GEORGIA DOT RESEARCH PROJECT 12-32

FINAL REPORT

Influence of Physical, Chemical and Biological Conditions on the Infiltration Rate of Highway Stormwater Runoff

Georgia Department of Transportation

OFFICE OF RESEARCH
15 KENNEDY DRIVE
FOREST PARK, GA 30297-2534

<table>
<tr><td>1.Report No.:
FHWA-GA-17-12-32</td><td>2. Government Accession No.:</td><td>3. Recipient's Catalog No.:</td></tr>
<tr><td colspan="2" rowspan="2">4. Title and Subtitle:
Influence of Physical, Chemical and Biological Conditions on the Infiltration Rate of Highway Stormwater Runoff</td><td>5. Report Date:
February 2017</td></tr>
<tr><td>6. Performing Organization Code:</td></tr>
<tr><td colspan="2">7. Author(s):
W. Randall Pettyjohn, Jr., Xenia Wirth, and Susan E. Burns</td><td>8. Performing Organ. Report No.:
RP 12-32;</td></tr>
<tr><td colspan="2" rowspan="2">9. Performing Organization Name and Address:
Georgia Institute of Technology
School of Civil&Environmental Engineering
790 Atlantic Drive
Atlanta GA 300332-0355</td><td>10. Work Unit No.:</td></tr>
<tr><td>11. Contract or Grant No.:
PI# 0011760</td></tr>
<tr><td colspan="2" rowspan="2">12. Sponsoring Agency Name and Address:
Georgia Department of Transportation
Office of Research
15 Kennedy Drive
Forest Park, GA 30297-2534</td><td>13. Type of Report and Period Covered:
Final; December 2012 – February 2017</td></tr>
<tr><td>14. Sponsoring Agency Code:</td></tr>
<tr><td colspan="3">15. Supplementary Notes:</td></tr>
<tr><td colspan="3">Double-ring infiltrometer tests were used to measure infiltration rates in situ at three sites in the Piedmont physiographic province and one site in the Coastal Plain physiographic province of Georgia. The accuracy of predicting saturated hydraulic conductivity for Piedmont soils from the published soil surveys of the National Resource Conservation Service (NRCS) and the derived pedotransfer functions (PTF) was then investigated.
The major findings of this study include:
• Predicted values from the NRCS overestimated every measured value of in situ saturated hydraulic conductivity when compared to field values.
• Predicted values from the ROSETTA pedotransfer software (primarily by soil grain size) more closely estimated the saturated hydraulic conductivity of the Piedmont soils.
• The infiltrometer, which is known to work well in the sediments of the Coastal Plain, also performed well in the soils of the Piedmont physiographic province; however, care must be taken to ensure proper installation in soils in the Piedmont.</td></tr>
<tr><td colspan="2">17. Key Words:
Infiltration, hydraulic conductivity, infiltrometer, Piedmont, Coastal Plain, partial saturation</td><td>18. Distribution Statement:</td></tr>
<tr><td>19. Security Classification (of this report):
Unclassified</td><td>20. Security Classification (of this page):
Unclassified</td><td>21. Number of Pages:
87

22. Price:</td></tr>
</table>

Form DOT 1700.7 (8-69)

GDOT Research Project No. 12-32

Final Technical Report

Influence of Physical, Chemical and Biological Conditions on the Infiltration Rate of Highway Stormwater Runoff

By

W. Randall Pettyjohn, Jr.
Xenia Wirth
and
Susan E. Burns
Professor

Georgia Institute of Technology
School of Civil Engineering

In Contract with

Georgia Department of Transportation

In cooperation with

U.S. Department of Transportation

February 2017

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

GLOSSARY

A = sample area (L^2)

a = area of standpipe containing inflow water (L^2)

a = insertion depth of infiltrometer inner ring

a = dimensionless parameter equal to 0.9084

C = experimental constant in empirical formulations

D = insertion depth of the ring (L)

d = diameter of inner ring

f = correction factor based on infiltrometer ring geometry and soil

G* = length term (L)

g = acceleration due to gravity (L/T^2)

H = total head (L)

ΔH = change in total head (L)

H_o = total height of soil (L)

H = height of ponded water in infiltrometer test

h = change in head (L)

h_p = pressure head (L)

h_v = velocity head (L)

h_z = elevation head above a datum (L)

I = infiltration (L)

i = gradient (-)

i = steady-state infiltration rate (L/T)

k = hydraulic conductivity (L/T, typically cm/sec)

GLOSSARY (Continued)

k_{FS} = field saturated hydraulic conductivity

k_x = hydraulic conductivity in the x direction

L = sample length (L)

l = change in distance (L)

Q = volume of flow (L^3)

q_s = steady-state flow rate from the DRI

r = radius of ring (L)

t = time (T)

u = water pressure (M/(LT^2))

v = average flow velocity (L/T)

z = layer thickness (L)

α = macroscopic capillarity length

γ_w = unit weight of water (M/(L^2T^2))

θ = water content

θ_r = residual water content

μ = viscosity

Φ = porosity

ψ = matric suction

ψ_{ae} = air entry pressure

EXECUTIVE SUMMARY

In this study, a two-phase investigation of the hydraulic conductivity parameters of silty soils was performed. In the first phase, double-ring infiltrometer tests were used to measure infiltration rates in situ at three sites in the Piedmont physiographic province and one site in the Coastal Plain physiographic province of Georgia. The accuracy of predicting saturated hydraulic conductivity for Piedmont soils from the published soil surveys of the National Resource Conservation Service and the derived pedotransfer functions (PTF) was then investigated. A PTF is a function that references an existing database of measured soil properties, such as grain size, and then uses that information to determine hydraulic properties of the soil. For example, utilizing this existing database, input parameters such as sand/silt/clay fraction and in situ density can be used to estimate hydraulic properties (Wösten et al. 2001).

Work was focused on the development of a consistent test methodology for all soils studied in the Piedmont (e.g., sandy silts and sandy clays) in the Piedmont, and the final test method utilized was the constant head test, using a double-ring infiltrometer with Mariotte tubes (devices incorporated into the infiltrometer to maintain constant head).

The major findings of this study include:

- Predicted values from the NRCS overestimated every measured value of in situ saturated hydraulic conductivity when compared to field values.

- Predicted values from the ROSETTA pedotransfer software (primarily by soil grain size) more closely estimated the saturated hydraulic conductivity of the Piedmont soils.
- The infiltrometer, which is known to work well in the sediments of the Coastal Plain, also performed well in the soils of the Piedmont physiographic province; however, care must be taken to ensure proper installation in soils in the Piedmont.

Recommended Guidance for Determining Infiltration Rate

It is recommended that infiltration rate and saturated hydraulic conductivity initially be estimated based on known characteristics of the soils in the area. Correlation of the soil type with typical ranges of hydraulic conductivity should provide a first pass estimate. These values can then be compared to the database values obtained from the USDA's Natural Resources Conservation Service (NRCS) database, although it is important to note these database values tended high when compared to field measured values in this study. This study determined that the use of the pedotransfer function, ROSETTA (https://cals.arizona.edu/research/rosetta/), yielded infiltration and conductivity values that were in reasonable agreement with the field measured values; consequently, ROSETTA is recommended as a tool to apply for the first level estimate of infiltration values in the field. If the estimate is greater than the minimum rate allowed, then follow up testing should be performed to confirm site feasibility. The preferred field tests for determining infiltration rate include the borehole test and the infiltrometer test.

Recommended Resources and Tests for Determination of Infiltration and Saturated Hydraulic Conductivity Values

1. Site Feasibility Screening Infiltration and hydraulic conductivity estimated from: a) Soil classification b) NRCS database c) ROSETTA pedotransfer function
If a site is feasible for infiltration, conduct field infiltration tests.
2. Field Infiltration Testing a) Infiltrometer tests can easily be performed in most soils in the Coastal Plain and can be performed with care in the Piedmont. b) Borehole tests can be performed in virtually all soils.

ACKNOWLEDGEMENTS

The authors are grateful to the ongoing support, thoughtful guidance, and reviews of Mr. Jon D. Griffith, P.G., P.E., of the Georgia Department of Transportation. His insights have been valuable to the success of this work. The authors are also grateful to Mr. Brad McManus, P.E., Hydraulics and Hydrology Group Leader, of the Georgia Department of Transportation. His ongoing comments and reviews are much appreciated.

1.0 INTRODUCTION

The infiltration rate of a given soil is important to the proper design and sizing of stormwater infiltration structures such as infiltration trenches, which are an environmentally friendly low-impact construction practice. The infiltration rate for a given area is known to vary both as a function of distance and time (short and long-term conditions), making a selection of reasonable design values a difficult task. Overestimating the design infiltration rate results in the non-conservative design and undersizing infiltration structures, with possible detention overflow and direct stormwater discharge to receiving streams. Underestimating infiltration rates results in oversizing infiltration structures and corresponding construction and maintenance cost increases. Consequently, accurate estimation of the design infiltration rate of any facility that receives stormwater is extremely important. Infiltration is significant in reducing the amount of required stormwater runoff detention set forth in the Georgia Department of Transportation (GDOT) MS4 permit and the GDOT *Manual on Drainage Design for Highways*. Additionally, infiltration is a mechanism that naturally attenuates contaminants that may be present in stormwater runoff. Natural attenuation is an additional benefit.

The infiltration rate is a complex function of the hydraulic conductivity of the near surface soils and the hydraulic gradient of the infiltrating water, and some simulations have suggested that the geometry of the infiltration basin also impacts the rate at which the fluid flows into the subsurface (Massman, J.W., 2003). While the mechanics of hydraulic flow are relatively straightforward, the infiltration rate in the field is complicated by the existence of multiple factors, including layered and trending heterogeneous hydraulic conductivity of the surface soils, the effects of partial saturation, the soil type with respect to mineralogy, and the presence of biofouling and siltation of the infiltration structure lining. Groundwater mounding can also significantly impact infiltration rates.

Urbanization is typically accompanied by significant increases in the impervious surface area such as paved streets and highways, lawns, roofs, and many other surfaces disturbed by human activity. As a result, as stormwater runoff flows over low permeability surfaces, it accumulates chemicals, debris, and solid insoluble particles (total suspended solids, TSS), as well as a variety of biological contaminants, such as fecal matter. Consequently, interest in researching methods to efficiently and economically treat stormwater runoff has increased over the past decades. In the past, stormwater runoff was considered a nuisance to be dealt with by using detention facilities (WEF et al. 2012); however, a recent shift in thinking has emphasized stormwater runoff as a potential resource to be returned to groundwater, where it can be filtered and returned to receiving waterways. This shift in the approach of stormwater treatment is the basis for low-impact development (LID) as practiced in modern construction, and stormwater best management practices (BMPs) limits the impacts on natural water flows and cycles (WEF et al. 2012). BMPs include facilities used by GDOT, such as infiltration trenches and ponds.

The ASCE (2012) Manual of Practice (MOP) defines an infiltration BMP as a stormwater BMP that treats the design volume by allowing stormwater runoff to infiltrate into the native soil and shallow aquifers, where it can then make its way to the groundwater system, and eventually discharge as baseflow into receiving streams. Infiltration BMPs typically consist of a layer of gravel or coarse sand to store the captured volume overlaying the native soil (Figure 1).

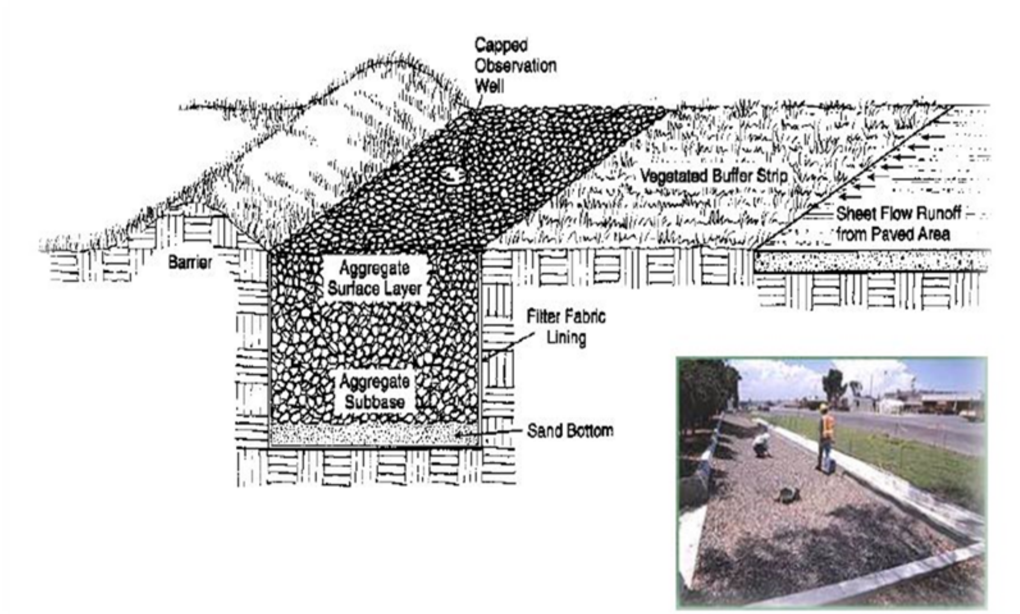


Figure 1. Cross section of a generic infiltration trench.
(Figure from Georgia Stormwater Manuel Volume 2).

Currently, infiltration trenches are designed with a vegetated buffer to enhance sedimentation and filtration, with the goal of reducing the quantity of suspended solids in the stormwater runoff before it enters the trench. The aim is to reduce the solids loading to slow the rate of clogging of the coarse fill in the infiltration trench. Additionally, geotextiles are used to establish a filter layer to further reduce transport of suspended solids into the infiltration trench. Sample details for typical infiltration structures used by GDOT are included in APPENDIX B.

The objective of this project was to develop a comprehensive approach to determining the infiltration rates throughout the State of Georgia with the ultimate goal of developing guidance to aid in selecting the most reasonable design values for the infiltration rate.

The work performed in this project includes:

1) Comprehensive literature review that examines the factors that control the infiltration rate in a variety of soil types. This review was focused on field methods of estimating and measuring the unsaturated and saturated hydraulic conductivity of soils, laboratory methods of measuring the unsaturated and saturated hydraulic conductivity, methods of providing the best estimate of the hydraulic gradient. Additionally, this review was focused on methods of assessing the effects of biological fouling including microbial and clutter accumulation, the impact of layering and siltation in infiltration ponds, and the grouping of soils by soil texture and shrink-swell potential for comparison of wetted versus dry infiltration rates that can be anticipated in the field.
2) Bench-scale testing (e.g., sand-tank modeling) to study the impact of physical, chemical, and biological process variables under controlled laboratory conditions. The results are included in Appendix C.
3) Field testing to assess best practices and procedures for field determination of infiltration rates in a variety of soil types.
4) Statewide guidance on estimating infiltration rates and site suitability for infiltration structure locations.

2.0 LITERATURE REVIEW

2.1 Georgia Geology

The state of Georgia is divided into four primary physiographic provinces: Valley and Ridge, Blue Ridge, Piedmont, and Coastal Plain (Figure 2). In some classifications, a fifth province, known as the Appalachian Plateau, is also included, although its areal extent in the state is limited to a small area in the northwest corner of the state, adjacent to the Alabama and Tennessee borders. The Valley and Ridge province and the Blue Ridge province are located in the northern portion of the state, which is adjacent to the Tennessee border. The Piedmont physiographic province covers the central portion of the state and extends south from its border with the Valley and Ridge and Blue Ridge provinces to the fall line. The fall line in Georgia, which is coincident with the historic northernmost shoreline during the transgression of the Atlantic Ocean in the Mesozoic era, runs from Columbus through Macon to Augusta. The Coastal Plain province covers the southern portion of the state from the fall line to Georgia's borders with Alabama, Florida, South Carolina, and the Atlantic Ocean.

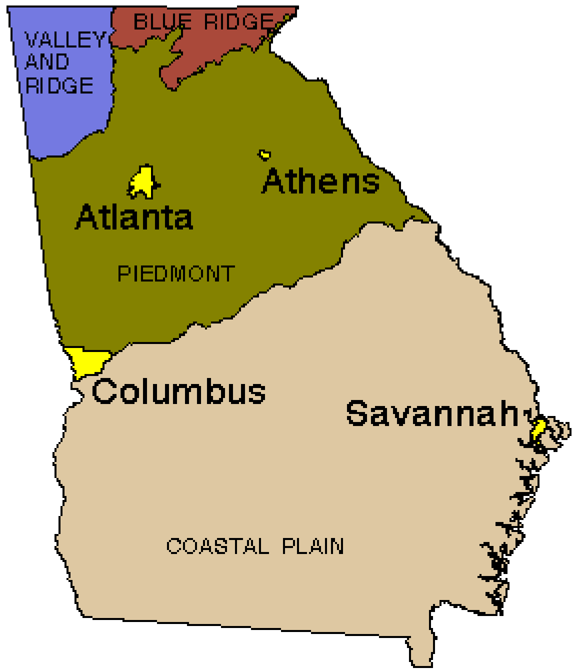


Figure 2. Primary physiographic provinces in the state of Georgia: Valley and Ridge, Blue Ridge, Piedmont, and Coastal Plain. Note: the Appalachian Plateau province is grouped with the Valley and Ridge province (northwest corner of the state). Figure credit: https://en.wikipedia.org/wiki/Geology_of_Georgia_(U.S._state) from Wikimedia Commons, the free media repository

Each province is defined by a characteristic geology, including bedrock and soil types. The Valley and Ridge Provinces are composed primarily of sedimentary rocks that were subjected to folding and faulting during tectonic collisions (Gore and Witherspoon, 2013). Soils in the Valley and Ridge province are heterogeneous and vary as a function of location, with the ridges covered primarily by thin veneers of soil overlying erosion-resistant hard bedrock, while the valleys are composed of soils that weathered from softer bedrock such as limestone, shale, and sandstone (Gore and Witherspoon, 2013). The Blue Ridge and the Piedmont provinces are composed primarily of metamorphic and igneous rocks that were also deformed during tectonic movements (Gore and Witherspoon, 2013). Soils in these geologic provinces consist of iron rich residual soils and partially weathered bedrock that has undergone physical and chemical weathering in place, and typically range from particle sizes of fine sands to silts. The Coastal Plain consists primarily of sediments overlying deep bedrock. Soils in the Coastal Plain province consist of sands, silts, and clays that were eroded, transported, and deposited over geologic timescales.

2.2 Saturated Flow in Soils

The direction of groundwater flow through soil is governed by the difference in total energy, or total head, and can be calculated according to Bernoulli's equation. The total head at any point is comprised of three components, including pressure head, elevation head, and velocity head, according to the following:

$$H = h_z + h_p + h_v = h_z + \frac{u}{\gamma_w} + \frac{v^2}{2g}$$

Where H = total head (units = L or length), u = water pressure ($M/(LT^2)$), v = velocity (L/T), g = acceleration due to gravity (L/T^2), γ_w = unit weight of water ($M/(L^2T^2)$), h_p = pressure head (L), h_z = elevation head above a datum (L), and h_v = velocity head (L). In soils, the velocity head of flow is so low it can be neglected under most circumstances, and Bernoulli’s Equation simplifies to:

$$H = h_z + h_p = h_z + \frac{u}{\gamma_w}$$

In addition to the direction of flow, the velocity of hydraulic flow in soils is also important. The velocity can be quantified using Darcy’s law for one-dimensional flow:

$$v = k\frac{\Delta H}{l} = ki$$

Where v = average flow velocity (L/T), k = hydraulic conductivity (L/T), ΔH = change in total head (L), l = change in distance (L), and i = gradient.

2.2.1 Hydraulic Conductivity

The essential information needed to estimate infiltration rate at a site includes the hydraulic parameters of the in situ soil, with the soil type and saturated hydraulic conductivity being the two most basic pieces of information required. Although the hydraulic conductivity and infiltration rate are not synonymous, the hydraulic conductivity is the physical property most commonly referred to when evaluating rates for infiltration BMPs. The hydraulic conductivity represents the ease of flow of water through a soil medium and depends on the viscous drag on the fluid by the particle surfaces (Santamarina et al. 2001). As the pore space in a graded soil is filled with finer particles, the volume available for fluid conduction decreases and the surface area that contributes to viscous drag increases. Quite often, determining the saturated hydraulic conductivity of a soil can be challenging. In situ measurements of hydraulic conductivity yield coefficients of variation as high as 400% (Reynolds et al. 2002).

Grain size and index parameters can be used to estimate the order of magnitude hydraulic conductivity values. Similarly, using correlations to estimate hydraulic conductivity, or tabular values of hydraulic parameters based on the soil classification, hydraulic conductivity values can be estimated. However, unsaturated parameters are required for many of the classical infiltration rate equations, including air entry pressure, water content at saturation, and residual water content. These unsaturated parameters are required to estimate the effects of capillarity on the infiltration rate before saturation is achieved.

Multiple properties impact the hydraulic conductivity of soils, including the following major factors: degree of particle size grading, and the particle shape, i.e., the degree of sphericity and rounding. For example, increased soil densities are obtained with the introduction of smaller grains into a matrix of larger grains. If the density, which is closely related to the void ratio, of the large particles is held constant, the addition of smaller particles into the intergranular pore space will increase the global density of the mixture, which is a function of the particle size ratio between the individual constituents of the mixture (McGeary 1961). The hydraulic conductivity of particle mixtures affects engineering behavior of groundwater transport, mining applications, slurry walls, and filter media.

Obtaining these hydraulic parameters economically is a challenge due to the extensive testing required for both saturated and unsaturated soils. Consequently, in practice, the required parameters typically are first estimated based on known characteristics of the soils in the area. As the project proceeds, in situ or lab tests are then performed to validate the estimations. This two-pronged approach is commonly found in municipal stormwater manuals and recent publications by the WEF and ASCE. Typical values for saturated hydraulic conductivity are summarized in **Table 1**.

Table 1. Ranges for Typical Saturated Hydraulic Conductivity Values for Soil (after Freeze and Cherry, 1979 and Budhu, 2011)

Soil Type	**Hydraulic Conductivity (cm/sec)**
Gravel (GW and GP)	1×10^{-1} to 1×10^{1}
Clean Sand (SW and SP)	1×10^{-3} to 1×10^{0}
Sand Mixtures (SM and SC)	9×10^{-6} to 9×10^{-2}
Silts (MH and ML)	1×10^{-7} to 1×10^{-4}
Clay (CH and CL)	1×10^{-10} to 1×10^{-7}

The hydraulic parameters required for site design can be obtained in the following order (Massman 2003; WEF et al. 2012): first: obtain estimates by referencing published soils data or historical site information, and second, if the estimate is greater than the minimum rate allowed follow up with in situ or lab testing to confirm site feasibility. The minimum acceptable rate for infiltration is determined by local municipalities, with values varying as a function of regional geography (**Table 2**). The additional design steps are carried out once the rate is measured and the feasibility of the site for construction is determined.

Table 2. Minimum Saturated Hydraulic Conductivity Values for Infiltration BMPs

Minimum Infiltration Rate (in/hr)	Organization
0.5	Atlanta (Georgia) Regional Commission
0.4	ASCE / WEF
0.2	Minnesota Pollution Control Agency
0.1	Pennsylvania Department of Environmental Protection

Good design requires review of the methods for estimation of conductivity parameters, and of the measurement techniques that are available in practice today. Additionally, ensuring a solid scientific understanding of the infiltration process is fundamentally important. A review of methods of hydraulic conductivity testing follows.

2.2.2 Laboratory Determination of Saturated Hydraulic Conductivity

Hydraulic conductivity is measured in laboratory tests on soil samples that are intact or reconstituted after sampling (common for sands). Lab-testing methods include the constant head test, where the gradient driving fluid flow through the soil does not change with time, and a constant pressure is exerted on the water flowing through the soil. Constant head testing is commonly used for tests performed on coarse-grained soils. For fine-grained soils, a falling head test is commonly performed. In the falling head test, the hydraulic gradient changes with time and decreases as the test progresses.

In the constant head test, the volume of flow is proportional to the following: the area of the sample, the velocity of the water moving through the soil, and the time, and can be determined according to:

$$k = \frac{QL}{hAt}$$

Where Q = volume of flow (L^3), L = sample length (L), h = change in head (L), A = sample area (L^2), t = time (T), and k = hydraulic conductivity (L/T, typically cm/sec). For a falling head test, the head difference in the test is constantly changing, which means that the velocity is a function of time, and the velocity must be written as a differential function of the head, resulting in the following relationship:

$$k = \frac{aL}{A\Delta t}\ln\frac{h_1}{h_2}$$

Where a = area of standpipe containing inflow water (L^2), and h_1 = head at the start of the test (L), and h_2 = head at the end of the test (L).

2.2.3 Hydraulic Conductivity in Layered Soils

Hydraulic flow also occurs in layered soils, and is frequently modeled as flow parallel to the soil layers or as flow perpendicular to the soil layers. Modeling as flow through parallel layers means that the gradient is the same at all points, and equivalent hydraulic conductivity is determined as:

$$k_{x(eq)} = \frac{1}{H_o}(z_1 k_{x1} + z_2 k_{x2} + ... + z_n k_{xn})$$

Where H_o = total height of soil, z = layer thickness, and k_x = hydraulic conductivity in each layer. For hydraulic flow that is perpendicular to the soil layers, head loss in each soil layer will sum to total head loss, and equivalent hydraulic conductivity is determined according to:

$$k_{x(eq)} = \frac{H_o}{(\frac{z_1}{k_{x1}} + \frac{z_2}{k_{x2}} + ... + \frac{z_n}{k_{xn}})}$$

2.2.4 Field Determination of Saturated Hydraulic Conductivity

Hydraulic conductivity tests can also be performed in the field, which is advantageous because the in situ test measurements include the impact of the actual pore fluid, field gradient, temperature, any flaws such as macropores, and field boundary conditions. Multiple types of field tests have been developed, with the borehole test and the infiltrometer test being most commonly implemented.

For a borehole test, a well is drilled and casing is placed within the well. The annular space between the casing and borehole is sealed with grout. After the well has been developed, water can be pumped in to maintain a constant head, while recording volume of inflow as a function of time, which is known as a constant head test. Alternately, the test can be performed as a falling head test, where the water level in the well is raised above the water table and the drop in water level is measured as a function of time.

An infiltrometer test can be performed as either a single ring or double ring configuration. In the infiltrometer test, a ring is embedded in the soil to be tested, and it is sealed with a bentonite grout. Next, the ring is filled with water, and the rate at which water infiltrates into the ground is measured. In the double ring configuration, the rate of infiltration of water is measured in the center ring, which is assumed to be primarily one-dimensional flow. Infiltrometers can also be sealed to limit evaporation in arid climates or long running tests. At steady state, the rate of infiltration approaches the saturated hydraulic conductivity of the soil. Infiltration is determined assuming that flow is driven by ponded water at the soil surface.

Percolation tests are also used to measure the rate at which water percolates into a soil. Percolation testing is carried out by measuring the rate that water flows through a soil, with no ponding at the surface. In the percolation, or perc test, a small hole is excavated and filled with water. The test is performed as a falling head test, and the change in water level over time is measured as a function of time. The test results are reported as percolation rate.

2.2.5 Empirical Correlations for Estimations of Saturated Hydraulic Conductivity

The hydraulic conductivity of a soil can also be estimated from empirical relationships. One of the most commonly used correlations for hydraulic conductivity is Hazen's formula, which was developed for sands (Hazen, 1930):

$$k = CD_{10}^2 \quad \text{k (cm/s)}$$

where k is in cm/sec when D_{10} is in mm, C is a constant that varies from 0.4 to 1.4, typically assumed C = 1.0.

Another commonly used empirical relationship is Taylor's equation (1948):

$$k = \frac{\gamma}{\mu} D_{50}^2 \frac{Ce^3}{1+e}$$

where γ = unit weight of pore fluid, μ = viscosity, and C = experimental constant.

It is important to note that both Taylor and Hazen's formulas are empirical relationships that should only be applied with caution.

2.3 Infiltration in Partially Saturated Soils

Most commonly in infiltration BMPs, the infiltration of water will occur when the filter media and surrounding soils are not fully saturated (Barbu and Ballestero 2014). Consequently, the initial infiltration period is governed by the influence of capillarity on early time infiltration rates. Capillarity is influenced by the surface tension of the fluid, as well as the contact properties of the solid and the liquid. For an initially saturated soil, the pressure required for air to enter the void space is referred to as the air entry pressure, ψ_{ae}. The air entry pressure is dependent on the radius of the pore openings and on the grain size of the soil, and increases as the pore size decreases. Consequently, air entry values are much higher for clays and silts than for sands and gravels (Cho and Santamarina 2001; Lu and Likos 2004). The air entry phase typically occurs during the capillary phase, in which the bulk soil is still primarily saturated and capillary bridges occur between particles (Urso et al. 1999). Meniscus geometry for coarse-grained particles exhibits several stages of wetting, including the capillary, funicular, and pendular bridges which show distinct structure as a function of water content (**Figure 3**).

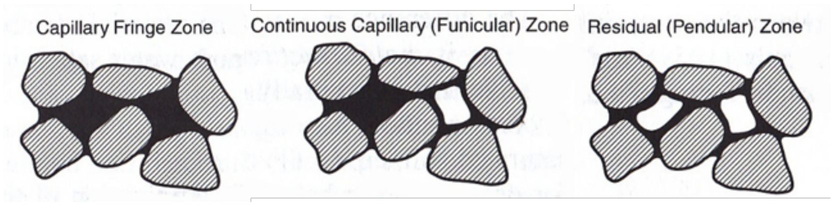


Figure 3. Saturation phases are shown. From left to right the saturation phases represented are the capillary, funicular, and pendular phases. Figure from Lu and Likos 2004.

Measurement techniques for matric suction most commonly involve the use of tensiometers in the field, and pore water extraction tests using a pressure plate apparatus in the lab. ASTM D6836 offers guidance in selecting the appropriate apparatus based on the soil type being tested. Methods using a centrifuge have also been proposed to increase the speed of obtaining the parameters necessary to measure the SWCC (Zornberg et al. 2010).

There are numerous models relating matric suction to water content. One of the most commonly implemented models was proposed by van Genuchten (1980):

$$\theta = \theta_r + \frac{\phi - \theta_r}{\left[1 + \left(\frac{\psi}{\psi_{ae}}\right)^n\right]^{1-\frac{1}{n}}} = \theta_r + \frac{\phi - \theta_r}{[1 + (\alpha\psi)^n]^m}$$

where ψ = matric suction, ψ_{ae} = air entry pressure, Φ = porosity, θ = water content, and θ_r = residual water content. The variable n is a function of grain size distribution and m = 1 – 1/n. The van Genuchten parameters can be fit to experimental values of matric suction measured in the lab, estimated by grain size distribution using one of many proposed pedotransfer functions (PTFs), or referenced in tabulated values by soil type.

The result of matric suction on the infiltration process is that water is pulled by the matric suction of the drier soils, in addition to gravity flow from the ponded water at the soil surface (Ferguson 1994). This combined driving gradient can be explained using Darcy's law combined with the matric suction into the total head:

$$q_z = -k\frac{dh_t}{dz}$$

The terms q_z = flow rate through a given cross sectional area of soil in the vertical (z) direction, k = coefficient of hydraulic conductivity, and h_t = total head, the total head in unsaturated flow is a function of both gravity flow and matric suction. Buckingham (1907) modified this form of Darcy's equation to represent unsaturated flow:

$$q_z = -k(\theta)\left[1 + \frac{d\psi(\theta)}{dz}\right]$$

Evaluating Darcy's law for unsaturated conditions, it can be seen that one-dimensional flow for unsaturated conditions is a function of both gravity flow and matric suction. The degree of matric suction depends on the water content of the soil and the soil type.

Richard's equation is obtained by taking the derivative of the above equation for one-dimensional flow, with respect to the z-direction, and combining Darcy's law and the conservation of mass. Richard's equation is the basic theoretical equation describing infiltration of water into a homogenous soil mass (Dingman 2008). The one-dimensional version of Richards's equation is expressed below as follows:

$$-\frac{\partial \mathrm{k}(\theta)}{\partial \mathrm{z}} + \frac{\partial}{\partial \mathrm{z}}\left[\partial \mathrm{k}(\theta)\frac{\partial \psi(\theta)}{\partial \mathrm{z}}\right] = \frac{\partial \theta}{\partial \mathrm{t}}$$

Richard's equation is non-linear, without closed form analytical solutions; however, it can be used for numerical modeling of infiltration by applying boundary conditions, initial conditions and then solving the equation for thin layers for small time changes (Dingman 2008).

2.4 Infiltration Models in Partially Saturated Soils

Due to the complexity of the forces governing infiltration and hydraulic conductivity in partially saturated conditions, many approximate solutions have been developed to obtain closed form solutions for infiltration rates. A summary of these methods follows.

2.4.1 Horton Equation

The Horton equation is an empirical model used in practice by stormwater designers (Ferguson 1994). The Horton equation is expressed simply in terms of initial infiltration rate (f_0), final infiltration rate (f_p), time (t), and an empirical constant (k):

$$f(t) = f_p + \left(f_o - f_p\right)^{-kt}$$

It is typically considered that f_p approaches the saturated hydraulic conductivity (k_{sat}) at steady state infiltration rates. The benefit of using the Horton equation is its simplicity. It is easily fitted to experimental data; however, the Horton equation is purely empirical and has no physical basis. Closed form analytical solutions are often needed for inclusion in hydrologic models (Dingman 2008; Lu and Likos 2004), which is one of the motivations for developing and applying empirical models.

2.4.2 Green and Ampt Model

The Green and Ampt model (1911) assumes a sharp, uniformly propagating wetting front (z_f), constant water contents above (Φ) and below (θo) the wetting front, and that the matric suction directly under the wetting front (ψ_f) is greater than the ponded water height (H) (Dingman 2008; Ferguson 1994; Lu and Likos 2004). Given cumulative infiltration (F(t)) as an input parameter, a non-linear expression can be solved iteratively to determine a value of ψ_f (Dingman 2008).

$$ln\left(1-\frac{F(t)}{(\phi-\theta_o)\psi_f}\right)=\frac{K_{sat}t-F(t)}{(\phi-\theta_o)\psi_f}$$

The depth of the wetting front (z_f) and infiltration rate can then be solved.

$$z_f=\frac{F(t)}{(\phi-\theta_o)}$$

2.4.3 Philip's Model

The Philip's model offers a simplified solution to Richards equation based on an infinite series solution for ponded water infiltrating into an indefinitely deep soil (Dingman 2008; Lu and Likos 2004):

$$f(t)=\frac{S_p}{2}t^{-\frac{1}{2}}+A_2+A_3\,t^{\frac{1}{2}}+A_4t+\cdots+A_n{}^{\frac{n}{2-1}}$$

Typically, only the first two terms are considered and A_2 is treated as k_{sat}, or the final infiltration rate:

$$f(t)=\frac{S_p}{2}t^{-\frac{1}{2}}+K_p$$

The terms K_P and S_p are the final infiltration rate, and sorptivity, defined as:

$$S_p=\left[(\phi-\theta_o)K_{sat}|\psi_{AE}|\left(\frac{2b+3}{b+3}\right)\right]^{\frac{1}{2}}$$

The term b is a constant related to the grain size distribution, and ψ_{ae} is obtained from the SWCC. It is common practice to estimate S_p and K_p as empirical parameters by fitting the values to measured infiltrometer data. The model works particularly well with the spatial variability of infiltrometer data (Dingman 2008; Ferguson 1994).

Practitioners commonly use tabulated input parameters by soil type to make initial estimates of infiltration rates (Ferguson 1994). Rawls et al. (1983) and Rawls et al.

(1982) have developed tables based on soil type that provide input parameters specifically for the Green-Ampt equation and SWCC, respectively.

Table 3 summarizes the advantages, disadvantages and required input parameters for the three infiltration models discussed above (Bedient et al. 2013; Dingman 2008; Ferguson 1994; Lu and Likos 2004). These three models include the models applied most frequently in literature, textbooks, and design manuals from professional practice. All three models provide a similar fit to the data when compared with numerical solutions based on Richard's equation, assuming a constant water supply (Hsu et al. 2002). There are numerous infiltration models in the literature and in practice, and a review of additional infiltration models can be found in EPA report EPA/600/R-97/128b (Chen et al. 1998).

Table 3. Summary of Three Commonly Used Infiltration Rate Equations

Model	Assumptions	Input Parameters	Advantages	Disadvantages
Green-Ampt (1911)	Uniform wetting front	k_{sat}, porosity, initial water content, cumulative infiltration	Few input variables required	Cumulative infiltration required input parameter
Horton (1940)	Empirical	Based on measured data	Simple	Empirical
Philip (1957)	Smooth wetting front	k_{sat}, air entry pressure, b, porosity, initial water content	Predicts cumulative infiltration, works well with infiltrometer data	Does not theoretically hold for time approaching zero and infinity, many input parameters required

2.5 Infiltration Determination: Saturated and Unsaturated

It has been shown that the infiltration rate is a controlled by both capillarity and saturated hydraulic conductivity. However, saturated hydraulic conductivity alone is the most often used parameter for the design rate for infiltration BMPs (WEF et al. 2012). This design approach neglects the unsaturated phase, which is a conservative approach because the saturated infiltration rate is always slower than the unsaturated rate of infiltration (Massman 2003). Despite this widespread practice, the following section will review methods for estimating and measuring both the saturated and unsaturated hydraulic conductivity parameters.

2.5.1 Published Soils Data

Soil Survey data available through USDA-NRCS is a tool used for initial estimates of soil properties. All the historical data from the NRCS soil surveys has been digitized by county and is available via the Web Soil Survey. Estimates of hydraulic conductivity referenced by NRCS data are based on texture and bulk density, but also take into account overriding parameters such as macropore flow (Arrington et al. 2013). The validity of using NRCS data to estimate saturated hydraulic conductivity for stormwater applications has been investigated by Fedler et al., who compared NRCS estimates to measured values for ten counties in the state of Texas. It was found that the data from NRCS did not correlate with the field measured values (Fedler et al. 2012). Arrington et al. (2013) found that NRCS predicted values had a lower root mean squared error when compared to measured values and values found in tables used by stormwater practitioners (Arrington et al. 2013).

2.5.2 Pedotransfer Functions

Hydraulic properties can also be estimated by pedotransfer functions (PTFs). A PTF is a function that references an existing database of measured soil properties, and uses this information to determine hydraulic properties of the soil. For example, utilizing this existing database, input parameters such as sand/silt/clay fraction and in situ density can be used to estimate hydraulic properties (Wösten et al. 2001).

Another widely used source is a table based on USDA soil textural classification developed by Rawls et al. (1982). Measured data from 1,323 soils with 5,350 horizons from 32 states were used as a basis (Rawls et al., 1982). Regression analysis was used to estimate the hydraulic properties of the eleven standard USDA soil types. The hydraulic properties included porosity, residual saturation, effective porosity, air entry pressure, pore size distribution, water retained at -0.33 bar and -15 bar, and saturated hydraulic conductivity (Rawls et al., 1982). Saxton et al. (1986) developed a model based on the same data that could estimate additional hydraulic properties by using sand/silt/clay fraction as the only input parameter. Rawls et al. (1992) developed a look-up table that estimated saturated hydraulic conductivity based on USDA soil types (Arrington et al. 2013). This table was updated in 1988 to include soil density and porosity categories as input parameters (Arrington et al. 2013, Rawls et al. 1998). Saxton and Rawls (2006) updated the original model from 1986 to include the updated regression equations from Rawls et al. (1998), organic matter (OM) as an input parameter, and the larger database of the USDA-NRCS soil survey as a reference. This updated model is available in a software package, Soil-Plant-Atmosphere-Water (SPAW), which is free to download via the USDA Hydrolab. In addition to hydraulic conductivity, SPAW provides the properties necessary to estimate the SWCC. The estimated SWCC curve can be used to determine parameters for Green – Ampt and Philip's equations to estimate the infiltration rate.

The ROSETTA model is another software package developed to predict hydraulic properties. For a detailed explanation of the ROSETTA process, see Appendix D in this report and https://cals.arizona.edu/research/rosetta/. ROSETTA utilizes four soil descriptors (texture, grain size, bulk density, and water retention) to hierarchically to estimate saturated and unsaturated hydraulic conductivity, and for estimation of the van Genuchten parameters (Schaap et al. 2001). It is based on neural network analysis that allows uncertainty estimates to be provided, which is useful when no measured values of k_{sat} are available for comparison (Schaap et al. 2001). The data set is composed of 2,134 soil samples from temperate to sub-tropical climates in North America and Europe; of those samples, 1,306 provide saturated hydraulic conductivity values (Schaap et al. 2001).

The validity of using PTFs to estimate hydraulic parameters has been investigated for stormwater applications. Values estimated using a PTF, Precision Agriculture-Landscape Modeling System (PALMS), yielded a lower root mean squared error (RMSE) than published values by Rawls et al. (1998), when compared to measured values for a study in Dane County, Wisconsin (Arrington et al. 2013). Values predicted using Saxton et al. were equally higher and lower than 28 measured values from ten counties in Texas (Fedler et al. 2012).

2.5.3 In situ Methods

The previous methods for estimating hydraulic properties of soils are useful for first cut estimates, but these values are based primarily on texture and do not take into account disturbance and site variability. A geotechnical investigation is necessary to confirm the conditions in situ are appropriate for infiltration BMPs. There are many methods to determine infiltration rate in situ, but the most common methods found in stormwater practice and literature include ring infiltrometers (single or double) (Bouwer 1986; Reynolds et al. 2002, ASTM D3385) and borehole testing (Bouwer and Rice 1976; Brown et al. 1995).

<u>Borehole Tests</u>

Borehole infiltration tests are widely employed for in situ measurements of hydraulic conductivity (Reynolds 2013). Bouwer and Rice proposed a method for slug test data analysis when groundwater is encountered in unconfined aquifers (Bouwer and Rice 1976). This method was shown to produce less error than alternative analysis techniques, such as the Hvorslev method (Brown et al. 1995). The United States Bureau of Reclamation (USBR) methods are used in the geology, water management, and engineering applications, while borehole permeameter (BP) methods are often used in agricultural and environmental sciences (Reynolds 2013). It has been shown that BP methods provide more accurate results than the USBR method for most scenarios (Reynolds 2013). Borehole tests can be relatively time consuming and expensive when taking into account the boring and casing required. A more economical alternative is a ring infiltrometer.

<u>Infiltrometer tests</u>

A ring infiltrometer consists of either a single or double concentric ring configuration. It is driven into the ground in a manner that minimally disturbs the soil, but is deep enough to prevent side-wall leakage. Falling head and/or constant head tests can then be performed. A concentric ring set up, or double-ring infiltrometers (DRI), is used to mitigate the impact of lateral flow (Reynolds et al. 2002). The measurements from a DRI are taken only from the center ring, with the annular space accounting for the lateral flow. The flow lines beneath a double ring infiltrometer, through the outer annular space, are vertical and lateral, as opposed to the flow lines below the center ring, which are primarily vertical (Figure 4).

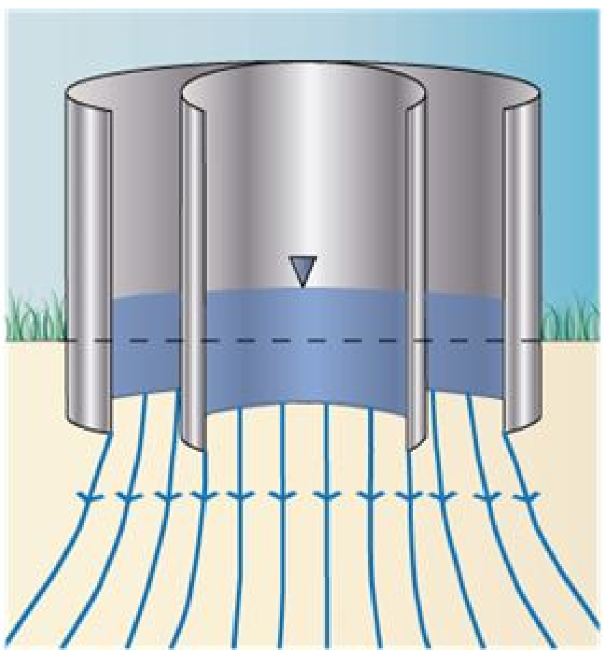


Figure 4. Schematic of a double-ring infiltrometer. The flow lines show the potential for lateral flow around the perimeter of the annular space.

Gregory et al. (2006) performed DRI tests in Northern Florida. Three DRI configurations were evaluated on a residential construction development. An ASTM standard, a Turf-tech 15 and 30 cm DRI with a constant head, and Turf-tech 15 and 30 cm DRI with a falling head were run on a single lot, predevelopment. The differences between the results of the constant head test of the smaller DRI and of the ASTM standard method were not statistically significant. The results of the falling head test with the smaller DRI had an unacceptably high COV (Gregory et al. 2006).

In a study in Auburn, AL, a DRI with 15 and 30 cm rings, similar to the device studied by Gregory et al. (2006), was fitted with a pressure transducer near the bottom of the center ring, and falling head tests were then performed. The results for the modified DRI were consistently lower than those from results using the ASTM standard method (Arriaga et al. 2010).

It has also been shown that using smaller infiltrometers can yield inaccurate results. Lai and Ren (2007) found that the variability of results decreased with an increase in inner ring diameter, using both numerical and experimental results. The larger center ring allows a better chance of capturing the heterogeneity of the soil and subsequently measuring a more stable hydraulic conductivity. An inner ring diameter of at least 80 cm was recommended (Lai and Ren 2007). The buffer index, the size ratio between the outer and inner rings, was also investigated. It was shown that the buffer index and the inner ring size affect the accuracy of the results. The inner ring size was a more important factor to consider (Lai et al. 2010). The effect of the embedment depth on the accuracy of results was also investigated. Numerical and experimental results from six different outer and inner ring insertion depths show that continuously increasing the depth of insertion would improve the accuracy. But driving the infiltrometer deeper will further disturb the soil and hence affect the accuracy of the results. The authors recommend in insertion depth of between 5 and 15 cm, or approximately the same as is recommended in ASTM D3385 (Lai et al. 2012).

The smaller inner ring diameter has been shown to decrease the accuracy of the results. However, an 80 cm inner ring diameter would be difficult to implement in the field for practical purposes. Of the aforementioned 15/30, 30/60 and larger infiltrometers used in the Lai et al. studies, the 30/60 offers a larger inner ring diameter within a practical size range.

ASTM D3385 recommends running the test for 6 hours or until steady-state values are observed. These steady-state values are then taken as the saturated hydraulic conductivity. This approach does not take into account head due to ponded water, depth of ring insertion, or the ring geometry. Reynolds et al. (2002) proposed a model that takes the aforementioned parameters into account:

$$\boldsymbol{k_{FS}} = \frac{\boldsymbol{q_s}}{\left[\frac{\boldsymbol{H}}{\boldsymbol{C_1 d + C_2 a}}\right] + \left\{\frac{\boldsymbol{1}}{[\boldsymbol{\alpha(C_1 d + C_2 a)}]}\right\} + \boldsymbol{1}}$$

where the k_{FS} = field saturated hydraulic conductivity, q_s = steady-state flow rate from the DRI, H = height of ponded water, d = diameter of inner ring, a = insertion depth of inner ring, α = macroscopic capillarity length, C_1 and C_2 are empirical constants.

Pilot Infiltration Test

Another method for measuring in situ hydraulic conductivity is a pilot infiltration test (PIT). A PIT consists of excavating a pit to the depth of the potential BMP, and filling the pit with water to a fixed depth. The test is performed with a constant head, and the amount of water required to maintain the water level is monitored. The technique is based on guidance provided by Massman (2003). The PIT overcomes some of the scaling error inherent in measurements made using a smaller apparatus, such as the DRI. Massman (2003) found that measurements made using the PIT method were on average 50 times lower than estimates provided by Hazen's correlation with D_{10} and regression equations based on grain size distribution. Some of the PIT tests were carried out in existing BMPs, so biofouling or physical clogging could have contributed to the disparity in rates (Massman 2003).

Factor of Safety

Factors of safety (FS) for measured rates vary. The U.S. EPA suggests using an FS between 25 and 50 (Philips and Kitch 2011). The State of Washington suggests FS based on the type of soil, and additional considerations such as frequency of inspection and maintenance. The FS range from 5.5 to 18 (Massman 2003). While WEF and ASCE recommend FS between 3.3 and 2, as well as modifiers based on soil type (WEF et al. 2012).

2.6 Infiltration in the Piedmont Physiographic Region

The Piedmont physiographic province begins in central Alabama and passes through Georgia and continues northeast to the northern tip of Virginia. Metamorphic and igneous rocks from the Precambrian and Paleozoic eras make up the primary bedrock in virtually this entire region (NRCS 2014, https://www.nrcs.usda.gov/). The soils in this region are saprolitic, formed from the in-place weathering of this bedrock. The upper portion of the soil is typically classified as silty-fine sand (SM) or low plasticity silt (ML) with less frequent occurrences of clayey sand (SC), sandy clay (SC) and plastic sandy silt (MH) (Finke et al. 2001).

There is limited literature available on stormwater infiltration in the Piedmont region. A study conducted by the U.S. Air Force and the U.S. Atomic Energy Commission (dissolved in 1975) investigated the infiltration of water into basins referred to as disposal pits located in Dawson County, Georgia. Rates were measured by continuously pumping water into the basins and maintaining constant heads of one, two, and three feet for several days. The measured infiltration rates ranged from 1.8×10^{-4} – 3.9×10^{-4} cm/s (Stewart 1964).

Ellington and Ferguson (1991) used a computer model from 1990 to simulate the effectiveness of replacing existing detention stormwater BMPs for two sites in the Piedmont region of Georgia. The simulations used data generated for a 50-year storm to show that both sites could use infiltration to reduce peak discharge to below pre-development levels at a significant cost savings (Ellington and Ferguson 1991). However, only saturated hydraulic conductivity data from published soil maps were used to estimate the final infiltration rates. No lab or in situ tests were carried out to validate these rates.

Another study monitored the infiltration rates of three BMPs that depended on infiltration in the Charlotte area of North Caroline utilizing three different infiltration BMPs. The BMPs consisted of a pervious pavement, a bio-retention pond and an infiltration basin. Preconstruction infiltration rates were measured using a DRI and subsequent infiltration rates were monitored using pore pressure transducers installed in PVC monitoring wells to measure change in water levels over several months post construction. Preconstruction infiltration rates varied from 1.7×10^{-4} – 2.2×10^{-4} cm/s. Post construction rates averaged at 9.9×10^{-5} cm/s. The decrease in post construction rates was attributed to construction activities; i.e. compaction from equipment and clogging of subgrade materials such as gravel and geotextile filters (Estes 2007).

3.0 FIELD INFILTRATION TESTING

Field hydraulic conductivity tests using the double ring infiltrometer were performed in the Piedmont and Coastal Plain Physiographic providences of Georgia. These tests were performed to measure infiltration rates in areas being considered for construction of infiltration trenches as stormwater BMPs. This experimental field-based investigation measured infiltration rates in situ to compare with published and predicted values for the Piedmont and Coastal Plain providences of Georgia. Application of infiltration in the Valley and Ridge providences and Blue Ridge providences is reduced by the presence of large rocks in the surficial soil, which causes leakage at the infiltrometer boundaries and can damage the infiltrometer during installation.

3.1 Materials – Infiltration Testing

Four sites were chosen for testing: three of those sites were located in the metro Atlanta region, and one was located on Skidaway Island, Georgia. The sites in metro Atlanta included Covington, GA, Lawrenceville, GA, and the GDOT sand filter in Canton, GA. Samples were taken from each site, for classification and index testing (Table 4). The rows are titled by acronyms for the site and test pit number; i.e., TPC1 is test pit Covington #1.

Table 4. Soil Description from Test Pits from In situ Testing Sites

Test Pit	**Depth (m)**	**Soil Type**	
		USDA	USCS
TPC1	0.6	Sandy loam – loamy sand	SM
TPC2	0.15	Sandy clay loam	CH
TPC3	0.3	Loam	MH
TPL1	1	Sandy Clay Loam	CL
TPL2	1	Sandy Clay	CH
TPL3	1	Sandy Clay Loam	SC
TPL4	1	Sandy Clay	CH
Canton	0.5	Silty Sand	SM
Skidaway	0.4	Sand	SW

Grain size distributions were determined for samples taken from the test pit in Covington and Lawrenceville according to ASTM D-422. The samples were taken from the bottom of the excavated pits. The soils tested ranged from sandy soils to silts to clays (Figure 5).

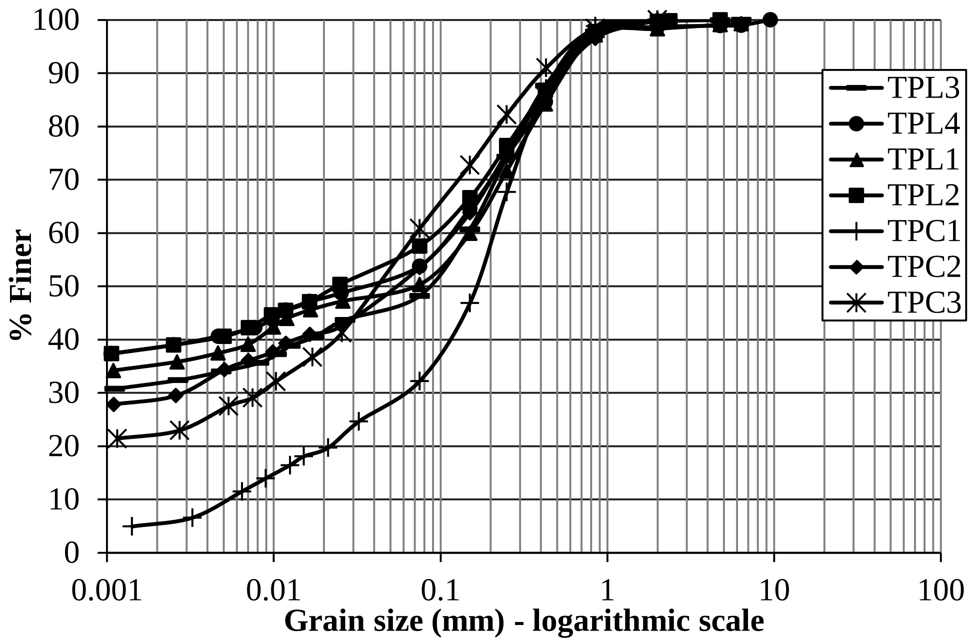


Figure 5. Grain size distributions are shown for the materials recovered from test pits during site work.

3.2 Methods – Infiltration Testing

Infiltration tests were performed according to ASTM D-3385, which recommends double-ring infiltrometer (DRI) ring diameters of 12 inches (30.5 cm) and 24 inches (61.0 cm) for the inside and outside rings, with an embedment depth of between two to four inches (5.1 – 10.2 cm) to prevent sidewall leakage. For this study, the 30/60 configuration was selected (IN14-W Heavy Duty infiltrometer, Turf Tec International). The infiltrometer used for the tests was composed of 14 gauge galvanized steel.

3.2.1 Covington GA Infiltration Test

The first site work was performed in Covington, GA. The lot was provided by Georgia Department of Transportation and was located at an intersection, where the right-of-way had been extended to include a site with relatively undisturbed pine forest for a future project, at the intersection of Highway 142 and Alcovy Road. Georgia 811 was contacted and the utilities were marked on the site before any work commenced.

Pits were excavated to avoid testing the hydraulic conductivity of top soil or fill and to avoid macropores created by root systems and other biological activity. The pits were excavated by manually using a shovel and pickaxe. After the pits were excavated and leveled, the DRI was placed on the ground, a steel driving plate was placed on top, a 2 x 12 (5.1 x 30.5 cm) piece of yellow pine was placed on top of the driving plate, and a 12 pound (5.4 kg) sledge hammer was used to drive the infiltrometer into the ground. An average embedment depth of 6 cm was achieved. Lines were then connected to Mariotte tubes that regulated the head in the infiltrometer. The Mariotte tubes were then driven into the ground beside the DRI so that the water heads within the inner and annular rings would be within 5 mm. The Mariotte tubes were then filled with tap water. A 15 cm soil thermometer was then installed outside the infiltrometer to monitor ground temperatures for the duration of the test. Water was siphoned from a barrel to initially fill the DRI to the desired level. The Mariotte tubes were then opened. Time was started as soon as the water level stabilized within the DRI, and readings were taken every fifteen minutes for the first four readings. Time increments for the remainder of the test were based on the rates recorded. More frequent readings were made for higher rates. Each test was run for a minimum of six hours (Figure 6). The first three trials had to be terminated within the first few hours due to sidewall leakage. This was alleviated by placing bentonite around the perimeter of the DRI and tamping into place to minimize leakage.

Figure 6. Test set up for in situ measurement of saturated hydraulic conductivity using a double-ring infiltrometer.

3.2.2 Lawrenceville, GA Infiltration Test

Access to the Lawrenceville, GA site was provided by Bowen & Watson Inc. An excavator was available on this site and was used to excavate four test pits. There was a layer of gravelly sand fill approximately 60 cm deep at each test pit. Hence, the pits were excavated to approximately one meter. The excavator was also used to drive the infiltrometers into place for testing. Initial water content samples were taken before testing began. All procedures were identical to the test methods followed at the Covington site, and were followed after installation of the infiltrometer. Final water contents were taken after test termination (Figure 7). For the Lawrenceville site, the excavation, manual clearing of cuttings and leveling of the pits, and the installation of the DRIs were accomplished in one day. All four infiltration tests were completed the next day. The disturbance due to driving the infiltrometers was considerably reduced at this site by using the excavator for insertion of the infiltrometer, compared to the previous site, which required between one and two hours to drive the DRIs with a sledge hammer, balanced with intervals of wetting the soil.

Figure 7. Two test pits located at the Lawrenceville, GA site are shown. A test set up is also shown in one of the test pits

For this test location, some changes were made to the operation of the Mariotte tubes. Stainless steel sleeves to hold the tubes in place during testing were fabricated, and a wing nut was welded to the sleeve to allow the height of the Mariotte tube to be adjusted, rather than driving the Mariotte tube into the ground. An auger bit was advanced into the soil for up to 3 inches (7.6 cm). The sleeve was then driven the remaining distance using a rubber mallet. This configuration provided a more automated method for adjusting the heads in the infiltrometers at the beginning of the test.

Soil samples from each pit were collected for index testing. The Atterberg limits, grain size distribution, and water contents were measured for each test pit at each of the two sites. The liquid limit was determined using methods described in BS 1377. A correlation by Feng (2004) was used to calculate the plastic limit.

3.2.3 Canton, GA Infiltration Test

Infiltrometer tests were performed in the GDOT sand filter BMP at the intersection of GA 20 and I-575 in Canton, GA. A double-ring infiltrometer was used as to measure the saturated field hydraulic conductivity. The sand filtration and drainage system, which was constructed to control highway stormwater run-off from I-575, has been in place for 10 years. The site is characterized by a thick layer of coarse sand overlain by a geotextile filter fabric (to mitigate fines migration). A 4 inch (10 cm) thick layer of top soil and native Georgia grasses covers the entire site (Figure 8). Root penetration through the geotextile fabric was observed, but overall the geotextile fabric is well-preserved.

Figure 8. Site view of GDOT storm water run-off control facility in Canton, Georgia with DRI equipment.

An IN14-W Heavy Duty 14 gauge galvanized steel double-ring infiltrometer (Turf Tec International) was used to perform the double-ring infiltrometer test. Because embedment would damage the geotextile filter fabric, the double-ring infiltrometer was placed directly on top of the filter fabric (did not penetrate the fabric). Topsoil and vegetation were cleared from the fabric before the infiltrometer was placed. To seal the outer ring, the outer ring edges were buried 2 – 4 inches (5-10 cm) deep in saturated fine-grained soil from the site. A bentonite layer was packed on top of the soil seal, and more topsoil was placed to hydrate the bentonite. Finally, well-graded play sand was added to a depth of 6 inches (15 cm) was added to increase the effective confining stress (Figure 9). The outer edges of the inner ring were sealed in a similar manner, except no sand was added. The packing reached a depth of approximately 4 inches (10 cm), and care was taken to only pack the edges of the inner ring. A thin layer of water was placed on top to saturate the bentonite. The inner edges of the inner ring were sealed with saturated fine grained soil around the edges, but no bentonite was added. Two falling head tests were performed. During the second falling head test, breakout occurred between the inner and outer rings. As soon as the water levels equilibrated between the rings, the test was restarted as a single-ring infiltrometer test with a diameter of 24 inches (60 cm). Measurements of water in the inner ring were taken at ten-minute increments until either the ponded water completely infiltrated the soil or the head level ceased to drop. Both tests finished in less than two hours.

Figure 9. Falling-head DRI test in coarse filter sand in Canton, Georgia.

3.2.4 Skidaway Island, GA Infiltration Test

An IN14-W Heavy Duty 14 gauge galvanized steel double-ring infiltrometer (Turf Tec International) was used to perform a double-ring infiltrometer test to measure saturated field hydraulic conductivity of coastal Georgia sands on Skidaway Island, a barrier island off the coast of Georgia (Xu et al. 2012). The tests were performed at the Saltmarsh Environment Research Facility (SERF) site on the west side of Skidaway Island, an unconfined freshwater aquifer lens overlaid by wooded forests upland and surrounded by an extensive salt marsh subjected to diurnal tidal flow. The soils at Skidaway Island were mostly clean sands upland, with highly organic muds in the salt marsh regions and higher contents of clay at the aquitard/aquifer interface at the SERF site. A thin layer of nutrient-rich topsoil supports the wooded areas at higher elevations from the marsh beds. The soil profile at the test pit was summarized as: (1) a thin layer of organic, nutrient-rich topsoil from 0-0.6 m BGS; (2) a thick layer of fine sand extending at least 4 m deep. The precise geographic location of the test pit was 31 58.520 N, 81 01.766 W.

A test pit large enough to hold the infiltrometer was dug deep enough to eliminate all top soil, root systems or other plant debris using a shovel and saw. The rings were then embedded an average depth of 6 inch (15 cm) into the soil using plywood and a sledgehammer. Bentonite was tamped along the edges of the inner and outer rings to seal the disturbed sand and minimize preferential water flow and leakage. A falling head test was performed using water from the nearby estuary (Figure 10). The water level in the outer ring was kept constant at 2 inch (5 cm) while the inner ring was filled completely with water. Measurements of water level in the inner ring were taken at ten-minute increments until the ponded water had completely infiltrated. Each test finished in less than three hours.

Figure 10. Falling-head DRI test in sandy coastal soils on Skidaway Island.

4.0 RESULTS and ANALYSIS

4.1 Infiltration Measurements at Covington and Lawrenceville Test Sites in Piedmont Physiographic Province

Results of the double ring infiltrometer infiltration tests performed on the Piedmont soil in Covington GA are given in Figure 11. Data are shown for Test Pit 2 and Test Pit 3 only, as the data gathered for Test Pit 1 were considered unreliable. The measured rates for TPC2 and TPC3 were relatively constant over the entire measuring interval, and were attributable to the wetting of the surface during test setup to aid in driving the infiltrometers. Consequently, the resulting measurements represent saturated hydraulic conductivity, with relatively little impact of capillarity during the initial operation of the test. The infiltration rates in these soils reached a steady state value of between $1 – 2 \times 10^{-4}$ cm/sec, which is consistent with previous measurements in similar soils (Estes 2007; Stewart 1964).

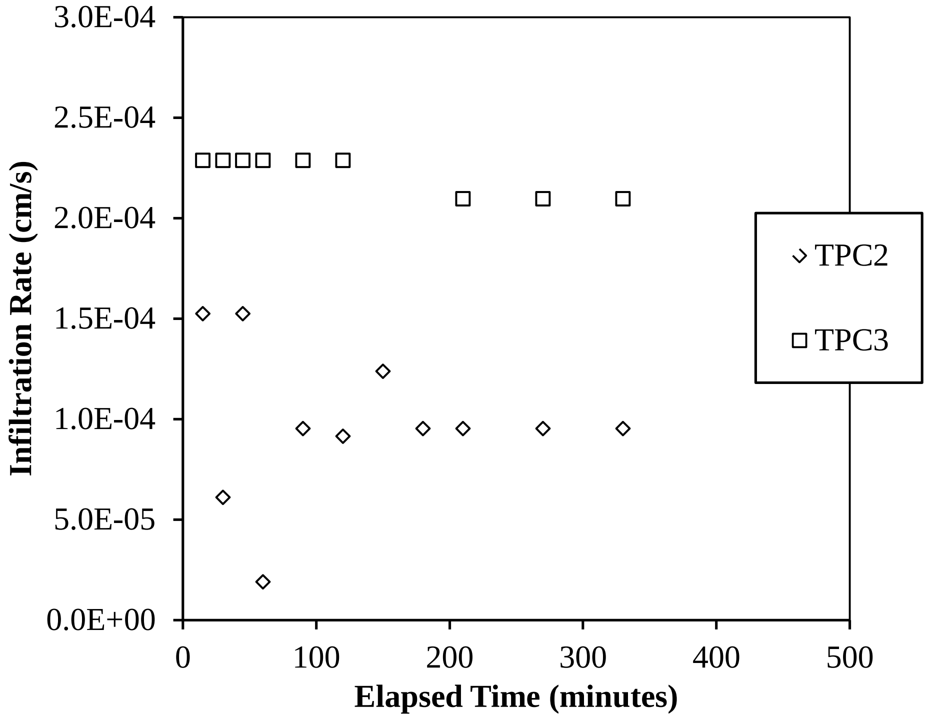


Figure 11. In situ infiltration rates in Covington GA measured using a double-ring infiltrometer. Piedmont physiographic region.

Infiltration rates measured for the four test pits at the Lawrenceville site demonstrated the clear influence of capillarity, with infiltration rates decreasing as elapsed time progressed (Figure 12). Steady state infiltration rates measured at the Lawrenceville site ranged over an order of magnitude, from a low of 1×10^{-5} cm/sec (Test Pit 1) to a high of 3×10^{-4} cm/sec (Test Pit 3). This level of variability in hydraulic parameters is not uncommon in soils of saprolitic origin. The saturated hydraulic conductivity measured during a double ring infiltrometer test is typically taken to be the final, or steady-state, rate of infiltration.

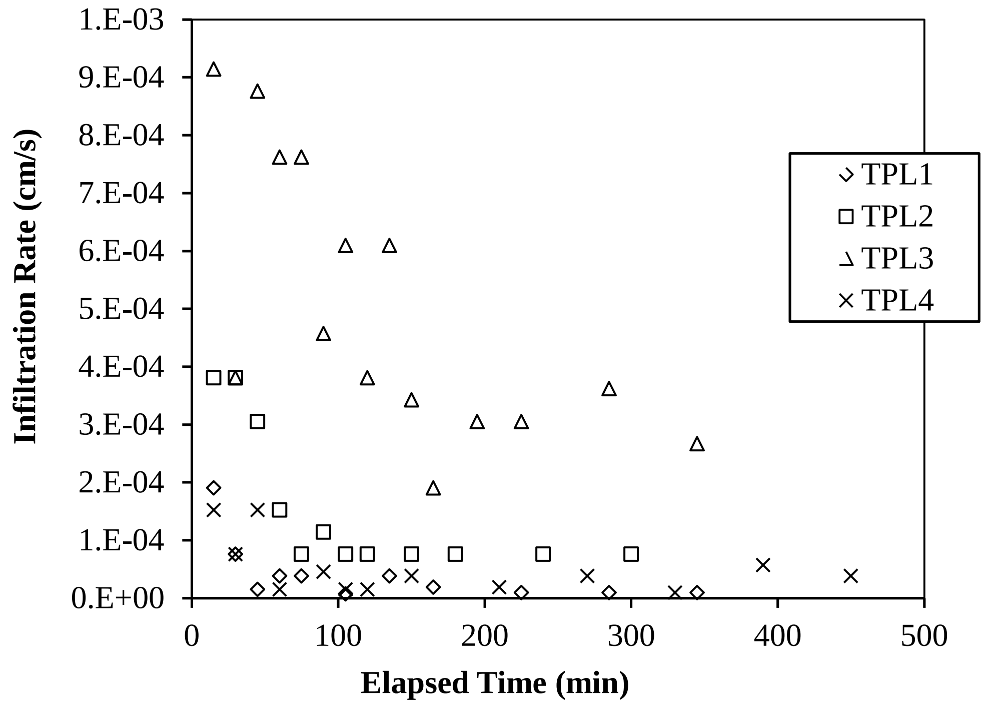


Figure 12. In situ infiltration rates in Lawrenceville, GA measured using a double-ring infiltrometer. Piedmont physiographic region.

4.2 Infiltration Measurements at Canton Test Site in Sand Filter

For the sand filter in Canton, GA, saturated hydraulic conductivity was measured using one double-ringed infiltrometer test and one single-ring infiltrometer test that was performed in the same test pit. The average steady state infiltration rate was measured from trend lines fitted to the water level drop over time data (Figure 13). Saturated hydraulic conductivity was then determined from the infiltration rate and other water retention curve and soil parameters referenced by Xu et al. (2012) (Table 1). The average vertical hydraulic conductivity for the combination of filter fabric and coarse sand using this method was 35.1 cm/s (Table 5).

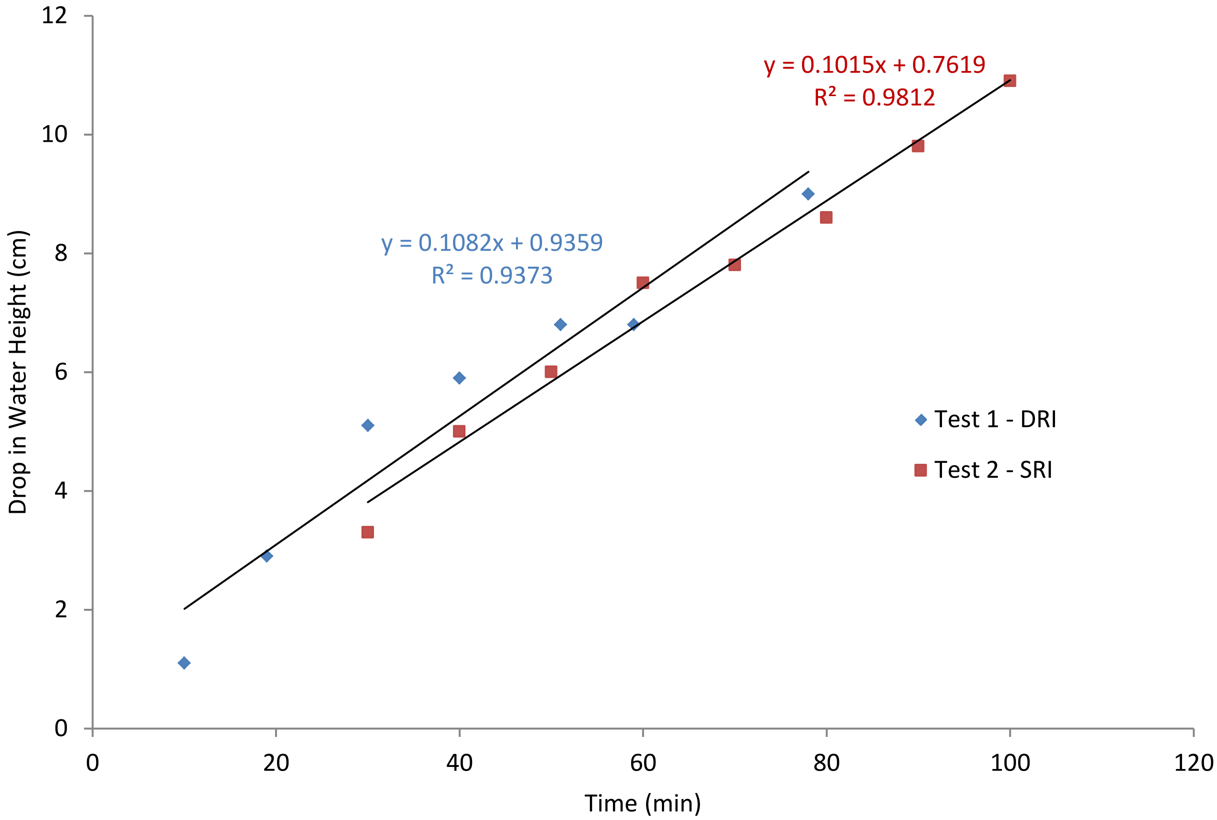


Figure 13. Steady state infiltration rates measured in a falling head test performed using a double-ringed infiltrometer (1) and a single-ringed infiltrometer (2) at a GDOT sand filter site, Canton, GA.

Table 5. Hydraulic Conductivities from the Infiltration Tests: GDOT Sand Filter Site, Canton, GA

Test	Infiltration Rate (cm/min)	Infiltration Rate (cm/s)	Hydraulic Conductivity (cm/s)
1 (DRI)	0.108	6.492	17.85
2 (SRI)	0.102	6.090	52.32

Saturated vertical hydraulic conductivity was calculated using the Wu2 method for a steady-state infiltration rate. This method has been used for both double ring and single ring infiltrometer tests (Wu et al. 1997). A linear equation was fit to the steady-state infiltration data, and parameters from the equation are used to calculate saturated hydraulic conductivity (Xu et al. 2012).

$$I = it + c = afK_s t + c$$

$$f = \frac{H + \frac{1}{\alpha}}{G^*}$$

$$G^* = d + \frac{r}{2}$$

$$K_s = \frac{1}{af}$$

where i: steady-state infiltration rate (cm/s); I: infiltration (cm); H: initial head (cm); d: insert depth of the ring (cm); f: a correction factor based on ring geometry and soil; r: radius of ring (cm); G*: length term (cm); a: a dimensionless parameter equal to 0.9084; and α: inverse soil macroscopic capillary length (sand =0.36 cm^{-1}) (Xu et al. 2012).

The derivation of hydraulic conductivity from infiltration rate was first developed by Reynolds and Elrick 1990) based on application of Darcy's Law describing flow rate through a soil column. Ponded water within the ring was modeled by mass balance, where flow rate out of the ring has two components, a gravity flux and a hydrostatic capillarity flux (Reynolds and Elrick 1990). The complexity of the solution was a result of the boundary conditions imposed by the flow geometry constraints of ponded water held inside a ring, instead of a point source (Reynolds and Elrick 1990).

Infiltration tests at the Canton sand filter resulted in saturated hydraulic conductivity values that correspond to medium to coarse sands. Water flowed rapidly through the geotextile fabric into the underlying coarse sand layer, which gives evidence that the sand filter has not clogged after ten years of service.

4.3 Infiltration Measurements at Skidaway Island, Coastal Plain

Saturated hydraulic conductivity was measured for three double-ringed infiltrometer tests performed in the same test pit. The average steady state infiltration rate was measured from trend lines fitted to the water level drop over time data using the same theory as applied at the Canton sand filter site (Figure 14). Infiltration rates calculated using this approach fluctuated from a high of 29.7 cm/s to a low value of 13.1 cm/s. There was a noticeable decrease in infiltration rate over time. Saturated hydraulic conductivity was then calculated from the infiltration rate and other soil parameters referenced by Xu et al. (2012) (Figure 14). The average vertical hydraulic conductivity for the coastal sediments using this method was 36.9 cm/s.

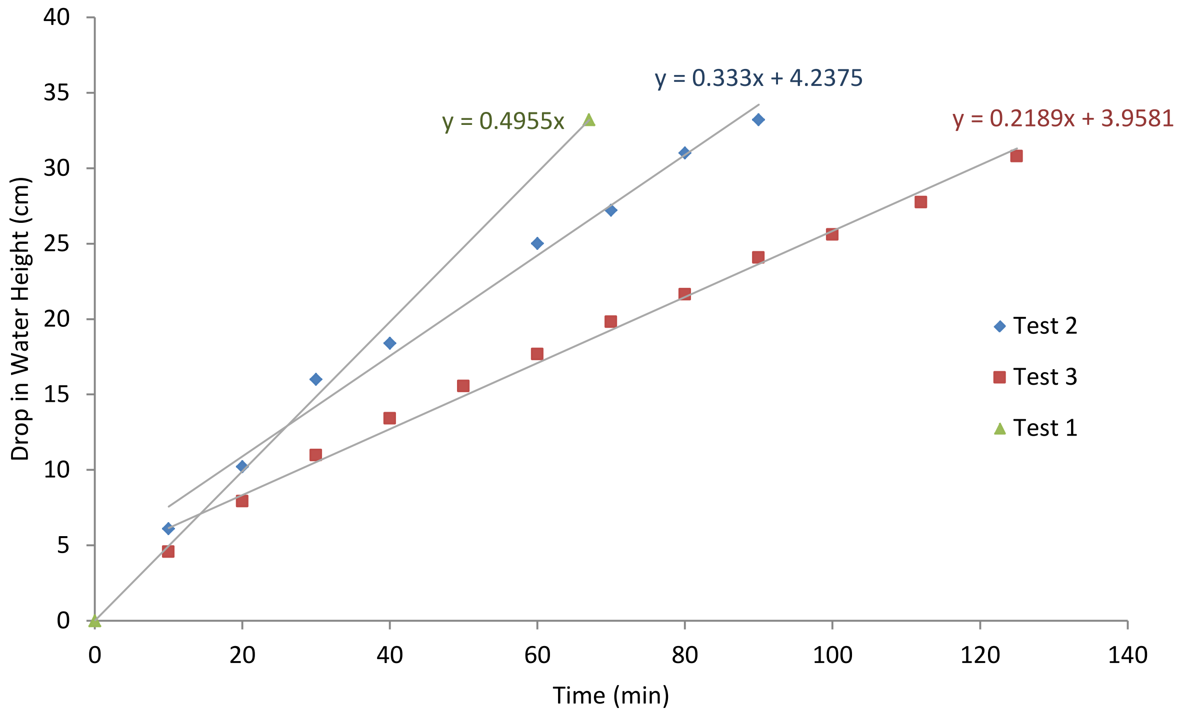


Figure 14. Steady state infiltration rates measured in a falling head test performed using a double-ringed infiltrometer at the SERF Site on Skidaway Island. Coastal Plain Physiographic region.

4.4 National Resource Conservation Service (NRCS) Database Comparison and Model Predictions at Covington and Lawrenceville Test Sites

4.4.1 NRCS Database and USDA ROSETTA Model

The Covington site and the Lawrenceville site were chosen for further analysis and comparison with data obtained from soil databases and from commonly used models. The National Resources Conservation Service (NRCS) database was used for reference, and the US Department of Agriculture’s ROSETTA model was used to predict conductivity using the sand, silt, and clay contents of the tested soils as inputs to the model. Comparison of the in situ measured values at Covington and Lawrenceville with the NRCS database and the values predicted using the ROSETTA model show that the NRCS figures over predicted the field measured values (Table 6). However, the ROSETTA model, using the soil grain size distribution curve as input, gave values that were more closely aligned with the field measured values (Table 6).

Table 6. Measured and Predicted Values of Hydraulic Conductivity at Covington and Lawrenceville

Test Pit	Saturated Hydraulic Conductivity, k_{sat} (cm/s)		
	NRCS Database Value	Measured In situ	ROSETTA Predicted Value
TPC2	2.8×10^{-3}	9.5×10^{-5}	3.3×10^{-4}
TPC3	9.0×10^{-4}	2.4×10^{-4}	7.8×10^{-5}
TPL1	9.0×10^{-4}	9.5×10^{-6}	1.2×10^{-4}
TPL2	9.0×10^{-4}	7.6×10^{-5}	1.1×10^{-4}
TPL3	9.0×10^{-4}	2.6×10^{-4}	1.2×10^{-4}
TPL4	9.0×10^{-4}	3.8×10^{-5}	1.2×10^{-4}

4.4.2 Philip's Parameters

In addition to the ROSETTA model, the experimental data were also fitted to the Philip's Equation for infiltration. The Philip's parameters were treated as empirical constants and estimated by the measured infiltrometers data using the method of least-squares (Figure 15 through Figure 18, Table 7). Reasonable agreement was found for the four test pits in Lawrenceville; however, the relatively sharp decrease in measured data for Test Pit 1 resulted in an unrealistically high sorptivity value. The measured infiltrometer data from Covington were recorded under saturated conditions due to repeated wetting in order to advance the infiltrometers, hence determining sorptivity parameters would be physically meaningless.

Table 7. Least-Squares Estimates of Philip's Parameters, Lawrenceville Test Site

Test Pit	Philip's Equation Parameters	
	Sorptivity S_p (cm/s$^{1/2}$)	K_p (cm/s)
TPL1	1.75×10^{-3}	9.42×10^{-6}
TPL2	2.29×10^{-3}	1.00×10^{-6}
TPL3	7.22×10^{-3}	9.01×10^{-5}
TPL4	7.75×10^{-4}	3.99×10^{-6}

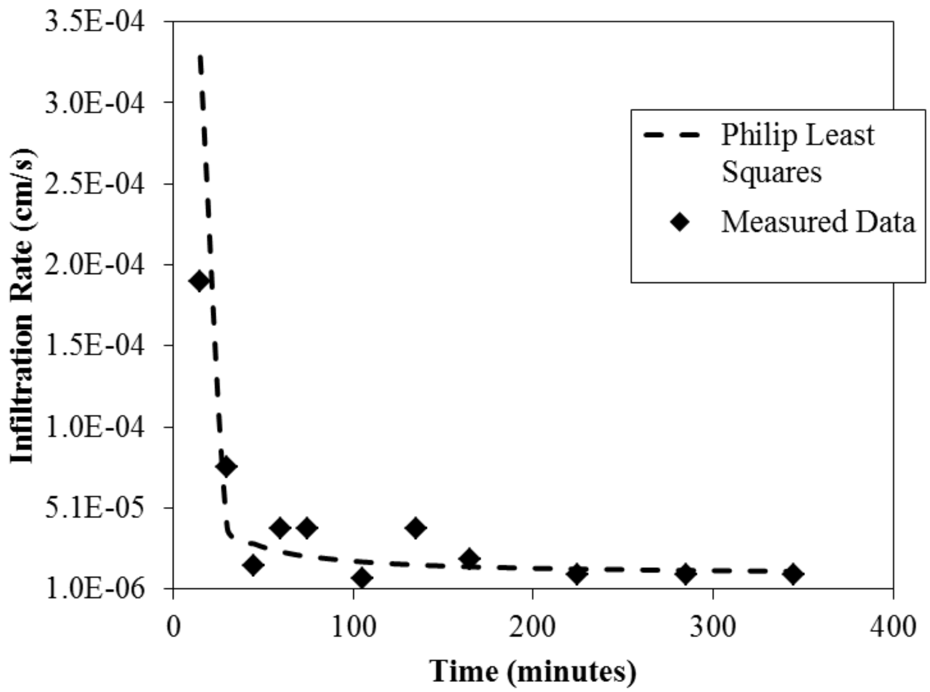


Figure 15. Philip's parameters estimated using least-squares fit with measured infiltrometer data. TPL1 Lawrenceville, GA.

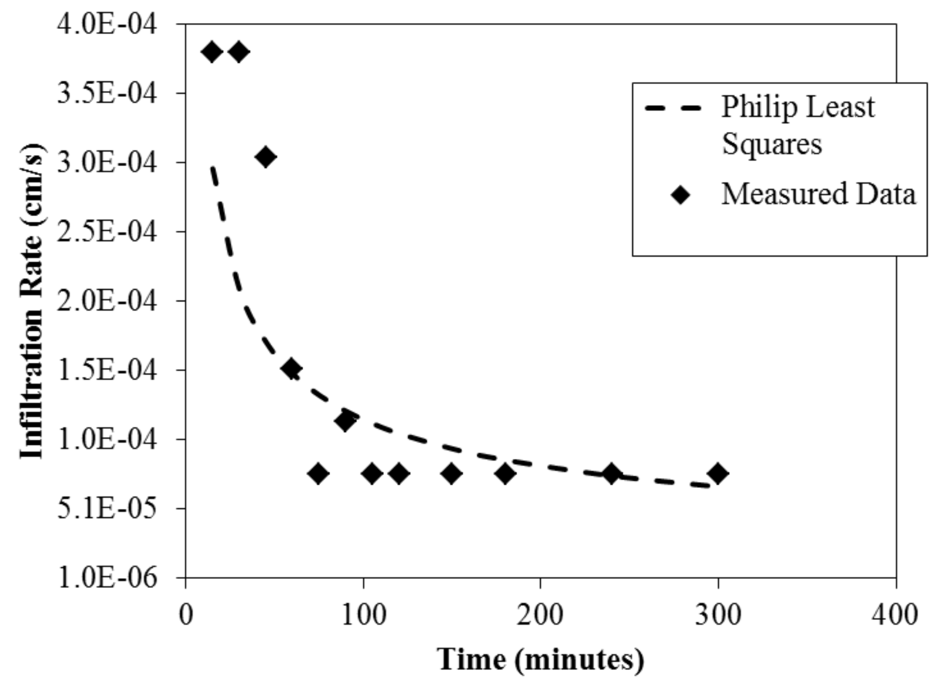


Figure 16. Philip's parameters estimated using least-squares fit with measured infiltrometer data. TPL2, Lawrenceville, GA.

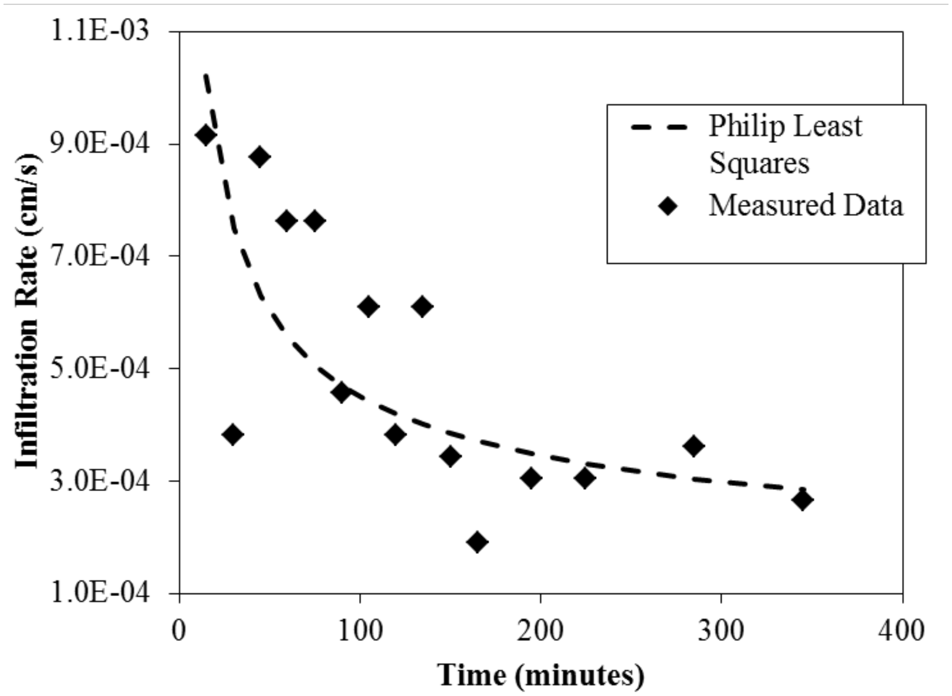


Figure 17. Philip's parameters estimated using least-squares fit with measured infiltrometer data. TPL3, Lawrenceville, GA.

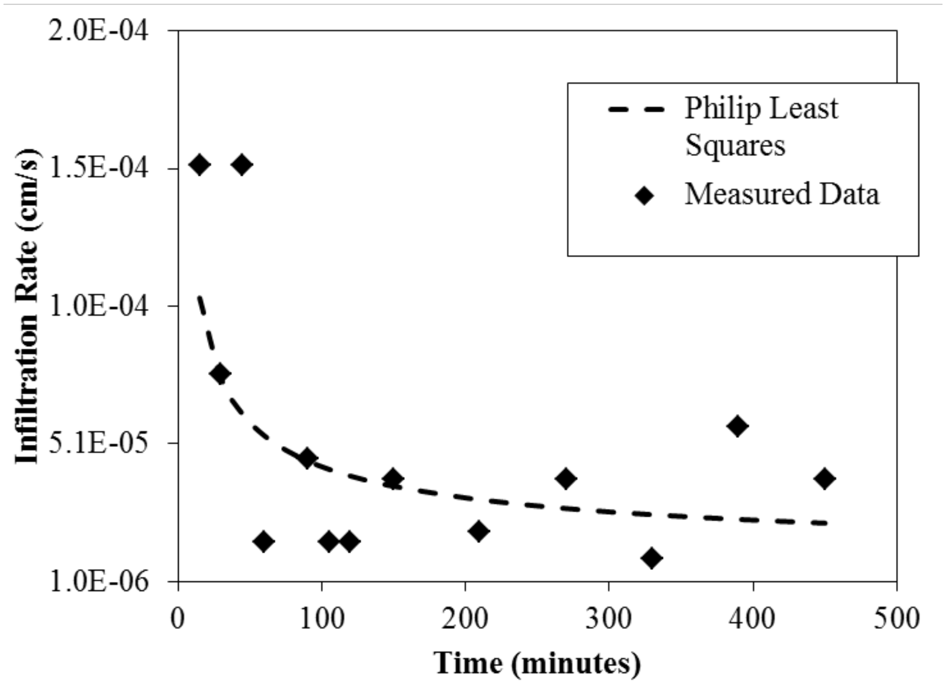


Figure 18. Philip's parameters estimated using least-squares fit with measured infiltrometer data. TPL4, Lawrenceville, GA.

4.5 Discussion: Infiltration

For the infiltration tests that were performed at the Lawrenceville site, there were variations between rates in early readings, particularly with TPL1. These variations were likely due to macropore flow from voids beneath the DRIs. Other known sources that produce overestimation of measured infiltration rates using a DRI are separation of the soil from the wall of the DRI and lateral divergence. Lateral divergence can be caused by capillarity on adjacent soils, restrictive layers causing a perched water table, and using high-pressure head (Bouwer 1986). At the test site, the soils encountered below the gravelly fill were relatively uniform. All material tested passed the #4 sieve (10mm) and no inclusions were discovered during excavation; however, extensive subsurface exploration was not performed before testing. Restrictive layers may have been present under the infiltrometers, and low head was maintained during testing.

The measured rates varied over two orders of magnitude for the Lawrenceville site and one order of magnitude for the Covington site. However, this was not unexpected because in situ values of hydraulic conductivity have been known to have a coefficient of variation as high as 800% (Reynolds et al. 2002). The results obtained in this study were within the bounds of variability found in the literature.

For the infiltration rates measured at the Skidaway Island location, the infiltration rates predicted higher average field saturated hydraulic conductivities than were estimated using other methods, notably empirical and theoretical correlations with grain size distributions and slug tests (36.9 cm/s versus 0.37 cm/s and 0.012 cm/s, respectively). During a falling head double-ring infiltrometer test, the hydraulic gradient decreases because the water level in the inner ring drops. This change in hydraulic pressure results in higher recorded infiltration rates, compared to a constant head test, which correlate to higher hydraulic conductivities (Gregory et. al. 2005).

The saturated hydraulic conductivity rates were higher than those measured by Schultz and Ruppel on Sapelo Island, another Georgia barrier island (Shultz and Ruppel 2001). For the empirical correlations from grain size distributions, falling head permeameter tests, amplitude attenuation models, and constant-rate pumping tests, the magnitude of hydraulic conductivity of an upland find sand overlaid by forest varied from $1x10^{-4}$ cm/s to 0.01 cm/s (Shultz and Ruppel 2001). However, both sites show higher values of hydraulic conductivity for the empirical correlations versus tests performed in wells (Shultz and Ruppel 2001).

From the very limited data collected, there seems to be a poor correlation between NRCS and measured infiltration rates. The NRCS estimates did over predict the measured rates in every case. However, the estimates provided by the ROSETTA pedotransfer function both under and over predicted the measured rates, but were a reasonable first pass approximation.

5.0 CONCLUSION

5.1 Infiltration Rates: Field, Database, and Predicted

In this study, the infiltration rates were measured in situ at three sites in the Piedmont physiographic province and one site in the Coastal Plain physiographic province of Georgia. The efficacy of predicting saturated hydraulic conductivity for Piedmont soils via published soil surveys from the National Resource Conservation Service and pedotransfer functions was investigated. Constant head methods were employed using a double-ring infiltrometers with Mariotte tubes to maintain the head. The soils encountered in situ ranged from sandy soils to silts to clays.

The major findings of this study include:

- Predicted values from the NRCS over predicted every measured value of in situ saturated hydraulic conductivity when compared to field tests.
- Predicted values from the ROSETTA pedotransfer software predicted the saturated hydraulic conductivity with reasonable accuracy when compared to field tests.
- The infiltrometer, which is known to work well in the sediments of the Coastal Plain, also performed well in the soils of the Piedmont physiographic province; however, care must be taken to ensure proper installation in soils in the Piedmont.

5.2 Recommended Guidance for Determining Infiltration Rates

It is recommended that the infiltration rate and the saturated hydraulic conductivity initially be estimated based on known characteristics of the soils in the area. Correlation of the soil type with typical ranges of hydraulic conductivity will provide a first pass estimate. These values can then be compared to the database values obtained from the USDA's Natural Resources Conservation Service (NRCS) database, although it is important to note these values tended high when compared to field measured values in this study. This study determined that the use of the pedotransfer function, ROSETTA (https://cals.arizona.edu/research/rosetta/), yielded infiltration and conductivity values that were in reasonable agreement with the field measured values; consequently, it is recommended as a tool to apply for the first level estimate of infiltration values in the field. If the estimate is greater than the minimum rate allowed, then follow up testing should be performed to confirm site feasibility. The preferred field tests for determining infiltration rate include the borehole test and the infiltrometer test. Recommended resources and test are given in the following (**Table 8**):

Table 8. Recommended Resources and Tests for Determination of Infiltration and Saturated Hydraulic Conductivity Values

1. Site Feasibility Screening Infiltration and hydraulic conductivity estimated from: d) Soil classification e) NRCS database f) ROSETTA pedotransfer function
If a site is feasible for infiltration, conduct field infiltration tests.
2. Field Infiltration Testing a) Infiltrometer tests can easily be performed in most soils in the Coastal Plain and can be performed with care to avoid rocks in the Piedmont. b) Borehole tests can be performed in virtually all soils.

REFERENCES

American Society of Civil Engineers (ASCE). (2012). Manual of Practice, http://ascelibrary.org/page/books/s-mop.

Arriaga, F. J., Kornecki, T. S., Balkcom, K. S., and Raper, R. L. (2010). “A method for automating data collection from a double-ring infiltrometer under falling head conditions.” *Soil Use and Management*, 26(1), 61–67.

Arrington, K. E., Ventura, S. J., and Norman, J. M. (2013). “Predicting Saturated Hydraulic Conductivity for Estimating Maximum Soil Infiltration Rates.” *Soil Science Society of America Journal*, 77(3), 748.

Atlanta Regional Commission (2016). Georgia Stormwater Management Manual, http://www.atlantaregional.com/environment/georgia-stormwater-manual.

Bandini, P., Asce, M., and Sathiskumar, S. (2009). “Effects of Silt Content and Void Ratio on the Saturated Hydraulic Conductivity and Compressibility of Sand-Silt Mixtures.” *Journal of Geotechnical and Geoenvironmental Engineering*, 135(12), 1976–1980.

Barbu, I. A., and Ballestero, T. P. (2014). “Unsaturated Flow Functions for Filter Media Used in Low-Impact Development — Stormwater Management Systems.” *Journal of Irrigation and Drainage*, 1–11.

Bedient, P. B., Huber, W. C., and Baxter, E. V. (2013). *Hydrology and Floodplain Analysis*. (H. Stark, C. Heinle, and S. Disanno, eds.), Pearson, Upper Saddle River, NJ.

Belkhatir, M., Arab, A., Della, N., and Schanz, T. (2014). “Laboratory Study on the Hydraulic Conductivity and Pore Pressure of Sand-Silt Mixtures.” *Marine Georesources & Geotechnology*, 32(2), 106–122.

Bouwer, H. (1986). “Intake Rate: Cylinder Infiltrometers.” *Methods of Soil Analysis, Part 1: Physical and Mineralogical Methods*, American Society of Agronomy and Soil Science Society of America, Madison, Wisconsin, 825–844.

Bouwer, H., and Rice, R. C. (1976). “A slug test for determining hydraulic conductivity of unconfined aquifers with completely or partially penetrating wells.” *Water Resources Research*, 12(3), 423–428.

Bouwer, H., and Rice, R. C. (1984). “Hydraulic Properties of Stony Vadose Zones.” *Groundwater*, 22(6), 696–705.

Brown, D. L., Narasimhan, T. N., and Demir, Z. (1995). “An evaluation of the Bouwer and Rice method of slug test analysis.” *Water Resource Research*, 31(5), 1239–1246.

Buckingham, E. 1907. Studies on the movement of soil moisture. Bull. 38. USDA, Bureau of Soils, Washington D.C.

Budhu, M. (2010). Soil Mechanics and Foundations, Wiley, 3rd Edition, 780 pp.

Chen, J. S., Ouyang, Y., Ravi, V., and Williams, J. R. (1998). *Estimation of Infiltration Rate in the Vadose Zone: Application of Selected Mathematical Models Volume II*. Washington, DC.

Cho, G. C., and Santamarina, J. C. (2001). “Unsaturated Particulate Materials - Particle Level Studies.” *Geotechnical and Geotenvironmental Engineering*, 127(1), 84–96.

Choo, H., and Burns, S. E. (2014). “Effect of over-consolidation ratio on dynamic properties of binary mixtures of silica particles.” *Soil Dynamics and Earthquake Engineering*, Elsevier, 60, 44–50.

Dingman, S. L. (2008). *Physical Hydrology*. Waveland Press, Inc, Long Grove, IL.

Ellington, M. M., and Ferguson, B. K. (1991). “Comparison Of Infiltration And Detention In The Georgia Piedmont Using Recent Hydrologic Models.” *1991 Georgia Water Resources Conference*, Athens, GA, 213–216.

Estes, C. J. (2007). “Storm Water Infiltration in Clay Soils : A Case Study of Storm Water Retention and Infiltration Techniques in The North Carolina Piedmont.” *2nd National Low Impact Development Conference*, ASCE, 159–170.

Fedler, C. B., Littlejohn, J., Duan, R., and Feng, L. (2012). *TCEQ Report No. 582-9-90350: Refining the Application Rates for Onsite Surface Application*. Lubbock, TX.

Feng, T. (2004). “Using a Small Ring and a Fall-Cone to Determine the Plastic Limit.” *Journal of Geotechnical and Geoenvironmental Engineering*, 130(6), 630–635.

Ferguson, B. K. (1994). *Stormwater Infiltration*. CRC Press, Inc, Boca Raton, Florida.

Finke, K. A., Mayne, P. W., and Klopp, R. A. (2001). “Piezocone Penetration Testing in Atlantic Piedmont Residuum.” *Journal of Geotechnical and Geoenvironmental Engineering*, 127(January), 48–54.

Freeze, R.A. and Cherry, J.A. (1979). Groundwater, Prentice Hall, 604 pp.

Gore, P.J. and Witherspoon, W. (2013). Roadside Geology of Georgia, Mountain Press Publishing Company.

Green, W.H. and G. Ampt. 1911. Studies of soil physics, part I – the flow of air and water through soils. J. Ag. Sci. 4:1-24.

Gregory, J. H., Dukes, M. D., Jones, P. H., and Miller, G. L. (2006). “Effect of urban soil compaction on.” *Journal of Soil and Water Conservation*, 61(3), 117–124.

Hazen, A. 1930. Water supply. American Civil Engineers Handbook. John Wiley & Sons, New York, NY., pp. 1444–1518.

Horton RE. 1940. An approach towards physical interpretation of infiltration capacity. Proceedings of the Soil Science Society of America 5: 399– 417.

Hsu, S. M., Asce, M., Ni, C., and Hung, P. (2002). “Assessment of Three Infiltration Formulas based on Model Fitting on Richards Equation.” *Journal of Hydrologic Engineering*, 7(5), 373–379.

Lai, J., Luo, Y., and Ren, L. (2010). “Buffer Index Effects on Hydraulic Conductivity Measurements Using Numerical Simulations of Double-Ring Infiltration.” *Soil Science Society of America Journal*, 74(5), 1526.

Lai, J., Luo, Y., and Ren, L. (2012). “Numerical Evaluation of Depth Effects of Double-Ring Infiltrometers on Soil Saturated Hydraulic Conductivity Measurements.” *Soil Science Society of America Journal*, 76(3), 867.

Lai, J., and Ren, L. (2007). “Assessing the Size Dependency of Measured Hydraulic Conductivity Using Double-Ring Infiltrometers and Numerical Simulation.” *Soil Science Society of America Journal*, 71(6), 1667.

Lu, N., and Likos, W. J. (2004). *Unsaturated Soil Mechanics*. John Wiley and Sons Ltd, Hoboken, New Jersey.

Massman, J. W. (2003). “A Design Manual for Sizing Infiltration Ponds.” *Washington State Department of Transportation*, (October).

McGeary, R. K. (1961). “Mechanical Packing of Spherical Particles.” *Journal of The American Ceramic Society*, 44(10), 513 – 522.

Philip, J. R., The theory of infiltration, 1, The infiltration equation and its solution, Soil Sci., 83, 345-357, 1957.

Philips, E. C., and Kitch, W. A. (2011). “A review of methods for characterization of site infiltration with design recommendations.” *43rd Symposium on Engineering Geology and Geotechnical Engineering*, Las Vegas, NV, 23–25.

Rawls, W. J., Brakensiek, D. L., and Miller, N. (1983). “Green-ampt Infiltration Parameters from Soils Data.” *Journal of Hydraulic Engineering*, 109(1), 62–70.

Rawls, W. J., Brakensiek, D. L., and Saxton, K. E. (1982). “Estimation of Soil Water Properties.” *American Society of Agricultural Engineers*, 25(5), 1316 – 1330.

Reynolds, W. D. (2013). “An assessment of borehole infiltration analyses for measuring field-saturated hydraulic conductivity in the vadose zone.” *Engineering Geology*, Elsevier B.V., 159, 119–130.

Reynolds, W.D., and Elrick, D.E. (1990). “Ponded infiltration from a single ring: I. Analysis of steady flow.” *Soil Science Society of America,* 54, 1233-1241.

Reynolds, W. D., Elrick, D. E., Youns, E. G., Amoozegar, A., Booltink, H. W. G., and Bouma, J. (2002). “Saturated and Field Saturated Water Flow Parameters.” *Methods of Soil Analysis: Part 4 - Physical Methods*, J. H. Dane and G. C. Topp, eds., Soil Science Society of America, Inc, Madison, Wisconsin, 797 – 877.

Santamarina, J. C., and Jang, J. (2010). “Energy geotechnology : Implications of mixed fluid conditions.” *5th International Conference on Unsaturated Soils*, A. Gens and E. Alonso, eds., Barcelona.

Santamarina, J. C., Klein, K. A., and Fam, M. A. (2001). *Soils and Waves Particulate Materials Behavior, Characterization and Process Monitoring*. John Wiley and Sons Ltd, West Sussex.

Sathees, T. (2006). “Saturated hydraulic conductivity of poorly graded sands with non-plastic silts using a flexible wall permeameter.” New Mexico State University.

Saxton, K. E., and Rawls, W. J. (2006). “Soil Water Characteristic Estimates by Texture and Organic Matter for Hydrologic Solutions.” *Soil Science Society of America Journal*, 70(5), 1569.

Schaap, M. G., Leij, F. J., and van Genuchten, M. T. (2001). “Rosetta: a Computer Program for Estimating Soil Hydraulic Parameters With Hierarchical Pedotransfer Functions.” *Journal of Hydrology*, 251(3-4), 163–176.

Shultz, G., and Ruppel, C. (2001). “Constraints on hydraulic parameters and implications for groundwater flux across the upland-estuary interface.” *Journal of Hydrology,* 260, 255-269.

Stewart, J. W. (1964). *Infiltration and Permeability of Weathered Crystalline Rocks Georgia Nuclear Laboratory Dawson County, Georgia - Studies of Sites for Nuclear Energy Facilities*. Dawson County, GA.

Taylor, D.W. (1948). Fundamentals of Soil Mechanics, John Wiley and Sons, 700 pp.

Thevanayagam, S., and Martin, G. R. (2000). “Liquefaction in silty soils - screening and remediation issues.” *International Workshop on Mitigation of Seismic Effects on*

Transportation Structures, National Center for Research on Earthquake Engineering, Taipei, Taiwan.

Urso, M., Lawrence, C., and Adams, M. (1999). “Pendular, Funicular, and Capillary Bridges: Results for Two Dimensions.” *Journal of colloid and interface science*, 220(1), 42–56.

Van Genuchten, M. T. (1980). “Closed-form Equation for Predicting the Hydraulic Conductivity of Unsaturated Soils, A.” *Soil Science Society of America Journal*, 44(8), 892–898.

WEF, ASCE, and EWRI. (2012). “Infiltrators.” *Design of Urban Stormwater Controls*, McGraw Hill, Alexandria and Reston, Virginia.

Wösten, J. H. M., Pachepsky, Y. A., and Rawls, W. J. (2001). “Pedotransfer functions: bridging the gap between available basic soil data and missing soil hydraulic characteristics.” *Journal of Hydrology*, 251(3-4), 123–150.

Wu, L., Pan, L., Roberson, M.J. & Shouse, P.J. 1997. Numerical evaluation of ring-infiltrometers under various soil conditions. Soil Science, 162, 771–777.

Xu, X., C. Lewis, W. Liu, J.D. Albertson, and G. Kiely. 2012. Analysis of single-ring infiltrometer data for soil hydraulic properties estimation: Comparison of BEST and Wu methods. Agric. Water Manage. 107:34– 41. doi:10.1016/j.agwat.2012.01.004.

Zhang, Z. F., Ward, A. L., and Keller, J. M. (2009). *Determining the Porosity and Saturated Hydraulic Conductivity of Binary Mixtures*. Richland, Washington.

Zornberg, J. G., Asce, M., Mccartney, J. S., and Asce, A. M. (2010). “Centrifuge Permeameter for Unsaturated Soils. I : Theoretical Basis and Experimental Developments.” *Journal of Geotechnical and Geoenvironmental Engineering*, 136(8), 1051–1063.

APPENDIX A

The Atterberg Limits were determined using method BS 1377. The correlation proposed by Feng (2004) was used to calculate the plastic limit.

$$PL = C(2)^m$$

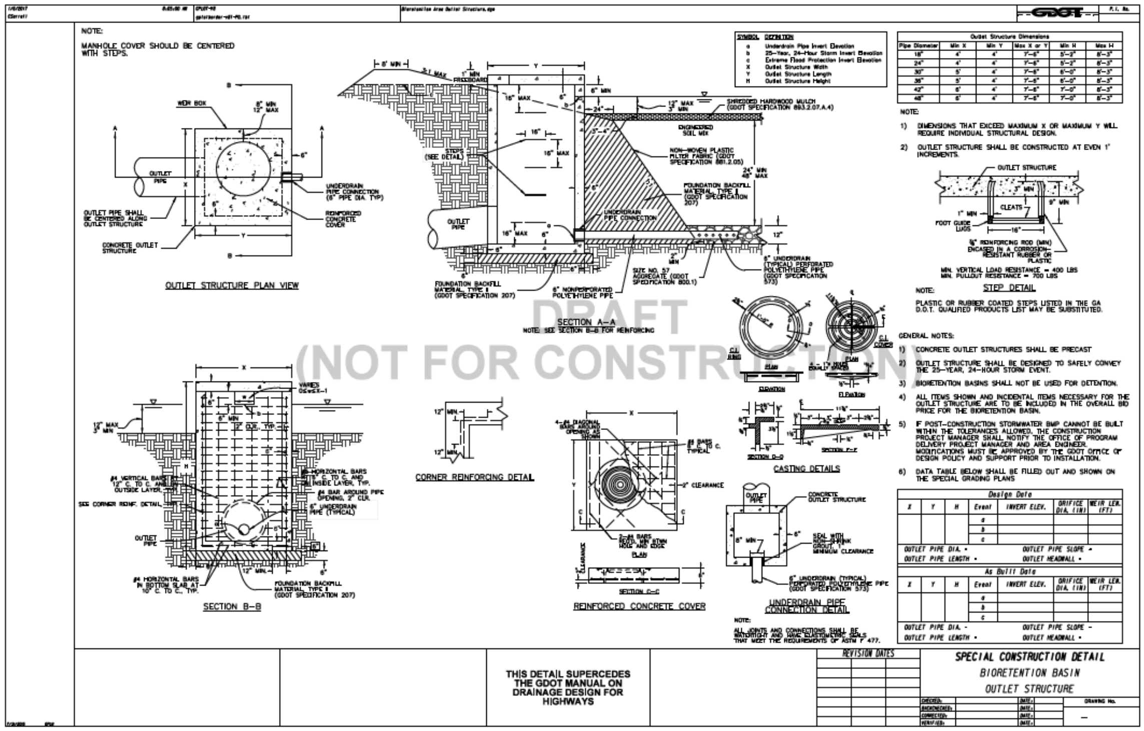
The LL, PL, PI. Feng (2004) parameters, and USCS soil types are summarized in Table A-1. Figures A-1, A-2, A-3, A-4, A-5 and A-6 show the water content with penetration depth, and Figure A-7 shows the plasticity chart used to classify the fine grained soils.

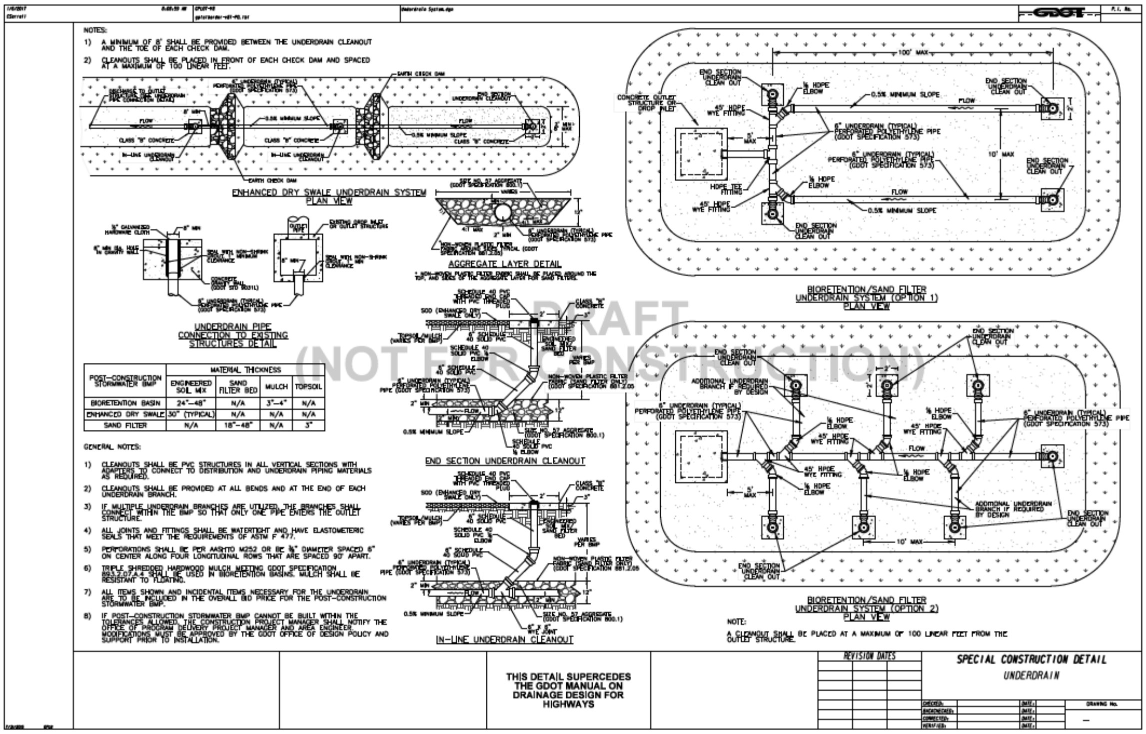
Table 9. Summary of Atterberg Limits and Soil Types per USCS

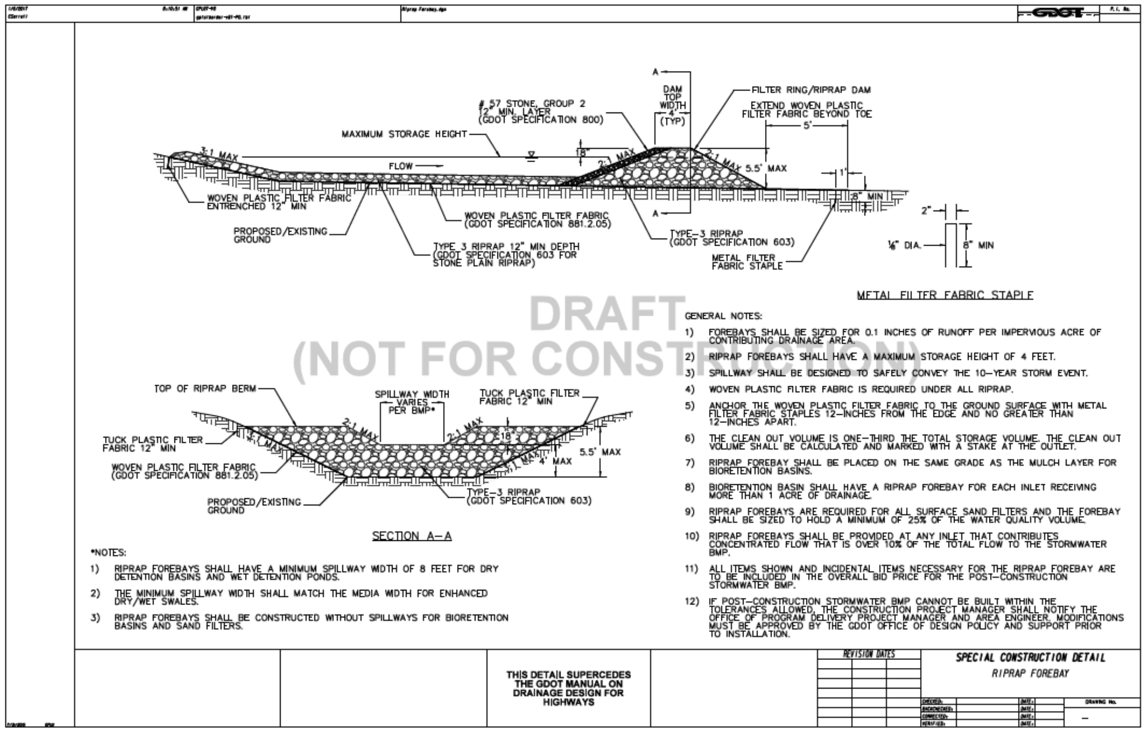
Test Pit	LL	C	m	PL	PI	USCS	Group Name
TPC2	53.0	20.730	0.372	26.8	26.2	CH	Sandy Fat Clay
TPC3	59.5	27.490	0.258	32.9	26.6	MH	Sandy Elastic Silt
TPL1	45.0	19.510	0.345	24.8	20.2	CL	Sandy Clay
TPL2	52.0	22.860	0.335	28.8	23.2	CH	Sandy Fat Clay
TPL3	50.0	20.249	0.359	26.0	24.0	SC	Clayey Sand
TPL4	60.0	20.458	0.406	27.1	32.9	CH	Sandy Fat Clay

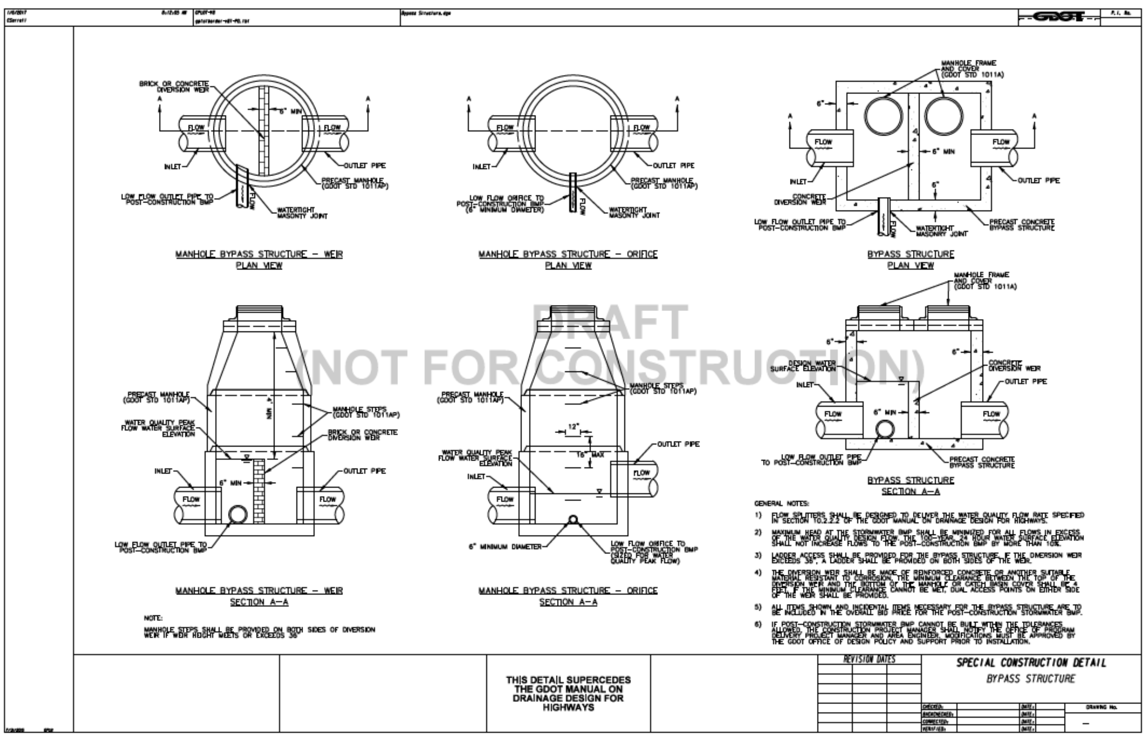
APPENDIX B

Bioretention details






DAM TOP WIDTH 4' (TYP)
FILTER RING/RIPRAP DAM
#57 STONE, GROUP 2 12" MIN. LAYER (GDOT SPECIFICATION 800)
EXTEND WOVEN PLASTIC FILTER FABRIC BEYOND TOE
MAXIMUM STORAGE HEIGHT
FLOW
3:1 MAX
2:1 MAX
5.5' MAX
WOVEN PLASTIC FILTER FABRIC ENTRENCHED 12" MIN
WOVEN PLASTIC FILTER FABRIC (GDOT SPECIFICATION 881.2.05)
PROPOSED/EXISTING GROUND
TYPE 3 RIPRAP 12" MIN DEPTH (GDOT SPECIFICATION 603 FOR STONE PLAIN RIPRAP)
TYPE-3 RIPRAP (GDOT SPECIFICATION 603)
METAL FILTER FABRIC STAPLE
1/8" DIA.
8" MIN
METAL FILTER FABRIC STAPLE
DRAFT (NOT FOR CONSTRUCTION)
TOP OF RIPRAP BERM
SPILLWAY WIDTH VARIES PER BMP*
TUCK PLASTIC FILTER FABRIC 12" MIN
WOVEN PLASTIC FILTER FABRIC (GDOT SPECIFICATION 881.2.05)
PROPOSED/EXISTING GROUND
TYPE-3 RIPRAP (GDOT SPECIFICATION 603)
SECTION A-A
*NOTES:
1) RIPRAP FOREBAYS SHALL HAVE A MINIMUM SPILLWAY WIDTH OF 8 FEET FOR DRY DETENTION BASINS AND WET DETENTION PONDS.
2) THE MINIMUM SPILLWAY WIDTH SHALL MATCH THE MEDIA WIDTH FOR ENHANCED DRY/WET SWALES.
3) RIPRAP FOREBAYS SHALL BE CONSTRUCTED WITHOUT SPILLWAYS FOR BIORETENTION BASINS AND SAND FILTERS.
GENERAL NOTES:
1) FOREBAYS SHALL BE SIZED FOR 0.1 INCHES OF RUNOFF PER IMPERVIOUS ACRE OF CONTRIBUTING DRAINAGE AREA.
2) RIPRAP FOREBAYS SHALL HAVE A MAXIMUM STORAGE HEIGHT OF 4 FEET.
3) SPILLWAY SHALL BE DESIGNED TO SAFELY CONVEY THE 10-YEAR STORM EVENT.
4) WOVEN PLASTIC FILTER FABRIC IS REQUIRED UNDER ALL RIPRAP.
5) ANCHOR THE WOVEN PLASTIC FILTER FABRIC TO THE GROUND SURFACE WITH METAL FILTER FABRIC STAPLES 12-INCHES FROM THE EDGE AND NO GREATER THAN 12-INCHES APART.
6) THE CLEAN OUT VOLUME IS ONE-THIRD THE TOTAL STORAGE VOLUME. THE CLEAN OUT VOLUME SHALL BE CALCULATED AND MARKED WITH A STAKE AT THE OUTLET.
7) RIPRAP FOREBAY SHALL BE PLACED ON THE SAME GRADE AS THE MULCH LAYER FOR BIORETENTION BASINS.
8) BIORETENTION BASIN SHALL HAVE A RIPRAP FOREBAY FOR EACH INLET RECEIVING MORE THAN 1 ACRE OF DRAINAGE.
9) RIPRAP FOREBAYS ARE REQUIRED FOR ALL SURFACE SAND FILTERS AND THE FOREBAY SHALL BE SIZED TO HOLD A MINIMUM OF 25% OF THE WATER QUALITY VOLUME.
10) RIPRAP FOREBAYS SHALL BE PROVIDED AT ANY INLET THAT CONTRIBUTES CONCENTRATED FLOW THAT IS OVER 10% OF THE TOTAL FLOW TO THE STORMWATER BMP.
11) ALL ITEMS SHOWN AND INCIDENTAL ITEMS NECESSARY FOR THE RIPRAP FOREBAY ARE TO BE INCLUDED IN THE OVERALL BID PRICE FOR THE POST-CONSTRUCTION STORMWATER BMP.
12) IF POST-CONSTRUCTION STORMWATER BMP CANNOT BE BUILT WITHIN THE TOLERANCES ALLOWED, THE CONSTRUCTION PROJECT MANAGER SHALL NOTIFY THE OFFICE OF PROGRAM DELIVERY PROJECT MANAGER AND AREA ENGINEER. MODIFICATIONS MUST BE APPROVED BY THE GDOT OFFICE OF DESIGN POLICY AND SUPPORT PRIOR TO INSTALLATION.
THIS DETAIL SUPERCEDES THE GDOT MANUAL ON DRAINAGE DESIGN FOR HIGHWAYS
REVISION DATES
SPECIAL CONSTRUCTION DETAIL
RIPRAP FOREBAY


BRICK OR CONCRETE DIVERSION WEIR
FLOW
INLET
OUTLET PIPE
PRECAST MANHOLE (GDOT STD 1011AP)
LOW FLOW OUTLET PIPE TO POST-CONSTRUCTION BMP
WATERTIGHT MASONRY JOINT
MANHOLE BYPASS STRUCTURE - WEIR PLAN VIEW
LOW FLOW ORIFICE TO POST-CONSTRUCTION BMP (6" MINIMUM DIAMETER)
MANHOLE BYPASS STRUCTURE - ORIFICE PLAN VIEW
MANHOLE FRAME AND COVER (GDOT STD 1011A)
CONCRETE DIVERSION WEIR
PRECAST CONCRETE BYPASS STRUCTURE
BYPASS STRUCTURE PLAN VIEW
DRAFT (NOT FOR CONSTRUCTION)
MANHOLE STEPS (GDOT STD 1011AP)
WATER QUALITY PEAK FLOW WATER SURFACE ELEVATION
6" MINIMUM DIAMETER
LOW FLOW ORIFICE TO POST-CONSTRUCTION BMP (SIZED FOR WATER QUALITY PEAK FLOW)
MANHOLE BYPASS STRUCTURE - WEIR SECTION A-A
MANHOLE BYPASS STRUCTURE - ORIFICE SECTION A-A
NOTE:
MANHOLE STEPS SHALL BE PROVIDED ON BOTH SIDES OF DIVERSION WEIR IF WEIR HEIGHT MEETS OR EXCEEDS 36"
DESIGN WATER SURFACE ELEVATION
BYPASS STRUCTURE SECTION A-A
GENERAL NOTES:
1) FLOW SPLITTERS SHALL BE DESIGNED TO DELIVER THE WATER QUALITY FLOW RATE SPECIFIED IN SECTION 10.2.2.2 OF THE GDOT MANUAL ON DRAINAGE DESIGN FOR HIGHWAYS.
2) MAXIMUM HEAD AT THE STORMWATER BMP SHALL BE MINIMIZED FOR ALL FLOWS IN EXCESS OF THE WATER QUALITY DESIGN FLOW. THE 100-YEAR, 24 HOUR WATER SURFACE ELEVATION SHALL NOT INCREASE FLOWS TO THE POST-CONSTRUCTION BMP BY MORE THAN 10%.
3) LADDER ACCESS SHALL BE PROVIDED FOR THE BYPASS STRUCTURE. IF THE DIVERSION WEIR EXCEEDS 36", A LADDER SHALL BE PROVIDED ON BOTH SIDES OF THE WEIR.
4) THE DIVERSION WEIR SHALL BE MADE OF REINFORCED CONCRETE OR ANOTHER SUITABLE MATERIAL RESISTANT TO CORROSION. THE MINIMUM CLEARANCE BETWEEN THE TOP OF THE DIVERSION WEIR AND THE BOTTOM OF THE MANHOLE OR CATCH BASIN COVER SHALL BE 4 FEET. IF THE MINIMUM CLEARANCE CANNOT BE MET, DUAL ACCESS POINTS ON EITHER SIDE OF THE WEIR SHALL BE PROVIDED.
5) ALL ITEMS SHOWN AND INCIDENTAL ITEMS NECESSARY FOR THE BYPASS STRUCTURE ARE TO BE INCLUDED IN THE OVERALL BID PRICE FOR THE POST-CONSTRUCTION STORMWATER BMP.
6) IF POST-CONSTRUCTION STORMWATER BMP CANNOT BE BUILT WITHIN THE TOLERANCES ALLOWED, THE CONSTRUCTION PROJECT MANAGER SHALL NOTIFY THE OFFICE OF PROGRAM DELIVERY PROJECT MANAGER AND AREA ENGINEER. MODIFICATIONS MUST BE APPROVED BY THE GDOT OFFICE OF DESIGN POLICY AND SUPPORT PRIOR TO INSTALLATION.
THIS DETAIL SUPERCEDES THE GDOT MANUAL ON DRAINAGE DESIGN FOR HIGHWAYS
REVISION DATES
SPECIAL CONSTRUCTION DETAIL
BYPASS STRUCTURE

APPENDIX C

Laboratory measurements of the saturated hydraulic conductivity of binary mixtures.

The saturated hydraulic conductivity of an ASTM 100/200 sand mixed with Sil-Co-Cil 40 non-plastic silt in a binary mixture was measured using a flexible wall permeameter. Falling head-rising tailwater methods were used as described ASTM D5084, and two target relative densities were used for the sand-silt mixtures: 20 and 70%. The measured hydraulic conductivity decreased with increasing silt content for both loose and dense specimens by two orders of magnitude (Figure 19 and Figure 20). The hydraulic conductivity also decreases with increasing density. Both of these findings agree with those found in the literature (Bandini et al. 2009; Belkhatir et al. 2014; Sathees 2006; Thevanayagam and Martin 2000). All values shown were measured with a confining pressure of 35 kPa. The 100% silt specimen was tested at one fixed density, $\rho_d = 1.3$ g/cm^3, for a reference value.

There is a greater than one order of magnitude drop between 0 and 17% silt contents for both the loosely and densely prepared specimens. After the 17% FC, the estimated FC* using the Choo and Burns (2014) model, the decrease is relatively smaller. As the fines content increased up to the FC*, the void space decreased and the surface area contributing to viscous drag increased, but the addition of fines beyond the FC* resulted in the loss of contact between the coarse particles (Figure 21). From this point, the saturated hydraulic conductivity, and global behavior of the soil specimen in general, exhibited behavior similar to that of the fines. Similar changes in the rate of decrease have been observed in recent literature (Belkhatir et al. 2014).

Although steps were taken to prepare homogenous specimens, segregation during specimen preparation or migration of silt during permeation is always a concern for sand-silt specimens. Relatively small segregation of silt was observed for the densely prepared specimens. However, the loosely prepared specimens were visually homogenous. Segregation in the densely prepared specimens could have induced lower conductivity values.

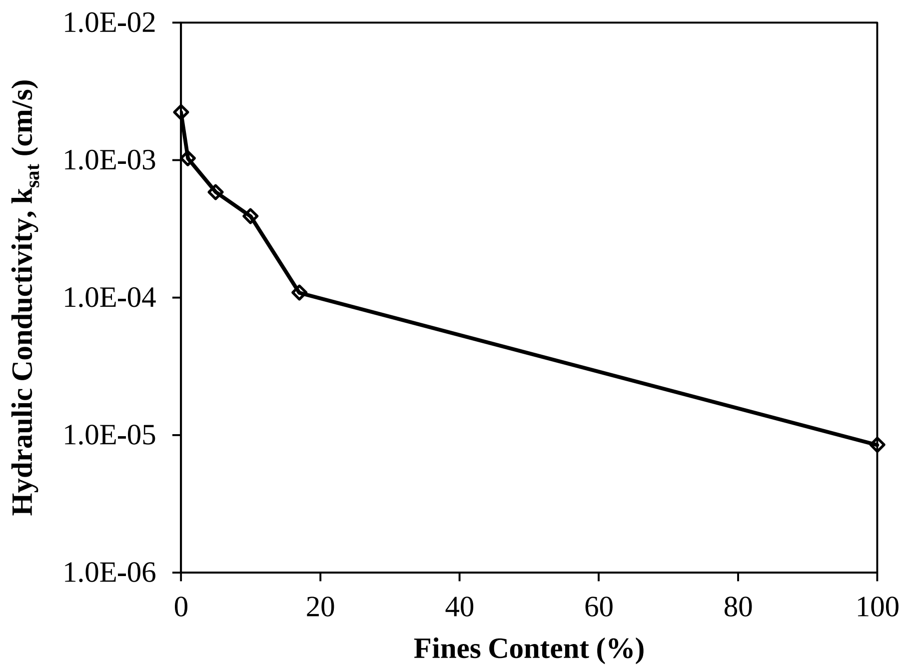


Figure 19. Saturated hydraulic conductivity of ASTM 100/200 sand with increasing Sil-Co-Sil 40 content. Specimens prepared relatively loose, Dr = 20%.

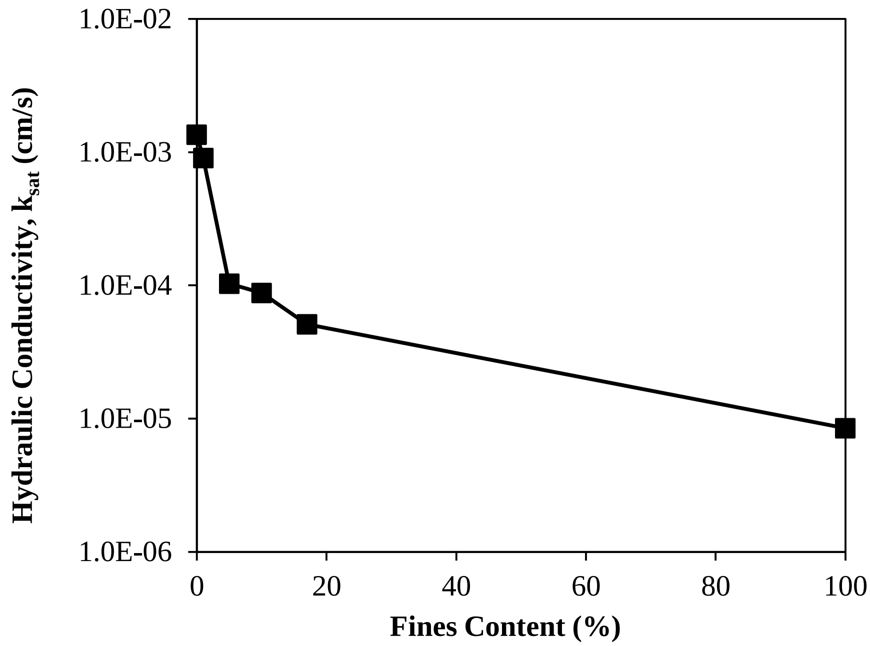


Figure 20. Saturated hydraulic conductivity of ASTM 100/200 sand with increasing Sil-Co-Sil 40 content. Specimens prepared relatively dense, Dr = 70%.

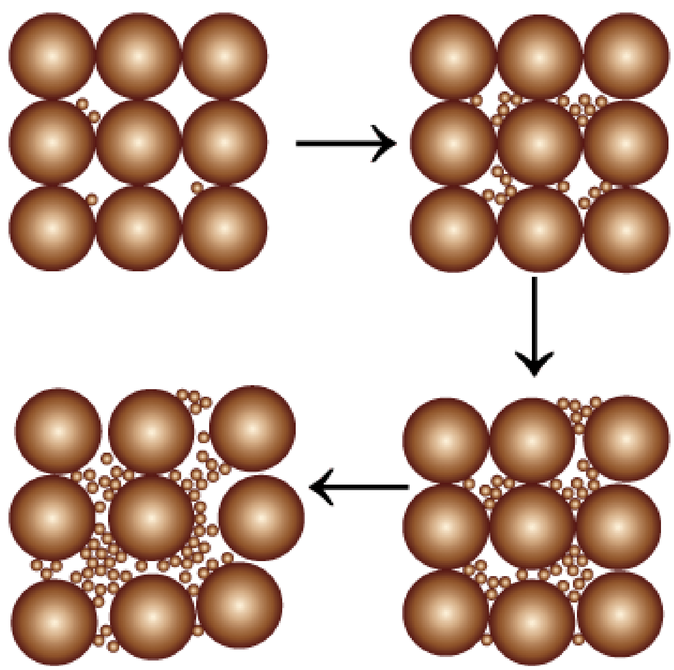


Figure 21. As the fines content increases the matrix of coarse particles is separated by the fine grains. At this point the global behavior of the soil is expected to behave similarly to the fine grained material.

The global void ratio was calculated for each specimen using the known mass of solids and total volume, with target void ratios to achieve target relative densities. The measured hydraulic conductivity was analyzed as a function of both the global and intergranular void ratio (Figure 22 and Figure 23). As expected, the hydraulic conductivity decreased with decreasing global void ratio and increasing intergranular void ratio for each sand silt mixture. The trend in Figure 22 shows the expected decrease in hydraulic conductivity with decreasing void ratio. However, the decrease in the void ratio resulted in larger decreases in hydraulic conductivity for specimens containing greater than 1% silt. The decrease in the void ratio of the sand/silt mixtures may lead to a greater decrease in volume due to the presence of fines already contained in the pore volume.

Figure 23 shows that as silt content increases, the intergranular void ratio increases and the hydraulic conductivity decreases. This observation agrees with those found in the literature (Belkhatir et al. 2014; Sathees 2006). All values shown were measured with a confining pressure of 35 kPa.

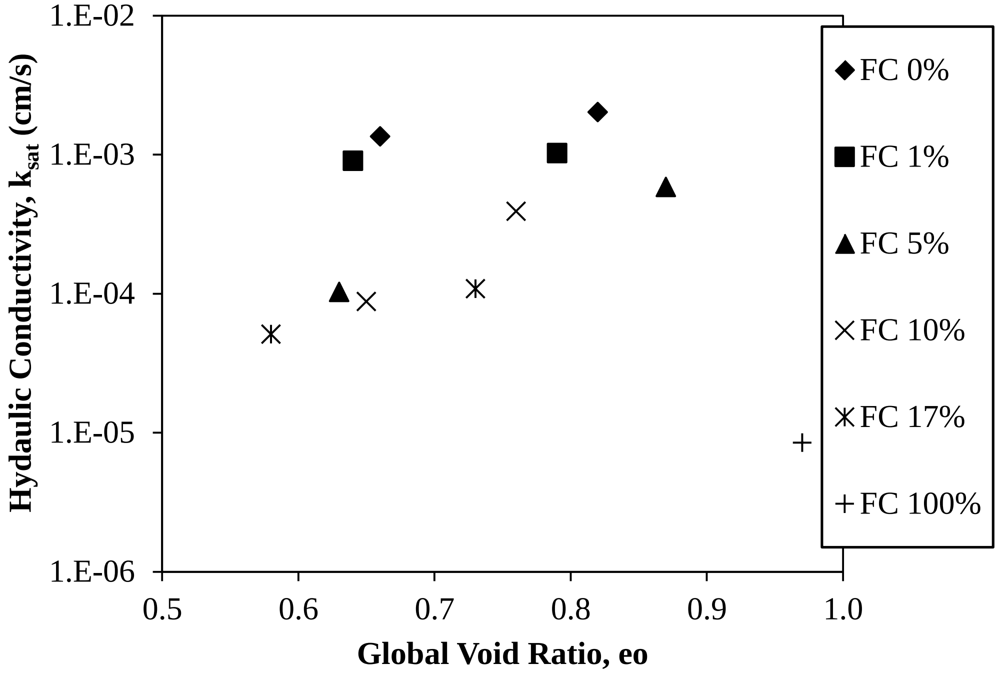


Figure 22. The saturated hydraulic conductivity of ASTM 100/200 sand with increasing contents of Sil-Co-Sil 40 is shown as a function of global void ratio.

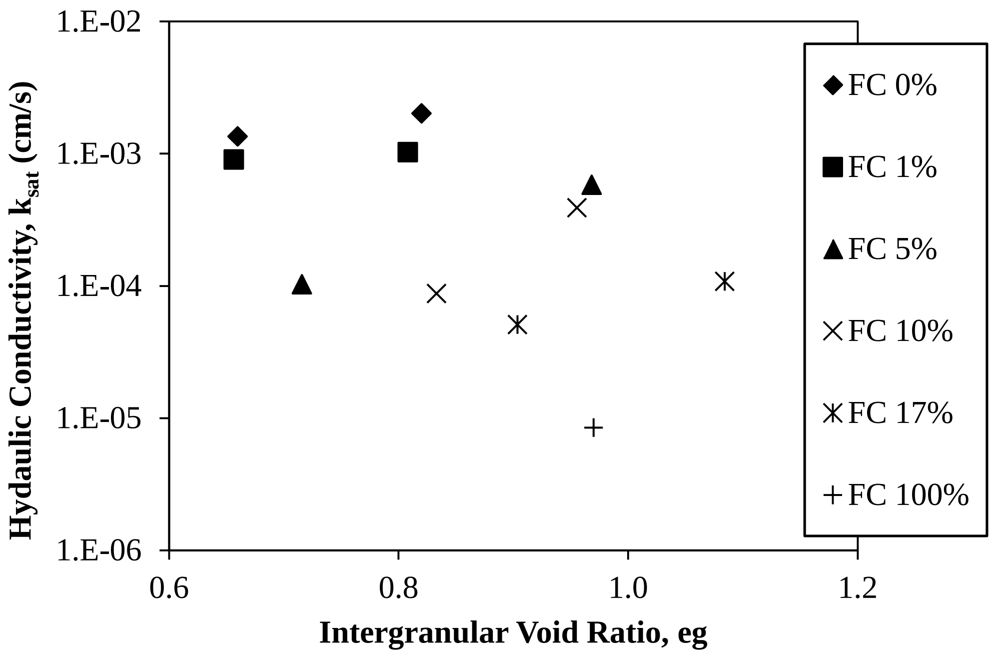


Figure 23. The saturated hydraulic conductivity of ASTM 100/200 sand with increasing contents of Sil-Co-Sil 40 is shown as a function of intergranular void ratio.

Each specimen was back pressured and tested with an effective stress of 35 kPa (5 psi). After measurements were taken at 35 kPa, the specimen was consolidated by an additional 35 kPa up to 140 kPa to assess the relationship between confining stress and hydraulic conductivity. Consolidation required less than five minutes for sand samples and over an hour for the 100% silt specimens. Figure 24 shows the hydraulic conductivity as a function of confining stress for loose specimens, and Figure 25 shows the values for dense specimens.

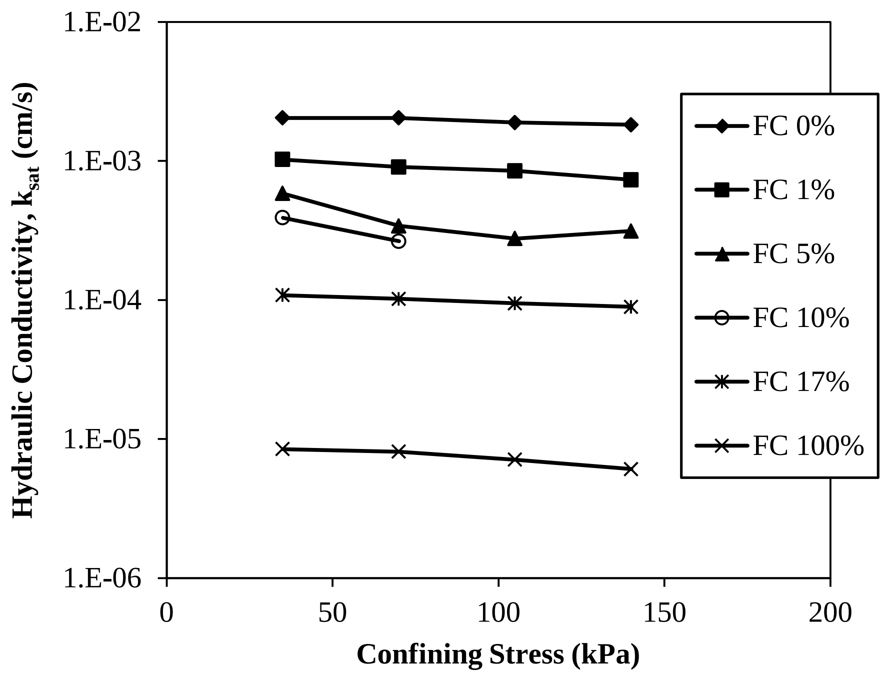


Figure 24. Hydraulic conductivity is shown as a function of confining stress for loosely prepared specimens of sand-silt mixtures.

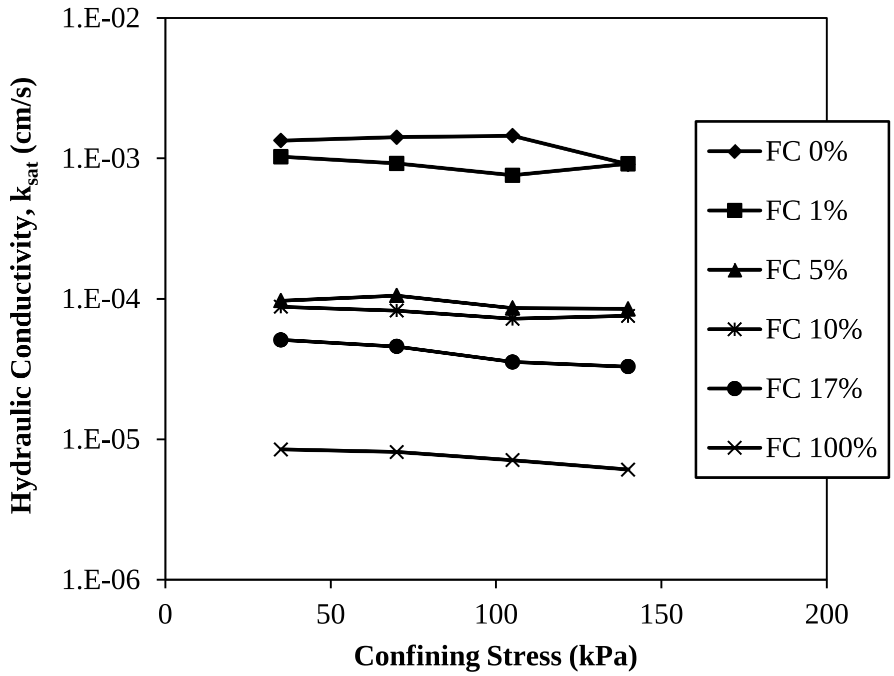


Figure 25. Hydraulic conductivity is shown as a function of confining stress for densely prepared specimens of sand-silt mixtures.

The mixing coefficient model was used to predict the saturated hydraulic conductivity of the sand-silt mixture (Figure 26 for loose specimens and Figure 27 for dense specimens). The terms UB, MCM, and LB are upper bound, mixing-coefficient model and lower bound, respectively. The UB prediction was generated assuming there was no mixing. The LB uses the ideal packing model, and the MCM used the estimated λ_{avg} value. λ_{avg} was calculated according to the methods outlined in Zhang et al.(2009). The λ value can be changed based on measured void ratios to more accurately reflect the level of mixing between the sand and silt for predicting extreme porosities, or void ratios. However, the mixing-coefficient model consistently over predicted hydraulic conductivity measured for the ASTM 100/200 and Sil-Co-Sil 40 mixture.

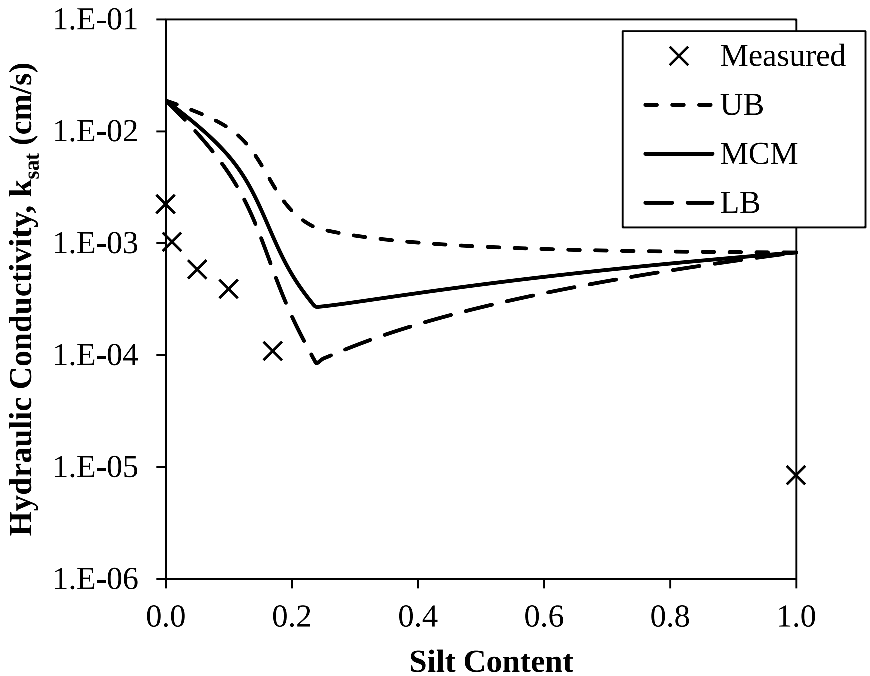


Figure 26. The mixing-coefficient model of Zhang et al. (2009) was used to predict the hydraulic conductivity of the loosely prepared specimens.

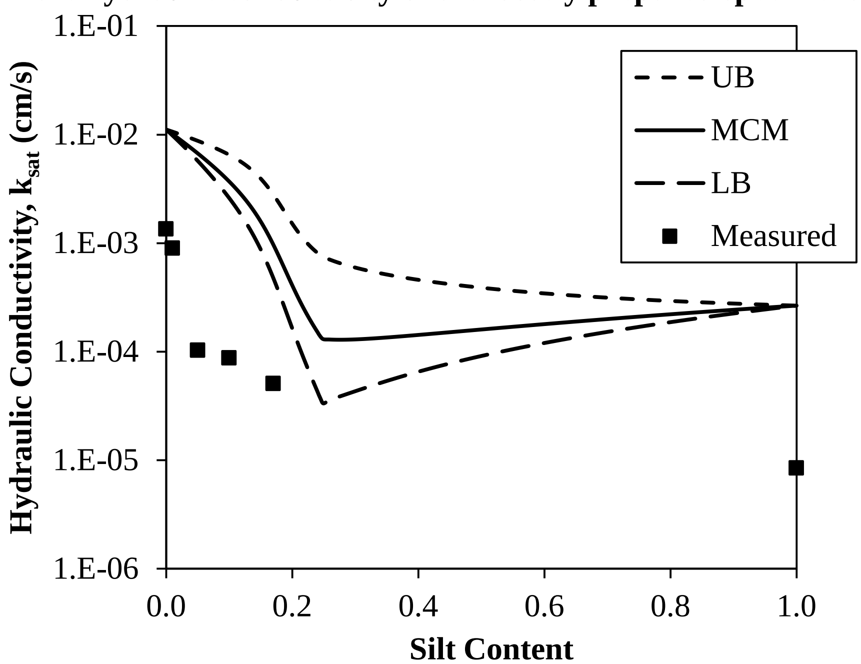


Figure 27. The mixing-coefficient model of Zhang et al. (2009) was used to predict the hydraulic conductivity of the densely prepared specimens.

APPENDIX D

ROSETTA is a hydraulic modeling program developed at the US Salinity Laboratory and UC Riverside in collaboration with funding from USDA-ARS, Army Research Office, NASA, and NSF. The mathematical model uses soil characteristics as input, including textural class, grain size, density, and water retention to predict the hydraulic conductivity of a soil deposit. The fundamental concept of the model is to take parameters that can be easily measured, such as soil grain size, and use those parameters to estimate hydraulic conductivity, which is more difficult to measure in the field. The model can be used for both saturated and unsaturated conductivity predictions, with multiple levels of data quality. In its most simple form, ROSETTA correlates soil textural class with a lookup table of hydraulic conductivity. In more complex calculations, the model predicts hydraulic conductivity using hydraulic conductivity models that are defined here: https://www.ars.usda.gov/pacific-west-area/riverside-ca/us-salinity-laboratory/docs/rosetta-hydraulic-functions/. The hydraulic conductivity is predicted based on the model of van Genuchten (1980). Additional detail on the ROSETTA model can be found at: https://cals.arizona.edu/research/rosetta/.