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EXAMINATION OF THE EFFECTIVENESS OF BIORETENTION CELLS AND POROUS PAVING PRACTICES IN AIKEN, SC

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EXAMINATION OF THE EFFECTIVENESS OF BIORETENTION CELLS AND POROUS PAVING PRACTICES IN AIKEN, SC

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ABSTRACT

This work seeks to quantify the impact and effectiveness of green infrastructure practices, specifically bioretention cells and porous asphalts, for the reduction of peak flow and volume of stormwater that discharges into the headwaters of the Sand River watershed in Aiken, SC. Stormwater runoff flows and volumes were monitored in the upper Sand River watershed that includes the urban Aiken area, along with two nested subwatersheds, prior to, during, and after the construction of the bioretention cells and porous asphalt sites. Flow data from these monitoring stations were analyzed and the data suggested that there was no significant reduction in volume of stormwater exiting the Sand River watershed. However, there was a significant reduction in the volume of stormwater exiting the subwatershed with a bioretention cell under wet conditions, and there was also a significant reduction in the volume of runoff for the control subwatershed under dry and wet conditions. Selected bioretention cells and porous asphalt sites were monitored to determine their as-built performance compared to their designed performance. One bioretention cell located along Park Avenue between Chesterfield Street and Newberry Street (PCN) was extensively monitored and analyzed. All of the monitored bioretention cells and porous asphalt sites functioned as designed although the data suggested that the bioretention cells were slightly over-designed. The porous asphalt sites were effective at capturing localized surface runoff and either infiltrating it back into the native subsoil or routing it into the bioretention cells. STELLA® modeling software was successfully used to model and characterize the water budget and hydraulic performance for two bioretention cells. Based on the results of this

study, while the green infrastructure retrofits investigated did function to reduce stormwater peak flow and volume, the limited size and area of the retrofitted practices did not significantly impact the peak flow and volume exiting the entire watershed. However, further construction will likely have a more significant impact, because the as-built stormwater control measures are functioning as designed.

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CHAPTER 1

INTRODUCTION

Site Description

The City of Aiken (*33.560°N, -81.719°W*) is located in the southwest corner of South Carolina near the South Carolina-Georgia border. Residential and commercial properties comprise the bulk of the 1220 acre watershed. Continuous development has reduced the amount of natural vegetation and soil available to infiltrate rain water and this development has increased the percentage of impervious surface within the watershed. As a result, stormwater runoff peak flow and volumes being discharged from the watershed have increased dramatically (Meadows et. al, 1992; Woolpert, 2003). The majority of the stormwater runoff from the downtown area is consolidated via underground conveyances to a single 10-foot pipe that forms the headwaters of the Sand River, Hydrologic Unit Code 030601060203. There is an elevation drop of 200 feet across the watershed with an average slope of 3%. As its name implies, the banks and stream bed of the Sand River are composed of mainly sand, and due to the large volume of water being discharged it is being eroded significantly (Julian and Torres, 2006). Additionally, a portion of the Sand River directly downstream from the stormwater outfall is a 303(d) impaired water body due to exceedance of the fecal coliform water quality standard (SCDHEC, 2010). In order to reduce the peak flow and volume of stormwater reaching the Sand River headwaters, the City of Aiken retrofitted existing parkways in the downtown area with several green infrastructure practices, including bioretention cells, porous asphalt, and a small underground cistern.

Project Description

Due to their location and design, the wide parkway medians in Aiken were well suited for stormwater practice retrofit using bioretention technology (Eidson et. al, 2010). The parkways make up the majority of the remaining vegetated pervious area that exists in the watershed (Woolpert, 2003). Bioretention cells are typically designed to capture the first flush of stormwater runoff from smaller, more frequent storms (Davis et. al, 2009). These smaller, more frequent storms typically cause the most damage at the 10-foot pipe outfall (Woolpert, 2003). Within the City of Aiken there are 105-acres of parkway medians, and in the initial phase of the Sand River Headwaters Green Infrastructure project 4.76-acres were converted to bioretention cells. In addition to parkway hydrologic modifications for bioretention cell function, the parallel parking areas adjacent to selected parkways were converted to porous asphalt. The porous paving materials, along with the bioretention cells, were expected to reduce peak flows and stormwater runoff quantity by functioning with the bioretention cells as a treatment train (Balades, 1995). In the initial phase of the project, there were a total of 6.24-acres of porous asphalts, porous concrete, and permeable pavers installed. The effectiveness of the bioretention cells were quantified using various instruments and analyses that will measure inflow, outflow, soil moisture content, and storage. These instruments were integrated into the Intelligent River® monitoring system.

Prior to the installation of the bioretention cells and porous asphalt, the existing stormwater system routed runoff from impervious surfaces underground into a network of pipes that eventually discharged into the Sand River headwaters. This practice has been effective in removing excess stormwater efficiently from the urban downtown area, but this rapid and intense stormwater conveyance with high energy at the point of discharge has resulted in significant erosion at the outfall. Gabion baskets and other energy dissipating devices were placed at the outlet of the 10-foot pipe, but erosion continued after these installations. The objective of Aiken Green Infrastructure project was to treat the problem at the source through the reduction of stormwater volumes and peak flow rates and enhanced infiltration within the upper areas of the urban watershed (Eidson et al., 2010). The parkways are a beneficial area to encourage infiltration because they already infiltrate direct rainfall. Through modification and improvement of the existing parkways, the volume of water infiltrated can be increased significantly and the parkways can become much more effective at capturing localized surface runoff.

Watershed Monitoring

The Sand River headwaters watershed has been modeled extensively (Meadows, et. al. 1992; Woolpert 2003), but there has been little monitoring of the actual volume of water entering the Sand River Headwaters via the 10-foot pipe storm sewer outfall. In order to more accurately quantify the amount of water entering the Sand River, this project installed several flow monitoring devices at locations throughout the watershed. The primary location for flow quantification was the 10-foot pipe outfall within

Hitchcock Woods. The flow at this outfall has been modeled extensively, and there were predicted flows in excess of 1000 cubic feet per second (cfs) for a 2-yr return period storm event (Woolpert 2003). Although modeling programs can be relatively accurate, much of the input information was either based on data that was as much as 40 years old, or simply estimated based on the best information available (Meadows et. al, 1992; Woolpert, 2003). As a result, the post-project hydrology would likely be better quantified using actual flow data from the monitoring equipment.

In addition to the monitoring done at the 10-foot pipe, there were two other monitoring devices placed in subwatersheds contributing to the Sand River headwaters. These monitoring stations were placed at the intersection of Hoods Lane and Newberry Street and South Boundary Avenue and Sumter Street. Quantification of the individual subwatershed contribution to overall watershed discharge both before and after the project will aid in determining the effectiveness of the bioretention cells.

Bioretention Cell Monitoring

Bioretention cells are areas constructed to temporarily retain and treat urban runoff (Davis et. al, 2009). There are three primary pathways for water to enter a bioretention cell: (1) rainfall interception, (2) surface runoff, and/or (3) storm sewer inlets. Generally, the most significant volume of water enters the bioretention cells via the storm sewer inlet from the existing storm sewer network, but this is dependent on the design and application of the bioretention cell. Water exits the cell in one or a combination of multiple ways: (1) infiltration into the native subsoil, (2) evapotranspiration, (3) via an overflow outlet, (4) and/or underdrain in the bioretention cell that connects with the existing storm sewer network. This project is investigating and quantifying the flow routes within the bioretention cell to quantify the effectiveness of bioretention cells for reducing the volume of stormwater being discharged from the watershed.

Porous Asphalt

Porous asphalt can be an integral part of a green infrastructure treatment train (Balades, 1995). It primarily intercepts surface runoff from streets and direct rainfall falling on the asphalt, and quickly infiltrates it into a sub-base and then back into the native soil. The porous asphalt in the Aiken Green Infrastructure project is located adjacent to many of the bioretention cells to form a treatment train. In some project designs, stormwater from the porous asphalt is routed from the sub-base material into the bioretention cell. Water levels within the sub-base were monitored via level loggers (pressure transducers), and the data from these level loggers were used to determine the volume of water being captured, stored, and infiltrated for given storm events.

Project Objectives

In order to determine the effectiveness of the green infrastructure practices installed in the City of Aiken, this work seeks to:

1. Characterize the Sand River watershed by:

a. Analyzing peak flow and volume data at selected locations for all storm events greater than 0.1 inches,

b. Developing runoff coefficients for the Sand River headwaters watershed, Hoods Lane (treatment) subwatershed, and Sumter Street (reference) subwatershed before and after bioretention cell installation,

2. Define, analyze, and quantify the impact, if any, of the bioretention cell construction on the volume of stormwater being discharged from the City of Aiken,
3. Characterize the unit functions occurring within the bioretention cells and porous asphalt sites and develop a water budget for these systems in order to better understand the small scale effectiveness of these practices,
4. Build, calibrate, and validate a model representing bioretention cell hydraulics and water budgets using available design parameters from the as-built bioretention cells as well as hydrologic monitoring data.

The results from this work will better quantify the impact of the green infrastructure retrofits on peak flow and volume reduction of stormwater and the enhancement of an urbanized watershed. Quantification of the impacts of these practices will aid in the future design and construction of green infrastructure practices as they become more widely accepted.

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CHAPTER 2

BACKGROUND AND LITERATURE REVIEW

Site Description

Aiken, SC (*33.560°N, -81.719°W*) has experienced problems with stormwater runoff because decades of expansion of the storm sewer system with the primary objective being conveying the runoff from more urbanized areas into less urbanized areas (Woolpert, 2003). As the city has grown, several problems have arisen concerning the volume, quality, and peak flow of stormwater runoff originating in the urbanized downtown watersheds. There have been multiple studies of the Sand River headwaters watershed (Meadows, 1992; Woolpert, 1994, 1995, 1998, 1999, 2001, 2003). The conclusion of much of this research is that large volumes of highly concentrated stormwater runoff from the downtown area of Aiken are causing significant ecological damage at the stormwater outfall which forms the Sand River headwaters (Woolpert, 2003; Eidson et. al, 2010). This is due to many factors, the most significant of which is the modification of the natural watershed to mostly impervious area (Meadows, 1992; Woolpert, 2003). The majority of area in the Sand River Headwaters watershed is in a developed land use (i.e. commercial, industrial, residential, right-of-way, etc.). The native soil surrounding the Sand River headwaters is comprised mainly of sand, which is highly susceptible to erosion by concentrated flows (Meadows et. al, 1992). Due to the large volume of water exiting the 10-foot pipe outfall at the headwaters, significant damage is being done by even small storms (Woolpert, 2003). Sharp peaks in flow rate are the most damaging to the river channel(Woolpert, 2003). Therefore, capturing and storing runoff from these events would likely have an impact on reducing the amount of damage being done to the Sand River headwaters. Conventional solutions to large peak runoff rates and volumes are not possible in the Sand River headwaters watershed because of extensive development. There is not enough contiguous space anywhere in the watershed available to construct a detention basin large enough to have a significant impact on the volume of water exiting the stormwater outfall (Woolpert, 2003). Other proposed solutions included:

- Installation of a gabion wall along the areas of the Sand River bank susceptible to erosion
- Extend outfall piping further downstream to an area with larger floodplains that would be less susceptible to erosion
- Further vegetative and structural stream bank stabilization
- Construction of Newbury riffles, wrapped face walls, gabion check dams, and j-weirs
- Construction of an earthen dam and lined reservoir on the Sand River

These solutions are costly, and many are not logistically feasible given the limitations on land use in the area.

Aside from the quantity of water entering the Sand River, another major concern was the quality discharged water. Urban runoff can contain many pollutants including metals, nutrients, and suspended sediment (Hunt et al., 2006). At the Sand River headwaters, historically, there is no evidence of impairment by any pollutant other than suspended sediment (Woolpert, 2003). However, a 303(d) impaired water body has been designated downstream of the urban discharge in the Sand River (SCDHEC, 2010). Presence of a high concentration of suspended solids (TSS) can be attributed to many factors. The primary source of the suspended sediments is the high intensity flow eroding and transporting the soils that constitute the bank of the Sand River (Julian and Torres, 2005). Although, the primary constituent of the bank is sand, there are silt-clay contents ranging from 2.4% to 17.5% (Julian and Torres, 2005). This amount of silt and clay cause the banks to become cohesive when exposed to flowing water and greatly reduce the amount of shear stress they can resist without being eroded (Julian and Torres, 2005). Suspended sediment is conventionally controlled using a sediment basin or detention pond (Haan et. al, 1994). However, as previously mentioned, there is insufficient land area for a practice large enough to make a significant impact at the headwaters.

Due to size limits and the scope of the problem, the most practical solution to the problems encountered at the stormwater outfall is a series of stormwater control measures (SCM's) or some combination of SCM's (Woolpert, 2003). The least expensive practices that could be employed to help reduce contaminant loading from within the watershed include (Woolpert, 2003):

- Public Education
- Land Use Planning
- Good Housekeeping Practices
- Use of Alternative Products

- Storm Drainage Signs
- Spill and Dumping Control
- Street Sweeping
- Storm System Maintenance
- Herbicide, Pesticide, and Fertilizer Control
- Illicit Discharge Detection and Elimination

Many of these practices are a practical means to reduce pollutant loadings, but they do little to reduce the volume of water exiting the watershed. Two of the most important problems to control on the banks of the Sand River headwaters watershed are the quantity of water exiting the watershed and the amount of erosion occurring within the Sand River. Sediment particles can act as sinks for contaminants under certain conditions, therefore controlling sediments within the watershed as well as preventing erosion will reduce pollutant loadings within the Sand River (Woolpert, 2003; Larrose et. al, 2010). The Woolpert study suggests that practices that require little space, reduce quantity of stormwater, and improve the quality of stormwater would be most practical for use within the Sand River headwaters watershed. Some of the appropriate practices would be (Woolpert, 2003):

- Vegetated Swales and Buffer Strips
- Water Quality Ponds

- Wetland Filters

- Sand Filters, Sediment Traps, and In-Line Devices

- Construction Site Controls

Many of these structures are very effective at attenuating stormwater runoff, however some (e.g. Water Quality Ponds and Wetland Filters) require contiguous space that is not readily available within the watershed. Other practices, such as construction site controls, would have only limited impact due to the fact that there is little ongoing construction within the watershed, and the potential for significant construction in the future is limited due to the amount of development already present. The large amount of development already present in the area limits the feasibility of any conventional solution to solving the problem with the volume of water exiting the watershed. A combination of SMC's that reduce the "connectivity" of the watershed and promote infiltration of stormwater would be most effective in reducing the volume and peak flow of water entering the Sand River.

Urban Hydrology

Development within an urban area can have a significant impact on the hydrological response of stormwater runoff. Increases in roadways, commercial and residential buildings, and parking areas are all consistent with urban development (Benedict and McMahon, 2006). This infrastructure has been shown to greatly increase the volume and peak flow rate of runoff because they reduce the availability permeable area for runoff to infiltrate as it occurs in a natural system (Chow, 1988; Booth and Jackson, 1997; Hunt, 1999). As the amount of impervious space within a watershed increases, the speed that stormwater enters and exits the system increases. This results in short, high-intensity peak flows that can cause erosion and flooding in downstream areas of the watershed (Chow, 1988). Other problems that may develop as a result of increased impervious area are increased stormwater volume entering downstream watersheds and reduced baseflow in natural streams due to lack of groundwater recharge (Hunt, 1999).

Generally, urbanization changes the shape, form, and function of natural channels within a watershed (Hunt, 1999). This change is common in the design of gutters, drains, and storm sewers to convey water efficiently to a downstream location (Booth and Jackson, 1997). Reducing the natural flow attenuation of a stream by straightening, deepening, or lining with concrete can significantly increase the effect of stormwater runoff at the outfall of the system. Although such channel modifications may reduce sedimentation and erosion within the modified channel, they may also increase flow velocities and erosion rates at the outfall (Julian and Torres, 2005). The negative effects of channel modification on the system at large are very significant (Julian and Torres, 2006).

Runoff is one of the most significant hydrological processes within an urbanized watershed (Chow, 1988). Within a natural hydrologic system runoff can be affected by many processes. Some of these processes are: ponding, infiltration, evapotranspiration, runoff, and interception. When a natural system becomes developed the effects of infiltration, evapotranspiration, and interception all can be significantly reduced

(Grimmond and Oke, 1991; Rodriguez et. al, 2006). Infiltration is essentially eliminated in a fully urbanized area because of the proliferation of impervious areas such as rooftops, roadways, and parking areas (Chow, 1988). Interception in greatly reduced because native vegetation is cleared and removed. And, evapotranspiration is affected because of the removal of vegetation (Grimmond and Oke, 1991). Studies have shown that evapotranspiration can play a significant role in the urban hydrology of some systems (Grimmond and Oke, 1991; Rodriguez et. al, 2006), but these were modeling based approaches that did extend their research to multiple locations.

Impervious area severely limits infiltration, and it increases the amount of runoff from any given site (Chow, 1988). However, impervious area that is not directly connected to a storm sewer system has a lesser effect on the hydrology of an urban system (Lee and Heaney, 2004). Directly Connected Impervious Area (DCIA) has been shown to contribute the majority of runoff from urban areas (Lee and Heaney, 2004). Connecting gutters and drains to storm sewers increases the speed and efficiency of stormwater transport (Lee and Heaney, 2004).

There are several limitations to calculating impervious area used in runoff models such as the Rational Method. Two of these limitations are that impervious area may not be directly connected to the storm sewer system and highly compacted pervious surfaces are exempted (Booth and Jackson, 1997). Runoff originating from areas not directly connected to the storm sewer system will take longer to travel to the outfall and will not contribute as much to peak flow. Compacted pervious surfaces have been shown to contribute a significant amount of runoff due to their limited infiltration capacity (Booth and Jackson, 1997; Shuster et. al, 2008). The main constraint on defining DCIA is that it is difficult to distinguish from total impervious due to limitations on remote sensing and accurate storm sewer mapping (Lee and Heaney, 2004).

Green Infrastructure

Urban areas are highly dependent on various types of infrastructure. Traffic control, power distribution, and storm sewer systems are all common types of infrastructure in any urban development. An important form of infrastructure that many cities are currently exploring is green infrastructure (Benedict and McMahon, 2006). Green infrastructure is defined as, "an interconnected network of green space that conserves natural ecosystem values and functions and provides associated benefits to human populations" (Benedict and McMahon, 2006). Most infrastructure can accomplish a task efficiently without regard to sustainability or environmental consequences. However, green infrastructure takes into account natural processes and the effect of infrastructure on the surrounding environment (Benedict and McMahon, 2006). Fragmented urban development and "urban sprawl" have significantly affected natural landscapes because of the explosive growth of these areas in comparison with the change in population for the same areas (Benedict and McMahon, 2006). Decreases in forested and agricultural land have led to habitat fragmentation and loss of natural processes, such as infiltration (Benedict and McMahon, 2006). Many natural areas mitigate events such as floods by providing an area for excess water to be stored. They also provide space for vegetative growth that facilitates sediment removal and nutrient uptake (Chow, 1988).

Pre-development conditions in most areas typically retain excess stormwater locally and store pollutants on site (Dietz, 2007). This pre-development retention reduces the transport and accumulation of sediments, nutrients, and pollutants downstream (Dietz, 2007). Green infrastructure mimics this pre-development hydrology by promoting in-situ stormwater retention and infiltration (Dietz, 2007). One review of two current green infrastructure practices, green roofs and grassed swales, showed that phosphorous is a common problem (Dietz, 2007). However, phosphorous export is generally attributed to excess levels in the soil media used and improper fertilization practices (Dietz, 2007).

An important function of infiltration practices, such as bioretention cells, is to retain the first inch of runoff from a site because majority of the sediment in runoff is contained in the first inch of runoff from a site (Dietz, 2007). The storage and infiltration of stormwater removes pollutants that would otherwise be transported downstream (Hunt et. al, 2006).

An important aspect of a successful implementation of any green infrastructure project is the treatment train (Balades, 1995). Many green infrastructure practices are very effective as isolated practices. However, once they are linked as a part of a larger system, their overall effectiveness increases drastically. One project evaluating two similar housing developments was studied to determine the effectiveness of green infrastructure practices (Dietz, 2007). One development was constructed using green infrastructure practices such as grassed swales, bioretention areas, and pervious pavements, and the other was constructed using traditional stormwater infrastructure. The development utilizing the green infrastructure practices had no change in stormwater or pollutant export after project completion (Dietz, 2007). But, the development constructed with traditional stormwater infrastructure exhibited increases in stormwater volume and pollutant export (Dietz, 2007). Numerous research studies have demonstrated that green infrastructure is a valuable and effective strategy in reproducing pre-development hydrology (Benedict and McMahon, 2006; Hitchcock et. al, 2010; Hunt et. al, 2006; Dietz, 2007).

Bioretention Cells

Bioretention cells have been widely accepted by many state and local governments as effective stormwater management practices, but there has been relatively little research done to examine performance in certain regions (SCDHEC, 2005; Davis et. al, 2009). There have been pilot studies conducted on different designs of bioretention cells and their effectiveness at reducing the quantity and improving the quality of stormwater runoff (Davis et. al, 2001; Hsieh and Davis, 2005; Hunt et. al, 2006; Hunt et. al 2008, and Davis et. al 2009). Depending on site conditions, bioretention cells have been shown to be effective for reducing peak flows from small to medium sized events (<2-year return period) (Hunt et. al, 2008). Design objectives for any low impact development stormwater control measure (SCM) are similar in that they often are installed in conjunction with other low impact development practices, they must function in smaller areas than conventional SCM’s, and they should improve water quality.

Bioretention cells are often designed to increase groundwater recharge and help maintain watershed base flow, remove surface and groundwater pollutants, protect natural channels, and reduce peak flow (Davis et. al, 2009).

Important hydrologic processes within bioretention cells include infiltration and evapotranspiration (ET) (Davis et. al, 2009). Infiltration of runoff allows groundwater systems to be recharged and helps maintain base flow in natural streams. It also reduces the amount of runoff that is generated from a land area. ET is a process largely dependent on climate, season, vegetation type and density, and geographic location. However, it has been shown that infiltration and ET can account for managing 50-90% of the runoff that can be reduced by a bioretention cell (Heasom et. al, 2006). Infiltration is also an important pollutant removal process. As runoff is infiltrated into the soil media within a bioretention cell, certain pollutants are chemically adsorbed to soil particles and others are chemically altered by in-situ microorganisms (Diblasi et. al, 2009). Other contaminants that were previously adsorbed to soil particles can settle out within the cell. Design specifications for the bioretention soil media (BSM) are very important for ensuring proper infiltration within the cell. Typically, sandy soils are assumed to have higher infiltration rates, but clayey soils are assumed to have higher cation exchange capacities. The amount of pollutant that a soil can remove is related to the cation exchange capacity (Hsieh and Davis, 2005). Therefore, a proper mixture of sandy and clayey soils will allow for adequate infiltration as well as pollutant removal within the cell. The most cost-effective and the simplest design of bioretention soil media to install is a uniform profile soil media with a combined filtration and vegetative layer (Hsieh and Davis, 2005).

Initial laboratory studies of the bioretention cell concept were very successful in mitigating flows and sequestering certain pollutants within the BSM (Davis et. al, 2001). The laboratory studies were used to develop a model for the amount of runoff infiltration as well as the rate of contaminant removal. Since bioretention cells are designed to remove pollutants from runoff, the soil media within the cell has a finite effective lifetime (Hsieh and Davis, 2005; Davis et. al, 2009). The lifetime of the soil media is a function of the depth of the bioretention cell, the amount of rainfall in a given time period, the area of the bioretention cell, the bulk density of the soil media, the runoff volume, and the adsorption coefficient for the contaminant of interest (Davis et. al, 2001). A common and undesired characteristic in many laboratory and field studies of bioretention cells is the export of nitrogen and phosphorous (Davis et. al, 2001; Hunt et. al, 2006; Hunt et. al, 2008). Nutrient exports are most likely due to the complex processes occurring within the cell that transform nitrogen based on many factors including: reduction/oxidation conditions, type of vegetation, and presence of microbes (Hunt et. al, 2006). Also, the original nitrogen content of the soil and fertilization practices may have a significant impact on nitrogen export from the cell. Phosphorous export is generally attributed to high phosphorous levels originating in the BSM and/or over-fertilization (Hunt et. al, 2006).

Many SCM's are initially designed to perform under certain conditions. However, if not properly maintained, the functionality of any SCM is significantly diminished, including that for bioretention cells and porous asphalt infiltration practices. The main physical process that inhibits the full functionality of any infiltration practice is clogging (Fujita, 1997). Clogging can occur for many reasons including sedimentation and accumulation of organic litter. In bioretention cells, infiltration into the in-situ soil media and exfiltration back into the native sub-soil are essential processes that must not be impeded in order for the cell to function properly (Hunt et. al, 2006). In order to maintain the designed infiltration rates, the mulch within the cell must be periodically removed and replaced (Hunt et. al, 2009). More common maintenance practices include aesthetic practices such as mowing, pruning, and removing trash (Hunt et. al, 2009). If water quality performance is diminished within the cell, then the top 2.5-5 cm of mulch should be removed and replaced because this is where majority of the pollutants tend to accumulate (Li and Davis, 2008). More research is required to better evaluate the type and frequency of common bioretention cell maintenance practices for effectiveness (Hunt, 2009).

Under certain conditions, infiltration practices may have some negative effects. Few human-made designs precisely and accurately mimic natural, pre-development conditions. Concentrating the infiltration of stormwater runoff into a relatively small, confined area has the potential to create groundwater mounding. Groundwater mounding can create a risk for subsurface infrastructure damage (Endreny and Collins, 2009). Subsurface water has been shown to rise substantially based on the spatial arrangement of bioretention cells (Endreny and Collins, 2009). Groundwater mounding seems to become an issue when there is a high concentration of bioretention cells with overlapping groundwater mounds in soils that are marginally suitable for the infiltration of the stormwater (Endreny and Collins, 2009). Other studies have shown that individual bioretention cells do not have a long-term effect on the height of the water table (Machusick et. al, 2011). However, it is conceded that the design of bioretention cells varies greatly depending on the site, and that adequate separation should be maintained between the groundwater elevation and the bottom of the cell (Machusick et. al, 2011).

In addition to groundwater mounding, the contamination of the subsurface water is plausible. Stormwater is a transport mechanism for a wide assortment of pollutants such as: hydrocarbons, zinc, copper, nitrogen, phosphorus, and organo-phosphates (Hunt et. al, 2006; Diblasi et. al, 2008; Li and Davis, 2008). When these pollutants are accumulated in a concentrated area, like in bioretention cell media, there is the potential for transport into the subsurface water system (Fujita, 1997). Bioretention cells, more specifically the media, are designed to become sinks for common pollutants. Maintenance becomes a very important aspect of long-term pollution control. Once the bioretention soil media has reached its maximum removal capacity, it must be replaced in order to prevent pollutant export from the cell (Hunt et. al, 2006). The selection of the soil media is also a very crucial step in the design process because soil media with high levels of nutrients like phosphorous and nitrogen can exacerbate existing pollution problems in both the groundwater and receiving water bodies (Hunt et. al, 2006).

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CHAPTER 3

WATERSHED EVALUATION FOR THE ASSESSMENT OF RETROFITTED GREEN INFRASTRUCTURE PRACTICES: AIKEN, SC

ABSTRACT

The goal of this study was to quantify the hydrologic impact of retrofitting an existing stormwater sewer system with Green Infrastructure practices in reducing the stormwater peak flow and volume exiting the Sand River headwaters watershed. The Sand River, located near Aiken, SC (*33.560°N, -81.719°W)* has been eroded by excessive stormwater being discharged from a portion of the downtown area of the city. Parkways within the downtown area were retrofitted with green infrastructure practices that came online in April 2011 as part of a project to reduce the volume and peak flow of stormwater being discharged from the Sand River watershed. These green infrastructure practices included the construction of bioretention cells and porous asphalts. The objectives of this study were to characterize the Sand River headwaters watershed and developing runoff coefficients prior to- and after the construction of the bioretention cells and porous asphalts, and then use these coefficients to analyze the impact of the green infrastructure practices. The runoff coefficient for the Sand River watershed was not significantly reduced ($p<0.05$) by the construction of the bioretention cells and porous asphalt. One of the subwatersheds, Hoods Lane, showed a significant runoff coefficient reduction under Antecedent Moisture Condition III (wet conditions). Another subwatershed, Sumter Street, had significant runoff coefficient reduction although there

no green infrastructure practices installed within the subwatershed. Based on the monitoring conducted within the watershed, further conversion of existing parkways to bioretention cells would likely have a more significant impact on reducing the peak flow and volume of stormwater being discharged at the Sand River headwaters.

INTRODUCTION

The Sand River is located near Aiken, SC (*33.560°N, -81.719°W*) with headwaters that were formed by a 10-foot diameter storm sewer outfall, and a major portion of the stormwater discharged from the outfall originated in the downtown area of Aiken at the time of this study. The existing stormwater infrastructure in the downtown area has been extensively developed. Dense urban land use and degree of impervious surface area has resulted in very rapid peak flows exiting the watershed during and after rain events. The Sand River headwaters area has been significantly impacted by these high-intensity flows. In some areas, the headwaters have eroded up to 70 feet. This erosion and corresponding incisement has resulted in a loss of ecological function for some areas of Hitchcock Woods, as well as a significant amount of sediment transport downstream.

When an area undergoes development, the hydrological processes of that watershed are typically altered, with increases in peak stormwater runoff flow and volume due to the conversion of native soil to impervious surface cover (Chow, 1988). These stormwater flow and volume increases are often mitigated by retention structures such as ponds (Chow, 1988). However, in fully developed areas, land area is typically limited for conventional stormwater management practices such as retention ponds. For more urban areas, green infrastructure retrofit strategies include a wide array of structures and management practices that seek to restore natural hydrological and ecological function (Benedict and McMahon, 2006). Practices such as bioretention cells and porous asphalt require less space than retention ponds, and they have been shown to be effective at infiltrating stormwater runoff (Davis et. al, 2009).

Numerous studies have been done to examine the quantity of water exiting the Sand River watershed (Meadows, 1992; Woolpert, 1994, 1995, 1997, 1998, 2001, 2003). However, none of these studies have conducted any monitoring to quantify the volume of water or peak flow exiting the watershed. The initial phase of this study included the installation of a flow monitoring device in the 10 foot pipe discharging to the headwaters to quantify the volume of water exiting the watershed for various storm events. The collection of measured flows exiting the downtown watershed for different storm events is very useful for watershed characterization as well as for the determination of SCM's to be deployed in the watershed drainage area. Models can be very helpful when characterizing watersheds with known dimensions, but many parts of the Sand River Headwaters watershed were unknown (Meadows et. al, 1992; Woolpert, 2003). Parts of the existing sewer network are poorly mapped, and several sections are not mapped at all (Meadows et. al, 1992; Woolpert, 2003). This lack of complete information makes accurate modeling through computer programs very difficult.

By quantifying the volume of water exiting the watershed for certain storms, the scope of the stormwater flow and volume reduction by urban retrofit with green infrastructure can be realized. There is very limited space opportunity within the watershed for conventional stormwater management practices (i.e. retention ponds) because the Sand River headwaters watershed is a fully developed commercial and residential area. This limitation increases the need for low impact stormwater management practices to be implemented as extensively as possible. Data from the storm sewer outfalls were collected for many storms prior to and after construction of the downtown bioretention cells and porous asphalts.

Site Description

The downtown Aiken urban area (Figure 3-1) is a completely urbanized watershed that – at the time of this study - drained to a single outfall located in the Hitchcock Woods (HUC 030601060203) and contributed significant discharge at the headwaters of the Sand River (Woolpert, 2003). The area of the entire Sand River watershed is 1220 acres, the area of the Hoods Lane subwatershed is 47 acres, and the area of the Sumter Street subwatershed is 340 acres. The center of the watershed is located at 33.560°N and -81.719°W. Elevation change in the watershed is approximately 200 feet from top to bottom with an average slope of 3%. A portion of the Sand River directly downstream from the stormwater outfall is a 303(d) impaired water body due to excessive levels of fecal coliform (SCDHEC, 2011).

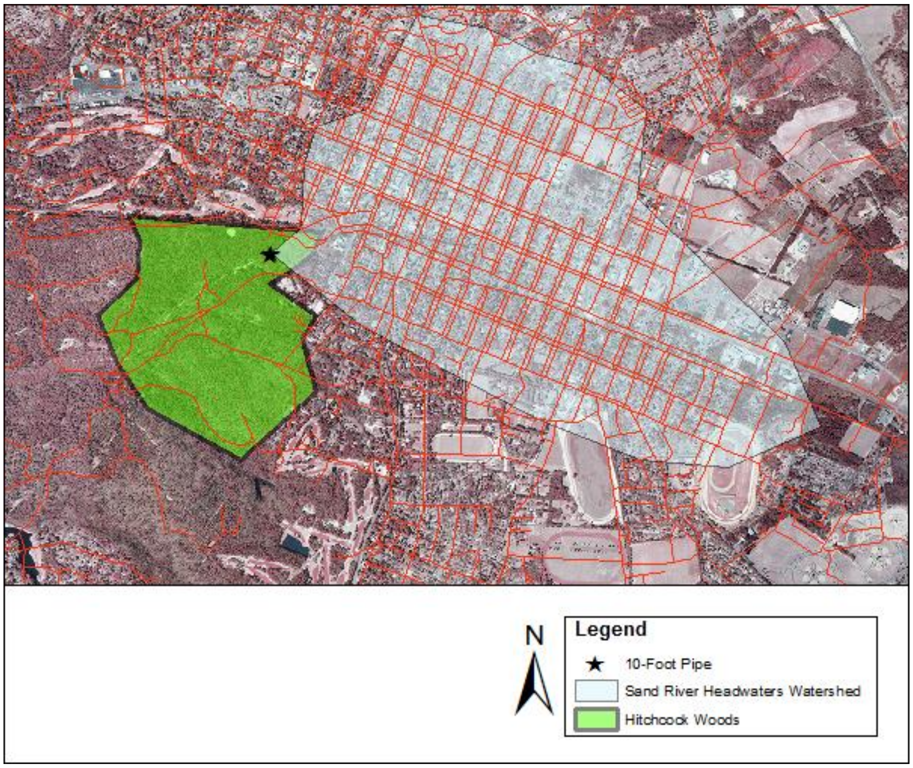

Figure 3-1. Map of downtown Aiken, SC. The street grid system indicates the highly urban area within the Sand River Headwater Watershed (1220 acres).

The area surrounding the stormwater outfall is the Hitchcock Woods (Figure 3-1), a 3.1 mi^2, wooded recreational area adjacent to both the downtown portion of the City of Aiken and many residential neighborhoods. As the stormwater infrastructure in the City has expanded and become more complex, the volume of water discharged from the 10 foot pipe has formed an unstable canyon 70 feet deep (Eidson et al., 2010). In the downtown area of Aiken, there are several green areas or parkways located between the roadways. The parkways are an important feature in the City, both aesthetically and for stormwater management. Historically, stormwater flow was partially attenuated with these green areas. However, many streets were curbed, and the water was routed directly from the street and into the stormwater pipe network. As a part of the Sand River Headwaters Green Infrastructure Project, much of this curbing was modified to allow stormwater from the street to enter the cell. Existing stormwater pipes were also modified so that the stormwater would enter some of the cells and then be discharged back into the same systems after being retained for a period of time. The Sand River Headwaters Green Infrastructure Project is a collaborative effort to reduce peak flows exiting the Sand River Headwaters watershed, and the erosion resulting from these flows.

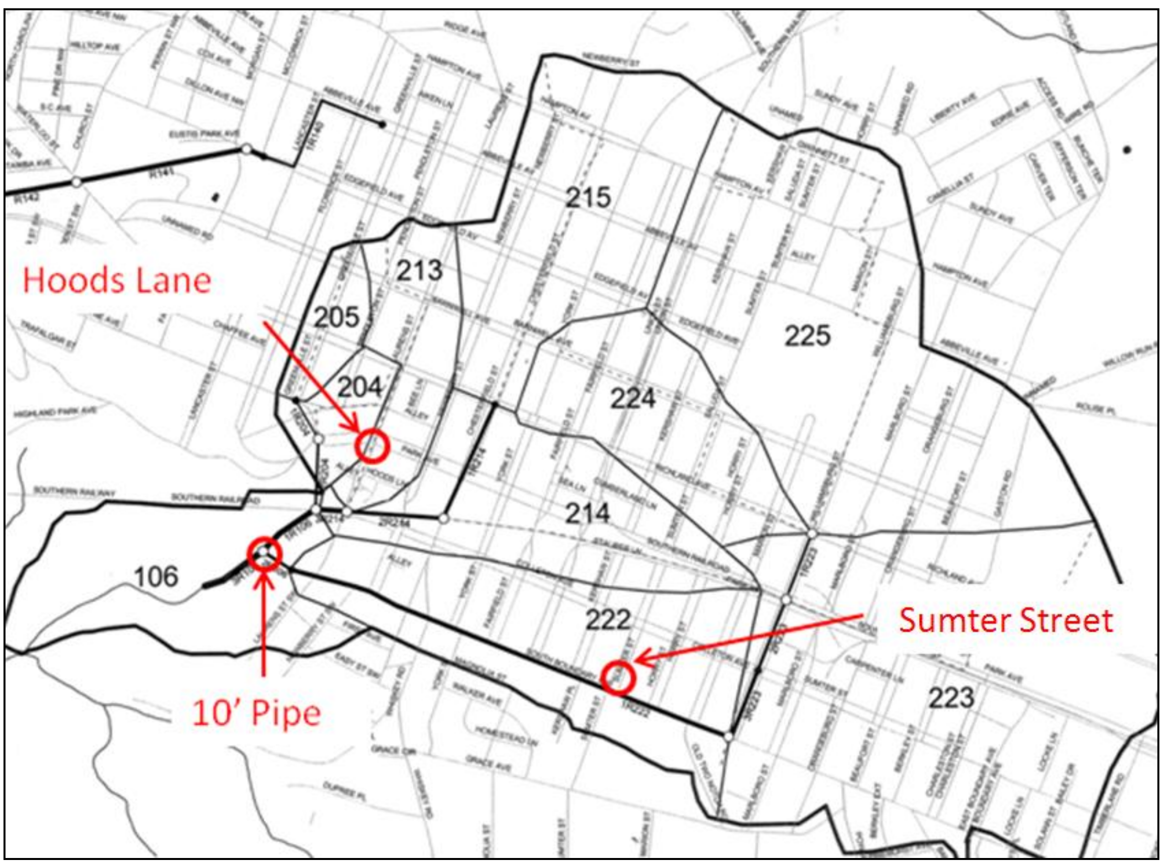

Figure 3-2. Map of the watershed with sampling locations shown, including the Sand River headwaters discharge point (10' pipe), the Hoods Lane (treatment) watershed, and the Sumter (reference) watershed.

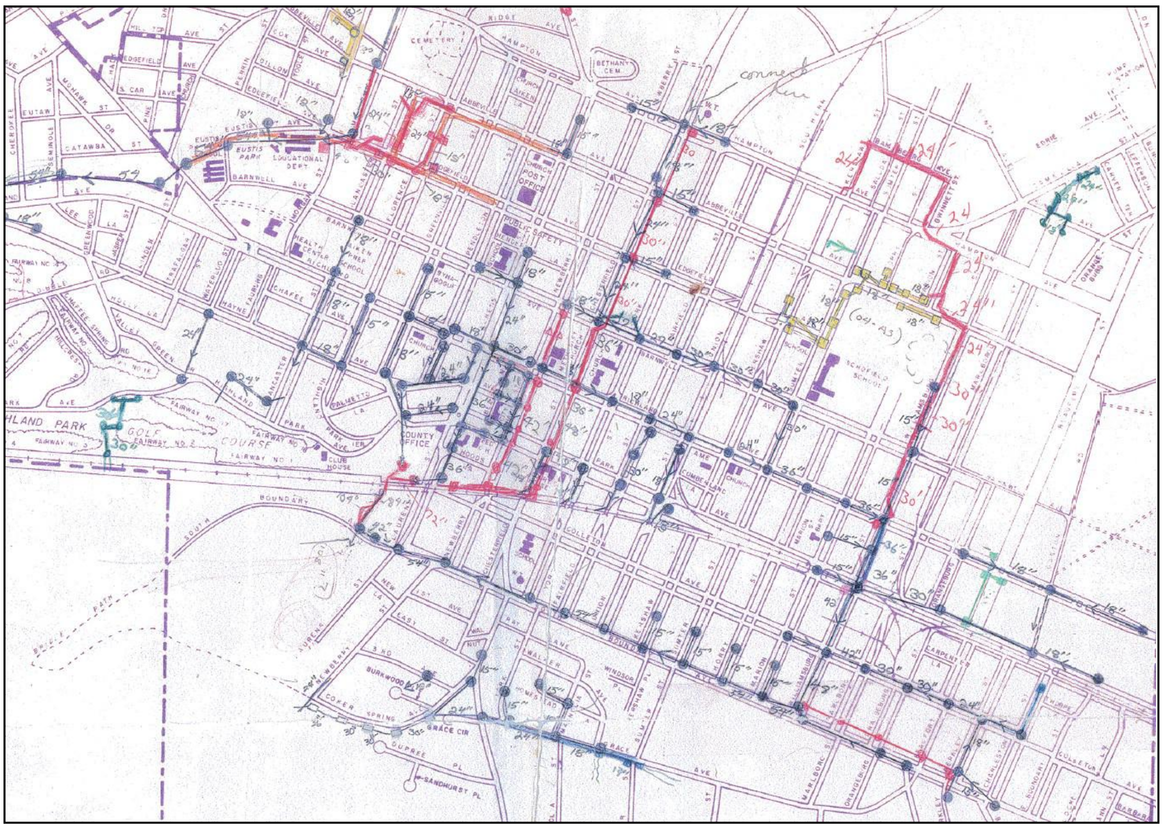

Figure 3-3. Existing stormwater trunk lines in Aiken, SC, developed from a combination of field verification and historical knowledge about installation of and modifications to the stormwater infrastructure. (L. Morris, City of Aiken)

OBJECTIVES

The installation of bioretention cells and porous asphalts is being examined as a potential solution to reducing the large volume of stormwater exiting the Sand River headwaters watershed. In order to determine the effectiveness of the green infrastructure practices installed in the City of Aiken, this research seeks to:

1. Characterize the Sand River headwaters watershed by:
   a. Analyzing peak flow and volume data for all storm events greater than 0.1 inches,
   b. Developing runoff coefficients for the Sand River watershed, Hoods Lane (treatment) watershed, and Sumter Street (reference) watershed before and after bioretention cell construction,
2. Define, analyze, and quantify the impact, if any, of the bioretention cell construction on the volume of stormwater being discharged from the Sand River watershed.

METHODS

Monitoring

Various meteorological and water quantity parameters were monitored within the contributing watershed. Rainfall was measured in order to determine the volume of water entering the watershed during any given storm event. The rainfall data was collected in two locations: in the center of the downtown area (*33.561°N, -81.719°W*) and near the stormwater outfall (*33.555°N, -81.722°W*). The rainfall data near the center of the downtown area was collected using a Campbell Scientific® tipping bucket rain gauge and Campbell Scientific® CR800 Series data logger on 1-minute intervals except for a brief period from April 2012 – June 2012 when the sampling frequency was changed to 10-minute intervals. These two instruments were part of a larger monitoring apparatus that also measured: temperature, relative humidity, barometric pressure, solar radiation, wind speed, and wind direction. Table 3-1 shows equipment used at this monitoring location.

Table 3-1. Equipment Deployed at the Downtown Aiken Monitoring Location.

Description	Manufacturer	Model	Reporting Units
Data Logger	Campbell Scientific® (Logan, UT)	CR800	-
Power Supply	Campbell Scientific® (Logan, UT)	PS100-SW	-
Solar Panel	BP® (London, UK)	SP10	-
Barometer	Setra® (Boxborough, MA)	CS100	kPa
Anemometer	RM Young® (Traverse City, MI)	03002-L13	m/s
Rain Bucket	Texas Electronics® (Lubbock, TX)	TE525-L13	mm
Temp/RH Sensor	Campbell Scientific® (Logan, UT)	CS215-L13	°C/%
Pyranometer	Li-Cor® (Lincoln, NE)	LI200X-L13	W/m^2
PAR sensor	Li-Cor® (Lincoln, NE)	LI190SB-L13	$mmol/m^2$

Collected data from the Campbell Scientific Weather Station (Logan, UT) were transmitted to an online database within the Intelligent River® network, and these data were accessed remotely and downloaded from the Intelligent River® database. Rainfall data at the stormwater outfall were collected using an ISCO® tipping bucket rain gauge that was downloaded in-situ from the ISCO® 6712 unit.

Stormwater flow was monitored in order to calculate the volume of stormwater contributed by two of the subwatersheds, as well as to determine the total volume of stormwater leaving the entire watershed for a given storm event. The stormwater flow as overall watershed and also subcatchment discharge was monitored in three different locations: the Sand River headwaters at the10-foot pipe outfall in Hitchcock Woods (*33.555°N, -81.722°W*), the treatment catchment at Hoods Lane draining Newberry St. from Park Ave. (*33.557°N, -81.722°W*), and the control or reference catchment at the intersection of South Boundary Street and Sumter Street (*33.552°N, -81.715°W*). An ISCO® 6712 unit equipped with an ISCO® 730 Bubbler Module monitored the level at the 10-pipe. At the Hoods Lane and Sumter Street monitoring locations, ISCO® 2150 Area/Velocity units were used to monitor flows. The data collected from the ISCO® units was exported into Microsoft Excel© for further analysis, including the calculation of flow based on the stormwater level and pipe characteristics.

Data Collection

All flow data from the various sensors were collected routinely for analysis. The ISCO® units were accessed using an ISCO® 581 Rapid Transfer Device (RTD). The

RTD was attached to the units and the data stored on the units were transferred temporarily to the RTD. Data from the RTD were then accessed using the Flowlink 5 software, and transferred onto a laptop. Data from the Campbell Scientific® Weather Station were accessed and downloaded from the Campbell Scientific® CR800 series data logger onto a laptop using the Campbell Scientific® PC200W software.

Spatial analysis of the watershed, including area and time of concentration, was taken from a study conducted by Woolpert in 2003, in which the entire watershed and several subwatersheds were delineated. This information is essential to understanding the origin of the runoff being monitored by the ISCO® units at the 10-foot pipe, Hoods Lane, and Sumter Street. Spatial information in this report was gathered using ArcGIS© and data from surveying various locations within the area of interest.

Data Analysis

Raw data collected from the 10-foot pipe had three separate components: rainfall, water level, and flow. Rainfall was reported in inches, level was reported in feet, and flow was reported in cubic feet per second. Flow was calculated by the ISCO® unit based on the characteristics of the pipe. These input characteristics were: diameter, Manning's roughness, slope, and water level. Flow data were then double-checked in an Excel© spreadsheet to make sure that the ISCO® unit was reporting the correct values. Raw data were double-checked using Manning's equation (Manning, 1890):

$$Q = \frac{1.49}{n} R^{2/3} S^{1/2} A \tag{3-1}$$

Where,

n = Manning's Roughness, n = 0.024

R = hydraulic radius, $R = \frac{D}{4}(1 - \frac{sin\Theta}{\Theta})$ [ft]

S = slope, [ft/ft]

A = area, $\frac{D^2}{8}\ (\Theta - sin\Theta)$ [ft^2]

Θ - Based on three flow scenarios: below half-full (0 < y < D/2), half-full (y = D/2), and above half-full (D/2 < y< D), where y is water depth.

Below half-full: $\Theta = 2 * tan^{-1}\left(\frac{\sqrt{\left(\frac{D}{2}\right)^2 - \left(\frac{D}{2} - y\right)^2}}{\frac{D}{2} - y}\right)$

Half-full: $\Theta = \pi$

Above half-full: $\Theta = 2\pi + 2 * tan^{-1}\left(\frac{\sqrt{\left(\frac{D}{2}\right)^2 - \left(\frac{D}{2} - y\right)^2}}{\frac{D}{2} - y}\right)$

The Manning's Roughness value, n, is determined empirically based on the material of the pipe. The 10-foot pipe is constructed from corrugated metal, n = 0.024 (SCS, 1951). Using the flow rate calculated from Manning's equation, a hydrograph was generated for every storm event that had at least 0.5 inches of rainfall. Volume of runoff is defined as the integral of runoff flow over a given time interval (Chow, 1988).

$$V = \int_0^t q\ dt \qquad (3\text{-}2)$$

Where,

V = runoff [ft^3]

q = flow [cfs]

This equation can be approximated by multiplying the flow rate at each sampling interval by the length of the sampling interval and taking the sum of these values over the time of the storm event as described in the following equation:

$$V = \sum Q \, \Delta t \tag{3-3}$$

Where,

V = Volume [ft^3]

Q = Sampled flow rate

Δt = Sampling interval

At the 10-foot pipe, baseflow was generally present due to continuous drainage from the upper reaches of the Sand River headwaters watershed via either groundwater, irrigation, or combinations of these sources. In order to account for the effect of baseflow in the volume of runoff, it must be subtracted from the total volume of water exiting the watershed for any given storm event. Baseflow varies seasonally, and in some cases from storm to storm. Therefore, the baseflow for each storm event was analyzed for 48 hours prior to rainfall, and the average baseflow subtracted from each time step accordingly (Chow, 1988).

$$V_r = \sum[(Q \, \Delta t) - (Q_{baseflow} \, \Delta t)] \tag{3-4}$$

Where,

V_r = runoff volume [ft^3]

Q = Sampled flow rate [cfs]

$Q_{baseflow}$ = Flow rate of baseflow based on the preceding 48 hours [cfs]

Δt = Sampling interval [s]

In addition to the volume of runoff and rainfall depth for each storm, other significant parameters were calculated and recorded such as rainfall intensity and antecedent moisture conditions.

Using the volume of runoff, the volume of rainfall, and the area of the watershed, a runoff coefficient was developed for each storm event. Runoff coefficient determination is a method for comparing the volume of water discharged from a watershed to the volume of rainfall within that watershed. When using a runoff coefficient, it is assumed that there is uniform depth of rainfall across the entire area of the watershed and that the measured volume of runoff is the total runoff for the entire area.

Once all of the storms (> 0.1-in) were analyzed in order to determine rainfall and runoff volume, the data were compiled into a single table in order to generate a relationship between the amount of rainfall and volume of runoff from the watershed. The ratio of the volume of rainfall to the volume of runoff is important to determine the effect of the bioretention cell construction on the volume of runoff exiting the watershed. The relationship between rainfall and runoff is defined by a runoff coefficient, C_r (Chow, 1988).

$$C_r = \frac{V_r}{V_{precip.} * A} \tag{3-5}$$

Where,

V_r = Runoff volume [ac-in]

V_{precip} = Precipitation volume [in]

A = Watershed Area [ac]

This coefficient was calculated for data gathered from the 10-foot pipe both before and after bioretention cell installation.

Statistical analyses were conducted for the two datasets to determine if there were significant differences in the pre-construction and post-construction mean runoff coefficients. Mean runoff coefficients were analyzed using a paired t-test for pre-construction and post-construction conditions and they were blocked by antecedent moisture conditions. The hypotheses are as follows:

H_0: There is no difference between the pre-construction and post-construction mean runoff coefficients

H_1: The mean pre-construction runoff coefficient is less than the post-construction mean runoff coefficient

The null hypothesis for the entire Sand River headwaters watershed and the Hoods Lane watershed is that the post-construction mean runoff coefficient is less than the pre-construction mean runoff coefficient. The null hypothesis is that there is no significant difference between the pre-construction mean runoff coefficient and the pre-construction mean runoff coefficient, with Hoods Lane catchment as the treatment watershed and the Sumter Street catchment as the control watershed. In addition to the t-test, a Pearson correlation test was conducted to determine what independent variables (rainfall, days since last rain, AMC, duration, and intensity) contributed to the dependent variable (runoff coefficient).

Runoff coefficients developed from the data acquired from the samplers at Sumter Street and Hoods Lane were both analyzed for significant differences. The purpose of analyzing these smaller watersheds is to narrow the focus of the analysis because the impact of the bioretention cells may be more easily examined on the scale of a smaller watershed. There were no bioretention cells in the watershed draining to the sensor on Sumter Street, so it was used as a control watershed. There was one cell within the watershed that discharged to the sensor on Hoods Lane. Land use and topography in both the Hoods Lane and Sumter Street subwatersheds are similar to that of the entire watershed.

RESULTS AND DISCUSSION

From January 2010 to January 2012 there were 132 storms greater than 0.1 inches. Due to various technical problems, there were 131 storms analyzed at the 10-foot pipe outfall, 119 storms analyzed at the Hoods Lane monitoring station, and 103 storms were analyzed at the Sumter Street monitoring station. A summary of the storms analyzed at each monitoring location is given in Table 3-2.

Table 3-2. Summary of storms occurring near Aiken, SC from January 2010 to January 2012 (pre-installation/post-installation).

Watershed	n	Mean (in.)	Median (in.)	St. Dev.
10' Pipe	131	0.556 / 0.468	0.390 / 0.295	0.465 / 0.410
Hoods Ln.	119	0.560 / 0.490	0.390 / 0.330	0.465 / 0.425
Sumter St.	104	0.612 / 0.562	0.465 / 0.450	0.505 / 0.463

On April 31, 2011, all bioretention cell construction was completed, and storms after this date are referred to as “post-construction”. Figures 3-4, 3-5, and 3-6 show the cumulative rainfall and the stormwater runoff volume for the entire Sand River watershed and the two subwatersheds, respectively.

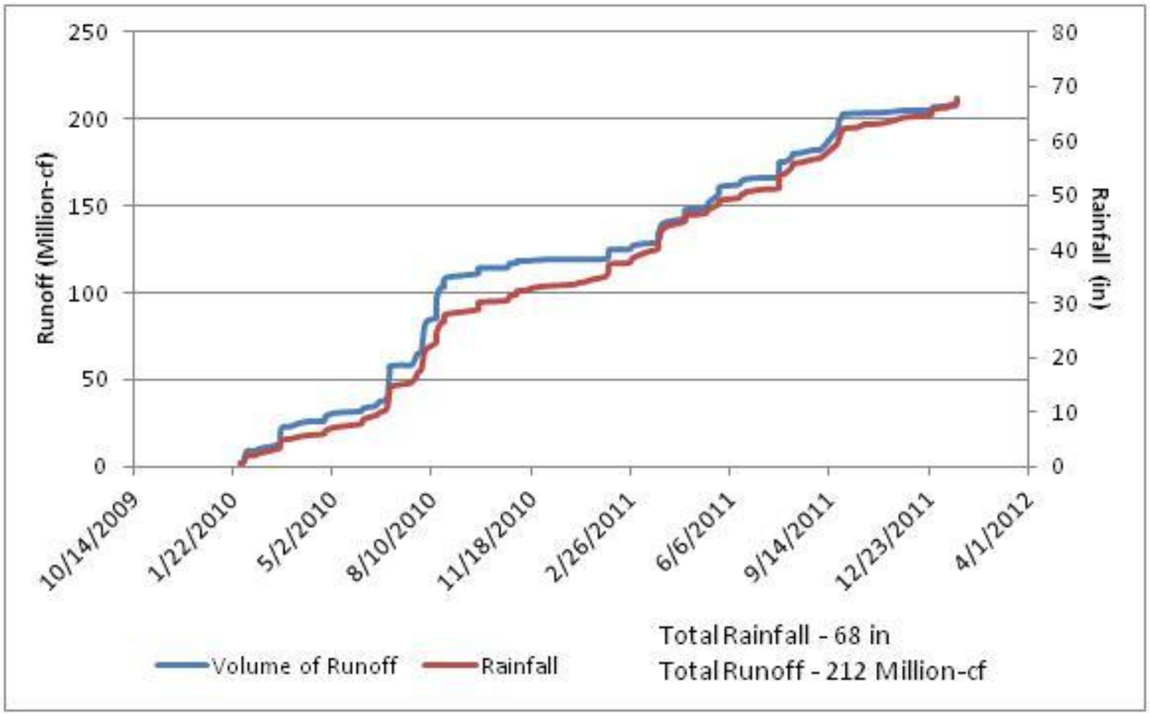

Figure 3-4. Cumulative rainfall and runoff from the entire watershed (Storms larger than 0.1 in., n = 131).

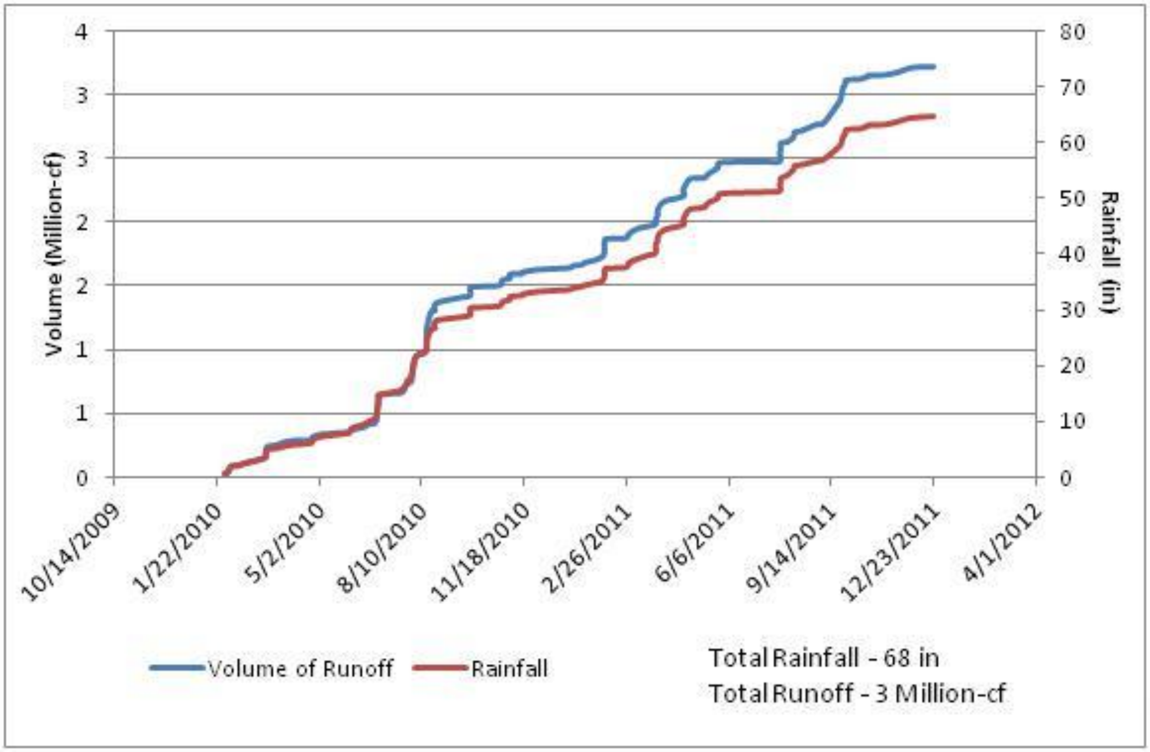

Figure 3-5. Cumulative rainfall and runoff from the Hoods Lane subwatershed (Storms larger than 0.1 in., n = 119).

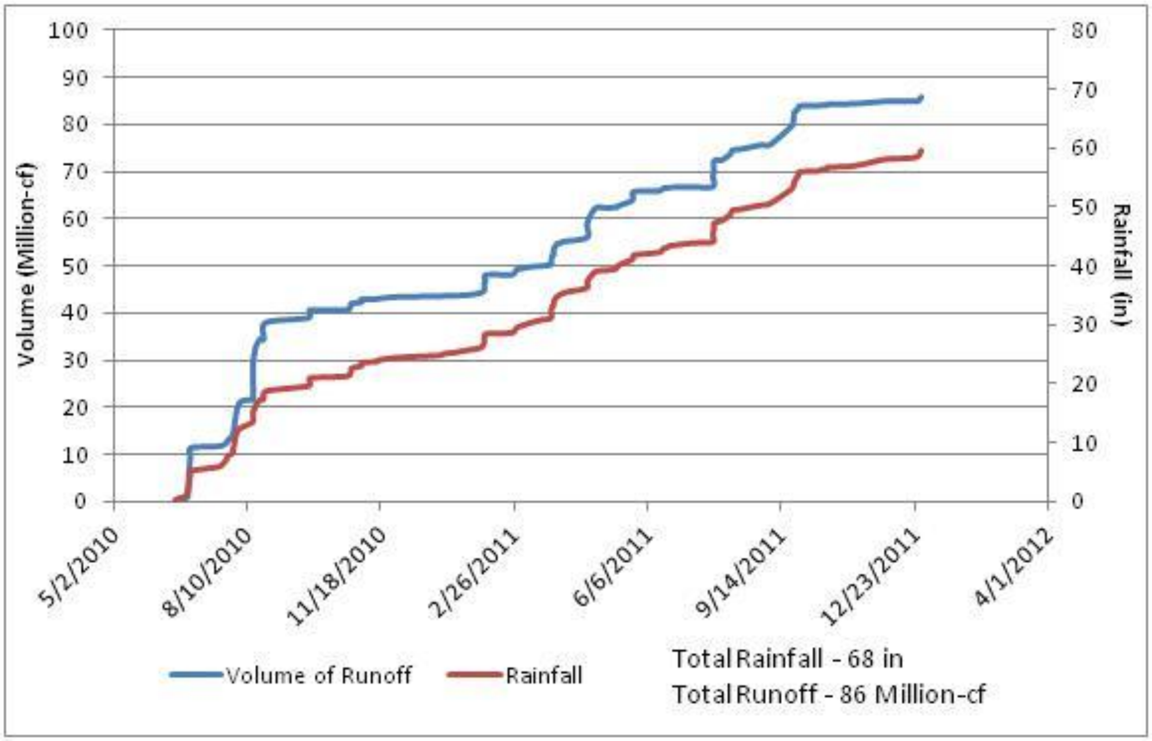

Figure 3-6. Cumulative rainfall and runoff from the Sumter Street subwatershed (Storms larger than 0.1 in., n = 104).

The quantity of water exiting the entire Sand River watershed and the two subwatersheds at each monitoring station were calculated based on the water level recorded by the sampling instrument at each location. These levels were then converted to flow rate using Equation 3-1. These flows were analyzed to create a runoff hydrograph for each storm event. Runoff volumes (V_r) for each storm were calculated using Equations 3-3 to 3-4. Figures 3-7, 3-8, and 3-9 show the rainfall and runoff volume relationship for the 10-ft pipe, Hoods Lane, and Sumter Street, respectively.

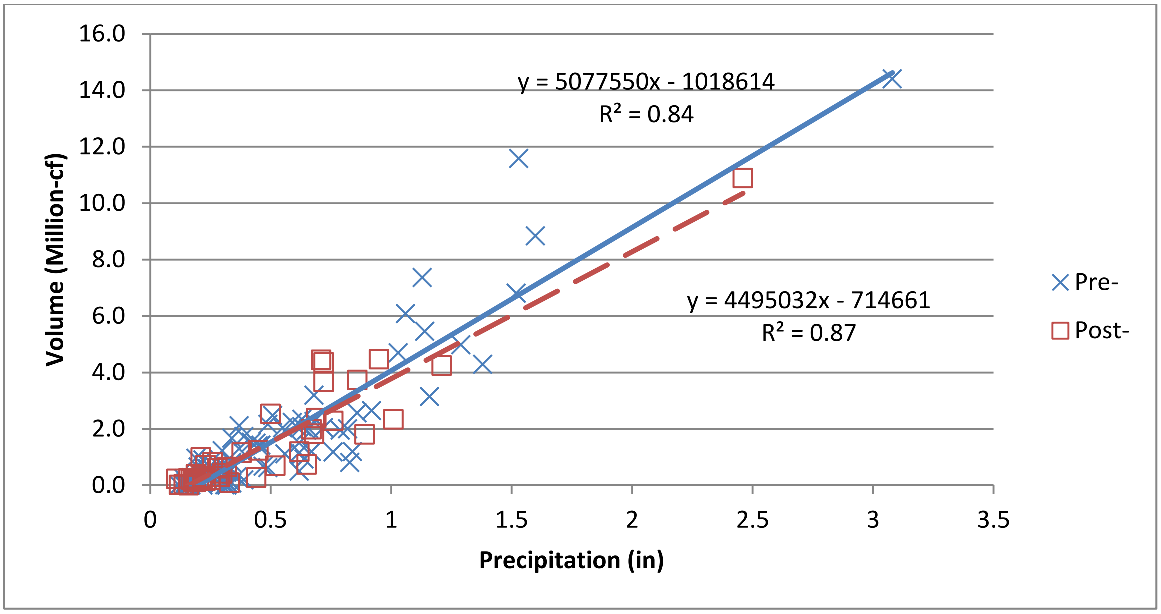

Figure 3-7. Rainfall-runoff volume relationship for the entire watershed as measured at the 10-foot pipe (n=121)

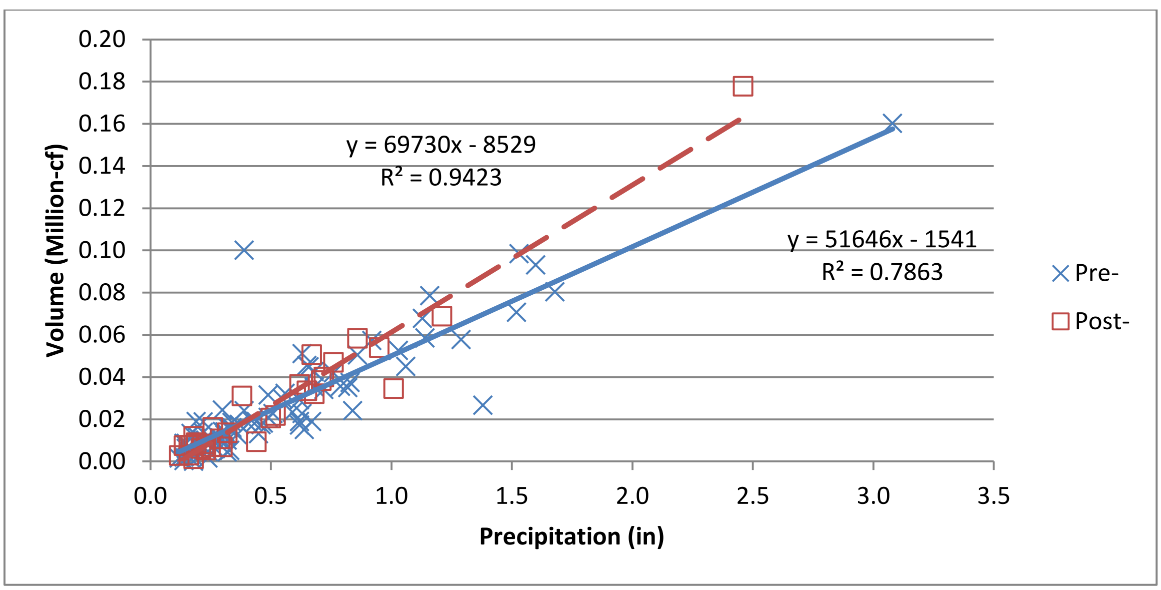

Figure 3-8. Rainfall-runoff relationship for the Hoods Lane watershed (n=119).

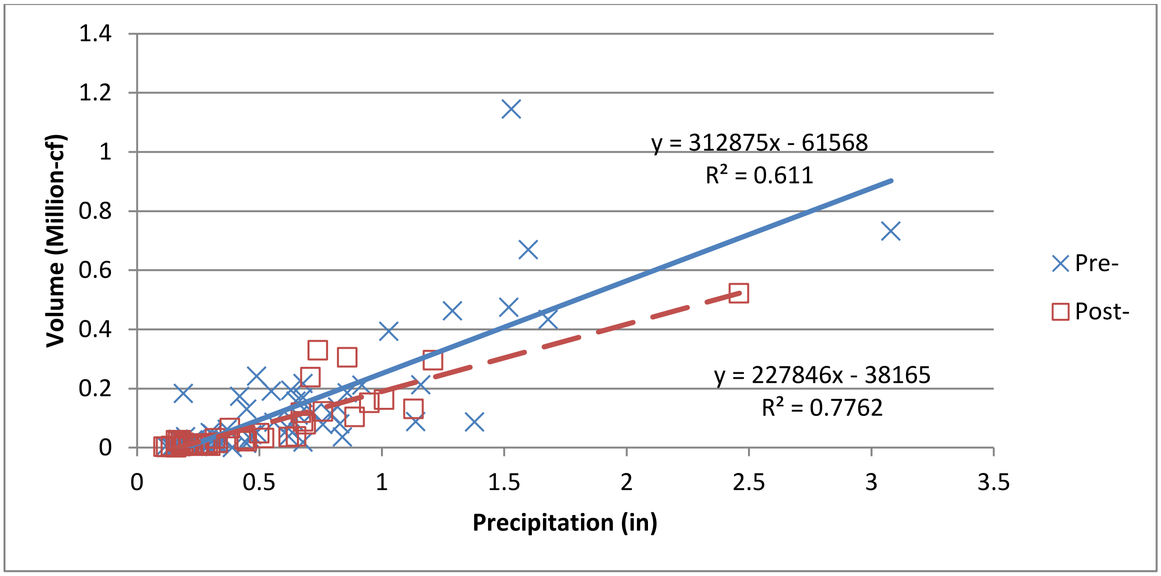

Figure 3-9. Rainfall-runoff relationship for the Sumter Street watershed (n=103).

For the 10-foot pipe and Sumter Street watersheds, the post construction trend line has a smaller slope than the pre-construction trend line. These results imply that there may have been less runoff from these watersheds after the construction of the bioretention cells. Data from the Hoods Lane watershed show an opposite trend - there was more runoff after the construction of the bioretention cells. To determine if these trend lines are an accurate representation of the change in the volume of runoff, the 95% confidence intervals were plotted.

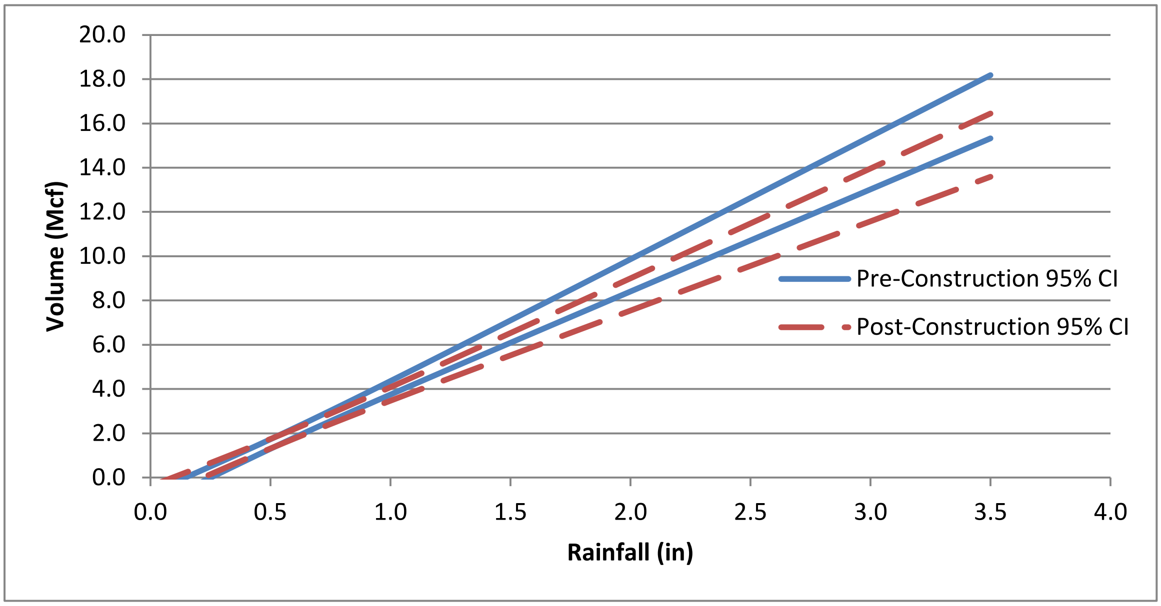

Figure 3-10. Rainfall and runoff volume 95% confidence intervals for the entire watershed as measured at the 10-foot pipe.

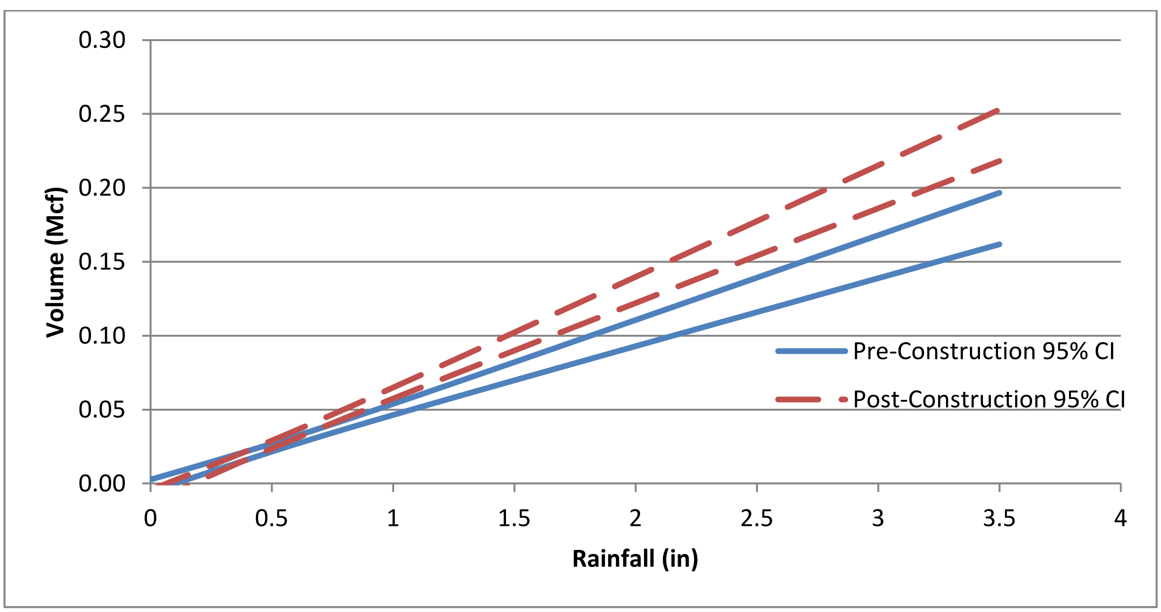

Figure 3-11. Rainfall and runoff volume 95% confidence intervals as measured at the Hoods Lane watershed.

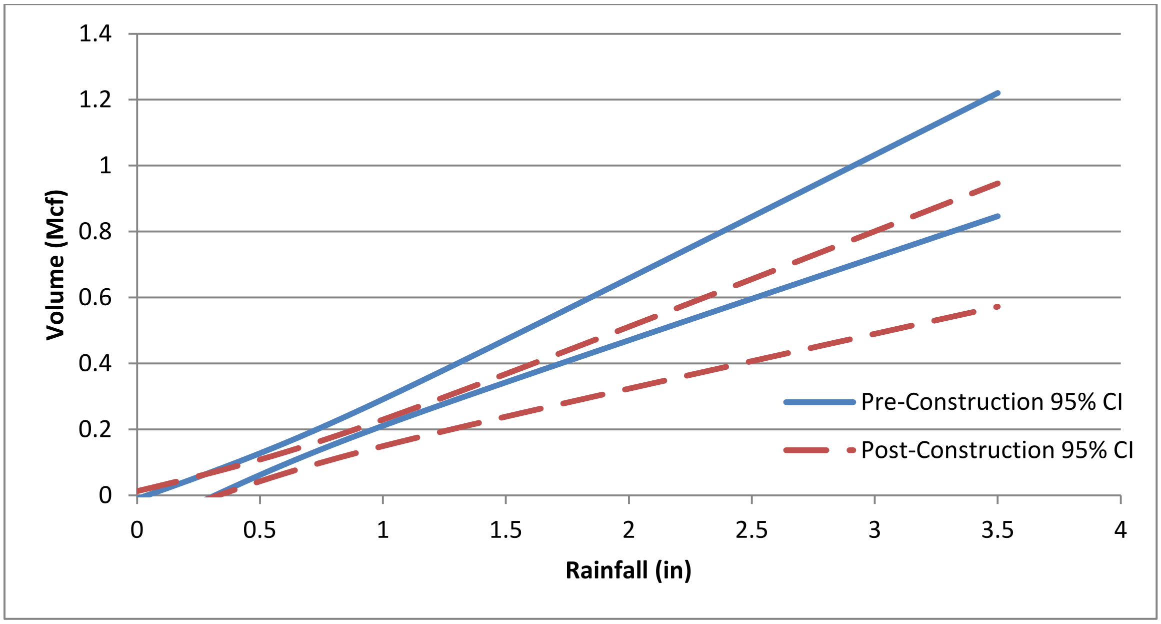

Figure 3-12. Rainfall and runoff volume 95% confidence intervals as measured at the Sumter Street watershed.

As shown in Figures 3-10, 3-11, and 3-12, the confidence intervals for the pre-construction and post-construction trendlines overlap, and therefore, solely fitting trend lines to the data is insufficient evidence to show that the bioretention cell construction significantly impacted the volume of water being discharged from the watershed.

Table 3-3 shows a summary of the average runoff coefficients for the entire Sand River watershed, Hoods Lane watershed, and Sumter Street watershed. A higher runoff coefficient is indicative of a larger volume of stormwater leaving an area per unit volume of precipitation.

Table 3-3. Summary of Average Runoff Coefficients.

Watershed	Pre-Construction	Post-Construction	n
10’ Pipe (Total)	0.545	0.497	131
Hoods Ln.	0.322	0.314	119
Sumter St.	0.134	0.0895	104

A t-test was conducted to determine if there was a significant ($\alpha = 0.05$) difference between the means. The results are summarized in Table 3-4.

Table 3-4. T-test for the Difference of the Runoff Coefficient Means. (“FTR” indicates a failure to reject the null hypothesis)

Watershed	Decision	*p-value*
10’ Pipe	Reject H_o	$p = 0.133$
Hoods Ln.	Reject H_o	$p = 0.363$
Sumter St.	FTR H_o	$p = 0.015$

The results show that although there was a consistent reduction in the calculated runoff coefficient means, there was no significant reduction in the mean runoff coefficients for either the entire Sand River watershed or the Hoods Lane subwatershed. This is most likely due to the fact that the total area of the bioretention cells was very small compared to the total area of the watershed, or approximately 0.4%. In the Hoods

Lane watershed, there was only one bioretention cell. Due to its design and collection from a smaller runoff contributing area, this cell did not perform as well as some of the other bioretention cells based on qualitative observations. A small contributing runoff area, combined with the small area of the one bioretention cell (PNL) in the subwatershed, most likely contributed to the lack of a significant reduction in the mean runoff coefficients.

The Antecedent Moisture Condition (AMC) of a soil can impact the initial abstraction from rainfall from a watershed (Chow, 1988). AMC was defined as the measure of the volume of rainfall in the 5-day period preceding any given storm event (Chow, 1988). Therefore, pre-construction and post-construction runoff coefficients were compared based on the AMC of the watershed as defined in Table 3-5 (SCS, 1972).

Table 3-5. Classification of Antecedent Moisture Conditions (SCS, 1972).

AMC group	Total 5-day antecedent rainfall (in)	
	Dormant Season	Growing Season
I	< 0.5	< 1.4
II	0.5 to 1.1	1.4 to 2.1
III	> 1.1	> 2.1

A Pearson correlation test was conducted to determine if there was a significant correlation between rainfall, duration, intensity, volume of stormwater discharged, runoff coefficient, antecedent runoff condition (ARC), and AMC. Table 3-6 summarizes the results of the Pearson correlation test.

Based on the results summarized in 3-6, there is a significant correlation between rainfall, volume discharged, and runoff coefficient. There is also a significant correlation between runoff coefficient, ARC, and AMC. The correlation between ARC and AMC is negative because the AMC increases with a higher frequency in rainfall and thus decreases with an increase in number of dry days (ARC). There are higher runoff coefficients as the time between storms decreases because of the increased soil moisture. As such, the storms analyzed in each watershed were separated based on AMC I, II, or III and their pre-construction and post-construction runoff coefficients were compared using a t-test. Tables 3-7 through 3-9 show the results comparing the pre-construction and post-construction runoff coefficients based on AMC.

Table 3-6. Pearson Correlation Test Summary.

	Rainfall (in)	Volume (ac-in)	Duration (hr)	Intensity (in/hr)	Runoff Coefficient	AMC	ARC (days)
Rainfall (in)	1	0.8816	0.0985	0.4382	0.5061	-0.0668	-0.1006
		p < 0.0001	*p < 0.2613*	*p < 0.0001*	*p < 0.0001*	*p < .4469*	*p < .2511*
Volume (ac-in)	-	1	0.0238	0.4723	0.7309	-0.0236	-0.2028
			p < 0.7869	*p < 0.0001*	*p < 0.0001*	*p < 0.7887*	*p < 0.0197*
Duration (hr)	-	-	1	-0.418	-0.0183	-0.2233	0.0545
				p < 0.0001	*p < 0.8346*	*p < 0.0101*	*p < 0.5351*
Intensity (in/hr)	-	-	-	1	0.3214	0.0456	-0.1051
					p < 0.0002	*p < 0.6039*	*p < 0.2305*
Runoff Coefficient	-	-	-	-	1	0.076	-0.2575
						p < 0.3865	*p < 0.0029*
AMC	-	-	-	-	-	1	-0.2073
							p < 0.0171
DSLR	-	-	-	-	-	-	1

Table 3-7. 10-foot Pipe Watershed Pre-Construction and Post-Construction Runoff Coefficients (* = statistically significant difference).

AMC	Pre-Construction	Post-Construction	*p-value*
I n=103	0.540	0.474	*0.164*
II n=17	0.447	0.578	*0.7915*
III n=11	0.800	0.522	*0.0513*

Table 3-8. Hoods Lane Watershed Pre-Construction and Post-Construction Runoff Coefficients (* = statistically significant difference).

AMC	Pre-Construction	Post-Construction	*p-value*
I n=97	0.314	0.307	*0.394*
II n=13	0.300	0.343	*0.739*
III n=9	0.473	0.336	*0.005**

Table 3-9. Sumter Street Watershed Pre-Construction and Post-Construction Runoff Coefficients (* = statistically significant difference).

AMC	Pre-Construction	Post-Construction	*p-value*
I n=91	0.117	0.081	*.002**
II n=4	0.108	ND	-
III n=9	0.395	0.15	*.006**

Table 3-7 shows that there are no significant differences in the pre-construction and post-construction mean runoff coefficients for any AMC in the Sand River watershed. The AMC III group is close to having a significant reduction, but the p-value is greater than 0.05. Table 3-8 shows that the Hoods Lane watershed displayed a significant reduction in the runoff coefficient for AMC III storms. Table 3-9 shows there are significant differences between the runoff coefficients for both AMC I and II in the Sumter Street watershed. There were no data for the Sumter Street watershed for storms occurring in AMC II post-construction.

The lack of a significant impact on the runoff coefficient of the entire watershed as measured at the 10-foot pipe was likely due to the relative size of the bioretention cells in comparison to the entire watershed, 0.4% of the watershed. The reduction of the runoff coefficient in the Hoods Lane watershed under AMC III would suggest that the sole bioretention cell in this watershed functions most effectively at a high level of soil moisture. This could be due in part to the design of this cell. There are porous asphalt cells directly adjacent to the cell, and for smaller storms most of the surface runoff is captured before entering the bioretention cell. However, higher soil moisture contents result in a greater surface stormwater runoff (Chow, 1988). Furthermore, this increase in runoff may allow for a larger volume of water to enter the bioretention cell and be retained. By solving Equation 3-5 for the volume of runoff (V_r) and replacing the runoff coefficient value with the reduction in runoff coefficient seen in the Hoods Lane watershed the volume of runoff captured per inch of rainfall can be calculated. For the Hoods Lane watershed in an AMC III, a reduction of 6.44 ac-in/inch of rainfall or 23,373 gallons/inch of rainfall is shown. Generally, in an urban watershed that has a large percentage of impervious area, the initial abstraction is smaller when compared to less developed area (SCS, 1972). However, the difference in the pre-construction runoff coefficients for the Hoods Lane watershed for AMC's I, II, and III, 0.314, 0.300, and 0.473, respectively, suggest that the amount of moisture present in the soil prior to a storm event does have an impact on the runoff coefficient for this watershed.

The significant reduction in the runoff coefficient in the Sumter Street watershed was not expected due to there being no bioretention cells or porous asphalt sites in that watershed. A possible explanation would be that there was an error in the data recording. The voltage on the sensor and recording unit dropped below 10V during a period from November, 2011 to January, 2012. The recommended battery life for the ISCO® 2150 module is 2.5 months, and the battery was installed in July, 2012 and not replaced until January, 2012. Another possible explanation would be the accuracy of the current stormwater maps. There have been difficulties mapping the subterranean stormwater pipes in previous studies, and to date no comprehensive mapping has been done (Meadows et. al, 1992; Woolpert, 2003). The most current map is a hand-drawn map shown in Figure 3-2. Figure 3-2 shows that most trunk lines from the downtown area flow directly south towards Hitchcock Woods. However, one block north of the bioretention cells on Park Avenue, all the trunk lines flow southeast and then turn south to intersect South Boundary Avenue. Further examination of the stormwater conveyance system should be done to conclusively determine if this was the cause of the Sumter Street watershed displaying an unexpected response to bioretention cell installation.

CONCLUSIONS

Fully developed urban watersheds, such as the Sand River watershed near Aiken, SC, can result in serious impairments to receiving water bodies. Stormwater peak flow and volume reduction can be achieved in developed urban areas by using green infrastructure practices such as bioretention cells and porous asphalt. In this study, bioretention cells and porous asphalt sites were constructed in the downtown area of Aiken, SC. The Sand River watershed and two subwatersheds were monitored and analyzed to determine the effectiveness of the construction of the green infrastructure practices. Runoff coefficients were developed for each watershed. The characteristic runoff coefficients for each watershed prior to the construction of the bioretention cells were compared to the runoff coefficients after bioretention cell construction on the basis of Antecedent Moisture Condition (AMC). Analysis of the runoff coefficients demonstrated that there was no significant difference for the Sand River watershed after the construction of the bioretention cells. The Hoods Lane watershed had a significant reduction in runoff coefficient for storms occurring shortly after previous storms (AMC III). For the Hoods Lane watershed, the reduction was 23,373 gal/in-rainfall in the volume of stormwater being discharged. The Sumter Street watershed demonstrated a significant decrease in runoff coefficient, despite there being no bioretention cell construction in the watershed. Based on the results of this study, the volume of water being discharged from the Sand River watershed was not significantly impacted by the construction of the bioretention cells.

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CHAPTER 4

BIORETENTION CELL CHARACTERIZATION AND MODELING

ABSTRACT

The Sand River, located near Aiken, SC (*33.560°N, -81.719°W)* has experienced severe bank and channel erosion by large stormwater flows being discharged from the downtown area of the city. Parkways within the downtown area were retrofitted with green infrastructure practices in April 2011 as part of a project to reduce the volume and peak flow of stormwater being discharged from the Sand River headwaters watershed. These green infrastructure practices included the installation of bioretention cells and porous paving materials. Objectives of this study include characterizing the bioretention cells, analyzing their performance as a part of the larger Aiken Green Infrastructure project, and modeling the water budget and hydraulic performance within individual cells. The monitored bioretention cells performed well at capturing direct rainfall, surface runoff, and storm sewer inlet flows. Level data suggested that 212,500 gallons of captured stormwater were infiltrated back into the native subsoil from March – June 2012 in one bioretention cell. Water quality data from one bioretention cell suggested that pollutants commonly associated with urban runoff were being introduced and captured within the cell as designed. Several porous asphalt sites were monitored and found to be effective at infiltrating captured surface runoff.

INTRODUCTION

The design of bioretention cells is hindered by a general lack of knowledge concerning the biological processes occurring within the bioretention cells (Davis et al., 2009). Current design practices are based on localized observations of various bioretention cells (SCDHEC, 2005; Davis et al., 2009). The hydraulic processes occurring within bioretention cells have been previously quantified using engineering principles in applications other than bioretention cells. These processes include inlet and outlet flow, pool depth, media depth, and underdrain configuration. The characterization and modeling of the bioretention cells constructed in this study is meant to contribute to the enhancement of the current bioretention cell design practice.

The Sand River is a small stream near Aiken, SC (*33.560°N, -81.719°W*). Its headwaters are formed by a 10-foot diameter storm sewer outfall, and a major portion of the stormwater discharged from the outfall originates in the downtown area of Aiken. The existing stormwater infrastructure in the downtown area is extensively developed. High urban land use and degree of impervious surface area result in very rapid peak flows exiting the watershed during and after rain events. The Sand River headwaters area has been significantly impacted by these high intensity flows. In some areas, the headwaters have been eroded down to 70 feet. This erosion has resulted in a loss of function for some areas of Hitchcock Woods, as well as a significant amount of sediment transport downstream. When a watershed is developed, the hydrology of that watershed is typically altered. Increases in peak stormwater runoff flow and volume are common (Chow, 1988). The effects of these increases are often reduced by retention structures such as ponds (Chow, 1988). However, in fully developed areas there is often limited space for conventional stormwater management practices like retention ponds. Green infrastructure practices seek to restore the natural functions of a given area (Benedict and McMahon, 2006). Practices such as the use of bioretention cells and porous paving materials require less space than retention ponds, and these practices have been shown to be effective at infiltrating stormwater runoff under most conditions (Davis et. al, 2009).

Numerous studies have been conducted to examine the quantity of water exiting the Sand River watershed (Meadows, 1992; Woolpert, 1994, 1995, 1998, 1999, 2001, 2003). However, none of these studies have conducted any monitoring to verify the volume of water or peak flow exiting the watershed. The initial part of the Aiken Green Infrastructure projected included the installation of a flow monitoring device in the storm sewer outfall to quantify the volume of water exiting the watershed for various storm events. The collection of measured flows at the 10-foot pipe outfall for different storm events is very useful for watershed characterization as well as for the determination of stormwater control measures (SCM's) to be deployed in the watershed drainage area. Models are very helpful when characterizing watersheds with known dimensions, but many parts of the Sand River Headwaters watershed were unknown (Meadows et. al, 1992; Woolpert, 2003). Some parts of the existing sewer network are poorly mapped, and many parts are not mapped at all (Meadows et. al, 1992; Woolpert, 2003). This makes accurate modeling through computer programs very difficult. By quantifying the volume of water exiting the watershed, the scope of the stormwater flow and volume reduction project by urban retrofit with green infrastructure can be realized. Due to limited land area within the watershed, there is an increased need for low impact stormwater management practices to be implemented as extensively as possible.

Site Description for Stormwater Control Measures

The downtown Aiken urban area (Figure 3-1) is an urbanized watershed that – at the time of this study - drained to a single outfall located in the Hitchcock Woods (HUC 030601060203) and contributed significant discharge at the headwaters of the Sand River (Woolpert, 2003). The center of the watershed is located at 33.560417°N and -81.719553°W. The elevation change in the watershed is approximately 200 feet with an average slope of 3%. The total watershed area is 1220 acres. A portion of the Sand River directly downstream from the stormwater outfall is a 303(d) impaired water body due to excessive levels of fecal coliform (SCDHEC, 2011).

The area surrounding the stormwater outfall is known as the Hitchcock Woods, a 3.1 mi^2, wooded recreational area adjacent to both the downtown portion of the City of Aiken and many residential neighborhoods. As the stormwater infrastructure in the City has expanded and become more complex, the volume of water discharged from the 10-foot pipe has formed an unstable canyon 70 feet deep (Eidson et al., 2010). In the downtown area of Aiken, there are several green areas, known as parkways, located between the roadways. However, many of the streets are curbed, and the water is routed directly from the street into the stormwater pipe network. As a part of the Sand River Headwaters Green Infrastructure Project, much of this curbing was modified to allow stormwater from the street to enter the cell. Existing stormwater pipes were also modified so that the stormwater would enter some of the cells and then be discharged back into the same systems after being retained for a period of time.

Figure 4-1. The bioretention cell between Richland Avenue and Park Avenue along Chesterfield Street (CRP) during construction.

Bioretention Cell Descriptions

Each of the bioretention cells installed downtown had similar features including, curb cuts, inlet and/or outlet structures, porous asphalt adjacent to the cell, and bioretention soil media. AutoCAD drawings of the cells can be found in Appendix D.

The bioretention cell located along Park Avenue between Union Street and Fairfield Street (PUF) had no inlet structure. All of the collected stormwater was either direct rainfall or surface runoff from the adjacent streets or overflow from the adjacent porous asphalt cells. An outlet structure was present to capture any overflow within the cell. Within this outlet structure was a weir box that was designed to measure excess flow from the cell underdrain via an upturned elbow. There were soil moisture sensors placed in this cell at various locations and depths.

The bioretention cell located along Park Avenue between Chesterfield Street and Newberry Street (PCN) was extensively monitored. Inflow and outflow were both monitored and there were several soil moisture sensors located in the cell. Inflow was routed into the bioretention cell from the existing storm sewer system. There was a consistent problem with backflow entering the inlet structure due to a poorly designed inlet invert elevation, resulting in many difficulties in calculating flow with Equation 4-2. A level logger was installed to measure the stage and thus storage of stormwater in the cell and to determine the time when the backflow condition occurred.

The bioretention cell located along Park Avenue between Newberry Street and Laurens Street (PNL) was also monitored. The inlet and outlet flows were monitored and there were also soil moisture sensors located throughout the cell. The inlet flow into the bioretention cell was not a part of the storm sewer system; it consisted of one stormwater drop inlet that captured surface runoff from an adjacent median.

Along Chesterfield Street between Richland Avenue and Park Avenue, there were two cells. The north basin was connected to the south basin via a 15-inch reinforced concrete pipe. Direct rainfall and surface runoff were responsible for the majority of runoff entering these cells. Due to difficulties measuring surface water inflow, a level logger was installed in the lowest part of the bioretention cell to measure the volume entering by inflow, direct rainfall, and surface runoff.

Porous Asphalt

There were several sites within the Sand River watershed that were retrofitted with porous asphalt, porous concrete, or permeable pavers. This study focuses solely on the sites directly adjacent to the bioretention cells along Park Avenue. Primary placement for the porous asphalt sites was in the areas adjacent to the bioretention cells used for parallel parking (Figure 4-4). Pervious concrete sites in the downtown Aiken area have been shown to have infiltration rates in excess of 1000 in/hr (Putman, 2010). A necessity for the proper function on porous asphalt, as well as, pervious concrete is a native subbase with a high infiltration rate (Balades, 1995; Hunt, 2006). The sandy soil in the downtown Aiken area facilitates this infiltration.

OBJECTIVES

Bioretention cells and porous asphalt sites were installed in several of the parkways in the downtown Aiken area as to reduce the peak flow and volume of the urban stormwater runoff originating within the Sand River watershed. The purpose of these bioretention cells and porous asphalt sites is to capture stormwater runoff and promote retention by infiltration and storage. The objectives of this study were to:

1. Characterize the hydrological functions occurring within the bioretention cells and porous asphalt sites and develop a water budget for these systems,
2. Analyze the in-situ performance of the bioretention cells and porous asphalt, and compare them to the designed performance,

3. Build, calibrate, and validate an effective model representing a bioretention cell water budget using available data from the as-built bioretention cells and monitoring instrumentation.

METHODS

Meteorological Monitoring

Meteorological data was collected near the bioretention cells. The rainfall data were collected in two locations: in the center of the downtown area (*33.561°N, -81.719°W*) and near the 10-ft pipe outfall (*33.555°N, -81.722°W*). The rainfall data near the center of the downtown area were collected using a Campbell Scientific® tipping bucket rain gauge and Campbell Scientific® CR800 Series data logger. These two pieces of equipment were part of a larger monitoring apparatus that also measured temperature, relative humidity, barometric pressure, solar radiation, wind speed, and wind direction. Table 4-1 shows the equipment used at this monitoring location.

Table 4-1. Equipment Deployed at the Downtown Aiken, SC Monitoring Location.

Description	Manufacturer	Model	Units
Data Logger	Campbell Scientific® (Logan, UT)	CR800	-
Power Supply	Campbell Scientific® (Logan, UT)	PS100-SW	-
Solar Panel	BP® (London, UK)	SP10	-
Barometer	Setra® (Boxborough, MA)	CS100	kPa
Anemometer	RM Young® (Traverse City, MI)	03002-L13	m/s
Rain Bucket	Texas Electronics® (Lubbock, TX)	TE525-L13	mm
Temp/RH Sensor	Campbell Scientific® (Logan, UT)	CS215-L13	°C/%
Pyranometer	Li-Cor® (Lincoln, NE)	LI200X-L13	W/m^2
PAR sensor	Li-Cor® (Lincoln, NE)	LI190SB-L13	$Mmol/m^2$

The data collected from the Campbell Scientific® weather station was transmitted to an online database within the Intelligent River® network, and this data was accessed remotely and downloaded from the Intelligent River® database. The rainfall data at the stormwater outfall was collected using an ISCO® tipping bucket rain gauge that was downloaded in-situ from the ISCO® 6712 unit.

Porous Asphalt Monitoring

At two of the bioretention cells (PUF and PNL), the water level in the adjoining porous asphalt cells was monitored. This water level was monitored using Global Water® level loggers installed at the bottom of the subbase material with a 10-minute sampling interval. Using level data, the area and depth of the porous asphalt site, and the porosity of the base material (0.40), the volume of stormwater stored for various storm events was calculated using Equation 4-2. The infiltration rate of the captured stormwater back into the native subsoil can also be calculated from this level data by determining the slope of the receding limb of the level data.

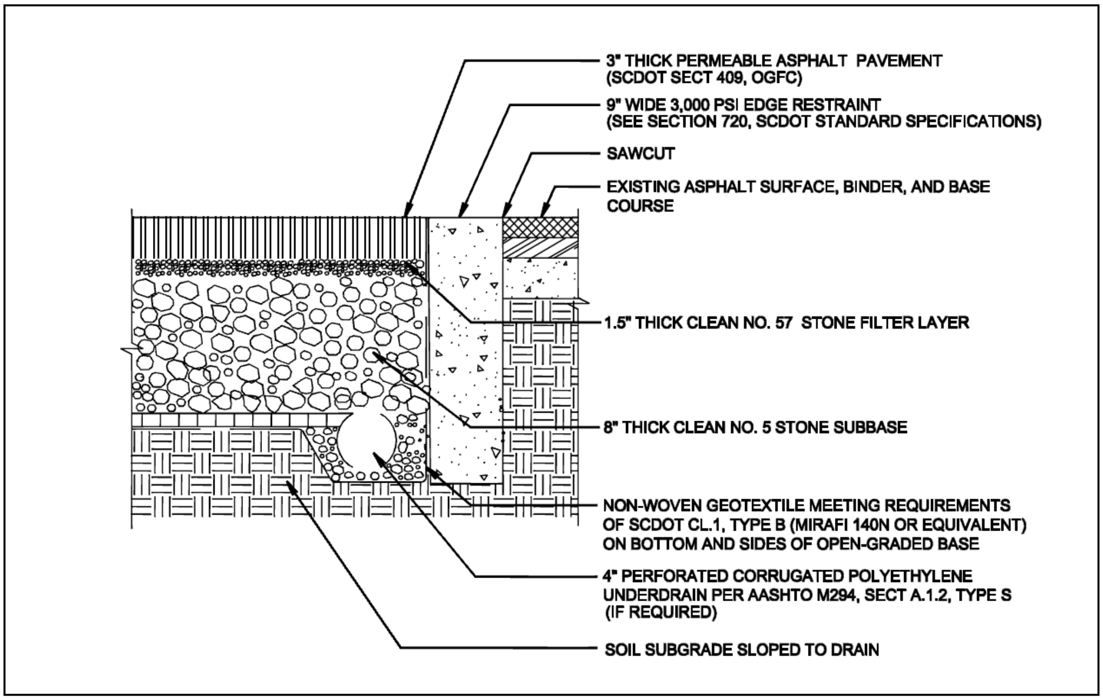

Figure 4-2. Porous asphalt design (Woolpert, 2010).

Data from the various sensors in the bioretention cells were collected routinely. Data from the ISCO® sampling units were collected using an ISCO® 581 Rapid Transfer Device (RTD) and uploaded to a laptop for analysis in the Flowlink5® software. The raw data were in the form of water level and the frequency of sampling events, if any occurred. Soil moisture data were accessed using the QAQC program to retrieve data from the Intelligent River® site where the data were stored. QAQC is a program used to access the data stored within the Intelligent River® database. These data was then exported to Excel® for further analysis. Meteorological data from the Campbell Scientific® CR 800 Series data logger were accessed and uploaded using the Campbell Scientific® PC 200W software. The Solinst® level data was retrieved using the companion software for the Solinst® level logger. Once these data were downloaded, they were exported to Excel for further analysis.

Porous asphalt level data were analyzed to quantify the volume of water that the porous asphalt paved areas were capturing and infiltrating. If the under drain for the porous asphalt cells were closed, then all of the water that entered the cell for any given event was infiltrated back into the native subsoil. The maximum volume of water stored in the cell was calculated based on the maximum level within the porous asphalt.

$$V_{max} = H_{max} * A * \Phi \qquad (4\text{-}1)$$

Where,

V_{max} = maximum volume of water stored in the cell, [ft^3]

H_{max} = maximum level of water in the cell, [ft]

A = area of the porous asphalt cell, [ft^2]

Φ = porosity of the aggregate in the base material

Bioretention Soil Media (BSM) Analyses

Schnabel Engineering, LLC conducted several boring and infiltration tests prior to bioretention cell construction in 2009. Two of the monitored bioretention cells were included in the infiltration tests. These infiltration tests showed that the native soil in PNL had an infiltration rate of 21.6 in/hr and the native soil in PUF had an infiltration rate of 10.8 in/hr (Schnabel, 2009). These high infiltration rates are very desirable because they do not limit exfiltration out of the bioretention cells constructed in these locations.

Proper gradation is important to bioretention cell function in order to promote infiltration, but it is also important for pollutant removal (Diblasi et. al, 2009). Coarser material is beneficial for rapid infiltration, but coarse material provides few adsorption sites for pollutants (Diblasi et. al, 2009). Finer material such as clay provides more adsorption sites, and organic matter is very efficient at removing certain hydrophobic constituents (Diblasi et. al, 2009). The gradation of the BSM was analyzed 1-year after construction at several of the bioretention cells to see if there were any significant changes from the time of installation. Bulk density and porosity of the BSM were also measured one year post-construction. Porosity is a measurement that can be used to determine the subsurface storage of saturated BSM based on volumetric soil moisture content.

Bioretention Cell Level and Flow Monitoring

Multiple sensors were placed in and around the bioretention cells to monitor their function and effectiveness. Some of the cells were configured differently than others. In general, the following parameters were monitored on most of the cells: inflow, outflow, soil moisture, and water quality.

Figure 4-3. Bioretention cell and monitoring locations in downtown Aiken, SC (Sand River Headwaters Green Infrastructure Project, 2010).

Bioretention cell inflow and outflow were measured using ISCO® 6712 sampling units equipped with ISCO ® 730 bubbler modules. Both inflow and outflow were routed through a box outfitted with a combination weir that consisted of a v-notch section and rectangular section.

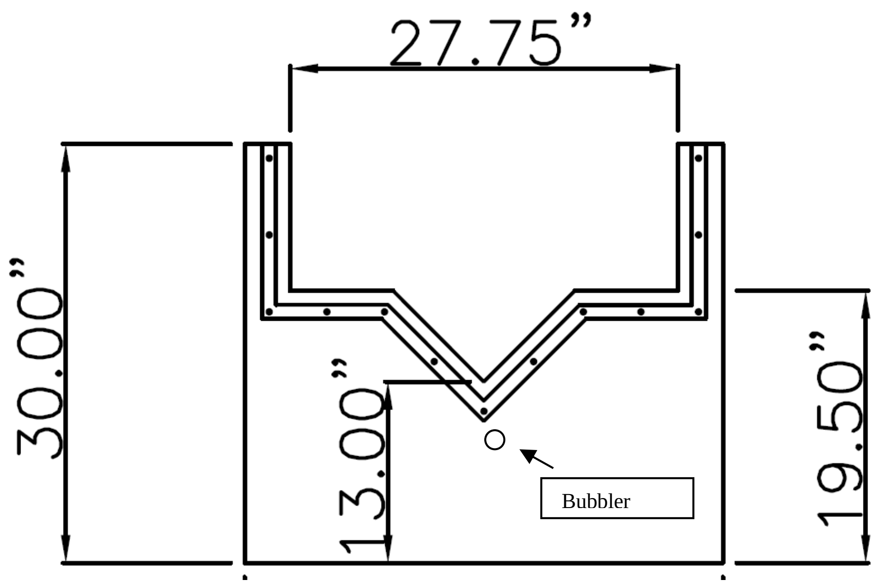

Figure 4-4. General combination weir and bubbler configuration for bioretention cell inlet and outlet.

Figure 4-5. Post-installation picture of combination weir configuration at the inlet of the bioretention cell between Newberry and Laurens Streets along Park Avenue (Bellamy, 2010).

Inflow was reported as the level behind a combination weir at the inlet to the cells. The equation for a combination weir is taken as the sum of the flow over a v-notch weir and a rectangular weir (Grant et. al, 2006) (Figures 4-4 and 4-5):

$$Q = C_1 \tan(\frac{\Theta}{2}) H_1^{2.5} + C_2 L H_2^{1.5} \quad (4\text{-}2)$$

Where,

Q = Flow [cfs]

C_1 = V-notch weir coefficient, C_1 = 2.5

Θ = Angle of v-notch weir, [radians]

H_1 = Head over the base of v-notch weir [ft], maximum value is the height of the v-notch section of the weir

C_2 = Rectangular weir coefficient, 3.0

L = Length of rectangular weir [ft]

H_2 = Head over the crest of rectangular weir [ft]

Within the cell, soil moisture sensors were placed in the bioretention soil media (BSM) at different locations and multiple depths. The data from the soil moisture sensors were accessed remotely from the Intelligent River® database. In the cells located at Chesterfield Street between Richland Avenue and Park Avenue (CRP) and Park Avenue between Chesterfield Street and Newberry Street (PCN) Solinst® level loggers were installed to measure surface water heights within the cells.

Outflow was analyzed in the same manner as inflow. Soil moisture data were analyzed to determine the infiltration rate of the bioretention soil media. This analysis was accomplished by comparing the time at the peak volumetric water content to the distance between the sensors.

$$i = \frac{(d_2 - d_1)}{(t_2 - t_1)} \tag{4-3}$$

Where,

i = Infiltration rate [in/hr]

d_1,d_2 = depth of sensors 1 and 2, respectively [in]

t_1,t_2 = time of peak volumetric water content at sensor 1 and 2, respectively [hours]

The level data was analyzed to determine the maximum amount of storage achieved by any given cell based on free water height within the cell. This relationship was established using the topography data from the as-built bioretention cell construction documents. The relationship used was:

$$V_{1,2} = \left(\frac{A_1 + A_2}{2}\right) * \ d \tag{4-4}$$

Where,

$V_{1,2}$ = volume between the height at point 1 and the height at point 2 [ft^3]

A_1 = area of the contour at height 1 [ft^2]

A_2 = area of the contour at height 2 [ft^2]

d = distance between points 1 and 2 [ft]

By determining this characteristic volume at different elevations within the cell, a stage-storage relationship was developed. Based on this relationship the level data were converted to stormwater volume in storage. Infiltration rates were also calculated from these level data. As the cell fills up with stormwater, the level quickly increased to a peak value, then declined at a steady rate until the cell no longer held any surface water. The rate of decline of the level in the cell was calculated as an infiltration rate.

One cell (PCN) had a reoccurring problem with backflow entering the inlet weir box as the cell filled with stormwater. The backflow occurred due to the invert elevation of the outlet pipe being higher than the invert elevation of the weir. While this does not necessarily significantly inhibit the function of the bioretention cell, it does create difficulties for quantifying inflow based on the level behind the inlet weir as described in Equation 4-2. Once level loggers were installed in the bioretention cell with the backflow problem, the time that the level of surface water reached the bottom of the v-notch weir was recorded. Using this time, the level and corresponding flow measurements on the inflow hydrograph were removed because the level being recorded no longer represented inflow. It was also assumed that the backflow reduced the velocity of the stormwater entering the cell to a degree that inflow was no longer considered to be significant.

Data from the level logger in the PCN cell were also used to determine the maximum volume of stormwater stored in the cell for a given rain event. Using these data for six storms during March 2012 and April 2012, the inflow volume of the stormwater entering the bioretention cell was determined. The fraction entering via the stormwater inlet structure was calculated using Equation 4-2 and eliminating the portion of the hydrograph that occurred after the backwater condition. The contributing fraction from direct rainfall was calculated by multiplying the area of the parkway by the equivalent depth of rainfall. The remaining volume of stormwater was assumed to be contributed from surface runoff, it was calculated by subtracting the inlet and direct rainfall fractions from the total volume stored within the cell. The relationship between contributing fractions was used to isolate previously acquired hydrographs in order to estimate the volume of stormwater captured by the cell.

Using the data for the storms occurring after the installation of the Solinst® level logger as a guide, the other hydrographs could be adjusted accordingly and the volume entering the PCN bioretention cell could be more accurately calculated.

An important input of water to the cell is direct rainfall. This quantity is based on the amount of rainfall and the area of the cell.

$$V_{direct} = P * A \tag{4-5}$$

Where,

V_{direct} – Volume from direct rainfall [ac-in]

P = precipitation [in]

A = area of the bioretention cell [acres]

Assuming that there are no other stormwater inputs to the cell other than the inlet flow, direct rainfall, and surface runoff from adjacent pavements, the total volume that is captured by the bioretention cells and porous asphalt can be determined. The storage of stormwater within a bioretention cell can be calculated by:

$$\Delta S = I - O \tag{4-6}$$

Where,

ΔS – change in storage

I – Inflows

O- Outflows

The outflows out of a cell are stormwater outflow, evapotranspiration, and infiltration. The storage is the amount of water ponded within the cell.

Influent and effluent water qualities were remotely sampled and monitored. The ISCO 6712® sampling units were programmed to sample the water flowing in the inlet and outlet to each cell if there was a sufficient volume of water passing through the cell. The automated sampling protocol had two components: (1) the first flush of stormwater and (2) a composited sample from the entire sampling event. This two-part sampling protocol was conducted to discriminate between any fluctuations in inlet concentrations over the duration of the sampling event. For storms where a water sampling event occurred, the samples were collected and sent to a certified lab for chemical analysis. After a qualifying storm event occurred, samples were removed from the ISCO® 6712 sampling unit, stored on ice, and transported to a certified lab. Pollutants of interest were: total suspended solids (TSS), nitrate, ammonia, potassium, zinc, copper, phosphorus, and oil and grease (O/G). The specific analyses performed on the samples are summarized in Table 4-2.

Table 4-2. Summary of Analyses Conducted on Stormwater Samples Taken from the Inlet of PCN.

Pollutant	Method	Detectable Limit
TSS	SM 2540-D	10 mg/L
Nitrate Wastewater	SM 4500NO3-E	0.02 mg-N/L
Ammonia	EPA 350.1	0.1 mg-N/L
Total Phosphorous	EPA 365.4	0.1 mg-P/L
Copper	EPA 200.7	0.02 mg/L
Zinc	EPA 200.7	0.02 mg/L
Nitrite	SM 4500NO2-B	0.01 mg-N/L
DRO in Water	SW846 SM 3510C/8015B	0.469 mg/L

The concentration from the composited sample was used to determine Event Mean Concentration (EMC) for a particular storm. The EMC is that mass of pollutant that passed through the inlet and/or outlet during the storm event and is calculated by the following equation (Li and Davis, 2009):

$$EMC = C_{comp.} * V \tag{4-7}$$

Where,

EMC = Event Mean Concentration [mg]

$C_{comp.}$ = pollutant concentration [mg/L]

V = volume of stormwater stored in cell [L]

The total mass of the pollutant stored in the cell is the difference of the concentration in the inflow and the outflow.

$$M_{pol.} = EMC_{in} - EMC_{out} \tag{4-8}$$

Where,

$M_{pol.}$ – mass of pollutant stored in cell, [mg]

EMC_{in} – event mean concentration of the inflow, [mg]

EMC_{out} – event mean concentration of the outflow, [mg]

STELLA® Modeling

The bioretention cells were modeled in STELLA® (ISEE Systems, Inc., 2007). This software program allows the user to create a water budget, control the physical parameters of the cell, and produce outputs that may be used to analyze bioretention cell performance and function. The water budget for a bioretention is no different from a general water budget for any retention area. There are inputs (precipitation, surface runoff, and inlet flow), outputs (evapotranspiration, exfiltration, and outlet flow), and storage.

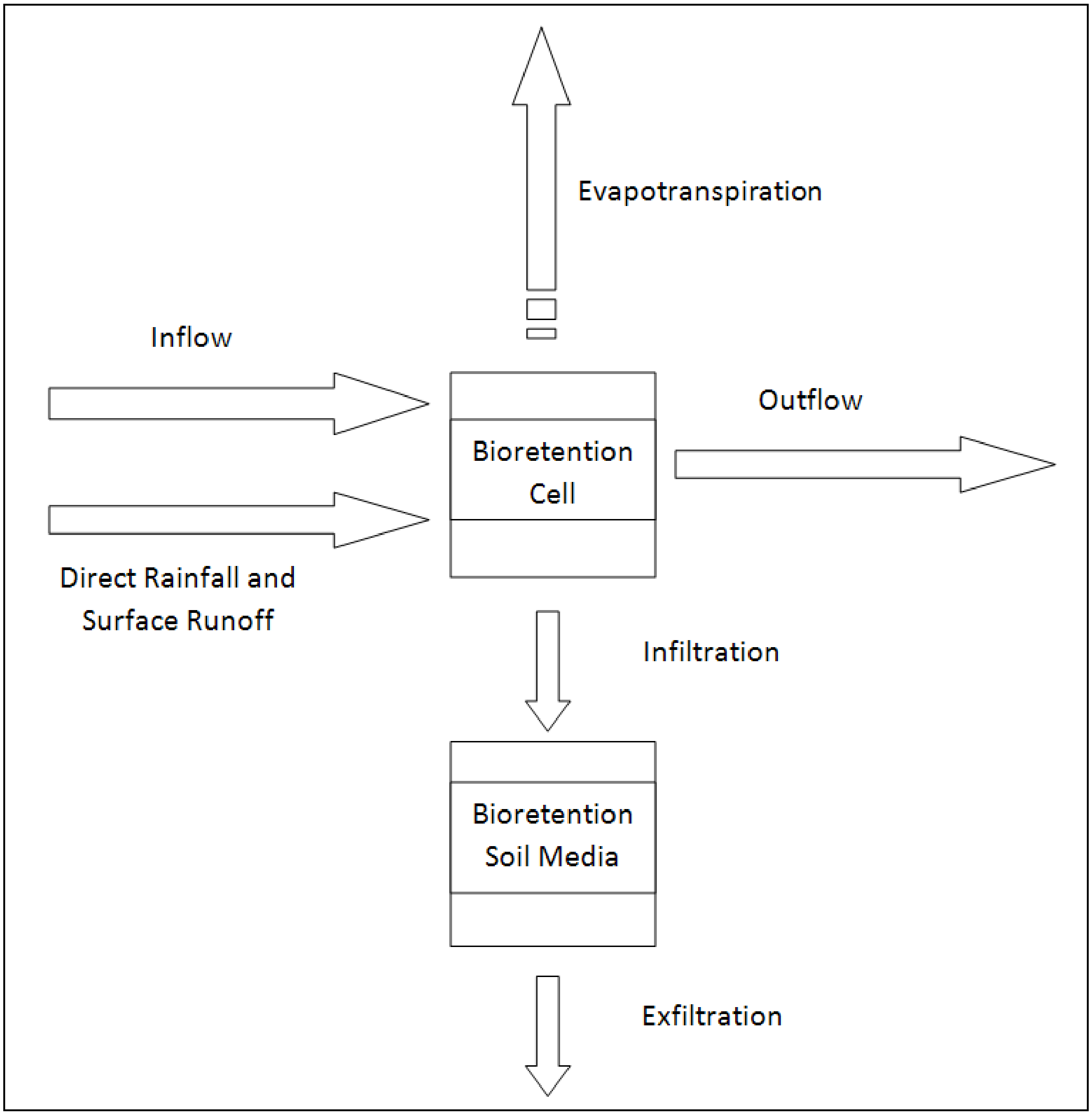

Figure 4-6. Conceptual water budget for a bioretention cell.

A bioretention cell is different from common retention areas in that part of the designed storage volume is contained within the soil media, and is referred to as the internal water storage zone (IWS). The volume of water stored in this zone is a design parameter and is a function of the media depth and porosity.

In the development of the STELLA® model, the inputs were data taken from various monitoring devices in or near the bioretention cell being monitored. Tables 4-3 and 4-4 are a summary of the input and output data, respectively, to the model and the units used.

Table 4-3. STELLA® Model Input Data.

Parameter	Symbol	Units	Data Source
Precipitation	P	in/hr	Measured
Inlet Flow	Q_{in}	ft^3/hr	Calculated from inlet level data
Outlet Flow	Q_{out}	ft^3/hr	Calculated from outlet level data
Infiltration Rate	i	in/hr	Calculated from BRC level data
Relative Humidity	RH	%	Measured
Temperature	T	°C	Measured
Solar Radiation	SR	cal/cm^2	Measured
Cell Area	A	ft^2	Measured
BSM Depth	d	ft	Measured
Porosity	n	-	Measured

Table 4-4. STELLA® Model Output Data.

Parameter	Symbol	Units	Data Source
Potential Evapotranspiration	PET	mm/day	Calculated from meteorological data
Storage	S	ft^3	Calculated from BRC level data
Level	L	ft	Measured

Other physical parameters needed are the stage-storage relationship pertaining to the specific bioretention cell. The general schematic for the model is shown in Figure 4-7.

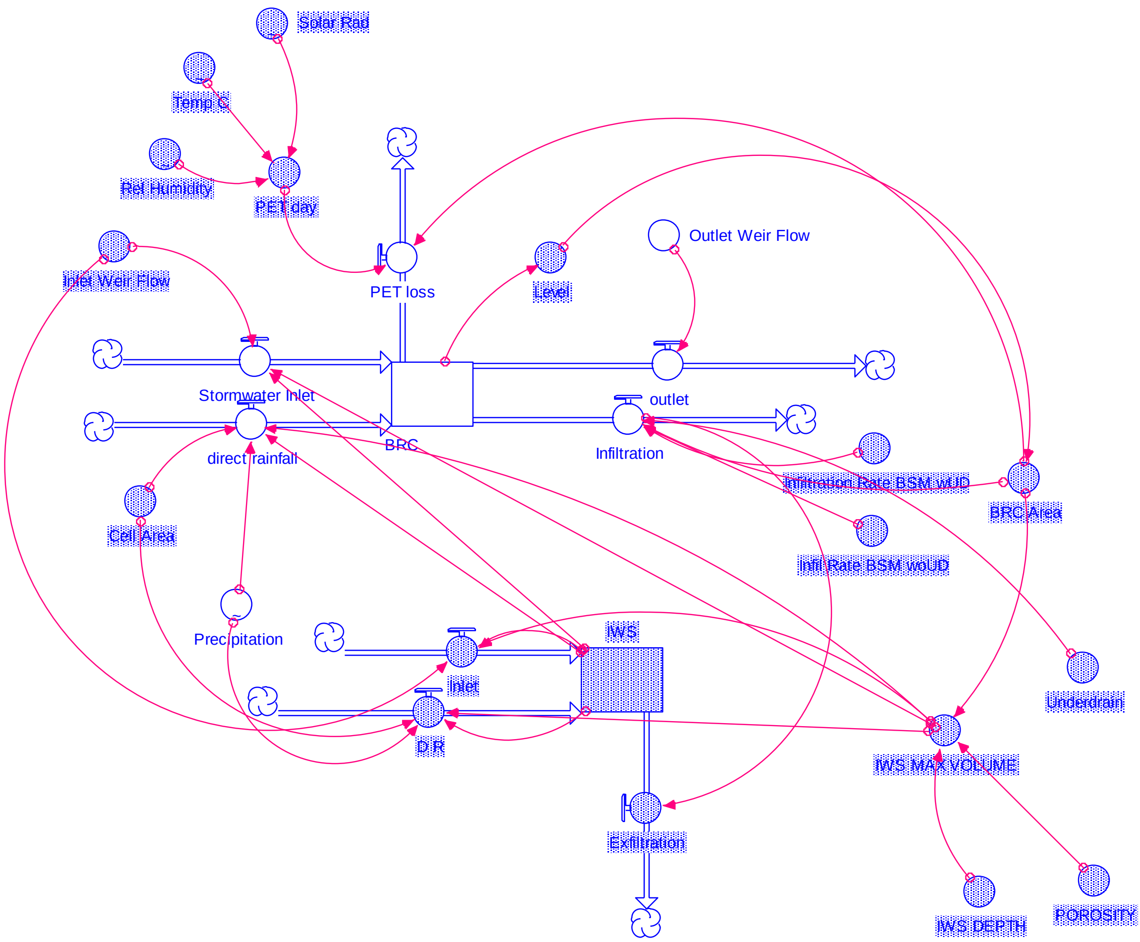

Figure 4-7. STELLA® model setup.

Utilizing the linking feature in the modeling program, input data can be entered and edited using Excel. Equation 4-10 shows the overall storage relationship for the bioretention cell based on the model input data.

BRC(t) = BRC(t - dt) + (Stormwater_Inlet + direct_rainfall - PET_loss - Infiltration - outlet) * dt

(4-10)

Where,

BRC – Storage within the bioretention cell [ft^3]

t – time [hr]

Stormwater_Inlet – inflow [ft^3/hr]

direct_rainfall – rainfall falling directly on the cell and the surrounding impervious surfaces [ft^3/hr]

PET_loss – loss due to potential evapotranspiration [ft^3/hr]

Exfiltration – loss from water within the cell leaving the soil media and infiltrating back into the native subsoil [ft^3/hr]

Outlet – outflow [ft^3/hr]

Several model parameters were calculated from input data to reconcile units and provide a uniform time step of one hour when running the model. The "direct_rainfall" variable in the equation is actually calculated from the precipitation and cell area using Equation 4-11.

$$direct\ rainfall = \frac{P}{12} * A \tag{4-11}$$

Exfiltration is calculated using Equation 4-12.

$$Ex = \frac{i}{12} * BRC\ Area \tag{4-12}$$

Where,

I – infiltration rate [in/hr]

BRC Area – surface area of the water stored in the bioretention cell [ft^2]

Potential Evapotranspiration (PET) was calculated using the Turc Equation (Turc, 1961. Evapotranspiration was calculated on a daily basis by the following equation:

If RH > 50:

$$PET = 0.013 * \left(\frac{T}{T+15}\right) * (S_r + 50) \qquad (4\text{-}13)$$

If RH ≤ 50:

$$PET = 0.013 * \left(\frac{T}{T+15}\right) * (S_r + 50) * (1 + \left(\frac{50-RH}{70}\right)) \qquad (4\text{-}14)$$

Where,

PET = potential evapotranspiration, [mm/day]

T = temperature, [°C]

S_r = solar radiation, [$cal/cm^2/day$]

RH = relative humidity [%]

The loss of ponded water due to potential evapotranspiration (PET) is calculated by modifying Equation 4-13 to account for loss on an hourly basis. This hourly PET rate is calculated by using the result from Equation 4-13 divided by 24.

Underdrain function in the model is important because some cells are outfitted with an underdrain and this can change the performance of the cell. In the model, the underdrain is activated by entering a “1” in the cell. Changing the value in the underdrain part of the model makes the infiltration rate change. When designing the bioretention cell, it may be difficult to determine how the infiltration rate will change. Using good engineering judgment based on the available data would be the appropriate approach to designing a bioretention cell with or without an underdrain.

Model validation was accomplished using the Nash-Sutcliffe coefficient as a standard metric to test hydrological models (Heasom et al., 2006). Values for the Nash-Sutcliffe coefficient can range from -∞ to 1. A coefficient of 1 means the model perfectly predicts the measured values, a coefficient less than 0 means that the mean of the measured values is a better predictor of the measured values than the model. The closer the coefficient is to 1, the better the model is at predicting the measured values.

$$R_{NS}^2 = 1 - \frac{\sum_{i=1}^{N}(Level_{i,measured} - Level_{i,predicted})^2}{\sum_{i=1}^{N}(Level_{i,measured} - Level_{average})^2} \tag{4-15}$$

RESULTS AND DISCUSSION

Porous Asphalt Performance

The level in the north PNL porous asphalt cell from a 0.68-inch storm event on 9/28/11 is shown in Figure 4-8. There was a maximum level of 0.315-ft of stormwater in the cell. This means of total volume of 3300 gallons of stormwater runoff was captured and infiltrated back into the subsoil over a period of 3.3 hours. Calculating the infiltration rate using the receding limb of the level data yielded an infiltration rate of 1-in/hr. This procedure was used to analyze the porous asphalt performance at the four monitored sites. Data from individual storms is detailed in Appendix E. Table 4-5 summarizes the capture volume and infiltration data for monitored porous asphalt sites.

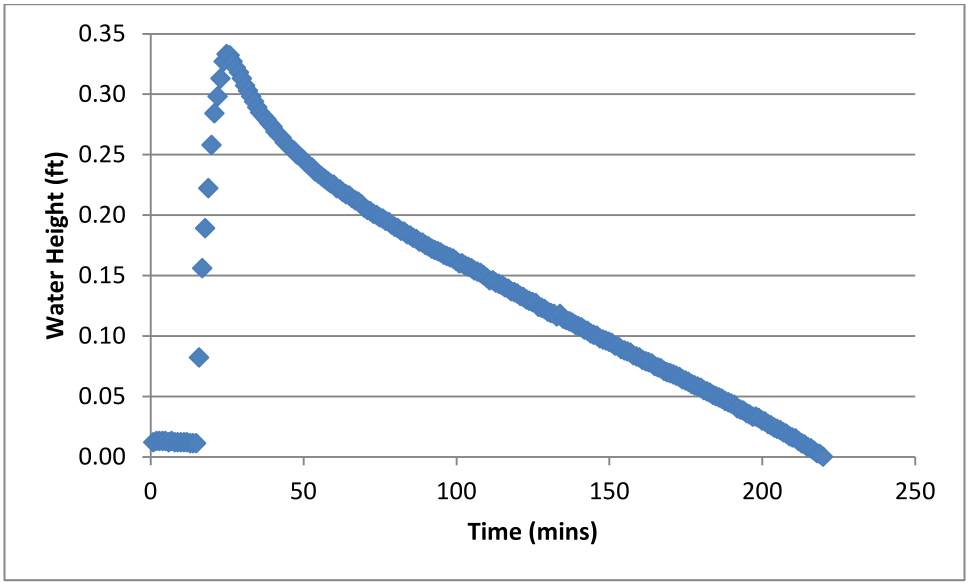

Figure 4-8. Level Data from PNL-N Cell on 9/28/11with 0.68-in of Rainfall.

Table 4-5. Volumes and Infiltration Rates at Three Porous Asphalt Sites in Downtown Aiken, SC.

Event Date	Rainfall (in)	PNL-S Storage (ft^3)	PNL-S Infiltration Rate (in/hr)	PNL-N Storage (ft^3)	PNL-N Infiltration Rate (in/hr)	PUF-S Storage (ft^3)	PUF-S Infiltration Rate (in/hr)
8/30/2011	0.67	58	11	366	1	47	3
9/6/2011	0.29	-	-	36	0.8	-	-
9/22/2011	2.84	431	6	775	1.1	1905	65
9/24/2011	0.72	181	6	423	0.6	1374	50
9/25/2011	0.62	-	-	252	0.65	-	-
9/28/2011	0.68	124	35	448	1	195	40
10/13/2011	0.38	-	-	110	0.5	-	-
11/29/2011	0.65	-	-	83	0.5	-	-
12/28/2011	0.89	-	-	229	0.5	-	-

The level data from the northern paved area at PUF (PUF-N) monitoring location was not included because the level logger was not functioning properly due to an internal electrical failure. As Table 4-9 shows, only the porous asphalt at the northern paved area at PNL (PNL-N) retained more stormwater for most of the storm events based on observed storage for most events versus the other areas that had limited event-based storage. This could be due to the subsoil in the other cells having such high infiltration rates. The subsoil at the southern paved area of PNL (PNL-S) and PUF (PUF-S) display extremely high infiltration rates compared to the infiltration rates seen at PNL-N. However, some of this infiltration may be due to the hydraulic effectiveness of the underdrains. The underdrain at the PNL-N site was observed to be capped during all field visits, but the underdrain in the PUF-S site was left open to discharge into the adjacent bioretention cell. The termination of the underdrain for the PNL-S site is in the western basin of the PNL bioretention cell and the high infiltration rates observed from the water level data suggest that this underdrain was usually left uncapped. Although the stormwater captured in the PNL-S and PUF-S sites was not infiltrated in-situ, they were nominally effective at reducing both the peak flow and volume discharged from their respective drainage areas because stormwater was routed into the bioretention cells and ultimately infiltrated. The level data from the PNL-N cell suggests that it is functioning as designed with respect to capturing stormwater runoff and infiltrating it back into the subsoil efficiently. Suggested infiltration rates for porous paving practices range from 0.5 – 3.0 in/hr (SCDHEC, 2005), and all of the porous asphalt sites monitored meet or exceed this criteria.

Other metrics used to evaluate porous asphalt performance are structural and surface performance. A visual inspection of all sites located along Park Avenue in June 2012 resulted in finding no major failures typically associated with open-graded paving practices. Due to low binder contents in the asphalt mix, porous pavements are especially prone to stripping and raveling, however none of these failures were observed. Some minor rutting was observed near the automated teller machine directly across the street from the Aiken Municipal Building, adjacent to the PNL bioretention cell. This rutting can most likely be attributed to the nature of the traffic entering and exiting the automated teller machine. Cars approaching this section of the porous asphalt site generally brake suddenly, turn their wheels while the car is not in motion, and accelerate.

The high traffic volume in this small area most likely exceeds the intended design of the paving practice, and a more robust pavement will be needed in the future. The surface of all the porous asphalt sites seems to be free of any significant clogging. The apparent lack of clogging is likely due in part to the routine vacuuming of the pavements by a street sweeper, as well as the lack of mobile sediment and organic material present in the urban downtown area. If clogging had occurred, ponding on the porous asphalt would be expected, and this result was not observed during the monitoring period.

Bioretention Cell Performance

Each of the bioretention cells were designed to reduce peak flow and capture a specific volume of stormwater. Table 4-6 summarizes the design peak inflows and captured volumes for a 2-year storm, as well as the maximum recorded inflows and capture volumes during the time data was collected for storms of at least a 2-year return period.

Table 4-6. Bioretention Cell Peak Flow and Capture Volume Summary.

	Design (2-year)		As-built (greater than 2-year storms)	
Cell	Peak Inflow (cfs)	Capture Volume (cf)	Max. Peak Inflow (cfs)	Max. Capture Volume (cf)
PUF	2.02	1196	ND	ND
PCN	11.55	11398	6	3700
CRP-N	2.76	2722	ND	ND
CRP-S	1.62	1597	4.6	2390
PNL	10.19	10055	0.495	ND

As Table 4-6 shows, each of the monitored cells except CRP-S appears to be over-designed in terms of peak inflow and maximum capture volume. The design of any retention structure is dependent on variables such as drainage area, land use, and rainfall (Chow, 1988). While land use and design rainfall can be determined based on assumptions, the drainage area contributing runoff to each cell is more difficult to ascertain. The design documents use varying drainage areas for each cell and they range from 0.34 acres to 3.14 acres (Woolpert, 2009). These drainage areas were determined by topographical maps and available stormwater piping diagrams. While many assumptions must be applied as design criteria for direct rainfall and surface runoff to each cell, it is more difficult to determine the appropriate drainage area when an existing storm sewer pipe is routed into the cell. Additional flow routed from the existing stormwater infrastructure is likely what accounts for the disparity between the design and as-built peak flows within the cells. Peak flows into PCN and PNL are only a fraction of what was designed and as a result, the cells are functioning at less capacity than they were designed. However, this "over-design" of the bioretention cells is more desirable than an under-sized system, which could result in flooding, short-circuiting, and/or poor hydraulic and treatment performance.

The monitored inlet into the PCN cell was not the only route stormwater could enter the cell. Table 4-7 shows the measured and calculated contributing sources of stormwater for the PCN cell based on measured inflow and level data

Table 4-7. Volume Fractions Entering PCN Bioretention Cell from March 2012 – April 2012.

Date	Rainfall (in)	Inlet Volume (ft^3)	Direct Rainfall (ft^3)	Surface Runoff (ft^3)	Total Storage (ft^3)
3/13/2012	0.19	667	483	1029	2178
3/16/2012	0.15	655	381	1202	2238
3/23/2012	0.27	519	686	961	2166
3/31/2012	0.55	1818	1398	487	3702
4/1/2012	0.2	763	508	1567	2838
4/2/2012	0.66	433	1677	96	2206
	Totals	4854	5133	5341	15328
	Percentage	32	33	35	100

Measured inlet flow accounts for approximately one-third of the stormwater entering the cell, with calculated direct rainfall accounting for another third. Surface runoff is calculated by subtracting the volumes entering via the inlet structure and falling directly on the cell from the total volume of storage measured within the cell. With the installation of the level logger, the contributing volume of surface runoff can be calculated. However, the peak flow from the surface runoff is almost impossible to accurately quantify because flow routing of surface runoff across the porous asphalt cells and through the numerous curb cuts. However, the relative volume of surface runoff entering the cell through the curb cuts could present a valid explanation for the small measured flows summarized in Tables 4-6 and 4-7. Thus the cells may be functioning closer to their designed capacity for peak flow reduction than based on measurements only at the inlet.

Infiltration rates for the BSM as designed, as-built, and as tested one year post-construction are detailed in Table 4-8.

Table 4-8. Bioretention Soil Media Infiltration Rates (Woolpert, 2010; Woolpert, 2009).

Cell	Design (in/hr)	Native Soil (in/hr)	After 1 year (in/hr)	BSM (S-M sensors) (in/hr)
PUF	10	10.8	ND	ND
PCN	10	ND	35.8	ND
CRP-N	10	16.8	ND	ND
CRP-S	10	ND	30.7	2.3
PNL	10	21.6	ND	ND

The tested BSM displayed infiltration rates in excess of the specified infiltration rates. The native soil also had infiltration rates larger than the design values. High infiltration rates in both the BSM and the native subsoil may explain why the maximum measured capture volume is much less than the designed capture volume. If the stormwater is infiltrating into the BSM and exfiltrating into the native subsoil at a rate higher than the design rate, there would be significantly less ponding measured on the surface of the bioretention cell.

Table 4-9. Particle Size Distributions for the As-Built Bioretention Cells and as Sampled 1 Year Later.

	As-Built		1-year later	
Cell	% Sand	%Silt/Clay	% Sand	%Silt/Clay
PCN	85.2	14.8	82.5	17.5
PNL	78.3	21.7	77.5	22.5
CRP	ND	ND	77.5	22.5
PUF	85	15	74.9	25.1

As Table 4-9 shows, there is very little change in the particle size distributions for the bioretention cells sampled. The PUF bioretention cell showed a 10% increase in the silt/clay fraction, suggesting that fine materials are being trapped in the cell. If this trend continues, the fine material could eventually lead to clogging of the BSM and porous asphalt adjacent to the cell. The results for BSM bulk density and porosity are presented in Table 4-10.

Table 4-10. Bulk Density and Porosity of BSM 1-year Post-Construction.

Cell	Bulk Density (g/cm^3)	Porosity
PUF	1.71	0.35
PCN	1.35	0.49
PNL	1.61	0.39
CRP	1.72	0.35

The BSM porosity for the PCN cell is higher than the other sampled BSM, suggesting that there are more coarse materials in the PCN cell compared to the other cells. Post-construction soil sampling summarized in Table 4-10 suggests that PCN has a larger fraction of coarse material as well.

Maintenance is a very important, yet often overlooked, aspect of any stormwater management practice (Davis et. al, 2009; Brown and Hunt, 2012). Bioretention cell maintenance is especially important due to the specificity of the materials and processes employed at any one site. The bioretention cells installed in Aiken, SC are all maintained by the City. Vegetation and perennial grasses are maintained to be aesthetically pleasing. However, the presence of deciduous trees in some of the bioretention cells (PCN, PNL, and PUF) may present a clogging problem in the future due to their leaves interfering with infiltration and flow routing within the cells. Organic litter accumulation regularly occurs behind the weir plate in PCN and should be removed as needed. Due to the large amount of runoff originating from the impervious surfaces bordering the bioretention cell, shallow concentrated flow is entering the CRP bioretention cell through two of the curb cuts, resulting in a serious erosion problem and could possibly contribute to an export of total suspended solids from the cell. Bank and BSM erosion at BRC inlets is highly undesirable and all efforts should be made to correct any erosion occurring and properly maintain the cells to prevent future erosion.

Water Quality Analysis

The effectiveness of the PCN bioretention cell in improving water quality was quantified by examining the influent and effluent event mean concentrations (EMC's) of several common pollutants. By comparing the EMC for the influent and effluent flows, the net capture or export of pollutants was determined. As previously mentioned, the PCN cell frequently experienced backflow conditions, which made the precise calculation of EMC's very difficult. Four storms (11/29/11, 12/28/11, 1/21/12, and 2/24/12) had a backflow occur, which led to an over-estimation in the EMC for the inlet. As a result, it was assumed that the bioretention cell likely received a maximum inflow of 7800 cubic feet (58,500 gallons). This value was calculated using the level and flow data from similar sized storms occurring after the installation of the level logger. Sampling events, rainfall, storage volume, and EMC's are detailed in Table 4-11.

Table 4-11. Sampling Event EMC Summary for Selected Pollutants in the PCN Cell.

Event Date	Rainfall (in)	Storage (gal)	TSS (kg)	Nitrate (g)	Ammonia (g)	Tot. P (g)	Cu (g)	Zn (g)	Nitrite (g)
10/19/2011	0.26	39085	2.5	12.3	17.0	-	-	8.6	-
11/16/2011	0.52	43440	2.9	6.9	20.2	-	1.1	11.4	-
11/29/2011*	0.78	58531	1.4	32.4	37.2	27.3	1.9	17.7	-
12/28/2011*	0.87	58531	9.8	66.9	-	-	-	21.6	-
1/12/2012	0.26	30365	2.1	15.6	16.2	-	-	-	-
1/18/2012	0.22	8262	NT	NT	NT	NT	NT	3.4	NT
1/21/2012*	1.01	58531	5.1	32.4	-	-	-	54.7	-
2/24/2012*	0.81	58531	4.3	36.6	-	-	4.8	14.6	-
3/3/2012	0.56	102396	0.1	1.0	2.1	-	0.1	2.0	-
3/31/2012	0.55	27691	2.4	12.5	42.0	-	-	6.9	1.4
Totals	5.84	485363	30.6	216.5	134.8	27.3	7.9	140.9	1.4

- – Below Detectable Limit *Backflow conditions

NT – Not Tested

The primary pollutants were Total Suspended Solids (TSS), Nitrate, Ammonia, and Zinc. Although Table 4-11 represents solely the inlet sampling concentrations, there was no outflow from the PCN cell for any of these events. Therefore, these EMC's represent the mass of each pollutant captured by the PCN cell. Metals and other pollutants can be transported via suspended solids, and the primary means of suspended solids removal within the bioretention cells is settling. Nitrate can come from many different sources within an urban watershed, but it is commonly associated with excessive fertilization. Nitrogen export could be a problem in the future if the bioretention cell continues to capture nitrate in the influent flows. Despite the fact that only 216 grams of nitrate were captured by the cell in a 5-month period, it may be advantageous to consider this contribution when fertilizing the perennial grasses and plants in the landscaping within the bioretention cell to prevent in-situ over-fertilization. Zinc is a common metal present in urban watersheds (Li and Davis, 2008). The relatively large mass captured in the PCN bioretention cell is beneficial because it prevents that mass from entering downstream water bodies. However, concentrations present in the BSM need to be carefully monitored because the bioretention cell is acting as a sink for zinc. Due to the close proximity of the PCN bioretention cell to sidewalks and the public, zinc concentrations could eventually pose a public health risk.

Improving water quality is an important performance characteristic of bioretention cells. Limited sampling of one bioretention cell in downtown Aiken, SC suggests that the bioretention cells are effective at removing monitored pollutants from being transported further downstream.

STELLA® Modeling

A STELLA® model was constructed and used as a tool to characterize the PCN bioretention cell. The model was structured to represent the water budget for the PCN bioretention cell. By changing the physical parameters of the cell and using the appropriate input data given in Table 4-3, the model could be used to design and evaluate any bioretention cell. Using data from March – June 2012, the level of captured stormwater in the PCN bioretention cell was modeled with a 1-hour time step and compared to the level data measured on a 10-minute time step during the same time period. A four month period was modeled using the Runge-Kutta method for solving equations (Kutta and Runge, 1900). During the modeled period there were 15 storm events of varying duration and intensity. Simulated versus observed results are provided in Figures 4-9 and 4-10 for PCN and CRP, respectively.

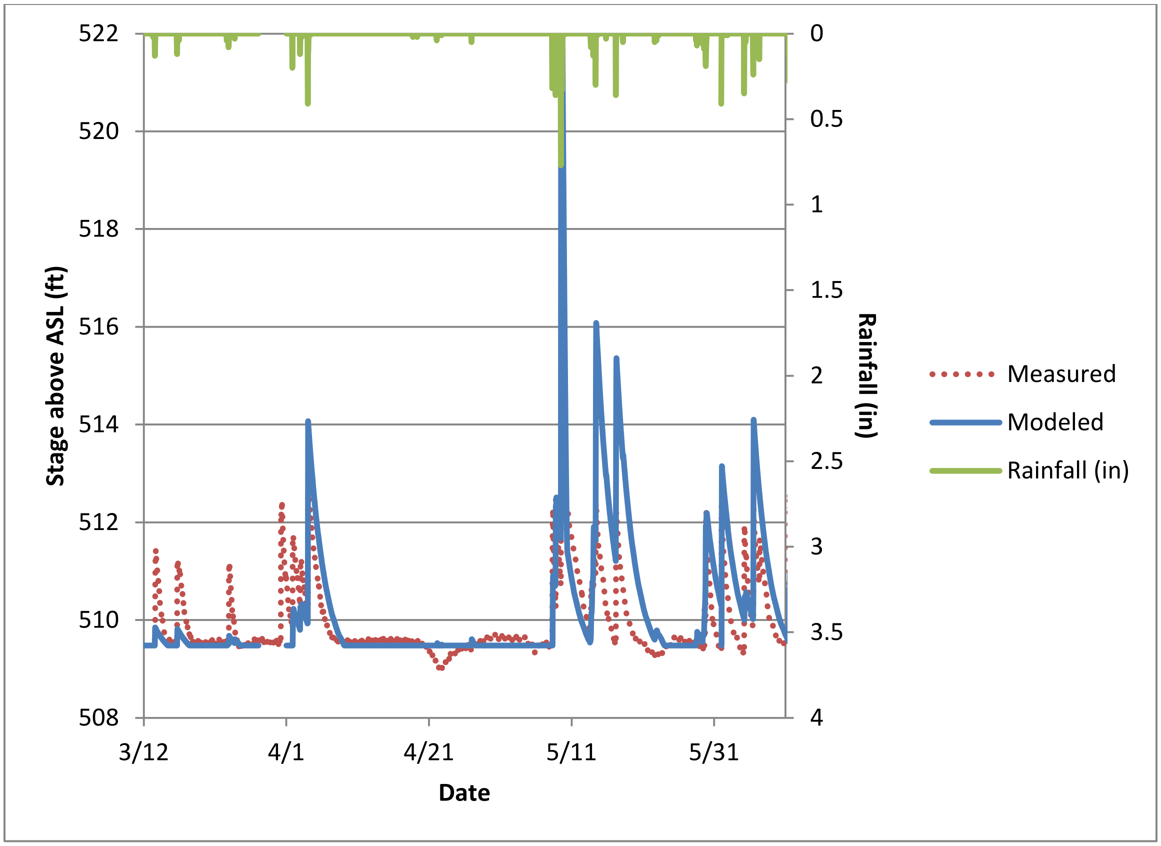

Figure 4-9. Predicted versus observed stormwater level of storage in the PCN bioretention cell [in feet above sea level (ASL)].

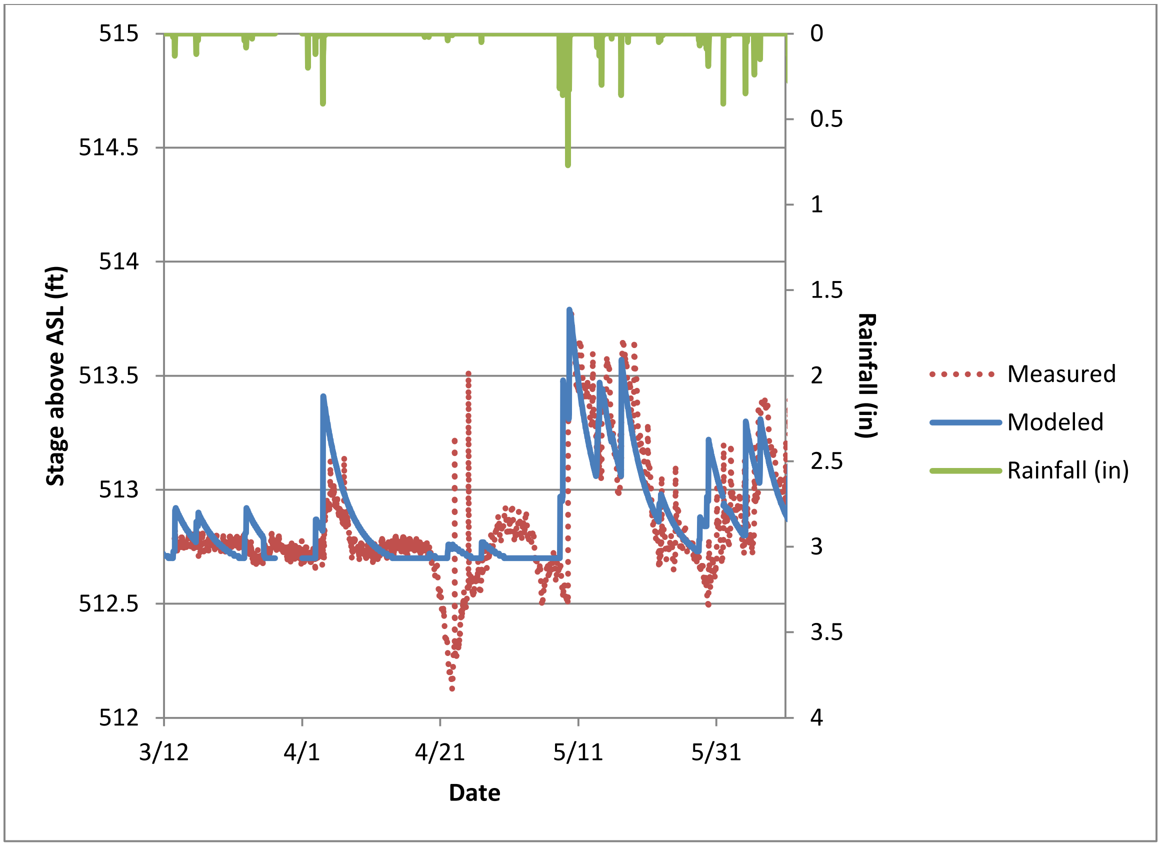

Figure 4-10. Predicted versus observed stormwater level of storage in the CRP bioretention cell [in feet above sea level (ASL)].

Modeled level peaks coincide with the measured level peaks for larger storms, and for one storm, occurring on 5/9/12, the model over-predicted the level in the PCN cell. Measured level and modeled level data for individual storms are located in Appendix F.

Nash-Sutcliffe coefficients were calculated from the measured and modeled data from each cell to determine the effectiveness of the model, and this information is summarized in Table 4-12.

Table 4-12. Nash-Sutcliffe Coefficients.

	PCN	CRP
R^2_{NS}	-0.70	0.68

The average and median rainfall for a storm during the modeled time period was 0.71 in. and 0.53 in., respectively. An analysis of the Nash-Sutcliffe coefficients suggests that the model is more effective at predicting the level in the CRP bioretention cell than using only the mean of the measured data, but not as effective at predicting the level in the PCN bioretention cell. This disparity could be due to several reasons including the time step of the model and the input data used to model the PCN cell. The model tended to under-predict storms with rainfall less than 0.25 in. Since the model had a time step of one hour, it is possible that small storms with short durations could be missed. Level data would be more sensitive because the sampling frequency was 10 minutes. Due to the larger time step used in the model and the relatively high infiltration rates of the bioretention soil media and the native subsoil, the smaller storms may have occurred and infiltrated within the one hour period. Using a smaller time step within the model may correct the problem for smaller storms, but it will cause the modeled time span to be much shorter due to the internal restrictions present in the program. Also, using a smaller time step may require more modification of the input data to ensure proper functioning within the program. Larger storms tended to result in the model over-predicting the level in the cell. With backflow being a significant problem with the PCN bioretention cell, each inflow hydrograph was compared to the level data and modified accordingly to represent the actual inflow based on the height of the inlet weir. However, the sampling interval for the level logger was 10 minutes and the sampling interval for the inflow measurements was 5 minutes. As a result, some of the inflow hydrographs may not have been cut off prior to the backflow occurring. This backflow timing issue and subsequent correction error would be most evident during a high intensity, short duration storm like the event that occurred on 5/9/2012. The inflow volume in the model for this storm forces it to grossly over-predict the actual level in the bioretention cell.

CONCLUSIONS

Bioretention cells in downtown Aiken, SC were evaluated to determine their effectiveness at reducing peak flow and volume of stormwater exiting the Sand River headwaters watershed. Based on cells that were instrumented with monitoring equipment, the bioretention cells were effective at capturing stormwater volume and infiltrating it back into the native soil. However, level and flow data suggested that some monitored cells were not capturing the stormwater volume for which they were designed to capture. The analyses of limited data suggested that bioretention cells were improving the quality of stormwater captured. This water quality improvement would be due mainly to the volume of water captured, stored, and infiltrated within the bioretention cells.

Porous asphalt sites located adjacent to bioretention cells are capturing surface runoff and direct rainfall, storing and infiltrating it back into the subsoil. Those bioretention cells designs with functioning underdrains, as well as, without underdrains seem to be effective in functioning as intended. In regards to structural and surface performance, the porous asphalt sites also seem to performing as designed.

STELLA® models used to evaluate the CRP bioretention cell were effective at predicting measured level. However, modeled data for the PCN bioretention cell were inadequate at predicting the measured level, likely because of the integrity of the input data being compromised by backflow. Quantifying the inflow for the PCN bioretention cell was difficult due to backflow problems frequently occurring and causing an over-calculation of inflow. Due to this difficulty in verifying the actual inflow and the relatively short period of time period of the level data collection, the model was not successfully validated. Further modeling and data collection should be done on the PCN and CRP bioretention cells in order to validate the model as it is currently designed, configured, and parameterized.

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CHAPTER 5

SUMMARY

In this study, bioretention cells and porous asphalt sites constructed in the downtown area of Aiken, SC were analyzed to determine their effectiveness. The effectiveness of the bioretention cells was determined at two scales: (1) the entire watershed, and (2) the individual, monitored bioretention cells.

The Sand River headwaters watershed and two sub-watersheds were monitored and analyzed to determine the effectiveness of the construction of the green infrastructure practices. Runoff coefficients were developed for each watershed, and the characteristic runoff coefficients for each watershed prior to the construction of the bioretention cells were compared to the runoff coefficients after bioretention cell construction on the basis of Antecedent Moisture Condition (AMC). Analysis of the runoff coefficients demonstrated that there was no significant difference for the Sand River watershed after the construction of the bioretention cells. The Hoods Lane watershed had a significant reduction in runoff coefficient for storms occurring shortly after previous storms (AMC III). For the Hoods Lane watershed, the reduction was 23,373 gal/in-rainfall in the volume of stormwater being discharged. The Sumter Street watershed demonstrated a significant decrease in runoff coefficient, despite there being no bioretention cell construction in the watershed. Based on the results of this study, the volume of water being discharged from the Sand River watershed was not significantly impacted by the construction of the bioretention cells.

Selected bioretention cells in downtown Aiken, SC were monitored and evaluated to determine their effectiveness at reducing the peak flow and volume of stormwater exiting the Sand River watershed. Based on the cells outfitted with monitoring equipment, the bioretention cells are effective at capturing stormwater and infiltrating it back into the native soil. However, level and flow data suggest that some of the monitored cells are capturing less stormwater than they were designed to capture. Preliminary data suggests that the bioretention cells are improving the quality of the stormwater captured. This water quality improvement is due mainly to the volume of water captured, stored, and infiltrated within the bioretention cells.

The porous asphalt sites located adjacent to the bioretention cells are capturing surface runoff and direct rainfall, storing and infiltrating it back into the subsoil. Designs with both underdrains and those without underdrains seem to be effective in functioning as designed. In regards to structural and surface performance, the porous asphalt sites also seem to performing as designed with little to no surface deterioration.

Modeling of the bioretention cells in STELLA® demonstrated that the level within the cells could successfully modeled if the input data was accurate. The CRP bioretention cell was successfully modeled during a period extending from March 2012 to June 2012. The PCN bioretention cell was modeled during the same period, but the model did not effectively predict the level within the cell because of the accuracy of the inlet flow data being used.

While further research needs to be done on the existing sewer network in the downtown Aiken area, results from this study demonstrate that bioretention cells and porous asphalt have to the potential to significantly impact the peak flow and volume of stormwater being discharged from the Sand River watershed. The small scope of the initial construction has limited the bioretention cells' impact on the peak flow and volume of stormwater being discharged from the entire watershed.

APPENDICES

APPENDIX A

10-FOOT PIPE DISCHARGE VOLUME SUMMARY

Event Date	Rainfall (in)	Volume Discharged (cf)	Volume (gal)	Volume (ac-in)	Duration (hr)	Intensity (in/hr)	Runoff Coefficient	AMC
1/30/2010	0.59	2214770	16566483	610	4.5	0.13	0.848	1
2/2/2010	0.23	476621	3565125	131	11.6	0.02	0.468	1
2/5/2010	1.06	6070975	45410893	1672	11.8	0.09	1.293	1
2/13/2010	0.15	118559	886821	33	1.6	0.09	0.178	1
2/15/2010	0.20	641915	4801521	177	3.1	0.06	0.725	1
2/22/2010	0.37	1285240	9613598	354	6.3	0.06	0.784	1
3/2/2010	0.34	802271	6000984	221	16.3	0.02	0.533	1
3/10/2010	0.44	1487086	11123404	410	5.8	0.08	0.763	1
3/11/2010	0.34	1661051	12424658	458	6.8	0.05	1.103	1
3/12/2010	1.13	7355507	55019191	2026	3.4	0.33	1.470	1
3/21/2010	0.22	895586	6698981	247	0.5	0.44	0.919	1
3/29/2010	0.39	1721360	12875771	474	0.8	0.52	0.997	1
4/8/2010	0.30	1222571	9144829	337	1.0	0.30	0.920	1
4/24/2010	0.24	203525	1522364	56	14.6	0.02	0.191	1
4/25/2010	0.51	2464287	18432866	679	5.2	0.10	1.091	1
5/3/2010	0.63	2076608	15533025	572	7.3	0.09	0.744	1
5/31/2010	0.67	1189165	8894954	328	4.3	0.16	0.401	1
6/1/2010	0.18	164924	1233633	45	0.6	0.31	0.207	1
6/2/2010	0.62	1492426	11163347	411	2.0	0.31	0.544	1
6/4/2010	0.18	252346	1887545	70	0.5	0.36	0.317	2
6/9/2010	0.32	458225	3427520	126	0.2	1.92	0.323	1
6/15/2010	0.29	651258	4871413	179	2.8	0.11	0.507	1
6/18/2010	0.37	2103383	15733306	579	1.3	0.30	1.284	1
6/20/2010	0.24	584582	4372673	161	2.3	0.10	0.550	1
6/25/2010	0.33	485296	3630014	134	4.6	0.07	0.332	1
6/26/2010	0.46	1412757	10567422	389	0.5	0.92	0.693	1
6/27/2010	0.27	444464	3324592	122	0.3	1.08	0.372	1
6/28/2010	1.60	8837926	66107689	2435	1.5	1.07	1.247	1
6/29/2010	1.52	6799082	50857130	1873	1.0	1.52	1.010	3
6/29/2010	0.49	2157554	16138502	594	1.4	0.35	0.994	3
7/12/2010	0.47	669023	5004290	184	1.1	0.43	0.321	1
7/21/2010	0.32	106856	799286	29	1.4	0.23	0.075	1
7/26/2010	1.14	5450110	40766821	1501	0.8	1.37	1.080	1
7/27/2010	0.64	928016	6941561	256	3.1	0.21	0.327	1
7/28/2010	0.12	2296	17177	1	3.7	0.03	0.004	2
7/31/2010	0.45	1389113	10390566	383	0.8	0.54	0.697	2
7/31/2010	0.21	112227	839461	31	0.4	0.50	0.121	2
8/3/2010	3.08	14397208	107691117	3966	4.3	0.72	1.056	1
8/6/2010	0.68	3179854	23785308	876	2.9	0.23	1.056	1
8/14/2010	0.84	1189342	8896278	328	3.0	0.28	0.320	1

8/15/2010	0.39	192515	1440013	53	4.5	0.09	0.111	1
8/15/2010	1.53	11578920	86610318	3190	0.7	2.30	1.709	1
8/16/2010	0.42	1522603	11389068	419	5.5	0.08	0.819	1
8/17/2010	0.63	2336957	17480438	644	0.6	1.08	0.838	2
8/20/2010	0.92	2643222	19771303	728	0.9	1.00	0.649	2
8/23/2010	0.14	63	472	0	0.3	0.56	0.000	1
8/24/2010	1.29	4980055	37250808	1372	0.3	3.87	0.872	1
9/17/2010	0.66	2006070	15005400	553	0.8	0.79	0.686	1
9/26/2010	0.31	653996	4891887	180	0.3	1.24	0.476	1
9/26/2010	0.86	2565109	19187017	707	4.7	0.18	0.674	1
9/26/2010	0.23	322323	2410973	89	1.7	0.14	0.316	1
9/27/2010	0.25	381112	2850717	105	1.1	0.23	0.344	1
10/25/2010	0.29	60992	456219	17	1.8	0.16	0.047	1
10/27/2010	0.68	2248925	16821957	620	1.7	0.41	0.747	1
10/28/2010	0.14	230768	1726148	64	1.0	0.14	0.372	2
11/4/2010	0.36	294354	2201766	81	2.8	0.13	0.185	1
11/4/2010	0.56	1106546	8276964	305	2.2	0.26	0.446	1
11/16/2010	0.16	1035	7744	0	3.3	0.05	0.001	1
11/16/2010	0.22	126007	942529	35	0.2	1.32	0.129	1
12/1/2010	0.49	625571	4679268	172	3.1	0.16	0.288	1
1/1/2011	0.34	73008	546098	20	4.3	0.08	0.048	1
1/5/2011	0.32	343	2567	0	12.1	0.03	0.000	1
1/12/2011	0.15	660	4940	0	3.1	0.05	0.001	1
1/13/2011	0.14	228	1703	0	2.2	0.06	0.000	1
1/17/2011	0.22	2188	16365	1	5.1	0.04	0.002	1
1/25/2011	0.31	7507	56156	2	4.6	0.07	0.005	1
2/1/2011	0.30	496978	3717397	137	4.3	0.07	0.374	1
2/4/2011	0.83	797208	5963117	220	9.8	0.09	0.217	1
2/4/2011	1.16	3138251	23474114	865	9.3	0.13	0.611	1
2/5/2011	0.20	454754	3401558	125	1.4	0.14	0.513	3
2/5/2011	0.19	949413	7101609	262	0.9	0.21	1.128	3
2/25/2011	0.17	4211	31500	1	2.2	0.08	0.006	1
2/28/2011	0.82	1991130	14893654	549	2.1	0.39	0.548	1
3/9/2011	0.76	1173969	8781288	323	4.4	0.17	0.349	1
3/19/2011	0.62	498981	3732381	137	2.2	0.29	0.182	1
3/26/2011	0.34	556380	4161719	153	0.7	0.51	0.370	1
3/26/2011	1.38	4286653	32064164	1181	9.9	0.14	0.701	1
3/27/2011	0.55	2032092	15200052	560	1.8	0.30	0.834	1
3/28/2011	0.20	139713	1045054	38	4.1	0.05	0.158	2
3/28/2011	0.65	1953432	14611671	538	0.5	1.30	0.679	2
3/30/2011	0.72	2036080	15229881	561	2.2	0.33	0.639	2
3/31/2011	0.17	269578	2016444	74	1.1	0.16	0.358	3
4/5/2011	0.62	1149137	8595545	317	1.9	0.32	0.419	1
4/22/2011	0.79	1953774	14614230	538	1.3	0.59	0.558	1
4/22/2011	1.03	4697744	35139126	1294	2.8	0.36	1.030	1
4/28/2011	1.68	0	0	0	2.7	0.63	0.000	1
5/6/2011	0.25	192078	1436742	53	1.0	0.25	0.173	1
5/13/2011	0.23	709616	5307931	195	0.3	0.69	0.697	1

5/14/2011	0.21	985369	7370558	271	2.2	0.10	1.060	1
5/16/2011	0.50	2528813	18915519	697	5.3	0.09	1.142	1
5/26/2011	0.95	4466473	33409215	1230	3.3	0.29	1.062	1
5/27/2011	0.71	4447801	33269551	1225	0.4	1.70	1.415	2
6/15/2011	0.45	1236654	9250175	341	0.9	0.49	0.621	1
6/18/2011	0.69	2379666	17799901	656	2.3	0.30	0.779	1
6/21/2011	0.11	220209	1647167	61	2.2	0.05	0.452	2
6/22/2011	0.28	593613	4440222	164	0.3	1.12	0.479	2
6/28/2011	0.19	387303	2897026	107	0.8	0.23	0.460	1
7/9/2011	0.30	418698	3131864	115	0.6	0.51	0.315	1
7/25/2011	0.18	163182	1220600	45	0.1	2.16	0.205	1
7/25/2011	1.21	4244645	31749945	1169	2.2	0.56	0.792	1
7/26/2011	0.26	817805	6117184	225	1.1	0.24	0.710	1
7/26/2011	0.86	3733490	27926503	1029	1.8	0.47	0.980	1
8/1/2011	0.44	273474	2045582	75	2.7	0.17	0.140	1
8/7/2011	1.01	2339454	17499116	644	0.7	1.52	0.523	2
8/9/2011	0.76	2276833	17030712	627	0.9	0.83	0.676	2
8/15/2011	0.17	201054	1503883	55	0.2	1.02	0.267	1
8/30/2011	0.67	1987065	14863249	547	0.3	2.01	0.670	1
9/6/2011	0.29	308669	2308842	85	0.7	0.44	0.240	1
9/22/2011	2.46	10887524	81438676	2999	0.8	2.95	0.999	1
9/23/2011	0.38	1156356	8649542	319	1.0	0.38	0.687	1
9/24/2011	0.72	4349894	32537204	1198	0.5	1.44	1.364	1
9/25/2011	0.62	1188267	8888238	327	1.2	0.53	0.433	3
9/26/2011	0.16	249598	1866991	69	0.6	0.27	0.352	3
9/27/2011	0.32	641060	4795127	177	0.5	0.64	0.452	3
9/28/2011	0.68	1820622	13618255	502	1.0	0.68	0.605	3
10/13/2011	0.19	198999	1488509	55	1.2	0.16	0.236	1
10/14/2011	0.19	120131	898583	33	0.3	0.76	0.143	1
10/19/2011	0.23	166589	1246089	46	2.2	0.11	0.164	1
10/19/2011	0.18	217033	1623405	60	0.9	0.20	0.272	1
11/4/2011	0.12	9824	73485	3	2.3	0.05	0.018	1
11/17/2011	0.52	685979	5131121	189	2.3	0.23	0.298	1
11/29/2011	0.65	731252	5469763	201	6.5	0.10	0.254	1
12/8/2011	0.14	29034	217176	8	1.0	0.14	0.047	1
12/22/2011	0.16	237	1774	0	1.3	0.13	0.000	1
12/26/2011	0.33	92937	695168	26	5.8	0.06	0.064	1
12/28/2011	0.89	1801803	13477484	496	9.6	0.09	0.457	1
1/11/2012	0.25	193664	1448606	53	0.7	0.38	0.175	1
1/12/2012	0.22	254752	1905542	70	1.5	0.15	0.261	2
1/12/2011	0.18	114793	858651	32	1.4	0.13	0.144	3
1/19/2012	0.23	169159	1265308	47	1.8	0.13	0.166	1
1/20/2012	0.28	299126	2237466	82	3.9	0.07	0.241	2
1/21/2012	0.72	3657975	27361656	1008	3.9	0.18	1.147	3

APPENDIX B

HOODS LANE DISCHARGE VOLUME SUMMARY

Event Date	Rainfall (in)	Volume Discharged (cf)	Volume (gal)	Volume (ac-in)	Duration (hr)	Intensity (in/hr)	Runoff Coefficient	AMC
1/30/2010	0.59	23152	173174	6.4	4.5	0.1	0.270	1
2/2/2010	0.23	9492	70999	2.6	11.6	0.0	0.284	1
2/5/2010	1.06	44911	335932	12.4	11.8	0.1	0.292	1
2/13/2010	0.15	8451	63213	2.3	1.6	0.1	0.388	1
2/15/2010	0.20	8391	62767	2.3	3.1	0.1	0.289	1
2/22/2010	0.37	17227	128854	4.7	6.3	0.1	0.321	1
3/2/2010	0.34	14929	111666	4.1	16.3	0.0	0.302	1
3/10/2010	0.44	18773	140423	5.2	5.8	0.1	0.294	1
3/11/2010	0.34	18360	137331	5.1	6.8	0.0	0.372	1
3/12/2010	1.13	67599	505644	18.6	3.4	0.3	0.412	1
3/21/2010	0.22	13687	102380	3.8	0.5	0.4	0.428	1
3/29/2010	0.39	23905	178809	6.6	0.8	0.5	0.422	1
4/8/2010	0.30	14083	105342	3.9	1.0	0.3	0.323	1
4/24/2010	0.24	1423	10640	0.4	14.6	0.0	0.041	1
4/25/2010	0.51	22594	169005	6.2	5.2	0.1	0.305	1
5/3/2010	0.63	22526	168493	6.2	7.3	0.1	0.246	1
5/31/2010	0.67	18840	140925	5.2	4.3	0.2	0.194	1
6/1/2010	0.18	2092	15649	0.6	0.6	0.3	0.080	1
6/2/2010	0.62	18787	140524	5.2	2.0	0.3	0.209	1
6/4/2010	0.18	49	368	0.0	0.5	0.4	0.002	2
6/9/2010	0.32	10053	75200	2.8	0.2	1.9	0.216	1
6/15/2010	0.29	10149	75915	2.8	2.8	0.1	0.241	1
6/18/2010	0.37	17460	130602	4.8	1.3	0.3	0.325	1
6/20/2010	0.24	4667	34911	1.3	2.3	0.1	0.134	1
6/25/2010	0.33	5134	38400	1.4	4.6	0.1	0.107	1
6/26/2010	0.46	17326	129598	4.8	0.5	0.9	0.259	1
6/27/2010	0.27	5868	43895	1.6	0.3	1.1	0.150	1
6/28/2010	1.60	92958	695325	25.6	1.5	1.1	0.400	1
6/29/2010	1.52	70577	527913	19.4	1.0	1.5	0.320	3
6/29/2010	0.49	31424	235051	8.7	1.4	0.3	0.442	3
7/12/2010	0.47	18034	134896	5.0	1.1	0.4	0.264	1
7/21/2010	0.32	4445	33246	1.2	1.4	0.2	0.096	1
7/26/2010	1.14	58356	436506	16.1	0.8	1.4	0.353	1
7/27/2010	0.64	15066	112696	4.2	3.1	0.2	0.162	1

7/28/2010	0.12	1499	11210	0.4	3.7	0.0	0.086	2
7/31/2010	0.45	13074	97794	3.6	0.8	0.5	0.200	2
7/31/2010	0.21	7018	52494	1.9	0.4	0.5	0.230	2
8/3/2010	3.08	160035	####	44.1	4.3	0.7	0.358	1
8/6/2011	0.68	42480	317751	11.7	2.9	0.2	0.430	1
8/14/2010	0.84	23966	179267	6.6	3.0	0.3	0.196	1
8/15/2010	0.39	100043	748318	27.6	4.5	0.1	1.767	1
8/15/2010	1.53	98320	735437	27.1	0.7	2.3	0.443	1
8/16/2010	0.42	19223	143791	5.3	5.5	0.1	0.315	1
8/17/2010	0.63	50974	381289	14.0	0.6	1.1	0.557	2
8/20/2010	0.92	57200	427854	15.8	0.9	1.0	0.428	2
8/23/2010	0.14	308	2304	0.1	0.3	0.6	0.015	1
8/24/2010	1.29	57588	430760	15.9	0.3	3.9	0.307	1
9/17/2010	0.66	44729	334574	12.3	0.8	0.8	0.467	1
9/26/2010	0.31	12458	93184	3.4	0.3	1.2	0.277	1
9/26/2010	0.86	50372	376783	13.9	4.7	0.2	0.403	1
9/26/2010	0.23	9936	74322	2.7	1.7	0.1	0.298	1
9/27/2010	0.25	10909	81596	3.0	1.1	0.2	0.301	1
10/25/2010	0.29	11116	83151	3.1	1.8	0.2	0.264	1
10/27/2010	0.68	35184	263174	9.7	1.7	0.4	0.356	1
10/28/2010	0.14	8489	63494	2.3	1.0	0.1	0.418	2
11/4/2010	0.36	12636	94519	3.5	2.8	0.1	0.242	1
11/4/2010	0.56	32143	240431	8.9	2.2	0.3	0.395	1
11/16/2010	0.16	4564	34138	1.3	3.3	0.0	0.196	1
11/16/2010	0.22	9825	73489	2.7	0.2	1.3	0.308	1
12/1/2010	0.49	20773	155379	5.7	3.1	0.2	0.292	1
1/1/2011	0.34	15084	112828	4.2	4.3	0.1	0.306	1
1/5/2011	0.32	16237	121453	4.5	12.1	0.0	0.349	1
1/12/2011	0.15	6020	45031	1.7	3.1	0.0	0.276	1
1/13/2011	0.14	1914	14316	0.5	2.2	0.1	0.094	1
1/17/2011	0.22	18650	139504	5.1	5.1	0.0	0.584	1
1/25/2011	0.31	13970	104493	3.8	4.6	0.1	0.310	1
2/1/2011	0.30	24265	181501	6.7	4.3	0.1	0.557	1
2/4/2011	0.83	37372	279541	10.3	9.8	0.1	0.310	1
2/4/2011	1.16	78410	586506	21.6	9.3	0.1	0.466	1
2/5/2011	0.20	11191	83707	3.1	1.4	0.1	0.385	3
2/5/2011	0.19	18818	140757	5.2	0.9	0.2	0.682	3
2/25/2011	0.17	5805	43420	1.6	2.2	0.1	0.235	1
2/28/2011	0.82	34810	260376	9.6	2.1	0.4	0.292	1
3/9/2011	0.76	41904	313439	11.5	4.4	0.2	0.380	1

3/19/2011	0.62	17197	128634	4.7	2.2	0.3	0.191	1
3/26/2011	0.34	15876	118754	4.4	0.7	0.5	0.322	1
3/26/2011	1.38	26579	198807	7.3	9.9	0.1	0.133	1
3/27/2011	0.55	29482	220529	8.1	1.8	0.3	0.369	1
3/28/2011	0.20	7972	59633	2.2	4.1	0.0	0.275	2
3/28/2011	0.65	45288	338752	12.5	0.5	1.3	0.480	2
3/30/2011	0.72	34006	254366	9.4	2.2	0.3	0.325	2
3/31/2011	0.17	13253	99133	3.7	1.1	0.2	0.537	3
4/5/2011	0.62	29054	217322	8.0	1.9	0.3	0.323	1
4/22/2011	0.79	35549	265908	9.8	1.3	0.6	0.310	1
4/22/2011	1.03	52459	392395	14.5	2.8	0.4	0.351	1
4/28/2011	1.68	80287	600548	22.1	2.7	0.6	0.329	1
5/6/2011	0.25	6927	51813	1.9	1.0	0.3	0.191	1
5/13/2011	0.23	5245	39233	1.4	0.3	0.7	0.157	1
5/14/2011	0.21	6105	45665	1.7	2.2	0.1	0.200	1
5/16/2011	0.50	20424	152769	5.6	5.3	0.1	0.281	1
5/26/2011	0.95	53902	403185	14.8	3.3	0.3	0.391	1
5/27/2011	0.71	38206	285778	10.5	0.4	1.7	0.371	2
6/15/2011	0.45	ND	ND	ND	0.9	0.5	ND	1
6/18/2011	0.69	ND	ND	ND	2.3	0.3	ND	1
6/21/2011	0.11	ND	ND	ND	2.2	0.1	ND	2
6/22/2011	0.28	ND	ND	ND	0.3	1.1	ND	2
6/28/2011	0.19	ND	ND	ND	0.8	0.2	ND	1
7/9/2011	0.30	6849	51231	1.9	0.6	0.5	0.157	1
7/25/2011	0.18	1255	9388	0.3	0.1	2.2	0.048	1
7/25/2011	1.21	68725	514061	18.9	2.2	0.6	0.391	1
7/26/2011	0.26	16090	120357	4.4	1.1	0.2	0.426	1
7/26/2011	0.86	58278	435920	16.1	1.8	0.5	0.467	1
8/1/2011	0.44	9209	68885	2.5	2.7	0.2	0.144	1
8/7/2011	1.01	34455	257721	9.5	0.7	1.5	0.235	2
8/9/2011	0.76	46829	350283	12.9	0.9	0.8	0.424	2
8/15/2011	0.17	7622	57015	2.1	0.2	1.0	0.309	1
8/30/2011	0.67	50547	378088	13.9	0.3	2.0	0.520	1
9/6/2011	0.29	10624	79468	2.9	0.7	0.4	0.252	1
9/22/2011	2.46	177692	1329138	49.0	0.8	3.0	0.497	1
9/23/2011	0.38	31012	231970	8.5	1.0	0.4	0.562	1
9/24/2011	0.72	39820	297855	11.0	0.5	1.4	0.381	1
9/25/2011	0.62	36357	271953	10.0	1.2	0.5	0.404	3
9/26/2011	0.16	7562	56562	2.1	0.6	0.3	0.325	3
9/27/2011	0.32	13499	100976	3.7	0.5	0.6	0.291	3

9/28/2011	0.68	32025	239551	8.8	1.0	0.7	0.324	3
10/13/2011	0.19	8742	65389	2.4	1.2	0.2	0.317	1
10/14/2011	0.19	8528	63790	2.3	0.3	0.8	0.309	1
10/19/2011	0.23	8176	61154	2.3	2.2	0.1	0.245	1
10/19/2011	0.18	11749	87882	3.2	0.9	0.2	0.450	1
11/4/2011	0.12	2707	20250	0.7	2.3	0.1	0.155	1
11/17/2011	0.52	21726	162511	6.0	2.3	0.2	0.288	1
11/29/2011	0.65	33302	249097	9.2	6.5	0.1	0.353	1
12/8/2011	0.14	7402	55366	2.0	1.0	0.1	0.364	1
12/22/2011	0.16	2820	21095	0.8	1.3	0.1	0.121	1
12/26/2011	0.33	ND	ND	ND	5.8	0.1	ND	1
12/28/2011	0.71	ND	ND	ND	9.6	0.1	ND	1

APPENDIX C

SUMTER STREET DISCHARGE VOLUME SUMMARY

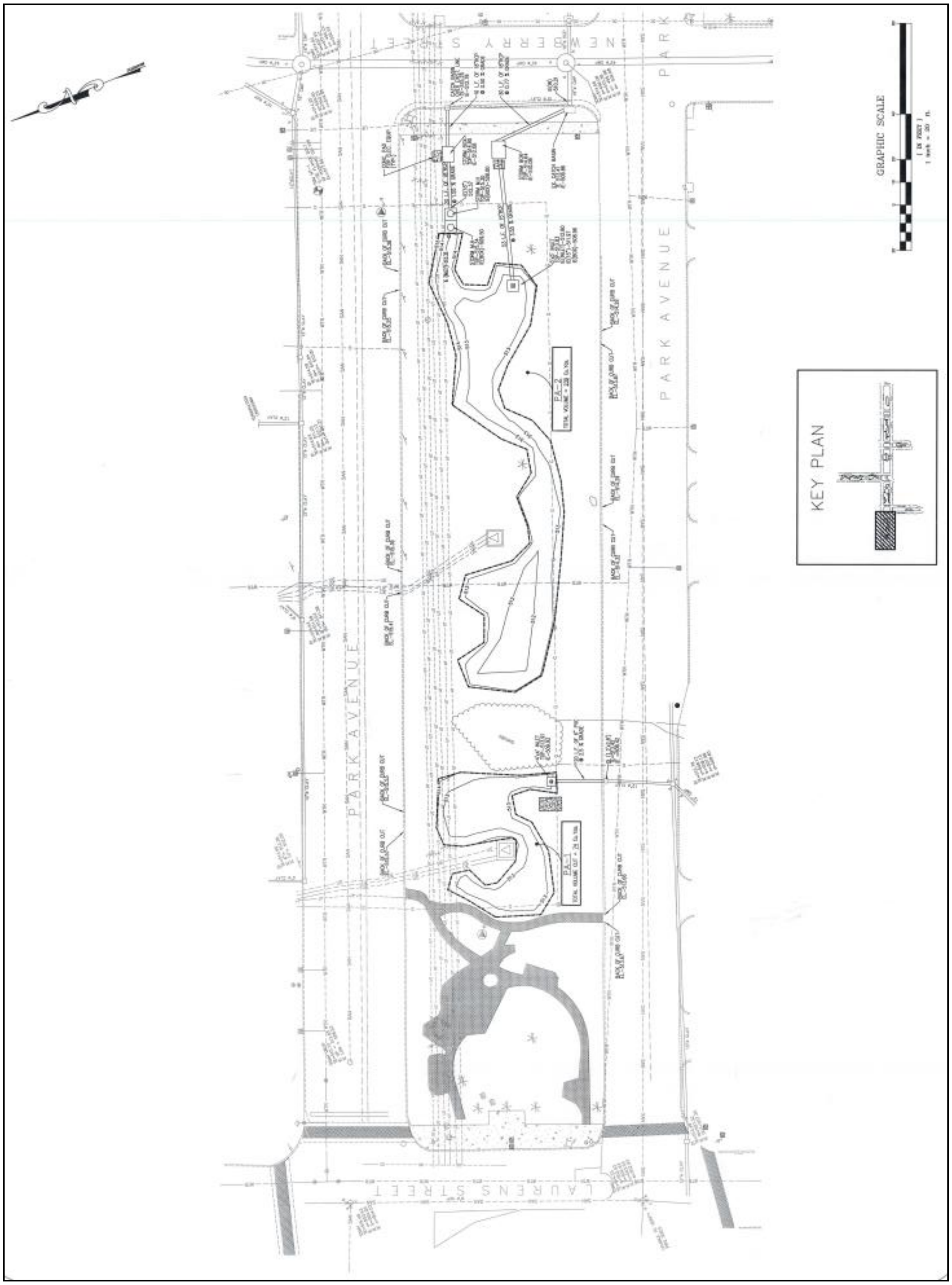
Event Date	Rainfall (in)	Volume Discharged (cf)	Volume (gal)	Volume (ac-in)	Duration (hr)	Intensity (in/hr)	Runoff Coefficient	AMC
6/18/2010	0.37	61765	462003	17	1.3	0.30	0.14	1
6/20/2010	0.24	28107	210242	8	2.3	0.10	0.09	1
6/25/2010	0.33	14765	110445	4	4.6	0.07	0.04	1
6/26/2010	0.46	33117	247712	9	0.5	0.92	0.06	1
6/27/2010	0.27	12895	96457	4	0.3	1.08	0.04	1
6/28/2010	1.6	668718	5002013	184	1.5	1.07	0.34	1
6/29/2010	1.52	474505	3549300	131	1.0	1.52	0.25	3
6/29/2010	0.49	241390	1805595	66	1.4	0.35	0.40	3
7/12/2010	0.47	42829	320365	12	1.1	0.43	0.07	1
7/21/2010	0.32	13922	104134	4	1.4	0.23	0.04	1
7/26/2010	1.14	87417	653880	24	0.8	1.37	0.06	1
7/27/2010	0.64	89906	672497	25	3.1	0.21	0.11	1
7/28/2010	0.12	5363	40113	1	3.7	0.03	0.04	2
7/31/2010	0.45	130252	974286	36	0.8	0.54	0.23	2
7/31/2010	0.45	14218	106352	4	0.4	0.50	0.03	2
8/3/2010	3.08	731510	5471693	202	4.3	0.72	0.19	1
8/6/2010	0.68	216027	1615885	60	2.9	0.23	0.26	1
8/14/2010	0.84	36018	269418	10	3.0	0.28	0.03	1
8/15/2010	0.39	15	113	0	4.5	0.09	0.00	1
8/15/2010	1.53	1144363	8559833	315	0.7	2.30	0.61	1
8/16/2011	0.42	173384	1296909	48	5.5	0.08	0.33	1
8/17/2010	0.63	193672	1448668	53	0.6	1.08	0.25	1
8/20/2010	0.92	210977	1578106	58	0.9	1.00	0.19	1
8/23/2010	0.14	514	3842	0	0.3	0.56	0.00	1
8/24/2010	1.29	462103	3456532	127	0.3	3.87	0.29	1
9/17/2010	0.66	108681	812935	30	0.8	0.79	0.13	1
9/26/2010	0.31	45474	340149	13	0.3	1.24	0.12	1
9/26/2010	0.86	184518	1380197	51	4.7	0.18	0.17	1
9/26/2010	0.23	25356	189660	7	1.7	0.14	0.09	1
9/27/2010	0.25	3072	22979	1	1.1	0.23	0.01	1
10/25/2010	0.29	8173	61135	2	1.8	0.16	0.02	1
10/27/2010	0.68	168285	1258772	46	1.7	0.41	0.20	1
10/28/2010	0.68	17930	134118	5	1.0	0.14	0.02	1
11/4/2010	0.36	25397	189972	7	2.8	0.13	0.06	1

11/4/2010	0.56	84949	635418	23	2.2	0.26	0.12	1
11/16/2010	0.16	2281	17060	1	3.3	0.05	0.01	1
11/16/2010	0.22	13655	102137	4	0.2	1.32	0.05	1
12/1/2010	0.49	50714	379344	14	3.1	0.16	0.08	1
1/1/2011	0.34	18276	136704	5	4.3	0.08	0.04	1
1/5/2011	0.32	12842	96060	4	12.1	0.03	0.03	1
1/12/2011	0.15	215	1610	0	3.1	0.05	0.00	1
1/13/2011	0.14	0	0	0	2.2	0.06	0.00	1
1/17/2011	0.22	13347	99839	4	5.1	0.04	0.05	1
1/25/2011	0.31	18887	141274	5	4.6	0.07	0.05	1
2/1/2011	0.3	50854	380391	14	4.3	0.07	0.14	1
2/4/2011	0.83	79495	594623	22	9.8	0.09	0.08	1
2/4/2011	1.16	212159	1586953	58	9.3	0.13	0.15	1
2/5/2011	0.2	36653	274164	10	1.4	0.14	0.15	3
2/5/2011	0.19	182576	1365670	50	0.9	0.21	0.78	3
2/25/2011	0.17	4787	35804	1	2.2	0.08	0.02	1
2/28/2011	0.82	137371	1027537	38	2.1	0.39	0.14	1
3/9/2011	0.76	77863	582416	21	4.4	0.17	0.08	1
3/19/2011	0.62	37550	280877	10	2.2	0.29	0.05	1
3/26/2011	0.34	32455	242765	9	0.7	0.51	0.08	1
3/26/2011	1.38	86059	643722	24	9.9	0.14	0.05	1
3/27/2011	0.55	190048	1421557	52	1.8	0.30	0.28	1
3/28/2011	0.2	15850	118555	4	4.1	0.05	0.06	1
3/28/2011	0.65	157195	1175822	43	0.5	1.30	0.20	1
3/30/2011	0.72	118288	884793	33	2.2	0.33	0.13	1
3/31/2011	0.17	27766	207693	8	1.1	0.16	0.13	2
4/5/2011	0.62	63550	475357	18	1.9	0.32	0.08	1
4/22/2011	0.79	115593	864639	32	1.3	0.59	0.12	1
4/22/2011	1.03	392994	2939597	108	2.8	0.36	0.31	1
4/28/2011	1.68	433199	3240331	119	2.7	0.63	0.21	1
5/6/2011	0.25	12138	90794	3	1.0	0.25	0.04	1
5/13/2011	0.23	6224	46559	2	0.3	0.69	0.02	1
5/14/2011	0.21	12290	91932	3	2.2	0.10	0.05	1
5/16/2011	0.5	48709	364346	13	5.3	0.09	0.08	1
5/26/2011	0.95	151275	1131537	42	3.3	0.29	0.13	1
5/27/2011	0.71	238580	1784581	66	0.4	1.70	0.27	1
6/15/2011	0.45	21971	164341	6	0.9	0.49	0.04	1
6/18/2011	0.69	78907	590226	22	2.3	0.30	0.09	1
6/21/2011	0.11	2785	20832	1	2.2	0.05	0.02	1
6/22/2011	0.28	13410	100309	4	0.3	1.12	0.04	1

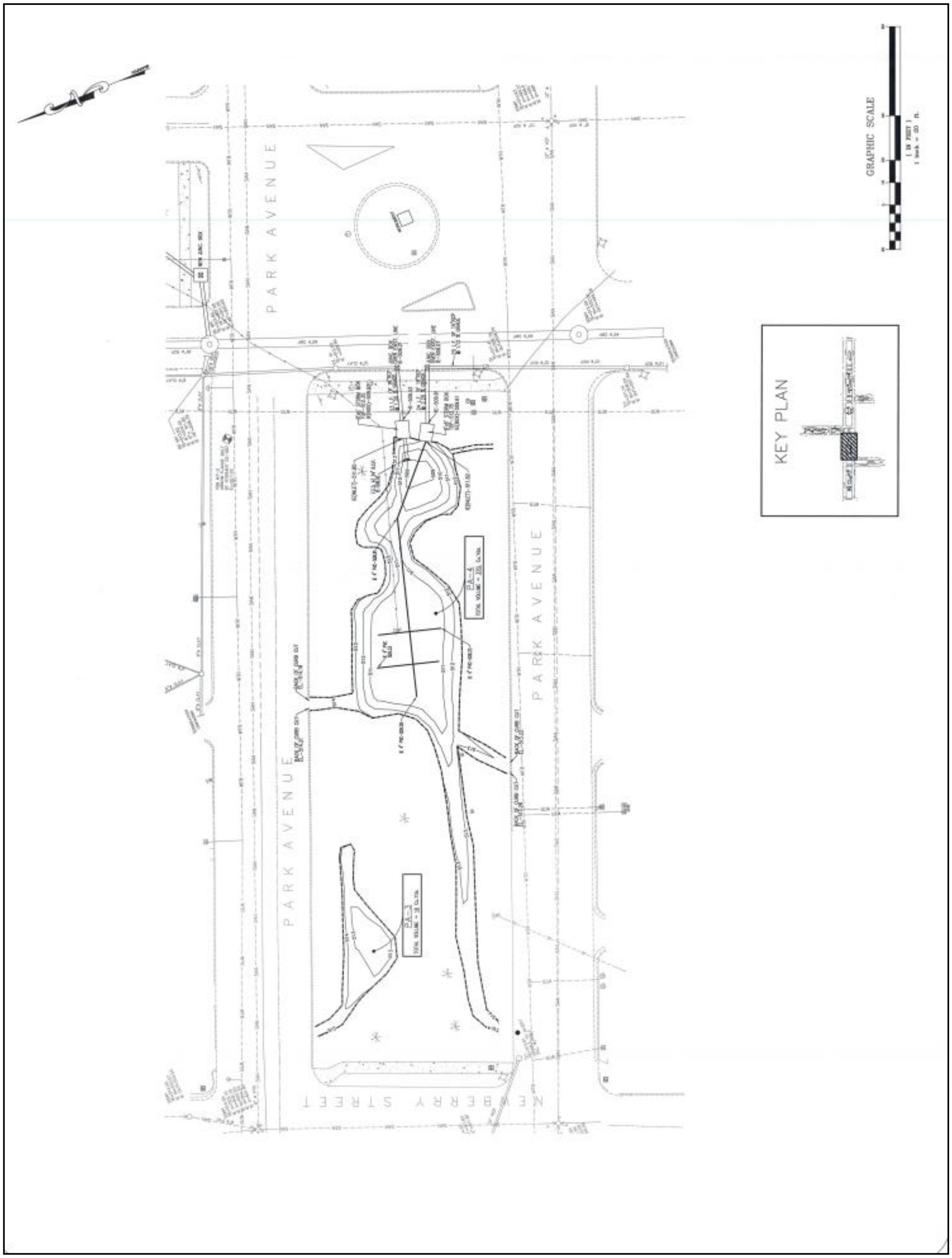
6/28/2011	0.19	15611	116768	4	0.8	0.23	0.07	1
7/9/2011	0.3	5837	43664	2	0.6	0.51	0.02	1
7/25/2011	0.18	4819	36047	1	0.1	2.16	0.02	1
7/25/2011	1.21	295304	2208872	81	2.2	0.56	0.20	1
7/26/2011	1.13	131391	982807	36	1.1	0.24	0.09	1
7/26/2011	0.86	305151	2282530	84	1.8	0.47	0.29	1
8/1/2011	0.44	28275	211498	8	2.7	0.17	0.05	1
8/7/2011	1.01	162641	1216553	45	0.7	1.52	0.13	1
8/9/2011	0.76	121804	911091	34	0.9	0.83	0.13	1
8/15/2011	0.17	20039	149893	6	0.2	1.02	0.10	1
8/30/2011	0.67	118227	884340	33	0.3	2.01	0.14	1
9/6/2011	0.29	11052	82672	3	0.7	0.44	0.03	1
9/22/2011	2.46	522093	3905255	144	0.8	2.95	0.17	1
9/23/2011	0.38	65554	490341	18	1.0	0.38	0.14	3
9/24/2011	0.74	328575	2457744	91	0.5	1.44	0.36	3
9/25/2011	0.62	35707	267088	10	1.2	0.53	0.05	3
9/26/2011	0.16	24288	181676	7	0.6	0.27	0.12	3
9/27/2011	0.32	30907	231186	9	0.5	0.64	0.08	3
9/28/2011	0.68	89067	666224	25	1.0	0.68	0.11	1
10/13/2011	0.19	12653	94648	3	1.2	0.16	0.05	1
10/14/2011	0.19	6192	46318	2	0.3	0.76	0.03	1
10/19/2011	0.23	11915	89123	3	2.2	0.11	0.04	1
10/19/2011	0.18	24656	184425	7	0.9	0.20	0.11	1
11/4/2011	0.12	1845	13801	1	2.3	0.05	0.01	1
11/17/2011	0.52	32204	240886	9	2.3	0.23	0.05	1
11/29/2011	0.65	37861	283199	10	6.5	0.10	0.05	1
12/8/2011	0.14	5396	40365	1	1.0	0.14	0.03	1
12/22/2011	0.16	769	5749	0	1.3	0.13	0.00	1
12/26/2011	0.33	16018	119816	4	5.8	0.06	0.04	1
12/28/2011	0.89	103480	774031	29	9.6	0.09	0.09	1

APPENDIX D

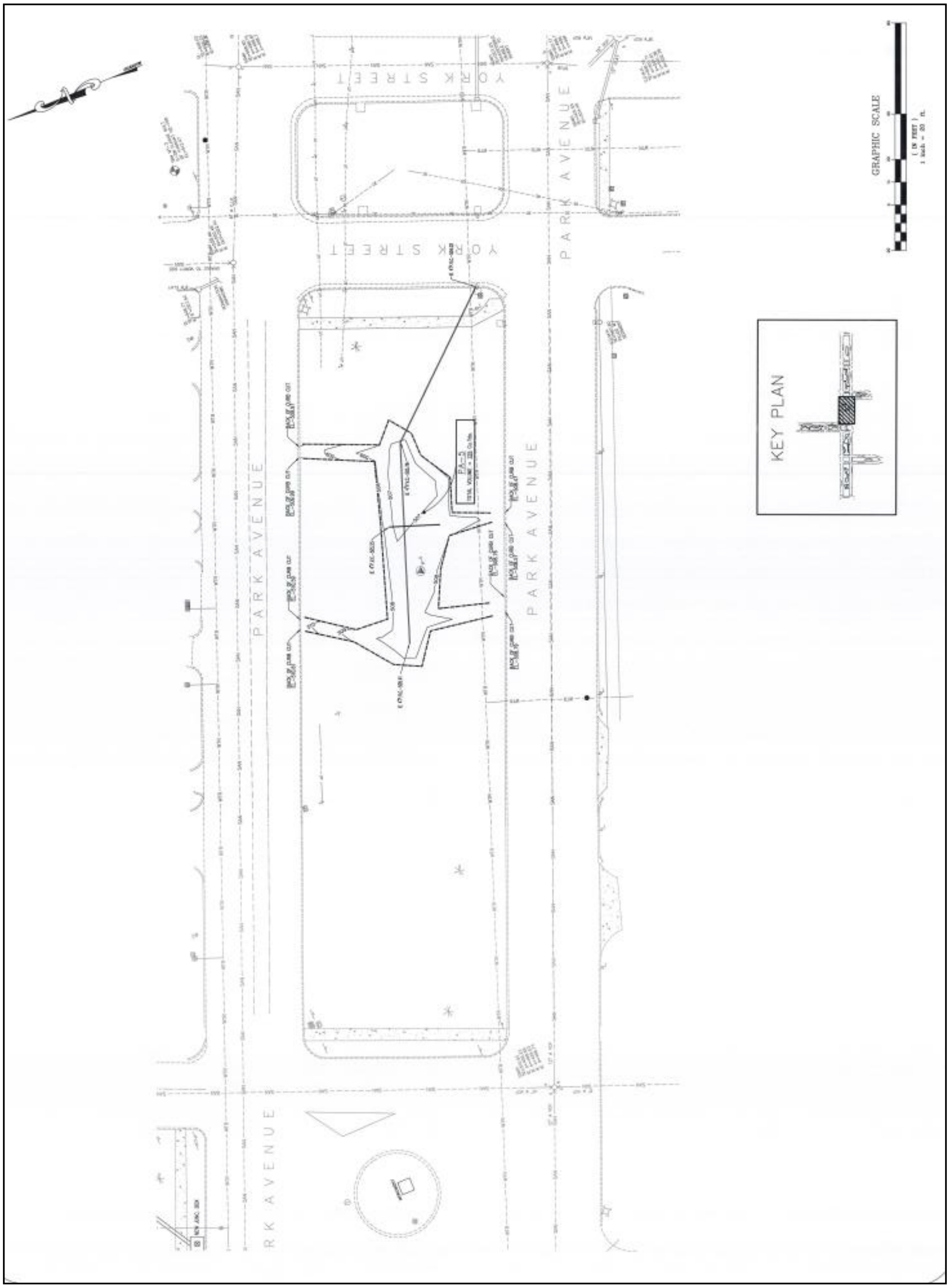
AS-BUILT PLANS OF BIORETENTION CELLS



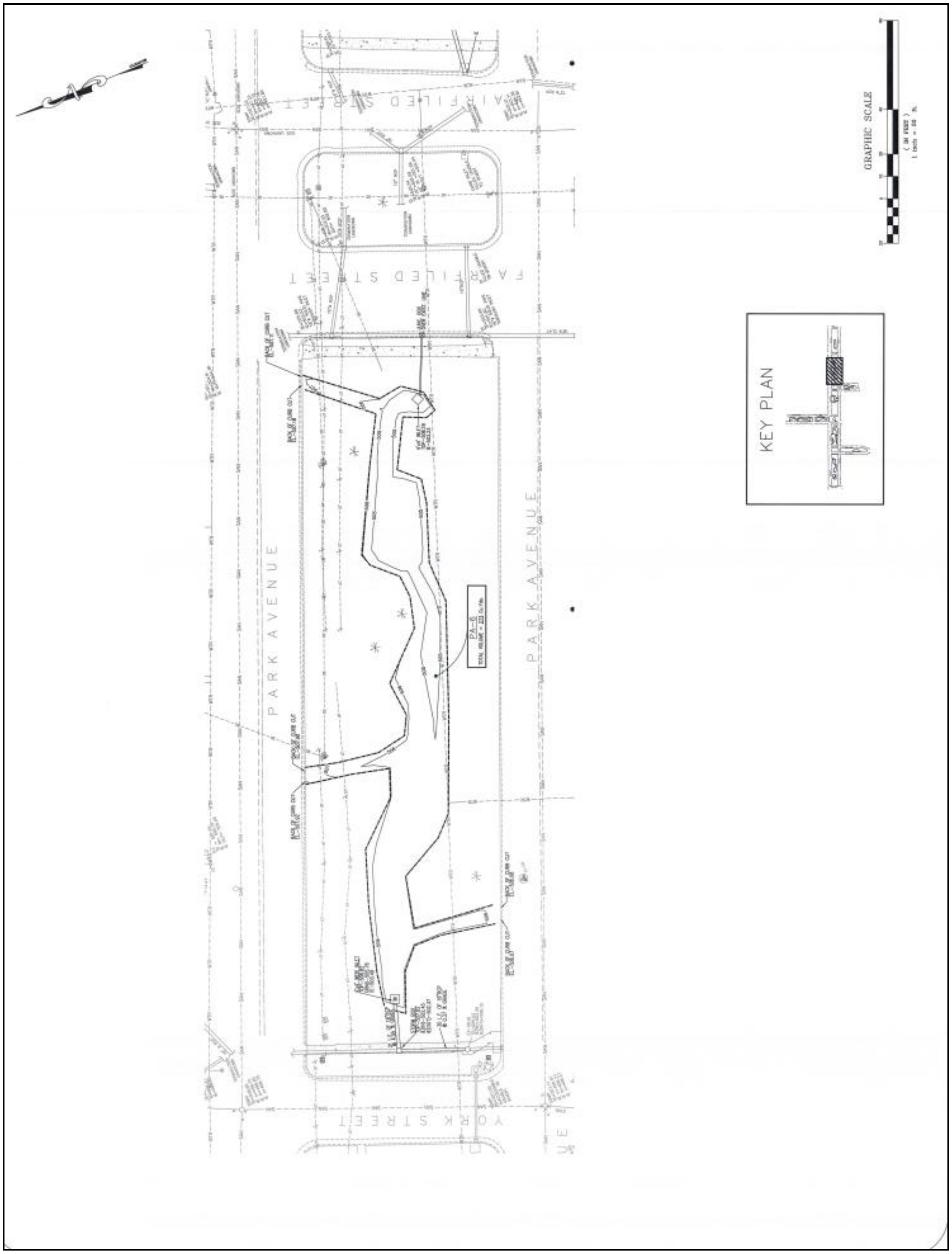
Appendix D-1: PNL Bioretention Cell (H. Lawson Graham & Associates, INC.)



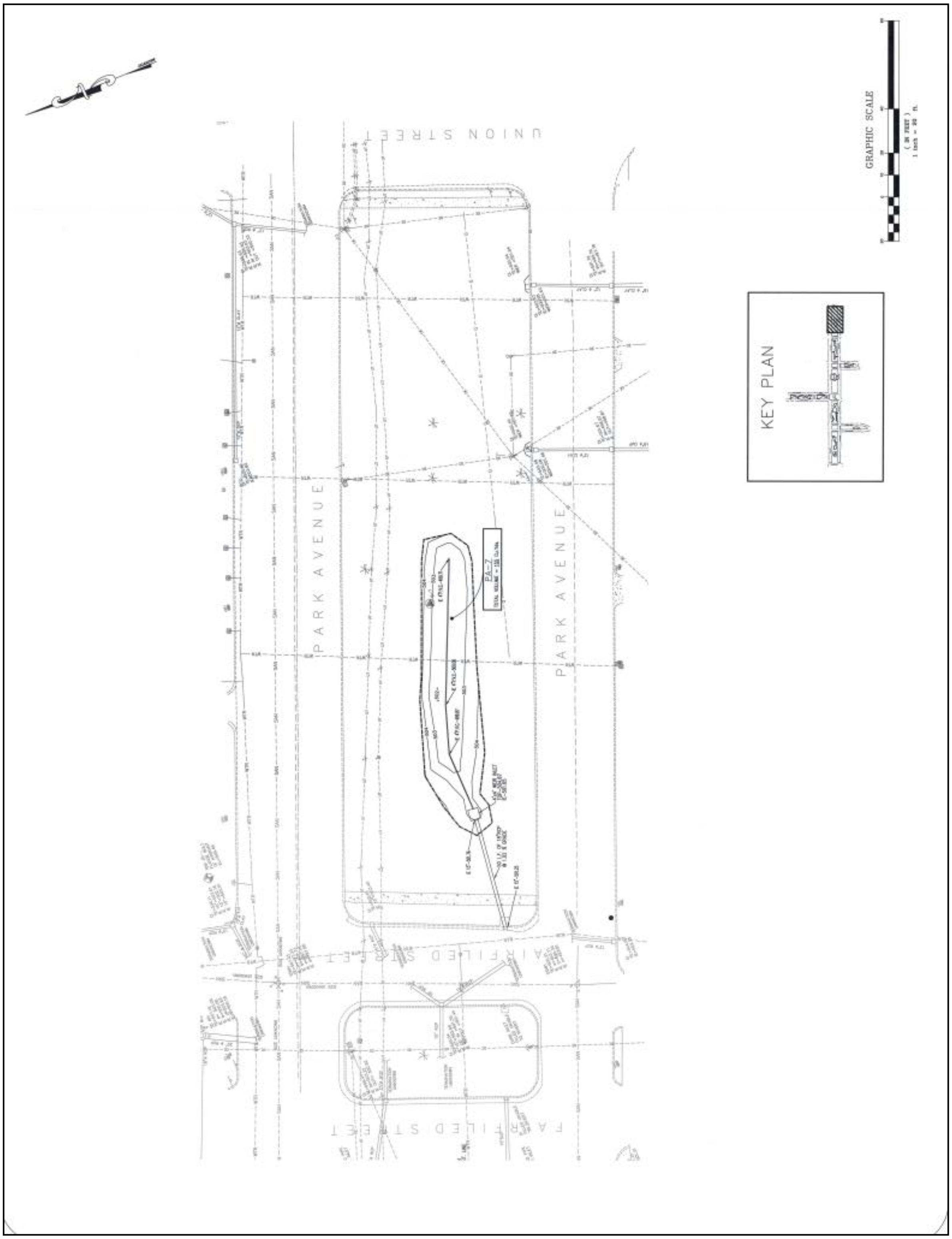
Appendix D-2: PCN Bioretention Cell (H. Lawson Graham & Associates, INC.)



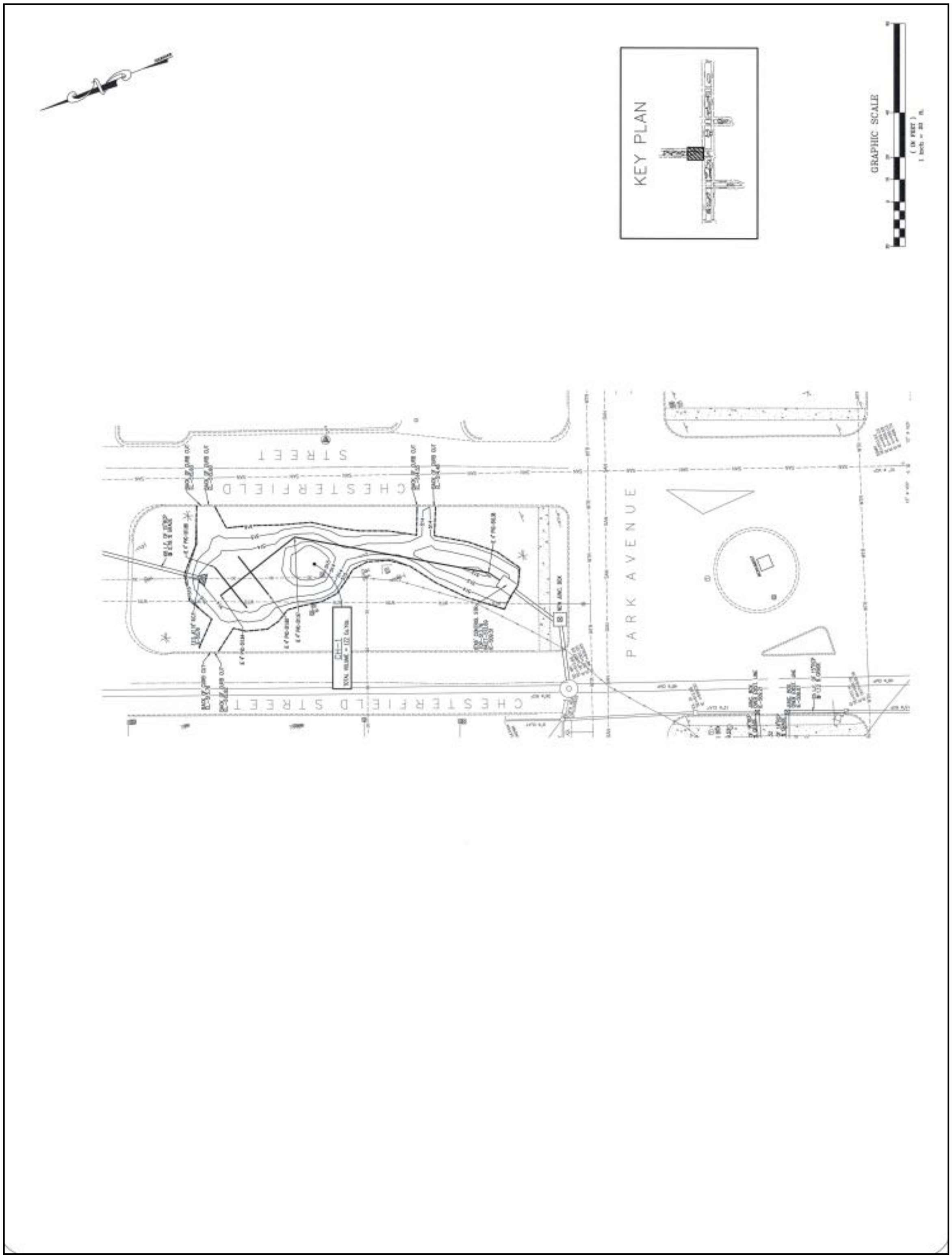
Appendix D-3: PCY Bioretention Cell (H. Lawson Graham & Associates, INC.)



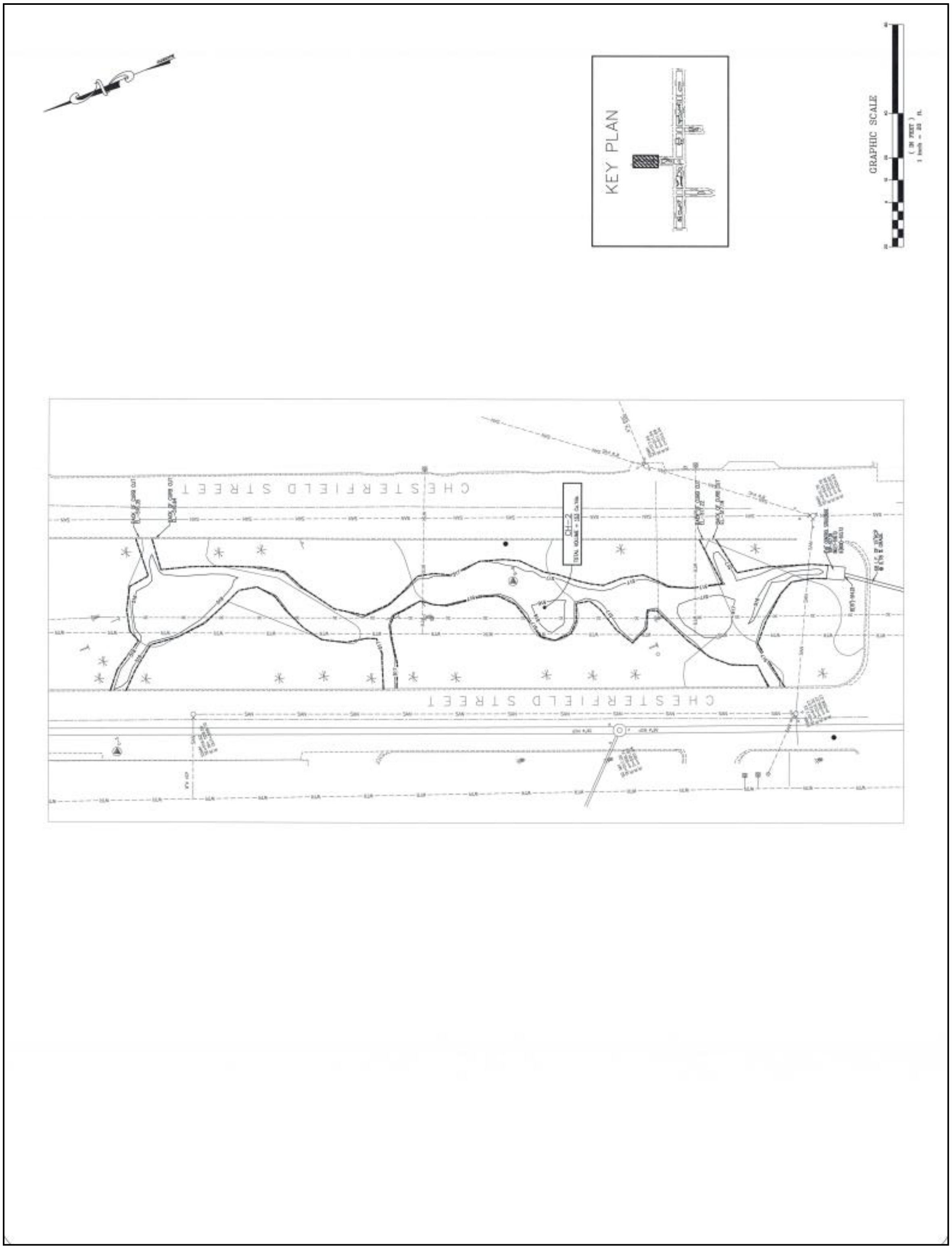
Appendix D-4: PFY Bioretention Cell (H. Lawson Graham & Associates, INC.)



Appendix D-5: PUF Bioretention Cell (H. Lawson Graham & Associates, INC.)



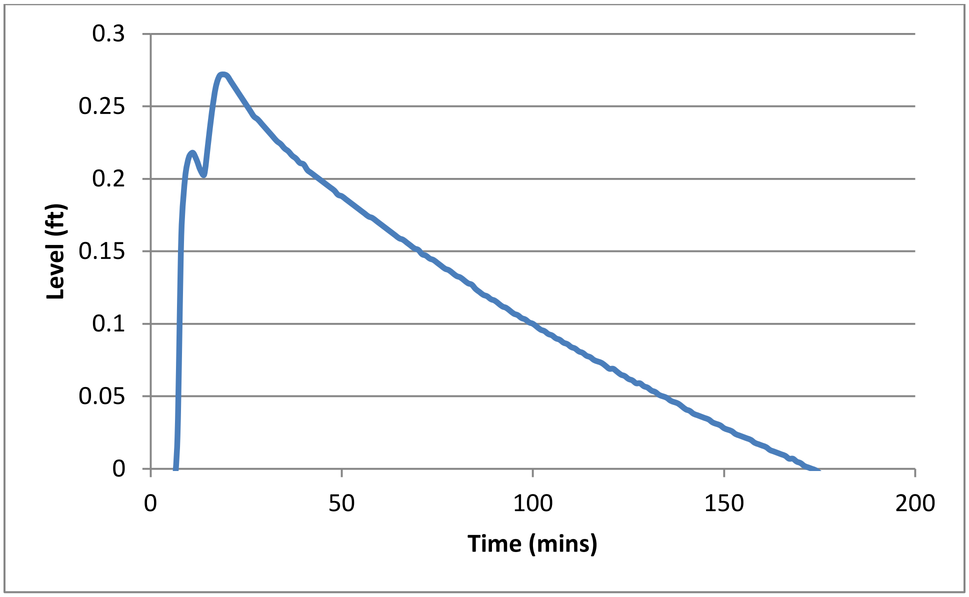
Appendix D-6: CRP-S Bioretention Cell (H. Lawson Graham & Associates, INC.)



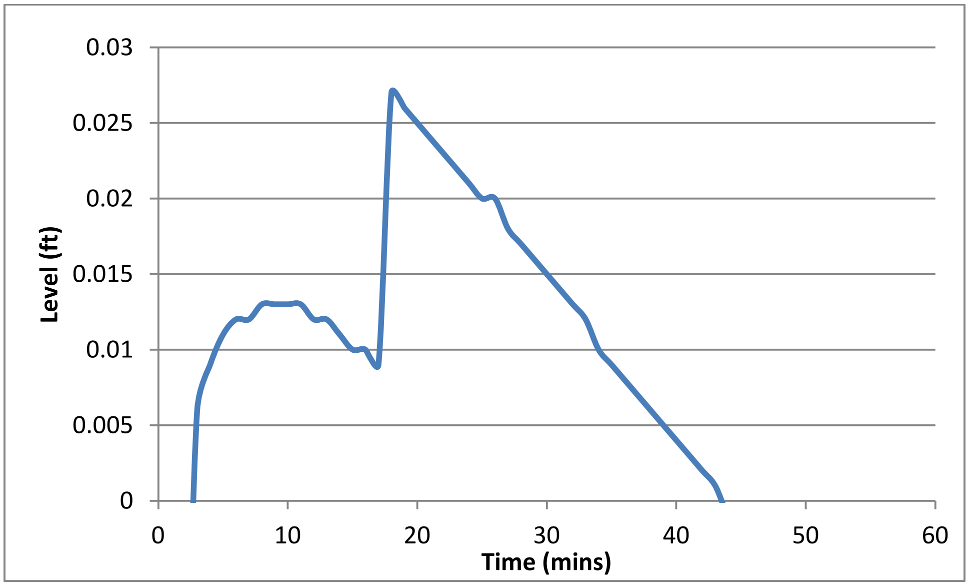
Appendix D-7: CRP-N Bioretention Cell (H. Lawson Graham & Associates, INC.)

APPENDIX E

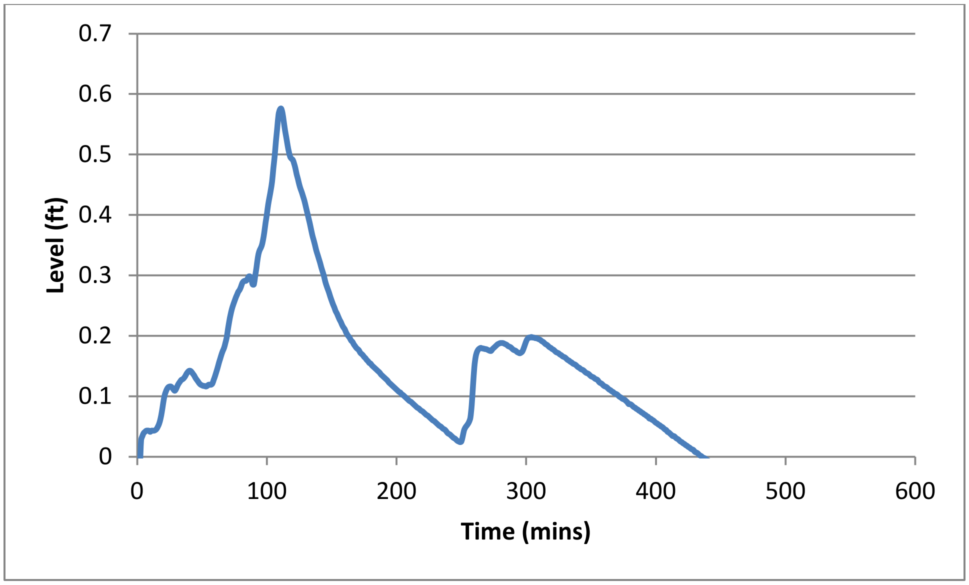
PORQUS ASPHALT LEVEL DATA



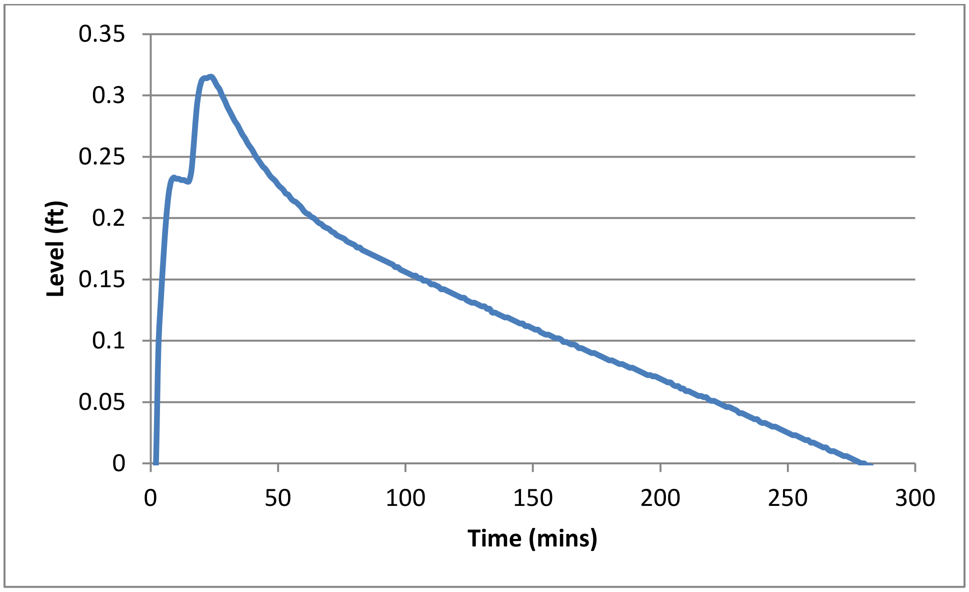
E-1: Storm event on 8/30/11 at PNL-N.



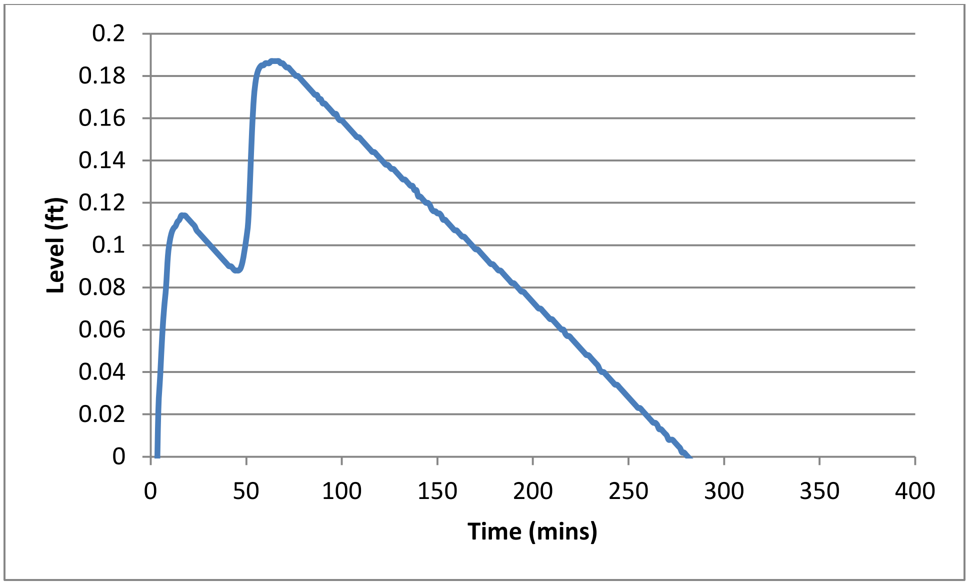
E-2: Storm event on 9/6/11 at PNL-N.



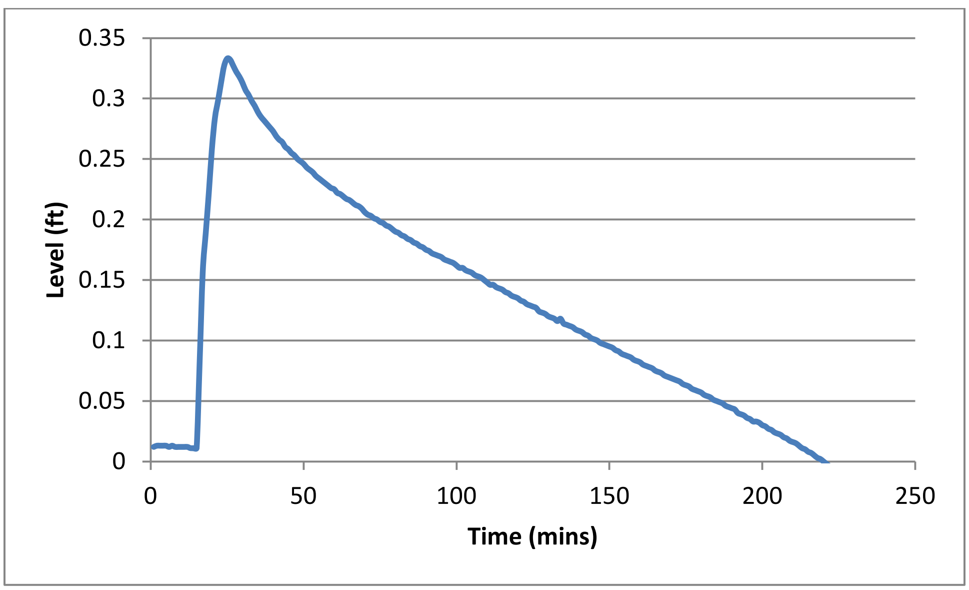
E-3: Storm event on 9/22/11 at PNL-N.



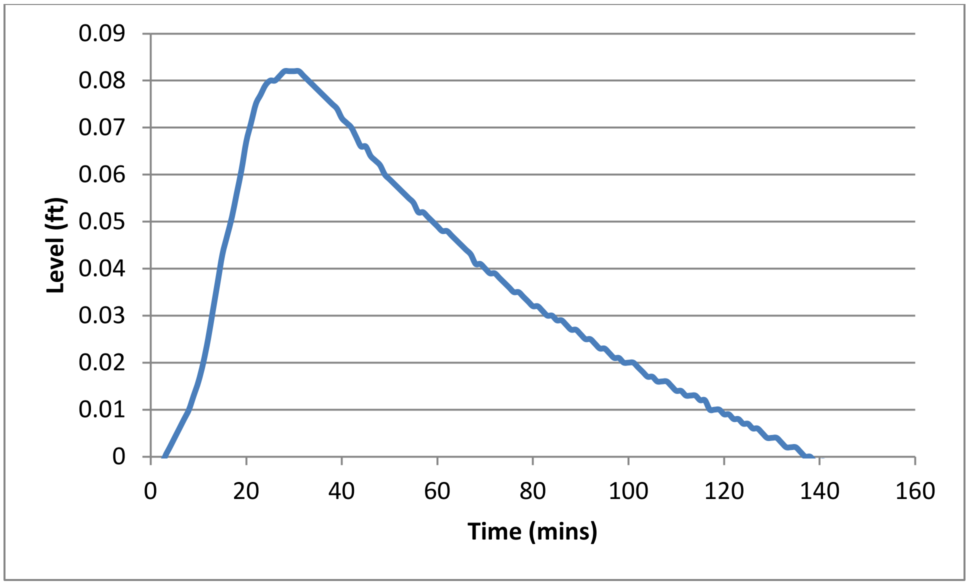
E-4: Storm event on 9/24/11 at PNL-N.



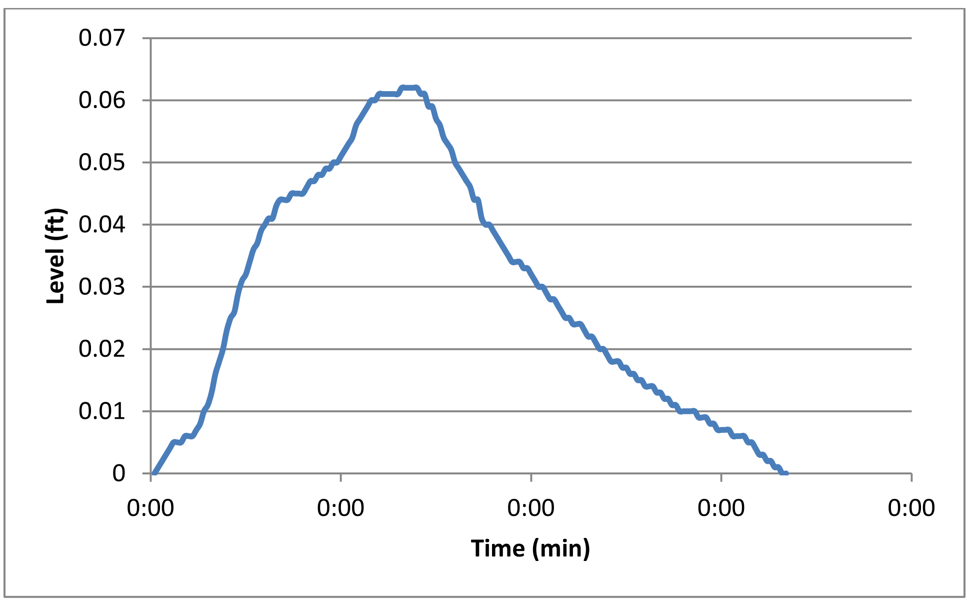
E-5: Storm event on 9/25/11 at PNL-N.



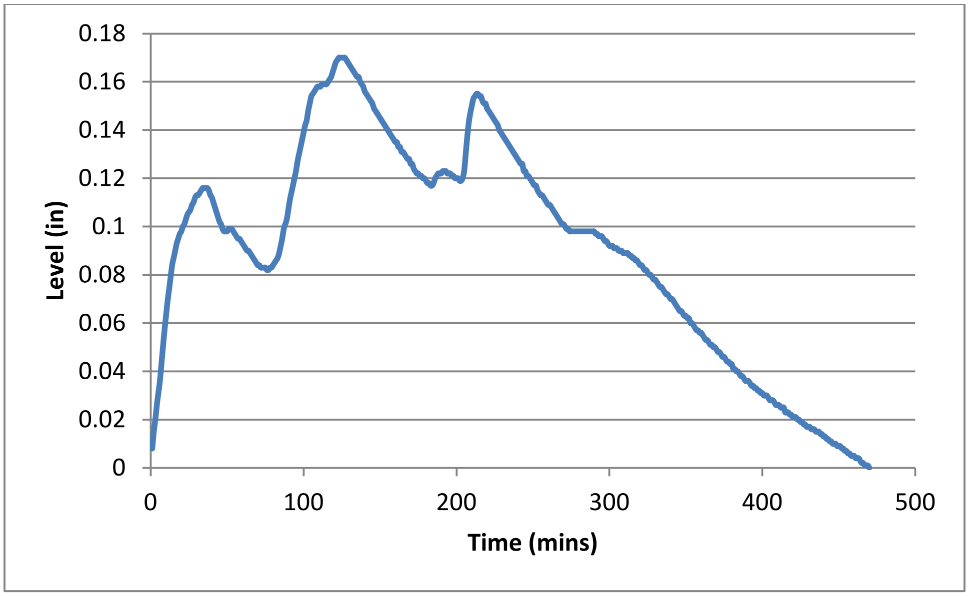
E-6: Storm event on 9/28/11 at PNL-N.



E-7: Storm event on 10/13/11 at PNL-N.



E-8: Storm event on 11/29/11 at PNL-N.

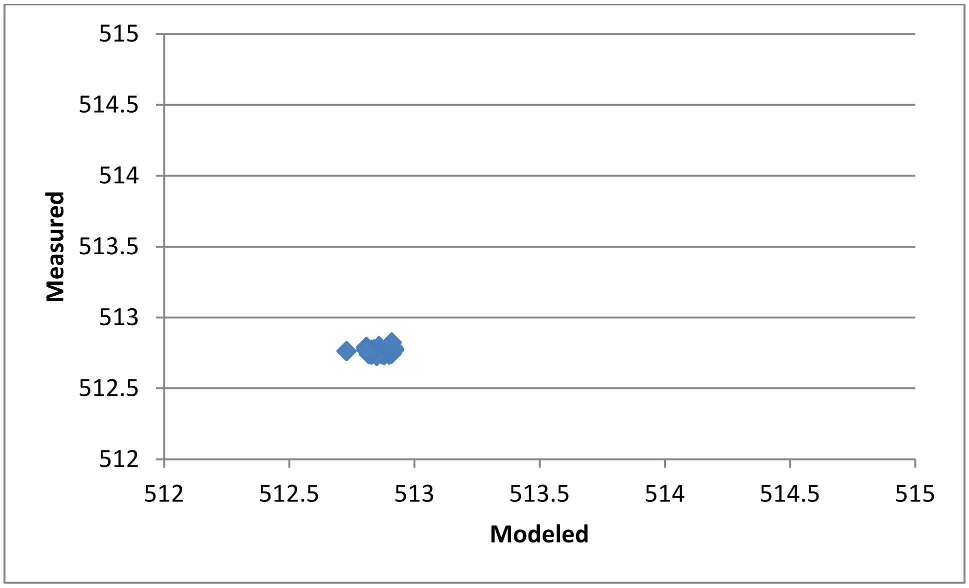


E-9: Storm event on 12/28/11 at PNL-N

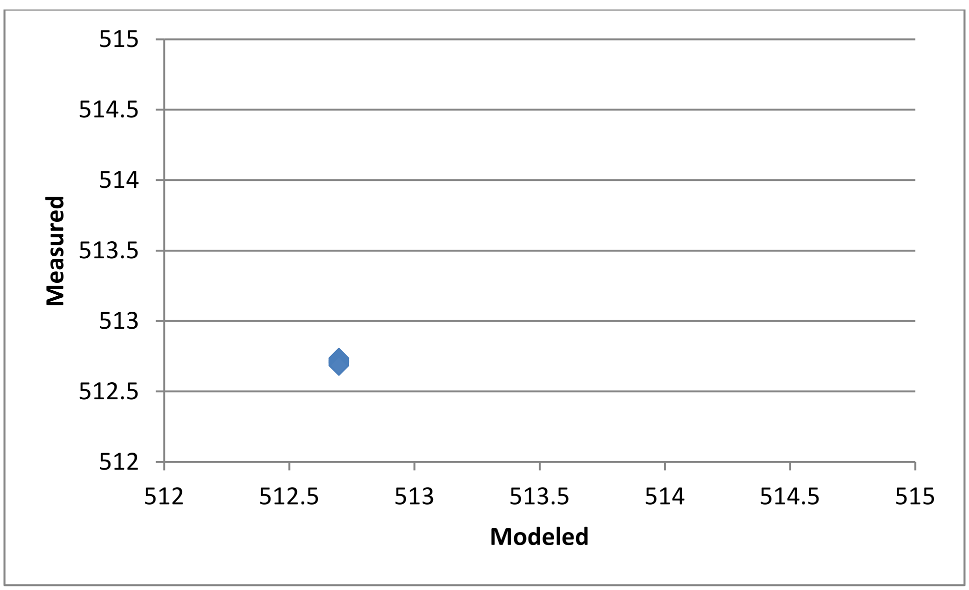
APPENDIX F

STORM EVENT SEPARTATIONS FOR THE MODELED AND MEASURED

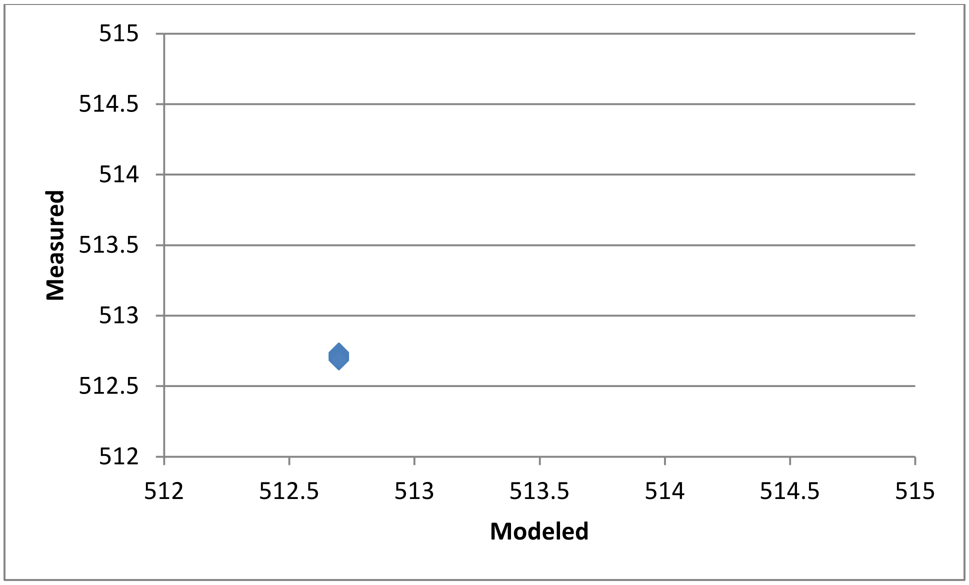
LEVELS IN THE PCN AND CRP BIORETENTION CELLS.



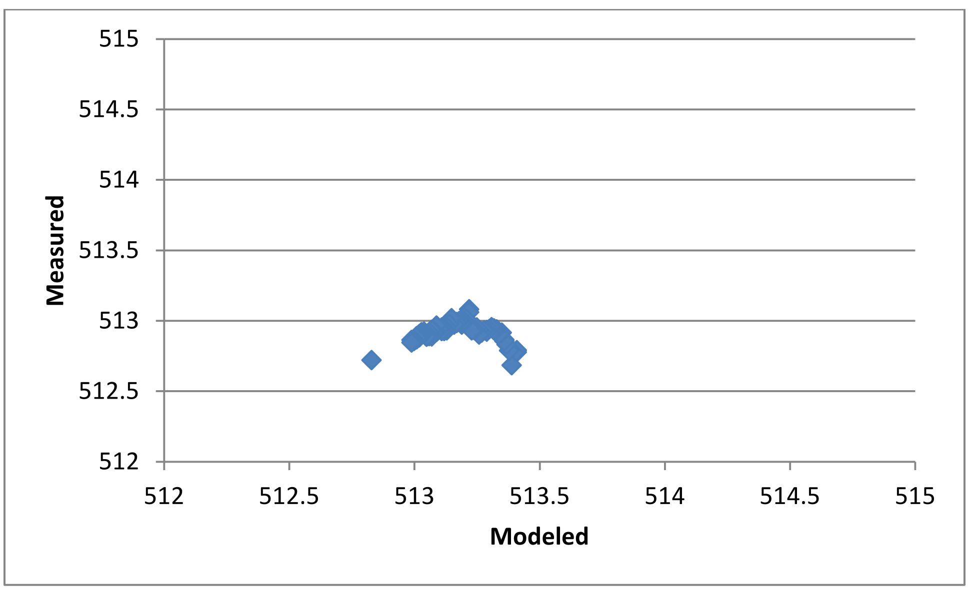
Appendix F-1: Storm event on 3/13/12 in CRP bioretention cell.



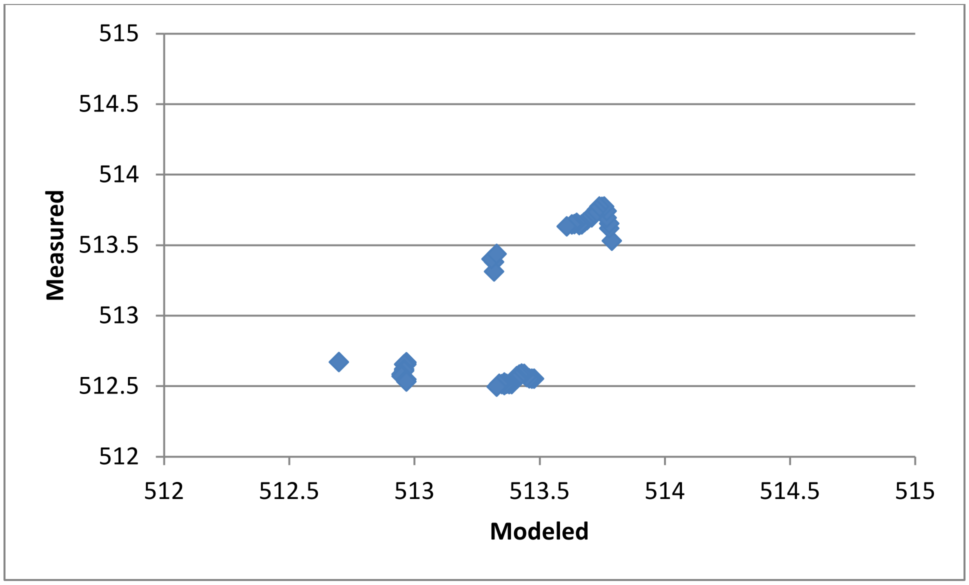
Appendix F-2: Storm event on 4/1/12 in CRP bioretention cell.



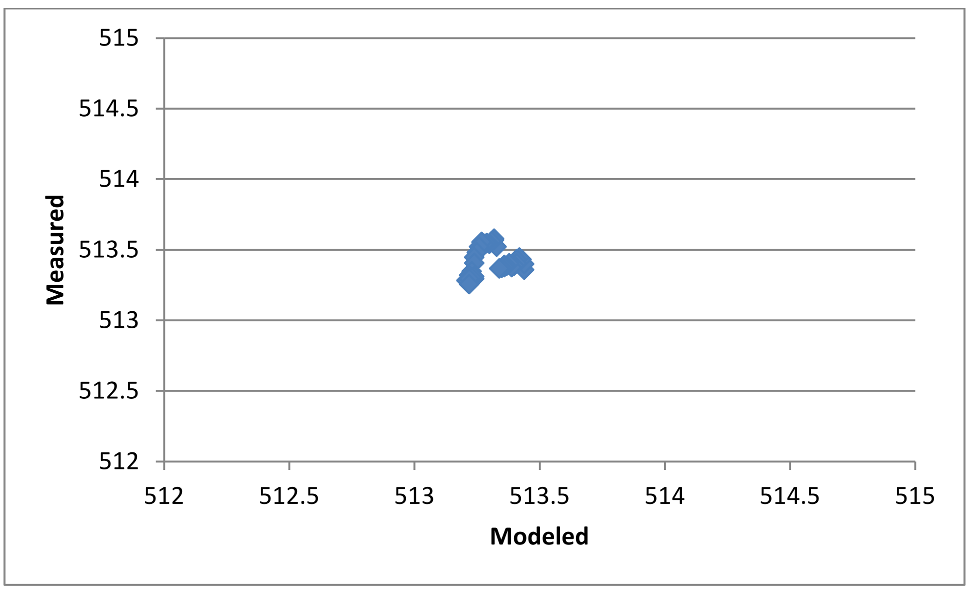
Appendix F-3: Storm event on 4/3/12 in CRP bioretention cell.



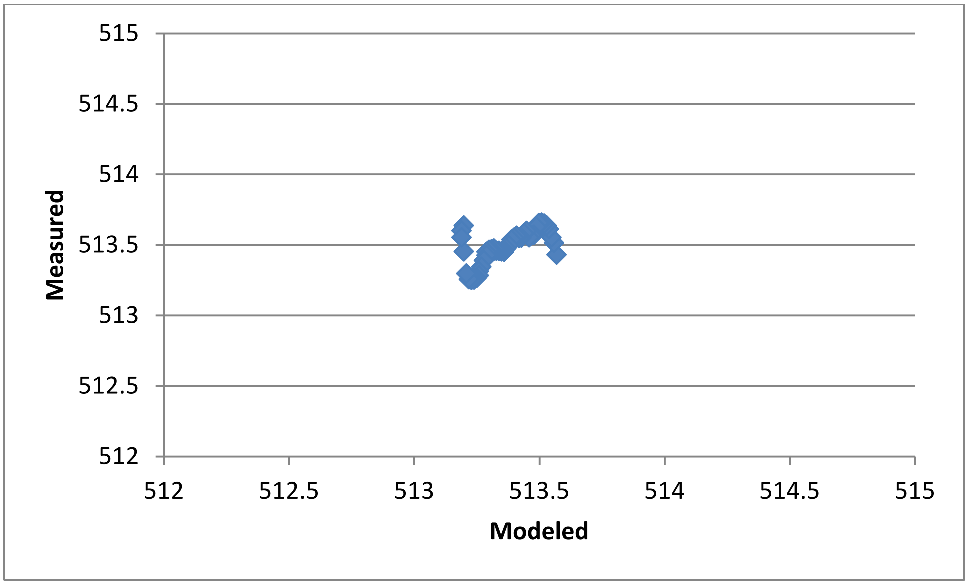
Appendix F-4: Storm event on 4/4/12 in CRP bioretention cell.



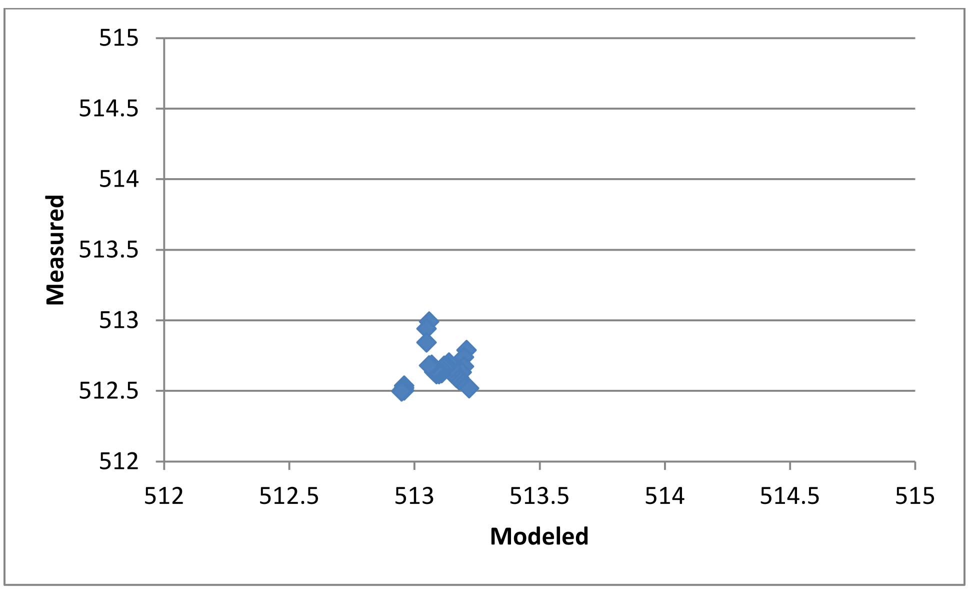
Appendix F-5: Storm event on 5/9/12 in CRP bioretention cell.



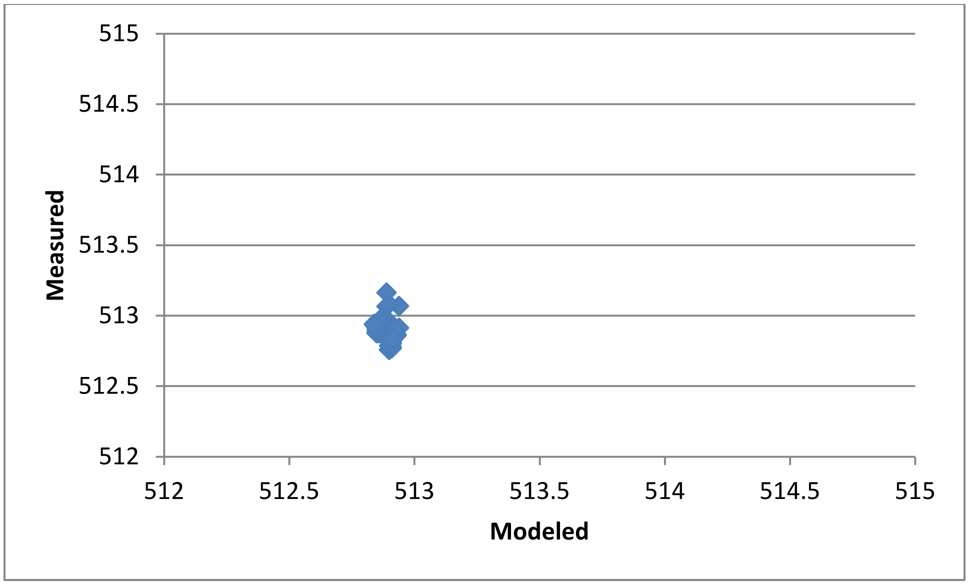
Appendix F-6: Storm event on 5/14/12 in CRP bioretention cell.



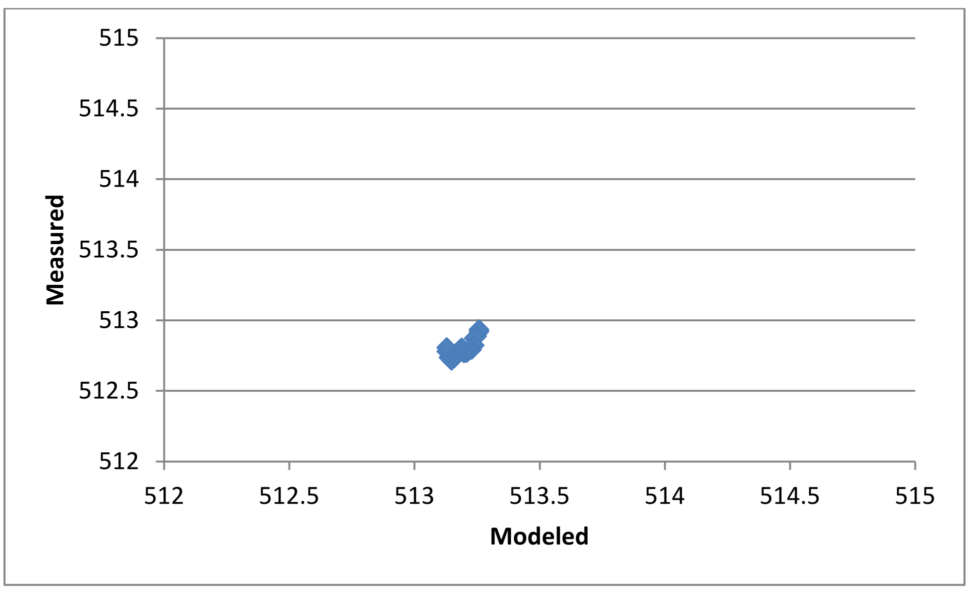
Appendix F-7: Storm event on 5/17/12 in CRP bioretention cell.



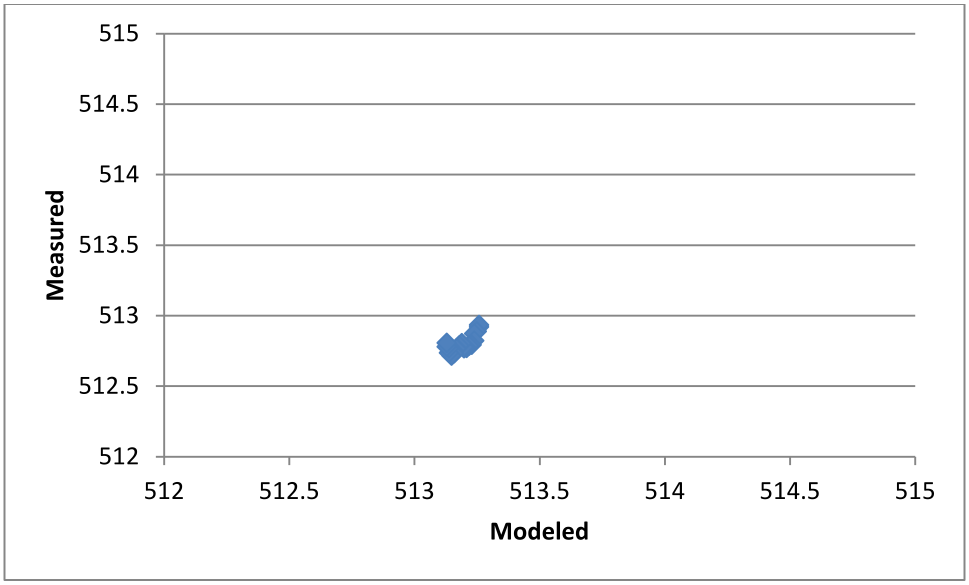
Appendix F-8: Storm event on 5/29/12 in CRP bioretention cell.



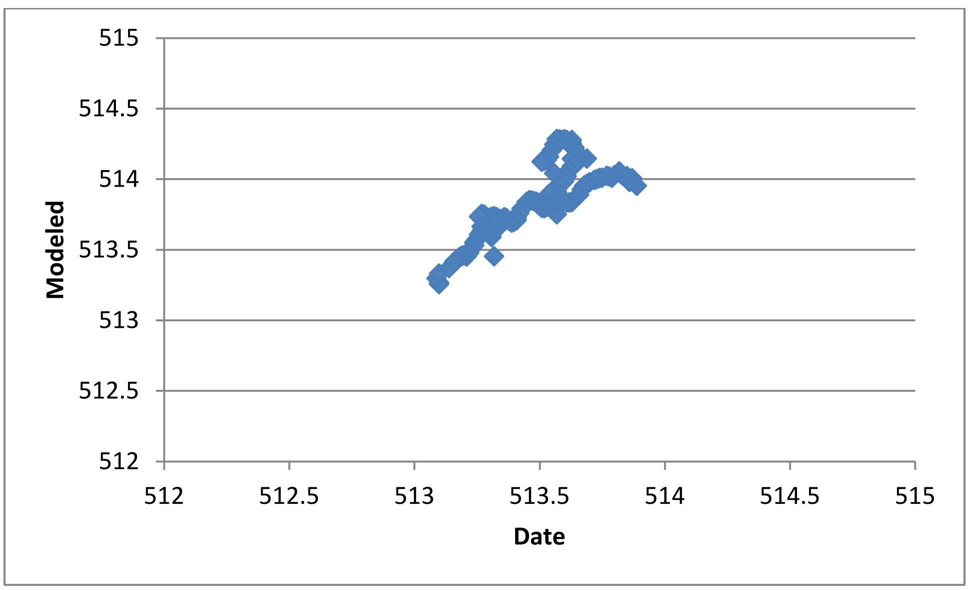
Appendix F-9: Storm event on 6/1/12 in CRP bioretention cell.



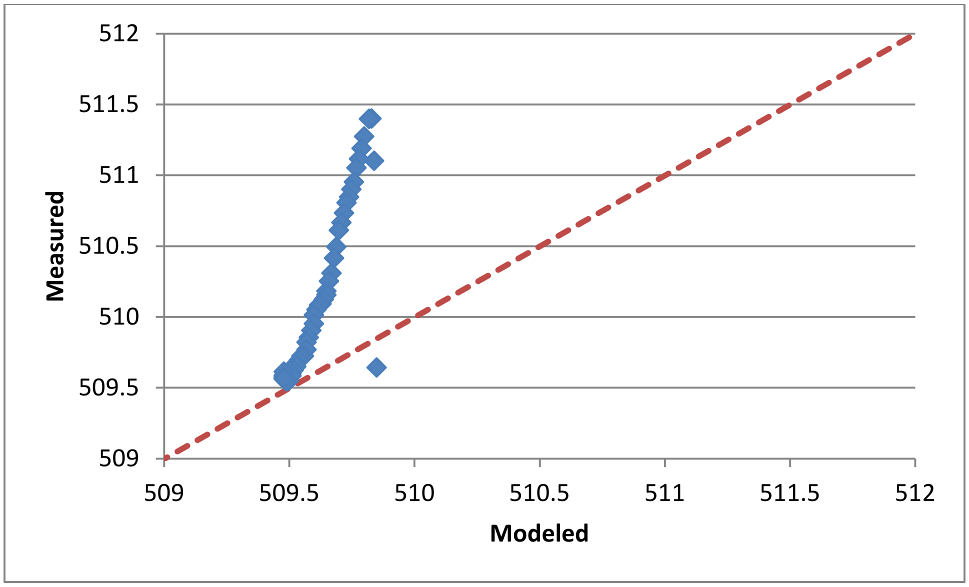
Appendix F-10: Storm event on 6/4/12 in CRP bioretention cell.



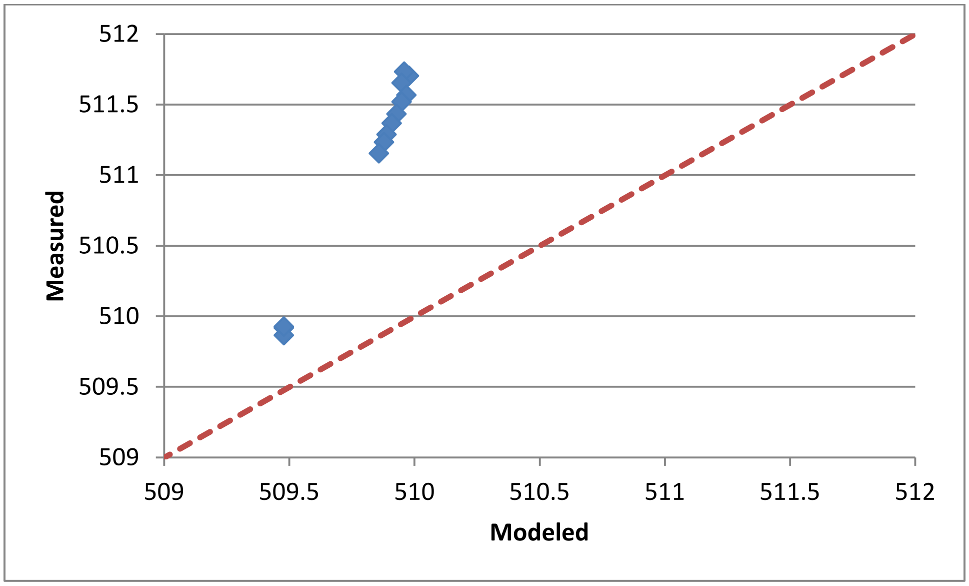
Appendix F-11: Storm event on 6/5/12 in CRP bioretention cell.



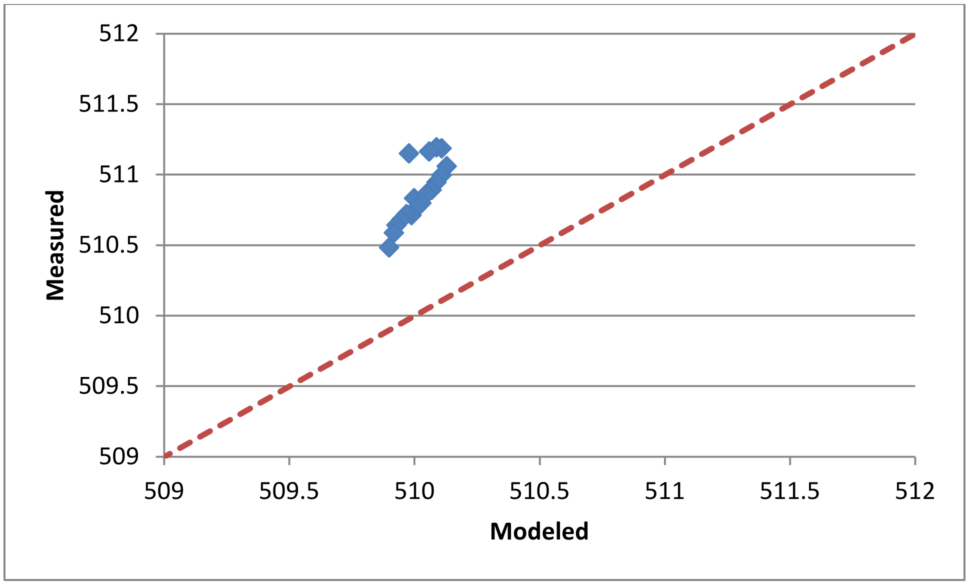
Appendix F-12: Storm event on 6/10/12 in CRP bioretention cell.



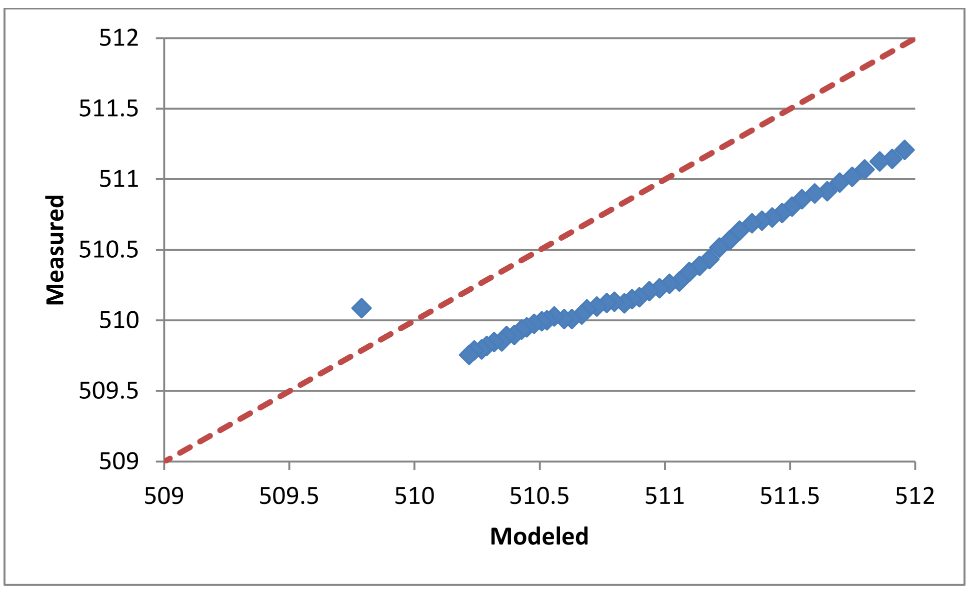
Appendix F-13: Storm event on 3/13/12 in PCN bioretention cell.



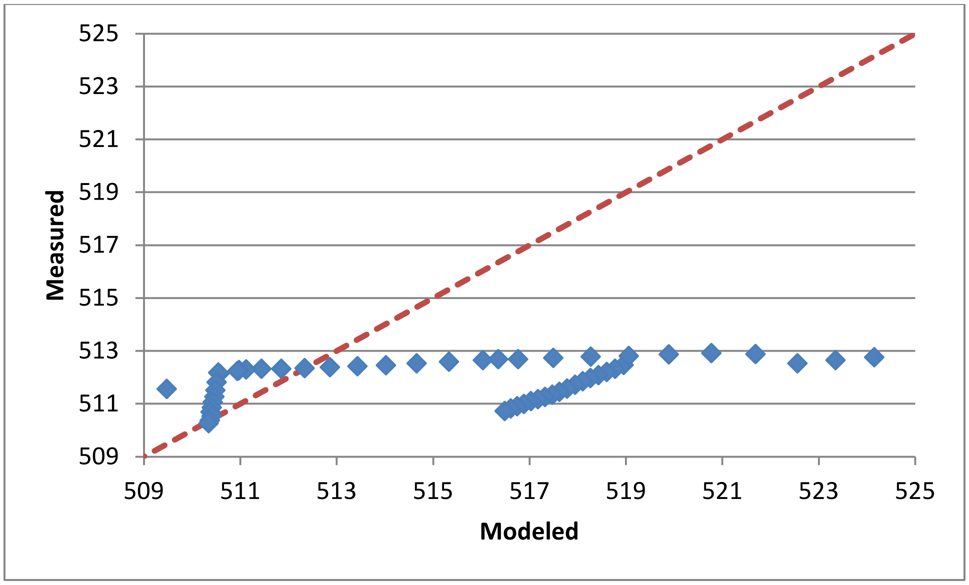
Appendix F-14: Storm event on 4/1/12 in PCN bioretention cell.



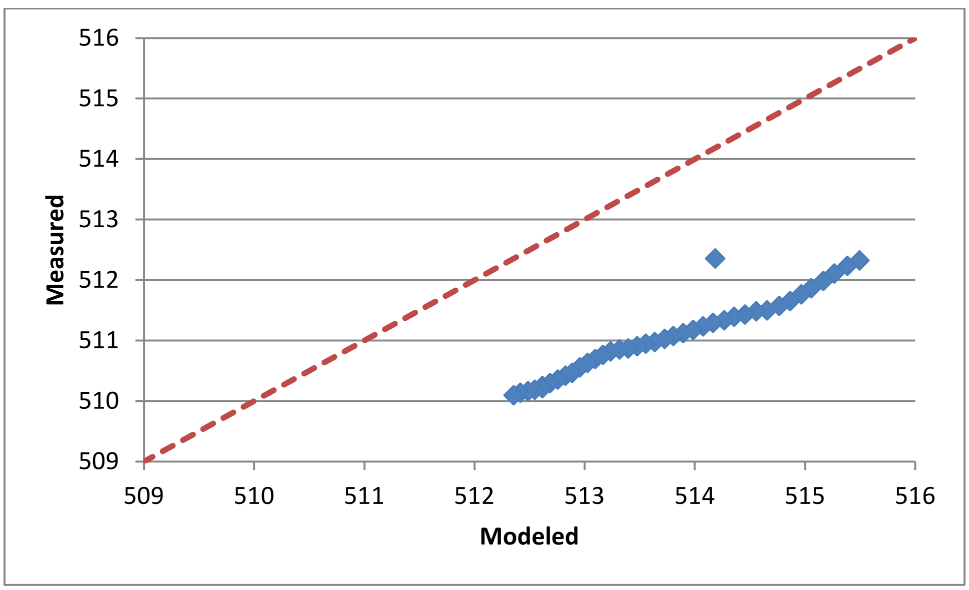
Appendix F-15: Storm event on 4/3/12 in PCN bioretention cell.



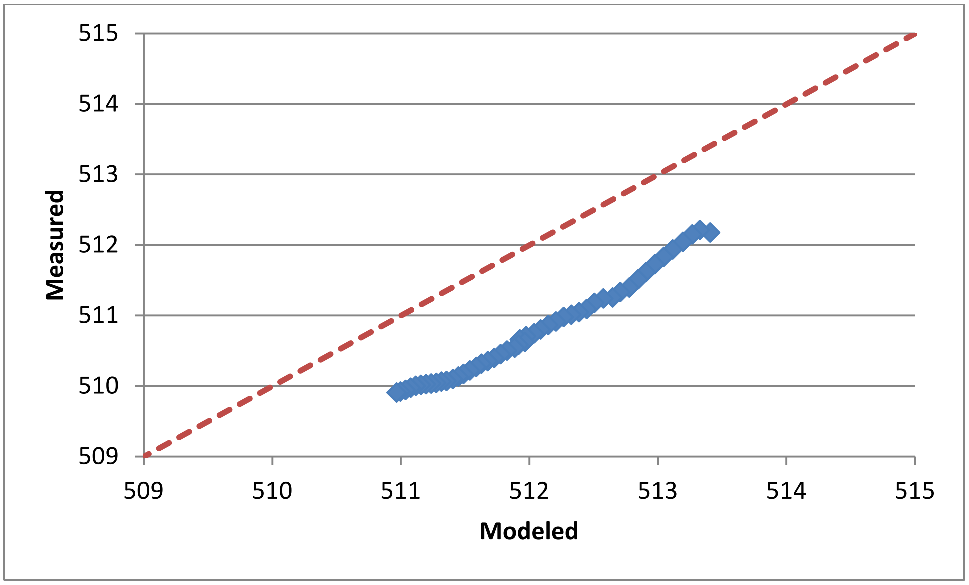
Appendix F-16: Storm event on 4/4/12 in PCN bioretention cell.



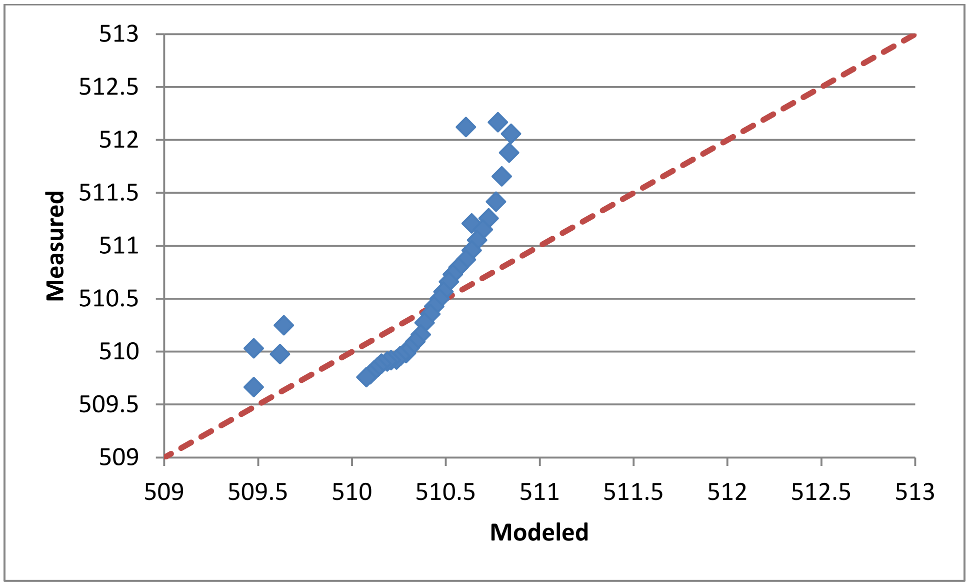
Appendix F-17: Storm event on 5/9/12 in PCN bioretention cell.



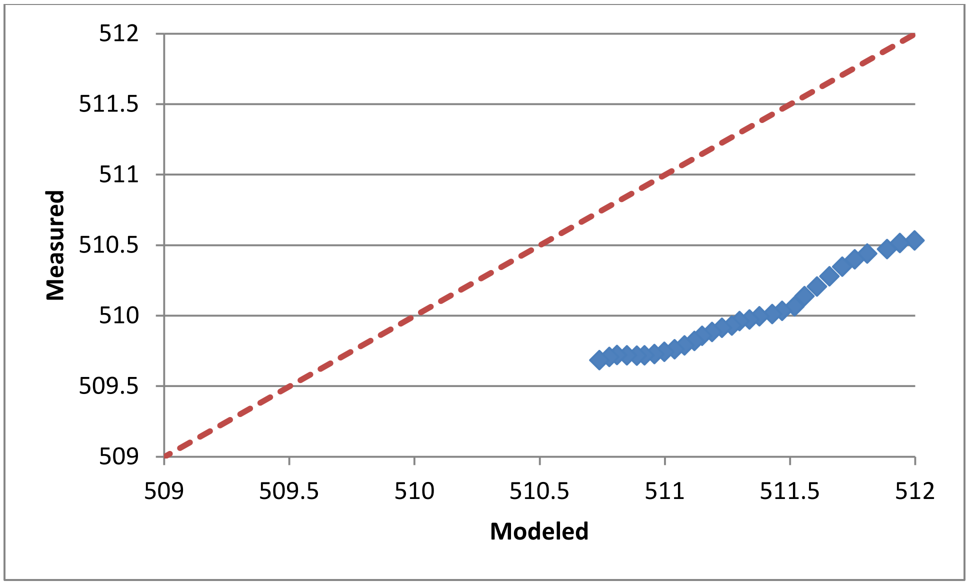
Appendix F-18: Storm event on 5/14/12 in PCN bioretention cell.



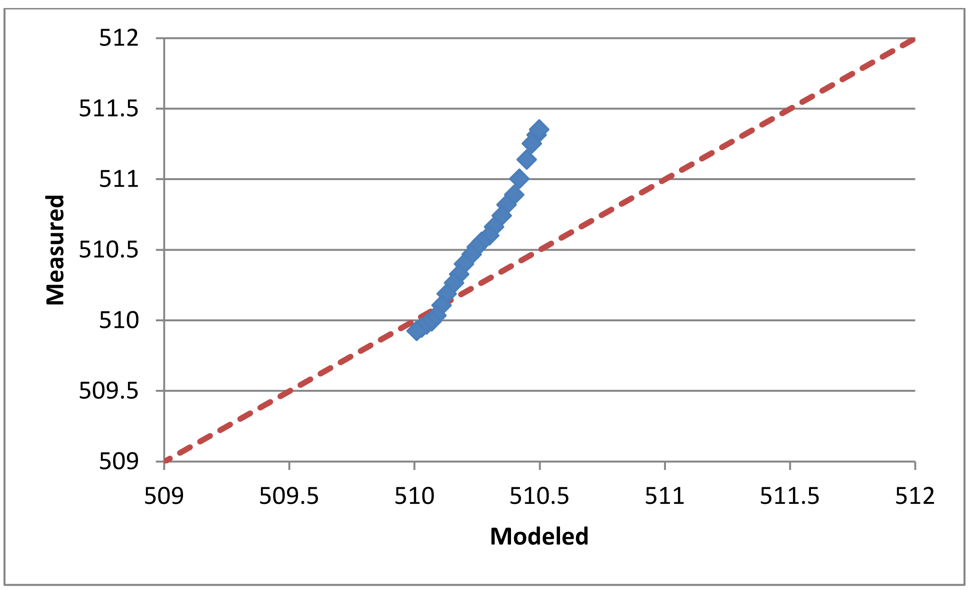
Appendix F-19: Storm event on 5/17/12 in PCN bioretention cell.



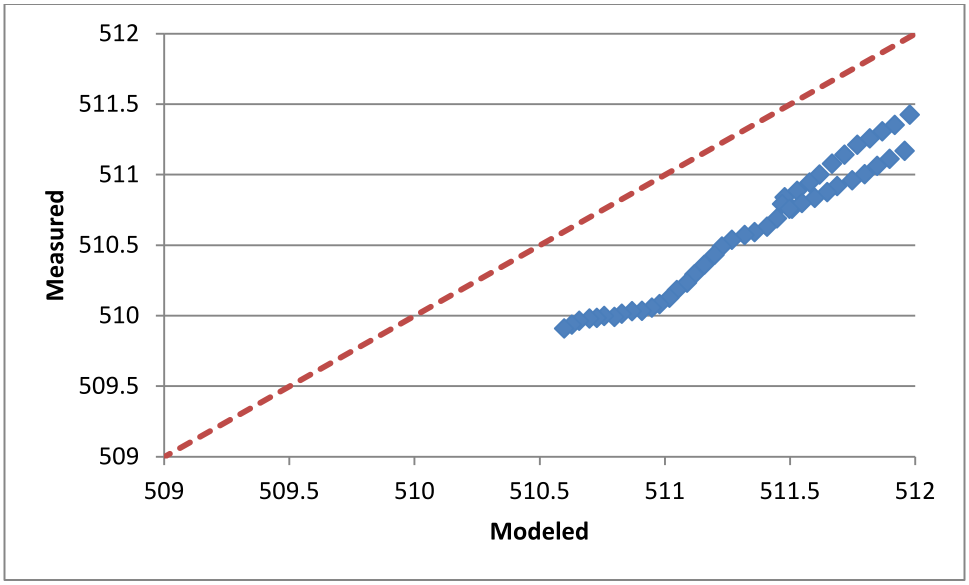
Appendix F-20: Storm event on 5/29/12 in PCN bioretention cell.



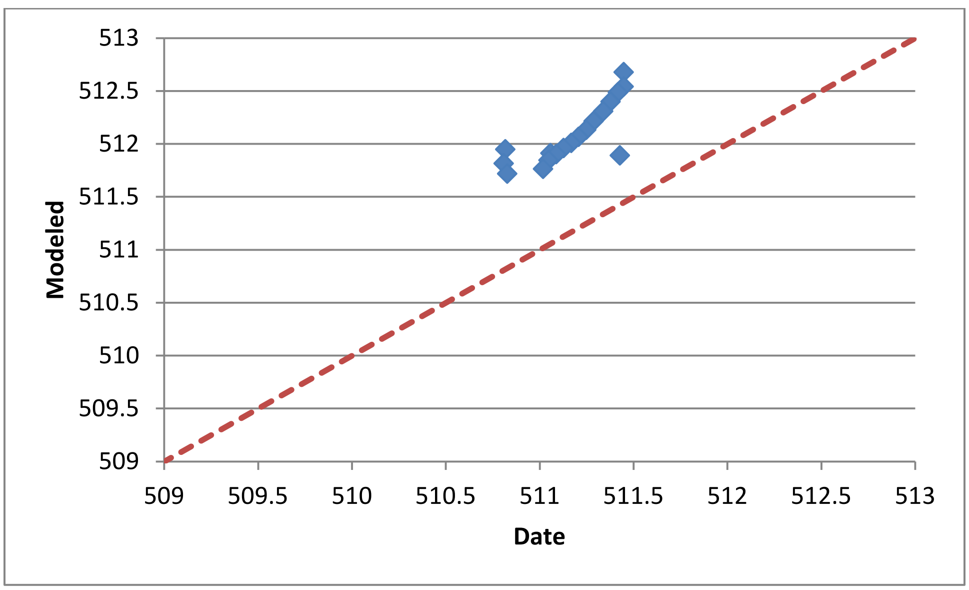
Appendix F-21: Storm event on 6/1/12 in PCN bioretention cell.



Appendix F-22: Storm event on 6/4/12 in PCN bioretention cell.



Appendix F-23: Storm event on 6/5/12 in PCN bioretention cell.



Appendix F-24: Storm event on 6/10/12 in PCN bioretention cell.