Regenerative Stormwater Conveyances in Knoxville, TN: A Performance Analysis and Investigation of Design Feature Alternatives

A Dissertation Presented for the
Doctor of Philosophy
Degree
The University of Tennessee, Knoxville

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December 2018

ACKNOWLEDGEMENTS

I would like to thank Dr. Jon Hathaway for providing me with the opportunity to perform this research at UT under his guidance. His experience, patience, and open-door policy made completing this endeavor a reality. Next, I would like to thank my co-advisor, Dr. John Schwartz. His years of experience and knowledge have proven to be an invaluable asset in my work. Next, I'm not certain where this project would have ended up without Tim Gangaware's expertise and advise during the project design and implementation stages. His access to numerous resources and connections were crucial in completing the project. A big thank you to Roy Arthur for taking us all over Knoxville during the site selection process and always being one of my biggest fans at conferences. Thank you to Ruthanne Hanahan, as there would be no education component without her. I've never met someone with even close to the same ability as she does for multitasking and organization – it is a true inspiration.

I am extremely thankful to my dissertation committee, Drs. Jon Hathaway, John Schwartz, Edmund Perfect, and Andrea Ludwig, for providing valuable insight, guidance, and time from their busy schedules in directing my research.

I would not have survived without the physical and mental help from all of the other graduate students and undergraduate research assistants that were willing to lend a hand (or ear) when needed, so a big thank you to them.

Thank you to Larry, Ken, and the rest of the shop crew for building monitoring equipment, installing monitoring wells, and everything else they do to make our projects a reality.

This project would not be possible without funding from the Tennessee Healthy Watershed Initiative, comprised of The Tennessee Department of Environment and Conservation, The Tennessee Valley Authority, The Tennessee Chapter of the Nature Conservancy, and West Tennessee River Basin Authority.

Thank you to The City of Knoxville, Knox County, and Knox County Schools Maintenance and Operations for the use of their property and taking on any grounds maintenance. Further, South Doyle Middle School (Dave Gorman and Jennifer Sauer) and various Americorp members have played a huge part in this project, whether through planting grass seed, picking up litter, performing monitoring, or general education.

Thank you to my parents for supporting my dreams since day one. I got my inspiration for engineering from my dad and his brothers, all of whom are engineers. I started my educational journey on a much different path (art history, also inspired by my parents), so wouldn't be where I am without an engineering influence.

Last but not least, thank you to my partner, Blake and our sweet baby girl, Lilly. He always made sure my head was on straight and always had my back 100%. Lilly has been especially good at requiring me to take breaks from my research, even if it is to just change a diaper.

ABSTRACT

Regenerative Stormwater Conveyances (RSCs) are an innovative approach to repairing eroded stormwater conveyances while simultaneously treating runoff and providing groundwater recharge. Although RSCs are increasingly being utilized as a Stormwater Control Measure (SCM) for watershed restoration, research is limited. The objectives of this research were to: 1) build and utilize a 3D computational fluid dynamics model to investigate design feature alternatives, 2) perform a groundwater analysis to determine the subsurface interactions of infiltrated stormwater, and 3) conduct a hydrologic and water quality investigation to evaluate RSC performance in terms of volume, peak flow rate, and pollutant reductions. The modeling study determined that it is possible to create a system that maximizes velocity and energy reductions. Specifically, it was determined that lower pool length to width ratios (L/W) were preferred for reducing velocities and energies, with results suggesting an optimal L/W of 0.7. The results also suggested that design of the first two pools of the system were the most critical and that a minimum number of pools was necessary for adequately dissipating erosive flows before discharging to the receiving stream. The groundwater investigation found that there were significant influences on the groundwater system resulting from infiltrated stormwater within the RSC. The degree of influence of the RSC resulted from seasonal fluxes in water table elevation, varying weather patterns, and most significantly, the presence of two buried sewer lines. Despite these confounding factors, it was discovered that the RSC acted as a source to the local water table during small to medium events, and a sink during large events. Lastly, the hydrologic and water quality investigation yielded mixed results, with hydrologic and pollutant load performance showing modest reductions, despite relatively poor reductions in event mean concentrations. The study also found that performance was best during the growing season (compared to the non-growing season), with volume reductions of 37% possible. In conclusion, RSCs are recommended as a viable SCM in the Knoxville area, given the right site conditions. Optimal site conditions include locations with lower water tables, highly infiltrative soils, treatment train applications, or as a retrofit where head cutting has occurred.

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1. Introduction

1.1. General

Stormwater runoff has been identified as a major contributor to water quality degradation across the United States. According to the most recent Water Quality Assessment Report, available for reporting year 2012, urban runoff is one of the top probable causes of river and stream impairment, preventing the designated uses of recreation and fish/aquatic life from being met, and potentially posing harm to human health (EPA 2014b). As such, methods to reduce the impacts of stormwater runoff to surface waters are being sought.

Regenerative Stormwater Conveyances (RSCs), available in either dry channel or wet channel variants, are an innovative approach to repairing local channel incision at stormwater outfalls, while simultaneously treating stormwater runoff, providing groundwater recharge, and establishing floodplain connectivity (Brown 2010). Essentially, RSCs bridge the gap between sustainable stormwater management and stream restoration practices. RSCs utilize aquatic pools consisting of a sand and wood chip media to promote microbial action and infiltration; riffles formed from cobble and weir grade control structures constructed from large boulders to dissipate energy; a sand seam to provide filtration and a conduit for subsurface flow; and established native vegetation to promote microbial activity, nutrient reduction, and evapotranspiration (Cizek 2014). Their dual functionality allows small storms to infiltrate and travel through the system as shallow groundwater flow, and large storms to travel through the series of step pools to reduce erosive velocities prior to discharge. Further, these systems are especially beneficial because they can be used to retrofit areas with steep slopes (up to 50% with appropriate design), making them particularly useful when other standard practices would fail (Flores et al. 2012). This combination of stability under large flows, infiltration under small flows, and applicability on varying terrains creates a versatile, robust system.

Since the implementation of RSCs as a stormwater control measure (SCM), a limited number of studies have been performed to quantify their flow and pollutant load reductions (Brown 2010, Browning 2008, Cizek 2013, Cizek 2014, Cizek et al. 2017, Cook et al. 2014, Filoso and Palmer 2011, Koryto 2016, Koryto et al. 2018, Koryto et al. 2017, Palmer et al. 2014). Because this research only investigated dry channel RSCs, defined as a retrofit practice applied to ephemeral stormwater conveyances, the available literature is even more limited (Cizek 2013, Cizek 2014, Cizek et al. 2017, Koryto 2016, Koryto et al. 2018, Koryto et al. 2017). That being said, this research aims to build upon the already existing bank of knowledge regarding RSC hydrologic and pollutant load reductions. Additionally, only one attempt (to our knowledge) has been made to optimize system design and performance through any form of modeling (Cizek 2014). A portion of this research utilized the commercially available computational fluid dynamics (CFD) modeling program, FLOW-3D™, to provide insight into possible sources of physical failure, whereby design parameters (i.e. depth, length, width of pools) were modified to achieve the most robust RSC design. Lastly, little is known regarding the groundwater interactions resulting from the infiltration of stormwater through RSCs, or other SCMs for that matter. Monitoring wells installed at the site will aid in the investigation of the conversion of surface flow to subsurface flow, as well as provide insight into how these systems function under varying site conditions.

1.2. Site Characteristics

A failing stormwater conveyance draining to Baker Creek in Knoxville, TN, has been selected as an experimental site to retrofit with an RSC and investigate performance. The conveyance is located on the South Doyle Middle School campus and receives water from a 2.0 ha, 74% impervious watershed comprised of a school building, parking lot, and associated grassed areas.

Stormwater enters a series of inlets located in the school parking lot and travels through a 0.61 m diameter concrete pipe until it reaches the RSC. Due to the high imperviousness and level of soil compaction within the drainage area, surface flow in the RSC occurs with as little as 0.2 cm of precipitation. Once stormwater leaves the conveyance, it discharges to Baker Creek, a tributary of the Tennessee River located within the Watts Bar Lake subbasin (HUC-8: TN06010201). Baker Creek is listed on the 303(d) list of impaired streams for the designated uses of Fish/Aquatic Life and Recreation, resulting from excessive pollutant loadings from pathogens, nutrients, and habitat alteration (EPA 2014a). Prior to installation of the RSC, the conveyance was a riprap lined channel, providing minimal storage and acting as a source of suspended sediment to Baker Creek resulting from bank failure.

1.3. Evolution of Stormwater Management

1.3.1. Urbanization

In 2008, the global urban population surpassed the rural population (Seto et al. 2010), signifying a shift in world demographics. What's more significant, these global urban areas are expanding at twice the rate as overall population growth (Angel et al. 2011). While new estimates predict that all urban areas make up less than 3% of the global terrestrial surface (Grimm et al. 2008), the ecological impact is much greater, with these same areas accounting for 78% of carbon emissions, 60% of residential water use, 76% of wood used for industrial purposes (Brown 2001), and heavily contributing to stream degradation (Klein 1979). Klein (1979) further explained that the growth of urban areas ultimately results in increased volumes of stormwater runoff, reduced stream base flow, alteration of the natural stream temperature, alteration of the character and volume of energy inputs to the stream, and increased pollutant loadings, including heavy metals, pesticides, oil, road salt, detergents, nutrients, and sediment. Many of these hydrologic and water quality effects can be traced to an increase in impervious surfaces as a result of urbanization, resulting in a substantial disruption of the hydrologic regime.

1.3.2. Impervious Cover

Impervious cover is defined as any surface preventing the infiltration of water to the soil, including roads, rooftops, and sidewalks (Arnold and Gibbons 1996). Therefore, as cities grow, the amount of impervious cover increases. Historically, impervious cover has been the variable used to relate the degree of impairment to extent of urbanization (Klein 1979). However, due to inconsistencies in efforts quantifying degree of urban development using impervious cover, Booth and Jackson (1997) identified a need for distinction between total impervious area (TIA) and effective impervious area (EIA). TIA is equivalent to the typical definition of imperviousness, presented previously. EIA is defined as the impervious surfaces that have direct hydraulic connection to the receiving stream. The primary argument for this distinction is that TIA does not account for highly compacted soils that prevent infiltration (similar to paved surfaces), and that it includes impervious surfaces that do not contribute stormwater to the receiving stream, for example a gazebo in a field, where the roof drains directly to grassy area and eventually infiltrates into soils (Booth and Jackson 1997). Regardless, the effects of these surfaces have resulted in a multitude of stream impairments requiring amelioration and spurring environmental regulation.

1.3.3. The Clean Water Act

Amendments made in 1972 to the Federal Water Pollution Control Act of 1948 became known as the Clean Water Act (CWA) (EPA 2017a). Ultimately, the CWA required all states to restore their waters to a fishable and swimmable status. The CWA made the following amendments: provided the basic structure for regulating pollutant discharges; gave the Environmental

Protection Agency (EPA) the authority to implement pollution control programs; maintained existing requirements for water quality standards for polluted waterways; made it illegal to discharge point source pollutants into waterways without a permit; funded construction of sewage treatment plants; and, recognized the need to address nonpoint source pollution (EPA 2017a). These amendments paved the way for the future evolution of stormwater regulation in the United States.

1.3.3.1. NPDES

As part of the CWA, the National Pollution Discharge Elimination System (NPDES) was created to regulate the discharge of point source pollution from municipal and industrial operations, including, treated wastewater, process water, cooling water, and stormwater runoff from drainage systems (EPA 2017e). In 1990, the NPDES further evolved to include the NPDES Stormwater Program to regulate discharges from municipal separate storm sewer systems (MS4s), construction activities, and industrial activities (EPA 2017b). Yet, streams are still enduring significant degradation from nonpoint sources of pollution, as they are more difficult to identify, and thus mitigate.

1.3.3.2. 303(d) list

Section 303(d) of the Clean Water Act established a methodology for identification and remediation of polluted waters. Under this section of the CWA, each state is required to perform a water quality assessment of all surface waters in the state (where data is available) every two years. If a waterbody is identified as not meeting its designated use (i.e. drinking, recreation, aquatic habitat, and industrial), that waterbody is placed on the 303(d) list (EPA 2017c). Once a waterbody is added to the 303(d) list, the state must then develop a Total Maximum Daily Loading (TMDL) for the given waterbody. The waterbody will remain on the 303(d) list until the

TMDL is developed, after which, water quality will continue to be tracked until it has been fully restored (EPA 2017c). The creation of this listing methodology provides a means to tract impaired streams in order to provide municipalities a starting point for water quality improvements – beginning with the development of TMDLs.

1.3.3.3. Total Maximum Daily Loading (TMDL)

A TMDL is the maximum pollutant load a waterbody can receive while still being able to meet its intended use. For all point sources, a waste load allocation (WLA) is assigned, and for all nonpoint sources, a load allocation (LA) is assigned. Point sources include all sources outlined in NPDES program, while nonpoint sources include remaining sources of pollution and any natural background concentrations. In short, a TMDL is the sum of all WLA and LA, plus a margin of safety. A TMDL must be developed for each pollutant causing degradation for a given waterbody. Particularly relevant to this study are the significant contributions made by urban stormwater runoff carrying pollutants, thus causing impairments. This source of degradation often plays a major role in watershed restoration efforts. Typically, a TMDL implementation plan accompanies the TDML in order to provide information regarding pollutant sources in a watershed and how the municipality anticipates controlling them (EPA 2017d). Although not required, TMDL implementation plans are the critical framework necessary for preliminary stream restoration efforts.

1.3.4. Evolution of Stormwater Control Measures

Stormwater management dates back to ancient Mesopotamia where practices were used for flood control, waste conveyance, and rainwater storage (NRC 2008). Modern-day stormwater management began with the onset of urbanization, to address the occurrence of flash flooding (NRC 2008) and has continued to evolve to encompass a wide range of ecosystem services, or benefits people obtain from the ecosystem (Figure 1.1) (Fletcher et al. 2014, MEA 2005). The initial solution was to route stormwater out of cities and into receiving waters as quickly as possible (Brown et al. 2009). However, this approach had consequences, resulting in downstream flooding and bank erosion due to the influx in stormwater runoff. In turn, streams were channelized, which consisted of straightening, enlarging, and lining with concrete to provide more space for flood waters and prevent erosion; however, the addition of more impervious surfaces in the stream channel resulted in more flooding downstream and virtually eliminated the habitat necessary for a healthy aquatic ecosystem (NRC 2008). As a result, it became apparent that new methods were still needed to safely manage and convey stormwater runoff.

Beginning in the early 1970's was the onset of detention basins, implemented as a means to reduce peak flows from developed areas (NRC 2008). McCuen (1979) found that while detention basins are capable of reducing the peak flows at the site outlet, the combined effect of all detention basins within a watershed can actually increase peak flows further downstream. This occurs by delaying the release of flows, and although lowered peaks are released from each individual basin, the downstream cumulative peak for all releases can be greater than if there had been no detention basin at all (McCuen 1979). Thus, these methods proved problematic and new methods of stormwater management were still being sought to adequately control stormwater runoff, with a shifting focus of maintaining predevelopment hydrology and water quality at a site scale, rather than watershed scale.

More recently, a focus has been placed on water quality improvements, not just water quantity management. Increased pollutant loadings found in streams have led to the recognition of stormwater runoff as a source of environmental degradation, resulting in new management objectives focused on load reductions in addition to volume reductions (Burns et al. 2012). In

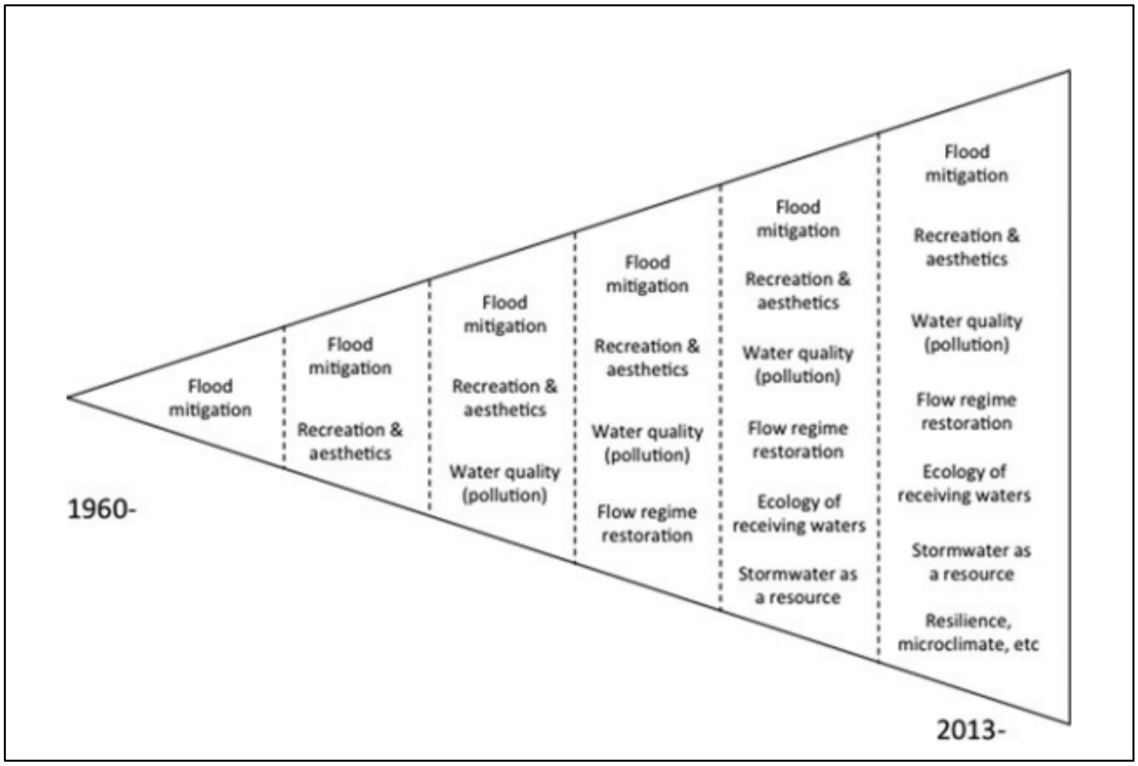

Figure 1.1 Evolution of stormwater management goals. Figure taken from Fletcher et al. (2014), who adapted from Whelans et al. (1994).

order to meet these new stormwater management objectives, many states have adopted pollutant load reduction targets as a preventative practice, rather than relying on the retroactive practice of TMDL development. Tennessee, for example, has a goal to prevent runoff from the first inch of rainfall, and in cases where that is not possible, to remove 80% of total suspended solids (TSS) from that first inch of runoff (TDEC 2015). TSS was chosen because it is typically the leading pollutant of concern and acts as a surrogate for removal of other pollutants, such as nutrients and heavy metals (TDEC 2015). With these evolving trends of stormwater management, traditional grey infrastructure practices (i.e. storm sewers and concrete lined conveyances) are no longer able to meet these stormwater management targets, resulting in the need for new, innovative approaches for stormwater management, commonly referred to as stormwater control measures (SCM).

SCMs are defined as "a technique, measure, or structural control that is used for a given set of conditions to manage the quantity and improve the quality of stormwater runoff in the most cost-effective manner" (NRC 2008). NRC (2008) states that SCMs have evolved to meet larger watershed goals of protecting biology and geomorphology of receiving waters. SCMs can be both structural (i.e. stormwater wetlands, infiltration basins, bioretention systems, and regenerative stormwater conveyances) or nonstructural practices (i.e. education, downspout disconnection programs, and better site design to limit stormwater runoff) (Fletcher et al. 2014, NRC 2008). Further, Cizek and Hunt (2013) conclude that structural SCMs should aim to mimic predevelopment hydrology, which must take into account all flow pathways (surface runoff, shallow interflow, and groundwater surge) in order to meet predevelopment water quality. The argument is that unique water quality treatment mechanisms are present in each of the flow pathways, and must be considered in order to truly meet predevelopment conditions (Cizek and

Hunt 2013). Therefore, it proves beneficial from a water quality and hydrologic viewpoint to identify and implement SCMs, such as RSCs, capable of conveying flows in all possible pathways.

1.4. Role of Regenerative Stormwater Conveyances

Regenerative stormwater conveyances (RSC) are also known at coastal plain outfalls, regenerative step-pool storm conveyances, and biofiltration conveyances, and can be classified as either wet channel or dry channel (Berg et al. 2013). Berg et al. (2013) describes dry channel RSCs as the restoration of ephemeral streams or eroding gullies and are considered a stormwater retrofit practice. Wet channel RSCs are located in perennial streams, promote floodplain connectivity, and are considered a stream restoration practice (Berg et al. 2013). This research focuses solely on dry channel RSCs which will simply be referred to as RSCs herein.

RSCs were first implemented in Maryland as a retrofit to previously failing SCMs. Anne Arundel County, MD, conducted an inventory of all stormwater outfalls in the county and found that the majority of all pipe outfalls, rip-rap and gabion level spreaders, and energy dissipation devices have failed. The estimated damage to streams, wetlands, and steep slopes has been estimated at more than $600 million (Brown 2010). As a result, the municipality has turned to RSCs as the SCM of choice, as it is capable of providing water quality improvements, increased infiltration, thus groundwater recharge and reduced surface flows, as well as a suite of ecosystem services. Through the implementation of RSCs in place of conventional stormwater management practices, Anne Arundel County, MD has recognized benefit-cost ratios ranging from 2:1 to 6:1 (Brown 2010). Based on past evaluation, it is evident that RSCs are capable of providing move than just hydrologic and water quality benefits, including cost savings and ecosystem services, making them an even more attractive SCM.

1.4.1. Design Components

The primary components of RSCs include natural sections of pools, riffles, weirs, and underlying sand seam. The aquatic pools consist of an approximate mixture of 80% sand and 20% hardwood mulch media to promote microbial action and infiltration; riffles are formed from cobble and weir grade control structures constructed from large boulders to dissipate energy; a sand seam to act as a filtration media and conduit for shallow subsurface flow; and established native vegetation to promote microbial activity, nutrient reduction, and evapotranspiration (Figure 1.2) (Cizek 2014, Flores et al. 2012). Under small, more frequent storm events, stormwater ponds in the pools and infiltrates to the underlying sand seam where it either travels through the system as shallow groundwater flow or exfiltrates to parent soil. During large storm events, stormwater travels predominately as surface flow through the series of step pools and across the riffle/weir grade control structures to reduce erosive flows prior to discharge. In instances of slopes greater than 10% (or 5% when used for water quality improvements), Flores et al. (2012) recommends the use of a cascade structure to be placed at slopes up to 50% with no greater than a five-foot vertical drop over the length of the cascade. Cascades are sections of large boulders intended to stabilize the area of steep slopes and are recommended to be followed by three pools at equal elevation. The design guidelines also recommend that typical pool depths be 0.46-0.91 m (1.5-3.0 ft), pool/riffle widths be a minimum of 2.4 m (8 ft), and pool length be twice the length of the riffle section (Flores et al. 2012), yet nothing more specific is provided in regards to design of RSC features, and design which does exist is typically empirical. Development of more rigid design guidelines supported by field and/or modeling data is critical in ensuring long-term stability of RSCs, especially as these systems are continuously gaining attention and are becoming a more widely implemented SCM.

1.4.2. Stormwater Management Benefits

RSCs are commonly used as a retrofit practice where previous conveyances have failed, typically as a result of steep slopes leading to massive head cutting where stormwater outfalls discharge. RSCs combine features of swales, infiltration, filtering, and wetland practices to provide a wide array of treatment benefits (Brown 2010). RSCs are especially desirable because they are capable of providing all three flow pathways necessary to mimic predevelopment conditions, which is an important consideration when considering the treatment mechanisms that occur along the flow paths (Cizek and Hunt 2013). Although few monitoring studies exist regarding performance of RSC, the results are positive for providing water quality improvements and volume reductions. Studies have also highlighted important future design considerations, such as established vegetation, installation of exfiltration trench under saturated pools, or constructing SCMs in series. Studies concluded that if these design features had been implemented prior to construction, it would have resulted in greater water quality and hydrologic performance (Cizek 2014, Koryto 2016). However, existing data is still very limited and further research is necessary to be confident in RSC performance, especially when considering varying site conditions.

To date, only a handful of monitoring studies have been completed in regards to RSC water quality and quantity performance (Brown 2010, Browning 2008, Cizek 2013, Cizek 2014, Cizek et al. 2017, Cook et al. 2014, Filoso and Palmer 2011, Koryto 2016, Koryto et al. 2018, Koryto et al. 2017, Palmer et al. 2014). Specifically looking at dry channel RSCs, the available monitoring data is even more limited to three sites studied as part of research at North Carolina State University – (Cizek 2014, Koryto 2016), with a portion of each being published as peer reviewed literature (Cizek 2013, Cizek et al. 2017, Koryto et al. 2018, Koryto et al. 2017). Cizek's (2014) research involved the monitoring of two RSCs located in distinctly different soil types: one in the Coastal Plains region of North Carolina (Brunswick County) with predominately sandy soils of Hydrologic Soil Group (HSG) A classification, and the second RSC in the Piedmont Region of North Carolina (Alamance County) with more dense, clay soils of HSG D classification. Koryto (2016) investigated a third RSC, also located in the Piedmont region in Durham County, North Carolina (HSG D). The results of the two RSCs located in the Piedmont region are considered to be most relevant to the research in Knoxville, TN, as the soil types are very similar. In short, it was found that volume, peak flow, and pollutant loading reductions are possible, however, site conditions and water table location play a huge role in the extent of reductions realized, as evidenced by the varying performance between Koryto (2016) and Cizek (2014).

1.4.3. Ecosystem Services and Associated Costs

A study by Adrienne Cizek (2014) at North Carolina State University (NCSU) discusses in detail the ecosystem services provided by RSCs. Key findings of this work will be briefly discussed. The study compared 14 RSCs (both wet and dry systems) across North Carolina and Maryland to two reference ephemeral headwater streams to determine ecosystem service indicators present. Ecosystem service indicators used in the study include carbon sequestration and climate regulation, nutrient cycling and soil formation, biodiversity, habitat provision, and cultural services. In total, it was determined that the RSCs studied could provide $67,300/ha of ecosystem services over the first 15-year life of the system. It was also found that many of the services provided increased with the age of the system. It was also noted that incorporating recreational and/or educational features into the system, planting seedlings rather than seeds, and creating wet areas would further increase the value per hectare provided (Cizek 2014). While not investigated in this research, it is beneficial to realize the additional benefits provided by properly functioning RSCs, which can further promote them as a viable SCM option.

1.5. Knowledge Gaps

As previously mentioned, monitoring studies performed on (dry channel) RSCs are limited to three sites in North Carolina. This is not enough data to truly understand the benefits of RSCs and their contribution to watershed restoration. Previous research available had the primary focus of monitoring water quality and hydrologic performance, with additional conclusions made based on the findings. For example, Cizek (2014) monitored each pool individually, and thus was able to provide insight into the minimum recommended number of pools necessary for maximum pollutant removal. However, no recommendations were made in regard to length of system or internal geomorphic design for velocity/energy dissipation to ensure long-term stability. Design recommendations for these systems can still greatly benefit from fundamental studies to understand how changing RSC internal geomorphology, such as energy dissipating pools, will influence performance. Such studies will lend a scientific backing to design suggestions, leading to more robust and sustainable systems.

Further, very little research has been completed studying groundwater interactions that occur within RSCs, or even infiltration based SCMs in general. This is an important component to investigate to determine the amount of surface water being exfiltrated from the system and to determine the degree of groundwater mounding present. The majority of the SCM infiltration studies currently available only focus on infiltration within the SCM and do not investigate exfiltration to the surrounding water table. Groundwater mounding has the potential to cause negative consequences – posing harm to underground utilities, structural foundations, and overall flow regime, thus understanding how and where the surface water is being transported is an important consideration to investigate. Further, the potential influence underground utilities play is an important consideration, given they pose the risk of short-circuiting flow pathways, thus bypassing water quality treatment mechanisms. As urban environments continue to grow, the occurrence of underground utilities is becoming more of a concern for infiltration based SCMs.

1.6. Research Objectives

In order for RSCs to gain wide acceptances as a viable SCM, it is necessary to build upon the existing bank of knowledge and fill in data gaps. This research aims to accomplish the following:

1. The objectives of the modeling investigation of RSCs are to (1) identify what pool geometry (length and width combination) provides the greatest velocity and energy reductions under a range of flow rates, thus maximizing internal system stability, and (2) identify areas prone to erosion and scour through the evaluation of shear stresses.

   The significance of this research is that it works to refine the already existing, but somewhat empirical, design guidelines for RSCs, by providing recommendations for creating a strategically designed system that maximizes velocity and energy reductions, ensuring long-term system stability – an insight that is currently lacking regarding design of RSCs.

2. The objectives of the groundwater investigation are to (1) identify the surface and subsurface flow pathways present in the RSC to determine whether infiltrated stormwater remains mounded locally, is exfiltrated to the groundwater system, or travels along a utility line directly to the stream, and (2) determine if infiltrated stormwater reaches the water table and evaluate any subsequent influences to the groundwater system.

The significance of this study is that it provides insight into the occurrence of groundwater mounding, or lack thereof, within and around RSCs (with implications for infiltration-based stormwater control measures in general). Further, stormwater that infiltrates during storm events is typically considered “lost” from the surface water balance of stormwater controls. This study will help determine if infiltrated water is truly lost to the groundwater supply or is simply stored temporarily and then released during non-storm flow conditions.

3. The objectives of the RSC water quality and hydrologic study are to (1) quantify pollutant loadings at the inlet and outlet of the RSC to determine if reductions are occurring and (2) quantify the stormwater volume and peak flow reduction performance of the RSC.

   The significance of this study is that it will add to the existing, but limited, bank of knowledge regarding water quality and hydrologic performance of RSCs. Specifically, this research will fill data gaps regarding performance of RSCs within differing soil types and weather patterns that are present in existing literature.

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2. Three-Dimensional Modeling of the Hydraulic Function and Channel Stability of Regenerative Stormwater Conveyances

This article has been published by Jessica M. Thompson, Jon M. Hathaway, and John S. Schwartz:

Thompson, Jessica M., Jon M. Hathaway, and John S. Schwartz. "Three-Dimensional Modeling of the Hydraulic Function and Channel Stability of Regenerative Stormwater Conveyances." Journal of Sustainable Water in the Built Environment 4.3 (2018): 04018008.

Jessica M. Thompson generated this manuscript as part of the dissertation research with guidance from Jon M. Hathaway and John S. Schwartz.

Abstract

Regenerative Stormwater Conveyances (RSCs) are an innovative approach to repairing local channel incision at stormwater outfalls while simultaneously treating stormwater runoff, providing groundwater recharge, and establishing floodplain connectivity. Although RSCs are increasingly being utilized for watershed restoration, little research has been performed on these systems. This study uses the commercially available computational fluid dynamics program, FLOW-3D™, and hypothesizes that RSC system design can be improved to maximize velocity and energy dissipation while minimizing erosion potential. Results found that it is possible to create a strategically designed system that maximizes velocity and energy reductions, providing insight that is currently lacking regarding design of RSCs. Overall it was determined that lower length to width ratios are preferred for reducing velocities and energies, with study results suggestion designing with a L/W pool ratio of 0.7. It was also found that design of the first two pools of the system appears to be most critical for dissipating erosive flows and that a minimum number of pools is necessary for adequately reducing velocities and energy before discharging to the receiving stream. These findings are beneficial for to the design community as they allow a better understanding of RSC function and how manipulation of system geomorphology can improve stability.

2.1. Introduction

Regenerative Stormwater Conveyances (RSC) are an innovative approach to repairing local channel incision at stormwater outfalls while simultaneously managing stormwater runoff (Brown 2010). Essentially, RSCs bridge the gap between sustainable stormwater management and stream restoration practices. These systems utilize aquatic pools consisting of a sand and wood chip media to promote microbial action and infiltration; cobble riffles and boulder weir grade control structures to dissipate energy; and established native vegetation to promote microbial activity, nutrient reduction, and evapotranspiration (Cizek 2014a). Their dual functionality allows runoff from small storms to infiltrate and travel through the system as shallow groundwater flow, and runoff from large storms to travel through the series of step pools as surface flow, reducing erosive velocities prior to discharging into a receiving stream. Furthermore, these systems are especially beneficial because they can be used to retrofit areas with steep slopes (up to 50% with appropriate design), making them useful when other standard practices would fail (Flores et al. 2012). This combination of stability under large flows, infiltration under small flows, and applicability on varying terrains creates a versatile, robust system.

Based on previous research in areas of sandy soils, it has been determined that a considerable amount of runoff entering RSCs infiltrates and travels to the stream via shallow, subsurface flow (Cizek 2014a). However, research conducted on an RSC in the Piedmont Region of North Carolina indicates that substantial surface flow occurs in systems with less permeable soils, especially during storms with large amounts of rainfall and/or high rainfall intensity (Koryto et al. 2017). As a result of increased surface flow volumes through RSCs in regions of less permeable soils, stability becomes a critical system component and must be designed in such a way to prevent bed and bank scour, similar to the goals of stream restoration practices (Shields et al. 2003).

RSCs were first implemented in 2005 as a means to help restore the Chesapeake Bay watershed (Brown 2010, Underwood 2009). Since then, a limited number of studies have been performed to quantify their flow and pollutant load reduction (Brown 2010, Browning 2008, Cizek 2014b, Cizek et al. 2017, Cook et al. 2014, Hayes 2016, Koryto 2016, Palmer et al. 2014). However, only one attempt (to our knowledge) has been made to optimize system design and performance through any form of modeling. Researchers at North Carolina State University developed a preliminary RSC Hydrology Performance Model for quantifying the water balance within RSC systems (Cizek 2014b). Results from Cizek (2014b) provided general suggestions for design modifications (i.e. increasing pool and media depth); however, it does not take into account flow rate, velocities, and flow paths, all vital elements to consider when modifying system design. These system components are particularly important when aiming to reduce the potential for internal erosion and subsequent system failure while the RSC is operating under surface flow. Computational fluid dynamics (CFD) modeling programs provide insight into possible sources of physical failure, where design parameters (i.e. depth, length, width of pools and riffles) can be modified to achieve the most stable RSC design.

Three-dimensional (3D) CFD applies numerical analysis utilizing the continuity, momentum, and energy equations to solve for flow through discretized cells within a larger finite mesh (Anderson and Wendt 1995, Flow Science 2012). Because of complex flow pathways, a 3D CFD model is necessary to simulate the hydraulics through RSCs (Sturm 2010). CFD programs are a highly useful tool in the design and analysis of many hydraulic applications, including, but not limited to, spillways (Bombardelli et al. 2011, Chanel and Doering 2008, Chatila and Tabbara 2004, Teklemariam et al. 2002), fish passages (Duguay and Lacey 2014, Ho et al. 2011), weirs (Abad et al. 2008, Bhuiyan A. B. M. and Hey 2007), and fluvial geomorphology (Abad et al. 2008, Lane et al. 1999, Lofthouse and Robert 2008, Ma et al. 2002, Morvan et al. 2002, Qiu 2008, Rodriguez et al. 2004, Rüther and Olsen 2007, Schwartz et al. 2015). However, the use of 3D CFD to model stormwater control measures (SCM) is far less common. Examples of the use of CFD for SCM modeling includes Ho et al. (2010) who used the commercially available FLOW-3D™ program to evaluate the performance of a stormwater filter designed to remove debris from supercritical flows, and Rashid et al. (2012) who also used FLOW-3D™ to study the flow through modular permeable pavement. More relevant to RSC function, with its strong surface flow component, is the work of Han et al. (2014) to simulate flow patterns and suspended sediment transfer in surface flow constructed wetlands using Fluent, another commercially available CFD program. A similar study performed by Min and Wise (2009) models short-circuiting flow patterns in constructed wetlands; however, the analysis was only two-dimensional, thus providing limited insight into the flow patterns and turbulence structure as compared to three-dimensional models. Outcomes from all studies presented indicate CFD programs are a valid tool for predicting complex flows, providing further confidence in their use for assessment of SCM design components.

RSCs incorporate elements of stream design such as weir/check dam structures constructed from large boulders, riffles formed from smaller cobble, and pools that mimic bioretention and wetland facilities. Therefore, three-dimensional, rather than 1- or 2-dimensional, analysis becomes essential as more complex flow patterns are present. Literature review has found examples of individual components of RSCs modeled as part of other hydraulic structures. For example, the highly vegetated, shallow aquatic pools within RSCs are similar to shallow ponds found in constructed wetland systems (Min and Wise 2009). Flow across weir grade control structures constructed from large boulders is similar to simulations that analyzed flow across boulders and dumped rock in rivers and spillways (Nielsen et al. 2013, Oertel et al. 2010, Shen and Diplas 2008). Lastly, several studies exist that examine water discharged over stepped spillways, which is similar to the steps from each riffle to pool in RSCs (Bombardelli et al. 2011, Chatila and Tabbara 2004, Nikseresht et al. 2013). Yet, no numerical simulations of RSCs as a whole have been completed. Such studies are needed to examine boundary shear stress (τ0) and turbulence characteristics for pools to provide better design guidance on pool sizing whereby erosion potential is most efficiently reduced and structural stability is maximized, as stabilizing eroding channels is a key objective for installing RSCs to reduce fine sediment loads to receiving streams.

Currently, the most established design guidelines for RSCs were developed by Anne Arundel County, Maryland (used in the design of the RSC under investigation) (Flores et al. 2012). These guidelines are intended to provide information for the designer to properly size an RSC that will adequately convey the 100-year storm event. Specifications such as maximum and minimum length and depths are provided (Flores et al. 2012); however, sizing of pool width is more vague. While these guidelines provide an excellent starting point for design, it is hypothesized that design parameters can be further fine-tuned to design a system that will target velocity and energy reductions, thus decreasing the potential for erosion and scour. This numerical modeling study aims to provide this insight.

This study uses the commercially available CFD program, FLOW-3D™, and hypothesizes that RSC system design can be improved to ensure stability of the RSC bed and bank, focusing on locations near the boundaries of each pool. The objectives are to (1) identify what pool geometry (length and width combination) provides the greatest velocity and energy reductions, thus maximizing internal system stability, (2) determine how high flow rates alter the previously established relationship between design geometry and performance, and (3) identify areas prone to erosion and scour through the evaluation of shear stresses. The significance of this work is that it aims to fine-tune the already existing design guidelines for RSCs by providing recommendations for creating a strategically designed system that maximizes velocity and energy reductions, insight that is currently lacking regarding design of RSCs. More specifically, this research provides recommended length to width ratios that help to strategically design RSC systems. Such improvements will help ensure long-term function by fostering a stable system capable of safely conveying flows.

2.2. Field Site Description

2.2.1. Site Location

The RSC investigated in this study is located at South Doyle Middle School in Knoxville, TN (lat: 35.952003, long: -83.893768). The conveyance collects water from a 2.0 ha, 74% impervious watershed comprised of a school building, parking lot, and associated grassed areas. Stormwater enters a series of inlets located in the school parking lot and travels through a 0.61 m diameter concrete pipe until it reaches the RSC. Due to the high imperviousness of the drainage area, surface flow in the RSC occurs with as little as 0.2 cm of precipitation. Once stormwater leaves the conveyance, it discharges to Baker Creek, a tributary of the Tennessee River located within the Watts Bar Lake subbasin (Hydrologic unit code (HUC) 8: TN06010201). Baker Creek is listed on the 303(d) list of impaired streams for the designated uses of Fish and Aquatic Life and Recreation, resulting from excessive pollutant loadings from pathogens, nutrients, and habitat alteration (EPA 2014). Prior to installation of the RSC, the conveyance was lined with riprap on the channel bottom, providing minimal storage and acting as a source of suspended sediment to Baker Creek from bank failures.

2.2.2. RSC Design

The RSC consists of a cascade and three riffle-pools sections, totaling 18.9 meters from the outfall to the stream bank, and having an average slope measuring 0.05 m/m. The cascade (an area of increased elevation drop relative to the riffles) is necessary for this particular system because of road constraints and the location of existing bedrock. Half of the elevation loss occurs in the cascade, leaving an elevation change of -0.09 m and -0.06 m to occur at each of the subsequent riffle sections. The short riffle sections are comprised of sandstone cobble (d50 = 23 cm) with boulders (approximate dimensions: 0.3m x 0.6m x 1m each) serving as steps from one pool to the next. The pools are underlain with a soil media consisting roughly of 80% sand and 20% hardwood mulch, backfilled to a depth of approximately 0.5 m (Figure 2.1).

While designing the RSC, the pool was intended to have a residual depth of 0.46 m, measured from the bottom of the pool to top of the riffle leaving the pool, (Lisle 1982, Thompson and Hoffman 2001). This is the minimum pool depth per Anne Arundel County, Maryland design specifications (Flores et al. 2012). However, during construction, bedrock was encountered, preventing the pool from reaching the targeted design depth. Resulting pool depths were roughly 0.35 m.

2.2.3. Monitoring

The outfall of the culvert into the RSC has been retrofitted with a compression ring mounted with a compound triangular-rectangular sharp-crested weir to determine flow rate (Figure 2.2a). An additional compound triangular-rectangular sharp crested weir was installed at the discharge point of the RSC to Baker Creek (Figure 2.2b). Flow depth was determined using a Teledyne

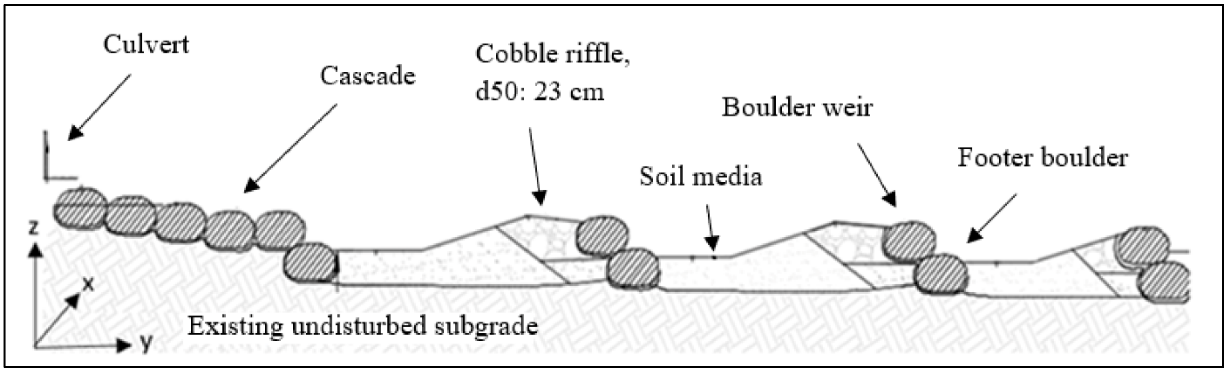


Figure 2.1 Longitudinal cross section of Regenerative Stormwater Conveyance.

Isco 4230 Flow Meter, with the bubbler tubing being mounted upstream of each of the weirs. A stage-discharge relationship for each weir was used to convert flow depth to flow rate (Jan et al. 2006, Martinez et al. 2005).

2.3. Methods

In order to create the three-dimensional CFD model, a LIDAR (Light Detection and Ranging) survey was performed at the project site. This remote sensing method uses pulsed light to measure the distance to an object, in our case, the surface of the conveyance system. The LIDAR scan was performed using the Faro Focus 3D X330 scanner, completed at a one-centimeter resolution. The resulting point cloud dataset was then imported into Civil3D (version 2015) to generate a three-dimensional (3D) solid for use in the model. Due to the high resolution of the scan (1 cm) and the associated computational requirements, the file was simplified to reduce the density of the point cloud by a factor of 0.33 prior to creating the 3D solid. Three-dimensional solid representation of the RSC was imported into the commercially available CFD program, FLOW-3D™ (version 10.1), developed by Flow Science, Inc. Using the FLOW-3D™ graphical user interface, the stormwater pipe conveying water into the system was created and all objects embedded into a rectangular, computational mesh. Field verified inflows (based on flow monitoring at the inlet) were simulated through the system and resulting velocities were spot checked for model calibration (further detail provided below). After successful model calibration, simulations were run to complete a performance analysis on system geometry. In the performance analysis, parameters were modified in AutoCAD Civil 3D, simulated in FLOW-3D™, and subsequent results used to develop dimensionless ratios (i.e. length:width) to determine the optimum design parameters for RSCs when internal erosion, velocities, and energies are minimized. Length to width (L/W) ratio was chosen as the explanatory variable

a. b.

Figure 2.2a-b Compound weir located at a) pipe outlet/RSC inlet and b) outlet to Baker Creek.

because it best describes the surface geometry of the pools. Once the ideal depth was determined, it remained constant for the remainder of the investigation, making the L/W ratio the primary variable for subsequent sensitivity analysis.

2.3.1. Numerical Model Development

CFD programs use a series of equations and algorithms to approximate a solution for the individual cells within the mesh. The governing flow equations used in FLOW-3D™ are conservation of mass and momentum via the Navier-Stokes equation, which is solved using the finite-difference approximation method. Average values for each of the dependent variables are taken at the center of the cell block, except for velocity, which is determined along the face of the cell (Flow Science 2012). The Volume of Fluid (VOF) method (Hirt and Nichols 1981) is used to model free surfaces of fluids and account for boundary conditions of a structured computational mesh. With this method, the program determines the fraction of cell volume occupied by liquid, represented by the variable "F", thus, the surface is defined at the locations where F changes from 0.0 to 1.0 (Flow Science 2012). A system of rectangular cells is used to form the structured computational mesh, along with the FAVOR (Fractional Area/Volume Obstacle Representation) technique to compute open area fractions on cell faces and open volume fractions to reconstruct object geometry. The advantage of this technique is it provides a simpler, less time consuming procedure than body-fitted grids, while maintaining accurate object representation (Flow Science 2012).

For the RSC model, the following boundary types were used: "volumetric flow rate" defined at the beginning of the pipe section, y_{min}; "outflow" set at conveyance outlet, y_{max}; and all other boundaries (x_{min}, x_{max}, z_{min}, z_{max}) set to "symmetry". The simulated pipe length was 9.14 meters to allow fluid to reach equilibrium before entering the conveyance system. For this study, the Renormalized Group (RNG) k-ε model, a more robust version of the standard k-ε model, was chosen. The RNG k-ε model calculates turbulent kinetic energy, k, dissipation rate, ε, and dynamically determines the turbulent mixing length, LT (Flow Science 2012, Yakhot and Smith 1992). The RNG k-ε model is better suited for calculating turbulent flows within complex bedforms (Bradbrook et al. 1998), such as the riffle-pool sections with varying sizes of cobble and boulders found in the RSC.

While creating the model in FLOW-3D™, there were a few discrepancies that existed between the model and field site. First, the numerical model did not consider the porosity of the sand/wood chip media, and thus did not account for storage within the system. Secondly, the model did not represent the void spaces between the cobble and boulders, only simulating flow over the riffles, rather than through. However, these two simplifications will only result in overestimation of velocities and volume within in the system, and provide insight in to worst case scenarios (i.e. flow through a fully saturated system). Further, as understanding the internal stability of the system is the primary goal, function during larger storms (when saturation of the underlying soils is likely) is of primary importance.

2.3.2. Calibration and Sensitivity Analysis of Numerical Model

To calibrate the model, surface roughness was adjusted until field measured velocity magnitudes matched model output velocity magnitudes as closely as possible. Velocity magnitudes within the system were spot checked in the field using a portable Marsh-McBirney Model 2000 Flo-Mate flow meter. As this instrument only measures velocity, and not direction, it was assumed that the meter was parallel to the flow vector at the time of measurement. The Nash-Sutcliffe efficiency (NSE) coefficient ($-\infty$ to 1) was used to determine the model's capability of predicting surface flow velocities (Nash and Sutcliffe 1970). Surface roughness was then adjusted until the greatest efficiency coefficient was achieved. One outlier was identified and removed from the goodness of fit analysis under the criteria that the percent error was greater than 1.5 times the interquartile range. The removal of the outlier was further justified by the likelihood that there was a discrepancy in flow conditions for the field location versus the modeled location. Specifically, it is suspected that the location of the modeled data point was in the line of the concentrated flow path, when the field measured data point was not, resulting an extremely large percent error. The resulting NSE was 0.66, indicating the model is a relatively good predictor of velocities within the system. Results of the goodness of fit for the calibration analysis can be found in Figure 2.3. Simulated flow velocities were found to be slightly lower than measured in the first pool of the system and slightly higher in the last pool of the system due to the uniform application of the roughness parameter across the grid. FLOW-3D™ does not allow for roughness to be adjusted by region while using a single mesh block; and use of multiple mesh blocks introduced instabilities. An additional source of error between measured and modeled velocities resulted from the model not accounting for porosity or flow through cobble/boulders, causing modeled velocities at the end of the system to be greater than field velocities.

The model output was found to be sensitive to changes in mesh size. The sensitivity analysis consisted of reducing the mesh size until only minimal changes in output were detected. A final mesh size of 6.1 cm was chosen to represent the model. Initially, simulations were run with a mesh size of 7.6 cm; however, it was determined that instabilities existed at this larger resolution. Further, a simulation was run using a smaller mesh size of 4.6 cm, resulting in a velocity difference of only 5% between the 4.6 cm and 6.1 cm size; however, simulation run time was much longer with the smaller mesh size. As the research relies on numerous simulations of varying design parameters, any mesh finer than 6.1 cm resulted in processing time

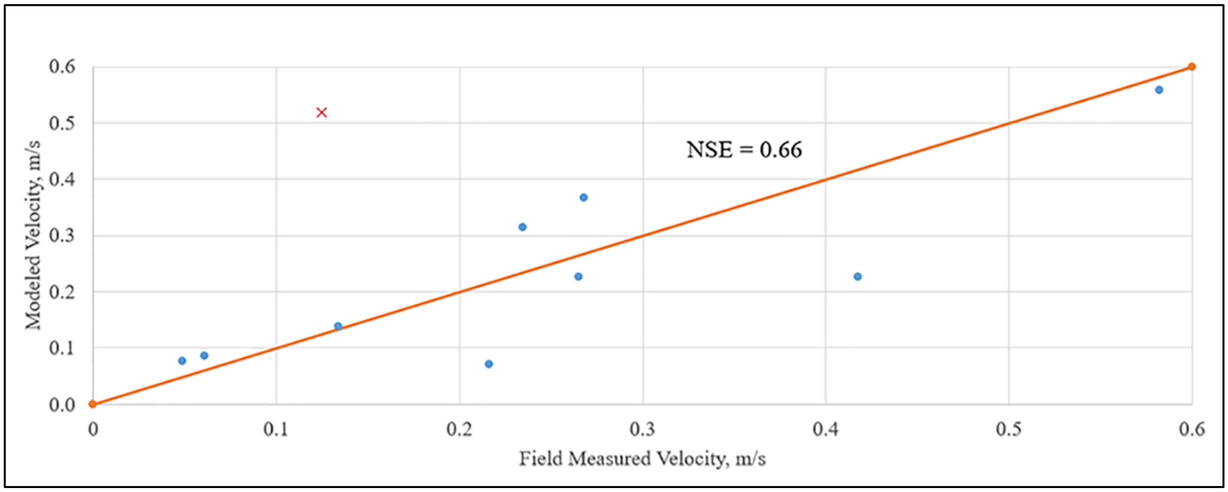


Figure 2.3 Goodness of fit for modeled vs. observed velocities.

and output file sizes infeasible for the scope of the research. Thus, the authors concluded a mesh size of 6.1 cm was sufficient for the simulations given the relatively minor increase in accuracy and increased time requirements of a finer mesh.

2.3.3. Performance Analysis of RSC Design Parameters

The performance analysis of the design parameters within the RSC pools was intended to aid engineers in designing the most stable system that maximizes velocity and energy dissipation while minimizing erosion potential (without over use of resources). RSC pools are oval in shape, with length and width referring to the maximum dimension for each of the measurements. System modifications were made to the original LIDAR scan, individually adjusting depth, length, and width, with a primary goal of keeping the remainder of the geomorphology intact. This was done by selecting the bottom section of pools and adjusting elevation to modify depth, then scaling pools in the x and y directions to modify width and length, respectively. Once the optimal design depth was identified through a qualitative, visual investigation of simulation results, it was held constant for all future simulations while modifying length and width. Length and width parameters were then analyzed through quantitative measures, investigating velocity and energy dissipation as well as bed and boundary shear stresses. All design modifications were made in AutoCAD Civil 3D (2015) and imported into FLOW-3D™ for simulations.

A total of 23 simulations were run with varying design parameters (i.e. pool width, length, and depth) and flow rates. For the analysis of pool depth only, a visual analysis looked at the velocity magnitude and direction of velocity vectors at the centerline cross-section, perpendicular to flow through each of the pools. Simulations with a velocity jet or downward directed vectors present were considered undesirable, as these are conditions where scour is likely to occur. Further, values were compared against maximum allowable velocity criteria for coarse sand (1.22 m/s) as determined by the Army Corp of Engineers (Li 2008). Any value above the allowable velocity indicates the likelihood of bed material to be transported.

For the length and width analysis, both depth averaged velocity (DAV) and depth averaged turbulent kinetic energy (TKE) were investigated to quantify velocity and energy dissipation between various design geometries and from pool to pool. The analysis variable, DAV, was chosen as a measure of mass and momentum routing between vertical columns of fluid motion. TKE was chosen because it considers 3D velocity fluctuations and turbulence pattern as an indicator of energy dissipation spatially within the RSC (Noss et al. 2010, Shvidchenko and Pender 2001a, Shvidchenko and Pender 2001b, Tamburrino and Gulliver 1999). Further, bed and bank shear stresses were calculated and compared against critical values found in literature. Boundary shear stresses ($\tau 0$) were calculated using the "law of the wall" method and used to identify locations of bank/bed sediment erosion (Sturm 2010). The calculated values were compared to critical bed values determined from literature for medium grain sand ($\tau c = 0.194$ Pa) (Berenbrock 2008) and critical bank shear values for Steadman Silt Loam ($\tau c = 7.8$ Pa) (Mahalder et al. 2017).

Simulations were run at low flow (0.37 m^3/s), corresponding to field measured data and representative of a typical storm event (0.2 cm storm over 2 hours), as well as the 10-year (2.45 m^3/s), and 25-year (3.13 m^3/s), 24-hour duration storm recurrence intervals for Knoxville, TN. Storm events beyond the 25-year, 24-hour recurrence interval were not simulated due to the presence of instabilities in the model resulting from high turbulent fluctuations at greater flow rates. Numerical outputs from the simulation results were analyzed at nine locations within each of the three pools (Figure 2.4). These nine analysis locations were spaced at one-fourth the length and one-fourth the width, measured from the center of the pool to ensure the output was

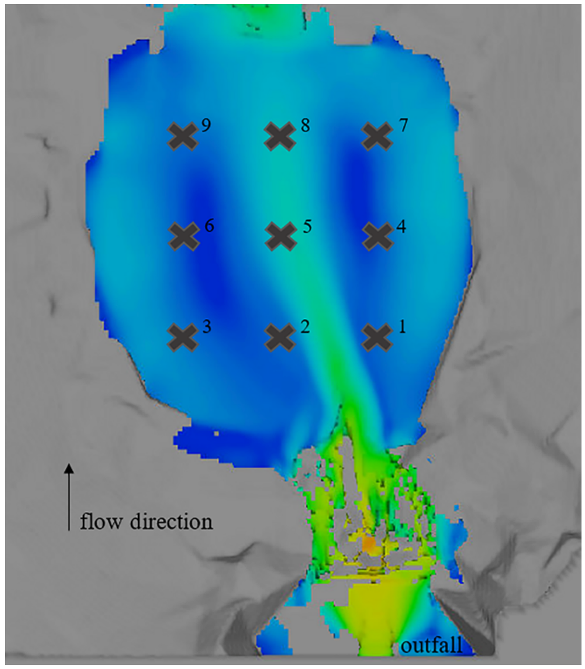


Figure 2.4 Nine analysis locations for each pool.

consistently measured for each design variation. Depth-averaged TKE and DAV, averaged for each pool using data from a vertical column at each of the nine analysis locations, were then plotted against length to width ratios to determine how varying design parameters affect flow through the system. The goal of the study was to identify which design maximizes velocity and energy dissipation while also minimizing $\tau 0$ at the bed and bank to prevent erosion, thus ensuring long-term system stability and performance.

2.3.4. Statistical Analysis

Results investigating the correlation between the hydraulic parameters (TKE and DAV) and L/W ratio were tested for statistical significance using the non-parametric Spearman's rank-order correlation. Analysis of the data sets were tested using SAS Enterprise Guide 7.1. Correlations were tested for significance at the 95% confidence level.

2.4. Results and Discussion

2.4.1. Optimal Design Depth

The first stages of the performance analysis focused on modifying residual pool depth only while holding the design length and width constant (see Table 2.1 for specific design length and width dimensions for each pool). Simulations were run at average pool depths of 0.23 m, 0.35 (constructed depth), 0.46 m, and 0.91 m. During surface geometry modifications, it was important to keep topography constant, therefore only the pool bed elevation was increased or decreased. As a result, the side slopes for the 0.91 m design depth exceeded the recommended 3:1 side slope. Since reaching this design threshold would require additional grading (which would make results from various depths incomparable) and because the scope of this research is not concerned with modifying side slope recommendations per the design guidelines, it was concluded the 0.91 m was undesirable and disregarded from the results analysis.

Table 2.1. Pool geometries for various simulations.

Pool	Scaled	Direction	Width, X (m)	Length, Y (m)	L/W
1	*Constructed*		*4.9*	*5.1*	*1.0*
	0.9	X	4.4	5.1	1.2
	0.8		3.9	5.1	1.3
	0.7		3.5	5.1	1.5
	0.9	Y	4.9	4.6	0.9
	0.8		4.9	4.0	0.8
	0.7		4.9	3.5	0.7
2	*Constructed*		*4.0*	*4.2*	*1.0*
	0.9	X	3.7	4.2	1.1
	0.8		3.3	4.3	1.3
	0.7		2.8	4.2	1.5
	0.9	Y	4.1	3.8	0.9
	0.8		4.1	3.4	0.8
	0.7		4.1	3.0	0.7
3	*Constructed*		*4.3*	*4.7*	*1.1*
	0.9	X	3.9	4.4	1.1
	0.8		3.5	4.3	1.2
	0.7		3.0	4.3	1.4
	0.9	Y	4.3	3.9	0.9
	0.8		4.3	3.5	0.8
	0.7		4.3	3.1	0.7

Note: The “scaled” column refers to the factor by which the original measurement was multiplied. “Direction” indicates the lateral direction of the measurement, where x applies to the pool width and y applies to the pool length.

Quantitative results of DAV and TKE in the depth analysis were inconclusive in determining optimal design depth. The variability observed through the quantitative analysis was likely a result of reducing the analysis to a pool averaged value for velocity, which does not properly describe the presence of jets through the pools – a characteristic that is intended to be minimized. Thus, to adequately describe the presence of high velocity jets found in pools, a decision for optimal design depth was made based on visual investigation of velocity magnitude, direction of velocity vectors, and analysis of literature. Figure 2.5 illustrates the velocity magnitude and vectors taken at cross sections located at the center of each pool (y=10.52 m, 16.35 m, and 20.76 m for pools 1, 2, and 3, respectively) for varying depths. It should be noted that velocity magnitudes at all depths investigated (Figure 2.5) are less than the 1.22 m/s maximum allowable velocities in channels with coarse sand (Li 2008), meaning the pools are stable from scour at this simulated low flow event, yet is important to eliminate velocity jets as they will intensify at higher flow rates, potentially leading to scour.

Visual investigation of the velocity magnitude at the center of each pool identified the presence of a high velocity jet at depths 0.23 m and 0.35 m in pool 1. At the 0.46 m depth, this high-velocity had been mostly dissipated. This adheres to mass continuity, in that velocity must decrease as cross-sectional area of flow increases under constant flow rates. Further, locations of jets through pools should be avoided when possible because they produce high shear stress as well as initiate movement of sediments (Caamaño et al. 2012). Pool 1 illustrated the greatest differences in velocities at varying depths, with little variation seen for subsequent pools. Higher dispersed secondary circulation was observed with the pool depth of 0.46 m, indicating a well-mixed flow. Of the modeled depths, it was concluded the 0.46 m residual depth was most optimal in minimizing velocity magnitude, while also maintaining stable side slopes. Thus, this

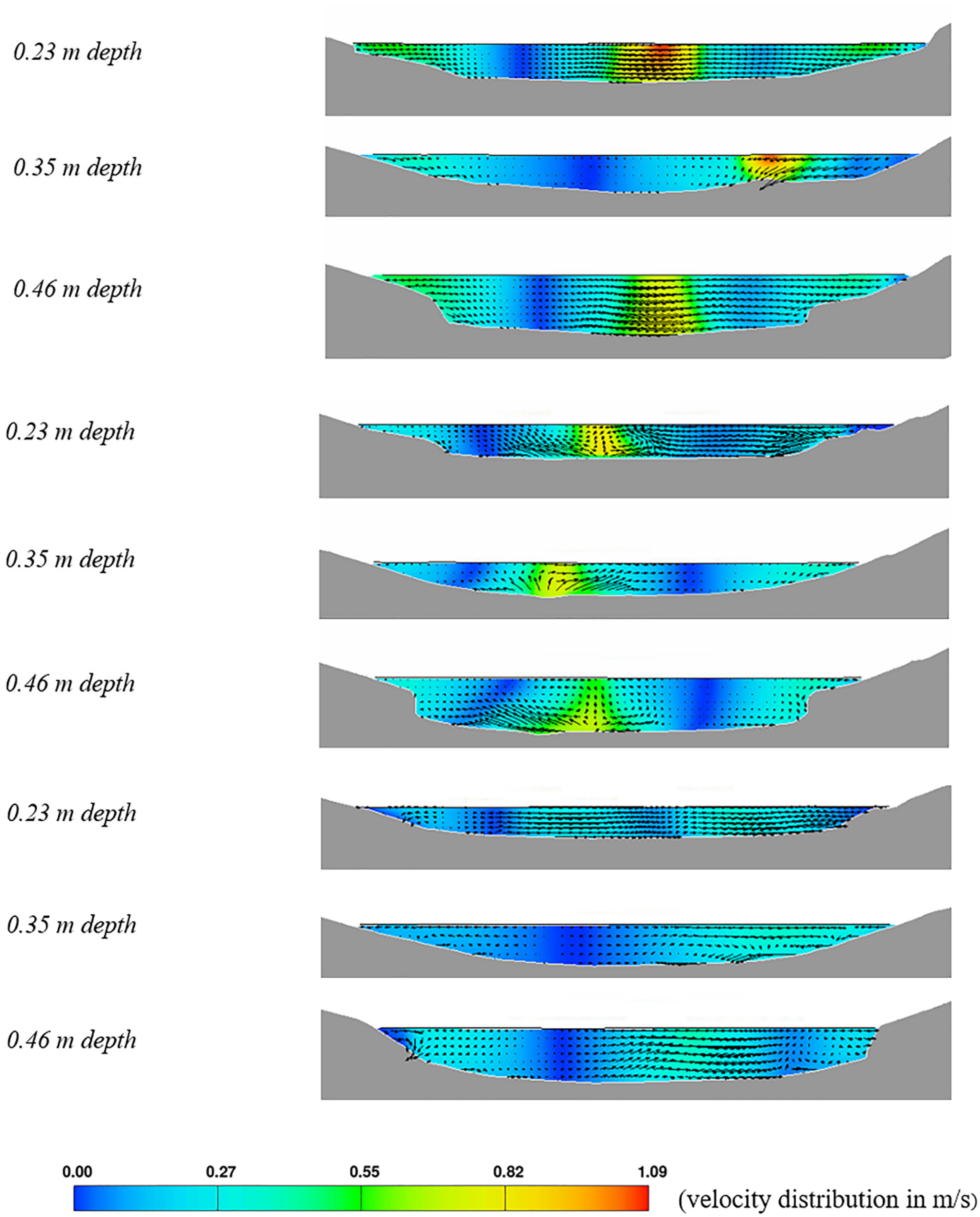


Figure 2.5 Velocity magnitude in a) pool 1, b) pool 2, and c) pool 3 at varying depths for flow rate = 0.37 m^3/s. Cross section for each depth taken at center of pool (pool 1: y=10.52, pool 2: y=16.35 m, and pool 3: y=20.76 m). Velocity units are m/s. The 0.35 m depth is the constructed depth.

depth was used for future simulations modifying length and width. It should be noted that this depth also corresponds with the minimum depth requirement specified in the design guidelines from Anne Arundel County, Maryland. Deeper pools (greater than 0.46 m) may be equally or more adequate under varying site constraints and are worthy of future analysis.

2.4.2. Optimal Length and Width

The second phase of the performance analysis held the residual pool depth constant and modified length and width independently from one another. Table 2.1 lists the original and modified pool dimensions for each of the simulations at a 0.46 m depth. The intent of this portion of the study was to determine if a relationship exists between design parameters and flow characteristics. In order to illustrate this, depth averaged velocity (DAV) and depth averaged TKE for each pool were compared against the dimensionless length to width ratio (L/W), at low flow (0.37 m^3/s) (Figures 2.6-2.7). It was found that there is strong correlation between the hydraulic parameters and L/W ratio in the first pool (DAV: $r_s=0.96$, and $p<0.05$; TKE: $r_s=0.96$, and $p<0.05$), but correlation decreased in the subsequent pools and was no longer considered statistically significant (Table 2.2). In regard to pool 1, the plots indicate that pools with lower L/W ratios (i.e. short and wide; large cross-sectional area) have lower DAV and TKE. Again, this was expected as it adheres to flow continuity. It should be noted that Anne Arundel County Design Guidelines state pools should be at least 3.05 m in length and twice the riffle length (Flores et al. 2012). It is expected that there is a minimum L/W ratio where DAV and TKE are no longer minimized; however, this research did not investigate any pool dimensions below the recommended threshold. Additionally, this research was limited to L/W ratios that would not lead to substantial changes in the required area of the project. Total system length was held constant to accommodate the upstream and downstream connection points (i.e. the stormwater

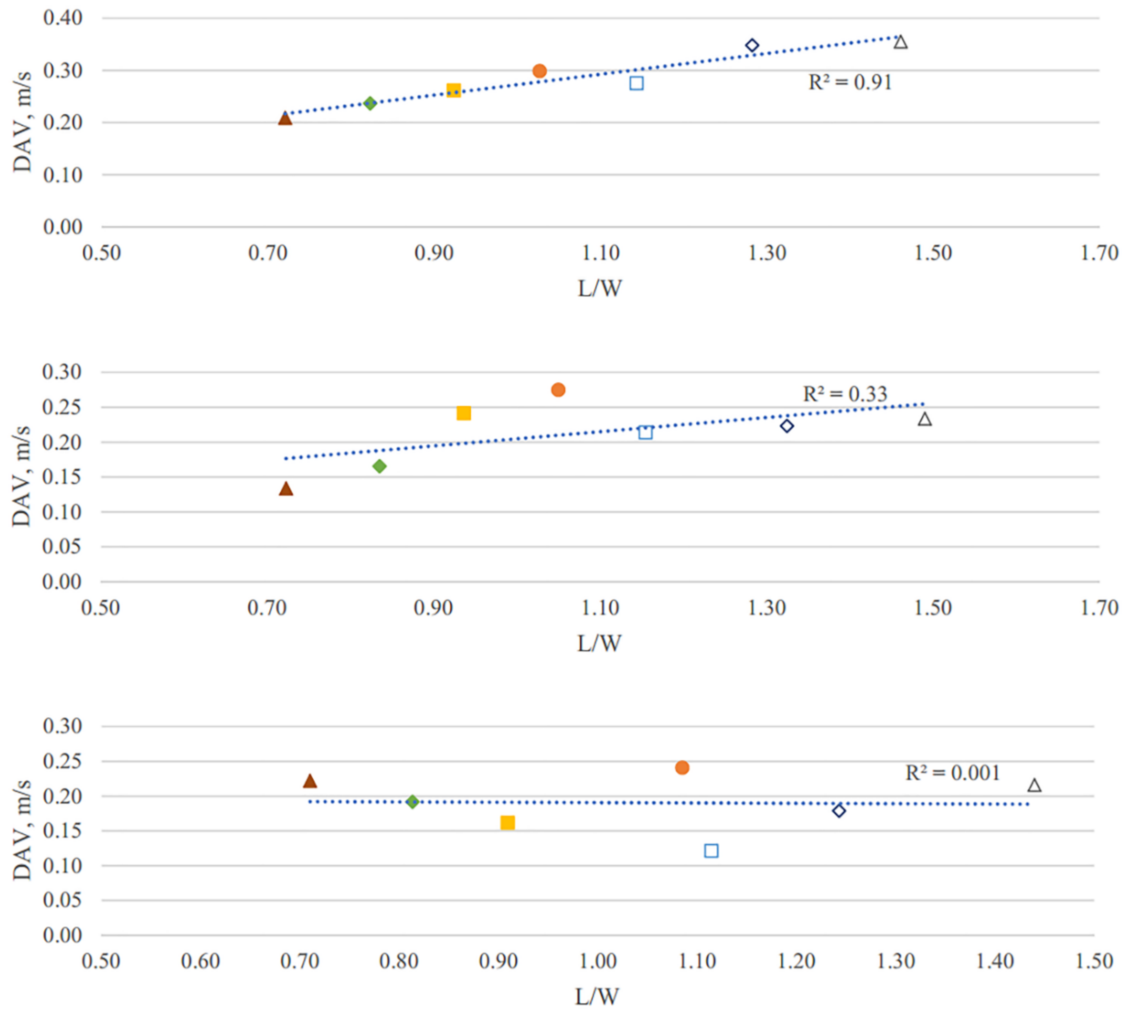
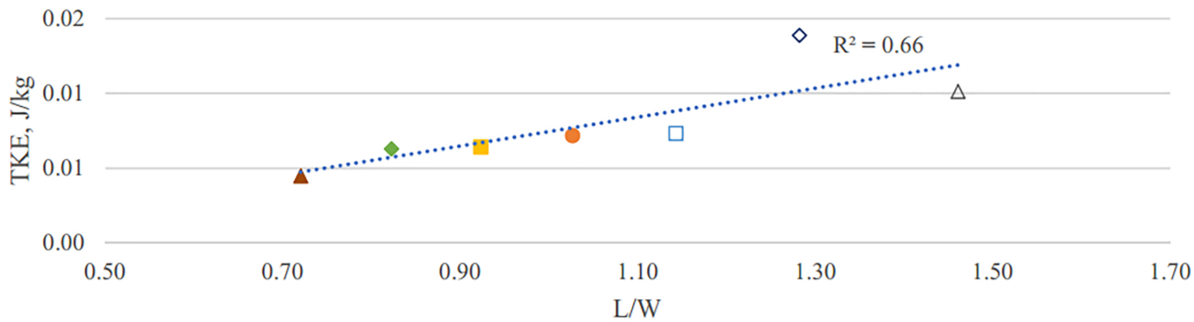
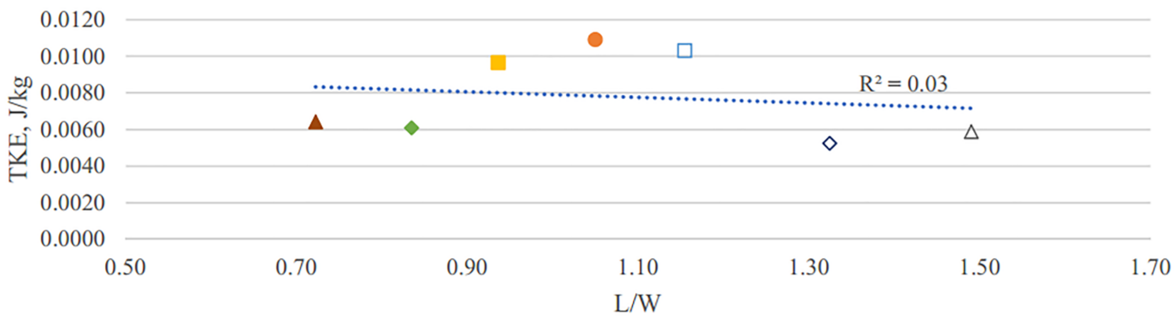


Figure 2.6 DAV vs L/W ratio for a) pool 1, b) pool 2, and c) pool 3. It should be noted that correlation between DAV and L/W weakens as flow progress through each pool. The data point represented by a circle in each plot represents the constructed pool geometry. See Table 2.1 for description of L/W parameters.





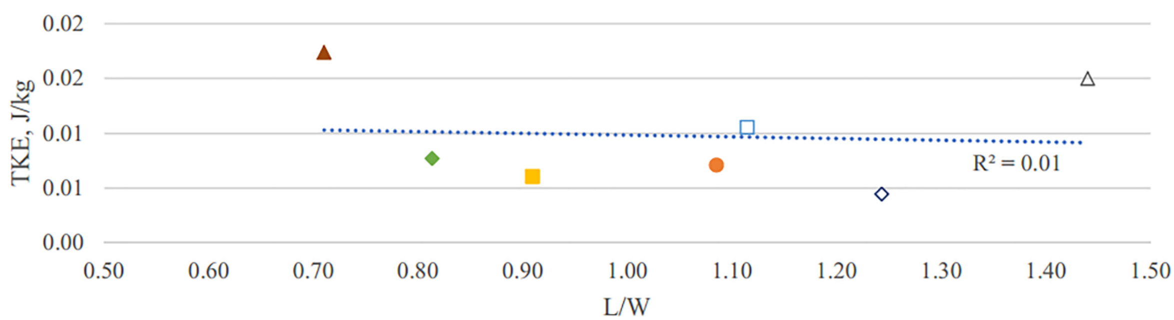


Figure 2.7 TKE vs L/W ratio for a) pool 1, b) pool 2, and c) pool 3 at the 0.46 m depth. It should be noted that correlation between TKE and L/W weakens as flow progress through each pool. The data point represented by a circle in each plot represents the constructed pool geometry. See Table 2.1 for description of L/W parameters.

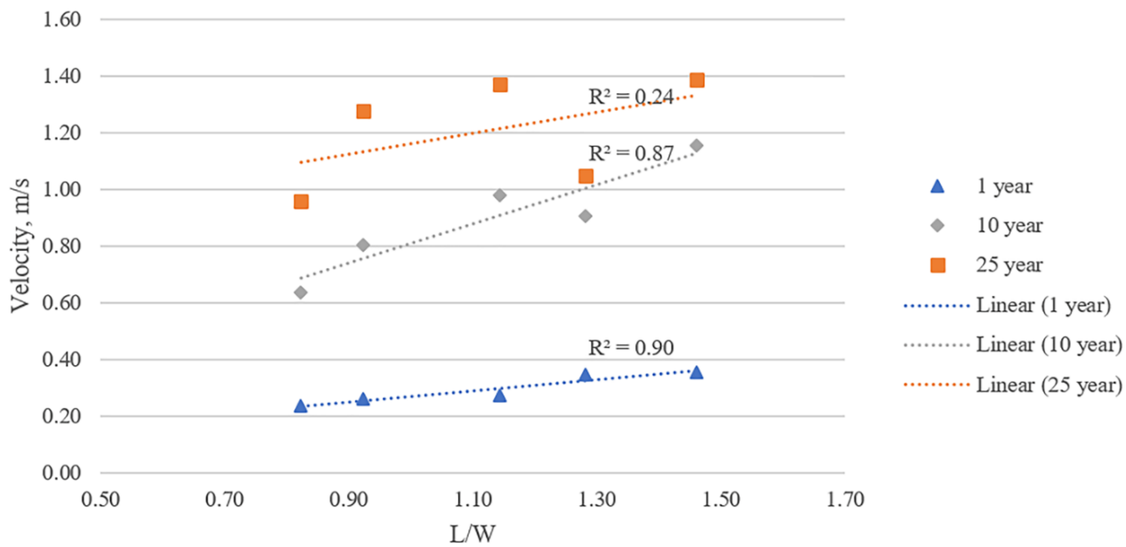
Table 2.2. Results of the Spearman's Rank Order Correlation analysis to determine significance in correlation between hydraulic parameters and L/W ratios.

Simulation	r_s	p	Confidence Level = 95% (α=0.05)	Significance
DAV				
Pool 1	**0.96**	**0.0005**	0.05	**significant**
Pool 2	0.46	0.29		not significant
Pool 3	-0.21	0.64		not significant
TKE				
Pool 1	**0.96**	**0.0005**		**significant**
Pool 2	-0.32	0.48		not significant
Pool 3	-0.21	0.64		not significant

outfall at the upper end and Baker Creek at the lower end). While looking at the trends in pools 2 and 3, there is little to no correlation found for varying L/W ratios. This is most likely attributed to the majority of the velocity and energy being dissipated in pool 1. The variability observed in pools 1 and 2 is likely dependent on the unique morphology of the pool's physical structure. Therefore, these results indicate that dimensions of the first pool are most critical for design of RSC systems, and it appears a lower L/W ratio is desirable for greater energy dissipation.

2.4.3. Varying Flow Rates

To investigate the impact of variable flow rates on system design, simulations were performed at a depth of 0.46 m with the constructed length and width dimensions held constant at L/W ratios of 1.03, 1.05, and 1.09, for pools 1, 2, and 3, respectively. The only significant correlation among varying flow rates was found within pool 1, therefore only that pool is presented in this discussion (Figures 2.8a-b). In pool 1, it was found that as flows increase, the slope of the regression line also increases for TKE (the trend was less consistent for DAV). This suggests that the hydraulic parameters appear more responsive to changes in pool geometry at higher flow rates, further reinforcing the previous finding that the design of the first pool is most critical and becomes even more important at higher flow rates. Additionally, the correlation decreased significantly under the 10-year and 25-year storm event flow rates. The low flow event showed strong correlation between DAV and TKE versus L/W, but drastically diminished under higher flow rates (Table 2.3). It is expected that at some point between the low flow and the 10-year event, the flows within the RSC are no longer captured within the dimensions of the pool, and instead are completely inundating the system. At this point, design of the pool will have little to no effect on flows within the system and the focus of the RSC shifts to safely conveying surface flows from the inlet to outlet with minimal erosion.



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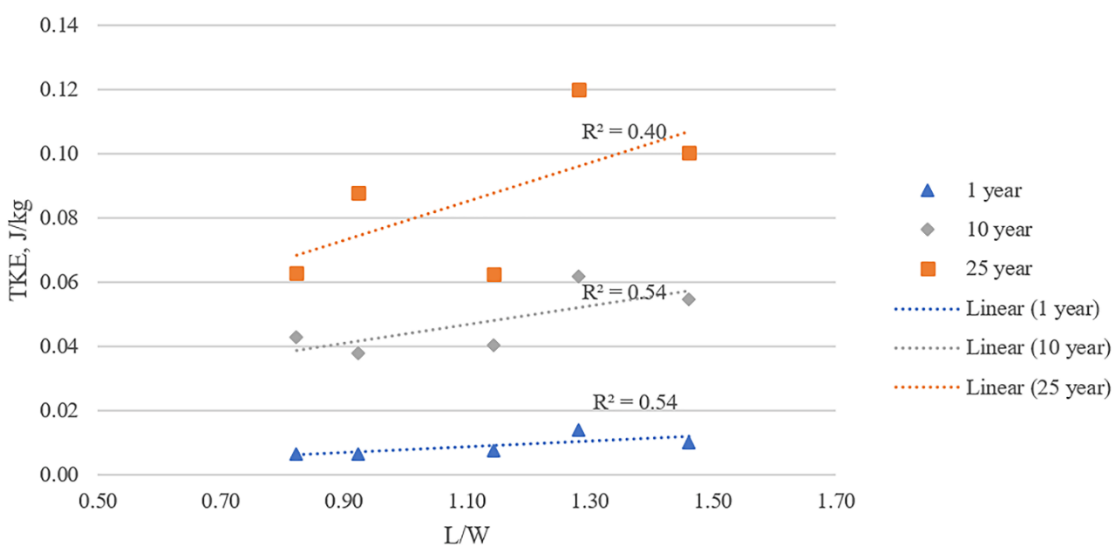


Figure 2.8 Hydraulic parameters a) DAV and b) Depth averaged TKE plotted against length to width ratios for <1, 10, and 25-year storm recurrence interval. Only data for pool 1 is represented. Values for DAV and TKE are averaged across each pool for each design simulation.

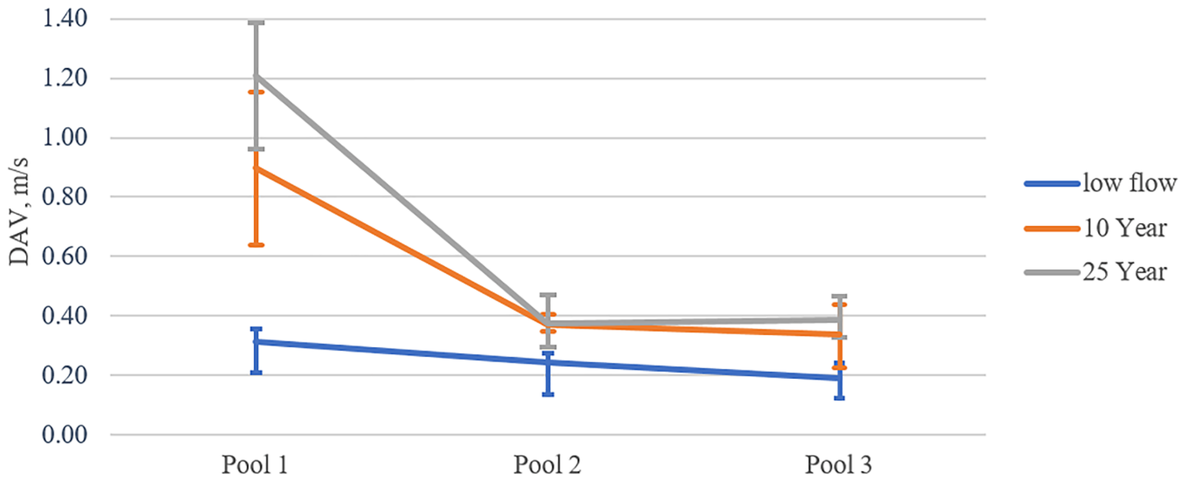
Table 2.3. Results of the Spearman's rank order correlation analysis to determine significance in correlation between hydraulic parameters and L/W ratios at varying flow rates.

Simulation	r_s	p	Confidence Level = 95% (α=0.05)	Significance
DAV				
Low flow	**1.0**	**<0.0001**	0.05	**significant**
10	0.1	0.87		not significant
25	0.3	0.62		not significant
TKE				
Low flow	**0.9**	**0.04**		**significant**
10	-0.1	0.87		not significant
25	-0.1	0.87		not significant

Figures 2.9a-b show DAV and depth averaged TKE averaged for all simulations under each storm size, which are then plotted by pool. The results show that the greatest reductions in DAV and TKE occur from the first to second pool at high flow rates (10-year and 25-year recurrence interval). From pool 2 to 3, there are little reductions seen in regard to DAV, but for TKE, energy dissipation is still occurring. This illustrates that the hydraulic parameters have not reached a state of equilibrium and should eventually reach a point where no further reductions are observed under steady flow conditions. For the case of TKE, based on the results, it is expected that the energy dissipation would plateau if one or two more pools were present. Thus, for larger storms, the design of the first few pools may be critical in terms of velocity and energy dissipation, and ultimately system stability. However, the exact number of pools needed would depend on the specific site conditions, such as incoming flow rate, available width, and slope. This is an important finding, as there is also little guidance presented regarding the required length of RSCs. These findings indicate that there is a minimum length (i.e. number of riffles and pools) needed to bring influent flows down to steady flow.

2.4.4. Evaluation of Shear Stresses

One of the primary water quality treatment mechanisms of RSCs is the reduction of suspended sediments from stormwater; therefore, it important to design in a way such that sediment transport is minimized. Further, points of scour within the systems may act to excavate the underlying sand seam, and have the potential to reduce the stability of boulder structures. Lastly, severe erosion of the bank has the potential to result in bank failure, further contributing to system instabilities and sediment transport. In this section, shear stresses (τ_0) were evaluated to determine if bed and bank erosion were occurring.



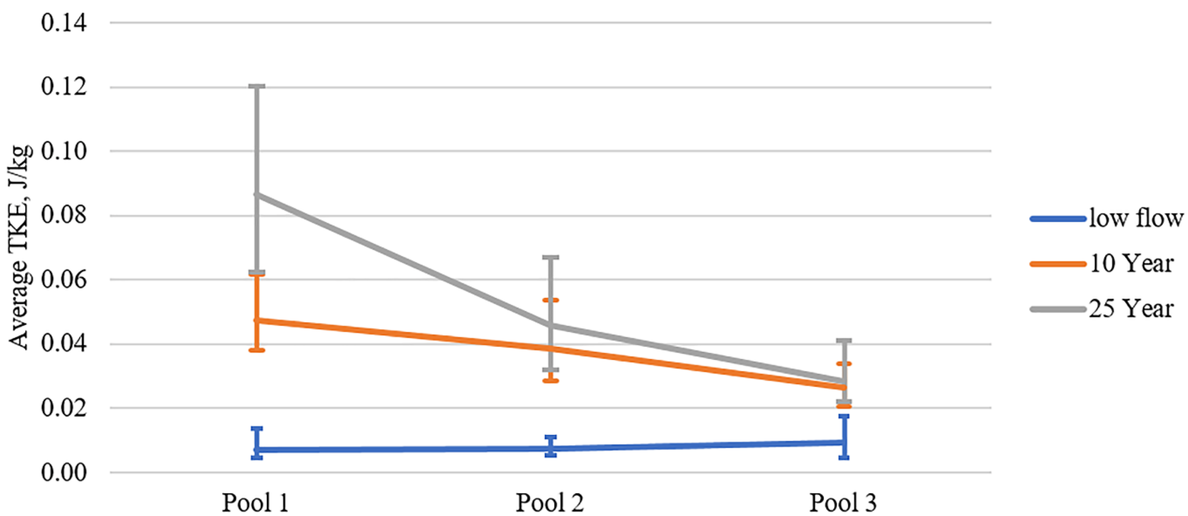


Figure 2.9 a) Depth averaged velocity and b) depth averaged turbulent kinetic energy for low flow, 10, and 25-year storms across each pool. Each storm represents the average of all simulations, with maximum and minimum values represented by error bars.

Results found that the only simulations to produce bank shear stresses greater than the critical value (τc = 7.8 Pa) (Mahalder et al. 2017) were under the 10-year and 25-year storm recurrence interval. Further, these instances were confined to the first pool and were only observed at greater length to width ratios. For the investigation of critical bed shear stresses (τ_C), locations 1, 2, and 3 are disregarded because footer boulders are located in each pool at these locations for added bed stability (See Figure 2.4). Table 2.4 lists the results of shear stress for all locations at various simulations under low flow conditions (0.37 m^3/s). Values that exceed the critical value for movement of bed material (τc = 0.194 Pa) are in bold. There are only five instances where critical bed shear was exceeded, with the majority of them being at the median L/W ratio or greater. These results from bed and bank shear stress analysis reinforce the conclusions made previously, that lower L/W ratios are the ideal design parameters for ensuring system stability by maximizing velocity and energy dissipation. However, the results do not point to any one particular pool as having greater occurrence of sediment transport than the other. Lastly, results of the shear stresses at higher flow rates are inconclusive in determining a specific pool geometry where shear stresses are minimized, only that lower L/W ratios appear superior.

2.5. Conclusions

The results of the qualitative assessment of pool depth on RSC stability suggest the validity of the currently proposed design specifications, that is, 0.46 m depth is the minimum depth that should be considered for design. This was primarily a concern for the first pool of the system, where erosive flow rates were greatest, and the most care should be taken to optimize design. Greater depths may be considered, but could not be analyzed in this study while maintaining

Table 2.4. Bed shear stresses. Values in bold indicate exceedance of critical value (τc = 0.194 Pa). Refer to Figure 2.3 for detailed description of analysis locations. All results listed below are for a pool depth of 0.46 m. Each column represents a different pool dimension, scaled from the original pool dimension. For example, "Scaled 0.7 Y" is 70% of the original Y dimension.

Shear Stresses, Pa								
	Analysis Location	Scaled 0.7 Y	Scaled 0.8 Y	Scaled 0.9 Y	Original Dimensions	Scaled 0.9 X	Scaled 0.8 X	Scaled 0.7 X
		Lowest L/W			Median L/W		Highest L/W	
Pool 1	4	0.0020	0.0003	0.0002	0.0438	0.0727	**0.1993**	**0.7626**
	5	0.0396	0.0003	0.0558	0.0180	0.0206	0.0076	0.0087
	6	0.0055	0.0005	0.0003	0.0001	0.0041	0.0022	0.0082
	7	0.0225	0.0322	0.0227	0.0007	0.0132	0.0766	0.0146
	8	0.0920	0.0188	0.0000	0.0030	0.0013	0.0005	0.0461
	9	0.0144	0.0029	0.0977	0.0113	0.0536	0.0072	0.0061
Pool 2	4	0.0000	0.0038	0.0015	0.0090	0.0004	0.0258	0.0024
	5	0.0007	0.0006	0.0016	0.0078	0.0608	0.0002	0.0466
	6	0.0282	0.1738	0.0227	0.0678	0.0411	0.1833	**0.2064**
	7	0.0061	0.0001	0.0000	0.0003	0.0025	0.0044	0.0006
	8	0.0101	0.0214	0.0136	0.0484	0.0264	0.0014	0.0035
	9	0.0171	0.0515	0.0275	0.1646	0.0200	0.0827	0.0233
Pool 3	4	0.0003	0.0029	0.0000	0.0001	0.0356	0.0104	0.0005
	5	0.0336	0.0007	0.0666	**0.3642**	0.0020	0.0036	0.0006
	6	0.0822	0.0012	0.0003	0.0030	0.0072	0.0005	0.0334
	7	0.0030	0.0001	0.0007	0.0020	0.0051	0.1107	0.0008
	8	0.1757	**0.2143**	0.0863	0.0171	0.0202	0.0424	0.0782
	9	0.0066	0.0105	0.0006	0.0005	0.0000	0.0019	0.0048

comparability among simulations. Investigation of greater pool depths would be an area of potential future study.

The second portion of the study focused on varying pool lengths and widths, concluding that lower L/W ratios are superior for reducing velocities and energies, and that the design of the first pool in the RSC system is the most critical. Specifically, the results of the study found that lower DAV and TKE were found at lower L/W ratios, with the hydraulic parameters increasing as the L/W ratio increased. This trend was most evidenced by a strong correlation of pool hydraulic parameters to geometries in the first pool, but weakened correlation as flow progressed through the subsequent pools. This conclusion was further supported by the finding that at high flows, the greatest velocity and TKE reductions occurred in the first pool of the system. However, for larger flows, TKE reductions were still found to occur in the second pool, suggesting that geometry of the second pool also has influence on hydraulic parameters during extreme events. It was also suspected that if additional pools were in the system, TKE would reach a point of no further reduction (i.e. steady flow), similar to the trends seen in DAV. Therefore, to ensure adequate stability under a wide range of storm events, it is recommended that, at minimum, the first two pools be designed with optimized L/W ratios to reduce velocities and energies. However, more “critical” pools (those where substantial energy dissipation occurs) may be necessary depending on specific site conditions, such as flow rate, width, and slope. As current design requirements do not specify a necessary length of RSC, these results suggest that thresholds for length do exist and are important in establishing steady, low velocity flow conditions prior to discharge to surface waters. More specifically, this study suggests that pools be sized at a L/W ratio of 0.7; however, additional field scale studies are necessary to refine this initial estimate of a threshold ratio.

Despite the advancement in understanding RSC function during large storms events provided by this study, further study is needed to continue to improve design specifications for these increasingly utilized SCMs. Primarily, field scale studies are necessary to validate model results in order to refine the initial suggestions made herein. Secondly, as this analysis was performed on one location, future studies targeting sites with varied site characteristics from those herein should be performed to more fully understand the applicability of these design parameters under a wide range of conditions. Future modeling of RSCs would also be greatly enhanced by inclusion of the underlying sand seam, particularly for small storms. Determining the influence of the size of the underlying seam on stormwater volume reductions would be a novel contribution. Last, further refining guidance on the design lengths required for RSCs to perform optimally is an important future step.

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3. Groundwater Interactions with Regenerative Stormwater Conveyance Infiltration: A Case Study, Knoxville, TN

Abstract

A regenerative stormwater conveyance (RSC) investigated in Knoxville, TN, found that there were significant influences on the groundwater system resulting from infiltrated stormwater within the RSC. The degree of influence the RSC had, and the variability of the groundwater elevations in general, resulted from seasonal fluxes in water table elevation, varying weather patterns and storm event size, and most significantly, the presence of two buried sewer lines. Mounding was observed below the RSC during small to medium storm events (with the sewer acting as a sink), but the mounding shifted to the areas adjacent to the conveyance during larger events. The most prominent factor influencing trends in groundwater interactions at this research site appeared to be the presence of the sewer lines. It is important to understand how the urban subsurface infrastructure network will influence groundwater interactions in order to avoid any unintended consequences or short circuiting of water quality treatment mechanisms. These findings will benefit the design community, informing them of potential challenges when constructing RSCs in urban environments where there can be significant buried infrastructure.

3.1. Introduction

Urbanization is known to fundamentally shift the water balance in watersheds, with severe implications for stream biogeochemical systems. Even prior to the 1980s, the potential of urbanization to deplete aquifers and reduce stream baseflow by shifting stormwater transport from subsurface flow to predominately surface flow was recognized and described literature (Leopold 1968). While this can be true in certain scenarios, without external influences, studies have also indicated that urbanization may contribute toward an overall water recharge as a result of added infrastructure (i.e. Bhaskar et al. 2016, Howard and Gerber 2018, Hutchinson and Woodside 2002, Lerner 1986). As outlined in a review by Howard and Gerber (2018), these recharge sources may be intentional or unintentional and include leakage from water supply lines and sewer pipes, exfiltration from septic systems, excessive watering of gardens and parks, and infiltration from stormwater control measures (SCMs).

Infiltration based SCMs have become a key component in modern-day strategies to mimic predevelopment conditions, preserve groundwater and baseflow supplies, and ultimately restore ecosystem function. Extensive performance research is available for common types of infiltration based SCMs, such as bioretention (Birch et al. 2005, Carpenter and Hallam 2009, Davis et al. 2009, Hatt et al. 2007, Le Coustumer et al. 2007, Winogradoff 2002), rain gardens (a slightly varied version of bioretention - (Dietz and Clausen 2005, Jenkins et al. 2010, Shuster et al. 2007)), and permeable pavement (Collins et al. 2009, Dreelin et al. 2006, Emerson and Traver 2008). However, these studies often only quantify infiltration leaving the SCM itself, not making observations as to how this infiltration influences the surrounding water table, an infrequently studied topic of great importance.

Regenerative stormwater conveyances (RSCs) are gaining popularity as one such SCM that may help restore more natural hydrology in urban watersheds, in part by promoting infiltration. RSCs use aquatic pools, cobble riffles/boulder weirs, and native vegetation to encourage infiltration, energy dissipation, microbial activity, nutrient reduction, and evapotranspiration (Cizek 2014b). The three transport mechanisms present in RSCs are surface flow, shallow subsurface flow, and exfiltration to the surrounding water table. Despite appearing to have some contribution to peak flow and volume reductions from urban watersheds, especially for smaller storms, these systems have been studied far less frequently than other types of SCMs (Cizek et al. 2017, Koryto et al. 2017, Thompson et al. 2018). In particular, their infiltration capacities and interactions with groundwater are not well understood and appear to be highly variable based on location (Koryto et al. 2017).

With the growing popularity of infiltration based SCMs, concern has risen over whether infiltration of stormwater runoff poses negative consequences. For example, when stormwater is infiltrated from SCMs, there is the potential for groundwater contamination (Fischer et al. 2003, Mikkelsen et al. 1997, Pitt et al. 1999), undesired effects of groundwater mounding (Carleton 2010, Endreny and Collins 2009, Guo 2001, Machusick et al. 2011, Rai et al. 1998), and short-circuiting of the natural infiltration process as a result of highly permeable underground utility trenches (Kaushal and Belt 2012). Groundwater mounding naturally occurs beneath infiltration based SCMs but is exacerbated when the surrounding parent soils are less permeable than the soils located within the SCM (Guo 2001). Mounding is undesirable because it has the potential to harm underground utilities, compromise structural foundations, and negatively impact a watershed's groundwater flow regime (Machusick et al. 2011). Further, groundwater mounding has potential negative implications on the performance of SCMs by reducing stormwater infiltration during subsequent rain events. Despite this understanding, actual field studies investigating groundwater interactions from SCMs are limited (Cizek 2014a, Machusick et al. 2011).

Complicating these subsurface dynamics are underground utilities, which have the potential to influence the movement of groundwater, in ways quite opposite of mounding, also affecting the performance of infiltration based SCMs (Kaushal and Belt 2012). Underground sewer lines, both stormwater and sanitary, commonly have cracks leading to them acting as a source or sink for groundwater flow depending on the capacity at which the line is flowing. Additionally, these lines are encased in highly permeable gravel, creating preferential flow pathways away from the SCM. As an example, according to the Knoxville Utilities Board's Standards and Specifications, standard practice for sewer line installation is to encase sanitary sewer lines with crushed stone, 15 cm below and 30 cm above the sewer line (KUB 2015). The influence of these complex pipe networks on groundwater have been termed "urban karst" (Kaushal and Belt 2012). The influence of the "urban karst" is a particularly important consideration when implementing infiltration based SCMs because it can be difficult, if not impossible, to avoid these structures in urban and suburban settings. As indicated in a review of literature by Bonneau et al. (2017), the success of infiltration based SCMs is reliant upon future research targeted towards understanding catchment scale impacts of stormwater infiltration.

An RSC was constructed and monitored in Knoxville, TN. Data from a series of wells installed around the site indicated that the magnitude of water level change and responsivity of the groundwater system at the site varied spatially post construction. The RSC had two sanitary sewer lines that ran beneath, one active and one inactive. This lead to the hypotheses that the RSC had a significant influence on groundwater, but that interactions were highly influenced by the sewer line that crossed beneath the system. This research aims to characterize the well fluctuations and determine if mounding, short-circuiting, or a combination of both were occurring. Specifically, the objectives of this study are to (1) identify the surface and subsurface flow pathways present in the RSC to determine whether infiltrated stormwater remains mounded locally, is exfiltrated to the groundwater system, or travels along a utility line directly to the stream, and (2) determine if infiltrated stormwater reaches the water table and evaluate any subsequent influences to the groundwater system. The significance of this study is that it provides insight into the occurrence of groundwater mounding, or lack thereof, within and around RSCs (with implications for infiltration-based stormwater control measures in general).

3.2. Site Description

3.2.1. General Characteristics

This study investigated an RSC located at the South Doyle Middle School campus in Knoxville, TN (latitude: 35.952003 and longitude: -83.893768). The contributing drainage area was 2.0 ha and 74% impervious, consisting of the school building, parking lot, and associated grassed areas. Stormwater entered multiple catch basins dispersed through the parking lot, then traveled to the RSC via a 0.61 m diameter concrete pipe. Surface flow occurred in the RSC with rain events as small as 0.2 cm due to the high imperviousness of watershed. The stormwater that was not infiltrated via the RSC was discharged to Baker Creek, a tributary of the Tennessee River (Watts Bar Lake subbasin, hydrologic unit code 8: TN06010201). The RSC was located at the bottom of the drainage area, bound by the stream to the north, roadway to the east and south, and grassed field to the west (Figure 3.1). Beneath the RSC were two sewer lines, one active and one decommissioned, which were not discovered until during and after construction, respectively. While excavating the footprint of the RSC, gravel was encountered, indicating that the system may have had strong connectivity to the gravel surrounding the active sewer line. The presence and influence of underground utilities on SCMs is increasingly of interest (and being recognized as influential to performance) as infiltration-based practices are installed in locations throughout the world. Thus, this site provided an opportunity to study RSC performance under this condition.

3.2.2. RSC Description

Design of the RSC was completed using guidelines established by Anne Arundel County, Maryland (Flores et al. 2012) and constructed early August 2015, with vegetation planted early November of the same year. According to the Knox County Soil Survey, soils at the study site

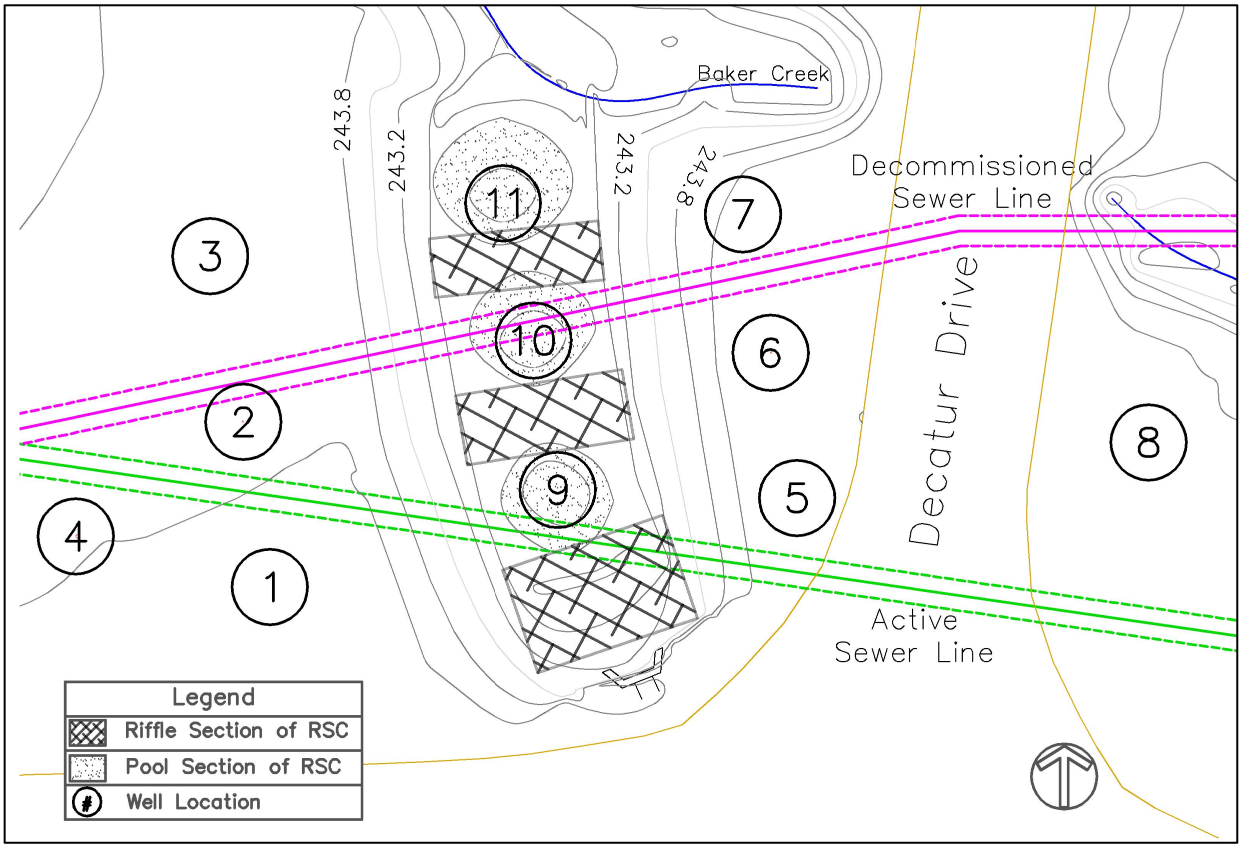


Figure 3.1 RSC study site, monitoring well locations, and approximate sewer line alignment. Contour line units are meters and represent 0.3 m change in elevation.

were Steadman Silt Loam and HSG C (Hartgrove 2006). The RSC consisted of a cascade and three riffle-pool sections, totaling 18.9 meters from the outfall to the stream bank. After excavation of the RSC footprint, the site was underlain with an approximately 0.30 m layer of sand (80% sand, 20% hardwood mulch) to promote subsurface seepage and provide a source of organic material. The remaining structural elements of the RSC were built upon this sand bed. The cascade was formed from existing, exposed bedrock with sandstone cobble used to fill in uneven areas. Sandstone cobble (d50 = 23 cm) was also used to build riffle sections that acted as the approach to the boulder weirs, which formed steps into each pool (approximate boulder dimensions: 0.3m x 0.6m x 1m each).

3.2.3. Well Installation

After construction, a series of eleven wells were installed within and around the RSC (Figure 3.1), using methods outlined by Sprecher (2008). Monitoring wells were installed using a truck mounted drill auger and drilled to the point of refusal (Table 3.1). The point of refusal in all wells surrounding the RSC was located below the water table, making it an unconfined aquifer. However, due to seasonal variations in water table height, the wells within the RSC only intersected the water table at times of high groundwater elevation. The exact depth of each well was determined from surveyed measurements; however, wells were typically between 1.8 and 2.7 m deep. The wells were made of fully screened PVC and covered with a well sock over the entire depth. The annulus around the well and bore hole was backfilled with sand and sealed with a bentonite plug (Sprecher 2008).

Table 3.1 Depth of monitoring wells

Well	Depth* (m)
1	2.4
2	2.6
3	2.7
4	2.2
5	2.4
6	2.7
7	2.5
8	1.8

*depth below ground, to point of refusal

3.3. Methods

3.3.1. Monitoring Setup

The eleven monitoring wells, each coupled with a HOBO water level data logger (typical error ± 0.4 cm), continuously recorded groundwater elevations for approximately 16 months (March 2016 to July 2017) at one-minute increments, which began approximately 8 months after installation (and 5 months after vegetation was planted). Wells 4 and 8 were placed as control wells to represent uninfluenced groundwater elevations and determine the lateral movement of groundwater. Rainfall was measured using a tipping bucket and recorded using a HOBO pendant® data logger, located approximately 40 m from the RSC. Recorded measurements were verified using a manual rain gauge. Individual rain event totals were calculated from on-site data; however, due to equipment failure resulting in discontinuous long-term data, monthly and seasonal rainfall amounts were determined from a rain gauge located at the Tyson McGhee Airport, approximately 19.3 km from the site.

3.3.2. Data Analysis of Monitoring Wells

Preliminary investigation of the data determined that the monitoring wells were highly responsive and showed an increase in groundwater elevation almost instantaneously after the initial rainfall of a given event. Therefore, it was necessary to maintain the data at a relatively high temporal resolution (as opposed to aggregating). A five-minute frequency was determined to be the largest increment allowable while still accurately representing the peaks of the wells. Dry weather groundwater elevations were determined monthly for each of the wells by averaging one, 24-hour period in each month that was preceded by 72 hours of dry weather conditions. These data were used to investigate monthly and seasonal variations in dry weather groundwater levels.

The start of an event was defined as the first recorded tip in the rain gauge tipping bucket. The end of the storm event was defined as the point where no further decreases in groundwater elevation occurred after smoothing the data to eliminate diurnal fluctuations. Lastly, the peak was simply the maximum groundwater elevation that occurred between the start and end of the event, with the height of the peak being calculated relative to the elevation at the beginning of the storm. The groundwater elevations were based off an arbitrary reference datum of 243.84 m (800 ft). Only storms that occurred after 72 hours of dry weather were considered in the wet weather analysis to isolate groundwater signatures for individual events. Additionally, one event was discarded from the analysis because it followed the highest magnitude storm event recorded during the monitoring period and, as a result of substantially saturated soils, the wells were more responsive and not an accurate representation of typical patterns. The non-parametric Spearman's rank-order correlation was used to test the relationship between height of peak and precipitation depth for each of the wells. Although all storms were used for overall statistical analysis, for the purposes of this paper, examples of a small (0.30 cm), medium (2.74 cm), and large (10.57 cm) storm event have been pulled from the dataset to describe how the groundwater responds under varying storm sizes. The three example storms chosen were selected within the same season and year (Fall 2016) to avoid variations in groundwater elevation due to seasonal and yearly fluctuations. Events were also analyzed during the Spring of 2016, when the water table was high, but no significant differences from the Fall were observed (thus the results are not reported herein).

3.4. Results and Discussion

3.4.1. Dry Weather Characterization

As previously noted, the RSC was located adjacent and perpendicular to Baker Creek with the system discharging surface flow to the stream. It is believed that the groundwater in some areas of the site was influenced by the stream, while in other areas groundwater was influenced by infiltrated stormwater within the RSC. More significantly, a sewer line running beneath the RSC is believed to have had substantial impact on groundwater transport in conjunction with infiltrated stormwater from the RSC.

Dry weather groundwater elevations were investigated for each of the wells for the monitoring period from March 2016 to July 2017. Monthly trends showed that the wells were typically lower during the fall and higher during the spring, displaying seasonal variability in dry weather water table elevations (Figure 3.2). This was expected, as water tables are typically lowest during late summer to early fall as a result of increased evapotranspiration, coupled with dryer weather patterns (Conlon et al. 2005). It should also be noted that Knox County experienced extreme drought conditions that peaked late November 2016, which likely exacerbated the seasonal groundwater fluxes (NDMC 2018). As a general observation, during all four seasons, wells 2 and 6 had the lowest water table elevation (Figure 3.3), thus the gradient of groundwater movement during dry weather conditions was consistently towards the areas of wells 2 and 6, presumably towards the zone of influence of the sewer line.

In looking at more specific trends, it is evident that wells 4 and 8 had the highest water table elevation during dry weather conditions (Figure 3.3). These were the control wells located furthest from the RSC, and likely to have the least influence from the RSC. Wells 2, 3, and 6 had the lowest water table elevation (Figure 3.2). As wells 2 and 6 were located closest to the sewer

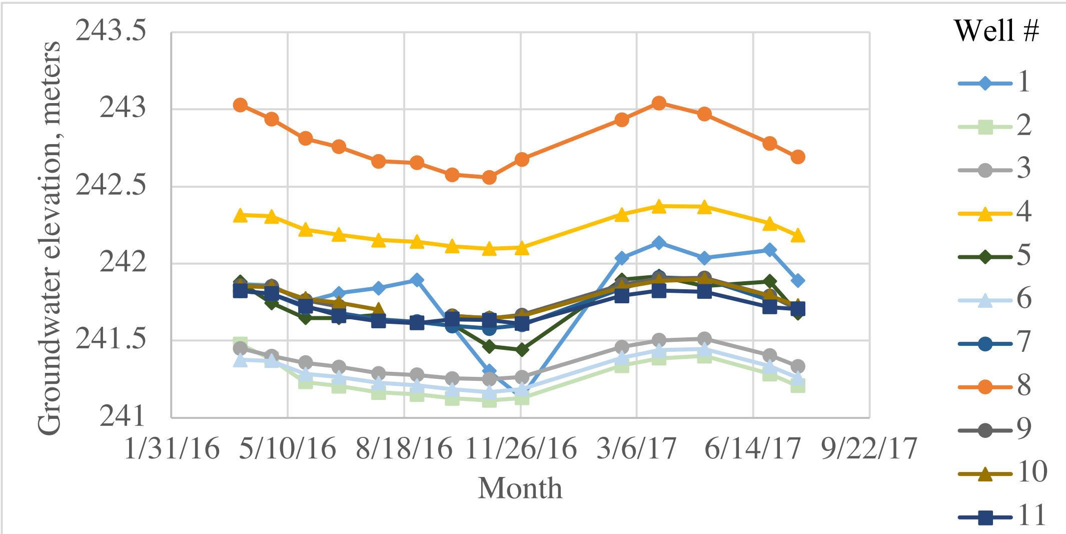


Figure 3.2 Monthly, dry weather, groundwater elevations. Elevations were averaged for a 24-hour period identified as the day closest to the last day of month that was preceded by 72-hours of dry weather.

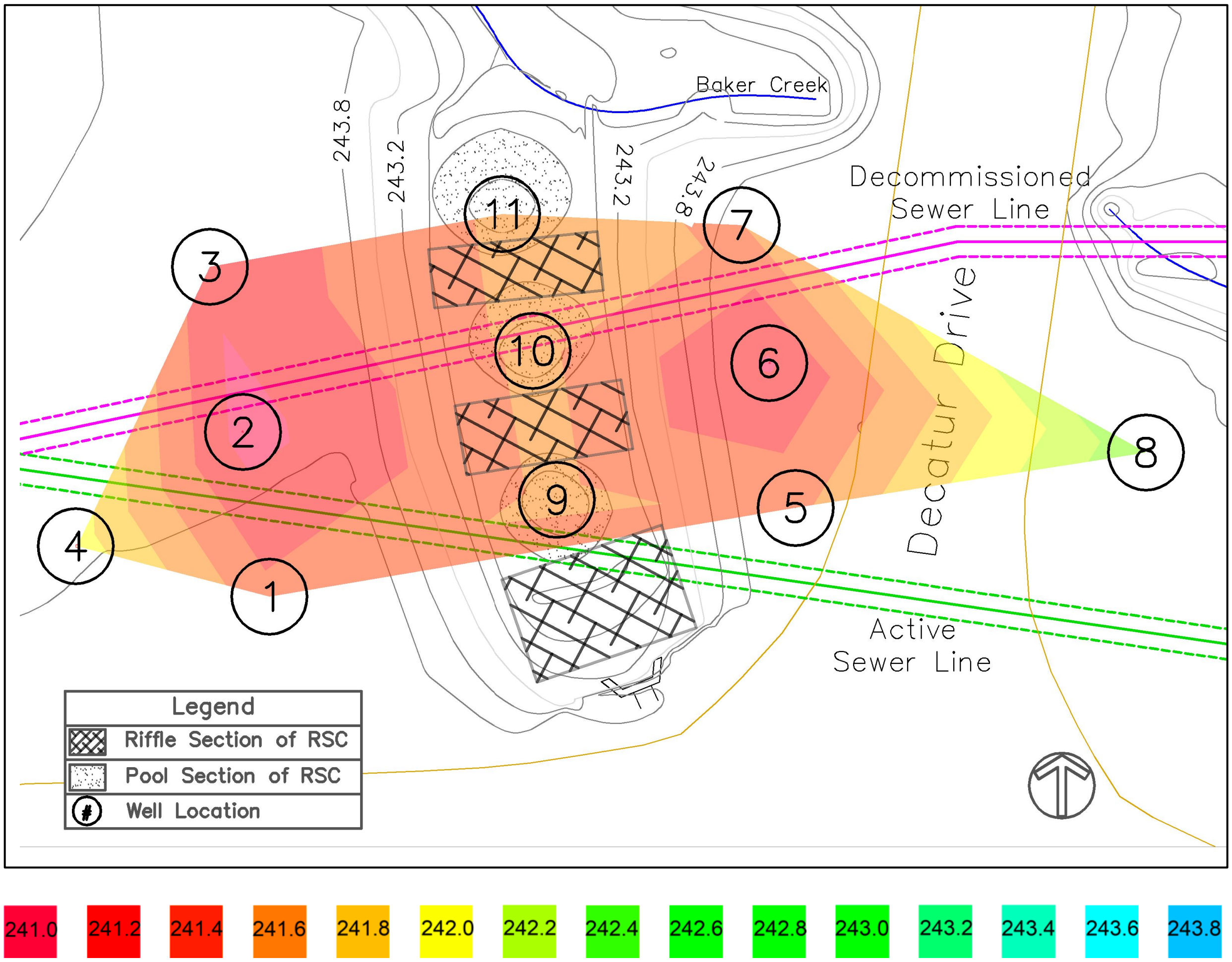


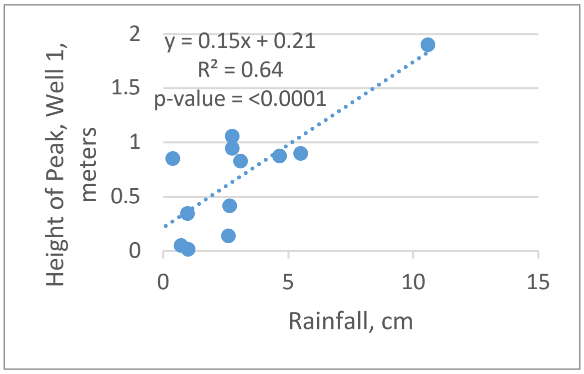
Figure 3.3 Contour map of averaged elevations for Fall 2016 under dry weather conditions. Contours generated using Delaunay triangulation of points in AutoCAD Civil3D. Surface contour line units are meters and represent 0.3 m change in elevation.

line (Figure 3.1), they were believed to be regulated by the highly porous gravel encasing the sewer pipe. Although Figure 3.1 suggests well 3 has some separation from the abandoned sewer line, the exact location of this sewer was highly uncertain as no manholes remain at the site. The researchers only had historical GIS data available to identify the possible location of this infrastructure. The well data from this study suggest the sewer line may actually be closer to well 3 than previously thought (discussed further in *Wet Weather Characterization)*.

Wells 1 and 5 behaved similarly to one another and were both located upgradient of the flow of groundwater (Figure 3.3). Wells 1 and 5 had much greater variability when looking at monthly trends (Figure 3.2), which might have been caused by fluctuating dynamics depending on the overall groundwater level resulting from seasonal influences. That is, during times of low water table elevation, well 1 acted more similarly to wells 2, 3, and 6, but in times of higher water table elevations, it acted more closely to the wells located within the RSC (wells 9, 10, and 11). Well 7 was located close to both the RSC and Baker Creek and appeared to behave most similarly to the RSC. The RSC was connected to the stream by a zone of permeable cobble, boulders, and sand backfill. Thus, it was likely that these wells were most influenced by the stream elevation more than the groundwater system. Overall, based on the wells not within the RSC, the movement of local groundwater appeared to be more influenced by the presence of sewer lines than the stream. This general observation allows context for the other results discussed herein.

3.4.2. Wet Weather Characterization

The change in groundwater elevation for a given event was largely correlated to the amount of rainfall the site received. However, spatial variation at the research site influenced these trends, as wells showed differential responses to increased rainfall. Figures 3.4a-h illustrate the height of a.



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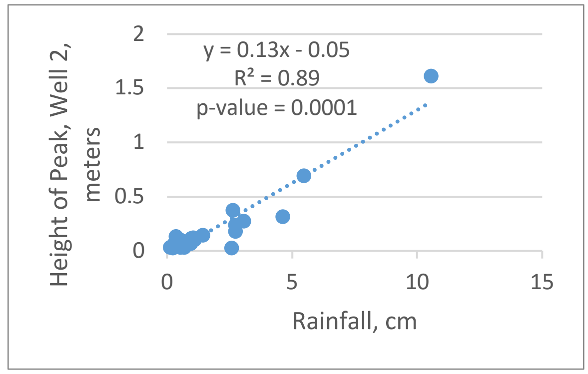

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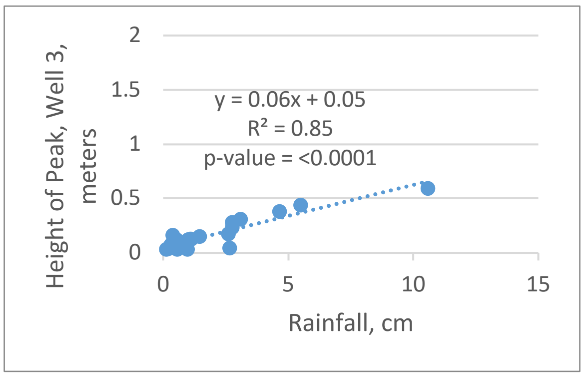

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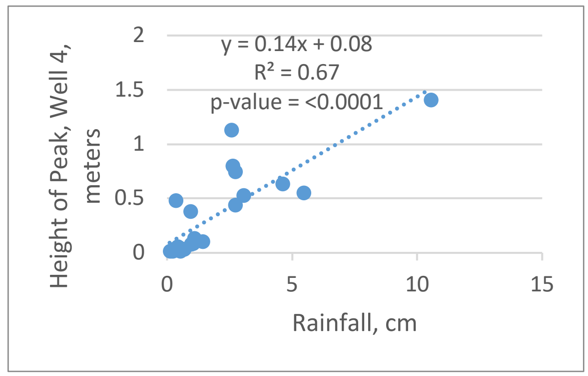

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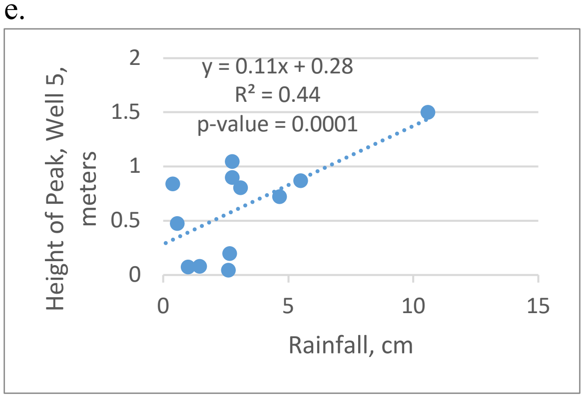

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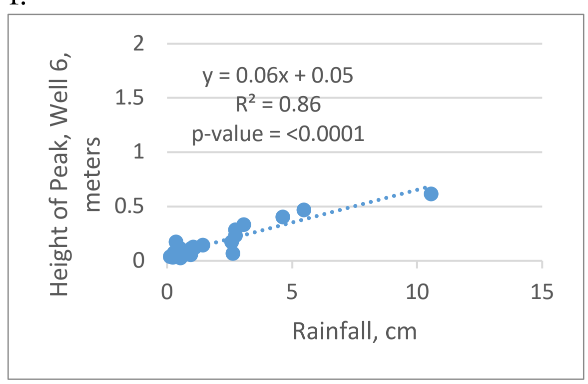

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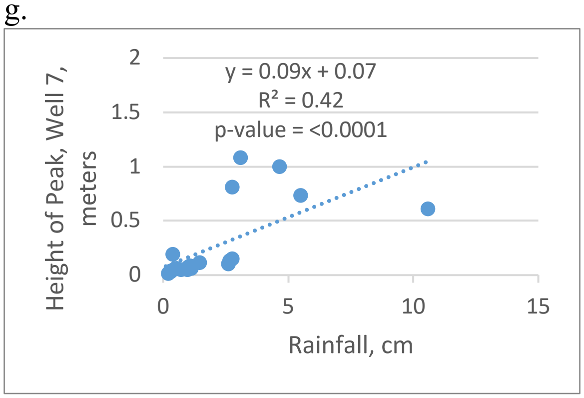

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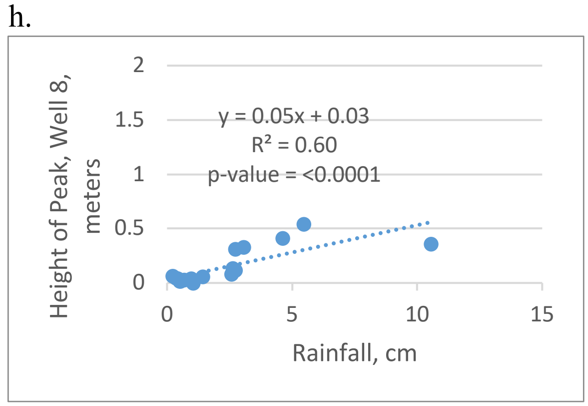

Figure 3.4a-h Height of peak vs rainfall for wells 1-8, illustrated in a-h, respectively.

peak vs. rainfall and show that varying correlations existed for each well. A strong positive relationship was found ($R^2 > 0.85$) for wells 2, 3, and 6 (Figures 3.4b, c, and f, respectively) and weaker relationships ($R^2 = <0.7$) for wells 1, 4, 5, 7, and 8 (Figures 3.4a, d, e, g, and h, respectively). It is logical that the correlation in wells 4 and 8 was weaker because they were located furthest from the RSC and least likely to be influenced by the RSC and rainfall. Further, it was also expected that wells 2, 3, and 6 were strongly influenced by the sewer line in that it appeared to act as a direct connection between these wells and the RSC, and thus was strongly tied to rainfall patterns. Wells 1, 5, and 7 were all located upgradient as the groundwater moved towards wells 2, 3, and 6. However, it is expected that several external factors might influence trends, such as influences from the larger surrounding groundwater profile, variable connectivity to the sewer line depending on height of water table (wells 1 and 5), or stream influences (well 7); thus, resulting in weakened correlation with rainfall for these wells.

To further explore groundwater elevations under wet weather conditions, three storms were selected to represent a range of event sizes – small (0.30 cm), medium (2.74 cm), and large (10.57 cm). All selected events occurred during late Fall of 2016, when the water table was low, to best illustrate groundwater movement without additional seasonal differences amongst the wells. Large variations were observed across the site for the three storm events in regard to height of peak, lag time, and time to peak (Table 3.2). The measured height of groundwater increase (height of peak) across the wells differed by up to 1.54 m during a single event (10.57 cm event on 11/28/16, with well 1 having the highest height to peak and well 8 having the lowest). For this same event, the lag time, measured from the first recorded tip on the tipping bucket to the start of increase in groundwater elevation, ranged from 1.1 hours to over 17 hours (wells 3 and 1, respectively) before an increase in groundwater elevation was realized. Further,

Table 3.2 Well responses for different storm size, excluding wells 9, 10, and 11

Precip. (cm)	Intensity (cm/hr)	Date	Lag time (hours)		Time to Peak (hours)		Height of Peak (meters)	
			Low	High	Low	High	Low	High
0.30	0.026	11/23/16	3.75 (2, 6)*	No peak (1, 5, 8)	4.75 (3, 6)	No peak (1, 5, 8)	0 (1, 5, 8)	0.08 (3)
2.74	0.026	12/12/16	3.08 (3)	5.58 (5)	7.25 (1, 3)	31.42 (8)	0.24 (2)	0.95 (1)
10.57	0.033	11/28/16	1.08 (3)	17.67 (1)	35.67 (4)	42.17 (8)	0.36 (8)	1.90 (1)

*Values represented as ranges to show values across all wells. The value in parentheses is the well that corresponds to the given value. Wells 9, 10, and 11 were excluded because they were located within the RSC and do not represent changes in groundwater.

the time to peak, measured from the start of increase in groundwater elevation to the maximum height of groundwater elevation for that storm, ranged from 35.7 to 42.2 hours before the maximum peak was observed (wells 4 and 8, respectively). One major factor for variations across storm events were the weather patterns. For example, a short intense storm will have a quicker lag time and peak than a longer, less intense storm event. Additionally, soil conductivity (i.e. well location) also plays a major factor. Whether the well was surrounded by the dense, silty loam, native to the site (well 8), or porous gravel encasing a sewer line (well 2), influenced the time it took for the groundwater elevation to peak.

The groundwater contour maps and groundwater elevation plots illustrate the spatial and temporal peaks of the groundwater elevation, respectively (Figures 3.5a-f). Figure 3.5a shows a 0.30 cm rain event, with groundwater moving away from the RSC towards wells 2 and 6. This indicates that infiltrated stormwater was mounded beneath the RSC, resulting in the conveyance acting as a source under wet weather conditions and wells 2 and 6 acting as a sink. Similar trends were realized for the 2.74 cm storm event (Figure 3.5b), with well 2 (i.e. the sewer line) still acting as a sink, but with higher elevations observed in the perimeter wells of the research site. This indicates that the RSC may be a sink for water traveling from the east, but was also acting as a source for well 2. This implies some mounding may still be occurring under the RSC during moderately sized storms, but additional dynamics are also impacting groundwater levels. Figure 3.5c illustrates a 10.74 cm rainfall event where it can be seen that the groundwater elevation at all wells except the RSC was elevated, with a gradient of movement towards the RSC. This indicates that minimal mounding was occurring due to the RSC, and it generally acts as a sink during extreme precipitation events.

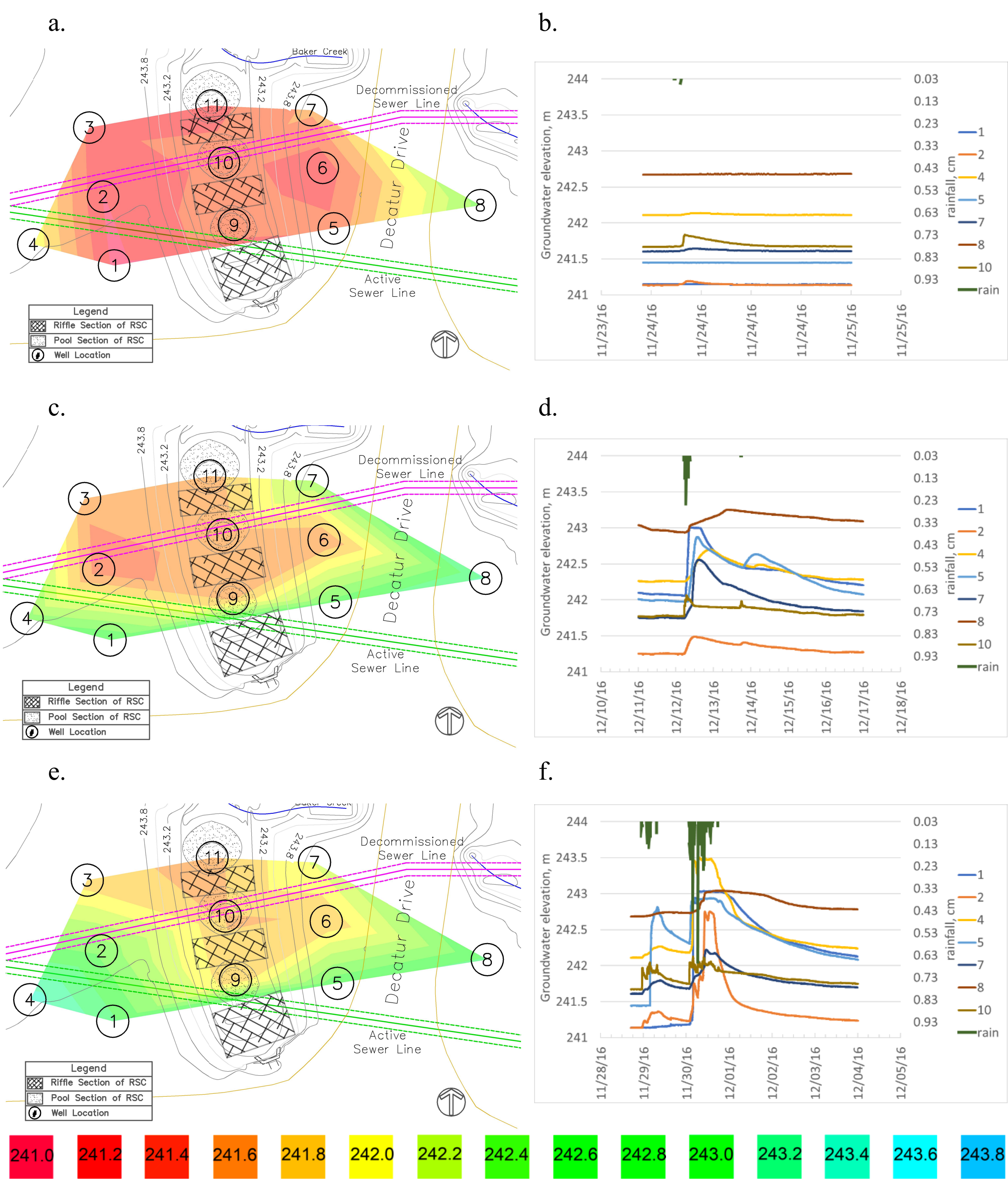

Figure 3.5a-f Groundwater elevation (m) contour map and groundwater elevation plots under wet weather conditions for the following storm events during Winter 2016 a-b) 0.30 cm, c-d) 2.74 cm e-f) 10.57 cm. Each contour line represents a 0.3 m change in elevation

From the groundwater elevation plots for each of the three events (Figures 3.5b, d, f), it was evident that there was high variation in how the wells responded to each rain event, both amongst wells and between storms. The most notable patterns have been extracted and are discussed below. In the groundwater elevation plots for the three storm events, wells 2, 3, and 6 (represented only by well 2 in the figure) are almost identical in shape to the groundwater level plot for wells 9, 10, and 11 (located within the RSC and represented by well 10 in the figure), just slightly more smoothed. Based on previous analyses herein (see Figures 3.2-3.4) it appears these wells have a strong hydrologic connection to the RSC via the sewer line, thus creating a short-circuit between the two. As a result, even the smallest storm events created a rise in the water table. Sharp rises and recessions corresponding to the rainfall patterns were characteristic of these wells, which was another indicator that short circuiting was occurring. As mentioned previously, because well 3 behaved so similarly to wells 2 and 6, it is believed some discrepancy exists in the proximity of well 3 to the sewer line, in that the well may actually be located closer to the abandoned sewer line than indicated by the aerial imagery.

Well 8, located furthest from the system, had very gradual increases and decreases, similar to what would be expected for undisturbed groundwater conditions (i.e. the project "control"). Well 4 was also one of the control wells; however, it showed a greater response to rain events than well 8. The plots for well 4 have similar shapes to wells 1 and 5, only slightly more muted, suggesting some influence of the RSC and sewer lines.

Wells 1, 5, and 7 showed less predictable results than the other wells. Well 7 had very gradual increases and decreases, and based on its spatial location, it was expected this well was influenced by both the RSC and the stream. For example, for the 2.74 cm rainfall (Figure 3.5d), there was a gradual increase in response to the rainfall, then, as the well began to decrease, there was a slight uptick in groundwater elevation that was not correlated to precipitation. Based on the contour maps for the 2.74 rainfall event (Figure 3.5c), the gradient of movement was away from well 7, thus, the increase in groundwater elevation at well 7 was most likely a result of stream influences. Wells 1 and 5 also show sharp water level increases, but their height of peak was significantly greater than the surrounding wells. Additionally, wells 1 and 5 plateaued before gradually decreasing, most notably illustrated in Figure 3.5f. These wells appear to be influenced by the combined effects of the RSC and sewer and may be unable to recede until other areas of the groundwater system drain.

3.5. Conclusions

The results of this groundwater monitoring case study indicate that there were substantial, yet variable, influences on the groundwater system resulting from stormwater infiltrating into the RSC. The degree of influence of the RSC, and variability of the groundwater elevations in general, resulted from seasonal fluxes in water table elevation, varying weather patterns and storm event size, and most significantly, the presence of two buried sewer lines. Seasonal, dry weather, trends show that groundwater elevations were highest during spring and lowest during fall, with a consistent gradient of movement towards the areas where a sewer line was present. Additionally, it was found that the connectivity of the groundwater system to the sewer lines appeared to change seasonally, as illustrated with wells 1 and 5. It was also observed that the areas in close proximity to the sewer lines were most influenced by rainfall (wells 2, 3, and 6), indicating that these areas of the site were highly connected to the RSC. Lastly, event size played a large part in whether the RSC acted as a source or sink for the groundwater system. Mounding was observed below the RSC during small to medium storm events (with the sewer acting as a sink), but the mounding shifted to the areas adjacent to the conveyance during larger events. This suggested that as groundwater from the surrounding watershed leads to high water table elevations, the RSC acts as a sink for the overall groundwater system.

The most prominent factor influencing trends in groundwater interactions at this research site appeared to be the presence of the sewer lines. However, this is not a localized issue, as urban areas continue to grow, the presence of buried infrastructure (i.e. urban karst) will continue to grow. It is important to understand how the urban karst network will influence groundwater interactions in order to avoid any unintended consequences or short circuiting of water quality treatment mechanisms. Although this case study offers one example of these processes, further study is needed to understand how RSCs, and infiltration-based SCMs in general, interact with surrounding groundwater, particularly in light of the preponderance of active and inactive underground utilities in urban environments.

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4. Water Quality and Quantity Performance of a Regenerative Stormwater Conveyance in Knoxville, TN

Abstract

A regenerative stormwater conveyance in Knoxville, TN, was monitored for water quality and hydrologic performance for a 16-month period between March 2016 and June 2017. Parameters investigated included volume, peak flow rate, and pollutant event mean concentrations (Ammonium, Nitrite, Nitrate, Phosphate, Sulfate, Copper, Zinc, and Lead) at the inlet and outlet of the system. Results indicated that reductions were 6%, 23%, and 16% for cumulative volume, median event-based volume, and peak flow rate reductions, respectively, though not statistically significant at the 95% confidence level. Mixed results were found for concentration reductions of each of the pollutants investigated; however, pollutant loadings suggested better performance, though still not statistically significant. An investigation into volumetric, peak flow rate, and water quality pollutant load reductions for the growing season and non-growing season did show a significant difference, indicating that seasonal processes such as evapotranspiration and water table elevation were influential. The outcomes of this work suggest implementation of these systems in the Knoxville area (or regions with similar geological characteristics) will be most impactful in areas with a naturally low water table, when part of a treatment train application, or where the opportunity exists to backfill an area with sandy media (i.e. massive head cutting).

4.1. Introduction

With the evolution of municipal regulations targeting stormwater management, policy makers and stakeholders welcome new practices that will fulfill ever stringent requirements for stormwater management. An existing suite of stormwater control measures (SCMs) has proven effective for stormwater management (bioretention cells, wetlands, grassed swales, etc.); however, certain constraints require the need for innovative SCMs to meet site-specific challenges. Regenerative stormwater conveyances (RSC) were developed as a practice to retrofit head cutting at stormwater outfalls, a common condition that is particularly observed in areas of sandy soils. Additionally, a unique feature of RSCs is that they can accommodate slopes up to 50%, making them a viable SCM option in locations where other practices are not feasible.

Principle components of RSCs include aquatic pools to promote infiltration; cobble riffles and boulder weir grade control structures to dissipate energy and erosive flows; and established native vegetation to promote stability and evapotranspiration. Further, RSCs are constructed with an underlying sand seam comprised of an 80% sand and 20% hardwood mulch, which, when coupled with established root systems from vegetation, creates a system rich in organic material that encourages microbial activity. All of these components combined make RSCs a promising SCM for targeting volume reductions and pollutant removal. However, while RSCs are gaining popularity, they have only been investigated for hydrologic and water quality performance at four locations (including this research) (Cizek 2014, Koryto 2016, Thompson et al. 2018). The results of these studies have yielded mixed results regarding water quality and hydrologic performance, indicating further investigation is needed.

Cizek (2014) monitored two RSCs located in distinctly different soil types: one in the Coastal Plains region of North Carolina (Brunswick County) with predominately sandy soils of Hydrologic Soil Group (HSG) A classification, and the second RSC in the Piedmont Region of North Carolina (Alamance County) with denser, clay soils of HSG D classification. Koryto (2016) investigated a third RSC, also located in the Piedmont region in Durham County, North Carolina (HSG D). The results of the two RSCs located in the Piedmont region are considered to be most relevant to the research in Knoxville, TN, as the soil types are very similar. A summary of the findings are outlined in Table 4.1. Cizek (2014) found significant reductions for surface flows, peak flows, total suspended solids (TSS), total nitrogen (TN), and total phosphorus (TP)

Table 4.1 Outline of results from previous RSC research

Study	Location	HSG	Volume Reductions	Peak Flow Reductions	TSS Reductions	Additional Findings
(Cizek 2014)	Brunswick Co, NC	A	94-100%	90-96%	na	Able to mimic predevelopment hydrograph for events 1 year or less, but not all events. **High water table.**
(Cizek 2014)	Alamance Co, NC	D	84%	80%	72%	Able to mimic predevelopment hydrograph and flow pathways. **Low water table for majority** of system and high water table in last section.
(Koryto 2016)	Durham Co, NC	D	8%	49%	17%	Hydrologic reductions were only seen for rainfall less than 12.7 mm. **High water table.**

across both sites. However, additional research conducted by Koryto (2016) showed much lower hydrologic reductions, and thus pollutant load reductions, most likely a result of the combination of a number of factors described below.

The RSC in Brunswick County, North Carolina (HSG A) had a high water table, intersecting the second pool for much of the monitoring period, but still realized significant volume and peak flow reductions. However, it was observed that exfiltration was limited, meaning surface flows were infiltrated into the system but remained mounded locally in the underlying sand seam, traveling as shallow subsurface flow (Cizek 2014). The RSC in Durham County, North Carolina (HSG D), also with a high water table intersecting the system, showed significantly less volume and peak flow reductions and pollutant removal rates (Koryto 2016). This implies that RSCs are capable of performing with either high water table (e.g. Cizek 2014 – Brunswick site) or clay-like soils (e.g. Cizek 2014 – Alamance site) but may be limited when both conditions are present (e.g. Koryto 2016 – Durham site). Further, at a minimum, high water tables are observed to negatively influence partitioning of inflows to infiltration, as seen by Cizek (2014), even when surface flows are reduced. In summary, literature shows that volume, peak flow, and pollutant loading reductions are possible in RSCs, but site conditions (location of water table and infiltration capacity of soils) play a role in the extent of reductions realized. As such, there is a substantial need to add to the literature regarding RSC performance to better understand how site conditions influence outcomes.

This study uses hydrologic and water quality data to better understand the ability of RSCs to mitigate the effects of urban stormwater runoff. The objectives of the study are to (1) quantify the stormwater volume and peak flow reduction performance of the RSC and (2) quantify pollutant loadings at the inlet and outlet of the RSC to determine if reductions are occurring. This study adds to the existing, but limited, bank of knowledge regarding water quality and hydrologic performance of RSCs by providing data from another unique location. Specifically, this research will fill data gaps regarding performance of RSCs within differing physiographic regions than are present in existing literature and examine the factors (i.e. seasonality, water table depth, etc.) leading to the poor performance noted in literature for sites located in high water tables and clayey soils.

4.2. Field Site Description

4.2.1. Site Location

The RSC investigated in this study was located in Knoxville, TN, on the South Doyle Middle School campus (lat: 35.952003, long: -83.893768). The RSC collected stormwater from a 2.0 ha, 74% impervious catchment. The primary land use types were the school building, parking lot, and associated grassed areas. The watershed was highly responsive due to its amount of imperviousness, with flow into the RSC occurring after just 0.2 cm of precipitation. Stormwater entered the RSC via a 0.61 m diameter concrete pipe. After traveling through the RSC, stormwater was discharged to Baker Creek, a tributary of the Tennessee River located within the Watts Bar Lake subbasin (HUC-8: TN06010201). Baker Creek is listed on the United States Environmental Protection Agency's 303(d) list of impaired streams for excessive pollutant loadings from pathogens, nutrients, and habitat alteration, causing degradation to the designated uses of Fish/Aquatic Life and Recreation (EPA 2014). Prior to installation of the RSC (August 2015), an eroding, riprap lined channel, conveyed flows from the catchment, providing minimal storage and acting as a source of suspended sediment to Baker Creek.

4.2.2. RSC Design

The RSC consisted of a cascade and three riffle-pools sections, totaling 18.9 meters from the outfall to the stream bank, and having an average slope measuring 0.05 m/m. A cascade (an area of increased elevation drop relative to the riffles) was necessary for this particular system because of road constraints and the location of existing bedrock (Flores et al. 2012). Half of the elevation loss occurred in the cascade, leaving an elevation change of -0.09 m and -0.06 m at each of the subsequent riffle sections. The short riffle sections were comprised of sandstone cobble (d50 = 23 cm) with boulders (approximate dimensions: 0.3m x 0.6m x 1m each) serving as steps from one pool to the next. The pools were underlain with a soil media consisting of roughly 80% sand and 20% hardwood mulch, backfilled to a depth of approximately 0.5 m (Figure 4.1).

While designing the RSC, the pool was intended to have a depth of 0.46 m, measured from the bottom of the pool to the top of the riffle leaving the pool. This was the minimum pool depth per Anne Arundel County, Maryland, design specifications (Flores et al. 2012). However, bedrock was encountered during construction, preventing the pool from reaching the targeted design and constraining the depth to approximately 0.35 m. A previous modeling studying that investigated optimal pool depths determined that depths less than 0.46 m were likely to generate high velocity jets and should be avoided (Thompson et al. 2018).

4.2.3. Vegetation

A principle component of RSC systems is a thriving vegetative community. The vegetated nature of the system allows for increased evapotranspiration, soil stability, carbon content, energy dissipation in surface flows, and aesthetics. Resources available through the Lady Bird Johnson Wildflower Center's website (Lady Bird Johnson Wildflower Center 2015) and the USDA

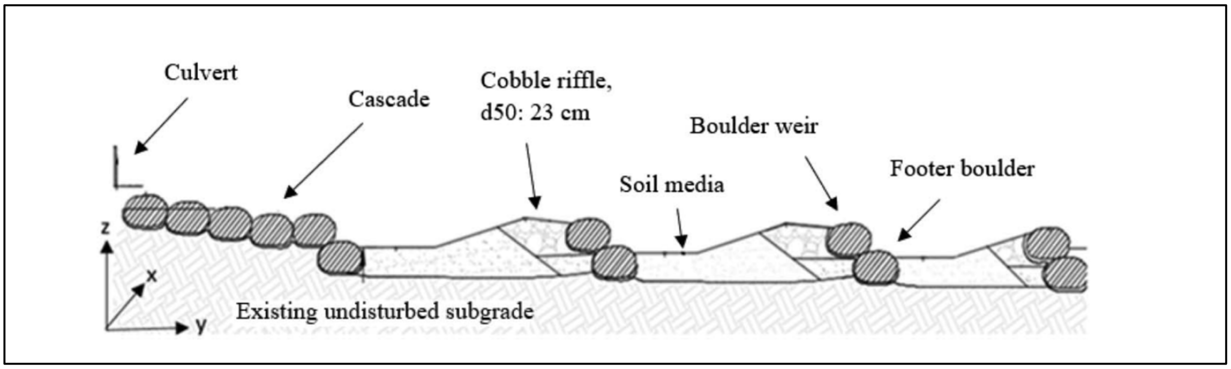

Figure 4.1 Cross sectional image of RSC in Knoxville, TN. Taken from Thompson et al. (2018)

NRCS PLANTS database (USDA 2006) were used to aid plant selection. When designing this system, care was taken to choose native plants that would flourish in the conditions present within the system, and to site plantings based on their preferences (i.e. wetland like conditions in the pools and more dry conditions on the banks). The vegetation was planted in the Fall of 2015. Plugs, rather than seeds, were chosen to expedite the establishment period. In the pools of the RSC, Carex cherokeensis (Cherokee Sedge), Juncus effuses (Common Rush), Caltha palustris (Marsh Marigold), and Symphyotrichum novae-angliae (New England Aster) were planted. On the banks, Schizachyrium scoparium (Little Bluestem), Zizia Aurea (Golden Alexanders), Lobelia siphilitica (Great Blue Lobelia), Baptisia alba (White Wild Indigo), and Lobelia cardinalis (Cardinal Flower) were planted. Conoclinium coelestinum (L.) (Blue Mistflower), Asclepias incarnata (Swamp Milkweed), and Lindera benzoin (Spicebush) were planted on the top of bank.

4.3. Methods

4.3.1. Monitoring

Hydrologic data was collected at the inlet and outlet of the RSC. Compound triangular-rectangular sharp-crested weirs were mounted at both locations, and stage was measured using a Teledyne ISCO 4230™ bubbler flow meter (Figure 4.2a-b). Flow depth was converted to flow rate using a stage-discharge relationship developed by Jan et al. (2006) and Martinez et al. (2005). A Teledyne ISCO 3700™ automatic sampler equipped with a 24 bottle configuration was used to collect stormwater samples at inlet and outlet of RSC. The automatic sampler was programmed to collect aliquots at volumetric intervals (i.e. flow paced sampling) to accurately represent the hydrograph of a sampled storm event. Rainfall was recorded using a tipping bucket coupled with HOBO™ pendant data logger and verified with a manual rain gauge. The rain

a. b.

Figure 4.2a-b a) Compound weir located at a) pipe outlet/RSC inlet b) RSC outlet to Baker Creek. Image taken from Thompson et al. (2018).

gauge was positioned approximately 40 m from the research site. However, occasional equipment malfunction occurred, resulting in some events lacking precipitation data readings. As a result, a correlation analysis was performed to interpolate rainfall from inflow volumes (discussed in section 4.3.5) to allow some investigations discussed in the results section.

Stormwater monitoring at the site occurred during a 16-month monitoring period between March 2016 and June 2017 and represents storms ranging in size from 0.15-8.6 cm. For the purposes of this research, a storm event began when flow started to discharge from the RSC inlet (which was dry between events) and ended once the inlet flow rate fell below 0.0004 m^3/s (considered residual flow), at which point the outlet flow throughout the study was observed to have stopped. A total of 19 storm events were sampled and analyzed for water quality and 58 events were characterized for hydrology.

4.3.2. Data Collection and Lab Analysis

Sample bottles were collected in a timely manner after each storm event (no more than 24 hours), transported to the University of Tennessee Water Quality Lab, composited based on flow, and either analyzed immediately or prepared for storage and future analysis. Water quality analysis included total suspended solids (TSS); ion chromatography (IC) analysis to determine ammonium, nitrate, phosphate, and sulfate; and inductively coupled plasma atomic emission spectroscopy (ICP-AES), to determine dissolved copper, zinc, and lead. Analytical methods and method detection limits (MDL) for each of these parameters are outlined in Table 4.2.

4.3.3. Hydrologic Data Analysis

Volume reductions and peak flow reductions (Eq. 1) were calculated for the 58 hydrologic events during the monitoring period, with both cumulative and median event-based values used to represent hydrologic performance. Volume and peak reductions were calculated as follows:

Table 4.2 Pollutants investigated and analytical methods used

Pollutant		Analytical Method	MDL (mg/L)
TSS	Total suspended solids	SM 2540D[a]	1
NO_3^-	Nitrate/nitrite, as nitrogen	EPA 300.1[b]	0.027
SO_4^{2-}	Sulfate	EPA 300.1[b]	0.048
PO_4^{3-}	Phosphate	EPA 300.1[b]	0.121
NH_4^+	Ammonium	EPA 300.1[b]	0.011
Cu	Copper	EPA 200.7[c]	0.0006
Zn	Zinc	EPA 200.7[c]	0.016
Pb	Lead	EPA 200.7[c]	0.003

a. (Association and Association 2012)

b. (EPA 1997)

c. (EPA 1994)

$$Hydrologic\ Reduction = \frac{X_{in} - X_{out}}{X_{in}}\ x\ 100 \qquad (1)$$

where x represents either the volume (L) or peak flow rate (m^3/s) and "in" refers to the inlet value and "out" refers to outlet value.

4.3.4. Water Quality Analysis

Event mean concentration (EMC) (Eq. 2), pollutant concentration reduction (Eq. 3), event load (Eq. 4), and load reduction (Eq. 5), were calculated for each of the pollutants investigated. In the analysis, a value of one half the MDL was used where results were less than the detection limit (Clausen and Spooner 1993). In instances where both inlet and outlet values were below detection limit, the event was excluded from the load analysis. Results are presented as cumulative and median event-based values for the monitoring period.

$$EMC = \frac{\sum_{i=1}^{n} V_i C_i}{\sum_{i=1}^{n} V_i} \qquad (2)$$

$$Concentration\ Reduction = \frac{EMC_{in} - EMC_{out}}{EMC_{in}}\ x\ 100 \qquad (3)$$

$$Event\ load = EMC\ x\ V_{total} \qquad (4)$$

$$Load\ reduction = \frac{EL_{in} - EL_{out}}{EL_{in}}\ x\ 100 \qquad (5)$$

where pollutant removal and load reduction are expressed as percentages, EMC_{in} is the EMC at the inlet (mg/L), EMC_{out} is the EMC at the outlet (mg/L), EL_{in} is the event load at the inlet (g), and EL_{out} is the event load at the outlet (g).

4.3.5. Interpolation of Precipitation from Inflow Volume

Of interest for this study was how volume reductions were correlated with rainfall depths. However, due to occasional equipment failure, several storm events were missing precipitation data, necessitating a way to estimate rainfall based on monitored runoff. Because the drainage area was highly impervious, it was expected that the majority of the precipitation falling on the drainage area entered the RSC as stormwater runoff via the inlet A regression analysis was performed for rainfall vs. inflow volume (Figure 4.3), with good correlation between rainfall and runoff observed ($R^2 = 0.7$). Thus, the resulting equation for the regression line was used to estimate precipitation.

4.3.6. Statistical Analysis

Inlet and outlet data were tested for normality and log normality through visual assessment and the Shapiro-Wilk test. It was determined that none of the data fit a normal or log normal distribution, thus nonparametric analyses were performed. The Wilcoxon signed-rank test was used to determine whether a significant difference occurred at the inlet and outlet of the RSC for volume, peak flow rate, and pollutant concentrations, as well as to test differences in reductions between the growing season versus the non-growing season. The Spearman's rank-order correlation was used to test the strength of the correlation between inflow and rainfall. Results were deemed statistically significant at the 95% confidence level ($\alpha=0.05$).

4.4. Results and Discussion

4.4.1. General Hydrologic Performance

Results for the investigation into hydrologic performance of the RSC found that event-based volumetric reductions ranged from -41% to 100%, with negative values observed that were largely attributed to storm event size and amount of groundwater intrusion. More specifically, for the 16-month monitoring period, the results yielded a cumulative volume reduction of 6%, median event-based volumetric reduction of 23%, and median event-based peak flow rate reduction of 16% (Table 4.3), with statistical tests indicating no significant differences between inlet and outlet. While this hydrologic performance was poorer than studies in coastal plain regions that realized reductions in exceedance of 80% (Cizek 2014, Table 4.1), it is comparable

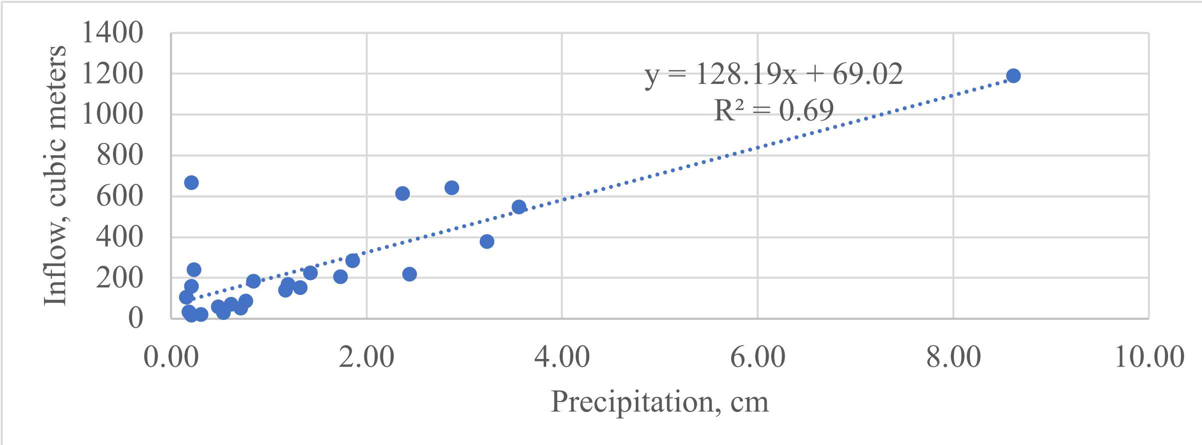

Figure 4.3 Precipitation vs. inflow volume for the 25 events with paired rainfall data. Results from Spearman's Correlation analysis showed a p-value of 0.0005, thus statistically significant.

Table 4.3 Hydrologic data for the monitoring period, including volume reduction, peak flow reduction, and significance between inlet and outlet

	n	Cumulative			Event Based						
		Inlet	Outlet	Red. (%)	Inlet			Outlet			Median Red. (SD)
					Min	Median (SD)	Max	Min	Median (SD)	Max	
Volume (m^3)	58	11,754	11,075	6%	4.9	155.3 (207.9)	1,190	0.0	114.2 (259.7)	1,453	23% (38%)
Peak (m^3/s)	58	-	-	-	0.003	0.038 (0.039)	0.16	0.0	0.0354 (0.030)	0.12	16% (35%)

to other studies with dense soils and a high water table present (Koryto 2016, Table 4.1) which was more similar to the conditions present herein.

Another factor found to influence hydrologic performance was the amount of precipitation the site received. Figure 4.4 illustrates precipitation versus volume reduction, and it was determined that the system performed best at less than 2.0 cm of rainfall. Perhaps most interesting, however, is the performance for events up to 2.5 cm (or the first inch), a common design metric for SCMs, resulted in cumulative and event-based volume reductions of 15% and 26%, respectively. For events over 2.5 cm, performance greatly decreased to -7% and -22%, respectively, indicating the RSC is best suited for treating stormwater runoff from smaller, more frequent storm events. Of note in this analysis was that negative event-based volumetric reductions were observed even for smaller events, leading to the realization that other factors are also influencing performance.

Visual observation at the site revealed a highly variable groundwater elevation based on season, prompting further investigation into the hydrologic data. Results found that during the winter (when the water table was higher), median volume reductions were negative, meaning there was a net export of stormwater or, more likely, some external source of water entering. Outside of winter, only four events had negative volume reductions, and all instances were either late fall or early spring (i.e. dates surrounding winter - Figure 4.5). Thus, the poor volume reductions were attributed to the influence of groundwater on overall outflow volumes. This is likely due to either the storage space in the sand/woodchip mix being occupied, or groundwater intrusion into the system during storms (which is probable based on the results from Chapter 3). Additionally, during construction, it was found that bedrock was present under the last two pools of the system, likely further limiting infiltration. Conversely, when the water table was lowest in

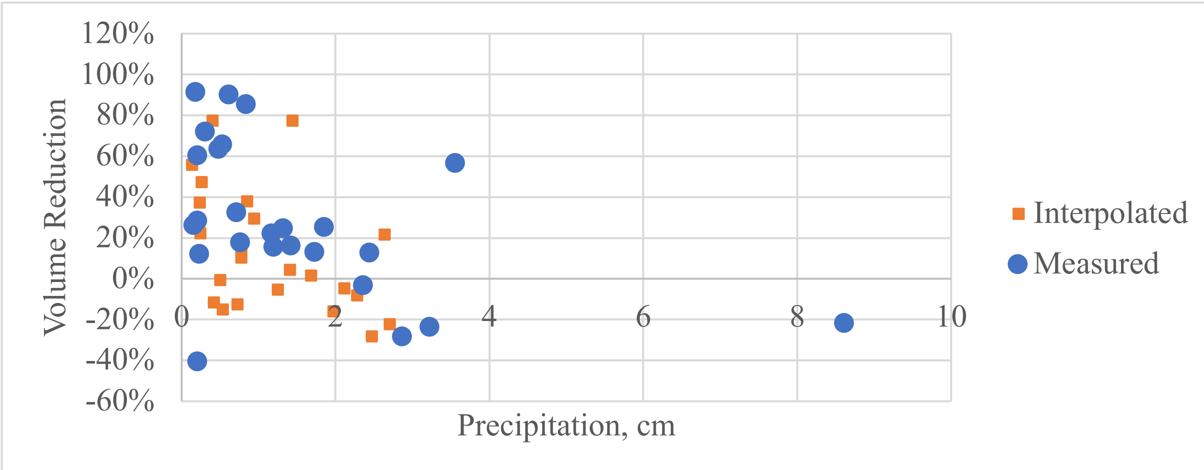

Figure 4.4 Graph of precipitation vs. volume reduction, illustrating decreasing performance with increasing precipitation.

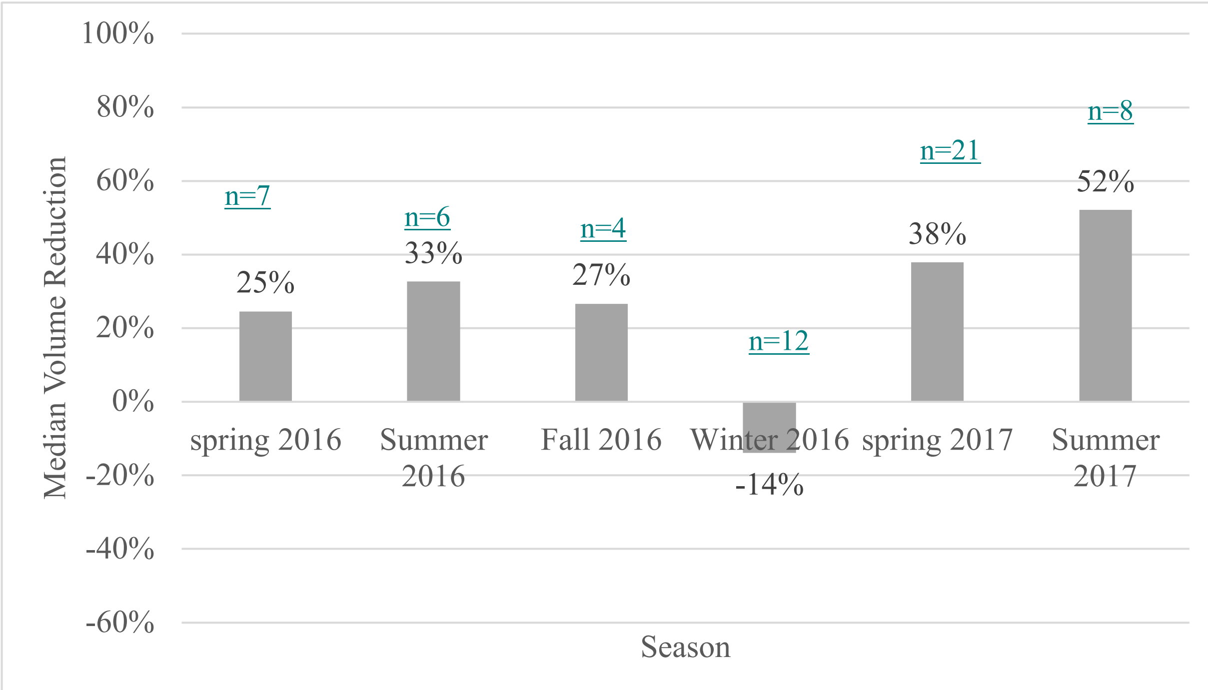

Figure 4.5 Median volume reductions for each season during the study period.

the summer, performance was at its peak. If the winter events were excluded from the analysis, cumulative and median event-based reductions became 19% and 35%, respectively, a marked improvement over the full water balance.

4.4.2. Water Quality Performance

A total of 19 storms were sampled and tested for each of the water quality parameters. Performance was quantified based on EMC reduction and load reduction between the inlet and the outlet. While results indicated minimal EMC reduction, investigation of pollutant loading showed water quality improvements.

Median reduction in EMC ranged from -36% (TSS) to 16% (SO_4^{2-}), with more than half of the parameters having a concentration reduction equal to 0% (Table 4.4) and no pollutant showing a significant difference between the inlet and outlet. One suspected cause was the occurrence of irreducible concentrations entering the system (Hathaway and Hunt 2010, Lenhart and Hunt 2011, Schueler and Holland 2000). The watershed is 74% impervious with minimal sources of contaminants, especially for TSS. Other studies have shown similar results for SCMs with low influent concentrations. For example, in an analysis by Lenhart and Hunt (2011) on stormwater wetlands, 80% of pollutant reductions occurred within the first cell, resulting in low influent concentrations entering subsequent cells and no significant reductions occurring, indicating the pollutants investigated had already reached the background concentration level, with no further reductions possible. Lastly, the shallow pool depth of 0.35 m (compared to the design depth of 0.5 m) that resulted from intersecting bedrock could have led to sediment export due to geomorphic changes in the system itself. Coupled with low concentrations entering system, this appeared to result in export of sediment.

Table 4.4 Median EMC reduction performance values.

Pollutant	EMC							
	N	Inlet (mg/L)			Outlet (mg/L)			Median Reduction (SD)
		Min	Median (SD)	Max	Min	Median (SD)	Max	
TSS	19	3.333	29.33 (52.45)	226.0	10.00	43.00 (46.34)	203.8	-36% (126%)
NO_3^-	19	0.014	0.674 (1.107)	4.650	0.014	0.465 (1.531)	6.392	5% (55%)
SO_4^{2-}	19	0.023	3.474 (3.552)	14.28	0.023	3.179 (4.336)	18.77	16% (62%)
PO_4^{3-}*	19	0.060	0.060 (0.051)	0.211	0.060	0.060 (0.065)	0.235	0% (62%)
NH_4^+ *	19	0.011	0.011 (0.103)	0.366	0.011	0.011 (0.084)	0.258	0% (379%)
Cu	19	0.0003	0.001 (0.002)	0.008	0.0003	0.001 (0.002)	0.009	-3% (71%)
Zn	19	0.008	0.008 (0.032)	0.130	0.008	0.017 (0.079)	0.322	0% (518%)
Pb**	19	0.002	0.002 (0)	0.002	0.002	0.002 (0)	0.002	0% (0%)

*50% or more of samples were below detection limit.

**all values were below detection limit

Investigation of load reductions (Table 4.5) showed much better performance than the EMC reductions; however, still not statistically significant for the inlet versus the outlet. Median load reductions ranged from -27% (Zn) to 38% (SO_4^{2-}); and, just as hydrologic performance was suspected to be hindered due to groundwater intrusion, so were loadings. Additionally, potential for error in load export calculations was possible as a result of applying a uniform EMC to the volume of water leaving the RSC. In reality, based on the hydrologic analysis, a portion of the flow came from an external source (groundwater). Therefore, if the groundwater did not contain high concentrations of the analyzed parameters, the calculations presented in this research would overestimate the export load. However, if high concentrations were present in the groundwater entering the system, load calculations would be under estimated. Water quality analysis was not performed on groundwater samples; thus, it was not possible to explicitly determine the influence, positive or negative, on the surface flow leaving the RSC.

Results of the water quality investigation found nutrient concentrations to be minimal entering the system, with several of the sample concentrations below detection level at the inlet. Possible sources of nutrients include fertilizer, decomposing organic material, atmospheric deposition, and animal waste (Bannerman et al. 1993, Khwanboonbumpen 2006, NADP 2018). However, it was suspected that these sources were limited in the watershed. A large portion of the watershed consisted of impervious surfaces, including the school building and parking lot, with the remaining portion of the watershed being low maintenance landscaping (i.e. grass and trees) that likely did not receive a significant amount of fertilizer application. Further, the school campus was not believed to be a significant source for decomposing organic material (few trees – minimal leaf litter), atmospheric deposition (not anticipated to be more elevated than that of other similar studies), or animal waste (relative to residential land uses). As a result, low nitrate,

Table 4.5 Median load reduction performance values.

Pollutant	Load							
	N	Inlet (g)			Outlet (g)			Median Reduction (SD)
		Min	Median (SD)	Max	Min	Median (SD)	Max	
TSS	19	438.3	5,404 (13,327)	53,981	86.62	5,080 (31,429)	124,066	-3% (126%)
NO_3^-	14	40.71	135.9 (310.0)	1,112	3.819	66.22 (252.8)	992.1	38% (44%)
SO_4^{2-}	15	135.9	640.2 (1,464)	5,931	5.554	408.0 (1,420)	5,307	37% (36%)
PO_4^{3-}*	4	1.765	16.06 (14.26)	32.26	1.545	11.24 (10.62)	25.46	25% (30%)
NH_4^+ *	8	0.630	7.471 (5.374)	18.01	0.773	2.931 (3.946)	11.85	19% (201%)
Cu	18	0.076	0.181 (0.202)	0.938	0.027	0.158 (0.373)	1.633	27% (94%)
Zn	12	0.939	2.797 (1.958)	7.879	0.285	3.516 (25.40)	92.43	-27% (784%)
Pb**	0	-		-	-		-	na

Instances where inlet and outlet were both below detection limit were removed from the load analysis

*50% or more of samples were below detection limit. For instances where more than have of the samples were below detection limit, load analysis was excluded.

**all values were below detection limit

phosphate, and ammonium concentrations entered the system and, thus, little to no concertation reductions (0-5%) were realized. More than half of the concentrations at both the inlet and outlet were below detection limit for phosphate and ammonium, which were thus removed from the load analysis. Median concentration reductions for sulfate were 16%, and nitrate were 5%, which might have indicated these pollutants were being removed through plant uptake and microbial activity, yet not significantly because the concentrations entering the system were already low (Gabriel et al. 2008, Khwanboonbumpen 2006). For all nutrients investigated, load reduction performances were higher than EMC reductions, the result of the combined effect of volume reduction.

Investigation into concentrations of dissolved copper, zinc, and lead entering the RSC found that zinc was present in varying concentrations, copper concentrations were low, and lead was not present (all values below detection limit). Primary sources for zinc and copper include motor oil and brake/tire wear (Davis et al. 2001). With the primary land use type being a parking lot used for school pick up and drop off, zinc and copper were expected to be seen in stormwater inlet concentrations. However, because the parking lot only experienced traffic twice a day (low traffic density, <250 vehicles per hour), the concentrations were expected to be less than what is typically realized for metals found in highway runoff (medium to high traffic density, 250+ vehicles per hour). Studies have shown that metal concentrations increase with increasing traffic density (Fakayode and Olu-Owolabi 2003). Typically, metals are removed from stormwater through sedimentation as they adsorb to soil particles. However, studies have showed that metals adsorbed to sediment have the ability to release to overlying water, with zinc having the greatest percent release (Lundy et al. 2017). While low concentrations were found to be entering the RSC, it is possible that they were entering the system as particulate species, accumulating in the pools and then released later as dissolved species, which would explain the -3% median copper concentration reduction and the -27% median load reduction for zinc, as chemical analysis tested for dissolved concentrations.

Another factor that had the potential to contribute to zinc export was the accumulation of fragmented asphalt in the system as it degraded from the parking lot and washed into the RSC with rain events. While asphalt has low concentrations of zinc, and is usually not enough to be considered a major contributor to stormwater pollution in highway runoff (Golding 2006), if the pieces of asphalt were regularly submerged for extended periods of time, as they were in the pools of the RSC, it is possible that zinc could have been transferred to the water resulting in export from the system.

4.4.3. Influences of Growing Season

More substantial differences in both hydrologic and water quality performance were found to exist between growing and non-growing seasons compared to the four annual seasons. This is likely, in part, because seasonal categorization does not always capture the considerable variation in biogeochemical processes that occurs throughout the year. To quantify this, events that occurred between April 18 and October 24 were categorized as growing season, and all other events were the non-growing season (OFA 2018). For growing and nongrowing seasons, cumulative volume reductions were 29% and 0%, respectively; median event-based volume reductions were 37% and 2%, respectively; and, median event-based peak flow rate reductions were 28% and 4%, respectively; with all reductions being significantly different between the two periods. Additionally, load reductions were typically greater during the growing season, with the exception of nitrate, but were not statistically significant (Table 4.6). Influence of vegetation was observed to increase from the first growing season (2016) to the second (2017) as volumetric

Table 4.6 Median percent reduction values for growing season and non-growing season

Parameter	N	Growing	Non-growing	p-value
Hydrologic Reductions				
Cumulative	58	29%	0%	na
Event -Based Volume	58	37% (32%)	2% (39%)	**0.002**
Peak	58	28% (38%)	4% (29%)	**0.007**
Water Quality Load Reductions				
TSS	19	-1% (110%)	-13% (137%)	0.72
NO_3^-	14	38% (49%)	44% (36%)	0.98
SO_4^{2-}	15	40% (37%)	11% (36%)	0.54
PO_4^{3-}*	4			
NH_4^+*	8			
Cu	18	40% (45%)	-1% (111%)	0.12
Zn	12	69% (73%)	-185% (989%)	0.17
Pb*	0			

*indicates not enough values to perform analysis
Values in bold indicate significant p-values (α=0.05)

reductions increased from 27% to 46%, while all other variables remained constant. This can be attributed to minimally established vegetation in the first year and an increased vegetative establishment in the second year. The increased hydrologic and water quality performance in the growing season was suspected to result from a few factors: (1) because the system is highly vegetated, the plants took up more water during the growing season and released through evapotranspiration, (2) the dense vegetation slowed down the flow of stormwater, resulting in higher residence times and greater opportunity for infiltration and settling of pollutants, and (3) during the growing season, the surrounding water table was lower. Seasonal water table changes result from increased uptake from plants in the watershed as well as drier weather conditions. In Knoxville, the lowest monthly average precipitation occurs between August and October, which coincides with the end of the growing season, thus less groundwater intrusion contributed to the volume leaving the system (NWS 2018). These combined factors allowed for improved hydrologic and water quality performance during the growing season.

4.5. Conclusions

This research showed how several factors substantially influenced RSC performance. Most obvious was the groundwater influence on the RSC resulting from a high water table. These groundwater influences fluctuated with season, resulting in differential performance for both hydrology and water quality. While reductions were observed to be best during the summer and worst during the winter, an even more descriptive characteristic than the four seasons were the trends realized between growing and non-growing season. During the growing season, hydrologic performance of the RSC was highest, with cumulative reductions of 29% and event-based reductions of 37%, compared to cumulative reductions of 6% and event-based reductions of 23%, for the 16-month monitoring period. This was attributed to more storage available within the sand/woodchip media underlying the system resulting from low groundwater table, less groundwater intrusion, and increased vegetative presence. Conversely, performance was severely hindered during the non-growing season when the water table was highest and no vegetation was present. However, the exact extent of the contributing groundwater to the system was unknown and an investigation of such would have been beneficial this research.

The results of the Knoxville RSC study were most similar to the Koryto et al. (2017) investigation of an RSC in Durham Co, NC, because it also had dense soils (HSG D) and a high water table. Both studies showed hindered hydrologic and water quality performance, further reinforcing that both variables should not be present at a site for these systems to be an effective SCM. While Koryto et al. (2017) show positive pollutant concentration reductions and poor load reductions due to export of stormwater, the RSC in this study showed the opposite – poor pollutant concentration reductions and improved load reductions. For TSS, which consistently had negative reduction values (or net export), it is suspected this can be attributed to more shallow pool depths than the recommended design. This occurred due to the presence of bedrock during construction and is theorized to have resulted in resuspension of sediments due to high velocity jets.

Although performance was hindered at the site due to preexisting subsurface dynamics, this work shows the potential for urban stormwater runoff mitigation by RSCs under the right site conditions (i.e. low water table, more permeable soils, no bedrock). This is best observed by the performance of the system when winter months, and thus high water table conditions, were removed and the overall volume reduction provided by the system increased from 6% (when all seasons are considered) to 19%. However, because stormwater conveyances often connect a drainage area to a receiving stream, locating a potential site where the water table is low and groundwater influences are minimized could prove to be a challenge. Therefore, utilizing RSCs in a treatment train approach to stormwater management is one recommendation for these practices. When used as a treatment train application, a practice can be chosen that maximizes volume reductions and coupled with the RSC to maximize energy dissipation. A second potential application would be an existing stormwater conveyance where head cutting has occurred and an opportunity is presented to backfill with a sandy media, creating the infiltration capacity and media storage necessary to successfully reduce inflow volumes and peak flow rates.

Additional research is still needed to fully quantify RSC performance values. In comparison to other green infrastructure controls there is substantially fewer studies performed on these systems. A continuation of this research that quantifies the extent of groundwater intrusion into the RSC investigated in this study would be beneficial to more accurately determine hydrologic and water quality performance. Lastly, an investigation of RSC performance at locations with the recommended site characteristics (as treatment train application or where occurrence of head cutting exists) would also be a novel contribution to RSC research.

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5. Summary of Key Findings, Future Research, and Final Recommendations

5.1. Summary of Key Findings

This multi-year study investigated several aspects of RSC performance including the potential for system design improvements to ensure stability, RSC-groundwater interactions, and stormwater runoff quality and quantity mitigation. The analyses in Chapter 2 stemmed from the limited design guidelines that existed for RSCs. This lead the researchers to hypothesize that system design and guidelines could be further fine-tuned to create a more effective system that strategically targets velocity and turbulent kinetic energy reductions, thus ensuring system stability. Specifically lacking in existing design guidance were recommendations for overall system length, or number of pools required. Additionally, vague guidance was given for the design of each pool's dimensions. This research was able to advance these previously established design guidelines through the use of three-dimensional, computational fluid dynamics software, Flow-3D^{TM}, providing insight into minimum system length and optimal length to width ratios for the pools.

In Chapter 3, the interactions between the RSC, which encourages stormwater infiltration, and the surrounding groundwater system was explored. The groundwater investigation showed that the RSC acted as a source to the overall groundwater system under small to medium sized events; however, under large events it acted as a sink. In Chapter 4 the performance of the system itself was quantified for water quality and quantity improvements. The findings from Chapter 3 were evident in the Chapter 4 hydrologic study as well, which showed percent reductions greatly decreased as event size increased. Further, it was discovered that the RSC had a net export of stormwater from the system during some events. This was attributed to groundwater intrusion into the RSC which appeared to vary seasonally and with event size. Additionally, an important factor found to influence groundwater interactions were the two sewer lines that ran beneath the system. The wells that were directly connected to the pervious zone of gravel surrounding the sewer line were found to be directly connected to the RSC, and thus stormwater runoff, potentially creating a short-circuiting of the natural flow path.

In addition to the sewer lines found at the site, bedrock was encountered during the construction of the RSC, requiring the pool depths to be shortened in order to fit into the site constraints. As indicated from the investigation of pool depths in the Chapter 2 modeling study, it was thought that the modification could have been partially responsible for the negative TSS reductions observed. While water quality and hydrologic reductions were not comparable to some of the previous studies found in literature, they were comparable to the only other site studied with similar site conditions. The primary confining factors to the water quality and hydrologic performance of the system were the high water table and dense soils (HSG C) found at the site. Further, low pollutant concentrations entering the system were believed to be another reason for poor reduction performance, as the concentrations were already at irreducible levels. Specific key findings of this work are presented as follows:

1. The modeling study concluded, in agreement with design recommendations, that pool design should have a minimum depth of 0.46 m (1.5ft). Design of the first pool of the system was most critical for dissipating erosive flows and should be carefully considered when implementing these practices.
2. A pool length to width ratio of 0.7 was the optimal design for this study based on the model, but lower length to width ratios, in general, were best for reducing velocities and energies. Again, it was determined that the first pool of the system was most critical, but subsequent pools will become increasingly critical as storm sizes increase. It should also

be noted that more pools than utilized herein may be needed to accommodate specific site conditions, such as varying flow rate, width, and/or slope.

3. System length (or number of pools) played a role in dissipating TKE and that there was a minimum length for which this occurs, especially under larger events, although this was not explicitly determined in this research.
4. The primary factors that influenced groundwater interactions were determined to be seasonal fluxes in water table elevation, varying weather patterns and storm sizes, and most significantly, the presence of underground infrastructure. It was determined that mounding did occur as a result of stormwater infiltration, particularly under small to medium sized events, and conversely, under large events the RSC became a sink for the surrounding water table.
5. As buried infrastructure is near ubiquitous in urban watersheds, this study determined that it was critical to locate this infrastructure prior to placement of infiltration based SCMs. The presence of buried utilities can have significant influence on movement of groundwater, often times short circuiting the water quality treatment mechanisms, in addition to host of other problems posed by buried infrastructure (i.e. leaking sewage, groundwater intrusion into conduits, etc.).
6. Cumulative volume reductions were 6%, median event-based volume reductions were 23%, and median peak flow reductions were 16% during the monitoring period; although, not statistically significant from inlet to outlet. Greater volume reductions were realized between the growing season (29% cumulative and 37% median event-based reductions), and non-growing season (0% cumulative and 2% median event-based reductions), and

were found to be statistically significant from one another, indicating the system performed best when the water table was lowest with ample vegetation (growing season).

7. The water quality investigation found that pollutant load reductions were higher than EMC reductions. One reason EMC reductions were suspected to be low was that pollutant concentrations entering the system were already at irreducible levels. The greater load reduction performance can be attributed to the volume reductions realized in the system. It was also found that significant differences in pollutant load reductions occurred between growing and non-growing seasons, while year-round load reductions occurred, they were not significant. This further reinforces the role of vegetation in these "regenerative" systems.

5.2. Future Research

1. Modeling studies are best validated with field scale studies; thus, the modeling work herein would benefit from additional empirical studies to support the results of the research which suggested a length to width ratio of 0.7. Further, this study made model simplifications, such as exclusion of the underlying sand seam and only modeling flow over the riffle sections (rather than through). Greater insight would be gained if these aspects were included in future simulations.
2. This study concluded that a minimum pool depth of 0.46 m was best for design, but also noted that greater depths may be equally or more efficient; however, these were not analyzed in this study and would warrant further investigation. Additionally, it would be beneficial to investigate the length thresholds required for establishing steady, low velocity flow of surface waters.

3. The groundwater investigation would benefit from an investigation of an RSC's infiltration capacity and potential mounding where buried infrastructure was not posing an influence on the system. Further, investigation of potential mounding and groundwater movement in varying soil conditions not previously investigated would be valuable.
4. This study was performed in East Tennessee, an area generally characterized by dense soils and varying water table depths. Previous research has been performed in coastal and piedmont regions of North Carolina. Research would benefit from additional studies in regions, soil types, and weather patterns not already investigated.
5. A hydrologic and water quality performance investigation testing the design feature alternatives suggested from the modeling study would be a valuable contribution to the research.

5.3. Final Recommendations

Performance of RSC systems as a whole was variable based on local groundwater, but this study indicated there was potential for improvement under the right site conditions (i.e. low groundwater table or more permeable soils). Because stormwater conveyances often connect a drainage area to a receiving stream, locating a potential site where the water table is low and groundwater influences are minimized could prove to be a challenge. Therefore, in the Knoxville region (and other areas with similar soil types) there are a few options. The following are examples of potential options for RSC application:

1. If utilizing an RSC as a stand-alone practice, care should be taken to locate a site with naturally low water table year-round and avoidance of bedrock or buried utilities.
2. RSCs may be utilized in a treatment train approach to stormwater management. An example of this would be utilizing bioretention cells in parking lots (primary treatment

practice) with drainage from the cells traveling to an RSC (secondary treatment practice) via the storm sewer for energy dissipation and final treatment before being discharged to the receiving stream. Another example would be to locate the RSC as the primary treatment mechanism, which then discharges to a secondary treatment practice.

3. An existing stormwater conveyance where head cutting has occurred could be replaced with an RSC, thus creating the opportunity to backfill with a sandy media. This would result in significant improvement of infiltration capacity and storage relative to alternative gray infrastructure, creating another ideal site for implementation of RSCs.

While RSCs have proven to be limited to adequate site conditions, they have potential to be an excellent choice of SCM, for both hydrologic and water quality treatment, given the right conditions. Thus, to aid designers in siting and sizing these practices, the previously mentioned recommendations have been used to develop guidance tools for designing RSCs. These tools including a decision tree for placing RSCs under varying site conditions (Figure 5.1) and a regression analysis (Figure 5.2) to suggest the number of pools necessary to adequately dissipate erosive velocities and energies, depending on incoming flow rates (Table 5.1). It should be noted that this is purely suggestive and not intended to be a concrete recommendation as the regression was based on a limited number of data points.

Figure 5.1 guides the designer through a series of questions, starting with site conditions, and ending with final recommendation for practice application. The first step is to determine if bedrock or buried infrastructure is located at the site. If so, it is suggested to avoid the use of RSCs. Any instances where head cutting has occurred, creating the opportunity to backfill, RSCs may be used as either stand-alone or treatment train practices. Otherwise, if bedrock or buried

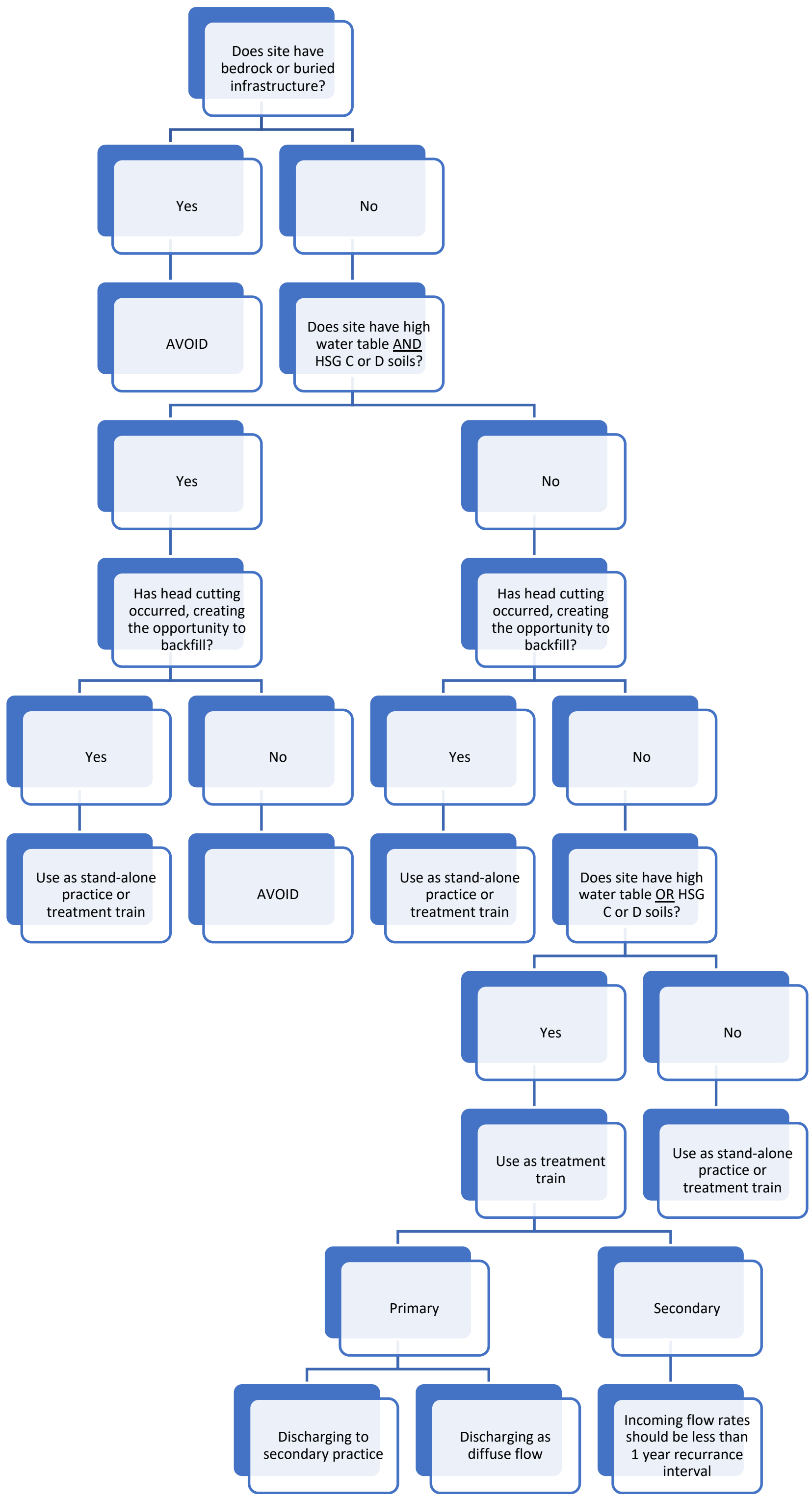

Figure 5.1 Decision tree for siting RSCs based on existing site conditions, including when to avoid use of the practice, when to use as a treatment train application, or when to use as a stand-alone practice

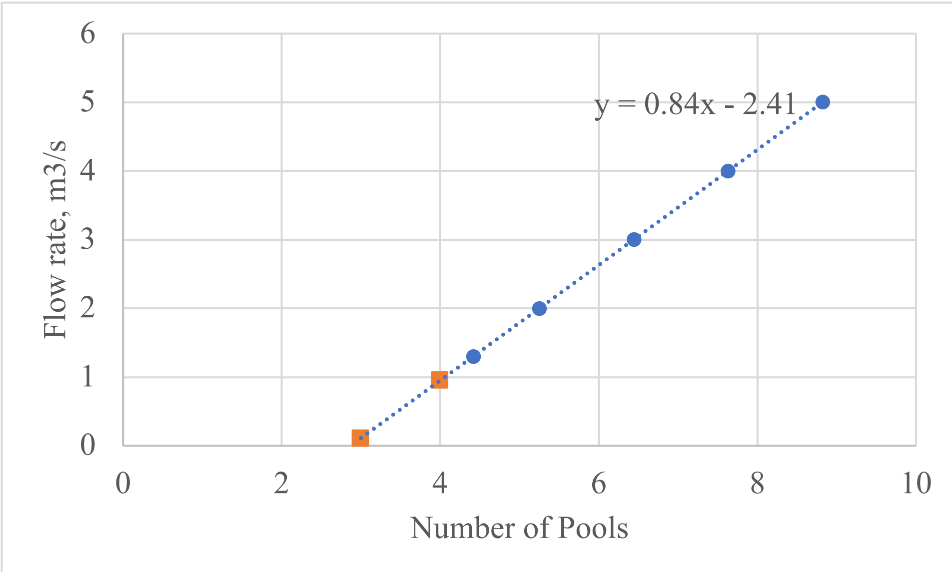

Figure 5.2 Correlation between flow rate and number of pools. The squares represent the two data points from the modeling study (Chapter 2) and the circles represent predicted values determined using the resulting equation of the regression line.

Table 5.1 Correlation between flow rate and number of pools. The first two flow rates correspond to the low flow event (field measured) and 25-year event and the number of pools that was suggested from the modeling study in Chapter 2. The remaining flow rates are hypothetical values.

Flow rate, m3/s	Number of Pools	Notes
0.11	3	low flow (field measured)
0.95	4	25-year event
1	4	
2	5	
3	6	
4	8	
5	9	

infrastructure are not present, but both high water table and dense soils are present, it is again recommended to avoid the use of RSCs. If there is no head cutting, high water table, or dense soils present RSCs may be used as either a stand-alone or treatment train practice. However, if either high water table or dense soils are present, the practice must be used as a treatment train application; and, if used as the secondary practice, it is suggested that the receiving flows are equal to the 1-year recurrence interval or less.

Appendices

Appendix A: Three-Dimensional Modeling of the Hydraulic Function and Channel Stability of Regenerative Stormwater Conveyances

Table A.1 Velocity measurements at each analysis location for the nine analysis locations in each of the three pools for all length to width combinations evaluated

			Velocity Measurement Location								
	Simulation Name	l/w	1	2	3	4	5	6	7	8	9
Pool 1	**FINAL_1.5ft pools_0.7 Y**	0.72	0.0201	0.0032	0.0024	0.0020	0.0396	0.0055	0.0225	0.0920	0.0144
	FINAL_1.5ft pools_0.8 Y	0.82	0.0202	0.0003	0.0302	0.0003	0.0003	0.0005	0.0322	0.0188	0.0029
	FINAL_1.5ft pools_0.9 Y	0.92	0.0346	0.0000	0.0391	0.0002	0.0558	0.0003	0.0227	0.0000	0.0977
	FINAL_1.5ft pools	1.03	0.0384	0.0041	0.0083	0.0438	0.0180	0.0001	0.0007	0.0030	0.0113
	FINAL_1.5ft pools_0.9 X	1.14	0.0478	0.0266	0.0338	0.0727	0.0206	0.0041	0.0132	0.0013	0.0536
	FINAL_1.5ft pools_0.8 X	1.28	0.5704	0.0002	0.0070	0.1993	0.0076	0.0022	0.0766	0.0005	0.0072
	FINAL_1.5ft pools_0.7 X	1.46	0.7943	0.0003	0.0054	0.7626	0.0087	0.0082	0.0146	0.0461	0.0061
Pool 2	**FINAL_1.5ft pools_0.7 Y**	0.72	0.0001	0.2873	0.0030	0.0000	0.0007	0.0282	0.0061	0.0101	0.0171
	FINAL_1.5ft pools_0.8 Y	0.83	0.0012	0.0703	0.0510	0.0038	0.0006	0.1738	0.0001	0.0214	0.0515
	FINAL_1.5ft pools_0.9 Y	0.94	0.0064	0.3671	0.0147	0.0015	0.0016	0.0227	0.0000	0.0136	0.0275
	FINAL_1.5ft pools	1.05	0.0003	0.2039	0.0000	0.0090	0.0078	0.0678	0.0003	0.0484	0.1646
	FINAL_1.5ft pools_0.9 X	1.15	0.0006	0.2852	0.0005	0.0004	0.0608	0.0411	0.0025	0.0264	0.0200
	FINAL_1.5ft pools_0.8 X	1.32	0.0262	0.0012	0.0394	0.0258	0.0002	0.1833	0.0044	0.0014	0.0827
	FINAL_1.5ft pools_0.7 X	1.49	0.0033	0.0025	0.0252	0.0024	0.0466	0.2064	0.0006	0.0035	0.0233
Pool 3	**FINAL_1.5ft pools_0.7 Y**	0.71	0.4126	0.4350	0.7857	0.0003	0.0336	0.0822	0.0030	0.1757	0.0066
	FINAL_1.5ft pools_0.8 Y	0.81	0.1186	0.2700	0.0199	0.0029	0.0007	0.0012	0.0001	0.2143	0.0105
	FINAL_1.5ft pools_0.9 Y	0.91	0.0034	0.1687	0.0138	0.0000	0.0666	0.0003	0.0007	0.0863	0.0006
	FINAL_1.5ft pools	1.09	0.3456	0.1571	0.0050	0.0001	0.3642	0.0030	0.0020	0.0171	0.0005
	FINAL_1.5ft pools_0.9 X	1.11	0.0035	0.0049	0.0271	0.0356	0.0020	0.0072	0.0051	0.0202	0.0000
	FINAL_1.5ft pools_0.8 X	1.24	0.0001	0.2132	0.0020	0.0104	0.0036	0.0005	0.1107	0.0424	0.0019
	FINAL_1.5ft pools_0.7 X	1.44	0.0039	0.0483	0.3650	0.0005	0.0006	0.0334	0.0008	0.0782	0.0048

Pool=1 DAV/TKE=TKE

1 With Variables:	L/W
1 Variables:	Value

Simple Statistics						
Variable	N	Mean	Std Dev	Median	Minimum	Maximum
L/W	7	1.05419	0.26097	1.02664	0.72022	1.46120
Value	7	0.00797	0.00310	0.00719	0.00450	0.01388

Spearman Correlation Coefficients, N = 7 Prob > |r| under H0: Rho=0	
	Value
L/W	0.96429 0.0005

Figure A.1 Statistical Analysis Results from SAS Enterprise Guide 7.1, TKE in Pool 1

Pool=2 DAV/TKE=TKE

1 With Variables:	L/W
1 Variables:	Value

Simple Statistics						
Variable	N	Mean	Std Dev	Median	Minimum	Maximum
L/W	7	1.07326	0.27167	1.04974	0.72243	1.49034
Value	7	0.00779	0.00240	0.00643	0.00524	0.01091

Spearman Correlation Coefficients, N = 7 Prob > |r| under H0: Rho=0	
	Value
L/W	-0.32143 0.4821

Figure A.2 Statistical Analysis Results from SAS Enterprise Guide 7.1, TKE in Pool 2

Pool=3 DAV/TKE=TKE

1 With Variables:	L/W
1 Variables:	Value

Simple Statistics						
Variable	N	Mean	Std Dev	Median	Minimum	Maximum
L/W	7	1.04504	0.25361	1.08505	0.71016	1.43996
Value	7	0.00976	0.00482	0.00771	0.00446	0.01741

Spearman Correlation Coefficients, N = 7 Prob > |r| under H0: Rho=0	
	Value
L/W	-0.21429 0.6445

Figure A.3 Statistical Analysis Results from SAS Enterprise Guide 7.1, TKE in Pool 3

Pool=1 DAV/TKE=DAV

1 With Variables:	L/W
1 Variables:	Value

Simple Statistics						
Variable	N	Mean	Std Dev	Median	Minimum	Maximum
L/W	7	1.05419	0.26097	1.02664	0.72022	1.46120
Value	7	0.28353	0.05465	0.27507	0.20895	0.35539

Spearman Correlation Coefficients, N = 7 Prob > |r| under H0: Rho=0	
	Value
L/W	0.96429 0.0005

Figure A.4 Statistical Analysis Results from SAS Enterprise Guide 7.1, DAV in Pool 1

Pool=2 DAV/TKE=DAV

1 With Variables:	L/W
1 Variables:	Value

Simple Statistics						
Variable	N	Mean	Std Dev	Median	Minimum	Maximum
L/W	7	1.07326	0.27167	1.04974	0.72243	1.49034
Value	7	0.21230	0.04780	0.22336	0.13357	0.27475

Spearman Correlation Coefficients, N = 7 Prob > |r| under H0: Rho=0	
	Value
L/W	0.46429 0.2939

Figure A.5 Statistical Analysis Results from SAS Enterprise Guide 7.1, DAV in Pool 2

Pool=3 DAV/TKE=DAV

1 With Variables:	L/W
1 Variables:	Value

Simple Statistics						
Variable	N	Mean	Std Dev	Median	Minimum	Maximum
L/W	7	1.04504	0.25361	1.08505	0.71016	1.43996
Value	7	0.19042	0.04080	0.19193	0.12120	0.24112

Spearman Correlation Coefficients, N = 7 Prob > |r| under H0: Rho=0	
	Value
L/W	-0.21429 0.6445

Figure A.6 Statistical Analysis Results from SAS Enterprise Guide 7.1, DAV in Pool 3

Page Break

Correlation Analysis

The CORR Procedure

Flow=low flow

1 With Variables:	L/W
1 Variables:	Value

Simple Statistics						
Variable	N	Mean	Std Dev	Median	Minimum	Maximum
L/W	10	1.12649	0.24505	1.14306	0.82258	1.46120
Value	10	0.15209	0.15518	0.12523	0.00630	0.35539

Spearman Correlation Coefficients, N = 10 Prob > |r| under H0: Rho=0	
	Value
L/W	0.46775 0.1728

Figure A.7 Statistical Analysis Results from SAS Enterprise Guide 7.1, L/W for low flow

Correlation Analysis

The CORR Procedure

Flow=10

1 With Variables:	L/W
1 Variables:	Value

Simple Statistics						
Variable	N	Mean	Std Dev	Median	Minimum	Maximum
L/W	10	1.12649	0.24505	1.14306	0.82258	1.46120
Value	10	0.47211	0.46578	0.35000	0.03792	1.15534

Spearman Correlation Coefficients, N = 10 Prob > |r| under H0: Rho=0	
	Value
L/W	0.00000 1.0000

Figure A.8 Statistical Analysis Results from SAS Enterprise Guide 7.1, L/W for 10-year event

Page Break

Correlation Analysis

The CORR Procedure

Flow=25

1 With Variables:	L/W
1 Variables:	Value

Simple Statistics						
Variable	N	Mean	Std Dev	Median	Minimum	Maximum
L/W	10	1.12649	0.24505	1.14306	0.82258	1.46120
Value	10	0.64798	0.60589	0.53995	0.06235	1.38680

Spearman Correlation Coefficients, N = 10 Prob > |r| under H0: Rho=0	
	Value
L/W	0.04924 0.8926

Figure A.9 Statistical Analysis Results from SAS Enterprise Guide 7.1, L/W for 25-year event

Appendix B: Groundwater Interactions with Regenerative Stormwater Conveyance Infiltration: A Case Study, Knoxville, TN

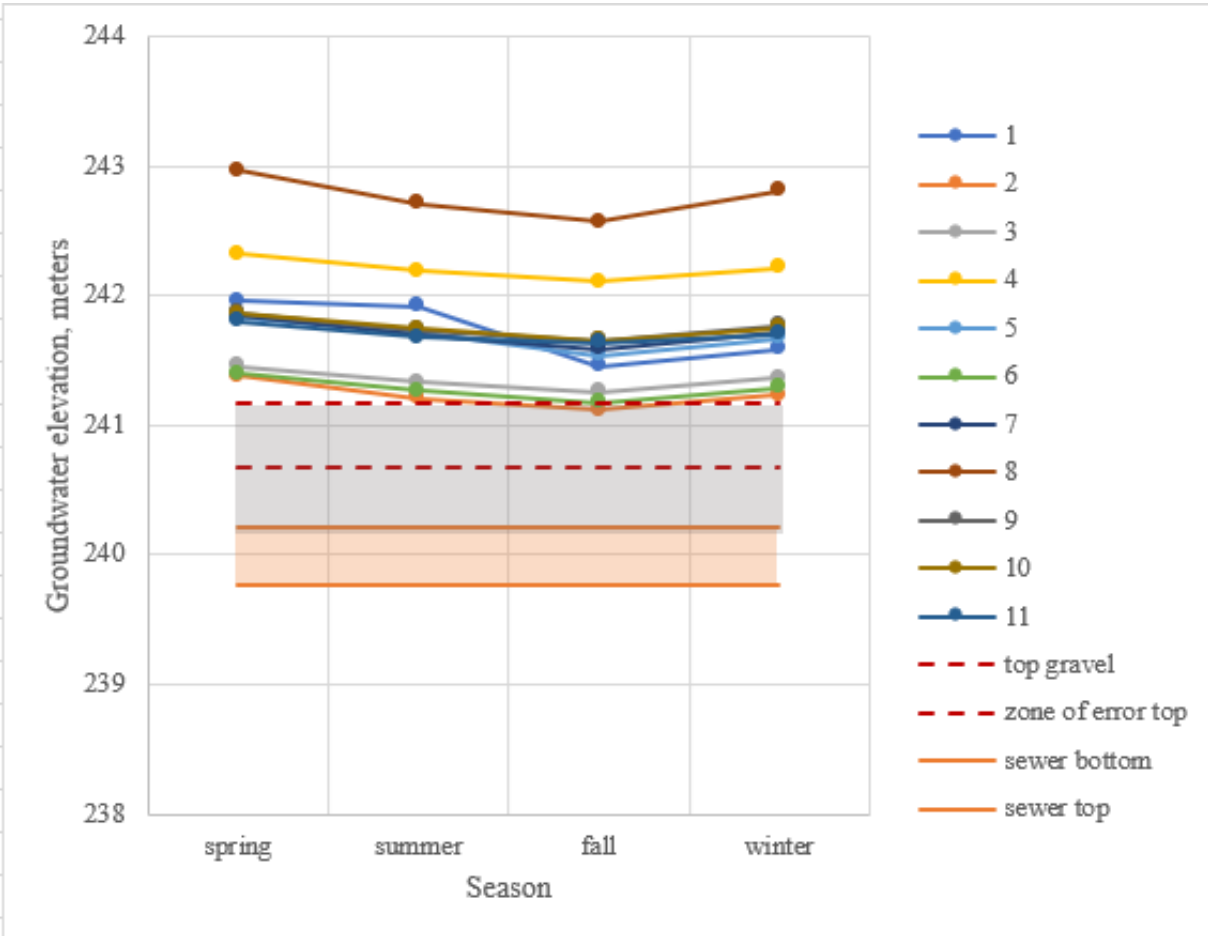

Figure B.1 Groundwater elevation for each of the wells, averaged by season, plotted in comparison to location of sewer line.

Appendix C: Water Quality and Quantity Performance of a Regenerative Stormwater Conveyance in Knoxville, TN

Table C.1 EMC for each pollutant investigated

Storm	Rain	TSS			NO3			SO4			PO4			NH4			Cu			Zn			Pb		
	cm	Inlet	Outlet	% red	inlet	outlet	% red	inlet	outlet	% red	inlet	outlet	% red	inlet	outlet	% red	inlet	outlet	% red	inlet	outlet	% red	inlet	outlet	% red
		(mg/L)			(mg/L)			(mg/L)			(mg/L)			(mg/L)			(mg/L)			(mg/L)			(mg/L)		
4/1/16	na	76.82	86.67	-13%	0.443	0.014	97%	2.86	2.67	7%	0.06	0.06	0%	0.011	0.011	0%	0.0006	0.0006	-14%	0.008	0.008	0%	0.002	0.002	0%
4/7/16	na	28.57	34.46	-21%	0.014	0.014	0%	3.47	0.02	99%	0.06	0.06	0%	0.054	0.050	7%	0.0013	0.0012	5%	0.008	0.008	0%	0.002	0.002	0%
4/12/16	0.48	22.73	31.82	-40%	0.014	0.014	0%	0.02	0.02	0%	0.06	0.06	0%	0.011	0.192	-1691%	0.0022	0.0024	-10%	0.026	0.017	37%	0.002	0.002	0%
4/28/16	0.30	37.14	14.71	60%	0.014	0.014	0%	0.02	0.02	0%	0.06	0.06	0%	0.366	0.253	31%	0.0081	0.0053	34%	0.088	0.067	24%	0.002	0.002	0%
4/30/16	0.20	44.10	85.71	-94%	4.650	3.557	24%	8.20	5.97	27%	0.17	0.23	-34%	0.288	0.258	10%	0.0061	0.0040	33%	0.057	0.043	24%	0.002	0.002	0%
4/30/16	1.17	70.51	79.75	-13%	1.054	0.842	20%	2.66	1.90	28%	0.21	0.24	-12%	0.059	0.083	-40%	0.0009	0.0011	-27%	0.022	0.008	64%	0.002	0.002	0%
5/2/16	1.32	149.7	203.8	-36%	0.964	0.920	5%	3.79	3.17	16%	0.21	0.17	17%	0.048	0.067	-39%	0.0018	0.0018	-3%	0.018	0.008	57%	0.002	0.002	0%
7/5/16	0.76	19.33	53.33	-176%	0.674	0.898	-33%	7.32	5.65	23%	0.06	0.06	0%	0.206	0.011	95%	0.0022	0.0019	16%	0.008	0.008	0%	0.002	0.002	0%
7/7/16	0.15	51.33	110.7	-116%	0.723	0.395	45%	4.31	3.18	26%	0.06	0.06	0%	0.011	0.011	0%	0.0009	0.0006	31%	0.008	0.008	0%	0.002	0.002	0%
9/18/16	0.20	26.00	21.33	18%	0.954	0.507	47%	2.44	3.41	-40%	0.06	0.06	0%	0.011	0.011	0%	0.0022	0.0009	60%	0.050	0.127	-155%	0.002	0.002	0%
11/19/16	0.53	15.00	43.00	-187%	2.601	6.392	-146%	5.88	18.77	-219%	0.06	0.23	-278%	0.064	0.137	-115%	0.0084	0.0087	-3%	0.130	0.322	-147%	0.002	0.002	0%
11/28/16	1.85	46.00	18.00	61%	0.597	0.539	10%	6.04	4.90	19%	0.06	0.06	0%	0.011	0.011	0%	0.0017	0.0012	27%	0.008	0.018	-125%	0.002	0.002	0%
11/30/16	8.61	45.33	85.33	-88%	0.934	0.303	68%	2.66	2.41	9%	0.06	0.06	0%	0.011	0.011	0%	0.0008	0.0011	-43%	0.008	0.008	0%	0.002	0.002	0%
12/6/16	2.90	29.33	104.7	-257%	0.014	0.014	0%	0.02	0.02	0%	0.06	0.06	0%	0.011	0.011	0%	0.0006	0.0008	-38%	0.008	0.008	0%	0.002	0.002	0%
1/1/17	na	3.33	10.00	-200%	2.021	1.953	3%	14.28	10.45	27%	0.06	0.06	0%	0.011	0.011	0%	0.0008	0.0011	-38%	0.008	0.182	-2199%	0.002	0.002	0%
2/7/17	na	226.0	83.33	63%	1.145	0.505	56%	8.33	4.76	43%	0.06	0.06	0%	0.152	0.152	0%	0.0006	0.0003	52%	0.008	0.032	-299%	0.002	0.002	0%
3/1/17	3.20	26.67	34.00	-28%	0.014	0.014	0%	0.02	0.02	0%	0.06	0.06	0%	0.011	0.011	0%	0.0003	0.0010	-261%	0.008	0.080	-913%	0.002	0.002	0%
6/5/17	na	4.67	26.67	-471%	0.447	0.389	13%	6.56	3.60	45%	0.06	0.06	0%	0.011	0.011	0%	0.0006	0.0014	-131%	0.032	0.084	-160%	0.002	0.002	0%
6/23/17	1.73	21.33	24.67	-16%	0.215	0.465	-116%	2.30	4.02	-75%	0.06	0.06	0%	0.011	0.011	0%	0.0003	0.0003	0%	0.008	0.008	0%	0.002	0.002	0%
MEDIAN		**29.33**	**43.00**	**-36%**	**0.674**	**0.465**	**5%**	**3.474**	**3.179**	**16%**	**0.060**	**0.060**	**0%**	**0.011**	**0.011**	**0%**	**0.001**	**0.001**	**-3%**	**0.008**	**0.017**	**0%**	**0.002**	**0.002**	**0%**

Table C.2 Pollutant loading values for each pollutant investigated

Storm	Rain	Volume, L		TSS			NO3			SO4			P04			NH4			Cu			Zn			Pb		
	cm	Inlet	Outlet	inlet	outlet	% red	inlet	outlet	% red	inlet	outlet	% red	inlet	outlet	% red	inlet	outlet	% red	inlet	outlet	% red	inlet	outlet	% red	inlet	outlet	% red
				g			g			g			g			g			g			g			g		
4/1/16	na	2.8E+05	2.8E+05	21840	24261	-11%	125.8	3.8	97%	811.8	747.3	8%							0.162	0.181	-12%						
4/7/16	na	2.5E+05	2.4E+05	7105	8185	-15%				863.9	5.6	99%				13.329	11.852	11%	0.318	0.288	9%						
4/12/16	0.5	5.9E+04	2.2E+04	1339	689	49%										0.630	4.151	-559%	0.127	0.052	59%	1.555	0.363	77%			
4/28/16	0.3	2.1E+04	5.9E+03	773	87	89%										7.624	1.489	80%	0.168	0.031	81%	1.824	0.394	78%			
4/30/16	0.2	1.7E+04	6.6E+03	731	568	22%	77.0	23.6	69%	135.9	39.6	71%	2.884	1.545	46%	4.766	1.712	64%	0.101	0.027	73%	0.939	0.285	70%			
4/30/16	1.2	1.4E+05	1.1E+05	9768	8617	12%	146.0	90.9	38%	368.6	205.6	44%	29.23	25.46	13%	8.215	9.001	-10%	0.119	0.118	1%	3.068	0.855	72%			
5/2/16	1.3	1.5E+05	1.2E+05	22919	23545	-3%	147.6	106.2	28%	580.6	366.4	37%	32.26	20.18	37%	7.319	7.683	-5%	0.270	0.209	22%	2.828	0.914	68%			
7/5/16	0.8	8.7E+04	7.2E+04	1692	3853	-128%	59.0	64.8	-10%	640.2	408.0	36%				18.007	0.773	96%	0.194	0.134	31%						
7/7/16	0.2	1.1E+05	7.8E+04	5404	8633	-60%	76.2	30.8	60%	454.0	248.0	45%							0.096	0.049	49%						
9/18/16	0.2	1.6E+05	1.1E+05	4097	2411	41%	150.4	57.3	62%	384.0	385.9	0%							0.345	0.100	71%	7.879	14.403	-83%			
11/19/16	0.5	2.9E+04	1.0E+04	438	432	1%	76.0	64.3	15%	171.9	188.7	-10%	1.765	2.298	-30%	1.859	1.372	26%	0.245	0.087	64%	3.809	3.241	15%			
11/28/16	1.9	2.8E+05	2.1E+05	13056	3829	71%	169.6	114.8	32%	1713.9	1042.7	39%							0.480	0.263	45%	2.246	3.792	-69%			
11/30/16	8.6	1.2E+06	1.5E+06	53981	124066	-130%	1111.9	439.8	60%	3169.9	3509.3	-11%							0.938	1.633	-74%						
12/6/16	2.9	6.4E+05	8.3E+05	18829	86399	-359%													0.379	0.673	-78%						
1/1/17	na	4.2E+05	5.1E+05	1385	5081	-267%	839.6	992.1	-18%	5930.7	5307.4	11%							0.330	0.556	-68%	3.287	92.427	-2712%			
2/7/17	na	1.3E+05	1.3E+05	30065	11147	63%	152.4	67.6	56%	1108.1	637.0	43%							0.076	0.036	52%	1.053	4.227	-302%			
3/1/17	3.2	3.5E+05	4.1E+05	9319	14012	-50%													0.095	0.405	-325%	2.765	33.037	-1095%			
6/5/17	na	1.9E+05	1.3E+05	882	3554	-303%	84.6	51.8	39%	1240.8	479.9	61%							0.113	0.183	-63%	6.075	11.147	-83%			
6/23/17	1.7	2.0E+05	1.8E+05	4362	4391	-1%	44.0	82.7	-88%	470.0	715.1	-52%															
MEDIAN		**1.6E+05**	**1.3E+05**	**5404.5**	**5080.6**	**-3%**	**135.9**	**66.22**	**38%**	**640.2**	**408.0**	**37%**	**16.06**	**11.24**	**25%**	**7.471**	**2.931**	**19%**	**0.181**	**0.158**	**27%**	**2.797**	**3.516**	**-27%**			

Table C.3 Volume and Peak flow rate values for each storm event recorded

Storm	Season	Year	Growing Season	Volume, m3			Peak Flowrate, m3/s		
				Inlet	Outlet	Reduction	Inlet	Outlet	Reduction
4/1/2016	Spring	2016	growing	284.13	279.77	2%	0.14	0.10	31%
4/7/2016	Spring	2016	growing	248.53	237.38	4%	0.04	0.03	8%
4/12/2016	Spring	2016	growing	58.86	21.65	63%	0.01	0.01	45%
4/28/2016	Spring	2016	growing	20.80	5.89	72%	0.01	0.00	60%
4/30/2016	Spring	2016	growing	16.56	6.62	60%	0.01	0.01	57%
4/30/2016	Spring	2016	growing	138.44	107.99	22%	0.10	0.07	25%
5/2/2016	Spring	2016	growing	153.02	115.46	25%	0.10	0.07	25%
7/5/2016	Summer	2016	growing	87.45	72.19	17%	0.04	0.04	6%
7/7/2016	Summer	2016	growing	105.22	77.97	26%	0.16	0.10	38%
8/12/2016	Summer	2016	growing	101.80	53.66	47%	0.16	0.09	40%
8/14/2016	Summer	2016	growing	26.09	8.43	68%	0.03	0.02	41%
8/15/2016	Summer	2016	growing	43.58	26.38	39%	0.05	0.04	13%
8/16/2016	Summer	2016	growing	41.97	53.49	-27%	0.10	0.07	25%
9/18/2016	Fall	2016	growing	157.49	112.95	28%	0.04	0.05	-23%
11/19/2016	Fall	2016	non-growing	29.21	10.05	66%	0.01	0.01	27%
11/28/2016	Fall	2016	non-growing	283.65	212.62	25%	0.04	0.04	4%
11/30/2016	Fall	2016	non-growing	1190.04	1453.04	-22%	0.10	0.07	25%
12/3/2016	Winter	2016	non-growing	612.84	634.40	-4%	0.03	0.03	-10%
12/6/2016	Winter	2016	non-growing	641.53	824.97	-29%	0.06	0.07	-14%
12/24/2016	Winter	2016	non-growing	339.88	355.84	-5%	0.03	0.03	-5%
12/27/2016	Winter	2016	non-growing	161.56	182.08	-13%	0.02	0.03	-7%
12/28/2016	Winter	2016	non-growing	385.59	493.89	-28%	0.04	0.05	-30%
1/1/2017	Winter	2016	non-growing	415.19	507.76	-22%	0.03	0.03	2%
1/10/2017	Winter	2016	non-growing	320.95	372.82	-16%	0.03	0.03	-4%
1/22/2017	Winter	2016	non-growing	666.86	939.05	-41%	0.07	0.11	-56%
1/25/2017	Winter	2016	non-growing	137.23	157.74	-15%	0.04	0.04	1%
2/7/2017	Winter	2016	non-growing	132.95	133.69	-1%	0.04	0.05	-29%
2/8/2017	Winter	2016	non-growing	122.31	136.35	-11%	0.06	0.07	-23%
2/28/2017	Winter	2016	non-growing	225.30	189.24	16%	0.10	0.07	25%
3/1/2017	Spring	2017	non-growing	377.81	467.80	-24%	0.14	0.10	27%
3/7/2017	Spring	2017	non-growing	241.34	212.53	12%	0.03	0.03	3%
3/10/2017	Spring	2017	non-growing	228.18	240.75	-6%	0.04	0.04	5%
3/13/2017	Spring	2017	non-growing	361.28	390.91	-8%	0.03	0.03	4%
3/17/2017	Spring	2017	non-growing	407.10	318.91	22%	0.03	0.03	3%
3/21/2017	Spring	2017	non-growing	177.89	110.60	38%	0.04	0.05	-14%
3/26/2017	Spring	2017	non-growing	119.79	27.25	77%	0.02	0.02	12%
3/27/2017	Spring	2017	non-growing	253.23	57.28	77%	0.04	0.03	18%
3/30/2017	Spring	2017	non-growing	183.30	27.08	85%	0.04	0.03	21%
4/3/2017	Spring	2017	non-growing	546.50	238.46	56%	0.04	0.04	14%
4/5/2017	Spring	2017	non-growing	168.22	142.24	15%	0.06	0.07	-16%
4/17/2017	Spring	2017	non-growing	28.95	4.73	84%	0.01	0.00	62%
4/17/2017	Spring	2017	non-growing	33.37	5.80	83%	0.01	0.00	100%
4/18/2017	Spring	2017	growing	85.58	37.98	56%	0.01	0.01	34%
5/21/2017	Spring	2017	growing	28.84	9.79	66%	0.02	0.01	47%

Table C3, continued. Volume and Peak flow rate values for each storm event recorded

Storm	Season	Year	Growing Season	Volume, m3			Peak Flowrate, m3/s		
				Inlet	Outlet	Reduction	Inlet	Outlet	Reduction
5/23/2017	Spring	2017	growing	20.55	3.08	85%	0.03	0.02	28%
5/23/2017	Spring	2017	growing	98.82	61.95	37%	0.03	0.02	28%
5/24/2017	Spring	2017	growing	167.85	144.87	14%	0.04	0.07	-81%
5/26/2017	Spring	2017	growing	4.87	0.17	97%	0.00	0.00	97%
5/27/2017	Spring	2017	growing	168.19	150.75	10%	0.08	0.08	4%
5/30/2017	Spring	2017	growing	99.59	77.60	22%	0.10	0.07	25%
6/1/2017	Summer	2017	growing	5.72	0.00	100%	0.00	0.00	100%
6/4/2017	Summer	2017	growing	21.79	6.08	72%	0.01	0.01	53%
6/5/2017	Summer	2017	growing	188.99	133.21	30%	0.03	0.02	9%
6/13/2017	Summer	2017	growing	217.80	190.57	13%	0.04	0.06	-35%
6/14/2017	Summer	2017	growing	34.67	3.11	91%	0.01	0.00	70%
6/19/2017	Summer	2017	growing	53.57	36.37	32%	0.04	0.05	-37%
6/22/2017	Summer	2017	growing	69.70	6.99	90%	0.01	0.01	35%
6/23/2017	Summer	2017	growing	204.35	177.89	13%	0.10	0.07	25%

Nonparametric One-Way ANOVA

The NPAR1WAY Procedure

Wilcoxon Scores (Rank Sums) for Variable peak,m3/s Classified by Variable location					
location	N	Sum of Scores	Expected Under H0	Std Dev Under H0	Mean Score
inlet	58	5075.0	3393.0	181.091017	87.50
outlet	58	1711.0	3393.0	181.091017	29.50
Average scores were used for ties.					

Wilcoxon Two-Sample Test	
Statistic (S)	5075.0000
Normal Approximation	
Z	9.2854
One-Sided Pr > Z	<.0001
Two-Sided Pr > |Z|	<.0001
t Approximation	
One-Sided Pr > Z	<.0001
Two-Sided Pr > |Z|	<.0001
Exact Test	
One-Sided Pr >= S	<.0001
Two-Sided Pr >= |S - Mean|	<.0001
Z includes a continuity correction of 0.5.	

Kruskal-Wallis Test	
Chi-Square	86.2697
DF	1
Pr > Chi-Square	<.0001

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Figure C.1 Results of statistical analysis using SAS Enterprise Guide 7.1, peak flow at inlet and outlet

Nonparametric One-Way ANOVA

The NPAR1WAY Procedure

Wilcoxon Scores (Rank Sums) for Variable volume, l Classified by Variable location					
location	N	Sum of Scores	Expected Under H0	Std Dev Under H0	Mean Score
inlet	58	3647.50	3393.0	181.104594	62.887931
outlet	58	3138.50	3393.0	181.104594	54.112069
Average scores were used for ties.					

Wilcoxon Two-Sample Test	
Statistic (S)	3647.5000
Normal Approximation	
Z	1.4025
One-Sided Pr > Z	0.0804
Two-Sided Pr > |Z|	0.1608
t Approximation	
One-Sided Pr > Z	0.0817
Two-Sided Pr > |Z|	0.1635
Exact Test	
One-Sided Pr >= S	0.0805
Two-Sided Pr >= |S - Mean|	0.1609
Z includes a continuity correction of 0.5.	

Kruskal-Wallis Test	
Chi-Square	1.9748
DF	1
Pr > Chi-Square	0.1599

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Figure C.2 Results of statistical analysis using SAS Enterprise Guide 7.1, volume at inlet and outlet

Nonparametric One-Way ANOVA

The NPAR1WAY Procedure

Parameter=Cu

Wilcoxon Scores (Rank Sums) for Variable Concentration Classified by Variable Location					
Location	N	Sum of Scores	Expected Under H0	Std Dev Under H0	Mean Score
inlet	19	356.0	370.50	34.233992	18.736842
outlet	19	385.0	370.50	34.233992	20.263158
Average scores were used for ties.					

Wilcoxon Two-Sample Test	
Statistic	356.0000
Normal Approximation	
Z	-0.4090
One-Sided Pr < Z	0.3413
Two-Sided Pr > |Z|	0.6826
t Approximation	
One-Sided Pr < Z	0.3425
Two-Sided Pr > |Z|	0.6849
Z includes a continuity correction of 0.5.	

Kruskal-Wallis Test	
Chi-Square	0.1794
DF	1
Pr > Chi-Square	0.6719

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Figure C.3 Results of statistical analysis using SAS Enterprise Guide 7.1, Cu EMC at inlet and outlet

Nonparametric One-Way ANOVA

The NPAR1WAY Procedure

Parameter=Cu

Wilcoxon Scores (Rank Sums) for Variable Load Classified by Variable Location					
Location	N	Sum of Scores	Expected Under H0	Std Dev Under H0	Mean Score
inlet	19	403.0	370.50	34.252737	21.210526
outlet	19	338.0	370.50	34.252737	17.789474

Wilcoxon Two-Sample Test	
Statistic	403.0000
Normal Approximation	
Z	0.9342
One-Sided Pr > Z	0.1751
Two-Sided Pr > |Z|	0.3502
t Approximation	
One-Sided Pr > Z	0.1781
Two-Sided Pr > |Z|	0.3562
Z includes a continuity correction of 0.5.	

Kruskal-Wallis Test	
Chi-Square	0.9003
DF	1
Pr > Chi-Square	0.3427

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Figure C.4 Results of statistical analysis using SAS Enterprise Guide 7.1, Cu load at inlet and outlet

Nonparametric One-Way ANOVA

The NPAR1WAY Procedure

Parameter=HPO4

Wilcoxon Scores (Rank Sums) for Variable Concentration Classified by Variable Location					
Location	N	Sum of Scores	Expected Under H0	Std Dev Under H0	Mean Score
inlet	19	357.0	370.50	23.162324	18.789474
outlet	19	384.0	370.50	23.162324	20.210526
Average scores were used for ties.					

Wilcoxon Two-Sample Test	
Statistic	357.0000
Normal Approximation	
Z	-0.5613
One-Sided Pr < Z	0.2873
Two-Sided Pr > |Z|	0.5746
t Approximation	
One-Sided Pr < Z	0.2890
Two-Sided Pr > |Z|	0.5780
Z includes a continuity correction of 0.5.	

Kruskal-Wallis Test	
Chi-Square	0.3397
DF	1
Pr > Chi-Square	0.5600

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Figure C.5 Results of statistical analysis using SAS Enterprise Guide 7.1, PO_4^{3-} EMC at inlet and outlet

Nonparametric One-Way ANOVA

The NPAR1WAY Procedure

Parameter=HPO4

Wilcoxon Scores (Rank Sums) for Variable Load Classified by Variable Location					
Location	N	Sum of Scores	Expected Under H0	Std Dev Under H0	Mean Score
inlet	19	385.0	370.50	34.252737	20.263158
outlet	19	356.0	370.50	34.252737	18.736842

Wilcoxon Two-Sample Test	
Statistic	385.0000
Normal Approximation	
Z	0.4087
One-Sided Pr > Z	0.3414
Two-Sided Pr > |Z|	0.6827
t Approximation	
One-Sided Pr > Z	0.3425
Two-Sided Pr > |Z|	0.6851
Z includes a continuity correction of 0.5.	

Kruskal-Wallis Test	
Chi-Square	0.1792
DF	1
Pr > Chi-Square	0.6721

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Figure C.6 Results of statistical analysis using SAS Enterprise Guide 7.1, PO_4^{3-} load at inlet and outlet

Nonparametric One-Way ANOVA

The NPAR1WAY Procedure

Parameter=NH4

Wilcoxon Scores (Rank Sums) for Variable Concentration Classified by Variable Location					
Location	N	Sum of Scores	Expected Under H0	Std Dev Under H0	Mean Score
inlet	19	367.50	370.50	30.755355	19.342105
outlet	19	373.50	370.50	30.755355	19.657895
Average scores were used for ties.					

Wilcoxon Two-Sample Test	
Statistic	367.5000
Normal Approximation	
Z	-0.0813
One-Sided Pr < Z	0.4676
Two-Sided Pr > |Z|	0.9352
t Approximation	
One-Sided Pr < Z	0.4678
Two-Sided Pr > |Z|	0.9357
Z includes a continuity correction of 0.5.	

Kruskal-Wallis Test	
Chi-Square	0.0095
DF	1
Pr > Chi-Square	0.9223

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Figure C.7 Results of statistical analysis using SAS Enterprise Guide 7.1, NH_4^+ EMC at inlet and outlet

Nonparametric One-Way ANOVA

The NPAR1WAY Procedure

Parameter=NH4

Wilcoxon Scores (Rank Sums) for Variable Load Classified by Variable Location					
Location	N	Sum of Scores	Expected Under H0	Std Dev Under H0	Mean Score
inlet	19	399.0	370.50	34.243366	21.0
outlet	19	342.0	370.50	34.243366	18.0
Average scores were used for ties.					

Wilcoxon Two-Sample Test	
Statistic	399.0000
Normal Approximation	
Z	0.8177
One-Sided Pr > Z	0.2068
Two-Sided Pr > |Z|	0.4135
t Approximation	
One-Sided Pr > Z	0.2094
Two-Sided Pr > |Z|	0.4188
Z includes a continuity correction of 0.5.	

Kruskal-Wallis Test	
Chi-Square	0.6927
DF	1
Pr > Chi-Square	0.4053

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Figure C.8 Results of statistical analysis using SAS Enterprise Guide 7.1, NH_4^+ load at inlet and outlet

Nonparametric One-Way ANOVA

The NPAR1WAY Procedure

Parameter=NO3

Wilcoxon Scores (Rank Sums) for Variable Concentration Classified by Variable Location					
Location	N	Sum of Scores	Expected Under H0	Std Dev Under H0	Mean Score
inlet	19	391.0	370.50	34.230242	20.578947
outlet	19	350.0	370.50	34.230242	18.421053
Average scores were used for ties.					

Wilcoxon Two-Sample Test	
Statistic	391.0000
Normal Approximation	
Z	0.5843
One-Sided Pr > Z	0.2795
Two-Sided Pr > |Z|	0.5590
t Approximation	
One-Sided Pr > Z	0.2813
Two-Sided Pr > |Z|	0.5626
Z includes a continuity correction of 0.5.	

Kruskal-Wallis Test	
Chi-Square	0.3587
DF	1
Pr > Chi-Square	0.5492

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Figure C.9 Results of statistical analysis using SAS Enterprise Guide 7.1, NO_3^- EMC at inlet and outlet

Nonparametric One-Way ANOVA

The NPAR1WAY Procedure

Parameter=NO3

Wilcoxon Scores (Rank Sums) for Variable Load Classified by Variable Location					
Location	N	Sum of Scores	Expected Under H0	Std Dev Under H0	Mean Score
inlet	19	406.0	370.50	34.248989	21.368421
outlet	19	335.0	370.50	34.248989	17.631579
Average scores were used for ties.					

Wilcoxon Two-Sample Test	
Statistic	406.0000
Normal Approximation	
Z	1.0219
One-Sided Pr > Z	0.1534
Two-Sided Pr > |Z|	0.3068
t Approximation	
One-Sided Pr > Z	0.1567
Two-Sided Pr > |Z|	0.3135
Z includes a continuity correction of 0.5.	

Kruskal-Wallis Test	
Chi-Square	1.0744
DF	1
Pr > Chi-Square	0.3000

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Figure C.10 Results of statistical analysis using SAS Enterprise Guide 7.1, NO_3^- load at inlet and outlet

Nonparametric One-Way ANOVA

The NPAR1WAY Procedure

Parameter=Pb

Wilcoxon Scores (Rank Sums) for Variable Concentration Classified by Variable Location					
Location	N	Sum of Scores	Expected Under H0	Std Dev Under H0	Mean Score
inlet	19	370.50	370.50	0.0	19.50
outlet	19	370.50	370.50	0.0	19.50
Average scores were used for ties.					

Wilcoxon Two-Sample Test	
Statistic	370.5000
Normal Approximation	
Z	0.0000
One-Sided Pr < Z	0.5000
Two-Sided Pr > |Z|	1.0000
t Approximation	
One-Sided Pr < Z	0.5000
Two-Sided Pr > |Z|	1.0000
Z includes a continuity correction of 0.5.	

Kruskal-Wallis Test	
Chi-Square	0.0000
DF	1
Pr > Chi-Square	1.0000

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Figure C.11 Results of statistical analysis using SAS Enterprise Guide 7.1, Pb EMC at inlet and outlet

Nonparametric One-Way ANOVA

The NPAR1WAY Procedure

Parameter=Pb

Wilcoxon Scores (Rank Sums) for Variable Load Classified by Variable Location					
Location	N	Sum of Scores	Expected Under H0	Std Dev Under H0	Mean Score
inlet	19	392.0	370.50	34.252737	20.631579
outlet	19	349.0	370.50	34.252737	18.368421

Wilcoxon Two-Sample Test	
Statistic	392.0000
Normal Approximation	
Z	0.6131
One-Sided Pr > Z	0.2699
Two-Sided Pr > |Z|	0.5398
t Approximation	
One-Sided Pr > Z	0.2718
Two-Sided Pr > |Z|	0.5436
Z includes a continuity correction of 0.5.	

Kruskal-Wallis Test	
Chi-Square	0.3940
DF	1
Pr > Chi-Square	0.5302

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Figure C.12 Results of statistical analysis using SAS Enterprise Guide 7.1, Pb load at inlet and outlet

Nonparametric One-Way ANOVA

The NPAR1WAY Procedure

Parameter=SO4

Wilcoxon Scores (Rank Sums) for Variable Concentration Classified by Variable Location					
Location	N	Sum of Scores	Expected Under H0	Std Dev Under H0	Mean Score
inlet	19	378.0	370.50	34.023343	19.894737
outlet	19	363.0	370.50	34.023343	19.105263
Average scores were used for ties.					

Wilcoxon Two-Sample Test	
Statistic	378.0000
Normal Approximation	
Z	0.2057
One-Sided Pr > Z	0.4185
Two-Sided Pr > |Z|	0.8370
t Approximation	
One-Sided Pr > Z	0.4191
Two-Sided Pr > |Z|	0.8381
Z includes a continuity correction of 0.5.	

Kruskal-Wallis Test	
Chi-Square	0.0486
DF	1
Pr > Chi-Square	0.8255

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Figure C.13 Results of statistical analysis using SAS Enterprise Guide 7.1, SO_4^{2-} EMC at inlet and outlet

Nonparametric One-Way ANOVA

The NPAR1WAY Procedure

Parameter=SO4

Wilcoxon Scores (Rank Sums) for Variable Load Classified by Variable Location					
Location	N	Sum of Scores	Expected Under H0	Std Dev Under H0	Mean Score
inlet	19	397.0	370.50	34.252737	20.894737
outlet	19	344.0	370.50	34.252737	18.105263

Wilcoxon Two-Sample Test	
Statistic	397.0000
Normal Approximation	
Z	0.7591
One-Sided Pr > Z	0.2239
Two-Sided Pr > |Z|	0.4478
t Approximation	
One-Sided Pr > Z	0.2263
Two-Sided Pr > |Z|	0.4526
Z includes a continuity correction of 0.5.	

Kruskal-Wallis Test	
Chi-Square	0.5986
DF	1
Pr > Chi-Square	0.4391

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Figure C.14 Results of statistical analysis using SAS Enterprise Guide 7.1, SO_4^{2-} load at inlet and outlet

Nonparametric One-Way ANOVA

The NPAR1WAY Procedure

Parameter=TSS

Wilcoxon Scores (Rank Sums) for Variable Concentration Classified by Variable Location					
Location	N	Sum of Scores	Expected Under H0	Std Dev Under H0	Mean Score
inlet	19	333.50	370.50	34.250863	17.552632
outlet	19	407.50	370.50	34.250863	21.447368
Average scores were used for ties.					

Wilcoxon Two-Sample Test	
Statistic	333.5000
Normal Approximation	
Z	-1.0657
One-Sided Pr < Z	0.1433
Two-Sided Pr > |Z|	0.2866
t Approximation	
One-Sided Pr < Z	0.1467
Two-Sided Pr > |Z|	0.2935
Z includes a continuity correction of 0.5.	

Kruskal-Wallis Test	
Chi-Square	1.1670
DF	1
Pr > Chi-Square	0.2800

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Figure C.15 Results of statistical analysis using SAS Enterprise Guide 7.1, TSS EMC at inlet and outlet

Nonparametric One-Way ANOVA

The NPAR1WAY Procedure

Parameter=TSS

Wilcoxon Scores (Rank Sums) for Variable Load Classified by Variable Location					
Location	N	Sum of Scores	Expected Under H0	Std Dev Under H0	Mean Score
inlet	19	371.0	370.50	34.252737	19.526316
outlet	19	370.0	370.50	34.252737	19.473684

Wilcoxon Two-Sample Test	
Statistic	371.0000
Normal Approximation	
Z	0.0000
One-Sided Pr < Z	0.5000
Two-Sided Pr > |Z|	1.0000
t Approximation	
One-Sided Pr < Z	0.5000
Two-Sided Pr > |Z|	1.0000
Z includes a continuity correction of 0.5.	

Kruskal-Wallis Test	
Chi-Square	0.0002
DF	1
Pr > Chi-Square	0.9884

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Figure C.16 Results of statistical analysis using SAS Enterprise Guide 7.1, TSS load at inlet and outlet

Nonparametric One-Way ANOVA

The NPAR1WAY Procedure

Parameter=Zn

Wilcoxon Scores (Rank Sums) for Variable Concentration Classified by Variable Location					
Location	**N**	**Sum of Scores**	**Expected Under H0**	**Std Dev Under H0**	**Mean Score**
inlet	19	343.50	370.50	31.662387	18.078947
outlet	19	397.50	370.50	31.662387	20.921053
Average scores were used for ties.					

Wilcoxon Two-Sample Test	
Statistic	343.5000
Normal Approximation	
Z	-0.8370
One-Sided Pr < Z	0.2013
Two-Sided Pr > |Z|	0.4026
t Approximation	
One-Sided Pr < Z	0.2040
Two-Sided Pr > |Z|	0.4080
Z includes a continuity correction of 0.5.	

Kruskal-Wallis Test	
Chi-Square	0.7272
DF	1
Pr > Chi-Square	0.3938

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Figure C.17 Results of statistical analysis using SAS Enterprise Guide 7.1, Zn EMC at inlet and outlet

Nonparametric One-Way ANOVA

The NPAR1WAY Procedure

Parameter=Zn

Wilcoxon Scores (Rank Sums) for Variable Load Classified by Variable Location					
Location	N	Sum of Scores	Expected Under H0	Std Dev Under H0	Mean Score
inlet	19	374.0	370.50	34.252737	19.684211
outlet	19	367.0	370.50	34.252737	19.315789

Wilcoxon Two-Sample Test	
Statistic	374.0000
Normal Approximation	
Z	0.0876
One-Sided Pr > Z	0.4651
Two-Sided Pr > |Z|	0.9302
t Approximation	
One-Sided Pr > Z	0.4653
Two-Sided Pr > |Z|	0.9307
Z includes a continuity correction of 0.5.	

Kruskal-Wallis Test	
Chi-Square	0.0104
DF	1
Pr > Chi-Square	0.9186

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Figure C.18 Results of statistical analysis using SAS Enterprise Guide 7.1, Zn load at inlet and outlet

Nonparametric One-Way ANOVA

The NPAR1WAYProcedure

Growing season=growing

Wilcoxon Scores (Rank Sums) for Variable Volume Reduction Classified by Variable Year					
Year	N	Sum of Scores	Expected Under H0	Std Dev Under H0	Mean Score
2016	11	134.0	154.0	20.264912	12.181818
2017	16	244.0	224.0	20.264912	15.250000

Wilcoxon Two-Sample Test	
Statistic	134.0000
Normal Approximation	
Z	-0.9623
One-Sided Pr < Z	0.1680
Two-Sided Pr > |Z|	0.3359
t Approximation	
One-Sided Pr < Z	0.1724
Two-Sided Pr > |Z|	0.3448
Z includes a continuity correction of 0.5.	

Kruskal-Wallis Test	
Chi-Square	0.9740
DF	1
Pr > Chi-Square	0.3237

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Figure C.19 Results of statistical analysis using SAS Enterprise Guide 7.1, Growing 2016 vs 2017

Nonparametric One-Way ANOVA

The NPAR1WAYProcedure

Wilcoxon Scores (Rank Sums) for Variable tss Classified by Variable growing?					
growing?	N	Sum of Scores	Expected Under H0	Std Dev Under H0	Mean Score
nongrowing	10	95.0	100.0	12.247449	9.500000
growing	9	95.0	90.0	12.247449	10.555556

Wilcoxon Two-Sample Test	
Statistic (S)	95.0000
Normal Approximation	
Z	0.3674
One-Sided Pr > Z	0.3567
Two-Sided Pr > |Z|	0.7133
t Approximation	
One-Sided Pr > Z	0.3588
Two-Sided Pr > |Z|	0.7176
Exact Test	
One-Sided Pr >= S	0.3598
Two-Sided Pr >= |S - Mean|	0.7197
Z includes a continuity correction of 0.5.	

Kruskal-Wallis Test	
Chi-Square	0.1667
DF	1
Pr > Chi-Square	0.6831

Figure C.20 Results of statistical analysis using SAS Enterprise Guide 7.1, TSS load for growing vs nongrowing seasons.

Nonparametric One-Way ANOVA

The NPAR1WAY Procedure

Wilcoxon Scores (Rank Sums) for Variable zn Classified by Variable growing?					
growing?	N	Sum of Scores	Expected Under H0	Std Dev Under H0	Mean Score
nongrowing	6	30.0	39.0	6.234071	5.0
growing	6	48.0	39.0	6.234071	8.0
Average scores were used for ties.					

Wilcoxon Two-Sample Test	
Statistic (S)	30.0000
Normal Approximation	
Z	-1.3635
One-Sided Pr < Z	0.0864
Two-Sided Pr > |Z|	0.1727
t Approximation	
One-Sided Pr < Z	0.1000
Two-Sided Pr > |Z|	0.2000
Exact Test	
One-Sided Pr <= S	0.0833
Two-Sided Pr >= |S - Mean|	0.1667
Z includes a continuity correction of 0.5.	

Kruskal-Wallis Test	
Chi-Square	2.0842
DF	1
Pr > Chi-Square	0.1488

Figure C.21 Results of statistical analysis using SAS Enterprise Guide 7.1, Zn load for growing vs nongrowing seasons.

Nonparametric One-Way ANOVA

The NPAR1WAY Procedure

Wilcoxon Scores (Rank Sums) for Variable cu Classified by Variable growing?					
growing?	N	Sum of Scores	Expected Under H0	Std Dev Under H0	Mean Score
nongrowing	10	77.0	95.0	11.254629	7.700
growing	8	94.0	76.0	11.254629	11.750

Wilcoxon Two-Sample Test	
Statistic (S)	94.0000
Normal Approximation	
Z	1.5549
One-Sided Pr > Z	0.0600
Two-Sided Pr > |Z|	0.1200
t Approximation	
One-Sided Pr > Z	0.0692
Two-Sided Pr > |Z|	0.1384
Exact Test	
One-Sided Pr >= S	0.0610
Two-Sided Pr >= |S - Mean|	0.1220
Z includes a continuity correction of 0.5.	

Kruskal-Wallis Test	
Chi-Square	2.5579
DF	1
Pr > Chi-Square	0.1097

Figure C.22 Results of statistical analysis using SAS Enterprise Guide 7.1, Cu load for growing vs nongrowing seasons.

Nonparametric One-Way ANOVA

The NPAR1WAY Procedure

Wilcoxon Scores (Rank Sums) for Variable so4 Classified by Variable growing?					
growing?	N	Sum of Scores	Expected Under H0	Std Dev Under H0	Mean Score
nongrowing	7	50.0	56.0	8.640988	7.142857
growing	8	70.0	64.0	8.640988	8.750000

Wilcoxon Two-Sample Test	
Statistic (S)	50.0000
Normal Approximation	
Z	-0.6365
One-Sided Pr < Z	0.2622
Two-Sided Pr > |Z|	0.5244
t Approximation	
One-Sided Pr < Z	0.2674
Two-Sided Pr > |Z|	0.5347
Exact Test	
One-Sided Pr <= S	0.2679
Two-Sided Pr >= |S - Mean|	0.5358
Z includes a continuity correction of 0.5.	

Kruskal-Wallis Test	
Chi-Square	0.4821
DF	1
Pr > Chi-Square	0.4875

Figure C.23 Results of statistical analysis using SAS Enterprise Guide 7.1, SO_4^{2-} load for growing vs nongrowing seasons.

Nonparametric One-Way ANOVA

The NPAR1WAY Procedure

Wilcoxon Scores (Rank Sums) for Variable no3 Classified by Variable growing?					
growing?	N	Sum of Scores	Expected Under H0	Std Dev Under H0	Mean Score
nongrowing	6	45.50	45.0	7.737450	7.583333
growing	8	59.50	60.0	7.737450	7.437500
Average scores were used for ties.					

Wilcoxon Two-Sample Test	
Statistic (S)	45.5000
Normal Approximation	
Z	0.0000
One-Sided Pr < Z	0.5000
Two-Sided Pr > |Z|	1.0000
t Approximation	
One-Sided Pr < Z	0.5000
Two-Sided Pr > |Z|	1.0000
Exact Test	
One-Sided Pr >= S	0.4885
Two-Sided Pr >= |S - Mean|	0.9770
Z includes a continuity correction of 0.5.	

Kruskal-Wallis Test	
Chi-Square	0.0042
DF	1
Pr > Chi-Square	0.9485

Figure C.24 Results of statistical analysis using SAS Enterprise Guide 7.1, NO_3^- load for growing vs nongrowing seasons.

Vita

Jessica Thompson is from Birmingham, Alabama and attended the University of Alabama at Birmingham, where she received and Bachelor of Science degree in Civil Engineering in 2010. After graduation, she spent some time traveling, both internationally and within the states. In 2013, she began working for the City of Chattanooga Office of Sustainability, which then transitioned into a position with the City's Water Quality Program. While higher education had always been an option, it was in this position that convinced her to pursue a doctorate in Environmental Engineering, focusing on water resources. After meeting Jon Hathaway at the University of Tennessee's Watershed Symposium, she decided he would be the best professor to pursue this endeavor and began graduate school in August 2014. Jessica graduated in December 2018 with a Ph.D. in Environmental Engineering.