Determination of hydraulic conductivity using the Permeafor

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DETERMINATION OF HYDRAULIC CONDUCTIVITY USING THE
PERMÉAFOR

BY

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BS, Merrimack College, 2008

THESIS

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# TABLE OF CONTENTS

ACKNOWLEDGEMENTS ......................................................................................... iii

TABLE OF CONTENTS ....................................................................................... iv

LIST OF TABLES ................................................................................................ ix

LIST OF FIGURES ............................................................................................. x

ABSTRACT ........................................................................................................ xv

1. INTRODUCTION ............................................................................................. 1

2. LITERATURE REVIEW .................................................................................... 5

2.1 Background .................................................................................................. 5

2.2 Fluid Fundamentals .................................................................................... 6

2.2.1 Compressibility ....................................................................................... 6

2.2.2 Fluid Statics ............................................................................................ 7

2.2.3 Concept of Head ...................................................................................... 9

2.2.4 Bernoulli's Equation ............................................................................. 10

2.2.5 Fluid Flow ............................................................................................... 10

2.3 Hydraulic Conductivity ............................................................................... 15

2.3.1 Darcy's Law .......................................................................................... 15

2.3.2 Hydraulic Conductivity .......................................................................... 16

2.4 Standard Testing Procedures ...................................................................... 18
2.4.1 Laboratory Tests ................................................................. 19

2.4.1.1 Sample Preparation ......................................................... 19

2.4.1.2 Constant Head Test ....................................................... 19

2.4.1.3 Falling Head Test ......................................................... 22

2.4.1.4 Hydraulic Conductivity as a Function of the Molding Water Content ........................................................................... 24

2.4.2 Field Tests ........................................................................... 27

2.4.2.1 Lefranc Permeability Test ............................................... 27

2.3.2.2 Well Pumping Test ......................................................... 28

2.3.2.3 Percolation Test .............................................................. 31

2.5 Hydraulic Conductivity Correlations ........................................ 32

2.5.1 Hazen Formula ................................................................. 32

2.5.2 Shepherd Formula ............................................................. 33

2.5.3 Kozeny-Carman Formula ................................................... 34

2.5.4 Alyamani and Sen Formula ............................................... 35

2.5.5 Rawls and Brakensiek Formula ......................................... 35

2.5.6 Summary of Hydraulic Conductivity Based on Grain Size .... 37

2.6 The Perméafor ...................................................................... 38

2.6.1 Standard Testing With the Perméafor ................................. 41

2.6.2 Measurement Devices .......................................................... 45
3.4.3 Determination of Head Losses within the System ...................... 92
3.4.4 Laboratory Perméafor Testing ........................................... 97
3.5 Perméafor Field Testing .................................................. 99
  3.5.1 Field Testing Location .............................................. 99
  3.5.2 Field Testing Setup ................................................ 100
  3.5.3 Soil Profile .......................................................... 102
  3.5.4 Determination of Head Losses .................................... 106
4. RESULTS ........................................................................ 108
  4.1 Introduction ................................................................ 108
  4.2 Laboratory Testing Results .......................................... 108
    4.2.1 Measured and Calculated Testing Parameters ............... 109
    4.2.2 Five Layer Soil Test .............................................. 110
    4.2.3 Seven Layer Soil Test .......................................... 116
    4.2.4 Ten Layer Soil Test ............................................. 121
    4.2.5 Saturation Test Results ......................................... 127
    4.2.6 Correlation of Laboratory Results to Hydraulic Conductivity .... 128
  4.3 Field Testing Results .................................................. 131
    4.3.1 Field Testing With the Perméafor ......................... 132
    4.3.2 Effect of External Head on Measurements .................. 140
    4.3.3 Hydraulic Conductivity Based On Grain Size .............. 141
<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.4 Field Testing Variations</td>
<td>143</td>
</tr>
<tr>
<td>4.4.1 Change in Constant Head Level</td>
<td>143</td>
</tr>
<tr>
<td>4.4.2 Portable Hydraulic Rig versus Dynamic Penetrometer Hammer</td>
<td>144</td>
</tr>
<tr>
<td>4.4.3 Screen Section</td>
<td>145</td>
</tr>
<tr>
<td>4.4.4 Tapered Design</td>
<td>146</td>
</tr>
<tr>
<td>4.5 Scale Model Perméafor Test Comparisons</td>
<td>147</td>
</tr>
<tr>
<td>4.5.1 Comparison of Laboratory and Field Perméafor Tests</td>
<td>148</td>
</tr>
<tr>
<td>4.5.2 Comparison to Additional Scale Model Laboratory Testing</td>
<td>151</td>
</tr>
<tr>
<td>4.5.3 Comparison to French Perméafor Results</td>
<td>155</td>
</tr>
<tr>
<td>5. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS</td>
<td>156</td>
</tr>
<tr>
<td>5.1 Summary</td>
<td>156</td>
</tr>
<tr>
<td>5.2 Conclusions</td>
<td>156</td>
</tr>
<tr>
<td>5.3 Recommendations and Future Work</td>
<td>159</td>
</tr>
<tr>
<td>LIST OF REFERENCES</td>
<td>161</td>
</tr>
<tr>
<td>APPENDIX A: Japanese Method for the Determination of Minimum and Maximum Void Ratio</td>
<td>164</td>
</tr>
<tr>
<td>APPENDIX B: Preliminary Soil Testing Results</td>
<td>168</td>
</tr>
<tr>
<td>APPENDIX C: Perméafor Laboratory Testing Results</td>
<td>173</td>
</tr>
<tr>
<td>APPENDIX D: Perméafor Field Testing Results</td>
<td>182</td>
</tr>
</tbody>
</table>
LIST OF TABLES

Table 2.1: Typical Values of Hydraulic Conductivity of Saturated Soils (Das, 2006) ............................................................... 17
Table 2.2: Variations of $n_{10C}/n_{200C}$ with Testing Temperature (Das, 2006) ........ 18
Table 2.3: Variations of $m$ Based on $l/D$ (Ursat, 1989) ......................................................... 58
Table 2.4: Example of Calculated $Q/H'$ with Depth (Ursat, 1989) ................................. 60
Table 2.5: Example of Calculated Hydraulic Conductivity with Depth .................. 60
Table 3.1: Results of Lee Sand Sieve Analysis ................................................................. 66
Table 3.2: Energy of Compaction for Each Soil Sample ..................................................... 87
Table 3.3: Laboratory Head Loss Parameters ................................................................. 93
Table 3.4: Parameters Used to Determine Head Losses .................................................. 95
Table 4.1: List of Parameters for Testing with the Perméafor ........................................... 109
Table 4.2: Results of $Q/H'$ for 5 Layer Sample Laboratory Test .................................. 111
Table 4.3: Results of $Q/H'$ for 7 Layer Sample Laboratory Test .................................. 117
Table 4.4: Results of $Q/H'$ for 10 Layer Sample Laboratory Test ................................ 122
Table 4.5: Results of $Q/H'$ for Dynamic Cone Hammer (DCH) 1 ............................. 133
Table 4.6: Results of $Q/H'$ for Dynamic Cone Hammer (DCH) 2 ............................. 134
Table 4.7: $Q/H'$ Results for all Field Tests .................................................................... 138
Table 4.8: Energy of Compaction for 10 and 12 Blow Per Layer .......................... 151
LIST OF FIGURES

Figure 2.1: Variation in Pressure with Elevation (Sleigh and Noakes, 2009)........ 7
Figure 2.2: Turbulent Flow and Laminar Flow (University of Cambridge CREST) ................................................................. 11
Figure 2.3: Pressure, Elevation, and Total Heads for Flow of Water Through Soil (Das, 2006).................................................................................................................. 13
Figure 2.4: Variation of Velocity, $v$, with Hydraulic Gradient, $i$ (Das, 2006)........ 14
Figure 2.5: Standard Proctor Mold Permeameter for Hydraulic Conductivity Testing (HUMBOLDT, 2010) ................................................................. 20
Figure 2.6: Constant Head Hydraulic Conductivity Testing Apparatus with Permeameter.................................................................................................................. 21
Figure 2.7: Falling Head Hydraulic Conductivity Testing Apparatus with Permeameter.................................................................................................................. 23
Figure 2.8: Hydraulic Conductivity as a Function of Molding Water Content (as shown in Sharma and Reddy, 2004) ................................................................. 26
Figure 2.9: Lefranc Permeability Test Well (as shown in Cassan, 2005)............. 28
Figure 2.10: Diagram of flow of water toward well during pumping test: (Left) if piezometric level lies above pervious layer; (Right) if free-water surface lies within pervious layer (Terzaghi, Peck, and Mesri, 1996)................................. 29
Figure 2.11: Rawls and Brakensiek Graphical Method for Estimating $k$ in cm/s . 37
Figure 2.12: Original Perméafor ........................................................................ 39
Figure 2.13: Perforated Midsection of the Perméafor ........................................ 39
Figure 2.14: Schematic of the Perméafor Testing ............................................. 42
Figure 2.15: (Left) Results of Permeafor Testing (thick blue line), Penetration Effort (thin brown line), (Right) Soil Profile with Depth (Centre D'Etudes Techniques de L'Equipement) .................................................. 43

Figure 2.16: Schematic of Permeafor measurement device locations .......... 46

Figure 2.17: Head Loss Calibration with the Permeafor (Left) Flow is Recorded with $H_e$ Determined, (Right) $H_c$ Water Level is Measured at Same $H_e$ .............. 48

Figure 2.18: $H_c$ at a Test Location ............................................................. 49

Figure 2.19: Calibration Curve ................................................................. 51

Figure 2.20: Total Head in the Field ......................................................... 52

Figure 2.21: Evaluation of Critical Flow Based on the Effective Diameter .... 55

Figure 2.22: Influence of External Head on $Q/H'$ Results (Ursat, 1989) .... 56

Figure 2.23: Measured Parameters in Field Testing ................................. 59

Figure 2.24: Results of Laboratory Tests Conducted by Torterat using the Half Scale Model Permeafor ................................................................. 62

Figure 3.1: Location of Sand Borrow on Garrity Road in Lee, New Hampshire (Google, 2010) ................................................................. 65

Figure 3.2: Soil Gradation Curves for Lee Sand ........................................... 66

Figure 3.3: Proctor Compaction Curve for Lee Sand .................................. 68

Figure 3.4: LEE Sand (Top) Hydraulic Conductivity Test, (Bottom) Compaction Test ................................................................. 73

Figure 3.5: Hydraulic Conductivity as a Function of Molding Water Content (Sharma and Reddy, 2004) ................................................................. 75

Figure 3.6: (Left) Full Scale French Permeafor, (Right) Scale Model Permeafor 77
Figure 3.7: Evaluation of Critical Flow Based on the Effective Diameter, \( D_{10} \) .... 78

Figure 3.8: Laboratory Perméafor Layout .................................................................................. 81

Figure 3.9: Overflow Protection for Perméafor Water Supply .............................................. 82

Figure 3.10: Split Piece Connector for Dynamic Cone Hammer ............................................. 83

Figure 3.11: Soil Layer Labeling of the Test Tank ................................................................. 85

Figure 3.12: LWFD Used for Soil Compaction (Vennapusa and White, 2008) .......................... 86

Figure 3.13: Dynamic Cone Penetrometer (Bottom) Hammer, (Middle) Cone Tip With Rod Attached, (Top) Additional Rods ................................................................. 88

Figure 3.14: Dynamic Cone Penetrometer Test in Laboratory Specimen .............................. 89

Figure 3.15: Results of Laboratory Dynamic Cone Penetrometer Test in 5, 7, and 10 Soil Lift Specimens .................................................................................................................. 90

Figure 3.16: (Left) Soil Specimen Ring Locations, (Right) Plan View of the Location of Rings in Soil Specimen .............................................................................................................. 92

Figure 3.17: Head Loss Calibration with the Perméafor (Left) Flow is Recorded with \( H_e \) Determined, (Right) \( H_c \) Water Level is Measured at Same \( H_e \) .................................. 93

Figure 3.18: Head Losses as a Function of Flow with and without the Flow Meter in Conjunction with the Reported French Head Losses ......................................................... 96

Figure 3.19: Test Site Location for Field Testing ....................................................................... 99

Figure 3.20: Water Source with Variable Head Levels for Field Testing ............................... 100

Figure 3.21: Perméafor Field Testing Setup ............................................................................. 101

Figure 3.22: Sieve Analysis Results from Field Samples ......................................................... 103

Figure 3.23: Moisture Content Profile from Field Samples ..................................................... 104

Figure 3.24: Dynamic Cone Penetration Field Testing Results .................................................. 105
Figure 3.25: Head Losses for Field and Laboratory Testing Setup

Figure 4.1: Results of $Q/H'$ for 5 Layer Sample Laboratory Test

Figure 4.2: Results of $Q/H$ with Time for 5 Layer Sample

Figure 4.3: Index Properties from 5 Layer Sample Test after Testing

Figure 4.4: Results of $Q/H'$ for 7 Layer Sample Laboratory Test

Figure 4.5: Results of $Q/H$ with Time for 7 Layer Sample

Figure 4.6: Index Properties from 7 Layer Sample Test after Testing

Figure 4.7: Results of $Q/H'$ for 10 Layer Sample Laboratory Test

Figure 4.8: Results of $Q/H$ with Time for 10 Layer Sample

Figure 4.9: Index Properties from 10 Layer Sample Test after Testing

Figure 4.10: Test Results of Saturation Tests Conducted on 5 and 7 Layer Soils

Figure 4.11: Variations of Hydraulic Conductivity from the Perméafor tests with Depth

Figure 4.12: Results of Field Testing with the Perméafor using the Dynamic Cone Penetrometer Hammer (DCH)

Figure 4.13: Field Tests Conducted with the Perméafor using the Dynamic Cone Penetrometer Hammer (DCH) and the Hydraulic Rig (HR)

Figure 4.14: Variation of $Q/H'$ as a Function of External Head

Figure 4.15: Hydraulic Conductivity Based on Grain Size

Figure 4.16: Increase Diameter Section of the Probe

Figure 4.17: Dynamic Cone Penetrometer Results for Laboratory and Field Testing
Figure 4.18: Results of $Q/H'$ for Laboratory and DCH Field Tests .................. 150
Figure 4.19: Head Losses in Laboratory Layer for Torterat and this Research 153
Figure 4.20: Results of $Q/H'$ for Torterat and this Research .......................... 154
ABSTRACT

DETERMINATION OF HYDRAULIC CONDUCTIVITY USING THE
PERMÉAFORE 

by

Amy C. Larrabee

University of New Hampshire, December, 2010

Hydraulic conductivity is a soil characteristic that describes the flow of water through soil due to the presence of interconnected voids. This property is difficult to accurately and rapidly evaluate in the laboratory and in the field. A device known as the Perméafor has been developed in Strasbourg, France aimed at evaluating hydraulic conductivity in situ. The tool is approximately 80 centimeters in length, and is specially designed to test lateral hydraulic conductivity. A half-scale of the Perméafor was constructed at the University of New Hampshire in order to test soils at shallow depth using a portable penetration system. The half-scale model was used in laboratory simulations and field testing of a fine-grained sand for the determination of hydraulic conductivity. Preliminary results seem to indicate the reduced size tool has the potential to evaluate permeability in situ.
CHAPTER 1

1. INTRODUCTION

Hydraulic conductivity is a characteristic that describes the flow of water or other liquids through soil. All soils are permeable due to the presence of interconnected voids which allows fluids to flow from points of high to low energy. Understanding hydraulic conductivity is necessary in a variety of engineering problems including dam design, septic system leachfield design, drainage applications, landfills and retention ponds. However, it is typically difficult to evaluate accurately.

Current practices for evaluating hydraulic conductivity rely on laboratory and field testing. Laboratory testing involves using discrete samples that may not necessarily be representative of the field conditions. These small samples represent a fraction of the volume of soil for any specific site. To address the shortcomings associated with laboratory testing, field test methods have gained increased popularity in spite of their cost and the time required for each test.

One commonly used method is the field pumping test. This involves the construction of a pump well and several monitoring wells, which measure the changes in water level over time from pumping. This test is costly as it involves the construction of several wells and testing may take days to months to carry
out. Additionally, well pumping tests result in an average hydraulic conductivity for the site as a whole. They do not have the ability to estimate the varying hydraulic conductivity with depth.

Other common in situ tests include the Lefranc test, the Piezocone Cone Penetration Test (CPTU), the Flat Plate Dilatometer Test (DMT) and piezometers tests. In response to the shortcomings of the current in situ and laboratory tests in providing rapid and accurate measurements, a device known as the Perméafor has been developed in Strasbourg, France. The Perméafor is aimed to aid in the evaluation of field hydraulic conductivity. The tool is approximately 80 centimeters in length and 5 centimeters in diameter with a perforated section at the center of the probe. The tapered design forces the flow of water to occur in the radial direction. The conical tip allows the probe to be inserted into the ground using a percussion drill rig.

Water is supplied to the Perméafor through small flexible tubing that runs inside the drive rods. During penetration, water continuously flows through the center and out laterally into the surrounding soil through the perforated section. At testing intervals of 20 centimeters, the advance is stopped and the flow is monitored for ten seconds before proceeding to the next test depth. A full profile can be carried out in a few hours. Variations of flow rate normalized by the applied pressure head are good indicators of hydraulic conductivity. For instance, in a gravelly sand layer, the flow will tend to increase compared to similar test conditions in a fine sand layer. These normalized flow readings can be used to identify soil types and estimate density and hydraulic conductivity.
The Permeafor can provide a continuous qualitative and quantitative profile of the subsurface showing the variation of hydraulic conductivity with depth. This test method is quick, efficient and cost effective providing instant results.

The research described in this thesis investigates the applicability of a half-scale Permeafor for use in shallow test applications for the evaluation of hydraulic conductivity. The model is approximately 40 centimeters in length and 2.5 to 4.0 centimeters in diameter, measuring half the size of the original tool.

The objectives of the research described in this thesis were the following:

1) Evaluate and understand the soil index and hydraulic properties of the Lee sand used in Permeafor testing. The sand is uniform and fine grained and was obtained from an abandoned sand borrow located in Lee, New Hampshire three miles from the University of New Hampshire campus. This site allowed for ample supply for laboratory testing while also providing the ability to conduct field tests at the home of Professor Jean Benoît, directly abutting the borrow area.

2) Develop laboratory and field testing procedures for the half scale Permeafor model.

3) Perform Permeafor laboratory tests in a test tank with reconstituted Lee sand of varying densities.

4) Perform field tests with the Permeafor.

5) Compare field Permeafor results with results from the laboratory hydraulic conductivity and the simulation test tank tests using the Permeafor.
Chapter II of this thesis provides some background information of hydraulic conductivity along with standard testing practices for the French Perméafor. Chapter III explains the materials and methods used to evaluate the half scale Perméafor in both a laboratory and field test setting. The test results of the laboratory and field testing are provided in Chapter IV. Finally, the summary, conclusions and recommendations for future work for the Perméafor are found in Chapter V.
CHAPTER 2

2. LITERATURE REVIEW

2.1 Background

In geotechnical engineering, hydraulic conductivity is a characteristic describing the ability for fluid to flow through a porous medium. Soils are permeable due to the presence of interconnected voids which allow water to flow from high to low energy. In geotechnical engineering, hydraulic conductivity is often interchangeable with the term coefficient of permeability and has units of length over time. From this point the term hydraulic conductivity will be used and it is assumed to be equivalent to the coefficient of permeability.

Hydraulic conductivity is a function of several factors including temperature and viscosity of the flowing fluid, pore and grain size and distribution, void ratio, material roughness, and degree of saturation. For saturated soils, hydraulic conductivity can vary more than ten orders of magnitude for soils ranging from gravel to clay.

Understanding hydraulic conductivity, or the flow of water through soils, is necessary in a variety of engineering problems including:

- Estimating the flow and amount of underground seepage in different hydraulic conditions
• Dewatering for construction below the water table
• Evaluating seepage forces in the analysis of earth dams or retaining walls

The understanding of hydraulic conductivity starts with a basic knowledge of fluid mechanics.

2.2 Fluid Fundamentals

Fluid mechanics deals with forces that act on fluids. Unlike solids, fluids are defined as substances that continuously deform. This deformation causes flow as a result of an applied shear stress to the fluid. Both liquids and gases are considered fluids.

2.2.1 Compressibility

All fluids can be compressed when an external pressure is applied. This pressure can then be released causing fluids to expand back to their original volumes. In conventional fluid mechanics, fluids are considered to be elastic media (Street, Watters and Vennard, 1996). The elasticity of fluid is related to the amount of deformation that occurs over a given pressure change. Elasticity is often called compressibility.

In geotechnical applications it is common to consider fluids to be incompressible, which simplifies the initial conditions. Soil in this case includes the combination of solids, liquids and gases. This simplification is valid when dealing with soils because the compressibility of the fluid is minimal in ordinary stress levels that occur in civil engineering applications (Holtz and Kovacs, 1981).
2.2.2 Fluid Statics

Fluids exert both normal and shear forces on the surfaces of which they are in contact with. Only normal forces are present if a fluid is static, or at rest. These normal forces in fluids are called pressure forces. Under equilibrium, the pressure at a point in a static fluid acts with the same magnitude in all directions and is referred to as hydrostatic pressure.

For static fluid the pressure only varies with the change in elevation of the fluid. Figure 2.1 illustrates a fluid element under equilibrium which is oriented in space in the \( s \) direction at an inclination \( \theta \) from the vertical. The element has a length \( \delta s \) and a cross-sectional area \( A \). The fluid is in static condition, therefore the only forces acting on the body are gravitational forces and pressure forces.

![Figure 2.1: Variation in Pressure with Elevation (Sleigh and Noakes, 2009)](image-url)
The gravitational force, \( mg \), can be defined as:

\[ mg = \gamma \cdot A \cdot \delta s \quad [2.1] \]

where the term \( \gamma \) is denoted as specific weight, which is the gravitational force per unit volume of fluid. The equation for equilibrium is then shown in Equation 2.2.

\[ \sum F_s = 0 = p \cdot A - (p + \delta p)A - \gamma \cdot A \cdot \delta s \cdot \sin \theta \quad [2.2] \]

Equation 2.2 can then be simplified as:

\[ \frac{\delta p}{\delta s} = -\gamma \sin \theta \quad [2.3] \]

If the length of the element is allowed to approach zero, then the limit becomes \( \frac{\delta p}{\delta s} = \frac{dp}{ds} \) and \( \sin \theta = \frac{dz}{ds} \) leading to Equation 2.4.

\[ \frac{dp}{ds} = -\gamma \left(\frac{dz}{ds}\right) \quad [2.4] \]

Further simplification yields Equation 2.5.

\[ \frac{dp}{dz} = -\gamma \quad [2.5] \]

Equation 2.5 is the basic equation for hydrostatic pressure variation with elevation. Equation 2.4 states that a change in pressure in the \( s \) direction \( (\frac{dp}{ds}) \) will occur only when there is a change in elevation in the \( s \) direction \( (\frac{dz}{ds}) \). This means that the fluid that lies on the same horizontal plane will have the same pressure. Changes in elevation will result in a change in hydrostatic pressure.

In practical applications several simplifications are made including that the density, therefore, the specific weight of the fluid are uniform throughout. This allows \( \gamma \) to be a constant leading to Equation 2.6.
\[ p + \gamma z = \text{constant} \] \hspace{1cm} [2.6]

The sum of the pressure and \( \gamma z \) is known as the piezometric pressure.

Equation 2.6 can be divided by \( \gamma \) resulting in Equation 2.7.

\[ \left( \frac{p}{\gamma} \right) + z = \text{constant} \] \hspace{1cm} [2.7]

The sum of the terms on the left side is called the piezometric or total head and has units of length.

### 2.2.3 Concept of Head

The concept of head relates the energy of the fluid to the height of the equivalent static column of that fluid. There are also several other energies that are combined to produce the total energy of a given fluid. These energies include the energy from the movement of the fluid, the energy associated with the pressure in the fluid, and the energy associated with the elevation of the fluid above a defined datum.

As stated previously, the sum of the pressure head \((p/\gamma)\) and the elevation head results in the piezometric head. The elevation head is defined as the elevation of the fluid above a datum or reference horizontal plane.

The energy that is associated with the movement of fluid is the velocity head. As the fluid accelerates it gains energy within the system known as kinetic energy. Kinetic energy is a function of the mass of the fluid and its viscosity. In conventional geotechnical engineering the velocity head is assumed to be minimal, and therefore is typically neglected in analysis. The three energy terms elevation, velocity, and pressure head are the available energy in a given fluid (Streeter, 1971).
In an ideal system, there are no energy losses. Since systems typically are not ideal, dissipation of energy is evident. This loss in energy is called head loss. In any moving fluid there are head losses due to the frictional forces acting against the fluid’s motion as it flows through the porous medium.

2.2.4 Bernoulli’s Equation

The relationship between velocity head, pressure head, and elevation head is known as the Bernoulli equation. The energy equation for incompressible steady flow of a fluid, Bernoulli’s equation, is shown in Equation 2.8.

\[ h = \left( \frac{u}{\gamma_w} \right) + \left( \frac{v^2}{2g} \right) + z \]  

where

- \( h \) = total head
- \( u \) = fluid pressure
- \( \gamma_w \) = unit weight of water
- \( v \) = fluid velocity
- \( g \) = acceleration due to gravity
- \( z \) = elevation

The pressure head is defined by the term \( u/\gamma_w \) if the fluid is water. The term \( v^2/2g \) is the velocity head. The elevation head is denoted by \( z \). The sum of all three energy terms produces the total head, \( h \), within the system.

2.2.5 Fluid Flow

In basic fluid mechanics, flow is described as steady or unsteady corresponding to the conditions being constant or varying with time. Flow can be
described as one, two, or three dimensional. For example, in the case of one dimensional flow all parameters including pressure, velocity and temperature are constant in any cross-section perpendicular to the direction of flow. Fluid flow problems in geotechnical engineering are mostly considered to be one or two dimensional.

Fluid flow is also described as laminar or turbulent. Laminar flow is when the fluid flows in parallel layers without mixing or intercrossing streamlines. The motion of the fluid is essentially in parallel paths as in the lower portion of Figure 2.2. The flow lines are illustrated by linear arrows which show that the particle motions are parallel to one another.

In turbulent flow the fluid particles do not remain in the parallel paths as in laminar flow. The fluid particles move throughout the fluid by sliding past one another and colliding with others. Turbulent flow has random velocity fluctuations which then mixes the fluid to dissipate the internal energy (Holtz and Kovacs, 11).
1981). An illustration of the flow lines for turbulent flow is shown in Figure 2.2 where the flow lines are denoted by arrows moving in random directions.

The zone in-between laminar and turbulent flow is called the intermediate or transition state. In laminar flow the head loss increases linearly as the velocity increases. To illustrate the concept of head loss, a simple geotechnical flow problem can be used as shown in Figure 2.3. This figure shows a soil sample with water flowing from left to right. Two open standpipes, or piezometers, are inserted at points A and B. These piezometers allow water to rise up into the tube to measure the piezometric levels at these points. The height to which the water rises within the tube is the pressure head. The pressure head at point A is $u_A/\gamma_w$, and the pressure head at point B is $u_B/\gamma_w$. The elevation head is simply the heights of points A and B above the specified datum. The elevation head at point A is $Z_A$ and at point B is $Z_B$. The velocity head in the system is negligible, and therefore not included. The total head within the system is the sum of the pressure and elevation which is denoted as $h_A$ and $h_B$, respectively.
The head loss that occur as water flows from point A to point B is defined as $\Delta h$ which is the difference between the total heads at points A and B as shown in Equation 2.9.

$$\Delta h = h_A - h_B = \left[\frac{u_A}{\gamma_w} + Z_A\right] - \left[\frac{u_B}{\gamma_w} + Z_B\right] \tag{2.9}$$

The total head loss $\Delta h$ that occurs over the table path $L$ through the soil can be used to define a non-dimensional term known as the hydraulic gradient as shown in Equation 2.10.

$$i = \frac{\Delta h}{L} \tag{2.10}$$

where $i = \text{hydraulic gradient}$

$\Delta h = \text{head loss}$

$L = \text{unit length over which the head loss occurred (distance}$
between point A and point B in the direction of flow)

The hydraulic gradient changes with the velocity of the flowing fluid. As seen in Figure 2.4, three zones can be outlined, Zone 1: Laminar Flow Zone, Zone 2: Transition Zone, and Zone 3: Turbulent Flow Zone. When flow is considered laminar, the hydraulic gradient increases linearly with increasing velocity. With a gradual increase of the hydraulic gradient through Zones 1 and 2, flow will remain laminar. At a higher hydraulic gradient the flow will become turbulent corresponding in Zone 3.

![Figure 2.4: Variation of Velocity, $v$, with Hydraulic Gradient, $i$ (Das, 2006)](image)

For laminar flow the velocity is proportional to the hydraulic gradient as shown in Equation 2.11.

$$v = ki$$  \[2.11\]

where $v = \text{discharge velocity}$

$i = \text{hydraulic gradient}$

$k = \text{hydraulic conductivity}$
The discharge velocity is a measure of the quantity of water that flows over time across a unit area. The flow, or percolation, occurs through the cross-sectional area at right angles to the direction of flow. This equation is primarily based on the observations of Darcy about the flow of water through clean sands. The seepage velocity, or actual velocity of the water, is greater than the discharge velocity. Therefore when the term velocity is used in conjunction with permeability it indicates the discharge velocity and not the seepage velocity.

2.3 Hydraulic Conductivity

2.3.1 Darcy’s Law

In 1856, Darcy, a French scientist, observed that the rate of flow in clean filter sands is proportional to the hydraulic gradient as outlined previously by Equation 2.11. Combining this, with the equation for continuity produces Darcy’s Law as stated in Equation 2.12.

\[ q = vA = kiA = k\left(\frac{\Delta h}{L}\right)A \]  \[2.12\]

where

- \( q \) = total rate of flow
- \( v \) = discharge velocity
- \( A \) = cross sectional area
- \( k \) = hydraulic conductivity
- \( i \) = hydraulic gradient, head loss (\( \Delta h \)) per unit length (\( L \))

Darcy’s law implies the discharge velocity has a linear relationship with the hydraulic gradient. As previously shown in Figure 2.4, this only holds true for laminar flow.
2.3.2 Hydraulic Conductivity

Hydraulic conductivity is also the measure of the ease of water to flow through permeable materials. Hydraulic conductivity is expressed in SI units as centimeters per second or meters per second (cm/s, m/s), or in English units as feet per minute or feet per day (ft/min, ft/day). Through mathematical analysis and measurements of the flow through permeable materials it has been illustrated that hydraulic conductivity is mainly influenced by the area of individual pores normal to the direction of flow, the shape of the pores, and the total area of pores (Terzaghi, Peck and Mesri, 1996). In geotechnical terms the factors that influence hydraulic conductivity are the following:

- Fluid viscosity
- Pore size and pore size distribution
- Grain size and grain size distribution
- Void ratio
- Roughness of mineral particles
- Degree of saturation

Typical values of hydraulic conductivity are often given based on the soil type because of the strong influence of pore size which is controlled by grain size. Typical values for hydraulic conductivity can be seen in Table 2.1.
<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Hydraulic Conductivity, k</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>cm/s</td>
</tr>
<tr>
<td>Clean Gravel</td>
<td>$10^2$-$10^0$</td>
</tr>
<tr>
<td>Coarse Sand</td>
<td>$10^0$-$10^{-2}$</td>
</tr>
<tr>
<td>Fine Sand</td>
<td>$10^{-2}$-$10^{-3}$</td>
</tr>
<tr>
<td>Silty Clay</td>
<td>$10^{-3}$-$10^{-5}$</td>
</tr>
<tr>
<td>Clay</td>
<td>&lt;10$^{-6}$</td>
</tr>
</tbody>
</table>

Table 2.1: Typical Values of Hydraulic Conductivity of Saturated Soils (Das, 2006)

Fluid properties such as viscosity, temperature, and unit weight need to be properly evaluated in the reporting of hydraulic conductivity values. The temperature at which tests are conducted will change the viscosity of the water. The hydraulic conductivity is inversely proportional to the viscosity of water. As the temperature of the water increases, the viscosity will decrease.

Conventionally, hydraulic conductivity is normalized to a standard of twenty degrees Celsius. Equation 2.13 shows the correction of hydraulic conductivity using the testing water viscosity and the viscosity of water at twenty degrees Celsius.

$$k_{20°C} = \left(\frac{\eta_{T°C}}{\eta_{20°C}}\right) \times k_{T°C}$$  \hspace{1cm} [2.13]

where

- $k_{20°C}$ = hydraulic conductivity at 20°C
- $\eta_{T°C}$ = viscosity of the water at temperature of test, T (°C)
- $\eta_{20°C}$ = viscosity of the water at 20°C
\( k_{T_{\text{OC}}} \) = hydraulic conductivity of soil at temperature of test, \( T \) (°C)

The ratios of viscosity for a range of typical testing temperatures are shown in Table 2.2.

<table>
<thead>
<tr>
<th>Temperature, ( T ) (°C)</th>
<th>( \eta_{T_{\text{OC}}} / \eta_{20_{\text{OC}}} )</th>
<th>Temperature, ( T ) (°C)</th>
<th>( \eta_{T_{\text{OC}}} / \eta_{20_{\text{OC}}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>1.135</td>
<td>23</td>
<td>0.931</td>
</tr>
<tr>
<td>16</td>
<td>1.106</td>
<td>24</td>
<td>0.910</td>
</tr>
<tr>
<td>17</td>
<td>1.077</td>
<td>25</td>
<td>0.889</td>
</tr>
<tr>
<td>18</td>
<td>1.051</td>
<td>26</td>
<td>0.869</td>
</tr>
<tr>
<td>19</td>
<td>1.025</td>
<td>27</td>
<td>0.850</td>
</tr>
<tr>
<td>20</td>
<td>1.000</td>
<td>28</td>
<td>0.832</td>
</tr>
<tr>
<td>21</td>
<td>0.976</td>
<td>29</td>
<td>0.814</td>
</tr>
<tr>
<td>22</td>
<td>0.953</td>
<td>30</td>
<td>0.797</td>
</tr>
</tbody>
</table>

Table 2.2: Variations of \( \eta_{T_{\text{OC}}} / \eta_{20_{\text{OC}}} \) with Testing Temperature (Das, 2006)

When water temperatures are below the standard of 20° Celsius the ratios from Table 2.2 are greater than one. The viscosity of water at the testing temperature is greater than the viscosity at twenty degrees Celsius. The ratio will then increase the reported value of hydraulic conductivity for the 20° Celsius standard.

**2.4 Standard Testing Procedures**

Standard laboratory tests for hydraulic conductivity include the constant head and the falling head tests, both applicable for various soil types. In general the constant head test is more suitable for permeable soils and the falling head test is better adapted for less permeable soils. Common field tests for hydraulic
conductivity in soils include the Lefranc test, well pumping test, and the percolation test. A brief description of each follows.

2.4.1 Laboratory Tests

2.4.1.1 Sample Preparation. For laboratory permeability tests a soil specimen must be prepared at a prescribed density. This can be easily accomplished using a Proctor mold with top and base plates specially designed to allow flow of water through the compacted soil specimen. Standard or Modified Proctor Compaction Test methods can be used as outlined by American Society for Testing and Materials (ASTM) Standards D 698 and D 1557.

The energy of compaction is a function of the hammer drop height and weight, the number of layers and number of blows per layer. As more energy is inputted into the soil the resulting dry unit weight will be greater. The energy of compaction can be evaluated by Equation 2.14.

\[ E = \frac{\text{No. Layers} \times \text{No. Blows/Layer} \times \text{Weight of Hammer} \times \text{Drop Height}}{\text{Volume of Mold}} \]  

[2.14]

The standard methods produce a compaction energy of approximately 12,400 ft-lbf/ft\(^3\), while the modified produces a compaction energy of 56,000 ft-lbf/ft\(^3\).

2.4.1.2 Constant Head Test. The constant head laboratory test determines the hydraulic conductivity of granular soil, as outlined in ASTM Standard D 2434. The test is limited to soils that contain no more than ten percent passing the No. 200 sieve. The soil sample is tested in a standard four inch Proctor mold permeameter as shown in Figure 2.5. The base of the Proctor
mold houses a porous stone that allows water to exit the mold during testing. The soil is compacted using either the standard or modified compaction method. A porous stone is then placed on top of the compacted soil. A spring is placed between the top cap and the porous stone to prevent the soil specimen from swelling during saturation.

![Figure 2.5: Standard Proctor Mold Permeameter for Hydraulic Conductivity Testing (HUMBOLDT, 2010)](image)

Once the soil specimen is compacted and the dry unit weight and molding water content are determined, the soil sample must be saturated. To achieve saturation, water is allowed to percolate from the bottom connection through the soil specimen. Using gravity, a head of water is maintained above the sample. Water percolates through the sample until it reaches the top cap where it can exit. Saturation is assumed to occur when air bubbles no longer exit the top cap. This process is typically achieved in less than ten minutes.
At UNH, the constant head hydraulic conductivity test is conducted by supplying water through the top cap of the permeameter using a Mariotte's bottle system as shown in Figure 2.6.

Once a constant flow rate has been established, water flowing out of the bottom of the permeameter is collected in a graduated cylinder for a known duration. Three sets of measurements (volume and time) are recorded and an
average is used to determine the hydraulic conductivity based on Darcy’s Law expressed by Equation 2.15.

\[ Q = A v t = A (k i) t \]  

where \( Q \) = volume of water collected

\( A \) = cross-sectional area of the soil specimen

\( v \) = discharge velocity

\( t \) = duration of water collection

\( k \) = hydraulic conductivity of sample

\( i \) = hydraulic gradient

Using the hydraulic gradient, \( i \), as \( h / L \), where \( L \) is the length of the specimen and \( h \) is the constant head of water maintained during testing yields Equation 2.16.

\[ k = \frac{(Q * L)}{(A * h * t)} \]  

The temperature of the water flowing through the sample is also recorded so the value of hydraulic conductivity can be reported to the standard of 20°C.

2.4.1.3 Falling Head Test. The falling head hydraulic conductivity laboratory test can be performed on the same soil specimen after the constant head test is done as the specimen is already saturated. The test itself is conducted similarly to the constant head test but with a variable head. Water is supplied by a standpipe above the soil specimen. The initial head of water within the standpipe is recorded at \( h_1 \) at a corresponding time \( t_0 \). The water is then allowed to flow through the soil to a final head position, \( h_2 \), where time is then
stopped and recorded as $t_2$. The standard falling head hydraulic conductivity testing apparatus with permeameter is shown in Figure 2.7.

Three tests are performed for each soil sample in order to achieve an average hydraulic conductivity for the specimen. The rate of flow through the specimen at any time $t$ can be expressed as shown in Equation 2.17.

$$ q = k \left( \frac{h}{L} \right) A = -a \left( \frac{dh}{dt} \right) $$  \hspace{1cm} [2.17]$$

where

- $q = \text{flow rate}$
- $k = \text{hydraulic conductivity of sample}$
\[ L = \text{length of specimen} \]
\[ a = \text{cross-sectional area of the standpipe} \]
\[ A = \text{cross-sectional area of the soil specimen} \]

Equation 2.17 can be rearranged to yield Equation 2.18.

\[ dt = \left( \frac{aL}{Ak} \right) \left( -\frac{dh}{h} \right) \]  \[ [2.18] \]

The left side of Equation 2.18 can be integrated with limits of time from 0 to \( t \), and the right side can be integrated with limits of head difference from \( h_1 \) to \( h_2 \) resulting in Equation 2.19.

\[ t = \left( \frac{aL}{Ak} \right) \log_e \left( \frac{h_1}{h_2} \right) \]  \[ [2.19] \]

Finally, solving for \( k \) results in Equation 2.20, which is used for the determination of hydraulic conductivity for the falling head test.

\[ k = 2.303 \times \left[ \frac{(a * L)}{(A * t)} \right] \times \log \left( \frac{h_1}{h_2} \right) \]  \[ [2.20] \]

where \( t = \text{time from } t_0 \text{ to } t_1 \)
\( h_1 = \text{initial head} \)
\( h_2 = \text{final head} \)

\[ 2.4.1.4 \text{ Hydraulic Conductivity as a Function of the Molding Water Content} \]

Similarly to the compaction levels, the hydraulic conductivity is a function of the molding water content prior to testing. Figure 2.8 illustrates three compaction curves and the corresponding hydraulic conductivity tests performed on each soil mold. The compaction curves at the top of Figure 2.8 shows the dry unit weight of soil with increasing molding water content for three different soil...
compaction energies. A reduced Proctor, a standard Proctor, and a modified Proctor test were used to compact the same soil. The solid black line on the top figure is the zero air void curve representing 100% saturation.

Hydraulic conductivity tests were performed on each compaction mold. The molding water content is noted prior to testing as the soil sample is saturated before hydraulic conductivity testing is conducted. As shown in the figure, hydraulic conductivity tends to decrease as the molding water content increases. As the molding water content increased past the optimum water content, the hydraulic conductivity levels off and in some cases starts increasing as a function of insufficient compaction effort. This figure also shows that as the compaction energy increases the hydraulic conductivity decreases which is reasonable considering that the increased energy creates a higher dry density.
Figure 2.8: Hydraulic Conductivity as a Function of Molding Water Content (as shown in Sharma and Reddy, 2004)
2.4.2 Field Tests

There are a variety of in situ tests available that are designed to evaluate the hydraulic conductivity. These tests include the Lefranc Permeability Test, the Well Pumping Test, and the Percolation test and are discussed in the following sections.

2.4.2.1 Lefranc Permeability Test. The Lefranc permeability test is an in situ test that originated in France and is commonly used in the United States. The test is performed within a borehole and can be used to determine the hydraulic conductivity above and below the water table. The test is quick and relatively inexpensive to perform while also providing information to both the structure and heterogeneity of the subsurface (Cassan, 2005).

Testing is performed in an open hole at a specified test depth. To prevent caving or collapse of a borehole, the walls of the hole may be cased to the top of the test zone. The test zone is extended a suitable length below the casing. The preparation of the test zone is dependent on the soil type and is the most delicate portion of the test. The zone of interest is filled using gravel and then plugged or capped to isolate the test area. Testing can be performed by pumping water either into or out of the borehole. The measure of flow through the filter system can be correlated to hydraulic conductivity. Figure 2.9 illustrates the Lefranc permeability test.
The test is only reliable for hydraulic conductivities greater than approximately $10^{-4}$ cm/s (Cassan, 2005). The results are highly dependent on the quality and integrity of the test zone. Fines that are suspended in the water can possibly form a filter skin over the bottom and the sides of the test zone reducing or even plugging the flow of water resulting in a reduction of the actual hydraulic conductivity. The Lefranc method for determining hydraulic conductivity focuses on specific test zones and must be completed at several depths and within several boreholes to evaluate the hydraulic conductivity of an entire site.

2.3.2.2 Well Pumping Test. As the Lefranc testing only provides information on the order of magnitude of the coefficient of permeability, well pump tests allow for
more reliable information. Unlike the Lefranc test, a well pumping test has the ability to measure hydraulic conductivity of an entire site with one test.

Either specific test zones or larger test areas can be evaluated during well pumping. Testing is completed either by measuring one value of hydraulic conductivity for the site or at specific depths. Several variations of the well pump test can be applied to aid in the understanding of hydraulic conductivity both in depth and area.

Well pumping usually includes a pump well along with several observation wells. Observation wells are located away from the pump well at various distances in the direction(s) of interest to measure the drawdown as water is pumped out of the pump well. Figure 2.10 illustrates the concept of the pump test for the case of a confined aquifer and an unconfined gravity aquifer. The number of observation wells is site and application specific.

Figure 2.10: Diagram of flow of water toward well during pumping test: (Left) if piezometric level lies above pervious layer; (Right) if free-water surface lies within pervious layer (Terzaghi, Peck, and Mesri, 1996)

Pumping is started at a constant rate until a steady state has been reached while also monitoring the initial and change in water level at all
observation wells. Using the theory of radial flow to a well the hydraulic conductivity can be determined for the aquifer. For the case of the confined aquifer the hydraulic conductivity can be evaluated using Equation 2.21.

\[ k = \left[ \frac{q}{(2 \pi H_0 (h_2 - h_1))} \right] \times \ln \left( \frac{r_2}{r_1} \right) \]  

where \( k \) = hydraulic conductivity of test zone
\( q \) = constant pumping flow rate
\( H_0 \) = saturated thickness of pervious layer
\( h_1 \) = height of the water in the first observation well with respect to the bottom of the permeable layer
\( h_2 \) = height of the water in the second observation well with respect to the bottom of the permeable layer
\( r_1 \) = distance to first observation well from pumping well
\( r_2 \) = distance to second observation well from pumping well

For the case of the unconfined aquifer the hydraulic conductivity is evaluating using Equation 2.22.

\[ k = \left[ \frac{q}{(\pi (h_2^2 - h_1^2))} \right] \times \ln \left( \frac{r_2}{r_1} \right) \]  

Overall, well pumping provides an accurate depiction of the hydraulic conductivity of a site as a whole. If the site contains variable soil strata conditions, this test may not be ideal as the hydraulic conductivity varies with soil types and densities. Also, well pumping tests are time consuming and costly due to the need for several observation wells that are often abandoned after the completion of the test.
2.3.2.3 Percolation Test. The percolation test, or Perc test, is mainly used to determine gross relative permeability in the application of septic design. In order to define the hydraulic conductivity, Henry Ryon, in the 1920s, developed a test to estimate the hydraulic conductivity by evaluating the soils of nearly failing and non-failing leachfields. The hydraulic conductivity of soil is important in the design of septic systems because it will “influence aeration, water flow, water retention, biological activities, and filtration of parasites and pathogens” (Kaplan, 1988). The final resulting test became a crude measure of hydraulic conductivity that can be associated with the design leachfield size and the estimated design lifetime.

A Perc test is performed by having a test hole of four to twelve inches in diameter and in depth near the proposed leachline. The bottom of the hole is then cleaned of any smeared soil surfaces and about two inches of fine gravel or coarse sand is added to prevent scouring. With the hole prepared, soaking is conducted by maintaining a high water level in the hole. This soaking is maintained for at least four hours but preferably overnight.

Depending on the elevation of the water in the hole after the overnight soaking, the hole is refilled to a depth of six inches above the gravel layer. From a fixed position the height of the water within the hole is measured over time. The water drop that occurs can then be used to determine the percolation rate (Kaplan, 1988).

The measurement of the ability for water to flow through soils cannot be determined by simply measuring how fast water disappears from a hole.
Therefore the Perc test is actually measuring gross relative permeability that is only relevant for the design of septic systems.

2.5 Hydraulic Conductivity Correlations

In an attempt to evaluate the hydraulic conductivity of soils without performing costly and time consuming laboratory or field tests, several empirical relationships have been developed based on particle index properties. The correlations used for granular materials in particular are discussed in the following sections.

2.5.1 Hazen Formula

About a century ago, based on his observations on loose clean filter sands, Hazen expected that the permeability of granular soils may increase as a function of the square of some grain size characteristic (Terzaghi, Peck and Mesri, 1996). Hazen proposed an empirical relationship describing hydraulic conductivity as a function of the effective size as shown in Equation 2.23.

\[
k = c \cdot D_{10}^2
\]  

[2.23]

where \( k \) = hydraulic conductivity (cm/s) 
\( c \) = Hazen empirical coefficient 
\( D_{10} \) = effective grain size for which 10% of the soil is finer (cm)

The Hazen empirical coefficient is a parameter that takes into account the effects of the shape of the pore channels in the direction of flow and the total volume of pores. The empirical correlation is best suited for clean sands with a coefficient of uniformity \( C_u = D_{60}/D_{10} \) less than two. Typically, the coefficient is
taken as 1 when $k$ is expressed in m/s and $D_{10}$ is expressed in centimeters. This equation does have several limitations and may over or underestimate the hydraulic conductivity of granular soils by a factor of about two (Terzaghi, Peck and Mesri, 1996). Generally the formula is applicable for soils that have an effective size from 0.01-0.3 centimeters (Carrier, 2003). Although the Hazen empirical formula was conducted on loose clean filter sands, it does provide an estimation of the hydraulic conductivity.

### 2.5.2 Shepherd Formula

In 1989, Shepherd expanded upon the work of Hazen by performing regression analysis on 19 different sets of published data. The data sets consisted of unconsolidated sediments (Cronican and Gribb, 2004). Throughout testing, Shepherd found that the Hazen empirical coefficient ($c$) is most often between 0.05 and 1.18, but can also reach as high as 9.85.

In addition, Shepherd found that the exponent for the $D_{10}$ term in Equation 2.23 varies from 1.11 and 2.05. The average value for this exponent is 1.72. Values for both the exponent and empirical coefficient are typically higher for poorly graded samples with spherical grain particles.

Overall, the Hazen formula is not a universal correlation. Research completed by Shepherd shows the wide variation that is associated with hydraulic conductivity.
2.5.3 Kozeny-Carman Formula

About a half-century ago Kozeny and Carman developed a semi-empirical, semi-theoretical formula to predict the hydraulic conductivity of porous media. The Kozeny-Carman formula is shown in Equation 2.24.

\[
k = \left(\frac{\gamma}{\mu}\right) * \left(\frac{1}{C_{K-C}}\right) * \left(\frac{1}{S_0^2}\right) * \left(e^3/(1 + e)\right)
\]

[2.24]

where

- \( k \) = hydraulic conductivity (cm/s)
- \( \gamma \) = unit weight of the permeant
- \( \mu \) = viscosity of the permeant
- \( C_{K-C} \) = Kozeny-Carman empirical coefficient
- \( S_0 \) = specific surface area per unit volume of particles (1/cm)
- \( e \) = void ratio

The first term in the formula relates the unit weight and the viscosity of the permeate. The Kozeny-Carman empirical coefficient is reported to be 4.8±0.3, and the average is usually taken as 5. If the hydraulic conductivity is reported at the standard temperature of 20°C, the equation then becomes Equation 2.25.

\[
k = 1.99 \times 10^4 \times \left(\frac{1}{S_0^2}\right) * \left(e^3/(1 + e)\right)
\]

[2.25]

Similarly to the Hazen formula, the Kozeny-Carman formula has several limitations. The formula is not applicable to clays and silts because it assumes that there are no electrochemical reactions between the soil particles and water. This formula also assumes that the fluid passing through the porous medium follows Darcy's law. Consequently, if turbulent flow exists, such as in gravel, the Kozeny-Carman formula would not be applicable.
2.5.4 Alyamani and Sen Formula

Alyamani and Sen (1993) set out to derive an empirical relationship for hydraulic conductivity also based on the grain size distribution curve. Using 32 sandy soil samples from Saudi Arabia and Australia the equation to determine hydraulic conductivity is shown in Equation 2.26.

\[ k = 1.505 \times [I_o + 0.025(D_{50} - D_{10})]^2 \]  

where
\[ k = \text{hydraulic conductivity (cm/s)} \]
\[ I_o = \text{the intercept (in mm) of the line formed by } D_{50} \text{ and } D_{10} \text{ with the grain size axis} \]
\[ D_{50} = \text{grain size (mm) for which 50\% of the soil is finer} \]
\[ D_{10} = \text{effective grain size (mm) for which 10\% of the soil is finer} \]

It is important that the units used in Equation 2.26 are consistent with the stated units. The \( I_o \) term is the intercept on the x-axis of the straight line that is formed by joining \( D_{50} \) and \( D_{10} \) on the grain size distribution curve. Alyamani and Sen also found that a log-log plot of the hydraulic conductivity versus \([I_o + 0.025(D_{50} - D_{10})]\) for the provided data set produced a linear fit with a coefficient of determination \( (R^2) \) of 0.94 (Cronican and Gribb, 2004). This method considers both the grain sizes \( D_{50} \) and \( D_{10} \) as well as the influence of grain size distribution.

2.5.5 Rawls and Brakensiek Formula

In 1989, Rawls and Brakensiek used field data from 1323 soils from across the United States to estimate hydraulic conductivity. Using a regression analysis the resulting equation relates the porosity \( (n) \), the percentage of sand-sized particles \( (S) \), and the percentage of clay-sized particles \( (C) \). The equation
developed by Rawls and Brakensiek estimates hydraulic conductivity in centimeters per hour and is shown in Equation 2.27.

\[ k = \exp[19.52348 \times n - 8.96847 \times C - 0.00018107 \times S^2 - 0.0094125 \times C^2 - 8.395215 \times n^2 + 0.077718 \times S \times n - 0.00298 \times S^2 \times n^2 - 0.019492C^2 \times n^2 + 0.0000173 \times S^2 \times C + 0.02733 \times C^2 \times n0.001434 \times S^2 \times n - 0.0000035 \times C^2 \times S] \]

[2.27]

where  
\[ k = \text{hydraulic conductivity (cm/h)} \]  
\[ n = \text{porosity} \]  
\[ C = \text{percentage of clay-sized particles by weight} \]  
\[ S = \text{percentage of sand-sized particles by weight} \]

The Rawls and Brakensiek equation is also produced in graphical form. Hydraulic conductivity is expressed in centimeters per second with the assumption that the fraction of organic matter assumed to be 1.5%. The graphic form of Equation 2.27 is shown in Figure 2.11. To determine the hydraulic conductivity of the soil sample both the percentage of sand and clay sized particles are needed. The soil sample can then be plotted on the chart. From the plotted point the hydraulic conductivity is estimated by projecting the point onto the hydraulic conductivity axis.

For example, a soil has 35 percent sand and 35 percent clay sized particles. The soil is then plotted on Figure 2.11 as a clay loam material. From this point the hydraulic conductivity is determined by projecting the point onto the diagonal axis. The point is projected by approximating the distance between the darkened boundaries established by Rawls and Brakensiek to the hydraulic
conductivity axis. For this example, the hydraulic conductivity is approximately 4.3E-5 centimeters per second.

![Graphical Method for Estimating k in cm/s](image)

Figure 2.11: Rawls and Brakensiek Graphical Method for Estimating k in cm/s

Both the equation and graphical method developed by Rawls and Brakensiek were derived for soils containing 5 to 70% sand-sized particles and 5 to 60% clay-sized particles (Cronican and Gribb, 2004).

**2.5.6 Summary of Hydraulic Conductivity Based on Grain Size**

Several methods have been developed to try to evaluate the hydraulic conductivity based on grain size and grain size distribution. One experiment was conducted on 84 filter pack sands to estimate the hydraulic conductivity using the
Hazen formula, the Kozeny-Carman formula, and the Alyamani and Sen formula. The best performer was the Hazen formula predicting hydraulic conductivity values within 12% of the actual value. The Kozeny-Carman equation estimated values 73 to 83% lower than the measured values. The Alyamani and Sen formula estimated values 30 to 36% greater than the measured values.

It appears that the simplest method produces the most reliable results. The Hazen formula requires knowledge of only one variable and predicts hydraulic conductivity values on the same order of magnitude of the measured results. Introducing more complex variable, as in the Kozeny-Carman formula, does not always imply that the result is more reliable.

In general, empirical relationships are not ideal for the estimation of hydraulic conductivity. The current relationships are outlined for specific soil types and are not universal. For design considerations, a true value of hydraulic conductivity needs to be established, rapidly and economically. Every site is different which creates a need for a tool that can be used in a variety of conditions.

2.6 The Perméafor

The Perméafor is an in situ soil testing device developed in the early 1980s at the laboratory of the Strasbourg Highway Department, located in western France (Ursat, 1992). This tool allows for rapid and continuous probing for the evaluation of hydraulic conductivity in subsurface soils.

The Perméafor probe as shown in Figure 2.12 is approximately 80 centimeter in length. The Perméafor is pushed or vibrated into the subsurface
using a conventional or percussion drill rig. The tool is hollow to allow water to flow from the ground surface to the center portion of the tool. The midsection of the probe is perforated allowing water to flow into the soil during advancement and testing. The original Permeafor from France is shown in Figure 2.12.

Figure 2.12: Original Permeafor

To ensure that the flow of water is indeed in the lateral direction, the perforated section is recessed from the surrounding probe body. The flow then becomes well defined in the horizontal direction and is perpendicular to the screen itself (Ursat, 1992). Figure 2.13 illustrates the screen as it is smaller in diameter then the surrounding probe.

Figure 2.13: Perforated Midsection of the Permeafor
In addition the probe also has a tapered design to induce lateral flow. The tip of the Perméafor has a minimum diameter of 3.5 centimeters and is tapered to the top of the probe having a final diameter of 5 centimeters. As the probe is penetrated into the soil the space created is gradually increased in diameter. The increase in diameter from 3.5 to 5 centimeters creates a doubling of the volume which approximately corresponds to the limit pressure of the soil. The gradual transition constrains flow in the lateral direction such that the water will not flow up the outside of the rods back to the surface. Water may still have a tendency to flow upwards along the drill rods for the first two to three meters of penetration. Once the seal has been created lateral flow is established. For the case of overconsolidated soils lateral flow is established at the surface.

Water is supplied to the Perméafor using tubing that runs inside the push rods. Water then exists through the screen area into the surrounding soil. This perforated section is approximately 5 centimeters in length and has a length to diameter ratio of one. The geometry of the tool allows for uninterrupted testing without requiring pre-drilling (Reiffsteck et al., 2007).

Water flow begins before the probe is inserted into the ground and continues until the probe is removed at the end of a boring. Allowing flow during the entire sounding prevents fines from migrating into the probe which would potentially block flow. The probe can then be checked at the end of the boring to ensure that flow was continuous throughout the entire boring.

In situ conditions can be evaluated rapidly in a semi-continuous manner. The advancement of the probe is stopped every 20 centimeters to perform a
single test lasting 10 seconds. Water is allowed to flow into the soil while measurements of depth, flow, and pressure head are recorded. With the ability to test in such a rapid manner, the Perméafor can test over 40 meters linearly per day which proves to be a cost effective method providing on the spot results (Reiffsteck, 2009).

2.6.1 Standard Testing With the Perméafor

The overall objective of the probe is to measure lateral hydraulic conductivity. Measurement of the volume of water flowing into the subsurface is correlated to the permeability of the soil. Flexible tubing runs from the surface water source to the top of the probe. Due to the elevation and pressure head produced by the water column above the probe, water is propelled into the surrounding soil.

A schematic of the Perméafor test is shown in Figure 2.14. The flow of water, $Q$ (m$^3$/sec), is measured at the surface. As water flows through the tubing to the probe, head losses ($dH$) occur due to friction. Head losses are also present in the flow meter and the tubing connection areas. The effective head, $H'$, is then reported which is a function of the pressure head and the pressure losses. The water then flows into the subsurface through the screen.
Results from testing with the probe are reported for each test depth as a ratio of flow to the effective head, $Q/H'$, and the penetration effort to advance the probe. Typical results from Permeafor testing are shown in Figure 2.15.

The right portion of the figure shows the soil profile as logged through standard sampling. The left side of the figure shows a profile of $Q/H'$ and the penetration effort. Every 20 centimeters the probe advancement is stopped and the flow is recorded after it is allowed to stabilize for 10 seconds. The vertical load is maintained on the probe while flow measurements are recorded.

The values of $Q/H'$ are bounded by the dashed vertical lines located at the values of $10^{-3}$ and $10^{-6}$ square meters per second. Values within this region are considered acceptable for measurements with the probe. These provisional bounds are a function of several parameters surrounding the probe.
Figure 2.15: (Left) Results of Permeafor Testing (thick blue line), Penetration Effort (thin brown line), (Right) Soil Profile with Depth (Centre D'Etudes Techniques de L'Equipement)

The upper bound is established for high flow situations. When soils are highly permeable and flows are great, the accuracy is reduced. Sources of error during high flow include the measurement of the height of the water source, the
depth to the water table and the calibration of head losses. For the French probe these readings are measured using computer software. The total error is less than 10 to 12 centimeters at the worst case scenario, but errors are cumulative.

The upper bound is also relevant as the ratio of $Q/H'$ can be negative. When the ratio becomes negative it is a function of the calculated head losses as they are larger than the head provided above the Perméafor screen. This anomaly is typically seen at the surface of the probe sounding where the flow is not restricted by the surrounding soil.

The lower bound is a function of the soil type at which the probe is located. When soils are not highly permeable flow will be minimal. Flow is recorded after a ten second time period when a steady state is achieved. In low permeable soils the time to achieve a steady state is significantly greater than ten seconds. For this reason, the values of $Q/H'$ have a lower bound at $10^{-6}$ square meters per second. In the French system it is possible to have values as low as $3E-07$ square meters per second.

Trends correlating the soil stiffness and $Q/H'$ are evident in Figure 2.15. When the penetration effort is large, the flow is decreased. When a soil is stiff or compact the void ratio is small leading to a lower hydraulic conductivity. This trend is also seen in reverse, as the penetration effort is less, the void ratio is greater, and flow is higher. The silt layer is also interesting as it is easy to push the probe through, but the flow is still small. Silts have a lower hydraulic conductivity compared to sands and gravels.
The Perméafor results shown in Figure 2.15 were used to evaluate the effectiveness of a dike. The results illustrate that the dike is more permeable than the underlying silt layer. This implies that the dike is not performing to the design standards. The Perméafor tool can be used in a variety of other applications such as detecting thin seams of variable soil which may not be detected using conventional in situ methods. These soil variations may alter the design of the structure to be built on the site. For example, the Perméafor is capable of detecting thin silt seams under foundations which may cause differential settlement.

2.6.2 Measurement Devices

Flow meters used for the Perméafor make it possible to evaluate flows from as little as 0.1 to 180 liters per minute with a precision of ± 0.5%. At the surface the application of a hydraulic load from the pumps have the ability to apply 3 to 5 meters of head. The pressure, \( H_p \), or charge applied by the device is measured at the level of the pump (Ursat, 1992).

An incremental position sensor is used to determine the depth at which the Perméafor is located with respect to a point of reference above the ground surface. A schematic of the testing with defined location of measurement tools can be seen in Figure 2.16.
2.6.3 Determination of the Head Losses and System Calibration

Important pressure losses occur in the Perméafor system as a function of the friction imposed on the water from the small diameter tubing and the friction caused by the flow meter. The friction reduces the water pressure that is seen at the screen section of the probe compared to the theoretical pressure. These losses are summed together and used to find the effective pressure during testing. The pressure losses are obtained during the calibration of the probe at various flow values. These pressure losses are primarily dependent on:

- Composition of the injected water (salinity of water)
- Ambient air temperature and groundwater temperature
- Length of flexible tubing supplying water to the system
Flow meters used to measure flow throughout testing

Perméafor head loss calibration is typically carried out using various length of tubing. The effects of water temperature and salinity are minimal. Head losses are a function of the flow through the probe. A quadratic relationship can be established to explain the relationship of the head losses in the system, $\Delta H_{HT}$, and the flow, $Q$, through the tubing. This equation is shown in Equation 2.28.

$$\Delta H_{HT} = aQ^2 + bQ + c$$  \[2.28\]

where

$\Delta H_{HT} =$ theoretical head losses within the system

$a, b, c =$ coefficients

$Q =$ flow (liters per minute)

Calibration is conducted in two separate steps performed with and without the Perméafor. Before calibration the Perméafor and the flexible tubing is purged of air. If any air bubbles remain in the tubing or flow meter flow will be minimized or even stopped. The full sized Perméafor uses a pressurized system for the water injection process which is also purged.

The first step in calibration is performed by placing the Perméafor into a large bucket filled with water as shown on the left in Figure 2.17. The water supply is connected to the probe with a length of flexible tubing. The flow is measured at least at four different heights. The height from the top of the water column to the top of the bucket is noted as $H_e$, which is the parameter that changes at the four locations of testing. The total amount of head produced by any additional pump is also recorded.
The second step in the calibration process does not use the Perméafor.

The probe is detached from the flexible tubing. At the same testing height locations as previously completed with the bucket ($H_e$), the value of $H_c$ is measured. $H_c$ is the height of the water as it is ejected out of the tubing caused by the head of water above the test location. Figure 2.18 demonstrates the value of $H_c$ in a laboratory test.
For each of the four test heights the experimental head losses are determined as a function of flow, $Q$ and head, $H_e$. The additional head that is added from the hydraulic pumping system, $H_p$, is also involved in the determination of the head losses. Using the measurements from testing, the head losses within the system can then be defined using Equation 2.29.

$$\Delta H_{\text{exp}} = H_e - H_c + H_p$$  \hspace{1cm} [2.29]$$

where

$\Delta H_{\text{exp}} =$ experimental head losses within the system (meters)

$H_e =$ height of water above the top of the bucket (meters)

$H_c =$ height of water that is ejected out of the tubing (meters)

$H_p =$ the head of water produced by the hydraulic pumping system (meters)
Finally, to determine the coefficients $a, b, c$ as defined by Equation 2.28, the Gauss matrix is solved, as shown in Equation 2.30. The unknown parameters $a, b, c$ are determined by taking the inverse of the matrix. The input parameters are the four values of flow, the four values of experimental head losses as defined in Equation 2.29 and the number of test heights used which is four. The sum of the flow values are then raised to the fourth, third, second, and first power and used as the input on the left side matrix in Equation 2.30. The sum of the flow raised to the second or first power multiplied by the sum of the experimental head losses and the sum of the experimental head losses are used as the input parameters for the right side matrix in Equation 2.30.

\[
\begin{bmatrix}
\sum Q_i^4 & \sum Q_i^3 & \sum Q_i^2 \\
\sum Q_i^3 & \sum Q_i^2 & \sum Q_i \\
\sum Q_i^2 & \sum Q_i & N
\end{bmatrix} * \begin{pmatrix} a \\ b \\ c \end{pmatrix} = \begin{bmatrix}
\sum Q_i^2 * \Delta H_{exp} \\
\sum Q_i * \Delta H_{exp} \\
\sum \Delta H_{exp}
\end{bmatrix}
\]

where

- $Q =$ flow in liters per minute
- $N =$ number of test heights used
- $a, b, c =$ coefficients
- $\Delta H_{exp} =$ experimental head losses within the system (meters)

Using the results from the calibration, the head losses can be plotted against flow. The result is the quadratic function known as the calibration curve. The calibration curve allows the operator to know the head losses at any given flow. The head losses are then easily incorporated to report the results in the form of $Q/H'$. An example of a calibration curve is shown in Figure 2.19.
This calibration can also be conducted using a graphing program such as Excel. By graphing the flow and experimental head losses determined by Equation 2.30 the same graph as seen in Figure 2.19 is produced. This calibration process was outlined prior to the currently readily available graphing programs.

The effective head, $H'$, is determined by taking into account the losses that occur throughout the system. The head of water produced by the system includes $H_e$ and the head supplied by a hydraulic system if in use. The effective head, $H'$, is then calculated by Equation 2.31.
\[ H' = H - \Delta H_{th} \]  \[2.31\]

In field testing the head of water acting on the Perméafor system is also influenced by the ground water table. The head level in field testing is a combination of the height of the water column above the test zone, the depth of the water table, and the head added from the hydraulic pumping system. The total head in the field is shown in Equation 2.32 and illustrated in Figure 2.20.

\[ H = H_e + D_w + H_p \]  \[2.32\]

where
- \( H \) = total head of water in field testing (meters)
- \( H_e \) = head of water column above the ground surface (meters)
- \( D_w \) = depth of water table (meters)
- \( H_p \) = head produced by hydraulic pump (meters)

Figure 2.20: Total Head in the Field

The total head in the field also must be corrected for losses within the system. The total head, \( H \), is corrected to the effective head, \( H' \).
2.6.4 Hydraulic Conditions in Soil for the Start of Turbulent Stage

Measurements using the Permeafor are only applicable when water flow is within the range of laminar flow. To evaluate if soil conditions are within the range to produce laminar flow it is necessary to evaluate and understand the critical velocity \(v_c\). The limit velocity where the effect of fluid inertia makes Darcy’s Law inappropriate needs to be established. Flow is concentrated around the area of the screen in the center of the probe. During traditional testing the only measured value that can be associated with velocity is flow. Velocity is a function of flow using the dimensions of the probe, shown by Equation 2.33.

\[
Q = vs
\]

where \(v = \text{velocity}\)

\(s = \text{area of screen}\)

The theoretical critical flow corresponding to the turbulence threshold in soil can be estimated using Hazen’s formula by assuming that the average diameter where flow occurs is approximately equal to the effective grain size \((D_{10})\).

The critical Reynolds number can be established as it is equal to the ratio of the inertial forces to viscosity forces, as shown in Equation 2.34.

\[
Re_c = v_c \ast \left(\frac{d}{\eta}\right)
\]

where \(Re_c = \text{critical Reynolds number}\)

\(d = \text{diameter of flow space, } D_{10} \text{ (cm)}\)

\(\eta = \text{dynamic viscosity (g/cm*sec)}\)
A typical range of critical Reynolds values for granular materials is five to ten. The initial velocity is considered laminar when \( v_c > v \). The dynamic viscosity can also be defined using Equation 3.35.

\[ \eta = J \rho \]  

where  
\( J = \) kinematic viscosity (cm\(^2\)/sec)  
\( \rho = \) specific mass (g/cm\(^3\))

Rearranging Equations 2.33 and 2.34 produces a relationship of the critical flow with known parameters of the soil to be tested and the geometry of the probe. The critical flow is shown in Equation 2.36.

\[ Q_c = \frac{(Re_c \times J \times \pi \times D \times L)}{D_{10}} \]  

The length and diameter of the probe is five centimeters and the kinematic viscosity \( 13.5 \times 10^{-3} \) cm\(^2\)/s. The critical flow for the probe is then defined by Equation 2.37.

\[ Q_c \left( \frac{cm^3}{s} \right) = 1.06 \times \left( \frac{Re_c}{D_{10}} \right) \]  

Also, the flow can be defined in conventional units of liters per minute by Equation 2.38.

\[ Q_c \left( \frac{l}{min} \right) = 0.064 \times \left( \frac{Re_c}{D_{10}} \right) \]  

These calculation steps can also be determined graphically. The flow can be established as laminar or turbulent by using the given effective grain size diameter and the critical Reynolds number. Figure 2.21 illustrates the varying critical Reynolds number as a function of critical flow and the effective diameter.
By plotting the effective grain size and the critical Reynolds number the critical flow is outlined on the y-axis. For a given effective diameter, when the Permeafor flow during testing becomes greater than the critical flow it is considered to be turbulent. Testing is not acceptable within the turbulent zone.

2.6.5 Effects of External Head above the Probe

Depending on the test site it may be practical to have a larger amount of head above the probe. The resulting measurements are not sensitive to the external head applied except in the case of high flow where the head losses can significantly affect the accuracy of the effective head. Figure 2.22 shows the logarithm of $Q/H'$ corresponding to different head levels at the same test depth in laboratory simulation tests. Three different test depths are evaluated by changing the head above the probe.
Prior to the readily available graphing programs the profiles of $Q/H'$ were reported as $\log_{10} Q/H'$. By doing so, the profiles could be presented on conventional graph paper. Currently, results are plotted with the $Q/H'$ on a log scale.

The varying head level does not seem to influence the results of the $Q/H'$ ratio. It should be noted that the head variation was conducted while the Perméafor was in the same test hole location.
2.6.6 Permeafor Correlation to Hydraulic Conductivity

The flow from testing with the Permeafor can be correlated to hydraulic conductivity. Equation 2.39 indicates a common expression of flow.

\[ Q = \left[ \frac{2\pi(l/D)}{\log(l/D + \sqrt{l^2/D^2 + 1})} \right] * k * H * D \]  

Equation [2.39]

where

- \( l \) = length of probe screen
- \( D \) = diameter of probe screen
- \( k \) = hydraulic conductivity (meters per second)
- \( H \) = head above test zone (meters)

The first term on the right hand side of Equation 2.39 is the constant \( m \) which is a shape factor based on the cavity dimensions. By substituting the shape factor into Equation 2.39 the result is Equation 2.40.

\[ Q = m * k * H' * D \]  

Equation [2.40]

The Permeafor cavity has an \( l/D \) ratio of one therefore, \( m \) is defined by Equation 2.43. Values of \( m \) as any function of \( l/D \) are outlined in Table 2.3.
Table 2.3: Variations of \( m \) Based on \( l/D \) (Ursat, 1989)

<table>
<thead>
<tr>
<th>( l/D )</th>
<th>( m )</th>
<th>Equation No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>( l/D &gt; 10 )</td>
<td>( \frac{2\pi l/D}{\log(2^{l/D})} )</td>
<td>[2.41]</td>
</tr>
<tr>
<td>( 1.2 \leq l/D \leq 10 )</td>
<td>( \frac{2\pi l/D}{\log(l/D + \sqrt{l^2/D^2 + 1})} )</td>
<td>[2.42]</td>
</tr>
<tr>
<td>( 0.3 \leq l/D &lt; 1.2 )</td>
<td>( 2\pi \sqrt{l/D + 1/4} )</td>
<td>[2.43]</td>
</tr>
<tr>
<td>( 0.3 \leq l/D &lt; 0.7 )</td>
<td>( \pi \sqrt{2^{l/D} + 1/2} )</td>
<td>[2.44]</td>
</tr>
</tbody>
</table>

By rearranging Equation 2.39 to solve for hydraulic conductivity the result is Equation 2.45. The values of \( m \) and \( D \) are constant for the probe therefore, the coefficient \( c \) is substituted for the product of the two values resulting in the right side of Equation 2.45. The coefficient \( c \) is called the pocket coefficient.

\[
k \left( \frac{m}{s} \right) = \left[ \frac{Q}{(m \ast H' \ast D)} \right] = c \ast \left( \frac{Q}{H'} \right)
\]

For the case of the French Perméafor Equation 2.43 is used for the value of \( m \) which is equal to 7.02. The diameter of the probe is 5 centimeters (0.05 meters) resulting in a value of 2.8 for the coefficient \( c \). The hydraulic conductivity is then defined by Equation 2.46.

\[
k \left( \frac{m}{s} \right) = \left( \frac{Q}{H'} \right) \ast \left( \frac{1}{(7.02 \ast 0.05)} \right) = 2.8 \left( \frac{Q}{H'} \right)
\]

[2.45]  

[2.46]
The value of hydraulic conductivity produced by Equation 2.46 is a function of the results of Perméafor testing. For the French probe the pocket coefficient remains constant.

**2.6.7 Sample Testing Measurements**

Figure 2.23 shows a typical schematic of testing with the Perméafor. The top of the water source is located above the ground surface at a distance of $H_e$. The depth of the water table is denoted as $D_w$, and the probe is located at a depth below the surface. Note that the depth is measured from the center of the screen.

![Figure 2.23: Measured Parameters in Field Testing](image)

As testing is conducted with the probe the depth and flow are recorded. For example, the water table is located at a depth of 2.8 meters below the ground surface and the water source is located 3.0 meters above the ground surface. As a sounding is conducted the results are outlined by Table 2.4.
<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Q (l/min)</th>
<th>$dH$ (m)</th>
<th>$H' = H_e + D_w - dH$ (m)</th>
<th>$Q/H'$ (m²/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>0.00</td>
<td>-0.02</td>
<td>5.82 = 3.0 + 2.8 - (-0.02)</td>
<td>0</td>
</tr>
<tr>
<td>1.0</td>
<td>1.10</td>
<td>0.02</td>
<td>5.78 = 3.0 + 2.8 - 0.02</td>
<td>3.16*10⁻⁶</td>
</tr>
<tr>
<td>2.0</td>
<td>2.48</td>
<td>0.11</td>
<td>5.69 = 3.0 + 2.8 - 0.11</td>
<td>7.24*10⁻⁶</td>
</tr>
<tr>
<td>3.0</td>
<td>0.06</td>
<td>-0.02</td>
<td>5.82 = 3.0 + 2.8 - (-0.02)</td>
<td>1.58*10⁻⁷</td>
</tr>
<tr>
<td>4.0</td>
<td>1.65</td>
<td>0.05</td>
<td>5.75 = 3.0 + 2.8 - 0.05</td>
<td>4.79*10⁻⁶</td>
</tr>
</tbody>
</table>

Table 2.4: Example of Calculated $Q/H'$ with Depth (Ursat, 1989)

The hydraulic conductivity can then be calculated using the pocket coefficient. The hydraulic conductivity is calculated for each Permeafor test depth in Table 2.5.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>$Q/H'$ (m²/s)</th>
<th>$k = 2.849 \times Q/H'$ (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1.0</td>
<td>3.16*10⁻⁶</td>
<td>7.87*10⁻⁴</td>
</tr>
<tr>
<td>2.0</td>
<td>7.24*10⁻⁶</td>
<td>1.81*10⁻³</td>
</tr>
<tr>
<td>3.0</td>
<td>1.58*10⁻⁷</td>
<td>3.93*10⁻⁵</td>
</tr>
<tr>
<td>4.0</td>
<td>4.79*10⁻⁶</td>
<td>1.19*10⁻³</td>
</tr>
</tbody>
</table>

Table 2.5: Example of Calculated Hydraulic Conductivity with Depth

Using the Permeafor produces results of the hydraulic conductivity with depth as a function of four measured values. Only the height of the water source, depth of water table, depth of probe testing and the flow at each test depth are needed. The Permeafor provides a quick and easy test to measure in situ hydraulic conductivity.
2.6.8 Scale Model Perméafor testing

Over the summer of 2009 a scale model of the Perméafor was created for use in shallow field applications. The scale model works on the same principles as the full scale French Perméafor. The scale probe is approximately 39 centimeters in length and has a test zone of 2.8 centimeters. The taper in the probe produces the smallest diameter of 2.5 centimeters at the tip and the largest diameter at the rear of 3.9 centimeters.

Julien Torterat developed and implemented a laboratory testing program for the Perméafor model. Throughout the summer Torterat developed methods to create soil specimens, supply water for the probe and create an instrument to advance the probe through the soil specimens.

Some laboratory tests were conducted over the course of the summer of 2009 by Julien Torterat. The laboratory tests were conducted in a 55 gallon test tank compacted to varying densities. The soil was compacted using a 5, 7 and 10 lifts. Torterat’s results are presented in Figure 2.24.
More laboratory tests are needed to evaluate the half scale probe and the feasibility of its use in determining the hydraulic conductivity in shallow applications.

2.7 Summary

Hydraulic conductivity is a function of several soil properties. A single empirical correlation cannot accurately define the hydraulic conductivity for a wide variety of soil types. For this reason, in situ testing is essential in the evaluation of any site where hydraulic conductivity is a major design aspect.
Current practices for evaluating hydraulic conductivity are costly and do not give an accurate depiction of the changing permeable layers with depth. The Perméafor is a new technology that will aid in the evaluation of hydraulic conductivity in a relatively fast and inexpensive manner. To fully understand the applications and advantages of the Perméafor, a multitude of tests are needed to establish a baseline of data.
CHAPTER 3

3. MATERIALS AND METHODS

3.1 Introduction

The Permeafor was evaluated in the laboratory in a testing chamber filled with sand acquired from an abandoned sand borrow located in Lee, New Hampshire. Field testing was performed at a site immediately adjacent to the borrow area with the same soil conditions.

3.2 Lee Sand

An abandoned sand borrow, located three miles from the University of New Hampshire in the town of Lee, NH, allowed for ample sand supply and provides an ideal location for field testing. Several preliminary laboratory soil tests were completed on the Lee sand prior to testing with the Permeafor probe. A site map of the Lee sand borrow is shown in Figure 3.1.
3.2.1 Gradation

Sieve analyses were completed to evaluate the gradation of the Lee sand. Testing was conducted following ASTM Standard D 6913-04. Three tests using different samples were carried out on the Lee sand. The result of these sieve analyses are shown in Figure 3.2.
Figure 3.2: Soil Gradation Curves for Lee Sand

The uniformity coefficient and the coefficient of gradation were evaluated for the three gradation curves and the results are shown in Table 3.1.

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$D_{10}$ (mm)</td>
<td>0.09 ± 0.002</td>
</tr>
<tr>
<td>$D_{30}$ (mm)</td>
<td>0.19 ± 0.009</td>
</tr>
<tr>
<td>$D_{60}$ (mm)</td>
<td>0.29 ± 0.009</td>
</tr>
<tr>
<td>Uniformity Coefficient, $C_u = \frac{D_{60}}{D_{10}}$</td>
<td>3.07 ± 0.09</td>
</tr>
<tr>
<td>Coefficient of Gradation, $C_c = \frac{D_{30}^2}{(D_{60} \times D_{10})}$</td>
<td>1.40 ± 0.09</td>
</tr>
</tbody>
</table>

Table 3.1: Results of Lee Sand Sieve Analysis
Using these results the Lee sand can be classified as SP, a poorly graded sand with trace gravel, using the Unified Soil Classification System. Under the American Association of State Highway and Transportation Officials (AASHTO) Classification System the Lee sand is classified as group A-3.

3.2.2 Compaction

Both the Standard and Modified Proctor Compaction tests were conducted on the Lee sand following ASTM Standard D 698 and D 1557-02, respectively. Compaction curves were created to determine the maximum dry unit weight and the optimum moisture content. The results of the soil compaction test are shown in Figure 3.3.

In the standard Proctor compaction test, the dry unit weight remained nearly constant ranging from 104.9 to 105.5 lb/ft$^3$. As the moisture content reached about 12%, the sample was unable to retain water as it began to leak out of the bottom of the Proctor mold. Due to the small variation in the dry unit weight, the optimum moisture content is not apparent.

The results from the modified Proctor test show a maximum dry unit weight of 113.0 lb/ft$^3$ at an optimum moisture content of about 11%. Similarly to the standard Proctor test, the mold began to leak at a moisture content of 12%. An o-ring was used to attempt to prevent the Proctor mold from leaking excessively.
Both the standard and modified Proctor compaction tests did not exhibit a well defined peak at the optimum moisture content. There are several factors that affect the compaction of soil. The level of compaction is a function of the molding water content, compaction effort and the type of soil. The grain size distribution, soil shape and specific gravity affect the maximum dry unit weight and optimum moisture content.

The Lee sand is a uniform fine grained soil. Typically, uniformly graded soils do not respond well to compaction as all the soil particles are the same size. Based on the USCS soil classification system the Lee sand has an average
maximum dry unit weight of 110-120 pounds per cubic foot. AASHTO supplies a range of 110-115 pounds per cubic foot and an optimum moisture content range of 9-15 percent for the Lee sand (McCarthy, 2002). Using the standard Proctor test the maximum dry unit weight is slightly below the expected results provided by the literature.

### 3.2.3 Minimum and Maximum Void Ratio

There are several methods to determine both the minimum and maximum void ratio. In this research the Japanese and ASTM D4254-00 (2006) Standard Test Methods for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density methods were used. A full explanation of the Japanese test method is outlined in Appendix A. The minimum and maximum void ratios are used to evaluate the relative density of all laboratory tests tank samples using Equation 3.1.

\[
D_r = \left( \frac{e_{\text{max}} - e_o}{e_{\text{max}} - e_{\text{min}}} \right) \times 100
\]  

[3.1]

where  

- \( e_{\text{max}} \) = highest void ratio possible for the soil  
- \( e_{\text{min}} \) = lowest void ratio possible for the soil  
- \( e_o \) = void ratio of soil in place

For the Japanese method, the initial guidelines call for sand that has 100% passing the number 10 sieve, and 95% retained on the 200 sieve. Throughout three different sieve tests of the Lee sand, an average of 99.1% passed the number 10 sieve and an average of 95.4% was retained on the number 200 sieve. This implies that there was a small amount of large pea size
gravel in the soil sample. To modify the sand to fit in the given testing criteria these gravel size particles were removed. With the modified soil the minimum and maximum void ratios were determined as 0.57 and 0.97, respectively. These results are typical of fine grained sand.

To remove the issues that stem from the small mold size in the Japanese method the ASTM method was also used to determine the minimum and maximum void ratio. The test is conducted in exactly the same manner as the Japanese method the only difference being that the mold is a standard Proctor mold. The large volume of the Proctor mold eliminates the need to modify the soil sample before conducting the test. The ASTM method produced similar results as the Japanese method of a minimum void ratio of 0.56 and a maximum of 0.97. The similarities of the results prove that the small manipulation to the soil did not affect the final product.

3.2.4 Hydraulic Conductivity

Hydraulic conductivity tests were conducted on the Lee sand using samples prepared by the standard and modified Proctor methods outlined by ASTM Standard D 698, and ASTM Standard D 1557-02, respectively. Testing involved preparing twelve Proctor molds, six using the standard and six using the modified hammer. For each mold a constant head and falling head test was performed following the outlined standards.

The results of the hydraulic conductivity testing are shown in a graphical manner in Figure 3.4. The upper section of the figure represents the falling head and constant head tests for both the standard and modified Proctor. As
expected, the results from the modified Proctor produced a lower hydraulic conductivity consistent with the higher compaction energy and thus the increased compaction. For the standard Proctor the results also show a decreasing hydraulic conductivity with increasing molding water content. The modified Proctor on the other hand, tends to show a more linear trend where the hydraulic conductivity does not vary with the molding water content.

One outlier exists for the modified Proctor at approximately five percent molding water content. This could be due to insufficient saturation prior to testing. Another possibility could be due to the location of the moisture content sample. The moisture content is taken by using the soil from the trimmings of the mold. These are not necessarily an accurate representation of the moisture content in the sample as a whole. A more accurate place to measure moisture content is at the center of the sample, but that would require destroying the sample and then not allowing for hydraulic conductivity testing. The moisture content has a direct correlation to the dry density such that a reduced measurement of moisture content will increase the measured dry density.

The standard Proctor hydraulic conductivity test produced a range from 1.16E-03 to 3.96E-03 centimeters per second. The modified Proctor hydraulic conductivity test ranges from 1.21E-03 to 5.05E-04 centimeters per second. Typical values for hydraulic conductivity are outlined by soil types. Clean sand and sand and gravel mixtures range in hydraulic conductivity from 100 to $10^{-3}$ centimeters per second. Fine sand and silts have a typical hydraulic conductivity ranging from $10^{-3}$ to $10^{-5}$ cm/s (McCarthy, 2002). The hydraulic conductivity of
the fine grained Lee sand ranges from $10^{-3}$ to $10^{-4}$ cm/s which are within the typical range for fine sand and silt.

The bottom graph in Figure 3.4 shows the results of the compaction test for each Proctor mold. Neither the standard or modified produce a typical compaction curve where a peak is reached before it starts to decrease. For both compaction efforts the optimum moisture content and maximum dry unit weight are not obvious. In addition, there is little variation of dry unit weight with the increasing moisture content. The compaction tests completed following ASTM Standards D 698 and D 1557 are also plotted on Figure 3.4 for comparison. For those tests, hydraulic conductivity testing was not carried out.

The standard Proctor molds for both the Proctor test and hydraulic conductivity test have similar dry densities in the molding water content range of 2 to 8 percent. Beyond 8 percent water the hydraulic conductivity molds begin to have a higher dry density as the molding water content increments are the same as the standard Proctor test. The increasing dry density in the hydraulic conductivity testing implies that the optimum moisture content was never achieved. The standard Proctor test produces a linear trend, as molding water increases the dry unit weight remains constant, which created the assumption that the optimum moisture content was passed during testing.
Figure 3.4: LEE Sand (Top) Hydraulic Conductivity Test, (Bottom) Compaction Test
The modified Proctor test produces a more traditional curve with a more defined optimum moisture content at 11 percent and a maximum dry density of 113 pounds per cubic foot. The modified Proctor hydraulic conductivity testing is a reverse of the expected curve. The maximum dry density of 113 pounds per cubic foot occurs at a molding water content of 4.7 percent. This point corresponds to the outlier shown on the hydraulic conductivity curve in Figure 3.4. From this peak point, the compaction curve begins to decrease. The molding water content for the modified Proctor hydraulic conductivity testing only reached a maximum of approximately 9 percent. More tests past this moisture content were not conducted due to the linear fit of the hydraulic conductivity with varying molding water content.

Traditionally, after the optimum moisture content has been reached the hydraulic conductivity curve begins to flatten. Figure 3.5 illustrates the decreasing hydraulic conductivity with increasing molding water content. Once the curve passes the optimum water content, labeled by the grey circles, the hydraulic conductivity values begin to level off or for low energy compaction may actually start increasing again.
The optimum water content for the standard Proctor is not apparent from the compaction test. The molds created during hydraulic conductivity testing only reached 12% as they began to leak during compaction. The hydraulic conductivity curve is expected to taper and level off after the 12% molding water content. Due to the insufficient number of testing molds, the curve continues to decrease never having a leveled off portion. The modified Proctor hydraulic conductivity tests on the other hand only had a level portion of the expected trend with the exception of the one outlier.
3.3 Perméafor Model

Over the summer of 2009 a scale model of the Perméafor was created for use in shallow field applications. The scale model works on the same principles as the full scale French Perméafor. In order to optimize the tool the probe was created in several interchangeable sections. The scale probe is approximately 39 centimeters in length, an approximate diameter of 3 centimeters and has a test zone of 2.8 centimeters. The taper in the probe produces the smallest diameter of 2.5 centimeters at the tip and the largest diameter at the rear of 3.9 centimeters. In the rear of the probe a Swagelok connection allows for the attachment of ½ inch flexible tubing which supplies water from the surface. Figure 3.6 illustrates the full size French Perméafor next to the disassembled scale model.

Similarly to the French Perméafor, the diameter above the screen section in the scale model expands the surrounding soil to a pressure corresponding to what is required to reach the limit pressure of the soil. In other words, the diameter of the upper section of the probe is such that the volume is double that of the lower probe section. The $L/D$ ratio of the screen section is one, and the tip has an apex of 60 degrees.
3.3.1 Hydraulic Conditions in Soil for the Start of Turbulent Stage

Measurements using the Perméafor are only applicable when water flow is within the range of laminar flow. During traditional testing the only measured value that can be associated with velocity is flow. The theoretical critical flow corresponding to the turbulence threshold in soil is estimated using the Hazen’s formula by assuming that the average diameter where flow occurs is approximately equal to the effective grain size ($D_{10}$, cm). The critical flow for the half scale model is shown in Equation 3.2.
The flow can be established as laminar or turbulent by using the given effective grain size diameter and the critical Reynolds number. The effective diameter is 0.009 centimeters. Critical Reynolds number ranges from 5 to 10 for granular materials. Figure 3.7 illustrates the varying critical Reynolds number as a function of critical flow and the effective diameter for the half scale model.

\[
Q_c \left( \frac{i}{i_{\text{min}}} \right) = 0.02 \left( \frac{Re_c}{D_{10}} \right)
\]  

[3.2]
When the Permeafor flow during testing becomes greater than the critical flow it is considered to be turbulent. For the half scale model the critical flow is 11.1 liters per minute when the Reynolds number is equal to five. If the Reynolds number is assumed to be ten the critical flow is 22.2 liters per minute. In all laboratory and field testing flow never exceeded 4 liters per minute therefore, flow is always within the boundary of laminar flow.

3.3.2 Pocket Coefficient

As previously discussed in Section 2.6.6, to estimate the hydraulic conductivity while using the half scale model a coefficient must be determined relating the $Q/H'$ ratio to the hydraulic conductivity. This coefficient, $m$, is in essence a shape factor and is a function of the screen dimensions. The screen section has an $l/D$ ratio of one resulting in the coefficient $m$ to be evaluated using the previously defined Equation 2.43.

$$m = 2\pi \sqrt{\left(\frac{l}{D}\right) + \frac{1}{4}} = 7.02$$ [2.43]

The pocket coefficient $c$ is then calculated by Equation 3.3.

$$c = \frac{1}{(m * D)} = \frac{1}{(7.02 * 0.03)} = 4.75$$ [3.3]

The hydraulic conductivity for all Permeafor testing with the half scale model can then be estimated using Equation 3.4.

$$k \left(\frac{m}{s}\right) = 4.75 \ast \left(\frac{Q}{H'}\right)$$ [3.4]
3.4 Permeafor Laboratory Tank Testing

After characterizing the Lee sand through standard laboratory testing, the Permeafor tool was evaluated through a series of tests in a 55 gallon test tank.

3.4.1 Laboratory Layout

The scale Permeafor was tested in the laboratory using the setup shown in Figure 3.8. The system consists of a water source, a flow meter, flexible tubing and the test tank. A hydrometer bath is used for water supply as it allows for ample volume and it is also easily modified to connect for Permeafor testing. The bath has a length of 0.9 meters, height of 0.5 meters, and width of 0.2 meters producing a total volume of 0.09 cubic meters.

The hydrometer bath is open at the top which provides for static head only. There is no hydraulic pump to add additional head to the system. A length of 6.15 meters of tubing then connects to the flow meter. The flow meter is a Gilmont Model E floating ball flow meter. The meter can read a range of flows from 1 to 9 liters per minute. From the flow meter a length of 2.95 meters of tubing is then connected to the water source.
The hydrometer bath is continuously supplied water from the laboratory sink. To maintain the constant head level needed, an overflow valve at the top of the bath allows water to return to the sink when the bath is full. In Figure 3.9 the overflow system is shown.
The bath is connected to a flow meter using 0.5 inch outer diameter 0.0623 inner diameter flexible nylon tubing. The Permeafor probe is driven into a compacted soil sample using a falling weight system part of the dynamic cone penetrometer tool. Drill rods are attached to the probe with the nylon tubing running through the rods to a split piece that connects to the dynamic cone hammer. The split piece allows for the connection of the hammer system with a steel pin while also allowing for the tubing to not become pinched from the falling weight. Figure 3.10 shows the split piece attached to the dynamic cone hammer.
Figure 3.10: Split Piece Connector for Dynamic Cone Hammer

The laboratory testing setup allows for the evaluation of the flow of water through soils in a similar manner to the full scale method. The test tank is a 55 gallon polyethylene drum that holds a total of 0.192 cubic meters of compacted soil. On the inside at the base of the tank a wooden platform is used to elevate the soil from the bottom of the drum. This platform has several drilled holes to promote the percolation of water through the sample. A geotextile is placed on top of the wooden platform to ensure that the soil is maintained within the testing area.
The test tank size is sufficient such that the sides do not interfere with the flow conditions. During testing water from the probe percolates horizontally but never reaches the tank wall before the probe is advanced to the next test depth.

3.4.2 Laboratory Sample Preparation

Each specimen was compacted at different relative densities. Three laboratory specimens of Lee sand were prepared in the test tank for evaluating the Perméafor. These include one sample that is created in five lifts, the next in seven lifts, and the final in ten lifts.

The 55 gallon polyethylene tank has been modified with two water faucet taps at the base and top which are used during the saturation process. The bottom tap allows a water source to saturate the sample from the bottom and the top tap is used as an overflow.

During the creation of the soil specimen each lift is approximately the same amount of soil. To ensure that each lift is consistent the tank is divided and labeled as shown in Figure 3.11. The five lift specimen is labeled with a black line around the tank, the seven lift specimen is orange, and the ten lift specimen is green.
Soil Denominations:
Black: 5 Layers
Orange: 7 Layers
Green: 10 Layers

Soil Sample:
0.192 cubic meters

Figure 3.11: Soil Layer Labeling of the Test Tank

After each lift is placed in the test tank the soil is compacted. To achieve proper compaction a Light Weight Falling Deflectometer (LWFD) hammer was used as the energy source. The system used is a Zorn 2000 LWFD designed for measuring the modulus of elasticity in pavement design. The Zorn Deflectometer is composed of a 10 kilogram sliding drop weight that falls a distance of 72 centimeters on a guide rod. The LWFD rod and weight is connected to a 300 millimeter loading plate of which the energy is transmitted through the soil. A schematic of the Zorn LWFD is shown in Figure 3.12.
Using the LWFD, compaction is achieved by dropping the 10 Kg weight around the specimen surface ten times for each soil lift. When soil is added for each lift it is made level and then compacted. The LWFD weight is allowed to fall from the full 72 cm for each drop. As the weight slides down the guide rod to the base plate it is allowed to bounce, due to the steel springs, until it comes to rest at the top of the rod.

The energy of compaction, a function of the number of lifts, blows per lift, the weight of the hammer, the drop height, and the volume of the specimen mold is calculated for each specimen using Equation 2.14. The energy of compaction for each specimen is shown in Table 3.2.
A dynamic cone penetrometer was used to measure the soil resistance to penetration in an attempt to correlate the energy of compaction to the density of the soil.

The Dynamic Cone Penetrometer (DCP) was developed by Professor George Sowers in 1959. The purpose of the DCP is for field exploration and the evaluation of lightly loaded shallow spread footings during the construction phase (Durham Geo Slope Indicator). The tool itself is portable and can be used in locations where a drill rig would not have access. Due to the lightweight construction of the tool, it is only applicable for shallow applications.

The Dynamic Cone Penetrometer consists of a 6.8 kilogram (15 pound) steel donut shaped mass falling a distance of 50.8 centimeters (20 inches) to strike an anvil. The falling action of the weight is used to penetrate a 3.81 centimeters (1.5 inch) diameter rod with a 45 degree cone. The cone is placed at the bottom of a hand augered hole. Similarly to the SPT test, the number of blows are recorded for a certain standard penetration distance. Figure 3.13 shows the hammer system that attaches to rods to drive the cone tip into the subsurface.

<table>
<thead>
<tr>
<th>Soil Specimen</th>
<th>Compaction Energy (kJ/m³)</th>
<th>Compaction Energy (ft-lbf/ft³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5 Layer</td>
<td>1875</td>
<td>384</td>
</tr>
<tr>
<td>7 Layer</td>
<td>2625</td>
<td>537</td>
</tr>
<tr>
<td>10 Layer</td>
<td>3750</td>
<td>767</td>
</tr>
</tbody>
</table>

Table 3.2: Energy of Compaction for Each Soil Sample
Testing is performed from the bottom of a hand augered hole which is generally 7.62 to 15.24 centimeters (3 to 6 inches) in diameter. The bottom of the hole must be cleaned of all the cuttings. The sliding hammer system with attached rods and cone are lowered into the hole. The tip of the 5.08 centimeters (2 inch) cone is inserted and embedded fully into the bottom of the undisturbed hole. The system must remain plumb throughout testing. The cone is then driven into the subsurface by raising and dropping the donut hammer the full height of 50.8 centimeters (20 inches). The number of blow it takes to advance the cone tip 44 millimeters (1 3/4 inches) is recorded. A second and third penetration of the same distance can also be done. Beyond three penetration increments the skin friction on the rods may become significant. The DCP system is then removed from the hole, augering proceeds to the next test depth and the procedure is repeated. The results of the DCP test can then be correlated to standard penetration numbers.
In the laboratory it is not practical to auger the sand specimen as it would just collapse. The testing procedure was modified such that the penetration was recorded every five centimeters. The system was not removed until the cone was pushed the full depth of 70 centimeters into the sample specimen. Figure 3.14 shows the Dynamic Cone Penetrometer during a laboratory test.

![Dynamic Cone Penetrometer Test in Laboratory Specimen](image)

Figure 3.14: Dynamic Cone Penetrometer Test in Laboratory Specimen

DCP testing was conducted for each level of compaction. The results of the DCP tests for the five, seven, and ten lift soil samples are shown in Figure 3.15.
As expected the ten lift soil specimen required the largest number of blows to advance the tool. For the top 20 centimeters all three soil samples tended to have about the same resistance. The cone was pressed into the soil under the weight of the rods and hammer system over five centimeters for each test. Between 40 and 70 centimeters the soil samples have the most variation. As layers of soil are added the compaction effort is dispersed through the entire soil specimen. For this reason, the bottom layers will be denser than the top in addition to having more skin friction on the rods.
To evaluate the density and moisture content, rings were placed into the sample during the preparation and compaction. Three locations throughout the specimen were chosen to place rings for testing. The locations were based on the result of the Dynamic Cone Penetrometer testing. The rings are placed at depths of 40, 50, and 60 centimeters from the top of the sample tank. Three rings are placed at each depth around the center of the soil specimen. Permeafor testing is conducted in the center of the soil sample; therefore, the rings do not interfere with the probe testing.

After testing with the Permeafor, the rings are carefully removed and weighed to determine soil properties. Some rings may become compromised as they are removed, for this reason, there are three rings located at each test depth. The rings have a diameter of 7.37 centimeters (2.9 inches), and a height of 3.05 centimeters (1.2 inches). Figure 3.16 illustrated the location of the rings in the test specimen and the plan view of the rings at each test layer.

The purpose of the rings is to measure the soil properties of the sample with depth. The rings are then removed, weighed, dried, and weighed again. With this data the moist unit weight, dry unit weight, moisture content, void ratio, porosity, degree of saturation, and relative density are determined for each ring.
3.4.3 Determination of Head Losses within the System

The calibration of the Perméafor begins with the determination of $H_e$ and $H_c$ values as previously discussed in Section 2.6.3. The $H_e$ is the height from the top of the water source to the top of the soil tank or ground surface. $H_c$ is the height of water ejected from the flexible tubing under each head $H_e$. The calibration readings are shown in Figure 3.17.
A minimum of four locations are chosen to measure the head loss. With a larger head above the test zone the flow will be greater as the hydraulic head is increased. By increasing $H_e$, both the $H_c$ and the flow will increase. The measurements at the four locations are shown in Table 3.3.

<table>
<thead>
<tr>
<th>Location</th>
<th>$H_e$ (cm)</th>
<th>$H_c$ (cm)</th>
<th>Flow (l/min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>235.9</td>
<td>5.08</td>
<td>4.60</td>
</tr>
<tr>
<td>2</td>
<td>200.4</td>
<td>4.45</td>
<td>4.20</td>
</tr>
<tr>
<td>3</td>
<td>181.8</td>
<td>3.81</td>
<td>3.95</td>
</tr>
<tr>
<td>4</td>
<td>145.1</td>
<td>3.18</td>
<td>3.50</td>
</tr>
</tbody>
</table>

Table 3.3: Laboratory Head Loss Parameters

For each test location the experimental head losses are determined using Equation 3.5. In the laboratory setting a hydraulic pump is not used to create
additional head within the system, and therefore $H_p$ was not included in the determination of the experimental head losses.

$$\Delta H_{\text{exp}} = H_e - H_c + H_p$$  \[3.5\]

As clearly shown, the experimental head losses are a function of both $H_e$ and $H_c$. The determination of the head losses as a function of flow is performed by solving the Gauss matrix shown in Equation 3.6. By inverting the matrix and solving for the coefficients $a, b, c$, the resulting equation describes head loss as a function of flow.

$$\begin{bmatrix} \sum Q_i^4 & \sum Q_i^3 & \sum Q_i^2 \\ \sum Q_i^3 & \sum Q_i^2 & \sum Q_i \\ \sum Q_i^2 & \sum Q_i & N \end{bmatrix} \begin{bmatrix} a \\ b \\ c \end{bmatrix} = \begin{bmatrix} \sum Q_i^2 \Delta H_{\text{exp}} \\ \sum Q_i \Delta H_{\text{exp}} \\ \sum \Delta H_{\text{exp}} \end{bmatrix}$$  \[3.6\]

By taking the measurements of $H_e$, $H_c$ and flow, all additional parameters can be solved. Table 3.4 outlines all measured and calculated parameters used to determine the head losses in the laboratory setup which includes the flow meter.
The parameters outlined in Table 3.4 are then entered into the Gauss matrix as shown in Equation 3.7.

\[
\begin{bmatrix}
1152.4 & 275.93 & 66.65 \\
275.93 & 66.65 & 16.25 \\
66.65 & 16.25 & 4
\end{bmatrix} \begin{bmatrix}
a \\ b \\ c
\end{bmatrix} = \begin{bmatrix}
128.56 \\ 30.84 \\ 7.47
\end{bmatrix} \tag{3.7}
\]

The final result of the Gauss matrix for the laboratory Perméafor testing is shown in Equation 3.8.

\[
\Delta H_{HT} = 0.049Q^2 + 0.406Q - 0.601 \tag{3.8}
\]

where \( a = 0.049 \)

\( b = 0.406 \)

\( c = 0.601 \)

The head losses can also be evaluated in a graphical manner. To understand the head losses that are attributed to the flow meter, the calibration can be conducted without the flow meter. For comparison purposes Figure 3.18 shows the reported head losses in the French system with the flow meter and 38 meters of tubing in conjunction with the results from the laboratory head losses...
for both with and without the flow meter. It should be noted that the French
system uses three different flow meters which are used based on the amount of
flow expected. Their flow meters allow measurement increments of 0 to 5.4, 5 to
24 and 15 to 74 liters per minute. The head losses in Figure 3.18 are
representative of the low range flow meter.

\[
\Delta H_{HT} = 0.049Q^2 + 0.406Q - 0.601
\]

Figure 3.18: Head Losses as a Function of Flow with and without the Flow Meter
in Conjunction with the Reported French Head Losses

Flow is restricted in the small flexible tubing and increasing the flow
increases the friction which causes greater head loss. The flow meter creates an
increase in head loss from 0.25 to 0.75 meters. Typical flow readings during
testing range from 2.0 to 3.5 liter per minute creating an average range of head loss of 0.4 to 1.5 meters. The French system uses flexible tubing significantly larger and longer compared to the half-scale model. The large tubing does not cause high amounts of friction to cause the head losses.

The determination of head losses were originally outlined prior to the use of graphing programs. The resulting graph of head loss versus flow can also be determined in an easier fashion by using Excel or any similar graphing program. Using a second order polynomial fit for the experimental head losses and flow produces the same result as the Gauss matrix.

The results of these calibrations allows for each Permeafor test to be corrected for head losses. The effective head $H'$ is determined using Equation 3.9.

$$H' = H - \Delta H_{th}$$  \hspace{1cm} [3.9]

Final results using the Permeafor are reported in terms of $Q/H'$. The flow recorded during testing in cubic meters per minute and the effective head in meters describe the relative hydraulic conductivity as a function of depth.

**3.4.4 Laboratory Permeafor Testing**

Prior to testing the Permeafor probe is fully assembled and connected to the flexible tubing. For these tests, one drill rod is sufficient. All of the tubing and the flow meter must be purged of air bubbles for proper operation. Throughout testing the water is allowed to continuously flow. If the water is stopped while the probe is in the soil particles will migrate into the perforated screen causing clogging and thus restricting future flow. The probe, rod, and hammer assembly
are then placed in the center of the soil sample. The weight of the 21.6 kilogram (47.6 lbs) system initially sinks into the soil sample. The amount of penetration is measured along with the subsequent number of blows needed to advance the probe every five centimeters.

The system is then allowed to progress into the soil until it reaches the bottom of the tank. The primary objective of the Permeafor is to measure the lateral flow of water through the soil. During initial penetration, water has a tendency to partially flow up the drill rod to the surface of the soil sample. The probe is advanced into the soil sample and flow readings are recorded every five centimeters.

In the full scale Permeafor, tests are conducted every ten to twenty centimeters. Due to the size of the soil sample in laboratory testing penetration was stopped every five centimeters after vertical flow was no longer present. The test zones ranged from 35 to 55 centimeters below the top of the soil sample. The test zone corresponds to the depth of the center of the screen section below the surface of the soil. At these points the flow is recorded for a period of time, but at least for ten seconds as outlined by the French investigators.

When the Permeafor is at the final test depth the soil is saturated by slowly percolating water through the sample from the bottom upward. Once the soil has had time to saturate the flow at the final test depth is recorded once more.
3.5 Permeafor Field Testing

The testing conducted in the field for this research is the first field testing with the half scale Permeafor. After a series of laboratory testing, the field testing is aimed to evaluate the applicability of the half-scale Permeafor and to modify the probe and testing procedures as appropriate.

3.5.1 Field Testing Location

The site chosen for field testing with the Permeafor directly abuts the Lee sand borrow. The site is at the home of Dr. Jean Benoît which allows for the availability of ample water supply. Figure 3.19 shows the location of the test site with respects to the sand borrow.

Figure 3.19: Test Site Location for Field Testing
3.5.2 Field Testing Setup

The setup of the field testing is similar to that of laboratory testing. A water source (garden hose) supplies a 55 gallon tank allowing for a constant head above the probe. The 55 gallon tank has five removable fittings which allow for variation of head level within the bucket. At the base of the bucket a quick connect fitting attaches to the $\frac{1}{4}$ inch flexible tubing. The water source bucket is shown in Figure 3.20 illustrating the five locations for variable head levels.

![Figure 3.20: Water Source with Variable Head Levels for Field Testing](image)

Figure 3.21 shows the field Permeafor testing setup. The water source, with the second port open for overflow, sits on top of scaffolding at a height of
five feet above the ground surface. The total head above the probe is 1.9 meters before penetration into the ground. The flexible tubing is then connected to the same flow meter used for laboratory testing. A length of 15.2 meters of tubing is then connected to the Perméafor from the flow meter. Drill rods are pre-strung with the flexible tubing such that a total of over three meters of testing can be conducted.

![Perméafor Field Testing Setup](image)

Figure 3.21: Perméafor Field Testing Setup

The probe is pushed using a portable hydraulic frame or driven using the sliding weight from the dynamic cone penetrometer. Pictured in Figure 3.21 is the portable hydraulic rig which can provide up to seven tons of pushing pressure. To counteract the pushing force several 55 gallon buckets filled with
water are used to hold the rig down. The buckets sit on two \( \frac{3}{4} \) inch aluminum sheets weighing approximately 50 pounds. These sheets are placed on the H-Frame at the base of the rig. Testing was also conducted with the sliding hammer from the dynamic cone penetrometer which was accomplished in the same manner as in the laboratory testing.

3.5.3 Soil Profile

The reason for performing testing adjacent to the sand borrow was to test the same material as used in the laboratory. Using a hand auger the test site was profiled to determine if the field conditions matched the samples collected for laboratory testing. Samples were collected every 30 centimeters to perform sieve analysis and moisture content testing. The results of the sieve analysis for all samples are shown in Figure 3.22.
Figure 3.22: Sieve Analysis Results from Field Samples

The sieve analysis from the borrow sample used in laboratory testing is also shown in Figure 3.22. The grain-size curve is similar to the soil used in the laboratory testing. Large gravel size particles were seen at depths of 30 and 290 centimeters. Laboratory testing was conducted on sand that had all large gravel removed. Other than the large gravel particles, the sand in the laboratory and the field are the same. The results of the moisture content for each depth are shown in Figure 3.23.
To understand the relative density of the soil profile with depth in the field, a dynamic cone penetration test was conducted. The results of the dynamic cone penetration test for the field and laboratory samples are shown in Figure 3.24. The top 40 centimeters of the field soil profile is denser than any laboratory test sample. Between 40 and 75 centimeters the field and 7 layer soil sample have similar densities. After 40 centimeters the 10 layer sample is denser than the field conditions. The five layer sample is significantly less than the field dynamic cone results. Below 120 centimeters the field density increases rapidly.
As the tip of the cone advances further into the soil skin friction becomes a factor. For this reason, the hole is augered to depth and testing is resumed. The hole was augered at depths of 85, 110, 140 and 160 centimeters. The first blow counts after the augering process produced lower results than the previous depth. This is in part due to the augering process as loose softer soil remained in the hole causing a lower density and due to the lack of skin friction on the rods above the tip.
3.5.4 Determination of Head Losses

The head losses for the field testing are determined in the same manner as the laboratory testing. The results of the head losses for the field and laboratory setup are shown in Figure 3.25.

![Figure 3.25: Head Losses for Field and Laboratory Testing Setup](image)

It should be noted that this method produces a fit of the data measured. For example, the laboratory head losses were calculated using the measured flow values ranging from 3.5 to 4.7 liters per minute. The quadratic fit is an approximation of the expected head losses for flows beyond this range.
Therefore, flows experienced below 3.5 l/min the head losses are simply estimation. The same issue is seen for flows exceeding 4.7 l/min.

There are variations in head losses between the field and laboratory. This is due to an additional nine meters of tubing using in the field. More tubing is needed in the field as several drill rods are used. The water source remains in one locations and there needs to be ample slack in the tubing to be able to maneuver the probe to different locations on the site. With all parameters established, testing can begin with the Perméafor probe.
4. RESULTS

4.1 Introduction

This chapter presents the results of Permeafor laboratory and field tests conducted on a uniform fine-grained sand from Lee, New Hampshire. Test results are discussed and presented graphically showing the variation of \( \frac{Q}{H'} \) with depth for tests conducted in a soil test tank and in the field. In addition, the results are compared to previous tests using the reduced size probe, tests conducted using the French probe and other laboratory hydraulic conductivity tests.

4.2 Laboratory Testing Results

Laboratory testing was conducted in a test tank on the three soil samples varying in relative density. Medium dense to dense samples were created in a 55 gallon drum used for Permeafor laboratory testing. The varying densities were created by increasing the number of soil lifts within the testing drum. The samples were created using five, seven and ten soil lifts as the compaction energy for each layer remained constant. The results of the laboratory tests are discussed in the following section.
4.2.1 Measured and Calculated Testing Parameters

Tests conducted with the Perméafor result in a combination of both measured and calculated parameters that are used to illustrate the changing relative hydraulic conductivity with depth. A single test uses pressure head to propel water into the surrounding soil. The parameters of each test include the flow, pressure head and the depth of the probe. The depth of the probe in the sample is measured from the center of the test screen to the top of the soil sample. From these measurements additional parameters are calculated. Table 4.1 lists all relevant parameters needed for testing conducted with the Perméafor.

<table>
<thead>
<tr>
<th>Measured Parameters</th>
<th>Calculated Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flow, ( Q ) (L/min)</td>
<td>Corrected Head, ( H' = H - \Delta H ) (m)</td>
</tr>
<tr>
<td>Head Above Test Zone, ( H ) (m)</td>
<td>( Q/H' ) (m²/s)</td>
</tr>
<tr>
<td>Test Depth (m)</td>
<td></td>
</tr>
<tr>
<td>Head Losses, ( \Delta H ) (m)</td>
<td></td>
</tr>
</tbody>
</table>

Table 4.1: List of Parameters for Testing with the Perméafor

Head losses within the testing system are a function of the flow of water through the narrow flexible tubing. For each flow value the head losses are determined using the quadratic formula shown in Equation 4.1.

\[
\Delta H = aQ^2 + bQ + c
\]  

[4.1]

The determination of the coefficients \( a, b \) and \( c \) is outlined in Section 2.6.3. The corrected head is simply the head of water above the test zone subtracted by the head losses at the specified flow. Finally, the ratio \( Q/H' \) is
determined by dividing the flow in cubic meters per second by the corrected head in meters.

Both laboratory and field testing results are calculated in the same manner. Final results are presented graphically showing the variation of $Q/H'$ with depth. The penetration effort used to push the probe into the soil is also presented to show the changing density with depth. The graphs are used to show the changing relative hydraulic conductivity as a function of depth.

4.2.2 Five Layer Soil Test

The dynamic cone penetrometer hammer was used to penetrate the probe through the soil sample. The weight of the probe and hammer system forces the Permeafor into the soil a few inches before hammering was initiated for penetration. As the probe progressed into the compacted soil specimen, water began to flow up the drill rods to the soil surface. Once water stopped flowing vertically to the surface it was then assumed that lateral flow had been fully established. Lateral flow is assumed, further investigation may prove that flow is both in laterally and downward. For the scope of this work, once vertical flow is stopped, lateral flow is assumed.

The flow and hydraulic head were recorded throughout testing. Lateral flow was established in the five layer soil sample at a depth of 35 centimeters below the surface. Figure 4.1 shows the number of blows needed to penetrate the probe into the soil sample and the results of the Permeafor testing in terms of $Q/H'$ which are also outlined in Table 4.2. Note that the number of blows are measured at the tip of the probe. Therefore, at each test interval the screen
section is above the tip of the probe. Table 4.2 outlines the approximate number of blows at the screen section to progress the probe for each five centimeter interval.

<table>
<thead>
<tr>
<th>Test Depth (cm)</th>
<th>Number of Blows at Test Depth</th>
<th>Head Above Test Zone (m)</th>
<th>Flow, $Q$ (l/min)</th>
<th>Head Losses, $\Delta H$ (m)</th>
<th>Corrected Head, $H'$ (m)</th>
<th>$Q/H'$ (m$^2$/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>0</td>
<td>1.78</td>
<td>3.7</td>
<td>1.57</td>
<td>0.21</td>
<td>2.96E-04</td>
</tr>
<tr>
<td>15</td>
<td>0</td>
<td>1.83</td>
<td>3.7</td>
<td>1.57</td>
<td>0.26</td>
<td>2.39E-04</td>
</tr>
<tr>
<td>20</td>
<td>4</td>
<td>1.88</td>
<td>3.7</td>
<td>1.57</td>
<td>0.31</td>
<td>2.00E-04</td>
</tr>
<tr>
<td>25</td>
<td>4</td>
<td>1.93</td>
<td>3.6</td>
<td>1.50</td>
<td>0.43</td>
<td>1.38E-04</td>
</tr>
<tr>
<td>30</td>
<td>3</td>
<td>1.98</td>
<td>3.6</td>
<td>1.50</td>
<td>0.48</td>
<td>1.24E-04</td>
</tr>
<tr>
<td>35</td>
<td>4</td>
<td>2.03</td>
<td>3.5</td>
<td>1.42</td>
<td>0.61</td>
<td>9.56E-05</td>
</tr>
<tr>
<td>40</td>
<td>5</td>
<td>2.08</td>
<td>3.3</td>
<td>1.27</td>
<td>0.81</td>
<td>6.81E-05</td>
</tr>
<tr>
<td>45</td>
<td>6</td>
<td>2.13</td>
<td>3.2</td>
<td>1.20</td>
<td>0.93</td>
<td>5.73E-05</td>
</tr>
<tr>
<td>50</td>
<td>7</td>
<td>2.18</td>
<td>3.0</td>
<td>1.06</td>
<td>1.12</td>
<td>4.45E-05</td>
</tr>
<tr>
<td>52.5</td>
<td>8</td>
<td>2.21</td>
<td>2.95</td>
<td>1.02</td>
<td>1.19</td>
<td>4.16E-05</td>
</tr>
</tbody>
</table>

Table 4.2: Results of $Q/H'$ for 5 Layer Sample Laboratory Test
The probe progressively becomes harder to penetrate into the soil specimen as the sample is increasingly denser. The $Q/H'$ results show that the ratio decreases as the density increases, similarly to hydraulic conductivity. After testing is completed the probe is allowed to remain in the soil sample while saturation of the soil takes place from the bottom of the test container.

The slope of the $Q/H'$ plot remains relatively constant through the entire test sample despite the presence of vertical flow. It would be assumed that there should be a dramatic change in the flow once horizontal flow is fully established.
The water takes the path of least resistance, prior to horizontal flow this path is partially up the rods to the surface of the soil. Once this path is no longer available the flow should decrease as water is forced laterally into the soil. The results seem to suggest that the portion of the flow in the vertical direction is only a small contribution of the total flow.

Standard testing with the Perméafor requires a ten second time period prior to recording the flow, \( Q \), to allow for flow stabilization. When the probe is at a desired test depth the flow is monitored over time. Tests at various depths are shown in Figure 4.2 illustrating the variation of flow with time for the five layer soil test.

As shown in the figure, it appears that the flow does fluctuate significantly after the ten seconds outlined by the French guidelines. The reading after the first ten seconds is the recorded flow for the test depth. The variation of flow as a function of time illustrates that the relative hydraulic conductivity decreases as the saturation level near the probe increases.
Rings were placed throughout the soil sample at depths of 40, 50 and 60 centimeters below the top of the sample drum. Soil becomes contained in the rings which are excavated after testing with the probe is complete. These rings are used to evaluate index properties variations with depth. The moisture content, relative density, degree of saturation and the void ratio are measured for each Perméafor test. These index properties for the five layer soil test are shown in Figure 4.3.
For the saturated test, the probe is left in the drum with water continuously flowing. As time passes the drum becomes saturated as water was allowed to pond on top of the sample. When a significant time had passed with constant water ponded on top of the sample it was assumed that the soil was 100 percent saturated. The rings are excavated after testing with the probe is complete allowing the ponded water to dissipate throughout the sample by opening the water release valve on the bottom of the testing tank.

The average moisture content throughout the sample was 17.8 percent and the degree of saturation is 69.2 percent. Due to the nature of the saturation process it is still assumed that the soil is saturated during the saturated test. Water will filter out of the ring while the excavation is being conducted.
The overall average dry unit weight of the soil sample is 16 kN/m$^3$. The relative density increases with depth from 66 to 72 percent which is consistent as the soil sample is denser near the bottom. Based on the relative density, the soil sample is considered medium dense to dense. The void ratio tends to decrease with depth as the soil particles are closer together ranging from 0.65 to 0.74 with an average value of 0.70.

4.2.3 Seven Layer Soil Test

Vertical flow was present for the top 45 centimeters of the seven layer sample. For this reason, only two tests could be conducted after horizontal flow was established. The vertical flow was only present for the top 35 centimeters in the five layer soil test. The results of the seven layer soil sample are shown in Figure 4.4 and tabulated in Table 4.3.
<table>
<thead>
<tr>
<th>Test Depth (cm)</th>
<th>Number of Blows at Test Depth</th>
<th>Head Above Test Zone (m)</th>
<th>Flow, Q (l/min)</th>
<th>Head Losses, ΔH (m)</th>
<th>Corrected Head, H' (m)</th>
<th>Q/H' (m²/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>0</td>
<td>1.73</td>
<td>3.7</td>
<td>1.57</td>
<td>0.16</td>
<td>3.90E-04</td>
</tr>
<tr>
<td>10</td>
<td>1</td>
<td>1.78</td>
<td>3.7</td>
<td>1.57</td>
<td>0.21</td>
<td>2.96E-04</td>
</tr>
<tr>
<td>15</td>
<td>1</td>
<td>1.83</td>
<td>3.7</td>
<td>1.57</td>
<td>0.26</td>
<td>2.39E-04</td>
</tr>
<tr>
<td>20</td>
<td>2</td>
<td>1.88</td>
<td>3.7</td>
<td>1.57</td>
<td>0.31</td>
<td>2.00E-04</td>
</tr>
<tr>
<td>25</td>
<td>2</td>
<td>1.93</td>
<td>3.7</td>
<td>1.57</td>
<td>0.36</td>
<td>1.72E-04</td>
</tr>
<tr>
<td>30</td>
<td>3</td>
<td>1.98</td>
<td>3.7</td>
<td>1.57</td>
<td>0.41</td>
<td>1.51E-04</td>
</tr>
<tr>
<td>35</td>
<td>4</td>
<td>2.03</td>
<td>3.7</td>
<td>1.57</td>
<td>0.46</td>
<td>1.35E-04</td>
</tr>
<tr>
<td>40</td>
<td>5</td>
<td>2.08</td>
<td>3.7</td>
<td>1.57</td>
<td>0.51</td>
<td>1.21E-04</td>
</tr>
<tr>
<td>45</td>
<td>6</td>
<td>2.13</td>
<td>3.7</td>
<td>1.57</td>
<td>0.56</td>
<td>1.10E-04</td>
</tr>
<tr>
<td>50</td>
<td>8</td>
<td>2.18</td>
<td>3.4</td>
<td>1.35</td>
<td>1.83</td>
<td>6.79E-05</td>
</tr>
</tbody>
</table>

Table 4.3: Results of $Q/H'$ for 7 Layer Sample Laboratory Test
The penetration of the probe was also similar to that of the five layer sample test. Results in terms of $Q/H'$ for the entire profile do not follow the expected trend as they are greater than those of the five layer testing. It is expected that the flow would decrease in the higher density soil. However, these results may not be conclusive due to the small number of test locations with only lateral flow.

Figure 4.4: Results of $Q/H'$ for 7 Layer Sample Laboratory Test
At test depths where horizontal flow is fully present the variation of flow as a function of time were recorded as shown in Figure 4.5. Any fluctuations in the flow are minimized after a 70 second time frame.

Figure 4.5: Results of $Q/H$ with Time for 7 Layer Sample

As in the five layer test, rings were used to measure a variety of index properties. The results of the soil index properties for the seven layer soil test are shown in Figure 4.6.
The seven layer soil test has a dry unit weight of 16.5 kN/m$^3$, which is greater than the five layer sample. The average moisture content of the sand was 17.2 percent and the average degree of saturation was 68.6 percent. Both of these index properties are within one percent of the results from the five layer test.

The relative density of the soil is approximately 73.0 percent which is larger than the five layers by four percent. Based on the relative density the soil sample is still considered a medium dense to dense sand. The void ratio ranges from 0.66 to 0.73 with an average value of 0.68. The void ratio is expected to be lower than the previous test as the soil particles are packed closer together. The previous test resulted in a higher average void ratio of 0.70. All soil index
properties show that the soil in the seven layer sample is denser than the five layer sample.

4.2.4 Ten Layer Soil Test

The final laboratory test was conducted in a slightly different manner than both previous tests. In this test vertical flow stopped at a depth of 40 centimeters. The soil was not allowed to saturate after the final test depth was reached with the probe. Rather than evaluating the $Q/H'$ after saturation, instead the probe was removed from the test drum to examine the hole created by the probe during the test. The results of testing with the Perméafor for the ten layer soil test are shown in Figure 4.7 and are also outlined in Table 4.4.
<table>
<thead>
<tr>
<th>Test Depth (cm)</th>
<th>Number of Blows at Test Depth</th>
<th>Head Above Test Zone (m)</th>
<th>Flow, $Q$ (l/min)</th>
<th>Head Losses, $\Delta H$ (m)</th>
<th>Corrected Head, $H'$ (m)</th>
<th>$Q/H'$ (m²/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>0</td>
<td>1.73</td>
<td>3.6</td>
<td>1.50</td>
<td>0.23</td>
<td>2.56E-04</td>
</tr>
<tr>
<td>10</td>
<td>1</td>
<td>1.78</td>
<td>3.6</td>
<td>1.50</td>
<td>0.28</td>
<td>2.11E-04</td>
</tr>
<tr>
<td>15</td>
<td>2</td>
<td>1.83</td>
<td>3.6</td>
<td>1.50</td>
<td>0.33</td>
<td>1.79E-04</td>
</tr>
<tr>
<td>20</td>
<td>2</td>
<td>1.88</td>
<td>3.6</td>
<td>1.50</td>
<td>0.38</td>
<td>1.56E-04</td>
</tr>
<tr>
<td>25</td>
<td>3</td>
<td>1.93</td>
<td>3.6</td>
<td>1.50</td>
<td>0.43</td>
<td>1.38E-04</td>
</tr>
<tr>
<td>30</td>
<td>3</td>
<td>1.98</td>
<td>3.6</td>
<td>1.50</td>
<td>0.48</td>
<td>1.24E-04</td>
</tr>
<tr>
<td>35</td>
<td>4</td>
<td>2.03</td>
<td>3.5</td>
<td>1.42</td>
<td>0.61</td>
<td>9.56E-05</td>
</tr>
<tr>
<td>40</td>
<td>5</td>
<td>2.08</td>
<td>3.5</td>
<td>1.42</td>
<td>0.66</td>
<td>8.84E-05</td>
</tr>
<tr>
<td>45</td>
<td>6</td>
<td>2.13</td>
<td>2.9</td>
<td>1.01</td>
<td>1.12</td>
<td>4.35E-05</td>
</tr>
<tr>
<td>50</td>
<td>7</td>
<td>2.18</td>
<td>2.7</td>
<td>0.85</td>
<td>1.33</td>
<td>3.39E-05</td>
</tr>
<tr>
<td>55</td>
<td>9</td>
<td>2.23</td>
<td>2.5</td>
<td>0.72</td>
<td>1.511</td>
<td>2.76E-05</td>
</tr>
</tbody>
</table>

Table 4.4: Results of $Q/H'$ for 10 Layer Sample Laboratory Test
The test zone after vertical flow had stopped shows a significant decrease in flow resulting in a lower value of $Q/H'$. The final three test zones have a $Q/H'$ value significantly less than the test conducted at 40 centimeters. The trend of the final three tests conducted at depths of 45, 50 and 55 centimeters have a slope three times greater than the tests conducted at depths of 30, 35 and 40 centimeters. This trend is expected since the flow below 40 centimeters appeared to be strictly horizontal.

Figure 4.7: Results of $Q/H'$ for 10 Layer Sample Laboratory Test
The trend of decreasing flow after the establishment of vertical flow is also present in the five layer soil drum but is not as dramatic. The trend is not present in the seven layer drum likely due to the lack of test zones below the area where horizontal flow was established. The trend demonstrates that the flow decreases once water no longer flows in the vertical direction up the rods.

As before, the variation of flow with respects to time was monitored at all test zones after horizontal flow was established. The results of the variation of $Q/H'$ with respects to time for the ten layer soil test are shown in Figure 4.8.

![Figure 4.8: Results of $Q/H$ with Time for 10 Layer Sample](image)

Unlike all prior tests there were no variations of flow with respect to time.

It appears that a steady state was present for each test zone once the probe was
at the desired test depth. Rings were again used in the soil to measure index properties. Because the soil was not saturated during testing the excavation of the rings proved more difficult. The water would tend to bond the soil so the rings could be removed. Several rings were compromised during the excavation process. The results of the soil index property tests for the ten layer soil sample are shown in Figure 4.9.

<table>
<thead>
<tr>
<th>Moisture Content (%)</th>
<th>Relative Density (%)</th>
<th>Degree of Saturation (%)</th>
<th>Void Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>25</td>
<td>0</td>
<td>100</td>
</tr>
<tr>
<td>100</td>
<td>0</td>
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</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1</td>
</tr>
</tbody>
</table>

![Figure 4.9: Index Properties from 10 Layer Sample Test after Testing](image)

The ten layer soil sample has an average dry unit weight of 17.0 kN/m$^3$, which is greater than the five and seven layer tests with average dry unit weight of 16.0 kN/m$^3$ and 16.5 kN/m$^3$, respectively. The moisture content decreases with depth from 17.5 to 7.5 percent due to the soil not being fully saturated. The probe begins to saturate the soil at the top layer at the beginning of the test.
Water becomes ponded on top of the soil sample encouraging a higher moisture content in the upper zones of the soil sample. The moisture content of both previous tests was 17.8 and 17.2 percent. The degree of saturation is also a function of the moisture content therefore it too decreases with depth. The maximum degree of saturation is 68.8 percent and the minimum is 28.0 percent.

The void ratio of the soil ranges from 0.67 to 0.73 with an average value of 0.70. The average void ratio of the five and seven layer tests were 0.70 and 0.68, respectively. The void ratio is expected to be smaller than the previous tests as the soil becomes denser with the increasing amount of soil lifts used to create the test sample. Due to the inconsistencies in the integrity of the soil rings the measured index properties may not be fully representative of the actual properties of the sample.

The average relative density is 67.2 percent which corresponds to a medium dense to dense sand. The relative densities of the previous two tests were 69.2 and 70.0 percent showing that the ten layer test was less dense. This is not an accurate representation of the density of the soil as both the dynamic cone penetrometer and the probe blow counts are greater in the ten layer soil test. The increase blow counts are a function of the increasing density of the soil. This suggests that by not saturating the soil the rings do not provide accurate index properties. It is possible that the saturation process actually helps increase the density the sample.
4.2.5 Saturation Test Results

In both the five and seven layer soil tests with the Permeafor the soil was
allowed to saturate after testing was conducted. The saturation was achieved by
maintaining the probe in the test drum with the water constantly flowing. Once
the soil was fully saturated one final test with the probe was conducted at the
same test depth as the prior test before saturation. Figure 4.10 shows the test
results of the saturation tests for both the five and seven layer soil test.

![Test Results of Saturation Tests Conducted on 5 and 7 Layer Soils](image)

Figure 4.10: Test Results of Saturation Tests Conducted on 5 and 7 Layer Soils
The flow tends to decrease with time as previously shown in Figures 4.1, 4.4 and 4.7 for the variation of flow with respects to time. Hydraulic conductivity is a function of the degree of saturation of the soil. Similarly, the ratio of $Q/H'$ decreases as the saturation level increases. The five layer soil test has a larger variation between the saturated and unsaturated state. For the five layer test, the saturated sample $Q/H'$ is 2.03E-05 m$^2$/s where the unsaturated test produced a $Q/H'$ of 4.16E-05 m$^2$/s. The saturated sample $Q/H'$ is 6.15E-05 m$^2$/s and the unsaturated test produced a $Q/H'$ of 6.79E-05 m$^2$/s indicating that the seven layer tests for the saturated and non-saturated samples were similar.

It is expected that the saturation test would produce a lower value of $Q/H'$, which is does not. As the saturation levels increase the flow decreases which in turn creates smaller head losses. The results between the saturated and so-called unsaturated state do not show a large difference. Because flow is established immediately around the probe screen the soil actually behaves as a saturated soil.

As the density of the soil increases the variation of hydraulic conductivity as a function of molding water content becomes less dramatic. This is consistent with the hydraulic conductivity tests conducted on the Lee sand in Proctor molds the higher compacted soils which resulted in a relatively constant hydraulic conductivity with the varying molding water content.

4.2.6 Correlation of Laboratory Results to Hydraulic Conductivity

Testing conducted with the Perméafor defines the soil profile using the ratio of $Q/H'$. Hydraulic conductivity is reported in the units of length over time
whereas results with the probe are reported in units of length squared over time. Manipulations of the data sets from the Perméafor are needed in order to report the results in the same units as hydraulic conductivity. The French have outlined a procedure to determine hydraulic conductivity based on the geometry of the probe. The pocket coefficient, defined by the geometry of the probe, can be used to relate hydraulic conductivity to the $Q/H'$ ratio as shown in Equation 4.2.

$$k \left( \frac{\text{cm}}{s} \right) = 475 \left( \frac{Q}{H'} \right)$$  \[4.2\]

Using Equation 4.2 the laboratory tests can be expressed as changing hydraulic conductivity with depth. Using the Hazen equation to estimate the hydraulic conductivity based on the Lee sand sieve analysis and the effective diameter, the hydraulic conductivity is on the order of magnitude of $1.0\text{E}-02$ to $1.0\text{E}-03$ cm/s. The results of the correlation of Perméafor tests to hydraulic conductivity are shown in Figure 4.11. Also outlined on the right of this figure are the locations at which vertical flow stopped. In addition, the number of blows needed to progress the dynamic cone penetrometer through the soil are shown as an indication of variation in density through the soil sample.

Prior to the establishment of horizontal flow, the hydraulic conductivity determined by the Perméafor is on the order of magnitude of $1.0\text{E}-01$ cm/s. The expected values based on the Hazen equation are on the order of magnitude of $1.0\text{E}-02$ to $1.0\text{E}-03$ cm/s. After horizontal flow is established the Lee sand reports values of hydraulic conductivity tending toward the range of the hydraulic conductivity expected using the Hazen equation.
The seven layer soil test and the modified Proctor sample have energy compaction levels of 2,625 and 2,694 kJ/m$^3$, respectively. Since the soils have similar compaction energy the hydraulic conductivity can be related. Vertical flow limits the interpretation of the hydraulic conductivity for the soil sample as only
two test depths were performed with horizontal flow. The hydraulic conductivity
does decrease with depth, but additional tests are needed to determine if the
probe produces similar hydraulic conductivity values compare to conventional
laboratory tests.

Overall, laboratory testing proved the ease of testing with the Perméafor.
Tests are both fast and easy to interpret; the major time consumer for laboratory
testing is the preparation of the sample. The number of blows needed to
advance the probe for all three tests were similar with the region above 40
centimeters. Once vertical flow is established the average number of blows per 5
centimeters for the five layer sample is 6 blows, 9 blows for the seven layer
sample and 10 blows for the ten layer sample.

The probe and drill rods are marked using a wax chalk every five
centimeters from the center of the perforated screen section. The number of
blows needed to advance the probe are counted between every five centimeter
interval. On occasion, the number of blows do not lie on the 5 centimeter marks
on the probe therefore some interpretation is needed. The blow counts for this
reason are variable but still a good measurement of the probe resistance to
penetration.

4.3 Field Testing Results

A multitude of field tests were conducted with the Perméafor. Tests were
conducted by advancing the probe using either the dynamic cone penetrometer
hammer or a hydraulic rig. The results of the field testing conducted at the test
site in Lee, New Hampshire are summarized in the following section.
4.3.1 Field Testing With the Perméafor

Two test profiles were conducted using the dynamic cone penetrometer hammer (DCH) as in the laboratory testing. For both profiles the probe was advanced to a depth of 30 centimeters before flow was recorded. The first profile conducted with the probe used an $H_e$ of 1.90 meters. Flow recordings were taken from 30 centimeters to a final depth of 100 centimeters where flow was no longer able to be measured using the current flow meter. The second field profile with the dynamic cone penetrometer hammer was conducted at an $H_e$ of 3.60 meters. Flow was recorded from 30 to 150 centimeters. Lateral flow was not established until 70 centimeters for the first profile and 80 centimeters for the second profile. The results of tests conducted at the Lee, New Hampshire site using the dynamic cone hammer as the advance method are shown in Tables 4.5 and 4.6 and Figure 4.12 along with the resistance to penetration. Note that due to the different levels of the water source, two separate head loss equations are used. This means that the head losses at a flow of 3.0 L/m in test one are different then the head losses at the same flow in test two.
<table>
<thead>
<tr>
<th>Test Depth (cm)</th>
<th>Number of Blows at Depth</th>
<th>Head Above Test Zone (m)</th>
<th>*Flow, Q (l/min)</th>
<th>Head Losses, ΔH (m)</th>
<th>Corrected Head, H' (m)</th>
<th>Q/H' (m²/s)</th>
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<tr>
<td>80</td>
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<td>14</td>
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<td>3.0</td>
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<td>0.99</td>
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<td>20</td>
<td>2.91</td>
<td>1.3</td>
<td>0.04</td>
<td>2.87</td>
<td>7.27E-06</td>
</tr>
</tbody>
</table>

* The flow is recorded on the flow meter in liters per minute which is converted to cubic meters per second using the following conversion:

\[
Q \left( \frac{m^3}{sec} \right) = \text{Value of } Q \left( \frac{l}{min} \right) \times \left( \frac{1 \text{ min}}{60 \text{ sec}} \right) \times \left( \frac{m^3}{1000 L} \right)
\]

Table 4.5: Results of \( Q/H' \) for Dynamic Cone Hammer (DCH) 1
<table>
<thead>
<tr>
<th>Test Depth (cm)</th>
<th>Number of Blows at Depth</th>
<th>Head Above Test Zone (m)</th>
<th>Flow, $Q$ (l/min)</th>
<th>Head Losses, $\Delta H$ (m)</th>
<th>Corrected Head, $H'$ (m)</th>
<th>$Q/H'$ (m²/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>30</td>
<td>8</td>
<td>3.91</td>
<td>5.2</td>
<td>3.28</td>
<td>0.63</td>
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<td>5.2</td>
<td>3.28</td>
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<tr>
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<td>3.19</td>
<td>1.12</td>
<td>7.63E-05</td>
</tr>
<tr>
<td>80</td>
<td>12</td>
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<td>3.19</td>
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<td>7.00E-05</td>
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<tr>
<td>90</td>
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<td>4.6</td>
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<td>3.72E-05</td>
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<tr>
<td>120</td>
<td>23</td>
<td>4.81</td>
<td>3.7</td>
<td>1.71</td>
<td>3.10</td>
<td>1.99E-05</td>
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<tr>
<td>130</td>
<td>32</td>
<td>4.91</td>
<td>3.0</td>
<td>0.80</td>
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<td>1.22E-05</td>
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<tr>
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<td>2.65</td>
<td>0.30</td>
<td>4.81</td>
<td>9.18E-06</td>
</tr>
</tbody>
</table>

Table 4.6: Results of $Q/H'$ for Dynamic Cone Hammer (DCH) 2
To advance the probe throughout the soil strata a similar number of blows of the hammer system were needed. The ratio of $Q/H'$ is expected to be the same as the soil is the same material as the profiles were conducted within 3 meters of each other. Closer to the surface prior to complete horizontal flow the ratio of $Q/H'$ are similar. Once the probe creates a seal in the ground and flow is lateral the flow begins to decrease.
After lateral flow is established it is assumed that the ratio of $Q/H'$ would be the same for both profiles. The two profiles were conducted using different pressure heads. The flow was significantly higher for the second profile as the head was increased by approximately 1.7 meters. The head above the test zone affected the results as it increased the flow. As flow increases the head losses in the system also increase. The increasing head losses will decrease the corrected head, $H'$. For this reason, the decrease corrected head will produce an increase ratio of $Q/H'$. In this case, the external head on the system played a major role in the resulting $Q/H'$.

Similar to the laboratory testing, the parameter of $Q/H'$ decreases as the density increases. The density of the soil become greater as the probe is progressed further. Overall, the results follow the same trend where the relative hydraulic conductivity decreases with depth.

At depths greater than 150 centimeters a total of 45 or more blows of the dynamic cone penetrometer hammer were needed to advance the tool a distance of five centimeters. This method is both time consuming and physically exhausting. Water is continuously flowing while the probe is driven which saturates the soil prior to being able to read the flow of water into the test zone area. Additional tests with the Perméafor were conducted using the hydraulic rig to advance to tool.

Three profiles were conducted with the hydraulic system which all had vertical flow present throughout the entire push of the probe. Several variations of this layout were evaluated, but proved unsuccessful such that lateral flow was
never fully established. The probe and one drill rod are attached to the hydraulic rig. A 20 centimeter preparation hole was augered due to the length of the rods. The results of the tests conducted with the dynamic cone penetrometer hammer (DCH) and the hydraulic rig (HC) are shown in Table 4.7 and Figure 4.13.

Using the hydraulic rig the ratio of $Q/H'$ at depths above 20 centimeters is negative. In order to use the rig the top 20 centimeters of soil is augered as the frame is not tall enough to support the probe and one drill rod. In this area there is no soil to provide resistance to the flow of water. The flow is high which in turn produces high head losses through the flexible tubing. The effective head then is negative which creates a negative ratio of $Q/H'$. Negative values are not possible in normal circumstances they would indicate a very permeable soil. The results are bounded between values of $10^{-6}$ and $10^{-3} \text{ m}^2/\text{s}$ therefore, in Figure 4.13 the negative $Q/H'$ values are not plotted. Values are not possible beyond these bounds outlined by the French standards.
<table>
<thead>
<tr>
<th>Depth (cm)</th>
<th>$Q/H'$ (m$^2$/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>DCH 1</td>
</tr>
<tr>
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</tr>
<tr>
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<tr>
<td>140</td>
<td></td>
</tr>
<tr>
<td>150</td>
<td></td>
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</table>

Table 4.7: $Q/H'$ Results for all Field Tests
The variation of $Q/H'$ is similar for all three tests conducted with the hydraulic rig. The trend of the curve follows the same path as both tests conducted with the DCH. The hydraulic rig would be ideal for testing as it provides easy and fast testing. Due to lateral flow never being established, the results can only provide a basis for partial lateral flow.
4.3.2 Effect of External Head on Measurements

At the field testing location two different head levels were used while advancing the probe. The two head positions were located at 1.9 and 3.6 meters above the ground surface. Figure 4.14 shows the $Q/H'$ value as a function of head at depths of 30 to 100 centimeters below the ground surface. Also shown are published the results for the same experiments conducted with the full scale French probe in a test chamber.

Figure 4.14: Variation of $Q/H'$ as a Function of External Head

The French testing is conducted by measuring the flow at more than one external head level for the same test depth. Tests conducted with the half scale...
model were performed in two separate test locations of what is assumed to be the same soil. Due to the limitation of the testing layout it was not possible to change the head above the probe while it is at a single test depth.

The effects of changing head in the half scale probe are significant at depths of 0.3, 0.4, 0.8, 0.9 and 1.0 meters. It is expected that at each head level the plot would have a slope of zero. The largest slope is two which is seen at 1.0 meters. Test locations within the expected boundaries occurred at depths of 0.5, 0.6 and 0.7 meters. It is assumed that these variations are a function of the changing soil conditions between the two test holes.

In conjunction with the soil fluctuations with depth a second issue is presented due to the horizontal flow not being established until a depth of 0.7 meters for the test conducted with a head of 1.9 meters and 0.8 meters for the test at a head level of 3.6 meters. Overall, more testing is needed to understand the effects of external head on the half scale model.

4.3.3 Hydraulic Conductivity Based On Grain Size

Empirical correlations for hydraulic conductivity can be assessed based on the grain size analysis of the samples collected in the field. As outlined in Section 2.5, there are several equations available. For the purpose of this research the Hazen formula, the Alyamani and Sen formula and the Rawls and Brakensiek formula were evaluated. Perméafor test results are corrected to represent values of hydraulic conductivity using the pocket coefficient defined in Section 2.6.6. The results of the Perméafor field testing, empirical correlations and measured laboratory testing are shown in Figure 4.15.
Figure 4.15: Hydraulic Conductivity Based on Grain Size
The laboratory testing results shown are the range of applicable values of hydraulic conductivity measured throughout both the standard and modified Proctor tests. After horizontal flow is established with the Permeafor the results prove to be approximately an order of magnitude higher than the measured values of hydraulic conductivity based on laboratory Proctor testing. On the other hand, the empirical correlations are similar to the results with the probe after vertical flow has stopped.

All of the empirical correlations produce a larger value of hydraulic conductivity compared to the measured laboratory results. The correlations also tend to follow the same trend. The Hazen formula proves to be the median value.

4.4 Field Testing Variations

Several variations and adaptations were developed and implemented to optimize the Permeafor probe field testing. Issues that stem from vertical flow coming up the drill rods and having flow stop at shallow depths needed to be addressed.

4.4.1 Change in Constant Head Level

During testing with the dynamic cone penetrometer hammer flow stopped at a depth of 100 centimeters. With the water source located at 1.9 meters above the ground surface, more head was needed to increase flow through the soil. The water source was then placed at a height of over 3.6 meters above the ground surface in addition to a hydraulic pump. The pump used created an additional 2.2 meters of head.
The increases in flow due to the large elevation head and hydraulic head added by the pump only compounded the issue of water flowing vertically up the drill rods. The water source was reverted back to the 1.9 meter level of the scaffolding, and the pump was not used. Once the probe is advanced into the soil the pump cannot simply be turned on because it must be purged of air and introduces a significant amount air into the system. The pump must be on when the probe is at the surface in order to eliminate this problem.

Overall, a pressurized water system would be more practical compared to the water source used in field testing. Simply using elevation head does not allow for enough flexibility in varying in the head to adjust to the changing conditions of the subsurface.

4.4.2 Portable Hydraulic Rig versus Dynamic Penetrometer Hammer

Using the dynamic penetrometer hammer to advance the probe is ideal for laboratory testing. The hammer is easy to maneuver and advances the probe in a reasonable manner within laboratory samples. The location chosen for field testing is significantly denser then all of the laboratory samples. To advance the probe five centimeters at depths below 1.5 meters, an average of 45 blows of the hammer were needed. The process became excessive and time consuming as the probe would saturate the soil before testing could be conducted.

The hammer also is connected to the drill rods with a steel pin causing a large amount of lateral movement. This movement causes the probe to create a larger hole compared to the diameter of the probe. The large hole also facilitates
water to travel up the outside of the drill rods rather than moving laterally into the soil.

On the other hand, no values could be recorded past 150 centimeters as the flow meter was not able to record flow. With the length of time needed to progress the probe, the soil becomes saturated reducing flow. In order to push the probe both faster and with relative ease the portable hydraulic rig was used. Several different variations were used with the rig including a range from two to four 55 gallon drums of water to counteract the force applied by the system. The portable rig only was only able to push the probe to a depth of 115 centimeters but testing took only minutes versus hours.

Using a constant pushing system vertical flow was evident throughout the entire testing. The French probe uses a percussion drill rig to push the probe. The dynamic cone penetrometer hammer vibrates the soil allowing it to collapse back onto the drill rods. The collapsing soil seals the flow of water from moving vertically up the rods. The hydraulic rig creates a straight hole allowing water to flow upward throughout the entire test.

Overall the dynamic cone penetrometer hammer produced results with the Perméafor. The ease of testing with the hydraulic rig is favorable, but results appear less reliable.

4.4.3 Screen Section

In attempts to obtain results above 70 centimeters using the portable hydraulic rig, a new screen section was created. The velocity of water being ejected out of the probe near the surface was too great to be absorbed by the
surrounding soil. To decrease the velocity the screen section length was increased.

The screen section, as originally designed, has eight rows of four perforations resulting in a total area of 1.26 square centimeters. The average flow of water during testing is 3.5 liters per minute providing an exit velocity of 46.3 centimeters per second. By doubling the size of the screen, the exit velocity is reduced to 23.3 centimeters per second. The decreased velocity allows the soil more time to absorb water from the probe.

The original probe as designed in France has a screen section of five centimeters in length and a length of diameter ratio of one. The new screen section should not be larger than the full scale model. The overall objective of the Perméafor is to measure the incremental variation of $Q/H'$ with depth. The small screen section allows for the detection of thin soil seams. Increasing the screen section any more than five centimeters would defeat the original purpose of the probe. Even with the decrease in velocity from the increased screen section vertical flow was still present during testing.

4.4.4 Tapered Design

The original design of the half scale model included an increase diameter section near the top of the probe. As shown in Figure 4.16, the increased section includes a flare of 45 degrees. The purpose of this section is to create a seal such that water does not flow up the drill rods to the surface and isolates flow to the test area.
As the probe is being pushed into the subsurface this section increases the diameter of the hole. Using the direct push method of the portable drill rig the soil could not create a seal so water continuously rises to the surface. The sharp change in diameter causes a bearing capacity failure in the sand. To ease the transition in this section the probe was tapered at a constant rate. After this modification water still continued to flow vertically. The probe was easier to push into the subsurface and also easier to remove.

4.5 Scale Model Perméafor Test Comparisons

In the following section comparisons are made from the results of the laboratory and field testing. In addition to comparing tests conducted with the
probe, additional comparisons are made to previous testing with the half scale model and the French full scale model.

4.5.1 Comparison of Laboratory and Field Perméafor Tests

The dynamic cone penetrometer was performed in all three laboratory soil samples and at the testing site location in Lee, New Hampshire. For depths above 40 centimeters the field blow counts are significantly greater than any of the laboratory tests. The field blow counts values are approximately 1.5 times greater than lab samples at depths above 40 centimeters.

Beyond 40 centimeters the blow counts in the field are similar to those seen in the 7 layer soil sample. Similar blow counts relate to approximately the same relative density. The blow counts to advance the probe for the 5 and 7 layer soil samples are similar to the number of blows needed to penetrate the probe in the field. When soil samples have similar relative density the ratio of $\frac{Q}{H'}$ should be comparable in both laboratory and field testing.

Beginning at approximately 80 centimeters the dynamic cone penetrometer and rods had to be removed from the subsurface. The hole was augered then use of the dynamic cone penetrometer resumed. As the bottom of the hole was not fully cleaned the blow counts are less than the blows above the test zone. This was performed four times throughout the test. Figure 4.17 shows the blow counts of the dynamic cone penetrometer for laboratory and field tests.
Figure 4.17: Dynamic Cone Penetrometer Results for Laboratory and Field Testing

Comparisons for laboratory and field ratios of $Q/H'$ proves more difficult. Lateral flow for all field tests was only present at depths below 70 centimeters and testing in the lab was only possible to depths of 60 centimeters due to the limitations of sample size. Results are only comparable after horizontal flow is established. The 10 layer soil sample produces a ratio of $Q/H'$ on the same order of magnitude as the first DCH field test. The 7 layer soil sample, on the other hand, is on the same order of magnitude as the second DCH field test. Figure 4.18 shows the results of $Q/H'$ for the two field tests conducted with the
dynamic cone penetrometer hammer and the three laboratory samples with varying density. Each test shows a similar trend of decreasing $Q/H'$ ratio with increasing depth. In the top zone of all tests about 40 centimeters the ratio of $Q/H'$ is similar even with the varying soil densities.

Figure 4.18: Results of $Q/H'$ for Laboratory and DCH Field Tests
4.5.2 Comparison to Additional Scale Model Laboratory Testing

During the summer of 2009 Julien Torterat performed laboratory testing with the half scale Perméafor. Results of his tests were presented in an excel file in conjunction with a written report in French. The work of Torterat at the University of New Hampshire was not completed before returning to France therefore, additional calculations were needed to present his data. Three tests were conducted using the same laboratory set up as discussed in Section 3.4.1. There are differences between the two testing methods which include the method of compaction, the hammer system used to progress the probe and the soil type tested.

The major difference in compaction methods is the number of blows from the LWFD used to compact the soil. Torterat implemented 12 blows for each soil lift, whereas in this research only 10 blows per lift were used. The difference in the number of blows will change the energy of compaction. Table 4.8 outlines the differences of the energy of compaction between the 10 and 12 blows per layer for all soil samples.

<table>
<thead>
<tr>
<th>Soil Sample Lifts</th>
<th>10 Blows Per Layer</th>
<th>12 Blows Per Layer</th>
</tr>
</thead>
<tbody>
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<td>1875</td>
<td>2250</td>
</tr>
<tr>
<td>7</td>
<td>2625</td>
<td>3150</td>
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<tr>
<td>10</td>
<td>3750</td>
<td>4500</td>
</tr>
</tbody>
</table>

Table 4.8: Energy of Compaction for 10 and 12 Blow Per Layer
To advance the probe though the soil specimen a different hammer system was used. In this research the dynamic cone penetrometer hammer was modified to be able to attach to the drill rods. Torterat created a hammer system that consisted of a rod with several free weights which slid up and down the rod to advance the probe. The drop height and approximate weight of the hammer system were not reported.

The final difference between the Perméafor tests is that Torterat used a Dover sand where as this research focused on a Lee sand. The Dover sand is a larger grain size and a more permeable soil. No hydraulic conductivity tests were performed on the sand prior to testing with the Perméafor.

The excel file obtained was not the final draft, therefore calculations were not completed. In order to compare results, the head above the test zone is estimated as 170 centimeters. The test tank in this research was elevated from the ground surface by a pallet to allow for the tap on the base of the tank to be used. It is not known if this was also done during Torterat's tests.

Other calculations were needed to report the effective head for each test depth. Although the same laboratory layout was used the head losses for the system were not consistent with work completed in this research. The calculations completed for Torterat were done using his reported head losses. The head losses for both this research and that completed by Torterat are shown in Figure 4.19.
Figure 4.19: Head Losses in Laboratory Layer for Torterat and this Research

One reason that could be attributed to the discrepancy in the results is the reading of the flow meter. It appears that Torterat used an equation to approximate flow rather than using the provided calibration sheet for the flow meter. The calibration sheet uses a non-linear graphical approach to take the numerical readings from the flow meter to be able to report the flow in liters per minute. For example, a flow reading of 34 on the flow meter corresponds to a flow of 2.95 liters per minute where as Torterat's equation reports a flow of 3.03 liters per minute. Torterat's results are presented using his interpretation of flow and the approximation of the total head above the probe. The results of the
laboratory tests conducted by Torterat and those completed in this research are shown in Figure 4.20.

![Figure 4.20: Results of $Q/H'$ for Torterat and this Research](image)

The trend seen in all of Torterat's data shows an increasing $Q/H'$ ratio with depth. This is opposite of the expected results as hydraulic conductivity decreases with depth as the soil becomes denser. Additional issues that were reported for Torterat's results were air present in the flexible tubing. This poses an issue such that the flow will decrease significantly in turn not producing an accurate reading with the probe. Overall, it is difficult to compare tests
conducted in this research with T述职at's results due significantly different soil type.

4.5.3 Comparison to French Perméafor Results

The half scale model and the full scale French Perméafor produce similar results. Basic similarities include that hydraulic conductivity decreases with depth, the presence of negative $Q/H'$ ratios and reading beyond the bounded results of $10^{-3}$ to $10^{-6}$. Tests with the probe are not completed in the same soil such that the ratio of $Q/H'$ could be compared.

At shallow depths both probes produce a negative ratio of $Q/H'$ which is considered an anomaly as the head losses are greater than the head above the screen section. The French have outlined that the results of the probe are bounded at values of $10^{-3}$ to $10^{-6}$ square meters per second. The French probe and the soundings completed with the hydraulic rig report values outside of the bounded range. One of the final major similarities between the two systems is that the probe produces vertical flow at shallow depths. The probe must create a seal within the soil to produce the lateral flow needed to recorded $Q/H'$ with depth.
5. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

5.1 Summary

The Permeafor is a new in situ testing tool which under certain conditions provides a continuous qualitative and quantitative profile of changing hydraulic conductivity with depth. The results from three laboratory simulation tests and five field profiles are presented in terms of the ratio of flow and corrected hydraulic head versus depth for a fine grained uniform sand from Lee, New Hampshire. The results were compared to other hydraulic conductivity measurements conducted on the same Lee sand. Throughout testing with the Permeafor, the procedures and set up were modified and optimized such that testing could be conducted with more efficiency.

5.2 Conclusions

The conclusions of the testing of the Permeafor in the Lee sand are summarized as follows:

1) The Permeafor demonstrated its ability to measure small variations in relative hydraulic conductivity with depth. Testing with the probe can be conducted in five centimeter intervals which can profile many soil conditions.
2) Laboratory profiles after vertical flow was established produced an average ratio of \(Q/H'\) of 6.14E-05 m\(^2\)/s for the five layer sample, 8.92E-05 m\(^2\)/s for the seven layer sample and 4.83E-05 m\(^2\)/s for the ten layer sample. It is expected that each increase in energy of compaction would decrease the hydraulic conductivity. The seven layer sample did not follow this trend.

3) Field profiles after vertical flow was established produced an average ratio of \(Q/H'\) of 3.77E-05 m\(^2\)/s for the profile with an \(H_e\) of 1.9 meters and 3.48E-05 m\(^2\)/s for the profile with an \(H_e\) of 3.6 meters. This illustrates that the external head above the probe does not alter expected results.

4) The average value of hydraulic conductivity for the five layer laboratory sample was 2.92E-02 cm/s, 4.24E-02 cm/s for the seven layer sample and 2.30E-02 cm/s for the ten layer sample. The average hydraulic conductivity for the first field profile was 1.79E-02 cm/s and 1.66E-02 cm/s for the second field profile.

5) The hydraulic conductivity determined using the Permeafor is an order of magnitude larger than that of the reported results using the Hazen equation.

6) The hydraulic conductivity determined with the probe in the laboratory were an order of magnitude larger than the results presented in the laboratory constant and falling head permeability tests. This is expected as the proctor molds are compacted vertically and tested vertically. On the other hand, tests conducted with the probe are compacted vertically.
but measured horizontally. The ability for water to flow laterally is greater than the ability for the water to flow vertically.

7) One major issue that needs to be addressed is that horizontal flow was not established until a depth of 70 centimeters below the surface. With the current methods used to advance the probe and the testing protocol, test depths were severely limited.

8) The probe has the ability to report a negative $Q/H'$ ratio. A ratio of flow over corrected head that is negative is considered an anomaly as it implies that the head losses are greater than the head supplied above the probe. A negative ratio would correlate to a highly permeable soil.

9) The results of $Q/H'$ can also be outside the bounds of $10^{-3}$ to $10^{-6}$ square meters per second as stated in the French standards. These bounds are representative of typical values seen with the probe. Recent tests with the French probe report values of 3.0E-07 square meters per second.

10) Laboratory testing is practical for initial calibration of the probe but proves to be both time consuming and difficult. Field testing is an ideal method to further evaluate the Perméafor.

11) With further evaluation, the Perméafor will be applicable for shallow depth testing to report the changing hydraulic conductivity with depth. The results gained with the probe will aid in the design of septic systems, drainage, retention ponds, and many more.

There are significant improvements that would be imperative before testing is practical in everyday geotechnical practice.
5.3 Recommendations and Future Work

There are several different aspects of both the testing setup and testing procedures that need to be analyzed further. The most critical changes for Permeafor testing are as follows:

1) One of the major changes to the field testing setup includes the use of a percussion system to drive the probe into the subsurface. A major issue with testing in both the laboratory and the field was that water would continue to travel vertically instead of laterally into the soil. Although this issue is also present in the French probe, the half scale model is intended for shallow applications. A percussion system will likely collapse the soil around the probe to seal the screen area sooner. By sealing the hole created by penetration sooner the water will travel laterally and readings could be gained closer to the ground surface.

2) The water supply system for the probe is based on pressure head. If more pressure is needed, there is only a limited amount of additional elevation head which can be added. A pressurized water system is more practical for testing with the Permeafor. This system would allow for smaller head near the surface and for larger head when the probe is at greater depths.

3) Currently, flow into the probe is recorded using a floating ball flow meter. To capture the variations in flow throughout testing a video recorder was used. After testing the video was used to evaluate the flow at changing
depths. A more reliable system is desirable which includes a digital flow meter with a build-in data acquisition system.

4) Further laboratory testing should be conducted using variable soil. Creating a soil sample with seams of varying soil will further illustrate the ability for the probe to determine the changing hydraulic conductivity with depth.

5) Re-evaluation of the pocket coefficient for the half scale model may allow for a stronger correlation between the ratio of $Q/H'$ and hydraulic conductivity.

Testing conducted throughout this thesis was the first of its kind with the scale model of the Perméafor. All laboratory testing was completed using the original 2.5 inch screen. Throughout field testing the concept of increasing the size of the screen was introduced therefore, laboratory testing with the new screen was needed. Laboratory testing would be more practical if the sample size was changed. The current 55 gallon tank is not practical to create a sample due to the tapered side walls. A cylindrical sample would be more practical.

Lateral flow is assumed when water is no longer flowing to the surface. Testing needs to be conducted to prove that flow is actually going in the lateral direction using the half scale model. More tests at greater depths will also aid in the understanding of the vertical flow.

Additional field work is also needed to optimize the testing tool. In conjunction with both a pressurized water injection system and a percussion penetration system the field work will become more reliable.
LIST OF REFERENCES


Torterat, J. (2009). Développement d'un outil de mesure de la perméabilité des sols in situ: la sonde UNH.


APPENDIX A: Japanese Method for the Determination of Minimum and Maximum Void Ratio
To determine the minimum and maximum void ratio the Japanese method can be used. This method is valid for oven dried sands that have 100% passing the Number 10 sieve, and have more than 95% remaining on the Number 200 sieve. The test is conducted in a mold which has an inner diameter of 60 millimeters and an inner height of 40 millimeters. The soil is poured into the mold using a paper funnel with specified dimensions. Soil compaction is achieved using a standard rice hammer. The tools used in the Japanese method are shown in Figure A.1.

![Japanese Method Tools](image)

Figure A.1: Japanese Method Tools, (Left) Funnel, (Center) Rice Hammer, (Right) Mold with Extension

For the determination of the minimum void ratio and the maximum dry unit weight, sand is placed into the mold in ten lifts each containing about 20 grams of sand. To compact the soil the rice hammer is slid along a flat surface where it then strikes the outside of the mold. Consistent blows of the hammer are needed
by maintaining a constant frequency and amplitude. The mold should be struck at a rate of five hits per seconds, with an amplitude of about five centimeters. The hammer is allowed to slide along the table hitting the bottom of the mold five times then the mold is turned 45 to 90 degrees. This process is repeated until 100 blows have been completed for each lift for a total of ten lifts.

Once all lifts have been compacted the extension is removed from the mold. The sand is made level using a straight edge, and the mass of the soil with the mold is recorded. The mass of the soil is then calculated using Equation A.1.

\[ M_{soil} = M_{soil+mold} - M_{mold} \]  

where \( M_{soil} \) = mass of soil  
\( M_{soil+mold} \) = mass of soil in the Japanese mold  
\( M_{mold} \) = mass of Japanese mold

The dry unit weight of the soil is then determined using Equation A.2.

\[ \gamma_d = \frac{M_{soil}}{V_{mold}} \]  

where \( \gamma_d \) = dry unit weight (kg/m\(^3\) or lb/ft\(^3\))  
\( V_{mold} \) = volume of the Japanese mold (m\(^3\) or ft\(^3\))

Next, the volume of solids is determined using Equation A.3.

\[ V_{solids} = \frac{M_{solids}}{G_s \cdot \gamma_w} \]  

where \( V_{solids} \) = volume of solids  
\( G_s \) = specific gravity of soil  
\( \gamma_w \) = unit weight of water

Finally, the minimum void ratio can be determined using Equation A.4.
\[ e_{\text{min}} = \frac{V_{\text{mold}}}{V_{\text{solids}}} - 1 \]  \[\text{[A.4]}\]

where \( e_{\text{min}} \) = volume of solids

In conjunction with the determination of the minimum void ratio, the maximum dry unit weight is can established. As the void ratio is at its smallest, meaning that the soil is compacted the most, the maximum dry unit weight is was previously stated in Equation A.2. These steps are repeated through three trials to establish an average minimum void ratio and maximum dry unit weight.

The maximum void ratio and minimum dry unit weight tests are conducted in the same mold. Using the funnel, soil is allowed to pour into the mold from a constant height above the soil. Once the soil has reached above the extension of the mold, it can be removed and a straight edge is used to level off the soil. The mass of the mold and soil is then taken and Equation A.4 and Equation A.2 are used, respectively, to determine the now maximum void ratio, and minimum dry unit weight.
APPENDIX B: Preliminary Soil Testing Results
Sieve Analysis Sample 1

Figure B.1: Sieve Analysis for Sample 1

Table B.1: Sieve Analysis Results for Sample 1

<table>
<thead>
<tr>
<th>Percent Finer</th>
<th>Diameter</th>
<th>Coefficient of Uniformity, $C_u$</th>
<th>Coefficient of Gradation, $C_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$D_{10}$</td>
<td>0.10</td>
<td>$D_{60}/D_{10}$</td>
<td>$D_{30}/(D_{60} \times D_{10})$</td>
</tr>
<tr>
<td>$D_{30}$</td>
<td>0.20</td>
<td>$D_{50}/D_{10}$</td>
<td></td>
</tr>
<tr>
<td>$D_{60}$</td>
<td>0.28</td>
<td>2.95</td>
<td>1.50</td>
</tr>
</tbody>
</table>

Sieve Analysis Sample 2

Figure B.2: Sieve Analysis for Sample 2

Table B.2: Sieve Analysis Results for Sample 2

<table>
<thead>
<tr>
<th>Percent Finer</th>
<th>Diameter</th>
<th>Coefficient of Uniformity, $C_u$</th>
<th>Coefficient of Gradation, $C_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$D_{10}$</td>
<td>0.09</td>
<td>$D_{60}/D_{10}$</td>
<td>$D_{30}/(D_{60} \times D_{10})$</td>
</tr>
<tr>
<td>$D_{30}$</td>
<td>0.18</td>
<td>$D_{50}/D_{10}$</td>
<td></td>
</tr>
<tr>
<td>$D_{60}$</td>
<td>0.28</td>
<td>3.11</td>
<td>1.29</td>
</tr>
</tbody>
</table>
Figure B.3: Sieve Analysis for Sample 3

Table B.3: Sieve Analysis Results for Sample 3

<table>
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<tr>
<th>Percent Finer</th>
<th>Diameter</th>
<th>Coefficient of Uniformity, $C_u$</th>
<th>Coefficient of Gradation, $C_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$D_{10}$</td>
<td>0.10</td>
<td>$C_u = D_{60}/D_{10}$</td>
<td>$C_c = D_{30}^2/(D_{60} \cdot D_{10})$</td>
</tr>
<tr>
<td>$D_{30}$</td>
<td>0.20</td>
<td></td>
<td>1.40</td>
</tr>
<tr>
<td>$D_{60}$</td>
<td>0.30</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Figure B.4: Standard Proctor Compaction Test Results

Figure B.5: Modified Proctor Compaction Test Results
Hydraulic Conductivity Standard Compaction

- LEE Standard Compation Falling Head
- LEE Standard Compation Constand Head
- Poly. (LEE Standard Compation Falling Head)
- Poly. (LEE Standard Compation Constand Head)

Figure B.6: Hydraulic Conductivity Standard Compaction Test Results

Hydraulic Conductivity Modified Compaction

- LEE Modified Compation Falling Head
- LEE Modified Compation Constand Head
- Poly. (LEE Modified Compation Falling Head)
- Poly. (LEE Modified Compation Constand Head)

Figure B.7: Hydraulic Conductivity Modified Compaction Test Results
APPENDIX C: Perméafor Laboratory Testing Results
\[ \Delta H_{HT} = 0.049Q^2 + 0.406Q - 0.601 \]

C.1: Laboratory Head Losses for Permeafor

Number of Blows

C.2: Laboratory Dynamic Cone Penetrometer Testing
C.5: 10 Lift Soil Sample Soil Properties
C.6: 5 Lift Soil Sample $Q/H'$ with Time Test

C.7: 7 Lift Soil Sample $Q/H'$ with Time Test
C.8: 10 Lift Soil Sample $Q/H'$ with Time Test
No. of Blows

0 4 8 12 16

Q/H' (m²/s)

0 1E-06 1E-05 1E-04 1E-03

Vertical Flow Stopped

Depth (cm)

0 20 40 60 80 100

C.9: 5 Lift Soil Sample Q/H'

179
C.10: 7 Lift Soil Sample $Q/H'$
C.11: 10 Lift Soil Sample $Q/H'$
APPENDIX D: Permeafor Field Testing Results
Figure D.1: Field Head Losses for Permeafor

Dynamic Cone Penetrometer

Number of Blows

Depth (cm)

Figure D.2 Field Dynamic Cone Penetrometer Test Results
Figure D.3: Moisture Content Profile of Field Testing Site
Figure D.4: Field Testing Results with Permérafor Test 1
Figure D.5: Field Testing Results with Perméafor Test 2
Figure D.6: Additional Field Testing Results with Permeafor