A HYDROLOGIC EVALUATION OF PRETREATMENT AND VARIATIONS IN SEASONAL AND LARGE STORM PERFORMANCE OF INFILTRATING STORMWATER CONTROL MEASURES

By

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A	weir plate width adjacent to the notch
A_{IT}	bottom area of the infiltration trench (IT)
$A_{IT,total}$	total surface area available for infiltration in the IT
A_{paver}	area of pavers
H	height of the weir above its notch
h	height of water over the weir notch
I_{i-j}	infiltration rate for a specific depth increment (i, j) in the IT
L	cross-sectional width of a weir
l	length of the IT
Q	discharge over the notch of a weir
RR_{i-j}	recession rate for a specific depth increment (i, j) in the IT
SA_{i-j}	surface area of a specific depth increment (i, j) in the IT
t_{final}	time that y_{final} occurs
t_o	time when runoff first enters the IT
t_{over}	duration of overflow
$t_{peak,i-j}$	time that $y_{peak,i-j}$ occurs
V_{inf}	volume of infiltration
V_{IT}	volume of runoff into the IT
$V_{IT,total}$	total volume reaching the IT
V_{over}	overflow volume
V_{runoff}	volume of runoff
V_{sys}	total runoff volume captured by the treatment train

w	width of the IT
y_{final}	lowest depth in the IT following a rain event
y_o	height of water in the IT when runoff first enters
y_{peak}	peak depth reached in the IT
$y_{peak,i\text{-}j}$	peak depth achieved in a specific depth increment (i, j) of the IT
Z	total height of a weir

Abstract

Green infrastructure, specifically stormwater control measures (SCMs), meant to reduce runoff volumes are typically designed for smaller, frequently occurring events (e.g., 2.5 cm or the 2-year event) and their performance are not always considered for larger, more infrequent extreme event management. If a SCM relies on infiltration as its primary volume reduction strategy, infiltration continues throughout the rainfall event providing more volume capture than design, even during large storms. The present work uses the hydrologic performance of the Villanova treatment train comprised of several SCMs in series, including an infiltration trench, designed for the 2.5 cm rainfall event as a basis for discussion. Additionally, volume capture performance was compared with an extremely undersized infiltration trench (the "old" infiltration trench) on campus to determine the effect of proper design and pretreatment of an SCM.

Results at the old infiltration trench (IT) showed a rapid decrease in volume capture due to sediment loading from the large drainage area. Recession rates decreased from 8.9 cm/h to 0.7 cm/h within the month of July over the first three years of operation. Infiltration performance at the old IT was greater than the IT at the treatment train in its first year of operation but quickly decreased, while the performance increased following the first year at the treatment train IT. For the IT at the treatment train, overall there was a general increase in infiltration rates over the course of the study period despite slight decreases in winter and spring of the systems third year. Further, the system has consistently met and usually exceeded the volume reduction design goals by capturing at least 59% of the volume of every storm, and on average 96% of all events. These performance results demonstrate that infiltration SCMs can reduce significant volumes of runoff during larger events, and that new design strategies are needed to account for their performance.

1 Introduction

Two green infrastructure sites featuring infiltration trenches (IT) were evaluated as part of this research. The overall objective of this study was to evaluate the hydrologic performance of the two separate stormwater control measure (SCM) systems and to determine what effect pretreatment had on volume capture during large storms and different seasons

1.1 Background

Urbanization changes a watershed's landscape through decreases in pervious grass or vegetated areas and increases in impervious areas such as buildings and roadways. Increases in impervious areas result in an excess of stormwater runoff quantity, as well as a decrease in water quality. Stormwater control measures (SCMs) are being increasingly used to mitigate the stresses that development places on urban watersheds. Stormwater control measures include features such as vegetated swales, rain gardens, and ITs, among others. Agencies around the United States are developing and implementing SCMs to better reduce stormwater runoff volumes, peak flows and pollutant loads to receiving water bodies (NRC 2009; WEF 2012). For example, Philadelphia is implementing a $1.2 billion long-term control plan to primarily use SCMs to manage stormwater and reduce combined sewer overflows (CSOs) (PWD 2011). Additionally, New York City has committed approximately $1.5 billion toward implementing a Green Infrastructure Plan to help reduce CSOs and control stormwater runoff from impervious areas over a 30 year period (NYCDEP 2012). Furthermore, Washington DC has committed approximately $100 million of a $2.6 billion long-term CSO control plan to implementing "as much green infrastructure as possible" (DCWSA 2014). Similarly, communities such as Portland, OR and Onondaga County, NY have committed substantial resources to implementing green infrastructure features

(CH2MHill 2012; Entrix 2010). As part of any municipal-wide or watershed-wide stormwater management plan, optimal system performance while minimizing cost is desired.

The shift to SCMs as a stormwater management technique was in response to the understanding that small, frequent storms can cause environmental damage and that large events were not the only event that should be managed (Prince George's County, Maryland 1999). Stormwater control measures are designed, generally, to control smaller storms – typically less than 2.5–3.8 cm of rainfall over a drainage area (NRC 2009), or events that are greater than 95% of all other annual rainfall events and often just at the event scale. With the focus on smaller events, the usefulness of SCMs is commonly thought to be inconsequential for larger and extreme events (EPA 2014). However, the results of the present research indicate that the benefits of SCMs can be substantial during large events and annually if designed adequately and appropriately. For example, the Philadelphia area receives on average 97 cm of rainfall annually (PWD 2013). Events that are 2.5 cm or less account for approximately 50% of the annual events (Figure 1, dashed grey line). From a volume perspective, if the first 2.5 cm of a large storm is absorbed by a SCM, only the volume above 2.5 cm will be discharged to the receiving body. In this case, nearly 80% of annual rainfall volume is captured for a SCM designed to hold 2.5 cm (Figure 1, black line). When SCMs are designed too small for a drainage area, both water quality and quantity performance will likely suffer over time. One example of this is an infiltration trench at Villanova University. This "old" Villanova IT was designed with a drainage area 160 times larger than its surface area, well above the five times larger recommendation, to accelerate and study decreased hydrologic performance over time. However, adequately designed SCMs can provide resilience or "graceful extensibility" during large storm events. Similar to resiliency,

graceful extensibility is a systems ability to continue to perform beyond designed boundaries (Woods 2014). An SCM shows graceful extensibility if it captures and treats more stormwater runoff than it was designed for. As a result of potentially increasing rainfall volume and intensity due to climate change, understanding the resiliency and graceful extensibility of SCMs can provide insight on performance during large storm events and stormwater management planning. High rainfall intensities can at times be greater than the infiltration rate of an SCM, which can limit the volume going to infiltration at an SCM and reduce capture performance during rain events. However, if a system is designed to detain flow through the system, the rainfall intensity impact is curbed and the system presents more flexibility under different rainfall scenarios.

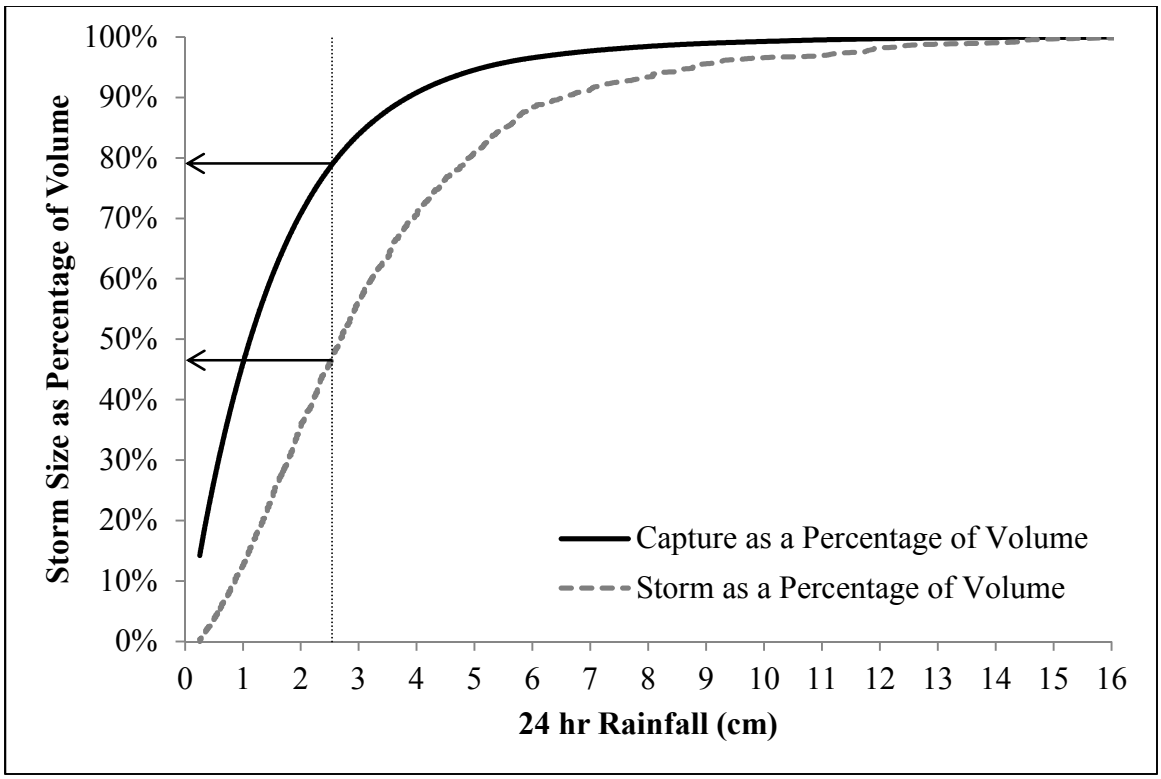

Figure 1.1: Percent storm by volume and percent capture by volume. Data from Philadelphia Airport Gage 24 hour rainfall data (1948-2011)

Pretreatment measures are part of adequate design for most SCM measures. These pretreatment measures can act similar to a stream's riparian buffer, which can slow down flood waters and prevent pollutants, such as nitrogen, phosphorus, and sediment, from entering a stream (Cunningham et al. ND). Similarly, the goal of pretreatment is to remove sediment, debris, and pollutants before they enter an SCM (MDE 2009; PADEP 2006). Further, pretreatment methods can add extensibility to the hydrologic performance of an SCM system. Even undersized SCMs have shown to provide adequate water quality benefits. Luell et al. (2011) found that the difference between pollutant capture between undersized and properly sized bioretention systems was not significantly different. The undersized bioretention cell captured 84% of sediment loading, and 50% of nitrogen loading from the drainage area. Understanding the extent of SCM performance for the full range of rainfall volumes and intensities is necessary for urban areas to develop resilient and cost effective stormwater management. Additionally, understanding the benefits of pretreatment on system longevity and performance could lead to substantial monetary savings from reducing maintenance costs and increasing the lifespan of SCMs.

One type of SCM used to reduce stormwater runoff volumes and provide water quality treatment is an IT. Infiltration trenches are below ground practices that allow stormwater runoff to infiltrate into the surrounding soil (Akan 2002). The performance of two separate ITs was monitored as part of the present study. The "old" Villanova IT (old IT) was constructed in 2004 with no pretreatment measures, and is composed of washed aggregate. The old IT was constructed with a 160:1 drainage area to surface area, well above the recommended 5-10:1 ratio. Hydrologic performance at the extremely undersized IT drastically decreased over the first three years of operation as a result of sediment loading (Emerson et al. 2010). The treatment train IT was

constructed in 2011 and is the final feature in a series of SCMs. The treatment train features approximately 130 feet of a vegetated swale, two small rain gardens in series, and the IT. The concept of the treatment train is to utilize the benefits of each SCM in the system to provide ideal water quantity control, peak flow reduction, and water quality management (Lyons 2012). Seasonal and large storm performance for the treatment train was monitored with a focus on the IT to determine how the system functioned in terms of volume capture. Furthermore, the system was designed to provide pretreatment to each SCM and reduce maintenance and increase the lifespan and longevity of the system. The treatment train IT performance was compared to the old IT to determine what effect pretreatment had on system performance and longevity.

1.2 Research Objectives

To meet the overall goal of this study, several research objectives were developed:

- To analyze an extremely undersized infiltration trench over a ten plus year period and determine what affects a large contributing drainage area and no pretreatment has had on performance
- To quantify the performance of the IT at the treatment train based on infiltration rates and determine how performance varied over the study period
- To assess how the IT at the treatment train performed during large storm events and how factors such as rainfall intensity, antecedent soil moisture, and seasonal variation in temperature affect its performance
- To evaluate water quantity performance of the treatment train system and investigate the pattern of rainfall events that caused overflow to the system

- To compare performance of each infiltration trench to determine what effect proper design and pretreatment can have on system performance
- To examine methods of determining flow rates through the treatment train system and determine instrumentation that will be most effective in doing so.

2 Literature Review

A review of current literature was conducted to gain knowledge of IT performance and design, methods of pretreatment for SCMs, and how states and municipalities credit SCMs depending on performance. Previous research was examined to obtain a basis of knowledge of which the current study could build upon.

2.1 Infiltration Trench Design and Performance

Infiltration trenches (ITs) are one type of SCM that provide significant volume reduction, peak attenuation and groundwater recharge through storage (Shaver 1986). Additionally, ITs provide water quality benefits as a result of sorption and filtration methods (Akan 2002; Birch et al. 2005; Emerson et al. 2010; Siriwardene et al. 2007; Scholz and Yazdi 2008). Stormwater runoff is stored in a concentrated area that allows infiltration into the surrounding soil (Emerson et al. 2010). Infiltration trenches are often utilized in urban stormwater management as they are compact and can readily fit into the linear urban landscape. Design of ITs often includes lining an excavated trench with geotextile fabric and filling the trench with washed stone aggregate (EPA 1999; PADEP 2006). The aggregate in the trench acts as a filter to remove sediment, debris, and other pollutants contained in stormwater runoff prior to infiltrating into the surrounding soils while providing structural support within the trench. Another option are designs that use modular plastic unit tanks with significant void space (approximately 95%) and provide structural support within the trench as they are resistant to high loads (Clark and Acomb 2008). The plastic modular units are similarly wrapped in geotextile fabrics to prevent soil migration into the IT. Modular systems can provide additional storage capacity to the IT while providing water quality benefits. Recommended drainage area to SCM surface area ratios for ITs are between 5-10 to 1 (IDEQ 2005; PADEP 2006; VDOT 2013). Furthermore, maximum

recommended drainage areas for ITs are approximately 5 acres (EPA 1999; MDE 2009). The restriction on drainage area is to prevent large sediment and debris loads to the IT that can lead to decreased infiltration rates due to clogging of the bottom layer affecting long term performance and longevity of the system (Schueler 1994).

Infiltration trenches provide water quality benefits. For example, ITs can significantly reduce levels of metals, and nutrients in surface runoff (Maniquiz et al. 2010). Additionally, ITs are effective at removing suspended solids from runoff, however the water quality benefit is offset by the potential for clogging pore spaces (Siriwardene 2007). Several previous studies have shown that infiltration SCMs have clogged from sediment buildup as a result of improper site location, improper design, and lack of maintenance (Lindsey and Roberts 1992; Galli 1993; Hilding 1996; New Jersey Pinelands Commission 2005). Previous research at the old IT performed by Emerson et al. (2010) demonstrated a decrease in ability to infiltrate stormwater runoff over time, most likely due to clogging of the bottom of the IT. Within the first three years of the study period, the volume reduction capability of the IT decreased by approximately 45% and has experienced a pattern of exponential decay over the operation (Emerson et al. 2010; Lewellyn et al. 2015). A decrease in IT performance was also shown in Bergman et al. (2011), Dechesne et al. (2005), Duchene et al. (1994), Lindsey and Roberts (1992), and Schuh (1990). However, studies have shown that infiltration practices can maintain their performance for long periods of time with proper design and location (Welker et al. 2006; Emerson and Traver 2008). Even older and severely undersized infiltrating SCMs have been shown to outperform designed volume capture early in their lifespan (Bergman et al. 2011; Emerson et al. 2010). Additionally, Barraud et al. (2014), Emerson et al. (2010), and Gonzalez-Merchan et al. (2012) found that

infiltration is substantial through the sidewalls in addition to the bottom of an IT, enabling greater actual infiltration than design.

Infiltration is a key component of volume capture for the ITs and other infiltrating SCMs. Aside from clogging due to sediment loading, several other factors have been shown to affect SCM infiltration performance. Emerson and Traver (2008) and Braga et al. (2007) showed a correlation between infiltration rate and temperature based on water viscosity's dependence on temperature. Infiltration rates were shown to rise when water was less viscous in warmer temperatures, and fall when the viscosity of water lowers in colder temperatures. Seasonal variability in temperature has been shown to impact the infiltration rate of runoff to surrounding soils by as much as 56% (Braga et al. 2007; Lin et al. 2003).

Warnaars et al. (1999) evaluated the performance of two ITs over a 2.75 year study period in central Copenhagen. The ITs were located in densely built-up area with a drainage area of approximately 600 m^2 directly connected impervious area. Both ITs displayed a 30% to 70% decrease in hydrologic performance between over the study period as a result of potential clogging due to sediment deposition. Bergman et al. (2011) continued this research to determine how the ITs were performing hydrologically 15 years after monitoring began. Hydraulic conductivity for the sides and the bottom of one of the ITs was found to have decreased by approximately 70% during the lifetime of the IT. It was concluded that infiltration through the bottom and sides of the IT were affected as a result of sediment deposition.

Dechesne et al. (2005) studied the long-term evolution of clogging in non-vegetated, soil and gravel covered infiltration basins ranging from 10 to 21 years by evaluating hydraulic resistance. Each of the four studied infiltration basins were determined to have equivalent soils, contributing land uses and depths to groundwater table. All of the infiltration basins included settling basins and flow regulators to control the runoff entering the infiltration basin as pretreatment methods. These basins were found to have relatively low hydraulic resistance even after 10 to 21 years of operation. However, Barraud et al. (2005) performed a statistical analysis on pollution in the same infiltration basins over the study period. The statistical analysis showed that pollutants attached to fine particles collect in the top 30 to 40 cm of the soil profile, clogging soil pores and changing soil texture. The study also showed a strong link between fine soil particles and high pollutant concentrations when examining the infiltration basins.

Previous research at Villanova's old IT focused on pollutant capture and decreasing hydrologic performance over time. Batroney et al. (2010) examined pollutant loading from the IT drainage area to determine which pollutants found in the runoff exhibited a first flush. The study examined runoff from the drainage area during 2006 and 2007. Concentration first flush is described as an initial high concentration of a pollutant at the onset of runoff, followed by a rapid decline in pollutant concentration and a relatively low concentration throughout the remainder of the runoff duration (Batroney et al. 2010). Total suspended solids (TSS), dissolved copper, dissolved cadmium, nitrate, and chloride all exhibited a first flush; total dissolved solids (TDS), total nitrogen, total phosphorus, nitrate phosphate, and dissolved chromium did not exhibit first flush concentrations. Emerson et al. (2010) examined the hydrologic performance and TSS capture ability of the old IT over the first three years of operation. The IT captured

approximately 36% of the TSS load from the drainage area over the three year study period. During the first three years of the study it was shown that the extremely undersized IT can perform better than designed by capturing 0.65 cm of runoff from its drainage area when it was originally designed to capture only 0.5 cm. This outperformance is likely a result of continuous infiltration occurring during a rain event. Concurrently, there was a drastic decrease in infiltration performance over the three year period, with the bottom area of the IT becoming negligible from clogging due to sediment loading and most of the infiltration occurring out of the sides of the IT.

Previous research performed at the Villanova treatment train showed a distinct seasonal variation in infiltration rates at the treatment train IT (Lyons 2012). Further, the treatment train continuously outperformed design capture volume of the first year of monitoring (Lyons 2012). The present research builds on previous findings at the treatment train.

2.2 Pretreatment Measures

Pretreatment methods are often included in the design of infiltrating SCMs to prevent sediment and pollutants from entering systems. Pretreatment measures, such as sedimentation basins, vegetated filters or swales, and other vegetative practices, are often recommended to remove sediment and debris from the runoff before it enters the IT (MDE 2009; PADEP 2006). Utilization of pretreatment is essential with infiltration SCMs such as ITs because incoming sediment can clog ITs by reducing soil pore availability and reduce the volume reduction and pollutant removal ability (Claytor and Schueler 1996). As a result, pretreatment can result in better water quality and quantity performance at an SCM.

Sedimentation basins are recommended in many state stormwater manuals as a means of pretreatment (MDE 2009; MSSC 2008; McCarthy 2008; NJDEP 2004). Sedimentation basins are constructed at the upstream point of an SCM and allow sediment and debris from incoming stormwater runoff to settle out prior to the runoff entering the SCM (VADEQ 2011). Basins can be used for pretreatment of large SCMs, such as wetlands, and smaller scale SCMs, such as rain gardens and underground filters (NYSDEC 2011; MDE 2009). There is minimal volume reduction benefit provided by sedimentation basins aside from designed static storage.

Filter strips are vegetated surfaces that are usually adjacent to impervious areas, such as roadways, that are designed to treat sheet flow runoff by filtering out sediment, providing infiltration, and slowing down runoff velocity (EPA 1999). Vegetated swales are densely planted trapezoidal or parabolic channels that are designed to remove pollutants by sedimentation, filtration by plants, and infiltration into the soil (PADEP 2006; Weiss et al. 2010). For optimum stormwater treatment, swales should be designed to be at least 60 feet long, and have a bottom slope of no more than 3% to prevent high velocities and increase hydraulic residence time within the swale (Ferguson 1988; Yu et al. 2001). Vegetated swales have been shown to have the potential to reduce stormwater volume by between 50% and 89%, and reduce nitrogen and solids loading by 97% and 95%, respectively (Barrett 2005; Xiao and McPherson 2011). However, performance of these systems is largely based on design and underlying soils (Barrett 2005).

A combination of pretreatment measures and SCMs in series is referred to as a stormwater treatment train. The purpose of a treatment train is to utilize implemented SCMs in series to reduce the volume of stormwater runoff and improve water quality before stormwater is

discharged to a receiving body of water (AES 2006). Treatment trains are also utilized to provide additional pollution prevention and runoff volume minimization at SCM sites (MSSC 2008). Runoff volume reduction is additive based on the type and amount of SCMs used as part of the treatment train (NCDENR 2009). Therefore, utilizing multiple SCMs in series may increase the amount of runoff volume being treated from a drainage area.

2.3 Volume Reduction

Stormwater control measures, such as ITs, vegetated swales, and rain gardens, are designed to capture the first 2.5-3.8 cm of a rainfall event. Volume reduction for SCMs is often credited based on the amount of above ground storage combined with storage potential in the media layer of an SCM (PADEP 2006; MDE 2009). However, in some cases volume reduction credit only accounts for above ground storage potential (NCDENR 2009). Credit for volumetric design of SCMs sometimes neglects or reduces the effect of runoff volume reduction as a result of infiltration (NCDENR 2009; WVDEP 2012). Therefore, performance of certain SCMs may be underestimated for volume reduction during storm events. While soil infiltration rates decrease as the soil becomes saturated, infiltration does not stop occurring. Infiltration rates can also fluctuate based on seasonal temperature variation and antecedent soil moisture conditions, which can affect a systems ability to capture stormwater runoff. Many states and the federal government have set minimum infiltration guidelines based on a soils saturated hydraulic conductivity to determine if SCMs are performing adequately and considered effective (Clar et. al 2004; PADEP 2006; MSSC 2008; MDE 2009). However, infiltration tests performed for SCM siting and design may not be conducted during winter months when infiltration rates would be the lowest.

While rainfall volume is used as metric for SCM sizing, rainfall intensity plays a key role in overflow and drainage performance at an SCM (Davis et al. 2012). Long periods of intense rainfall will likely exceed the infiltration rate and capacity of the soil in an SCM and cause the system to overflow (Davis et al. 2012). However, previous studies have shown SCMs ability to outperform their design volume capacity. Lord et al. (2013) showed that a bioinfiltration system removed 50% of the runoff volume for storms that exceeded the design target rainfall utilizing over a decade of monitoring data; similar results were seen in Davis (2008). The bioretention facility studied by Lord et al. (2013) was designed to capture approximately 2.5 cm of runoff: 1.25 cm in above ground storage, and 1.25 cm in soil storage. Volume was reduced from inflow to outflow by approximately 82% on average over the course of the study period.

Gilbert Jenkins et al. (2010) showed that sediment deposition within a vegetated SCM may reduce infiltration capacity at specific point within a facility, but does not have a significant impact on performance. It was shown that there was no significant difference in infiltration rates at the bioretention sites following a three year period of sediment loading. Even with an accumulation of fine soil particles, infiltration rates were still quite high (between 0.4 cm/h and 25.5 cm/h).

Further, Horst et al. (2011) showed that a pervious concrete infiltration basin captured an average of 91% of total monthly rainfall over a three year monitoring period, even during months with higher than average rainfall. Part of the outperformance is because the design focus is only on small events and SCMs are not considered to contribute to management of larger, more intense events. Volume reduction in infiltrating SCMs is a result of relatively high infiltration rates

(approximately 1.3 cm/h) of surrounding soils that is required by design. However, the equivalent rainfall intensity occurs infrequently, which allows infiltrating SCMs to capture rainfall runoff during storm events.

3 Site Design and Instrumentation

The old IT and the treatment train are both located adjacent to the Saint Augustine Center (SAC) parking garage on the University's campus (Figure 3.1). The old IT is located directly to the west of the parking garage, while the treatment train is directly to the east. Both sites are located in the Mill Creek watershed, which drains to the Schuylkill River and eventually the Delaware River. The parking garage is 100% impervious and used year round, which provides an adequate representation of stormwater runoff from an urban area.

Figure 3.1: Aerial photo of the old IT and Treatment Train (Adapted from Google Earth)

3.1 Infiltration Trench

In 2004, an IT was constructed at Villanova University to monitor performance of an undersized IT compared to its drainage area; this site is referred to as the old IT henceforth. The site was

constructed as a retrofit of an eroding sloped area between a two story parking garage and the SAC on Villanova's campus. In addition to providing a research site, the construction of the IT provided a portion of the parking garage with stormwater treatment and provided an aesthetically pleasing patio area.

3.1.1 Infiltration Trench Design

Many state stormwater manuals recommend a drainage area to SCM surface area of approximately 5-10:1. The old infiltration trench was designed with a 100% impervious drainage area to SCM surface area of approximately 160:1 with minimal pretreatment. The IT was designed to capture approximately 0.5 cm of runoff from the SAC parking garage. The system was undersized to accelerate long term loading effects and monitor system performance.

A network of 10 cm PVC pipes collect runoff from approximately 1,900 m^2 (20,400 ft^2) of the parking garage adjacent to the IT (Figure 3.2). Runoff is routed through a monitoring bench to a 12" HDPE perforated distribution pipe where it is released into the stone bed. The inside of the monitoring bench consists of a wire screen to separate out large particles that enter along with the inflow, baffle plates to dissipate energy and slow down inflow from the parking garage, and a 45 degree V-notch weir that was used for to determine flow rate into the IT when monitoring of the site began from 2004 to 2007 (Figure 3.3) (Batroney 2007).

Figure 3.2: Drainage area of the old IT outlined in blue, and the infiltration trench area highlighted in red. (Adapted from Batroney 2007)

Figure 3.3: Monitoring bench at the old IT

The stone bed is approximately 1.8 m deep with 40% void space and is composed of washed aggregate (Emerson and Traver 2008). The stone bed provides temporary storage as the water soaks into the surrounding soil at the bottom and sides of the trench. An overflow pipe allows ponded water in excess of 1.6 m to flow into an existing stormwater inlet. The stone bed was lined with a pervious geotextile to prevent soil migration into the stone bed. The surface area of

the IT is approximately 12 m^2 with 7.7 m^3 of effective storage. The top of the IT is composed of a porous paver patio that was donated by EP Henry Corporation. During intense periods of rainfall the porous paver patio acts as a secondary overflow and the water flows over a grass strip and into a stormwater inlet.

In 2005, soil samples were collected during excavation and analyzed using a wash sieve analysis and hydrometer test (ASTM Method D422). The underlying IT soil was 77% sand, 10% silt, and 13% clay (Figure 3.4). The United States Department of Agriculture (USDA) classification was a sandy loam and the estimated saturated hydraulic conductivity for the soil was between 3.6 cm/h and 12.2 cm/h (Rawls et al. 1998).

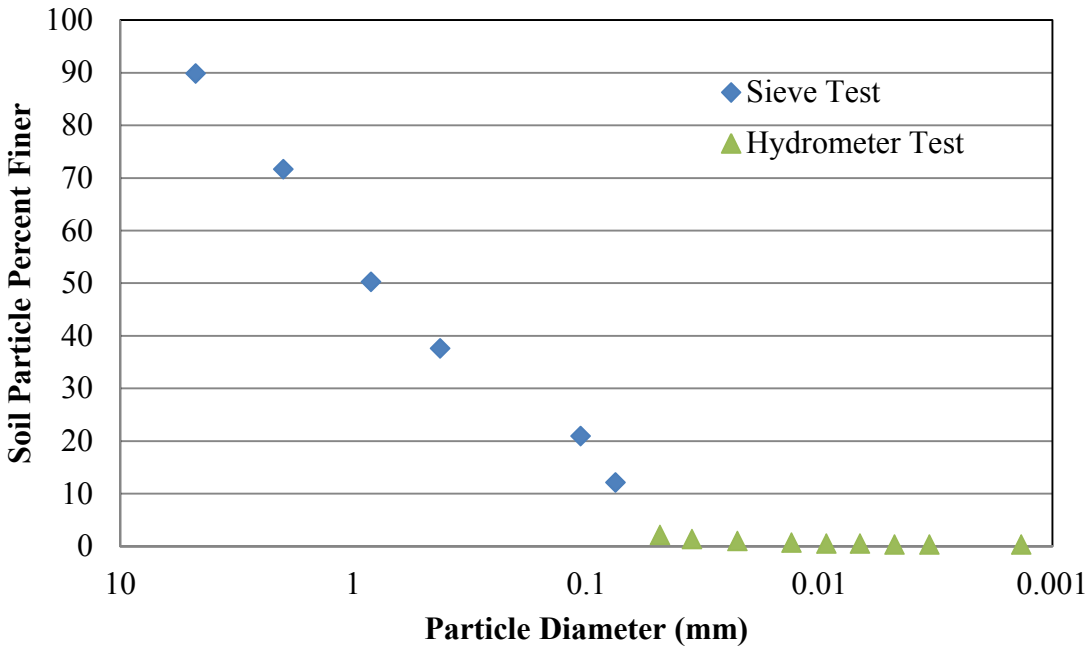

Figure 3.4: Grain size distribution at old IT from 2005.

3.1.2 Old IT Instrumentation

An INW pressure transducer (PS9800) rated up to 3.5 m with ±0.1% accuracy was used to record water depth in the IT. Water depth measurements taken using the pressure transducer were used to determine recession and infiltration rates as part of the hydrologic analysis of the old IT. Rainfall was measured on site with an American Sigma Bucket Rain Gage Model 2149 that measures 0.025 cm of rainfall per bucket tip at 0.5% accuracy for intensities up to 1.3 cm/h (Figure 3.5). The rain gage was located within the drainage area for the old IT and was also used for rainfall measurements at the treatment train. Rainfall events were considered once measured rainfall was greater than 0.13 cm, as 0.13 cm was considered the initial abstraction for the drainage area. A period of at least six hours between measurable rainfall was chosen to differentiate events; therefore, it was possible to have multiple storm events in a single day (Driscoll et al. 1989). Event duration was defined from rainfall start to finish (Geosyntec Consultants and Wright Water Engineers 2009). Rainfall intensity was calculated from the measured rainfall depth for the entire event duration. A temperature sensor (Campbell Scientific model 107) was placed in an adjacent monitoring well in the IT. All measurement devices were connected to a Campbell Scientific datalogger (CR1000) and data was recorded at one-minute intervals.

Figure 3.5: Rainfall gauge used for measurements at both the old IT and treatment train

3.2 Treatment Train

In 2011, a vegetated treatment train ending in an IT was constructed at Villanova University. The linear vegetated techniques included a vegetated swale and two rain gardens in series, which reduced stormwater runoff volume reaching the IT with the intent to increase the IT's longevity and reduce maintenance needs.

3.2.1 Treatment Train Design

The site has a 930 m^2 (10,000 ft^2), 100% impervious drainage area (upper level of a nearby parking garage), and the entire system (IT and vegetated portion) was designed for a 2.5 cm storm with a 7.5:1 drainage area to SCM area ratio (Figure 3.6).

A network of 10 cm PVC pipes conveys runoff from the drainage area to the weir box at the upstream end of the treatment train. The weir box is a 2.0 m long, 0.9 m wide, and 0.9 m deep concrete box used to obtain flow measurements of runoff entering the treatment train. A 2.4 cm PVC pipe delivers inflow to the weir box approximately 15 cm from the top of the box. Two baffle plates slow down runoff coming into weir box and allow settling of sediment and debris from the parking garage. A 90-degree v-notch weir is affixed to the downstream end of the box to allow flow measurement into the system. A metal Bilco door was installed to allow access into the weir box for instrumentation and maintenance (Figure 3.7 and Figure 3.8).

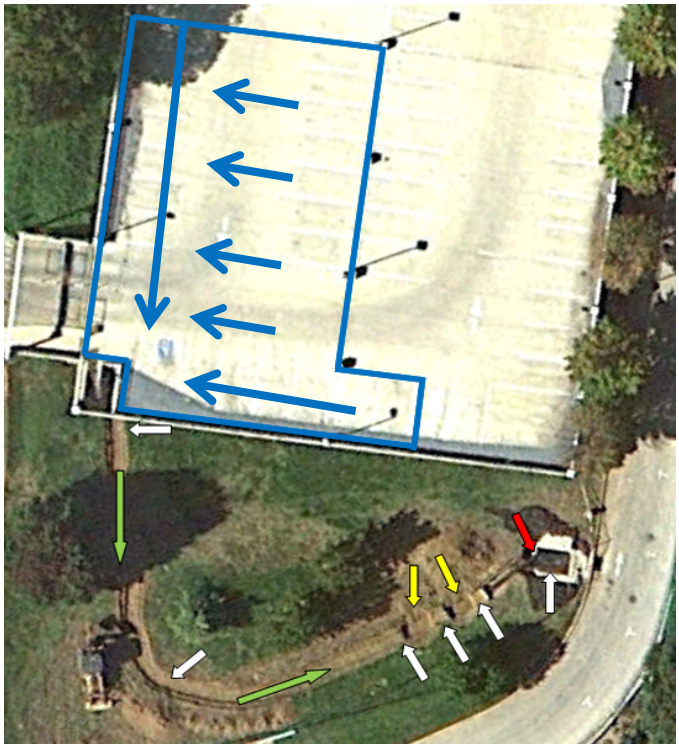

Figure 3.6: Treatment train drainage area and layout. Green arrows depict the vegetated swale, yellow arrows depict the rain gardens, the red arrow depicts the IT, white arrows depict monitoring stations, and blue arrows and outlines depict approximate flow paths and drainage area boundaries, respectively. (Adapted from Lyons 2012)

Figure 3.7: The weir box at the treatment train

Figure 3.8: Instrumentation units in weir box

A vegetated swale and two rain gardens were constructed upstream of the IT to provide pretreatment of stormwater runoff from the parking garage. The vegetated swale is approximately 40 meters long and designed to capture the first 0.7 cm of runoff from the parking garage. The bottom width of the vegetated swale is approximately 1 m with 2:1 side slopes. The swale was designed with approximately 23 cm of engineered media and a ponding depth of approximately 15.2 cm. The vegetated swale and rain gardens were designed with engineered media (85% sand, 10% fines, and 5% organics). Two check dams were placed in the swale to slow the flow of incoming runoff and maximize volume retention and pollution capture. There are two monitoring stations within the swale, which were intended to measure flow. Each monitoring station consists of a pressure transducer used for water depth measurement and a 90 degree v-notch weir for flow measurement. Instrumentation and flow measurement is discussed in Section 3.2.2 and Section 3.2.3, respectively.

Following the vegetated swale, two rain gardens in series were constructed upstream of the IT. Each rain garden was designed to capture 0.5 cm of runoff from the parking garage. The volume capture calculated for each SCM only includes bowl volume. The oval shaped rain gardens are approximately 4 m long, have a bottom width of 1 m, a top width of 2 m, and side slopes of 2:1. Approximately 46 cm of engineered media was used as fill for the rain gardens. Further, there are monitoring stations at the downstream end of each rain garden for flow measurement.

The final SCM in the treatment train is the IT. The 5.4 m^2 x 1.3 m deep IT was formed with R-Tanks, donated by ACF Environmental, with a porosity of approximately 95% (Figure 3.9). The

R-tanks were wrapped in a geotextile fabric to prevent soil migration into the IT and surrounded by crushed stone. The IT was designed to store 0.8 cm of a 2.5 cm storm. There is no underdrain or controlled overflow system. The IT is covered with a metal Bilco door for experimental and maintenance access (0.6 m^2), which is surrounded by Xeripave pervious pavers (4.8 m^2 x 0.05 m) that allow flow out through the top when the IT is filled. The pavers are surrounded by grass and overflow is allowed to run overland into a street inlet.

Figure 3.9: R-tanks used for the IT at the treatment train

Soil samples were collected during excavation for the upper portion of the swale, the lower portion of the swale and the IT. The samples were analyzed using a wash sieve analysis and hydrometer test (ASTM Method D422). Underlying soils were classified as either sandy loam or gravelly sands (Table 3.1). Saturated hydraulic conductivity estimates were obtained from Rawls (1998).

Surveys of the treatment train were performed in November 2013 and May 2014 to determine dimensions of each SCM and to see if the geometry of the system had changed over the first

years of operation. Further, these measurements were used to determine total storage capacity of the systems. Results of both surveys are provided in Appendix A.

Table 3.1: Soil classifications from the upper swale, lower swale, and IT at the treatment train (adapted from Lyons 2012)

Location	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	USDA Classification	Estimated Saturated Hydraulic Conductivity (cm/h)
Upper Swale	7.6	74.4	15.0	3.0	Sandy Loam	2.3
Lower Swale	17.6	76.4	5.0	1.0	Gravelly Sand	20
IT	3	61.0	28.0	8.0	Sandy Loam	2.3

The treatment train was designed similarly to other SCMs and green infrastructure practices to capture the 2.5 cm storm event to conform to Pennsylvania requirements. Table 3.2 shows the design capture volumes for each SCM in the treatment train. Soil storage within the system was considered negligible and was not included in volume capture totals.

Table 3.2: Design summary capture volume for the treatment train (Lyons 2012)

SCM	Purpose	Rainfall (in)/(cm)	Estimated Volume Capture (ft^3)/(m^3)
Vegetated Swale	Slow flow with check dams, volume retention, and pollution capture	0.3 in / 0.7 cm	240 ft^3 / 6.8 m^3
Rain Gardens	Retain water for infiltration, pollution capture	0.4 in / 1.0 cm	340 ft^3 / 8.7 m^3
Infiltration Trench	Hold volumes from larger storms (> 0.7 in)	0.3 in / 0.8 cm	245 ft^3 / 7.0 m^3
Design Capture Totals		**1 in / 2.5 cm**	**825 ft^3 / 22.5m^3**

3.2.2 Treatment Train Instrumentation

A Campbell Scientific pressure transducer (CS450-L) rated up to 5.1 m with ±0.1% accuracy and field calibrated quarterly was used to record water depth and temperature in the IT. Rainfall was measured using the same rain gauge used for the old IT, therefore rainfall totals during storm events at each site are equal in volume and intensity. Rainfall event characteristics were determined using the same method described in Section 3.1.2 Antecedent dry time was calculated from the end of an event to the start of the next event. A temperature sensor (CS450-L) was placed at the upstream end of the vegetated treatment train and air temperature was recorded by a Campbell Scientific model 107 temperature sensor. All measurement devices were connected to a Campbell Scientific datalogger (CR1000) and data was recorded at one-minute intervals.

Campbell Scientific pressure transducers (CS450-L) were also placed in the weir box, at the vegetated swale, and in each rain garden to determine water depth. Further, 90 degree v-notch weirs were placed just downstream of each pressure transducer for flow measurement calculations (Figure 3.10, Table 3.3). However, due to inaccurate flow measurements throughout the system, discussed in section 3.2.3, other flow measurement techniques were sought. In the weir box, a Gems XT-1000 level sensor was installed to measure inflow from the parking garage into the treatment train. The Gems XT-1000 is a magnetostrictive level sensor with accuracy to 0.02 cm for depth measurements.

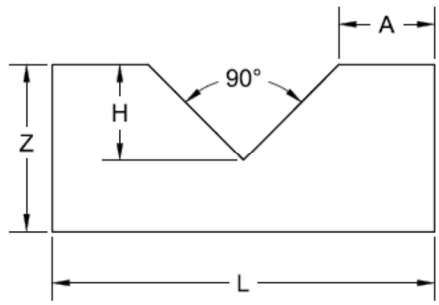

Figure 3.10: 90-degree v-notch weir used throughout the treatment train

Table 3.3: Dimensions for weirs used throughout the treatment train. Where Z is the total height of the weir, H is the height of the weir above the notch, L is the cross sectional width of the weir, and A is the plate width adjacent to the notch.

	Z	H	L	A
Feet	1.75	1	4	1
Meters	0.5	0.3	1.2	0.3

There were several challenges with flow measurement throughout the treatment train. Adequately quantifying flow entering and leaving the system proved to be more difficult than initially thought.

3.2.3 Treatment Train Flow Measurement

Measuring runoff flow rate through the treatment train was an integral part of the hydrologic performance analysis. However, accurate measurement of flow proved to be a difficult task throughout the study. Initially, flow rates were calculated using water depth readings from pressure transducers and the 90 degree, sharp crested v-notch weirs in the treatment train. The Cone equation was used to determine flow rates at each of the 90 degree, v-notch weirs (Equation 1):

$$Q = 2.49h^{2.48} \tag{1}$$

where Q is the discharge over a weir (cubic feet per second), and h is the head above the weir (feet) (USBR 2001). Units were converted to metric following calculations.

Flow rates were compared at the inflow point of the system by using the Small Storm Hydrology (SSH) methodology for a completely impervious area (Pitt 1999), and the Natural Resource Conservation Service curve number method (CN) for determining runoff. The flow rates obtained for calculations using the Cone equation were at times nearly five times greater than the results obtained using both the SSH and CN methods and can be found in Table 3.4. These large differences in measured flow rates were assumed to be due to the error associated with the pressure transducers during times of low flow. The error associated with the pressure transducers was approximately 0.5 cm for each depth readings. This could have caused large variability in recorded depth at very low levels. Further, the depth measurement error is amplified because the depth is raised to the 2.48 power in the Cone equation, leading to even larger increases in flow rates.

Table 3.4: Runoff volume comparisons using the SSH methodology and the uncalibrated Cone equation.

Storm Date	Rainfall (cm)	Calculated Runoff from Parking Garage (m^3)	Runoff Using Uncalibrated Cone Equation (m^3)	Percent Difference
11/1/2013	0.8	6.9	9.3	35.5%
11/26/2013	9.0	82.5	130.8	58.6%
12/6/2013	2.8	25.9	80.3	210.0%
12/9/2013	0.6	5.3	30.5	479.2%
12/22/2013	0.3	2.7	5.6	102.5%
12/23/2013	1.7	15.7	27.9	77.2%
12/29/2013	3.2	29.1	52.3	79.4%

To prevent these large differences in measured flow rate, the Gems XT-1000 was installed at the weir box to measure inflow into the treatment train. The error associated with depth measurement using the XT-1000 was approximately 0.02 cm, which would have less of an effect on depth readings and calculated flow rate. The XT-1000 was calibrated using constant level and variable water level tests. Constant water level tests were performed by recording XT-1000 depth measurements at a constant depth of water. Variable water level tests were performed by recording XT-1000 depth measurements with changing water levels over a period of several hours. Lab calibration tests were performed with the XT-1000 in the housing unit that was used at the treatment train (Figure 3.10). Results of the variable water level tests are provided in Appendix B.

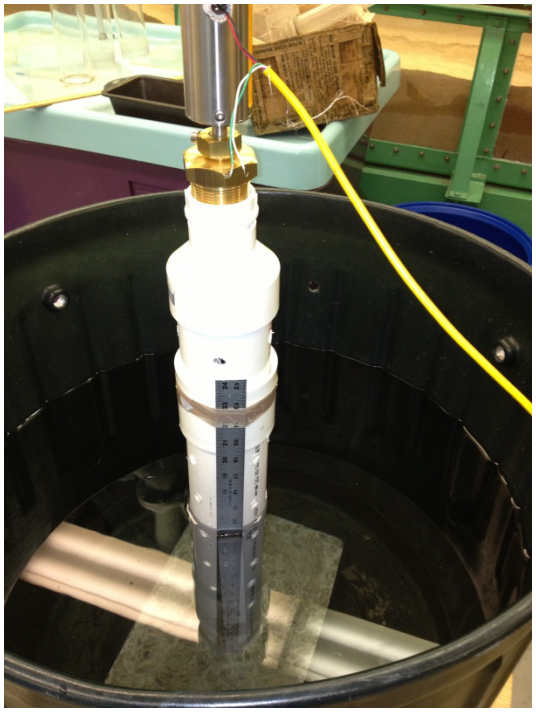

Figure 3.10: Calibration test setup for the XT-1000

When compared to the SSH and CN methods, the calculated flow rates using depth measurements from the XT-1000 were still significantly higher. It was determined that the high

flow rates calculated using depth measurements and the Cone equation were a result of water clinging to the downstream face of the weir at very low heads. The USBR Water Measurement Manual (2001) recommends a head above the weir crest of at least 6 cm to prevent clinging, which can result in approximately a 25% error in flow measurement.

A modified version of the Cone equation was used to account for the height of the weir (2.02 feet) above the bottom of the weir box. Further, the base of the housing unit for the XT-1000 raised the bottom of the instrument approximately 0.03 ft. This slight increase was taken into account in the modified Cone Equation (Equation 2).

$$Q = 2.49(h - 1.99)^{2.48} \tag{2}$$

The Cone equation was calibrated with the XT-1000 in the weir box to prevent future flow measurement errors for inflow at the treatment train. Inflow to the treatment train using the XT-1000 and the 90-degree v-notch weir was calibrated using the SSH method calculations to determine the total inflow volume. Flow rate calculations were obtained every minute using depth readings from the XT-1000. Volume was taken from inflow hydrographs and compared to calculated inflow volume using the SSH method. Acceptable error between flow measurements using the XT-1000 and the SSH method was determined to be 10% based on depth measurement accuracy and the angle of the v-notch weir. The Cone equation was calibrated based on the results of ten rain events during late 2013 and early 2014. The weir notch height in the calibrated equation is raised by 0.031 feet to account for volume overestimates that were occurring during flow measurements (Equation 3).

$$Q = 2.49(h - 2.021)^{2.48} \quad (3)$$

Results from the calibration of the Cone Equation are shown in Table 3.5. Flow rates obtained using the calibrated Cone Equation are within 10% of the results from the SSH method for 8 of the 10 storms that were analyzed. Volume of rainfall events ranged from 0.3 cm to 9.0 cm during the period of analysis. The range of absolute differences between measured and calculated flow rates was between 2.3% and 64%. The results of the December 9, 2013 event (0.23 cm) overestimate volume entering the treatment train by approximately 64%. Due to the much lower variability in runoff measurements in rain events prior to and following December 9, 2013, the volume overestimation may have been due to an error in rainfall measurement.

Table 3.5: Treatment train inflow volume comparisons using the calibrated Cone Equation and the SSH method

Storm Date	Rainfall (cm)	Calculated Runoff from Parking Garage (m^3)	Runoff Using XT-1000 (m^3)	Percent Difference
11/1/2013	0.8	6.9	6.5	-6.2%
11/26/2013	9.0	82.5	86.9	5.1%
12/6/2013	2.8	25.9	28.5	9.3%
12/9/2013	0.6	5.3	14.8	64.4%
12/22/2013	0.3	2.7	2.9	5.0%
12/23/2013	1.7	15.7	14.2	-10.8%
12/29/2013	3.2	29.1	31.0	5.9%
3/12/2014	0.9	8.5	8.3	-2.3%
3/19/2014	1.9	17.3	18.4	5.5%
3/30/2014	7.2	65.7	60.5	-8.6%

Depth measurements using the XT-1000 and the calibrated Cone equation resulted in inflow volumes that were within reasonable error of the SSH method for most rain events. Flow quantification through the treatment train continues to be a challenge. Although the XT-1000 and the calibrated Cone equation proved to produce accurate inflow values in most cases, the SSH method was utilized during this study to determine inflow to the treatment train. Reliable and accurate flow measurements through the treatment train have not been successfully determined yet. However, a proposed method for flow measurement is discussed in Section 5.3.1.

3.3 Water Quantity Analysis

Infiltration and recession rates were calculated for the old IT and the IT at the treatment train to determine hydrologic performance over time.

3.3.1 Old Infiltration Trench

The recession rate (i.e. how quickly the water depth in the IT dropped) can be used to calculate the infiltration rate (IR) over the entire IT surface area that includes the bottom (12.2 m^2) and sides. Recession rates (RR) in the IT were determined for different depths: 0-0.3 m, 0.3-0.6 m, 0.6-0.9 m, 0.9-1.2 m, and 1.2 to 1.6 m. For multiple peak storms, a depth increment may have several infiltration rates that are averaged. This analysis focused on the recession and infiltration rates in the bottom 0.3 m of the IT over the 10 year monitoring period. Recession rates were calculated using the linear slope of depth decrease in the bottom 0.3 m of the IT following a rain event (Equation 4):

$$RR_{0-0.3} = \frac{y_{peak,0-0.3} - y_{final}}{t_{peak,0-0.3} - t_{final}} \quad (4)$$

where $y_{peak,0\text{-}0.3}$ is the peak depth achieved in the bottom 0.3 m of the IT; y_{final} is the lowest depth in the IT following a rain event; $t_{peak,0\text{-}0.3}$ is the time at which $y_{peak,0\text{-}0.3}$ occurs; and t_{final} is the time at which y_{final} occurs.

Infiltration rates were determined using the RR for the bottom layer of the IT. The RR was converted to a flow rate by multiplying by the bottom area of the IT (12 m^2). The flow rate was divided by the surface area of the bottom 0.3 m ($SA_{0.3}$) (Equation 5):

$$SA_{0-0.3} = 12m^2 + 2(l * 0.3\ m) + 2(w * 0.3\ m) \tag{5}$$

where l is the length of the IT (4 m) and w is the width of the IT (3 m). Surface area will be constant (16.6 m^2) for all infiltration rates because only one depth increment in the IT is being analyzed. Infiltration rates were calculated by dividing the product of the RR in the 0.3 m of the IT and the bottom area by $SA_{0.3}$ (Equation 6):

$$I_{0-0.3} = \frac{12\ m^2 * RR_{0-0.3}}{16.6\ m^2} = 0.723RR_{0-0.3} \tag{6}$$

Recession and infiltration rates are reported in cm/h throughout the analysis. The focus of this analysis was events from the month of July from 2004 to 2010 and 2014. Data from 2011 through 2013 was not available due to a lack of site funding during that time; data from July of 2009 was not available due to issues with instrumentation.

3.3.2 Treatment Train

Runoff volume (V_{runoff}) was calculated using the Small Storm Hydrology method (Pitt 1999) for a large, completely impervious drainage area for each rainfall event. The runoff volume into the IT (V_{IT}) after the vegetated components was calculated as height of peak ponding multiplied by the IT bottom area and the porosity of the R-tanks (Equation 7):

$$V_{IT} = 0.95\left(y_{peak} - y_o\right)A_{IT} \tag{7}$$

where y_{peak} is the peak depth reached in the infiltration trench, y_o is the height of the water in the IT when runoff first enters, and A_{IT} is the IT bottom area (5.4 m^2).

Periods of IT overflow through the pavers were observed, although not measured directly. Overflow was considered to occur when the measured IT water depth remained constant at 1.3 m (i.e. the top of the R-Tanks) and higher. Overflow was calculated in two ways to develop a range for possible overflow values. To conservatively estimate the overflow volume (V_{over}), the rate of peak rise was multiplied by the duration of elevated ponding (t_{over}) and the paver area (A_{pavers} = 4.8 m^2) (Equation 8).

$$V_{over} = \left(\frac{y_{peak} - y_o}{t_{peak} - t_o}\right) t_{over} A_{pavers} \tag{8}$$

For a less conservative overflow estimate, volume of infiltration (V_{inf}) was calculated for each overflow event to account for infiltration occurring during the storm event, as well as runoff that reenters the IT after overflowing through the pervious paver. The V_{inf} was calculated multiplying

the infiltration rate from the upper depth increment of the IT ($I_{0.9\text{-}1.3}$), t_{over} and the total surface area available for infiltration (IT sidewalls and bottom, $A_{IT,\ total}$ 17.9 m^2) (Equation 9).

$$V_{inf} = I_{0.9-1.3} t_{over} A_{IT,total} \tag{9}$$

The V_{inf} was subtracted from the V_{over} estimate to yield a less conservative estimate of V_{over}. The total volume reaching the IT ($V_{IT,\ total}$) is the sum of V_{IT} and V_{over} based on the more conservative estimate overflow, and the sum of V_{IT}, V_{over}, and V_{inf} based on the less conservative estimate. In both overflow scenarios, $V_{IT,\ total}$ will be the same.

The volume captured by the system (V_{sys}) is V_{runoff} less V_{over}. Volume captured by the system was calculated using Equation 10:

$$V_{sys} = V_{runoff} - V_{over} \tag{10}$$

where V_{sys} is the total volume captured by the treatment train system (m^3).

Similar to the old IT, the recession rate was used to calculate the infiltration rate over the entire IT surface area that included the bottom (5.4 m^2) and sides (peak depth for each incremental depth range). Infiltration rates in the IT at the treatment train were determined for different depths: 0-0.3 m, 0.3-0.6 m, 0.6-0.9 m, 0.9-1.3 m (Figure 3.11). For multiple peak storms, a depth increment may have several infiltration rates that are averaged. Recession rates and infiltration

rates were calculated using a similar method presented in section 3.3.1. However, several depth increments were analyzed for the treatment train as opposed to only the bottom increment.

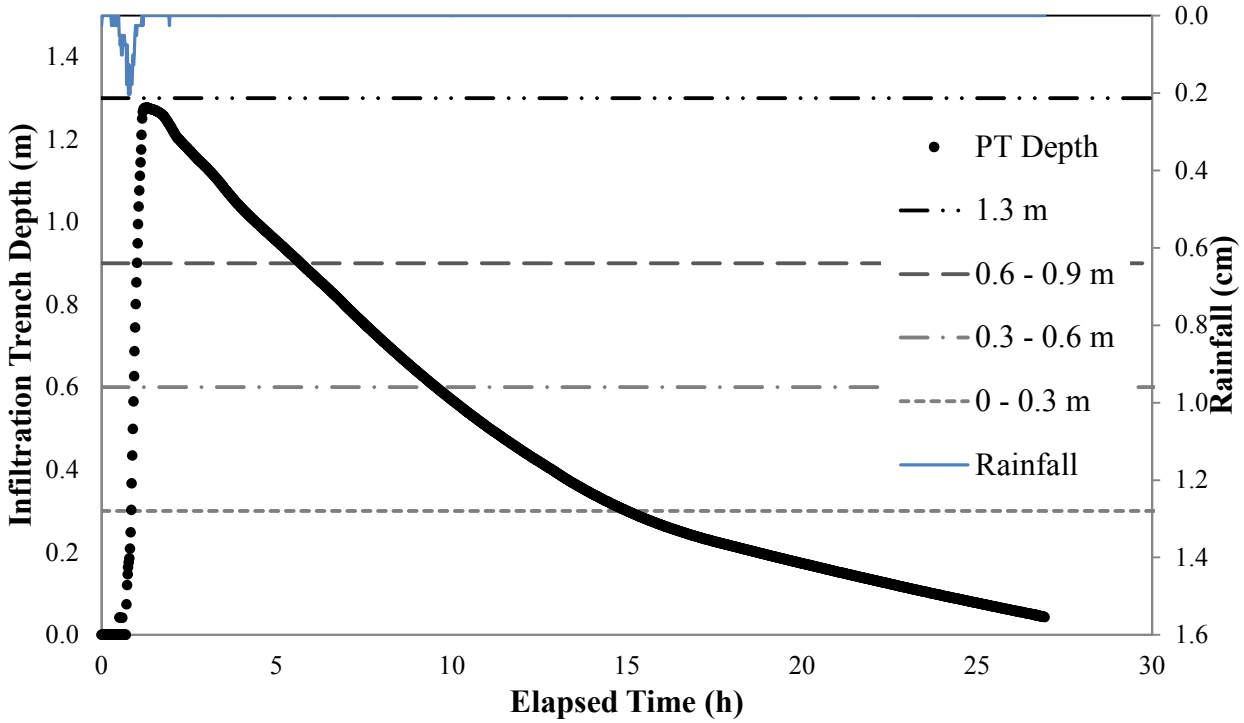

Figure 3.11: An example of the response in the IT due to a 3.6 cm rainfall event (light grey, solid line), which came close to the top of the IT but did not overtop. The observed depth (black dots) was used to calculate the infiltration rates at four different depth ranges (0-0.3 m, 0.3-0.6 m, 0.6-0.9 m and 0.9-1.3 m). The dashed lines indicate the upper boundary of each range.

Emerson et al. (2010) showed a correlation between infiltration rate and temperature based on water viscosity's dependence on temperature; this analysis was also done to determine if the relationship held. A Student t-test ($p = 0.05$) was applied to compare observed infiltration rates to the average temperature for each event.

4 Water Quantity Performance

4.1 Old IT Performance

Over the first three years of operation, the IT underwent a significant decrease in ability to infiltrate runoff (Figure 4.1, Table 4.1). Beginning in 2004, recession rates in the bottom layer (0 – 0.3 m) were as high as 14.4 cm/h with an average of 8.9 cm/h. Decrease in infiltration performance during the month of July can be seen on a yearly basis until approximately 2007 when recession rates become generally stagnant for the bottom layer of the IT. Yearly variance in recession decreases drastically from the beginning of the study period as well. Average recession rates for the month of July remain fairly unchanged from year to year at approximately 0.3 cm/h beginning in 2007.

Recession rates during the month of July were examined from year to year to determine statistical similarities. A Student's t-test was performed (p=0.05) to compare recession rates from year to year over the course of the study. Differences from 2004 and 2005, 2006 and 2007, 2008 and 2010, and 2010 and 2014 were statistically significant (Table 4.2).

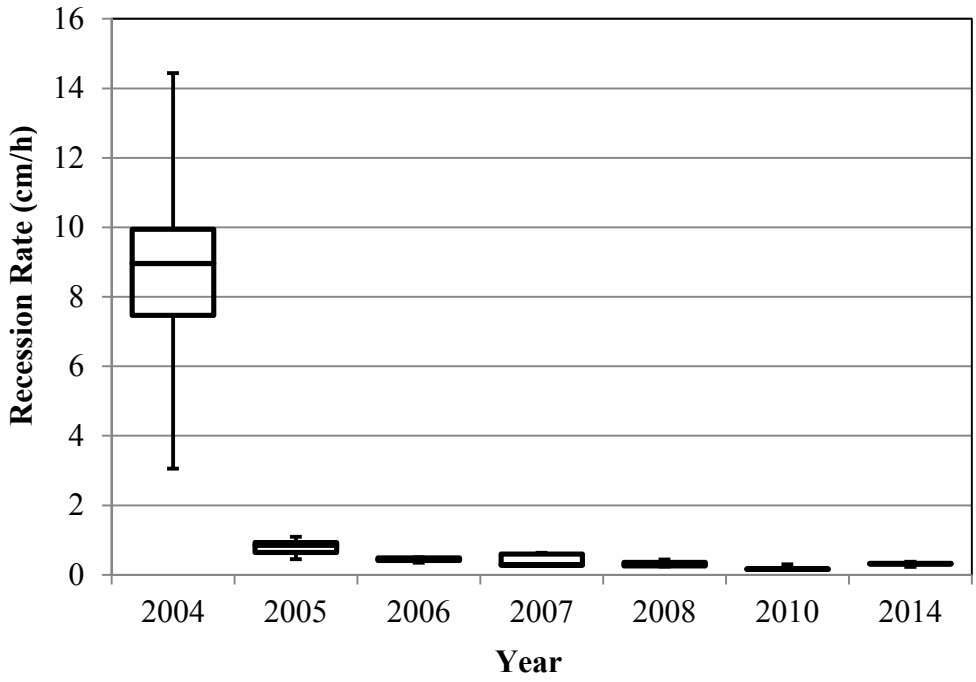

Figure 4.1: Box and whisker plots of recession rates in the month of July throughout the study period.

Table 4.1: Recession rate statistics during the study period for the month of July

Year	Sample size	Average (cm/h)	Maximum (cm/h)	Minimum (cm/h)	Std. Dev.	Variance
2004	12	8.9	14.4	3.1	3.3	10.6
2005	7	0.7	1.1	0.5	0.3	0.1
2006	6	0.4	0.5	0.4	0.1	0.0
2007	5	0.3	0.3	0.3	0.0	0.0
2008	6	0.3	0.4	0.2	0.1	0.0
2010	6	0.2	0.3	0.2	0.1	0.0
2014	4	0.3	0.4	0.2	0.1	0.0

Table 4.2: Comparison of recession rates from year to year. Bold indicates statistical significance.

Yearly Comparison	P-Value
2004 and 2005	**2.2E-06**
2005 and 2006	0.16
2006 and 2007	**1.6E-03**
2007 and 2008	0.68
2008 and 2010	**0.02**
2010 and 2014	**0.03**

The result of clogging in the IT is apparent when examining the minimum depth reached following a rain event. Figure 4.2 illustrates the minimum depth in the IT during rain events in the month of July. Ideally, after a rain event, water levels in the IT would recede until it is completely drained before the next rain event occurs. However, deposition of sediments at the bottom of the IT cause a decrease in recession rates and slows down the rate at which the IT is draining. As a result, the minimum depth achieved after a rain event increases over time. Although the minimum depth achieved in the IT following a storm is partially dependent on frequency of rain events, the minimum depth achieved is an indicator of decreasing recession rates over time. Early on in the life cycle, the IT routinely completely drained and reached an average minimum depth of 3.6 cm following each storm event. However, beginning in 2005, there was an increase in the minimum depth achieved in the IT following a rain event (Table 4.3). Although the IT completely drained in 2010, the average minimum depth achieved was also the highest during that year than any other year in the study period. The IT completely drained in 2010 due to a 8 day period with no rainfall. Increases in minimum depth are due to lower recession rates as a result of clogging from sediments and debris at the bottom of the IT. The minimum depth achieved in the IT is also dependent on storm occurrence during the month of July. However, the average minimum depth achieved from 2005 to 2014 was consistently

between 15.0 cm and 53.1 cm, which suggests that sedimentation was the main cause of increasing depth.

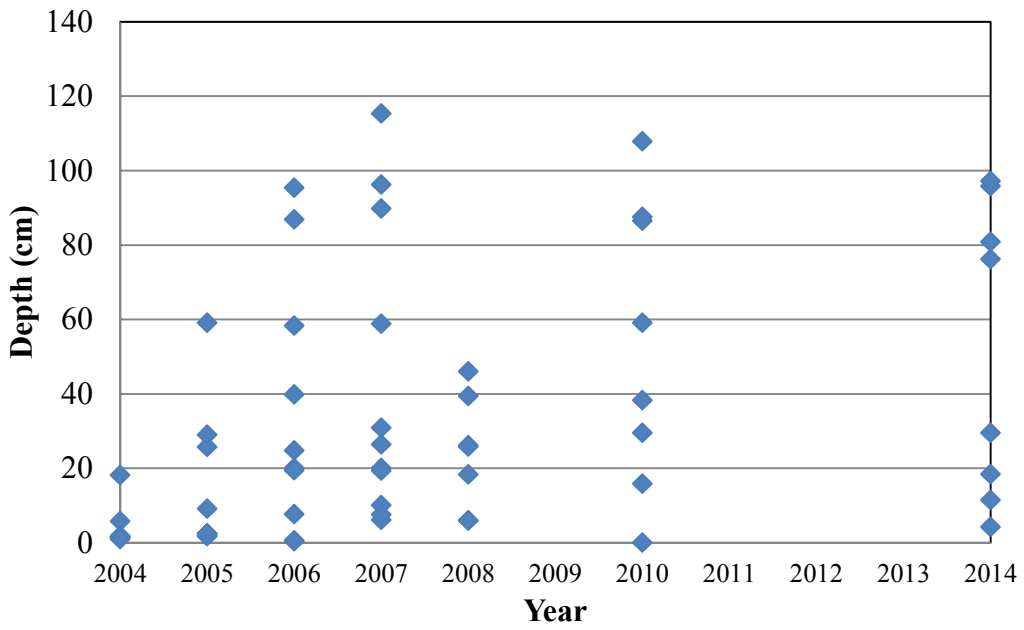

Figure 4.2: Minimum depths in the old IT for rain events occurring in the month of July during the study period

Table 4.3: Minimum depth statistics during the study period for the month of July

Year	Average (cm)	Maximum (cm)	Minimum (cm)	Std. Dev. (cm)	Variance (cm)
2004	3.6	18.2	1.0	5.3	28.3
2005	15.0	59.1	1.8	19.7	386.9
2006	35.3	95.4	0.4	34.3	1177.7
2007	43.7	115.3	6.1	39.6	1568.9
2008	24.0	46.0	6.0	15.4	235.7
2010	53.1	107.9	0.0	38.4	1474.3
2014	48.5	97.2	4.3	32.8	1076.0

Decreasing performance of the IT over the study period was a direct result of sediment loading from the large impervious drainage area. In February 2005, a particle size distribution and hydrometer test were performed at a test pit near the site of the IT to determine the USDA soil

classification of the surrounding soil (Figure 4.3). The soil was determined to be approximately 77% sand, 10% silt, and 13% clay (Emerson 2008), which is a sandy loam.

Soil distribution and hydrometer tests were performed in May 2014 to determine changes to the soil as a result of 10 years of sediment and debris loading from the 100 percent impervious drainage area (Figure 4.3). The sample from the bottom of the IT to approximately 45 cm below the bottom was obtained using a soil sampling probe. The soil was determined to be approximately 60% sand, 15% silt, and 25% clay, a sandy clay loam.

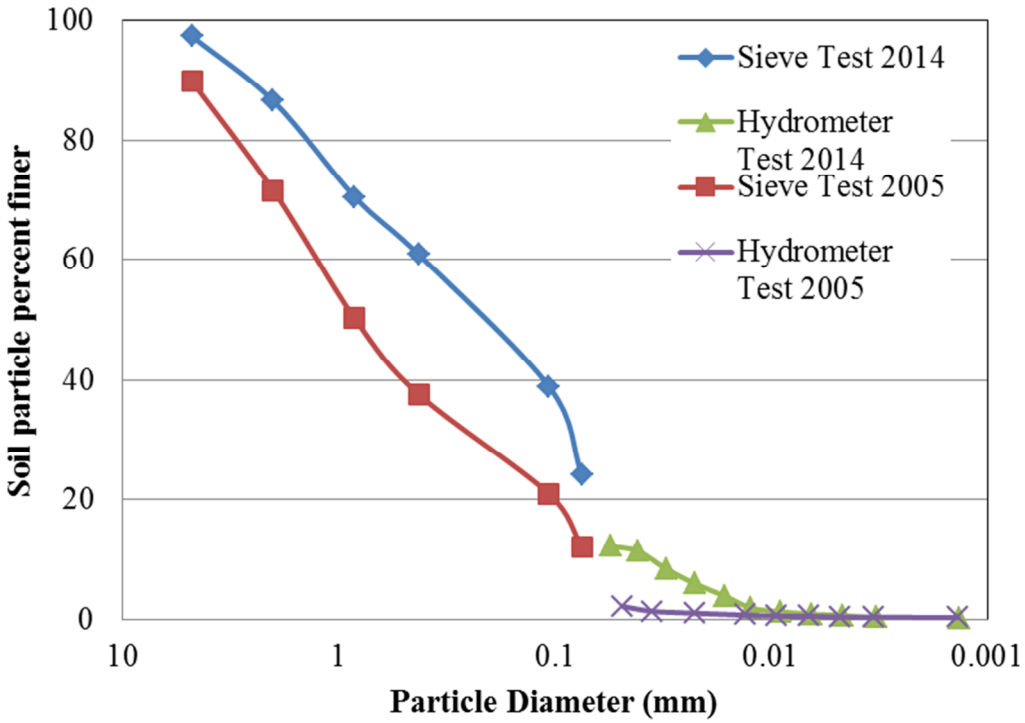

Figure 4.3: Grain size distribution and hydrometer analysis performed on test pit adjacent to IT in February 2005 and on bottom of IT in May 2014.

Results from the particle size distribution and hydrometer test show an increase in fine particles of the soil deposited at the bottom of the IT over 10 years. From 2004 to 2014, the percentage of clay in the soil was determined to increase from 13% to 25% and the percentage of sand in the

soil decreased from 77% to 60%. Saturated hydraulic conductivity (K_s) values for a sandy loam range from 3.6 cm/h to 12.2 cm/h, while K_s values for a sandy clay loam range from 0.1 cm/h to 1.6 cm/h (Rawls et al. 1998). Infiltration rates can be reduced in soils with high clay content due to smaller void space (NRCS 2008). Larger particles and debris likely settled out while runoff entered the monitoring bench at the IT and may not have had a significant effect on decreasing performance. However, smaller soil particles were able to enter the IT. Therefore, the observed decrease in the recession rate in the IT over time is corroborated by the increase in clay content in the soil at the bottom of the IT.

4.2 Treatment Train Performance

The hydrologic performance of the treatment train with a focus on the IT was monitored beginning in 2012. This portion of the study focused on seasonal variation of volume capture and volume capture during large storms.

4.2.1 Seasonal Performance

Emerson and Traver (2008) showed that infiltration rates can vary substantially with temperature as the viscosity of water is temperature dependent. It is interesting to examine runoff temperature as it moves through the IT system (Figure 4.4). The air and runoff temperatures are quite variable through seasonal changes, with a range over 30° C. It appears runoff entering the vegetated pretreatment is generally warmer than the air temperature because the surface of the parking lot warms stormwater runoff. Likewise, the temperature of the runoff entering the IT appears to be slightly greater than the air temperature. However, both runoff temperatures were statistically similar to the air temperature, indicating that the pretreatment does not provide significant temperature buffering and infiltration rates in the IT will vary greatly as seasons change.

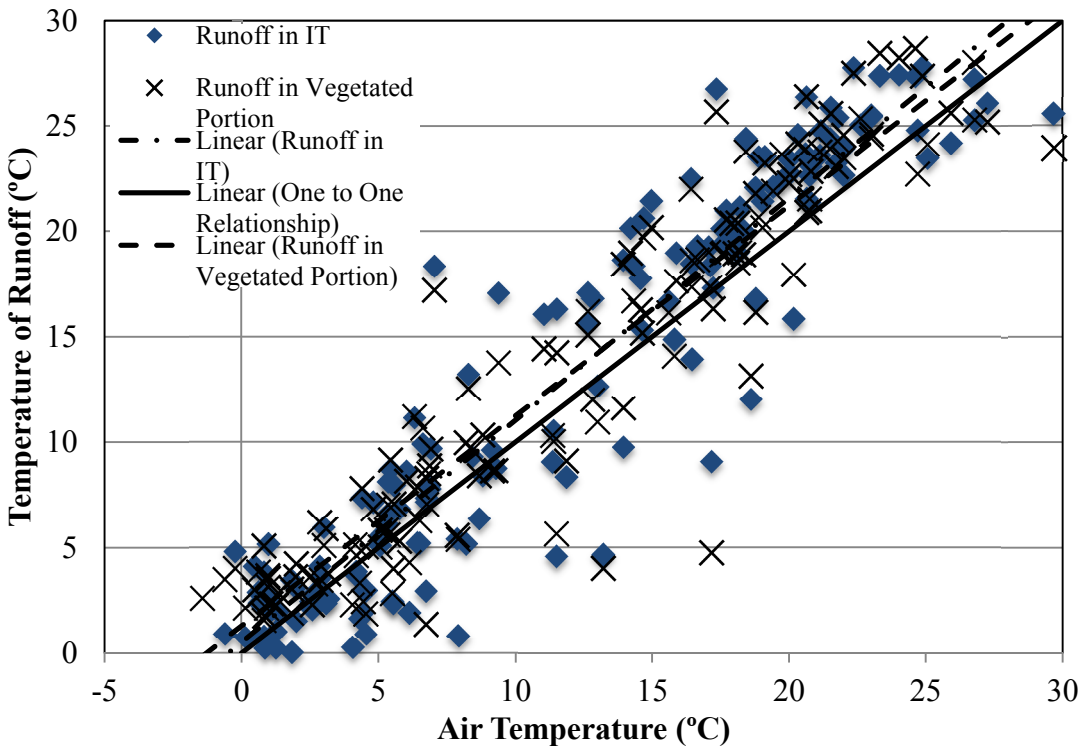

Figure 4.4: Comparisons of temperature of runoff in to the vegetated component (black x) and runoff in to the IT to air temperature (blue diamonds). The dashed line represents the linear regression for the runoff into the IT data set, the dashed and dotted line represents the linear regression for the runoff into the vegetated component data set, and the solid line represents a 1:1 relationship.

The infiltration rate, which dictates the capture volume, varies with temperature and IT ponding depth throughout the storm (Figure 4.5). Braga et al. (2007), Emerson and Traver (2008), Emerson et al. (2010) and Horst et al. (2011) found similar seasonal variations in SCM infiltration rates. There was seasonal variability in infiltration rates at each depth related to temperature – higher temperatures enable higher rates (Figure 4.6). The infiltration rate fluctuates over a wider range with warmer temperatures (> 15° C) than colder temperatures; there were both high and relatively low infiltration rates with warm temperatures. A Student t-test ($p = 0.05$) shows that the infiltration rates in the highest depth (0.9–1.3 m) was not significantly dependent on temperature (Table 4.4). Infiltration rates at the remaining depths (0–0.3 m, 0.3–

0.6 m, 0.6-0.9 m) were significantly dependent on temperature. The infiltration at the bottom of the IT may be more dependent on temperature, rather than the limited area available for infiltration and lower head (Figure 4.6d), exampled by several cases where infiltration rates are greater for shallower depths than the deeper depths. The maximum infiltration rate for the 0-0.3 m range is 14.8 cm/h (Figure 4.6d), and only 11.6 cm/h for the 0.9-1.3 m range (Figure 4.6a).

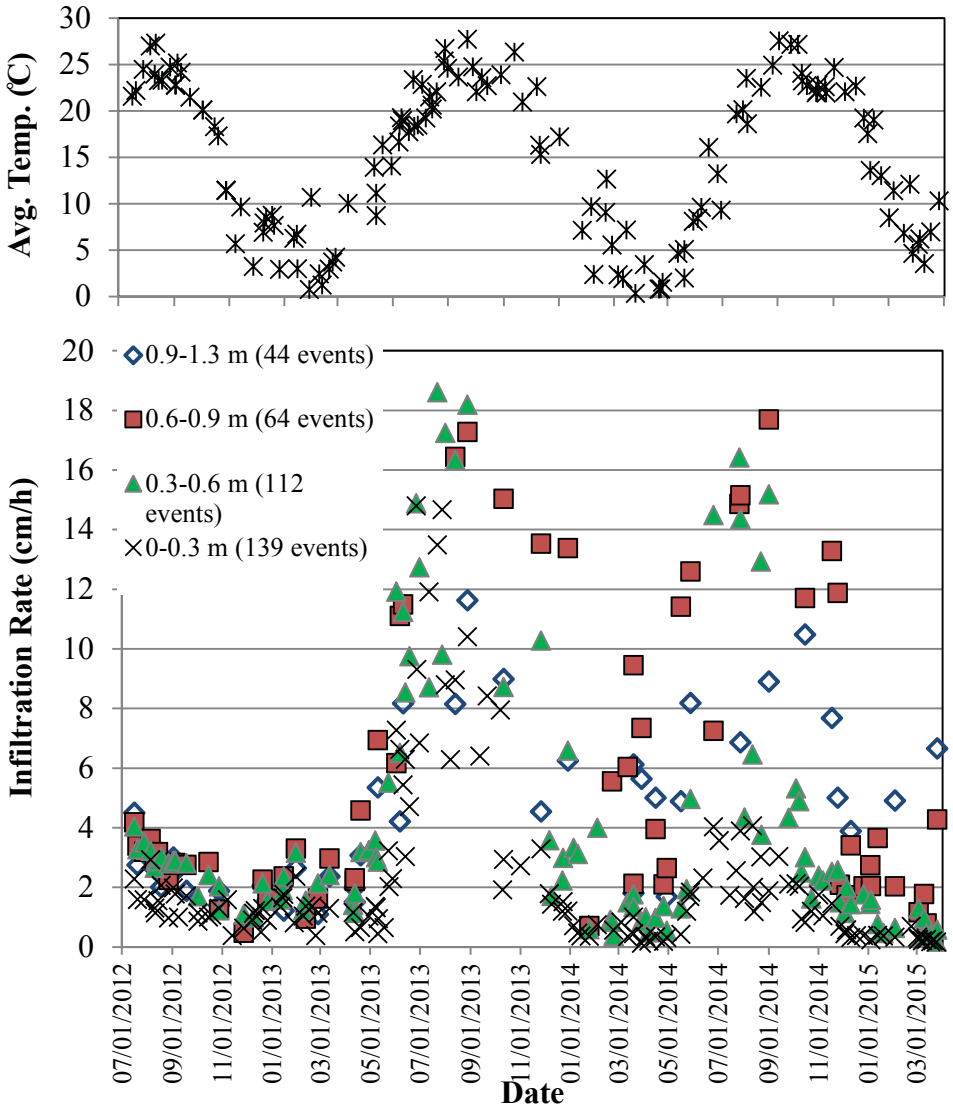

Figure 4.5: Temperature (top) and maximum infiltration rates (bottom) in the IT at each depth range throughout the study period.

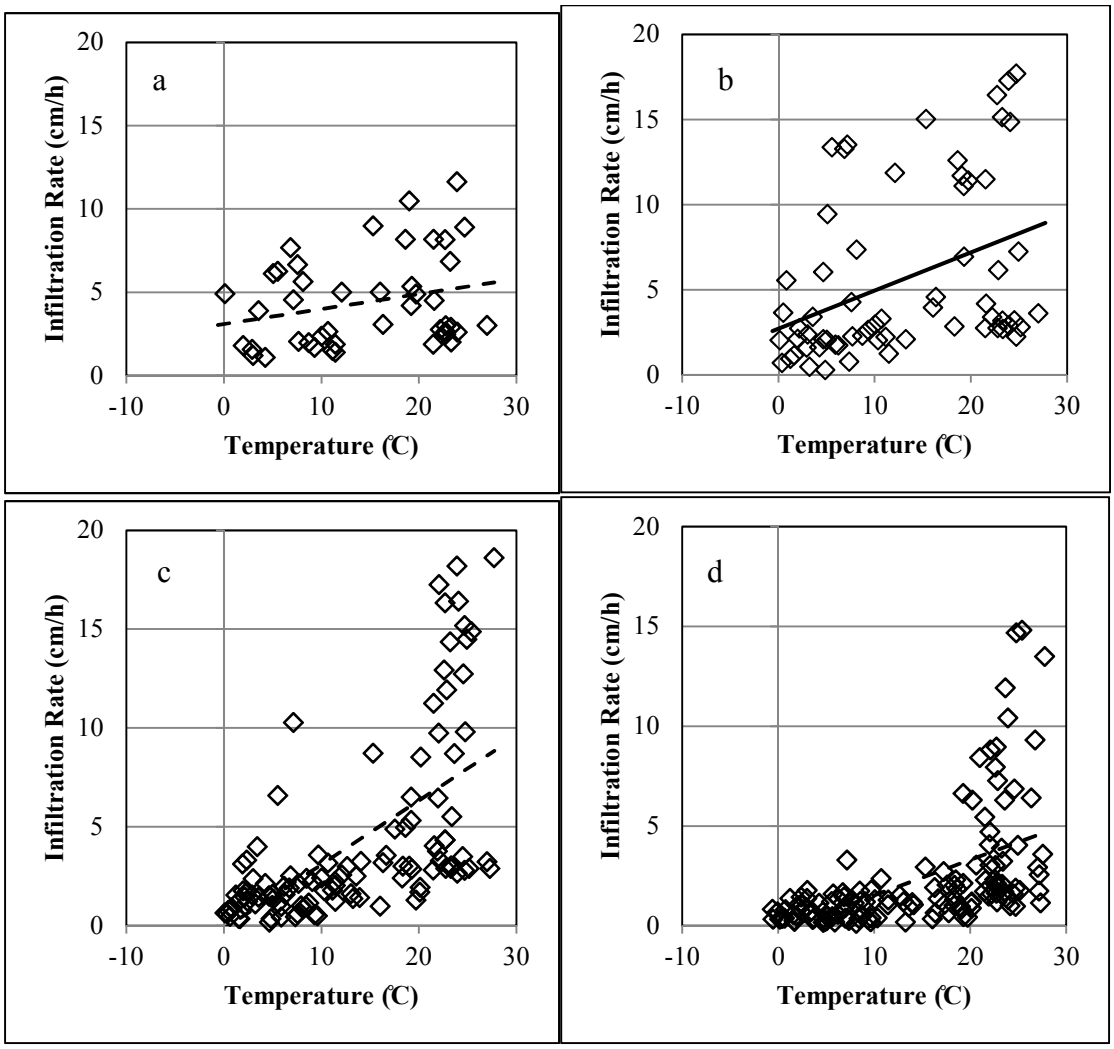

Figure 4.6: Temperature effect on infiltration rate at the IT for each depth range, a) 0.9-1.3m, b) 0.6-0.9m, c) 0.3-0.6m, and d) 0-0.3m. The dashed line represents the linear regression.

Table 4.4: Slope (m), R-squared, correlation coefficients, and t-test results for comparing infiltration rates to temperature at each ponding depth range; bold indicates statistical significance.

Infiltration vs. Temperature							
Range (m)	**m**	**R^2**	**Correlation**	**n**	**t-critical (p = 0.05)**	**$t_{(n-2)}$**	**Significant?**
0.9-1.3	0.09	0.07	0.26	44	2.02	1.74	No
0.6-0.9	**0.22**	**0.15**	**0.39**	**64**	**2.00**	**3.34**	**Yes**
0.3-0.6	**0.32**	**0.38**	**0.61**	**112**	**1.98**	**8.17**	**Yes**
0-0.3	**0.18**	**0.29**	**0.54**	**139**	**1.98**	**7.49**	**Yes**

Generally, a reduction in infiltrating surface area and a decrease in head results in lower infiltration rates. When the IT is completely full (1.3 m) runoff ponds in the upstream rain garden. As water continues to infiltrate, runoff continues to flow from the rain garden into the IT resulting in what appears to be lower infiltration rates at deeper depths. It is evident that infiltration rates are dependent on the ponded level in the IT (Table 4.5). Infiltration rates were highest for the highest two depth ranges (0.6-0.9 m and 0.9-1.3 m) and these rates were statistically similar. These middle depth infiltration rates were not statistically different than the highest range (0.9-1.3 m), but were statistically different than the lowest depth range (0-0.3 m). A decrease in the water level not only decreases the surface area, but decreases the head of water and resulting pressure in the IT. Based on findings from Emerson et al. (2010), side wall infiltration is substantial and therefore when the IT is full infiltration appears to be using all surfaces to infiltrate water out of the IT, but the sides are no longer a substantially large surface when the water depth drops below 0.3 m

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Table 4.5: Infiltration rate statistics (in cm/h) and statistical comparison of depth ranges of infiltration rates. Bold values indicate statistical significance.

						P-Value			
Range (m)	**Mean**	**Std. Dev.**	**Median**	**Q1**	**Q3**	**0.9-1.3**	**0.6-0.9**	**0.3-0.6**	**0-0.3**
0.9-1.3	4.40	2.74	3.48	2.04	6.15	-	0.12	0.63	**0.00**
0.6-0.9	5.58	4.98	3.18	2.11	7.87	-	-	0.05	**0.00**
0.3-0.6	4.11	4.52	2.45	1.35	4.11	-	-	-	**0.00**
0-0.3	2.26	2.89	1.29	0.63	2.20	-	-	-	-

Infiltration rates for the upper depth increment have increased when looking at monthly averages from year to year where the water in the IT reached above 0.9 m (Figure 4.7). A table of monthly infiltration rate averages are provided in Appendix C. Furthermore, infiltration rates in the 0.6-0.9 m depth increment have increased based on monthly average from year to year for every month except April. The average infiltration rate for April 2013 was 3.03 cm/h compared to 2.89 cm/h in April 2014 and was determined to be statistically different. This slight decrease in infiltration rate may have been due to warmer average temperatures in April 2013 (12.9^{o} C) compared to April 2014 (11.3^{o} C). Infiltration rates for the lowest depth range in the IT are variable from month to month and show a decrease in performance from January through June from 2013 to 2014. Infiltration rates at the lowest depth increment during January, March, April, and June were statistically different from 2013 to 2014. However, infiltration rates from February were statistically similar for 2013, 2014, and 2015. Further, infiltration rates for March were statistically similar for 2013 and 2015, and for 2014 and 2015. The decrease in infiltration rates during this time period may have been a result of storm size or antecedent dry time at the site and not clogging. Further, settling of soil surrounding the IT may have resulted in more compacted soil with less infiltration potential. Small storm events would result in less runoff

entering the IT and lower infiltration rates below 0.3 m due to low head. Short time periods between rainfall events may have resulted in reduced infiltration capacity due to soil moisture conditions. Infiltration models, such as the Natural Resource Conservation Service Curve Number method and the Green-Ampt method, are dependent on antecedent soil moisture conditions to predict runoff (Fennessey and Hawkins 2001, Rawls et al. 1983).

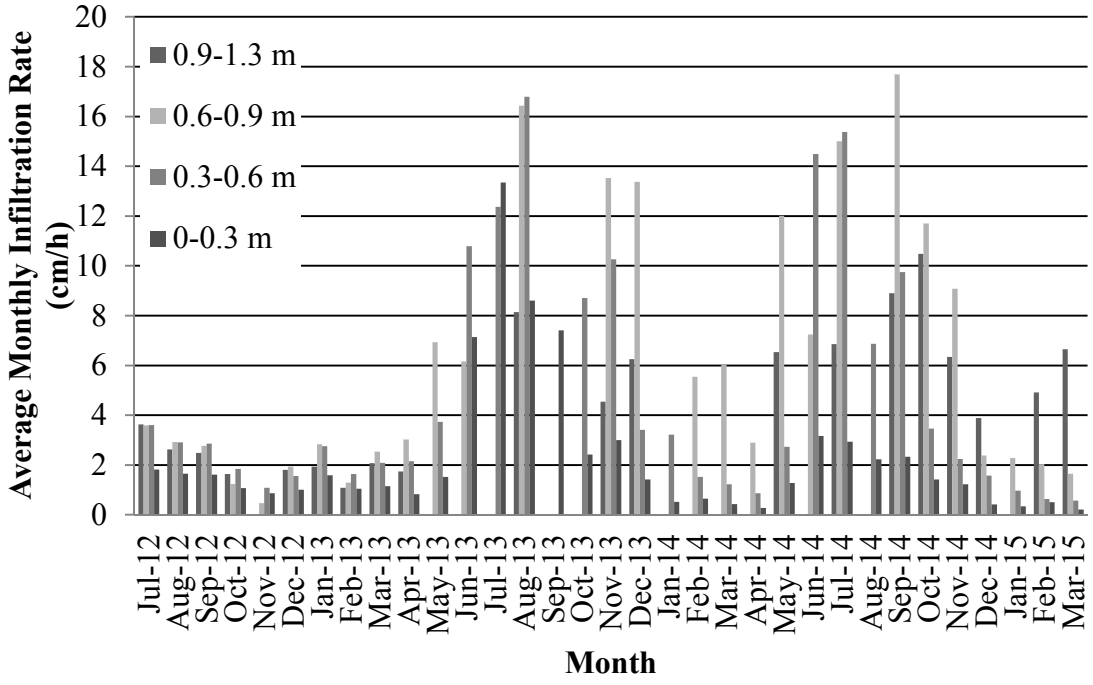

Figure 4.7: Monthly averages of infiltration rates at each depth range throughout the study period.

While infiltration rates at the IT vary based on season and soil moisture conditions and range from 0.12 cm/h to 18.6 cm/h, overall the average infiltration rates at the IT are high enough at all depths to be considered effective by EPA standards (at least 1.27 cm/h; Clar, et al. 2004), and the vegetative system provides some pretreatment so there is no immediate concern of decreasing performance. Additionally, the average infiltration rate in the IT exceeds the minimum requirements listed in various state manuals (e.g. PA BMP Manual (2006) states a rate of 0.64

cm/h is required for infiltrating SCMs in a sandy loam; Maryland Manual suggests a rate of at least 1.33 cm/h (MDE 2009); Minnesota Manual suggests a minimum 0.51 cm/h (MSSC 2008)). Average infiltration rates for all depth increments are similar to or greater than the average saturated hydraulic conductivity for the soil surrounding the IT (2.3 cm/h, Rawls et al.1998).

4.2.2 Large Storm Performance

Over the 33 month study period there were 218 events, with 151 events that produced enough runoff to reach the IT, and 20 events that overflowed (Figure 4.8). While there were events smaller than the design rain of 1.8 cm that reached the IT due to high rainfall intensity (Figure 4.9), seasonal variation in infiltration rates, and antecedent soil moisture, the vegetated components prevented 48% of these storms from reaching the IT, which will extend the longevity of the IT by inhibiting influent sediments from entering. When looking at Figure 1.1, this corresponds to an approximately 1.2 cm rainfall event in the Philadelphia area. It should be noted that the contribution to the IT for 19% of small events (<1.8 cm) was minor (V_{it}/V_{sys} below 0.1). The smallest rainfall event to reach the IT but not overflow was 0.2 cm; this event occurred on December 18, 2012 when the temperature at the IT was 6° C, with a peak ponding depth of 0.3 m, a relatively high average rainfall intensity of 0.7 cm/h, and a relatively low maximum infiltration rate of 0.9 cm/h, compared to a reference saturated hydraulic conductivity ranging between 1.0 cm/h and 6.9 cm/h (Rawls et al. 1998). Rainfall events as small as 1 cm generated overflow (April 12, 2013), but this was a higher intensity event (0.7 cm/h) and the system had only 7.6 h to recover since the previous rain event of 1.4 cm. Conversely, there are large rainfall events (e.g. November 26, 2013 had 9.0 cm, but a low average intensity event (0.30 cm/h) and an eight day antecedent dry period) where approximately 95% of the total volume was captured by the system. When viewed as a system, there was not one specific rainfall depth that

automatically triggered overflow, and of the events with overflow 89% were greater than the design capture volume (2.5 cm, Table 3).

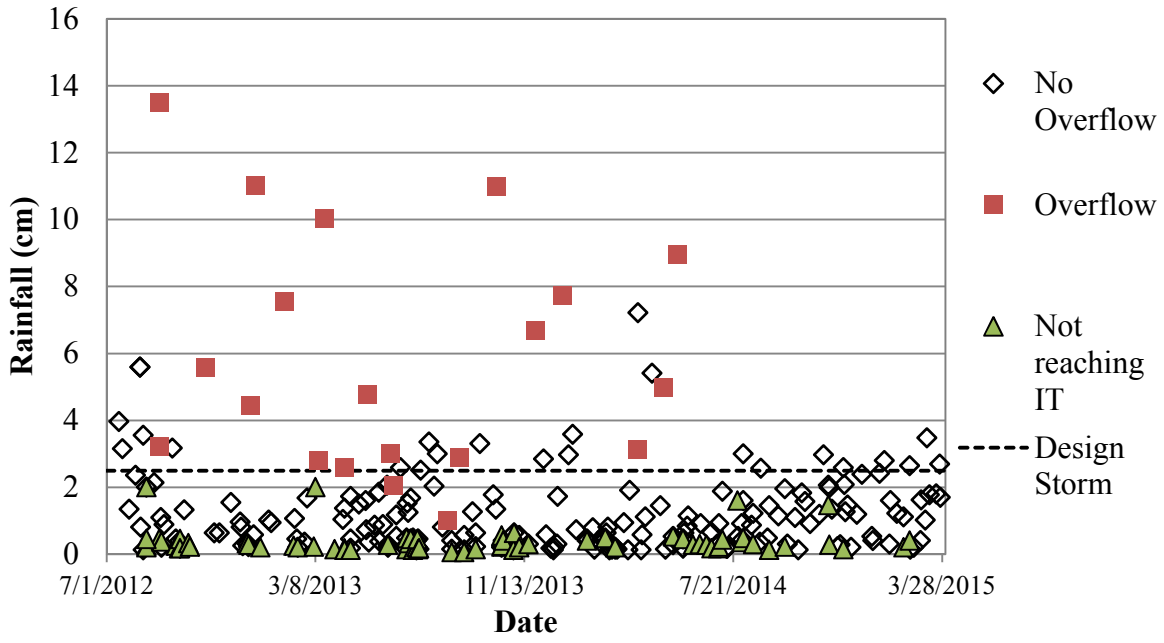

Figure 4.8: All rain events during the study period, delineated by events reaching the IT and overflowing.

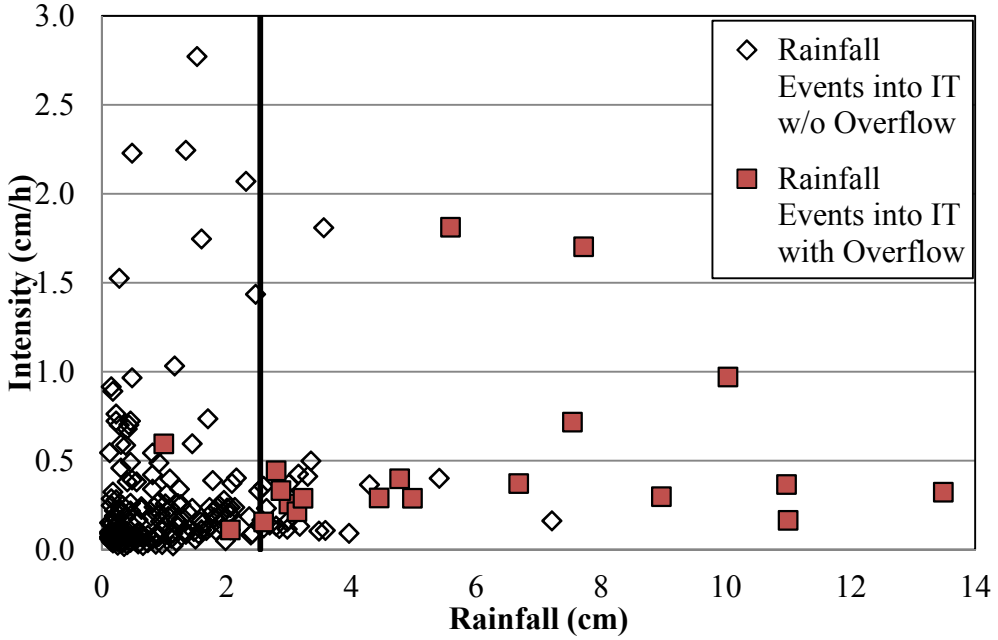

Figure 4.9: Events reaching the IT that did and did not overflow with rainfall depth and intensity. The design event for the entire system is 2.5 cm (black line) and the design event to reach the IT is 1.8 cm.

As a system, the strengths of the components are complementary. During an overflow event, infiltration is still occurring so storage is simultaneously being recovered as the IT is overflowing. Typically only a short duration of the event had overflow occur (Figure 4.10). Superstorm Sandy (October 28-29, 2012) yielded 11 cm of rainfall (V_{runoff} = 101 m^3) over 67 hours, so a large volume event but low average intensity (0.065 cm/h). The volume ponding and infiltrating in the IT ($V_{\neg IT}$) was 8 m^3. There was overflow for approximately 6 hours with an estimated overflow volume (V_{over}) of between 1 m^3 and 3 m^3. The V_{IT}, total was 11 m^3 and the volume captured by the system (V_{sys}) was between 98 m^3 and 100 m^3, yielding a total capture between 97% and 99%. Superstorm Sandy was not an outlier; there were several relatively large events that triggered some overflow, but the system was able to capture the majority of the runoff volume (Figure 4.11), far exceeding the expected capture. The expected capture is 100% of all events less than 2.5 cm and the first 2.5 cm of any event greater than 2.5 cm. For example, the system is expected to capture 50% of a 5 cm event. For all observed events greater than 2.5 cm, there was an observed average capture of 94%. The worst performing event with 59% volume capture was the aforementioned April 12, 2013 event.

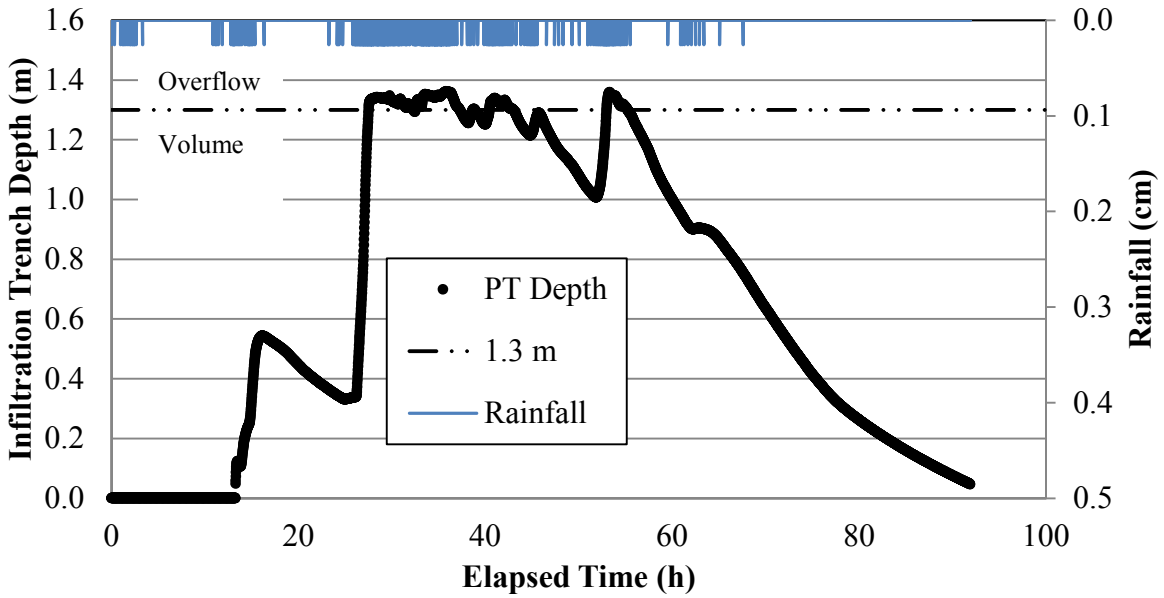

Figure 4.10 - Superstorm Sandy IT depth and rainfall. There was a large rainfall volume (11 cm), but low intensity as there was no 1-minute rainfall observation greater than 0.03 cm. The event was 67 hours long, but overflow was observed for only two periods of 5 hours and 1 hour, where the depth is greater than 1.3 m.

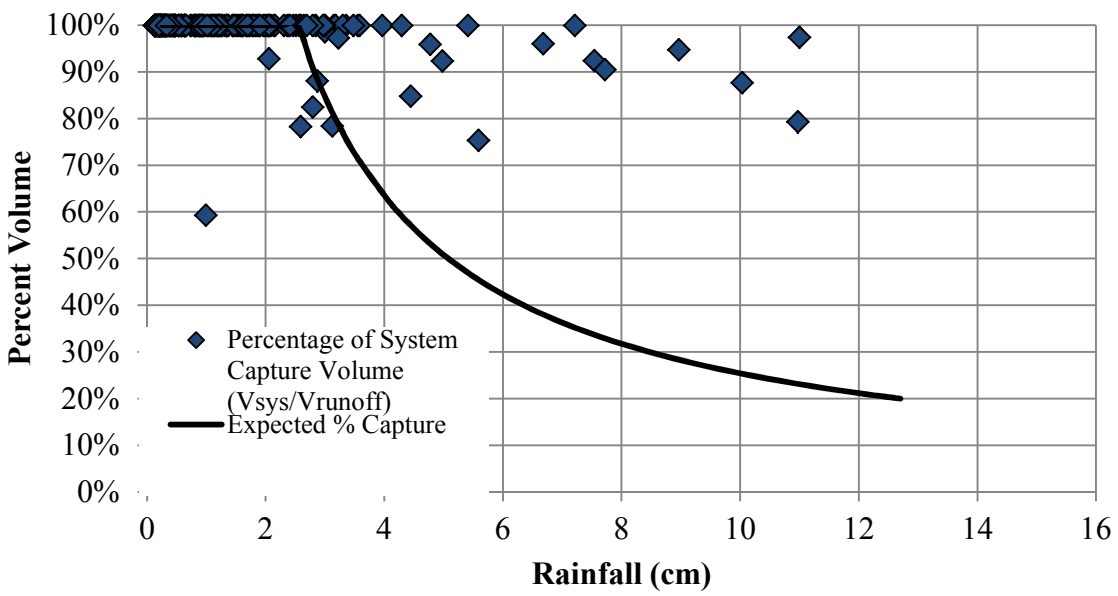

Figure 4.11: Total system performance based on system capture volume over total runoff. Expected system percent capture (solid black line) is shown as a basis for comparison. Overflow volumes were calculated assuming that no infiltration occurs during overflow (Equation 2). To the right of the solid black line are events that captured more than the design.

The analysis shows the IT system as a whole usually performs better than what would be expected from a volume approach during large storm events. The complementary strengths of the SCMs in the system has thus far provided resiliency during large storm events by continuing to capture larger volumes than expected. While this IT is just one example of how infiltrating SCMs continue to recover storage capacity during a large rainfall event via infiltration, the processes can be conveyed to any infiltrating SCM.

4.3 Site Comparison

Performance of the old IT and the IT at the treatment train were compared over the first three years of their operating life. The comparison evaluates infiltration rates in the bottom 0.3 m of both sites during the month of July. The infiltration rates from 2004, 2005 and 2006 were used for the old IT, while infiltration rates from 2012, 2013, and 2014 were used for the IT at the treatment train (Figure 4.12, Table 4.6 and Table 4.7). The old IT shows a drastic reduction in infiltration performance, while the IT at the treatment train experiences an increase in infiltration performance in Year 2 and 3.

Infiltration rates during the month of July were examined from year to year for each site to determine statistical similarities. A Student's t-test was performed (p=0.05) to compare infiltration rates at each site over the first three years of each study. Differences from year 1 and year 2 were statistically significant for both the Old IT and the IT at the treatment train. Differences from Year 1 to Year 3 were statistically significant for the old IT, while differences from Year 2 to Year 3 were statistically significant for the IT at the treatment train (Table 4.8).

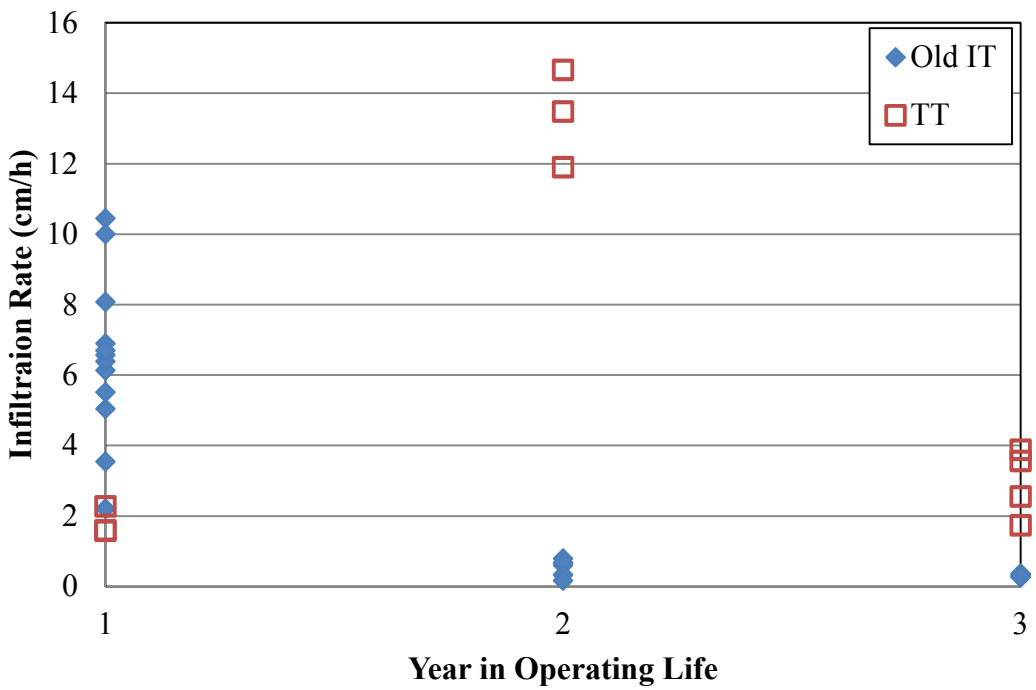

Figure 4.12: Infiltration rates over the first three years of operation for the old IT and the IT at the treatment train

Table 4.6: Infiltration rate statistics for the old IT during the first three years of operation

Year	Average (cm/h)	Max (cm/h)	Min (cm/h)	Std. Dev.	Variance
1	6.5	10.4	2.2	2.4	5.5
2	0.5	3.5	0.2	0.2	0.1
3	0.3	0.4	0.3	0.0	0.0

Table 4.7: Infiltration Rate Statistics for IT at the treatment train for the first three years of operation

Year	Average (cm/h)	Max (cm/h)	Min (cm/h)	Std. Dev.	Variance
1	1.8	2.3	1.6	0.4	0.2
2	13.3	14.7	11.9	1.4	1.9
3	2.9	3.9	1.7	1.0	1.0

Table 4.8: Comparison of infiltration rates from year to year. Bold indicates statistical significance

Yearly Comparison	Old IT	IT at Treatment Train
1 vs. 2	**0.00**	**0.00**
1 vs. 3	**0.00**	0.11
2 vs. 3	0.07	**0.00**

Although infiltration rates decreased from Year 2 to Year 3, infiltration rates in Year 3 were higher than infiltration rates in Year 1. This is likely a result of vegetation stabilization and growth in the pretreatment measures at the treatment train between Year 1 and Year 2. The decrease in infiltration rates at the IT at the treatment train from Year 2 to Year 3 may be a result of sedimentation in the pretreatment measures in the treatment train. Volume capture may have been reduced in the swale and rain gardens as a result of sediment loading from the parking garage. As a result, sediment may have entered the IT at the treatment train and reduced infiltration rates between Year 2 and Year 3. As discussed previously, infiltration performance decreased rapidly at the old IT as a result of sedimentation from a large drainage area. Conversely, infiltration rates increased in Year 2 at the IT at the treatment train. Further research on infiltration rates at the IT at the treatment train will be performed to determine infiltration performance over time.

5 Summary and Future Research

5.1 Old Infiltration Trench

The old IT showed a decrease in performance due to clogging as a result of sediment loading from the large impervious drainage area within the first year of the study period. Statistical analysis of recession rates in the bottom 0.3 m of the IT showed that performance continued to decrease during the month of July until approximately 2007, at which point average recession rates were consistently approximately 0.3 cm/h. Furthermore, a decrease in the variance of recession rates occurs as a result of decreasing maximum recession rates. Additionally, sedimentation from the large impervious drainage area caused the recession rates to slow, which led to an increase in the minimum depth achieved in the IT after a storm event and a decrease in storage available in the IT for the following rain event. Although the minimum depth achieved following a storm can depend on timing of storm events, there was an overall increase in the average minimum heights in IT following storm events in the month of July. The increase in minimum depth in the IT is likely due to a decrease in recession rates over time. Soil particle size distribution and hydrometer tests showed changing soil conditions at the IT. The amount of clay found in the soil increased from 13% to 25%. This finding is significant because high clay amounts in soils can reduce effective pore space available for infiltration. The decrease in effective pore space available for infiltration resulted in a decrease in hydrologic performance and volume capture over time (Table 5.1).

Table 5.1: Runoff volume capture at the old IT during the month of July

Year	Rainfall (cm)	Runoff Volume (m^3)	Volume Captured (m^3)	Fraction of Total Runoff Volume Captured
2004	27.8	521.9	93.5	0.18
2005	13.9	261.7	58.2	0.22
2006	11.5	215.9	72.3	0.33
2007	9.9	186.4	59.7	0.32
2008	8.0	150.1	70.1	0.47
2010	19.9	369.0	63.3	0.17
2014	12.5	234.5	45.5	0.19

The fraction of runoff volume captured in 2004 was 0.18, or 18% of the runoff from the total drainage area, which is the second lowest runoff percentage captured throughout the study. This was likely due to the large amount of rainfall in July of 2004. Although the fraction of volume captured for 2004 was one of the lowest during the study, the total volume captured was the greatest when compared to the remainder of the study. The largest percentage of runoff captured in the old IT occurred in years with the lowest rainfall totals.

Of course, proper site selection, design, construction, and maintenance should be ensured when using infiltrating SCMs in urban setting. Pretreatment, such as vegetated swales, rain gardens, or sedimentation basins, can be used separately, or in series, to reduce the amount of sediment and debris entering to an IT. Additionally, infiltrating pretreatment practices can reduce runoff volume entering an IT potentially resulting in decreased maintenance and increased longevity. Furthermore, an effort should be made to reduce the contributing drainage area to an IT to prevent sediment overload and an overall decrease in performance.

5.2 Treatment Train

Infiltration rates in the IT at the treatment train tend to vary with ponded depth and temperature with higher rates at the mid-depths and higher temperatures (> 15° C). Although infiltration rates in the lowest depth increment have decreased through the first half of 2014, there was a general increase in infiltration rates over the course of the study period. The infiltration trench system was designed for 2.5 cm of total volume control based on the dimensions of the vegetated swale, rain gardens, and IT. However, the system has been able to capture and remove a significantly greater amount of runoff volume. While periods of overflow have been observed, the IT system's ability to capture, retain, and infiltrate runoff from a 100% impervious area has shown its resiliency during large storms. The trench overflowed during 20 of the 218 analyzed storms (9%) over the period of study, which is less than the 36 storms greater than 2.5 cm that occurred and were expected to cause overflow based on the design volume capacity of the system. Capture performance based on the total volume into the system and overflow volume not accounting for infiltration during rain events was a minimum of 59%, with an average of 96%, meaning that the annual volume of runoff from almost every storm was completely captured. Of the storms analyzed, a total of 3,139 m^3 entered the system (338 cm of rain over the entire drainage area) and approximately 556 m^3 entered the infiltration trench. Between 73 m^3 and 103 m^3 of rainfall was determined to overflow from the system. This infiltration SCM system has proven to be very dynamic in terms of volume capture. Infiltration (recharge) capacity of the system has accounted for significant runoff volume reduction during storm events larger than 2.5 cm. Rainfall characteristics prove to be an important factor in the system performance. High intensity storms were more likely to cause overflow, regardless of total rainfall volume. While the results here are specific to an infiltration trench, given that this IT has similar seasonal performance to other

infiltration trench systems (Emerson et al. 2010; Horst et al. 2011), the performance during large events is transferrable. Additionally, these findings are applicable to other types of infiltrating SCMs, such as bioretention systems and pervious pavements (Braga et al. 2007; Emerson and Traver 2008; Gilbert Jenkins et al. 2010; Horst et al. 2011; Lord et al. 2013).

A system's ability to infiltrate runoff also has an impact on water quality treatment of polluted stormwater runoff. When an infiltrating SCM becomes clogged and infiltration capacity is lost, untreated stormwater can runoff into a nearby storm drain system or stream. A water quality analysis was performed for the treatment train based on the total volume infiltrated over the operating life. Several pollutants were analyzed, including: total suspended solids (TSS), total dissolved solids (TDS), total nitrogen (TN), total phosphorus (TP), nitrite, nitrate, phosphate, chloride, copper (Cu), cadmium (Cd), and chromium (Cr). High average pollutant concentrations from the parking deck were taken from the first flush analysis performed by Batroney et al. (2007). The treatment train has captured 3,034 m^3 out of 3,139 m^3 (96%) of total runoff from the SAC parking garage. Table 5.2 provides an estimate of total and captured pollutant loading to the treatment train over the course of study.

Estimated pollutant loading and capture at the treatment train emphasize the importance of volume capture, proper design, and system longevity. Pretreatment measures act as additional volume capture and prevent runoff from reaching the IT. Properly functioning infiltration systems have the potential to provide water quantity reduction as well as significant water quality benefits.

Table 5.2: Estimated pollutant loading and capture at the treatment train

Pollutant	Average high C (mg/L)	Pollutant loading to treatment train (lbs.)	Pollutant capture in treatment train (lbs.)
TSS	24.3	168	163
TDS	55.1	381	369
TN	3.2	22	21
TP	0.21	1.5	1.4
Nitrite	0.07	0.48	0.47
Nitrate	0.31	2.14	2.08
Phosphate	0.04	0.28	0.27
Chloride	2.57	18	17
Cu	0.011	0.08	0.07
Cd	0.0006	0.0042	0.0040
Cr	0.0039	0.027	0.026

Design standards and regulations for SCMs often only consider the volume capacity of a system based on surface storage, which is an underestimate of the observations from this study. Even considering the soil storage capacity in design still underestimates performance because this design standard is a static volume. The treatment train was designed to capture the 2.5 cm storm, however the treatment train has captured an average of 3.9 cm of runoff from the drainage area during rain events greater than 2.5 cm. Infiltration is a continual, dynamic process constantly recovering capacity during a storm event as opposed to a finite storage volume and thus always provides more actual storage capacity than designed. Many SCMs are designed with gravel, sand, or engineered media, which have relatively high saturated conductivity that allows for significant infiltration even under saturated conditions. Adjusting design standards and regulations to account for infiltration during storm events would properly account for additional volume capture and pollutant reduction for infiltrating SCMs. Accounting for additional volume capture and continual infiltration could help cities such as New York develop resiliency plans

including infiltrating green infrastructure practices. Additionally, infiltrating SCMs could be considered for flood control based on performance of the treatment train during large storm events. Approaches would need to account for an individual system's ability to infiltrate and retain runoff during storm events that could lead to more effective SCM design and stormwater management.

5.3 Future Research

Several research tasks are planned for the treatment train to determine volume capture performance of the whole system and of each individual SCM. New flow measurement techniques would allow runoff volume reduction to be measured at each individual SCM through accurate flow measurement moving through the system. Further, accurate modeling could provide flow, runoff and evapotranspiration (ET) quantification at the treatment train. Flow measurement could also provide calibration to ensure model accuracy.

5.3.1 Flow Measurement

Accurate flow measurement at the treatment train has been difficult since the project first began. Complications due to instrumentation and low flows at v-notch weirs have resulted in erroneous flow measurements that require significant calibration. Inflow and outflow measurement can be reasonably estimated with the help of rainfall runoff methods, and volume calculations based on the dimensions of the IT at the treatment train. However, flow measurement through the swale and two rain gardens has been difficult due to the lack of a calibration method.

One of the challenges with flow measurement at the treatment train has been accurately quantify low flow rates. Further, flow measurement at each SCM has been difficult due to unknown volume reduction as runoff travels through the treatment train. Based on rain events during the

study period, inflow to the treatment train can be as high as approximately 250 gallons per minute (gpm), while inflow to the IT at the treatment train has ranged between 0.7 gpm to 42 gpm. Significant runoff volume reduction is occurring, however this has been difficult to quantify for each SCM.

The implementation of magnetic flow meters has been investigated for use at the treatment train. Two separate magnetic flow meters would be used in series to account for both low and high flow rates in each SCM. Low flow rates (0.1 to 26.4 gpm) at each SCM could be measured using a 1-inch diameter IFM SM8001 magnetic inductive flow meter. Further, high flow rates would be measured using a 2-inch diameter Seametrics WMP-Series magnetic flow meter with a flow range of 6 gpm to 300 gpm. The IFM flow meter would be installed downstream of the Seametrics flow meter to prevent erroneous low flow readings. A vertical pipe outlet would be placed between the Seametrics and the IFM flow meter to allow an additional outflow point during periods of high flow. The system of PVC pipes would be designed with an upturned elbow at the downstream end to ensure full pipe flow at all times for accurate flow rate measurement. The current v-notch weirs would be replaced with a high density polyethylene (HDPE) plastic sheet to allow ponding at each SCM. Initial designs of the flow measurement system for the treatment train are included in Appendix D. Measurements from the treatment train survey and specification of each flow meter were used to adequately size the flow measurement stations.

Accurate flow measurements could be used to determine how much volume capture is occurring at each SCM. Quantifying volume capture in each SCM could lead to recommendations for

effective pretreatment measures and volume capture potential for each SCM type. Further, accurate monitoring and instrumentation has led to dynamic monitoring for SCMs. Dynamic monitoring through companies like OptiRTC could lead to increased volume capture and water quality as a result of optimal SCM performance.

5.3.2 Modeling

Several models were investigated to accurately simulate hydrologic processes at the treatment train. Models with continuous simulation capabilities were evaluated to perform simulations for the treatment train using multiple years of rainfall data. The EPA Stormwater Management Model (SWMM), RECARGA, and the Soil Water Atmosphere and Plants (SWAP) model capabilities were reviewed to determine which would be best for modeling the treatment train.

5.3.2.1 EPA SWMM

The EPA Stormwater Management Model (SWMM) is a tool used to model rainfall runoff in primarily developed areas. SWMM Version 5.1 has the capability of modeling low impact development (LID) practices, such as vegetated swales, rain gardens, and infiltration trenches. Users can input rainfall, evaporation, and inflow as part of modeling. Additionally, SWMM has the capability of performing continuous simulation. SWMM models LID practices by performing a water balance throughout the simulation to account for water movement in soils and accounts for storage and runoff in a LID practice (Rossman 2010).

Rain gardens and infiltration trenches are modeled using the Green-Ampt method for infiltration based on soil and storage layer properties input by the user. Infiltration for vegetated swales are modeled using the user defined subcatchment infiltration method within SWMM. Surface storage is calculated based on the dimensions of the swale and the velocity of runoff moving

through the swale. There is no input for multiple soil layers when modeling the vegetated swale. Therefore, additional infiltration and volume capture as a result of the layer of engineered media within the swale would be neglected during model runs. One potential solution would be to model the vegetated swale as a long rain garden to account for infiltration. While this could more accurately account for infiltration, the movement of surface runoff through the swale as a result of the bed slope would be ignored. SWMM is a dynamic rainfall runoff model with many applications, but there are limitations for modeling LID practices.

5.3.2.2 RECARGA

RECARGA was developed at the University of Wisconsin to model bioretention facilities, rain gardens, and infiltration basins (Atchison and Severson 2004). Movement of water is continuously simulated for up to three soil layers. Users can input precipitation and evaporation data for inclusion in the model. Similar to SWMM, RECARGA performs a water balance to account for movement of water within the facility and records results at user-specified time steps. Inflow to the model can be calculated using the SCS curve number methodology or by user input. The Green-Ampt infiltration method is used along with the van Genuchten relationship to account for drainage between soil layers. Further, users have the option to enter a regional average of evapotranspiration in the model.

RECARGA provides the results of the water balance to the user as part of the results. Additionally, plant survivability results are provided in terms of hours of observed ponding and overflow events. While RECARGA is a more specific model for infiltrating SCMs, there are still limitations for modeling the vegetated swale. The approach for modeling the swale as a rain

garden could be used in RECARGA as well as SWMM. Although there is no specific tool for modeling swales and ITs as there is in SWMM, the soil layers within the model could be adjusted to accurately represent each SCM. Unlike SWMM each SCM in the treatment train would need a separate model. Runoff hydrographs could be taken from each model output and used as model input for the next downstream SCM. RECARGA offers more detailed infiltration modeling of SCMs that is not available in SWMM, however creating separate models for each SCM in the treatment train is not an ideal approach.

5.3.2.3 SWAP

SWAP is a continuous simulation model that has the capability to simulate subsurface and surface water transport. The model was developed by Alterra and Wageningen University to model field scale transport processes during growing season (Kroes et al. 2009). The user can input a wide range of data into SWAP, including meteorological, evapotranspiration, and regional groundwater flow. Additionally, options for calculating ET are available, including Penman-Monteith and reference ET data. SWAP uses the one dimensional Richard's equation to model infiltration in a system. Further, the Van Genuchten function is used to simulate flow between soil layers with the model. Surface runoff is modeled through a water balance of inflow and infiltration terms. SWAP provides output for a variety of factors, including surface water flow, ground water flow, and evapotranspiration. Compared to SWMM and RECARGA, SWAP is a more robust model for the simulation of infiltration and performing a water balance within a SCM. However, like RECARGA, there are no specific options for modeling particular SCMs within the program. Additionally, each SCM would need to be modeled separately. Runoff hydrographs from SWAP output could be used as input to model the next downstream SCM in

the treatment train. With proper detailed model input, SWAP would likely provide the most realistic model results for each SCM.

Modeling approaches continue to be analyzed for the treatment train. While three models have been considered, a combination of models may be employed to properly simulate hydrologic processes at the treatment train. Future research will include selecting the proper model, or combination of models, and developing simulations to accurately model hydrologic processes at the treatment train.

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Appendix A: Survey results

Table A.1: Survey results from November 7, 2013

Point	Distance between points (ft.)	Cumulative Station (ft.)	Elevation (ft.)	Relative Elev. (ft.)
0	0.0	0	3.69	435.29
1	7.7	7.7	5.76	437.28
2	0.0	7.7	5.16	436.76
3	7.1	14.8	4.93	436.53
4	9.0	23.8	4.65	436.25
5	8.2	32	4.56	436.16
6	10.7	42.7	4.41	436.01
7	7.4	50.1	4.3	435.90
8	9.5	59.6	4.13	435.73
9	8.4	68	3.73	435.33
10	3.5	71.5	3.59	435.19
11	0.0	71.5	3.32	434.92
12	8.6	80.1	2.81	434.41
13	11.2	91.3	2.39	433.99
14	18.1	109.4	2	433.60
15	16.1	125.5	1.51	433.11
16	8.5	134	1.53	433.13
17	2.7	136.7	1.62	433.22
18	0.0	136.7	1.23	432.83
19	5.1	141.8	0.93	432.53
20	4.3	146.1	0.89	432.49
21	3.5	149.6	1.03	432.63
22	0.0	149.6	0.65	432.25
23	5.3	154.9	0.38	431.98
24	3.5	158.4	0.46	432.06
25	3.4	161.8	0.74	432.34
26	0.0	161.8	-0.32	431.28
27			1.49	433.09
28	U/S Left Corner of IT		1.07	432.67
29	U/S Right Corner of IT		1.14	432.74
30	D/S Left Corner of IT		1.34	432.94

Point	Distance between points (ft.)	Cumulative Station (ft.)	Elevation (ft.)	Relative Elev. (ft.)
31	D/S Right Corner of IT		1.31	432.91
32	Invert of IT		-2.96	428.64
33			0.32	431.92
34	Manhole cover (reference point)		0	431.60

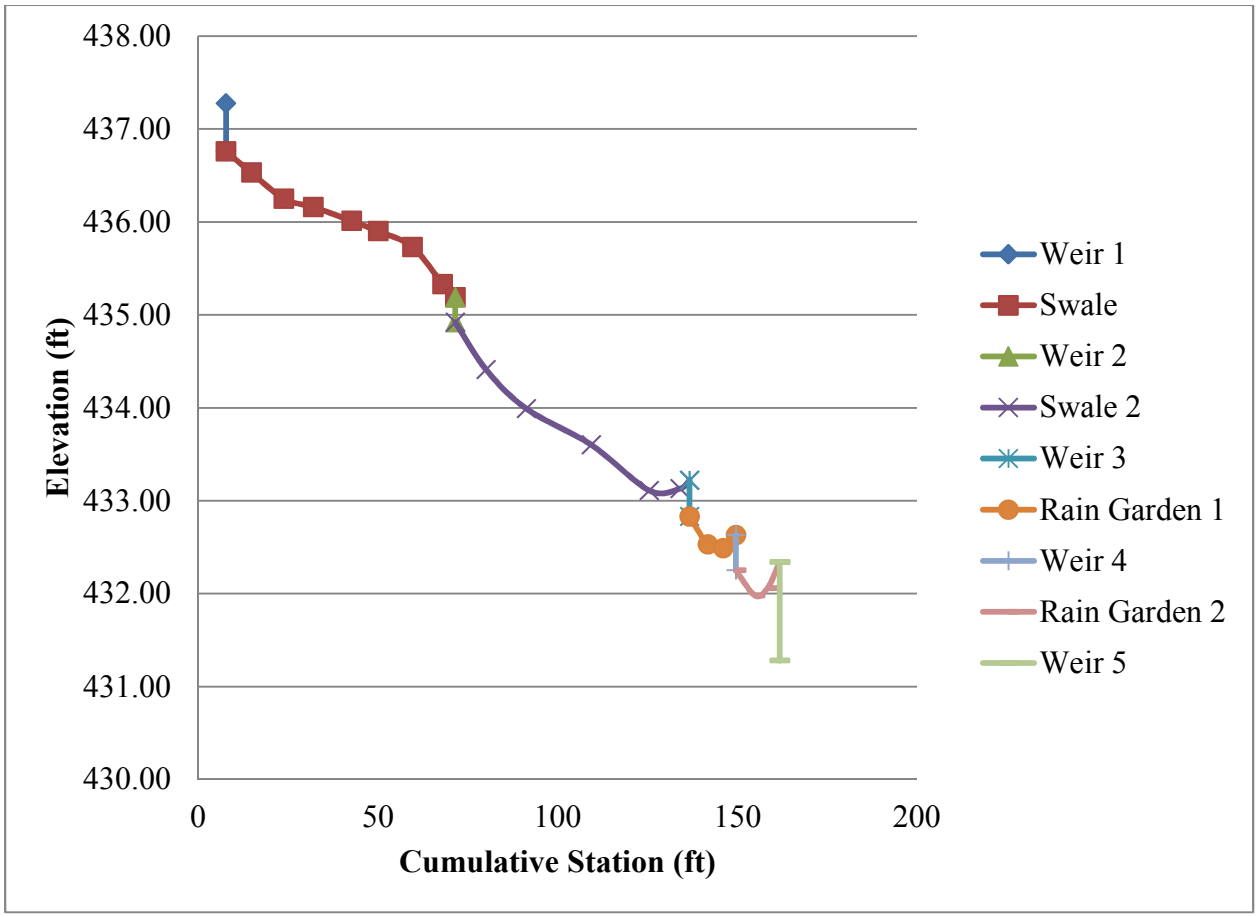

Figure A.1: Survey results from November 7, 2013

Table A.2: Survey results from May 28, 2014

Point	Cumulative station (ft.)	Elevation (ft.)	Relative Elev. (ft.)
9	0	5.69	437.29
10	0	5.22	436.82
11	32	4.39	435.99
12	68	3.49	435.09
13	71.5	3.55	435.15
14	71.5	3.34	434.94
15	103	1.97	433.57
16	134	1.41	433.01
17	136	1.88	433.48
18	136.7	1.72	433.32
19	137	1.19	432.79
20	143	0.74	432.34
21	149	0.99	432.59
21a	149.6	1.2	432.80
22	150	0.65	432.25
23	156	0.38	431.98
24	158.4	0.45	432.05
25	161.3	0.42	432.02
25a	161.8	0.72	432.32
26	162	-0.26	431.34
27a		1.1	432.70
27b		1.07	432.67
28	U/S Left Corner of IT	1.33	432.93
29	U/S Right Corner of IT	1.35	432.95
30	D/S Left Corner of IT	1.15	432.75
31	D/S Right Corner of IT	1.09	432.69
32	Invert of IT	-2.95	428.65
33		0.32	431.92
34	Manhole cover (reference point)	0	431.60

Appendix B: XT-1000 Calibration

Table B.1: Results from variable head test for XT-1000 on September 13, 2013

Height Reading from XT-1000 (in)	Actual height measured (in)	mV from XT-1000
5.46	6.75	1428.8
6.51	7.75	1534.5
7.64	8.75	1648.0
9.87	10.75	1872.7
10.90	11.625	1976.4
12.16	12.75	2103.1
14.37	14.75	2325.0
16.69	16.75	2588.4
17.79	17.75	2669.6
18.94	18.75	2784.8
21.23	20.75	3015.2

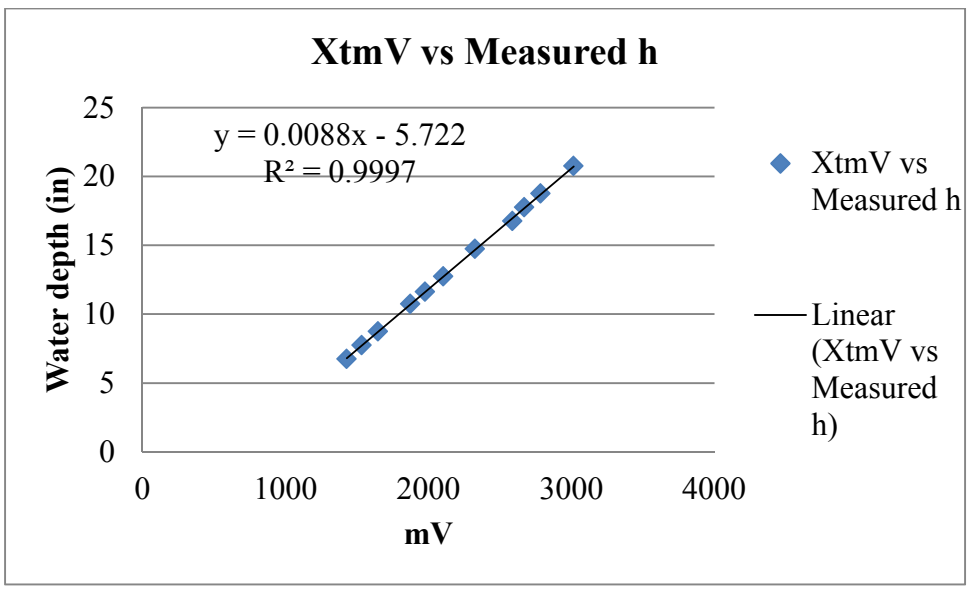

Figure B.1: Results of variable head test for XT-1000 on September 13, 2013. Actual head is plotted against the voltage reading from the XT-1000 to determine the best fit equation.

Table B.2: Results from variable head test for XT-1000 on September 17, 2013

Height Reading from XT-1000 (in)	Actual height measured (in)	mV from XT-1000
2.742	3.938	1155.8
3.692	4.875	1251.3
4.593	5.875	1341.9
6.087	6.875	1491.2
7.178	7.875	1601.9
8.322	8.875	1717.5
9.490	9.875	1834.2
10.592	10.875	1945.2
11.740	11.875	2060.4
12.910	12.938	2178.3
14.020	13.875	2290.1
15.200	14.938	2408.5
16.310	15.875	2520.4
17.390	16.875	2628.9
18.605	17.875	2751.2
19.720	18.875	2863.2
20.832	19.875	2975.1
22.033	20.875	3096.0

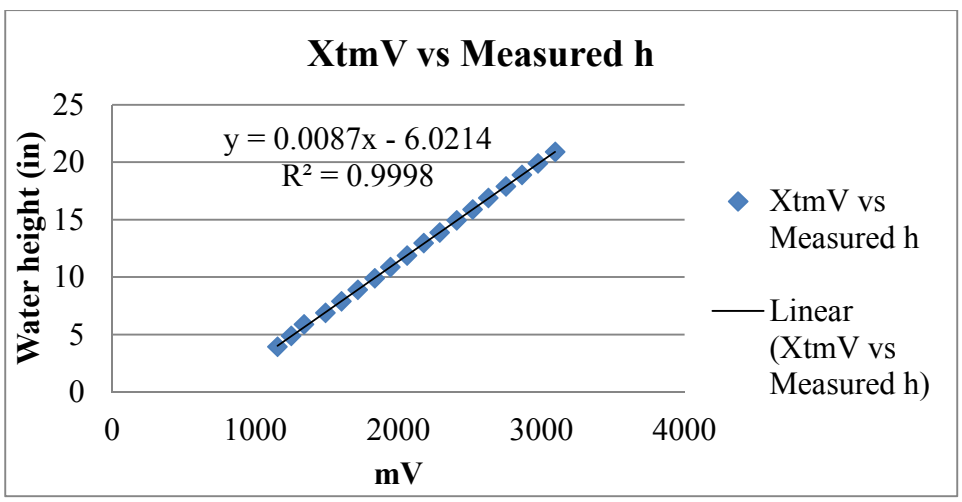

Figure B.2: Results of variable head test for XT-1000 on September 17, 2013. Actual head is plotted against the voltage reading from the XT-1000 to determine the best fit equation.

Table B.3: Results from variable head test for XT-1000 on September 18, 2013

Height Reading from XT-1000 (in)	Actual height measured (in)	mV from XT-1000
0.013	1.500	881.3
2.064	3.375	1087.5
3.228	4.375	1204.6
4.335	5.375	1315.9
5.175	6.250	1400.4
6.502	7.250	1533.9
7.611	8.250	1645.7
8.942	9.375	1779.9
10.030	10.375	1888.8
11.150	11.375	2001.7
12.280	12.375	2115.0
13.440	13.375	2231.5
14.580	14.375	2346.2
15.720	15.375	2461.1
16.850	16.375	2574.5
18.000	17.375	2690.2
19.110	18.375	2801.9
20.230	19.375	2914.7
20.8	19.875	2971.7
21.96	20.875	3089.0

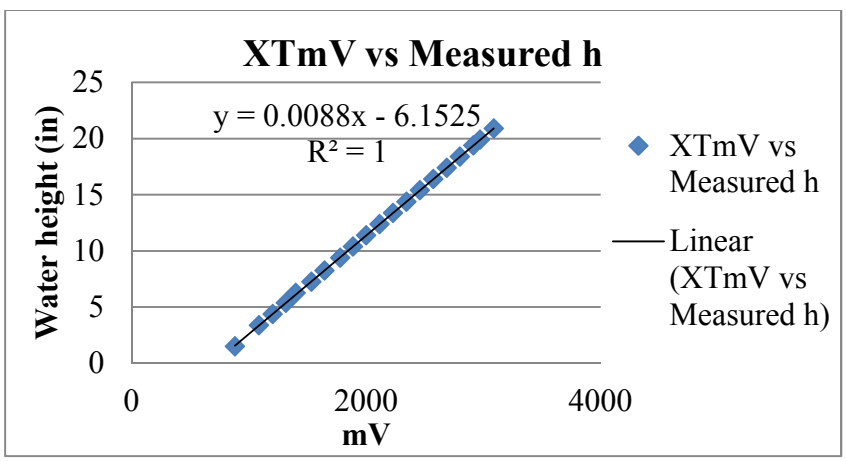

Figure B.3: Results of variable head test for XT-1000 on September 18, 2013. Actual head is plotted against the voltage reading from the XT-1000 to determine the best fit equation.

Table B.4: Results from variable head test for XT-1000 on September 19, 2013

Height Reading from XT-1000 (in)	Actual height measured (in)	mV from XT-1000
0.985	2.375	979.0
2.647	3.875	1146.2
3.775	4.875	1259.6
4.929	5.875	1375.8
6.032	6.875	1486.5
7.215	7.875	1605.6
8.330	8.875	1717.8
9.498	9.875	1835.1
10.620	10.875	1948.1
11.680	11.875	2054.7
12.880	12.875	2175.4
14.020	13.875	2290.0
15.158	14.875	2404.5
16.280	15.875	2517.3
17.382	16.875	2628.2
18.500	17.875	2740.5
19.640	18.875	2855.6
20.740	19.875	2965.9
21.91	20.875	3083.7

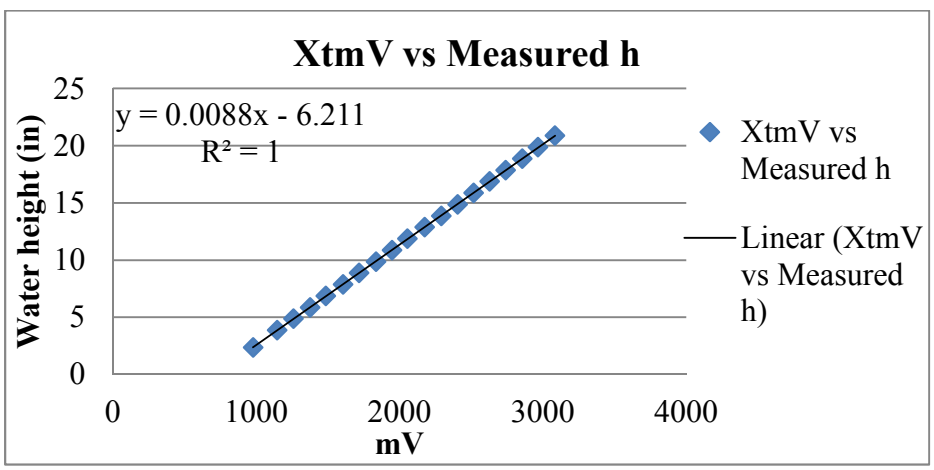

Figure B.4: Results of variable head test for XT-1000 on September 19, 2013. Actual head is plotted against the voltage reading from the XT-1000 to determine the best fit equation.

Table B.5: Results from variable head test for XT-1000 on September 20, 2013

Height Reading from XT-1000 (in)	Actual height measured (in)	mV from XT-1000
0.011	0.000	868.0
1.040	2.500	980.0
4.552	5.750	1361.0
7.645	8.750	1696.7
10.940	11.875	2054.2
14.060	14.875	2393.0
17.210	17.875	2735.0
20.390	20.875	3081.0
31.900	31.875	4329.8

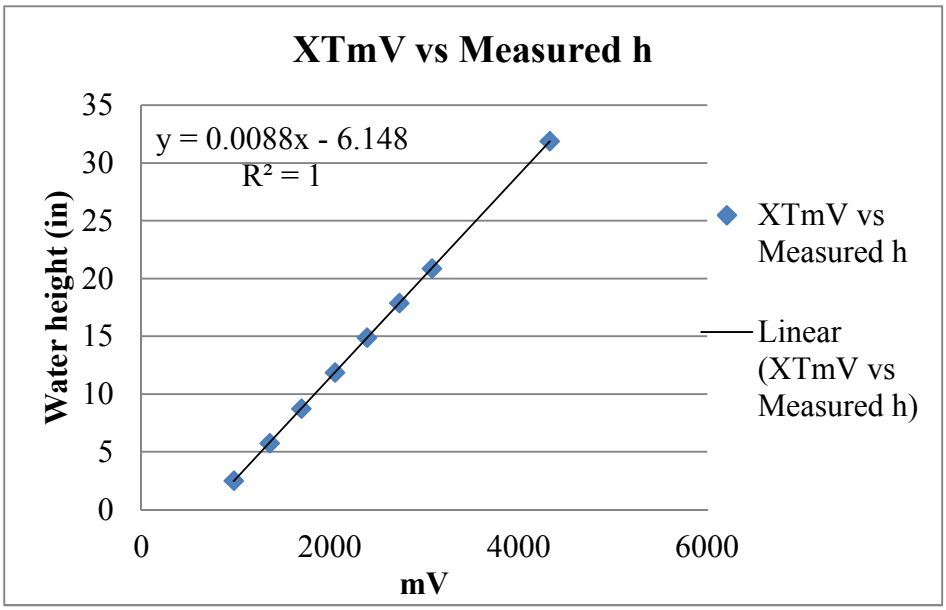

Figure B.5: Results of variable head test for XT-1000 on September 20, 2013. Actual head is plotted against the voltage reading from the XT-1000 to determine the best fit equation.

Appendix C: Average Monthly Infiltration Rates for the IT at the Treatment Train

Table C.1: Average Monthly Infiltration Rates for the IT at the Treatment Train

Month	Average Monthly Infiltration Rate (cm/h)			
	0.9-1.3 m	0.6-0.9 m	0.3-0.6 m	0-0.3 m
Jul-12	3.63	3.59	3.60	1.82
Aug-12	2.63	2.92	2.91	1.66
Sep-12	2.49	2.77	2.86	1.61
Oct-12	1.64	1.24	1.84	1.07
Nov-12	N/A	0.47	1.09	0.86
Dec-12	1.80	1.93	1.56	1.00
Jan-13	1.93	2.83	2.76	1.58
Feb-13	1.09	1.29	1.63	1.05
Mar-13	2.08	2.54	2.08	1.15
Apr-13	1.74	3.03	2.15	0.83
May-13	N/A	6.94	3.73	1.53
Jun-13	N/A	6.16	10.79	7.14
Jul-13	N/A	N/A	12.37	13.35
Aug-13	8.15	16.43	16.79	8.61
Sep-13	N/A	N/A	N/A	7.41
Oct-13	N/A	N/A	8.71	2.42
Nov-13	4.54	13.52	10.27	3.00
Dec-13	6.25	13.38	3.42	1.42
Jan-14	N/A	N/A	3.21	0.52
Feb-14	N/A	5.55	1.53	0.65
Mar-14	N/A	6.04	1.23	0.43
Apr-14	N/A	2.89	0.87	0.27
May-14	6.53	12.00	2.72	1.28
Jun-14	N/A	7.25	14.48	3.17
Jul-14	6.86	15.00	15.38	2.93
Aug-14	N/A	N/A	6.87	2.23
Sep-14	8.90	17.69	9.75	2.34
Oct-14	10.48	11.70	3.46	1.42
Nov-14	6.34	9.08	2.24	1.23
Dec-14	3.89	2.39	1.58	0.41
Jan-15	N/A	2.28	0.97	0.34
Feb-15	4.91	2.03	0.63	0.51
Mar-15	6.66	1.65	0.57	0.21

Appendix D: Initial Flow Measurement System Design Example