Water Quality and Hydrologic Performance of Roadside Biofiltration Conveyance (BFC) in the Coastal Plain and Piedmont of North Carolina

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Preface

This final report has been written to satisfy the requirement of the NCDOT – NCSU BAE research partnership, task order #6. This report will focus on the hydrologic and water quality performance of Biofiltration Conveyance (BFC) in North Carolina. The authors wish to thank NCDOT for funding this project, for supervising design and construction, and for their support and aid throughout the research.

Executive Summary

Biofiltration Conveyance (BFC) is an open channel, sand-filtering system composed of a series of shallow aquatic pools, riffles and weirs, native vegetation, and underlying media beds. Surface runoff entering a BFC is conveyed as non-erosive surface flow or subsurface seepage through the media, and exits the system as surface flow, seepage out, exfiltration into the parent soil, or evapotranspiration (ET). BFCs are expected to perform similar to other sand-media-based low impact development (LID) stormwater control measures (SCMs), but the hydrological and water quality efficiencies of BFC have not been sufficiently validated in a variety of hydrogeological conditions to date. Two BFCs were installed in North Carolina – one in the Coastal Plain (Brunswick County) and one in the Piedmont (Alamance County) eco-regions. Hydrologic performance was monitored at both BFCs, and water quality performance was monitored at the Alamance BFC.

At the Brunswick County BFC surface flow was reduced substantially through the BFC, with 86% of inflow converted to a shallow interflow-like seepage through the media, herein referred to as “seepage.” High groundwater levels resulted in small overall exfiltration rates, but increased evaporation rates due to extended ponding. The conversion of surface runoff to seepage has significant implications for stormwater mitigation, releasing filtered water at slower rates than conventional conveyance channels, similar to undeveloped watersheds. The Brunswick BFC released a similar fraction of seepage to that of shallow interflow observed in undeveloped watersheds.

Hydrologically, the Alamance BFC reduced volume and peak flow by 84% and 80%, respectively, while mimicking both predevelopment hydrograph shape and hydrologic flow pathways. The BFC was also able to reduce TSS, TP, and TN in surface flow by 72%, 28%, and 30%, respectively, likely due to filtration. The potential exists for further nutrient reduction if vegetated, wetland-like conditions are present. BFC outflow matches the modeled predevelopment hydrograph shape and pathway components, including both pre-event and event water, as determined by deuterium isotope concentrations. Optimal storm mitigation performance is expected when BFCs include a minimum of three pool/riffle cells, established vegetation, and exfiltration trenches to promote exfiltration into parent soils through extended subsurface ponding.

Introduction

Urban areas are associated with a high degree of impervious surfaces, resulting in large volumes of stormwater runoff and associated pollutants entering nearby waterbodies. Stormwater runoff has detrimental effects on ecosystem function. Low impact development (LID) strives to maintain predevelopment (native) hydrologic conditions on developed lands. A new LID stormwater control measure (SCM), referred to as Biofiltration Conveyance (BFC), uses principles of stream restoration to treat and manage stormwater prior to it entering receiving waters.

Urbanization and Pollution

By some estimates, the population of the world's urban areas will be 4.9 billion by 2030, more than quadrupling since 1950 (UN, 2005). As urban populations increase, the amount of land required to sustain them also increases. Folke et al. (1997) found that cities in Baltic Europe require land area 500 to 1000 times larger than cities themselves for resource production and waste assimilation. This means that, despite the fact that urban areas only account for 2% of the land area (Grubler, 1994), additional land must be altered to provide for the needs (and wants) of these dense populations (Lambin et al., 2001). These land-use changes – urban, suburban, and supporting - disturb the native vegetation and soil structure, resulting in increased stormwater runoff and pollutant loads.

Stormwater Pollution from Urbanization

Klein (1979) identified major factors of urbanization which affect stream quality, to include: reduced base flow, alteration in the natural stream temperature regimen, alteration of the character and energy of inputs, increased presence of toxic substances, and elevated nutrient inputs, all of which can result from stormwater runoff from urbanized landscapes. Urbanization has: 1) destroyed ecosystems that naturally mitigate stormwater and associated pollutants (Bernhardt and Palmer, 2007); 2) increased the degree of imperviousness resulting in an increased amount of stormwater runoff volume, flow, and duration (Bernhardt and Palmer, 2007; Arnold and Gibbons, 1996; Walsh et al., 2005; NRC, 2009); and 3) contributed additional pollutants, such as nitrogen, phosphorus, fecal bacteria, or heavy metals, to the environment to ultimately be transferred by runoff into receiving waters (Bennett et al., 1999; Waschbusch et al., 1999; Passeport et al., 2009; Hathaway and Hunt, 2011). The culmination of these things has detrimental effects on surface waters and ecosystems across the globe.

Loss of Natural Stormwater Mitigation

Vegetation and biological activity in undeveloped watersheds support a highly porous soil structure allowing for infiltration. The organic matter in the soil helps reduce erosion of small particles and holds moisture in the soil. Consequently, in an undeveloped watershed, stormwater is either infiltrated or evapotranspired for all but the most intense storm events (NRC, 2009). In contrast, developed or supporting lands are often stripped of their vegetation and topsoil and compacted for forestry or agricultural purposes. Even "green space" in developed areas have reduced infiltration rates. Pitt et al. (2002) found that nearly one third of urban soils tested in Milwaukee, Wisconsin had infiltration rates of nearly zero. These combined effects result in a developed landscape with little to no natural ability to mitigate stormwater.

Imperviousness

Impervious surface cover is often classified as pavement or rooftops, where water is unable to infiltrate. However, as described by Pitt et al. (2002), compacted green spaces currently regarded by stormwater models as pervious, can be virtually impervious as well. The reduction or elimination of infiltration results in larger volumes of runoff entering receiving waters. In the case where the receiving water is a stream, the larger volumes increase the stream velocity causing severe bank erosion and incision, which eventually lowers the water table around the stream (Groffman et al., 2003). Additionally, flow over impervious surfaces is more efficient such that runoff travels to receiving waters faster (NRC, 2009). The increase in runoff volumes and peak flows, combined with the decrease in time of concentration, causes rapid fluctuations in streams levels and higher chances of flooding.

Augmented stream flow velocities also result in increased sediment transport. The transport capacity within a stream increases non-linearly with flow velocity such that a small increase in stream velocity will result in a larger increase in transport capacity (Vogel et al., 2003). When the amount of sediment entering the stream is smaller than the amount of sediment being transported through stream (often the case with stormwater from concrete or rooftops), then incision, or down cutting, occurs (Groffman et al., 2003; Booth, 1990; MacRae, 1996). Incision is especially characteristic in older, stable urban and suburban developments with few sources of sediment to replace that scoured from the stream (Groffman et al., 2003). Incision lowers the elevation of the streambed, and in turn lowers the base flow and ground water level along the stream. This effect, termed groundwater drought, harms vegetation in the riparian zone, further reducing this zone’s ability to prevent movement of pollutants, such as nitrate, from uplands into the streams (Groffman et al., 2003). Increased velocities and incision also limit opportunities for in-stream nutrient removal (Bukaveckas, 2007; Galloway et al., 2004). Altogether, urbanization has a profound effect on stream geomorphology and hydrology, which ultimately affect the stream function and ability to provide ecosystem services.

Pollutants

Urban land-use changes are often associated with increases in pollutant loadings. Nitrogen and phosphorus loadings are high in runoff from agricultural lands. However, these nutrients are also found in runoff from urban and suburban land uses in concentrations nearly a magnitude greater than in runoff from forested landscapes (NRC, 2009). Urban sources of nitrogen, namely fossil fuel combustion, play a major role in increased nitrogen concentrations in coastal areas through atmospheric nitrogen deposition (NRC, 2000). Anthropogenic nitrogen fixation has increased 2- to 3-fold over the past 40 years, and continues to grow (Galloway et al., 1995). Stormwater runoff from roadways and commercial land uses contribute annual lead, copper, and zinc loadings of approximately 3, 0.4, and 2 lbs/ac, respectively, into receiving waters (Burton and Pitt, 2001). Additionally several studies have measured indicator organism concentrations, as well as pathogen bacteria, in stormwater runoff from urban and suburban land uses well above the EPA recommended limits (Hathaway and Hunt, 2011; Cizek et al., 2008; Krometis et al., 2011). The additional pollutants entering receiving waters, combined with the degradation of natural system's ability to mitigate pollutants, have detrimental consequences for public and ecosystem health.

Stream Health and Impervious Cover

Klein (1979) studied 27 small catchments in the Piedmont province of Maryland and identified impervious surface cover as a reliable indicator of stream degradation. Since then, many studies have agreed with Klein's findings, and also suggest that stream quality consistently drops from good to fair when the watershed impervious cover reaches 10-15% (Arnold and Gibbons, 1996; MacRae, 1996; Walsh, 2000; Wang et al., 2000; Schleuler, 1994; Paul and Meyer, 2001). Arnold and Gibbons (1996) found associations between streams labeled as "degraded" and impervious cover greater than 30%. Several other studies argue that, although imperviousness is important, how impervious surfaces are connected to the receiving waters determines the severity of the degradation (Hatt et al., 2004; Taylor et al., 2004; Walsh et al., 2004; Newall and Walsh, 2005). Directly connected impervious area (DCIA), or impervious surfaces that directly drain to receiving waters (i.e. via pipe, curb, etc.), transfer more runoff and associated pollutants than impervious surfaces that drain to pervious surfaces prior to entering a receiving water. Although disconnecting impervious areas may reduce the impact of imperviousness on stream health, Miltner et al. (2004) ultimately showed that few sites with greater than 27% total impervious area can meet interim Clean Water Act goals.

Effects of Urbanization and SCMs on Stream Health

Several studies of the past 30 years have shown decreases in macroinvertebrate diversity with increasing imperviousness (Klein, 1979; Walsh et al., 2005; Hogg and Norris, 1991; Walsh, 2000; Roy et al., 2003). In a study performed examining benthic macroinvertebrate indices in small streams of three regions (Maryland, Texas, and Washington), Horner et al. (2003) showed that high urbanization and loss of natural cover *always* led to biological degradation. Increased runoff volumes and velocities associated with urbanization alter the sediment distribution in receiving streams, causing decline in macroinvertebrate populations (Roy et al., 2003). Observations of fish species decline have also been associated with the urbanization of watersheds (Klein, 1979; Horner et al., 2003; Wang et al., 2000).

SCMs are designed to mitigate the increased runoff associated with urbanization in order to protect receiving waters. Stormwater management strategies have shifted from flood control by diverting runoff via pipe networks to designing systems which attempt to treat pollutants and reduce runoff volume and peak flow by providing opportunity for infiltration and evapotranspiration (NRC, 2009). Horner et al. (2003) examined how structural SCMs could moderate the effects of urbanization on stream health using a macroinvertebrate index as an indicator. The study concluded that highly urbanized watersheds showed considerable improvements with SCMs, however, there was no instances of the best category of macroinvertebrate indices (>75%). It should be noted that this study did not include low impact development or nature-based SCMs, which aim to match pre-development hydrologic conditions, and may therefore, better preserve receiving stream integrity. Ultimately, conventional SCMs may be able to shift the impervious cover thresholds mentioned previously, but current development practices will continue to degrade streams.

Urbanization and Public Welfare

Increases in stormwater runoff pollution are a concern to the public. Public safety and welfare concerns associated with severe incision include steep, unstable stream banks and flashy stream levels leading to frequent and unpredictable flooding. Public health is also a concern, as beaches are often closed due to high fecal bacteria concentrations. Water related disease outbreaks in the United States are often associated with extreme precipitation (Curriero et al. 2001; Gaffield et al. 2003). Furthermore, ecosystem degradation can be costly to municipalities and industry. Beach closings and restricted fishing are often consequences of large rainfall events in coastal areas (Ajuzie and Altobello, 1997). New York City's drinking water supply, located in the pristine Catskill Mountains, is under threat as upstream development increases, despite a 28,000 hectare (ha) conservation easement (DePalma, 2006).

Stormwater Control Measures for Urban and Suburban Areas

Until recently, the goal of stormwater management was flood control for public safety. However, a series of regulations, beginning with the Clean Water Act in 1972, provided the foundation for conventional stormwater management, with an emphasis on water quality and peak flow mitigation. Most recently, emerging LID strategies are emphasizing SCMs that reduce runoff volume by aiming to achieve predevelopment hydrology on developed sites.

Evolution of Regulations

The recognition of the stormwater runoff contribution to waterbody impairment is recent - within the past 30 years. The Clean Water Act (CWA) of 1972 set the goal of restoring and maintaining chemical, biological, and physical integrity of the nation's water bodies through the National Pollutant Discharge Elimination System (NPDES). This required inventories of pollutant sources for currently impaired water bodies by establishing a pollutant total maximum daily load (TMDL). Implementation of the TMDL program began at the state level during the 2000's. As TMDLs evolved, it became evident that waterbody impairment is often dominated by non-point pollution sources, as opposed to the point discharge pollution sources monitored under the NPDES permitting (NRCS, 2009). As a result, the recent focus for stormwater management has been controlling non-point stormwater pollution through structural and non-structural SCMs.

Stormwater Management Goals

The CWA of 1972 shifted the focus of stormwater management from flood control to pollutant removal. Permits were issued with limits on water quality. Although runoff volume is often considered under these permits, it is only used as a surrogate for water quality (NRC, 2009). Consequently, SCMs are chosen based on their pollutant concentration reduction (e.g. wet ponds achieve 85% TSS removal credit according to the 2009 North Carolina Department of Water Quality (NC DWQ) Stormwater Best Management Practices Manual).

Throughout the past decade, LID has gained more traction. LID allows for the development of a site while maintaining pre-development hydrology. In 2007, the Energy Independence and Security Act (EISA) required all federal development and redevelopment projects of 465 m^2 or larger to achieve pre-development hydrology to the "maximum extent technically feasible" (Energy Independence and Security Act, 2007). As the stormwater engineering field moves towards hydrology driven regulations (Low Impact Development Center, 2007), it becomes imperative to understand what constitutes pre-development hydrology, and how to best achieve it.

Stormwater Control Measures

Managing stormwater on a site is generally achieved by piece-working several different types of SCMs together. For the sake of this report, SCMs can be categorized as those that focus on 1) peak flow attenuation, 2) stormwater conveyance, and 3) runoff volume reduction. As the field is progressing towards designing systems to achieve pre-development hydrology, SCMs which focus on reducing runoff volume will be given the most attention as they have the greatest potential to meet this goal.

Peak Flow Attenuation

An example of a peak flow attenuating SCM is the detention/retention basin (wet or dry). These basins are designed to capture the runoff from a site and slowly release it through an outlet structure over the course of two to five days. By controlling the release of runoff, detention basins are able to reduce downstream erosion that would result from high velocity flows characteristic of impervious areas. Although these structures effectively regulate large flows and achieve some degree of treatment (Geosyntec Consultants and Wright Water Engineers 2008), they offer little improvement to, and may actually worsen, watershed scale flooding (NRC, 2009), as there is virtually no overall volume reduction (Geosyntec Consultants and Wright Water Engineers 2011). Despite demonstrating water quality mitigation of sediment-bound pollutants and a suite of ecosystem benefits (Moore and Hunt, 2012), wet ponds act similarly to detention/retention basins in that they often do not effectively reduce overall runoff volume (Geosyntec Consultants and Wright Water Engineers 2011). Wetlands do offer some opportunity of meeting pre-development hydrology as they move water through several pathways within the system: the vegetated surface, the organic soil layer at the bottom, and potentially slow infiltration into the clay layer. Furthermore, stormwater wetlands are generally implemented for watersheds greater than 1.6 ha

(Wossink and Hunt, 2003), and therefore, often do not receive highly urban runoff flows directly.

Conveyance

Conveyance structures in urban areas have traditionally consisted of pipes, which efficiently moving water from a collection point to a waterbody. In this case, stormwater runoff undergoes no treatment or volume reduction. In many urban areas, new construction is required to treat stormwater runoff onsite prior to discharge (often via pipe) into streams. Outside of dense urban areas, swales are a less expensive and more beneficial option for conveying stormwater. Swales are vegetated channels (often grassed) that direct runoff towards an endpoint. Swales are able to provide some treatment of less intense storms by maintaining a water level during flows below the grass height (hence, achieving a level of vegetated filtration). Sedimentation, exfiltration to parent soils, and soil surface chemical treatment are also possible pollutant removal mechanisms. Deletic and Fletcher (2006) examined an artificially dosed grassed swale in Brisbane, Australia. Their results showed good removal of TSS for a variety of flow rates: 85% and 65% removal for inflow rates of 2 and 15 L/s, respectively. The study also showed an exponential relationship between nutrient removal and swale length, achieving an overall removal of 46% and 56% for TP and TN, respectively.

As the treatment of runoff becomes more important, especially in nutrient sensitive waters, researchers and engineers are examining ways to enhance the performance of swales. Yu et al. (2001) examined swale performance in Taiwan and Virginia. With check dams in place, removal efficiencies increased from 48% to 70%, 14% to 21%, and 29% to 77% for TSS, TN, and TP, respectively. The check dams increased the hydraulic residence time and decreased velocity. Winston et al. (2012) compared the performance of grassed swales and wetland swales (swales located in soil with a high water table and were thus permanently submerged) along Interstate-40 in North Carolina. Although concentrations of TSS and TP were similar between wet and dry swales, the wetland swales showed significantly lower concentrations of TN being exported. Thus, in areas with high water tables, wetland swales may be used to further reduce nitrogen loadings from stormwater runoff. The swales examined in the NC study did not show any significant exfiltration, likely due to the high degree of compaction during construction.

Volume Reduction

Many studies have shown that increased runoff from urban areas causes augmented pollutant loads, erosive discharge and stream velocities, and a drop in groundwater level and stream base flow level (Geosyntec Consultants and Wright Water Engineers, 2011; USEPA, 2009). Recently, efforts have been made to understand the hydrology within SCMs, in particular, how well SCMs are able to reduce runoff volume (Geosyntec Consultants and Wright Water Engineers, 2011; Brown et al., 2010; Li et al., 2009). As the increased impervious surfaces associated with urbanization reduce evapotranspiration and infiltration into parent soil, SCMs that encourage exfiltration and evapotranspiration are able to reduce the overall runoff volume leaving the watershed.

Bioretention is an excellent example of an SCM that promotes both exfiltration and evapotranspiration. A bioretention cell is composed of vegetation and mulch over a bed of porous, sandy media. Runoff passes through the media bed and then exits through underdrains. Davis et al. (2012) studied the hydrology of three bioretention cells in Maryland, North Carolina, and Pennsylvania. All three cells reduced outflow as compared to inflow into the cell, such that only 23%, 14%, and 48% of the inflow volume was discharged at each of the sites, respectively. These results suggest that there are additional losses in the water budget, most likely exfiltration and evapotranspiration. In examining six existing bioretention cells with different design parameters, including varying media depths, ponding depths, and surface to drainage ratios, throughout Maryland and North Carolina, Li et al. (2009) saw consistent trends in peak reduction, peak delay, and outflow duration. Peak flow reduction occurred at all six sites, with a minimum reduction of 94%. Over 70% of the events tested met the peak flow target criteria described by Davis (2008) (i.e. ratio of time of peak outflow and peak inflow is greater than six). The outflow target (less than 1/3 of the inflow runoff volume discharged from the site 24 hours after peak) was met 40% of the time. A shallow bioretention cell with a liner preventing exfiltration achieved this goal only 15% of the time.

Hunt et al. (2006) studied several bioretention cells in the Piedmont, North Carolina. The cells demonstrated seasonal variation in their ability to reduce runoff volume, with a mean 93% volume reduction in summer and a mean 46% volume reduction in winter. As higher evapotranspiration rates occur in summer, this variation suggests that evapotranspiration plays an important role in the fate of water from a bioretention cell. Nitrogen loads exiting the cells were reduced up to 75%, primarily due to reduced runoff volume. Hunt et al. (2008) showed reductions in nutrient and TSS concentrations between 31% and 73% in a bioretention cell in Charlotte, NC. Thus, in addition to reducing overall runoff volume, bioretention cells also provide pollutant concentration mitigation, as well as attenuating peak flows for small and medium size storm events.

Biofiltration Conveyance

Biofiltration Conveyance (BFC), also described as regenerative stormwater conveyance (RSC) by their inventors (Keith Underwood and Biohabitats in Maryland) and step pool stormwater conveyance (SPSC) by MD Dept. of the Environment, are open channel conveyances consisting of a porous sand media bed, riffle/weir step pools, and vegetation. Unlike most other SCMs, these systems are considered regenerative, in that they provide a positive feedback supporting surrounding environments, such that as the system establishes the surrounding environment is improved, hence further improving the system. Although limited data exist for BFC performance (none of which is peer-reviewed), preliminary data show peak flow attenuation, water quality treatment, conveyance, and runoff volume reduction.

Regenerative Systems

Conventional approaches to engineering have detrimentally impacted the environment, as described previously. These impacts are recognized, and in an effort to stop and reverse them, engineering design can take on several design philosophies from solely reducing impacts to actively promoting living systems. Regenerative design requires examining and engaging the system as a whole, such that all aspects of the system are integral to the life process and evolution of the system. When this occurs, the design process functions as a catalyst for evolutionary change (Integrative Design Collaborative et al., 2006), such that all parts of the systems are able to grow and adapt, supported by the remaining parts of the system (positive feedback). In order to design regeneratively, an interdisciplinary effort must be made to articulate the project objectives and understand the broader context of the ecosystem. For example, Walter and Merritts (2008) show that natural stream beds throughout the eastern United States prior to colonial settlement actually consisted of branching channels within extensive carbon-rich vegetated wetlands, as opposed to the gravel bed meandering streams bordered by a self-formed fine-grained floodplain that typically drive stream restoration. In understanding this, ecological engineers are able to alter the river restoration approach to include a base flow channel incorporated with its floodplain, aided by a series of pools and weirs (Berg, 2009). Additionally, regenerative systems depend entirely on the quality of their ecosystem function, such regenerative systems will demonstrate ecosystem services beyond that of stormwater mitigation. Engineers are able to use this knowledge in order to more holistically and regeneratively design stormwater conveyance as zero-order ephemeral stream ecosystems.

BFC Components

BFCs, being ecosystem-based, are specifically engineered to achieve predevelopment hydrology. Water is expected to exit the system as seepage for smaller storms, and as non-erosive surface flows for extreme floods (up to the 100-yr storm event) (Brown et al. 2010; Flores et al. 2009). Specific BFC components are required to achieve these goals – sand bed material, riffles and pools, and an established plant community (Figure 1). Each component offers opportunities for hydrologic and water quality improvement.

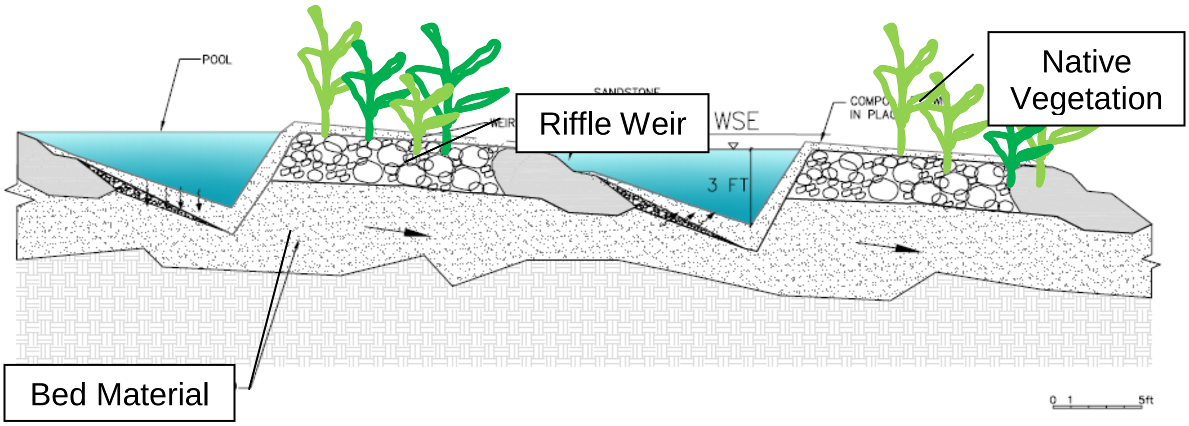


Figure 1 Profile view of a BFC system, including bed material (sand), riffles and pools, and native vegetation (Image courtesy of Biohabitats, used with permission).

Bed Material

The bed material, located below the riffles and pools (Figure 1), is designed to have a high hydraulic conductivity such that runoff will easily infiltrate from the pools into the media bed. In this way, the runoff will

1) Undergo media filtration removing sediment and associated pollutants,
2) Be stored in the media pores, reducing and possibly eliminating the need for downstream detention (Berg and Underwood, 2009),
3) Seep through bed material and into the parent soil, and
4) Recharge the groundwater.

Unlike bioretention media, the carbon-rich bed material is an 80:20 to 70:30 blend of sand and shredded hardwood mulch. Therefore, the media bed is both highly porous and able to support fungal and microbial communities necessary for enhanced nutrient reduction (Berg and Underwood 2009).

Riffles and Pools

As with stream geomorphology, the surface of the system is composed of a series of riffles and pools (Figure 1) to dissipate surface runoff energy, and reduce depth and velocity. Water leaving the pools passes over the riffle section followed by a parabolic weir. By slowing down the water, BFCs achieve:

1) Non-erosive flows,
2) Sedimentation of solids and associated pollutants within the pools, and
3) Opportunities for thick vegetation growth, offering further water quality treatment, habitat, and aesthetics.

As runoff volumes increase, the weirs direct the flow horizontally, as opposed to increasing the stream depth, which allow treatment mechanisms to occur as they would in a stream floodplain (Kaushal et al. 2008). Additionally, the pools temporarily store water and, therefore, reduce or eliminate downstream detention needs.

Established Plant Community

An established plant community is important to the regenerative characteristics of BFCs. Plants offer a variety of opportunities for runoff volume and pollutant removal by

1) Providing a site for microbial activity and, therefore, nutrient reduction,
2) Taking up nutrients for plant growth,
3) Long term carbon sequestration through a healthy root mass, and
4) Evapotranspiring stormwater

Additionally, thriving plant communities improve the aesthetics of a site by adding shade, privacy, and color. A plant community also provides the opportunity to restore a range of site ecologies.

Costs

Anne Arundel County in Maryland (MD) estimates that the majority of traditional stormwater conveyance devices throughout their county, including pipe outfalls and rip-rap/gabion level spreaders, have failed, costing the county more than $600 million in damage (Brown et al., 2010). As an alternative to such devices, Anne Arundel County is interested in the potential of BFCs as SCMs. At a medium density residential site, Preserve at Severn, the installation of a BFC saved the developer approximately $400K, more than half of the cost of using conventional storm drain pipe or other related drainage infrastructure (Brown et al., 2010). In addition to the capital savings, BFCs are easy to maintain, requiring invasive plant management and excess debris removal during the first five years (Flores et al., 2009). Overall, BFC systems are potentially a less expensive and highly beneficial alternative to traditional stormwater control management practices.

BFCs as SCMs

Although BFCs are now a credited SCM in MD (MD DOE, 2014), very little data is available on their stormwater mitigation performance. Hydrologic TR-20 modeling of a BFC site receiving runoff from 7.2-ha, low-density, residential development in Anne Arundel County showed a four-fold reduction in peak flow (Brown et al., 2010). Field scale stage monitoring of another BFC receiving water from a 5.7-ha, medium-density, residential neighborhood in the same area saw up to 50% reduction in peak flow for rainfall events less than 3.8 cm (Filoso, 2013).

Documented water quality performance in BFCs is limited to non-peer-reviewed reports. Filoso (2013) compared BFC performance to a control headwater stream in a neighboring watershed. TSS concentrations exiting the BFC were 60% less than the control stream, with median outflow concentrations of 20 mg/L. Total phosphorus removal mirrored TSS removal. However, there was no overall difference in Nitrate-N (NO3), Ammonium (TAN), total dissolved nitrogen (TDN), or total nitrogen (TN) outflow concentrations between the control stream and the BFC. Browning (2008) monitored a flat, wet BFC in MD. She reported 0% to 50% TSS removal in the BFC, but with similar median effluent concentrations to those in Filoso et al. (2013). In contrast to Filoso et al. (2013), this flat, wet BFC consistently reduced NO3 and TAN by 20% to 40%, respectively, suggesting that a wetter system may provide more opportunities for nitrogen biotransformation (i.e. nitrification and denitrification).

In addition to stormwater mitigation benefits, BFC systems have been reported as regenerative, in that they offer dynamic and diverse ecosystems for a range of plants, animals, amphibians, and insects, while also providing educational and aesthetic opportunities (Brown et al. 2010). BFC systems appear to be thriving, healthy ecosystems able to offer benefits towards the wellbeing of humans, while also achieving stormwater management goals at a lower cost.

Need for Further Research

BFCs appear to be a viable approach to stormwater management, both in terms of pollutant reduction and matching predevelopment hydrology. Further research is needed to determine the extent that these stormwater management objectives are met. Additionally, BFC systems are advertised as nature-based systems, in that they appear to function, sustain, and regenerate healthy ecosystem function. Although, healthy ecosystem function has been observed in specifically designed habitat restoration applications (Underwood et al., 2006), these qualities have not been observed in the systems designed as SCMs.

Research Objectives

Despite the minimal research on field performance, BFCs (referred to as step pool stormwater conveyance [SPSC] by MD DOE) are a choice tool in MD and Washington, D.C., for stormwater mitigation at tidal inlets, pipe outfalls, and eroded headwater streams (ND DOE, 2014; Ralph Spangolo, US EPA, personal communication). The North Carolina Department of Transportation (NC DOT) has begun to use BFCs as retrofit SCMs along roadsides. One BFC was installed at the Alamance County, NC, I-85 Southbound Rest Area and extensively monitored for hydrology and water quality. The second BFC was located in Brunswick County (at the coast), The Brunswick County BFC was located along US Hwy 17 at the bridge over the Lockwood Folly River. Using the data collected from these two sites, this research aims to:

i. Quantify the flow pathways of runoff entering a BFC,
ii. Determine the ability of BFCs to mitigate stormwater runoff vis-a-vis predevelopment hydrology,
iii. Quantify the ability of BFCs to remove nutrients and TSS from stormwater runoff, and

iv. Identify pollutant removal mechanisms and design characteristics associated with nutrient and TSS reductions.

Materials and Methods

BFC sites were chosen by NC DOT and monitored by NCSU. Hydrologic monitoring at the Brunswick BFC occurred from October 2012 to March 2014. Hydrologic and water quality monitoring at the Alamance BFC occurred from July 2013 through June 2014. Site descriptions and monitoring strategies are described below.

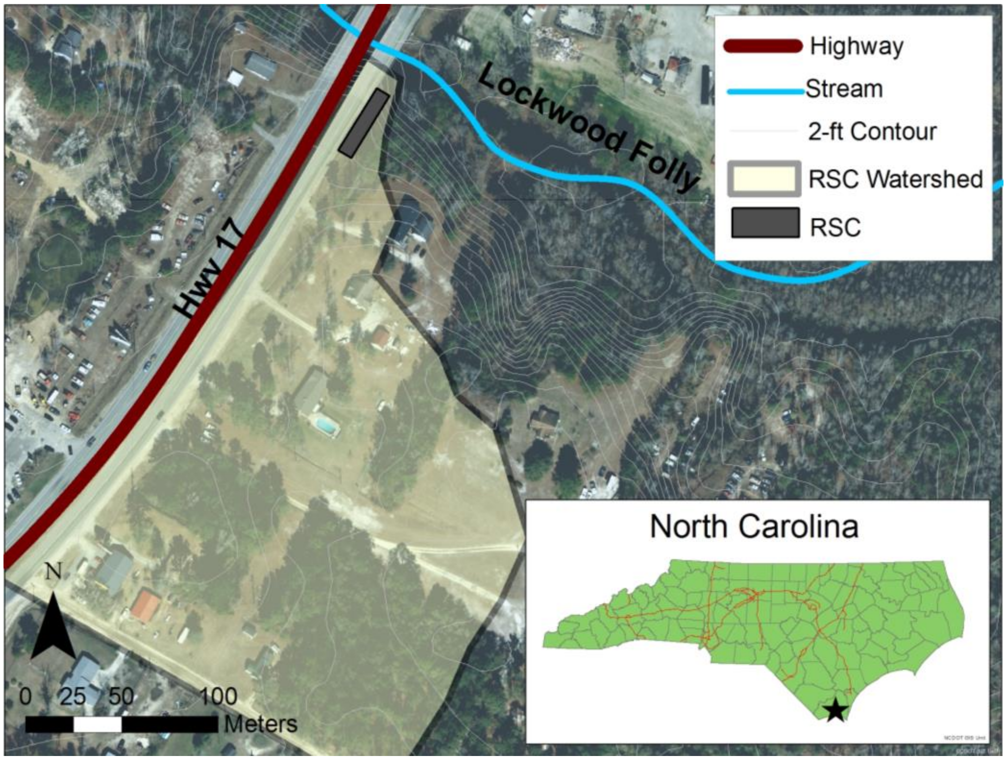
Site Descriptions

Brunswick County BFC

The Brunswick County BFC was located along US Hwy 17 at the Royal Oak Bridge over the Lockwood Folly River near Supply, NC (34.02^{o} N, 78.26^{o} W). The watershed was 5.2 ha including 0.64 ha of impervious area, with Hwy 17 accounting for 0.23 ha of imperviousness (Table 1, Figure 2). The remaining land cover was composed of pine forest and brush. The site was dominated by Baymeade/Marvyn and Muckalee loam soils, both predominantly sandy Hydrologic Soil Group A soils (NRCS, 2007). Portions of the original swale in which the BFC was installed were heavily eroded, with a 2 m head cut at its downstream end. The 40-m long and 4.3-m wide BFC, constructed during summer 2012, began at the end of a driveway culvert and is comprised of three pool/riffles with an average system slope of 4% (Figure 3). The sand media bed was 0.6 m deep separated by geotextile fabric from a 0.46 m thick layer of Class A rip rap to stabilize the channel (Table 1). The geotextile layer in the second pool exhibited some evidence of clogging immediately after construction, which may have contributed to slower infiltration rates through this layer (discussed in Results). However, saturation of the sand-media below the fabric indicated that water would still eventually penetrate this layer. Precast concrete weirs (Figure 2) were used in lieu of boulder weirs, as large boulders were not locally available. The entire SCM was covered in 50 mm of composted hardwood mulch and seeded with a stabilization mix. The BFC did not have any substantial vegetation during the monitoring period.

Table 1 Characteristics of the Brunswick County BFC

	Cell 1	**Cell 2**	**Cell 3**
Contributing Watershed Area	5.1 ha		
Sand Media Depth	0.61 m		
Rip Rap Depth	0.46 m		
Ponding Depth	0.91 m		
Length	11.9 m	18.3 m	9.1 m
Width	4.3 m	4.3 m	4.3 m
Surface Area	50 m^2	78 m^2	39 m^2
Contributing Run On Area	395 m^2	368 m^2	184 m^2
Average Slope	2.5 %	3.7 %	6.5 %
Sand Media Available Storage	9.1 m^3	13 m^3	6.8 m^3
Rip Rap Available Storage	4.2 m^3	6.1 m^3	1.7 m^3
Pond Available Storage	20 m^3	56 m^3	55 m^3



(a)

(b)

Figure 2 (a) Brunswick County BFC and watershed map, and (b) Brunswick BFC one month after completion of construction.

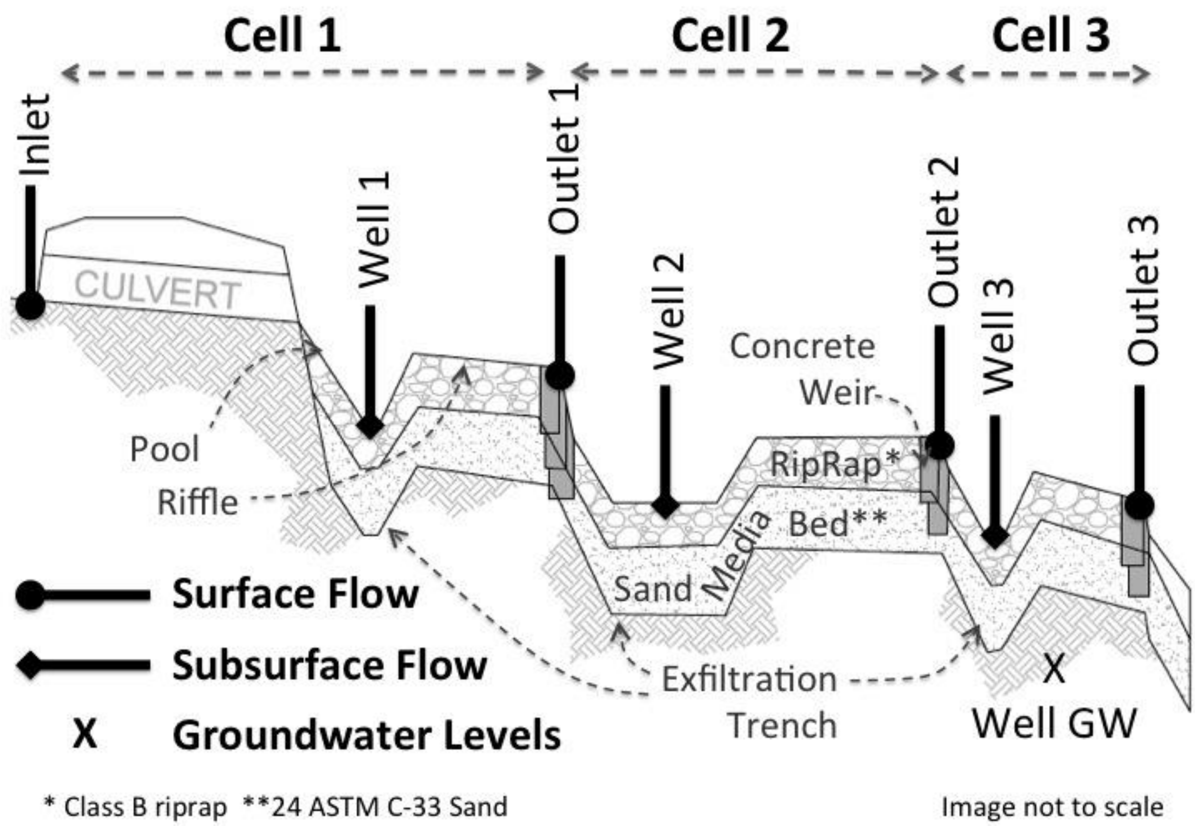


Figure 3 Brunswick County BFC cross-section with major design components and monitoring locations.

Alamance County

The Alamance BFC was installed along an entrance ramp from the rest area (36.06° N, 79.54° W) to I-85 during summer of 2013. The BFC's 1.6-ha watershed was 63% impervious, primarily consisting of parking areas and building rooftop (Figure 4, Table 2). The parking spaces (0.76 ha) were directly connected to the inlet of the BFC. The underlying soil was composed of Wilkes (HSG D, K_{SAT} 0 to 0.25 mm hr^{-1}) soil series with 15 cm sandy loam covering 15 to 20 cm of tight clay, overlying weathered diorite, gabbro, diabase, and gneiss bedrock. Runoff entered the 33.5-m BFC via a sewer grate and 61-cm concrete pipe. The BFC was comprised of three pool/riffles with an average system slope of 2.5%, followed by a 2.9-m cascade drop into a series of three "wetland pools" of equal elevation (Figure 5). The sand media bed was 0.6 m deep with 0.46 m of Class 1 (diameter 15 to 30 cm) rip-rap to stabilize the channel (Table 2). An exfiltration trench was located beneath the "wetland pools" such that water stored within the sand media will leave the system via exfiltration (Figure 4). The entire BFC was covered in 10 to 15 cm of composted hardwood mulch and seeded (Figure 3b).

Table 2 Alamance County BFC site description

	Cell 1	**Cell 2**	**Cell 3**	**Cell 4**
Contributing Watershed Area	1.6 ha			
% Imperviousness / % DCIA[1]	63% / 48%			
HSG / Infiltration Rate	D / 0.25 mm hr^{-1}			
Sand Media Depth	0.61 m			
Rip Rap Depth	0.46 m			
Ponding Depth	0.46 m			
Length	5.8 m			
Width	4.9 m	6.1 m	6.7 m	14.6 m
Surface Area	28.2 m^2	4.9 m	4.9 m	4.9 m
Contributing Run On Area	335 m^2	29.7 m^2	32.7 m^2	71.3 m^2
Average Slope	1.3 %	66 m^2	91.8 m^2	254 m^2
Sand Media Storage	5.6 m^3	3.75 %	2.3 %	8.33%
Rip Rap Storage	3.0 m^3	8.1 m^3	6.5 m^3	15.2 m^3
Pond Storage	3.42 m^3	2.2 m^3	3.7 m^3	9.0 m^3

[1] DCIA = directly connected imperviousness area

(a)

(b)

Figure 4 Alamance County BFC (a) watershed and (b) photo with monitoring weir

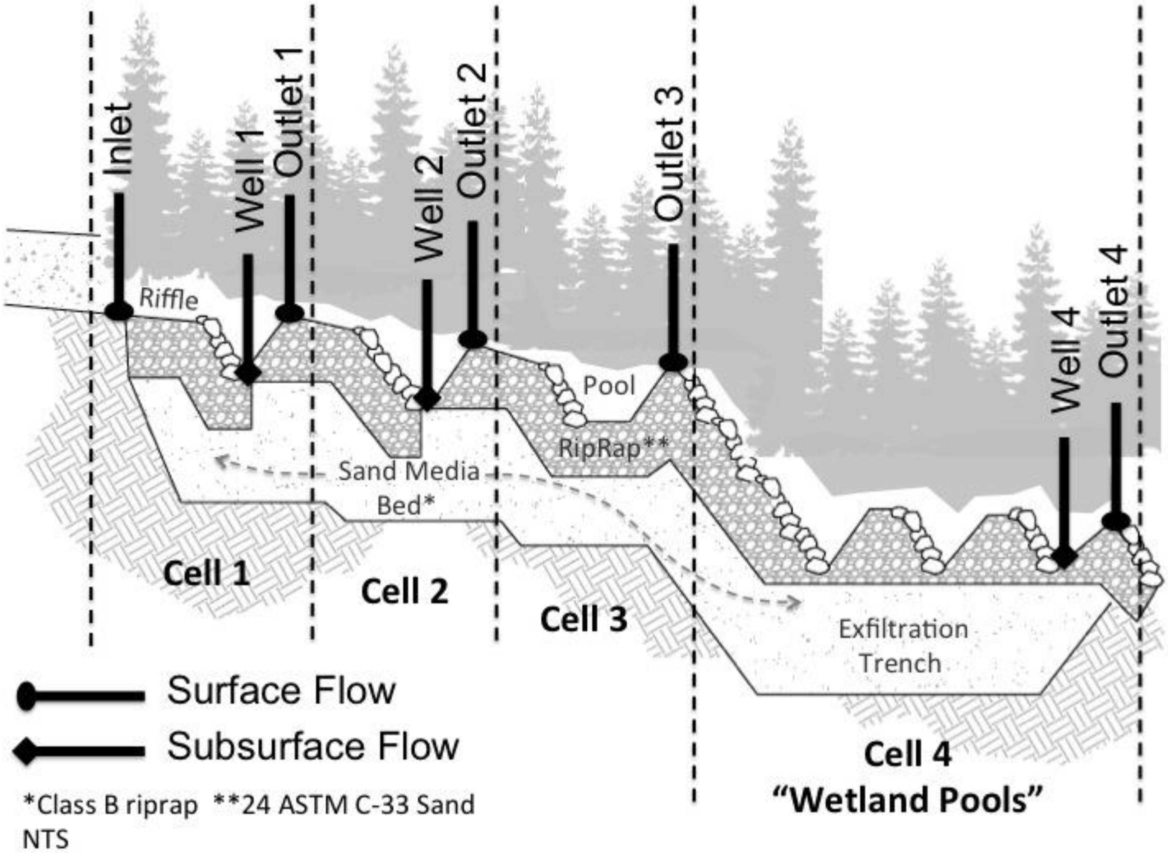


Figure 5 Alamance BFC profile with important design aspects and monitoring points

Hydrologic Monitoring

Each pool/riffle series was monitored (Figures 3 and 5). Surface flow and exfiltration were measured using a series of weirs, wells, and pressure transducers. Evapotranspiration was calculated using reference values measured at nearby weather stations, and seepage was calculated by completing the water balance. Rainfall was measured using a tipping bucket rain gauge and a manual rain gauge located adjacent to the BFC. Groundwater levels in the Brunswick BFC were monitored using a 3-m deep groundwater well located on the bank adjacent to the third pool.

Surface Water

At the Brunswick BFC, Hobo™ U-20 pressure transducers were used to collect 2-min interval water depth measurements and temperature measurements at the inlet and outlet of each cell within the BFC. The inlet weir was located immediately upslope of the driveway culvert to avoid backwater submerging the weir (Figure 3). Each of the concrete riffle weirs forming the outlet of each cell were fitted with a compound weir composed of a 12.7-cm tall 90-deg V-notch lower section and a 0.9-m wide broad crested upper section.

Runoff levels and velocities entering the Alamance BFC were recorded at 2-min intervals using an ISCO 750 area-velocity meter located 1 m within the 61-cm diameter concrete pipe conveying water to the BFC. Flow entering the BFC was calculated for each 2-min interval using Eq. 1.

$$Q = v\frac{r^2(\theta - sin\theta)}{2\mathrm{E}{-3}} \qquad \text{Eq. 1}$$

where Q is the flow rate, in L/s

θ is determined by $2\arccos\left(\frac{r-h}{r}\right)$, in radians

r is the pipe radius in meters, 0.3 m

h is the height of the water in the pipe in meters

v is the velocity of the water, m/s

When ponded water created backwater in the inlet pipe, runoff entering the BFC was estimated using the SCS Curve Number Method (SCS, 1985) and Rational Method (ASCE, 1996), with the curve number and runoff coefficient estimated from known storm inflows. ISCO 730 bubblers were used to collect 2-min interval water depth measurements at the inlet and outlet monitoring weirs of the remaining BFC cells (Figure 5). The monitoring weirs located at the outlet of each pool consisted of compound weir with13-cm tall, 90-deg V-notch lower section and a 0.9-m wide broad crested upper section (Figure 4).

A compound 90-degree v-notch and broad crested weir equation was used to determine flow through the Brunswick and Alamance inlet and outlet weirs, using the water level above the invert of the weir (Eq. 2) for each 2-min interval.

$$Q = \begin{cases} 2.31h^{2.49} & h \leq 12.7 \\ 2.31h^{2.49} + 10L(h - 12.7)^{1.5} & h > 12.7 \end{cases} \quad \text{Eq. 2}$$

where Q is the flow rate, in L/s, when water level is h cm

L is the length of the broad crested weir, 0.91 m

Flow measurements were used to determine the peak flow reduction and overall volume reduction of the BFC system for each measured rainfall event.

In addition to surface flows entering the BFCs via the inlet, runoff from the adjacent land was expected to enter each cell as overland flow. This surface volume contribution is calculated using the SCS Curve Number Method (SCS, 1985). Curve numbers (CN) for the roadway and brushy shoulder were estimated as 98 and 48, respectively. Contributing run-on areas for each cell are described in Table 1 and Table 2. Precipitation falling directly on each cell was accounted as a direct contributor to inflow.

Exfiltration

The media bed was designed to facilitate exfiltration into the parent soil, and eventually into the groundwater, similar to the function of bioretention media (Brown, 2010). An exfiltration trench, where the bottom of the media bed followed the surface pool contours, was located within each cell enabling the estimation of exfiltration rates (Figure 3, Figure 5). Hobo™ U-20 pressure transducers within wells were used to measure the height of water within the media bed in each pool. Water stored below the top of the exfiltration trench could only leave the BFC via exfiltration into the parent soil. From that point the drawdown of the water level and associated volume reductions were solely attributed to exfiltration; volume changes were used to calculate the instantaneous exfiltration rate at 2-min intervals. A regression analysis was used to determine the relationship between the volume of water exfiltrated and the stage of the water within the media. Exfiltration was then back calculated over the entire storm event for water stages exceeding the height of the exfiltration trench.

Evapotranspiration

Despite the BFCs in Alamance and Brunswick counties being seeded with stabilization grass mix and mulched, vegetation did not establish during the monitoring periods. As such, evaporation rates were estimated *only* when water was ponded on the BFC. Pan evaporation values from KSUT-Brunswick Co Airport in Southport, NC (33.93 N, 78.07 W), located 25.75 km from the Brunswick BFC, and KBUY-Burlington Alamance Airport in Burlington, NC (36.05 N, 79.47 W), located 6.4 km from the Alamance BFC, were multiplied by a PAN coefficient of 0.9 (estimated from Allen and Pruitt, 1991), and used to estimate evaporation. Extended ponding at the Brunswick BFC allowed for continuous evaporation between rainfall events, which, if considered, would overestimate evaporative losses associated with a single event. Therefore, evaporative losses specific to a rainfall events at the Brunswick BFC were only considered when water stored within the BFC increased due to incoming runoff until it returned to its pre-rainfall amount (i.e. when final storage was equal to initial storage).

Seepage

Water levels monitored within the sampling wells were originally intended to be used for calculating seepage, or horizontal subsurface flow from one cell to another, within the BFC media bed. However, unanticipated high groundwater levels were observed in the on-site groundwater well at the Brunswick site, resulting in long-term ponding within the pools of each cell. These persistently high water levels within the media rendered any seepage calculations impossible. Instead, seepage was calculated using all the previously determined inflow and outflows. For each cell, the following water balance was calculated (Eq. 2).

$$\boldsymbol{\Delta Storage = V_{surf,in} + V_{seep,in} + V_{RO} - V_{surf,out} - V_{seep,out} - E - Ex} \qquad \text{Eq. 2}$$

where ΔStorage is the change in storage

V_{surf} is the surface volume into and out of the cell

V_{seep} is the seepage volume into and out of the cell
V_{RO} is the precipitation and run-on volume entering the cell through overland flow, as opposed to through the inlet weir
E is the calculated evaporation
Ex is the calculated exfiltration

BFC storage for each cell was determined based on stage-storage calculations. As the system was frequently wet, specific yield for the sand media and rip rap was estimated as 30% and 25%, respectively (Stephens et al., 1998).

Pre-development Conditions

Pre-development runoff conditions were compared to BFC outflow to determine the extent to which the BFC was able to mimic pre-development hydrology during the monitoring period. Pre-development runoff volumes and flows of each storm event were calculated using the SCS Curve Number Method (SCS, 1985) and the Rational Method (ASCE, 1996). Vegetation in the 5.2-ha Brunswick County watershed was primarily woody. Conservative pre-development conditions were modeled, with an SCS curve number of 35 and a rational runoff coefficient of 0.15. Vegetation in the 1.6-ha Alamance County watershed was un-grazed wooded or pine forests. A predevelopment SCS curve number for the watershed was estimated as 77 (considering HSG D soils), and a rational runoff coefficient of 0.15 was chosen. Given the calculated runoff volume and peak flow, a step-function described in Eq. 3, (Malcom, 1989), was used to estimate the center-weighted pre-development hydrograph for each storm event.

$$q_i = \begin{Bmatrix} \frac{Q_p}{2}\left[1 - cos\left(\frac{\pi t_i}{T_p}\right)\right] & for\ t_i \leq 1.25T_p \\ or & \\ 4.34Q_p exp\left(-1.30\,\frac{t_i}{T_p}\right) & for\ t_i > 1.25T_p \end{Bmatrix} \qquad \text{Eq. 3}$$

where q_i is the respective flow rate for time $t_{\text{-}i}$ m^3/s
Q_p is peak design flow rate in m^3/s

T_p is the time to Q_p in seconds, calculated by $T_p = \frac{V}{1.39Q_p}$

V is the total runoff volume in m^3

Additionally, undeveloped watersheds show clear evidence of three different flow pathways contributing to a storm hydrograph: 1) surface runoff, 2) groundwater surge, and 3) shallow interflow (Williams and Pinder, 1990; Brown et al., 1999; Kendell et al., 2001). To truly achieve predevelopment hydrology, all three pathways should be considered. Previous studies of BFCs suggest that the surface runoff and shallow interflow (also referred to as seepage) pathways are present at the outlet of BFCs (Brown, 1999). Stable isotopes have been used in the past to distinguish between water from that rainfall event and stored water from a previous event in SCMs (Cizek and Hunt, 2013). Samples were taken at the Alamance BFC in the Spring 2014 from the inflow and the outflow of the system using ISCO 6712 automated samplers over the course of the storm hydrograph. The cumulative inflow sample and samples at select points within the outflow hydrograph underwent analysis for deuterium isotope concentration at Duke DEVIL Labs in Durham, NC. Deuterium levels over the course of the hydrograph were used to determine if the initial water leaving the BFC was water from a previous rainfall event, thus mimicking the predevelopment groundwater surge pathway.

Water Quality Monitoring

Water quality samples were taken from the surface water at the Alamance BFC, with sampling locations at the inlet and the outlet of each cell. Pollutant concentrations and total pollutant loadings were calculated using flow volumes as described in the section above.

Field Measurements

ISCO 6712 automated samplers collected flow-weighted composite samples at the sampling sites indicated in Figure 5 to determine event mean concentrations (EMCs) for each storm event. Samples were collected within 36 hours of the storm event, placed on ice, and submitted to the NCSU Center for Applied Aquatic Ecology for Total

Suspended Solids (TSS), Total Phosphorus (TP), Ortho-Phosphate (OP), Nitrate/Nitrite-Nitrogen (NO3), Total Kjeldahl Nitrogen (TKN), and Ammonium-Nitrogen (TAN) analysis. Total Nitrogen (TN) was calculated using Eq. 4

$$[TN] = [TKN] + [NO_3/NO_2 - N] \qquad \text{Eq. 4}$$

Additionally, pH and temperature were measured for each sample. Recorded EMCs show pollutant concentration reduction throughout the system, providing valuable insight into possible pollutant removal processes occurring within the BFC.

Loading Calculations

The total loading of pollutants exiting the system measures BFC pollutant contribution to receiving waters. The overall pollutant loading was calculated for each contaminant measured in field using Eq. 5

$$TL = EMC \sum V_{Weir,t} \qquad \text{Eq. 5}$$

where TL is the total loading of a pollutant for one storm event (mg)
EMC is the event mean concentration for the pollutant (mg/L)
$V_{Weir,t}$ is the volume (L) measured at the specified at time *t,* for all *t*'s over the course of a storm event.

Statistics

Water quality and hydrologic data were tested for normalcy using the Sharpiro-Wilk test and visual assessment. The data were uniformly non-normal; therefore, non-parametric statistical methods were used for further comparative analysis. Differences in inflow and outflow volume, flows, pollutant concentrations, and pollutant loads were tested for significance using the Wilcoxon Rank-Sum Test. Differences in seasonality was tested using the Kruskal-Wallis non-parametric ANOVA. For all statistical analyses, the data were considered significant when Type I error (α) was less than 0.05.

Results

Brunswick County BFC

A total of 27 inflow-producing events were monitored from October 2012 to March 2014. Inflow producing precipitation depths ranged between 5.8 mm and 74.3 mm (Table 3). The SCS curve number was back-calculated and plotted for each event to verify that inflow data was reasonable given known watershed characteristics (Figure 6). Admittedly, for most of these storm depths the SCS did not intend curve numbers to be used, but the data did show a trend asymptotically approaching a watershed curve number in the low 40's (Hawkins, 1993, Mullem et al., 2000). Given the aforementioned land has 0.64 ha impervious area (CN of 98) and 4.56 ha of brush/forest cover (CN of 35), the estimated composite curve number for the watershed is 43, which corresponds very well with inflows observed at the BFC.

Of 27 events, only *two* events resulted in surface outflow from the BFC. The first, 1.4 m^3 of inflow occurring on October 8, 2012, followed within 24 hours of a previous event, and produced < 1 m^3 of outflow. The second outflow-producing event resulted from 22 m^3 of inflow from Tropical Storm Andrea (June 6-7, 2013). Additionally, only two events resulted in surface outflow exiting Cell 1, one of which was Tropical Storm Andrea. The other, occurring on October 7, 2012, resulted from 30 mm of rainfall, an amount that never again produced surface outflow from Cell 1. There was initially a problem with alternative subsurface flow pathways around the concrete weirs, hence short-circuiting around Outlet 1 (Figure 3), where the concrete weir tied into the parent soil. This was repaired in late October 2012 using sand bags and riprap, which likely prevented high volumes of flow from passing this way, but probably did not altogether eliminate the alternative pathway around the weir. Groundwater levels monitored in the neighboring (control) well were higher than the base of the BFC, indicating groundwater was indeed present in BFC media, interacting with the stormwater runoff.

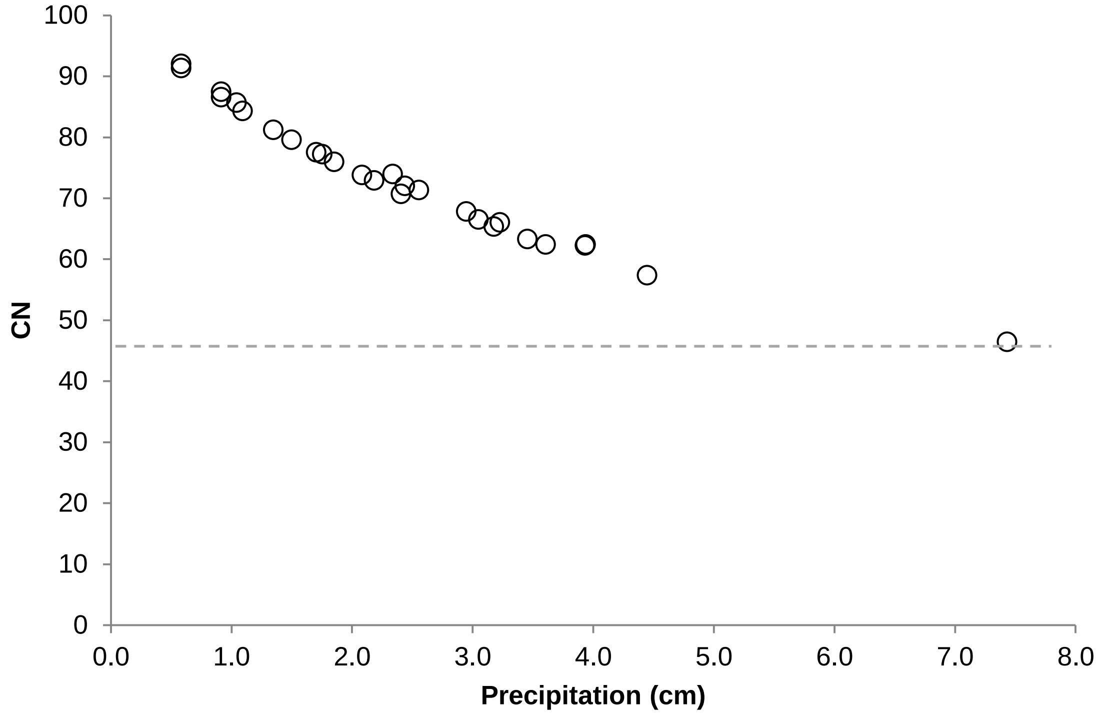


Figure 6 Relationship between watershed runoff contributions to BFC and back-calculated curve numbers (CN).

Table 3 Overall water balance in the Brunswick BFC from October 2012 to March 2013, including surface inflow/outflow, exfiltration, evaporation, and run on.

		Cell 1						Cell 2							Cell 3						
		IN (mm)		OUT (mm)				IN (mm)			OUT (mm)				IN (mm)			OUT (mm)			
Date	*Precip (mm)*	*Sur*	*Run On*	*Sur*	*Seep*	*E*	*Exfil*	*Sur*	*Run On*	*Seep*	*Sur*	*Seep*	*E*	*Exfil*	*Sur*	*Run On*	*Seep*	*Sur*	*Seep*	*E*	*Exfil*
10/7/12	30	0.12	0.03	0.00	0.15	0.00	0.00	0.00	0.05	0.15	0.00	0.20	0.00	0.00	0.00	0.02	0.20	0.00	0.22	0.00	0.00
10/8/12	18	0.03	0.02	0.00	0.04	0.00	0.00	0.00	0.03	0.04	0.00	0.07	0.00	0.00	0.00	0.01	0.07	0.00	0.08	0.00	0.00
11/18/12	36	0.05	0.04	0.00	0.09	0.00	0.00	0.00	0.06	0.09	0.00	0.15	0.00	0.00	0.00	0.03	0.15	0.00	0.18	0.00	0.00
12/12/12	22	0.02	0.02	0.00	0.04	0.00	0.00	0.00	0.03	0.04	0.00	0.06	0.01	0.00	0.00	0.02	0.06	0.00	0.07	0.00	0.00
12/25/12	31	0.07	0.03	0.00	0.10	0.00	0.00	0.00	0.05	0.10	0.00	0.14	0.01	0.00	0.00	0.02	0.14	0.00	0.16	0.00	0.00
3/19/13	14	0.00	0.01	0.00	0.01	0.00	0.00	0.00	0.02	0.01	0.00	0.02	0.01	0.00	0.00	0.01	0.02	0.00	0.03	0.00	0.00
3/24/13	32	0.05	0.03	0.00	0.08	0.01	0.00	0.00	0.05	0.08	0.00	0.11	0.02	0.00	0.00	0.02	0.11	0.00	0.13	0.00	0.00
4/4/13	21	0.01	0.02	0.00	0.03	0.00	0.00	0.00	0.03	0.03	0.00	0.06	0.00	0.00	0.00	0.02	0.06	0.00	0.08	0.00	0.00
4/19/13	24	0.00	0.02	0.00	0.02	0.01	0.00	0.00	0.04	0.02	0.00	0.03	0.02	0.00	0.00	0.02	0.03	0.00	0.04	0.01	0.00
5/20/13	19	0.01	0.02	0.00	0.01	0.01	0.00	0.00	0.03	0.01	0.00	0.02	0.02	0.00	0.00	0.01	0.02	0.00	0.03	0.00	0.00
5/23/13	9	0.01	0.01	0.00	0.00	0.02	0.00	0.00	0.01	0.00	0.00	0.00	0.01	0.00	0.00	0.01	0.00	0.00	0.01	0.00	0.00
6/6/13	74	0.43	0.18	0.00	0.59	0.01	0.00	0.00	0.22	0.59	0.01	0.79	0.01	0.00	0.01	0.09	0.79	0.02	0.87	0.01	0.00
6/29/13	15	0.01	0.01	0.00	0.02	0.00	0.00	0.00	0.02	0.02	0.00	0.04	0.00	0.00	0.00	0.01	0.04	0.00	0.05	0.00	0.00
6/30/13	6	0.00	0.03	0.00	0.03	0.00	0.00	0.00	0.03	0.03	0.00	0.06	0.00	0.00	0.00	0.02	0.06	0.00	0.08	0.00	0.00
6/30/13	10	0.01	0.02	0.00	0.03	0.01	0.00	0.00	0.03	0.03	0.00	0.04	0.01	0.00	0.00	0.02	0.04	0.00	0.06	0.00	0.00
7/2/13	9	0.01	0.03	0.00	0.03	0.01	0.00	0.00	0.03	0.03	0.00	0.04	0.02	0.00	0.00	0.02	0.04	0.00	0.06	0.00	0.00
7/11/13	39	0.31	0.05	0.00	0.36	0.00	0.00	0.00	0.07	0.36	0.00	0.43	0.00	0.00	0.00	0.03	0.43	0.00	0.46	0.00	0.00
7/13/13	26	0.16	0.02	0.00	0.17	0.01	0.00	0.00	0.04	0.17	0.00	0.21	0.01	0.00	0.00	0.02	0.21	0.00	0.22	0.01	0.00
7/29/13	24	0.13	0.02	0.00	0.11	0.04	0.00	0.00	0.04	0.11	0.00	0.08	0.06	0.00	0.00	0.02	0.08	0.00	0.07	0.03	0.00
8/14/13	35	0.04	0.04	0.00	0.08	0.00	0.00	0.00	0.06	0.08	0.00	0.14	0.00	0.00	0.00	0.03	0.14	0.00	0.17	0.00	0.00
8/16/13	39	0.28	0.05	0.00	0.33	0.00	0.00	0.00	0.07	0.33	0.00	0.40	0.00	0.00	0.00	0.03	0.40	0.00	0.43	0.00	0.00

Table 3-2 (continued)

		Cell 1						Cell 2							Cell 3						
		IN (mm)		OUT (mm)				IN (mm)			OUT (mm)				IN (mm)			OUT (mm)			
Date	*Precip (mm)*	*Sur*	*Run On*	*Sur*	*Seep*	*E*	*Exfil*	*Sur*	*Run On*	*Seep*	*Sur*	*Seep*	*E*	*Exfil*	*Sur*	*Run On*	*Seep*	*Sur*	*Seep*	*E*	*Exfil*
8/23/13	11	0.01	0.01	0.00	0.02	0.00	0.00	0.00	0.02	0.02	0.00	0.04	0.00	0.00	0.00	0.01	0.04	0.00	0.04	0.00	0.00
9/22/13	23	0.23	0.02	0.00	0.26	0.00	0.00	0.00	0.04	0.26	0.00	0.29	0.00	0.00	0.00	0.02	0.29	0.00	0.31	0.00	0.00
11/26/13	32	0.16	0.03	0.00	0.07	0.00	0.12	0.00	0.05	0.07	0.00	0.12	0.00	0.00	0.00	0.02	0.12	0.00	0.12	0.00	0.02
12/14/13	17	0.01	0.02	0.00	0.03	0.00	0.00	0.00	0.03	0.03	0.00	0.05	0.00	0.00	0.00	0.01	0.05	0.00	0.06	0.00	0.00
2/11/14	45	0.06	0.05	0.00	0.11	0.01	0.00	0.00	0.07	0.11	0.00	0.17	0.01	0.00	0.00	0.04	0.17	0.00	0.21	0.00	0.00
2/15/14	6	0.02	0.01	0.00	0.03	0.00	0.00	0.00	0.01	0.03	0.00	0.03	0.00	0.00	0.00	0.00	0.03	0.00	0.04	0.00	0.00
Total	**665**	**2.26**	**0.85**	**0.01**	**2.84**	**0.14**	**0.12**	**0.01**	**1.21**	**2.84**	**0.01**	**3.79**	**0.25**	**0.00**	**0.01**	**0.59**	**3.79**	**0.03**	**4.26**	**0.08**	**0.02**

Water Balance

Because the watershed was sandy, the majority of runoff entering the system (53%) did so as run-on from adjacent Hwy 17, as opposed to through the driveway culvert (Table 3). As described earlier, this part of the watershed contained the impermeable surface of Hwy 17 followed by a steeply sloped shoulder leading into the BFC. Therefore, it is reasonable that run-on contributed a large amount of additional inflow volume to each cell. The cells effectively converted most (86%) surface flows to subsurface seepage through the media, and eventually seepage from the BFC.

Many exchanges between surface flows and subsurface flows occur throughout the BFC. The "conversion" of a drop of runoff from surface flow to subsurface flow in Cell 1 does not mean that that same drop will not re-emerge as surface flow in Cell 2, as observed by water seeping between concrete weirs during Tropical Storm Andrea (June 6-7, 2013). During Andrea, measured surface flow volume was reduced to 0.11 m^3 in Cell 1; then gradually increased to a surface flow volume of 1.25 m^3 in subsequent Cells 2 and 3. These exchanges have an important implication for effluent water quality; in this case, surface flow almost always underwent some level of media treatment via subsurface flow prior to leaving the BFC (Peterjohn and Correll, 1984; Cooper, 1990; Lowrance et al., 2000).

In addition to surface/subsurface exchanges for the stormwater entering the BFC, a fluctuating groundwater table resulted in interchanges between stormwater and groundwater. The seasonally high water table (SHWT) was estimated to be 2.4 m below the bottom of Cell 2 prior to construction of the BFC from historical records. Construction was completed in June 2012. When monitoring began in October 2012, permanent ponding was observed within the BFC sand media, with corresponding groundwater levels in the adjacent monitoring wells. Cell 2 had ponded water in its pool from October 2012 through June 2013. Ponding in this cell was again observed in February 2014 and through the remainder of the monitoring period. Increases in groundwater levels have been associated with stream restoration projects (Hammersmark et al., 2008) and beneath bioretention cells due to mounding (Endreny and Collins, 2009; Machusick et al., 2011), although the mounding only occurred for short durations after the precipitation event. Water was periodically ponded in Cells 1 and 3, but for shorter durations than Cell 2. The continuously ponded water in Cell 2 resulted in wetland-like conditions, leading to the emergence of a thriving *Typha* (cattail) community (Figure 7). Extended ponding enabled 10% of the overall volume to be evaporated. In November 2013, water levels within the media of Cell 1 drained below top of the exfiltration trench (Figure 3), allowing exfiltration rates to be calculated. Regression analysis found an irregular relationship between water level in the media and exfiltration loss normalized by area (Figure 8). Initial high rate of exfiltration may be due to the parent soil being unsaturated. As more water is present in the media around the parent soil, the exfiltration rate moves towards the saturated exfiltration rate (K_{SAT}), represented by the second line. It is possible that the steep curve of the third line includes both lateral flow and exfiltration, suggesting that the top of the exfiltration trench is not precisely at 0.63 m, but slightly lower. As no survey was taken prior to filling the media bed, the top of the exfiltration trench was considered to be 0.57 m from this point further, as indicated in Eq. 6.

$$r_{vol} = \begin{Bmatrix} 208.9x - 51.3 & 0.24 < x < 0.26 \\ 11.7x + 0.075 & 0.26 < x < 0.63 \end{Bmatrix} \qquad \text{Eq. 6}$$

where r_{vol} is the change in storage (L/h/m^2)

d is the stage of water in exfiltration trench above the parent soil (m)

Exfiltration was expected to occur when groundwater levels were below the top of the trench at the beginning of the storm. When it occurred, exfiltration accounted for up to 53% of water loss on a storm-by-storm basis, ultimately comprising 3% of the total runoff loss during the entire monitoring period (Table 3).

Figure 7 Typha community in Cell 2, July 2014

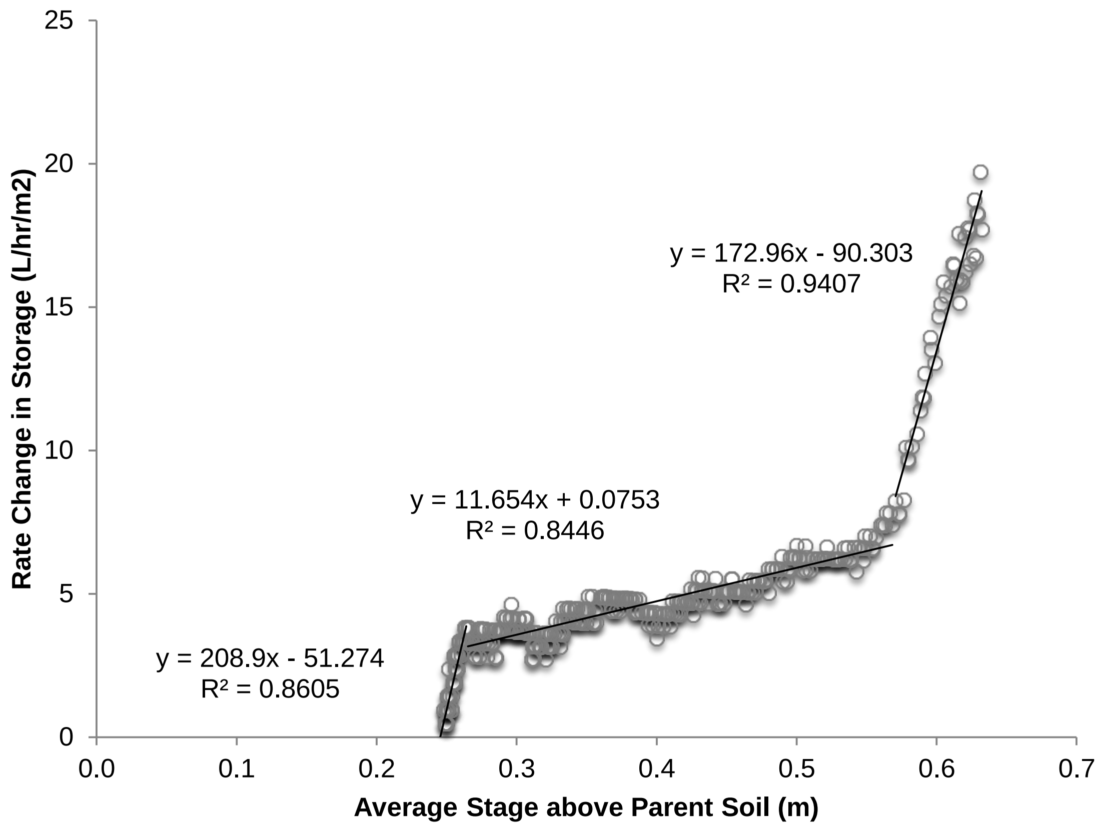


Figure 8 Relationship between exfiltration rates per area and the average stage above parent soil.

As designed, the majority (86% in this case) of the stormwater entering the BFC left the system as seepage from both the sand and rip rap media layers (Table 3). This may have important implications for water quality mitigation, as any seepage out has been subject to some level of media treatment through the BFC sand layer, although the distance water has traveled through this layer likely varies.

Volume and Peak Flow Mitigation

Surface flow volumes were reduced between 94% and 100% on a storm-by-storm basis. When surface outflow occurred, peak flow discharge rates decreased between 90% and 96%. Flow was observed being well distributed across the 4.6-m wide, 0.3-m deep channel, and overtopped the v-notch weir at a maximum non-erosive velocity of

0.3 m/s. This same flow spread out over the entire concrete weir in the absence of monitoring equipment would result in a outflow velocity of 0.6 cm/s.

Because many jurisdictions are adopting pre-development hydrograph-based stormwater goals (Low Impact Development Center, 2007), the modeled pre- and measured post-development hydrographs were compared. The pre-development watershed was sandy and woody so that a runoff-producing event would need to exceed 94 mm of rainfall, just less than the 1-yr, 24-hr rainfall event for the area. As no event this large occurred during the monitoring period, no runoff would theoretically have been yielded under pre-development conditions for the monitored storm events. The post-development inflow hydrograph was represented by the inflow hydrograph recorded at the inlet of the BFC, although this is a conservative representation as it does not consider the contribution of direct run-on from Hwy 17. The post-development effluent hydrograph from the BFC is represented by the recorded BFC surface outflow from the outlet of Cell 3. Because the SCS Curve Number Method used to estimate the pre-development conditions only accounts for direct runoff contributions to the storm hydrograph (SCS, 1985; NBFC, 1986), only surface outflow from the BFC was considered in the post-development hydrograph. The BFC did mimic the pre-development hydrograph for 25 of the 27 runoff producing events (or 94% of the time). However, Tropical Storm Andrea on June 6-7, 2013 (74.4 mm) produced 1.25 m^3 of surface outflow from the BFC, where the pre-development would have yielded zero outflow (Figure 9). So, despite the superior surface flow reductions observed, performance of the Brunswick County BFC did not quite achieve pre-development hydrologic conditions in every instance, with the notable exceptions being extreme weather events such as a Tropical Storm.

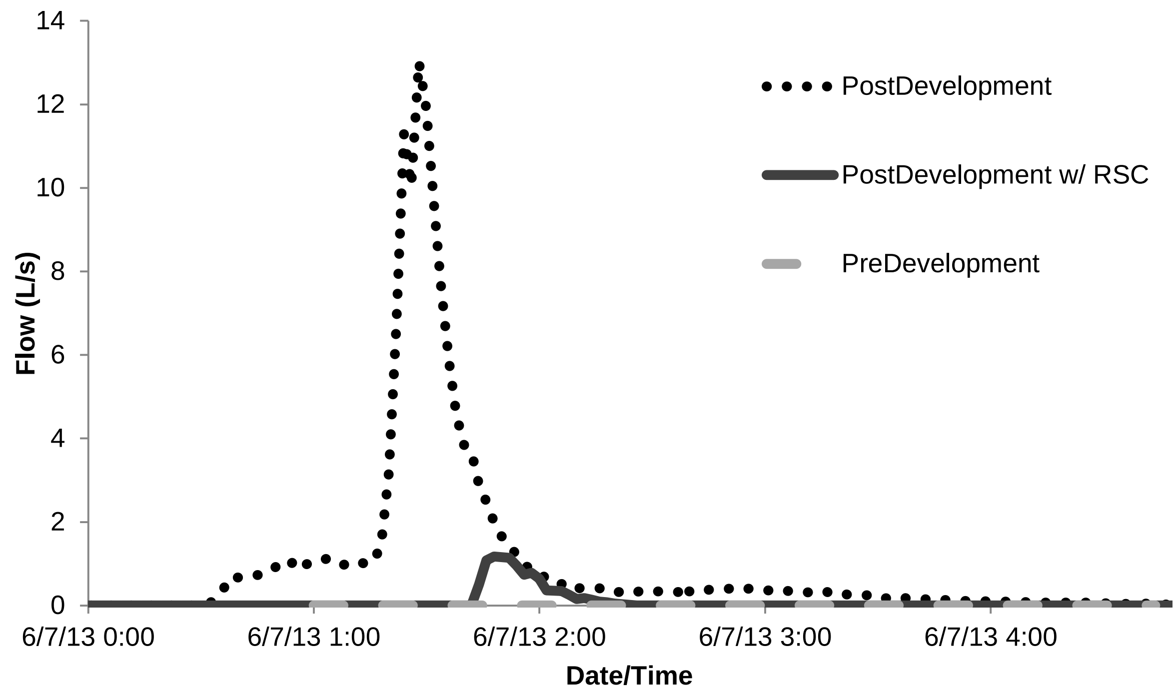


Figure 9 Pre-development (modeled) and post-development (measured) hydrographs for June 6-7 rainfall event, 74.3 mm

Alamance BFC

Hydrologic Performance

The Alamance BFC received 978 mm of total monitored rainfall between July 10, 2013, and June 10, 2014, resulting in 43 inflow-producing rainfall events. The largest event, occurring on September 1, 2013, had 81 mm of total precipitation falling at maximum intensity of 74 mm/h. This event produced in 660 m^3 of runoff at a peak flow of 246 L/s. During this most extreme event, the BFC discharged 235 m^3 of surface outflow at a maximum rate of 102 L/s, effectively reducing surface runoff by 57% and peak flow by 68%. Over the course of the monitoring period, median runoff and peak flow reductions were 84% and 80%, respectively (Table 4). Two of the three cells prior to the cascade significantly reduced surface volume and flow (Cell 3 being the exception). Cell 4 had a 36% median *increase* in surface volume between the inlet and outlet. This is likely because of the frequently high subsurface water levels present in the media of Cell 4, resulting in little additional subsurface storage for the seepage entering from Cell 3. Ultimately, the presence of the "wetland pools" (Cell 4) in the BFC slightly decreased median hydrologic performance from 89% surface volume and 85% peak flow reduction to 84% surface volume and 77% peak flow reduction.

Table 4 Median measured volume and peak flow over each weir. Bold values are statistically significant based on α = 0.05.

	Volume			Peak Flow		
	Median (m^3)	Cell Reduction	Cum. Reduction	Median (L/s)	Cell Reduction	Cum. Reduction
Inlet	129.5			80.4		
Outlet 1	78.6	**30%**	**30%**	33.7	**55%**	**55%**
Outlet 2	29.0	**61%**	**79%**	23.2	**44%**	**69%**
Outlet 3	15.9	**58%**	**89%**	14.2	**47%**	**78%**
Outlet 4	20.9	**-36%**	**84%**	15.0	-6%	**77%**

As predicted by Brown (2009) and observed at the Brunswick BFC, 77% of the runoff that entered the BFC was effectively converted to subsurface seepage (cognate to shallow interflow) (Table 5). As the surface water infiltrated into the media, the amount of water leaving each cell as subsurface seepage increased progressively through the BFC until Cell 4. The subsurface geometry of Cell 4 retained water in the media, thus restricting some subsurface seepage from leaving the BFC (Figure 4). Despite the increase in surface flow, the retention of runoff in Cell 4 did promote some (albeit slow) exfiltration into the parent soil (nearly 280 m^3 over the course of the monitoring period) (Table 5). Volumetric exfiltration rate regression showed a high level of variability, perhaps due to fluctuating water table and underlying clay soils. Exfiltration rates area exponentially influenced by soil water content (Mahmood-ul-Hassan et al., 2013; de Faria and Bowen, 2003), which can vary in clay soils depending on how recently the soil was saturated. Ultimately, an exponential relationship was found as described by Eq. 7

$$Vol_{EXFIL} = 0.0497 \exp(3.9794d) \qquad R^2 = 0.225 \qquad \text{Eq. 7}$$

where Vol_{EXFIL} is the volume of water exfiltrated in L/hr/m^3

d is the water level in m above the base of media

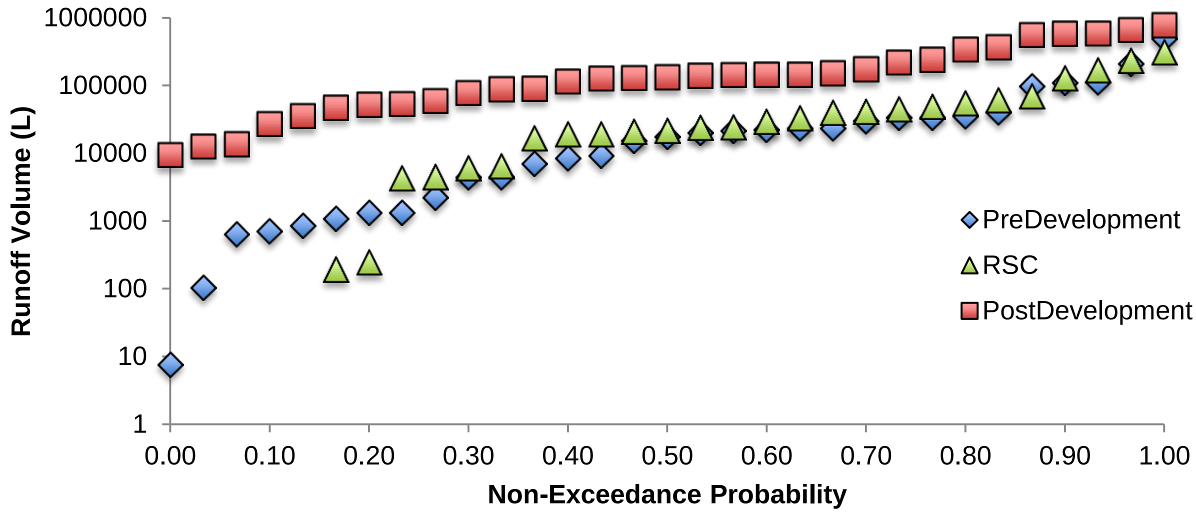
A negligible 24 m^3 of water were evaporated when water was ponded within the pools of the BFC. Based on studies done in bioretention cells, it is expected that this number could increase substantially (orders of magnitude) with the presence of established vegetation within the pools (Brown et al., 2013).

Table 5 Fraction of inflow and outflow sources and fates for each BFC cell.

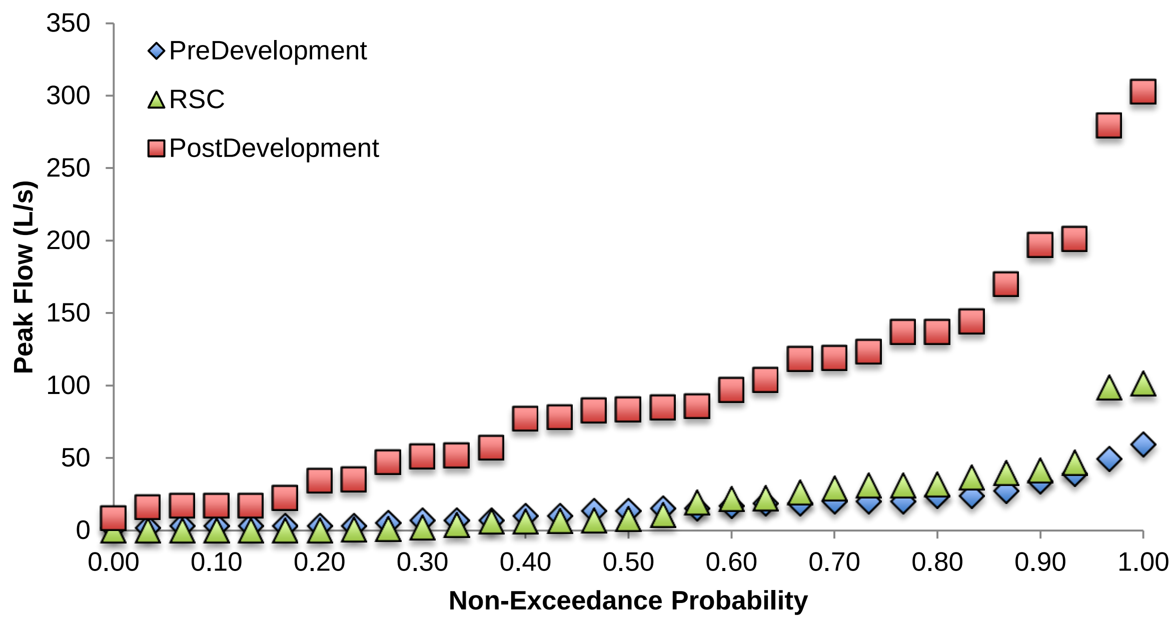
		INFLOW			OUTFLOW			
		Surface	Run On	Seep	Surface	Seep	ET	Exfil
Cell 1	*Sum*	0.91	0.04	NA	0.64	0.32	0.00	0.00
	Fall	0.92	0.03	NA	0.70	0.27	0.00	0.00
	Win	0.92	0.03	NA	0.76	0.19	0.00	0.00
	Spr	0.92	0.03	NA	0.53	0.39	0.00	0.00
	Total	**0.93**	**0.03**	**NA**	**0.60**	**0.35**	**0.00**	**0.00**
Cell 2	*Sum*	0.64	0.01	0.32	0.22	0.74	0.00	0.00
	Fall	0.70	0.01	0.27	0.10	0.84	0.00	0.00
	Win	0.76	0.01	0.19	0.30	0.65	0.00	0.00
	Spr	0.53	0.01	0.39	0.13	0.82	0.00	0.00
	Total	**0.60**	**0.01**	**0.35**	**0.30**	**0.66**	**0.00**	**0.00**
Cell 3	*Sum*	0.22	0.01	0.74	0.13	0.85	0.00	0.00
	Fall	0.10	0.01	0.84	0.07	0.89	0.00	0.00
	Win	0.30	0.01	0.65	0.13	0.82	0.00	0.00
	Spr	0.13	0.01	0.82	0.03	0.91	0.00	0.00
	Total	**0.30**	**0.01**	**0.66**	**0.15**	**0.81**	**0.00**	**0.00**
Cell 4	*Sum*	0.13	0.03	0.85	0.26	0.70	0.00	0.03
	Fall	0.07	0.03	0.89	0.11	0.95	0.00	0.04
	Win	0.13	0.03	0.82	0.07	0.86	0.00	0.05
	Spr	0.03	0.03	0.91	0.04	0.82	0.00	0.02
	Total	**0.15**	**0.03**	**0.81**	**0.20**	**0.77**	**0.00**	**0.03**

The data were tested for seasonal influences using the following criteria – Summer was considered Jun 1 through Sept 14, Fall was Sept 15 through Dec 14, Winter was Dec 15 through Mar 14, and Spring was Mar 15 through Jun 1 (Table 5). Inflow volume and peak flows did not show significant differences among the seasons. There was a significant difference between the fraction of water leaving Cell 1 as surface flow in the winter (median = 0.76) and spring (0.53). This is unexpected given that the inflow volume and peak flow is relatively constant. One possible explanation is the frequent freezing temperatures during winter 2013, thus creating occasional impervious layers of ice over the surface and in pore spaces of the media. Three of the eight winter rainfall events occurred on days with minimum temperature below 0^{o}C. Median surface flow reduction for these three events in Cell 1 was 17%. Median surface reductions from the other five "warmer" winter events was 34%, the latter being similar to that observed during other seasons (36% in summer, 30% in fall, 47% in spring) (Table 5). Similarly, this phenomenon may have occurred in the other cells, but was not as apparent as in Cell 1, where the greatest overall volumetric conversion from surface flow to seepage occurred. The fraction of surface flow leaving Cell 4 was also significantly greater during summer than fall, winter, or spring - a median of 26% as compared to 11%, 7%, and 4%, respectively. Summer also experienced the longest duration of high subsurface water levels. Median pre-storm Cell 4 subsurface water level was 1.31 m above the bottom of the media, or 0.21 m from the invert of the weir, as compared to 0.58 m, 1.03 m, and 1.10 m in the fall, winter, and spring, respectively. Thus, the additional surface flow from Cell 4 is likely related to the reduced subsurface storage available due to high subsurface saturation levels.

Modeled predevelopment, and measured post-development runoff and BFC outlet volumes and peak flows were compared for each monitored storm event (Figure 10). Runoff volume and peak flow from the BFC outlet were much less than or similar to modeled predevelopment conditions 95% of the time. As these are the two parameters that drive hydrograph shape, the data *strongly suggest* that BFCs can mimic the overall predevelopment hydrology. Both modeled predevelopment runoff and BFC outlet flows and volumes are significantly less than post-development conditions, reinforcing the need for an SCM to mitigate runoff from such development.



(a)



(b)

Figure 10 Modeled predevelopment, monitored post-development, and monitored BFC (a) outflow volume and (b) peak flow for each monitored rainfall event.

Inflow and partitioned outflow samples from five rainfall events April 2014 through June 2014 underwent deuterium isotope analysis to determine the age of the water leaving Cell 4. Samples displayed initially high deuterium concentrations, relative to inflow, progressing towards concentrations similar to and lower than those found in the inflow as more water left the BFC (Figure 11). Processes like evapotranspiration, plant uptake, and microbial digestion preferentially choose the lighter single neutron protium atom over the heavier deuterium atom (Barnes and Allison, 1988; Friedmand et al., 1964). Therefore, water stored in the cell between storm events (also referred to as pre-event water), which has undergone some level of evaporation and microbial digestion, is relatively enriched in deuterium as compared to the precipitation and inflow. Median ratios of deuterium outflow and inflow concentrations for the first 55 m^3, 110 m^3, and > 110 m^3 of BFC outflow are 0.69, 0.98, and 1.01, respectively. Although statistical significance was limited due to the number of samples available for said analysis, the first 55 m^3 of water released from the BFC appeared to be composed, at least in part, of pre-event water. Beyond the surface runoff and shallow interflow (or subsurface seepage) pathways evident in the water balance described herein, BFC may also be able to mimic the groundwater surge pathway when subsurface runoff retention is present. Therefore, in addition to mimicking overall predevelopment shape (volume and peak flow), BFCs also provide opportunities for all predevelopment hydrograph pathways described by Cizek and Hunt (2012). In other words, a ‘complete’ water balance may be possible when using a sufficiently large BFC.

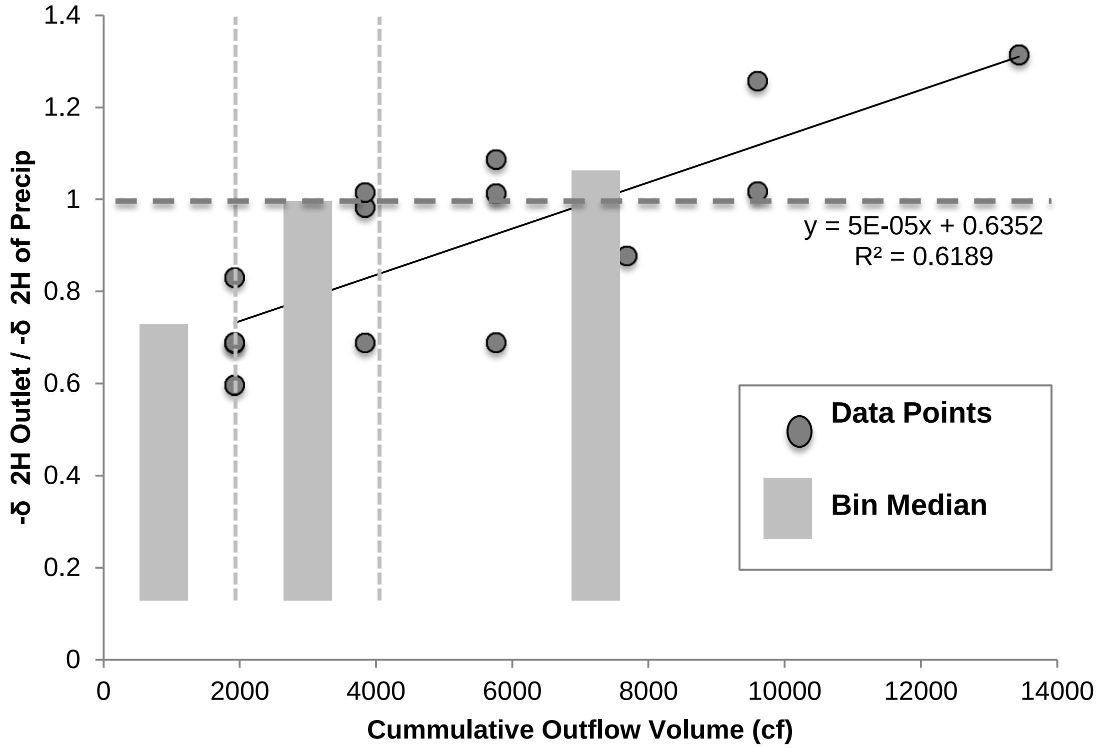


Figure 11 Summary of the deuterium isotope concentration ratio between outflow and inflow as compared to cumulative outflow. A ratio of 1 or higher suggests the water is predominantly event water.

Water Quality Performance

During the course of the monitoring period, 20 events were sampled for water quality. Influent nutrient concentrations varied widely based on activity within the watershed. Median TN concentrations were 2.4 mg/L, but ranged from 1.63 mg/L to 9.96 mg/L (Table 6). High TN concentrations often corresponded with high Total Kjedahl Nitrogen (TKN), Total Ammoniacal Nitrogen (TAN), and, in most cases high Total Phosphorus (TP) and ortho-Phosphate (OP) concentrations. It is known that the landscaping crew fertilized portions of the watershed in October and March, and some instances of high nutrient concentrations corresponded with these events. On the other hand, some instances of high concentrations of nutrients occurred mid-winter and are likely not related to fertilizers (See Appendix B). The nutrient source of the winter spikes remains unknown.

Despite the variable inflow nutrient loadings, the BFC relatively consistently removed pollutants from surface flows. The BFC significantly reduced concentrations of TN by 30%, TKN by 37%, TAN by 33%, TP by 28%, and TSS by 72%. The first cell acted as a forebay; more than 25% of the inflow TSS was removed here, but no significant nutrient reductions occurred. TSS continued to be successively removed as surface flow moved from cell to cell. Most of the nutrient reduction occurred in Cell 2, with slight increases in NO3 (0.02 mg/L or 8%) and TAN (0.03 mg/L or 14%) in Cell 3. These modest increases of some nutrient speicies may be evidence of the relatively high organic content in the media and the large amounts of composted wood chips present on the surface being flushed through the system. As this particular cell is further downstream and more difficult to access by the public, any nutrient increase is unlikely due to dumping, pet waste, etc. It is also unlikely related to fertilizer application, as the increase is isolated to Cell 3 and not observed throughout the entire BFC. Despite the modest uptick in annual concentration, significantly smaller concentrations of NO3 and TAN left Cell 3 in the spring (0.21 mg/L and 0.12 mg/L, respectively) than in summer (0.45 mg/L and 0.31 mg/L, respectively). TAN was further reduced in Cell 4, possibly as a result of additional aeration through the cascade entering the pools, oxygenating water for nitrification. Significant TP reduction also occurred in Cell 4. The overall contribution of the “wetland pools” appeared to be to further nutrient and TSS removal from the surface water. However, the Cell 4 water quality benefits were not necessarily greater than those provided by Cell 2, which was well drained. It is possible that adding vegetation to the “wetland pools” may increase microbial activity, and subsequent microbial nutrient reductions. However, it is not clear whether concentrations in runoff leaving Cell 3 were further reducible in a wetland environment (Moore et al., 2011).

Pollutant loadings were progressively reduced throughout the first three cells of the BFC (Table 6). This well-drained BFC section removed 14.8 kg of TN, 2.62 kg of TP, and 347 metric tons of TSS *from the surface flow* over the course of the monitoring period, equivalent to a median load reduction of 86% for TN and TP, and 95% for TSS. This, however, does not account for nutrients released via seep out (which went unaccounted for in these measurements). The addition of the “wetland pools” modestly increased loadings, due to the increase in surface outflow at times of high subsurface saturation. Overall, the BFC still reduced 81%, 84%, and 94% of the TN, TP, and TSS load. This corresponds to storm surface discharge loads of 2.5 kg TN, 0.57 kg TP, and 29.2 metric tons of TSS.

Table 6 Median pollutant event mean concentrations and loadings at each weir over the course of the monitoring period.

		TN		TKN		NO3		TAN		TP		OP		TSS	
		Conc (mg/L)	Load (kg)	Conc (mg/L)	Load (kg)	Conc (mg/L)	Load (kg)	Conc (mg/L)	Load (kg)	Conc (mg/L)	Load (kg)	Conc (mg/L)	Load (kg)	Conc (g/L)	Load (m.ton)
Inlet	*Sum*	1.84	471.5	1.63	4b25.3	0.21	87.3	0.20	47.2	0.33	71.6	0.22	47.1	39.5	6665
	Fall	3.95	455.9	3.08	355.4	0.56	100.5	0.88	86.7	0.66	75.9	0.50	57.9	71.2	10320
	Win	2.68	1074.4	2.35	942.6	0.23	78.9	0.35	75.2	0.58	111.6	0.20	75.7	90.1	46931
	Spr	3.14	1155.4	2.93	999.3	0.23	92.7	0.33	123.7	0.46	208.2	0.19	73.9	77.0	32413
	Total	**2.40**	**907.5**	**2.17**	**782.1**	**0.24**	**87.3**	**0.35**	**86.7**	**0.44**	**111.6**	**0.22**	**73.2**	**69.1**	**10713**
Outlet 1	*Sum*	2.03	276.5	1.79	209.6	0.30	66.8	0.21	68.5	0.43	41.7	0.30	22.3	11.3	3325
	Fall	2.68	752.6	2.40	695.9	0.28	56.7	0.55	172.1	0.44	106.8	0.17	55.0	64.6	37357
	Win	1.54	233.4	1.30	185.2	0.30	48.3	0.34	59.0	0.53	104.5	0.35	82.3	14.2	6086
	Spr	2.14	655.2	1.90	625.8	0.19	29.5	0.32	68.5	0.31	104.0	0.17	40.5	29.6	13875
	Total	**2.21**	**525.0**	**1.96**	**477.8**	**0.25**	**47.9**	**0.34**	**69.1**	**0.44**	**93.2**	**0.22**	**51.4**	**29.4**	**6044**
Outlet 2	*Sum*	1.74	92.2	1.49	73.3	0.24	19.0	0.21	11.2	0.40	15.7	0.31	12.0	9.9	1786
	Fall	1.54	62.3	1.28	54.5	0.31	7.8	0.22	14.4	0.47	23.9	0.36	17.6	33.2	1776
	Win	2.18	282.1	1.90	268.6	0.20	13.5	0.32	35.8	0.51	46.7	0.25	10.4	72.2	14347
	Spr	1.39	220.6	1.22	199.4	0.16	21.3	0.09	10.6	0.28	42.2	0.14	19.1	41.7	5711
	Total	**1.72**	**102.5**	**1.43**	**82.9**	**0.24**	**15.8**	**0.21**	**14.6**	**0.41**	**25.3**	**0.22**	**14.8**	**38.2**	**1879**
Outlet 3	*Sum*	3.80	96.4	3.36	88.9	0.45	10.9	0.31	18.6	0.63	10.1	0.22	3.8	12.8	1073
	Fall	1.54	47.4	1.13	35.0	0.31	10.3	0.24	8.6	0.49	15.0	0.38	11.6	22.4	914
	Win	2.20	112.1	1.96	105.5	0.24	6.6	0.30	10.2	0.44	14.3	0.25	3.9	32.5	2343
	Spr	1.39	146.7	1.17	127.5	0.21	19.2	0.12	9.7	0.26	27.2	0.13	12.0	27.6	2439
	Total	**1.92**	**63.1**	**1.57**	**55.9**	**0.26**	**10.6**	**0.24**	**9.1**	**0.41**	**14.9**	**0.22**	**6.8**	**22.0**	**952**
Outlet 4	*Sum*	1.86	116.8	1.52	88.0	0.35	28.9	0.29	16.2	0.30	17.0	0.19	11.8	10.0	466
	Fall	1.68	56.1	1.43	38.0	0.46	15.0	0.25	9.1	0.41	16.3	0.31	11.3	14.3	686
	Win	1.50	253.4	1.25	215.0	0.22	38.4	0.20	23.4	0.35	52.6	0.18	34.5	18.7	2608
	Spr	1.44	146.2	1.18	120.0	0.30	26.2	0.13	10.0	0.26	26.7	0.15	13.2	17.3	1706
	Total	**1.76**	**101.4**	**1.43**	**79.7**	**0.29**	**20.3**	**0.20**	**9.8**	**0.34**	**19.3**	**0.19**	**11.5**	**11.6**	**1050**

Discussion

The Alamance County BFC met traditional SCM goals of peak flow mitigation and volume reduction and LID goals of mimicking the predevelopment hydrograph. Surface volume and peak flow reductions exceeded that frequently observed in other media-based (Hunt et al., 2006; Passeport et al. 2009) or conveyance (Winston et al., 2012) SCMs located in HSG D soils. Furthermore, the outflow volume and discharge rates were strikingly similar to that modeled for the watershed under predeveloped conditions. BFCs also exhibit all flow pathways present in predeveloped watersheds, namely shallow interflow (seepage) and groundwater surge. In fact, nearly 80% of the BFC outflow hydrograph is comprised of these two pathways, a fraction within the range observed in undeveloped watershed (Brown et al., 1999; Kendell et al., 2001; Soulsby, 1995; Wenninger et al., 2004; Williams and Pinder, 1990). Surface flow could be further reduced through encouraging evapotranspiration via an established plant community and through the installation of exfiltration trenches beneath any appropriate cell.

The Brunswick County BFC also mitigated surface runoff flows by converting inflow to predominantly subsurface seepage (cognate to shallow interflow). Eighty-four percent of the runoff entering the BFC exited as seepage. Previous research on undeveloped watersheds suggests that surface runoff comprises a relatively small fraction (a median of 20% on a storm-by-storm basis) of the overall stream hydrograph (Brown et al., 1999). The fraction of surface runoff leaving this BFC was even smaller (2%). Many studies have found that storm hydrographs from undeveloped watersheds are comprised of 10% to 35% event water (surface runoff and shallow interflow) and 65% to 90% pre-event water (groundwater) (Brown et al., 1999; Soulsby, 1995, Wenninger et al., 2004; Williams and Pinder, 1990). As groundwater was present in the media bed for 90% of the monitoring period, it is likely that a large portion of the seepage was comprised of pre-event water, though this was not measured. A temperature mass balance, as described by Nath (1996) and in Eq. 8, was used to estimate the total amount of water leaving the system as seepage (including stormwater and groundwater) herein.

$$\frac{dT}{dt} = \frac{Q_i T_i}{V} - \frac{Q_e T_e}{V} + \frac{\phi_{net}}{\rho_w c_{pw} d} - \frac{T}{V}\left(\frac{dV}{dt}\right) \qquad \text{Eq. 8}$$

where T is the temperature (^{o}C) of the water in the cell, the inflow (i) and the outflow (e)

Q is the flow rate (m^3s^{-1}) of the inflow (i) and the outflow (e)

V is the total volume of water stored in the cell (m^3)

ρ_w is the density of water (kg m^{-3})

c_{pw} is the heat capacity of water (kJ kg^{-1} $^{o}C^{-1}$)

d is the depth of the ponded water (m)

ϕ_{net} is the interfacial heat transfer due to various processes occurring at the water surface (kJ m^{-2} s^{-1})

Heat transfer processes considered for ϕ_{net} included net short-wave radiation penetrating the water surface, net atmospheric long-wave radiation, long-wave water surface radiation, evaporative heat transfer, and conductive heat transfer. Detailed equations are described by Nath (1996). Total seepage is *estimated* here, as a true calculation of the water balance requires more extensive groundwater monitoring. If the seepage calculated through Eq. 8 is considered to represent total seepage leaving the BFC ($Seep_{tot}$), and the *runoff* leaving as seepage is calculated using Eq. 2 ($V_{seep.out}$, renamed $Seep_{RO}$ for the purpose of this discussion), then groundwater leaving the BFC as seepage ($Seep_{GW}$) would be described by Eq. 9.

$$Seep_{tot} = Seep_{RO} + Seep_{GW} \qquad \text{Eq. 9}$$

Using this approach, the median fraction of pre-event water (or groundwater) leaving the system per storm is 95%, similar to that measured in undeveloped watersheds on the Coastal Plain (Williams and Pinder, 1990). This finding is important as it appears the

Brunswick BFC is very close to mimicking the pre-development surface *and* subsurface flow pathways and volumes.

The greatest pollutant reductions were observed in TN, TKN, TP, and TSS. TKN concentrations comprised the majority of TN. TSS and associated TP, as well as organic nitrogen, a large component of TKN, are effectively removed via physical pollutant removal mechanisms such as sedimentation and filtration, which are likely to be the main treatment processes occurring as surface water flows through the riffle pools and is filtered through the underlying media. Additionally, observations of increasing surface flow in downstream pools at the Brunswick County BFC, suggest that water leaving the system as surface water may have experienced some subsurface filtration and reemerged as surface water in downstream pools. Significant reductions in TAN suggest some adsorption or nitrification may be occurring throughout the system, but there is not any evidence of denitrification as NO3 is not significantly reduced. Only one of the cells, (Cell #4) with the "wetland pools" harbors conditions that seem suitable for denitrification to occur. A BFC in Maryland exhibiting wetland-like conditions reduced NO3 concentrations by an average of 35% (Browning, 2009), despite that BFC having lower inflow concentrations. The main differences between the Alamance BFC and the MD site is (1) the age – the MD was 3 years established at the time of monitoring – and (2) the density, diversity, and establishment of vegetation – the Alamance site had little to no vegetation throughout the entire monitoring period while the MD site was planted as an Atlantic White Cedar habitat. Plants may be needed to provide denitrifying bacteria with an available carbon source, and the woodchips in the media may not have been sufficiently reduced within the first year of the BFC establishment. Previous studies on other SCMs have observed relationships between vegetation density and health with denitrification rates (Bachand and Horne, 2000; Liu et al., 2011; Lucas and Greenway, 2008). It is possible, then, that NO3 reductions in the BFC could improve with the planting and establishment of vegetation.

Another important consideration for water quality performance is that most of the water leaving the system is leaving as seep/ shallow interflow. If similar pollutant removal mechanisms occur to BFC subsurface seepage, then median pollutant loading reductions for this BFC would decrease to 30% TN, 27% TP, and 72% TSS. Research has shown that in riparian buffers, water traveling via shallow interflow undergoes a higher degree of physical, chemical, and in proper conditions, biological treatment process than that observed in surface flow (Cooper,1990; McDowell et al., 1992; Lowrance et al., 2000; Peterjohn and Correll, 1984). Other research conducted by Cizek (2014) explored nitrogen reduction in BFC seepage in a well-drained BFC (equivalent to Cells 1 through 3) during the winter. TKN from this BFC underwent reductions similar to that observed in the Alamance surface flow. Additional reduction of NO3 was observed in the subsurface seepage, suggesting an overall higher reduction of TN, and, therefore, smaller total TN loadings than predicted herein. To truly understand the ability of BFCs to mitigate water quality, however, pollutant removal mechanisms in BFC seepage must be explored in the field for a variety of hydrologic and seasonal conditions.

BFCs have the potential to be very adaptable to field constraints, depending on design goals for a specific project. As stated herein, for example, the addition of exfiltration trenches can reduce surface outflow volume via increasing exfiltration. These trenches also may encourage a wetter, more slowly drained system observed the Alamance BFC Cell 4, as opposed to a well-drained system experienced in Cell 1 through 3, particularly in tight clay soils. Well-drained systems provide greater surface volume reduction, with available soil pore space to encourage the greatest amount of subsurface flow. Wetter systems, on the other hand, provide opportunities for exfiltration, and if well vegetated, will likely provide additional pollutant removal mechanisms, namely nitrification/ denitrification. If underlying conditions allow, BFCs can be designed to both exhibit the water quality benefits of wetland-like conditions, but have the additional surface flows caused by subsurface saturation be re-converted to seepage before the BFC discharges to receiving waters.

The number of cells used in BFCs is somewhat flexible, and has a clear connection to performance. This research suggests a minimum of three sequential cells with slopes less than 5%. The first cell is a de-facto forebay (collects larger sediment, but very little nutrient processing). However, there is potential for nutrient processing in the subsequent two or more cells. Despite slight increases in nutrient concentrations, the third cell in the Alamance BFC reduced overall pollutant loadings because of its ability to reduce surface runoff, and, therefore, was valuable to this BFC's performance. It should be noted that sediment accumulation was observed in the first cell resulting in some loss of storage. This is likely to continue over the life of the BFC, unless addressed by regular maintenance. Subsequent cells would also provide a safety factor for maintaining BFC hydrologic and water quality performance as the BFC changes with age, particularly if the BFC is not properly maintained.

Several important design implications arise from this study. The exfiltration trenches used to measure exfiltration rates in the media also promoted exfiltration that would not have otherwise occurred. When groundwater levels were below the entire exfiltration trench, exfiltration accounted for up to 53% of a storm event's runoff due to the parent soil's high hydraulic conductivity (5 to 45 mm/hr). Even in non-sandy parent soils where exfiltration rates may not be high, the exfiltration trenches should enhance volume reduction. Brown and Hunt (2011) also showed an increase in exfiltration volume and rates from bioretention cells when water is retained in the media via internal water storage (IWS), further supporting the volume reduction benefits of water detention design, such as exfiltration trenches, in BFC design.

Conclusion

Biofiltration Conveyance effectively converts surface flow to a cognate of shallow interflow, with the potential to achieve further volume reductions via evapotranspiration and exfiltration. In these case studies, BFCs were able to provide hydrologic stormwater mitigation through surface volume reduction, peak flow mitigation, and non-erosive flow velocities. Furthermore, the Alamance BFC was able to significantly reduce volume and peak flow, as well as provide opportunity to mimic the predevelopment hydrographs and flow pathways despite the challenge of being located in HSG D soils. Additionally, the Alamance BFC provided physical pollutant removal, leading to significant reductions in surface flow TSS, TKN, and TP concentrations, similar to or greater than that observed by other media-based systems in tight soils (Hunt et al., 2006; Passeport et al. 2009; Winston et al., 2012). Further potential nutrient removal exists if a plant community is established. With careful consideration of design goals and objectives, BFCs can be a valuable tool for managing the detrimental effects of stormwater runoff from urban and suburban development.

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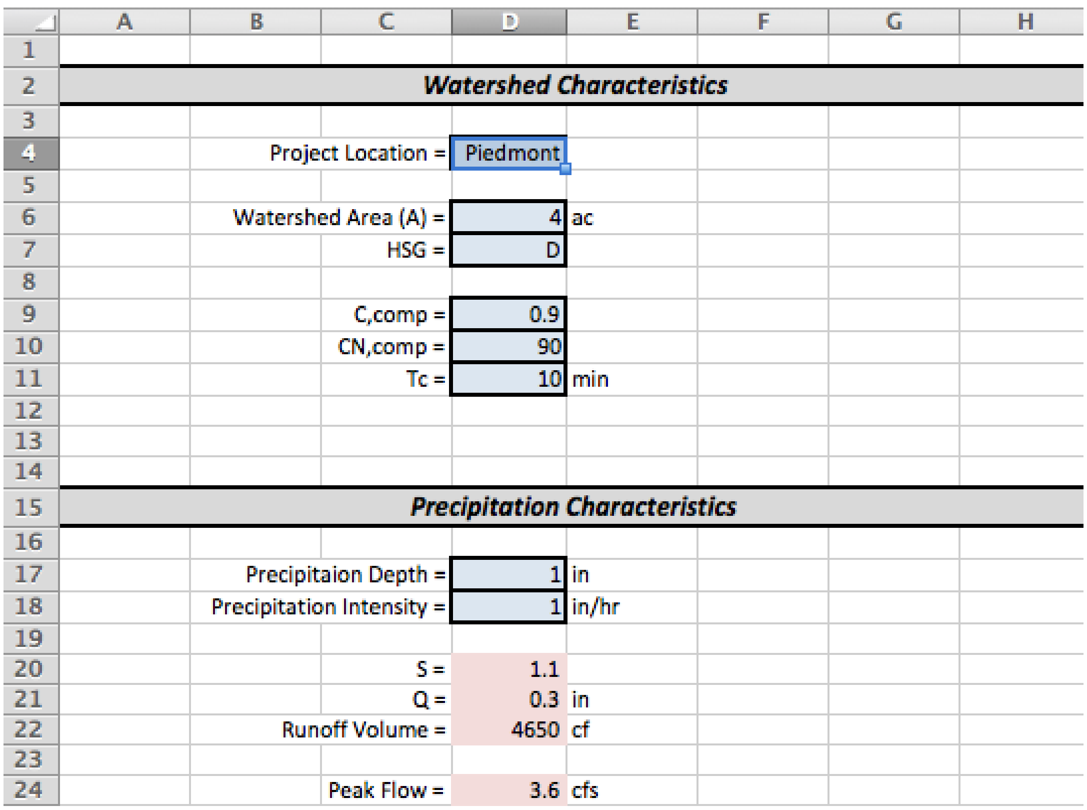
Appendix

A. Appendix: Introduction and User Manual for BFC Hydrology Performance Model

Using the Model

a. “Watershed” Tab (blue)

Selection of the blue tab labeled “Watershed” should result in the following screen, with varying values in the cells (Figure 12). Blue cells are model user inputs. Red cells are determined based on the inputs and may be useful to know. The model user should not adjust red cells.



	A	B	C	D	E	F	G	H
1								
2				Watershed Characteristics				
3								
4			Project Location =	Piedmont				
5								
6			Watershed Area (A) =	4	ac			
7			HSG =	D				
8								
9			C,comp =	0.9				
10			CN,comp =	90				
11			Tc =	10	min			
12								
13								
14								
15				Precipitation Characteristics				
16								
17			Precipitaion Depth =	1	in			
18			Precipitation Intensity =	1	in/hr			
19								
20			S =	1.1				
21			Q =	0.3	in			
22			Runoff Volume =	4650	cf			
23								
24			Peak Flow =	3.6	cfs			

Figure 12 Screen shot of blue “Watershed” tab

i. Inputs

Watershed Characteristics

- *Project Location* – Model designed for sites in North Carolina, located in the Coastal Plains, Piedmont, or Mountain per Figure 13.

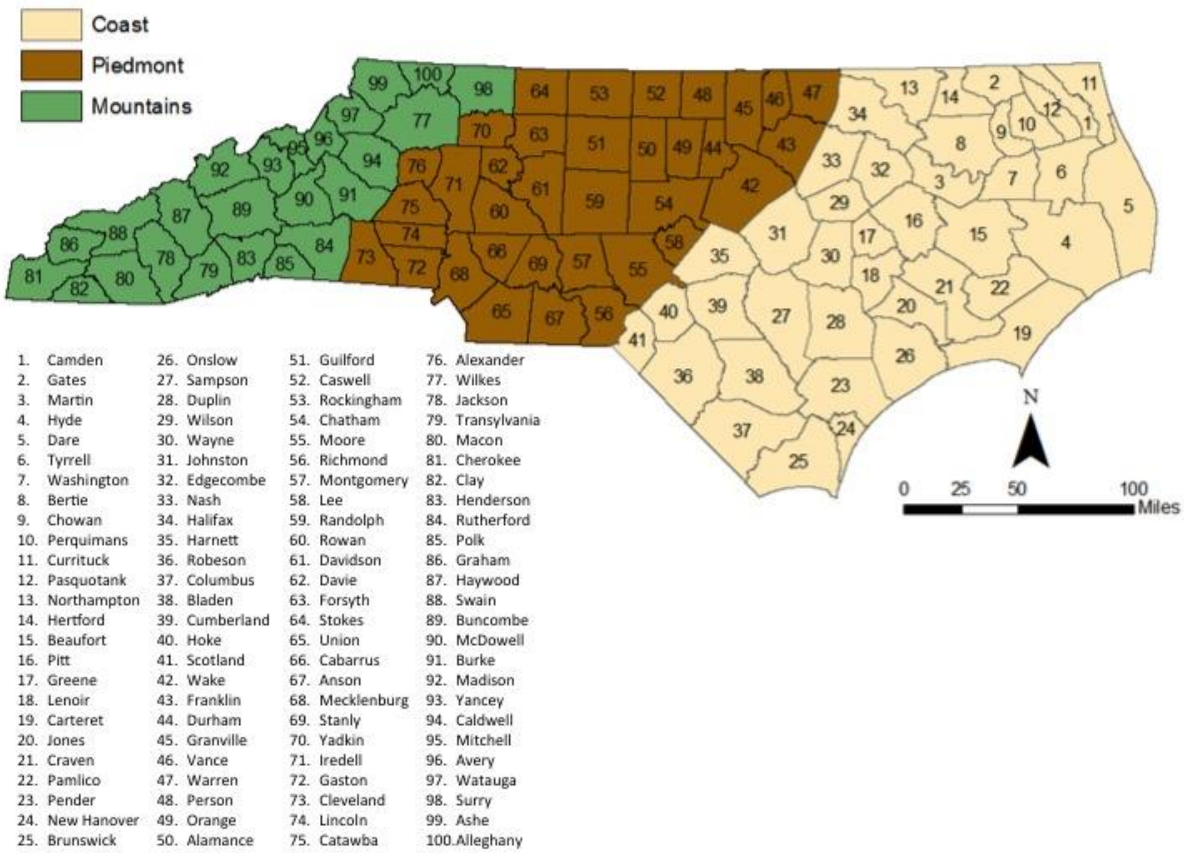


Figure 13 North Carolina counties by ecoregion

- *Watershed Area* – Total area of contributing watershed to the point where BFC will discharge, in acres
- *Hydrologic Soil Group (HSG)* – Classified as A, B, C, or D per descriptions in Table 7.
- *Curve Number (CN)* – 1 to 100 based on land use and soil characteristics calculated for the Discrete SCS Curve Number Number Method (Table 8). If

contributing watershed is composed of multiple land uses, a composite CN can be calculated by taking the weighted average CN, based on land use area.

- *Rational Runoff Coefficient (C)* – Based on land use determined for Rational Method (Table 9). If contributing watershed is composed of multiple land uses, a composite C can be calculated by taking the weighted average C, based on land use area.

Table 7 Four hydrologic soil groups, defined. (from NC DENR Stormwater BMP Manual)

Group A	A soils have low runoff potential and high infiltration rates even when thoroughly wetted. They consist chiefly of deep, well to excessively drained sand or gravel and have a high rate of water transmission (greater than 0.30 in/hr). The textures of these soils are typically sand, loamy sand, or sandy loam.
Group B	B soils have moderate infiltration rates when thoroughly wetted and consist chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission (0.15-0.30 in/hr). The textures of these soils are typically silt loam or loam.
Group C	C soils have low infiltration rates when thoroughly wetted and consist chiefly of soils with a layer that impedes downward movement of water and soils with moderately fine to fine texture. These soils have a low rate of water transmission (0.05-0.15 in/hr). The texture of these soils is typically sandy clay loam.
Group D	D soils have high runoff potential. They have very low infiltration rates when thoroughly wetted and consist chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very low rate of water transmission (0-0.05 in/hr). The textures of these soils are typically clay loam, silty clay loam, sandy clay, silty clay, or clay.

Table 8 Runoff curve numbers (CN) in urban areas from NC DENR Stormwater BMP Manual

Cover Description	Curve Numbers for Hydrologic Soil Group			
Fully developed urban areas	A	B	C	D
Open Space (lawns, parks, golf courses, etc.)				
Poor condition (< 50% grass cover)	68	79	86	89
Fair condition (50% to 75% grass cover)	49	69	79	84
Good condition (> 75% grass cover)	39	61	74	80
Impervious areas:				
Paved parking lots, roofs, driveways, etc.	98	98	98	98
Streets and roads:				
Paved; curbs and storm sewers	98	98	98	98
Paved; open ditches	83	89	98	98
Gravel	76	85	89	91
Dirt	72	82	85	88
Developing urban areas				
Newly graded areas	77	86	91	94
Pasture (< 50% ground cover or heavily grazed)	68	79	86	89
Pasture (50% to 75% ground cover or not heavily grazed)	49	69	79	84
Pasture (>75% ground cover or lightly grazed)	39	61	74	80
Meadow – continuous grass, protected from grazing and generally mowed for hay	30	58	71	78
Brush (< 50% ground cover)	48	67	77	83
Brush (50% to 75% ground cover)	35	56	70	77
Brush (>75% ground cover)	30	48	65	73
Woods (Forest litter, small trees, and brush destroyed by heavy grazing or regular burning)	45	66	77	83
Woods (Woods are grazed but not burned, and some forest litter covers the soil)	36	60	73	79
Woods (Woods are protected from grazing, and litter and brush adequately cover the soil)	30	55	70	77

Table 9 Rational runoff coefficients from NC DENR Stormwater BMP Manual

Description of Surface	Rational Runoff Coefficients, C
Unimproved Areas	0.35
Asphalt	0.95
Concrete	0.95
Brick	0.85
Roofs, inclined	1.00
Roofs, flat	0.90
Lawns, sandy soil, flat (<2%)	0.10
Lawns, sandy soil, average (2-7%)	0.15
Lawns, sandy soil, steep (>7%)	0.20
Lawns, heavy soil, flat (<2%)	0.15
Lawns, heavy soil, average (2-5%)	0.20
Lawns, heavy soil, steep (>7%)	0.30
Wooded areas	0.15

Precipitation Characteristics

- *Design Precipitation Depth (P)* – Based on location and ARI, inches
- *Design Precipitation Intensity (I)* – Based on location and ARI, inches per hour
- *Time of Concentration (Tc)* – Calculated for the contributing watershed using any regulation approved method. Some examples of these include
    - *US Federal Aviation Administration equation*

      $$t = 1.8\ (1.1 - C)\ L^{0.5} / (100\ S)^{1/3}$$

    - *Kirpich equation*
        - $t = 0.0078\ k\ (L / S^{0.5})^{0.77}$
    - *Kerby equation*

      $$t = 0.8268\ (L\ r / S^{0.5})^{0.467}$$

where C = is the runoff coefficient from the Rational Method (as determined in “Watershed Characteristics”

L = longest watercourse length (ft)

S = average slope of watercourse, ft/ft

t = time of concentration, min

k = Kirpich adjustment factor (Table 10)

r = Kerby retardance roughness coefficient (Table 11)

Table 10 Kirpich adjustment factors for different ground covers

Ground Cover	Kirpich Adjustment Factor, k (Chow et al., 1988; Chin, 2000)
General overland flow and natural grass channels	2.0
Overland flow on bare soil or roadside ditches	1.0
Overland flow on concrete or asphalt surfaces	0.4
Flow in concrete channels	0.2

Table 11 Kerby retardance roughness coefficients for different ground covers

Ground Cover	Kerby Retardance Coefficient, r (Chin, 2000)
Conifer timberland, dense grass	0.80
Deciduous timberland	0.60
Average grass	0.40
Poor grass, bare sod	0.30
Smooth bare packed soil, free of stones	0.10
Smooth pavements	0.02

ii. Outputs

- *S* – potential maximum retention after rainfall begins, in inches, calculated by

  $$S = 1000/CN - 10$$

- *Q* – Runoff volume from the contributing watershed, in inches, calculated by

  $$Q = (P - 0.2S)^2 / (P + 0.8S)$$

  where P is rainfall depth (in)

- *Runoff Volume* – Runoff volume from the contributing watershed converted from inches to cubic feet
- *Peak Flow* – Maximum design discharge for the storm event, in cfs, calculated by

  $$Q = C*I*A$$

  where C is the composite rational runoff coefficient for the watershed, calculated above

I is rainfall intensity (in/hr) for the design storm

A is the area of the watershed (ac)

b. "BFC Design" Tab (orange)

Pool Design Characteristics

	Pool 1	Pool 2	Pool 3	Pool 4	Pool 5
Length of Pool (lp), ft =	30	20	20	20	20
Width of Pool (wp), ft =	15	15	15	15	15
Depth of Pool (dp), ft =	1.5	1.5	1.5	1.5	1.5
Depth of Cobble Layer (dc), ft =	1.5	1.5	1.5	1.5	1.5
Depth of Sand Media Layer (dm), ft =	2	2	2	2	2
Width of Meida Bed Base (bw), ft =	8	8	8	8	8
Width of Cobble Base (cbw), ft =	12	12	12	12	12
Width of Pool Bed (pbw), ft =	9	9	9	9	9
Length of Media Bed Base (bl), ft =	30	20	20	20	20
Length of Cobble Bed Base (cbl), ft =	21	11	11	11	11
Length of Pool Base (pbl), ft =	21	11	11	11	11
Total Depth, ft =	5	5	5	5	5

Riffle Design Characteristics

	Riffle 1	Riffle 2	Riffle 3	Riffle 4	Riffle 5
Length of Riffle (rl), ft =	8	8	8	8	8
Wdith of Riffle (rw), ft =	15	15	15	15	15
Depth of Riffle (rd), ft =	1	1	1	1	1
Total Elev Drop over Riffle (rh), ft =	0.5	0.5	0.5	0.5	0.5
Riffle Slope =	16	16	16	16	16
	6%	6%	6%	6%	6%
Parabolic Riffle Coefficient (a) =	14.0625	14.0625	14.0625	14.0625	14.0625

Planting Characteristics

		K
Species Water Requirements	Mod	0.5
Planting Density	High	1.2
Microclimate	High	1.25
	KL	0.75

* Microclimate - increased (high) or decreased (low) ET conditions from open field (mod), i.e. sunny/ pavement = high

Figure 14 Screen shot of BFC design parameters on orange "BFC Design" tab

Selection of the orange tab labeled “BFC Design” should result in the left side of the screen as shown in Figure 14, with varying values in the cells. Blue cells are model user inputs. Red cells are determined based on the inputs and may be useful to know. The model user should not adjust red cells.

i. Inputs

The BFC design for this model allows for five consecutive cells (pool/riffles). Fixed design parameters include side slopes of pools and media bed, as depicted in Figure 15. Input dimensions for each component, in feet, are described below

Pool Design Characteristics

- *Length (lp)*
- *Width (wp)*
- *Depth (dp)*
- *Depth of Cobble Layer (dc)*
- *Depth of Sand Media Layer (dm)*

Riffle Design Characteristics

- *Length (lr)*
- *Width (wr)*
- *Depth (dr)*
- *Drop (hr)*

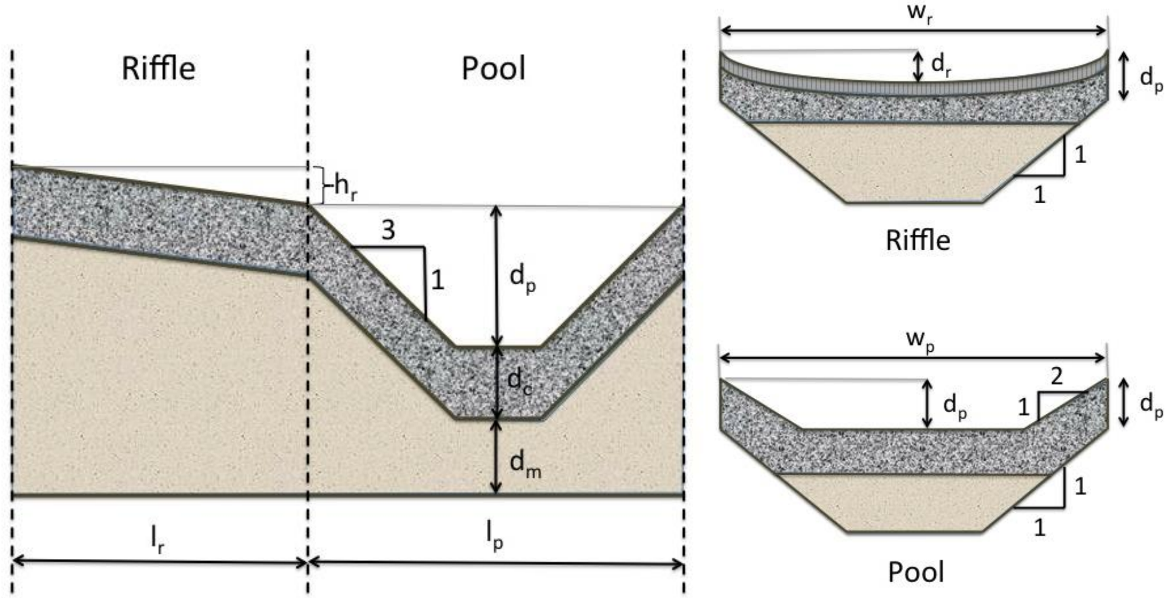


Figure 15 BFC cell and associated dimensions

Planting Characteristics

Planting characteristics are important for determining evapotranspiration within the BFC. Evapotranspiration, in this model, is calculated using the ETc formula, described as

$$ETc = (Kc)(ET_o)$$

where ETc is the evapotranspiration attributed to that specific crop

Kc is the crop coefficient associated with the specific crop

ET_o is a reference evapotranspiration value given for the location

When groundcover is not planted as a monocrop, this equation can still be used by replacing Kc with a landscape coefficient (K_L) (Costello et al., 2000), described by

$$K_L = (Ks)(Kd)(Kmc)$$

This model requires inputs for Ks, Kd, and Kmc as described below

- *Vegetation water requirements (K_s)* – dependent on the plant species and region planted, a good rule of thumb is using species irrigation requirements (Costello and Jones, 1999), such that
  - Low = irrigation requirements are < 30% that of ET_o
  - Moderate = irrigation requirements are 40% to 60% that of ET_o
  - High = irrigation requirements are 70% to 90% that of ET_o
- *Planting diversity (K_d)* – Defined by
  - Low = Immature, sparse cover
  - Moderate = full cover, but primarily one species
  - High = mixed species, trees, shrubs, and groundcovers
- *Microclimate (K_{mc})* – Defined by
  - Low = Shaded, protected from winds
  - Moderate = Similar to open-field setting
  - High = Around features which increase ET rates, such as median or parking lots
- Output values associated with each category (Low, Mod, High) are determined based on Costello et al. (2000).

c. Model Output

The model output, based on input parameters, is located on the right-hand side of the "BFC Design" tab (Figure 16).

- *Hydrographs* – The modeled inflow and cell outflow surface hydrographs are plotted using the Malcom Method (see Model Calculations below). This is a commonly used metric of SCM hydrologic performance, and illustrates both peak flow and runoff volume reductions.

- *Runoff Fate* – A pie chart associated with each cell shows the end fate of the runoff. Runoff can leave the BFC as surface flow, subsurface seepage, evapotranspiration, or exfiltration into parent soils.

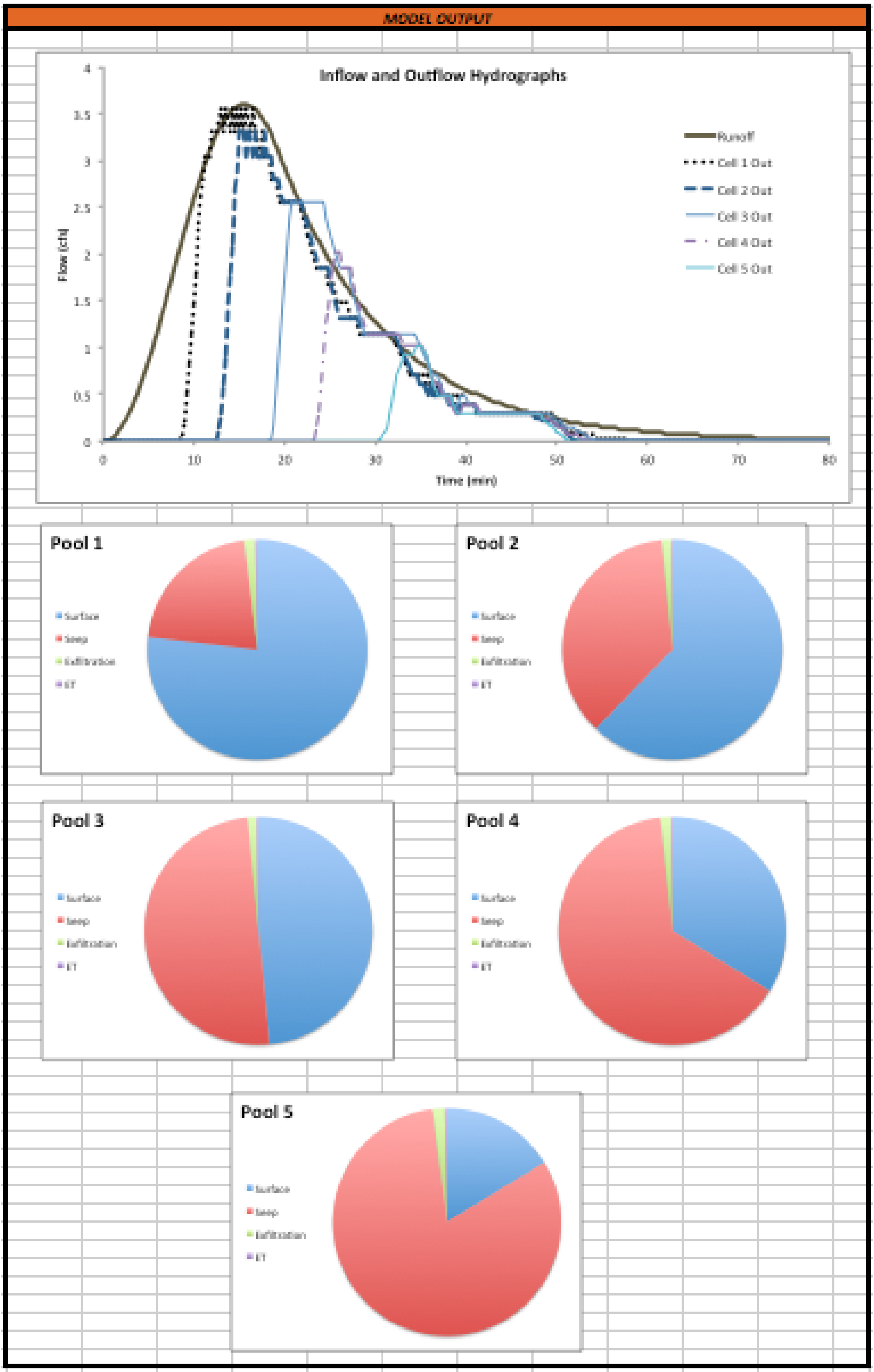


Figure 16 Screen shot of “Model Output” section on “BFC Design” tab

Model Calculations

a. Stage-Storage Tables

Stage-storage tables for the BFC cell (black tab, Cell # S-S), including the pore space within the media were created for each BFC cell within the design (up to 5 cells in series). These tables were based on BFC cell geometry provided in the inputs and standard side slopes. Porosities for the cobble and sand media were set as 0.25 and 0.3, respectively, based on Figure 6.

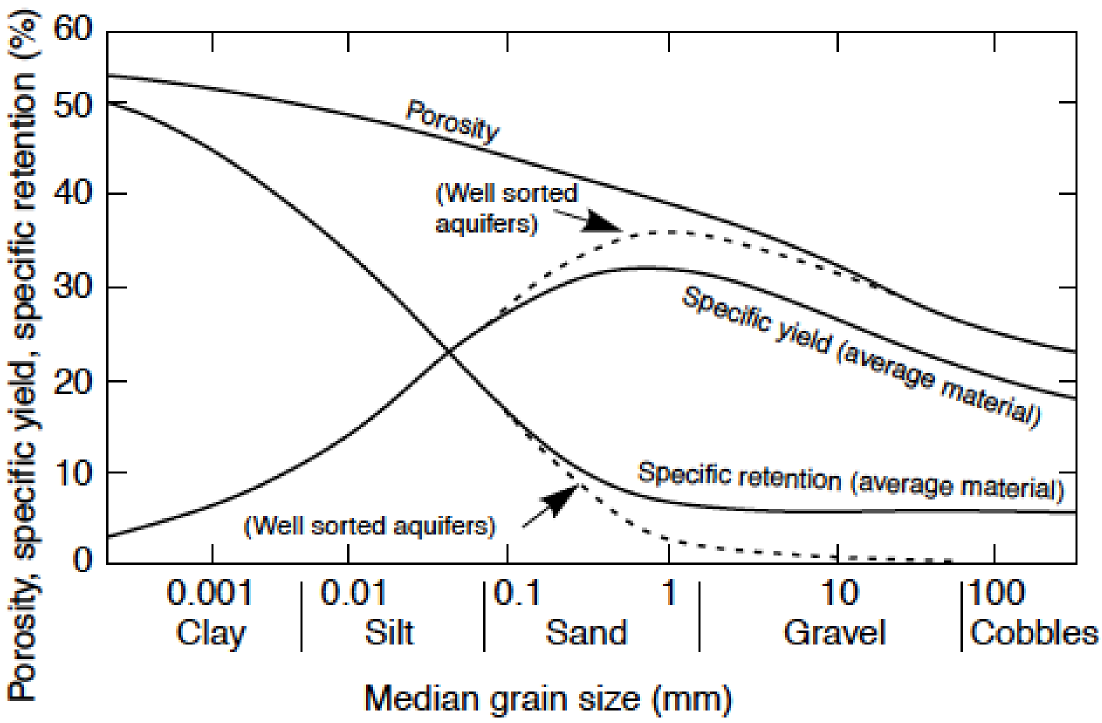


Figure 17 Porosity of media based on grain size (from Stephens et al., 1998)

Stage-storage tables were also created for the open pool space (black tab, Cell # Surf S-S) to aid in surface accumulation (prior to infiltration), based, again, on pool geometry provided by inputs.

b. Hydrographs

A runoff inflow hydrograph is estimated using Malcom's Method as described by Malcom (1995), given the design storm characteristic put into the model. Surface outflow from each pool is calculated using an empirical model for flow over parabolic weirs, as described by Sommerfeld and Stallybrass (1996), as follows

$$Q = C_0 \frac{\pi h^2 \sqrt{2ag}}{2}$$

where, C_0 is a discharge coefficient to account for friction loss over the weir, 0.6 for circular weirs and used in these calculations as well

h is the driving head over the center of the weir

g is the acceleration due to gravity (32.2 ft/s)

a is a measure of flatness of the parabola, such that

$$x^2 = 4ay$$

For the BFC cell described, a is calculated as follows

$$a = \frac{w_r^2}{16d_r}$$

Using the above equations, flow rates are calculated at time increments of 0.25 minutes for the first 120 minutes, and then 10 minutes for the remaining time interval.

c. Water Balance

Once runoff enters the system, it can leave as surface flow, subsurface seepage, evapotranspiration, and exfiltration into the parent soil.

i. Surface flow

Surface flows were calculated at 0.25-min increments for the first 120 min, and then at 10-min intervals for the remaining time span by balancing the *in's* and *out's*, such that

$$S_{pool,t} = S_{pool,t-\Delta t} + Q_{in,t}\,\Delta t - Q_{out}\,\Delta t - \mathit{Infil}$$

where, $S_{pool,t}$ is the water stored in the pool at time t or t- Δt

Δt is equal to the time increment between the current time and the previous time

$Q_{in,t}$ is the inflow at time t

$Q_{out,t}$ is the outflow from the pool at time t

Infil is the water infiltrated into the BFC media

The infiltration rate into the sand media is a function of the stage in the pool, and is described by a regression model of well data at a BFC in Alamance County (Figure 18), such that

$$K_{sand} = 0.167*h-0.2093 \text{ [=] ft/hr}$$

where, k is the instantaneous infiltration rate at stage h

From the infiltration rate, we can calculate the volume infiltrated during time interval t as

$$\textit{infil} = (K_{sand})(SA)\ \Delta t$$

It is important to note that the model will not allow infiltration beyond what is in storage. Therefore, if there is a volume stored in the pool at time t that is less than what is calculated to be infiltrated, the model will only allow what is stored to be infiltrated. The inflow to the cell is either from the Malcom inflow hydrograph or the outflow from the preceding cell.

ii. Exfiltration

Once the media is filled, BFCs are expected to promote exfiltration into parent soils, hence reducing overall runoff volumes. A lag is expected between when water begins infiltrating into the media and when the water has contact with the bottom of the system. This lag is estimated as depth of the media bed divided by the infiltration rate at the interface of the media. Once the water reaches the bottom of the cell, exfiltration is estimated to occur at a constant rate determined by the parent soil based on the HSG.

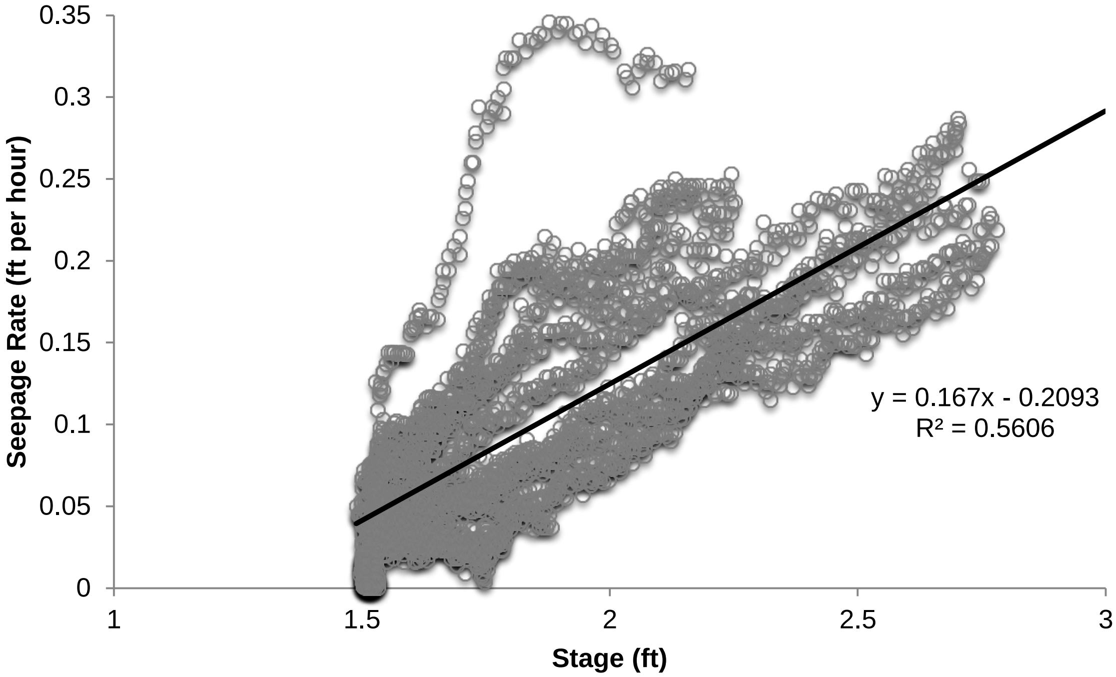


Figure 18 Regression analysis of summer 2013 well data within sand media at Alamance BFC

iii. Evapotranspiration

Evapotranspiration (ET) is estimated using the Penman Monteith reference ET values (ET_0). Average daily reference values were calculated for a station in each of NC's eco-regions (Coastal Plain – Wilmington, Piedmont – Raleigh, Mountains – Boone) using monthly measured data from February 2010. The landscape coefficient is calculated based on the amount of water the plant species chosen use, density and diversity of plantings, and the surrounding microclimate (described in inputs). The final landscape coefficient (K_L) is calculated as

$$K_L = K_s K_d K_{mc}$$

Therefore, the ET for each time interval is

$$ET_t = K_L ET_0 SA$$

where SA is the surface area of the cell

iv. Seepage

Once water enters the media, it is expected to preferentially flow along the cobble/sand interface. The sand has a smaller infiltration rate, so the same media infiltration rate as that for infiltration into the sand media bed is not used for seepage. Instead the infiltration rate (K_{cob}) is calculated from regression analysis of well data water levels in the cobble layer from the Alamance BFC (Figure 18), such that

$$K_{cob} = 15.387h - 31.124 \text{ [=] ft/hr}$$

Darcy's Law is then used to calculate the horizontal seepage flux as follows

$$q_{seep} = K\frac{dh}{dL} = K_{cob}\frac{h_r}{l_r + l_p}$$

It is assumed that the media interface generally follows the slope of the surface, and therefore, the overall dh is estimated by the drop between the inlet of a cell and the outlet of the cell. The flux of seepage is through the vertical cross-section, which is calculated using by the geometry of the cell at the riffle times the porosity of the media. As with the infiltration and exfiltration calculations, seepage will not occur unless the appropriate storage is available. Additionally, seepage may enter the system if the cell is not the first cell in the series.

v. Overall Water Balance

An overall water balance for each cell, and subsequently a series of cells using the following water balance

Surface IN = Surface OUT + Exfiltration + ET + Seepage

Using the above water balance, system performance can be hydrologically evaluated.

d. Model Assumptions

There are many assumptions made in the design of this model. Important assumptions include

- Media bed is longitudinally smooth, with a constant slope for the duration of BFC length
- Precipitation into each cell is negligible
- Once water enters media, flux is primarily downward at a constant rate until the next media interface (i.e. cobble/sand interface and sand/parent soil interface)
- Media drains completely between storm events

Example Model Validation

The model output is compared to an actual storm event on July 14, 2013 at the Alamance County BFC. Based on the model assumptions, only the first three cells were assessed (Figure 19, Figure 20). The fourth cell at the site is designed to store water in its subsurface, which was not considered in the model. Model inputs and outputs are compared to the actual system performance (Table 12).

The model does not simulate actual flow through this BFC system particularly well, although there are several good signs. The model shows the predominant flow through the system as surface flow, where the monitored site shows predominantly seepage. This may be a result of the monitoring equipment – a compound v-notch/broad crested weir, which allows much less surface flow through than the full width boulder parabolic weir. With less water leaving each cell as surface, there is more ponding and, therefore, more infiltration. It will be necessary to monitor BFCs at the inlet and outlet, without internal monitoring weirs to confirm this theory. The accuracy of the models predictions at this site may reveal a more validated model. Additionally, the model appears to predict ET and exfiltration well. ET is such a small piece of the water budget that this milestone may not be overly significant for performance predictions. The close representation of exfiltration is encouraging, as this is a predominant way of reducing outflow volume, and therefore, of high interest to engineers and regulators. Future adjustments and calibration of the model will with no doubt produce more accurate results.

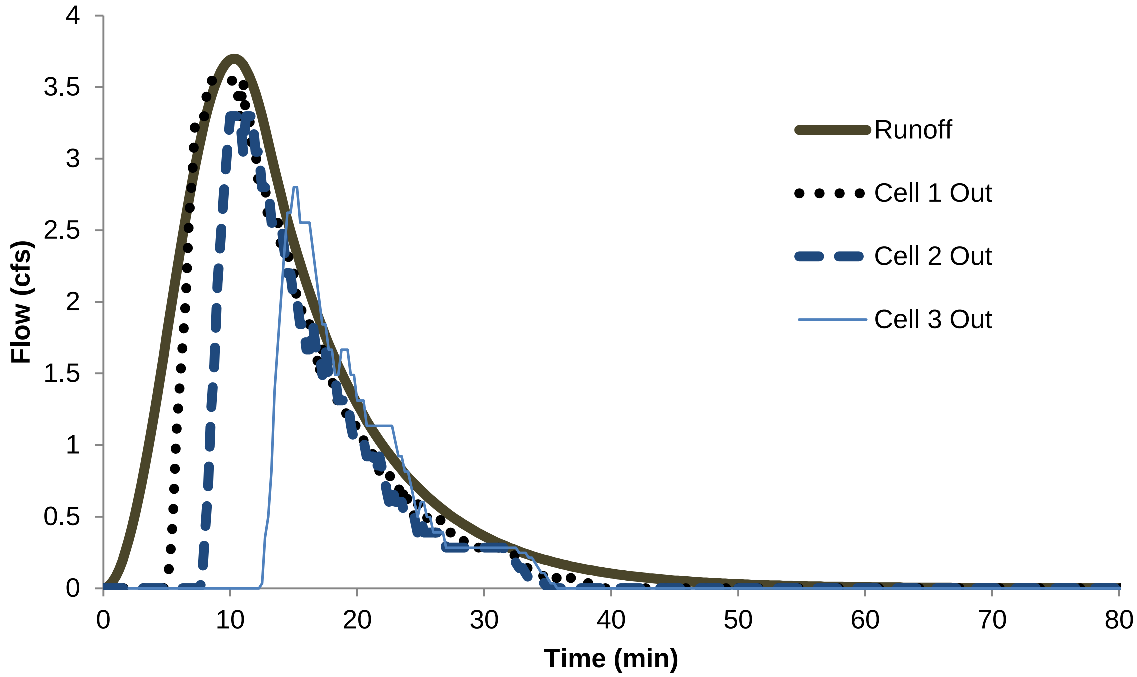


Figure 19 Hydrograph output from modeled July 14 storm event at Alamance BFC

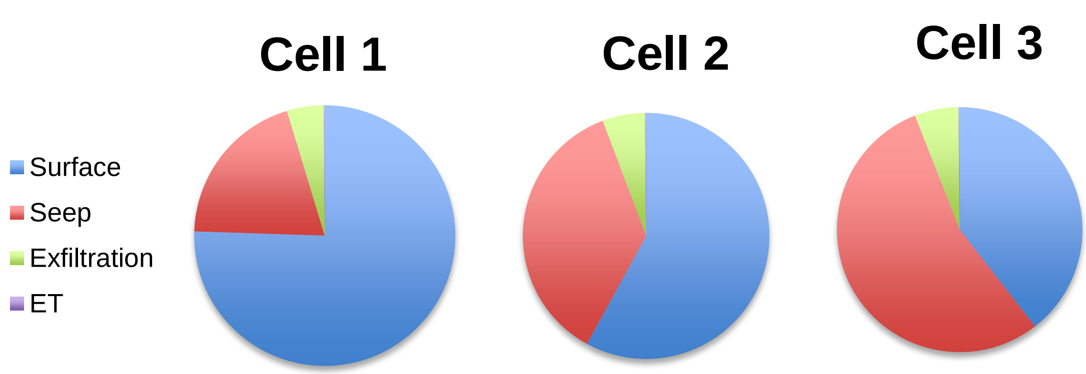


Figure 20 Runoff fate from each cell in series

Table 12 Model inputs and outputs for July 14, 2013 storm at Alamance BFC

Model Inputs

Runoff Volume =	3178	cf	
Peak Flow =	3.67	cfs	

	Pool 1	Pool 2	Pool 3
Length of Pool (lp), ft =	19	20.5	21.75
Width of Pool (wp), ft =	15	15	15
Depth of Pool (dp), ft =	1.5	1.5	1.5
Depth of Cobble Layer (dc), ft =	1.5	1.5	1.5
Depth of Sand Media Layer (dm), ft =	3	3	3

	Riffle 1	Riffle 2	Riffle 3
Length of Riffle (lr), ft =	8	8	8
Wdith of Riffle (wr), ft =	15	15	15
Depth of Riffle (dr), ft =	1	1	1
Total Elev Drop over Riffle (hr), ft =	0.5	0.25	0.5

Water Balance

		Monitored	Modeled
Pool 1			
Surface	*In*	3189	3189
	Out	857	2527
Seepage	*In*	0	0
	Out	2191	622
Exfiltration	*Out*	140	152
ET	*Out*	1	4
Pool 2			
Surface	*In*	857	2527
	Out	761	1918
Seepage	*In*	2191	662
	Out	2172	1201
Exfiltration	*Out*	133	184
ET	*Out*	1	5
Pool 3			
Surface	*In*	761	1918
	Out	186	1310
Seepage	*In*	2172	1201
	Out	2616	1810
Exfiltration	*Out*	149	192
ET	*Out*	1	6

Model Implications for BFC Design and Performance

A BFC was designed for a standard one-acre watershed (C = 0.65, CN = 78, HSG C), and its performance was compared for different design scenarios – P = 0.25 inch, P = 1 inch, and P = 2 in (Table 13). As expected, there was very little surface flow exiting the first pool during the smallest storm, and the proportion of surface flow gradually increased as the storm size increased. There was not any surface flow exiting the BFC system for either the 0.25-in and 1-in storm events, indicating that the system can very adequately mitigate the water quality event for the Piedmont.

Table 13 Model outputs for BFC designed for 1-acre watershed during different storm events

	Cell 1					
	P = 0.25 inch		P = 1.0 inch		P = 2.0 inch	
	In	Out	In	Out	In	Out
Surface	143	47	213	109	1758	1527
Seepage		93		100		229
Exfiltration		41		42		95
ET		4		4		4
	Total System (3 Cells)					
Surface	143	0	213	0	1758	1166
Seepage		136		203		596
Exfiltration		72		98		228
ET		15		15		15

BFC design parameters were adjusted on the same hypothetical BFC in the same hypothetical watershed under a 1-inch storm event (Table 14). When the depth of the pool was decreased, but the pool dimensions were adjusted such the storage volume was not affected, very little change in surface flow occurs. However, if the pool depth is increased and other dimensions maintained (not shown), then the ponding volume increases, decreasing surface flow and increasing seepage and exfiltration. Increasing media depth in bioretention cells have been shown to increase exfiltration by holding water within the system in contact with parent soil for a longer period of time. When the media depth is increased in a BFC, exfiltration does not appear to increase, and may decrease. However, seepage appears to increases with media depth. This makes more sense for a BFC, as these are conveyance systems, intending to move water. Moreover, seepage is managed, treated, slow release stormwater. When the slope of the riffle decreases, the hydraulic seepage rate (dh/dl) will decrease. This is why we see a decrease in seepage. The model also seems to struggle with lingering seepage and associated exfiltration (beyond 5 days) when seepage rates get too low. Also, increasing the number of cells in a series decreases surface flow and increases the other pathways.

Table 14 BFC design changes and associated fate of runoff

	In	Cell 1 - Out					
		Pool Depth		Media Depth		Riffle Slope	
		d_p = 1.5 ft, V_p = 79 cf	d_p = 1 ft, V_p = 80 cf	d_m = 2	d_m = 2.5	13% Slope	3% Slope
Surface	213	109	107	109	107	109	109
Seepage		100	105	100	105	100	97
Exfiltration		42	39	42	39	42	42
ET		4	4	4	4	4	4
	In	Total System (3 Cells) - Out					
Surface	213	0	0	0	0	0	0
Seepage		203	196	203	70+	203	96+
Exfiltration		98	93	98	97+	98	88+
ET		15	19	15	15	15	10

Conclusions

The “BFC Working Model” has room for much improvement as more data becomes available on BFC performance, but it already is a useful tool in estimating how system design factors may affect overall cell and system performance. General design recommendations include increasing pool volume and depth, increasing and media depth. Unfortunately, each of these recommendations is associated with an increase in cost. These design considerations must be balanced. In each of the simulations, three cells appeared adequate to manage the water quality event with greatly reduced surface flows. More work is needed in incorporating data for better quantifying seepage rates and subsurface flow (regressions are low). The model also needs validating from a variety of different BFC sites.

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B. Hydrologic Data from Alamance County BFC

Table 15 Water balance summary for inflow producing storms during monitoring period

			In m³	Cell 1 m³					Cell 2 m³					Cell 3 m³					Cell 4 m³				
Ss[1]	Date	Rain	Srf	RO	Srf Out	Seep	ET	Ex	RO	Srf Out	Seep	ET	Ex	RO	Srf Out	Seep	ET	Ex	RO	Srf Out	Seep	ET	Ex
1	7/10/13	28	218	10	8	220	0	0	3	39	192	0	0	3	24	210	0	0	9	61	171	1	10
1	7/14/13	12	90	4	78	15	0	0	1	13	82	0	0	1	8	88	0	0	8	17	71	1	10
1	7/21/13	4	9	1	0	10	0	0	0	0	11	0	0	0	0	11	0	0	1	0	0	0	11
1	7/25/13	15	139	5	116	28	0	0	1	50	95	0	0	2	24	123	0	0	5	41	115	0	1
1	7/27/13	17	127	6	129	15	0	0	2	50	106	0	0	2	24	146	0	0	6	45	138	0	5
1	7/28/13	5	52	1	36	16	0	0	0	13	40	0	0	0	5	48	0	0	1	21	35	0	2
1	8/6/13	14	173	5	65	114	0	0	1	32	148	0	0	2	18	163	0	0	5	33	146	1	10
1	8/17/13	16	43	6	30	19	0	1	1	7	43	0	1	2	10	40	0	1	5	33	21	0	1
1	8/19/13	6	129	2	90	40	0	0	1	74	57	0	1	1	35	95	0	1	2	29	97	0	6
1	9/1/13	81	657	29	537	149	0	0	8	181	513	0	0	10	175	529	0	0	26	234	507	0	6
2	9/20/13	6	59	2	41	20	0	0	1	4	58	0	0	1	5	57	0	0	2	0	61	0	3
2	9/21/13	13	35	5	36	3	0	0	1	26	14	0	0	2	17	25	0	0	4	24	15	6	0
2	10/7/13	13	144	5	91	58	0	0	1	19	131	0	0	2	20	131	0	0	4	19	155	0	7
2	10/10/13	13	142	5	123	24	0	0	1	53	95	0	0	2	31	119	0	0	4	40	110	0	4
2	11/1/13	10	115	3	96	23	0	0	1	17	102	0	0	1	16	104	0	0	3	19	117	0	6
2	11/12/13	3	27	1	0	28	0	0	0	0	28	0	0	0	0	28	0	0	1	0	28	0	1
2	11/15/13	3	13	1	10	4	0	0	0	0	14	0	0	0	0	15	0	0	1	0	17	0	1
2	11/17/13	4	14	1	13	2	0	0	0	1	14	0	0	0	0	15	0	0	1	0	29	0	6
2	11/26/13	42	587	15	436	166	0	1	4	66	539	0	1	5	41	569	0	1	14	70	567	1	7

[1] Ss = Season, 1 = Summer, 2 = Fall, 3 = Winter, 4 = Spring

Table 15 (continued)

			In m³	Cell 1 m³					Cell 2 m³					Cell 3 m³					Cell 4 m³				
Ss[1]	Date	Rain	Srf	RO	Srf Out	Seep	ET	Ex	RO	Srf Out	Seep	ET	Ex	RO	Srf Out	Seep	ET	Ex	RO	Srf Out	Seep	ET	Ex
3	12/7/13	9	89	3	73	18	0	0	1	29	63	0	0	1	13	80	0	0	3	6	84	0	5
3	12/9/13	6	45	2	44	3	0	0	1	3	44	0	0	1	1	47	0	0	2	1	46	0	3
3	12/14/13	17	89	3	73	18	0	0	1	29	63	0	0	1	13	80	0	0	3	6	84	0	5
3	12/22/13	34	502	12	333	180	0	1	3	103	413	0	1	4	41	478	0	1	11	25	495	0	10
3	1/10/14	42	582	30	508	103	0	1	8	436	182	0	1	10	230	398	0	1	27	313	333	1	8
3	1/14/14	9	77	6	48	35	0	0	2	27	57	0	0	2	12	74	0	0	6	24	60	0	8
3	2/4/14	10	96	3	81	19	0	0	1	31	68	0	0	1	12	89	0	1	3	20	80	0	3
3	2/12/14	24	326	9	66	268	0	1	2	56	280	0	1	3	25	313	0	1	8	10	330	0	5
3	2/21/14	14	158	5	142	20	0	0	1	68	95	0	0	2	28	137	0	0	5	40	118	0	10
3	3/3/14	12	121	4	93	33	0	0	1	26	99	0	0	1	10	117	0	0	4	9	114	0	7
3	3/6/14	62	910	22	754	178	0	0	6	401	537	0	0	8	170	775	0	0	20	275	686	0	4
4	3/16/14	13	131	5	52	84	0	0	1	13	123	0	0	2	3	135	0	0	4	4	129	0	8
4	3/17/14	26	366	9	113	262	0	0	2	52	325	0	0	3	16	364	0	0	8	22	359	0	8
4	3/18/14	6	44	2	4	42	0	0	1	0	47	0	0	1	0	48	0	0	2	0	50	0	0
4	3/23/14	5	24	2	16	9	0	0	0	5	20	0	0	1	1	25	0	0	1	0	21	0	5
4	3/28/14	7	47	2	28	21	0	1	1	5	44	0	1	1	0	49	0	1	2	0	40	1	10
4	4/7/14	40	554	14	412	156	0	1	4	303	268	0	0	5	127	336	0	1	13	127	341	0	8
4	4/15/14	26	341	10	195	155	0	1	3	136	215	0	0	3	56	298	0	0	9	56	299	0	8
4	4/18/14	13	153	5	101	56	0	1	1	18	139	0	0	2	4	155	0	0	4	6	148	0	8
4	5/10/14	7	54	3	41	14	0	1	1	23	33	0	0	1	7	49	0	0	2	4	46	0	8
4	5/15/14	54	777	19	488	307	0	1	5	378	422	0	0	7	156	650	0	0	17	170	646	0	8
4	5/27/14	20	144	7	15	136	0	0	2	4	149	0	0	2	0	155	0	0	6	0	158	0	4
1	6/10/14	20	240	7	156	90	0	0	2	53	196	0	0	2	18	233	0	0	6	49	201	0	7

[1] Ss = Season, 1 = Summer, 2 = Fall, 3 = Winter, 4 = Spring

C. Appendix: Water Quality Data for Alamance County BFC

Table 16 Data for each water quality event at the Inlet location

Ss[1]	Flow m^3	TN $mg\ L^{-1}$	TKN $mg\ L^{-1}$	NOx $mg\ L^{-1}$	NH_3 $mg\ L^{-1}$	TP $mg\ L^{-1}$	OP $mg\ L^{-1}$	TSS $mg\ L^{-1}$
1	218	2.15	1.94	0.21	0.11	0.33	0.22	25.71
1	90	1.59	1.39	0.20	0.14	0.31	0.10	42.33
1	752	1.84	1.63	0.21	0.32	0.36	0.23	8.82
1	657	2.40	2.17	0.23	0.31	0.64	0.49	69.14
2	59	2.93	2.55	0.37	0.88	0.36	0.25	26.25
2	144	6.79	5.86	0.93	2.66	2.14	1.65	71.16
2	142	1.85	1.55	0.29	0.36	0.44	0.29	74.87
2	115	3.95	3.08	0.87	0.75	0.66	0.50	86.79
2	587	5.45	4.89	0.56	1.94	0.76	0.53	44.22
3	582	1.84	1.61	0.23	0.13	0.58	0.29	97.43
3	77	1.68	1.45	0.23	0.31	0.28	0.15	41.22
3	158	4.28	4.07	0.21	0.35	0.71	0.13	296.60
3	121	9.96	9.31	0.65	1.75	0.88	0.62	90.08
3	910	2.68	2.35	0.33	0.77	0.39	0.20	52.58
4	47	5.44	5.16	0.27	1.45	0.55	0.19	81.46
4	554	1.63	1.41	0.23	0.27	0.37	0.13	72.50
4	341	4.47	4.30	0.18	0.39	0.61	0.22	103.08
4	153	ND	ND	ND	ND	ND	ND	ND
4	777	1.80	1.56	0.24	0.15	0.34	0.18	37.81
1	240	1.73	1.37	0.36	0.20	0.24	0.11	39.52

[1] Ss = Season, 1 = Summer, 2 = Fall, 3 = Winter, 4 = Spring

ND = no data

Table 17 Data for each water quality event at the Outlet 1 location

Ss[1]	Flow m^3	TN $mg L^{-1}$	TKN $mg L^{-1}$	NOx $mg L^{-1}$	NH_3 $mg L^{-1}$	TP $mg L^{-1}$	OP $mg L^{-1}$	TSS $mg L^{-1}$
1	8	2.03	1.79	0.25	0.11	0.41	0.30	10.51
1	78	3.24	2.63	0.61	0.89	0.47	0.28	20.22
1	320	1.58	1.36	0.23	0.21	0.43	0.34	10.36
1	537	3.27	2.97	0.30	0.38	0.54	0.35	11.28
2	41	1.38	1.13	0.25	0.31	0.24	0.17	14.18
2	91	0.84	0.35	0.49	1.81	1.52	1.28	68.06
2	123	1.89	1.50	0.39	0.34	0.53	0.39	59.11
2	96	5.64	5.41	0.23	1.41	0.86	0.68	46.95
2	436	3.59	3.04	0.54	1.00	0.55	0.43	19.04
3	508	2.58	2.17	0.41	0.55	0.44	0.16	75.42
3	48	1.64	1.39	0.25	0.34	0.28	0.17	23.73
3	142	4.26	4.03	0.23	0.33	0.75	0.14	261.10
3	93	8.10	7.49	0.61	1.85	0.80	0.59	64.59
3	754	2.68	2.40	0.28	0.67	0.42	0.23	60.65
4	28	2.28	2.03	0.25	0.77	0.30	0.13	29.64
4	412	2.14	1.90	0.24	0.32	0.35	0.17	55.56
4	195	3.34	3.19	0.15	0.35	0.53	0.21	76.99
4	101	1.89	1.71	0.18	0.21	0.23	0.13	18.54
4	488	1.55	1.36	0.19	0.15	0.31	0.18	28.30
1	156	1.76	1.33	0.43	0.15	0.27	0.13	29.21

[1] Ss = Season, 1 = Summer, 2 = Fall, 3 = Winter, 4 = Spring

ND = no data

Table 18 Data for each water quality event at the Outlet 2 location

Ss[1]	Flow m^3	TN $mg\ L^{-1}$	TKN $mg\ L^{-1}$	NOx $mg\ L^{-1}$	NH_3 $mg\ L^{-1}$	TP $mg\ L^{-1}$	OP $mg\ L^{-1}$	TSS $mg\ L^{-1}$
1	39	1.74	1.55	0.18	0.10	0.40	0.31	6.70
1	13	1.82	1.58	0.24	0.41	0.33	0.18	21.28
1	239	1.54	1.30	0.24	0.25	0.44	0.34	7.69
1	181	1.80	1.49	0.31	0.18	0.60	0.49	9.85
2	4	0.72	0.58	0.14	0.15	0.16	0.13	10.12
2	19	3.21	2.81	0.40	1.47	1.38	1.07	91.51
2	53	1.39	1.06	0.34	0.28	0.45	0.33	54.23
2	17	1.54	1.28	0.27	0.08	0.52	0.39	33.20
2	66	1.71	1.40	0.31	0.22	0.47	0.36	30.66
3	436	1.73	1.53	0.20	0.12	0.51	0.28	83.79
3	27	1.16	0.98	0.18	0.24	0.25	0.13	25.12
3	68	4.13	3.93	0.20	0.32	0.68	0.14	209.81
3	26	5.46	4.95	0.51	1.35	0.53	0.39	72.24
3	401	2.18	1.90	0.27	0.85	0.42	0.25	51.50
4	5	2.35	2.03	0.32	0.60	0.38	0.18	76.46
4	303	1.39	1.22	0.17	0.09	0.28	0.12	48.92
4	136	1.61	1.46	0.16	0.08	0.31	0.14	41.73
4	18	1.11	0.99	0.13	0.10	0.17	0.09	13.39
4	378	1.36	1.21	0.15	0.08	0.27	0.16	34.76
1	53	1.73	1.38	0.36	0.21	0.25	0.13	51.68

[1] Ss = Season, 1 = Summer, 2 = Fall, 3 = Winter, 4 = Spring

ND = no data

Table 19 Data for each water quality event at the Outlet 3 location

Ss[1]	Flow m^3	TN mg L^{-1}	TKN mg L^{-1}	NOx mg L^{-1}	NH_3 mg L^{-1}	TP mg L^{-1}	OP mg L^{-1}	TSS mg L^{-1}
1	24	5.64	5.16	0.47	0.39	0.61	ND	ND
1	8	6.88	6.45	0.43	3.41	0.66	0.47	ND
1	125	ND	ND	ND	ND	ND	ND	ND
1	175	1.89	1.56	0.33	0.20	0.64	0.00	10.74
2	5	1.13	0.90	0.22	0.24	0.20	0.15	13.70
2	20	3.43	2.90	0.53	1.79	1.53	1.15	48.40
2	31	1.54	1.13	0.40	0.29	0.49	0.38	29.66
2	16	1.89	1.58	0.31	0.24	0.55	0.43	21.67
2	41	1.28	1.03	0.25	0.21	0.37	0.30	22.38
3	230	2.20	1.96	0.24	0.30	0.44	0.28	75.10
3	12	1.11	0.90	0.21	0.24	0.25	0.16	13.64
3	28	4.03	3.79	0.24	0.27	0.51	0.14	84.15
3	10	5.16	4.65	0.51	1.05	0.49	0.40	32.52
3	170	1.96	1.68	0.28	0.93	0.37	0.25	19.02
4	0	ND	ND	ND	ND	ND	ND	ND
4	127	1.44	1.24	0.20	0.17	0.26	0.13	38.20
4	56	1.94	1.71	0.22	0.06	0.37	0.13	37.82
4	4	1.07	0.88	0.19	0.13	0.17	0.10	11.08
4	156	1.34	1.10	0.24	0.10	0.25	0.17	17.44
1	18	1.97	1.38	0.59	0.24	0.28	0.22	14.87

[1] Ss = Season, 1 = Summer, 2 = Fall, 3 = Winter, 4 = Spring

ND = no data

Table 20 Data for each water quality event at the Outlet 4 location

Ss[1]	Flow m^3	TN $mg\ L^{-1}$	TKN $mg\ L^{-1}$	NOx $mg\ L^{-1}$	NH_3 $mg\ L^{-1}$	TP $mg\ L^{-1}$	OP $mg\ L^{-1}$	TSS $mg\ L^{-1}$
1	61	2.24	1.83	0.41	0.31	0.34	0.19	7.57
1	17	1.77	1.58	0.19	0.39	0.27	0.10	13.80
1	222	ND	ND	ND	ND	ND	0.26	10.14
1	234	1.76	1.46	0.29	0.20	0.54	0.43	9.98
2	0	1.68	1.43	0.24	0.25	0.18	0.08	7.70
2	19	3.85	3.07	0.78	1.35	1.23	0.99	48.19
2	40	1.41	0.95	0.46	0.25	0.41	0.28	29.37
2	19	2.08	1.56	0.52	0.14	0.45	0.33	14.31
2	70	1.44	1.13	0.31	0.13	0.39	0.31	9.73
3	313	1.20	0.99	0.22	0.12	0.37	0.26	27.26
3	24	1.05	0.86	0.19	0.17	0.20	0.13	10.21
3	40	3.19	2.97	0.23	0.24	0.41	0.13	68.50
3	9	ND	ND	ND	ND	ND	ND	ND
3	275	1.80	1.52	0.28	0.78	0.32	0.23	8.88
4	0	ND	ND	ND	ND	ND	ND	ND
4	127	1.39	1.14	0.25	0.12	0.27	0.13	22.93
4	56	2.06	1.70	0.36	0.08	0.35	0.17	21.72
4	6	1.30	0.98	0.32	0.19	0.18	0.10	8.88
4	170	1.49	1.21	0.28	0.13	0.25	0.16	12.90
1	49	1.95	1.29	0.66	0.27	0.27	0.18	9.45

[1] Ss = Season, 1 = Summer, 2 = Fall, 3 = Winter, 4 = Spring

ND = no data

D. Appendix: R-code for BFC Hydrology and Water Quality Analysis

Hydrologic Analysis

```
#### Create Subsets ####
# Subset hydro by season
h2oSum = subset(R.Hydro, Season==1)
h2oFall = subset(R.Hydro, Season==2)
h2oWin = subset(R.Hydro, Season==3)
h2oSpr = subset(R.Hydro, Season==4)

# Subset hydro by cell
h2oC1 = subset(R.Hydro, Cell==1)
h2oC2 = subset(R.Hydro, Cell==2)
h2oC3 = subset(R.Hydro, Cell==3)
h2oC4 = subset(R.Hydro, Cell==4)

# Subset WB by Cell
WB.C1 = subset(R.WB, Cell==1)
WB.C2 = subset(R.WB, Cell==2)
WB.C3 = subset(R.WB, Cell==3)
WB.C4 = subset(R.WB, Cell==4)

## Inflow and Outflow
#Wilcoxon Rank Sum Test: Volume by Cell
wilcox.test(h2oC1$Surface.IN, h2oC1$Surface.OUT, paired=TRUE)
> p < 0.0001
wilcox.test(h2oC2$Surface.IN, h2oC2$Surface.OUT, paired=TRUE)
> p < 0.0001
wilcox.test(h2oC3$Surface.IN, h2oC3$Surface.OUT, paired=TRUE)
> p < 0.0001
wilcox.test(h2oC4$Surface.IN, h2oC4$Surface.OUT, paired=TRUE)
> p = 0.0014
wilcox.test(h2oC1$Surface.IN, h2oC3$Surface.OUT, paired=TRUE)
> p < 0.0001
wilcox.test(h2oC1$Surface.IN, h2oC4$Surface.OUT, paired=TRUE)
> p < 0.0001

#Wilcoxon Rank Sum Test: Peak by Cell
wilcox.test(h2oC1$Peak.IN, h2oC1$Peak.OUT, paired=TRUE)
> p < 0.0001
wilcox.test(h2oC2$Peak.IN, h2oC2$Peak.OUT, paired=TRUE)
> p = 0.0003
wilcox.test(h2oC3$Peak.IN, h2oC3$Peak.OUT, paired=TRUE)
> p < 0.0001
wilcox.test(h2oC4$Peak.IN, h2oC4$Peak.OUT, paired=TRUE)
> p = 0.2618
```

```
wilcox.test(h2oC1$Peak.IN, h2oC3$Peak.OUT, paired=TRUE)
> p = 0.0001
wilcox.test(h2oC1$Peak.IN, h2oC4$Peak.OUT, paired=TRUE)
> p < 0.0001

#Seasonal Inflow differences
kruskal.test(Surface.IN~Season, data=h2oC1)
> p = 0.4014
kruskal.test(Surface.OUT~Season, data=h2oC1)
> p = 0.2591
kruskal.test(Surface.IN~Season, data=h2oC2)
> p = 0.2591
kruskal.test(Surface.OUT~Season, data=h2oC2)
> p = 0.1314
kruskal.test(Surface.IN~Season, data=h2oC3)
> p = 0.1314
kruskal.test(Surface.OUT~Season, data=h2oC3)
> p = 0.297
kruskal.test(Surface.IN~Season, data=h2oC4)
> p = 0.297
kruskal.test(Surface.OUT~Season, data=h2oC4)
> p = 0.0957

#Kruskal test for differences: Cell and Fate
#Seasonality in WB per cell

#Cell 1
kruskal.test(Surf.in~Season, data=WB.C1)
> p = 0.9643
kruskal.test(Surf.out~Season, data=WB.C1)
> p = 0.0361
kruskal.test(RO.in~Season, data=WB.C1)
> p = 0.9643
kruskal.test(Seep.out~Season, data=WB.C1)
> p = 0.0242
kruskal.test(ET~Season, data=WB.C1)
> p = 0.8922
kruskal.test(Exfil~Season, data=WB.C1)
> p = 0.6255

#Cell 2
kruskal.test(Surf.in~Season, data=WB.C2)
> p = 0.0361
kruskal.test(Surf.out~Season, data=WB.C2)
> p = 0.1462
kruskal.test(RO.in~Season, data=WB.C2)
> p = 0.9643
kruskal.test(Seep.in~Season, data=WB.C2)
> p = 0.0242
kruskal.test(Seep.out~Season, data=WB.C2)
```

```
> p = 0.1951
kruskal.test(ET~Season, data=WB.C2)
> p = 0.6741
kruskal.test(Exfil~Season, data=WB.C2)
> p = 0.6801

#Cell 3
kruskal.test(Surf.in~Season, data=WB.C3)
> p = 0.1462
kruskal.test(Surf.out~Season, data=WB.C3)
> p = 0.1402
kruskal.test(RO.in~Season, data=WB.C3)
> p = 0.9643
kruskal.test(Seep.in~Season, data=WB.C3)
> p = 0.1951
kruskal.test(Seep.out~Season, data=WB.C3)
> p = 0.524
kruskal.test(ET~Season, data=WB.C3)
> p = 0.8585
kruskal.test(Exfil~Season, data=WB.C3)
> p = 0.6698

#Cell 4
kruskal.test(Surf.in~Season, data=WB.C4)
> p = 0.1402
kruskal.test(Surf.out~Season, data=WB.C4)
> p = 0.0056
kruskal.test(RO.in~Season, data=WB.C4)
> p = 0.9643
kruskal.test(Seep.in~Season, data=WB.C4)
> p = 0.524
kruskal.test(Seep.out~Season, data=WB.C4)
> p = 0.0686
kruskal.test(ET~Season, data=WB.C4)
> p = 0.7626
kruskal.test(Exfil~Season, data=WB.C4)
> 0.9854
```

Water Quality Analysis

```
# Subset WQ by weir
WQ.W1 = subset(R.WQ, Weir==1)
WQ.W2 = subset(R.WQ, Weir==2)
WQ.W3 = subset(R.WQ, Weir==3)
WQ.W4 = subset(R.WQ, Weir==4)
WQ.W5 = subset(R.WQ, Weir==5)

# Subset WQ by sesason
WQ.W1.Sum = subset(WQ.W1, Season==1)
WQ.W1.Fall = subset(WQ.W1, Season==2)
```

```
WQ.W1.Win = subset(WQ.W1, Season==3)
WQ.W1.Spr = subset(WQ.W1, Season==4)

WQ.W3.Sum = subset(WQ.W3, Season==1)
WQ.W3.Fall = subset(WQ.W3, Season==2)
WQ.W3.Win = subset(WQ.W3, Season==3)
WQ.W3.Spr = subset(WQ.W3, Season==4)

WQ.W3.Sum = subset(WQ.W3, Season==1)
WQ.W3.Fall = subset(WQ.W3, Season==2)
WQ.W3.Win = subset(WQ.W3, Season==3)
WQ.W3.Spr = subset(WQ.W3, Season==4)

WQ.W4.Sum = subset(WQ.W4, Season==1)
WQ.W4.Fall = subset(WQ.W4, Season==2)
WQ.W4.Win = subset(WQ.W4, Season==3)
WQ.W4.Spr = subset(WQ.W4, Season==4)

#Shapiro-Wilk Normality Test: Weir 1
shapiro.test(WQ.W1$TN)
> p < 0.0001
shapiro.test(WQ.W1$TKN)
> p = 0.0007
shapiro.test(WQ.W1$Nox)
> p = 0.0001
shapiro.test(WQ.W1$NH3)
> p = 0.0004
shapiro.test(WQ.W1$TP)
> p < 0.0001
shapiro.test(WQ.W1$OP)
> p < 0.0001
shapiro.test(WQ.W1$TSS)
> p < 0.0001

#Shapiro-Wilk Normality Test: Weir 2
shapiro.test(WQ.W3$TN)
> p = 0.0002
shapiro.test(WQ.W3$TKN)
> p = 0.0001
shapiro.test(WQ.W3$Nox)
> p = 0.191
shapiro.test(WQ.W3$NH3)
> p < 0.0001
shapiro.test(WQ.W3$TP)
> p = 0.0002
shapiro.test(WQ.W3$OP)
> p < 0.0001
shapiro.test(WQ.W3$TSS)
> p = 0.0002
```

```
#Shapiro-Wilk Normality Test: Weir 3
shapiro.test(WQ.W3$TN)
> p = 0.0002
shapiro.test(WQ.W3$TKN)
> p = 0.0001
shapiro.test(WQ.W3$Nox)
> p = 0.191
shapiro.test(WQ.W3$NH3)
> p < 0.0001
shapiro.test(WQ.W3$TP)
> p = 0.0002
shapiro.test(WQ.W3$OP)
> p < 0.0001
shapiro.test(WQ.W3$TSS)
> p = 0.0003

#Shapiro-Wilk Normality Test: Weir 4
shapiro.test(WQ.W4$TN)
> p = 0.0009
shapiro.test(WQ.W4$TKN)
> p = 0.0006
shapiro.test(WQ.W4$Nox)
> p = 0.0125
shapiro.test(WQ.W4$NH3)
> p < 0.0001
shapiro.test(WQ.W4$TP)
> p = 0.0002
shapiro.test(WQ.W4$OP)
> p = 0.0003
shapiro.test(WQ.W4$TSS)
> p = 0.0040

#Shapiro-Wilk Normality Test: Weir 5
shapiro.test(WQ.W5$TN)
> p = 0.0050
shapiro.test(WQ.W5$TKN)
> p = 0.002
shapiro.test(WQ.W5$Nox)
> p = 0.0081
shapiro.test(WQ.W5$NH3)
> p < 0.0001
shapiro.test(WQ.W5$TP)
> p < 0.0001
shapiro.test(WQ.W5$OP)
> p < 0.0001
shapiro.test(WQ.W5$TSS)
> p < 0.0001

## Inflow and Outflow concentrations
#Wilcoxon Rank Sum Test: Weir 1 v. Weir 2
```

```
wilcox.test(WQ.W1$TKN, WQ.W2$TKN, paired=TRUE)
> p = 0.210
wilcox.test(WQ.W1$Nox, WQ.W2$Nox, paired=TRUE)
> p = 0.953
wilcox.test(WQ.W1$NH3, WQ.W2$NH3, paired=TRUE)
> p = 0.541
wilcox.test(WQ.W1$TN, WQ.W2$TN, paired=TRUE)
> p = 0.258
wilcox.test(WQ.W1$TP, WQ.W2$TP, paired=TRUE)
> p = 0.418
wilcox.test(WQ.W1$OP, WQ.W2$OP, paired=TRUE)
> p = 0.891
wilcox.test(WQ.W1$TSS, WQ.W2$TSS, paired=TRUE)
> p < 0.0001

#Wilcoxon Rank Sum Test: Weir 2 v. Weir 3
wilcox.test(WQ.W2$TKN, WQ.W3$TKN, paired=TRUE)
> p = 0.0007
wilcox.test(WQ.W2$Nox, WQ.W3$Nox, paired=TRUE)
> p = 0.0012
wilcox.test(WQ.W2$NH3, WQ.W3$NH3, paired=TRUE)
> p = 0.0005
wilcox.test(WQ.W2$TN, WQ.W3$TN, paired=TRUE)
> p = 0.0018
wilcox.test(WQ.W2$TP, WQ.W3$TP, paired=TRUE)
> p = 0.0056
wilcox.test(WQ.W2$OP, WQ.W3$OP, paired=TRUE)
> p = 0.0637
wilcox.test(WQ.W2$TSS, WQ.W3$TSS, paired=TRUE)
> p =  0.8983

#Wilcoxon Rank Sum Test: Weir 3 v. Weir 4
wilcox.test(WQ.W3$TKN, WQ.W4$TKN, paired=TRUE)
> p = 0.3927
wilcox.test(WQ.W3$Nox, WQ.W4$Nox, paired=TRUE)
> p = 0.0003
wilcox.test(WQ.W3$NH3, WQ.W4$NH3, paired=TRUE)
> p = 0.0204
wilcox.test(WQ.W3$TN, WQ.W4$TN, paired=TRUE)
> p = 0.0936
wilcox.test(WQ.W3$TP, WQ.W4$TP, paired=TRUE)
> p = 0.5226
wilcox.test(WQ.W3$OP, WQ.W4$OP, paired=TRUE)
> p = 0.0569
wilcox.test(WQ.W3$TSS, WQ.W4$TSS, paired=TRUE)
> p = 0.0002

#Wilcoxon Rank Sum Test: Weir 4 v. Weir 5
wilcox.test(WQ.W4$TKN, WQ.W5$TKN, paired=TRUE)
> p = 0.1454
```

```
wilcox.test(WQ.W4$Nox, WQ.W5$Nox, paired=TRUE)
> p = 0.1454
wilcox.test(WQ.W4$NH3, WQ.W5$NH3, paired=TRUE)
> 0.0105
wilcox.test(WQ.W4$TN, WQ.W5$TN, paired=TRUE)
> p = 0.6191
wilcox.test(WQ.W4$TP, WQ.W5$TP, paired=TRUE)
> p = 0.0007
wilcox.test(WQ.W4$OP, WQ.W5$OP, paired=TRUE)
> p = 0.0654
wilcox.test(WQ.W4$TSS, WQ.W5$TSS, paired=TRUE)
> p < 0.0001

#Wilcoxon Rank Sum Test: Weir 1 v. Weir 5
wilcox.test(WQ.W1$TKN, WQ.W5$TKN, paired=TRUE)
> p = 0.0002
wilcox.test(WQ.W1$Nox, WQ.W5$Nox, paired=TRUE)
> p = 0.8603
wilcox.test(WQ.W1$NH3, WQ.W5$NH3, paired=TRUE)
> p = 0.0290
wilcox.test(WQ.W1$TN, WQ.W5$TN, paired=TRUE)
> p = 0.0004
wilcox.test(WQ.W1$TP, WQ.W5$TP, paired=TRUE)
> p = 0.0002
wilcox.test(WQ.W1$OP, WQ.W5$OP, paired=TRUE)
> p = 0.0638
wilcox.test(WQ.W1$TSS, WQ.W5$TSS, paired=TRUE)
> p < 0.0001

#Wilcoxon Rank Sum Test: Weir 1 v. Weir 4
wilcox.test(WQ.W1$TKN, WQ.W4$TKN, paired=TRUE)
> p = 0.0569
wilcox.test(WQ.W1$Nox, WQ.W4$Nox, paired=TRUE)
> 0.7819
wilcox.test(WQ.W1$NH3, WQ.W4$NH3, paired=TRUE)
> p= 0.1454
wilcox.test(WQ.W1$TN, WQ.W4$TN, paired=TRUE)
> p = 0.0714
wilcox.test(WQ.W1$TP, WQ.W4$TP, paired=TRUE)
> p = 0.0569
wilcox.test(WQ.W1$OP, WQ.W4$OP, paired=TRUE)
> p = 0.3225
wilcox.test(WQ.W1$TSS, WQ.W4$TSS, paired=TRUE)
> p < 0.0001

## Inflow and Outflow loads
#Wilcoxon Rank Sum Test: Weir 1 v. Weir 2
wilcox.test(WQ.W1$TKN_Ld, WQ.W2$TKN_Ld, paired=TRUE)
> p = 0.0108
wilcox.test(WQ.W1$Nox_Ld, WQ.W2$Nox_Ld, paired=TRUE)
```

```
> p = 0.0095
wilcox.test(WQ.W1$NH3_Ld, WQ.W2$NH3_Ld, paired=TRUE)
> p = 0.0323
wilcox.test(WQ.W1$TN_Ld, WQ.W2$TN_Ld, paired=TRUE)
> p = 0.0082
wilcox.test(WQ.W1$TP_Ld, WQ.W2$TP_Ld, paired=TRUE)
> p = 0.0001
wilcox.test(WQ.W1$OP_Ld, WQ.W2$OP_Ld, paired=TRUE)
> p = 0.0024
wilcox.test(WQ.W1$TSS_Ld, WQ.W2$TSS_Ld, paired=TRUE)
> p < 0.0001

#Wilcoxon Rank Sum Test: Weir 2 v. Weir 3
wilcox.test(WQ.W2$TKN_Ld, WQ.W3$TKN_Ld, paired=TRUE)
> p < 0.0001
wilcox.test(WQ.W2$Nox_Ld, WQ.W3$Nox_Ld, paired=TRUE)
> p < 0.0001
wilcox.test(WQ.W2$NH3_Ld, WQ.W3$NH3_Ld, paired=TRUE)
> p < 0.0001
wilcox.test(WQ.W2$TN_Ld, WQ.W3$TN_Ld, paired=TRUE)
> p < 0.0001
wilcox.test(WQ.W2$TP_Ld, WQ.W3$TP_Ld, paired=TRUE)
> p < 0.0001
wilcox.test(WQ.W2$OP_Ld, WQ.W3$OP_Ld, paired=TRUE)
> p = 0.0005
wilcox.test(WQ.W2$TSS_Ld, WQ.W3$TSS_Ld, paired=TRUE)
> p < 0.0001

#Wilcoxon Rank Sum Test: Weir 3 v. Weir 4
wilcox.test(WQ.W3$TKN_Ld, WQ.W4$TKN_Ld, paired=TRUE)
> p = 0.0120
wilcox.test(WQ.W3$Nox_Ld, WQ.W4$Nox_Ld, paired=TRUE)
> p = 0.0077
wilcox.test(WQ.W3$NH3_Ld, WQ.W4$NH3_Ld, paired=TRUE)
> p = 0.2288
wilcox.test(WQ.W3$TN_Ld, WQ.W4$TN_Ld, paired=TRUE)
> p = 0.0120
wilcox.test(WQ.W3$TP_Ld, WQ.W4$TP_Ld, paired=TRUE)
> p = 0.0040
wilcox.test(WQ.W3$OP_Ld, WQ.W4$OP_Ld, paired=TRUE)
> p 0.0013
wilcox.test(WQ.W3$TSS_Ld, WQ.W4$TSS_Ld, paired=TRUE)
> p = 0.0002

#Wilcoxon Rank Sum Test: Weir 4 v. Weir 5
wilcox.test(WQ.W4$TKN_Ld, WQ.W5$TKN_Ld, paired=TRUE)
> p = 0.2633
wilcox.test(WQ.W4$Nox_Ld, WQ.W5$Nox_Ld, paired=TRUE)
> p < 0.0001
wilcox.test(WQ.W4$NH3_Ld, WQ.W5$NH3_Ld, paired=TRUE)
```

```
> p = 0.6441
wilcox.test(WQ.W4$TN_Ld, WQ.W5$TN_Ld, paired=TRUE)
> p = 0.0638
wilcox.test(WQ.W4$TP_Ld, WQ.W5$TP_Ld, paired=TRUE)
> p = 0.0232
wilcox.test(WQ.W4$OP_Ld, WQ.W5$OP_Ld, paired=TRUE)
> p = 0.0654
wilcox.test(WQ.W4$TSS_Ld, WQ.W5$TSS_Ld, paired=TRUE)
> p = 0.2524

#Wilcoxon Rank Sum Test: Weir 1 v. Weir 5
wilcox.test(WQ.W1$TKN_Ld, WQ.W5$TKN_Ld, paired=TRUE)
> p < 0.0001
wilcox.test(WQ.W1$Nox_Ld, WQ.W5$Nox_Ld, paired=TRUE)
> p < 0.0001
wilcox.test(WQ.W1$NH3_Ld, WQ.W5$NH3_Ld, paired=TRUE)
> p < 0.0001
wilcox.test(WQ.W1$TN_Ld, WQ.W5$TN_Ld, paired=TRUE)
> p < 0.0001
wilcox.test(WQ.W1$TP_Ld, WQ.W5$TP_Ld, paired=TRUE)
> p < 0.0001
wilcox.test(WQ.W1$OP_Ld, WQ.W5$OP_Ld, paired=TRUE)
> p < 0.0001
wilcox.test(WQ.W1$TSS_Ld, WQ.W5$TSS_Ld, paired=TRUE)
> p < 0.0001

#Wilcoxon Rank Sum Test: Weir 1 v. Weir 4
wilcox.test(WQ.W1$TKN_Ld, WQ.W4$TKN_Ld, paired=TRUE)
> p < 0.0001
wilcox.test(WQ.W1$Nox_Ld, WQ.W4$Nox_Ld, paired=TRUE)
> p < 0.0001
wilcox.test(WQ.W1$NH3_Ld, WQ.W4$NH3_Ld, paired=TRUE)
> p < 0.0001
wilcox.test(WQ.W1$TN_Ld, WQ.W4$TN_Ld, paired=TRUE)
> p < 0.0001
wilcox.test(WQ.W1$TP_Ld, WQ.W4$TP_Ld, paired=TRUE)
> p < 0.0001
wilcox.test(WQ.W1$OP_Ld, WQ.W4$OP_Ld, paired=TRUE)
> p < 0.0001
wilcox.test(WQ.W1$TSS_Ld, WQ.W4$TSS_Ld, paired=TRUE)
> p > 0.0001

#Kruskal-Wallis Rank Sum Test: Seasonality Effects on Pollutants
#Inflow
kruskal.test(TN~Season, data=WQ.W1)
> p = 0.2069
kruskal.test(TKN~Season, data=WQ.W1)
> p = 0.2691
kruskal.test(Nox~Season, data=WQ.W1)
> p = 0.0355
```

```
kruskal.test(NH3~Season, data=WQ.W1)
> p = 0.0473
kruskal.test(TP~Season, data=WQ.W1)
> p = 0.1725
kruskal.test(OP~Season, data=WQ.W1)
> p = 0.0643
kruskal.test(TSS~Season, data=WQ.W1)
> p = 0.0989

#Weir 1
kruskal.test(TN~Season, data=WQ.W2)
> p = 0.6066
kruskal.test(TKN~Season, data=WQ.W2)
> p = 0.5297
kruskal.test(Nox~Season, data=WQ.W2)
> p = 0.1008
kruskal.test(NH3~Season, data=WQ.W2)
> p = 0.171
kruskal.test(TP~Season, data=WQ.W2)
> p = 0.194
kruskal.test(OP~Season, data=WQ.W2)
> p = 0.0593
kruskal.test(TSS~Season, data=WQ.W2)
> p = 0.0364

#Weir 2
kruskal.test(TN~Season, data=WQ.W3)
> p = 0.2627
kruskal.test(TKN~Season, data=WQ.W3)
> p = 0.3279
kruskal.test(Nox~Season, data=WQ.W3)
> p = 0.2209
kruskal.test(NH3~Season, data=WQ.W3)
> p = 0.2006
kruskal.test(TP~Season, data=WQ.W3)
> p = 0.1866
kruskal.test(OP~Season, data=WQ.W3)
> p = 0.1176
kruskal.test(TSS~Season, data=WQ.W3)
> p = 0.0812

#Weir 3
kruskal.test(TN~Season, data=WQ.W4)
> p = 0.0928
kruskal.test(TKN~Season, data=WQ.W4)
> p = 0.2028
kruskal.test(Nox~Season, data=WQ.W4)
> p = 0.0348
kruskal.test(NH3~Season, data=WQ.W4)
> p = 0.0271
```

```
kruskal.test(TP~Season, data=WQ.W4)
> p = 0.0926
kruskal.test(OP~Season, data=WQ.W4)
> p = 0.1057
kruskal.test(TSS~Season, data=WQ.W4)
> p = 0.3167
```