

Spring 1-1-2015

# Lost Depression Storage – A Physical Meaning for Low Impact Development Water Quality Volume

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LOST DEPRESSION STORAGE - A PHYSICAL MEANING FOR LOW IMPACT  
DEVELOPMENT WATER QUALITY VOLUME

by

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B.S., University of Iowa, 1985

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A thesis submitted to the  
Faculty of the Graduate School of the  
University of Colorado in partial fulfillment  
of the requirement for the degree of  
Masters of Science  
Department of Civil Engineering  
2015

This thesis entitled:

Lost Depression Storage – A Physical Meaning for Low Impact Development Water Quality  
Volume

written by Mary Jean O’Hair  
has been approved for the Department of Civil Engineering

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Date \_\_\_\_\_

The final copy of this thesis has been examined by the signatories, and we find that both the content and the form meet acceptable presentation standards of scholarly work in the above mentioned discipline.

O'Hair, Mary Jean (M.S., Civil Engineering)

Lost Depression Storage – A Physical Meaning for Low Impact Development Water Quality  
Volume

Thesis directed by Professor James C. Guo

## ABSTRACT

Urban sprawl resulting from unplanned or sub-optimal development is increasing at an alarming rate of two million acres per year in the US (Davis & McCuen, 2005). As urban sprawl increases so does the amount of impervious surfaces and vehicular traffic required to get from point A to point B. The pollutants from urbanization adhere to the impervious surfaces and are then dislodged and transported by a precipitation event, flowing into receiving waters or treatment plants. The polluted stormwater flow jeopardizes the water quality of the watersheds or strains the treatment facilities. To combat the issues that come with development and urban sprawl, municipalities are moving toward low impact development (LID). LID uses systems based on natural processes to reduce stormwater peak flows and runoff volume while increasing water quality to pre-development levels. States have developed best management practices (BMP) for the implementation of LID processes appropriate to their areas. BMPs that are used for pollutant removal such as a bioretention BMP, or rain garden, are designed to capture a volume of stormwater, which is then infiltrated or be absorbed by vegetation. While hydrologists and engineers use various methods to calculate rain garden volume, a key component to each method is capturing the first flush. The first flush contains the high pollutant content, which is dislodged at the inception of a precipitation event. Since LID mimics natural processes, it follows that the volume to capture for water quality is related to an aspect of the



hydrological cycle. This thesis shows the relationship of the volume necessary to capture for water quality to the depression storage lost due the increase in impervious area. The study is based on historical precipitation data in Denver, Colorado, and the depression storage design values for pre- and post-development for the area. The study shows that the lost depression storage is a reasonable estimate of the derived water quality capture volume (WQCV) used for the design of bioretention BMPs in Colorado. This finding is significant for the future design of water quality BMPs by providing a simple, robust, cost-effective and transferrable method to size water quality basins.

## DEDICATION

To my husband, Mike, for his unwavering support.

## ACKNOWLEDGEMENT

I would like to thank Dr. James Guo for his time and direction in the development of my thesis topic and data analysis. His knowledge, prior research, and expertise on the subject were invaluable. I would also like to thank Dr. Joesph Kasprzyk, whose class on Water Resource Management provided me with the tools for literature review and organization of my research. He also provided guidance in focusing on a thesis topic. I want to thank Dr. Balaji Rajagopalan and Dr. JoAnn Silverstein for mentoring me through the undergraduate and graduate engineering program and providing the contacts to pursue my interest in low impact development. My friends and family have put up with me during this process and have continued to provide their encouragement and support: thank you so much. Lastly, I would like to thank Jeff Walters, whose encouragement and first hand experience helped me in completing my thesis.

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# CHAPTER 1

## Introduction

### 1.1 Background

Urban sprawl, the result of unplanned or sub-optimal development is increasing at an alarming rate of two million acres per year in the US (Davis & McCuen, 2005). Poor planning during urban development results in more roads and vehicle traffic moving from point A to point B. This style of development means a need for more parking, driveways and rooftops which all increase the amount of impervious area. Urban planners are facing the challenges associated with the increasing amount of impervious surfaces caused by development and urban sprawl (Migues, Rezende, & Verol, 2014). An impervious surface precludes the ability of stormwater to infiltrate into the ground and minimizes the volume of stormwater stored in surface depressions. Reduced infiltration and depression storage causes an increase in stormwater runoff, higher peak flows, and an increase in pollutants entering into receiving waters. Stream quality will start to decline when the amount of impervious area is between 10% to 15% of the watershed (Brabec, Schulte, & Richards, 2002). Studies show that development increases surface runoff. In a pre-development area of forests and grassland, 10% to 30% of the rainfall will run off. In a developed area, depending on land use, 10% to 95% of the rainfall will run off (Davis & McCuen, 2005). A study done in Akron, Ohio, found that the average runoff depths increased by 220% for low-density housing, 1000% for high density housing and 1820% for commercial land use when compared to woodland runoff depths (Harbor, 1994). Urban sprawl changes the hydrology of an area increasing runoff volumes and also reduces the ability to remove of pollutants.



Figure 1-1: Urban Sprawl (Hull, n.d.)

The naturally occurring indentations, or depressions, in the ground surface that provide storage for precipitation are referred to as depression storage. In a simplified hydrological cycle, precipitation that reaches the ground will fill any depressions. Precipitation that fills the depressions, infiltrates into the ground, is absorbed by vegetation or evaporates. After the depression storage is filled the excess precipitation will run off. As runoff flows overland, a portion will infiltrate into the ground prior to reaching receiving waters. A pre-development area tends to be covered with vegetation. The vegetation allows higher infiltration of the runoff since it will slow the overland flow. Additionally, the vegetation aids in the removal of suspended solids through filtering (Perlman, 2014). The hydrological processes associated with depression storage, vegetation filtering and absorption, and infiltration all help to remove pollutants from runoff (US EPA, n.d.-c). The increase in impervious area with urban sprawl reduces these processes.

Developed areas have a “Catch 22”: not only are the hydrological systems that treat runoff diminished by the intensification of impervious areas, the pollutant load from urbanization is increased. As urban areas grow, so do transportation needs and industry. Vehicular traffic and industry increase the pollutant load of heavy metals, hydrocarbons, salt, sediment, polychlorinated biphenyls (PCBs), polycyclic aromatic hydrocarbons (PAHs), phenols mercury, and sulfur oxides. In residential areas lawns and gardens allow for infiltration but their maintenance increases non-point source pollutants including; phosphorus, nitrogen, fertilizers, pesticides, and organic debris (“Bioretention - Minnesota Stormwater Manual,” 2015).

The traditional approach to mitigate runoff issues in urban areas is the use of curb and gutters as shown in Figures 1-2 and 1-3. Figure 1-4 shows the runoff flowing through a grate directly into the lake. When a precipitation event occurs in a developed area, only a small amount of precipitation is stored in depressions. The remainder of the precipitation flows from pervious and impervious areas and is channeled through a storm sewer or combined sewer system into a receiving water body or a treatment plant. The inherent problem with this process is that pollutants adhering to the impervious surfaces are dislodged and transported by the stormwater. The unimpeded runoff contaminates the receiving water body and jeopardizes the riparian habitat. In the case of a combined sewer system, the additional stormwater has the potential to tax an already stressed treatment facility.

To ensure the water quality of receiving waters, The Clean Water Act, Section 402, now requires Nation Pollutant Discharge Elimination Systems (NPDES) permits for municipal separate storm sewer systems (US EPA, n.d.-b). In order to counteract the repercussions of development on watershed hydrology and to meet the NPDES requirements, urban planners are shifting toward low impact development (LID).



Figure 1-2: Storm Drain "Dispose No Waste - Drains to Creek", Boulder, Colorado (Photo by Mary Jean O'Hair)



Figure 1-3: Storm Event, Okoboji IA (Photo by Mary Jean O'Hair)





Figure 1-4: Tradition channeled flow directly into lake Okoboji IA (Photo by Mary Jean O'Hair)

## 1.2 Low Impact Development (LID)

LID is a methodology used to develop processes to manage the stormwater runoff from impervious surfaces, as well as address water quality (Urbonas, James, & MacKenzie, 2011). The premise of LID is to use natural processes, near the source, to achieve pre-development peak runoff, runoff volume, and water quality (US EPA, n.d.-a).

Dietz and Clausen preformed a study of two small watersheds in Waterford, CT. One neighborhood was developed using traditional method including curb and gutters, while the other

used LID processes. They found that in the traditional neighborhood, the runoff and pollutant load increased exponentially as the impervious area increased. In the LID neighborhood however, runoff volume and pollutant load remained consistent as the impervious area increased (Dietz & Clausen, 2008). These findings clearly demonstrate the efficiency of LID practices, but in order for them to be widely accepted and used, social, economic, political, and environmental impacts need to be considered. Best management practices (BMP), have been developed to create economically and technically feasible solutions to meet the requirements of LID (Wanielista et al., 1997).

A key component to the success of LID is determining the appropriate sizing of systems to achieve the watershed objectives. To meet the objective of sustaining pre-development water quality, the sizing of a basin to capture and treat the pollutants in stormwater runoff is critical.

Determining the optimum size for the design of a water quality basin requires money and expertise. BMPs throughout the United States use different methodologies to determine appropriate volumes for a water quality basin. Since LID mimics natural processes, it should correlate that a natural phenomenon can emulate the volume necessary to capture in order to provide pre-development water quality. The research for this thesis uses the optimized water quality capture volume (WQCV), derived by Guo and Urbonas, and shows that it is related to the naturally occurring depression storage. This finding provides a simple, transferrable, cost effective method to determine the sizing of a water quality basin.

### **1.3 Thesis Outline**

Chapter 2 will provide background and current methodology used to calculate the volume necessary to capture in a basin for water quality, exemplifying the motivation for a simple, robust and transferable method of calculation. Chapter 3 is self-contained and presented in a

format suitable for publishing in an academic journal. This chapter will outline the case study to show that lost depression storage is a reasonable estimate of the optimized WQCV derived by Guo and Urbonas. Chapter 4 provides conclusions, predicted impact of the research, and discussion for future research.



## CHAPTER 2

### Water Quality Volume

#### 2.1 Introduction

The bulk of urban pollutants are washed off and transported during the initial onset of a precipitation event referred to as the “first flush” (Lee, Bang, Ketchum Jr., Choe, & Yu, 2002). Water quality basins are designed to capture a volume of stormwater that will include the first flush. This is accomplished by capturing small precipitation events or the first portion of larger precipitation events. The overflow from the water quality basin during larger events is channeled into minor and major systems such as street gutters and storm sewers. If the first flush is captured in the water quality basin the overflow will have lower pollutant loads. Major and minor systems are used to manage peak flows and runoff volume from larger events (those ranging from 2-year to 100-year precipitation events) (Guo, Urbonas, & MacKenzie, 2014).

Determining the sizing for a water quality basin is a delicate balance. If it is too large, the cost and land requirements become a deterrent. Also, if the water quality BMP is too large the runoff will not have the required retention in the basin for plant absorption and biological breakdown of pollutants (“Bioretention Manual,” 2007). At the other end of the spectrum, if the basin is sized too small, the treatment will be limited.

This study focuses on the volume used for the sizing of a bioretention basin. A bioretention basin is a terrestrial-based process to control the quality and quantity of runoff from a storm event. Bioretention basins, or rain gardens, are shallow depressions placed in a strategic uniform distribution to create manageable subwatersheds to control surface runoff close to the source. The typical drainage area for a bioretention basin is 5 acres (“Bioretention - Minnesota Stormwater Manual,” 2015). Pollutant removal is achieved in a bioretention basin by a plethora

of processes; interception, infiltration, settling, evaporation, filtration, absorption, transpiration, evapotranspiration, assimilation, adsorption, nitrification, denitrification, volatilization, thermal attenuation, degradation, and decomposition (“Bioretention Manual,” 2007). The optimum volume of water captured in the bioretention basin was derived by Guo and Urbana and is referred to as the WQCV (Guo & Urbonas, 1996). Figure 2-1 displays the typical design of a rain garden and the WQCV. Figures 2-2 and 2-3 are pictures of rain gardens around Lake Okoboji in Iowa.

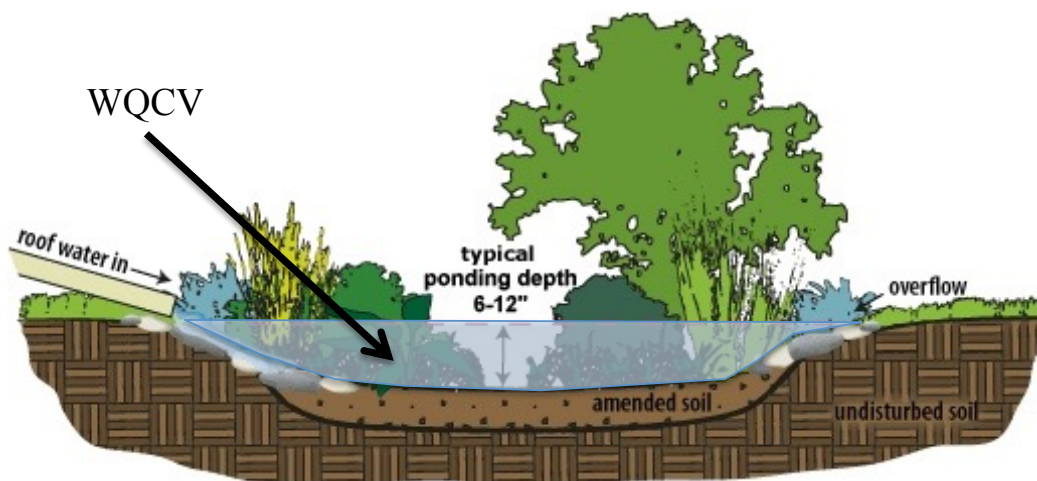


Figure 2-1: Rain Garden Diagram with the WQCV (“Rain Gardens | OSU Extension Service :: StormWater Assessment and Management,” n.d.)



Figure 2-2: Rain Garden, Okoboji IA (Photo by Mary Jean O'Hair)



Figure 2-3: Rain Garden, Okoboji IA (Photo by Mary Jean O'Hair)

Pollution removal data from a bioretention BMP varies across literature. The International Stormwater BMP Database has been collecting information on BMP performance

since 1994. The results of their analysis of pollutant removal by a bioretention BMP showed a decrease in total suspended solids (TSS) of 74%, a decrease in total nitrogen (TN) of 26%, and an increase in total phosphorus (TP) of 50%. These data were collected from 30 bioretention test sites in the Northeast/North Central, Mid-Atlantic/Upper Central and Pacific Northwest/ Pacific Central regions (Geosyntec Consultants & Wright Water Engineers, 2014). Table 2-1 shows different removal rates, some even within the same area. The variation in pollutant reduction emphasizes the importance of design parameters for a bioretention BMPs including the sizing, composition and depth of the infiltration material, as well as vegetation. The study in this thesis focuses on providing a consistent method of determining the volume of stormwater to be used for the sizing of a bioretention BMP.

Table 2-1: Bioretention Pollutant Removal

Study Area	TSS - Total Suspended Solids	TP - Total Phosphorus	Zn - Zinc	Source
University of Maryland	94%	60%	96%	(Tetra Tech Inc, 2010)
University of Maryland (based on 1" runoff capture)	47%	76%	62%	(Davis, 2007)
University of New Hampshire – Storm Water Center	99%	5%	99%	(Tetra Tech Inc, 2010)
New Hampshire Department of Environmental Services (Based on 1" runoff capture)	99%	65%	65%	(McCarthy, 2008)

## 2.2 Methods of Calculation

There is no standard method to determine the volume to capture in order to improve water quality. The idea is to capture the majority of the small precipitation events, which are considered water quality events. It is not practical to design a bioretention basin to capture all precipitation events. But what is the optimum value? The volume of the basin could be sized to capture a 2-year 1-hour storm. This storm will produce runoff larger than 95% of the runoff events in an urban catchment (Guo & Urbonas, 1996). A 90% or 94% capture rule is another method to determine sizing. This method sizes the basin to capture and treat 90% or 94% of the annual rainfall from precipitation events that produce runoff. This value is determined based on a rainfall frequency spectrum for the area. Some states, such as Minnesota and Maryland, size their water quality basins to capture an arbitrary one-inch depth of rainfall over the drainage area. A one-inch depth results in a similar sizing as the 90% capture rule for an area (Bauer, 2011). Iowa uses a 1.25” capture depth over the drainage area to size a water quality basin. This corresponds to a 90% capture volume in most areas of Iowa (“Iowa Stormwater Management Manual,” 2010).

Another option is to use the first flush concept, given that the majority of pollutants are washed off in the first half-inch of runoff. Using this method, the volume of the bioretention basin is calculated as being equivalent to a half-inch depth over the entire drainage area. A variation of this method is to consider the first flush from only the impervious area, calculated as a half-inch times the impervious area. The water quality volume can also be based on a 1-year, 24-hour storm obtained from the U.S. Weather Bureau’s TP40 reports. Minnesota Stormwater Manual Sub-Committee looked at the variations in water quality volume that each of the above scenarios would produce using the following assumptions:

- 100-acre residential area

- 35% impervious surfaces
- All impervious surfaces were new
- Twin Cities precipitation data were used
- Aggregate curve number = 72
- Impervious surfaces curve number = 98
- Pervious surfaces curve number =70
- Runoff coefficient = .30

The results of the calculations are shown in Table 2-2. The water quality volume ranges in size from 1.5 ac-ft to 6.4 ac-ft., a significant variation (Emmons & Olivier Resources & Center for Watershed Protection, 2005).

Table 2-2: Water Quality Volume Sizing Comparison (Emmons & Olivier Resources & Center for Watershed Protection, 2005)

Calculation Description	Rainfall Depth (inch)	Runoff Depth (inch)	Water Quality Volume (acre-feet)
90% of annual rainfall events	1.05		2.5
94% of annual rainfall events	1.4		3.3
1 inch times runoff coefficient	1		2.5
1-yr/24 hour storm	2.4		6.4
½ inch runoff over entire site		0.5	4.2
½ inch runoff over impervious surfaces		0.5	1.5

Research done by Guo and Urbonas in 1990 determined the point of diminishing returns in capturing precipitation events. Figure 2-4 depicts a sample of the results of the maximization of the WQCV, where “RVCR” stands for runoff volume capture ratio and “ECR” stands for event capture ratio.



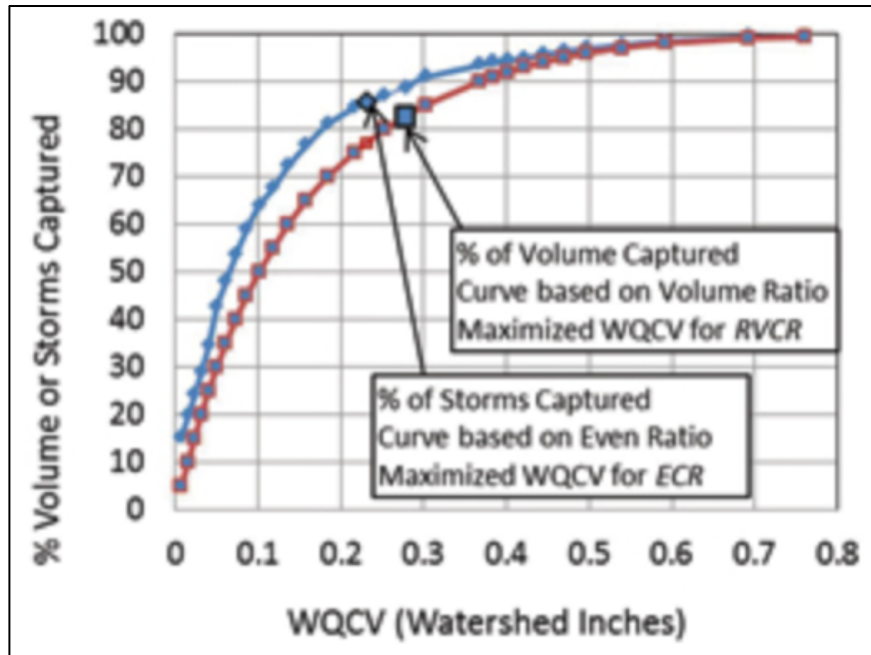


Figure 2-4: Point of diminishing return (maximized WQCV) (Urbonas et al., 2011)

This research led to the development by Guo, in 1992, of PondRisk software to automate the process to determine maximized sizing of the WQCV (Urbonas et al., 2011). WQ-COSM software was later developed based on the same principals used in PondRisk. Figure 2-5 shows the non-exceedance probability curve with the point of diminishing returns based on rain data from the Denver, Colorado, Stapleton Airport for a 50% impervious area with a 12-hour drain time, generated from WQ-COSM. The x axis in Figure 2-5 is normalized by the 99.5% rainfall volume (Guo et al., 2014). In this scenario, capturing 80% of the precipitation events reflects the optimum sizing for the WQCV.

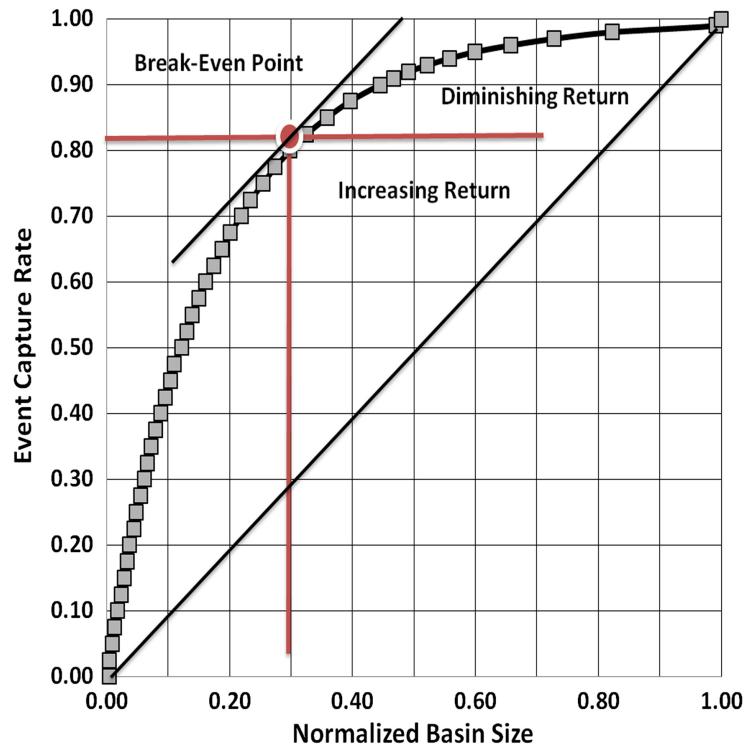


Figure 2-5: WQCV for Denver, Colorado, 50% impervious area, 12-hour drain time (Guo et al., 2014)

In 1996, Guo and Urbonas developed a relationship between the maximized WQCV to the imperviousness of an area and drain time for the BMP (Guo & Urbonas, 1996). The Urban Drainage and Flood Control District in Colorado is using a variation of this formula for the calculation of the WQCV for BMP design. This formula is described in Chapter 3 and is used for the research in this thesis.

The variation in design values for the water quality volume and complexity in determining the optimum value has motivated the need to relate this volume to the hydrology in a given area. Doing so will provide a robust and transferrable means to design water quality BMPs for LID.

### 2.3 Study – Context and Area

This study compared the runoff generated pre-development to the post-development runoff with a bioretention basin. The natural process replaced by the bioretention basin was



identified by stripping off components of the hydrological cycle until the runoff from pre- and post-development scenarios were similar. For the pre-development scenario, it was assumed that there were no impervious areas. The post-development runoff calculations were made with various levels of impervious area and a bioretention basin designed with a WQCV for the percentage of impervious area. Initially, the runoff generated factored in the amount of infiltration and depression storage, but as the percentage of impervious area increased, the pre- and post-development runoff volume diverged. This method resulted in higher runoff post-development. The runoff generated taking into account only depression storage provided similar pre- and post-development runoff volumes. This led to the conclusion that the WQCV is essentially replacing the lost depression storage due to proliferation of impervious surfaces during urbanization.

Denver, Colorado, was the study area used to relate water quality volume to the hydrologic cycle. The precipitation data for the study were obtained from NOAA at the Denver-Stapleton, Colorado, location shown in Figure 2-6. The precipitation data were taken over a 61-year period and are comprised of 1,821 individual events with a depth greater than 0.1 inches.

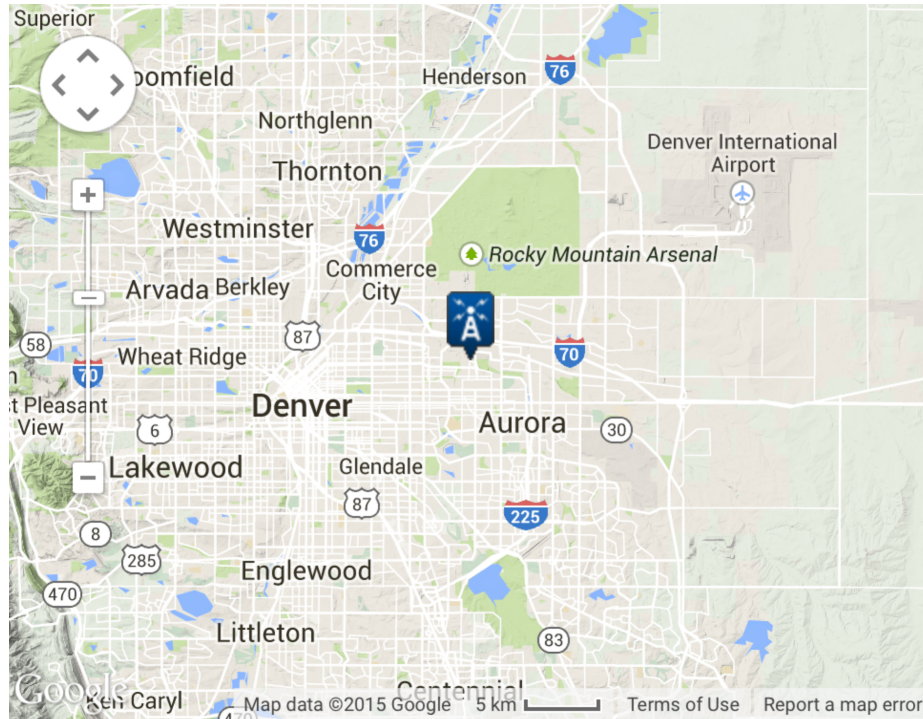


Figure 2-6: Denver-Stapleton NOAA Station Location (Google Maps)

The WQCV developed by Guo and Urbonas was used to size the bioretention basin for the study. This WQCV was derived using optimization and regression analysis in seven areas throughout the United States. It is discussed in greater detail in Chapter 3.

The identification of the physical process that is replaced by a water quality basin will provide a concrete understanding of the optimal sizing of a LID system for water quality. The study in this thesis will show that the water quality of the runoff can remain consistent with pre-development values if the change in volume of depression storage, due to the addition of impervious area, is treated with a water quality BMP. Using the lost depression storage volume as the design parameter for a LID water quality BMP provides a cost-effective and transferrable mechanism to achieve desired water quality benefits.

## CHAPTER 3

### Lost Depression Storage – A Physical Meaning for Low Impact Development Water Quality Volume

#### 3.1 Abstract

Low Impact Development (LID) is used by urban watershed managers to mitigate the increase in peak flows, volume, and pollution during a precipitation event caused by impervious surfaces (Urbonas et al., 2011). Bioretention best management practices (BMPs) are LID systems that treat pollutants in stormwater runoff. The sizing of a bioretention BMP is a critical factor in the design. Studies have been done to determine the volume to achieve optimal treatment levels of stormwater runoff; however, this value has not been related to a physical process in the hydrologic cycle. Relating the optimum water quality treatment volume to a physical process will provide tangible guidelines in the design of LID systems when the resources are not available for a detailed study. This study shows the correlation between the optimum volume of stormwater to treat for water quality and the lost depression storage. The study generated runoff depth calculated from precipitation data for the Denver-Stapleton, Colorado, area over a 61-year period. Pre-development runoff is assumed to have no impervious area and post-development runoff was calculated using multiple levels of imperviousness with a LID bioretention system optimized for water quality in place. The runoff generated in both scenarios is similar if only depression storage is considered. The bioretention LID system is essentially replacing the depression storage volume lost during development. The results of this study indicate that the optimum volume for a bioretention BMP is equal to the depression storage lost during development.

### **3.2 Introduction**

Urban development results in the replacement of pervious surfaces with impervious buildings, parking lots, driveways, and roads. By reducing the pervious area, natural infiltration and depression storage are also reduced. Infiltration and depression storage are key natural processes that reduce stormwater peak flows and runoff volume as well as aiding in pollutant removal (Perlman, 2014b). Stormwater runoff is one of the leading contributors to degradation of a watershed (Lee et al., 2002). The amount of impervious surface is a key consideration in watershed planning due to the repercussions on the surrounding habitat. Stream quality begins to decline when the amount of impervious area is between 10% to 15% (Brabec et al., 2002). Urban sprawl, the result of unplanned or sub-optimal development is increasing at an alarming rate of two million acres per year in the US (Davis & McCuen, 2005). The continued burgeoning of impervious areas combined with the impact of increased runoff and pollutant levels, illustrates the need to address these issues in urban development (Nowak & Greenfield, 2012).

There are five principal features that impact watershed hydrology: land use, vegetation, climate, geology, and soils. Of these five, only land use and vegetation can be controlled (Brabec et al., 2002). Municipalities are moving toward low impact development (LID) to control land use and the incorporation of vegetation to minimize the impact of urbanization on watershed hydrology. LID processes are used to manage the additional stormwater runoff from impervious surfaces as well as water quality (Urbonas et al., 2011). The premise of LID is to use natural processes near the source to achieve pre-development peak runoff, runoff volume and water quality (US EPA, n.d.-a). Bioretention systems, such as rain gardens, use LID methods to improve water quality and reduce runoff volume. For this type of LID system to achieve the

desired results, a volume of water is buffered and allowed to pass through a filter or infiltrate into the ground (Guo & Urbonas, 1996). This volume is referred to as a water quality volume.

The water quality volume must to be sized to allow the BMP to provide the necessary stormwater treatment to meet local standards, while not being too large so that it is cost and space prohibitive. Various methods are used to determine the water quality volume. Some methods are based on capturing a 2-year 1-hour event. This event produces runoff larger than 95% of the annual runoff events in an urban catchment (Guo & Urbonas, 1996). Other methods are based on capturing 90-95% of the volume of all annual runoff events. Some states use a 1” or 1.25” capture depth, which provides similar volumes to the 90% event capture volume (“Iowa Stormwater Management Manual,” 2010)(“Minnesota Stormwater Manual,” 2015)(Bauer, 2011).

Capturing either a percentage of events that produce runoff or a percentage of volume of runoff will have a similar impact on pollutant reduction due to the first flush concept. The first flush is the initial period of the storm where the pollutant concentration is significantly higher than later periods of the event (Lee et al., 2002). In the Colorado Front Range the recommendation is to treat the 80<sup>th</sup> percentile runoff event. Treating this volume of runoff will remove 80 to 90% of total suspended solids (TSS), while doubling this volume will increase the removal rate by a mere 1 to 2% (Guo & Urbonas, 1996). When designing a water quality BMP, the retention time must be considered as well as the volume. In a study in Washington, 65% of the suspended solid load was removed by a volume equal to the average runoff event and with a drain time of twelve hours. If the drain time is increased to forty hours, the removal rate was estimated to be 82% (Guo & Urbonas, 1996).

A detailed study of an area is the preferred method of determining the water quality volume; however, most municipalities do not have the necessary funds or expertise required for

such a study, leading to the need for a simple “rule of thumb” calculation. Guo and Urbonas (1996) created such a calculation using drain time and percent of impervious area based on hourly precipitation data in seven areas across the country (Seattle, Sacramento, Cincinnati, Boston, Phoenix, Denver, and Tampa). This study used the concept of diminishing returns where additional volume did not provide a significant increase in treatment. Equation 1 was developed from the results of the study:

$$C = 0.858I^3 - 0.780I^2 + 0.774I + 0.040 \quad (1)$$

where  $C$  is the runoff coefficient used for the rational method and  $I$  is the watershed impervious ratio. The maximized water quality capture volume or WQCV was calculated as

$$WQCV = (aC + b)P \quad (2)$$

where  $a$  and  $b$  are regression coefficients shown in Table 3-1 and  $P$  is the mean precipitation for the area in inches.

Table 3-1: Regression Coefficients (Guo & Urbonas, 1996)

<b>Brim full Drain Time (hr)</b>	<b><i>a</i></b>	<b><i>b</i></b>
12	1.096	0.010
24	1.256	0.030
48	1.457	0.063

The optimized volume for a water quality basin has been determined by Guo and Urbonas as the WQCV. This paper will provide a physical parallel for the WQCV by showing how the optimized WQCV in the Denver area is equal to the lost depression storage caused by the increase in impervious area.

### 3.3 Background

When a precipitation event begins, some rainfall will be intercepted prior to reaching the ground. The precipitation reaching the ground will first fill the depression storage in the area and the excess will then flow overland. Some of the precipitation will infiltrate into the soil and a

portion will evaporate or transpire into the atmosphere, see Figure 3-1. The overland flow or runoff is the transport mechanism for pollutants to receiving waters (Field & Tafuri, 2006). Impervious areas have a reduced amount of depression storage and allow no infiltration, thus increasing the amount of runoff. To counteract the increase in impervious areas, a LID bioretention system can be placed near the source of the runoff in order to reduce runoff volume and aid in improving water quality (US EPA, n.d.-a).

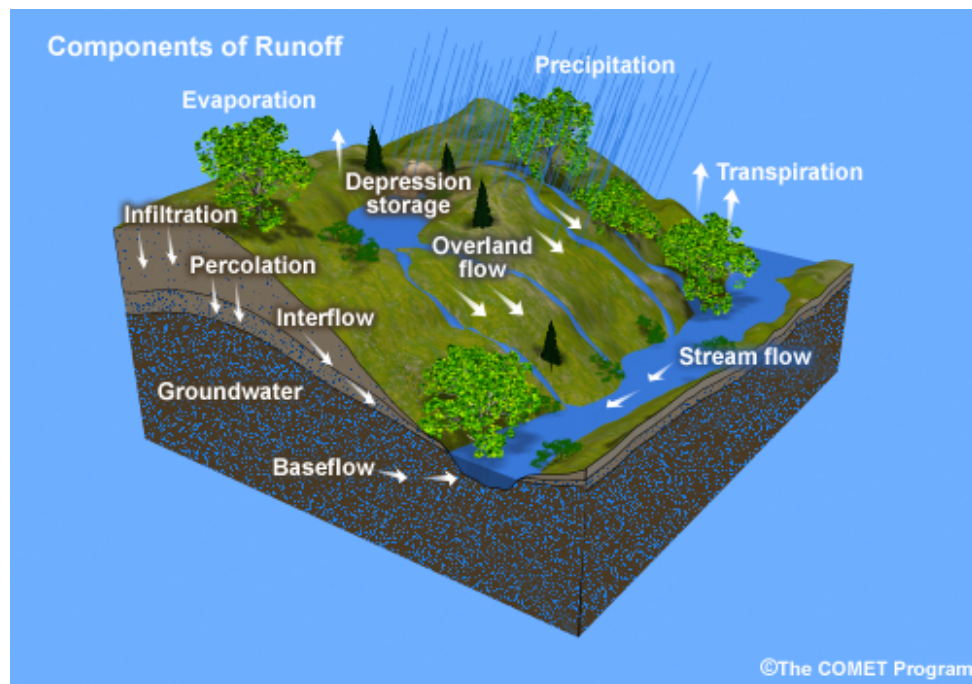


Figure 3-1: Precipitation Hydrology (The Comet Program, n.d.)

### 3.4 Methodology

In this study, the pre-development runoff depth, considering only depression storage, is compared against the post-development runoff depth with a 12-hour drain time bioretention BMP. A drain time of twelve hours is the suggested duration for a rain garden BMP (Urbonas et al., 2011). Drain time is the time required for the full storage volume in the water quality basin to completely drain. Subsurface infiltration in the basin was not factored into the runoff calculation to isolate the comparison between change in depression storage and the WQCV.

The data used for the study was obtained from rain gauges at Stapleton Airport in Denver, Colorado and consisted of 1,821 events over a 61-year period (“NOAA - The National Climatic Data Center’s (NCDC) Climate Data,” n.d.). Storms with a depth less than 0.1 inches produce virtually no runoff and do not impact the sizing of a BMP. Because of this, these storms were excluded from the precipitation data (Urbonas et al., 2011). The data used contained the storm events and resulting precipitation. These data were converted into runoff depth using a lumped-parameter runoff coefficient (Guo & Urbonas, 2014). The assumption was made that the infiltration on the depression area and the LID system was negligible during the event. At the start of each precipitation event, the depression storage and LID system were assumed to be dry. The runoff depth was calculated based on the water volume balance in which the depression storage was filled with the event precipitation and the excess running off. In the case of the post-development process, the depression storage is filled and excess precipitation flows into the water quality basin with the overflow from the basin considered runoff. Figure 3-2 depicts a schematic of the water balance used for the study. In Figure 3-2 the green line denotes pervious areas and black line denotes impervious area. The blue hatch area is the stored stormwater within the depressions and the LID system.

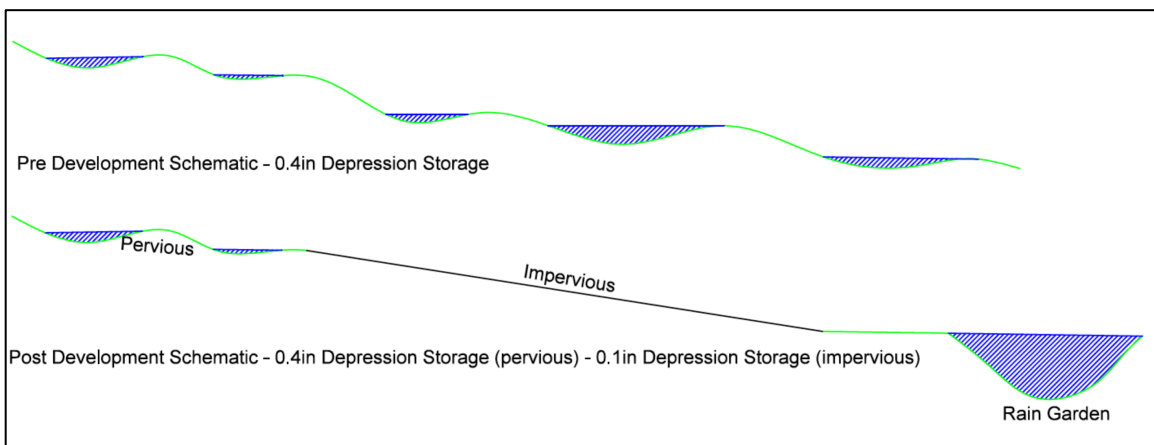


Figure 3-2: Schematic of pre- and post-development scenarios used in the study



For each of the 1,821 events, pre- and post-development runoff was calculated using the rational method. With the rational method, after the initial retention losses due to depression storage, runoff begins and it is equal to the rainfall times the runoff coefficient. The runoff coefficient was calculated using the volume-based runoff coefficient for urban catchments:

$$C = n \left[ \left( 1 - \frac{D_{vi}}{P} \right) I + m \left( 1 - \frac{D_{vp}}{P} - \frac{F}{P} \right) (1 - I) \right] \quad (3)$$

where  $C$  is the runoff coefficient,  $I$  is equal to the Impervious ratio (%/100),  $D_{vp}$  is the pervious depression storage,  $D_{vi}$  is the impervious depression storage,  $P$  is equal to the precipitation and  $F$  is the infiltration. The value of  $n$  and  $m$  are used to prevent a negative runoff coefficient corresponding to either the impervious or pervious area. The value of  $n$  is equal to one if the value in brackets, total runoff coefficient, is greater than or equal to zero. If it is less than zero, it is set to zero. The value of  $m$  is set to one if the value within the parenthesis, the pervious portion of the runoff coefficient, is greater than or equal to zero. Otherwise, it is set to zero (Guo & Urbonas, 2014). Since this study is comparing the lost depression storage to the WQCV, infiltration  $F$  is set to zero in the runoff coefficient calculation. Design values from USDCM Volume 1, Section 3.2.2 were used for the depression storage. For pre-development the design value for open fields of 0.4 inches was used for pervious depression storage. For post-development impervious depression storage, the design value of 0.1 inches was used (*Urban storm drainage criteria manual*, 2011). For the pre-development process,  $I$  was set to zero and the runoff depth was compared to post-development runoff depth with  $I$  values of 0% to 100% in increments of 10%.

Equations 1 and 2 were used to develop the  $WQCV$  for the Denver area. The  $WQCV$  for Denver is based on a precipitation volume of 0.6 inches, which captures 91.6% of storm events and 80% of the runoff producing events in the area. Capturing and treating 80% of the runoff

events will remove 80% to 90% of the annual total suspended solid load. The *WQCV* for the Denver area is calculated using the equation 4,

$$WQCV = a(0.91I^3 - 1.19I^2 + 0.78I) \quad (4)$$

where *a* is the coefficient that corresponds to the drain time of the BMP shown in Table 3-2 and *I* is the ratio of impervious area ( $\%/100$ )(*Urban Storm Drainage Criteria Manual*, 2011). In the study, the value of *a* was set to 0.8 for a 12-hour drain time.

Table 3-2: Drain Time Coefficients for WQCV Calculation (*Urban Storm Drainage Criteria Manual*, 2011)

Drain Time (hr)	<i>a</i>
12	0.8
24	0.9
48	1.0

Pre-development runoff was calculated using the rational method as

$$Rp_{pre} = CP \quad (5)$$

where *C* is derived from Equation 3, and *P* is the event precipitation in inches. *C* incorporates the depression storage of 0.4 inches for previous areas (see equation 3). Post-development runoff was also calculated using the rational method less the volume captured in the water quality basin, the *WQCV*:

$$Rp_{post} = CP - WQCV. \quad (6)$$

The *WQCV* in equation 6 was calculated using equation 4 shown above. For the post-development process, *C* incorporates 0.1 inches depression storage for impervious areas and 0.4 inches depression storage for pervious areas (see equation 3).

Two sets of summary calculations were generated. One was based on all 1,821 precipitation events and the other includes only the events that produced runoff. In the summary

calculations,  $N$  is equal to the total number of precipitation events, 1,821, for the “All Event” values. For the “Runoff Events” summary values,  $N$  is equal to the total number of events pre-development that produce runoff, 587. Mean runoff pre-development was calculated as

$$Rm_{pre} = \frac{1}{N} \sum_{i=1}^N Rp_{pre(i)}. \quad (7)$$

Post-development mean runoff was calculated as

$$Rm_{post} = \frac{1}{N} \sum_{i=1}^N Rp_{post(i)}. \quad (8)$$

The mean difference is calculated by first determining the difference in runoff for each event,

$$Rdf = Rp_{pre} - Rp_{post}, \quad (9)$$

and then summing the differences and dividing by the number of events

$$Rm_{diff} = \frac{1}{N} \sum_{i=1}^N Rdf_{(i)}. \quad (10)$$

The difference in runoff as a percent of post-development runoff is calculated by

$$Rm_{diff\%} = \frac{Rm_{diff}}{Rm_{post}}. \quad (11)$$

The root mean square error,  $RMSE$ , is calculated by

$$RMSE = \sqrt{\frac{1}{N} \sum_{i=1}^N (Rp_{post(i)} - Rp_{pre(i)})^2}. \quad (12)$$

The maximum water quality depth pre-development,  $Md_{pre}$ , is equal the design value for depression storage in the area. In this study the value of 0.4 inches is used, based on the

depression storage of an open field (*Urban Storm Drainage Criteria Manual*, 2011). The maximum water quality depth post-development is calculated as

$$Md_{post} = WQCV + D_{vi}(I) + D_{vp}(1 - I). \quad (13)$$

The difference between the pre- and post-development maximum water quality depth will make a difference when the runoff from an event exceeds the capacity of the water quality basin. The percent of events impacted by difference in capacity is

$$Di_{\%} = \frac{N_{post}}{N_{all}} \quad (14)$$

where  $N_{post}$  is equal to the total number of events that produce runoff post-development and  $N_{all}$  is equal to the total number of events in the study. The average annual impact of the deficit in terms of runoff depth is calculated as

$$Di_{ave} = \frac{\sum_{i=1}^N Rdf(i)}{Ny} \quad (15)$$

where  $Ny$  is the number of years in the study.

### 3.5 Results and Discussion

In this simplified evaluation of runoff using the design values for the Denver area and the precipitation events over a 61-year period, there is a correlation between the lost depression storage and the water quality volume. Table 3-3 shows the results of the analysis of runoff with varying levels of impervious surfaces. In the range of 40% to 80% total impervious area, the difference between the pre- and post-development runoff is less than 0.023 inches for runoff-producing events and less than 0.008 inches when all precipitation events are considered.

Table 3-3: Summary Results Pre- and Post-development Runoff

Post Development Impervious (f)		100%	90%	80%	70%	60%	50%	40%	30%	20%	10%	0%
WQCV	inch	0.400	0.321	0.263	0.220	0.189	0.165	0.144	0.121	0.093	0.054	0.000
$N_{all}$ - Total Number of Events in Study	event	1821	1821	1821	1821	1821	1821	1821	1821	1821	1821	1821
$N_{pre}$ - Pre-Development Runoff Events	event	597	597	597	597	597	597	597	597	597	597	597
$N_{post}$ - Post-Development Runoff Events	event	463	539	570	586	597	586	570	564	564	570	597
<b>All Events</b>												
$Rm_{pre}$ - Mean Runoff Pre Dev.	inch/event	0.184	0.184	0.184	0.184	0.184	0.184	0.184	0.184	0.184	0.184	0.184
$Rm_{post}$ - Mean Runoff Post Dev.	inch/event	0.154	0.168	0.176	0.181	0.181	0.179	0.176	0.174	0.173	0.176	0.184
$Rm_{diff}$ - Mean Runoff Difference	inch/event	0.030	0.016	0.007	0.003	0.003	0.005	0.008	0.010	0.010	0.008	0.000
$Rm_{diff\%}$ - Diff as % Post Dev		19.5%	9.6%	4.2%	1.8%	1.6%	2.7%	4.4%	5.7%	6.0%	4.3%	0.0%
RMSE	inch	0.054	0.028	0.013	0.006	0.005	0.009	0.014	0.018	0.018	0.013	0.000
<b>Runoff Producing Events</b>												
$Rm_{pre}$ - Mean Runoff Pre Dev.	inch/event	0.561	0.561	0.561	0.561	0.561	0.561	0.561	0.561	0.561	0.561	0.561
$Rm_{post}$ - Mean Runoff Post Dev.	inch/event	0.605	0.567	0.564	0.561	0.552	0.556	0.563	0.561	0.560	0.563	0.561
$Rm_{diff}$ - Mean runoff Difference	inch/event	0.091	0.049	0.022	0.010	0.009	0.015	0.024	0.030	0.032	0.023	0.000
$Rm_{diff\%}$ - Diff as % Post Dev		15.1%	8.7%	4.0%	1.8%	1.6%	2.7%	4.2%	5.4%	5.7%	4.1%	0.0%
RMSE	inch	0.094	0.050	0.022	0.010	0.009	0.015	0.024	0.031	0.032	0.023	0.000
Max Runoff Pre-Development	inch	4.42	4.42	4.42	4.42	4.42	4.42	4.42	4.42	4.42	4.42	4.42
Max Runoff Post-Development	inch	4.32	4.37	4.40	4.41	4.41	4.41	4.40	4.39	4.39	4.40	4.42
$Md_{pre}$ - Max WQ Depth Pre-Development (Depression storage)	inch	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40	0.40
$Md_{post}$ - Max WQ Depth Post-Development (WQCV+ Dep. Storage)	inch	0.50	0.45	0.42	0.41	0.41	0.42	0.42	0.43	0.43	0.42	0.40
$Di_{\%}$ - % of Events Impacted by Diff in WQ depth		25.4%	29.6%	31.3%	32.2%	32.8%	32.2%	31.3%	31.0%	31.0%	31.3%	32.8%
$Di_{ave}$ - Average Yearly Impact of Diff in WQ Depth	inch/yr	0.89	0.48	0.22	0.10	0.09	0.15	0.23	0.30	0.31	0.23	0.00

Figure 3-3 through 3-13 charts the comparison between the pre- and post-development sorted runoff depth for the 61-year period. These figures support the expectation that the pre- and post-development runoff depths are similar when only depression storage and WQCV are considered.

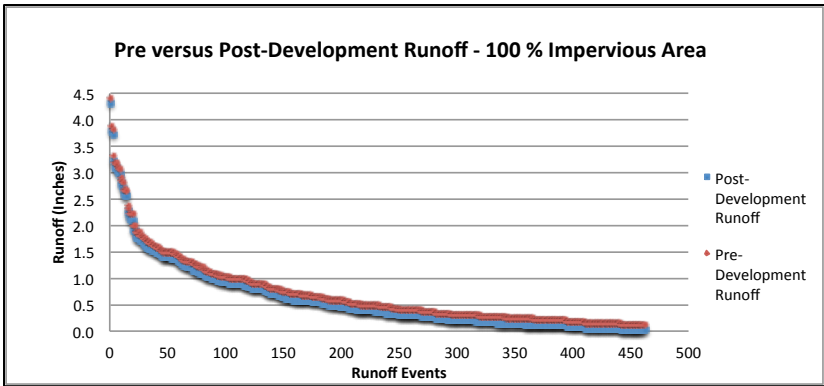


Figure 3-3: Ranked Runoff - 100% Impervious Area

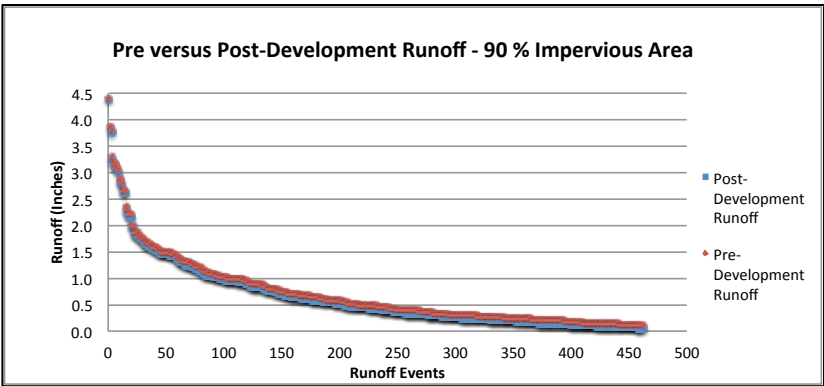


Figure 3-4: Ranked Runoff - 90% Impervious Area

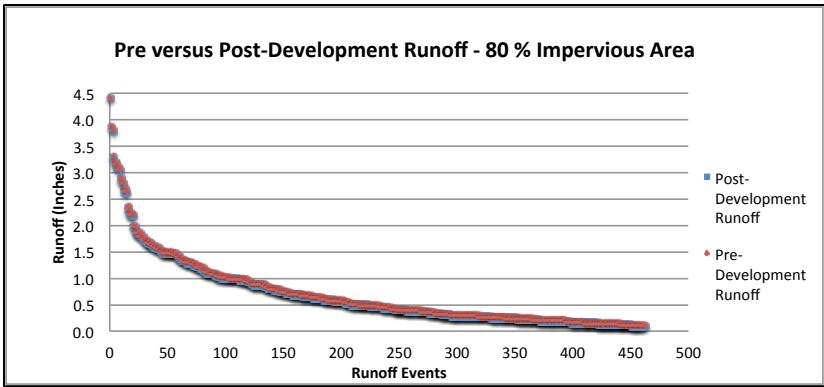


Figure 3-5: Ranked Runoff - 80% Impervious Area

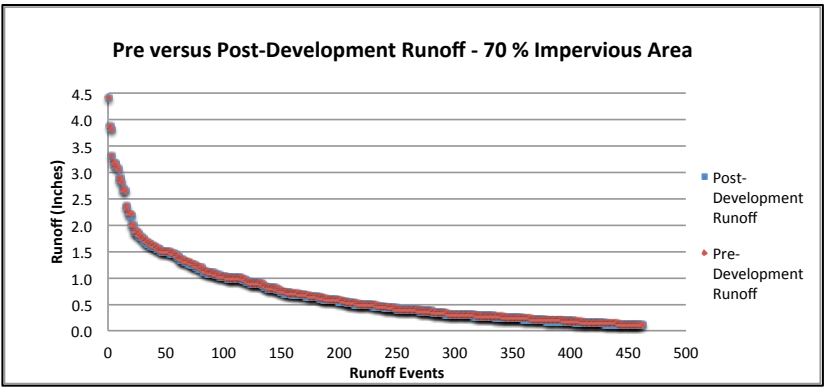


Figure 3-6: Ranked Runoff - 70% Impervious Area

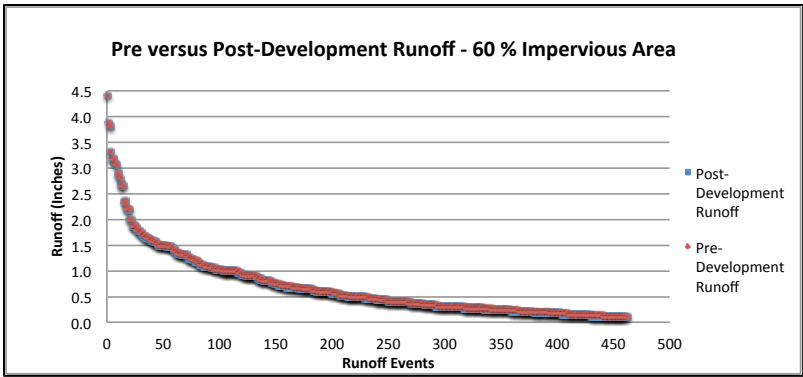


Figure 3-7: Ranked Runoff - 60% Impervious Area

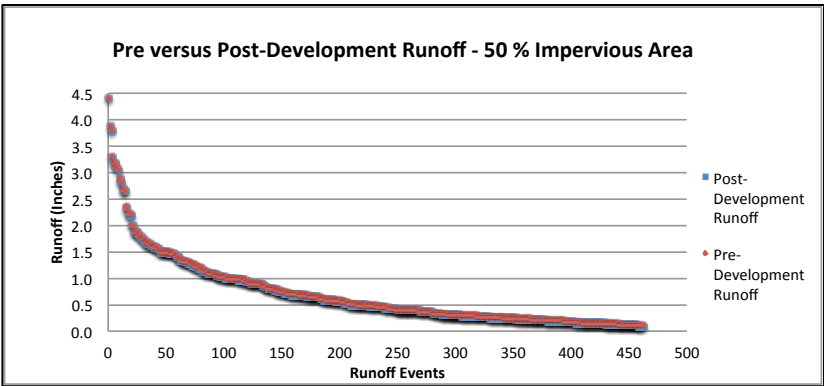


Figure 3-8: Ranked Runoff - 50% Impervious Area

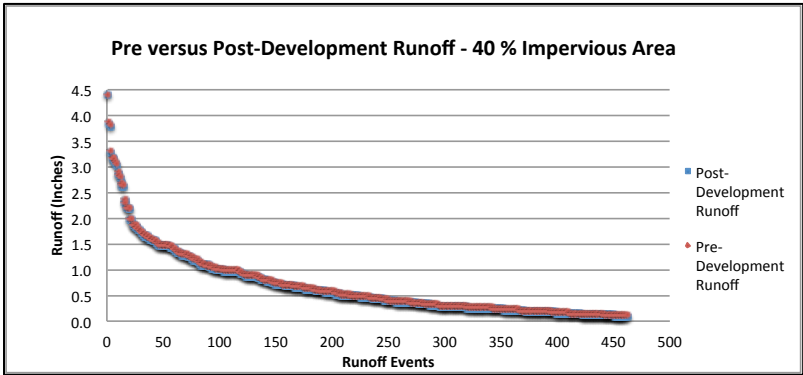


Figure 3-9: Ranked Runoff - 40% Impervious Area

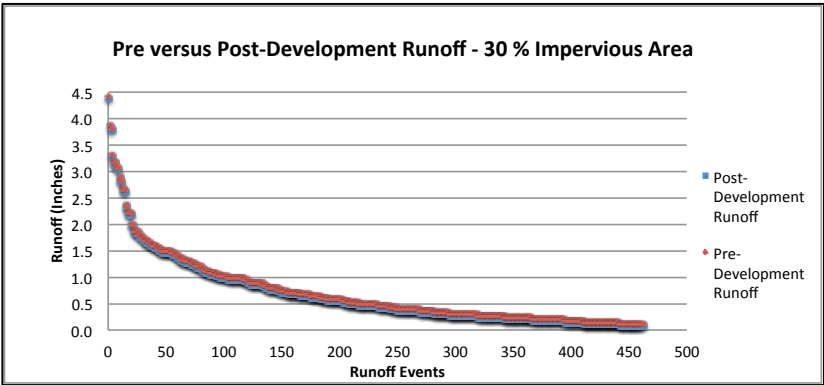


Figure 3-10: Ranked Runoff - 30% Impervious Area

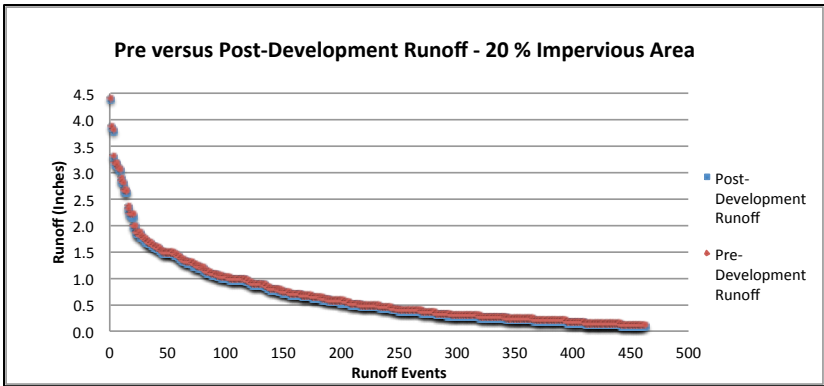


Figure 3-11: Ranked Runoff - 20% Impervious Area

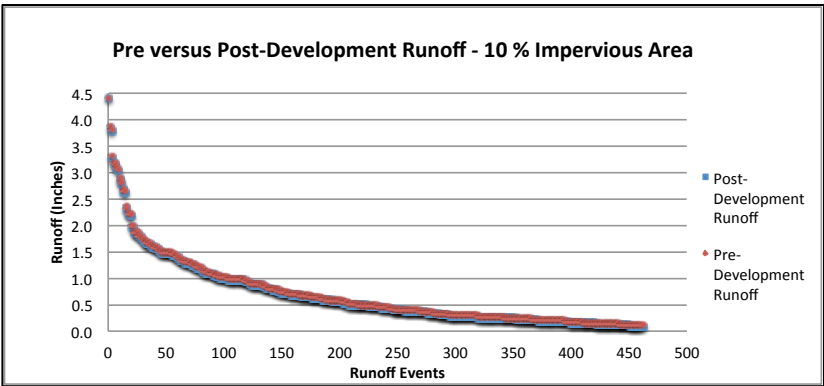


Figure 3-12: Ranked Runoff - 10% Impervious Area

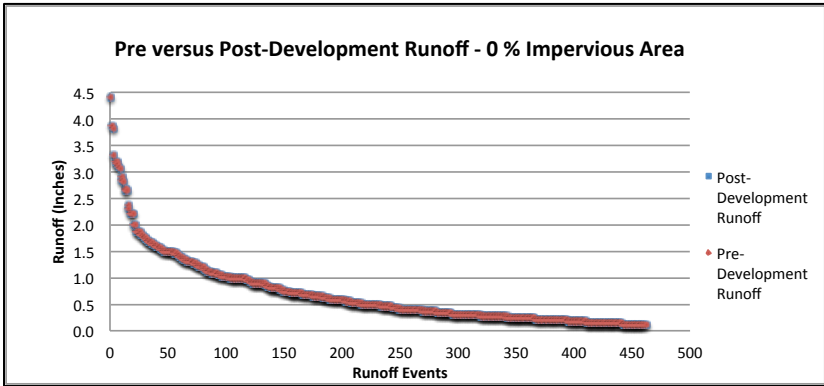


Figure 3-13: Ranked Runoff - 0% Impervious Area



Figures 3-14 and 3-15 show that between 40% and 80% impervious area, the difference in runoff depth is less than 0.01 inches and the RMSE is less than 0.015 for all events during the 61-year period. For runoff-producing events, the mean difference in the 40-80% range was less than 0.025 inches and the RMSE was less than 0.025. This error analysis implies that the lost depression storage is a reasonable estimate for the WQCV.

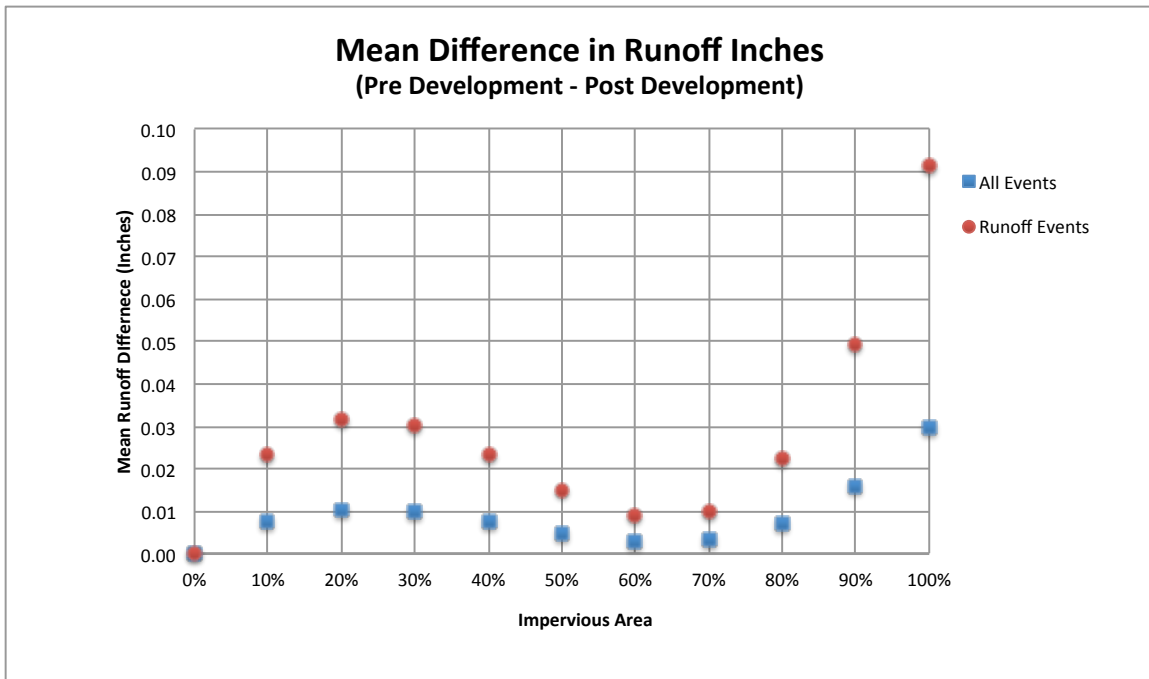


Figure 3-14: Mean Difference in Runoff Pre versus Post-development

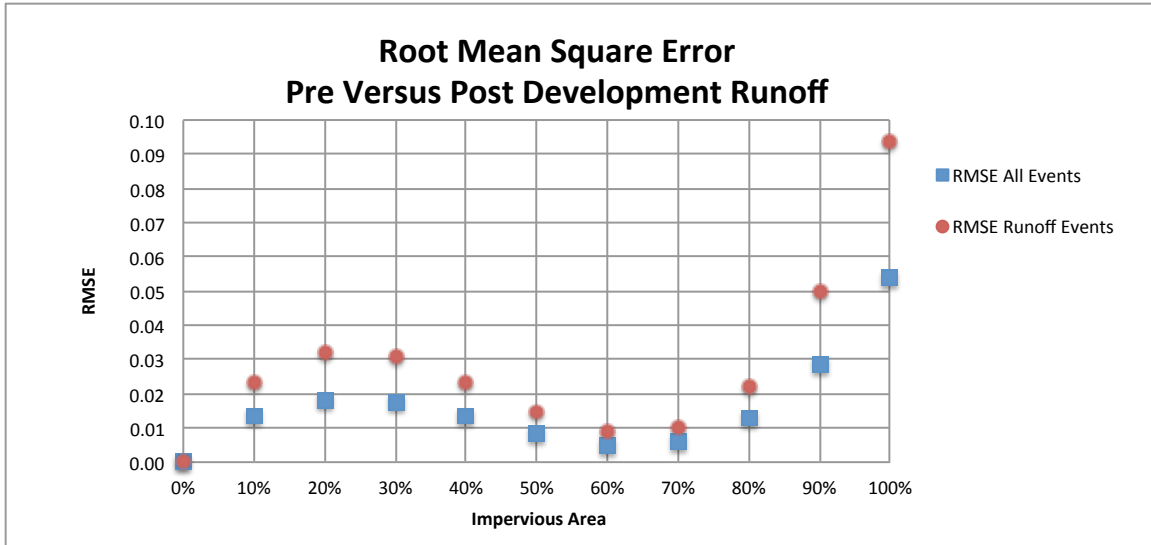


Figure 3-15: RMSE in Runoff Pre vs. Post Development

In Figure 3-16, the mean runoff difference is shown to be less than 5% of the post-development runoff when the impervious area is in the range of 40% to 80%. This amount is insubstantial when treating smaller drainage areas that are appropriate for the use of bioretention BMPs.

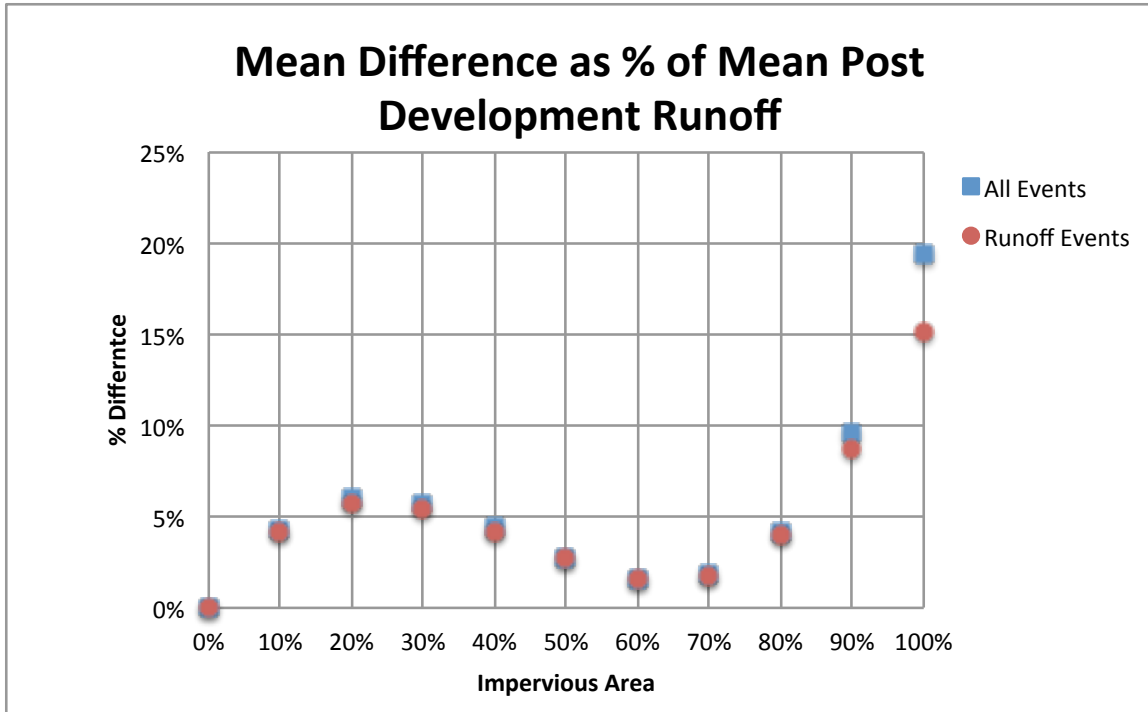


Figure 3-16: Mean Runoff as a percent of the Post-development Mean Runoff

The maximum capacity of the pre-development depression storage is less than the maximum capacity of the WQCV plus post-development depression storage. Figure 3-17 displays the difference in the maximum capacity of the water quality basin. The pre-development maximum capacity is based on the design value of 0.4 inch for an open field (*Urban Storm Drainage Criteria Manual*, 2011). The post-development maximum capacity is based on the precipitation depth captured in the water quality basin, WQCV, plus the depression storage post-development. The events impacted by the difference in capacity are the post-development runoff events. In the range of 40% to 80% impervious area, the difference in the maximum water captured, if depression storage is used to estimate the water quality volume, is less than 0.025 inches. Figure 3-18 shows the total average annual impact of the deficit.

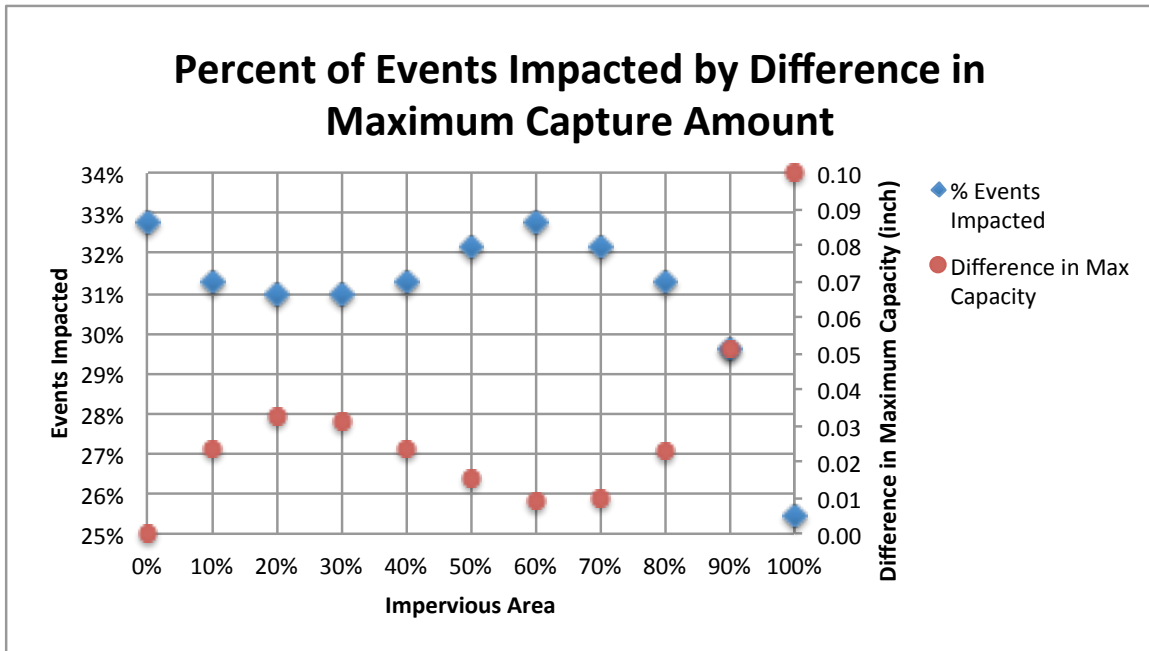


Figure 3-17: Maximum Water Quality Capacity Comparison

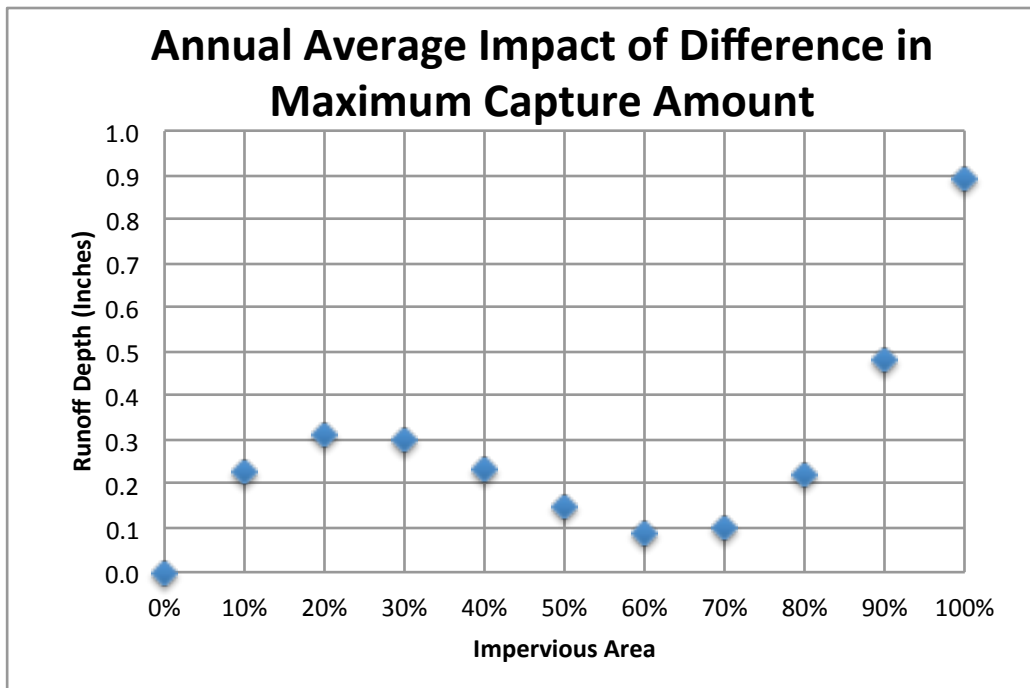


Figure 3-18: Annual Impact of Maximum Capacity Difference

The pre-development depression storage is providing the same function as the post-development water quality basin within the range of 40% to 80% impervious area. Stormwater within the depression storage or water quality basin will infiltrate, be absorbed and filtered by vegetation, or evaporate, providing treatment prior to entering the groundwater and receiving water bodies. The idea that the volume of stormwater that needs treatment by the basin is equivalent to the lost depression storage, provides a tangible basis for design of water quality BMPs when the expertise and funding is not available for a detailed study.

### **3.6 Conclusion**

Studies have been done to optimize the volume of stormwater to capture to achieve pre-development water quality. These studies have enabled the water quality volume to be related to a physical element in the hydrology cycle. This study shows the correlation between the lost depression storage and the water quality volume derived by Guo and Urbanos. The lost depression storage is a reasonable estimate for the water quality volume in the range of 40% to 80% impervious area. The relationship of the water quality volume to the lost depression storage provides a means for municipalities to effectively design a bioretention process without the cost and expertise required to do a detailed analytical study. This method of determining the water quality volume is easily transferrable to other areas. The lost depression storage can be obtained from design values for the area as used in this study or from geographic information system (GIS) analysis of the area (Chu, Zhang, Chi, & Yang, n.d.)

Future research should be done to understand the relationship of lost infiltration with the increase in impervious surfaces. The results of this research could lead to the understanding of the optimum sizing for detention BMPs used to manage peak flows.

## CHAPTER 4

### Conclusion

#### 4.1 Summary and Conclusion

LID is a practice that uses processes that mimic nature to manage stormwater. The goal of LID is for post-development runoff volume, peak flows, and water quality to match pre-development levels. In an undeveloped area, depression storage consists of indentations in the land surface where water pools and then infiltrates, evaporates, or is absorbed by vegetation. A bioretention BMP is performing the same function as the naturally occurring depression storage. Instead of runoff being stored in a large number of small depressions, one bioretention BMP is designed close to the source of the runoff to decrease pollutants.

Currently there are many different methods to determine the water quality volume of a bioretention BMP. The study in this thesis showed that the depression storage that is lost during development is a reasonable estimate of the water quality volume for a bioretention BMP. In the study, the pre-development runoff volumes, from storm events for a 61-year period in the Denver area, were compared against various levels of impervious area with a bioretention BMP. The design values from the Urban Storm Drainage Criteria Manual were used for the water quality volume of the bioretention BMP and depression storage. The study found that between 40% to 80% impervious area, the reduction in depression storage was a good estimate of the optimized water quality volume. This finding is significant for urban planners and watershed managers as it provides a simple, robust, transferrable and low cost means to design a bioretention BMP. Relating the water quality volume to the hydrological process simplifies the task of determining the necessary volume to capture and buffer for stormwater runoff volume control and water

quality to match pre-development levels. While conducting a detailed analysis of an area can be used to determine the water quality volume, this study suggests that depression storage design values for an area is a faster, easier, more cost effective method. For a more detailed estimate of the water quality volume, GIS could be used to determine the depression storage lost due to a new development project.

#### **4.2 Recommendation for Future Work**

This study was a simplified approach using historical data to associate depression storage to the water quality volume for a bioretention BMP. To provide additional validation of the concept in this thesis, a study should be done in a new development area. For this study, GIS should be used to identify the depression storage volume prior to construction. Also water quality measurements should be collected in the watershed prior to development. Post construction depression storage should again be measured with GIS. A BMP would then be designed and implemented with a water quality volume equal to the change in depression storage. The water quality measurements post-development would then be compared to pre-development values.

Additional research should also be done to understand the relationship of infiltration to LID processes. Is the change in infiltration pre-development versus post development an approximation of detention volumes needed to control runoff volumes and peak flows?

The study in this thesis provides a start to understanding the physical meaning of design parameters for low impact development best management practices. The association of the design parameters with natural processes enables watershed managers to make design decisions that are intuitive versus derived from complex algorithms. Using natural processes also provides a robust and transferable mechanism to determine BMP design parameters.



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## **Appendix A**

### **Excel Spreadsheet and Sample Calculations**

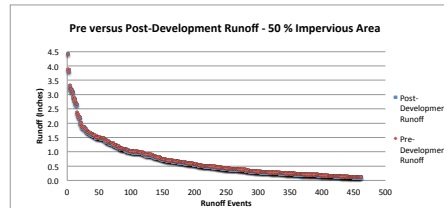
# A.1 Excel Spreadsheet – 50% Impervious Area

## Pre versus Post-Development Runoff - 50 % Impervious Area

### Summary

All Even N = 1821		Runoff Events N = 597	
Rm <sub>pre</sub> - Mean Runoff Pre-Development	0.184 inch	Rm <sub>post</sub> - Mean Runoff Post-Dev.	0.561 inch
Rm <sub>diff</sub> - Mean Runoff Post-Development	0.179 inch	Rm <sub>diff</sub> - Mean Runoff Post-Dev.	0.556 inch
Rm <sub>diff</sub> - Mean Runoff Difference	0.005 inch	Rm <sub>diff</sub> - Mean Runoff Difference	0.015 inch
Rm <sub>diff</sub> - Mean Diff. as % Post-Dev Runoff	2.7%	Rm <sub>diff</sub> - Mean Diff. as % Post-Dev. Runc	2.7%
RMSE	0.009	RMSE	0.015

Max Runoff Pre-Develop	4.42 inch	Md <sub>pre</sub> - Max WQ Depth Pre-Development (Depression storage)	0.40 inch
Max Runoff Post-Develop	4.41 inch	Md <sub>post</sub> - Max WQ Depth Post-Development (WQCV + Dep. Storage)	0.42 inch
D <sub>pre</sub> - % of Events Impacted by Diff in WQ depth	32.2%		
D <sub>post</sub> - Average Yearly Impact of Diff in WQ Depth	0.146 inch		



### Precipitation Data - Denver Stapleton Airport (NOAA)

N <sub>y</sub> - Number of years	61		
N <sub>st</sub> - Total Events in Study	1821		
(Used for All Events Summary)			
D <sub>pre</sub> - Impervious Depression Storage	0.1 inch	(USDCM)	
D <sub>post</sub> - Pervious Depression Storage	0.4 inch	(USDCM)	

### Pre-Development

T <sub>c</sub> - Time of Concentration	1.00 hr
f <sub>i</sub> - Infiltration rate	0 in/hr
N <sub>pre</sub> - Runoff Events	597
(Used for Runoff Events Summary)	
Non Runoff Events	1224

### Post-Development

T <sub>c</sub> - Time of Concentration	1.00 hr
f <sub>i</sub> - Infiltration rate	0 in/hr
N <sub>post</sub> - Runoff Events	586
(Used for Runoff Events Summary)	
Non Runoff Events	1235

f <sub>i</sub> - Impervious	0.5 %/100
D <sub>i</sub> - Drain time =	12 hr
a - used in WQCV	0.8
WQCV	0.165 inch

Drain Time (hr)	a
	12
	24
	48
	0.8
	0.9
	1

Storm ID	Event		Date		Event		Time		P (in)	D (hr)	T <sub>i</sub> (hr)	F (in)	C (in)	S <sub>d</sub> (in)	R <sub>p</sub> (in)	R <sub>p</sub> (in)	F (in)	C <sub>i</sub> (in)	C <sub>p</sub> (in)	C (in)	R <sub>p</sub> (in)	S <sub>b</sub> (in)	S <sub>d</sub> (in)	R <sub>p</sub> (in)	R <sub>pf</sub> (in)	R <sub>pf</sub> (in)
	(Year)	(Month)	(Day)	(Hour)	(Minute)	Pre-Development	Post-Development	Loss f X T <sub>c</sub>																		
1	1948	8	1	22	18	0	0.15	17	0	0.00	0.00	0.150	0.000	0.000	4.420	0.000	0.000	0.167	0.000	0.167	0.000	0.000	0.150	4.405	0.000	0.00000
2	1948	8	21	18	0	0.13	1	456	0.000	0.000	0.130	0.000	0.000	3.880	0.000	0.115	0.000	0.115	0.000	0.115	0.000	0.130	3.865	0.000	0.00000	
3	1948	9	19	18	0	0.40	3	695	0.000	0.000	0.400	0.000	0.000	3.810	0.000	0.375	0.000	0.375	0.000	0.375	0.000	0.150	3.795	0.000	0.00000	
4	1948	11	3	23	0	0.12	2	1062	0.000	0.000	0.120	0.000	0.000	3.310	0.000	0.083	0.000	0.083	0.000	0.083	0.000	0.120	3.295	0.000	0.00000	
5	1948	11	7	16	0	0.28	20	87	0.000	0.000	0.280	0.000	0.000	3.200	0.000	0.321	0.000	0.321	0.000	0.321	0.000	0.030	3.185	0.000	0.00000	
6	1948	11	20	9	0	0.18	22	285	0.000	0.000	0.180	0.000	0.000	3.180	0.000	0.222	0.000	0.222	0.000	0.222	0.000	0.180	3.165	0.000	0.00000	
7	1948	12	22	21	0	0.24	31	758	0.000	0.000	0.240	0.000	0.000	3.160	0.000	0.292	0.000	0.292	0.000	0.292	0.000	0.240	3.145	0.000	0.00000	
8	1949	1	2	15	0	0.85	49	227	0.000	0.529	0.400	0.450	3.100	0.000	0.441	0.265	0.706	0.435	0.165	0.250	0.000	0.000	3.085	0.015	0.00022	
9	1949	1	27	8	0	0.21	27	544	0.000	0.000	0.210	0.000	0.000	3.060	0.000	0.262	0.000	0.262	0.000	0.262	0.000	0.210	3.045	0.000	0.00000	
10	1949	3	5	19	0	0.25	7	872	0.000	0.000	0.250	0.000	0.000	2.900	0.000	0.300	0.000	0.300	0.000	0.300	0.000	0.250	2.885	0.000	0.00000	
11	1949	3	12	18	0	0.27	5	160	0.000	0.000	0.270	0.000	0.000	2.850	0.000	0.315	0.000	0.315	0.000	0.315	0.000	0.000	2.835	0.000	0.00000	
12	1949	3	23	21	0	0.2	8	292	0.000	0.000	0.200	0.000	0.000	2.800	0.000	0.250	0.000	0.250	0.000	0.250	0.000	0.200	2.785	0.000	0.00000	
13	1949	3	25	21	0	0.14	3	40	0.000	0.000	0.140	0.000	0.000	2.700	0.000	0.143	0.000	0.143	0.000	0.143	0.000	0.140	2.685	0.000	0.00000	
14	1949	3	28	9	0	1.3	53	57	0.000	0.692	0.400	0.900	2.660	0.000	0.462	0.348	0.808	0.889	0.165	0.250	0.000	0.000	2.645	0.015	0.00023	
15	1949	4	8	10	0	0.68	53	212	0.000	0.412	0.400	0.280	2.660	0.000	0.426	0.208	0.632	0.285	0.165	0.250	0.000	0.000	2.645	0.015	0.00023	
16	1949	4	13	11	0	0.32	28	68	0.000	0.000	0.320	0.000	0.000	2.370	0.000	0.344	0.000	0.344	0.000	0.344	0.000	0.070	2.355	0.000	0.00000	
17	1949	4	26	9	0	0.3	11	282	0.000	0.000	0.300	0.000	0.000	2.310	0.000	0.333	0.000	0.333	0.000	0.333	0.000	0.050	2.295	0.000	0.00000	
18	1949	5	5	3	0	1.75	51	199	0.000	0.771	0.400	1.350	2.230	0.000	0.471	0.386	0.857	1.335	0.165	0.250	0.000	0.000	2.215	0.015	0.00022	
19	1949	5	11	15	0	0.37	9	105	0.000	0.000	0.370	0.000	0.000	2.200	0.000	0.365	0.000	0.365	0.000	0.365	0.000	0.120	2.185	0.000	0.00000	
20	1949	5	13	7	0	0.71	14	311	0.000	0.437	0.400	0.310	2.200	0.000	0.430	0.218	0.648	0.295	0.165	0.250	0.000	0.000	2.185	0.015	0.00022	
21	1949	5	15	3	0	0.31	45	30	0.000	0.000	0.310	0.000	0.000	2.000	0.000	0.339	0.000	0.339	0.000	0.339	0.000	0.060	1.985	0.000	0.00000	
22	1949	6	2	20	0	2.27	58	404	0.000	0.824	0.400	1.870	2.000	0.000	0.478	0.412	0.890	1.855	0.165	0.250	0.000	0.000	1.985	0.015	0.00022	
23	1949	6	6	7	0	0.38	28	25	0.000	0.000	0.380	0.000	0.000	1.880	0.000	0.368	0.000	0.368	0.000	0.368	0.000	0.130	1.865	0.000	0.00000	
24	1949	6	8	15	0	0.16	8	28	0.000	0.000	0.160	0.000	0.000	1.880	0.000	0.188	0.000	0.188	0.000	0.188	0.000	0.000	1.865	0.000	0.00000	
25	1949	6	12	19	0	0.63	31	92	0.000	0.365	0.400	0.230	1.870	0.000	0.421	0.183	0.603	0.218	0.165	0.250	0.000	0.000	1.855	0.015	0.00023	
26	1949	6	23	19	0	0.6	3	233	0.000	0.353	0.400	0.200	1.860	0.000	0.417	0.167	0.563	0.165	0.165	0.250	0.000	0.000	1.845	0.015	0.00023	
27	1949	6	26	17	0	0.2	47	97	0.000	0.000	0.200	0.000	0.000	1.820	0.000	0.250	0.000	0.250	0.000	0.250	0.000	0.000	1.805	0.000	0.00000	
28	1949	7	5	14	0	0.63	77	166	0.000	0.385	0.400	0.230	1.800	0.000	0.421	0.183	0.603	0.215	0.165	0.250	0.000	0.000	1.785	0.015	0.00023	
29	1949	7	11	16	0	0.2	1	69	0.000	0.000	0.200	0.000	0.000	1.790	0.000	0.250	0.000	0.250	0.000	0.250	0.000	0.000	1.775	0.000	0.00000	
30	1949	7	13	17	0	0.12	24	48	0.000	0.000	0.120	0.000	0.000	1.750	0.000	0.083	0.000	0.083	0.000	0.083	0.000	0.120	1.735	0.000	0.00000	
31	1949	7	24	21	0	0.13	20	244	0.000	0.000	0.130	0.000	0.000	1.730	0.000	0.115	0.000	0.115	0.000	0.115	0.000	0.000	1.715	0.000	0.00000	
32	1949	7	28	19	0	0.12	9	74	0.000	0.000	0.120	0.000	0.000	1.700	0.000	0.083	0.000	0.083	0.000	0.083	0.000	0.000	1.685	0.000	0.00000	
33	1949	8	8	17	0	0.38	23	253	0.000	0.000	0.380	0.000	0.000	1.690	0.000	0.368	0.000	0.368	0.000	0.368	0.000	0.130	1.675	0.000	0.00000	
34	1949	8	24	19	0	0.28	23	363	0.000	0.000	0.280	0.000	0.000	1.670	0.000	0.321	0.000	0.321	0.000	0.321	0.000	0.030	1.655	0.000	0.00000	
35	1949	9	3	8	0	0.14	9	206	0.000	0.000	0.140	0.000	0.000	1.670	0.000	0.143	0.000	0.143	0.000	0.143	0.000	0.000	1.655	0.000	0.00000	
36	1949	9	30	10	0	0.14	5	641	0.000	0.000	0.140	0.000	0.000	1.650	0.000	0.143	0.000	0.143	0.000	0.143	0.000	0.000	1.645	0.000	0.00000	
37	1949	10	8	5	0	0.4	42	182	0.000	0.000	0.400	0.000	0.000	1.640	0.000	0.000	0.375	0.000	0.375	0.000	0.150	0.250	1.625	0.000	0.00000	

## A.2 Index of Variables

Variable Name	Description	Units
$a$	Drain time coefficient	
$C$	Volume runoff coefficient	
$C_i$	Impervious portion of runoff coefficient $C$	
$C_p$	Pervious portion of runoff coefficient $C$	
$D$	Duration	hr
$D_{i\%}$	% of events impacted by difference in WQ depth	%
$D_t$	Drain time of WQ basin	hr
$D_{vi}$	Impervious depression storage	inch
$D_{vp}$	Pervious depression storage	inch
$f$	Infiltration rate	inch/hr
$F$	Infiltration loss	inch
$I$	Impervious area	%/100
$Md_{post}$	Maximum WQ depth post-development (WQCV+ Dep. Storage)	inch
$Md_{pre}$	Maximum WQ depth pre-development (Depression storage)	inch
$N_{all}$	Total number of events in the study	event
$N_{post}$	Post-development runoff events	event
$N_{pre}$	Pre-development runoff events	event
$N_y$	Number of Years in Study	yr
$P$	Precipitation	inch
$Rdf$	Runoff difference: pre-development versus post-development	inch
$Rm_{diff}$	Mean difference of pre versus post-development runoff	inch/event
$Rm_{diff\%}$	Mean difference as % post-development runoff	%
$Rm_{post}$	Mean runoff post-development	inch/event
$Rm_{pre}$	Mean runoff pre-development	inch/event
$RMSE$	Root mean square error	inch
$R_p$	Peak runoff	inch
$S_b$	Event water quality basin storage	inch
$S_d$	Event depression storage	inch
$T_c$	Time of Concentration	hr
$T_i$	Time interval between events	hr
$WQCV$	Water quality capture volume	inch

## A.3 Calculation of General Variables

### A.3.1 $N_y$ - Number of Years

$N_y$  - Maximum year – Minimum year



### **A.3.2 $N_{all}$ – Number of Events in Study**

Count of all data rows in spreadsheet.

### **A.3.3 $D_{vi}$ – Impervious Depression Storage**

Manually entered based on impervious depression storage depth (*Urban Storm Drainage Criteria Manual*, 2011)

### **A.3.4 $D_{vp}$ – Pervious Depression Storage**

Manually entered based on an open field depression storage depth (*Urban Storm Drainage Criteria Manual*, 2011)

## **A.4 Pre-Development Sample Calculations (Storm ID - 14)**

### **A.4.1 $T_c$ – Time of Concentration**

Manually entered based on area.

### **A.4.2 $f$ – Infiltration Rate**

Manually entered based on area.

### **A.4.3 $N_{pre}$ – Number of Runoff Events Pre-Development**

Count of  $R_{p_{pre}} > 0$ in.

### **A.4.4 Non Runoff Events Pre-Development**

Count of  $R_{p_{pre}} = 0$ in.

### **A.4.5 $F$ - Infiltration Loss**

$$F = f \times \text{Min}(T_c, D)$$

$$F = 0 \frac{\text{in}}{\text{hr}} \times 1 \text{hr} = 0 \text{in}$$

### **A.4.6 $C$ – Volume Runoff Coefficient**

$$\begin{aligned}
 & \text{IF } P - D_{vp} - F > 0 \\
 & \quad C = \left[ \left( 1 - \frac{D_{vp}}{P} - \frac{F}{P} \right) \right] \\
 & \text{ELSE} \\
 & \quad C = 0 \\
 & \text{END}
 \end{aligned}$$

$$1.3in - 0.4in - 0in = 0.9in > 0in$$

$$C = \left[ \left( 1 - \frac{0.4in}{1.3in} - \frac{0in}{1.3in} \right) \right] = 0.69$$

#### A.4.7 *Sd* – Event Depression Storage

*Minimum of P or D<sub>vp</sub>*

$$P = 1.3in, D_{vp} = 0.4in \text{ therefore } Sd = 0.4in$$

#### A.4.8 *Rp* – Peak Runoff

$$Rp = C \times P$$

$$Rp = 0.69 \times 1.3in = 0.90in$$

### A.5 Post-Development Sample Calculations (50% impervious area, Storm ID - 14)

#### A.5.1 *Tc* – Time of Concentration

Manually entered based on area.

#### A.5.2 *f*– Infiltration Rate

Manually entered based on area.

#### A.5.3 *N<sub>post</sub>* – Number of Runoff Events Post-Development

Count of  $Rp_{post} > 0in$ .

#### A.5.4 Non Runoff Events Post-Development

Count of  $Rp_{post} = 0in$ .

### A.5.5 $a$ – Drain Time Coefficient

Calculated based in  $Dt$  entered in spreadsheet and corresponding entry in Drain time table.

### A.5.6 $I$ – Impervious Area

Manually entered

### A.5.7 $Dt$ – Drain Time of Water Quality Basin

Manually entered based on BMP design parameters.

### A.5.8 $WQCV$ – Water Quality Capture Volume

$$WQCV = a(0.91I^3 - 1.19I^2 + 0.78I)$$

$a=0.8$  for 12 hour drain time.

$$WQCV = 0.8(0.91(0.5)^3 - 1.19(0.5)^2 + 0.78(0.5)) = 0.165in$$

### A.5.9 $F$ - Infiltration Loss

$$F = f \times \text{Min}(Tc, D)$$

$$F = 0 \frac{in}{hr} \times 1hr = 0in$$

### A.5.10 $Ci$ - Impervious Volume Runoff Coefficient

$$C = n \left[ \left(1 - \frac{D_{vi}}{P}\right) I + m \left(1 - \frac{D_{vp}}{P} - \frac{F}{P}\right) (1 - I) \right] \quad (3)$$

$$Ci = \left[ \left(1 - \frac{D_{vi}}{P}\right) I \right]$$

$$Ci = \left[ \left(1 - \frac{0.1in}{1.3in}\right) 0.5 \right] = 0.462$$

### A.5.11 $C_p$ - Pervious Volume Runoff Coefficient

$$C = n \left[ \left( 1 - \frac{D_{vi}}{P} \right) I + m \left( 1 - \frac{D_{vp}}{P} - \frac{F}{P} \right) (1 - I) \right] \quad (3)$$

$$\begin{aligned} & \text{IF} \left( \left( 1 - \frac{D_{vp}}{P} - \frac{F}{P} \right) (1 - I) \right) > 0 \\ & \quad C_p = \left[ \left( 1 - \frac{D_{vp}}{P} - \frac{F}{P} \right) (1 - I) \right] \quad (m=1) \end{aligned}$$

$$\begin{aligned} & \text{ELSE} \\ & \quad C_p = 0 \quad (m=0) \\ & \text{END} \end{aligned}$$

$$C_p = \left( 1 - \frac{0.4in}{1.3in} - \frac{0in}{1.3in} \right) (1 - 0.5) = 0.346$$

### A.5.12 $C$ – Volume Runoff Coefficient

$$C = n \left[ \left( 1 - \frac{D_{vi}}{P} \right) I + m \left( 1 - \frac{D_{vp}}{P} - \frac{F}{P} \right) (1 - I) \right] \quad (3)$$

$$C = n(C_i + C_p)$$

$$\begin{aligned} & \text{IF} (C_i + C_p) > 0 \\ & \quad C = C_i + C_p \quad (n=1) \\ & \text{ELSE} \\ & \quad C = 0 \quad (n=0) \\ & \text{END} \end{aligned}$$

$$C = 0.461 + 0.346 = 0.808$$

### A.5.13 $R_p$ – Peak Runoff

$$R_p = C \times P - WQCV$$

$$R_p = 0.808 \times 1.3in - 0.165in = 0.885in$$

#### A.5.14 *Sb* – Event WQ Basin Storage

$$Sb = \text{Min} ( WQCV , CxP ) = \text{Min} ( 0.165in , 1.049in ) = 0.165in$$

The value in the spreadsheet is highlighted in red if the WQ Basin is full.

#### A.5.15 *Sd* – Event Depression Storage

$$Sd = \text{Min}(P, D_{vi} \times I + D_{vp}(1 - I))$$

$$Sd = \text{Min}(1.3in, 0.1in \times 0.5 + 0.4in(1 - 0.5))$$

$$Sd = \text{Min}(1.3in, 0.25in) = 0.25in$$

### A.6 Summary Sample Calculations (50% Impervious Area)

#### A.6.1 *Rm* – Mean Runoff

Pre-development – All Events

$$Rm_{pre} = \frac{1}{N_{all}} \sum_{i=1}^{N_{all}} Rp_{pre(i)}$$

$$Rm_{pre} = \frac{1}{1821 \text{ events}} (334.72in) = 0.184 \text{ in/event}$$

Post-development - All Events

$$Rm_{post} = \frac{1}{N_{all}} \sum_{i=1}^{N_{all}} Rp_{post(i)}$$

$$Rm_{post} = \frac{1}{1821 \text{ events}} (325.82in) = 0.179 \text{ in/event}$$

Pre-development – Runoff Events

$$Rm_{pre} = \frac{1}{N_{pre}} \sum_{i=1}^{N_{pre}} Rp_{pre(i)}$$

$$Rm_{pre} = \frac{1}{597 \text{ events}} (334.72in) = 0.561 \text{ in/event}$$

Post-development - Runoff Events

$$Rm_{post} = \frac{1}{N_{post}} \sum_{i=1}^{N_{post}} Rp_{post(i)}$$

$$Rm_{post} = \frac{1}{586 \text{ events}} (325.82in) = 0.556 \text{ in/event}$$

### A.6.2 $Rm_{diff}$ – Mean Runoff Difference

$$Rdf = Rp_{pre} - Rp_{post}$$

All Events

$$Rm_{diff} = \frac{1}{N_{all}} \sum_{i=1}^{N_{all}} Rdf(i)$$

$$Rm_{diff} = \frac{1}{1821 \text{ events}} 8.90in = 0.0049 \text{ in/event}$$

Runoff Events

$$Rm_{diff} = \frac{1}{N_{pre}} \sum_{i=1}^{N_{all}} Rdf(i)$$

$$Rm_{diff} = \frac{1}{597 \text{ event}} 8.90in = 0.015 \text{ in/event}$$

### A.6.3 $Rm_{diff\%}$ – Mean Diff as % of Post-Development Runoff

$$Rm_{diff\%} = \frac{Rm_{diff}}{Rm_{post}} \times 100$$

All Events

$$Rm_{diff\%} = \frac{0.0049in}{0.179in} \times 100 = 2.7\%$$

Runoff Events

$$Rm_{diff\%} = \frac{0.015in}{0.556in} \times 100 = 2.7\%$$

#### A.6.4 RMSE – Root Mean Square Error

All Events

$$RMSE = \sqrt{\frac{1}{N_{all}} \sum_{i=1}^{i=N_{all}} (Rp_{post(i)} - Rp_{pre(i)})^2}$$

$$RMSE = \sqrt{\frac{1}{1821} 0.133in^2} = 0.009in$$

Runoff Events

$$RMSE = \sqrt{\frac{1}{N_{pre}} \sum_{i=1}^{i=N_{all}} (Rp_{post(i)} - Rp_{pre(i)})^2}$$

$$RMSE = \sqrt{\frac{1}{597} 0.133in^2} = 0.015in$$

#### A.6.5 $Md_{pre}$ – Maximum Water Quality Depth Pre-Development

$$Md_{pre} = 0.4in$$

Depression storage of an open field (*Urban storm drainage criteria manual*, 2011).

### A.6.6 $Md_{post}$ – Maximum Water Quality Depth Post-Development

(WQCV +Depression Storage)

$$Md_{post} = WQCV + D_{vi}(I) + D_{vp}(1 - I)$$

$$Md_{post} = 0.165in + 0.1in(0.5) + 0.4in(1 - 0.5) = 0.415in$$

### A.6.7 $Di_{\%}$ - Percent of Events Impacted by Difference in Water Quality Depth

$$Di_{\%} = \frac{N_{post}}{N_{all}}$$

$$Di_{\%} = \frac{586 \text{ events}}{1821 \text{ events}} \times 100 = 32.2\%$$

### A.6.8 $Di_{ave}$ - Average Yearly Impact of Difference in Water Quality Depth

$$Di_{ave} = \frac{\sum_{i=1}^{N_{all}} Rdf_{(i)}}{\text{Total number of years}}$$

$$Di_{ave} = \frac{8.9in}{61yr} = 0.146 \text{ in/yr}$$