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Effectiveness Of Conventional And Lid Stormwater Management Approaches With SWMM Modeling: Rocky Branch Watershed, Columbia, SC

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EFFECTIVENESS OF CONVENTIONAL AND LID STORMWATER MANAGEMENT APPROACHES WITH SWMM MODELING: ROCKY BRANCH WATERSHED, COLUMBIA, SC

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ABSTRACT

Increases in percent impervious area and storm-sewer densities in an urbanized watershed lead to increased flood risk in urban areas. Conventional flood-risk management strategies such as detention ponds and low impact development (LID) can reduce peak flows. Research is needed to resolve questions about which strategy is best-suited for stormwater management under various schemes of sizing, distribution, and cost. Conventional and LID strategies differ in associated costs and benefits in addition to effectiveness and location feasibility. Previous research suggests that conventional strategies require less initial investment for design and construction, though LID is more cost effective in the long-term due to reduced annual maintenance requirements and the potential to distribution costs between centralized programs and public participation. This study used EPA's Storm Water Management Model (SWMM) for rainfall-runoff simulations to test and compare the effectiveness of conventional and LID management scenarios in reducing runoff depths and peak flows of moderate-magnitude storms in the upper Rocky Branch Watershed (RBW) in Columbia, SC. The SWMM was calibrated and validated with six independent storm events using flow-stage data at a very small, highly urbanized watershed, and Acoustic Doppler Current Profiler (ADCP) discharge data at a larger watershed. Model calibrations and validations were assessed with a Nash Sutcliffe Efficiency (NSE) and each of the six storms yielded NSEs ≥ 0.712. Various configurations and locations of detention ponds and LID were modeled to compare the effectiveness of individual strategies under two levels of initial investment based on unit storage costs

(\$/m3). Individual application of both strategies was only effective placed upstream in the smaller, highly impervious subcatchment, in which case detention ponds were more effective in reducing peak discharges at both initial investment levels. A localized scenario in which bioretention was clustered in the upper, most-urbanized sub-basin provided a 2.1% greater reduction of peak flow at the primary watershed than a distributed scenario in which bioretention was spread across three different locations.

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LIST OF ABBREVIATIONS

ADCP	Acoustic Doppler Current Profiler
BCA	Benefit-Cost Analysis
CoC	City of Columbia
DB	Devine-Blossom
FV	Future Value
GIS	Geographic Information System
GS	Gregg Street
HRH	Hollywood-Rose Hill
IL1	Investment Level 1
IL2	Investment Level 2
LCB	Life-Cycle Benefit
LCC	Life-Cycle Cost
LID	Low Impact Development
MLK	Martin Luther King Jr. Park
NSE	Nash-Sutcliffe Efficiency
PIA	Percent Impervious Area
Qpk	Peak Discharge
RBW	Rocky Branch Watershed
SS	Storm Sewer
SWMM	Storm Water Management Model
T1	Test 1

T2A .... Test 2A

T2B .... Test 2B

TIA .... Total Impervious Area

UH .... University Hill

WTP .... Willingness to Pay

CHAPTER 1

INTRODUCTION

1.1 Hydrologic Effects of Urbanization and Management Strategies

Urbanized watersheds can have a high percent impervious area (PIA) that leads to increased runoff (Jacobson, 2011; Walsh et al., 2005; Scheuler, 1994). They also tend to have an extensive network of storm sewers (SS) that accelerates the arrival of flood waves. Combined, PIA and SS systems can greatly multiply peak discharge in urban watersheds and accelerate stormflow arrival times (Leopold, 1968; Putnam, 1972; Bohman, 1992; Meierdiercks et al., 2010).

Stormwater management planning can best mitigate these hydrologic changes if flows through the physical infrastructure and processes are well understood, which can be assisted by the use of hydrologic simulations. Stormwater designs to reduce flood risks in urbanized watersheds typically can be classified as conventional or low impact development. Conventional stormwater management techniques emphasize the removal of water from developed sites via concentrated flows in ditches, gutters or storm sewers to local storage facilities, such as retention or detention ponds, that delay the release to streams (Wanielista and Yousef 1993). These stormwater-mitigation strategies focus primarily on reduction of peak flow rates for larger storm events (Sparkman et al., 2017). Due to the cost and scale of structures, conventional stormwater design is inherently centralized. Low impact development (LID), also known as green infrastructure, green engineering, spatially distributed, or source-control stormwater management, is also used to reduce flood risks, but aims to retain stormwater on site and often provides additional water pollution-reduction benefits (Hunt et al., 2006). LID uses a large number of small features, such as bioretention cells, rain gardens, green roofs, and permeable pavements to promote infiltration, storage, and evaporation (Rossman, 2015; Elliott and Trowsdale, 2006; Davis 2005). Implementation of LID options are becoming increasingly more attractive in urban areas experiencing flooding issues related to imperviousness (Sparkman et al., 2017). LID stormwater controls are frequently used due to their ability to mimic predevelopment site hydrology, reduce total impervious areas (TIA), and allow for clustered or distributed parcel-scale controls (Davis, 2005; Morsy et al., 2016). A study of three bioretention sites in North Carolina found that volumetric ratios of bioretention cell outflows to inflows varied from 0.07 in summer to 0.54 in winter (Hunt et al., 2006). A key advantage of LID is the ease of distributing stormwater management infrastructure across broad areas. LID flood management can be highly flexible at the watershed scale due to their lower requirements for space and site disturbance than conventional strategies. Little is known, however, of the advantages of centralized vs. decentralized applications of LID (Sitzenfrei et al., 2013). Further studies are needed of these strategies based not only on the resulting reductions in stormwater peaks and volumes, but also with analysis of initial and annual expenses as well as potential benefits of the life cycle of the management strategy.

1.2. Economics of Stormwater Management

Conventional and LID flood-mitigation strategies differ not only in size, scale, and function, but also in cost. Three methods can be used to assess the economic impact of LID practices, each with increasing complexity (Mateleska, 2016; Zhan and Chui, 2016). First, a simple cost comparison can be made between the initial construction costs of differing methods of LID treatment. Second, a life-cycle cost (LCC) analysis can be completed, adding another dimension of costs throughout the life of the stormwater control (Chui et al., 2016; Houle et al., 2013). Life-cycle costs (LCC) can be calculated starting with initial costs including land acquisition, design and construction, and annual operation and maintenance (O&M) costs over the expected life of the management strategy (Mateleska, 2016; Chui et al., 2015;). Mateleska (2016) found that detention pond design and construction had a lower unit storage cost ($/m3) than bioretention, $240.04 and $547.70 respectively, however bioretention cells were estimated to require 20.6% less annual investment related to O&M. Houle et al. (2013) reported similar results in a study reporting annual costs and required maintenance hours, with bioretention providing a 17.7% reduction in annual O&M costs over the course of their life-cycle. Benefit-cost analysis (BCA) provides a third means of comparing economic efficiency of management strategies, considering all relevant LCC and net life-cycle benefits (LCB), including environmental and social non-market benefits. While LCC analysis provides a more realistic long term understanding of stormwater management costs than a basic comparison of initial investment costs, a full BCA can provide a more accurate assessment of differences in total cost over the life cycle of a management strategy (Zhan and Chui, 2016; EPA, 2013). Typical life-cycles of stormwater controls range from 20-30 years, although these types of analysis often consider a longer period of 50 years or more (Zhan and Chui, 2016; Veseley, 2005). CBA can be significantly more complex than LCC analysis because many of these benefits do not have a direct economic value attached to them and therefore value must be inferred through non-market valuation strategies. Although both conventional and LID strategies provide direct economic benefits from runoff and discharge quantity reductions as well as various water quality benefits, LID provides more extensive environmental and societal benefits than conventional strategies including improved air quality, CO_2 sequestration, and thermal benefits from reduction of the urban heat island effect, as well as benefits to society such as improved citizen health and aesthetic benefits (Zhan et al., 2016; EPA 2013; Veseley, 2005). Environmental benefits can often be quantified through assessment of potential savings from avoided consequences, while social benefits must be inferred from contingent valuation strategies in which willingness to pay (WTP) for the infrastructure in question is evaluated by observing the preferences of an individual or group, either directly stated or 'observed' preferences (Zhan and Chui, 2016). A study based on contingent valuation surveys, experimental real estate negotiations, and spatial hedonic price methods found that individuals revealed an increased WTP for LID (Bowman et al., 2012). Individuals with prior knowledge of LID also showed higher WTP than those who were unfamiliar. This is an important advantage of LID, as public use of LID practices such as disconnecting rooftop runoff or the use of rain barrels can increase storage within a watershed without adding additional government and institutional expenses. Evaluation of the LCC and LCB of stormwater control must also include a discount rate and an inflation rate in order to observe these monetary values in the context of future value (FV). While there is no agreed upon value for these rates, previous research has used discount rates from 3.5 – 4.25% and inflation rates from 2 – 5% (Zhan and Chui, 2016; EPA, 2013; Veseley, 2005). While the management scenarios designed for this study based on initial design and construction costs per unit of storage ($/m3), a review of the economic benefits of LID revealed through BCA addresses the potential long-term efficiency of these strategies.

1.3. Hydrologic (Rainfall/Runoff) Modeling With SWMM

Rainfall-runoff models can be used to simulate hydrologic responses, such as infiltration, surface runoff, and channel and pipe flow to changes in land use or stormwater treatment practices. These models typically incorporate observed rainfall data, land use, and geospatial characteristics, as well as conveyance relationships between open channels, culverts, and floodplains. They are calibrated and validated with stream flow data. The models provide a tool that can be used to test scenarios of changes to the hydrologic system. One such model is the EPA's Storm Water Management Model (SWMM) (Rossman, 2015), which is an open-source model intended for urbanized watersheds. First developed in 1971 and now on its fifth version, SWMM is a dynamic rainfall-runoff simulation model capable of single-event or continuous simulations of water quantity and quality (Rossman, 2015; Barco et al., 2008). SWMM routes runoff for subcatchments with storm drains, combined sewers, and natural drainage through a network of pipes, channels, and storage/treatment units (Rossman, 2015; Barco 2008). The SWMM simulates three primary processes: stormwater infiltration, surface runoff, and flow routing. The latest versions of SWMM (e.g., version 5.1.012) can model hydrologic performance of typical conventional and LID controls with varying sizes, coverage, and geographic distributions. The LID controls include, but are not limited to, bioretention and green roofs (Rossman, 2015).

The primary applications of SWMM, as related to this research, include simulations of runoff volumes and discharge, that can be used for planning, analysis, and design of stormwater drainage systems. The SWMM has the ability to simulate runoff volumes and timing as stormwater flows through conventional detention infrastructure and eight different LID configurations, including bioretention cells, green roofs, and permeable pavements (Rossman, 2015). The model can also account for numerous hydrologic processes and water budgeting including precipitation time series, runoff storage, infiltration, evapotranspiration, and interaction with groundwater layers and interflow (Rossman, 2015). Three flow routing methods can be used for SWMM: steady flow, kinematic wave, or, where the purpose is for event-based storm events such as this study, dynamic wave, which accounts for backwater effects (Rossman, 2015). Once calibrated to observed rainfall and stream-flow data, SWMM can simulate the implementation of virtual conventional storage units or LID scenarios in order to evaluate their effectiveness in reducing surface runoff depth and peak discharge. Modeling flood-control implementation scenarios of varying type, size, and spatial location can accurately determine which stormwater management scenarios are most effective. Rosa et al. (2015) compared the accuracy of calibrated and uncalibrated SWMM models that incorporate LID controls and found that uncalibrated models underpredicted key runoff components by as much as 80%. In contrast, calibrated models produced results within 12% of observed values.

The objectives of this study are to (1) observe and compare the effectiveness of both conventional and LID strategies in reducing peak flow for moderate-magnitude storms in a small, highly urbanized watershed, (2) observe the effectiveness of LID in reducing total runoff volume (locally) within the test subcatchment of implementation (3) observe and compare the effectiveness of LID strategies when implemented in different patterns in the watershed, both localized (lumped) and distributed, and (4) observe SWMM results in reference to the economic effectiveness of conventional and LID strategies. A series of simple hypotheses are presented to structure objective tests for objectives one and two (Table 1.1). These hypotheses will be applied independently at each of the two stream gauges.

Table 1.1. Hypotheses regarding stormwater management scenarios for moderate-magnitude storms at given levels of investment

1. Conventional vs LID	
H1A	LID reduces peak discharges more than conventional management practices.
H1B	LID reduces local subcatchment runoff volume.
2. Spatial Patterns of LID	
H2	LID grouped in the most urbanized sub-basin is more effective in reducing peak discharge than LID distributed across multiple sub-basins.

CHAPTER 2

METHODS

2.1 Study Area

Rocky Branch Watershed (RBW) is contained within an area of roughly 10.3km2 and includes 14.5 km of open stream channels that flow into the Congaree River (Figure 2.1). The RBW contains most of the University of South Carolina campus, much of the downtown Columbia central business district, the Five Points Commercial District, and several old residential neighborhoods. With an imperviousness of 49%, the dominant land use in RBW is developed land of high, medium, and low intensity (McCormick Taylor, 2016). RBW falls entirely within the Sandhills ecoregion and physiographic province with topographically variable Cretaceous-age marine and aeolian sand (Sweezy et al., 2016). The high sand content of soils results in high contrasts in infiltration rates and runoff generation between impervious and pervious surfaces. Intense urban development over the course of the past century left RBW subject to extreme stormwater and water quality issues based on high longitudinal channel connectivity, low latitudinal floodplain connectivity, lack of open channel area, upstream imperviousness, and an absence of stormwater management (McCormick Taylor, 2016). The RBW demonstrates the increase in flood risk that often accompanies urbanization, as increased PIA and SS densities in the watershed have generated frequent flood events and water degradation. The area simulated in this study is the Pickens Basin in the upper RBW and the Gervais

Sub-basin nested within the Pickens Basin. Three rain gages provide data for the SWMM and two streamflow gages were used for calibration and validation.

An assessment for the City of Columbia (hereafter the Assessment) of the current condition of RBW utilized field mapping, hydrologic and hydraulic analysis, and GIS-based sub-watershed characterizations (McCormick Taylor, 2016). It subdivided RBW into 11 sub-watersheds as proposed by the Rocky Branch Watershed (Figure 2.2). This study refers to the total area of the upper watershed contributing to the Above Pickens gage, referred to as the Above Pickens Basin (Figure 2.1). Contained within this area are the Gregg Street (GS), Martin Luther King Park (MLK), Devine Blossom (DB), Hollywood-Rose Hill (HRH), and a portion of the University Hill (UH) subwatersheds. The Assessment made recommendations for watershed-restoration projects including five flood-water detention areas and thirty-two potential LID projects. Ultimately, their recommendations prioritized potential storm-water management projects for these areas using a cumulative ranking index system derived from potential reductions in peak discharge, total runoff, and unit runoff for the regional 2-year flood. The hydrologic analysis for the assessment was based largely on a SWMM model developed by KCI Technologies which was the initial basis for the model used in this study. The 11 sub-watersheds were further subdivided into sixty subcatchments in the SWMM model in order to produce a semi-distributed model (McCormick Taylor, 2016). This study is focused on the portion of the model above the Pickens Street streamflow gage, which includes 27 subcatchments. Of these subcatchments, five are used as locations for modeled stormwater controls: GS-1, GS-2, GS-3, MLK-9, and UH-3 (Figure 2.3). SWMM defines sections of open channel or SS conduits as 'links,' and peak flows within five links are observed in this study: Gervais-Link, Gregg St-Link, MLK-Link, Five Points-Link, and Pickens-Link (Figure 2.3).

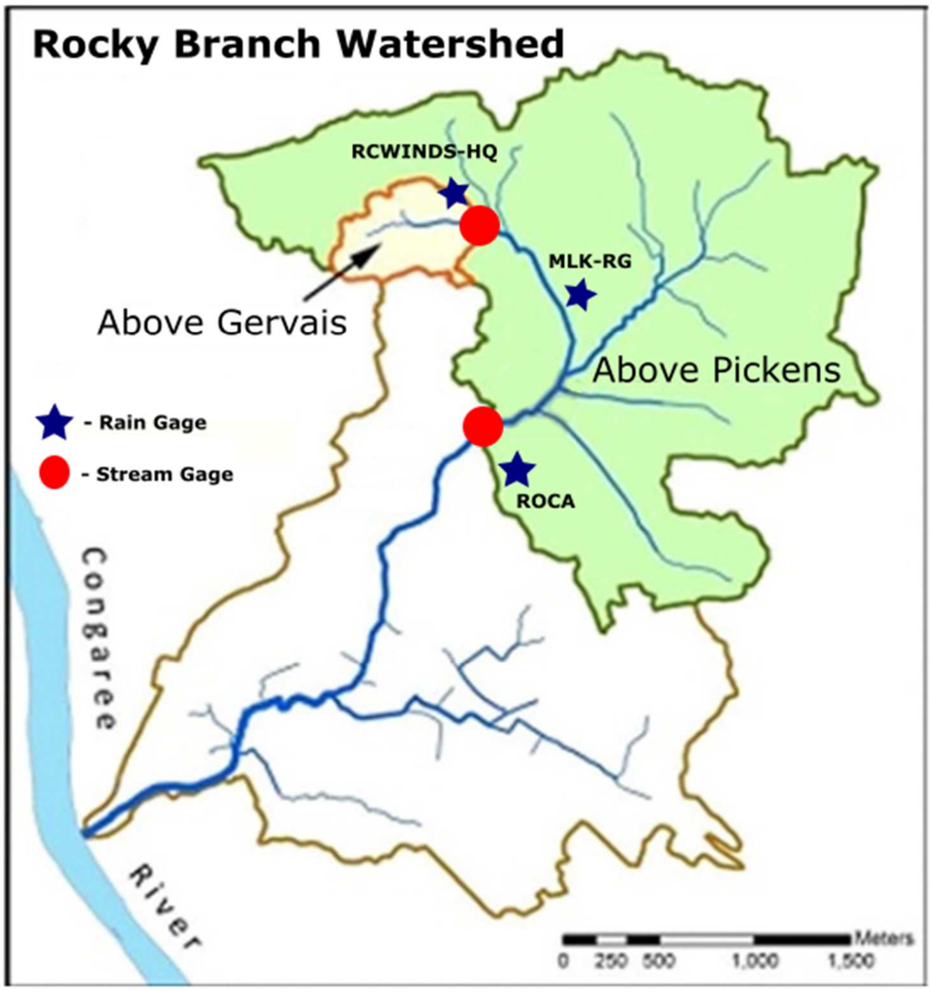

Figure 2.1. Study Area, Rain Gages, and Stream Gage

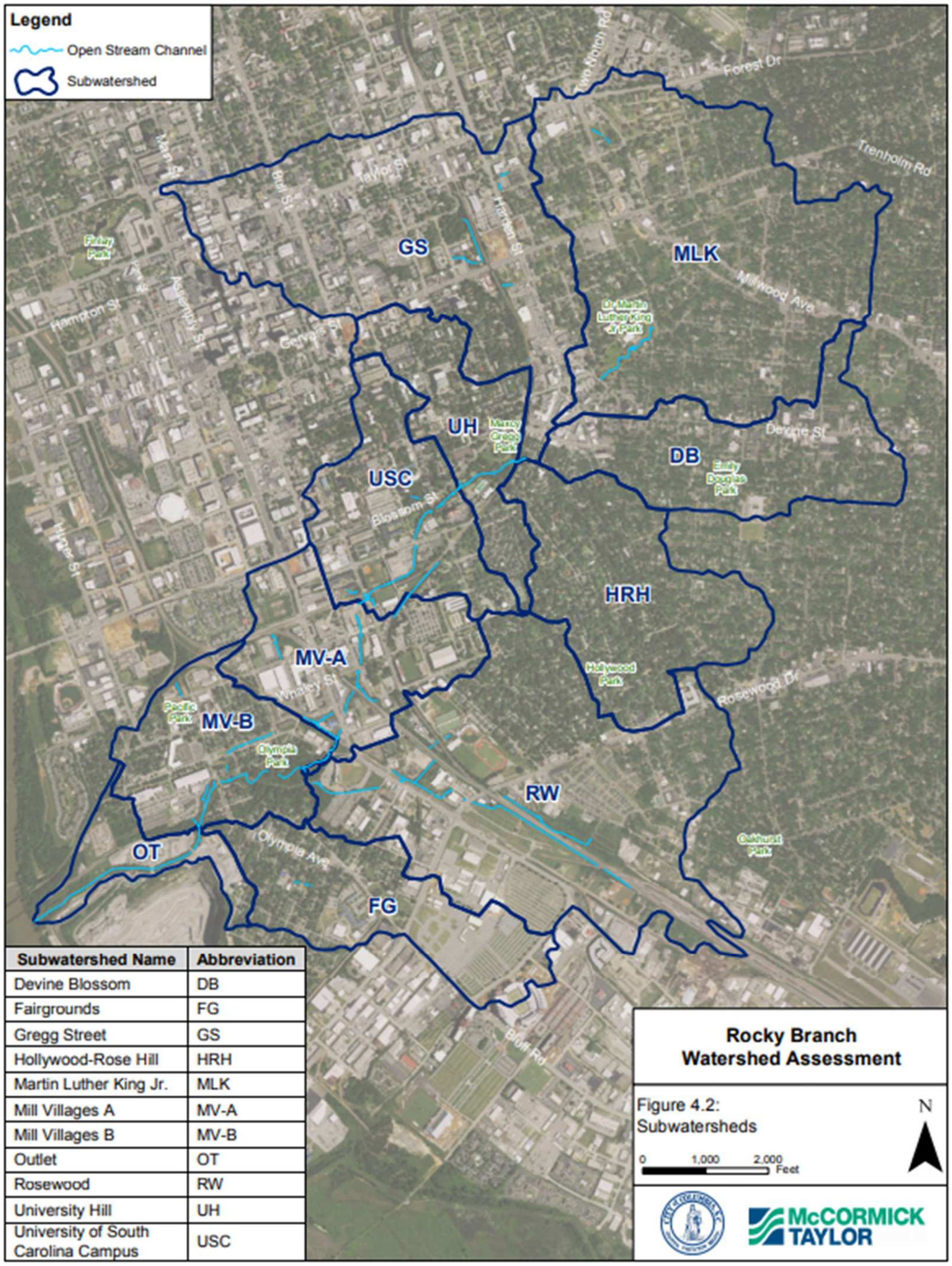

Figure 2.2. McCormick Taylor (2016) subwatershed delineation. The area observed for this study includes GS, MLK, UH, DB, and HRH

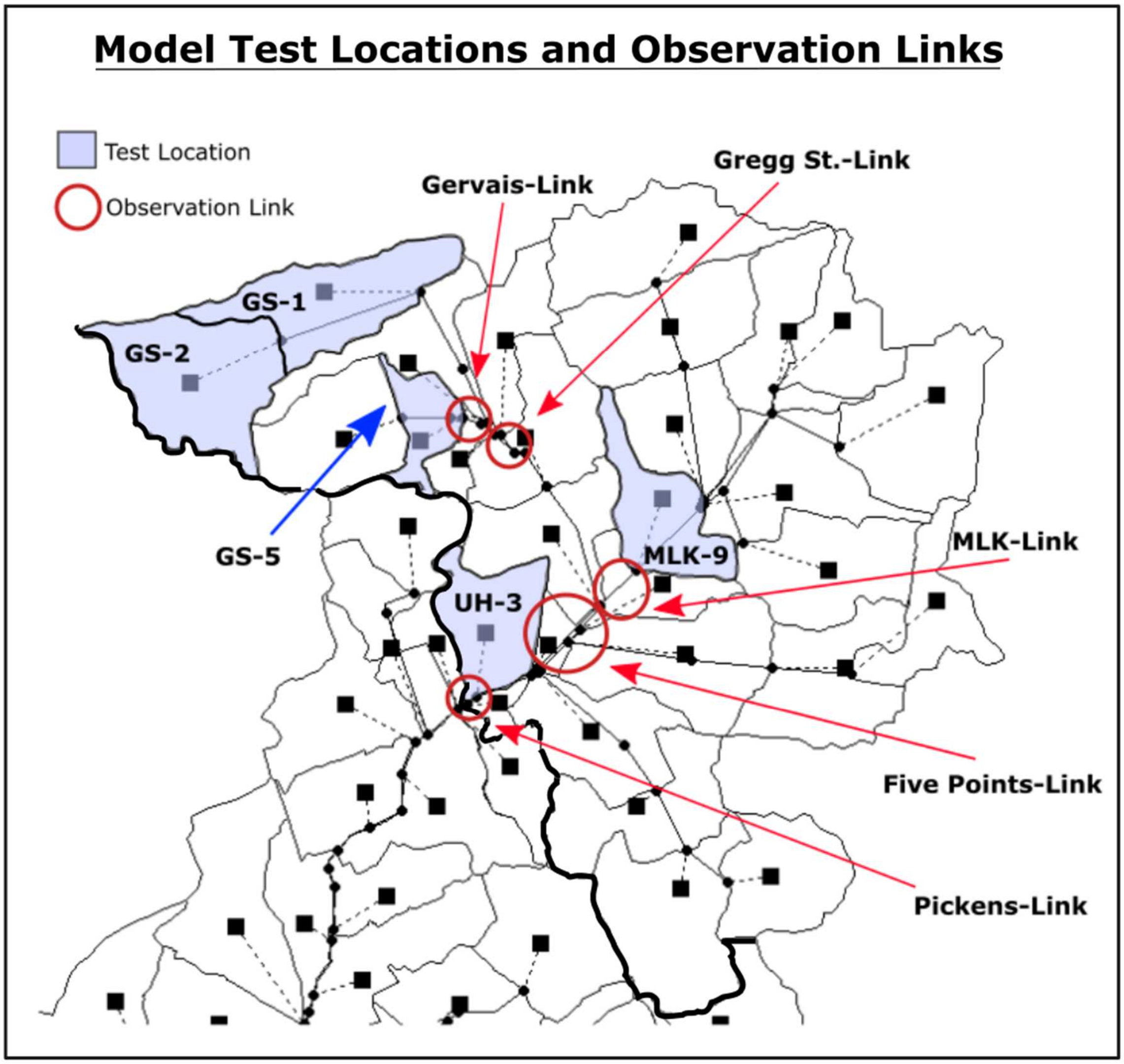

Figure 2.3. Upper RBW (above bold line) including 5 test subcatchments for stormwater controls and computation of runoff volumes and 5 test links examined for Qpk

A previous study utilized a SWMM model in RBW to model the effectiveness of LID controls, specifically rain gardens (Morsy et al., 2016). While incorporating parameters similar to the model developed in this study, the focus of that study was on flow-stage reduction based on different runoff-routing scenarios and focused on how much runoff must be diverted to proposed LID controls in order to account for runoff from various precipitation frequencies (Morsy et al., 2016). The study reported here compares the mitigation of stormwater resulting from the implementation of conventional and LID management practices.

2.2. Model Overview and Data Preparation

Initial model parameter estimation was based on existing literature, previous model settings, and model defaults. The GREEN-AMPT infiltration was adopted from previous versions of the model for RBW developed by KCI. The SS network was mapped by the City of Columbia (CoC) and imported the existing SWMM by KCI (McCormick Taylor, 2016). Soil characteristics were chosen based on SURGO digital data (USDA, 1978; 1994). The dynamic wave model was selected for flow routing because it accounts for channel storage, backwater effects, entrance and exit losses, flow reversals, and pressurized flow, all of which are known to occur during floods in RBW. Spatial data for RBW subcatchments, including drainage area, slope, and percent impervious area (PIA), were analyzed through geographic information system (GIS) procedures and used to update the model.

GREEN-AMPT parameters, such as suction head and hydraulic conductivity, were adjusted based on values appropriate for loamy sand, the dominant soil type in RBW (Rossman, 2015; Rawls and Brackensiek, 1993; Rawls et al., 1983;). Ranges for detention storage and Manning's roughness for overland flow were based on values cited in the 2016 SWMM Manual and other standard hydrology sources (Rossman, 2015; McCuen, 1996; ASCE, 1983; 1992). Stream channel and conduit profiles, dimensions, and roughness (Manning's n) were adopted from the existing model, although some open channel roughness values (Manning's n) were adjusted to more realistic values, and an updated channel profile was added to the model for the 'Above Gervais' calibration point. The

SWMM model used in this study utilized 5-minute rainfall data from three gages. Rain gage RCWINDS-HQ was operated by the Richland County Weather Information Network Data System and rain gages MLK-RG and ROCA were operated by Woolpert Inc., LLC for the CoC. The initial model was calibrated to stage data at Pickens and was recalibrated for this study using flow data from two locations: stage data (m) at the 'Above Gervais' gage and discharge data at the 'Above Pickens' gage (Figure 2.1). The stage data at Gervais were measured at two-minute intervals using a Solinst barometrically corrected level logger, and converted to five-minute intervals for model assessments. The discharge data at Pickens were collected by Woolpert Inc., LLC for the CoC using an acoustic Doppler current profiler (ADCP) at five-minute intervals (Figure 2.4).

Observed storm events for calibration and modeling were screened and selected for the study period of July 1, 2016 to February 1, 2018 at the RCWINDS-HQ and ROCA rainfall stations, the closest locations to the streamflow gages used for calibration at Above Gervais and Above Pickens, respectively. Precipitation events were screened visually and eliminated if rainfall was highly variable in time or between gage locations to avoid multi-modal hydrographs and spatially variate intensities. Events were discarded with discharges exceeding 15 m3/s at the Above Pickens gage due to observed difficulties with SWMM computation of overbank discharges. Precipitation durations for chosen storms ranged from 20 to 105 minutes, and precipitation depths ranged from 7.5 – 20 mm. Selected stormflow durations were determined using a factor of 5.4 times the time-to-peak following the time of peak discharge, calculated based on the end of stormflow for the storm event on May 29, 2017, a 35-min, 14 mm rainfall at HQ rain gage, resulting in the peak stage of 0.802 m at Gervais and the peak discharge of 9.97 m3/s at Pickens. Six storm events were selected, three for calibration and three for validation (Table 2.1). The 5/29/2017 event was selected as the base storm for scenario modeling due to its moderate-magnitude intensity and short duration (Figure 2.5).

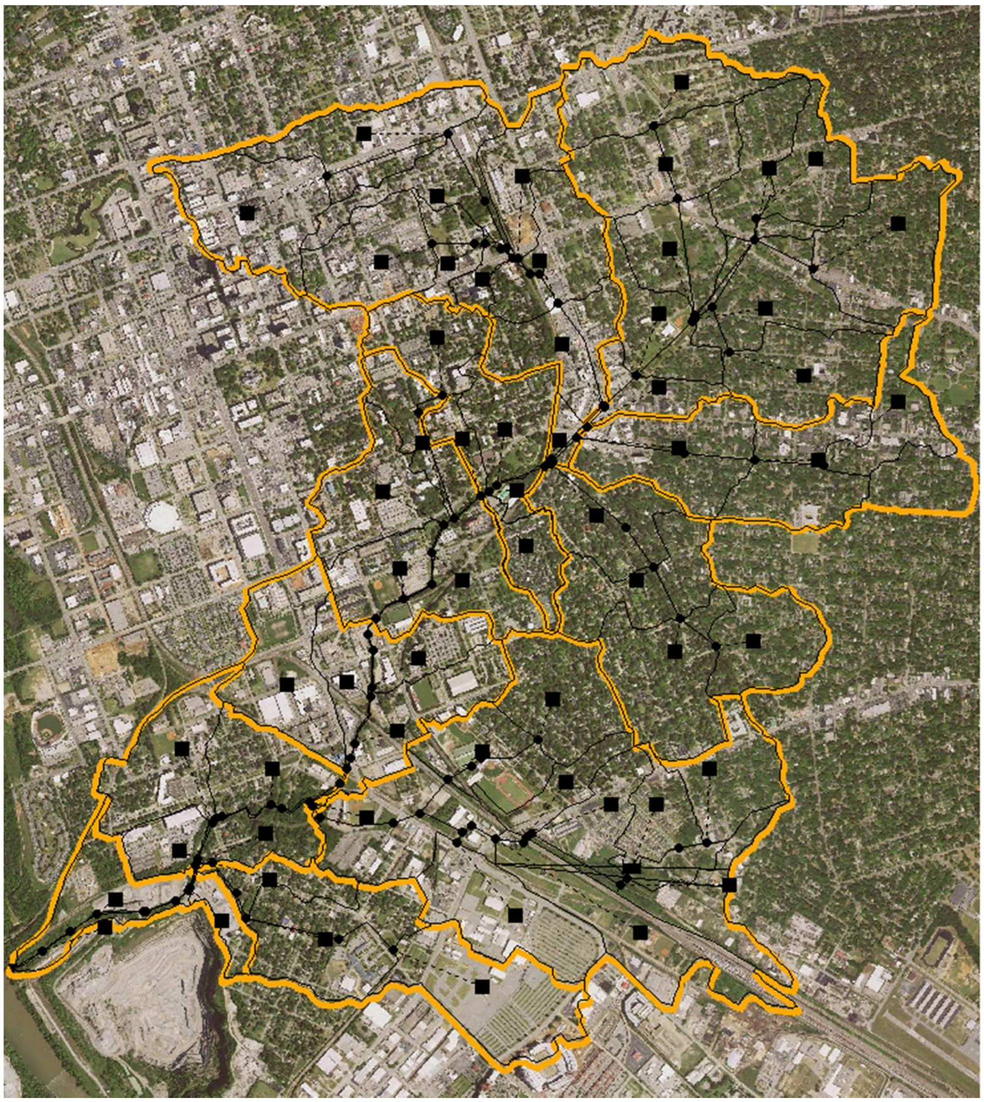

Figure 2.4. SWMM Model Layout including all subcatchments, links, and nodes. Only the upper portion of the model above Pickens was calibrated and used for this study

Table 2.1. Observed Storm Events

Event Code	Event Date	Duration	Cumulative Rainfall Depth	Rainfall Intensity	ROCA Qpk (m^3/s)
C1	5/22/2017	105	20	4.4	1.7
C2	5/29/2017	40	19.3	9.7	10.0
C3	8/13/2017	55	21	7.6	14.5
V1	5/24/2017	75	12	3.7	9.6
V2	7/25/2017	35	14	14.0	7.8
V3	10/16/2017	35	14	8.0	8.3

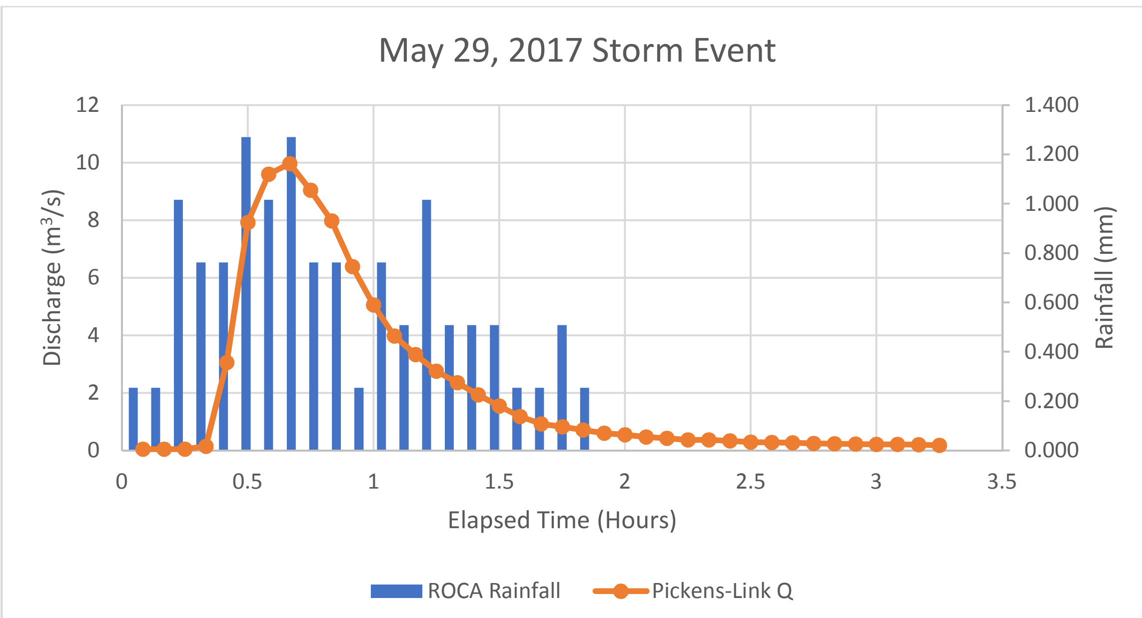

Figure 2.5. Observed rainfall at the ROCA rain gage and discharge at the Above Pickens stream gage for the base storm 5/29/2017

2.3. Model Sensitivity

Sensitivity analysis was performed to asses which parameter changes would be most effective in minimizing differences between simulated and observed concentrated stormflow values during calibration (Rosa et al., 2015). Parameters were adjusted over a range of ±50% of their original value with all other parameters remaining constant and the resulting changes in peak flow were noted at calibration locations. Relative sensitivity was computed by the method used by Rosa et al. (2015):

$$\text{Sensitivity}=(\partial R/\partial P)(P/R) \tag{1}$$

where ∂R is the difference between the original and new model output, ∂P is the difference between the original and adjusted parameter value, R is the original model output, and P is the original value of the chosen parameter of interest (Rosa et al., 2015; James and Burges, 1982). Green-Ampt infiltration parameters have been used as sensitive parameters for calibration, as well as Manning's n (roughness), saturated hydraulic conductivity, and initial soil moisture (Rosa et al., 2015). Parameters tested for sensitivity, as well as their initial value, calibration range, and final calibrated value/range can be seen in Table 2.2.

Table 2.2. Parameters for Sensitivity Analysis

Parameter	Initial Value	Calibration Range	Final Value/Range	Data Source
Manning's n (impervious)	0.015	0.01 - 0.024	0.015 - 0.024	(McCuen et al., 1996)
Manning's n (pervious)	0.4	0.01 - 0.8	0.4	(McCuen et al., 1996)
Dstore-Imperv (mm)	1.27	1.27 - 2.54	1.27 - 2.54	(ASCE,1992)
Dstore-Perv (mm)	2.54	2.54 - 5.08	2.54	(ASCE,1992)
Width	GIS Calculated	% of Orginal	% of Orginal	(Rossman, 2015)
Green Ampt Parameters				
Suction Head (mm)	2.4	49.0 - 320.0	60.96	(Rawls et al., 1983)
Conductivity (mm/hr)	1.18	0.254 - 120.4	29.97	(Rawls et al., 1983)
Initial Soil Moisture Deficit (fraction)	0.33	0.097 - 0.375	0.33	(Rawls et al., 1983)

2.4. Model Calibration and Validation

Event-based calibration was completed using three of the six events chosen during storm screening. Sensitive parameters were changed one at a time during calibration until differences between simulated and observed flows were minimized, or until a limit of the accepted range of the parameter was reached (Rosa et al., 2015; Morsy et al., 2016). Agreement between observed and simulated values was evaluated using the Nash-Sutcliffe model efficiency (NSE). The NSE is a dimensionless statistic that measures the relative magnitude of the residual variance (Moriasi et al., 2007), indicating how well the simulated data match the observed data compared to a 1:1 line:

$$NSE = 1 - [\frac{\sum_{i=1}^{n}(Yi\ obs - Yi\ sim)\hat{}2}{\sum_{i=1}^{n}(Yi\ obs - Y\ mean)\hat{}2}] \qquad (2)$$

where Yiobs is the ith observed value, Yisim is the ith simulated value, Ymean is the mean of the observed data, and n is the total number of observations (Moriasi et al., 2007). NSE values do not exceed an absolute value of 1, with an optimal value of NSE = 1 indicating a perfect fit, and negative NSE values indicating unacceptable performance (Nash and Sutcliffe, 1970; Moriasi et al., 2007; Dongquan et al., 2009; Rosa et al., 2015). Previous studies posit that an NSE > 0.5 indicates acceptable model performance for SWMM, with increased NSE values correlating with increased model accuracy (Rosa et al., 2015; Dongquan et al., 2009). Model assessments using NSE were based on a time period of 5.4 times the time-to-peak following the time of peak stormflow discharge.

Parameter changes were lumped initially across all subcatchments and then to specific subcatchments based on sensitivity, area, PIA, and proximity to the calibration point of interest. The focus of this study is on the management of stormwater in the upper watershed above Pickens, therefore, the model is not considered to be calibrated below this point.

2.5. Management Scenarios and Scenario Development

Management scenarios were designed to test the hypotheses concerning comparisons of LID and conventional treatment and geographic locations of treatments. The conventional and LID configurations were narrowly defined to control comparisons, but each of those configurations could be modified to substantially change results. For example, peak discharges under conventional management were highly sensitive to the outlet structures of detention structures that changed arrival times of flow peaks. Optimization of outlet structures for the moderate-magnitude flows in this study would not likely be optimal for larger flows, and optimizing over a large range of flows was beyond the scope of this study. Management scenarios were broken into a set of tests with varying locations of virtual stormwater management controls based on recommendations from the assessment for potential restoration opportunities within RBW (McCormick Taylor 2016). Although varying somewhat in size and treatment area, all bioretention cell locations were derived from the assessment. Conversely, placement of only one detention pond—located at GS-5—was derived from the assessment, as GS-5 was the only catchment with both a conventional and LID recommendation. Remaining detention pond locations were chosen based on recommended LID locations for the purpose of direct comparisons. All SWMM scenarios for this study were modeled using precipitation data from the May 29, 2017 calibration storm, a 40-minute event with rainfall intensities of 8 mm/20 min and 9.7 mm/20 min at RCWINDS-HQ and ROCA, respectively. This storm was selected as the base storm for modeling due to its relatively high intensities and consistency between intensities at both calibration locations. This study observes modeling results from two tests shown in Table 2.3.

Table 2.3. Test 1 and 2 Details

Test	Details	Links Observed for Qpk
Test 1 (T1)	Detention Ponds and bioretention implemented one at a time in GS-5, MLK9, and UH3. Tests H1A and H1B.	T1L1 (GS-5): Gervais Link, Pickens Link T1L2 (MLK-9): MLK-Link, Pickens Link T1L3 (UH-3): Pickens Link
Test 2A (T2A)	Localized Scenario; Bioretention implemented at GS-1, GS-2, and GS-3. Tests H2.	Gervais-Link; Gregg St-Link; MLK-Link; Five Points-Link; Pickens-Link
Test 2B (T2B)	Distributed Scenario; Bioretention implementd at GS-5, MLK-9, and UH-3. Tests H2.	Gervais-Link; Gregg St-Link; MLK-Link; Five Points-Link; Pickens-Link

Stormwater management Test 1 (T1) was designed to compare the effectiveness of conventional detention basins to that of bioretention cells (LID) in reducing peak discharge (m3/s) of concentrated flows within conduit and channel links. Bioretention scenarios were assessed for effectiveness in reduction of local runoff volume (106 m3), but simulations of detention ponds were not expected to show changes in runoff volume due to the way in which the model views the storage unit as a subcatchment outlet node rather than a part of the subcatchment itself. The configuration and size of detention ponds or bioretention cells were based on assumptions of initial investment for design and construction costs only, each modeled separately at three locations for a total of six SWMM runs. Location 1, Location 2, and Location 3 were at GS-5, MLK-9, and UH-3 respectively (Figure 2.6).

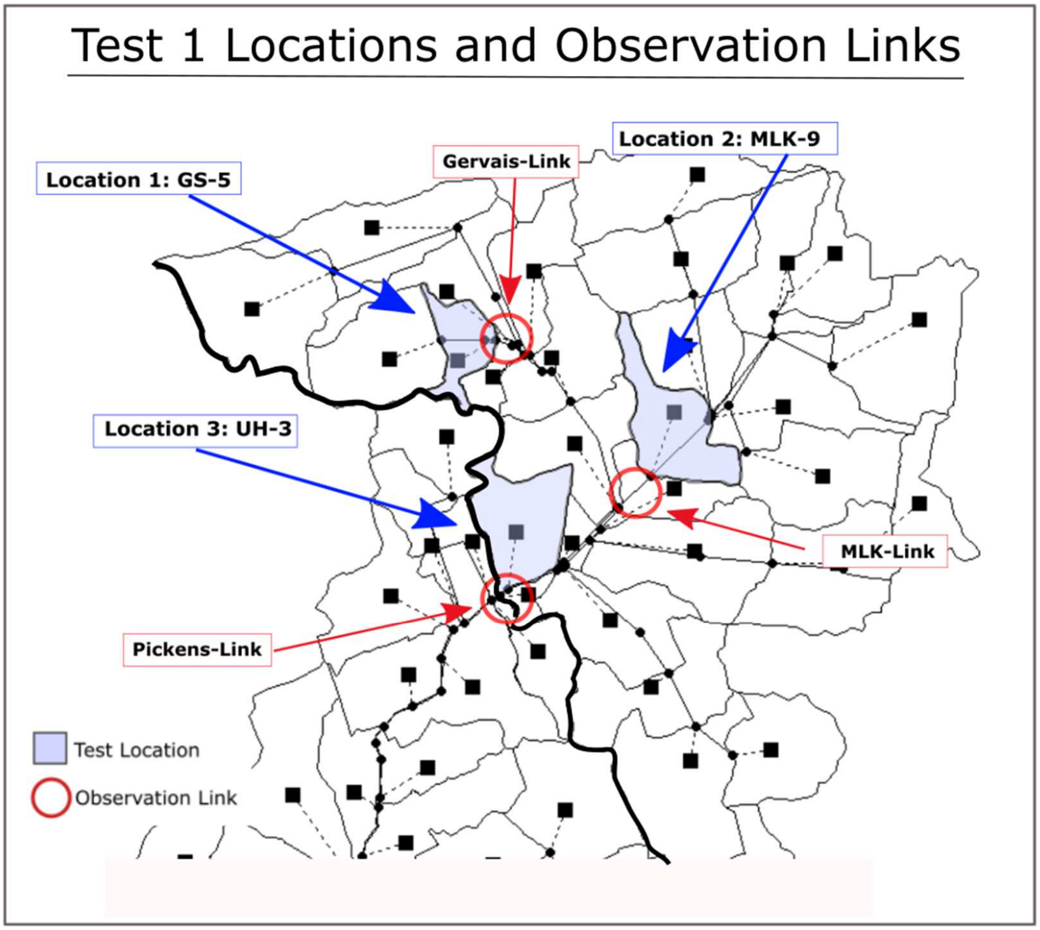

Figure 2.6. Test 1 locations and observation links. Each location models individual stormwater controls. Runoff results are observed at the test location, and Qpk is observed at the link closest to the test location as well as the Pickens-Link for all scenarios

Sizing of detention ponds and bioretention cells were based on unit storage costs ($/m3) of $240.04 and $545.70 for detention ponds and bioretention structures, respectively (Table 2.4) (Mateleska, 2016). Costs of bioretention structures were computed based on the volumes of their storage areas only, not including void space within the soil layer or surface ponding depth. These estimates indicate that the initial installation of bioretention cells cost more than twice that of detention ponds on a dollar-per-volume of storage basis. Each iteration of T1 was performed twice based on initial investment assumptions, with Investment Level 1 (IL1) and Investment Level 2 (IL2) equaling \$100,000 and \$200,000 respectively., Based on this doubling of investment between IL1 and IL2, the second model run in each case had twice the storage volume as the first. For example, the given unit storage costs, IL1 and IL2 resulted in detention pond storage of 417 m3 and 832 m3 and bioretention cell storage of 183 m3 and 367 m3, respectively. Changes in Qpk (m3/s) for all Test 1 scenarios are observed at the closest observation link to the location being tested and at the downstream Pickens-Link, which acts as a control observation point for all T1 locations. Test 1 also observes changes in total runoff volume (m3) for bioretention only within the test subcatchment of implementation.

Table 2.4. Unit Prices Per Cubic Meter of Storage in 2016 Dollars (Source: Mateleska, 2016)

Management Strategy	Unit Costs (\$/m^3) – 2016 Dollars	Volume (m^3) / \$100,000
Detention Pond	\$240.04	417
Bioretention Cell	\$545.70	183

Detention ponds were designed in SWMM as basic storage units with a depth/area relationship defined using the tabular curve method. Subcatchments within the SWMM model are set up to route all overland flow to an outlet node with no routing between subcatchments due to drainage divides. For this reason, modeled detention ponds were designed to function at the outlet node for their respective subcatchments, with pond inflows conveyed through a conduit to the original outlet node of the subcatchment. Based on a sensitivity analysis, outlet conduits for detention ponds used a 46-cm (18”) outlet pipe to be small enough to store and delay conveyance of inflows while draining outflows quickly enough for the pond to have available storage for storm events larger than the one chosen for this study.

Bioretention cells in SWMM include three vertical layers (Figure 2.7)—a surface layer where ponding can occur up to a specified height, a soil media layer, and a storage layer with the option of loss via infiltration, a drain outlet, or both (Rossman, 2010).

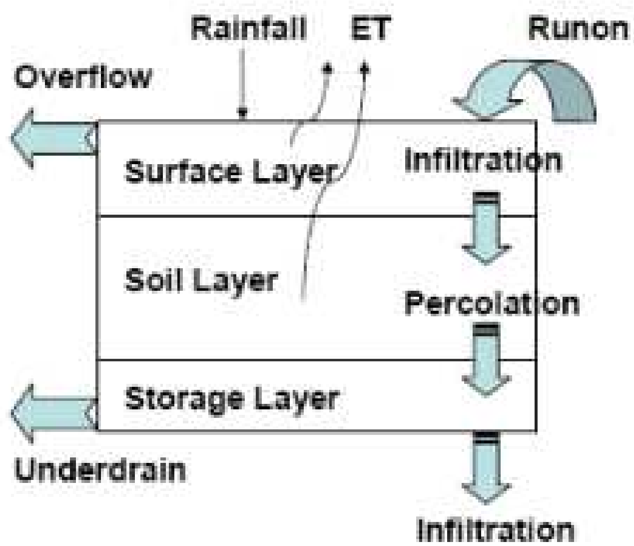

Figure 2.7. SWMM representation of bioretention (Rossman, 2010)

Bioretention cell parameters were selected largely based on existing literature (Table 2.5) (Lucas, 2005; Rossman, 2010; 2015). Storage capacity was calculated based on void space within the storage layer and did not include the surface or soil layers. The storage layer was assumed to have a depth of 1 m and a void ratio of 0.75, resulting in an area of 245 m2 at the $100,000 investment level. SWMM allows for LID to be designed separately and then applied to the desired subcatchment(s), with the ability to apply multiple identical units to the same subcatchment. For this reason, a bioretention cell design based on the assumed $100,000 initial investment was treated as the base unit, and a doubling of assumed investment level for IL2 was represented by an application of an additional identical unit, therefore doubling the storage. Bioretention cell drains were positioned 600 mm from the bottom of the bioretention cells so infiltration is the primary means of storage loss and drainage to the SS network occurs only for larger events where LID storage exceeds 60% of capacity. The proportion of sheet flow from impervious surfaces that flows into the bioretention cell was set at 25%, based on a sensitivity and optimization to reduce runoff and Qpk while leaving storage available for larger storms.

Stormwater management Test 2 compares localized and distributed applications for their effectiveness of LID in reducing peak discharge within SS and channel links. Test 2A (T2A) simulated a scenario with bioretention cells localized within the Gregg St. subwatershed--which is characterized by much higher PIA than the RBW average—and were placed at subcatchments GS-1, GS-2, and GS-5 (Figure 2.8). Test 2B (T2B) simulated a scenario where bioretention cells were spread throughout the upper watershed above Pickens in the same subcatchment locations observed in Test 1: GS-5, MLK-9, and UH-3 (Figure 2.9). Both the localized and the distributed scenarios applied identical bioretention units (using the previous design of 183 m3 per cell) to three different locations, for an assumed initial investment level of $300,000 total (IL1). T2A and T2B were each run a second time, adding another identical bioretention cell to each location to represent a doubling of both initial assumed investment and bioretention storage. Because the base bioretention cell placed at each location was identical to the assumed $100,000 investment from T1, Test 2 observed initial investment levels of $300,000 (IL1) and $600,000 (IL2). Both the localized and distributed scenarios modeled the cumulative effects of implementing three cells at once on Qpk (m3s/) at the five observation links shown in Figures 2.8 and 2.9, with emphasis on Qpk at the downstream Pickens-Link.

Table 2.5. SWMM Bioretention Parameters (Source: Lucas, 2005; Rossman, 2010; 2015)

Surface Layer	Value	Soil Layer	Value	Storage Layer	Value	Drain	Value
Berm Height (mm)	450	Thickness (mm)	750	Thickness (mm)	1000	Flow Coefficient	1
Vegetative Volume (Fraction)	0.1	Porosity (volume fraction)	0.5	Void Ratio (voids/solids)	0.75	Flow Exponent	0.5
Surface Roughness (Manning's n)	0.24	Field Capacity (volume fraction)	0.105	Infiltration Rate (mm/hr)	12.7	Offset Height (mm)	600
Surface Slope (percent)	1	Wilting Point (volume fraction)	0.047	Clogging Factor	0		
		Conductivity (mm/hr)	29.97				
		Conductivity Slope	10				
		Suction Head (mm)	60.97				

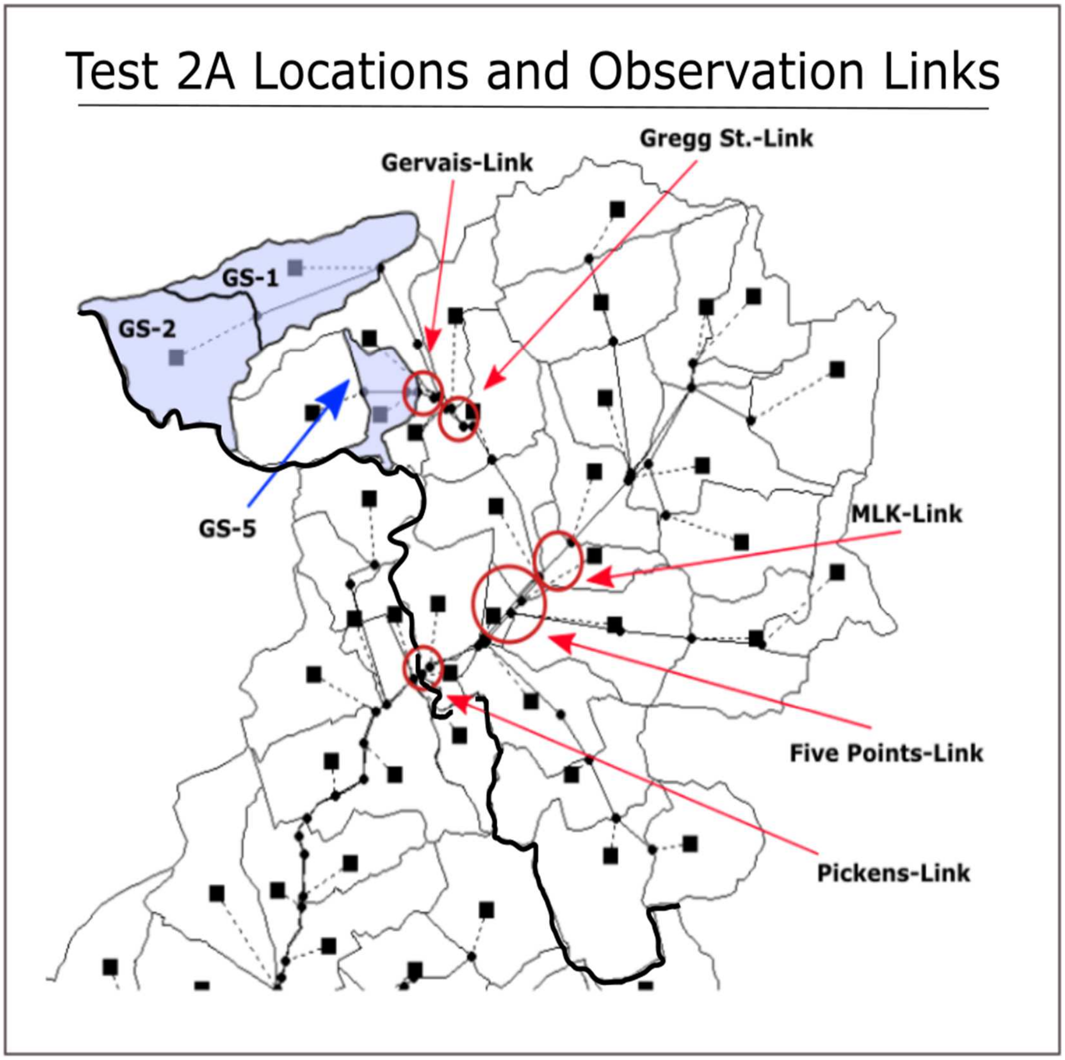

Figure 2.8. Test 2A (Localized) bioretention locations and observation links

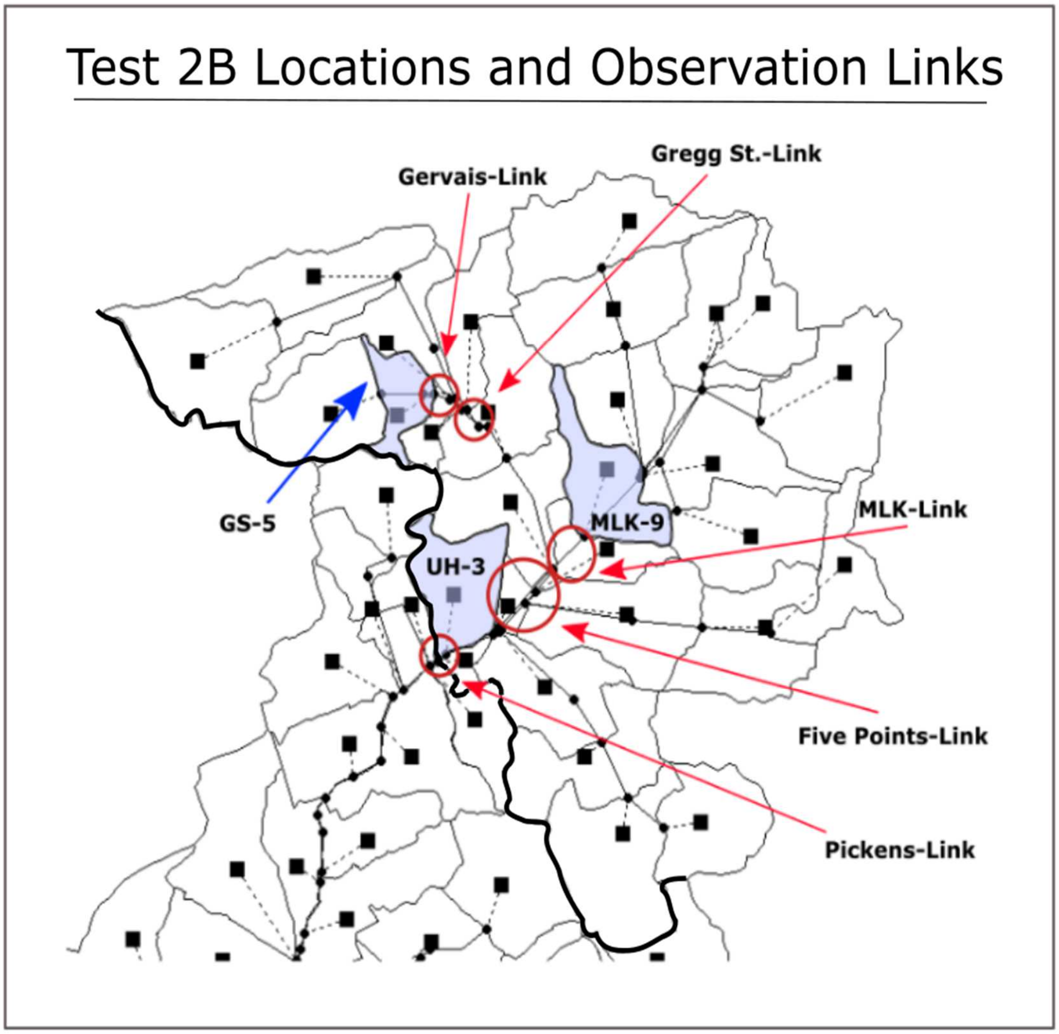

Figure 2.9. Test 2B bioretention locations and observation links

CHAPTER 3

RESULTS

3.1. Model Calibration and Validation

Sensitivity analysis identified subcatchment width, impervious detention storage, Manning's roughness (n) for impervious surfaces, and Manning's roughness (n) for channel links as sensitive parameters that were useful for model calibration. Initial hydrographs from the uncalibrated model demonstrated general tendencies for simulated stormwater flows to arrive earlier than observed, to overestimate peak flow, and to protract and overestimate flows in receding limbs. Calibrations involved increasing Manning's (n) for impervious surfaces and increasing impervious detention storage to slow the delivery of storm runoff and increase initial abstraction. These measures slowed storm hydrograph rising limbs and reduced peak flows to values closer to observations but produced receding limbs that were still too high. Therefore, the conveyance of storm water in distant subcatchments was accelerated by increasing subcatchment widths, which reduced the receding limbs to values in accordance with observations. The Three resulting NSE values for the three calibration storms at both gages were all >0.7 (Table 3.1). Three independent storm events used to validate the model also provide NSE values >0.7. NSE values for calibration and validation suggest that the SWMM for RBW is valid for flows within the range of those used in the calibration; that is, moderate magnitude, within-bank storm flows. Figures 3.1 and 3.2 show calibration results from the base storm 5/29/2017 (Storm

Event Code: C2) at the Above Gervais and Above Pickens gages.

Table 3.1. Nash Sutcliffe Efficiency for Observed Storm Events

	Calibration NSE			Validation NSE		
Gage	**22-May-17**	**29-May-17**	**13-Aug-16**	**24-May-17**	**25-Jul-17**	**16-Oct-17**
Gervais	0.773	0.931	0.838	0.953	0.742	0.759
Pickens	0.712	0.801	0.880	0.894	0.933	0.761

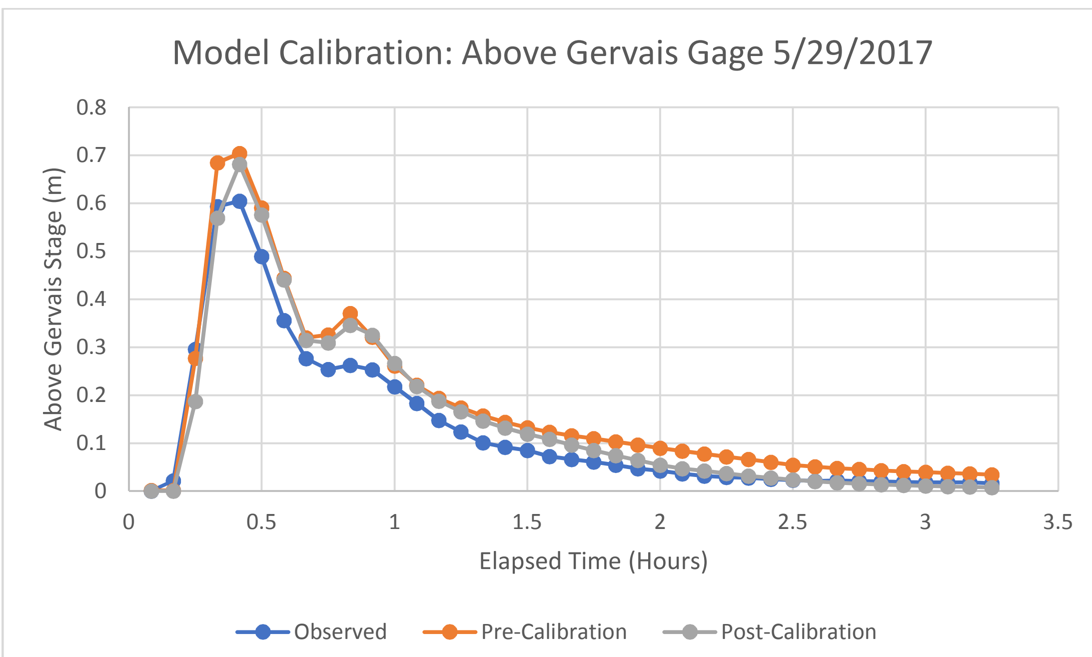

Figure 3.1. Above Gervais calibration results for 5/29/2017 (C2)

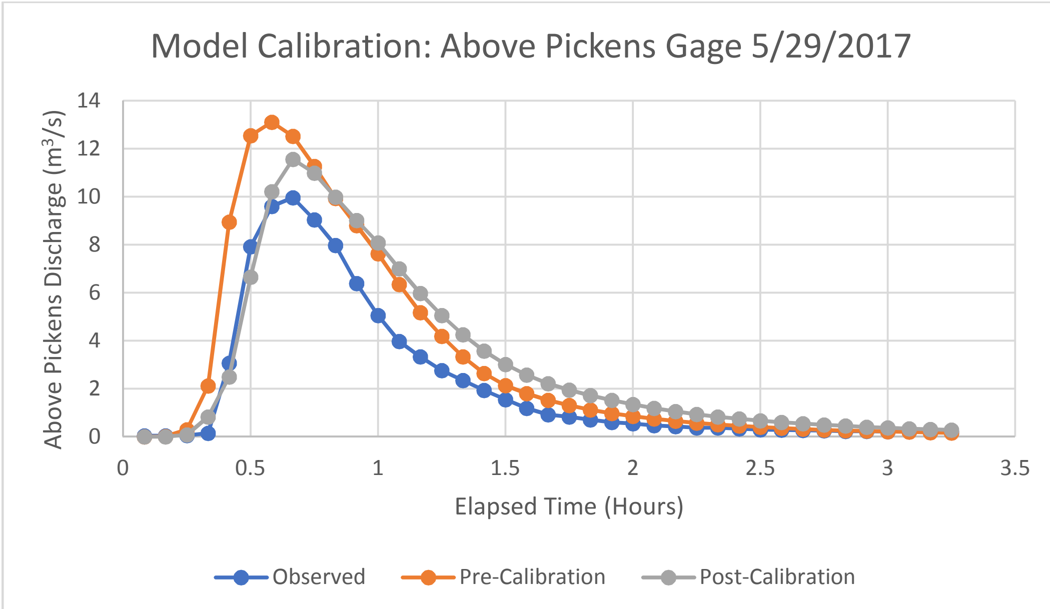

Figure 3.2. Above Pickens calibration results for 5/29/2017 (C2)

3.2. Test 1 Stormflow Reductions: Conventional and LID Strategies

Each of the Test 1 scenarios simulate a single stormwater management type concentrated in a single sub-basin. Peak discharge (Qpk) results were measured from specific channel or conduit links in the SWMM and represent all contributing flows above that point, which were reported as change in Qpk (m3/s), as well as percent change from initial values (Table 3.2). When management treatments are isolated, both detention ponds and bioretention cells influenced Qpk, although the effectiveness in stormwater reductions differed between scenarios. Both types of management were most effective when positioned at Location 1 (GS-5). The largest reductions were locally at the Gervais-Link, where detention pond Qpk percent reductions were -11.2% and -17.4% for Investment Level 1 (IL1) and Investment Level 2 (IL2).

Table 3.2. Test 1 Change in Peak Discharge Rates

Test Location	Management Control	Observation Link	Investment Level 1 (IL1): $100,000		Investment Level 2 (IL2): $200,000	
			Change in Qpk (m^3/s)	% Change	Change in Qpk (m^3/s)	% Change
Test Location 1 (T1L1): GS-5						
	Detention Pond					
		Gervais-Link	-0.26	-11.2%	-0.40	-17.4%
		Pickens-Link	-0.17	-1.5%	-0.23	-2.0%
	Bioretention					
		Gervais-Link	-0.14	-6.0%	-0.14	-6.0%
		Pickens-Link	-0.07	-0.6%	-0.09	-0.7%
Test Location 2 (T1L2): MLK-9						
	Detention Pond					
		MLK-Link	0.12	2.5%	0.07	1.3%
		Pickens-Link	0.01	0.1%	-0.04	-0.3%
	Bioretention					
		MLK-Link	-0.01	-0.2%	-0.04	-0.9%
		Pickens-Link	-0.04	-0.4%	-0.06	-0.6%
Test Location 3 (T1L3): UH-3						
	Detention Pond					
		Pickens-Link	0.15	1.3%	0.13	1.2%
	Bioretention					
		Pickens-Link	0.00	0.0%	0.01	0.1%

This scenario of detention pond placement at GS-5 also produced the greatest reductions in Qpk and percent change in Qpk downstream at the Pickens gage, although reductions were still modest ranging from -1.5% to -2% for IL1 and IL2. In comparison, bioretention Qpk reductions at the Gervais-Link were only 6% at both investment levels. The lack of increased reduction with a doubling in bioretention volume suggests that the increased storage capacity at that one subcatchment based on a doubled initial investment is not needed for the moderate magnitude storms examined in this study. This effect is only seen with the increase in investment level at GS-5, likely due to its lower drainage area compared to Location 2 (MLK-9) and Location 3 (UH-3). Implementation of detention ponds within the Gervais subcatchment led to Qpk reductions at Pickens that were more than twice the reductions achieved by bioretention at both investment levels, with the maximum reduction of 2% resulting from IL2.

Test 1 simulations of detention ponds at Location 2 (MLK) generally failed to produce reductions in Qpk at both the MLK-Link and Pickens-Link. The only reduction occurs downstream at Pickens at IL2. In every other case, detention pond implementation increased Qpk within the MLK-Link by 2.5% and 1.3%, respectively, at IL1 and IL2. In either case, the pond is never more than 53% filled, suggesting that the limitation is in the design of the pond. This increased Q, however, reveals a danger with conventional storage methods that may temporarily store flows and release them later when stormwater is arriving from distant catchments, adding to the peak discharge. Bioretention cells at Location 2 barely reduced flow rates at the MLK-Link and Pickens-Link at IL1, although at IL2 the local reduction at MLK of 0.2% was smaller than the downstream reduction at Pickens of 0.4%. At IL2 the Qpk reduction at the MLK-Link increased to 0.9% (the largest of any Location 2 percent Qpk reduction) but percent reductions downstream at the Pickens-Link were only 0.6%.

Simulations at Location 3 (UH-3) showed that neither detention pond nor bioretention controls were highly effective in reducing Qpk at the Pickens-Link. Detention ponds caused an increase of ~1.2% in Qpk at both investment levels. In contrast to Location 2 (MLK) detention pond results, Location 3 (UH-3) pond results suggest a need for greater storage capacity in order to be efficient, as ponds were filled to 88% and 100% capacity, for IL1 and IL2 respectively. Location 3 bioretention reductions were the least effective for both total runoff volume and Qpk and were negligible at both investment levels.

Results from all three of the Test 1 location scenarios show that the local and downstream discharge or percentage reductions for both detention ponds and bioretention structures are greatest when implemented at Location 1 (GS-5) and designed based on an assumed $200,000 initial investment (Figure 3.3 and 3.4). This may be explained by its higher PIA or lower drainage area and therefore total runoff volume as compared to both Location 2 (MLK-9) and Location 3 (UH-3). Although changing the location, type of treatment, or additional allocations had substantial local effects under Test 1 scenarios, the relatively small amount of variation in peak discharge responses at Pickens suggests that the effectiveness of the different scenarios in generating reductions in Qpk downstream are limited.

Test 1 simulations of bioretention cells show local reductions in runoff volume (106 m3) at all three locations and these reductions increase when doubling the assumed initial investment from IL1 to IL2 (Table 3.3). Percent reductions were greatest at GS-5 for both investment levels, with reductions of 21.7% and 25.3%, respectively, although greater volumetric and similar percent reductions were achieved with the MLK scenarios. Simulations of bioretention Location 3 (UH-3) showed the lowest percent reduction in total runoff volume, with reductions within the subcatchment of 13% at the IL1 and 19.2% at IL2, although these reductions were associated with the greatest volumetric reductions in Qpk (-0.25 to -0.37 m3/s). Simulations of detention ponds did not show changes in runoff volume due to the way in which they are designed within the SWMM. For this reason, the 0% reductions in runoff volume from detention ponds were omitted and not analyzed further.

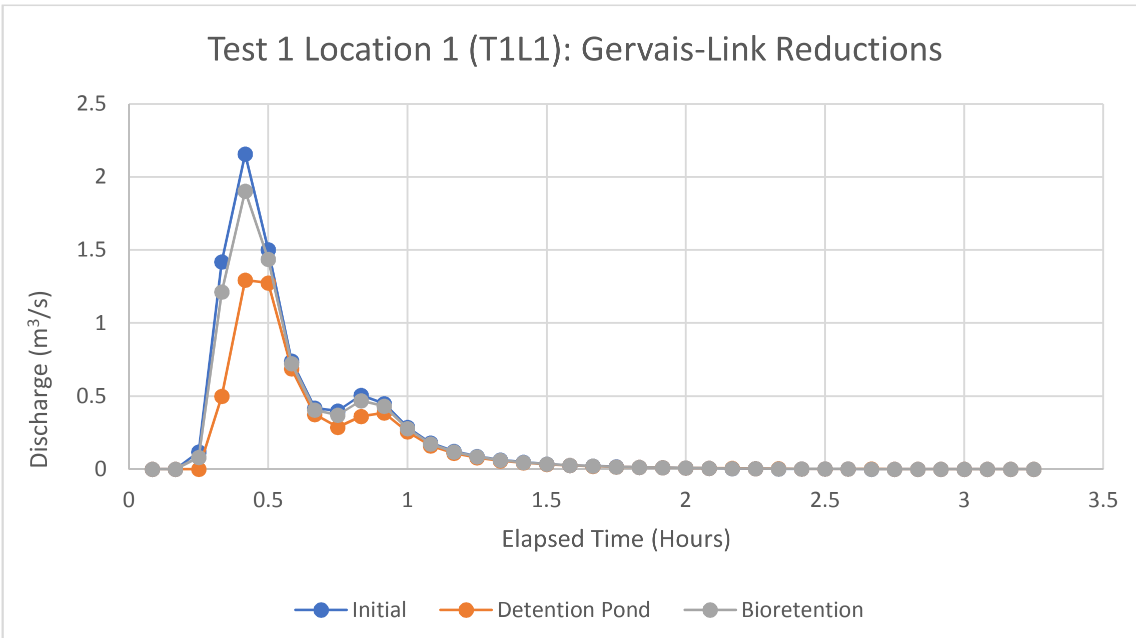

Figure 3.3. Reductions in discharge at the Gervais-Link

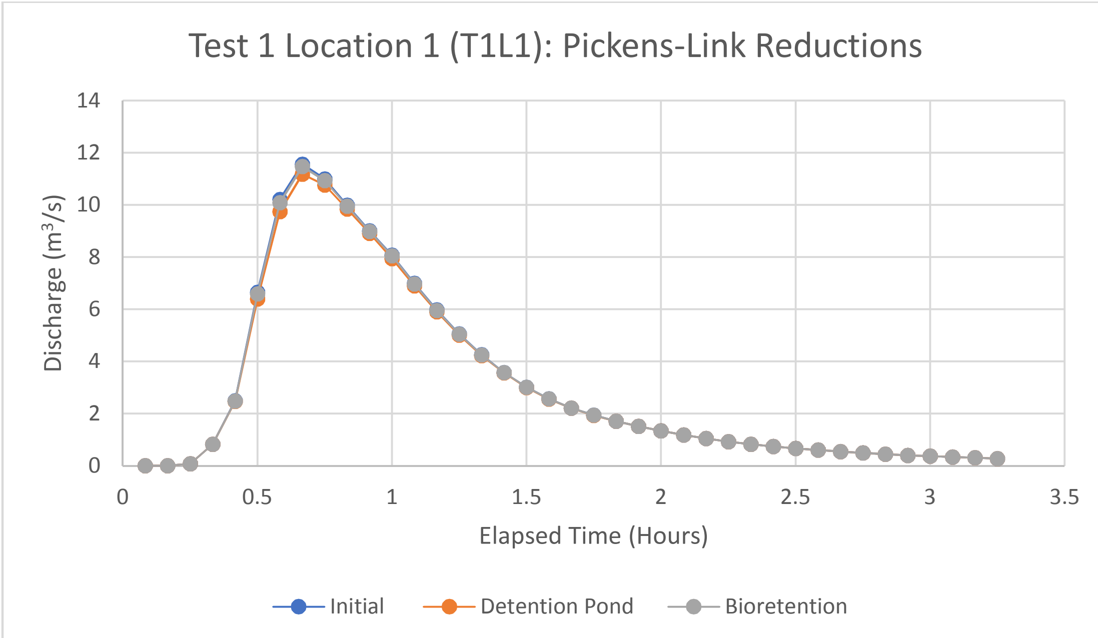

Figure 3.4. Reductions in discharge at the Pickens-Link

Table 3.3. Test 1 Change in Runoff Volume From Bioretention

	Investment Level 1 (IL1): $100,000		Investment Level 2 (IL2): $200,000	
Test Location	Change in Runoff Volume (10^6 m^3)	% Change	Change in Runoff Volume (10^6 m^3)	% Change
Test Location 1 (T1L1): GS-5	-0.18	-21.7%	-0.21	-25.3%
Test Location 2 (T1L2): MLK-9	-0.21	-20.0%	-0.26	-24.8%
Test Location 3 (T1L3): UH-3	-0.25	-13.0%	-0.37	-19.2%

3.3. Test 2 Stormflow Reductions: Localized and Distributed LID

Test 2 scenarios group LID stormwater management treatments in subcatchments to compare localized (Test 2A) versus distributed (Test 2B) bioretention approaches. Resulting Qpk values were observed at five channel or conduit links within the upper watershed, each representing flow from all contributing links above (Table 3.4).

Table 3.4. Test 2 Change in Peak Discharge Rates

	Investment Level 1 (IL1): $300,000		Investment Level 2 (IL2): $600,000	
Scenario	Change in Qpk (m^3/s)	% Change	Change in Qpk (m^3/s)	% Change
LOCALIZED (GS-1, GS-2, and GS-5)				
Conduit/Channel				
Gervais	-0.14	-6.0%	-0.14	-6.0%
Gregg St	-0.52	-13.7%	-0.55	-14.5%
MLK	0.00	0.0%	0.00	0.0%
Five Points	-0.36	-5.5%	-0.37	-5.7%
Pickens	-0.27	-2.4%	-0.37	-3.2%
DISTRIBUTED (GS-5, MLK-9, and UH-3)				
Conduit/Channel				
Gervais	-0.14	-6.0%	-0.14	-6.0%
Gregg St	-0.06	-1.6%	-0.06	-1.6%
MLK	0.00	-0.1%	-0.04	-0.8%
Five Points	-0.09	-1.4%	-0.11	-1.7%
Pickens	-0.11	-0.9%	-0.13	-1.1%

Of particular importance are the Pickens-Link due to its position downstream of all subcatchments with bioretention scenarios, as well as the Five Points-Link, which is located within the Five-Points commercial district that is associated with frequent flooding events. These tests represent four bioretention scenarios in which three LID treatments are lumped within the Gregg St subwatershed (Investment Level 1 (IL1) of $300,000 and Investment Level 2 (IL2) of $600,000) or are distributed between three sub-basins (at both IL1 and IL2). Both localized and distributed patterns of bioretention cells resulted in a change in peak discharges, although results varied between scenarios and investment levels (Table 3.4).

For the localized scenario (Test 2A), reductions were greatest at the Gregg St.-Link for both investment levels, with a 13.7% reduction at IL1 and 14.5% reduction at IL2. These results are consistent with initial expectations, as the Gregg St.-Link is at the confluence of the three contributing LID subcatchments, GS-1, GS-2, and GS-5. Reductions in Qpk at the Gervais-Link remained at a constant 6% for both Test 2A and Test 2B at both investment levels, which is consistent with findings from Test 1, which had the same configuration. Larger storms, however, would likely show an increase in effectiveness from a larger investment as is the case with the other subcatchments. Qpk was unchanged at the MLK-Link for both investment scenarios, as was expected because the link received no treatment by the Test 2A scenarios.

For the distributed bioretention scenario (Test 2B), the percent reduction in Qpk at the Gervais-Link was the largest observed for both investment levels, again at 6% for both investments. The MLK-Link showed the lowest reductions under this scenario at both investment levels, with reductions of only 0.1% and 0.8% for IL1 and IL2, respectively. These findings are consistent with those from Test 1 in which bioretention resulted in minimal reductions in local Qpk when placed at MLK-9. Aside from the Gervais-Link, all links observed in the distributed scenario demonstrated increased reduction of Qpk with a doubled initial investment assumption, but reductions were modest (≤ -1.7%).

A comparison of both spatial scenarios shows the localized pattern was more effective in Qpk reduction at all observation links except at the MLK-Link, where Qpk reductions are minimal. At Five Points, a heavily commercialized zone that is prone to high flood damages, the localized pattern has clear advantages over the distributed pattern in Qpk reduction at both investment levels. There, localized scenario reductions provided a reduction of Qpk 4.1% greater than distributed scenario at IL1 and 4% greater at IL2 (Figure 3.5). It should be noted however, that one third of the distributed treatment is downstream of the Five Points-link, so it is expected that the scenario with treatment localized upstream should be more effective. A good way to assess the cumulative effectiveness of the two LID spatial orientations modeled in Test 2, is by observing the reduction of Qpk at Pickens, where flows are contributed from all three LID implemented subcatchments. At Pickens the localized grouping of bioretention cells in the Gregg St sub-watershed is more effective in reducing Qpk with percent decreases of 2.4% versus 0.9% at IL1 and 3.2% versus 1.1% at IL2 for localized versus distributed reductions, respectively, although these reductions are modest in relation to the total volume of flow at the Pickens-Link, hydrographs of the cumulative effects of both scenarios at Investment Level 2 ($600,000) show these relationships (Figures 3.6).

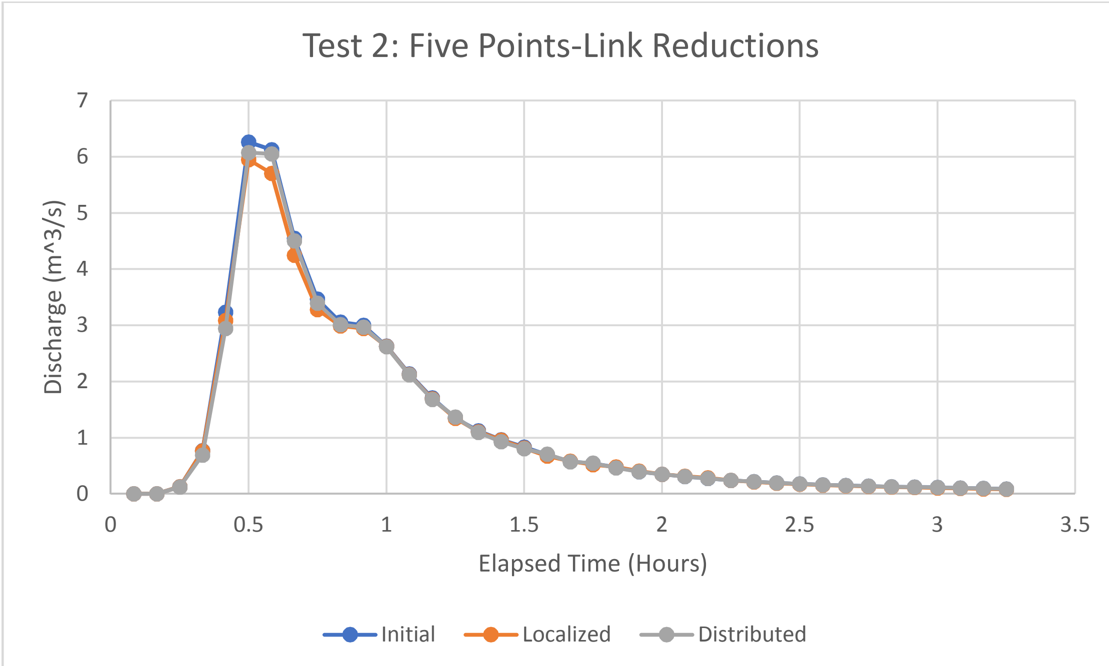

Figure 3.5. Five Points-Link discharge for initial conditions, Test 2A, and Test 2B

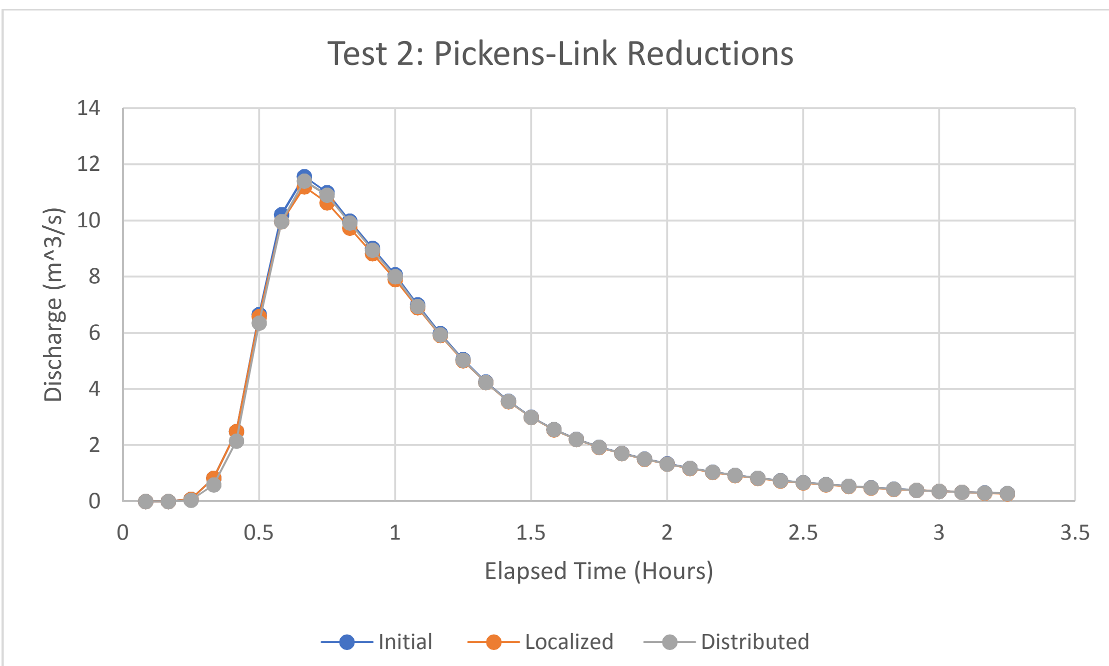

Figure 3.6. Pickens-Link discharge for initial conditions, Test 2A, and Test 2B

3.4. Approaches to Economic Analysis of Management Strategies

The stormwater management scenarios modeled in this study were designed using unit storage costs ($/m3) for only initial design and construction costs of detention ponds and bioretention cells (Mateleska, 2016). A long-term analysis of economic efficiencies, however, requires analysis of annual expenditures over the life-cycle of the stormwater control. Using estimates of both initial investment costs and annual estimated maintenance expenses from Houle et al. (2013), the following framework can be used to calculate the length of time required for a bioretention cell to become more cost effective than a detention pond by calculating the value of n (years) for which detention and bioretention expenses are equivalent:

$$LIDinitial + LIDo\&m(n) = PONDini + PONDo\&m(n) \quad (3)$$

Where LIDinitial is bioretention design and construction cost, LIDO&M is annual bioretention maintenance cost, PONDinitial is detention pond design and construction cost, PONDO&M is annual detention pond maintenance cost, and n is the number of years until total investment in both management strategies are equal, at which LID becomes cheaper over the remainder of the life-cycle. This framework was applied with estimates of bioretention initial and annual costs ($/acre treated) of $22,500 (initial) and $1,210 (annual) and detention pond initial and annual costs of $63,200 (initial) and $4,940 (annual) (Table 3.5). These calculations indicate that investments in controls of identical storage capacity would be equal after 18.6 years.

Table 3.5. Stormwater Control Initial Capital and O&M Cost

Management Strategy	Capital Cost ($/acre treated)	Annual O&M ($/acre treated)
Detention Pond	$40,700.00	$6,150.00
Bioretention Cell	$63,200.00	$4,940.00

CHAPTER 4

DISCUSSION

Reductions in stormwater peak flows come at a high cost. All 12 of the Test 1 scenarios (LID vs. conventional; $100,000 vs. $200,000 levels; and three sites) resulted in relatively small percent changes in peak flow rates at Pickens, which ranged from a decrease of 2.0% to an increase in 1.3% m3/s (-1.3% to 2.0%). The greatest reduction in Qpk achieved downstream at the Pickens gage site under any of the modeled scenarios was -2.0% at an initial cost of $200,000 or -3.2% at an initial cost of $600,00 (Tables 3.2 and 3.4). These costs do not include life-cycle costs such as operating costs or maintenance that tend to be cheaper for LID (Mateleska, 2016; Houle et al., 2013). Nor is it clear that reductions of 3.2% would be enough to counter projected increases in stormflow that could result from future land-use or climate changes. The economic analysis presented here assumes that a centralized program will be tasked with paying for the cost of LID, but much may be achieved through widespread applications by individuals distributed through the watershed. The high price of ex post facto, government-sponsored stormwater management measures to reduce discharge suggests that it is economically worthwhile for local governments to seek voluntary participation and to establish regulations to prevent further reductions in infiltration and increases in runoff generation. Citizens should be encouraged with education and incentive programs to install green infrastructure such as rain barrels, pervious driveways and patios, or disconnecting rooftops from impervious surfaces. Participation can also be ensured by requiring green infrastructure in future developments. Thus, initial economic analysis may show that conventional is cheaper than LID on the basis of $/m3 reductions in the short term, but it's still expensive, and voluntary efforts may greatly reduce the costs of distributed approaches to stormwater management.

CHAPTER 5

CONCLUSIONS

The first set of tests, based on twelve model runs, was designed to compare the effects of conventional detention ponds and bioretention cells on surface runoff volume and peak discharge rates of concentrated flows when modeled in different locations throughout the upper RBW. Test 1 demonstrated that—contrary to the first hypothesis—conventional stormwater management by construction of detention ponds at Location 1 was more effective on an initial unit-cost basis than bioretention cells both locally and downstream at the Pickens-Link. Bioretention was, however, effective in reducing local runoff volumes, therefore Hypothesis H1B was accepted. The analysis did not include life-cycle costs that are usually cheaper for LID, and extended only to moderate magnitude floods. In addition, detention ponds exacerbated peak discharges in some cases. In accordance with the second hypothesis, grouping of management strategies upstream in the highly impervious Gregg Street subwatershed (Location 1; GS-5) was most effective in reducing Qpk both locally and downstream at the Pickens streamflow gage. Presumably, this maximum reduction was due to the above-average PIA of this basin. Bioretention modeled at Location 1 (GS-5) showed no change in Qpk reduction with a doubled initial investment, suggesting that storage was sufficient at IL1 ($100,000), although detention ponds at Location 1 were more than twice as effective as bioretention in Qpk reduction locally and downstream at the lower investment level. Bioretention cells at Location 2 (MLK-9) resulted in minimal downstream reductions in Qpk for both strategies at both investment levels, while neither strategy was effective in reducing Qpk when placed at Location 3 (UH-3) for either investment level.

Test 2 modeling scenarios compared the effectiveness of grouping LID into various spatial configurations within the watersheds. Specifically, these scenarios tested the effect of clustering LID in a small, high priority area versus distributing an equal amount of LID storage across the upper RBW. Simulations indicate that clustering LID in the Gregg Street basin was more effective in reducing peak discharges at all observation links except the MLK-Link, which received no treatment. Focusing remedial measures within the highly impervious and heavily urbanized Gregg Street sub-basin was more effective than a distributed pattern in reducing stormwater in the Five Points commercial district, an area within the watershed with a history of flooding. Downstream at the Pickens-Link, localized use of LID within the Gregg Street subwatershed was more than twice as effective in reducing Qpk at IL1 ($300,000) and almost three times as effective at IL2 ($600,000) than the distributed approach. Based on these results, both Hypothesis H2A and H2B were accepted.

Analysis of economic efficiencies should go beyond a comparison of initial capital required for design and construction and should include assessment of life-cycle costs (LCC). In general, conventional stormwater management has proved to be cheaper based on initial cost, but LCC analysis has demonstrated a trend toward reduced annual O&M expenses for LID as opposed to conventional strategies. A comparison of LCC based on

Houle et al. (2016) demonstrates this, as a comparison of $/acre treated for both strategies reveals that LID management becomes cheaper after an estimated 18.6 years.

Within the range of treatments and distributions tested, results from this study show that (1) storage from both conventional and LID practices can reduce peak discharge rates for moderate magnitude storms, although in this case, detention ponds outperformed LID; (2) the application of LID strategies such as bioretention cells can be more flexible in scaling and pattern of deployment; and (3) clustering LID in priority locations such as highly urbanized headwaters characterized by above average PIA that generate large volumes of runoff can be a more effective strategy than distributing them across a watershed. These findings, when considered with life-cycle costs and benefits of various stormwater management strategies, as well as consideration of the potential for public participation through citizen use of LID strategies, suggest that LID is a more flexible and cost-effective means of flood risk reduction in RBW, although conventional strategies can provide immediate cost-effective means of reducing peak flows when sufficient space is available for their implantation.

REFERENCES

ASCE. (1982). *Gravity Sanitary Sewer Design and Construction.* ASCE Manual of Practice No. 60, New York, NY.

———. (1992). *Design & Construction of Urban Stormwater Management Systems.* New York, NY.

Barco, J., Wong, K.M., & Stenstrom, M.K. (2008). Automatic Calibration of the U.S. EPA SWMM Model for a Large Urban Catchment. *Journal of Hydraulic Engineering,* 134(4): 466-474. https://doi.org/10.1061/(ASCE)0733-9429(2008)134:4(466)

Bohman, L.R. (1992). Determination of Flood Hydrographs for Streams in South Carolina: Volume 2. Estimation of Peak-Discharge Frequency, Runoff Volumes, and Flood Hydrographs for Urban Watersheds. Water-Resources Investigations Report 92-4040. Retrieved from https://pubs.usgs.gov/wri/1992/4040/report.pdf

Bowman, T., Tyndall, J.C., Thompson, J., Kliebenstein, J., & Collet, J.P. (2012). Multiple approaches to valuation of conservation design and low-impact development features in residential subdivisions. *Journal of Environmental Management*, 104: 101-113. https://doi.org/ 10.1016/j.jenvman.2012.02.006

Chui, T.F.M., Liu, X., & Zhan, W. (2015). Assessing cost-effectiveness of specific LID practice designs in response to large storm events. *Journal of Hydrology*, 533: 353-364. https://doi.org/ 10.1016/j.jhydrol.2015.12.011

Davis, A.P. (2005). Green engineering principles promote low-impact development. *Environmental Science & Technology*, 39 (16):338A-344A. https://doi.org/10.1021/es053327e

Dongquan, Z., Jining, C., Haozheng, W., Qingyan, T., Shangbing, C., & Zheng, S. (2009). GIS-Based Urban Rainfall-Runoff Modeling Using an Automatic Catchment-Discretization Approach: A Case Study in Macau. *Environmental Earth Sciences*, 59(2):465-472. https://doi.org/10.1007/s12665-009-0045-1.

Elliott, A.H., & Trowsdale, S.A. (2007). A review of models for low impact urban stormwater drainage. *Environmental Modelling & Software*, 22: 394-405. https://doi.org/10.1016/j.envsoft.2005.12.005

Environmental Protection Agency. (2013). Case Studies Analyzing the Economic Benefits of Low Impact Development and Green Infrastructure Programs. Available at https://www.epa.gov/sites/production/files/2015-10/documents/lid-gi-programs_report_8-6-13_combined.pdf

Houle, J.J., Roseen, R.M., Ballestero, T.P., Puls, T.A., & Sherrard, J. (2013). Comparison of Maintenance Cost, Labor Demands, and System Performance for LID and Conventional Stormwater Management. *Journal of Environmental Engineering*, 139: 932-938.

Hunt, W.F., Jarrett, A.R., Smith, J.T., & Sharkey, L.J. (2006). Evaluating Bioretention Hydrology and Nutrient Removal at Three Field Sites in North Carolina. *Journal of Irrigation and Drainage Engineering*, 132(6): 600-608, https://doi.org/10.1061/(asce)0733-9437(2006)132:6(600).

Jacobson, C.R. (2011). Identification and quantification of the hydrological impacts of imperviousness in urban catchments: A review. *Journal of Environmental Management*, 92, 1438-1448. https://doi.org/ 10.1016/j.jenvman.2011.01.018

James, L.D., & S.J. Burges, (1982). Selection, Calibration, and Testing of Hydrologic Models. In: Hydrologic Modeling of Small Watersheds. *American Society of Agricultural Engineers.* St. Joseph, Michigan, pp. 437-472. Retrieved from https://www.researchgate.net/publication/309736708_Selection_Calibration_and_Testing_of_Hydrologic_Models_Chapter_11_in_CT_Haan_HP_Johnson_and_DL_Brakensiek_Eds_Hydrologic_Modeling_of_Small_Watersheds_American_Society_of_Agricultural_Engineers_Hydrolo

Leopold, L.B. (1968). Hydrology for Urban Land Planning—A Guidebook on the Hydrologic Effects of Urban Land Use. Geologic Survey Circular 554. Retrieved from https://pubs.usgs.gov/circ/1968/0554/report.pdf

Lucas, W.C. (2005). Green Technology: The Delaware Urban Runoff Management Approach. Prepared for Delaware Department of Natural Resources and Environmental Control Division of Soil and Water Conservation. Retrieved from http://arkansaswater.org/319/pdf/05-1100%20Urban%20Low%20Impact%20Appendix%202.pdf

Mateleska, K. (2016). Methodology for developing cost estimates for Opti-Tool. EPA Memorandum. Available at https://www3.epa.gov/region1/npdes/stormwater/ma/green-infrastructure-stormwater-bmp-cost-estimation.pdf

McCormick Taylor and KCI Technologies. (2016). Rocky Branch Watershed Assessment (Prepared for the City of Columbia). Available at https://www.columbiasc.net/depts/utilities-engineering/docs/sw/watershedplans/ws-plan-rocky-branch-2016-may20.pdf

McCuen, R. et al. (1996). *Hydrology*. FHWA-SA-96-067, Federal Highway Administration, Washington, DC.

Meierdiercks, K.L., Smith, J.A., Baeck, M.L., & Miller, A.J. (2010). Analyses of Urban Drainage Network Structure and Its Impact on Hydrologic Response. *Journal of the American Water Resources Association*, 46(5):932-943. https://doi.org/10.1111/j.1752-1688.2010.00465.x

Moriasi, D.N., Arnold, J.G., Van Liew, M.W., Bingner, R.K., Harmel, R.D., & Veith, T.L. (2007). Model Evaluation Guidelines for Systematic Quantification of Accuracy in Watershed Simulations. *American Society of Agricultural and Biological Engineers*, 50(3): 885-900.

Morsy, M.M., Goodall, J.L., Shatnawl, F.M., & Meadows, M.E. (2016). Distributed Stormwater Controls for Flood Mitigation within Urbanized Watersheds: Case Study of Rocky Branch Watershed in Columbia, South Carolina. *Journal of Hydrologic Engineering*, 21(11): 05016025-1:10. https://doi.org/10.1061/(ASCE)HE.1943-5584.0001430

Nash, J.E., & Sutcliffe, J.V. (1970). River Flow Forecasting through Conceptual Models Part I — A Discussion of Principles. *Journal of Hydrology,* 10(3):282-290, doi: 10.1016/0022-1694(70)90255-6.

Putnam, A. L. (1972). Effect of Urban Development on Floods in the Piedmont Province of North Carolina. U.S. Gelogical Survey Open – File Report. Retrieved from https://pubs.usgs.gov/of/1972/0304/report.pdf

Rawls, W.J., Brankensiek, D.L., & Saxton, K.E. (1983). Estimation of Soil Water Properties. *J. Hyd. Engr*., 109: 1316-1328.

Rawls, W.J., Ahuja, L.R., Brakensiek, D.L., and Shirmohammadi, A. (1993). *Infiltration and Soil Water Movement*. pp 5.1-5.51, Maidment, D. (Ed.). Handbook of Hydrology. McGraw-Hill, N.Y.

Rosa, J.D., Clausen, J.C., & Dietz, M.E. (2015). Calibration and Verification of SWMM for Low Impact Development. *Journal of the American Water Resource Association*, 51(3). 746-757. https://doi.org/10.1111/jawr.12272

Rossman, L.A. (2010). "Modeling Low Impact Development Alternatives with SWMM." *Journal of Water Management Modeling* R236-11. https://doi.org/10.14796/JWMM.R236-11

———. 2015. *Storm Water Management Model, User's Manual Version 5.1*. EPA/600/R-14/413b. Retrieved from https://nepis.epa.gov/Exe/ZyPDF.cgi?Dockey=P100N3J6.TXT

Schueler, T. (2000). The Importance of Imperviousness. *Watershed Protection Techniques* 1(3): 100-111.

Sitzenfrei, R., Möderl, M., & Rauch, W. (2013). Assessing the impact of transitions from centralised to decentralised water solutions on existing infrastructures e Integrated city-scale analysis with VIBe. *Water Research* 47(20): 7251–7263. https://doi/org/10.1016/j.watres.2013.10.038

Sparkman, S.A., Hogan, D.M., Hopkins, K.G., & Loperfido, J.V. (2017). Modeling watershed-scale impacts of stormwater management with traditional versus low impact development design. *Journal of the American Water Resources Association*, 53: 1081-1094. https://doi.org/10.1111/1752-1688.12559

Swezey, C.S., Fitzwater, B.A., Whittecar, G.R., Mahan, S.A., Garrity, C.P., Gonzalez, W.B.A., & Dobbs, K.M. (2016). The Carolina Sandhills: Quaternary eolian sand sheets and dunes along the updip margin of the Atlantic Coastal Plain province, southeastern United States. *Quaternary Research*, 86(3): 271-286. https://doi.org/10.1016/j.yqres.2016.08.007

U.S. Department of Agriculture. (1994). Soil Survey Geographic (SSURGO) Data Base: Data use information. Soil Conserv. Serv. Available at https://databasin.org/datasets/bdeb4bf4020b42679c37781436fb80eb

U.S. Department of Agriculture Natural Resource Conservation Service. (2016). Web Soil Survey. *Natural Resource Conservation Service*. Available at http://websoilsurvey.nrcs.usda.gov/app/

U.S. Department of Agriculture Soil Conservation Service. (1978). Soil Survey of Richland County, South Carolina. *Natural Resource Conservation Service*. Washington, D.C.

Veseley, E.T., Heijs, J., Stubles, C., & Kettle, D. (2005). The economics of low impact stormwater management in practice – Glencourt Place. *Proceedings of the 4th South Pacific Conference on Stormwater and Aquatic Resource Protection*, 4-6 May 2005 Auckland. Retrieved from http://www.landcareresearch.co.nz/research/urban/liudd/documents/Vesely_et_al_200 5.pdf

Walsh, C.J., Roy H.R, Feminella, J.W., Cottingham, P.D., Groffman, P.M., & Morgan II, R.P. (2005). The urban stream syndrome: current knowledge and the search for a cure. *Journal of the North American Benthological Society,* 24(3): 706-723. https://doi.org/10.1899/04-028.1

Wanielista, M.P. & Yousef, Y.A. (1993). Stormwater Management. J. Wiley & Sons. New York, NY.

Zhan, W. & Chui, T.F.M. (2016). Evaluating the life cycle net benefit of low impact development in a city. *Urban Forestry & Urban Greening*, 20: 295-304. https://doi.org/10.1016/j.ufug.2016.09.006