Using Bioretention Retrofits to Achieve the Goals of Virginia's New Stormwater Management Regulations

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ABSTRACT

Virginia's new stormwater regulations involve the use of the Runoff Reduction Method (RRM), which requires the product of the peak flow rate and runoff volume (Q*RV) from the one-year storm event in the post-development condition to be reduced to eighty percent of the pre-development Q*RV to protect against channel erosion. This study models different bioretention cell sizes in a developed watershed in Blacksburg, Virginia to determine the "performance" at both the sub-watershed and watershed levels. In addition, models of "optimal" bioretention cells sized to meet the RRM for each sub-watershed are evaluated. A direct relationship is determined between the size of the cell required to meet the RRM and the sub-watershed's Natural Resources Conservation Service (NRCS) curve number. However, the required size for some of the cells is much larger than those typically seen. With the RRM applied for all of the sub-watersheds, the resulting hydrograph at the watershed outlet has a lower peak than the pre-development condition.

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Chapter 1

Introduction

1.1 Background

In September 2011, the Commonwealth of Virginia's Department of Conservation and Recreation (DCR) made substantial revisions to the Virginia stormwater management regulations, which the Virginia Department of Environmental Quality (DEQ) has since taken the lead in development and implementation. The revised regulations include erosion and sediment control, as well as stormwater management, which was divided into two main categories: quantity and quality of stormwater runoff (DEQ 2013). While stormwater quality is an important part of the regulations, this study deals exclusively with the stormwater quantity aspect of the regulations.

The stormwater quantity regulations changed greatly with the recent revisions. The previous stormwater quantity regulations followed the "Simple Method," requiring the peak developed flow rates from the 2- and 10-year storm events be brought down to the pre-development values (DCR 1999). In the runoff quantity regulation revisions, channel protection and flood protection are the two primary aspects of interest. There are several different cases for the regulations, but when discharging into a natural conveyance system, the channel protection criteria require the study of the 1-year, 24-hour storm event's peak runoff flow rate and total runoff volume for both the pre- and post-development conditions. For flood protection, the study of the 10-year, 24-hour storm is required to ensure that the post-development peak flow rate is at, or below, the pre-development peak [Commonwealth of Virginia (VA) 2011]. The new regulations are based on the Virginia Runoff Reduction Method (RRM). The strategies for this method were developed for Virginia by the Center for Watershed Protection (CWP) and the Chesapeake Stormwater Network (CSN) to try to better emulate pre-development hydrologic conditions (CWP & CSN 2008).

Low Impact Development (LID) is a design methodology that seeks to restore a site's hydrologic response to a storm to how it would respond before development occurred upon the site (Prince George's County 1999). One practice that is considered to be an LID practice is bioretention. A bioretention cell, also sometimes called a "rain garden," accepts runoff, allows the water to pond on top of it, and then lets water percolate through its engineered soil media to either the underlying soil or an underdrain. Bioretention cells also often utilize an outlet structure or overflow weir to allow any excess water that enters the cell to be routed in an efficient manner to a desired location downstream. Bioretention will most likely become a popular stormwater Best Management Practice (BMP) used in Virginia with the adoption of the new stormwater management regulations because it improves both water quality and quantity aspects (James and Dymond 2012). With the implementation of the new Virginia stormwater regulations, the Virginia DCR (2011) also changed the design guidelines for bioretention cells. Retrofitting urbanized areas with LID and BMP technologies is an effective way of reducing runoff in a watershed (Damodaram et al. 2010), and bioretention is a practice that can be added into relatively small areas, which makes it feasible to be added to a site at a later time as a retrofit.

1.2 Problem Statement

According to the RRM (VA 2011), the channel protection requirements are dependent on the 1-year, 24-hour storm's peak flow rate (Q) and total volume of runoff (RV) leaving a site in both the pre- and post-development conditions. To meet these requirements, the product of the peak flow rate and the volume of runoff (Q*RV) in the developed condition must be below the same value for the pre-development due to an improvement factor. For more information on the channel protection requirements, see Section 3.1. The RRM also requires flood protection. For flood protection, the peak flow rate produced by the 10-year, 24-hour storm event in the developed condition must be reduced to the same, or lower, peak as the pre-development condition. With larger, less frequent storms, such as the 10-year storm, bioretention does not perform as well in reducing peak runoff as it does for smaller storms because it has a smaller capacity than large detention facilities that are solely designed to reduce peak runoff rates (Holman-Dodds et al. 2003). Therefore, detention facilities would still be needed in a watershed with added LID techniques, such as bioretention, to account for the flood protection requirements. However, these flood protection facilities should be smaller due to the inclusion of the bioretention added for channel protection within the watershed.

The purpose of this study is to improve understanding of the implications of the new VA stormwater regulations with regards to using bioretention as a retrofit. Its focus will be determining how much bioretention must be installed in a watershed to meet the goals of preventing channel erosion. Another aspect that will be addressed is how much the implementation of the bioretention for channel protection decreases the detention storage necessary for flood protection. With all of the areas within a watershed being retrofitted with bioretention to meet the channel protection requirements of the RRM, the impacts of this method will also be studied at the watershed outlet.

1.3 Objectives

There are several main objectives of this study:

1. *Develop and calibrate a model of a watershed within the Town of Blacksburg, Virginia.* The Town of Blacksburg is working to have functional models of the watersheds within the Town to improve their compliance with Phase II of their Municipal Separate Storm Sewer System (MS4) program. The development of this model helps the Town to perform this function and will help them make planning decisions in the future. Calibration of the model will provide more meaningful results both for this study and the Town's future work. The "North Stroubles" watershed is used in this study.

2. *Determine the optimal ways to both model the bioretention within the watershed and ensure that the bioretention is achieving the goals of the Runoff Reduction Method (RRM).*
   The modeling software used in this study contains a tool to model LID practices, but it requires the use of the Environmental Protection Agency (EPA) Storm Water Management Model's (SWMM) parameters, instead of the Natural Resources Conservation Service (NRCS) methodology for which the Town's data was initially

developed. Therefore, determining the best way to model a bioretention cell within this software environment is a preliminary step. Then, this knowledge must be used to determine the best way to implement the bioretention into the watershed, so that the RRM can be properly evaluated.

3. *Develop performance curves for bioretention as a function of its contributing drainage area and the NRCS curve number in the contributing sub-watershed.*
   Each bioretention cell will be sized at several different percentages of the area of the sub-watershed flowing to it. The resulting peak flows and volumes of runoff will be compared to pre-development values to determine the effectiveness of that size of bioretention. The results of this analysis will be compared against the curve number of the sub-watershed to provide some design guidance for sizing bioretention cells.

4. *Determine the amount of bioretention that must be added to each sub-watershed to achieve the channel protection requirements of the RRM for that area.*
   In order to determine the applicability of the RRM, the required sizes of the features used to achieve its goals need to be studied to see if they are feasible. In this study, each sub-watershed is considered to be a "site" that has the RRM applied to it. The optimal amount of bioretention for each sub-watershed will be determined so that the channel protection requirements for the flow leaving the sub-watershed will be met.

5. *Determine the reduction in storage volume required for flood protection due to the presence of the bioretention that is added to account for channel protection.*
   With the addition of the bioretention into the watershed to meet the channel protection requirements, there is an additional overall volume of storage in the watershed. This storage volume would also be in use during a 10-year storm event, which is the storm studied for flood protection, since all of the flows in the watershed are routed through the bioretention. A scenario will be modeled to determine what the difference in storage requirements for flood protection is between the existing conditions and when there is the added bioretention. This will be studied for each sub-watershed and for the whole watershed.

6. *Evaluate the effectiveness of the RRM at a level that aggregates multiple sites upon which the RRM has been implemented.*
   Once the RRM has been applied to each of the sub-watersheds within the whole watershed of interest, the impacts of this method need to be studied at the watershed outlet. This is evaluated in the study and should show that applying the RRM at the sub-watershed scale will result in hydrology more closely resembling the pre-development, not only at that scale, but also at the full watershed-wide scale.

1.4 Model Development

The data for this study were developed primarily by the research group of Dr. Randy Dymond in the Via Department of Civil and Environmental Engineering at Virginia Tech. The research group has worked with the Town of Blacksburg to map, survey, and model all of the stormwater infrastructure and watersheds within the Town. Data were, and still currently are, being collected for all of the watersheds in the Town and include stormwater conveyances and stormwater nodes, such as catch basins, manholes, and pond outlet structures.

The data collection process involves inputting the field-collected nodes, with information such as structure type, invert depth, and conveyance attributes such as number, size, shape, material, and azimuth, into a Geographic Information System (GIS). Channels and detention ponds are also located either in the field, or upon inspection of aerial imagery and topography in the GIS environment. Pond outlet control structure information is collected separately to account for its added complexity.

Once the nodes have been located, the stormwater network is connected by the appropriate conveyances. This is performed through a GIS script that uses the azimuth of the conveyances to make a pipe "stub" at each node in the correct direction so that the researchers can connect the appropriate nodes together with the stubs of matching attributes. Cross-sections of channels are taken at key locations and are based on LiDAR data provided by the Town. Pond elevation-area curves are also developed using the same LiDAR data.

The final process for creating a usable dataset is converting the field-collected attributes and initial post-processing work into a format that can be used for modeling. Several steps included in this process are using the LiDAR data and collected invert depths to get invert elevations for all of the structures and pipes and then ensuring that the upstream and downstream nodes of each pipe are identified correctly. Overall, the data development process is still similar to that described in Hixon (2009). This work has been completed for a majority of the Town and is planned to be finalized within a year.

The North Stroubles watershed (See Appendix A) being evaluated for this study is approximately 475 acres, and has been divided into 41 sub-watersheds, or catchments, ranging in size from just over 1 acre to approximately 34 acres. The sub-watersheds were delineated based on the direct drainage areas of stormwater management ponds and the intersection of major confluences in the network. This watershed is comprised of single-family homes, apartment complexes, and commercial, industrial, and institutional facilities. Most of the surveys and data processing for this section of the Town of Blacksburg was performed in 2009. Before beginning this study, the data had already been structured and input into the model software, but no analysis or calibration had been performed.

Once the data were input into the modeling environment, there were still several more process steps necessary to develop a working model. The flow in each of the sub-watersheds was routed to the most downstream element within it, even if there were multiple catch basins within the sub-watershed. This was done to simplify the model considerably because there are almost 400 catch basins within the watershed and modeling all of these would require the time-consuming process of delineating the watershed and calculating a time of concentration for each inlet. Also, it would very likely result in an unstable model due to its complexity. The points of interest were chosen based on their importance, such as at intersections of major conveyances and at detention ponds. The aggregation of stormwater elements was found to have no substantial effect on the overall output, according to a study by Elliott, et.al. (2009).

For this study, the NRCS's Technical Release 55 (TR-55) was used to determine the rainfall-runoff relationship. The NRCS TR-55 method is widely-used for this purpose, and has demonstrated relatively accurate results over larger areas (USDA 1986). TR-55 is based primarily on 3 conditions: location, which determines the rainfall depth and type, curve number, which determines runoff based on land cover/land use and soil type, and time of concentration, which is the time it takes for water to get to the point of study from the most hydraulically remote point. The "curve number" is one of the defining characteristics of the TR-55 method, and it helps to determine the volume of runoff. Student researchers at Virginia Tech determined area-weighted curve numbers for each of the sub-watersheds based on soil surveys and detailed digitized land cover data. Within a GIS environment, the students used detailed aerial photos from 2009 to trace areas in the Town of Blacksburg and assign them to certain land cover classes. The researchers then associated the land cover classes with corresponding TR-55 land use types and used an overlay of hydrologic soil groups (HSG) from the Soil Survey Geographic Database (SSURGO) (USDA 2009) to determine an area-weighted curve number for each sub-watershed. This detailed land cover dataset was shown by White (2011) to be more accurate for its estimation of curve numbers than the National Land Cover Dataset (NLCD).

For each sub-watershed that was delineated, a time of concentration path was also found. Multiple locations along the border of each sub-watershed were tested to see which would result in the most hydraulically remote point, as described in Aguilar and Dymond (2013). The final time of concentration path was broken up into up to three flow regimes: sheet flow, shallow concentrated flow, and concentrated, or channelized, flow. The travel time for each segment was calculated based on the land cover during that section of flow, the length of flow, and the average slope using the equations in Table 1.1.

For the modeling component of this study, SewerGEMS V8i was used (Bentley 2013). SewerGEMS was chosen because it offers several advantages for the research being performed. It is easily compatible with a GIS interface, and since most of the data are stored in the GIS format, it allows straightforward transfer between environments and should allow for updates of infrastructure in the future. SewerGEMS is also capable of modeling the rainfall-runoff relationship using TR-55 methodology, which allows for easy integration with the existing data structure. Using the pond elevation-area curves and pond outlet structure data collected in the field, such as weir and orifice sizes and heights above the invert of the pond, this software can also calculate stage-storage-discharge curves for each pond. Overall, using the SewerGEMS modeling software allows the researchers to use the compatibility with GIS to be more productive by evaluating the variables and results more without having to manually enter all of the characteristics associated with the watershed and its sub-entities.

Table 1.1: Time of Concentration Equations [adapted from James (2010)]

Description	Equation	Source
Sheet, Unpaved	$t_{t,1} = 0.225L^{0.42}S^{-0.19}0.3^{-1.0}$	Seelye (1945)
Sheet, Paved	$t_{t,1} = 0.225L^{0.42}S^{-0.19}0.9^{-1.0}$	
Shallow Concentrated, Unpaved	$t_{t,2} = \frac{L}{16.1345 * S^{0.5}}/60$	TR-55 (USDA 1986)
Shallow Concentrated, Paved	$t_{t,2} = \frac{L}{20.3282 * S^{0.5}}/60$	
Concentrated, Unpaved	$t_{t,3} = 0.00948(E_H - E_L)^{-0.38}L^{1.13}$	Kirpich (1940)
Concentrated, Paved/Piped	$t_{t,3} = 0.2 * 0.00948(E_H - E_L)^{-0.38}L^{1.13}$	
	$t_c = t_{t,1} + t_{t,2} + t_{t,3}$	

t_t = travel time (min)
t_c = time of concentration (min)
L = length (ft)
S = slope (ft/ft)
E_H = highest elevation of flow segment (ft)
E_L = lowest elevation of flow segment (ft)

Chapter 2

Literature Review

2.1 BMP and LID Implementation Strategies

Best Management Practices (BMPs) and Low Impact Development (LID) strategies are both popular techniques used to try to return the post-development hydrologic condition of a site or area to its pre-development hydrologic condition. The goals of these techniques are often improvements for both water quality and water quantity. Though bioretention is the focus of this study, there are many other practices used as BMP and LID techniques which have the similar goal of detaining or retaining stormwater runoff before it is released into the main stormwater infrastructure. LID strategies, especially those reliant on the infiltration of water through a soil media, have been widely shown to be effective in managing watershed hydrology for smaller, more frequent storms, and are less effective for larger, less frequent storms (Damodaram et al. 2010).

The location of BMPs and LID strategies within a watershed has been studied extensively to determine where the most effective locations for these practices would be located. Since the implementation of these practices is both cost and space consumptive, locating them in a more efficient manner is desirable. Using a synthetic watershed model, Chang et al. (2009) determined that the best place to locate BMPs is at the farthest downstream area, because it allows the whole watershed to be treated and has appreciable impacts on both peak flow and lag time. Su et al. (2010) studied a large watershed, with many sub-watersheds, and had some different, and somewhat conflicting, conclusions. They determined that adding detention in upstream areas decreases the peak at the watershed outlet and increases the length of time of high flows, but that adding detention in downstream areas has little effect on the watershed peak because the time of concentration to the overall watershed outlet is much longer than that from the sub-area located in the downstream location. Fang et al. (2010) found that mitigation results are dependent on both the location and sizing of the detention basin(s) and the location of development within the watershed. When multiple detention basins are installed throughout a watershed, they often have little overall impact on the peak flow rate leaving the watershed because not all of the flow is routed through them (Emerson 2005).

Another aspect to locating LID techniques involves evaluating the type of land cover contributing drainage to the LID practice. Treating highly impervious areas with bioretention was shown to have positive results for storm events of small sizes on the overall watershed scale by James and Dymond (2012). Gilroy and McCuen (2009) found that bioretention facilities that drain impervious areas are much more efficient with their use of space than those that drain pervious areas because of the pervious area’s natural ability to reduce runoff. They also determined that BMPs in series have their effects compounded no matter the distance between them. BMPs should be selectively located in important locations first and then installed in less important locations, as the budget allows (Perez-Pedini et al. 2005). Several programming approaches have also been developed in order to optimize the location of BMPs for efficiency in nonpoint source pollutant removal (Veith et al. 2003; Zhen et al. 2004).

As BMP and LID techniques in general are becoming increasingly popular, the implementation of bioretention is increasing as well, which has directed more research into sizing, maintaining, and determining the appropriate composition of bioretention cells. The Virginia Department of Conservation and Recreation (DCR 2011) has developed new design criteria for bioretention basins along with the new stormwater regulations. There are "Level 1" and "Level 2" designs with different guidelines that are given different credit for both their runoff reduction and phosphorous removal potentials. Prince George's County, Maryland was one of the first localities to implement LID strategies, and is considered to still be one of the leaders in the field. In their LID Manual (1999), many aspects of the design, including size, location, and composition, are discussed and recommended. Inadequately sizing these techniques can cause performance issues, and if accompanied by poor maintenance during and after construction, the reduction in performance can be substantial (Brown and Hunt 2010). Maintaining BMPs is important, and poor maintenance practices can lead to water quality issues, as shown in O'Connor (2010), where ammonium ions (NH_4^+) were found in high concentrations due to decaying leaves in the BMP.

2.2 BMP and LID Performance Assessment

Many researchers have studied BMPs and LID techniques that have been installed in the field to evaluate their performance. These studies allow for the field verification of models, but also allow for conclusions to be drawn and assumptions to be made for studies being performed in other locations, or for theoretical studies without a physical entity for validation. Partridgeberry Place is an LID subdivision in Ipswich, Massachusetts that has been monitored and has had a calibrated model built, which has led to a study concluding that clustered subdivisions, and especially clustered LID subdivisions, have watershed hydrology characteristics closer to those of the pre-development when compared to conventional subdivisions (Fitsik 2010). Hood et al. (2007) monitored two subdivisions near each other, one developed conventionally and the other with LID practices. Compared to the pre-development values, the conventional subdivision's runoff volume and nitrogen and phosphorous loading increased greatly, while the LID subdivision's values remained roughly the same. It was also confirmed that the LID subdivision had a lower peak discharge, a longer lag time, and a higher runoff threshold compared to the conventional subdivision.

Vegetated or green roofs are becoming more popular for stormwater mitigation and are also considered an LID practice. A green roof installed in Orlando, Florida was found to be able to reduce both the peak flow rate and volume of runoff from a roof due to its storage capacity in the soil media (Wanielista and Hardin 2007). Similar to other LID practices, green roofs perform well for smaller storms and poorer for larger storms, but do still reduce the flows leaving a site, while being a feasible retrofit option for many types of flat roofs (Stovin 2010).

As with all BMPs and LID practices, bioretention is also getting more research attention with more bioretention installed in the field. Field observations and the study of two bioretention cells installed in a Maryland parking lot suggest that bioretention can help to reduce the volume of runoff, lower peak flow rates, and increase lag times (Davis 2008). In this study, most events showed a peak discharge of about 50% of the inflow peak, and there was no flow leaving the underdrain of the cells for about 20% of the observed storm events, which means that all of the water was being absorbed into the media of the cell. Li et al. (2009) studied six bioretention cells in four locations in Maryland and North Carolina and found that cells with larger storage volumes, either through a larger cell area or deeper media depths, more closely replicated predevelopment conditions by reducing peak flow rates, reducing outflow volumes, and promoting more infiltration. This was found to be true even for larger storms, though to a smaller degree. Hatt et al. (2009) confirmed the hydrologic benefits of bioretention, or "biofiltration" as it is called in the paper, and also found substantial improvement in water quality with the bioretention generally removing more than 90% of suspended solids and heavy metals. Studying two bioretention cells with different depths and then fixing construction errors in both, Brown and Hunt (2010) demonstrated improved performance with the deeper media and correct construction. This confirms the importance of both design and construction.

2.3 Bioretention Retrofits

Stormwater retrofits offer a different sort of benefit to a watershed than developing a site in a low impact manner. The retrofits are often designed to reduce the flows of the existing condition of a watershed, without having to offset the increased flows that are coming from further new development. Depending on the goals of a retrofit and site characteristics, such as imperviousness in the drainage area and the size of the design storm, Guo et al. (2013) developed a procedure to determine the storage volume of an LID retrofit and found that more imperviousness leads to larger necessary retention volumes and that capturing 75-85% of yearly runoff volume is comparable to capture rates for most water quality measures. With the installation of bioretention "bumpouts," in addition to permeable pavement parking stalls and a tree filter device, a reduction (14%) in runoff volume can be achieved in a developed watershed (Winston et al. 2013). Using a small bioretention cell with a larger stone trench as a retrofit for a watershed, substantial reductions in runoff volume and in numbers of large peak flows were found, even for soil infiltration rates at the low end of those typically used for the design of such systems (Lucas 2010). A retrofit bioretention cell installed in the Stroubles Creek watershed in Blacksburg, Virginia was shown to reduce the average peak and volume of runoff by over 90%, even though its surface area to drainage area ratio of 2.1% is below the recommended and commonly used ratios (DeBusk and Wynn 2011).

2.4 Hydrologic/Hydraulic Modeling

In order to model and analyze BMPs and LID techniques, certain modeling approaches have been studied to determine the impacts that these approaches can have on the model, which can greatly affect a model and its results. Key components of simulating a BMP are the volume and peak reductions and the flow threshold where the BMP is still efficient and effective (Poresky et al. 2012). Damodaram et al. (2010) uses an innovative approach to applying LID practices over a large area by using the NRCS CN approach and adding in the storage volume of the LID practices to the initial abstraction volume. This allows for the widespread application of LID practices, without having to model each one independently. Regulatory agencies and some literature sources have tried to "score" certain BMPs and LID practices based on their hydrologic benefits, but Wild and Davis (2009) found that determining pollutant (and flow) reductions is a complex process that does not lend itself to a single score because there are many factors that go into the design and effectiveness of each one. Along these same lines, baseflow is often a characteristic that is desired to be achieved by LID practices, but it is extremely difficult to model because determining what the "correct" baseflow value should be is not a simple process or one that can be confidently calculated (Hamel et al. 2013).

One of the key parts of this study will be aggregating both BMPs and drainage areas. In order to do this aggregation, it had to be determined that it was appropriate and the results would prove applicable. In a study by Elliott et al. (2009), aggregation is shown to have little impact on the model results for most cases of bioretention. The only case with a large impact was when a network of 810 cells were attempted to be aggregated to 1 cell. James and Dymond (2012) aggregated drainage areas to important points of interest, such as ponds and the intersection of main conveyances, and aggregated a distributed network of bioretention cells, with effective results.

2.5 Summary

BMPs and LID techniques are effective methods for managing the hydrology of watersheds. The number of these approaches to stormwater management being implemented is growing, and bioretention is a key component of future stormwater management. Roy-Poirier et al. (2010) performed a thorough review of bioretention design and research and found that further modeling results were necessary to validate design guidelines and that more research was needed for consistency in sizing guidelines. This study attempts to expound upon the findings discussed, to help to fill in gaps in research, and to provide beneficial knowledge for those trying to optimize the design of bioretention cells in a watershed. Research has shown that bioretention performs well for smaller storms and is an effective manager of the overall hydrology of a watershed. However, little research has been performed with a specific goal for watershed improvement in mind, such as the goal expressed by the new VA stormwater regulations. More research into how much bioretention must be installed in a watershed to achieve stormwater management objectives is necessary.

Chapter 3

Using Bioretention Retrofits to Achieve the Goals of Virginia's New Stormwater Management Regulations

3.1 Introduction

Low Impact Development (LID) is a design methodology that seeks to restore a site's hydrologic response to a storm to how it would respond before development occurred upon the site (Prince George's County 1999). One practice that is considered to be an LID practice is bioretention. A bioretention cell, also sometimes called a "rain garden," accepts runoff, allows the water to pond on top of it, and then lets water percolate through its engineered soil media to either the underlying soil or an underdrain. Bioretention cells also often utilize an outlet structure or overflow weir to allow any excess water that enters the cell to be routed in an efficient manner to a desired location downstream. Retrofitting urbanized areas with LID and Best Management Practice (BMP) technologies is an effective way of reducing runoff in a watershed (Damodaram et al. 2010), and bioretention is a practice that can be added into relatively small areas, which makes it feasible to be added to a site at a later time as a retrofit.

In September 2011, the Commonwealth of Virginia's Department of Conservation and Recreation (DCR) made substantial revisions to the Virginia stormwater management regulations, which the Virginia Department of Environmental Quality (DEQ) has since taken the lead in development and implementation. The revised regulations include erosion and sediment control, as well as stormwater management, which was divided into two main categories: quantity and quality of stormwater runoff (DEQ 2013). While stormwater quality is an important part of the regulations, this study deals exclusively with the stormwater quantity aspect of the regulations.

The stormwater quantity regulations have changed significantly with the recent revisions. The previous stormwater quantity regulations followed the "Simple Method," requiring the peak developed flow rates from the 2- and 10-year storm events be returned to the pre-development values (DCR 1999). In the runoff quantity regulation revisions, channel protection and flood protection are the two primary aspects of interest. There are several different cases for the regulations, but when discharging into a natural conveyance system, a primary channel protection criterion requires the study of the 1-year, 24-hour storm event's peak runoff flow rate and total runoff volume for both the pre- and post-development conditions (VA 2011).

The new channel protection requirements use a unique method. Equation 3.1 (VA 2011) is used for the channel protection calculations. It calculates the maximum allowable peak flow rate for the developed condition during the 1-year storm event ($Q_{Developed}$). By multiplying both sides of Equation 3.1 by the developed runoff volume ($RV_{Developed}$), it becomes Equation 3.2, where the peak flow rates (Q) are multiplied by the volumes (RV) of flow for the 1-year storm event for both the pre- and post- development conditions, which is used as much of the basis for analysis in the rest of this study and referred to as Q*RV. Note that in Equation 3.2, the developed Q*RV must be less than or equal to 80% (*I.F.*) of the pre-development Q*RV.

$$Q_{Developed} \leq I.F. \; * \; Q_{Pre-Developed} * \frac{RV_{Pre-Developed}}{RV_{Developed}} \tag{3.1}$$

$$Q_{Developed} \; * \; RV_{Developed} \leq I.F. \; * \; Q_{Pre-Developed} * \; RV_{Pre-Developed} \tag{3.2}$$

Where: I.F. (Improvement Factor) = 0.8 for sites > 1 acre or 0.9 for sites ≤ 1 acre
$Q_{Developed}$ = peak flow rate for the drainage area in the developed condition for the 1-year storm event (cfs)
$Q_{Pre\text{-}Developed}$ = peak flow rate for the drainage area in the pre-developed condition for the 1-year storm event (cfs)
$RV_{Developed}$ = the volume of runoff from the site in the developed condition including runoff reduction for the 1-year storm event (inches)
$RV_{Pre\text{-}Developed}$ = the volume of runoff from the site in the pre-developed condition for the 1-year storm event (inches)

For flood protection, the study of the 10-year, 24-hour storm is required to ensure that the post-development peak flow rate is at, or below, the pre-development peak (Commonwealth of Virginia (VA) 2011). The new regulations are based on the Virginia Runoff Reduction Method (RRM). The strategies for this method were developed for Virginia by the Center for Watershed Protection (CWP) and the Chesapeake Stormwater Network (CSN) to try to better emulate pre-development hydrologic conditions (CWP & CSN 2008). The Q*RV value seems to be unique to the Virginia regulations, as none of the surrounding states have regulations that incorporate this value. Rolband and Graziano (2012) describe this method as the "Energy Balance" method, and they played a role in the method's development with VA's DCR. Bioretention will most likely become a popular stormwater BMP used in Virginia with the adoption of the new stormwater management regulations because it improves both water quality and quantity aspects (Brown and Hunt 2010).

Many studies have demonstrated that bioretention is an effective means of stormwater management quantity and quality, especially at the site scale. Bioretention is especially effective for less intense, more frequent storm events (Davis 2008; James and Dymond 2012). The results of the installation of two bioretention cells in a Maryland parking lot suggest that bioretention can significantly help to reduce the volume of runoff, lower peak flow rates, and increase lag times (Davis 2008). Bioretention retrofits are becoming more popular due to their hydrologic benefits. With the addition of bioretention "bumpouts," along with permeable pavement parking stalls and a tree filter device, Winston et al. (2013) found that a substantial reduction in runoff volume can be achieved in a developed watershed. A retrofit bioretention cell installed in the Stroubles Creek watershed in Blacksburg, Virginia was shown to reduce the average peak and volume of runoff by over 90%, even though its surface area to drainage area ratio is below the recommended and commonly used ratios (DeBusk and Wynn 2011).

The location of bioretention in a watershed is also important for maximizing its efficiency. James and Dymond (2012) found that bioretention is more efficient when it is treating large impervious areas, than when it is treating areas that have a higher percentage of pervious cover. Gilroy and McCuen (2009) had similar conclusions, and also determined that installing BMPs in series compounds their effects. However, neither one of these studies looked at how much bioretention would be necessary to meet water quantity goals or regulations. Proper sizing, maintenance, and construction practices are also critical to the performance of bioretention and, if done correctly, can result in a practice that reduces the flows leaving a site (Brown and Hunt 2010). Li et al. (2009) studied four locations with bioretention cells in Maryland and North Carolina and found that cells with larger storage volumes, either through a larger cell area or deeper media depths, more closely replicated pre-development conditions, even for larger storms, by reducing peak flow rates, reducing outflow volumes, and promoting more infiltration. Again, however, these studies do not investigate how much bioretention is necessary to meet a goal, just that an improvement is seen.

Previous research has shown that bioretention performs well for smaller storms and results in the overall hydrology of a watershed approaching the pre-development levels. However, little research has been performed with a goal for watershed improvement in mind, and more research into how much bioretention must be installed in a watershed to achieve stormwater management objectives is necessary. This study is performed with the goal of meeting actual stormwater regulations and providing insight into bioretention design and sizing. The study is performed on a modeled watershed in the Town of Blacksburg, Virginia. Using several different modeling scenarios, differing sizes of bioretention cells are modeled for their retrofitted installation throughout the watershed. Also, the "optimal" scenario is found for each sub-watershed in the watershed, so that it can meet the channel protection requirements of the RRM. Furthermore, the effect of the new regulations is studied at the watershed outlet for when all parts of the watershed are being treated with the RRM.

3.2 Methods

The watershed modeled in this study is the "North Stroubles" watershed in Blacksburg. It is a 475-acre watershed consisting of residential, commercial, industrial, institutional, and open space land uses in the headwaters of Stroubles Creek. There is a flow sensor within the stream located near Webb Street, and upstream of this, the watershed has been delineated into 41 sub-watersheds, or catchments, ranging from just over 1 acre up to approximately 34 acres and can be seen in Figure 3.1. The catchments were delineated based on key points of interest, such as ponds or the intersection of major conveyances. In addition to the flow sensor, there is a rain gauge less than a mile outside of the boundary of the watershed, and the data produced by these devices were used for calibration of the model.

This study looks at a number of different modeling scenarios. The pre-development scenario is used for comparison at the watershed-wide scale and to determine the target values for the design of each catchment's bioretention cell. The existing conditions scenario is used as a base for all of the scenarios with added bioretention. There are four different "performance" scenarios that evaluate performance of bioretention cells with surface areas sized as a percentage (3%, 5%, 7%, and 10%) of the contributing drainage areas. There are also two "optimal design" scenarios where the flow leaving the bioretention cell meets the channel protection requirements of the RRM.

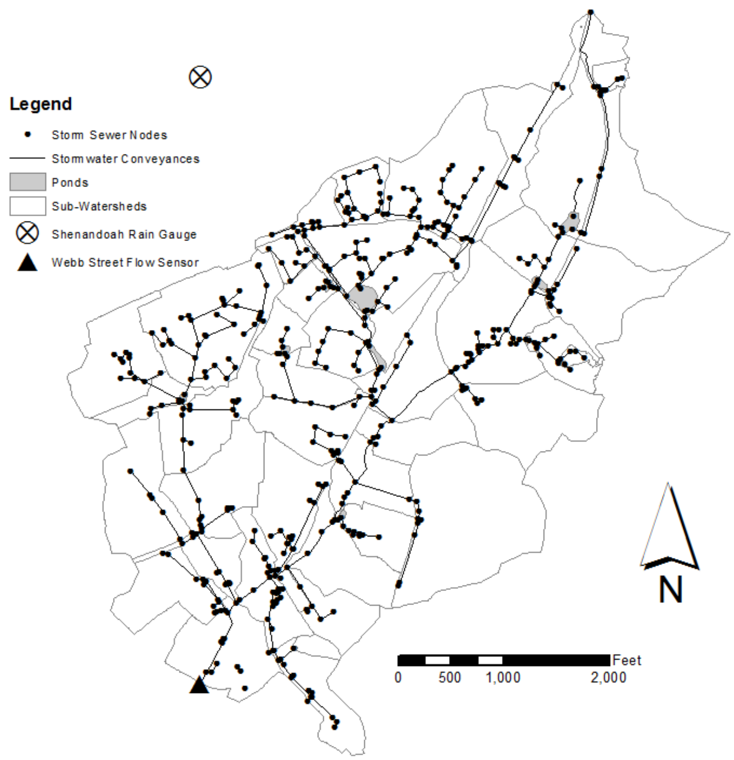

Figure 3.1: Study Watershed

3.2.1 Existing Conditions Model

In order to produce the most accurate results possible, the existing condition of the North Stroubles watershed was created and used for calibration. One existing bioretention cell and 11 existing ponds are present in the watershed and were included in the model along with other existing stormwater infrastructure, such as manholes, catch basins, pond outlet structures, pipes, and open channels.

The stormwater infrastructure that was included in this base model is part of an ongoing partnership between the Town of Blacksburg and researchers at Virginia Tech. This infrastructure information was field collected with the aid of GPS and aerial imagery. Attributes of the stormwater nodes and conveyances were entered into a Geographic Information System (GIS). Using LiDAR data, elevation and slope attributes were determined for the nodes and conveyances. The infrastructure was input into the SewerGEMS V8i modeling environment (Bentley 2013), which was utilized for the hydrologic and hydraulic modeling in this study.

Data collected from the field, aerial imagery, and LiDAR data were used to delineate the sub-watersheds based on the drainage areas of the existing ponds and the intersection of major confluences within the stormwater network. Following delineation of the sub-watersheds, the modeled flows contributed by each sub-watershed were introduced at their respective downstream node located in each sub-watershed. The flows were calculated for each sub-watershed with the NRCS TR-55 (USDA 1986) runoff methodology. After the sub-watershed flows are input, the modeling software used its implicit solver to route the flows downstream (Jin 2002).

Using detailed aerial photos to assign land cover classes and the Soil Survey Geographic Database (SSURGO) (USDA 2009), an area-weighted NRCS curve number (CN) for each sub-watershed was produced. The high-resolution aerial imagery was digitized into land cover classes, such as buildings, asphalt, concrete, meadow, and light and dense forest. This information was combined with the hydrologic soil group information in the SSURGO data to produce the NRCS CN. For each sub-watershed that was delineated, a time of concentration was also determined by commonly used flow equations and was based on the slopes developed from the LiDAR data and the land cover data. For each sub-watershed, the most hydraulically-remote point was estimated by checking the resulting time of concentration of several possible locations along the border of each sub-watershed and selecting the longest. This process, along with other aspects of the model development, is discussed by Aguilar and Dymond (2013).

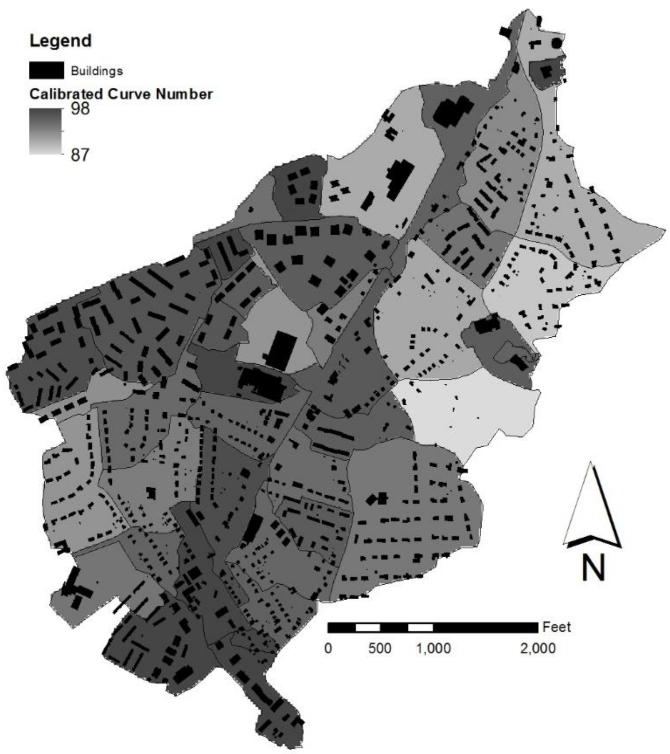

Figure 3.2: Calibrated Curve Numbers of Sub-Watersheds

The existing conditions model was designed to mimic the actual watershed, so it was calibrated using data from the flow measurement device at the watershed outlet. One of the main parameters altered during the calibration process was the NRCS CN of the sub-watersheds, with the calibrated values shown in Figure 3.2. For calibration, it was necessary to increase the CN values and modify the time of concentrations (t_c) for the sub-watersheds to produce more runoff. This was deemed necessary because the flow sensor's readings were much higher than those initially being output from the model with its original CNs. With the CN and t_c adjustments and some other slight modifications to the pond outlet structures, the model was calibrated from four actual, single-peak storm events in 2009. These particular storm events were selected to more closely replicate the single-peak nature of the NRCS design storms used in the rest of the analysis. One of the main calibration parameters used was the difference between the measured and modeled Q*RV because of the desire to meet the RRM's standards. For these four storm events, the differences in Q*RV between the modeled and measured values ranged from -40% to +21%, as shown in Table 3.1. The other main calibration tool used was the Nash-Sutcliffe model efficiency coefficient (Nash and Sutcliffe 1970), which ranged from 0.64 to 0.81. Some possible sources of these differences in the model could be errors in data collection, the effects of aggregating the parts of the sub-watersheds, or differences in actual and recorded precipitation due to the closest rain gauge being located just outside of the boundary of the watershed and some of the storms possibly being localized events. Figure 3.3 shows the measured flow and the calibrated model's flow for the May 14, 2009 storm event.

Table 3.1: Calibration Summary

Storm Event	5/8/2009	5/14/2009	6/15/2009	7/17/2009
Duration of Runoff (hr)	15.5	8.0	4.5	13.5
Time Step (hr)	0.25	0.25	0.25	0.25
Nash-Sutcliffe R^2 (1970)	0.78	0.81	0.66	0.64
Deviation in Volume	32%	34%	-24%	4%
Deviation in Peak Flow	-8%	-12%	-21%	-21%
Deviation in Q*RV	21%	18%	-40%	-18%
Peak Time Shift (hr)	0	-0.25	0	0
Model Continuity Error	0.0%	0.2%	0.0%	0.2%

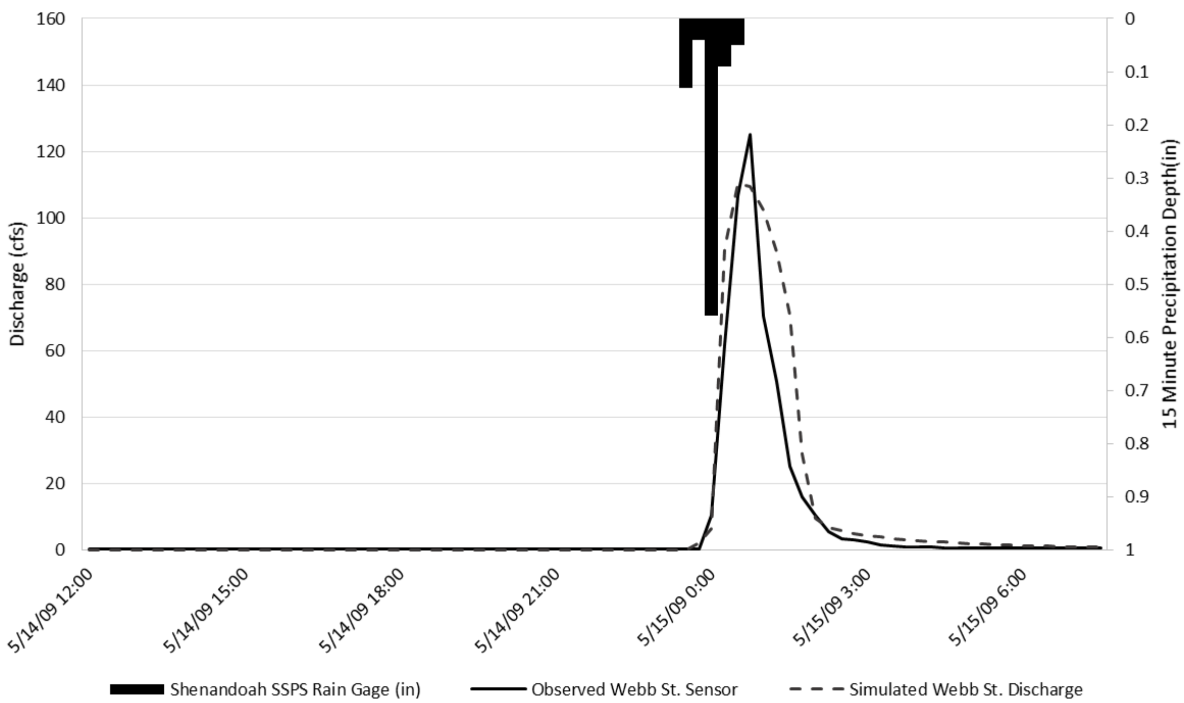

Figure 3.3: May 14, 2009 Storm Event

3.2.2 Pre-Development Model

To satisfy the regulations being studied, the developed condition has to be compared to the pre-development condition. For this study, the pre-development condition was assumed to be meadow in good condition with soils of the Hydrologic Soil Group C, which would be a typical pre-development condition in the study area and represents a CN of 71. For comparison at the watershed outlet, the whole watershed was combined into one catchment with no man-made infrastructure, and the path used for the time of concentration assumed that the elevation of the land within the watershed remained approximately the same. The same pre-development conditions were assumed for each of the sub-watersheds when comparisons were being made at the sub-watershed level.

3.2.3 Design of Bioretention Cells in Model

The “performance” scenarios modeled the sub-watersheds with the bioretention cells sized as a percentage of the drainage area and located at the local outfall. All runoff from each sub-watershed was routed through its bioretention cell. The areas of each cell differed, but the vertical structure of the cells remained the same. Primarily, the cells were designed with 3 feet of engineered soil media with a porosity of 25% and then 0.5 feet of surface storage (100% porosity), per the Level 2 Design in the Virginia Bioretention Design Specifications (VA DCR 2011). At the top of the surface storage, the weir outlet structure was designed to pass the flow rate of the 10-year storm for that sub-watershed. Flow entering the cell fills up the storage volume until the capacity is reached and then flows through the weir outlet structure to the most downstream structure within each sub-watershed. None of the water stored in the cell reenters the system. This modeling approach was assumed to be valid because of the high infiltration rates of the engineered soil media that is typically installed in bioretention cells and the lack of an underdrain in the design. Without an underdrain, the time that it would take the water to infiltrate into the groundwater and then make its way back to the surface water would be large enough to make it insignificant to the model.

Since the vertical structures of the cells remained the same, the varying areas of the cells provided the means for assessing the impacts of changing the storage volume. Large drainage areas were being routed into some of the cells, which resulted in some of the cells being much larger than those typically seen. However, these large cells would be representations of a distributed network of cells located throughout each of the sub-watersheds. Elliott et al. (2009) determined that it is acceptable to aggregate a network of bioretention cells for modeling purposes, and Gilroy and McCuen (2009) found that BMPs in series have their effects compounded regardless of the distance between them. The bioretention cells are designed as retrofits, but also could be implemented upon the initial development of the land.

By keeping the same vertical structure of each cell and changing the surface area, the volume of each cell could be changed in a consistent manner. Since each cell was only receiving flow from a single sub-watershed, they were sized based on a percentage of the area of their respective sub-watershed. The four consistent percentages used for sizing the surface areas of the cells were 3%, 5%, 7%, and 10% of the sub-watershed’s area.

3.2.4 “Optimal” Models

Along with the scenarios based solely on the percentage of drainage areas, two “optimal” scenarios were tested. The design of the “optimal” cells was achieved by changing the percentage of sub-watershed area for each cell until its Q*RV leaving the cell for the 1-year storm event equaled 80% of the related pre-development value. The 80% value was chosen to meet the RRM’s requirements.

The first of these two scenarios had the typical cell’s vertical structure used in all of the other scenarios. In the other optimal scenario, the engineered soil media depth was increased from 3 to 4 feet. This scenario with cells of increased depth would represent an urban area where space may be limited and constructing a deeper cell would be desired.

3.2.5 Flood Protection Analysis

In the RRM, the flood protection requirements call for bringing the peak flow rate from the 10-year storm event in the developed condition back down to, or below, the pre-development peak. The model was run with the 10-year storm event for the pre-development scenario, the existing conditions scenario, and the 3-foot optimal scenario. Due to the presence of stormwater management ponds and regional detention facilities already in the watershed, the flood protection was analyzed at the watershed scale, instead of the site or sub-watershed scale.

3.3 Results and Discussion

The model scenarios were run for the 24-hour, 1- and 10-year return frequency NRCS design storm events for Blacksburg, Virginia. The rainfall depth for each storm was obtained from the NOAA Atlas 14, Volume 2 (Bonnin et al. 2004) partial duration series. These values were 2.28 inches and 4.11 inches for the 24-hour storm events for return frequencies of 1 and 10 years, respectively.

For each sub-watershed, as well as the watershed as a whole, model results were obtained for each scenario and compared to the pre-development values with respect to the peak flow rate, the volume of flow, and the Q*RV. These values were plotted against the calibrated CN of the sub-watershed’s developed condition, as shown in Figure 3.4. In general, as the CN of the sub-watershed increased, the peak flow rate and volume of flow increased, and therefore the peak multiplied by the volume, increased as well. Also, as expected, as the area and volume of bioretention installed in each sub-watershed increased, the peak and volume of flow decreased.

When compared to the pre-development peak for the 1-year storm event, the sub-watersheds in the existing condition model (0% bioretention) produced peak flows between 2 and 10 times higher, as shown in Figure 3.4a. The 3% scenario had one sub-watershed that was brought below the pre-development peak, and the 5% scenario had five sub-watersheds achieving that reduction. Almost half of the 41 sub-watersheds in the 7% scenario had peaks at or below the pre-development value, and all of the watersheds in the 10% scenario had peak flows below the pre-development peak.

Meeting the pre-development values for volume was less successful. None of the 3%, only 1 of the 5%, and only 4 of the 7% sub-watersheds met the pre-development threshold (Figure 3.4b). Only about one-third of the sub-watersheds in the 10% scenario released less total flow than the pre-development scenario. Note that the storage volume in the 10% scenario was so large that it resulted in no flow leaving the bioretention cell for several of the sub-watersheds.

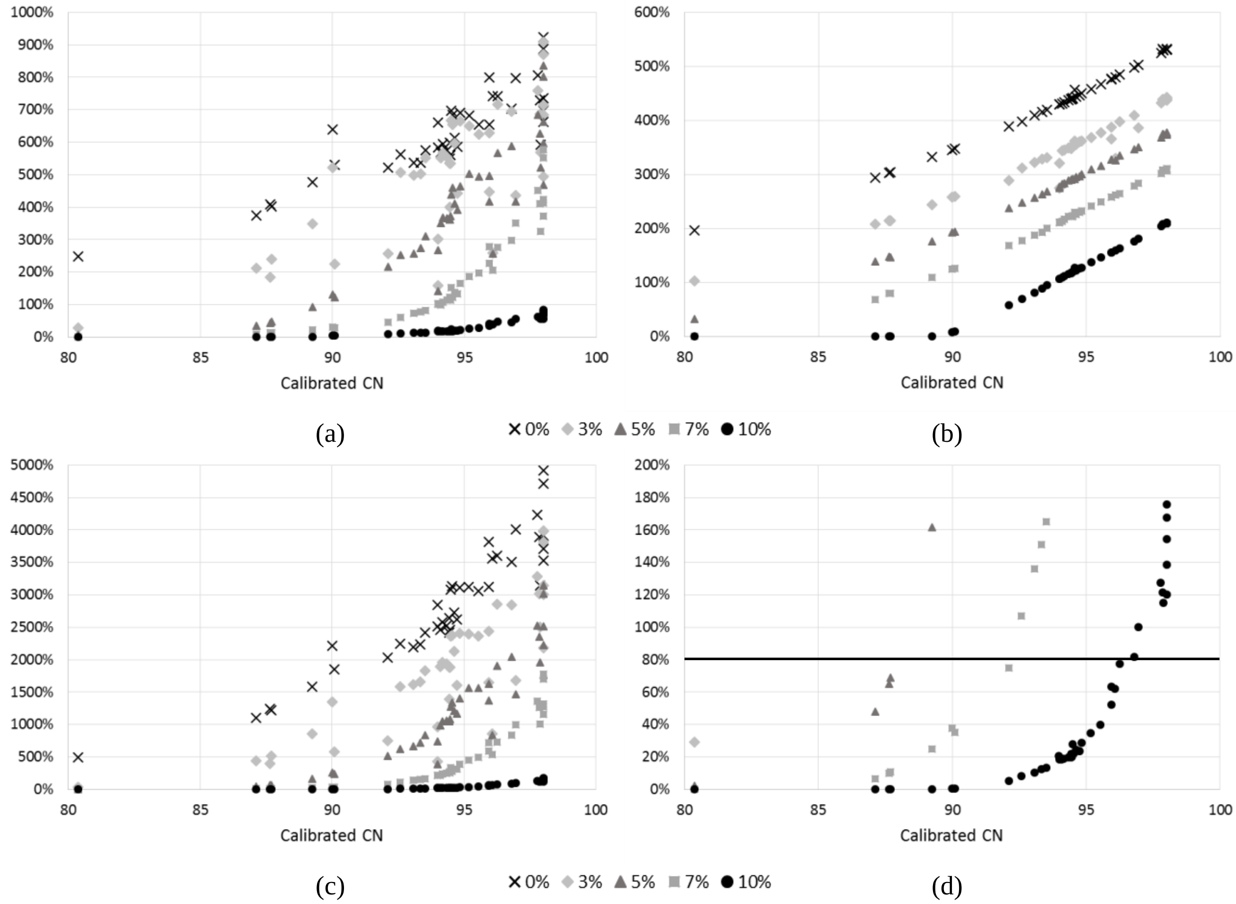

Figure 3.4: Comparison of Scenarios versus Pre-Development Values (1-Year Storm Event)
(a) Percent of Pre-Development Peak vs. CN, (b) Percent of Pre-Development Volume vs. CN,
(c) Percent of Pre-Development Q*RV vs. CN, and (d) Percent of Pre-Development Q*RV (under 200%) vs. CN
(**Note**: the y-axis of the above figures represent the percent ratio of the scenario and the pre-development values)

When reviewing the Q*RV, the range of results increased greatly. Several of the sub-watersheds in the existing condition had values almost 50 times greater than those in the pre-development condition. Figure 3.4c shows the full range of results for the scenarios compared to the pre-development, and Figure 3.4d shows the same information, but only for those data points below 200% of the pre-development. Seventy-six percent of the 41 sub-watersheds in the 10% scenario are below the 80% value (shown by the black line in Figure 3.4d) needed for the channel protection requirements of the RRM, with many fewer meeting this value in the other scenarios.

The trends shown in the previous figures demonstrate the need for a simple method of sizing bioretention based on the CN of the upstream watershed because of the correlation that is seen between the CN and the runoff metrics used in this study. There is no “one-size-fits-all” percentage to meeting the requirements in a sensible way. Therefore, for each sub-watershed, the bioretention cell was iteratively sized until it met the 80% value of the Q*RV. The results of this "optimal" design for each sub-watershed can be seen in Figure 3.5. As expected, more area and volume of bioretention is generally needed to achieve the same results when the CN of the contributing drainage area is higher. Linear trendlines were fit to the data for both the 3- and 4-foot depths of soil media to demonstrate the approximate linear relationship between the CN of the drainage area and the required size of the bioretention cell. Other forms of regression lines had similar goodness-of-fit measures, but the simpler linear regression line was used to show the basic trend in the data since no other forms of the line have an obvious relationship between CN and volume of runoff. Again, for this particular study, the pre-development CN is 71, so using the post-development CN and cell depth, Figure 3.5 could be used to size the area of a bioretention cell based on the size of the contributing drainage area.

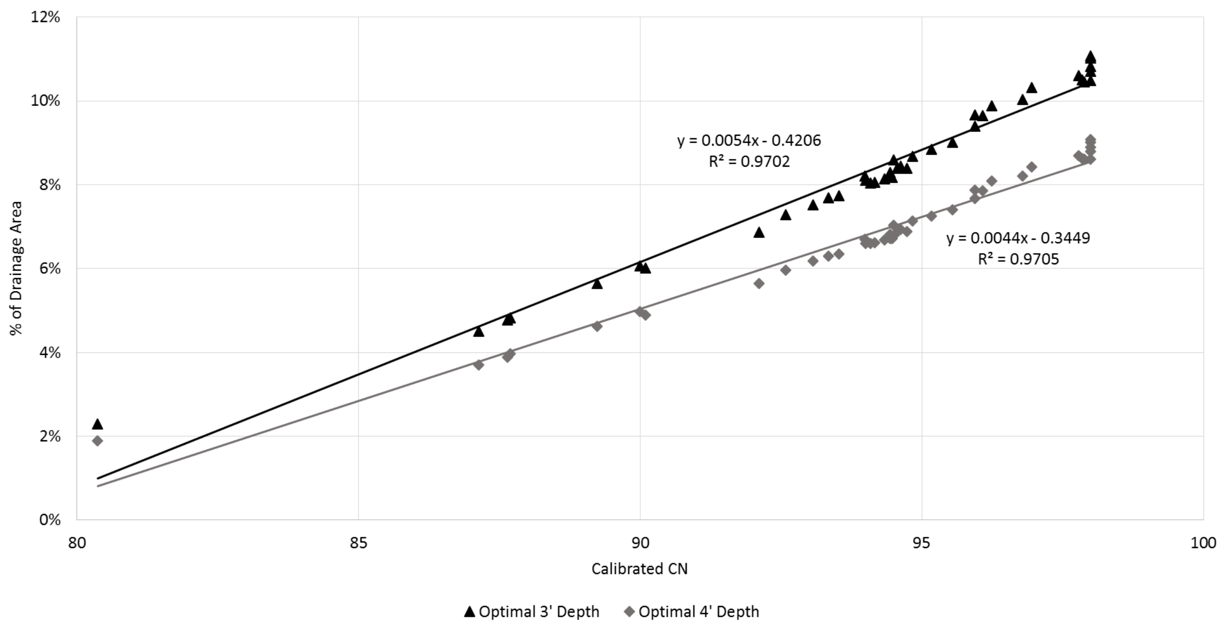

Figure 3.5: Optimal Bioretention Design

When increasing the depth of soil media from 3 to 4 feet, there is a consistent drop of 18% of surface area needed for the cell for all of the sub-watersheds. All of the data points from both the 3- and 4-foot depths meet the 80% of the pre-development Q*RV requirement, but it is interesting to note that the peak runoff rates and volumes of runoff do not equal each other for any sub-watershed. The deeper cells typically have a 2-3% higher peak with 2-3% less volume. Also, note that the size of the cell is plotted against the calibrated CN, so consideration should be given to CN values before using the equations for any reference or design practices.

To analyze the flood protection in the watershed, the 10-year storm event was modeled for the pre-development, existing conditions, and 3-foot optimal scenarios. Since the watershed already contains stormwater management ponds and regional detention facilities, the flood protection requirements were analyzed at the outlet to the whole watershed. The presence of the existing facilities limits the investigation into the effects of the added bioretention on flood protection because the facilities already bring the peak flow rate in the watershed below the pre-development peak. However, some limited analysis using TR-55 sizing methodology (USDA 1986) suggested that the bioretention, installed for the purpose of channel protection, could decrease the storage required for flood protection by 15-20%.

Another goal of this study other than the sizing of bioretention cells was determining the effect of implementing the RRM on many sites within a larger watershed. All of the percentage scenarios, along with the existing condition, pre-development, and “optimal” scenarios, were modeled for the 1-year storm for the whole North Stroubles watershed, as well as the individual sub-watersheds. The resulting hydrographs at the outlet of the watershed can be seen in Figure 3.6. Applying the RRM by treating each sub-watershed as a development site (demonstrated by the optimal scenario) results in an outflow hydrograph at the watershed outlet that has a lower peak than the pre-development, but a larger volume. This is due to an increased period of flows that are lower than the pre-development peak flow but higher than the pre-development flows after the peak. Another interesting result is that applying the 10% scenario resulted in so much storage throughout the watershed that the hydrograph does not resemble a typical hydrograph as in the other scenarios. The short rising limbs of the hydrographs in the scenarios with bioretention should be noted as well. This was also seen in all of the individual sub-watersheds and is most likely due to the cells filling up with the slower rates of runoff from the lower intensity rainfall at the beginning of the design storm, and then releasing the water at a high rate around the time of the higher intensity portion of the storm.

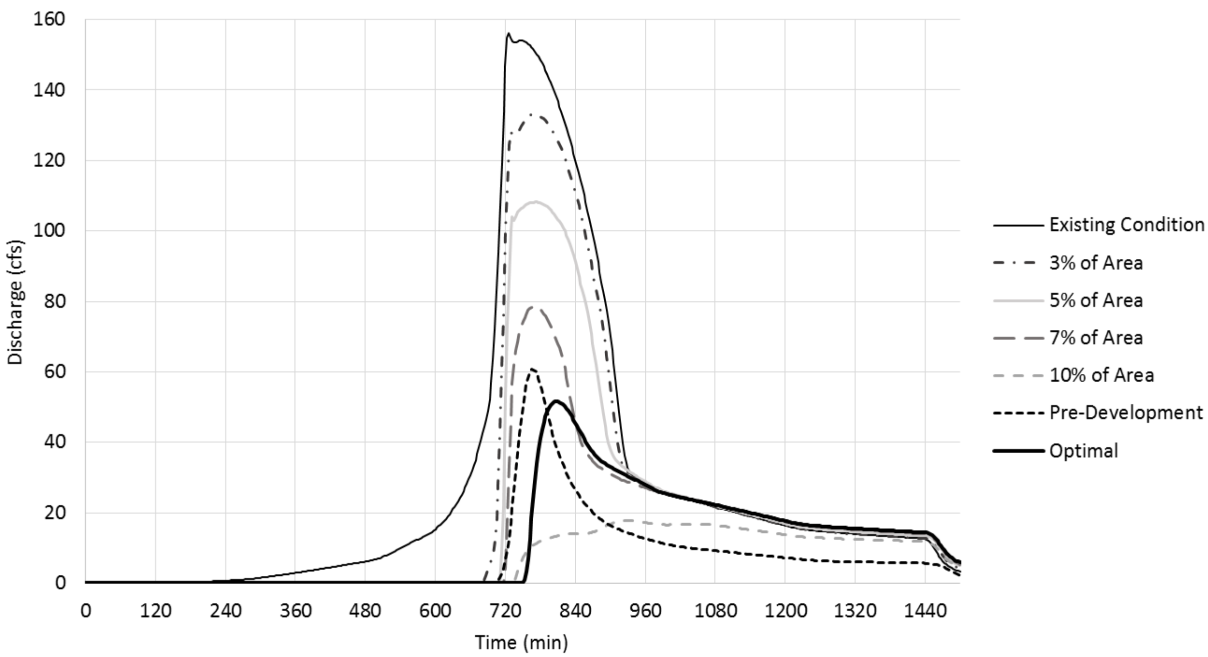

Figure 3.6: Modeled 1-Year Storm at the Webb Street Station

3.4 Conclusions

This study considered the new Virginia stormwater regulations, how bioretention could be used to meet these new regulations, and the effects that implementing bioretention to meet the new regulations would have on a larger watershed scale. Bioretention cells were located at the outfall of the sub-watersheds that make up a larger, calibrated watershed model and sized in a couple of different manners. The study resulted in several primary conclusions:

Bioretention

The installation of bioretention cells can result in the developed hydrology approaching the pre-development hydrology for both the site- and overall watershed-scales. All sizes of bioretention retrofits that were modeled showed decreased peak flows and volumes of flows from the developed, existing condition sub-watersheds. However, when larger percent area cells (7 and 10%) were modeled, the flows leaving some of the cells were very small, or non-existent, which could result in other hydrologic issues with water quantity and quality affecting aquatic life.

Optimal Sizing

The area, and resulting volume, of bioretention required to meet the RRM is directly related to the difference between the CN of the developed condition and that of the pre-development condition. As this difference increases, larger cells are needed to retain the larger amount of flow. The resulting sizes of the bioretention cells needed for the new standard of 80% of the Q*RV are typically larger in area than those seen in practice. Due to large differences between the developed and pre-development condition, modeling indicates that some sub-watersheds require cell area to be more than 10% of the drainage area. However, this begins to bring into question the feasibility of this process. The sub-watersheds with the most development, and therefore the larger necessary bioretention cells, would need to have available land area to accommodate the large cells, but based on preliminary observation, this amount of open space would not be available in many of the sub-watersheds in the area of study. Other forms of LID and BMP techniques would likely be required in this situation.

Depth Effects

If space is a major issue, modeling revealed that increasing the engineered soil media depth from 3 feet to 4 feet is a valid option. A consistent 18% reduction in area of the bioretention was shown for this change in design for meeting the regulations. This reduction was consistent across all of the sub-watersheds. Also, increasing the depth of the cells seems to result in slightly more volume attenuation, and slightly less peak attenuation. Therefore, this could be taken into account if regulations apply to either the volume or peak, but not necessarily their product.

Watershed-Scale Effects

Finally, the retrofit of bioretention in a watershed with the RRM principles results in a watershed that has hydrologic characteristics closely approaching the pre-development. When this method is applied at individual sites throughout a watershed, the model results at the watershed outlet are shown to be a peak lowered to below the pre-development, but with prolonged flows that are somewhat higher than the pre-development levels. This is a substantial improvement over previous development methods that have resulted in higher peaks and longer, higher flows in general, but requires much more space for the distributed network of smaller facilities.

In conclusion, the sizing of bioretention cells is critical to their performance. If they are sized to be too small, there is little benefit in their installation, and if they are too big, the outflow can be so small that it hurts the receiving waters. When sizing cells for certain performance goals, the required size is directly related to the CN of the contributing drainage area. However, the feasibility of the space requirements for meeting some of these goals, especially in a retrofit environment, is questionable. One possible way to overcome this is increasing the depth of the bioretention cell, which showed a consistent decrease in the required surface area. With the studied regulation applied throughout the study watershed, the hydrograph at the watershed outlet mimicked the pre-development hydrograph much closer than the hydrograph of traditional stormwater management techniques.

Chapter 4

Conclusions

4.1 Implications

This study investigated the impact that bioretention could have on the new Virginia stormwater management regulations and, more generally, the results of different sizes of bioretention cells. Whether through the use of a single bioretention cell or a network of cells within a watershed, the channel protection requirements of Virginia's Runoff Reduction Method can be achieved. The appropriate sizing of the cells was greatly dependent on the curve number of the developed condition. Applying the RRM to all of the sub-watersheds resulted in an outflow hydrograph at the watershed outfall that was similar in its characteristics to the pre-development condition. The resulting peak flow rate was lower than the pre-development, but there was a larger volume of runoff leaving the watershed. This is a large improvement over traditional development practices and would likely lead to a reduction in flows entering water bodies during flooding conditions. While it was possible to achieve these reductions, the cost in dedicated land for these facilities would be large. Also, the feasible availability of the large area for these facilities is questionable, especially in an already developed watershed.

As shown in other research (James and Dymond 2012), and confirmed in this study, the installation or retrofit of bioretention on a site or in a watershed consistently improves the hydrologic characteristics of the site and the watershed. As one would expect, cells with larger areas and volumes of storage resulted in larger reductions in peak flow and volume leaving the cell. When the developed conditions of the contributing watersheds had higher curve numbers, a larger cell was required to produce the same reductions seen by a smaller cell in a less intensely developed sub-watershed. Another factor found to affect the necessary area of bioretention to achieve the same results is the depth of the cell. Increasing the depth of each cell resulted in a consistent decrease in the area of the cells required to meet the new regulations.

4.2 Future Work

The results found in this study are most likely somewhat location-specific, and therefore, further research should be performed in other locations to verify or dispute the conclusions made. Since these regulations cover the entire state, verification of the results and trends in the state's physiographic regions should be pursued. Therefore, a future study could vary the pre-development CNs, then investigate the difference between the pre- and post-development CNs and the required bioretention for meeting a hydrologic goal.

Another aspect of this study that could use more research and refinement would be the design of the bioretention cells themselves. Additional research into how the cells are modeled may bring insight. This study assumed a constant cross-section that could infiltrate all of the runoff until it filled up and the water level rose to the weir outlet structure. Even though this study was similar in methods and scope to that of James and Dymond (2012), the method for modeling the bioretention varied greatly and centered on the assumed infiltration rate and capacity of the soil media. Including varying infiltration rates of the soil media, infiltration into the underlying soil, or an underdrain could yield different results or provide more guidance for the design of different systems. A modeling environment other than the SewerGEMS modeling approach used in this study may be required for some of these ideas due to software limitations. An ideal study for analyzing this would be geared towards specifically designing an optimal cell for differing characteristics of the contributing drainage area and other constraints that would affect the design.

Water quality benefits from bioretention and other LID practices also need further study. The data needed for model calibration for water quality were not available in this study, but water quality research could be combined with the quantity component to find optimal designs for meeting both, or either, set of regulations. Sizing bioretention to determine their water quality benefit requires more research for both the site and watershed scale. Also, as shown in this study for water quantity, the benefits of bioretention for water quality should also be investigated with an emphasis on how much is necessary to meet certain water quality goals.

Other LID practices also need further research with a similar scope to both this study and the other ideas mentioned previously. Practices such as green roofs, vegetated swales, rainwater harvesting, and other infiltration-based techniques need to be studied to determine their efficiencies. Cost and space constraints are critical on a site, especially when dealing with retrofits, and if these factors can be optimized, more of these LID techniques will be implemented with greater resulting watershed characteristics.

4.3 Final Words

In today's society and economy, growth is required. With growth will come further development of the land and more stress being placed on the natural world and its ability to sustain mankind. However, the planet's natural resources are limited and one of its key resources (fresh water) is threatened by this development. Best management practices and low impact development strategies attempt to decrease the threat to these natural systems from human development. Engineers and policy makers have started recognizing this, and the regulations in many areas are becoming more stringent on what flows are allowed to be entering natural conveyances. This should result in the prolonged improvement of the hydrologic characteristics of streams in developed areas. However, regulating agencies must also consider the cost and feasibility of the requirements of these regulations. This study analyzes the area requirements that are necessary for meeting a new set of regulations in Virginia. The required area to meet these new regulations is substantial, especially for intensely developed sites. It is the hope of this researcher that this study illustrates the importance both of maintaining the natural hydrologic characteristics of watersheds and of understanding the constraints that are required of developers and designers to meet these goals.

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Appendix A

Model Input Data

This section includes several tables that contain some of the characteristics of all of the sub-watersheds and bioretention cells modeled in the study. A map of the sub-watersheds is also included for reference.

Table A.1: Overall Pre-Development Watershed

Area (acres)	Tc (hours)	CN
474.8	1.21	71

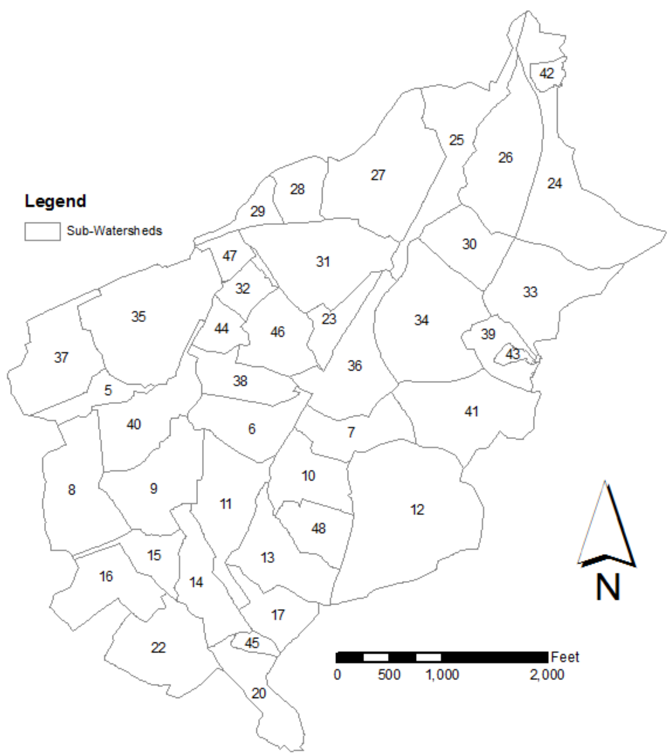

Figure A.1: Sub-Watersheds

Table A.2: Sub-Watershed Attributes

Sub-Watershed	Area (acres)	Tc (min)	Pre-Development CN	Calibrated CN
5	4.65	23.2	71	90
6	10.49	16.7	71	94
7	9.11	16.7	71	96
8	16.39	15.3	71	92
9	14.40	17.2	71	94
10	9.51	21.0	71	93
11	13.98	20.2	71	97
12	34.29	21.6	71	94
13	12.36	24.3	71	94
14	10.07	19.9	71	98
15	5.98	19.7	71	95
16	11.08	18.5	71	94
17	6.72	14.6	71	95
20	8.63	21.2	71	98
22	14.36	18.0	71	98
23	5.06	19.5	71	93
24	24.63	19.8	71	88
25	15.42	25.9	71	96
26	18.12	19.9	71	93
27	22.38	20.9	71	90
28	6.01	16.0	71	98
29	3.70	16.0	71	94
30	9.00	17.6	71	95
31	19.23	19.7	71	96
32	3.80	13.4	71	94
33	15.92	14.5	71	87
34	23.02	23.5	71	89
35	24.92	23.2	71	96
36	13.83	18.2	71	95
37	14.53	24.3	71	97
38	7.76	9.6	71	98
39	5.90	28.5	71	96
40	13.05	14.4	71	94
41	17.67	16.9	71	88
42	1.80	12.6	71	98
43	1.08	15.0	71	95
44	3.96	14.0	71	94
45	1.60	12.3	71	98
46	10.31	19.2	71	80
47	3.17	15.7	71	98
48	6.94	15.3	71	94

Table A.3: Bioretention Cells

Sub-Watershed	Area (acres)	Calibrated CN	Cell Invert (ft)	Weir Elevation (ft)	Weir Length (ft)	3% Cell Size (sf)	5% Cell Size (sf)	7% Cell Size (sf)	10% Cell Size (sf)	Optimal			
										3' % Area	3' Area (sf)	4' % Area	4' Area (sf)
5	4.65	90	2107.00	2110.50	16.3	6,080.56	10,134.26	14,187.96	20,268.52	6.06%	12,282.72	4.97%	10,071.43
6	10.49	94	2085.14	2088.64	39.6	13,704.26	22,840.43	31,976.60	45,680.86	8.29%	37,869.43	6.82%	31,154.34
7	9.11	96	2084.56	2088.06	36.7	11,899.63	19,832.72	27,765.80	39,665.43	9.01%	35,730.62	7.40%	29,352.42
8	16.39	92	2073.09	2076.59	57.8	21,416.20	35,693.67	49,971.14	71,387.34	6.86%	48,971.72	5.64%	40,262.46
9	14.40	94	2069.00	2072.50	52.2	18,822.18	31,370.30	43,918.42	62,740.60	7.73%	48,523.58	6.35%	39,834.01
10	9.51	93	2074.40	2077.90	29.5	12,426.08	20,710.14	28,994.19	41,420.27	7.29%	30,195.38	5.97%	24,727.90
11	13.98	97	2062.09	2065.59	54.6	18,264.52	30,440.86	42,617.21	60,881.73	10.03%	61,064.37	8.21%	49,983.90
12	34.29	94	2093.05	2096.55	121.1	44,815.58	74,692.63	104,569.68	149,385.26	8.20%	122,495.91	6.70%	100,088.12
13	12.36	94	2063.78	2067.28	41.7	16,154.61	26,924.36	37,694.10	53,848.71	8.59%	46,256.04	7.04%	37,909.49
14	10.07	98	2055.30	2058.80	55.6	13,158.52	21,930.86	30,703.21	43,861.73	11.02%	48,335.62	9.02%	39,563.28
15	5.98	95	2055.30	2058.80	23.1	7,808.15	13,013.59	18,219.02	26,027.17	8.68%	22,591.59	7.13%	18,557.38
16	11.08	94	2057.50	2061.00	37.3	14,477.99	24,129.99	33,781.99	48,259.98	8.10%	39,090.58	6.60%	31,851.59
17	6.72	95	2073.06	2076.56	26.2	8,776.31	14,627.18	20,478.05	29,254.35	8.39%	24,544.40	6.89%	20,156.25
20	8.63	98	2078.49	2081.99	47.7	11,276.31	18,793.85	26,311.38	37,587.69	11.08%	41,628.37	9.08%	34,129.62
22	14.36	98	2048.78	2052.28	64.2	18,761.88	31,269.80	43,777.72	62,539.60	10.82%	67,667.85	8.90%	55,660.24
23	5.06	93	2114.00	2117.50	15.4	6,607.88	11,013.14	15,418.40	22,026.28	7.68%	16,916.18	6.30%	13,876.56
24	24.63	88	2164.08	2167.58	62.3	32,187.00	53,645.01	75,103.01	107,290.02	4.83%	51,821.08	3.97%	42,540.49
25	15.42	96	2168.07	2171.57	57.9	20,150.13	33,583.55	47,016.98	67,167.11	9.67%	64,950.59	7.87%	52,860.51
26	18.12	93	2167.60	2171.10	54.7	23,684.64	39,474.41	55,264.17	78,948.81	7.52%	59,369.51	6.18%	48,790.37
27	22.38	90	2149.80	2153.30	67.9	29,245.67	48,742.78	68,239.89	97,485.56	6.02%	58,686.31	4.90%	47,738.68
28	6.01	98	2154.96	2158.46	25.0	7,857.96	13,096.60	18,335.23	26,193.19	10.70%	28,034.57	8.80%	23,050.01
29	3.70	94	2153.63	2157.13	13.5	4,830.10	8,050.16	11,270.23	16,100.33	8.04%	12,942.89	6.60%	10,618.17
30	9.00	95	2141.80	2145.30	33.3	11,761.54	19,602.56	27,443.59	39,205.13	8.44%	33,093.05	6.93%	27,169.15
31	19.23	96	2127.23	2130.73	68.8	25,130.36	41,883.94	58,637.51	83,767.87	9.39%	78,658.03	7.67%	64,249.96
32	3.80	94	2134.00	2137.50	15.1	4,963.51	8,272.52	11,581.52	16,545.03	8.18%	13,533.84	6.71%	11,101.72
33	15.92	87	2142.57	2146.07	45.3	20,807.15	34,678.59	48,550.02	69,357.18	4.50%	31,210.73	3.70%	25,662.16
34	23.02	89	2109.34	2112.84	59.5	30,086.75	50,144.58	70,202.41	100,289.16	5.64%	56,563.08	4.63%	46,433.88

Table A.3: Continued

Sub-Watershed	Area (acres)	Calibrated CN	Cell Invert (ft)	Weir Elevation (ft)	Weir Length (ft)	3% Cell Size (sf)	5% Cell Size (sf)	7% Cell Size (sf)	10% Cell Size (sf)	Optimal			
										3' % Area	3' Area (sf)	4' % Area	4' Area (sf)
35	24.92	96	2101.02	2104.52	92.7	32,567.44	54,279.07	75,990.69	108,558.13	9.64%	104,682.61	7.86%	85,304.98
36	13.83	95	2094.99	2098.49	56.1	18,072.92	30,121.53	42,170.15	60,243.07	8.84%	53,254.87	7.26%	43,736.47
37	14.53	97	2108.54	2112.04	55.8	18,982.26	31,637.09	44,291.93	63,274.19	10.32%	65,298.96	8.43%	53,308.50
38	7.76	98	2102.22	2105.72	42.9	10,142.78	16,904.64	23,666.49	33,809.28	10.49%	35,465.93	8.61%	29,092.88
39	5.90	96	2142.00	2145.50	18.9	7,716.44	12,860.73	18,005.02	25,721.46	9.88%	25,402.98	8.09%	20,813.81
40	13.05	94	2088.93	2092.43	50.4	17,051.93	28,419.89	39,787.84	56,839.78	8.15%	46,296.00	6.69%	38,025.81
41	17.67	88	2115.66	2119.16	49.1	23,092.96	38,488.27	53,883.57	76,976.54	4.77%	36,679.32	3.88%	29,866.90
42	1.80	98	2215.00	2218.50	9.6	2,353.45	3,922.42	5,491.38	7,844.83	10.51%	8,245.70	8.63%	6,770.09
43	1.08	95	2161.00	2164.50	5.2	1,414.56	2,357.61	3,300.65	4,715.21	8.40%	3,958.89	6.88%	3,241.71
44	3.96	94	2128.00	2131.50	14.5	5,176.31	8,627.19	12,078.06	17,254.38	8.18%	14,105.45	6.72%	11,594.94
45	1.60	98	2093.39	2096.89	6.8	2,094.09	3,490.15	4,886.22	6,980.31	10.45%	7,294.42	8.62%	6,014.23
46	10.31	80	2114.00	2117.50	20.2	13,475.48	22,459.13	31,442.78	44,918.26	2.29%	10,286.28	1.90%	8,512.01
47	3.17	98	2144.73	2148.23	16.9	4,139.33	6,898.89	9,658.44	13,797.78	10.60%	14,621.50	8.70%	12,004.07
48	6.94	94	2079.00	2082.50	26.8	9,064.65	15,107.75	21,150.85	30,215.50	8.07%	24,368.80	6.62%	20,002.66

Appendix B

Model Calibration

The existing conditions model was calibrated for four storm events that occurred during the summer of 2009 in order to have a more accurate representation of the actual watershed. The four events were chosen carefully. They were selected because they were single-peak events, like the NRCS design storms used in the analysis. It was also deemed to be important that they were from 2009 because the aerial imagery used for curve number development was from 2009 and an industrial facility had begun construction during that year. Several goodness-of-fit measures are shown in Table B.1 for the four storms. For these storms, only the immediate duration around the runoff from the storm was used. The primary two metrics consulted in the calibration process were the Nash-Sutcliffe model efficiency coefficient and the deviation in the product of the peak flow rate and volume of runoff. The model continuity value is calculated by SewerGEMS and represents water loss from elements in the model. Figures B.1 – B.4 show the resulting hydrographs from the model calibration results, and Table B.2 contains the measured and modeled data in tabular form.

Table B.1: Model Calibration Results

Storm Event	5/8/2009	5/14/2009	6/15/2009	7/17/2009
Duration of Runoff (hr)	15.5	8.0	4.5	13.5
Time Step (hr)	0.25	0.25	0.25	0.25
Nash-Sutcliffe R^2 (1970)	0.78	0.81	0.66	0.64
Deviation in Volume	32%	34%	-24%	4%
Deviation in Peak Flow	-8%	-12%	-21%	-21%
Deviation in Q*RV	21%	18%	-40%	-18%
Peak Time Shift (hr)	0	-0.25	0	0
Model Continuity Error	0.0%	0.2%	0.0%	0.2%

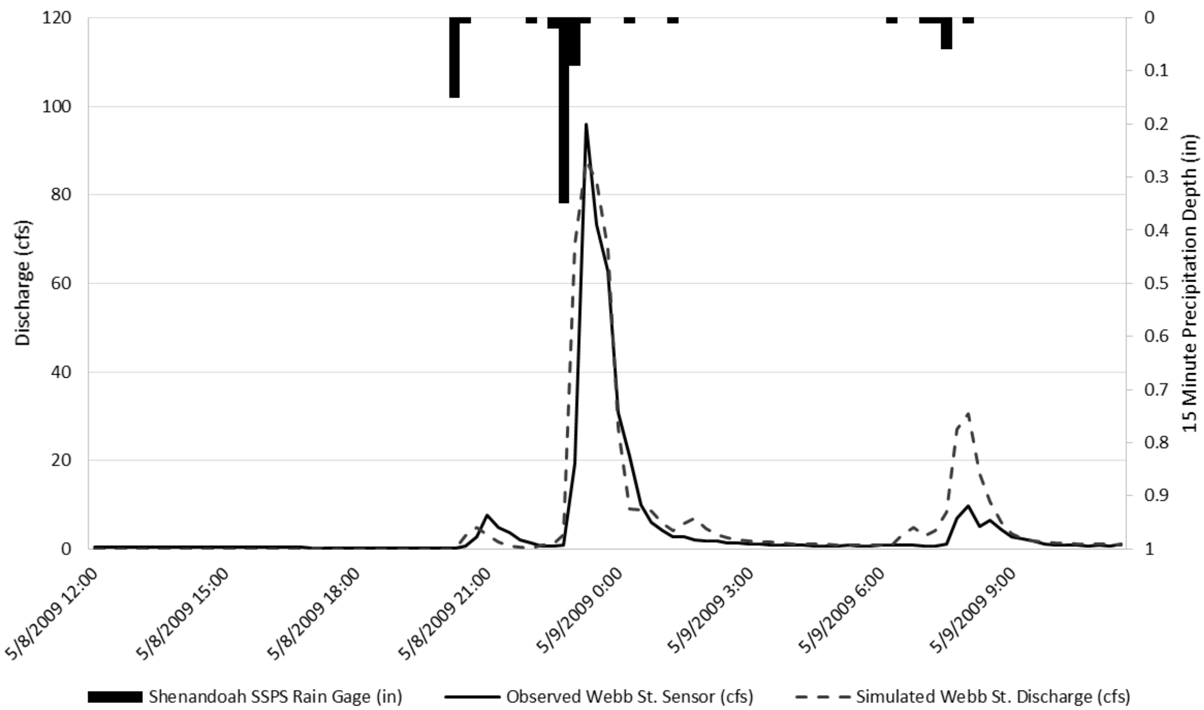

Figure B.1: May 8, 2009 Storm Event

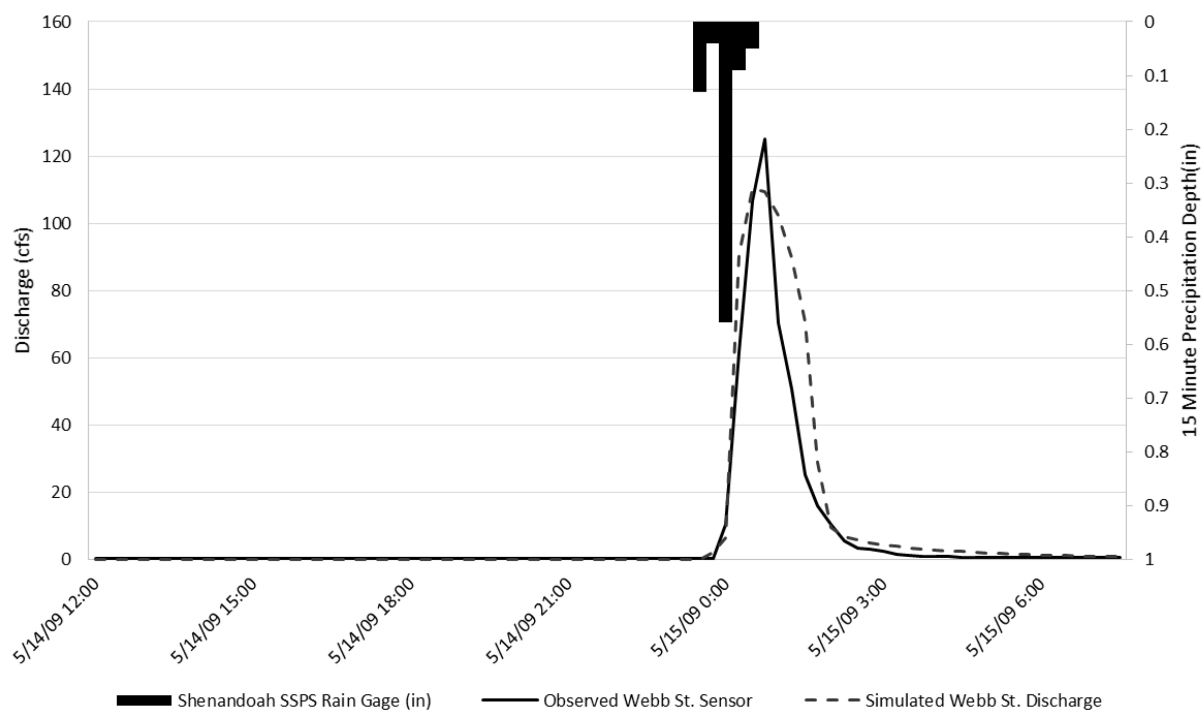

Figure B.2: May 14, 2009 Storm Event

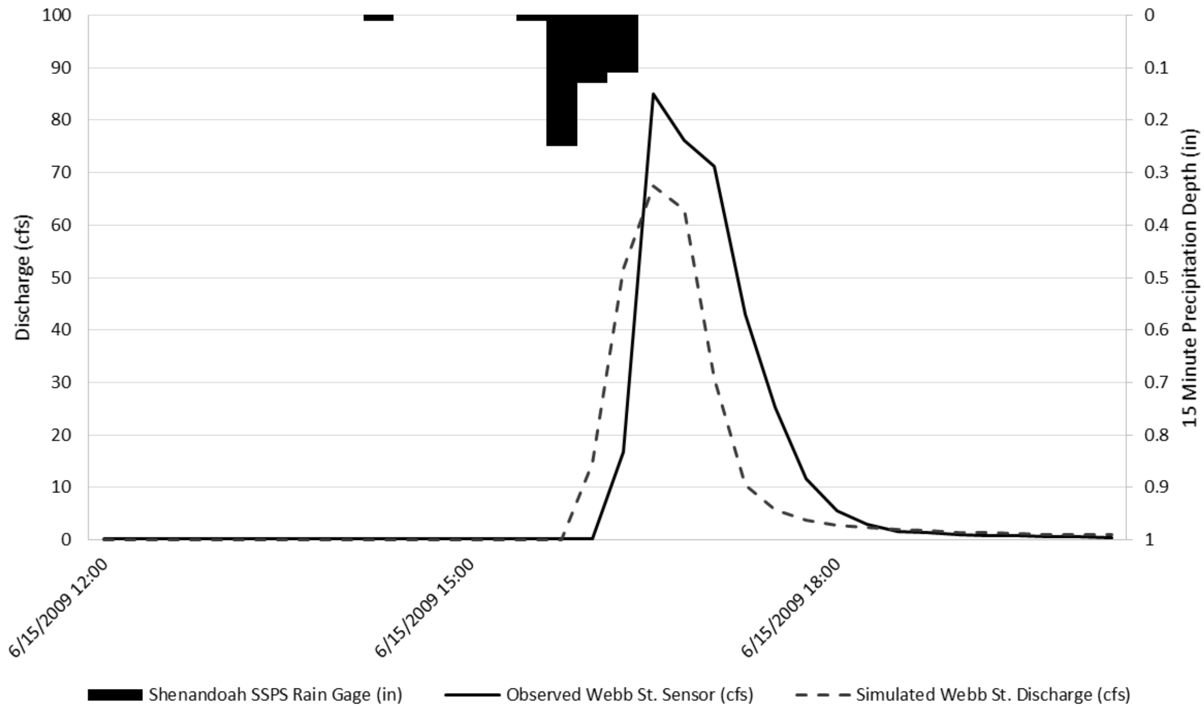

Figure B.3: June 15, 2009 Storm Event

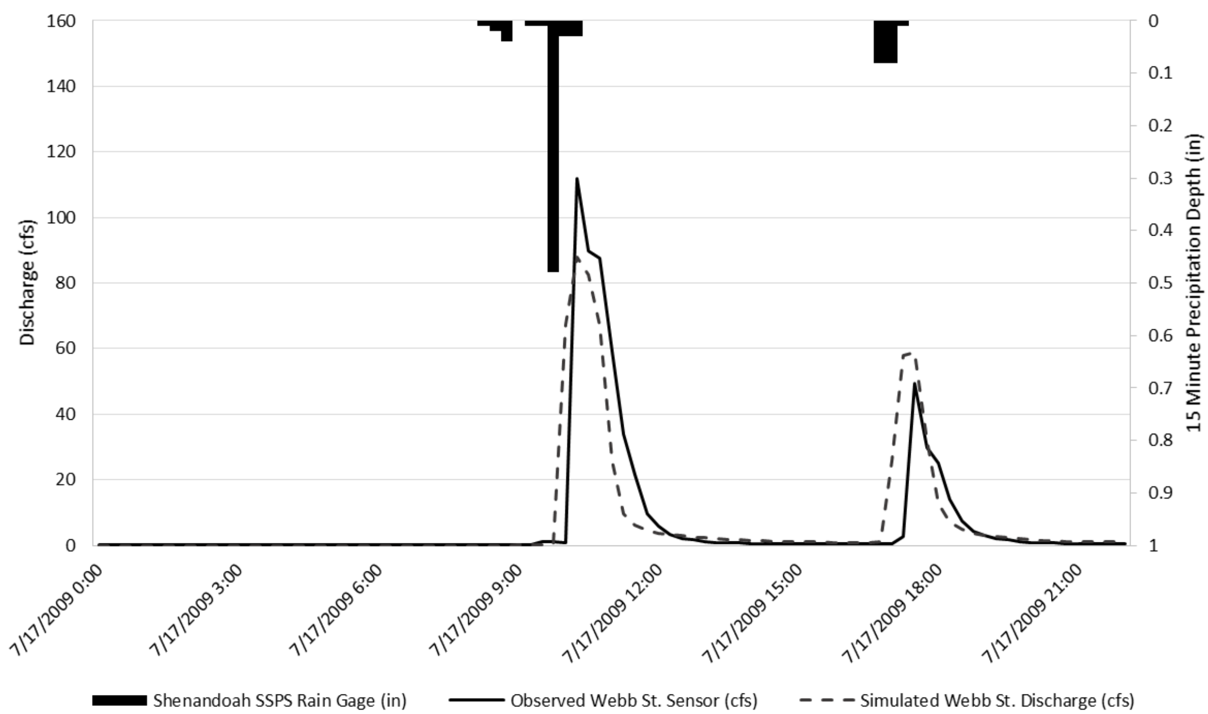

Figure B.4: July 17, 2009 Storm Event

Table B.2: Calibration Storm Events

Model Time (hr)	5/8/2009 Measured	5/8/2009 Modeled	5/14/2009 Measured	5/14/2009 Modeled	6/15/2009 Measured	6/15/2009 Modeled	7/17/2009 Measured	7/17/2009 Modeled
0.00	0.18	0.00	0.29	0.00	0.29	0.00	0.20	0.00
0.25	0.18	0.00	0.36	2.24	0.27	14.64	0.25	0.00
0.50	0.56	2.89	10.56	6.55	16.78	51.62	0.25	0.16
0.75	2.83	4.90	61.80	90.66	84.91	67.37	0.33	0.21
1.00	7.75	3.22	106.70	110.28	76.06	62.93	1.29	0.25
1.25	4.92	1.48	125.01	109.31	71.14	30.76	1.20	0.45
1.50	3.72	0.73	70.32	102.25	43.02	10.43	0.89	67.5
1.75	2.01	0.42	50.58	89.80	25.31	5.70	111.89	87.96
2.00	1.27	0.30	25.31	70.63	11.63	3.79	89.94	82.63
2.25	0.62	0.94	15.95	28.86	5.57	2.83	87.47	67.02
2.50	0.62	1.20	10.61	9.42	3.05	2.33	59.20	25.72
2.75	0.91	3.31	5.66	6.85	1.65	1.98	33.98	9.51
3.00	19.43	68.77	3.32	5.73	1.31	1.71	21.68	6.15
3.25	95.94	87.79	3.21	5.00	1.09	1.48	9.60	4.64
3.50	73.37	83.07	2.34	4.42	0.87	1.30	5.73	3.80
3.75	62.76	67.65	1.60	3.91	0.74	1.17	3.48	3.28
4.00	30.86	27.25	1.36	3.47	0.62	1.10	2.21	2.88
4.25	20.97	9.17	1.02	3.10	0.60	1.05	1.69	2.55
4.50	9.96	8.78	0.98	2.80	0.42	1.01	1.16	2.27
4.75	6.13	8.60	0.89	2.55			0.89	2.03
5.00	4.12	5.75	0.76	2.32			0.74	1.82
5.25	2.72	4.11	0.76	2.11			0.71	1.63
5.50	2.83	5.89	0.69	1.92			0.62	1.47
5.75	2.09	6.98	0.74	1.75			0.60	1.33
6.00	1.72	4.65	0.60	1.60			0.53	1.20
6.25	1.72	3.12	0.62	1.45			0.53	1.10
6.50	1.40	2.47	0.58	1.32			0.51	1.04
6.75	1.25	2.13	0.60	1.20			0.51	1.01
7.00	1.18	1.88	0.56	1.11			0.49	0.99
7.25	1.09	1.66	0.53	1.07			0.49	0.97
7.50	0.91	1.48	0.53	1.04			0.49	0.96
7.75	0.82	1.31	0.51	1.02			0.47	0.94
8.00	0.94	1.18	0.53	1.00			0.47	0.92
8.25	0.82	1.09					0.47	1.02
8.50	0.76	1.04					0.47	25.86
8.75	0.71	1.01					2.81	57.88
9.00	0.71	0.99					49.28	58.74
9.25	0.78	0.97					29.79	32.07
9.50	0.69	0.95					25.00	12.76
9.75	0.65	0.93					13.93	7.07

Table B.2: Continued

Model Time (hr)	5/8/2009		5/14/2009		6/15/2009		7/17/2009	
	Measured	Modeled	Measured	Modeled	Measured	Modeled	Measured	Modeled
10.00	0.78	0.91					7.60	4.78
10.25	0.91	0.91					4.26	3.62
10.50	0.78	3.25					2.96	2.98
10.75	0.82	4.88					2.05	2.57
11.00	0.71	3.00					1.67	2.24
11.25	0.76	4.20					1.02	1.96
11.50	1.23	8.32					0.74	1.73
11.75	7.02	27.1					0.69	1.54
12.00	9.71	30.44					0.67	1.36
12.25	5.17	16.89					0.60	1.23
12.50	6.53	10.68					0.58	1.13
12.75	4.32	5.92					0.56	1.08
13.00	2.81	3.47					0.56	1.05
13.25	2.29	2.38					0.56	1.02
13.50	1.80	1.91					0.53	1.00
13.75	1.20	1.66						
14.00	1.00	1.47						
14.25	0.91	1.32						
14.50	0.85	1.21						
14.75	0.76	1.12						
15.00	0.80	1.06						
15.25	0.71	1.01						
15.50	1.02	0.97						

Appendix C

Model Output Data

This section contains the results from the modeling scenarios. The peak flow rate and volume of runoff are shown for each sub-watershed and the overall watershed in Table C.1. The volume of runoff is given in inches to represent an inch of runoff across the whole sub-watershed/ watershed. Note that the previous discussion of these results was done by comparing the developed scenarios as percentages versus the pre-development condition.

Table C.1: Modeled Peak Flow Rates and Volumes

			Pre-Development		Ex. Cond. (0%)		3%		5%		7%		10%		Optimal 3'		Optimal 4'	
Watershed	Area (acres)	Calibrated CN	Peak (cfs)	Volume (in)	Peak (cfs)	Volume (in)	Peak (cfs)	Volume (in)	Peak (cfs)	Volume (in)	Peak (cfs)	Volume (in)	Peak (cfs)	Volume (in)	Peak (cfs)	Volume (in)	Peak (cfs)	Volume (in)
5	4.65	90	1.30	0.386	8.32	1.337	6.78	0.999	1.70	0.746	0.39	0.483	0.07	0.030	0.67	0.599	0.68	0.586
6	10.49	94	3.65	0.386	21.86	1.700	14.59	1.345	13.17	1.121	4.43	0.857	0.67	0.456	1.65	0.681	1.69	0.664
7	9.11	96	3.16	0.386	20.70	1.803	19.76	1.460	15.62	1.220	6.21	0.964	0.86	0.567	1.42	0.688	1.45	0.673
8	16.39	92	5.95	0.386	31.05	1.500	15.34	1.117	12.85	0.919	2.63	0.653	0.52	0.224	2.79	0.662	2.84	0.644
9	14.40	94	4.95	0.386	28.48	1.618	27.36	1.280	15.38	1.038	4.08	0.774	0.70	0.369	2.27	0.670	2.34	0.654
10	9.51	93	2.89	0.386	16.27	1.539	14.68	1.204	7.29	0.956	1.75	0.683	0.34	0.269	1.39	0.642	1.42	0.626
11	13.98	97	4.33	0.386	30.49	1.926	30.14	1.581	25.45	1.341	12.92	1.076	2.00	0.682	1.98	0.678	2.03	0.661
12	34.29	94	10.16	0.386	67.10	1.659	16.00	1.059	14.43	1.060	10.19	0.814	1.94	0.414	4.83	0.646	4.94	0.633
13	12.36	94	3.38	0.386	23.56	1.704	22.54	1.369	14.80	1.125	5.12	0.859	0.79	0.460	1.63	0.639	1.67	0.624
14	10.07	98	3.14	0.386	27.89	2.052	27.32	1.693	25.18	1.452	17.33	1.187	2.51	0.810	1.42	0.682	1.46	0.663
15	5.98	95	1.86	0.385	12.83	1.735	12.38	1.395	8.64	1.159	3.06	0.896	0.42	0.492	0.87	0.660	0.89	0.648
16	11.08	94	3.60	0.386	20.99	1.661	10.86	1.237	9.60	1.071	3.72	0.816	0.62	0.413	1.66	0.666	1.71	0.651
17	6.72	95	2.49	0.386	14.60	1.726	11.04	1.393	9.73	1.147	3.33	0.887	0.47	0.485	1.11	0.695	1.14	0.677
20	8.63	98	2.57	0.386	23.77	2.052	23.38	1.691	21.50	1.451	14.84	1.188	2.15	0.810	1.17	0.674	1.21	0.657
22	14.36	98	4.76	0.386	35.03	2.052	33.99	1.700	31.61	1.459	20.18	1.201	3.48	0.816	2.11	0.699	2.16	0.678
23	5.06	93	1.59	0.385	8.55	1.603	7.99	1.268	4.36	1.015	1.24	0.745	0.22	0.342	0.75	0.654	0.77	0.638
24	24.63	88	7.69	0.386	30.85	1.174	18.50	0.828	3.61	0.568	1.04	0.308	0.00	0.000	4.06	0.584	4.15	0.571
25	15.42	96	4.07	0.386	32.54	1.841	18.23	1.415	17.00	1.263	11.35	0.999	1.65	0.602	1.96	0.644	2.00	0.624
26	18.12	93	5.63	0.386	30.17	1.580	28.10	1.247	14.45	0.993	4.06	0.727	0.72	0.312	2.68	0.651	2.74	0.633
27	22.38	90	6.74	0.386	35.79	1.343	15.11	1.003	8.26	0.747	1.87	0.487	0.35	0.035	3.40	0.612	3.48	0.598
28	6.01	98	2.13	0.386	14.12	2.053	10.51	1.706	9.98	1.455	7.94	1.198	1.41	0.808	0.92	0.710	0.95	0.694
29	3.70	94	1.31	0.386	7.45	1.669	7.22	1.328	4.59	1.092	1.30	0.826	0.22	0.424	0.59	0.680	0.61	0.665
30	9.00	95	3.02	0.386	18.52	1.717	18.04	1.376	12.38	1.139	4.16	0.876	0.60	0.472	1.38	0.678	1.41	0.662
31	19.23	96	6.01	0.386	39.36	1.841	37.76	1.499	29.81	1.265	13.62	1.000	2.02	0.599	2.75	0.675	2.83	0.660
32	3.80	94	1.47	0.387	8.23	1.704	7.86	1.362	5.47	1.129	1.68	0.866	0.25	0.461	0.65	0.699	0.67	0.683
33	15.92	87	5.90	0.386	22.12	1.138	12.52	0.803	2.02	0.539	0.57	0.265	0.00	0.000	3.06	0.596	3.13	0.582
34	23.02	89	6.36	0.386	30.34	1.281	22.21	0.944	5.85	0.678	1.44	0.420	0.00	0.000	3.35	0.589	3.40	0.574
35	24.92	96	7.08	0.386	52.54	1.855	18.39	1.272	18.25	1.258	14.56	1.011	2.77	0.614	3.32	0.657	3.41	0.641

Table C.1: Continued

			Pre-Development		Ex. Cond. (0%)		3%		5%		7%		10%		Optimal 3'		Optimal 4'	
Watershed	Area (acres)	Calibrated CN	Peak (cfs)	Volume (in)	Peak (cfs)	Volume (in)	Peak (cfs)	Volume (in)	Peak (cfs)	Volume (in)	Peak (cfs)	Volume (in)	Peak (cfs)	Volume (in)	Peak (cfs)	Volume (in)	Peak (cfs)	Volume (in)
36	13.83	95	4.53	0.386	30.90	1.767	29.48	1.423	22.80	1.193	8.46	0.930	1.14	0.532	2.08	0.673	2.12	0.657
37	14.53	97	3.95	0.386	31.49	1.942	17.21	1.489	16.51	1.350	13.83	1.093	2.19	0.697	1.86	0.657	1.90	0.638
38	7.76	98	3.43	0.386	23.89	2.053	23.49	1.694	20.44	1.442	14.14	1.187	1.95	0.816	1.41	0.749	1.44	0.731
39	5.90	96	1.45	0.385	10.76	1.872	10.39	1.534	8.22	1.294	4.01	1.018	0.69	0.628	0.70	0.634	0.72	0.617
40	13.05	94	4.84	0.386	27.90	1.691	26.77	1.349	17.67	1.115	5.49	0.856	0.82	0.446	2.17	0.690	2.22	0.673
41	17.67	88	6.07	0.386	24.80	1.171	11.19	0.828	2.66	0.573	0.75	0.308	0.00	0.000	3.14	0.595	3.22	0.582
42	1.80	98	0.72	0.382	5.25	2.037	4.94	1.676	4.51	1.434	2.96	1.175	0.42	0.796	0.30	0.728	0.31	0.707
43	1.08	95	0.40	0.375	2.74	1.713	2.62	1.356	1.84	1.092	0.49	0.860	0.07	0.475	0.17	0.678	0.18	0.663
44	3.96	94	1.48	0.387	8.15	1.699	7.92	1.358	5.34	1.123	1.70	0.858	0.25	0.451	0.66	0.693	0.68	0.675
45	1.60	98	0.65	0.385	3.85	2.039	3.70	1.691	3.39	1.444	2.12	1.185	0.36	0.799	0.27	0.741	0.27	0.718
46	10.31	80	3.26	0.386	8.10	0.758	0.92	0.398	0.21	0.123	0.00	0.000	0.00	0.000	2.10	0.481	2.13	0.471
47	3.17	98	1.14	0.386	9.18	2.029	8.66	1.670	7.81	1.422	5.14	1.167	0.71	0.791	0.50	0.708	0.51	0.688
48	6.94	94	2.50	0.386	14.88	1.675	14.17	1.335	9.19	1.098	2.68	0.842	0.42	0.432	1.13	0.686	1.16	0.667
Overall	474.82	-	60.64	0.384	156.29	1.451	133.12	1.134	108.18	0.967	78.59	0.744	17.77	0.363	51.65	0.603	51.48	0.587