THESIS

LOW IMPACT DEVELOPMENT MODELING TO MANAGE URBAN STORM WATER RUNOFF AND RESTORE PREDEVELOPMENT SITE HYDROLOGY

Submitted by

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WE HEREBY RECOMMEND THAT THE THESIS PREPARED UNDER OUR SUPERVISION BY MATTHEW G. SIMPSON ENTITLED "LOW IMPACT DEVELOPMENT MODELING TO MANAGE URBAN STORM WATER RUNOFF AND RESTORE PREDEVELOPMENT SITE HYDROLOGY" BE ACCEPTED AS FULFILLING IN PART REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE.

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ABSTRACT OF THESIS

LOW IMPACT DEVELOPMENT MODELING TO MANAGE URBAN STORM WATER RUNOFF AND RESTORE PREDEVELOPMENT SITE HYDROLOGY

The hydrologic effects of urban development have been documented for some time. Urban streams experience dramatic changes to their natural flow regime, which is mostly due to the increased rate and volume of runoff. Conventional stormwater management focuses on peak rate control through the use of detention and retention basins while paying little attention to the increased volume of urban runoff. Low Impact Development (LID) is a land planning and stormwater management approach that seeks to control runoff as close as possible to its source. LID practices take advantage of natural processes, such as infiltration, to reduce the rate and volume of runoff while improving water quality at the same time. It is hypothesized that LID can be used to restore the predevelopment hydrology to a site. This thesis investigates if LID can be used exclusively to meet stormwater requirements and secondly whether LID can maintain the predevelopment site hydrology.

In order to examine if LID can restore predevelopment site hydrology, an EPA SWMM model was created based upon a proposed development in Fort Collins, CO. Several different scenarios were evaluated including: rainfall from Fort Collins, CO and Atlanta, GA; a high and a low infiltration soil; and BMPs with partial infiltration (with underdrain) and with full infiltration (without underdrain). The amount of LID in each model was increased until predevelopment peak flow rates and water balance were met; this was accomplished using design storm simulations. Each model was then analyzed with a continuous simulation using historic rainfall data from both locations. The LID BMPs that were modeled include grassed swales, rain gardens, infiltration trenches, and permeable pavement. Finally, a cost review of the LID designs was performed to explore the financial practicality of LID.

The results show that LID can restore predevelopment site hydrology, but the amount of LID required is substantial. However, the cost review shows that the extra LID expense could be recovered in certain locations through development of the detention pond land which is no longer needed.

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Above all, I want to praise my God and creator, Jesus Christ, who has given me this most wonderful planet as just a small demonstration of His unbelievable love for us. This Thesis is my contribution to helping us be a better 'gardener' of the planet He gave us. - *Soli Deo Gloria*

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1.0 INTRODUCTION

The hydrologic effects of urban development on a watershed have been well documented (Roesner & Bledsoe, 2003). Urban streams experience dramatic changes to their natural flow regime, including major increases in the volume, magnitude, frequency, and duration of stormwater flows, as well as the subsequent loss of base flow. Historically, conventional stormwater management has focused on peak rate control through the use of detention basins. Since about 1990, an emphasis on water quality has resulted in the use of features called Best Management Practices (BMPs) in addition to flood control facilities. Traditional BMPs include extended detention basins, retention basins, and constructed wetlands. These facilities are centralized “end-of-pipe” type facilities that generally require a large amount of space.

Low Impact Development (LID) is a land planning and stormwater management approach that seeks to control runoff as close as possible to its source. This is accomplished through the use of LID BMPs, which are smaller than end-of-pipe facilities and are distributed throughout the site. These LID BMPs utilize natural processes to reduce the rate and volume of runoff while improving water quality at the same time. The goal of LID is to restore the predevelopment hydrology to the site (Prince George's County, 2000a). The purposes of this thesis are first to demonstrate if LID drainage practices can be used exclusively to meet stormwater requirements, second to maintain the predevelopment site hydrology, and third to analyze the cost effectiveness of LID practices.

Many LID ideas are not really new to stormwater management. “French drains” have been used for at least 150 years (Bier, 2000), permeable pavement has been around for hundreds of years in the form of cobblestone roads, and natural surface drainage channels have been used for hundreds of years as well. However LID, as we know it today was pioneered by Prince George's County, Maryland in the 1980's and 1990's (DOD, 2004).

LID is important to study because it addresses the causes rather than the symptoms of urban stormwater problems. Conventional stormwater management seeks mainly to reduce peak flow rates but makes no major effort to reduce the increased volume of runoff associated with urbanization. As a result, large detention basins are required to store the excess runoff volume and these basins consume a relatively large area of valuable land. In addition, conventional stormwater practices struggle to treat dissolved pollutants and bacterial constituents in the stormwater (Clary, et al., 2008; WEF & ASCE, 1998). Last, conventional practices are not able to fully restore the natural flow regime to urban streams, which can result in severe erosion of stream channels (Roesner, Bledsoe, & Brashear, 2001) and the subsequent loss of healthy aquatic habitat and species (Poff, et al., 1997).

On the other hand, LID stormwater management practices are capable of reducing the volume of urban runoff and thus have the potential to greatly reduce the size and cost of stormwater basins and conveyance systems (EPA, 2007). Additionally where conventional stormwater BMP's struggle to address water quality issues, LID BMPs are robust. The filtration and infiltration of stormwater provided by many LID BMP's have been shown to reduce dissolved pollutants, nutrients (Davis, Hunt, Traver, & Clar, 2009), and bacteria (Clary, Jones, & Urbonas, 2009). Moreover, the total volume reduction of stormwater - through the process of infiltration - is crucial to maintaining the natural flow regime of urban streams upon which the health of aquatic species depends.

A site plan based on a proposed high density development in Fort Collins, CO was analyzed to determine the hydrological effects of LID. This proposed development made use of

LID practices to a moderate extent, and this thesis built on them to maximize the use of LID. The LID BMP's that were employed include rain gardens, permeable pavement, infiltration trenches, and grass swales. The EPA Storm Water Management Model (SWMM) software was used to quantify the hydrologic effects of LID. The EPA SWMM program is a dynamic rainfall-runoff model capable of unsteady gradually-varied flow routing. The LID BMP's were sized in SWMM using design storm simulations; then the model was analyzed with a continuous simulation using historic rainfall data. Rainfall patterns from two different locations were used: Fort Collins, CO and Atlanta, GA.

2.0 LITERATURE REVIEW

2.1 Effects of Urbanization

A direct consequence of urban development is that rainfall which was previously captured by the land now falls on impervious surfaces and is converted into surface runoff (Roesner, et al., 2001). Before a watershed is developed, pervious fallow land is able to store, infiltrate, and evapotranspirate a majority of the rainfall. After development, much of this pervious land is covered over with impervious surfaces that the rainwater cannot pass through, such as roads, sidewalks, parking lots, driveways, and rooftops. In addition, much of this impervious area is directly connected to a drainage system and the rainwater has no chance to infiltrate by passing over pervious ground; this is called Directly Connected Impervious Area (DCIA). The typical storm drainage system is designed to convey runoff quickly and efficiently to the receiving stream or water body. Combined, these result in larger, more frequent peak flows with more total volume (Jones, et al., 2005; Roesner, et al., 2001; WEF & ASCE, 1998). Furthermore, since a large portion of the impervious area is directly connected to the drainage system, biological processes are also largely removed; as a result, pollutants are also conveyed quickly and untreated to the receiving water bodies.

2.2 The Natural Flow Regime

The natural flow regime is a concept used to describe the dynamic hydrologic state of a stream or river in undeveloped, natural conditions. Poff (1997) presents five components of the flow regime that are thought to be the most ecologically important: magnitude, frequency, duration, timing (predictability), and rate of change (flashiness). These components are important because they describe the movement of water and sediment, which in large part determine the physical and environmental conditions that make up riverine ecosystems and habitat. Natural and human caused hydrologic changes both can disrupt the dynamic equilibrium between the water and sediment flow (Dunne & Leopold, 1978). The most healthy aquatic and riparian habitat will be found in streams with natural flow regimes (Poff, et al., 1997).

Urbanization significantly affects the natural flow regime by reducing infiltration, increasing runoff, and cutting off the overland sediment supply. By considerably reducing the amount of infiltration, urbanization reduces ground water recharge and lowers the base flow of streams (Leopold, 1968). Uncontrolled urban runoff causes streams to be very flashy with larger and more frequent peak flows. Development with traditional stormwater controls reduce peak rates to historic rates, but the duration of mid to low flows are severely extended (Roesner & Bledsoe, 2003; Rohrer, 2004). In addition, paved roads, rooftops, and grassed lawns cut off nearly all of the sediment supply. Combined, these result in a significant increase in the erosive capacity of urbanized streams during storm events.

2.3 Traditional Stormwater Management

2.3.1 Peak Rate Control

Traditional stormwater management typically addresses the increased peak flow rates from urban development through the use of large storm water detention basins or ponds. These detention ponds store the excess stormwater and release it through a designed orifice or weir so that the peak outflow rate does not exceed the historic flow rate for certain return interval storms. This practice, known as peak-shaving, was developed for the purpose of decreasing downstream flooding. Typical controlled release rates are the historical 10- and 100-year flow rates; more recently, 2-yr control has been added in an attempt to reduce channel erosion in receiving streams. Figure 2.3-1 demonstrates on a hydrograph, how peak shaving reduces the peak flow rate and extends the duration of mid to low flows.

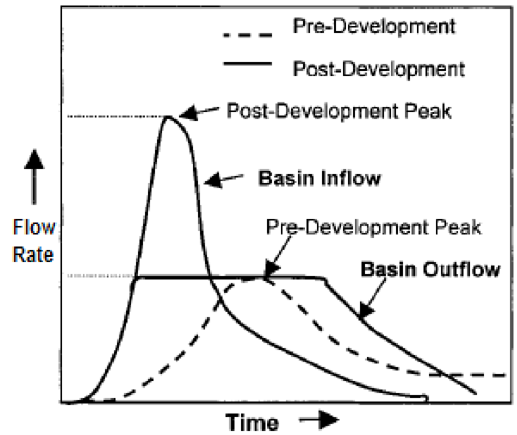


Figure 2.3-1 Peak Shaving Effect on a Hydrograph
Adapted from (Roesner, et al., 2001)

2.3.2 Water Quality

The Clean Water Act of 1987 requires municipal entities to obtain discharge permits for separate stormwater sewer systems (not combined with sanitary sewer) through the National Pollutant Discharge Elimination System (NPDES). The NPDES requires municipal entities to implement a stormwater program using ‘Best Management Practices’ (BMPs) that are designed to remove pollutants from urban runoff to the ‘maximum extent possible’ (EPA, 2005). BMPs are both engineered structural devices and procedural practices or methods that are intended to prevent, treat, or reduce pollution in urban runoff (Novotny, 2003).

Traditional stormwater management usually addresses water quality with an extended detention or retention BMP, which detains stormwater for a specified amount of time (12 – 40 hours) in order to settle suspended pollutants. These BMPs are typically located inside of larger flood control structures; the goal is to capture and treat the ‘first flush’ of runoff which contains higher amounts of pollution. The first flush is quantified with the Water Quality Capture Volume (WQCV) which is generally between 0.2 inches and 0.5 inches of runoff based on percent imperviousness and regional precipitation patterns (UDFCD, 2007). In this study, the WQCV will be determined with the Denver Urban Drainage and Flood Control District method (referred to as UDFCD hereafter).

2.3.3 Prolonged Flow Duration

While traditional stormwater management controls are generally straightforward to design, construct, and maintain, they fail to adequately address the increased volume of urban runoff. As seen in Figure 2.3-1, peak shaving practices reduce the post development peak flow rate by increasing the duration of mid to low flows.

Research has shown that these increased flow durations can do as much or more damage to streams than the pollution in urban runoff (Baker, et al., 2008; Roesner & Bledsoe, 2003; Roesner, et al., 2001; Rohrer & Roesner, 2006). For example, if the controlled 2-year release rate of a detention basin creates sheer stress that is higher than the critical sheer stress in the receiving stream, the stream is exposed to erosive flows for a longer duration of time (Roesner, et al., 2001). In fact, Rohrer (2004) found that the majority of sediment in a sand bed stream was transported by flow rates less than the 2-year flow rate, and when stormwater controls were applied, they actually increased the sediment transport for flows less than the 1-year return interval. These erosive flows degrade all intended uses and ecological functions of the stream; in particular they destroy the habitat for benthic macro-invertebrates, which live on the stream bottom and are an important part of the aquatic food chain (Thorp & Covich, 2001).

To deal with these stormwater and stream degradation issues, some communities have considered imposing impervious area limits on new development (Jones, et al., 2005). While a significant increase in watershed imperviousness does lead to stream erosion and subsequent loss of ecological functionality, as previously discussed and also as documented in literature (Schueler, 1994; 2009), overarching limits on imperviousness are misguided and fail to address the cause of urban stormwater problems: the increased volume of runoff. Enforcing percent impervious limits on new development does not reduce imperviousness, instead it effectively spreads out development, accelerates urban sprawl, and increases transportation related imperviousness (which is generally DCIA).

Rohrer (2005) found that if the critical portion of the post-development shear stress duration curve is matched with the predevelopment shear stress duration curve, the geomorphic stability of receiving streams can be maintained. This required peak shaving of the 1- and 10-year storms with a 40-hour extended detention BMP. However it is not possible to match the lowest flows portion of the predevelopment flow duration curve with detention.

As an alternative, stormwater runoff could be effectively controlled by clustering development into areas of medium and high density (Schueler, 1994) - which reduces transportation related imperviousness - while employing properly designed ‘treatment-train’ and volume reducing management practices such as infiltration and LID (Baker, et al., 2008; Jones, et al., 2005; Nehrke & Roesner, 2002).

2.4 Low-Impact Development

2.4.1 How Does LID Work?

LID is based on controlling runoff at the source by distributing LID BMPs throughout the site. LID takes full advantage of the practice of disconnecting impervious areas (called Non-Directly Connected Impervious Areas or NDCIA) by using pervious areas such as lawns, grassed street-strips, planter boxes, and ‘pocket-parks’ to store, infiltrate, evapotranspirate, filter, and slow runoff; this is called a ‘hydrologically functional landscape’ (Prince George's County, 2000a). In contrast to traditional stormwater management, LID addresses the cause of urban stormwater problems by dealing with the excess runoff volume near the source of runoff, through infiltration. The goal of LID is to restore the site's predevelopment hydrologic regime (Prince George's County, 2000a), however to date no studies could be found that have shown this. Table 2.4-1 presents commonly used LID BMPs, but by no means is an exclusive list.

Table 2.4-1 Common LID BMPs

Bioretention/ Rain Gardens
Disconnection of Impervious Areas
Dry Wells
Filter Strips
Grassed Swales/ Bioretention Swales
Infiltration Trenches
Permeable Pavement
Rain Barrels & Cisterns
Reducing Impervious Areas
Soil Amendments
Tree Box Filters
Vegetated Buffers
Vegetated Roofs

Source: (Prince George's County, 2000a)

LID takes advantage of nature's ability to process nutrients and pollutants, by routing runoff over as much pervious area as possible. Pollutants are strained out as runoff flows across filter strips and through grass lined swales. Vegetation is very good at removing nutrients, heavy metals, and degrading coliform bacteria (Coffman, 2009; Novotny, 2003). Infiltration filters out most solid pollutants and soil processes reduce dissolved pollutants and bacteria. Oil and grease are broken down by microbes in porous pavement (Ferguson, 2005). Table 2.4-2 summarizes the pollutant removal mechanisms of LID.

Table 2.4-2 LID Pollutant Removal Mechanisms

Pollutant Removal Mechanism	Pollutants
Absorption to Soil Particles	Dissolved metals and soluble phosphorus
Plant Uptake	Small amounts of nutrients including phosphorus and nitrogen
Microbial Processes	Organics (Oil and Grease), Pathogens
Exposure to Sunlight and Dryness	Pathogens
Infiltration	Filters runoff (see next point)
Sedimentation and Filtration	Total suspended solids, floating debris, trash, soil-bound phosphorus, some soil bound pathogens

Adapted from Hunt (2001)

2.4.2 Advantages and Disadvantages of LID

While the advantages of LID are considerable, it is not a 'silver-bullet' for stormwater management. Table 2.4-3 summarizes the advantages and disadvantages of LID when compared with traditional stormwater practices.

Table 2.4-3 Advantages and Disadvantages of LID

Criteria	Advantage	Disadvantage	Source
Design		More difficult to design than traditional storm drainage controls Design manual differences between entities, agencies, and academia can be confusing	(Earles, Rapp, Clary, & Lopitz, 2009) Author
Maintenance	Mowing & trash removal	More sensitive to maintenance than traditional stormwater controls Permeable Pavements require periodic vacuuming	(EPA, 2000)
Space Requirements	Fits in small areas Requires smaller storm drain pipes Reduces or eliminates the need for a detention pond		(EPA, 2000)
Cold Climate	Functions in all climates Perm. Pavements reduce need for deicing		(Houle, 2008) (Coffman, 2009)

Criteria	Advantage	Disadvantage	Source
Constructability		More difficult than traditional facilities Improper construction causes premature failure	(Ferguson, 2005)
Likelihood of Failure		More susceptible than traditional stormwater facilities	Author
Multi-Use	LID and conventional BMP’s can both be multi-functional amenities if designed well.		(Prince George's County, 2000a)
Aesthetics	Adds to landscape value Adds amenities to landscape (Example: City of Malmo)		(Prince George's County, 2000a) (Stahre, 2008)
Cost	Smaller storm drainage and detention system will be less costly than conventional stormwater systems	Permeable Pavements have higher upfront costs	(Prince George's County, 2000a) (EPA, 2007)
Peak Flow Rate Reduction	Can be accomplished if designed properly		(Prince George's County, 2000b)
Flow Duration Reduction	Can reduce flow duration		(Prince George's County, 2000b)
Volume Reduction	Potentially large reductions under the right conditions		(Prince George's County, 2000a)
Pollutant Removal	High potential for pollutant removal, including dissolved and bacterial constituents. Uses natural processes to store and break down pollutants		(Prince George's County, 2000a) (Coffman, 2009) (Clary, et al., 2009)
Ground Water Recharge/ Stream Base Flow	Depends on soils Potentially very high		(Prince George's County, 2000a) (Ferguson, 2005)
Regulation		Depends on region; many areas offer little to no stormwater credit for LID practices. Inconsistencies in regulations between entities Traditional stormwater management practices are prescribed	(Earles, et al., 2009)
Other	Reduces "urban heat island" effect	Swelling soils and frost heave can be issues	(Ferguson, 2005) (Prince George's County, 2000b)

2.5 LID BMPs in this Model

The LID BMPs that will be used in this model are grass swales, rain gardens, infiltration trenches, and permeable pavements. This section describes these in more detail.

2.5.1 Grassed Swales

A grassed swale is a shallow slow-flowing waterway lined with grass. The longitudinal and side slopes are low to keep flow velocity low and promote infiltration and sedimentation (UDFCD, 2007). Figure 2.5-1 shows a picture of a grassed swale.

Figure 2.5-1 Grassed Swale Photo
Source: (UCONN, 2010)

2.5.2 Rain Gardens

A rain garden, also known as bio-retention or porous landscape detention, is a low-lying area that has been excavated and backfilled with a porous material mixed with organics and is covered by natural vegetation and several inches of mulch (DOD, 2004). Rain gardens can be placed about anywhere that runoff can be directed to, such as street and parking lot medians, vegetated street side strips, and lawns. Sometimes they are underdrained and there also needs to be an overflow, either into a swale or inlet. They are very effective at treating stormwater runoff by infiltration and filtration in the porous soil mixture. Suspended pollutants are readily absorbed by the filter media, biological processes and bacteria in the soil further break down and process pollutants, and vegetation uptakes nutrients and heavy metals (Prince George's County, 2000a; UDFCD, 2007). Figure 2.5-2 shows an example of a rain garden.

Figure 2.5-2 Rain Garden Photo
Source: (Coffman, 2009)

2.5.3 Infiltration Trenches

An infiltration trench is an excavated trench that is backfilled with stone or aggregate; this becomes a storage reservoir that promotes additional infiltration of runoff. Just, like rain gardens, they can be placed about anywhere that runoff can be directed to; the bottom of a grassed swale is a good place because swales are designed to convey water and also provide pretreatment by filtering out sediment. Infiltration trenches also can be underdrained if needed. Pollution is removed from runoff by filtration, adsorption, plant uptake, and bacterial degradation (DOD, 2004; Prince George's County, 2000a). An example diagram of an infiltration trench is presented in Figure 2.5-3.

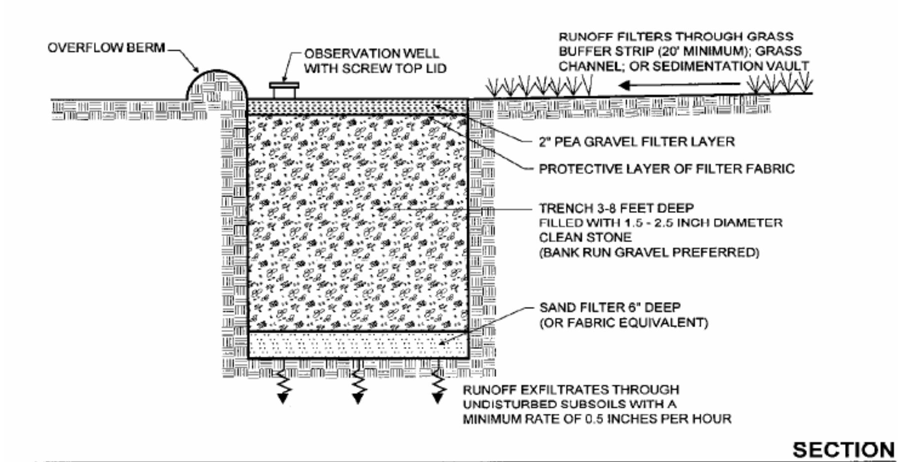


Figure 2.5-3 Infiltration Trench Diagram
Source: (CASQA, 2003)

2.5.4 Permeable Pavements

Conventional asphalt and concrete pavements do not allow significant amounts of water to percolate through the surface layer; consequently, rainfall is immediately converted into runoff, thus they are termed 'impervious.' To be termed 'pervious' or 'permeable,' the pavement must allow enough percolation of water through the pavement to significantly influence the hydrology of the surrounding area (Ferguson, 2005). The six main types of permeable pavement are presented in Table 2.5-1 and example photos are shown in Figure 2.5-5.

Table 2.5-1 Six Main Types of Permeable Pavements

Pervious Concrete
Porous Asphalt
Permeable Interlocking Concrete Pavers (PICP)
Concrete Grid Pavers
Porous Gravel Pavement
Reinforced Grass Pavement

Source: (Ferguson, 2005; UDFCD, 2007)

Hydrologically, all permeable pavements function by allowing water to percolate through the pavement surface layer into the pavement base. The pavement base is the bottom layer in the pavement section that is composed of relatively inexpensive aggregate; it serves the function of strengthening the pavement at lower cost than increasing the surface pavement thickness. The gradation of the aggregate forming the base of conventional pavements is ‘closed’ (or ‘dense’) graded, meaning there is very little void space and low permeability. When using permeable pavements, the base is ‘open’ graded which allows more void space for water to percolate through and this void space also functions as a stormwater storage reservoir. Once water percolates into the base, there are three possibilities: it either infiltrates to the subgrade, exfiltrates through a perforated drain pipe, or a combination of the two. These three drainage configurations are shown in Figure 2.5-4.

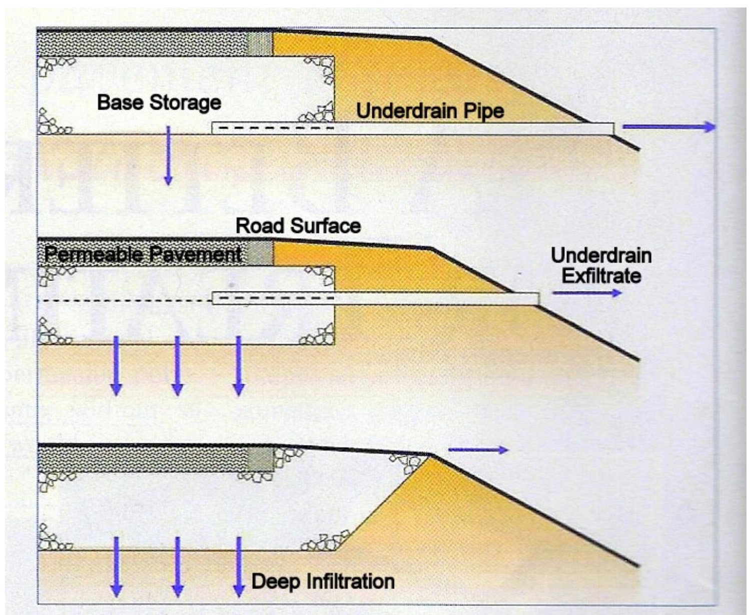


Figure 2.5-4 Permeable Pavement Underdrain Configurations
Adapted From: (Ferguson, 2009)

The top diagram in Figure 2.5-4 displays the underdrain pipe located at the bottom of the storage reservoir; this configuration will exfiltrate most or all of the stored water and would be used when the underlying soils have insufficient infiltration. The middle diagram shows the underdrain lifted off the bottom of the reservoir, which allows full infiltration of small storms and partial infiltration of large storms. The bottom diagram displays no underdrain, which would be used in situations where the underlying soils allow for full infiltration of the storm water.

A potential problem with permeable pavements is the loss of infiltration capacity, this can happen in several ways. The pavement surface can become clogged with sediment deposited by wind, automobiles, and from loose sediment adjacent the pavement. In this case the infiltration capacity can be partially or fully restored by vacuuming the pavement and should be done periodically to maintain the infiltration capacity (Bean, Hunt, & Bidelspach, 2007; UDFCD, 2007). A second way that clogging can occur, is if the filter fabric that separates different layers of base material clogs with fine sediment. Some have reported that is what happens (Ballestero, 2009; Boving, Stolt, Augunstern, & Brosnan, 2008), while others say that it is the use of incorrect fabrics and improper installation that are causing clogging (Urbonas, 2009). The alternative option is to use a graded Terzaghi filter, but these can be complicated to design and install (Ferguson, 2005; Urbonas, 2009). More research is needed on the efficacy of filter fabrics as separators (Ferguson, 2005). Third, the infiltration capacity of the subgrade can be severely reduced by intentional or inadvertent soil compaction during construction (UDFCD, 2007). Because permeable pavement functions structurally as a road and hydrologically as a stormwater control, there is disagreement about how to prepare the subgrade. Proper design of the pavement cross section should eliminate the structural need for compaction and care must to be taken to limit traffic over the areas where there will be permeable pavement and other BMP's that function by infiltration.

The different types of permeable pavement accomplish their permeability in different ways. Pervious concrete and porous asphalt accomplish this by removing fine aggregate from asphalt and mid-sized aggregate from concrete to achieve enough void space in the final pavement structure (Ferguson, 2005; UDFCD, 2007). Permeable pavers (sometimes called Permeable Interlocking Concrete Pavers or PICP) and concrete grid pavers are prefabricated concrete blocks; PICP have corners that are beveled and concrete grid pavers are shaped like a lattice. These void spaces are filled with a fine open-graded aggregate after placement, which preserves the permeability of the voids. Example photos are shown in Figure 2.5-5.

Figure 2.5-5 Permeable Pavement Photos

Pervious Concrete

Source:

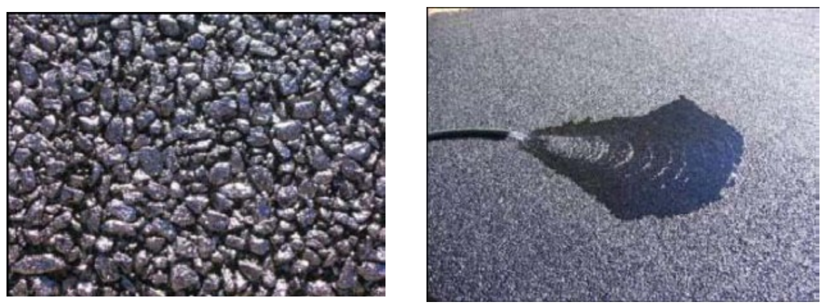
Left: (UDFCD, 2008)

Right: (CRMCA, 2009)

Porous Asphalt

Source:

(Ballestero, 2008)



Permeable Interlocking Concrete Pavers (PICP)

Source:
Left: (UDFCD, 2007)
Right: (Hunt W.F., 2006)

Concrete Grid Pavers

Source:

(Hunt W.F., 2006)

2.6 Summary of Literature Review

In summary, the literature review shows that urbanization significantly increases the volume of stormwater runoff, peak flow rates, and water pollution; these lead to many problems including flooding and the degradation of urbanized streams. Traditional stormwater management attempts to address these using detention basins with peak shaving controls and traditional BMPs; however these fail to adequately address the cause of urban stormwater problems. Additionally, improperly designed stormwater controls may actually exacerbate the degradation of streams by prolonging the duration flows greater than the critical shear stress of the receiving stream (Rohrer, 2004). LID presents a new way to manage stormwater that deals with the increased volume of runoff by using natural process and infiltration based BMPs. The future of stormwater management with LID seems promising; however there are many issues to be worked out and questions yet to be answered. For example: How much LID will it take to meet present stormwater requirements? Can LID actually maintain the predevelopment hydrologic regime? How much will it cost? Can it be profitable? What about the longevity of these BMPs?

3.0 Study Site

In order to quantify the amount of LID necessary to meet stormwater requirements and to restore predevelopment site hydrology, EPA SWMM was used to build and analyze an LID model based upon a proposed development in Fort Collins, CO. This development, named Union Place, was developed by Merten Inc. and the civil engineering was performed by Nolte Associates, Inc. The proposed site was designed with some LID and this model built on what was done so that the stormwater requirements were met with LID alone. The LID practices that were modeled include grassed swales, rain gardens, infiltration trenches, and permeable pavement.

3.1 Study Site Description

Union Place is a high density mixed use development with mostly residential and a few commercial buildings. The residential consists of single family, condos, and triplexes. The site is located in north Fort Collins, on the southwest corner of the intersection of N. College Ave. and W. Willox Lane. The full site encompasses 11 acres and this model analyzes 7.4 of those acres.

Before development, the site was a cultivated alfalfa field with an old house foundation and a grove of trees in the north-eastern corner. The site sloped at roughly 0.6% to the Southeast where there was a small farm swale that empties into a storm drain pipe.

Figure 3.1-1 presents the proposed Union Place site plan; a thick dashed line identifies the modeled 7.4 acres. Composing the 3.6 acres excluded from the model are a future city regional detention pond, a proposed piece of the future Mason Street extension, and another detention pond receiving offsite flow from the adjacent McDonalds parking lot. These areas are excluded from the model because they detract from the purpose of this thesis, which is to look directly at the effects of LID. The proposed drainage plan is shown by the blue line and arrows in Figure 3.1-1. The LID layout is shown in Figure 3.1-2 on the next page.

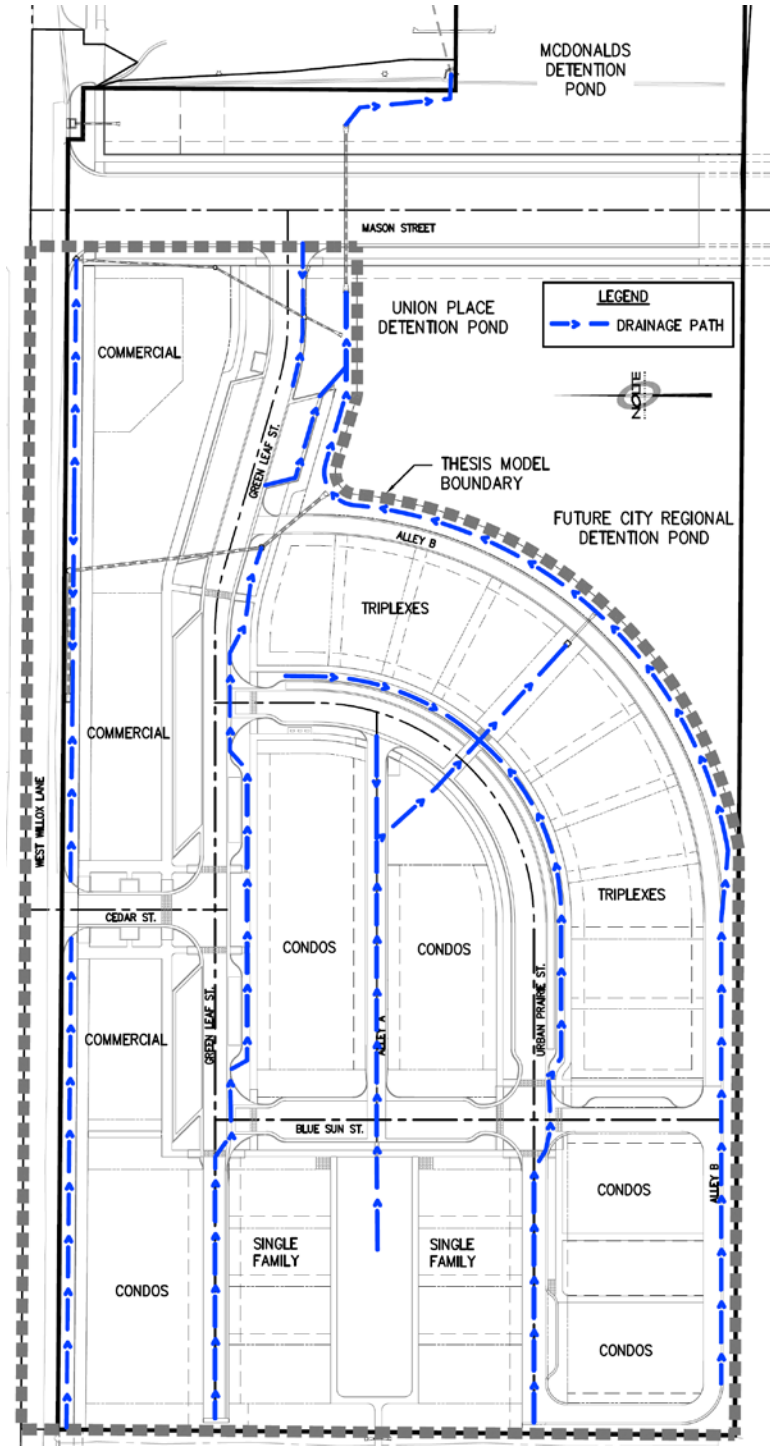


Figure 3.1-1 Thesis Model Site Plan

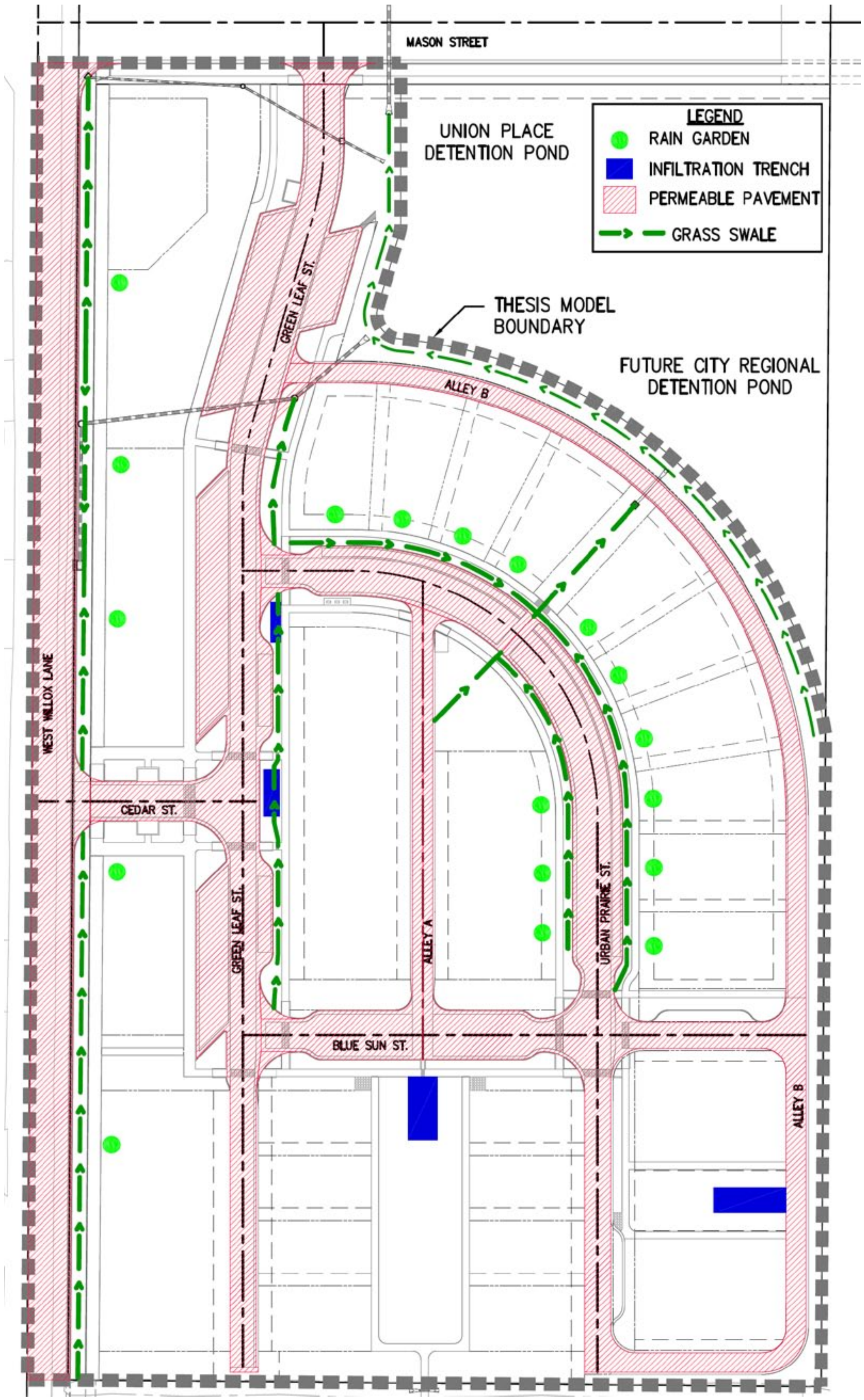


Figure 3.1-2 LID Model Layout

4.0 SYNTHESIS OF LID IN SWMM

Presently, the publically available EPA SWMM program is not explicitly capable of simulating LID BMPs. This chapter presents two methods to manually model LID in the current EPA SWMM program (version 5.0.018). The first method models LID BMPs in a full infiltration situation (see Figure 2.5-3). The second method models LID BMPs in a partial infiltration situation which requires underdrains and is much more complicated than the first method. Before presenting how to adjust SWMM to simulate LID infiltration practices, a brief infiltration and runoff discussion is needed.

4.1 Hydrology Overview

Determining runoff is the primary interest of urban stormwater models. Runoff is produced when the rainfall plus run-on rate is greater than the ability of the ground to infiltrate the water; this can occur in two ways:

1. The rainfall plus run-on rate is higher than the infiltration capacity of the surface. Figure 4.1-1 presents an example of this condition.
2. The soil storage has been exhausted and the soil is completely saturated.

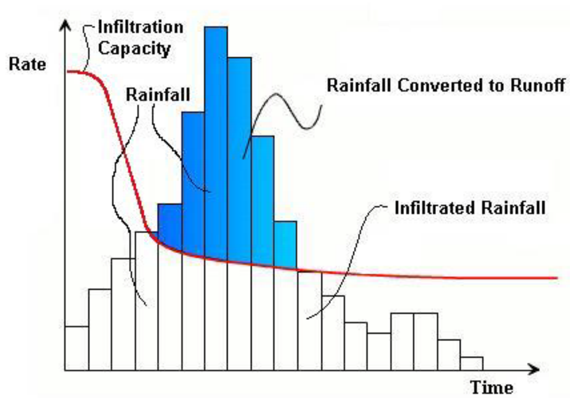


Figure 4.1-1 Rainfall-Runoff Curve
Adapted From: (Musy, 2001)

However, understanding infiltration is the key to determining runoff. When rainfall occurs, water ponds on the surface of the ground in small depressions, as the water collects it enters the soil through small pores and cracks due to the combined effects of capillary and gravitational forces. Capillary forces, also referred to as tension or suction forces, are the result of water's surface tension property and are inversely proportional to the size of the pores (Novotny, 2003). Water that is infiltrated due to capillary suction will remain in the soil matrix and will not gravity drain, this water is removed from the soil through evaporation and plant uptake (evapotranspiration). The moisture content below which water will no longer gravity drain from the soil is called *field capacity* (Smedema, Vlotman, & Rycroft, 2004). Water that is infiltrated as a result of gravity can be broken into two categories: saturated flow and unsaturated flow. Unsaturated flow is very complicated and will not be discussed here because of assumptions that will be made later. Saturated flow can be considered the minimum infiltration value of a soil and is measured by the *saturated hydraulic conductivity* or k_{sat}.

When rainfall begins, infiltration is driven by both capillary and gravitational forces. As pores in the soil accumulate water, the capillary forces and infiltration due to capillary forces decrease and eventually cease, at this point infiltration is driven solely by gravity and the rate is at the saturated hydraulic conductivity (Novotny, 2003). The sample infiltration curves in Figure 4.1-2 show the decrease in infiltration rate with time as described here.

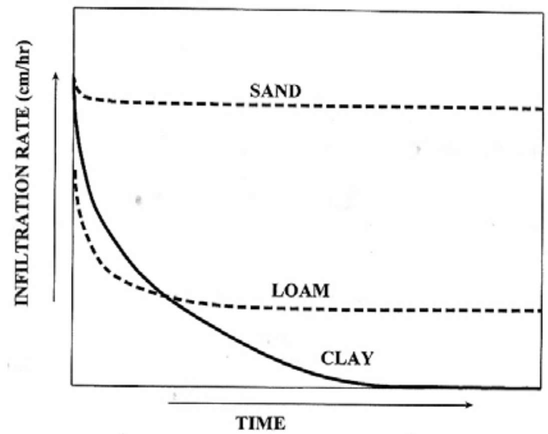


Figure 4.1-2 Idealized Infiltration Curves
Source: (Novotny, 2003)

Fine grained soils such as silt and clay have very small pore spaces and thus infiltration due to capillary forces is quite high. However, very little space is left for water to percolate through because the small pores hold water so strongly and as a result, infiltration due to gravity is low (low saturated hydraulic conductivity). Conversely, coarser grained soils, such as gravel and sand, have much larger pores and infiltration due to gravity dominates. They have high minimum infiltration rates and low field capacities (Novotny, 2003; Smedema, et al., 2004).

There have been many different methods proposed to estimate the temporal decrease in infiltration rate. The Horton infiltration option will be used in SWMM which iteratively solves the integral form of the volumetric Horton equation given below (Viessman & Lewis, 1995).

$$F(t) = \int_0^{t_p} f_p \, dt = f_{\min} \cdot t_p + \frac{f_{\max} - f_{\min}}{k} \cdot \left(1 - e^{-k \cdot t_p}\right)$$ where,

$F(t)$ = the cumulative infiltration volume at time, t

t = time from onset of rainfall (hr)

t_p = the equivalent time for the actual infiltrated volume (hr)

f_p = the infiltration rate at some time, t_p

f_{min} = minimum or constant infiltration rate (in/hr)

f_{max} = maximum or initial infiltration rate (in/hr)

k = decay constant (1/hr)

When the actual infiltration rate is less than the infiltration capacity of the soil, the infiltration capacity does not decay as rapidly. To account for this, this method keeps track of the actual volume of water infiltrated and then calculates an equivalent time, t_p, to represent the time at which the cumulative infiltration volume of the equation equals the actual infiltrated volume. This allows the decay of the infiltration rate to be a function of the infiltrated water volume instead of a function of time (Viessman & Lewis, 1995).

4.2 Full Infiltration Method

The following methods are presented assuming that the reader has a basic understanding of the SWMM model. These methods have been adapted in part from two publications on how to manually model LID in EPA SWMM: *BMP Modeling Concepts and Simulation* by Wayne Huber (2006) and *Storm Water Management Model Applications Manual* by Jorge Gironás (2009). Both are available on the EPA website.

4.2.1 Non-Directly Connected Impervious Areas (NDCIA) and Grass Buffers

To model NDCIA in SWMM, simply set the *subarea routing* to "pervious" for the subcatchment with NDCIA. The model will then flow runoff from the impervious area onto the pervious area. This method can be used to represent roof down spouts and patios draining over grass yards, and sidewalks draining to grassed areas, as seen in Figure 4.2-1. Since the properties of a buffer strip, such as slope, may be different than the adjacent area, another option is to use two subcatchments and discharge one onto the other, and adjust the properties of the downstream subcatchment to represent the buffer.

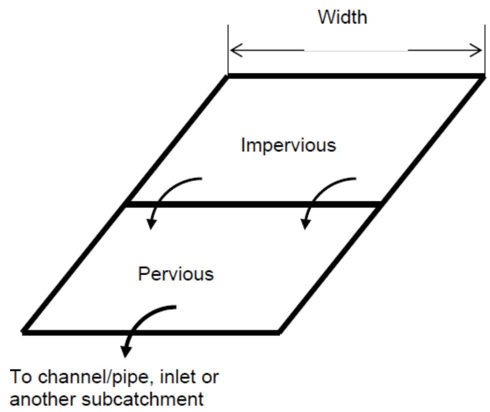


Figure 4.2-1 Pervious Subcatchment Routing

Source: (Huber, 2006)

4.2.2 Infiltration Trenches

The method presented here for infiltration trenches is a summary of the method presented in *Storm Water Management Model Applications Manual* (Gironás, et al., 2009). See this manual for a more detailed explanation. This method uses a subcatchment to model the surface area, storage volume, and infiltration of an infiltration trench. This is for situations where the soils allow full infiltration of the water captured by the infiltration trench.

Steps to create an Infiltration Trench in SWMM:

1. Draw a *subcatchment* defining the surface area of the infiltration trench. Since SWMM computes area with units of acres, the trench may be too small for SWMM to accurately figure the area. The area needs to be manually calculated and entered.
2. Set the *% Impervious* to zero.
3. Set the *Pervious Depression Storage* equal to the effective depth.

$$DStore_{perv} = d_{effective} = d_{trench} \cdot \Phi$$ where,

$DStore_{perv}$ = Pervious depression storage

$d_{effctive}$ = Effective water depth in trench

d_{trench} = Actual depth of infiltration trench

Φ = Porosity of trench fill

4. Set the *Pervious Manning's n* to the surface material of the trench.
5. Set the *maximum* and *minimum infiltration* values for the **subcatchment** (NOT the whole site) to the saturated hydraulic conductivity of the subsurface soil.

4.2.3 Permeable Pavement

The method presented here for permeable pavement is very similar to the infiltration trench method; it uses a subcatchment to model the surface area, storage volume, and infiltration of the permeable pavement. This method, again, is for situations where the soils allow full infiltration of the water captured by the permeable pavement.

Critical Assumption:

The method outlined below is critically dependent upon the assumption that the rate of infiltration through the permeable pavement surface is not a limiting factor; it is high enough that it can accept all of the rainfall plus run-on. In other words, runoff does not occur until the base reservoir has been filled; Figure 4.2-2 and Figure 4.2-3 graphically present this situation. This is the second runoff condition listed in section 4.1.

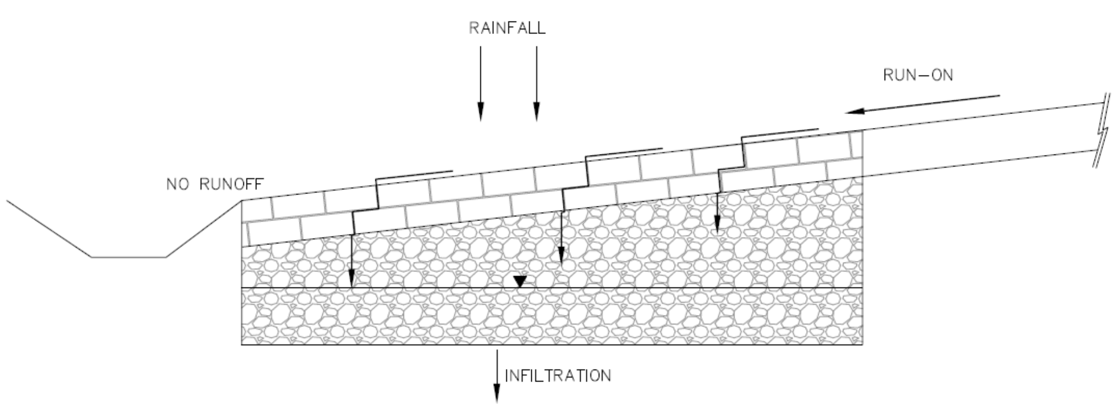


Figure 4.2-2 Permeable Pavement - No Runoff Condition

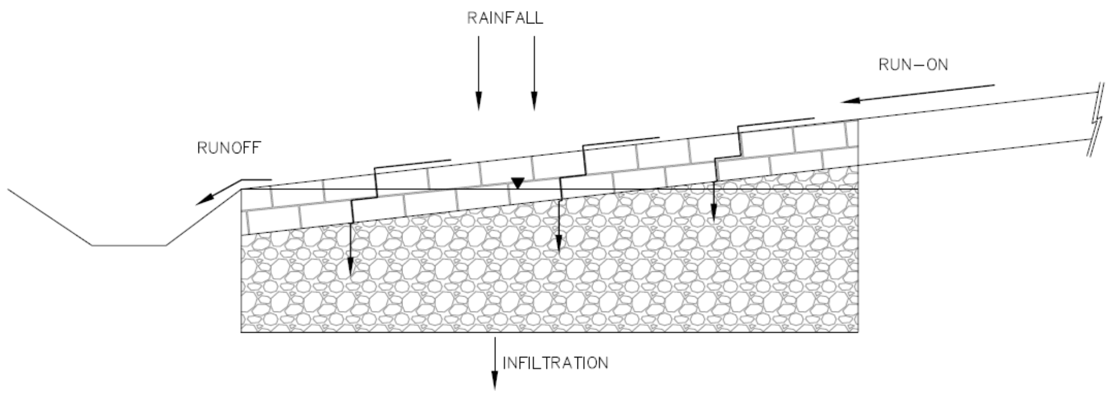


Figure 4.2-3 Permeable Pavement - Runoff Condition

A significant amount of research has been conducted on the infiltration rates of permeable pavements. Bean, Hunt et al. (2007) presented infiltration data on 14 PICP and 11 pervious concrete sites and found that infiltration rates of PICP and pervious concrete were not limited by their surface infiltration capacity as long as they were not clogged by fine sediment. The average infiltration rates for maintained sites were 800 in/hr for PCIP and 1500 in/hr for pervious concrete. Unmaintained sites contaminated with fines had rates on the order of 10 in/hr. Ferguson (2005) and Ballestero (2008) have presented similar data for pervious concrete.

Steps to create Permeable Pavement in SWMM:

1. Draw a *subcatchment* defining the surface area of the permeable pavement. It is important that the subcatchment only defines the permeable pavement area and does not overlap other subcatchments. Make sure that the "auto-length" setting is turned on.
2. Set the *% Impervious* to zero.
3. Set the *Pervious Depression Storage* equal to the depth of the base multiplied by porosity of the base. Typical porosity values are 0.3 to 0.4 (Ferguson, 2005).

$$d_{effective} = d_{base} \cdot \Phi \ , \quad \text{where} \quad \Phi = \ Porosity$$

4. Set the *<u>Pervious</u> Manning's n* to the permeable pavement surface material. This should be slightly higher than typical pavement values for Manning's n.

5. Set the *<u>maximum</u>* and *minimum infiltration* values for the subcatchment (NOT the whole site) to the saturated hydraulic conductivity of the <u>subsurface soil</u>. Due to potential compaction during construction, it would be conservative to apply a safety factor to the tested infiltration rates, Ferguson (2005) suggests a safety factor of 2.

4.2.4 Rain Gardens

The rain garden method presented here is for rain gardens that are at the subcatchment level in the model, the only flow going to them is from a subcatchment and not from a conduit. Roof drains directed towards a rain garden in the yard are an example.

<u>Steps to create a Rain Garden in SWMM:</u>

1. Set the *subarea routing* of the subcatchment to "pervious".
2. Calculate the storage volume of the rain garden:

$$\forall_{RG} = \forall_{RGvoids} + \forall_{Surface\_Storage}$$ where,

$$\forall_{RGvoids} = L \cdot W \cdot d_{Soil} \cdot \Phi$$

$$\forall_{Surface\_Storage} = L \cdot W \cdot d_{Surface\_Storage}$$

L = Length of rain garden (ft)

W = Width of rain garden(ft)

d_{Soil} = Depth of excavated and backfilled soil media (ft)

$d_{Surface\_Storage}$ = Maximum depth water can pond on top of rain garden (ft)

Φ = Porosity of rain garden fill

3. Multiply the storage volume by the number (n) of rain gardens per subcatchment:

$$\forall_{RGtotal} = \forall_{RG} \cdot n_{RainGardens} \quad \text{(volume in ft}^3\text{)}$$

4. Add that volume (in ft^3) to the pervious area as <u>depth</u> of *pervious depression storage*:

$$DStore_{Perv+RG} = DStore_{Perv} + \frac{\forall_{RGtotal} \cdot \left(12_{in\ /\ ft}\right)}{A_{subcatch.} \left(1 - \%\mathrm{Imp}\right)}$$

4.3 Partial Infiltration Method (With Underdrains)

4.3.1 Overview of Method

The method to model LID outlined in this section is more complicated than the previous method, but it does allow for underdrains and partial infiltration situations. The partial infiltration method uses a storage node to represent base storage instead of a subcatchment. Using a storage node allows the modeler to choose how to route water stored in the base of a BMP; in the previous method this could not be done. This method to manually model LID in SWMM is adapted from methods presented by Zhang (2009) and Lucas (2009).

4.3.2 Representation of Permeable Pavement with Underdrain in SWMM

Because this method changes the way SWMM would normally simulate infiltration and because underdrains add significant processes not previously accounted for, a schematic diagram of the SWMM representation is presented in Figure 4.3-1. Following the diagram is a section describing how certain physical processes are manually represented.

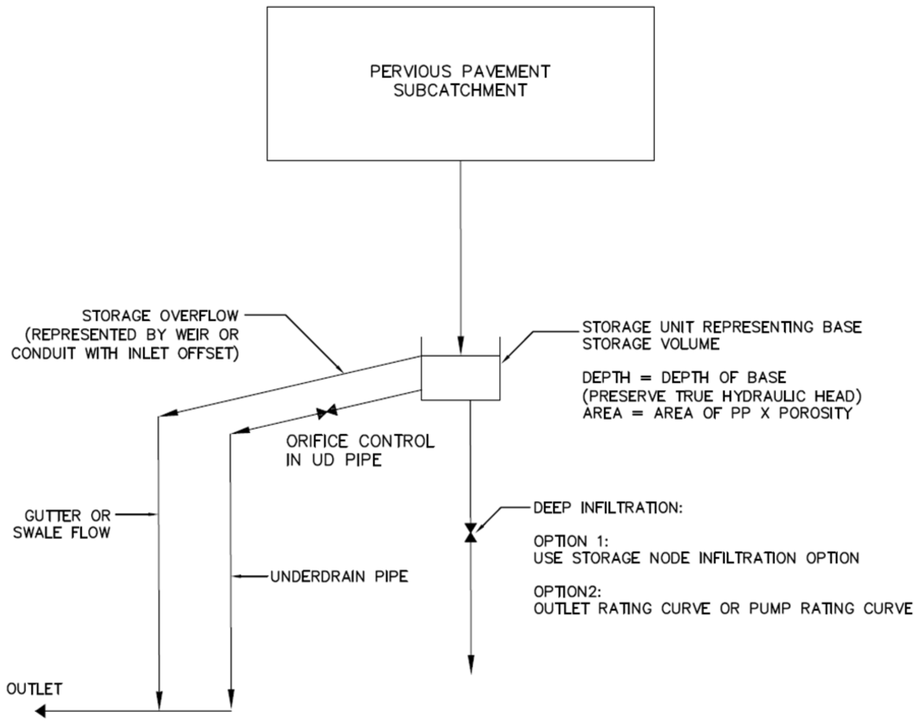


Figure 4.3-1 SWMM Representation of Permeable Pavement with Underdrain

4.3.2.1 How Physical Process are Manually Represented

1. **Rainfall –** Rainfall is simulated the same way it normally is.
2. **Runoff –** Create a subcatchment defining the permeable pavement area and set it to *100% impervious*. This subcatchment receives rainfall and converts the rainfall to runoff. The SWMM RUNOFF block then flows the runoff over the length of the subcatchment as sheet flow. At this point the user can decide where to put the water. Normally it is routed onto another subcatchment or into the conduit system, but in this case it is routed into a storage node representing the base storage. Runoff will commence when the base storage reservoir begins to overflow. This method makes the same **critical assumption** that the rate of infiltration through the permeable pavement surface is not a limiting factor.

To model storage overflow into the conduit system, a weir or a conduit is offset from the bottom of the storage unit the same distance as the base depth (see Figure 4.3-1 above). These overflow links need to have the *flap gate* setting turned "on"; this will prevent backflow into the storage reservoirs and prevent model oscillation and instability.

In reality, rainfall would first infiltrate into the permeable pavement and not flow across the pavement surface. This lag time should be comparatively small and may also off-set the percolation travel time which is not taken into account (see point 3).

3. **Percolation through Permeable Pavement -** This method does not take into account the travel time for water to percolate through the pavement and the base. It assumes that once the water has flowed across the subcatchment it instantaneously enters the base storage represented by the storage node. This is considered an acceptable assumption since the time for water to percolate through the pavement to the bottom of the storage reservoir is on the order of seconds and it may also be offset by the travel time for the water to flow across the subcatchment (see item 2 above).

4. **Base Storage –** The base storage is represented by a storage node whose volume is the same as that of the void space in the base. Water ponded on surface is not accounted for.

$$\forall_{storage} = A_{pavement} \cdot d_{base} \cdot \Phi_{base}$$ where,

$\forall_{storage}$ = Volume of the void space in the base (ft^3)

$A_{pavement}$ = Pavement surface area

d_{base} = Base storage depth

Φ = Porosity

The *storage node depth* should be set equal to the actual base depth, in order to preserve the true hydraulic head on the outlet orifices. Therefore the *storage unit area*, instead of the depth, is reduced to account for the porosity of the base. This creates a rectangular prismatic storage volume.

$$A_{storage} = \frac{\forall_{storage}}{d_{base}} = A_{pavement} \cdot \Phi_{base}$$

By terracing the subgrade, the base storage can be represented by a rectangular prismatic volume instead of a triangular prismatic volume for a conventional subgrade which is graded parallel to the road surface. The difference between these two is shown in Figure 4.3-2 and Figure 4.3-3.

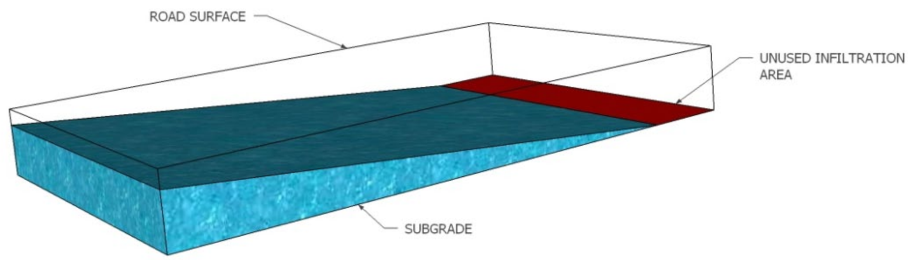


Figure 4.3-2 Conventional Base Storage Volume

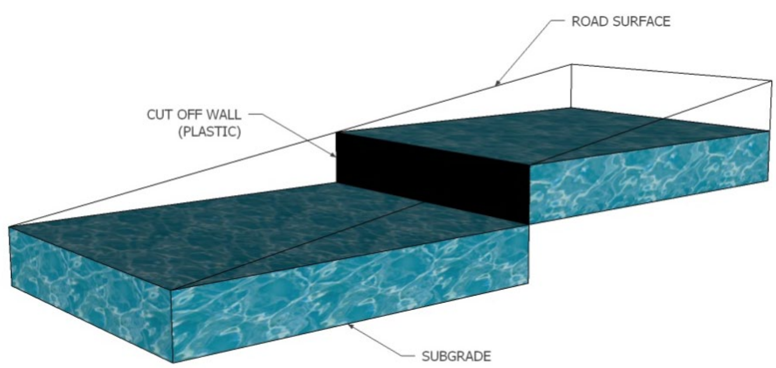


Figure 4.3-3 Terraced Base Storage Volume

The practice of terracing the subgrade also maximizes the effect of LID because it allows the full subgrade area to be available for infiltration at all times. In contrast, a triangular prismatic storage volume will have decreased subgrade surface area and subsequent infiltration capacity as the water depth decreases.

5. **Deep Infiltration to Subgrade –** Deep infiltration into the subgrade can be modeled three different ways, this model will use the first option presented below.

1. Use the built in storage node infiltration feature. This became available in version 5.0.015. The Green-Apt infiltration equation is the only method offered for storage nodes. It is recommended to set the *initial moisture deficit* (IMD) to zero which forces the infiltration rate of the subsurface to the *saturated hydraulic conductivity* from the start of the simulation. (Note: the *suction head* cannot be zero for SWMM to run.)

   Infiltration Rate Adjustment:

   It has been suggested that the infiltration rate be reduced by a safety factor of 2 to account for potential and likely loss of infiltration capacity due to construction compaction (Ferguson, 2005). However, since the storage area was reduced in the model to account for porosity (in point 4), the infiltration area was consequently reduced. To account for the reduced infiltration area, the *saturated hydraulic conductivity* for the subgrade should be increased by dividing by the porosity.

   $$k_{subgrade} = \frac{k_{sat}}{2}$$ *This is the safety factor adjustment.*

   $$k_{model} = \frac{k_{subgrade}}{\Phi} = \frac{k_{sat}}{2\Phi}$$

   Note: In the SWMM simulation status report, the storage unit infiltration is included with storage evaporation losses and is displayed as '*Storage Losses.*'

2. Use an outlet rating curve or pump rating curve to "hard code" the infiltration rate. Connect the link to a separate outlet designated "infiltration."

3. No infiltration, representing a BMP that is lined to prevent infiltration. Make sure that the storage node infiltration option is turned off.

6. **Flow into Underdrains –** Flow into underdrains is a very complex process. First, there is vertical unsaturated flow from the pavement down to the free water surface in the base (item 3 above states that this will be neglected). Second, there is saturated horizontal flow through the base to the underdrain. To get into the underdrain, water must flow through the fabric-wrap (to prevent fine particles from entering the pipe) around the pipe and through holes located in the side of the pipe. Then the water flows down the underdrain pipe to the junction with the main pipe. Finally, at the junction box with the main pipe, the Denver UDFCD suggests putting an orifice on the end of the pipe to restrict the flow and extending the storage duration.

   Modeling all of these processes would be very complicated. However, this thesis will assume that the orifice in the pipe, when sized properly, will control this whole system and the former processes can be neglected.

   The UDFCD *Manual* specifies that this orifice be sized to drain the permeable pavement cell in more than 6-hours (Figure PP-9 of the UDFCD *Manual*, vol. 3). The time that it takes water to percolate from the pavement surface to the base reservoir is on the order of seconds or minutes and horizontal flow to the underdrain pipe is on the order of a few minutes. Therefore, it is reasonable to conclude that the orifice in the pipe will control the underdrain outlet rate when sized to drain the cell in more than 6-hours.

   To avoid anoxic conditions in the base, the water should completely drain in 24 hours (48 maximum) (Hunt & Collins, 2008; Low Impact Development Center, 2007) Correct orifice sizing can be accomplished through iterative running of the SWMM model. Finally, a good option for lower permeability non-expansive soils is to raise the underdrain several inches off of the bottom of the reservoir, providing a permanent infiltration volume for high-frequency, low volume storms. The middle diagram in Figure 2.5-4 shows an example of a raised underdrain.

7. **Storage due to Capillary Suction (Field Capacity) –** The field capacity of typical base aggregate (no. 57 stone) is 5% moisture content (James, 2001); since this is relatively small it will be neglected in the model. This will have no effect on design storm results and will slightly affect continuous simulation results in the very low flows. If one desires to be more conservative, the volume represented between 0% moisture content and field capacity could be removed from the storage volume in the SWMM model.

5.0 SCENARIO DEVELOPMENT

This chapter describes the SWMM model requirements, scenarios, environmental conditions, and parameters used to build and run the models.

5.1 Stormwater Requirements

Nearly every municipal entity in the United States has stormwater requirements which regulate flood control and stormwater quality. This section describes the City of Fort Collins requirements and how they are adopted or modified for the thesis model.

5.1.1 Stormwater Quality

Stormwater treatment for this development is be required under Phase II of the NPDES and will be addressed with LID practices instead of a traditional BMP. Most LID practices are able to address the water quality requirements through their own respective natural mechanisms, as discussed in section 2.4. The WQCV will be determined with the Denver UDFCD method with the exception that permeable pavements will be allowed to treat 100% of the WQCV, as long as there is sufficient storage volume in the base.

5.1.2 Water Balance

The city of Fort Collins does not have a water balance requirement (the author is not aware of any municipality having a water balance requirement). However, in order to restore the predevelopment flow regime, a water balance requirement will be added to this model.

5.1.3 Flood Control

The City of Fort Collins master drainage plan requires ‘over-control’ of stormwater peak flow rates in this basin. The requirement is to reduce the postdevelopment 100-year peak flow rate to the predevelopment 2-year peak flow rate (City of Fort Collins, 1997). However, standard stormwater peak flow requirements are to maintain, not over-control, predevelopment peak flow rates in the postdevelopment condition for the 10- and 100-year storms; this is termed ‘normal-control.’ These requirements address the peak flow rate but not the total volume of runoff.

In this model LID practices will be used to control peak flow rates instead of a conventional detention pond. This study will relax the City of Fort Collins over-control requirements to normal-control in exchange for restoring the predevelopment runoff volume because the advantages of restoring the water balance are so significant. Normal-control requirements are also more representative of standard practice in stormwater management. Table 5.1-1 presents the stormwater control requirements for the City of Fort Collins and the modified requirements that will be used for this model.

Table 5.1-1 Stormwater Control Requirements

Criteria	Fort Collins Stormwater Requirement	Thesis Model Requirement
Peak Rate Control	Over-Control: 100-year developed back to 2-year undeveloped peak flow rate	Normal-Control: 2-yr and 100-yr peak flows developed back to respective undeveloped peak flow rates
Runoff Volume	No Regulation	Match undeveloped water balance for the 2-yr and 100-yr design storms.
Water Quality	Treat WQCV with a BMP (Typically an extended detention basin)	Treat WQCV from each area with a LID BMP

5.2 Study Site

As described in chapter 3.0, the study site, Union Place, is located in north Fort Collins, CO and is a high density mixed use development with mostly residential and a few commercial buildings. The proposed site was designed with some LID and this model builds on that in order to meet the stormwater requirements with LID alone.

Before development, the site was a cultivated alfalfa field with an old house foundation and a grove of trees in the north-eastern corner. The overland slopes are very flat, approximately 0.6%.

5.2.1 Soils Report

CTL Thompson, Inc. performed soil tests on the site, which revealed that a sandy clay layer covers the entire site at depths varying from 2 to 14 feet. The sandy clay is underlain by a clayey sand and then by cobbles. After applying a safety factor of 2 (Ferguson, 2005), design infiltration rates ranged from 0.5 inches/hour to 1.5 inches/hour. A design rate of 1.0 inches/hour was suggested by CTL Thompson (2009).

5.3 Model Design

This section explains how the proposed Union Place site plan was adapted and modeled in this thesis study. The goal was to deviate as little as possible from the actual layout, with the exception of adding more LID features. Slope, percent impervious, layout, lot configuration, and drainage network, all remained the same.

5.3.1 Union Place Models

Eight different models were created in order to evaluate the effects of LID on the hydrologic regime:

Table 5.3-1 Union Place Models

1. Predevelopment
2. Traditional Curb and Gutter with DCIA:
    a. Uncontrolled
    b. Detention with Normal Control
    c. Detention with Over Control
3. Street Side Swales with NDCIA
4. Permeable Pavement
5. LID 1 (Full Infiltration)
6. LID 2 (Partial Infiltration with Underdrains)

The following list describes the models listed above.

1. **Predevelopment Model:** Two predevelopment models were created to examine how the predevelopment hydrology actually functions. These models were based on the exact same area contained in the post-development models. The first (DCIA model) had 6% <u>directly</u> connected impervious area, the second (NDCIA model) had 6% <u>non-directly</u> connected impervious area. The DCIA model was used to establish peak flow rates for the design of post-development storm water controls. The NDCIA model was only used for comparison in

the continuous simulations. These will be referred to as the 'Predev.-DCIA' or 'NDCIA' models.

2. **Traditional Curb and Gutter Model:** This model was based on the Union Place site plan, as designed by Nolte, however the LID elements were removed and replaced with conventional ones. Conventional pavements were used on all road surfaces, traditional curb & gutter and storm pipes were used for conveyance, and all impervious areas were directly connected. Extra storm pipes and inlets were added to the Nolte design where curbs overtopped and where needed to remove swales inside the site (Pipes 7, 8, 10, and 11. Inlets D, E, G, and H), see Figure 5.3-1. A collector swale was used to connect the storm drain pipes to the outlet, but the swale was not treated as an infiltration device. Three models were based on this platform:
    a. **Uncontrolled:** This is the Traditional Curb and Gutter model with no detention pond. This model does not meet the water quality or flood control requirements. This will be referred to as the 'Traditional' or 'Trad.' model.
    b. **Normal-Control:** A detention pond with a 40-hour extended detention BMP and conventional 2-year and 100-year peak flow rate controls were added to the uncontrolled model. This will be referred to as the 'Trad. w/NC' model.
    c. **Over-Control:** A detention pond with a 40 hour extended detention BMP and 100-year to 2-year peak rate over-control was added to the uncontrolled model. This will be referred to as the 'Trad. w/ OC' model.

The detention ponds were sized using Nolte's pond grading (depth-area curve) shown in Table 5.4-8. Figure 5.3-1 on the next page shows the Traditional Curb and Gutter model layout.

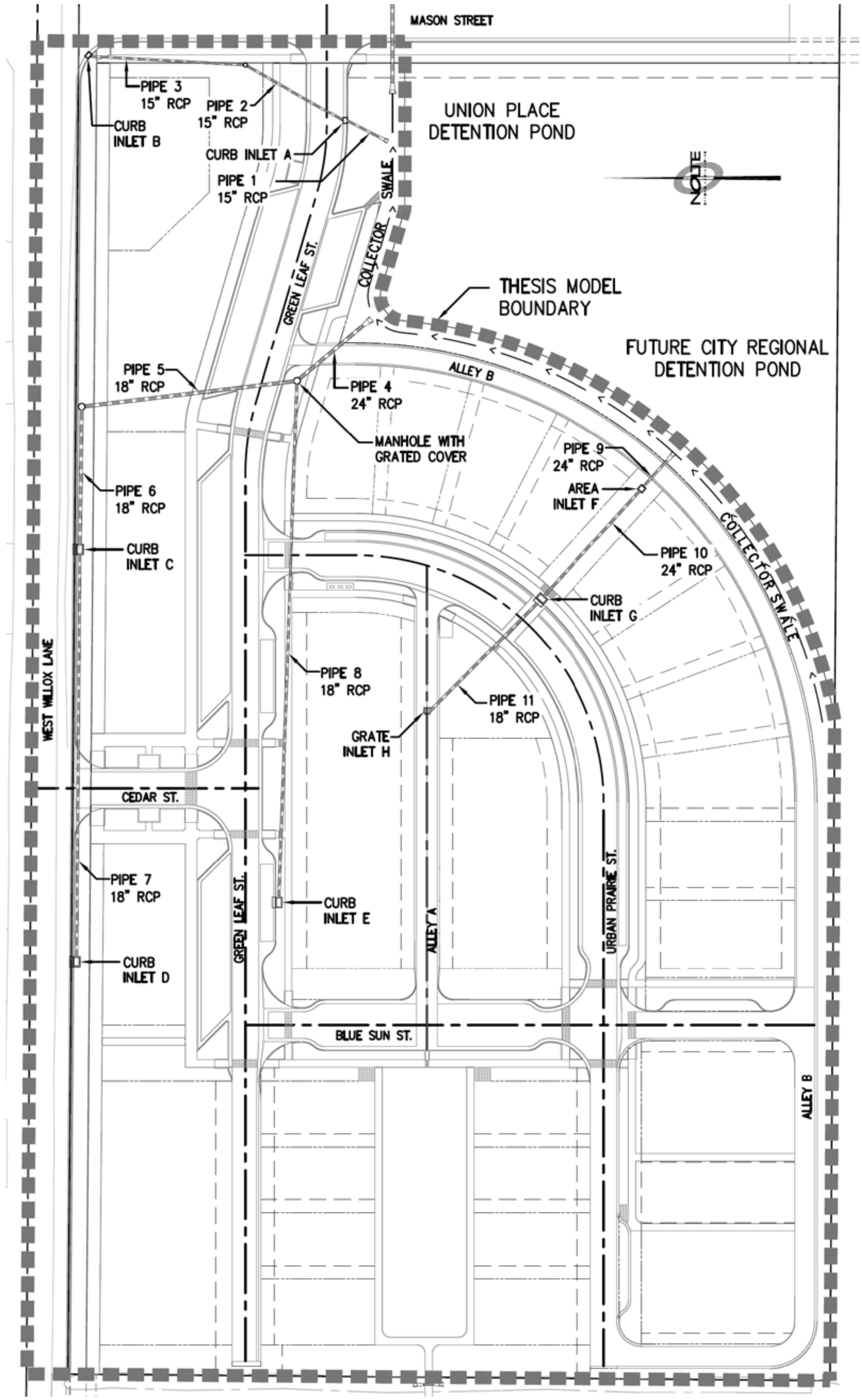


Figure 5.3-1 Traditional Curb and Gutter Model Layout

3. **Street Side Swales Model:** Based on the Traditional Curb and Gutter model, this model replaced the curb and gutter with street side swales. Where possible, the impervious areas were treated as "non-directly connected." The extra storm pipe and inlets that were added in the Traditional Curb and Gutter model were removed. The water quality requirement was not met in this model. This will be referred to as the 'Swales' model.
4. **Permeable Pavement Model:** Building on the Swales model, this model converted all road surfaces to permeable pavement. The permeable pavement was modeled with the *full infiltration* method from section 4.2; this method used a subcatchment's pervious depression storage to represent the pavement base storage and infiltration. The pavement base storage depths were the same as those in the LID 1 model. Water quality was addressed only for areas draining over permeable pavement. This will be referred to as the 'PP' model.
5. **LID 1 Model:** Starting with the Permeable Pavement model, this model used infiltration trenches and rain gardens to capture and treat the WQCV from areas not draining over permeable pavement. The LID BMPs were modeled with the *full infiltration* method from section 4.2, which used the pervious depression storage of a subcatchment to represent the storage and infiltration of a LID BMP. This model meets the water quality, water balance, and peak flow requirements given in Table 5.1-1.

Figure 5.3-2 on the next page displays the LID model layout

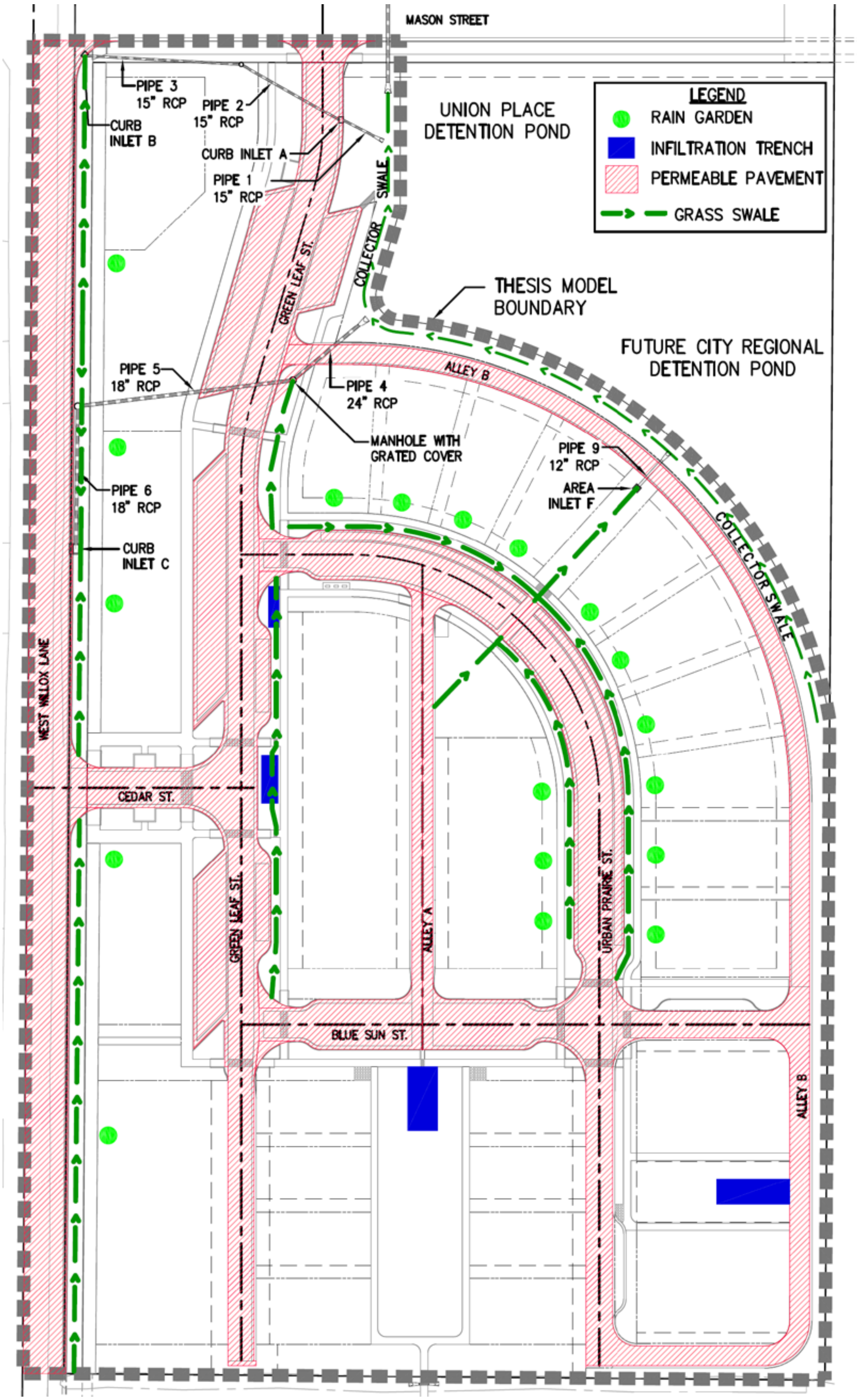


Figure 5.3-2 LID Model Layout

6. **LID 2 Model:** This model used the same BMP dimensions as the LID 1 model, but used the *partial infiltration* method from section 4.3. In this method, storage nodes were used to simulate the storage and infiltration of the permeable pavement and the infiltration trenches. This change from subcatchments to storage nodes allowed an underdrain system to be added. The underdrains were raised up 12 inches from the bottom of each storage node to provide a permanent infiltration volume and a detention storage volume. However, the rain gardens were modeled with the *full infiltration* method. This model also meets all of the stormwater requirements.

The reader should be cognizant that the LID 1 model is the only model designed to meet all of the requirements; the rest of the models are variations of the LID 1 model in order to understand the effects of changing or removing various elements. Also, the BMPs in the LID 2 model and the PP model were based on the BMP dimensions of the LID 1 model (the PP model did not have infiltration trenches or rain gardens).

5.3.2 Modeled Scenarios

The models were analyzed with precipitation data from two different locations and two different soils to better understand the effectiveness of LID practices. Fort Collins rainfall was selected because that is where the site is located, and Atlanta rainfall was selected because it has significantly higher amounts of rainfall including very intense storms (opposed to Seattle, which has high amounts of annual rainfall, but generally lower intensity storms). A high and a low infiltration soil were selected; the CTL Thompson soil test results were used for the high infiltration soil and a generic NRCS type 'C' soil was selected as the low infiltration soil.

Two different types of rainfall were simulated at each location: design storm and historical rainfall. A design storm simulation is a single synthetic rainfall event that is constructed from local precipitation patterns and run through the model; the duration of a design storm may vary from one hour to one day. A historical rainfall simulation (often referred to as a ‘continuous simulation’) is a historical rainfall record from a rain gauge that is run through the model; the duration of a continuous simulation can be as long as the rainfall record, generally between 10 and 70 years.

The LID 2 model was not simulated with the high infiltration soil because high infiltration soils generally are more than capable of handling large amounts of water and underdrains are unnecessary.

Table 5.3-2 presents the combinations of modeled scenarios.

Table 5.3-2 Modeled Scenarios

	Fort Collins				Atlanta			
	Design Storm		Continuous Simulation		Design Storm		Continuous Simulation	
Models	High Inf. Soil	Low Inf. Soil	High Inf. Soil	Low Inf. Soil	High Inf. Soil	Low Inf. Soil	High Inf. Soil	Low Inf. Soil
Pre Development:								
DCIA	X	X	X	X	X	X	X	X
NDCIA			X	X			X	X
Traditional Curb & Gutter:								
Uncontrolled	X	X	X	X	X	X	X	X
Normal Control	X	X	X	X	X	X	X	X
Over Control	X	X	X	X	X	X	X	X
Swales	X	X	X	X	X	X	X	X
Permeable Pavement	X	X	X	X	X	X	X	X
LID 1	X	X	X	X	X	X	X	X
LID 2		X		X		X		X

In order to shorten the lengthy scenario names, e.g. ‘Fort Collins Permeable Pavement Low Infiltration Soil’ scenario, they will be referred to in this manner: ‘FC-PP-L’. The ‘-L’ on the end indicates the low infiltration soil.

5.3.3 BMP Design

The following criteria were used to size the LID BMPs in the models.

Water quality was accomplished in the LID models by routing every impervious area through a LID BMP. A separate WQCV was determined for Fort Collins and for Atlanta using the UDFCD method assuming a 40-hour drain time (UDFCD, 2007). Wherever possible, the WQCV was addressed with rain gardens or infiltration trenches (the rain gardens and infiltration trenches were sized only to address water quality). Where rooftops drained directly onto pavement (e.g. residential alleys), the WQCV was included in the permeable pavement storage reservoirs, however the pavement reservoir sizing was controlled by the 100-year storm.

To control the 100-year storm, the permeable pavement storage reservoirs and stormwater conveyance channels were sized by iterative running of the SWMM model. The method for calculation of specific SWMM parameters for these BMP's was explained in sections 4.2 and 4.3. The underdrain orifices in the LID 2 model were sized so that each storage volume would drain in more than 6 and less than 24 hours (Hunt & Collins, 2008; UDFCD, 2007).

Once the BMPs were sized using the design storms, the continuous simulations were run to analyze the performance of the models with 60 years of historic rainfall data.

5.4 Model Input and Parameters

This section presents the parameters and values input to the SWMM model including precipitation, evaporation, and infiltration rates.

5.4.1 Precipitation and Evaporation Data

The climate in Atlanta, GA is wetter than Fort Collins, CO. Atlanta receives on average 48.4 inches of rainfall per year, compared with 13.2 inches in Fort Collins (from analysis of

NCDC data in section 5.4.1.2). The average storm depth is also larger in Atlanta with 0.70 inches, compared with 0.43 in Fort Collins (WEF & ASCE, 1998).

5.4.1.1 Design Storms

The design storms for Fort Collins were obtained from the City of Fort Collins Storm Drainage Design Criteria (1997). They were developed according to the method prescribed by the UDCFD *Manual* (2007), in which a one-hour design rainfall depth is distributed over two-hours. The Atlanta design storms were created using a 24-hour SCS Type II distribution (USDA, 1986, 1992). Rainfall depths to scale the Type II distribution were obtained from Technical Paper 40 (USDC, 1961).

Table 5.4-1 shows the differences between the Fort Collins and Atlanta design storms. The Fort Collins storms, which are 2-hours, have much less total depth than the Atlanta storms, which are 24-hours. Peak intensities of the 2- and 10-year Atlanta storms are about twice that of the Fort Collins storms, but the 100-year peak intensities are similar. Take note that the storm control requirements, listed in Table 5.1-1, state that this model will control the 2- and 100-year peak flow rates; the 10-year design storm is presented and used for comparison purposes only. Figure 5.4-1 on the next page graphically presents the design storms.

Table 5.4-1 Design Storm Comparison

		Design Storm		
		2-Year	10-Year	100-Year
Fort Collins, CO (prescribed storm depth for 2 hr. design storm)	Depth *(in)*	0.98	1.71	3.67
	Peak Intensity *(in/hr)*	2.85	4.87	9.95
Atlanta, GA (24 hr. storm)	Depth *(in)*	4.0	6.0	8.0
	Peak Intensity *(in/hr)*	5.48	8.22	10.97

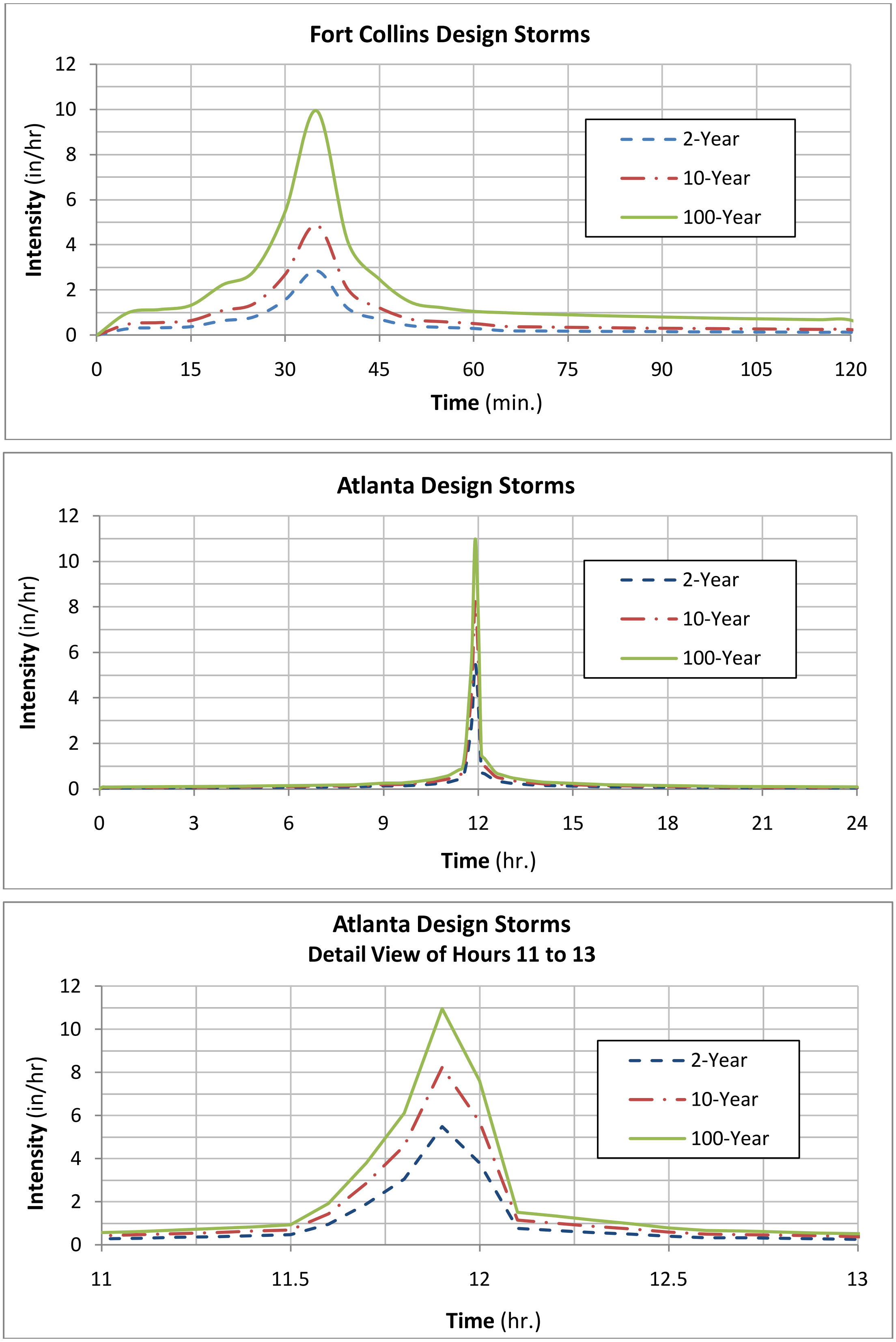


Figure 5.4-1 Design Storms

5.4.1.2 Continuous Simulation Rainfall Data

Historical rainfall data for both locations was obtained from the National Climatic Data Center (NCDC). 15-minute data was desired since it better represents the peaks and variations of actual rainfall and because the time of concentration of the site is less than 10 minutes. However, 15-minute data quality for both locations was found to be unsatisfactory; the available data was either missing too many recordings, the precision of the data was too low, or it did not exist in a 15-minute format. To remedy this, the continuous simulation rainfall data were acquired by disaggregating quality 1-hour rainfall data into 15-minute data, as described below.

The 1-hour data was disaggregated to 15-minute data using NetSTORM (Heineman & Prasad, 2009). The NetSTORM software utilizes a stochastic algorithm and a user-defined *spiking factor* to divide 1-hour rainfall depths into four 15-minute depths of varying magnitude with the same total volume as the 1-hour rainfall. The spiking factor is the user's control on how hard the resultant 15-minute data is spiked. To determine how the spiking factor should be set, an intensity-duration-frequency (IDF) analysis was performed on the disaggregated data and then compared against published IDF charts (Atlanta Regional Commission, 2001; City of Fort Collins, 1997). A *spiking factor* of 0.8 for Atlanta and 0.7 for Fort Collins provided satisfactory results.

The Fort Collins data were obtained from the 'Ft. Collins' gauge, station #053005, and data for Atlanta were obtained from the 'Atlanta Hartsfield International Airport' gauge, station #090451. They were obtained in the 1-hour format for the period of record from 1949 to 2009. The year 1973 was missing from the Fort Collins data.

Table 5.4-2 presents an IDF chart of the disaggregated rainfall data. A comparison of the disaggregated data with the raw 1-hour data and with published IDF charts can be found in APPENDIX A – IDF Comparison of Disaggregated Rainfall Data.

Table 5.4-2 Disaggregated Rainfall IDF Chart

	Storm Duration (hrs)	Return Period (years) 1	2	5	10	25	50	100
				Rainfall Intensity (in/hr)				
Fort Collins Disaggregated Data	0.5		1.26	1.84	2.38	3.24	4.06	5.08
	1		0.85	1.23	1.57	2.13	2.66	3.32
	2		0.50	0.73	0.94	1.28	1.61	2.02
Atlanta Disaggregated Data	0.5	2.24	2.6	3.1	3.54	4.26	4.96	5.84
	1	1.41	1.69	2.05	2.35	2.83	3.28	3.82
	2	0.81	0.99	1.21	1.40	1.69	1.97	2.29
	6	0.37	0.44	0.52	0.58	0.68	0.76	0.86
	12	0.21	0.26	0.32	0.36	0.43	0.49	0.55
	24	0.13	0.16	0.19	0.22	0.27	0.31	0.37

5.4.1.3 Evaporation Data

Design storm simulations typically neglect evaporation as it is not significant over the course of a few hours to few days. However over the course of years, as is the case in a continuous simulation, evaporation is very significant to the overall water budget (Gironás, et al., 2009). Evaporation is also the only mechanism to deplete the depression storage of impervious areas between rainfall events.

Pan evaporation data for Fort Collins were obtained from the Western Regional Climate Center (WRCC, 2009). Data for Atlanta were obtained from the NOAA Technical Report NWS 34 (Farnsworth & Thompson, 1982). The pan evaporation data were converted to natural surface evaporation data by multiplying the pan rate by 0.70, as described in the EPA SWMM Applications Manual (Gironás, et al., 2009). Table 5.4-3 shows the evaporation rates used in the continuous simulations.

Table 5.4-3 Evaporation from Natural Surfaces

Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	**Year**
Fort Collins												
					Monthly Average (in)							
0.00	0.00	1.75	3.16	3.79	4.42	4.84	4.25	3.32	2.15	1.04	0.00	**28.73**
					Average per Day (in./day)							
0.00	0.00	0.06	0.11	0.12	0.15	0.16	0.14	0.11	0.07	0.03	0.00	
Atlanta												
					Monthly Average (in.)							
1.48	1.91	3.00	4.05	4.92	4.97	4.95	4.69	3.65	2.90	2.02	1.58	**40.12**
					Average per Day (in./day)							
0.05	0.07	0.10	0.13	0.16	0.17	0.16	0.15	0.12	0.09	0.07	0.05	

Data corrected from pan evaporation rates by multiplying by 0.70.

5.4.2 Infiltration Rates

The onsite soils test yielded a design infiltration rate of 1.0 in./hr. (CTL Thompson, 2009) corresponding to a high ‘B’ or low ‘A’ NRCS soil type, and was used for the high infiltration soil. A generic NRCS type ‘C’ soil was selected for the low infiltration soil. Table 5.4-4 displays the infiltration rates used in the model; the Horton infiltration equation was used.

Table 5.4-4 Soil Infiltration Rates for Horton Equation

Model Soil Category	**Corresponding NRCS Soil Group**	**Infiltration Rate** *(in./hr.)*		**Decay Coefficient** *(1/hr.)*	**Drying Time** *(days)*
		Initial – f_i	Final – f_0		
High Inf. Soil	Low A or High B	3.0	1.0	4	7
Low Inf. Soil	C	3.0	0.25	4	7

The infiltration rates and drying time values were obtained from the UDFCD *Manual* (2007) and *Urban Runoff Quality Management* (WEF & ASCE, 1998). Selection of the decay coefficient was based on results from a dissertation on rainfall-runoff modeling in urban areas, which found that proper modeling of peak flow rates requires a decay coefficient slightly lower than 4 *1/hr* (Rivas-Acosta, 2009). Figure 5.4-2 shows the infiltration rate decay of the modeled soils using the simple form of the Horton equation.

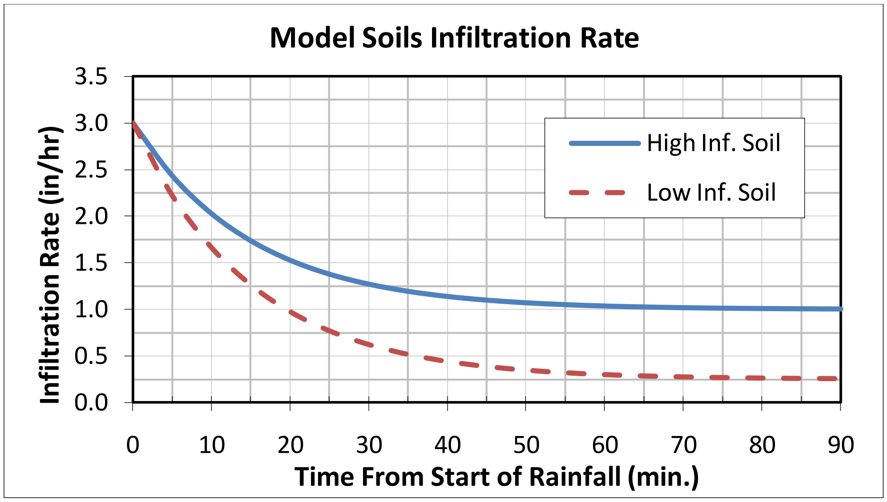


Figure 5.4-2 Model Soils Infiltration Rate Decay

5.4.3 Existing Conditions

5.4.3.1 Assumptions

It was assumed that the overland flow length could be reasonably estimated as the length of the field from north to south, a length of 400 feet, and that the flow does channelize before it leaves the field to the south east. With existing soil conditions, there was no runoff from most storms, in order to increase flow rates to reasonable values, 6% directly connected impervious area was added, which is typical.

5.4.3.2 Model Parameters

Table 5.4-5 Undeveloped Model Parameters

Parameter	Value	Source
Depression Storage		
Pervious	0.20 *in*	EPA SWMM help (Rossman, 2008)
Impervious	0.05 *in*	EPA SWMM help (Rossman, 2008)
Hydraulic Roughness (*Mannings n)*		
Overland Flow		
Undeveloped – Native Grass	0.2	UDSWM Manual (UDFCD, 2001)
Channelized Flow		
Farm Swale	0.054	EPA SWMM help (Rossman, 2008)

5.4.4 Developed Conditions

5.4.4.1 Assumptions

The major assumption in the PP and LID models is that the rate of infiltration through the permeable pavement and infiltration trench surfaces is not limiting. In other words, runoff does not occur from these devices until the storage capacity has been exceeded. Other assumptions are summarized in Table 5.4-6.

Table 5.4-6 Developed Model Assumptions

1. The subgrade is terraced to maximize infiltration and provide rectangular storage volumes.
2. Free outfall at the bottom of the development.
3. Rooftop slopes are 30% and lot slopes are 2%. A weighted average by area was used to determine each subcatchment slope.
4. All rain gardens are on the subcatchment level (opposed to in the conduit system) and the roof drains are directed towards the rain garden.
5. The effects of field capacity (capillary suction) in the storage reservoirs of permeable pavement, infiltration trenches, and rain gardens are negligible.
6. The underdrain orifice in the LID 2 model controls the outflow of the storage reservoir.

5.4.4.2 Model Parameters

Table 5.4-7 summarizes the general parameters used in the developed models. The overall percent impervious of the developed model is 66%; this does not account for reductions for permeable pavement.

Table 5.4-8 presents the detention pond depth-area relationship which was taken directly from the Nolte Design. Finally, the conduit cross sections, which were also developed from the Nolte design, are shown in APPENDIX B – Conduit Cross Sections.

Table 5.4-7 Developed Model Parameters

Parameter	Value	Source
Depression Storage		
Pervious	0.20 *in*	EPA SWMM help (Rossman, 2008)
Impervious	0.05 *in*	EPA SWMM help (Rossman, 2008)
% Impervious w/ Zero Dep. Storage		
Lots (roofs, driveways, sidewalks)	100%	(Roesner, 2009)
Streets	25%	SWMM Applications Manual (Gironás, et al., 2009)
Hydraulic Roughness (*Mannings n)*		
Overland Flow		
Asphalt	0.02	UDSWM Manual (UDFCD, 2001)
Lawns	0.25	UDSWM Manual (UDFCD, 2001)
Permeable Pavement	0.03	(James, 2001)
Rooftops	0.014	(Novotny, 2003)
Smooth Paving	0.014	(Novotny, 2003)
Channelized Flow		
Concrete Pipe	0.015	EPA SWMM help (Rossman, 2008)
Concrete Curb Gutter	0.016	(UDFCD, 2007) pg. ST-8
Cross Pan	0.016	(UDFCD, 2007) pg. ST-8
Grass Swale - Large	0.054	(UDFCD, 2007) pg. MD-85
Grass Swale – Small	0.14	(UDFCD, 2007) pg. MD-85
Underdrain Pipe (PVC)	0.015	EPA SWMM help (Rossman, 2008)
Slope		
Lot Slope	2%	Typical Slopes are taken from the Nolte Design. And from standard practice in engineering.
Rooftop Slopes	30%	
Street Cross Slope	2%	
Street Longitudinal Slope	0.6%	
Swale Longitudinal Slope	0.6%	
Porosity of Open Graded Base	0.3	(Ferguson, 2005; Smith, 2006)

Table 5.4-8 Detention Pond Depth Area Curve

Depth *ft*	Elevation Area ft^2
0	0
0.83	32,524
1.83	45,734
2.83	50,384
3.83	55,166

6.0 RESULTS

This chapter presents and discusses the SWMM modeling results of the scenarios described in chapter 5.0.

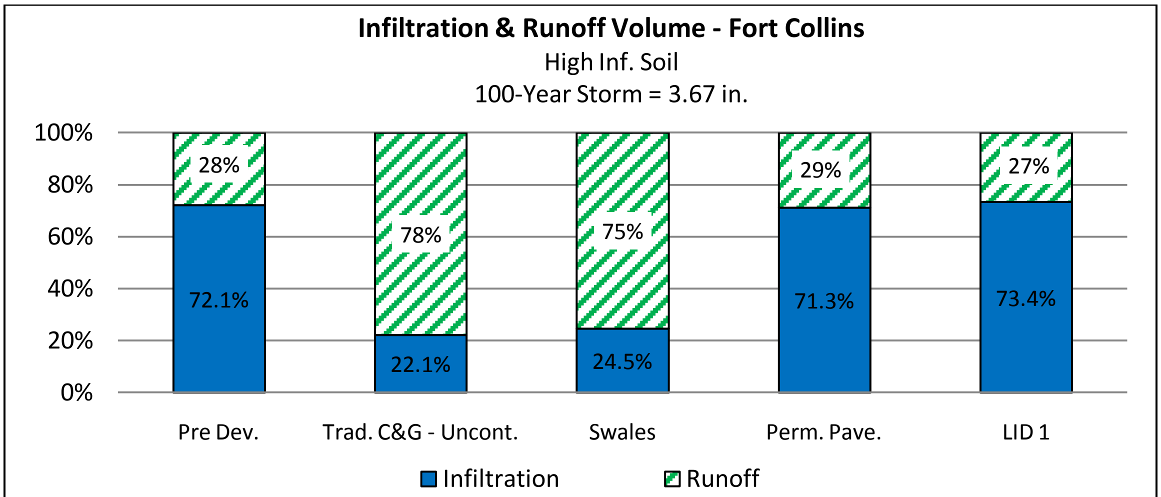
6.1 Fort Collins Design Storm Results and Discussion

Because this study attempted to control the 100-year design storm with LID, the 100-year results are presented in this section; the 2- and 10-year results can be found in APPENDIX C – Fort Collins Design Storm Results. Table 6.1-1 presents the resulting peak flow rates, water balances, and pond sizes for the Fort Collins 100-year design storm scenarios. Figure 6.1-1 displays the water balance in graphical form and Figure 6.1-2 shows the 100-year storm hydrographs.

Table 6.1-1 Tabular Fort Collins 100-Year Design Storm Results

100-Year Design Storm Depth = 3.67 in.						
Models	Peak Discharge *cfs*	Runoff Volume *in.*	Infiltration Volume *in.*	% Runoff	% Infiltrate	Detention Pond Size *ac.*
			High Infiltration Soil			
Pre Dev.	9.8	1.02	2.64	27.9%	72.1%	
Trad. C&G:						
Uncontrolled	54.1	2.85	0.81	77.7%	22.1%	
BMP + Normal Control	9.7					1.13
BMP + Over Control	1.2					1.20
Swales	42.5	2.76	0.90	75.3%	24.5%	
Perm. Pavement	8.1	1.06	2.62	28.9%	71.3%	
LID 1	7.6	0.98	2.69	26.8%	73.4%	

100-Year Design Storm Depth = 3.67 in.						
Models	Peak Discharge *cfs*	Runoff Volume *in.*	Infiltration Volume *in.*	% Runoff	% Infiltrate	Detention Pond Size *ac.*
			Low Infiltration Soil			
Pre Dev.	11.9	1.98	1.69	53.9%	46.1%	
Trad. C&G:						
Uncontrolled	55.8	3.19	0.47	87.0%	12.9%	
BMP + Normal Control	11.9					1.13
BMP + Over Control	1.2					1.20
Swales	43.8	3.18	0.49	86.6%	13.2%	
Perm. Pavement	13.4	1.67	2.01	45.6%	54.7%	
LID 1	11.6	1.60	2.08	43.5%	56.7%	
LID 2	11.0	1.85	1.81	50.4%	49.3%	



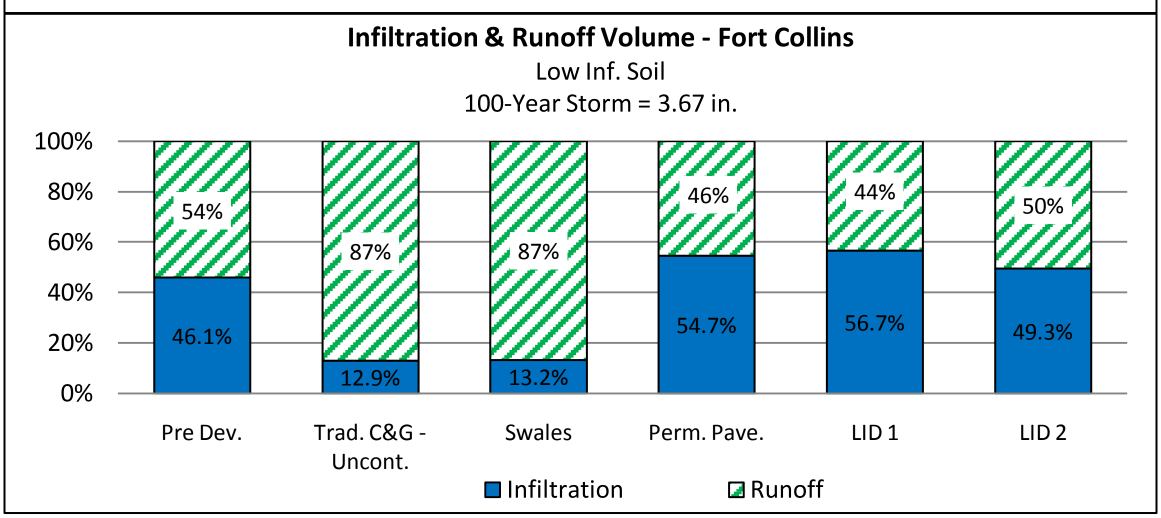


Figure 6.1-1 Water Balance Graphs – Fort Collins

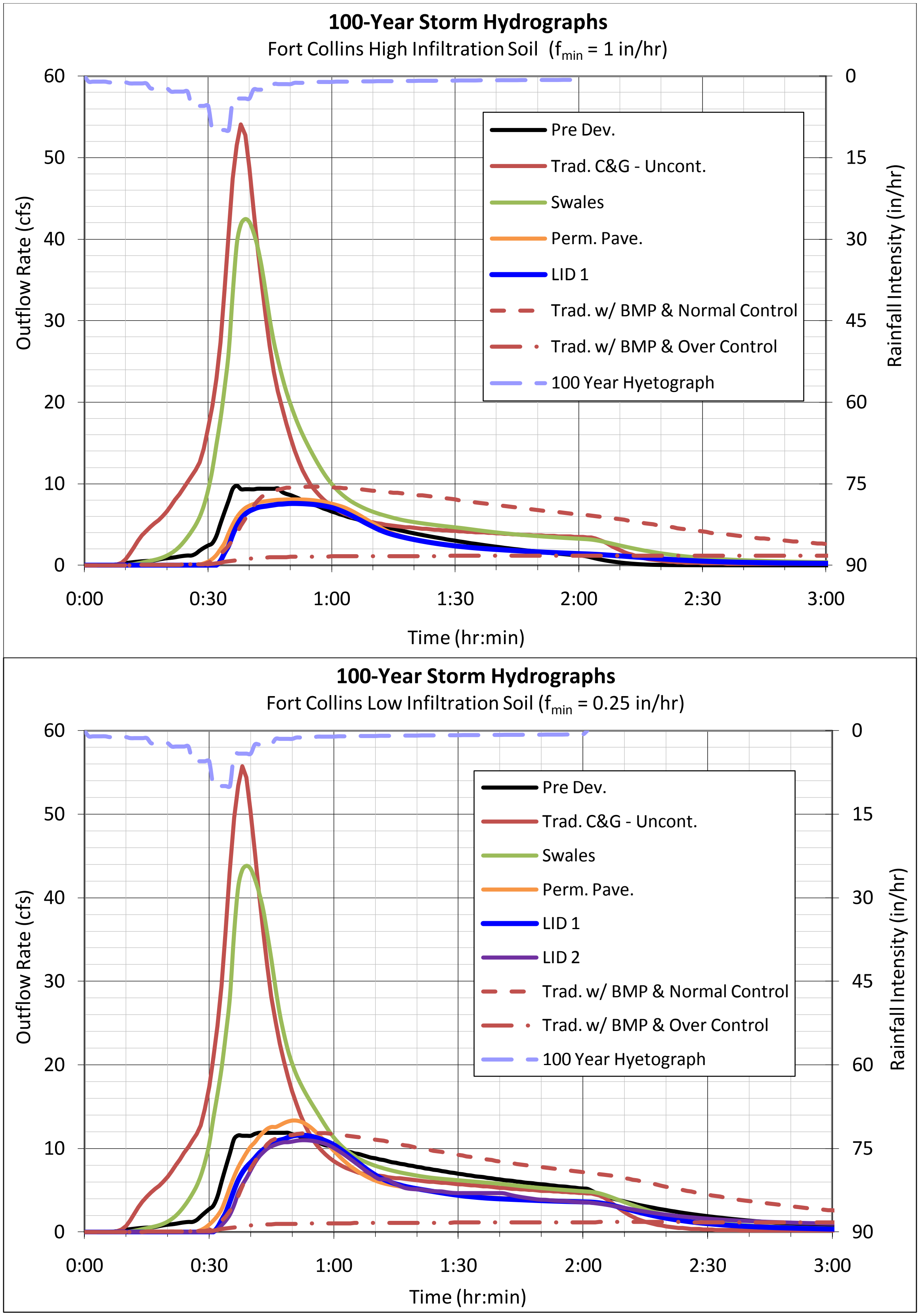


Figure 6.1-2 100-Year Design Storm Hydrographs – Fort Collins

As shown, LID was able to restore the predevelopment peak flow rate, water balance, and meet the stormwater requirements for the 2-year and 100-year design storms in Fort Collins (the 2- and 10-year results are found in APPENDIX C – Fort Collins Design Storm Results). The 10-year storm was not a model requirement, however the LID models did maintain the 10-year predevelopment peak flow rate as well. The 10-year water balance was not met by just a few percent; if desired, the size of the rain gardens and infiltration trenches could be slightly increased and this criteria would be met. The 10-year design storm is presented and used for comparison and reference purposes only.

The LID 1 model was the only model specifically designed to meet all of the stormwater requirements, the other models were variations of the LID 1 model. Table 6.1-2 presents the performance of each model against the thesis stormwater requirements.

Table 6.1-2 Fort Collins Models – Attainment of 100-Year Storm Requirements

Models	Meets Water Quality Requirements	Meets Water Balance Requirements	Meets Peak Flow Requirements
Trad. C&G:			
Uncontrolled	No	No	No
BMP + Normal Control	Yes	No	Yes
BMP + Over Control	Yes	No	Yes
Swales	No	No	No
Perm. Pavement	No[1]	High: No/ Low: Yes	High: Yes/ Low: No
LID 1	Yes	Yes	Yes
LID 2	Yes	Yes	Yes

[1] The Permeable Pavement model captures and treats much more than the WQCV, but does not capture the first flush from impervious areas not draining over permeable pavement.

These figures illustrate how the volume and peak flow rate of runoff are greatly increased by development with traditional drainage practices and no detention. The runoff volume was increased on average by 2 times in the 100-year storm and 10 times in the 2-year. The uncontrolled peak flow rate was increased on average by 5 times for the 100-year storm and 9 times for the 2-year storm. Detention ponds, when designed correctly, are effective at reducing these peak flows to historic rates, but do not reduce the excess runoff volume.

On the other hand, the LID hydrographs followed the predevelopment hydrographs reasonably well for the 100–year storm, and for the higher frequency storms they had much lower flow rates than predevelopment (see APPENDIX C – Fort Collins Design Storm Results). The reason for this is because the permeable pavement storage reservoirs were sized to control the 100-year storm, which resulted in higher than needed flow rate reduction for the more frequent storms; in fact, the LID model 2-year storm scenarios almost had no outflow. The inadvertent "over-control" of 2- and 10-year peak flow rates with LID may not be totally desirable as occasional flushing flows are thought to be helpful for streams.

It is interesting that both the LID 1 and LID 2 models performed very similarly for the Fort Collins design storms. The low infiltration graph in Figure 6.1-2 shows that both LID hydrographs are nearly on top of each other. This is insightful because these two models used very different methods to model LID. The underdrains in the LID 2 model resulted in more runoff than the LID 1 model which was expected, but only a small difference in peak flow rate. This validates that both of these methods work to model LID.

In the LID1-H scenario, the water balance instead of peak flow rate controlled the pavement reservoir sizing. This resulted in the LID1-H 100-year peak flow rate being 22% less than the predevelopment rate, as seen in Figure 6.1-2. The water balance is very different between the high and low infiltration soil scenarios. The volume of runoff in the Predev-DCIA-L scenario was 1.98 inches or 54%, but in the Predev-DCIA-H scenario it was 1.02 inches or 28%, this is a difference of 48%. This difference led to more permeable pavement storage being needed in the LID High scenario than the Low scenario, which is further explained in section 6.3.

The detention ponds for the Trad. drainage models required 1.13 acres and 1.20 acres for the NC and OC scenarios respectively. The LID models did not require the detention pond land that the traditional models did and were also able to restore predevelopment peak flow rates. A cost benefit analysis of this tradeoff is presented in Chapter 7.0.

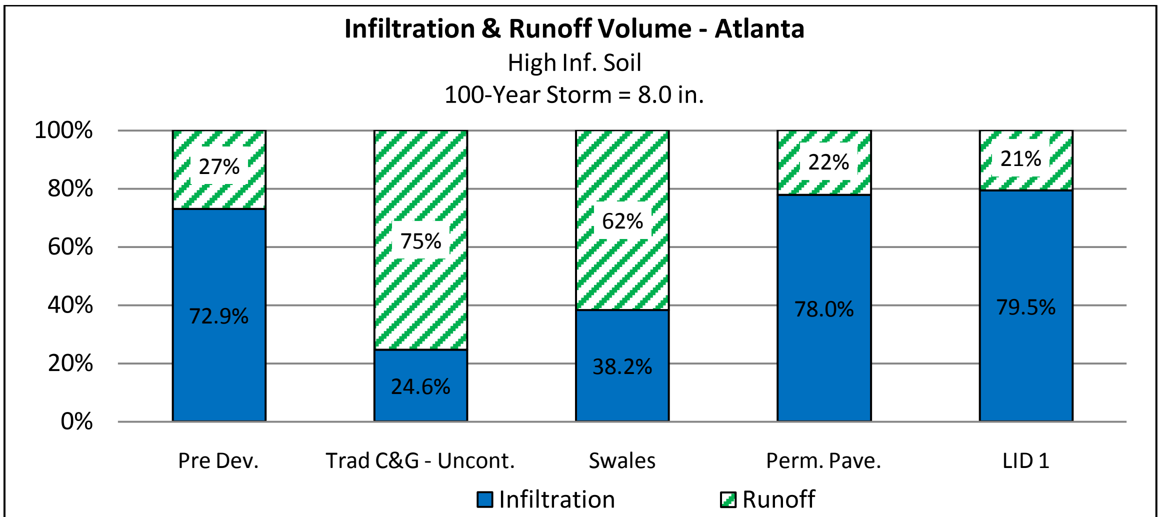
6.2 Atlanta Design Storm Results and Discussion

Because this study attempted to control the 100-year design storm with LID, the 100-year results are presented in this section; the 2- and 10-year results can be found in APPENDIX D – Atlanta Design Storm Results. Table 6.2-1 presents the simulated peak flow rates, water balances, and pond sizes for the Atlanta 100-year design storm scenarios. Figure 6.2-1 displays the water balance in graphical form and Figure 6.2-2 shows the 100-year storm hydrographs.

Table 6.2-1 Tabular Atlanta 100-Year Design Storm Results

100-Year Design Storm Depth = 8.00 in.						
Models	Peak Discharge *cfs*	Runoff Volume *in.*	Infiltration Volume *in.*	% Runoff	% Infiltrate	Detention Pond Size *ac.*
			High Infiltration Soil			
Pre Dev.	24.3	2.17	5.83	27.1%	72.9%	
Trad. C&G:						
Uncontrolled	64.4	6.03	1.97	75.3%	24.6%	
BMP + Normal Control	24.1					1.20
BMP + Over Control	5.8					1.25
Swales	53.0	4.94	3.06	61.7%	38.2%	
Perm. Pavement	24.9	1.78	6.24	22.2%	78.0%	
LID 1	24.2	1.65	6.36	20.7%	79.5%	

100-Year Design Storm Depth = 8.00 in.						
Models	Peak Discharge *cfs*	Runoff Volume *in.*	Infiltration Volume *in.*	% Runoff	% Infiltrate	Detention Pond Size *ac.*
			Low Infiltration Soil			
Pre Dev.	34.7	4.03	3.97	50.4%	49.6%	
Trad. C&G:						
Uncontrolled	64.9	6.59	1.41	82.3%	17.6%	
BMP + Normal Control	34.6					1.20
BMP + Over Control	9.8					1.25
Swales	54.4	6.20	1.80	77.5%	22.5%	
Perm. Pavement	34.8	3.24	4.77	40.5%	59.6%	
LID 1	34.4	3.12	4.89	39.0%	61.1%	
LID 2	17.7	3.72	4.27	46.5%	53.4%	



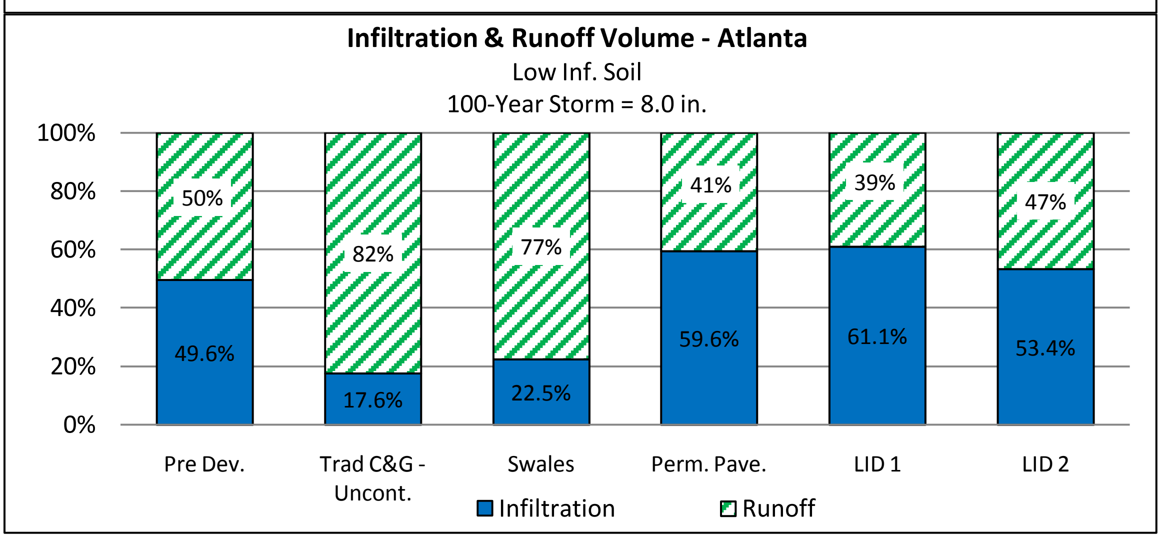


Figure 6.2-1 Water Balance Graphs - Atlanta

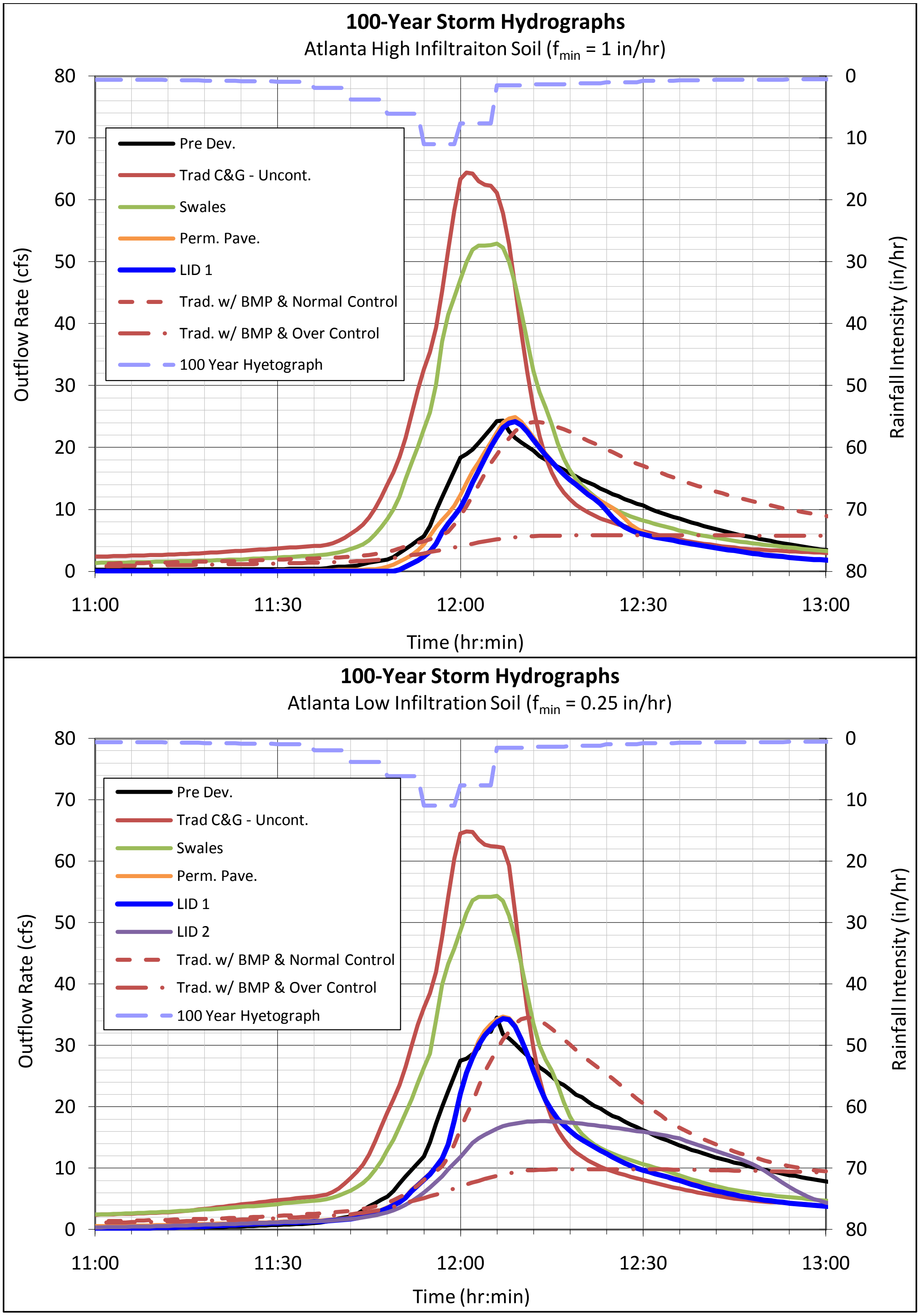


Figure 6.2-2 100-Year Design Storm Hydrographs - Atlanta

LID in the Atlanta location was also able to restore the predevelopment peak flow rate, water balance, and meet the stormwater requirements for the 2-year and 100-year design storms (the 2- and 10-year results are found in APPENDIX D – Atlanta Design Storm Results). In this case, as opposed to Fort Collins, both the predevelopment water balance and peak flow rates for the 10-year design storm were met by both LID models. In fact, the PP model also met the water balance for all three design storms and would only need a little more storage to meet predevelopment peak flows. Table 6.2-2 presents the performance of each model against the thesis stormwater requirements.

Table 6.2-2 Atlanta Models – Attainment of 100-Year Storm Requirements

Models	Meets Water Quality Requirements	Meets Water Balance Requirements	Meets Peak Flow Requirements
Trad. C&G:			
Uncontrolled	No	No	No
BMP + Normal Control	Yes	No	Yes
BMP + Over Control	Yes	No	Yes
Swales	No	No	No
Perm. Pavement	No[1]	Yes	No[2]
LID 1	Yes	Yes	Yes
LID 2	Yes	Yes	Yes

[1] The Permeable Pavement model captures and treats much more than the WQCV, but does not capture the first flush from areas not draining over permeable pavement.

[2] The Permeable Pavement model would only need a little more pavement storage to meet the peak flow requirements.

The LID 1 model results replicated the predevelopment flow regime very well for all three design storms in Atlanta (see APPENDIX D – Atlanta Design Storm Results); the total volume, peak flow rate, time to peak, duration, and shape of the hydrographs were very similar. However in this location, the LID 2 model performed differently than the LID 1 model. The peak flow rates in the LID 2 model were much lower than both the predevelopment and the LID 1 peak rates. This is a result of the longer design storm in combination with the underdrains. The design storm length in Fort Collins was 2-hours but in Atlanta it was 24-hours. What happened in the

Atlanta LID 2 scenario is that during the 11 hours of slow rainfall before the storm peak, the water level in the pavement storage had reached the raised underdrains and the water was gradually let out as the rain continued. When the storm peaked at 11.9 hours there was more available storage in the LID 2 model than in the LID 1 model and the runoff peak received greater attenuation. This suggests that runoff models with underdrains and longer storms are sensitive to the rate at which water is let out through the underdrains.

The detention ponds for the Trad. models in Atlanta required 1.20 acres and 1.25 acres for the NC and OC scenarios respectively. The LID models did not require the detention pond land that the Trad. models did and were also able to restore predevelopment peak flow rates. A cost benefit analysis of this tradeoff is presented in Chapter 7.0.

6.3 Permeable Pavement Base Depths

The BMPs in the LID 1 model were sized to meet the post-development stormwater criteria. The BMPs in the LID 2 model and the PP model were then based on the exact same dimensions (the PP model did not have infiltration trenches or rain gardens). This section presents the resulting permeable pavement base depths. Figure 6.3-1 shows the locations of each permeable pavement zone and Table 6.3-1 summarizes the depths for each model and zone.

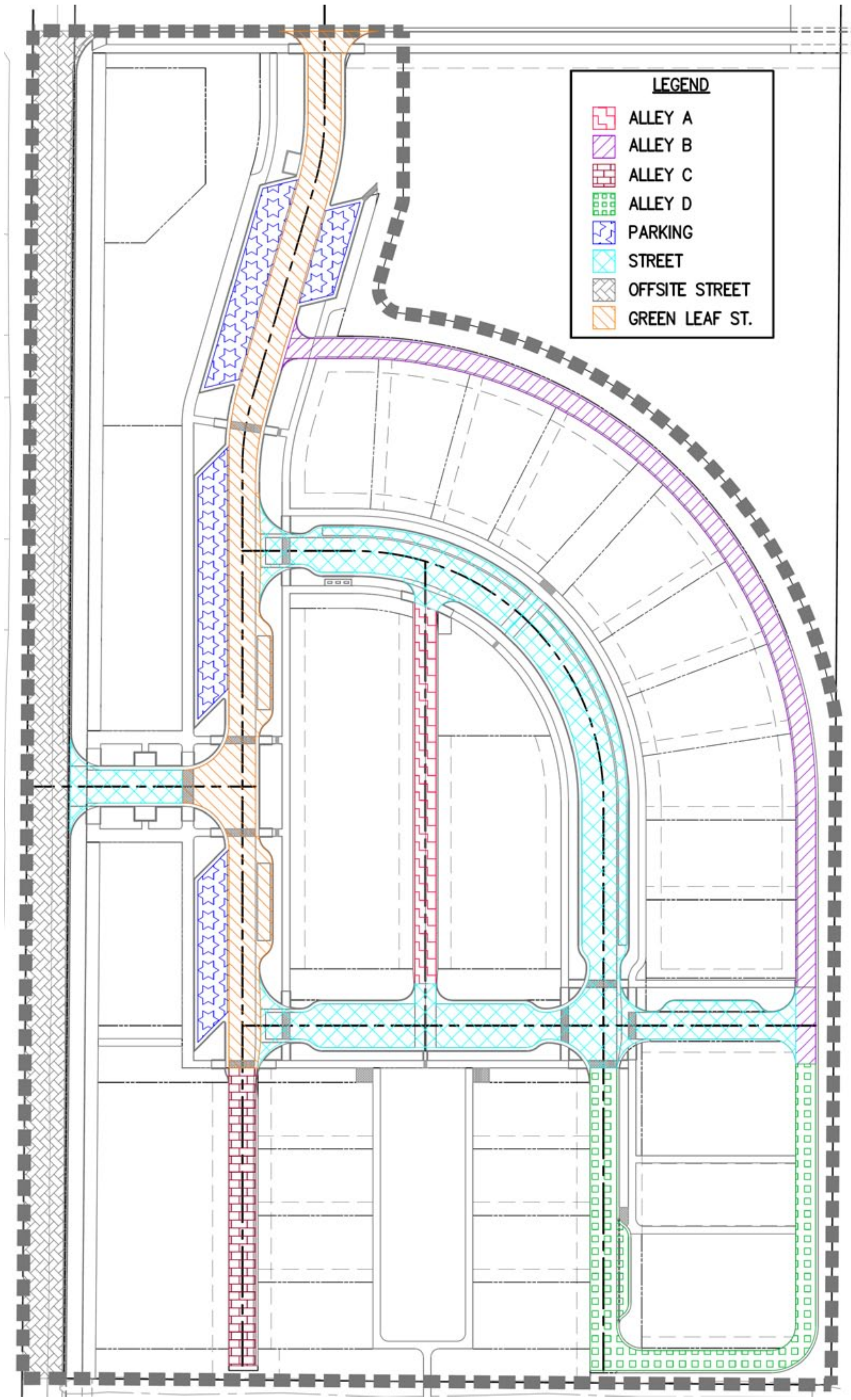


Figure 6.3-1 Permeable Pavement Zones

Table 6.3-1 Permeable Pavement Base Depths

Permeable Pavement Zone	Fort Collins High Inf. Soil in.	Fort Collins Low Inf. Soil in.	Atlanta High Inf. Soil in.	Atlanta Low Inf. Soil in.
ALLEY A	36	36	36	36
ALLEY B	36	36	36	36
ALLEY C	30	18	30	36
ALLEY D	30	18	30	36
PARKING	18	18	18	24
STREET	18	18	18	18
OFFSITE STREET	18	18	18	18
GREEN LEAF ST.	18	18	18	24
Controlled Sizing:	**Water Balance**	**Peak Rate**	**Peak Rate**	**Peak Rate**

Base porosity was 0.3.

As seen in the table above, the base storage depth in three of the four models was controlled by the peak rate requirement and not the water balance. The sole scenario where the water balance controlled was the FC-H models. In the three peak rate controlled models, a base depth of 18 inches in all pavement zones was more than enough to meet the water balance requirement.

Each model resulted in different amounts of base storage because, while the qualifiable stormwater requirements were the same for each model, the specific quantifiable stormwater requirements were different. The qualifiable requirements were to control the developed peak flow rate and the water balance back to predevelopment conditions. Different soils and different rainfall made the predevelopment peak flow and water balance different for each model. Thus each model had different specific requirements and subsequently needed different amounts of base storage to meet those requirements. Table 6.3-2 compares the predevelopment conditions.

Table 6.3-2 Comparison of Predevelopment Model Results

Models	Peak Discharge *cfs*	Runoff Volume *in.*	Infiltration Volume *in.*	% Runoff	% Infiltrate
		Fort Collins 100 Year Storm = 3.67 in.			
High Inf. Soil	9.8	1.02	2.64	27.9%	72.1%
Low Inf. Soil	11.9	1.98	1.69	53.9%	46.1%
		Atlanta 100 Year Storm = 8.00 in.			
High Inf. Soil	24.3	2.17	5.83	27.1%	72.9%
Low Inf. Soil	34.7	4.03	3.97	50.4%	49.6%

While the variation in the predevelopment conditions may account for much of the difference between the permeable pavement depths, the relationship between design storm length and infiltration decay rate also had an impact. When the Fort Collins design storms peak at 35 minutes after the start of rain, the soil infiltration rate has not fully decayed to the minimum value; whereas when the Atlanta design storms peak at 11.9 hours after the start of rain, the infiltration rate has been at a minimum value for many hours. This may explain why the FC-L models required the least amount of permeable pavement storage, but the ATL-L models required the most. It is also likely that the combined effects of the high infiltration soil and the short 2-hour design storm caused the FC-H model to be the only model where the water balance, not the peak rate controlled.

6.4 Continuous Simulation Results

This section presents the results of the continuous simulations. Sixty years of historical rainfall data, from 1949 to 2009, were run through each model for both Fort Collins and Atlanta locations (note, the Fort Collins data was missing 1973). This type of simulation analyzes the performance of the models over many years to see the full effect of the stormwater controls. The resulting flow data were analyzed in two ways, peak flow frequency and flow duration.

6.4.1 Peak Flow Frequency Curves – Results and Discussion

The peak flow frequency analysis picks the largest flow rate from each storm, using a 6-hour inter-event time, ranks them by magnitude, and then plots them against exceedance frequency on a chart. The step-by-step procedures used to produce these charts can be found in APPENDIX E – Continuous Simulation Graph Procedures.

Figure 6.4-1 and Figure 6.4-2 show the Fort Collins peak flow frequency results and Figure 6.4-3 and Figure 6.4-4 display the Atlanta peak flow frequency results.

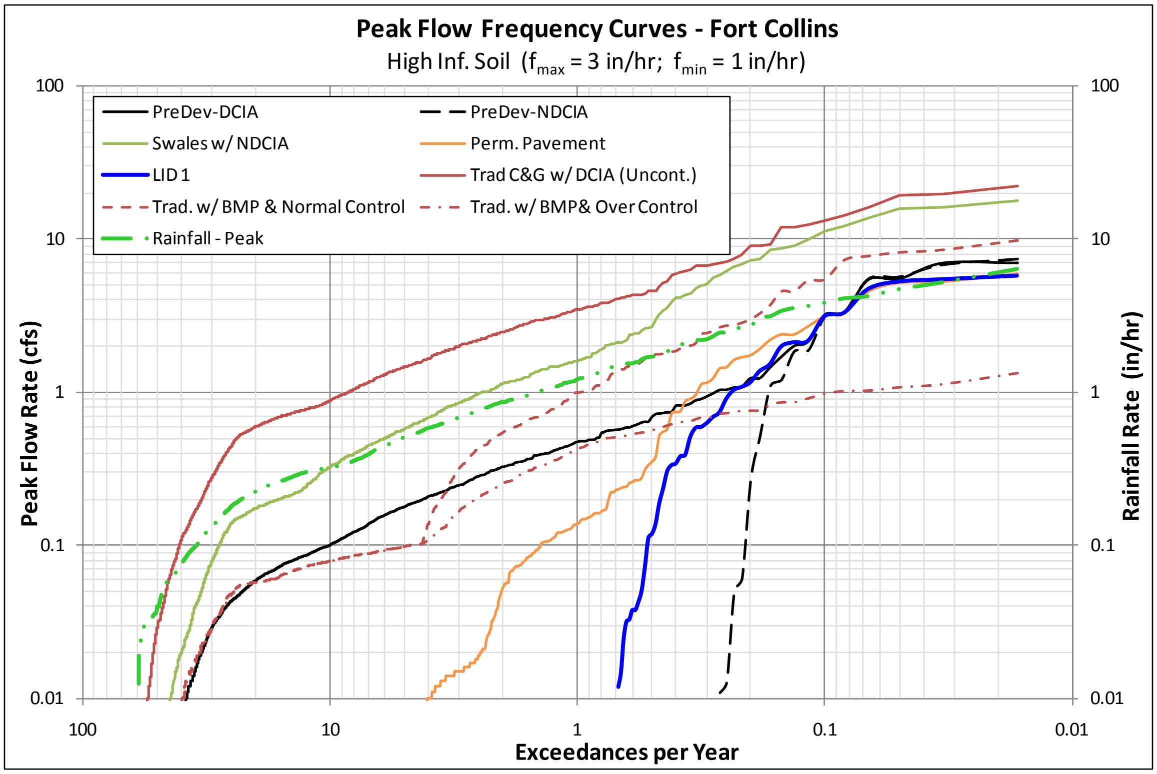


Figure 6.4-1 Peak Flow Frequency Curves – Fort Collins High Infiltration Soil

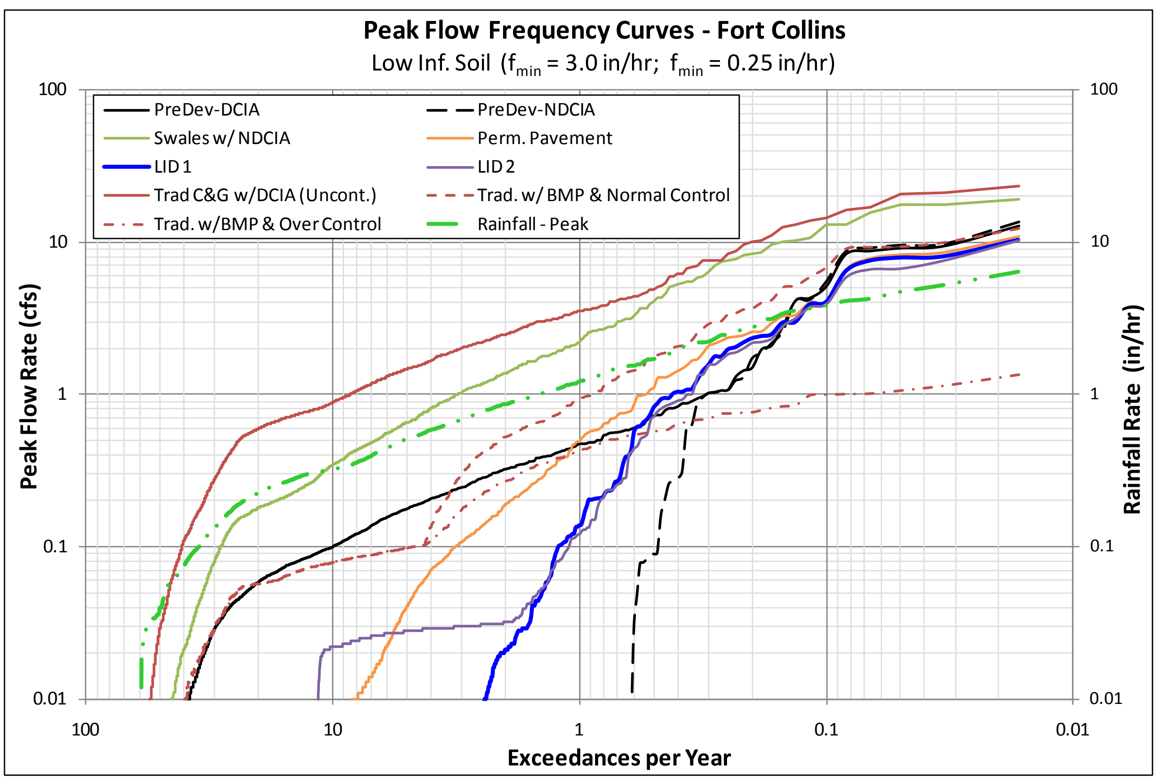


Figure 6.4-2 Peak Flow Frequency Curves – Fort Collins Low Infiltration Soil

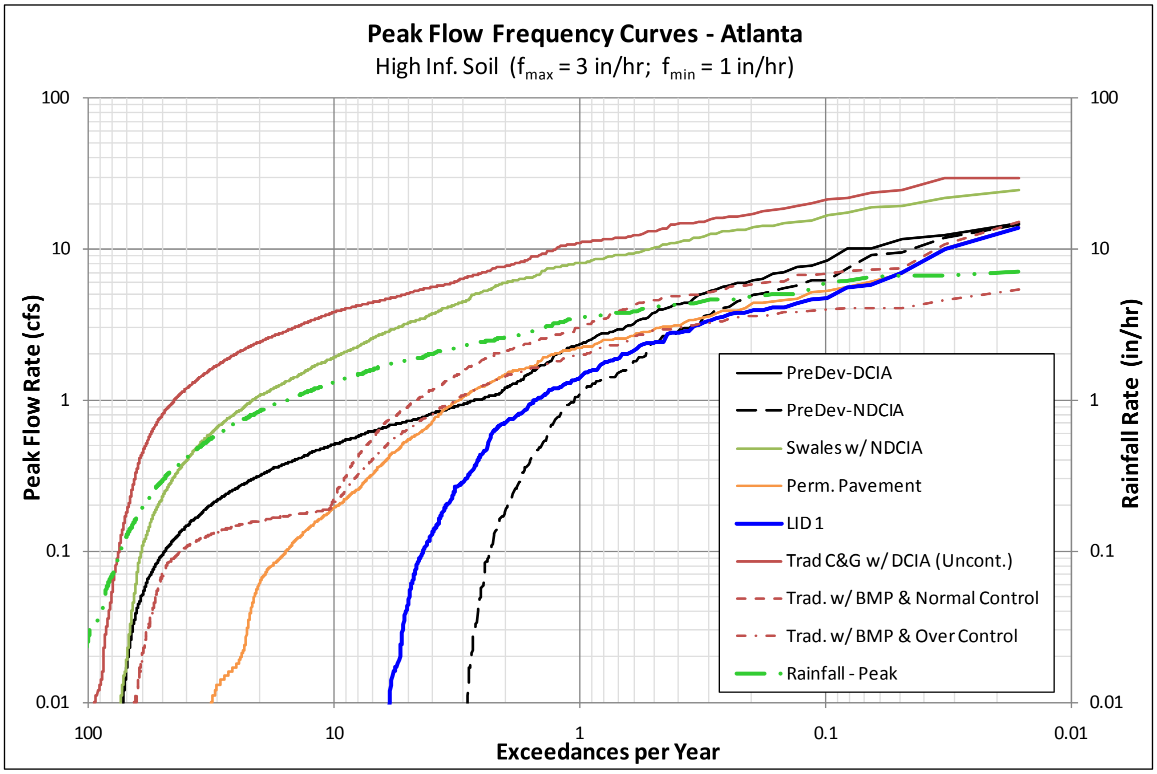


Figure 6.4-3 Peak Flow Frequency Curves – Atlanta High Infiltration Soil

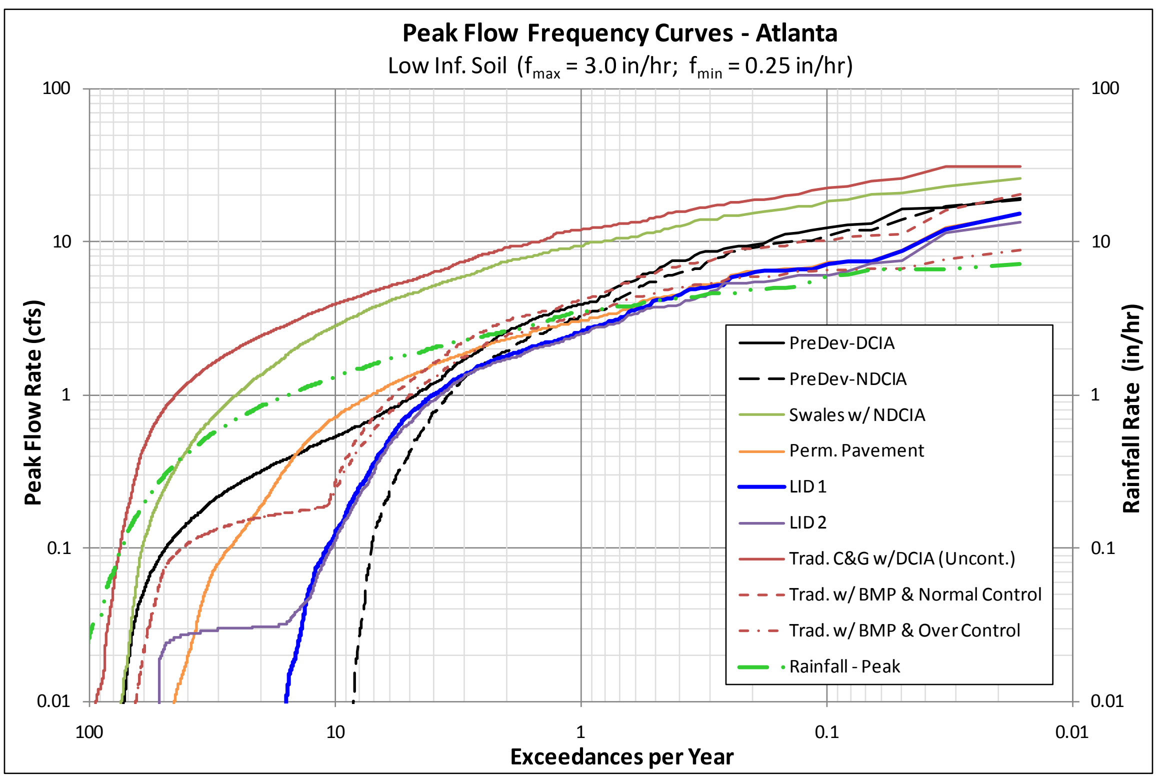


Figure 6.4-4 Peak Flow Frequency Curves – Atlanta Low Infiltration Soil

These peak flow frequency figures show that both LID models performed very well against the Predev. model curves. In fact for high frequency events, the LID curves split the difference between the Predev.-DCIA and NDCIA curves; for low frequency events, the LID curves tracked just lower and parallel to both Predev. curves. (As stated earlier, the predevelopment condition was assumed to have 6% impervious area and was modeled in the continuous simulations as both directly connected and non-directly connected.) That said, between the 2- and 5-year exceedance frequencies the FC-LID1-L and FC-LID2-L scenarios both produced peak flow rates slightly higher than predevelopment (see Figure 6.4-2). It is likely that this is caused by the discrepancy between design storm and continuous simulation peak flow rates. In the *design storm* simulations, both LID models were able to maintain the 2-, 10-, and 100-year peak flow rates. However, Figure 6.4-5 demonstrates how design storm and continuous simulation peak flow rates can be very different. The FC-Predev-DCIA-L design storm 10-year peak flow rate (2.4 cfs) was half that of the continuous simulation (5.2 cfs), and for the 2-year, the design storm peak flow rate (1.2 cfs) was twice that of the continuous simulation (0.7 cfs). Nehrke & Roesner (2002) explained this phenomenon in more detail.

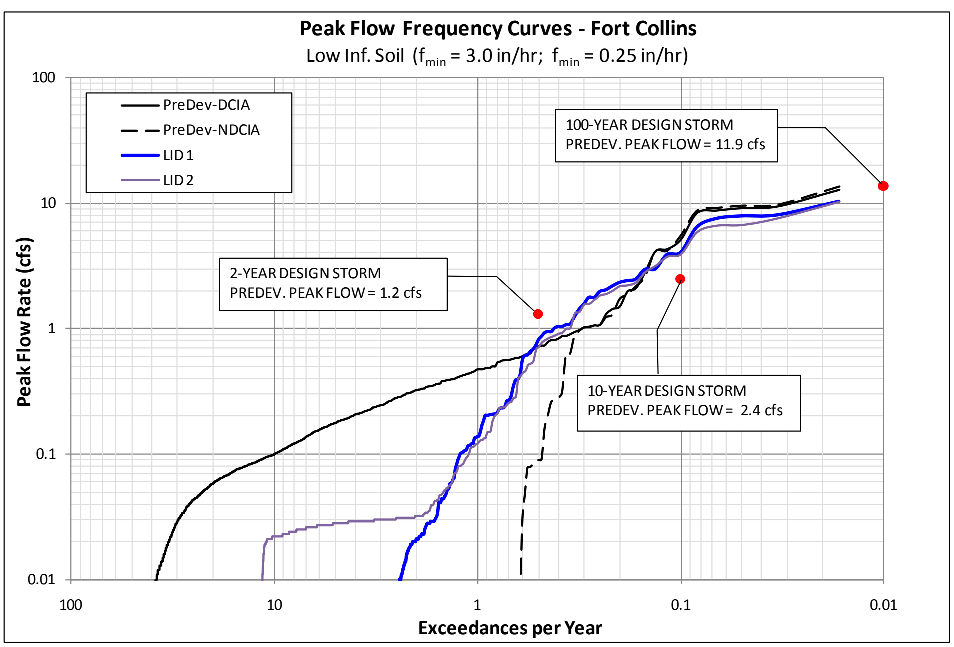


Figure 6.4-5 Design Storm vs. Continuous Simulation Peak Rate Differences

To remedy this in the FC-LID1-L and FC-LID2-L models, the infiltration trenches and rain gardens could be increased in size to attenuate more peak flow. Aside from this one issue, these graphs demonstrate that LID can effectively bring developed peak flows back to predevelopment conditions.

The effect of DCIA can be clearly seen in all of the peak flow frequency figures; using Fort Collins as an example, the Predev-NDCIA curve drops to zero flow sharply between 0.2 and 0.6 times per year while the Predev-DCIA curve continues on until 40 times per year before dropping to zero. The effect of the extended detention BMP can also be seen to left of the point where the Trad. w/NC and w/OC curves diverge from each other.

The LID 2 and PP models performed as expected. The LID 2 curve followed the LID 1 curve very closely for all but the very low flows. At low flows, the LID 2 curve plateaued at 0.03 cfs where the LID 1 continued to drop to zero; this is the effect of the raised underdrains in the LID 2 model which began flowing before surface flow commenced, producing a low flow at the outlet. The PP curve converged on the LID curves in the higher flows as expected. The only difference between the PP model and the LID 1 model, is that the LID 1 model has rain gardens and infiltration trenches for water quality which is why the PP curve diverged from the LID 1 curve at lower flows where the water quality size storms are.

The Swales model did not perform as expected, it's peak flow frequency curve tracked closer to the Uncontrolled Trad. model than expected and did not reduce the high frequency peak flows as much either. This likely happened for two reasons. First, infiltration in the swale conduits was not taken into account which is a short coming of the current SWMM software and no attempt was made to manually account for the swale infiltration. Second, the Swales model is not 100% NDCIA; this model employed the practice of NDCIA wherever possible, but some areas were not reasonable to treat as such. If it were possible to change these two things in the Swales model, the high frequency peak flows would be reduced.

The Uncontrolled Trad. peak flow frequency curve shows that the effects of development without stormwater controls are huge; the peak flow rates are increased over predevelopment between 2 and 10 times depending on return frequency. The effects of uncontrolled stormwater have been extensively discussed in literature and will not be further discussed here (Roesner, et al., 2001; Schueler, et al., 2009; WEF & ASCE, 1998). However the Trad. models with detention (NC and OC) tracked quite close to, or lower than, predevelopment for return frequencies greater than 1-year. Based on these peak flow frequency charts, it could be concluded that traditional stormwater controls are able to effectively address the hydrologic effects of development and that LID is not needed, however the changes to flow duration have not yet been examined.

6.4.2 Flow Duration Curves – Results and Discussion

The flow duration analysis records an outflow rate every 15-minutes over the duration of the continuous simulation, sorts those flows into 'bins' based on flow rate, and then plots the bins against exceedance frequency on a chart. The step-by-step procedures used to produce these charts can be found in APPENDIX E – Continuous Simulation Graph Procedures.

Figure 6.4-6 and Figure 6.4-7 present the Fort Collins flow duration charts and Figure 6.4-8 and Figure 6.4-9 show the Atlanta flow duration charts.

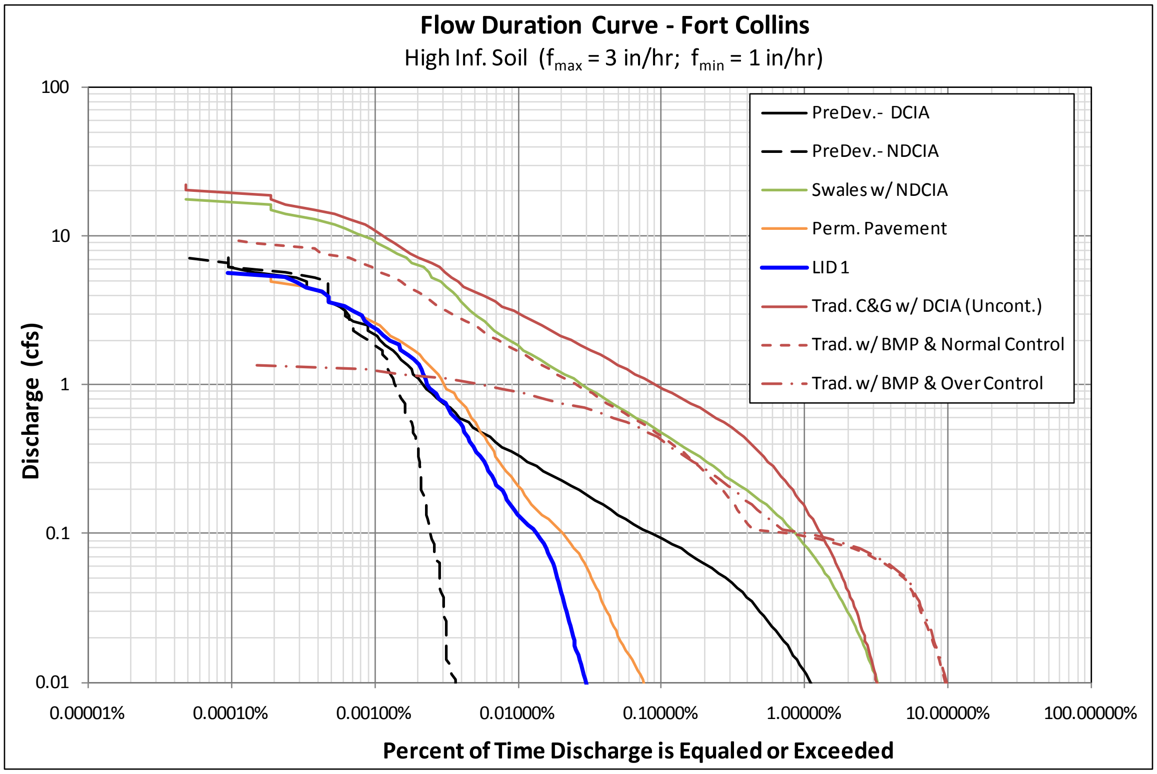


Figure 6.4-6 Flow Duration Curves – Fort Collins High Infiltration Soil

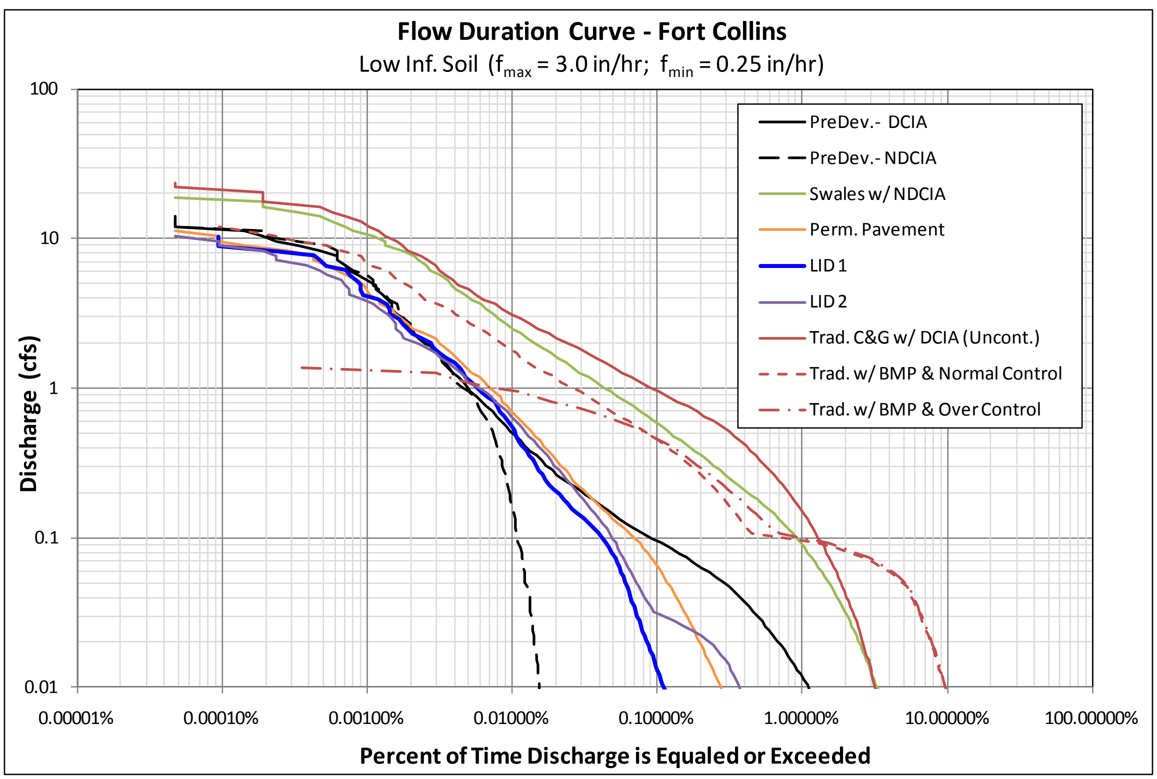


Figure 6.4-7 Flow Duration Curves – Fort Collins Low Infiltration Soil

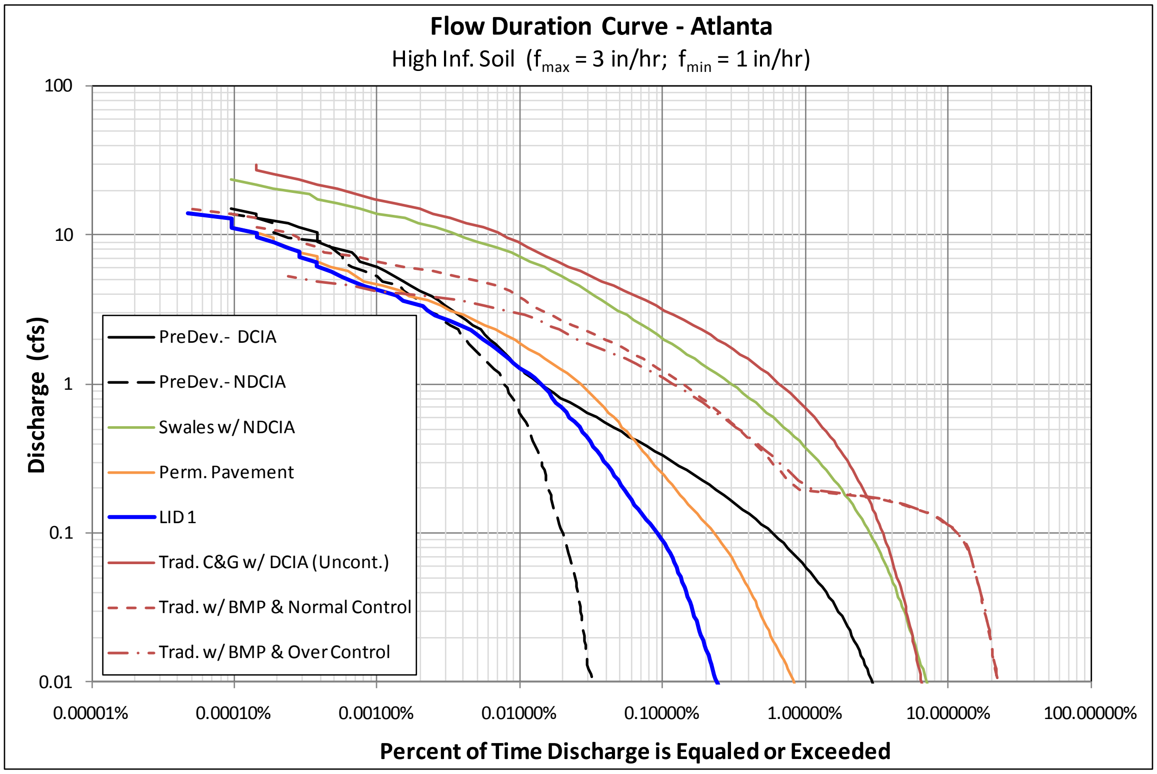


Figure 6.4-8 Flow Duration Curves – Atlanta High Infiltration Soil

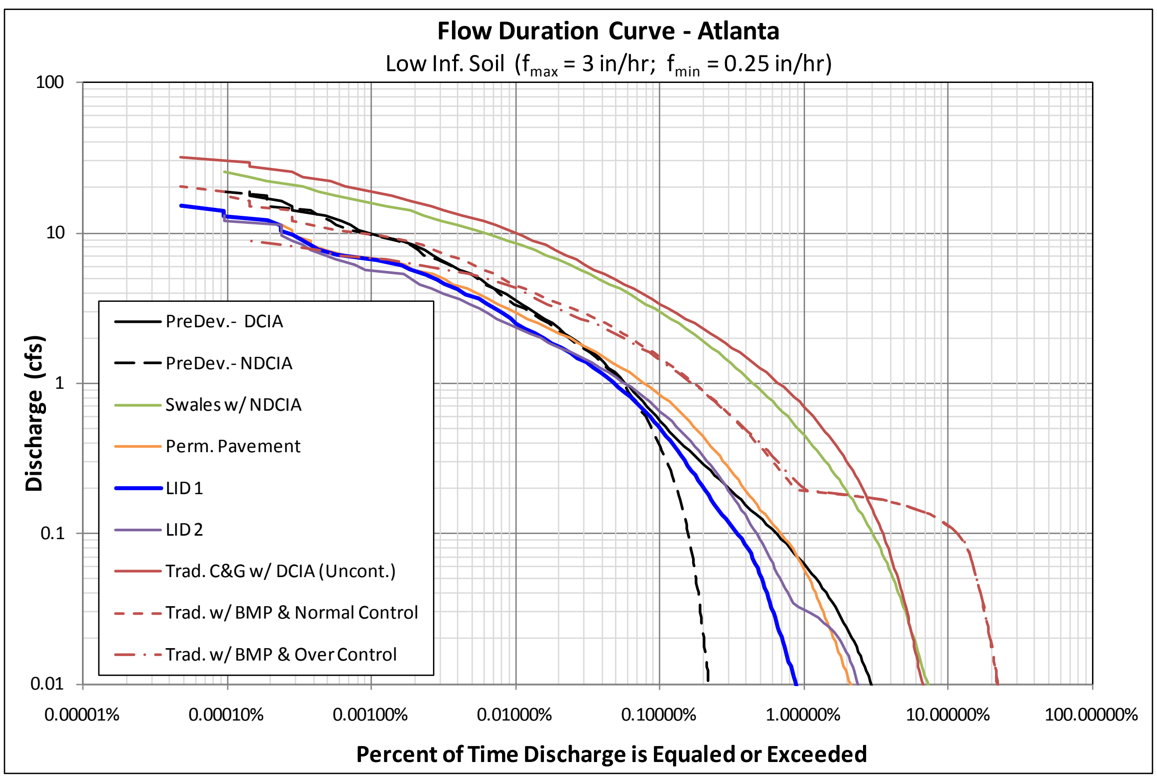


Figure 6.4-9 Flow Duration Curves – Atlanta Low Infiltration Soil

It is important to remember when examining the flow duration graphs that these are not only peak flows; these charts represent every single flow that came out of the model.

The flow duration figures show that once again, the LID models performed very well with the Predev. models. They split the difference between the Predev-DCIA and -NDCIA curves for high frequency events and tracked just lower than the Predevelopment curves for low frequency events. On the other hand, the Trad-w/NC and w/OC models significantly prolonged the duration of the majority of flows. In fact, for low flows, the stormwater controls extended the flow duration beyond that of the uncontrolled model. While it may not seem that flows on the order of 0.1 cfs are a big deal, remember that this model is 7.42 acres and 86 of these models will fit in 1 square mile, which transforms the 0.1 cfs into 8.6 cfs per square mile. Table 6.4-1 presents how the flow duration of 0.1 cfs was affected by different stormwater controls.

Table 6.4-1 Duration of Flows Exceeding 0.1 cfs

	0.1 cfs from this site = 8.6 cfs per sq. mi.					
	PreDev.			Trad. C&G		
models	NDCIA	DCIA	LID 1	Uncont.	BMP w/NC	BMP w/OC
			Days per Year			
FC High	0.01	0.31	0.04	4.38	3.10	3.65
FC Low	0.04	0.33	0.15	4.38	2.92	3.65
Atlanta High	0.07	2.19	0.33	12.78	40.15	40.15
Atlanta Low	0.55	2.19	1.28	12.78	40.15	40.15
			Ratio of Predevelopment (w/DCIA) Flow Duration			
FC High	0.03	1.0	0.1	14.1	10.0	11.8
FC Low	0.1	1.0	0.4	13.3	8.9	11.1
Atlanta High	0.03	1.0	0.2	5.8	18.3	18.3
Atlanta Low	0.3	1.0	0.6	5.8	18.3	18.3

This table shows that before development in Atlanta, the 0.1 cfs flow rate was exceeded 2 to 3 <u>days</u> per year, but after development with traditional stormwater controls, it was exceeded 5 to 6 <u>weeks</u> per year, or roughly 20 times longer duration. In Fort Collins the same flow exceedance duration was increased roughly 10 fold. In contrast, LID did not prolong the duration of low flows and fit nicely between the two predevelopment models. Looking at this effect a different way, Table 6.4-2, displays the increase in discharge at 0.1% exceedance frequency.

Table 6.4-2 Flow Discharge at 0.1% Exceedance Frequency

	PreDev.			Trad. C&G		
models	NDCIA	DCIA	LID 1	Uncont.	BMP w/NC	BMP w/OC
			cfs			
FC High	0.00	0.10	0.00	0.95	0.45	0.42
FC Low	0.00	0.10	0.03	0.95	0.45	0.45
Atlanta High	0.00	0.33	0.09	3.10	1.20	1.10
Atlanta Low	0.40	0.55	0.50	3.40	1.40	1.40
			Ratio of Predevelopment (w/DCIA) Flow Rate			
FC High	0.0	1.0	0.0	10.0	4.7	4.4
FC Low	0.0	1.0	0.3	10.0	4.7	4.7
Atlanta High	0.0	1.0	0.3	9.4	3.6	3.3
Atlanta Low	0.7	1.0	0.9	6.2	2.5	2.5

Again, development with traditional stormwater controls increased the flow discharge at the 0.1% exceedance frequency by 2 to 5 times while the LID models did not. In fact, both LID models followed or split the difference between the predevelopment peak flow frequency and flow duration curves very nicely for the whole range of return intervals.

6.5 Summary of Results

In summary, both LID and traditional stormwater controls, when designed properly, were able to meet stormwater peak flow requirements and maintain predevelopment peak flow rates for the whole range of flows. That said, it was demonstrated that traditional stormwater practices were not capable of reducing the excess runoff volume and they also significantly extended the duration of mid to low flows. On the other hand, LID practices were capable of restoring the predevelopment water balance and maintaining predevelopment flow durations.

7.0 COST COMPARISON

7.1 Purpose of Cost Comparison

Now that the question of whether LID can meet storm water requirements has been answered, this chapter explores the financial aspects. Is LID financially practical?

This analysis compares the Traditional model with the three LID models (LID1-H, LID1-L, and LID2-L). Costs were not estimated for the Permeable Pavement and Swales models because these models were used for hydrologic comparison only. The costs for the permeable pavement and swales contained within the LID 1 and LID 2 models were included in each respective LID estimate.

This is a cost *comparison* and not a complete cost estimate. The purpose is to compare the cost difference between the traditional stormwater system and the LID system; therefore, items that would remain the same between designs were not quantified, such as sidewalks and overlot grading. In addition to the storm drainage system, the road construction costs were also quantified, since the permeable pavement converts the roads into a functional part of the drainage system.

7.2 Initial Capital Cost

7.2.1 Computation Methodology

Construction quantities were calculated for each design using SWMM and AutoCAD. Unit costs for each item were obtained from the actual Union Place bid when available, the remaining unit costs were obtained from a RSMeans Site Work and Landscape Cost Data estimation guide (Spencer, 2008). Using the city cost indexes listed in RSMeans, costs were adjusted to Fort Collins, CO.

7.2.2 Assumptions

The following assumptions were made in the cost comparison:

1. General site costs such as mobilization and contingency are already covered as part of the main bid for the site; this is a cost comparison.
2. The land purchase cost is already covered; this is a comparison between two designs.
3. All storm pipes have 18 inches of cover.
4. Grate inlets are essentially a manhole with a grated cover, and thus the same price.
5. The rain gardens are on average 4 feet deep.
6. Impermeable cut off liners are installed laterally every 165 feet in the permeable pavement base. The area of each cut off liner is equal to the width of the road by 4 times the depth of the base. This is so that the liner can be folded over and tucked under the base, see figure PP-10 in volume 3 of the UDFCD Drainage Criteria Manual for more detail (UDFCD, 2007). The 165 foot spacing comes from the following equation from the Drainage Criteria Manual (see page S-24 of vol. 3):

$$L_{\max} = \frac{D}{1.5 \cdot S_0}$$

Where,

D = Depth of storage base in feet

S_0 = Slope of pavement surface

L_{max} = Maximum distance (ft) between cut-off barriers, normal to the flow direction.

7. Pond Excavation: The entire volume of the traditional model detention pond will be excavated and placed elsewhere on site. The detention volume to be excavated is

equal to the SWMM computed max pond depth for the 100-year storm plus 1 foot of freeboard using Nolte's proposed pond grading for the depth-area relationship.

8. Sodding and seeding of the site is assumed to be paid for as part of the conventional development and is not included in the cost comparison.

7.2.3 Cost Summary

Costs were calculated for two different permeable pavement options: permeable pavers and pervious concrete. Table 7.2-1 presents the initial capital costs for both the Fort Collins and the Atlanta designs. The difference in the designs, between locations, is the size of the LID BMPs necessary to meet the stormwater requirements. The supporting calculations, item unit costs, and quantities can be found in APPENDIX F – Cost Comparison Data.

Table 7.2-1 Initial Capital Cost Estimate

	LID 1[1] High Inf.	LID 1 Low Inf.	LID 2[2] Low Inf.	Traditional
Fort Collins Models				
Traditional Paving	-	-	-	$450,500
Permeable Pavers	$947,300	$934,100	$1,056,400	-
Pervious Concrete	$821,500	$808,300	$930,600	-
Pond Area (ac.)	-	-	-	1.20
Atlanta Models				
Traditional Paving	-	-	-	$454,200
Permeable Pavers	$954,700	$977,500	$1,099,800	-
Pervious Concrete	$828,900	$851,700	$974,000	-
Pond Area (ac.)	-	-	-	1.25

[1] LID 1 is full infiltration. [2] LID 2 is partial infiltration with an underdrain.

7.2.4 Opportunity Cost

Since the traditional design requires a detention pond and the LID designs do not, the area previously occupied by the detention pond is now available for development (1.20 and 1.25 acres for the Fort Collins and Atlanta models respectively). However the opportunity that LID provided to develop this land is relatively expensive; the cost difference between the Traditional design and LID designs will be called the opportunity cost. It is assumed that the land cost of the detention pond has already been paid for with the traditional development and does not need to be included in the cost comparison. Table 7.2-2 presents the opportunity cost for each scenario.

Table 7.2-2 Opportunity Cost

	LID 1 High Inf.	LID 1 Low Inf.	LID 2 Low Inf.
Fort Collins Models			
Permeable Pavers	$496,800	$483,600	$605,900
Pervious Concrete	$371,000	$357,800	$480,000
Atlanta Models			
Permeable Pavers	$500,500	$523,300	$645,600
Pervious Concrete	$374,700	$397,500	$519,800

Breaking this down further, the opportunity cost can be divided by the number of new lots that could be developed using the former detention pond land; this is the break-even sales price. Table 7.2-3 displays three potential ways to divide the pond area into lots and the associated break-even sales price. This does not include any extra infrastructure costs required to build on these lots, such as roads and utilities.

Table 7.2-3 Break Even Sales Price per Lot

	LID 1 High Inf.	LID 1 Low Inf.	LID 2 Low Inf.
Fort Collins Models			
Six 0.20 acre Lots			
Permeable Pavers	$82,800	$80,600	$101,000
Pervious Concrete	$61,900	$59,700	$80,100
Eight 0.15 acre Lots			
Permeable Pavers	$62,100	$60,500	$75,800
Pervious Concrete	$46,400	$44,800	$60,100
Twelve 0.10 acre Lots			
Permeable Pavers	$41,400	$40,300	$50,500
Pervious Concrete	$31,000	$29,900	$40,100

	LID 1 High Inf.	LID 1 Low Inf.	LID 2 Low Inf.
Atlanta Models			
Six 0.20 acre Lots			
Permeable Pavers	$83,500	$87,300	$107,600
Pervious Concrete	$62,500	$66,300	$86,700
Eight 0.15 acre Lots			
Permeable Pavers	$62,600	$65,500	$80,700
Pervious Concrete	$46,900	$49,700	$65,000
Twelve 0.10 acre Lots			
Permeable Pavers	$41,800	$43,700	$53,800
Pervious Concrete	$31,300	$33,200	$43,400

7.3 Discussion

7.3.1 Initial Capital Cost

A preliminary look at the LID price tag reveals that construction costs are twice as much as the traditional drainage system. Economically speaking, why should anyone consider it with this price tag? Only where the land value is high enough that the extra costs can be recovered by selling more lots. In rural areas, managing all of the storm water with LID may be cost prohibitive as land is readily available and it would be difficult to sell higher priced lots. In urbanized areas however, the economic incentive to maximize land use as well as the ability to sell lots at a premium may make LID profitable.

Between August 2009 and January 2010, 0.16 acre lots proximal to Fort Collins, CO have sold for $45,000 to $85,000, depending on location and amenities (Robinson, 2010). This shows that the break-even price for the recovered lots is reasonable. In order to make a profit, the appeal of the particular area, will determine the developers ability to sell these lots above the break-even price.

A very interesting point is that over half of the total LID cost is in the permeable pavement surface. Using the FC-H scenario as an example, the total LID 1 cost is $947,300 with pavers and $821,500 with pervious concrete; of that total cost, $587,100 and $461,300 are for pavers and pervious concrete respectively (see Table 15.2-3). That is 62% and 56%, respectively, of the entire cost! Table 7.3-1 shows the effect of reducing the permeable pavement cost on the total cost and the break-even price.

Table 7.3-1 Effects of Permeable Pavement Cost Reduction

	Original Paving Cost /SY	Reduced Paving Cost /SY	Percent Reduction	Original Total Cost	Reduced Total Cost	Percent Reduction	.15 acre Lot Price	Percent Reduction
Pavers	$63.00	$47.25	**25%**	$947,300	$800,500	**15%**	$43,800	**29%**
Pervious Concrete	$49.50	$37.17	**25%**	$821,500	$706,600	**14%**	$32,100	**31%**

This strongly suggests that if the cost of permeable pavement can be reduced, even by 25%, the profitability of LID projects greatly increases. If production costs could be brought lower or another method for decreasing the cost of permeable pavements were found, LID designs would become a more viable option for urban development. Developers would be more likely to consider LID in their projects if the costs were more comparable to conventional development. Trends also suggest that some buyers would consider paying a reasonable amount more for a home or business in a "green" development.

The underdrained scenario (LID 2 model), on the other hand, is not an economical option; as it pushes up the overall LID cost by an average of 14% and the opportunity cost by an average of 30%. So even though this design is able to meet the stormwater requirements, an extensive underdrain system could compromise the profitability of an LID project.

7.3.2 Reduction of Urban Sprawl

LID reduces urban sprawl by developing land that would normally be a detention pond. It is worth noting that well designed detention ponds can also serve as open space amenities, however the developable area on this site increased by 16%. Applying this over a large urban area translates into less roads, utilities, and other infrastructure. Allowing people to live slightly closer to where they work, shop, and socialize reduces miles traveled, fuel consumption, wear on vehicles, and air pollution.

7.3.3 Life Cycle Cost

In addition to the initial capital costs of LID, it is important to consider the life cycle cost of each BMP. Life cycle costs are the initial capital and construction cost plus the operation, maintenance, repair, and replacement costs that are incurred by the facility in the future (BusinessDictionary.com, 2010). This section discusses, but does not quantify, future costs of LID BMPs.

For many LID BMPs, such as rain gardens (bio-retention) and swales, the maintenance requirements are about the same as regular landscaping. They need mowing, irrigation, trash removal, periodic inspection, weeding, minor fertilization, and occasional sediment removal. If the BMP resides on residential property, generally it can be maintained by the homeowner. Infiltration trenches require yearly inspection to make sure they have not clogged; if clogged, the trench would need to be excavated and the aggregate washed and replaced with new filter fabric. Permeable pavements will generally require the most maintenance of the BMPs presented here. To prevent clogging, the pavement should be vacuumed with a vacuum sweeper between one and four times per year. For pavers, the gravel fill between the joints may need to be replaced after vacuuming; if vacuuming does not restore infiltration rates, the affected paver sections can be removed and the gravel bedding replaced. For all permeable pavements, it is important that sealer coats and winter sanding are not applied as they will significantly compromise the infiltration capacity of the pavement (DOD, 2004; Ferguson, 2005; Prince George's County, 2000a).

In contrast, traditional BMPs are not maintenance free either. Extended detention ponds need periodic inspection, mowing, mucking, cleaning, reseeding, and occasional pumping. Conventional pavements also have maintenance requirements such as resurfacing, sealing, painting, pothole repair, and crack filling (Ferguson, 2005). It is beyond the scope of this thesis to quantify and compare the maintenance costs of LID and traditional drainage systems. However, the performance of LID systems, particularly permeable pavements, is more sensitive to maintenance.

It is important for developers who employ LID to have maintenance agreements communicated with the owner verbally and in written form. These agreements need to include who pays for maintenance, a mechanism to exact the payments, and who will perform the maintenance. It is beyond the scope of this thesis to examine how these agreements work in theory and practice.

Lifespan is the last thing to be considered in this discussion. Rain gardens and infiltration trenches may need to be excavated and have the fill material washed or replaced every 5 to 15 years, depending on the reduction of infiltration rates. Grass swales may need to be re-graded and replanted if they become eroded or filled with sediment, otherwise their life span is indefinite. The life span of permeable pavers is between 10 to 25 years and for pervious concrete, 20 to 30 years (UDFCD, 2007). This is based upon both surface deterioration and excessive reduction of infiltration rates. If maintenance is performed frequently, it would be reasonable to assume that the pavements life span would be maximized. In contrast, conventional asphalt and concrete pavements can last 30 to 50 years, provided they are maintained (Croney & Croney, 1998). This includes overlays for asphalt and surface treatment to retain skid resistance for concrete. Based on this information, the life span of conventional paving is about twice the lifespan of permeable paving. This is a major issue to be considered by the developer and the final owner of the pavement.

8.0 CONCLUSIONS and RECOMMENDATIONS

The results in chapter 6.0 demonstrate that LID can be used exclusively to meet stormwater requirements in Fort Collins and Atlanta. The water quality requirement was met by routing each impervious area through a LID BMP which was sufficiently sized to capture the WQCV. Both LID models also demonstrated that they were able to maintain the predevelopment peak flow rates. Normally this would be considered sufficient to meet most municipal stormwater requirements. However, traditional stormwater management does not fully maintain the predevelopment hydrologic regime; it controls peak flow rates while paying little attention to the increased flow duration and volume of runoff associated with urbanization. To address this, the LID models were built using both peak flow and water balance as criteria. The result was that the LID 1 model replicated the predevelopment flow regime for the 2-, 10-, and 100-year design storms very well in Atlanta and acceptably in Fort Collins. The LID 2 model, which includes underdrains, also met the stormwater requirements, but did not replicate the shape of the Atlanta predevelopment hydrographs as closely as the LID 1 model did.

In the historical simulations, both LID models followed the predevelopment peak flow frequency and flow duration curves very well for the lower frequency flows, and split the difference between the DCIA and NDCIA curves for the higher frequency flows. In contrast, the Traditional model with Normal Control and with Over Control could not replicate the predevelopment water balance and significantly prolonged the duration of high frequency flows, revealing that it could not completely replicate the predevelopment hydrologic regime. This is where LID proved to have the upper hand and showed that it is more effective at controlling stormwater than traditional management practices.

These modeling results hinge on one major assumption, that the rate of infiltration through the permeable pavement surface is not a limiting factor. Literature on this subject suggests that when permeable pavements are maintained, the assumption is valid.

An interesting and valuable SWMM modeling finding is that the two different methods used to model LID performed very similarly for short duration storms but different for long duration storms. For the Fort Collins 2-hour design storms, they performed nearly identically. However in Atlanta, the LID 2 model yielded much lower peak flow rates than both the LID 1 and predevelopment models. This difference is the result of the underdrains releasing flow from the pavement storage reservoirs during the initial part of the 24-hour storm and then when the storm peaked, at 11.9 hours, the water levels in the storage reservoirs were lower in the LID 2 model. In the historical simulations, the LID 2 curve followed the LID 1 curve very closely for all but the very low flows. These findings suggest that for shorter duration storms, the full infiltration method (LID 1) could be used to model partial infiltration BMPs (LID 2). This is valuable because the modeling method used in the LID 2 model is much more complicated and less stable. The result would be that peak flows are slightly over estimated and the runoff volume is slightly under estimated.

The cost comparison in chapter 7.0 evaluated whether it is financially practical to manage stormwater exclusively with LID. This comparison revealed that the LID construction cost is about twice as much as the traditional storm drain system; this comparison included the road construction cost because the permeable pavement converts the road to a functional part of the drainage system. While this would seem to be cost prohibitive, a further investigation found that in urban areas this cost could be recovered by developing the detention basin land, which is no longer needed because of LID. It also revealed that in this site design, over half of the total LID cost was in the permeable pavement surface itself and thus the profitability of the project would be very sensitive to the construction cost of permeable pavement. Furthermore, the extensive underdrain system, in the LID 2 model, was relatively expensive and recovering this extra cost could be difficult. There are situations in which LID can be cost effective, as well as times in which the cost will outweigh the benefits of LID. If the cost of permeable pavement could be lowered, LID could more rapidly become a viable option for developers.

This thesis found that LID practices alone can meet the storm drainage requirements. However rather than using LID exclusively, a more practical option may be using LID to address water quality and to match the predevelopment water balance, while using a detention pond to restore predevelopment peak flows. In three of the four LID 1 models, the peak rate - not water balance - controlled the sizing of the pavement base depth. In fact, the LID 1 Low Infiltration model required less than half of the pavement storage to match the predevelopment water balance in both locations. Once the predevelopment water balance has been met, the size of the detention pond needed to control the 100-year peak flow should be substantially reduced. Quantifying the size of that pond was beyond the scope of this thesis; however it would be about 30-40% of the traditional scenario pond size. This option would significantly reduce the amount and cost of permeable pavement, and would free up a majority of the detention pond area for development.

LID is more complicated to model, design, construct, and maintain than its conventional counterparts. Care should be taken when planning and designing LID systems and those considering the use of LID should first count the cost and decide whether it is a worthwhile investment. While the benefits of LID are immense, it is not a ‘silver bullet’ for stormwater management.

As a result of this study, further investigation is called for on the following:

1. Research on permeable pavement cross section design. Where is clogging prone? How effective are geotextiles as permeable separators?

2. Further study and monitoring of the rate at which water passes through the permeable pavement surface in new pavement and existing pavement, both maintained and unmaintained.

3. What controls the rate at which water flows into underdrains? How it could be estimated?

4. Further modeling and evaluation of partial infiltration BMPs (LID 2 model) where the underdrains are not raised off of the reservoir bottom. Two cases should be considered: infiltration allowed and infiltration not allowed.

5. Research on the costs of permeable pavement installation, particularly why the pavements are so expensive and how the cost could be reduced.

6. Further investigation into the economics of LID.

7. Regulatory acceptance of using LID practices to meet detention and peak flow requirements.

8. Coding changes to the SWMM software to facilitate better modeling of LID. Particularly adding infiltration in swales (conduits), better options for LID BMP’s in the conduit system, and underdrained pavement storage reservoirs.

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10.0 APPENDIX A – IDF Comparison of Disaggregated Rainfall Data

10.1 Fort Collins IDF Charts

Table 10.1-1 Fort Collins IDF Comparison

	Storm Duration (hrs)	Return Period (years)					
		2	5	10	25	50	100
		Rainfall Depth (in)					
Fort Collins Stormwater Standards	0.5	0.65	0.9	1.1	1.45	1.7	2.3
	1	0.8	1.15	1.4	1.8	2.2	2.9
	2	1	1.4	1.7	2.4	2.8	3.7
FC 60 min. Raw Data	0.5						
	1	0.75	1.15	1.5	2.07	2.63	3.32
	2	0.95	1.41	1.81	2.48	3.12	3.91
FC Disaggregated Data	0.5	0.63	0.92	1.19	1.62	2.03	2.54
	1	0.85	1.23	1.57	2.13	2.66	3.32
	2	1	1.46	1.87	2.56	3.22	4.04
		Rainfall Intensity (in/hr)					
Fort Collins Stormwater Standards	0.5	1.3	1.8	2.2	2.9	3.4	4.6
	1	0.8	1.15	1.4	1.8	2.2	2.9
	2	0.5	0.7	0.85	1.2	1.4	1.85
FC 60 min. Raw Data	0.5						
	1	0.75	1.15	1.50	2.07	2.63	3.32
	2	0.48	0.71	0.91	1.24	1.56	1.96
FC Disaggregated Data	0.5	1.26	1.84	2.38	3.24	4.06	5.08
	1	0.85	1.23	1.57	2.13	2.66	3.32
	2	0.50	0.73	0.94	1.28	1.61	2.02
		Percent Difference from FC Stormwater IDF Chart					
Fort Collins Stormwater Standards	0.5						
	1						
	2						
FC 60 min. Raw Data	0.5						
	1	-6%	0%	7%	15%	20%	14%
	2	-5%	1%	6%	3%	11%	6%
FC Disaggregated Data	0.5	-3%	2%	8%	12%	19%	10%
	1	6%	7%	12%	18%	21%	14%
	2	0%	4%	10%	7%	15%	9%

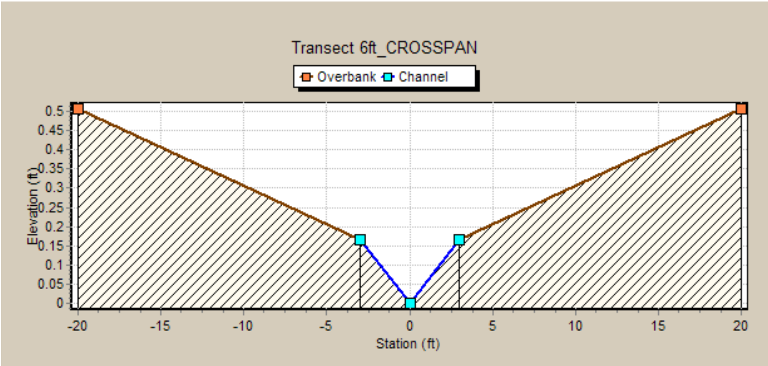
10.2 Atlanta IDF Comparison

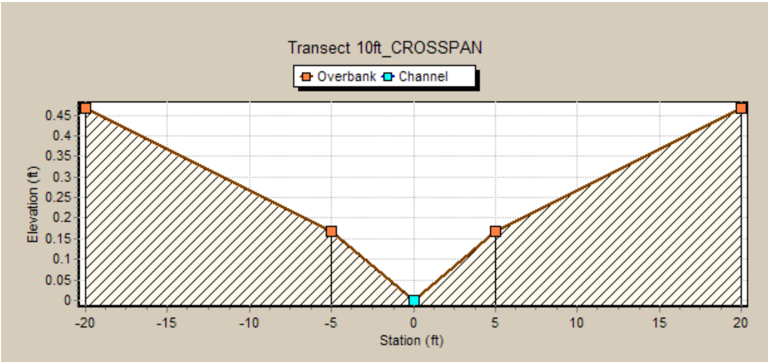
Table 10.2-1 Atlanta IDF Comparison

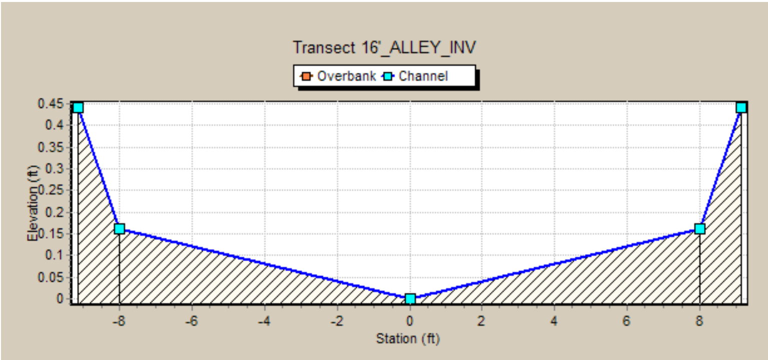
	Storm Duration (hrs)	Return Period (years) 1	2	5	10	25	50	100
					Rainfall Depth (in)			
Atlanta Stormwater Standards	0.5	1.17	1.36	1.66	1.88	2.19	2.44	2.68
	1	1.49	1.72	2.17	2.49	2.95	3.3	3.65
	2	1.92	2.28	2.8	3.16	3.68	4.04	4.42
	6	2.34	2.88	3.6	4.14	4.8	5.4	5.82
	12	2.76	3.36	4.32	4.92	5.64	6.36	6.96
	24	3.36	4.08	4.8	5.52	6.48	7.2	7.92
Atlanta 60 min. Raw Data	0.5							
	1	1.26	1.49	1.81	2.09	2.57	3.03	3.62
	2	1.56	1.9	2.31	2.66	3.23	3.77	4.41
	6	2.09	2.57	3.05	3.43	4	4.49	5.04
	12	2.57	3.11	3.73	4.24	5.02	5.71	6.53
	24	3.12	3.74	4.54	5.25	6.39	7.47	8.78
Atlanta Disaggregated Data	0.5	1.12	1.3	1.55	1.77	2.13	2.48	2.92
	1	1.41	1.69	2.05	2.35	2.83	3.28	3.82
	2	1.61	1.98	2.42	2.79	3.38	3.93	4.58
	6	2.2	2.62	3.1	3.49	4.07	4.57	5.15
	12	2.57	3.15	3.8	4.32	5.11	5.82	6.63
	24	3.15	3.75	4.56	5.27	6.41	7.48	8.79
					Rainfall Intensity (in/hr)			
Atlanta Stormwater Standards	0.5	2.33	2.72	3.32	3.75	4.38	4.87	5.36
	1	1.49	1.72	2.17	2.49	2.95	3.3	3.65
	2	0.96	1.14	1.4	1.58	1.84	2.02	2.21
	6	0.39	0.48	0.6	0.69	0.8	0.9	0.97
	12	0.23	0.28	0.36	0.41	0.47	0.53	0.58
	24	0.14	0.17	0.2	0.23	0.27	0.3	0.33
Atlanta 60 min. Raw Data	0.5							
	1	1.26	1.49	1.81	2.09	2.57	3.03	3.62
	2	0.78	0.95	1.16	1.33	1.62	1.89	2.21
	6	0.35	0.43	0.51	0.57	0.67	0.75	0.84
	12	0.21	0.26	0.31	0.35	0.42	0.48	0.54
	24	0.13	0.16	0.19	0.22	0.27	0.31	0.37

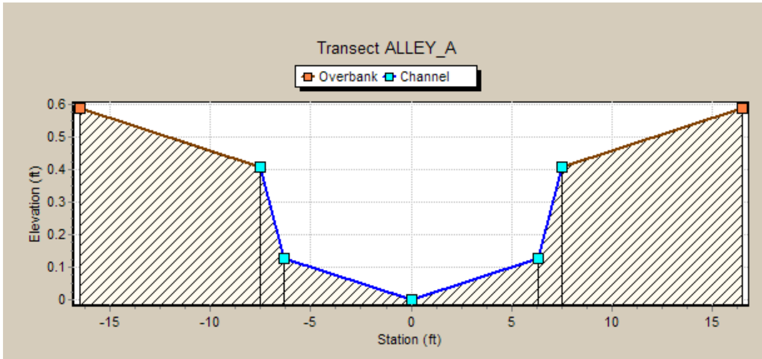
	Storm Duration (hrs)	Return Period (years)						
		1	2	5	10	25	50	100
Atlanta Disaggregated Data	0.5	2.24	2.6	3.1	3.54	4.26	4.96	5.84
	1	1.41	1.69	2.05	2.35	2.83	3.28	3.82
	2	0.81	0.99	1.21	1.40	1.69	1.97	2.29
	6	0.37	0.44	0.52	0.58	0.68	0.76	0.86
	12	0.21	0.26	0.32	0.36	0.43	0.49	0.55
	24	0.13	0.16	0.19	0.22	0.27	0.31	0.37
		Percent Difference from Atlanta Stormwater IDF Chart						
Atlanta Stormwater Standards	0.5							
	1							
	2							
	6							
	12							
	24							
Atlanta 60 min. Raw Data	0.5							
	1	-15%	-13%	-17%	-16%	-13%	-8%	-1%
	2	-19%	-17%	-18%	-16%	-12%	-7%	0%
	6	-11%	-11%	-15%	-17%	-17%	-17%	-13%
	12	-7%	-7%	-14%	-14%	-11%	-10%	-6%
	24	-7%	-8%	-5%	-5%	-1%	4%	11%
Atlanta Disaggregated Data	0.5	-4%	-4%	-7%	-6%	-3%	2%	9%
	1	-5%	-2%	-6%	-6%	-4%	-1%	5%
	2	-16%	-13%	-14%	-12%	-8%	-3%	4%
	6	-6%	-9%	-14%	-16%	-15%	-15%	-12%
	12	-7%	-6%	-12%	-12%	-9%	-8%	-5%
	24	-6%	-8%	-5%	-5%	-1%	4%	11%

11.0 APPENDIX B – Conduit Cross Sections








Transect ALLEY_A
Overbank
Channel
Elevation (ft)
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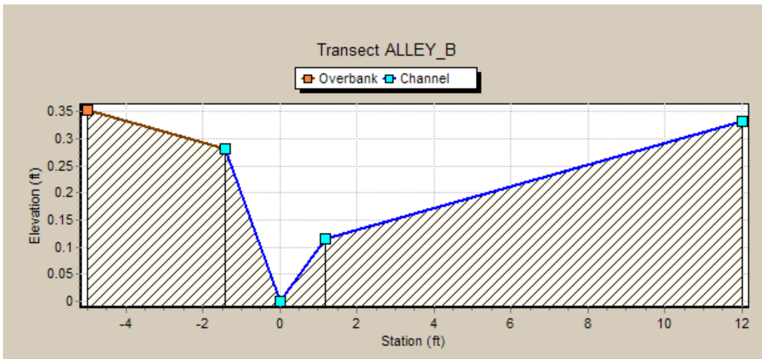
0.3
0.2
0.1
0
-15
-10
-5
0
5
10
15
Station (ft)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Channel
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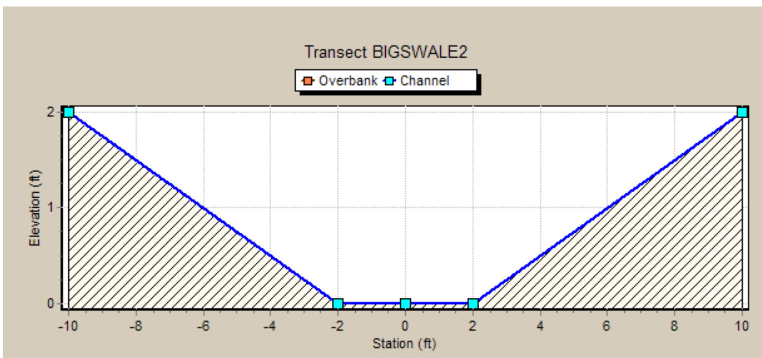
0.2
0.15
0.1
0.05
0
-4
-2
0
2
4
6
8
10
12
Station (ft)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Channel
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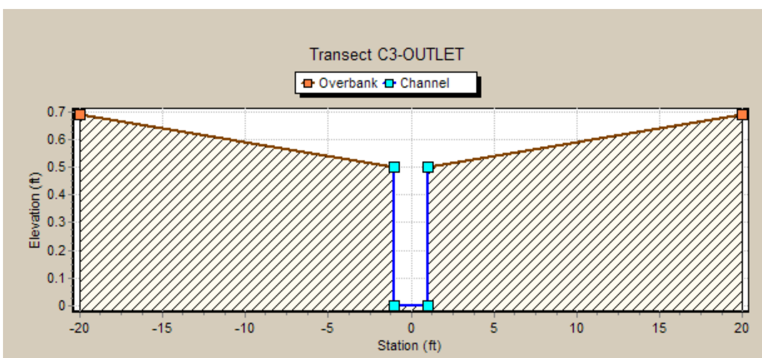
-8
-6
-4
-2
0
2
4
6
8
10
Station (ft)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Overbank
Channel
Elevation (ft)
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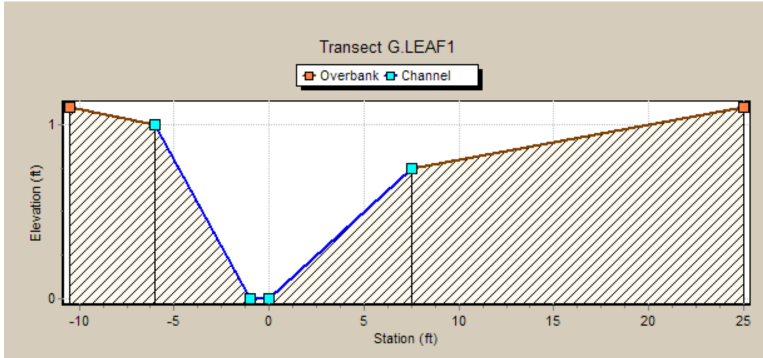
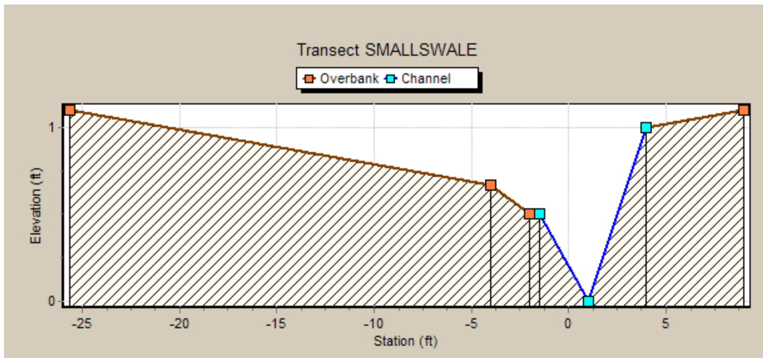
0.6
0.5
0.4
0.3
0.2
0.1
0
-20
-15
-10
-5
0
5
10
15
20
Station (ft)


Transect G.LEAF1
Overbank
Channel
Elevation (ft)
Station (ft)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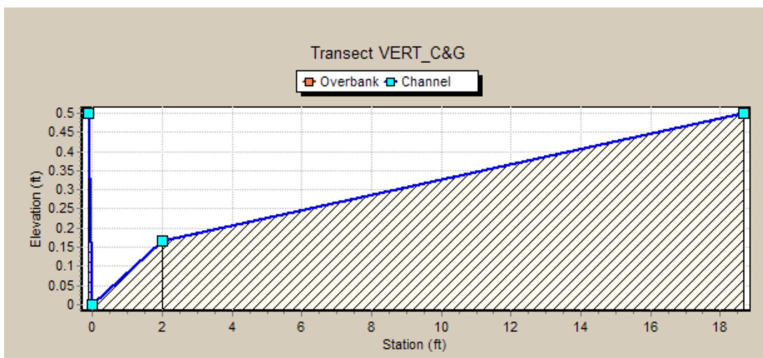
Channel
Elevation (ft)
Station (ft)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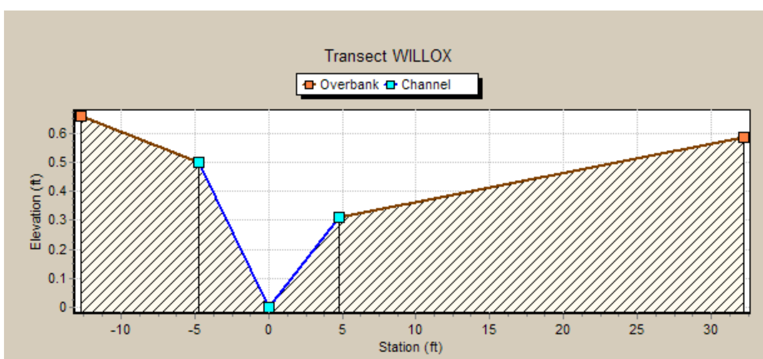
Channel
Elevation (ft)
Station (ft)


Transect WILLOX
Overbank
Channel
Elevation (ft)
Station (ft)

12.0 APPENDIX C – Fort Collins Design Storm Results

12.1 Design Storm Hydrographs – High Infiltration Soil

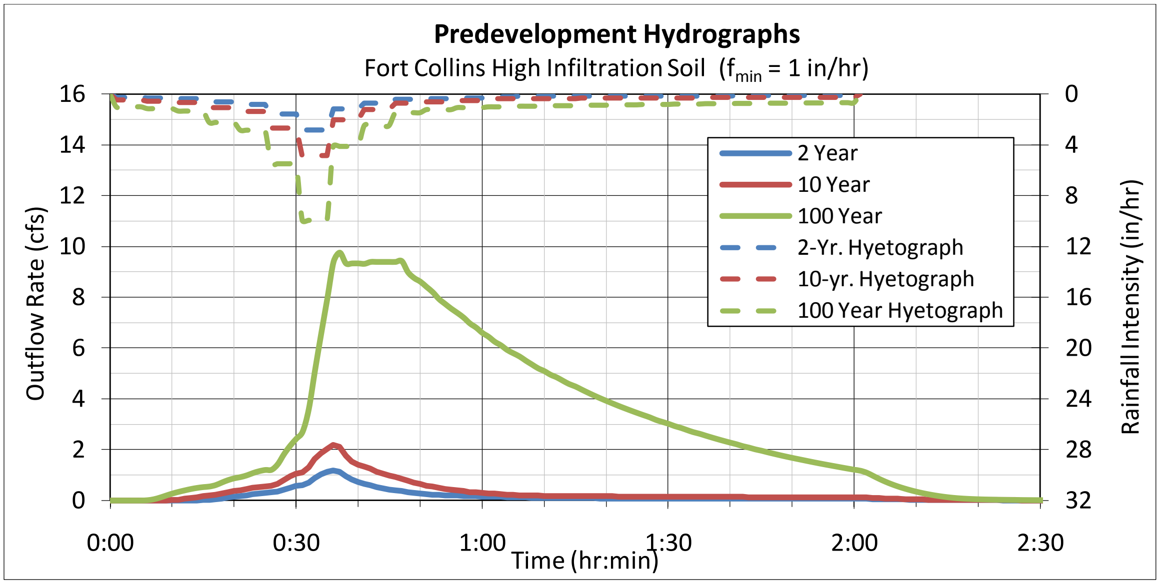


Figure 12.1-1 Predevelopment Hydrographs – Fort Collins High Infiltration Soil

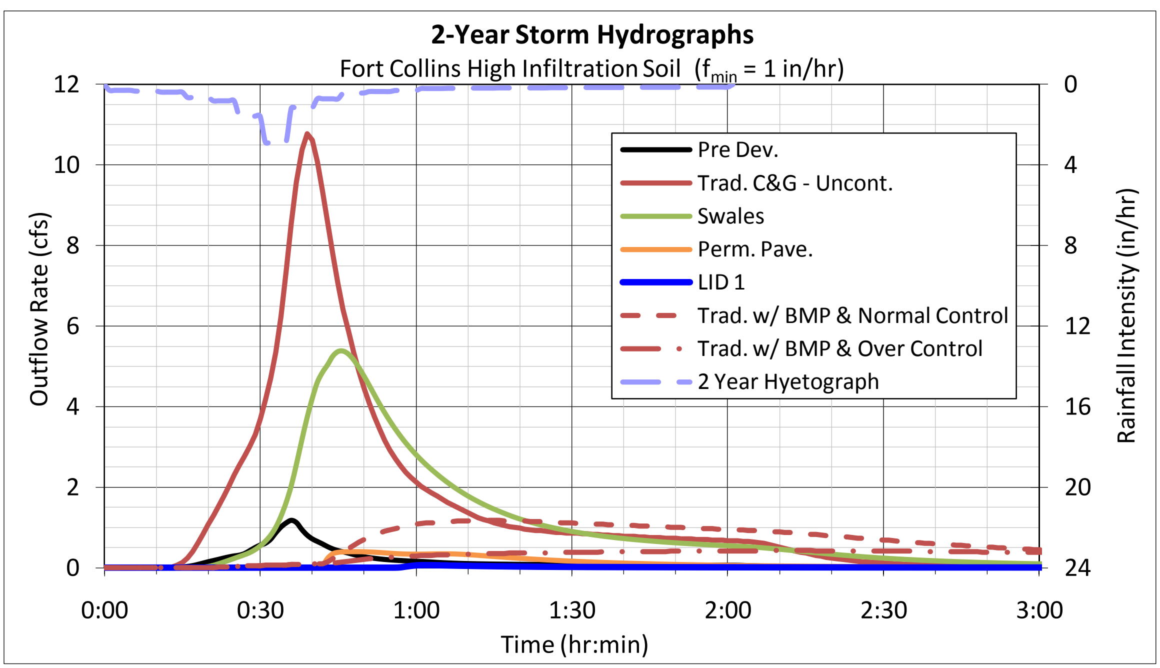


Figure 12.1-2 2-Year Storm Hydrographs – Fort Collins High Infiltration Soil

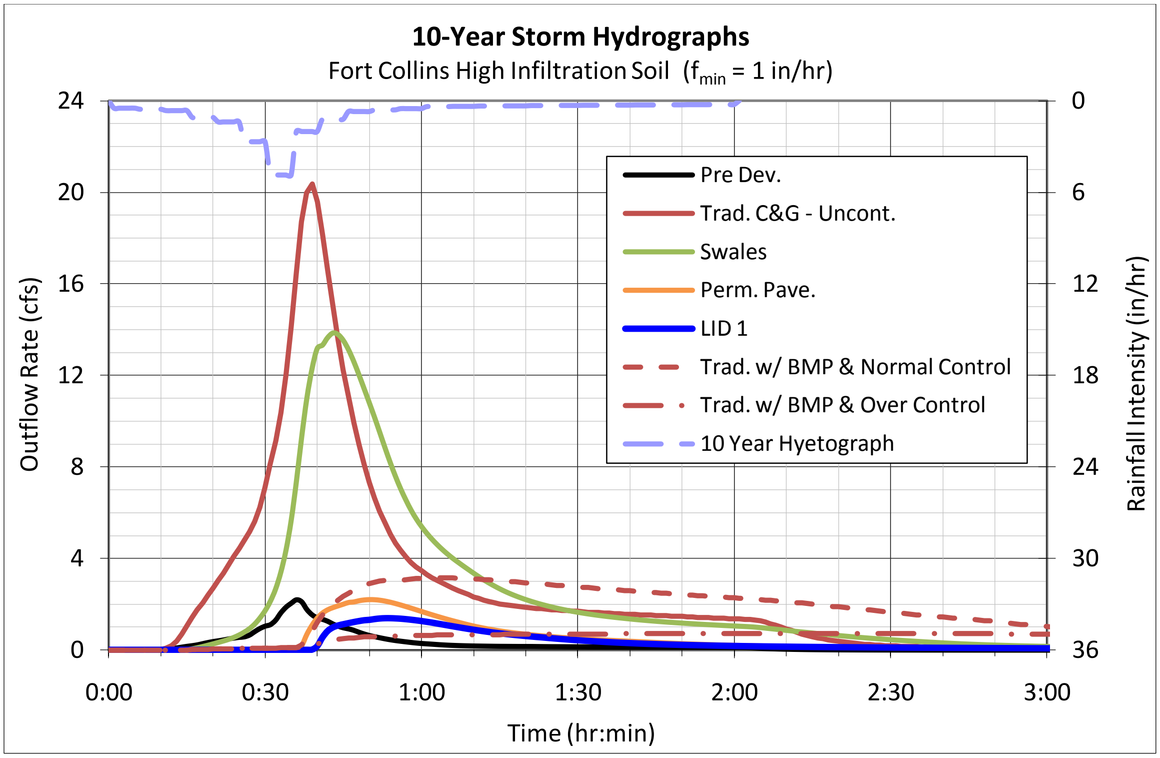


Figure 12.1-3 10-Year Storm Hydrographs – Fort Collins High Infiltration Soil

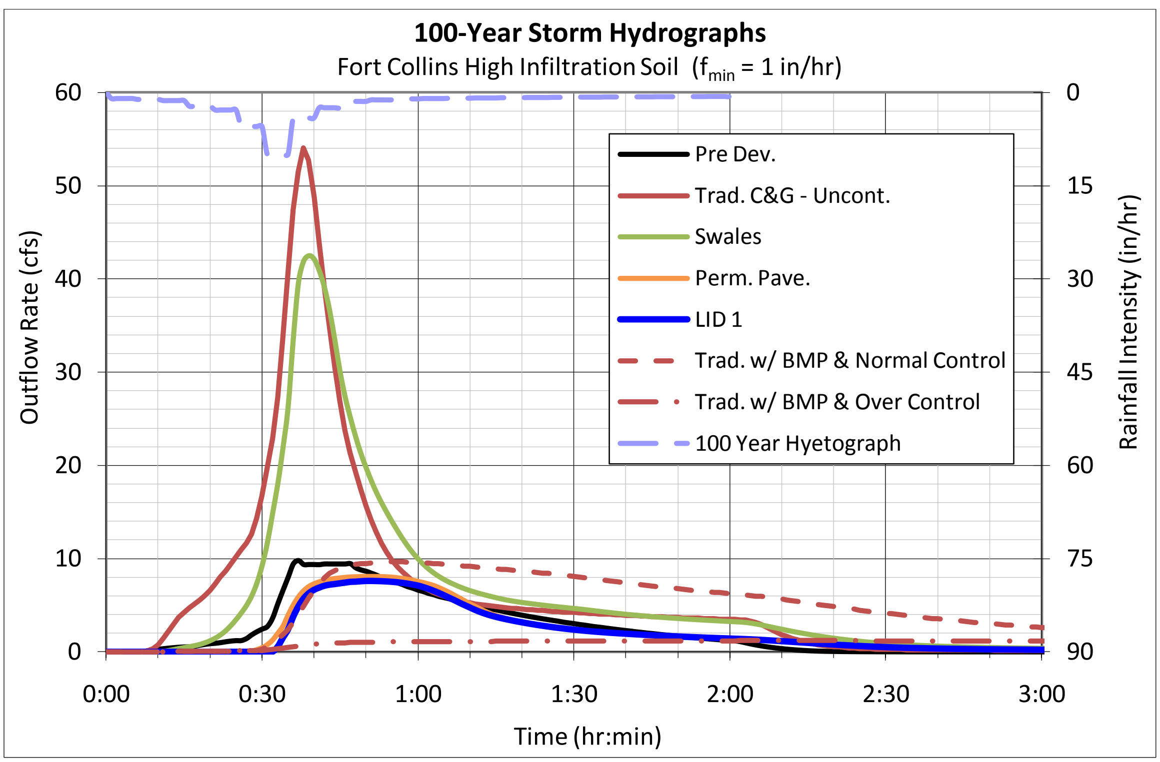


Figure 12.1-4 100-Year Storm Hydrographs – Fort Collins High Infiltration Soil

12.2 Design Storm Hydrographs – Low Infiltration Soil

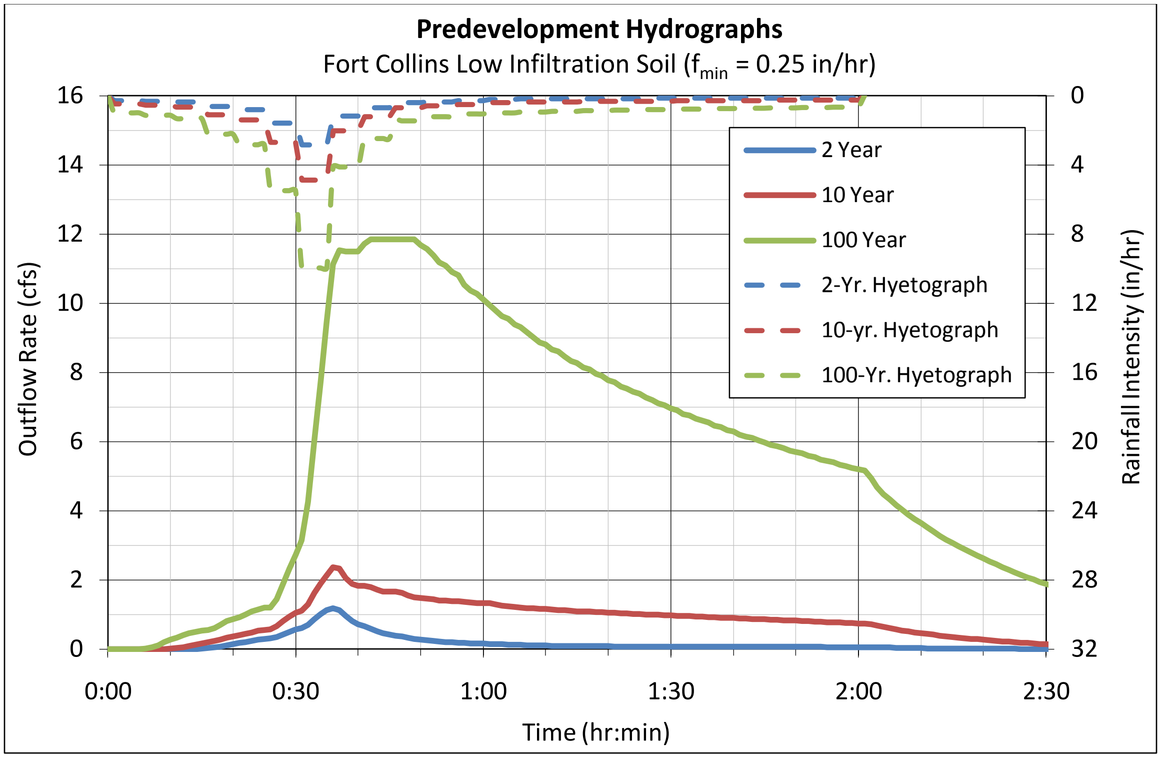


Figure 12.2-1 Predevelopment Hydrographs – Fort Collins Low Infiltration Soil

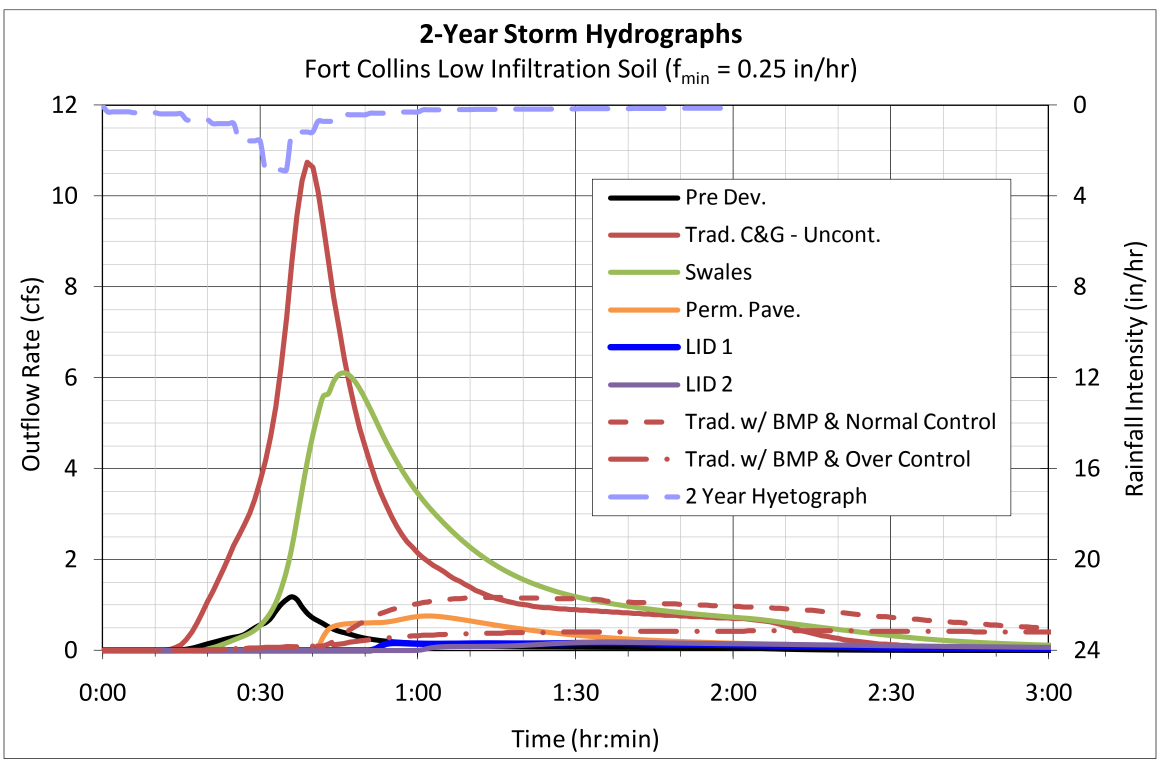


Figure 12.2-2 2-Year Storm Hydrographs – Fort Collins Low Infiltration Soil

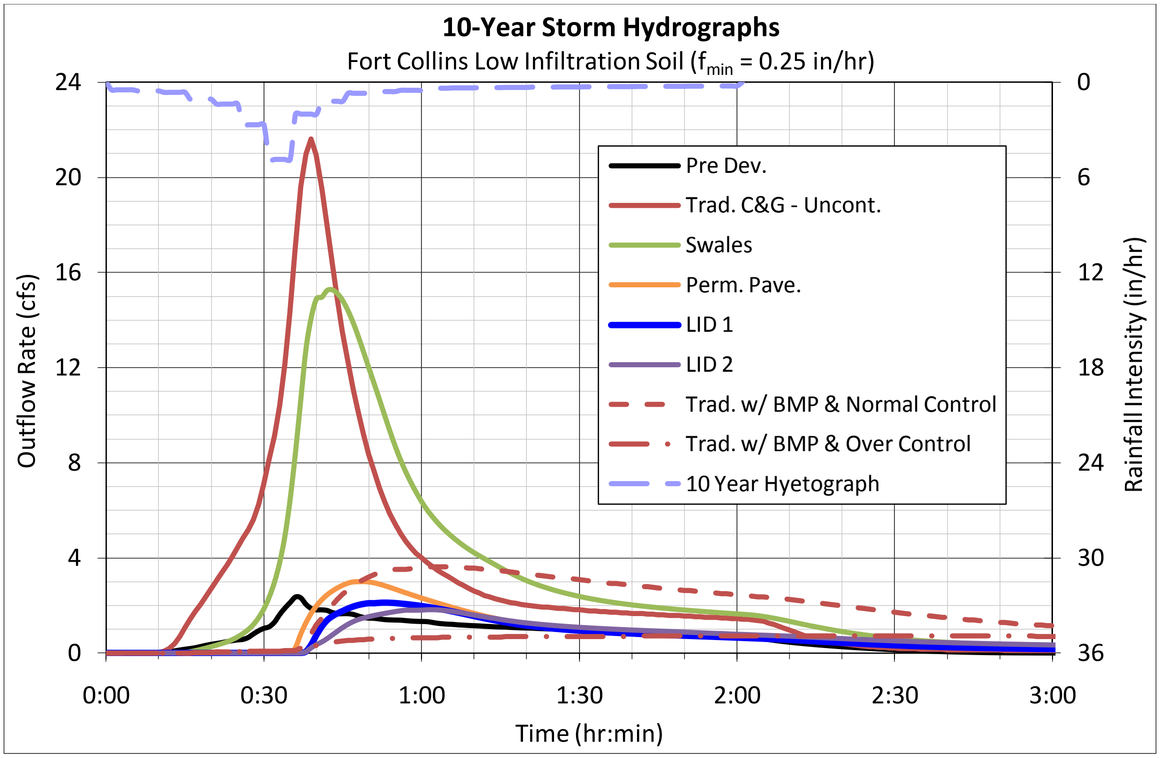


Figure 12.2-3 10-Year Storm Hydrographs – Fort Collins Low Infiltration Soil

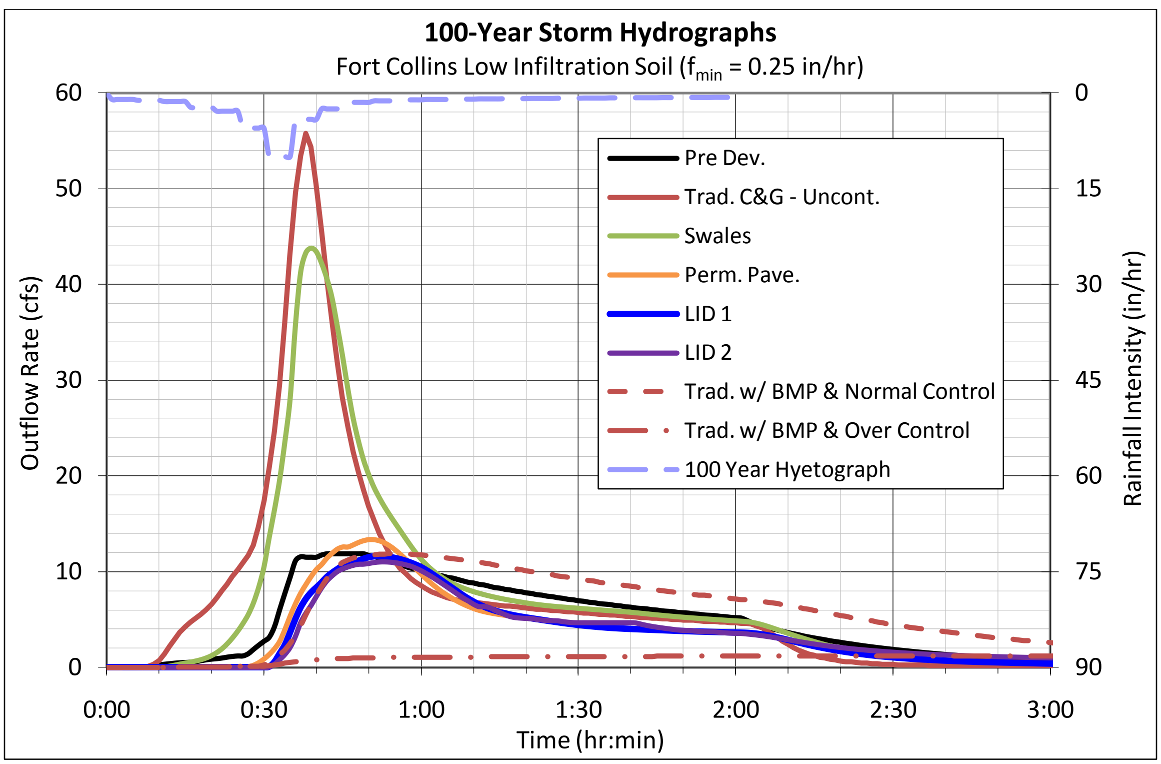


Figure 12.2-4 100-Year Storm Hydrographs – Fort Collins Low Infiltration Soil

12.3 Design Storm Water Balance

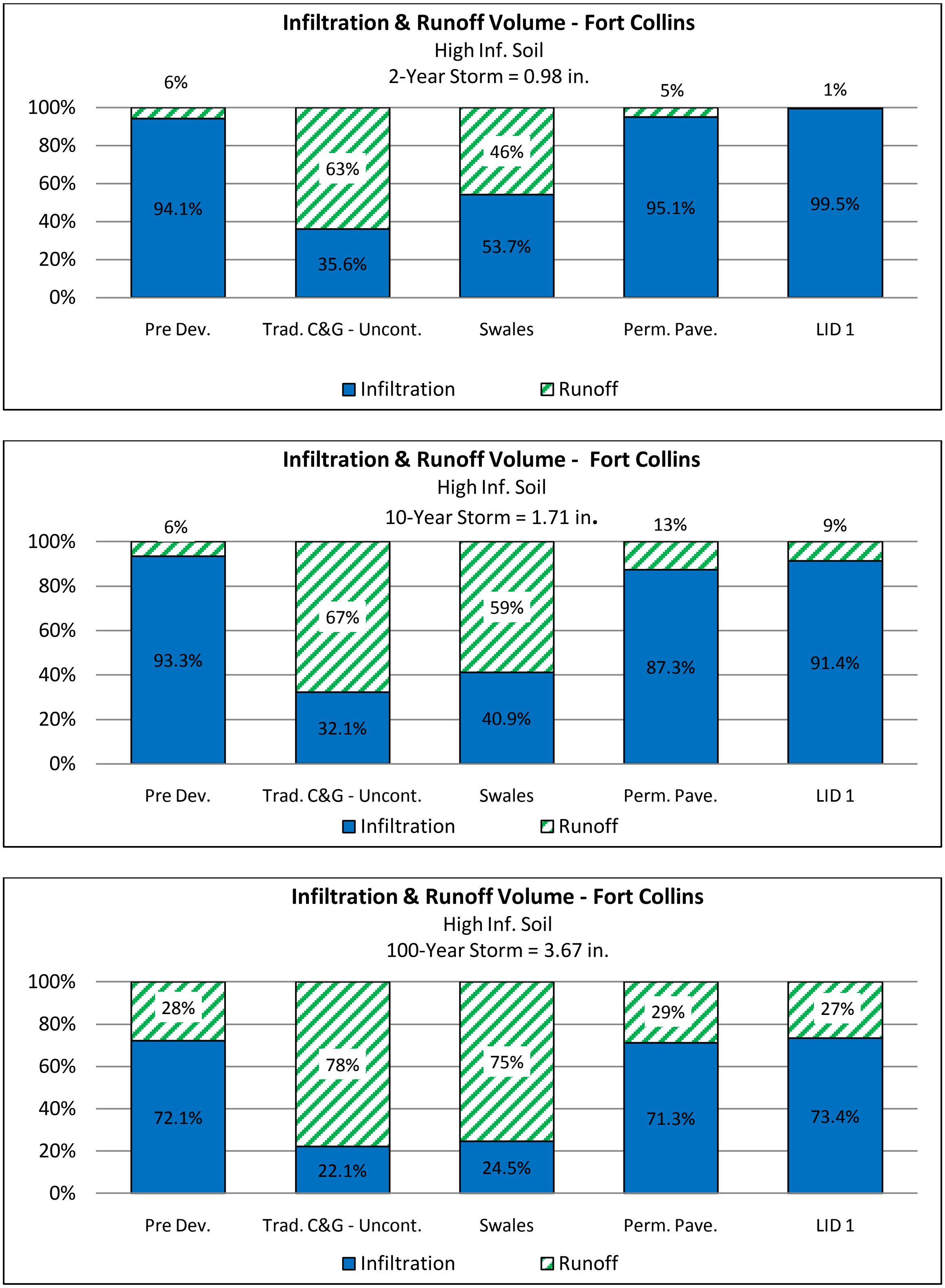


Figure 12.3-1 Water Balance Graphs – Fort Collins High Infiltration Soil

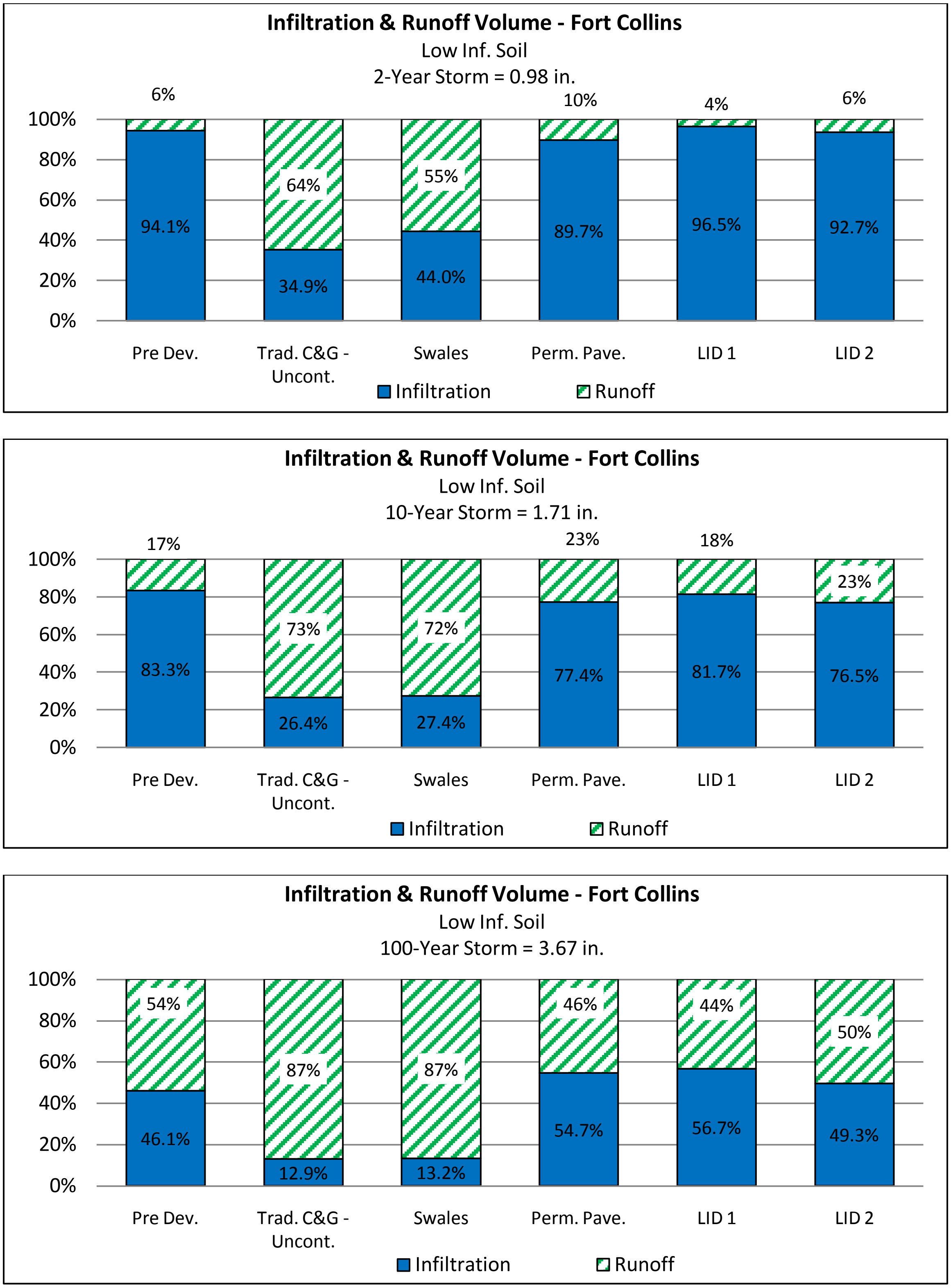


Figure 12.3-2 Water Balance Graphs – Fort Collins Low Infiltration Soil

12.4 Tabular Design Storm Results

Table 12.4-1 Fort Collins Design Storm Results – High Infiltration Soil

Models	Peak Discharge *cfs*	Runoff Volume *in.*	Infiltration Volume *in.*	% Runoff	% Infiltrate
		2-Year Storm = 0.98 in.			
Pre Dev.	1.2	0.06	0.92	5.7%	94.1%
Trad. C&G:					
Uncontrolled	10.8	0.62	0.35	63.5%	35.6%
BMP + Normal Control	1.2				
BMP + Over Control	0.4				
Swales	5.4	0.46	0.56	45.5%	53.7%
Perm. Pavement	0.4	0.05	0.93	5.0%	95.1%
LID 1	0.1	0.01	0.97	0.6%	99.5%
		10-Year Storm = 1.71 in.			
Pre Dev.	2.2	0.11	1.60	6.5%	93.3%
Trad. C&G:					
Uncontrolled	20.4	1.15	0.55	67.4%	32.1%
BMP + Normal Control	3.2				
BMP + Over Control	0.7				
Swales	13.9	1.00	0.70	58.7%	40.9%
Perm. Pavement	2.2	0.22	1.49	12.8%	87.3%
LID 1	1.4	0.15	1.56	8.7%	91.4%
		100-Year Storm = 3.67in.			
Pre Dev.	9.8	1.02	2.64	27.9%	72.1%
Trad. C&G:					
Uncontrolled	54.1	2.85	0.81	77.7%	22.1%
BMP + Normal Control	9.7				
BMP + Over Control	1.2				
Swales	42.5	2.76	0.90	75.3%	24.5%
Perm. Pavement	8.1	1.06	2.62	28.9%	71.3%
LID 1	7.6	0.98	2.69	26.8%	73.4%

Table 12.4-2 Fort Collins Design Storm Results – Low Infiltration Soil

Models	Peak Discharge *cfs*	Runoff Volume *in.*	Infiltration Volume *in.*	% Runoff	% Infiltrate
		2-Year Storm = 0.98 in.			
Pre Dev.	1.2	0.06	0.92	5.7%	94.1%
Trad. C&G:					
Uncontrolled	10.8	0.63	0.34	64.2%	34.9%
BMP + Normal Control	1.2				
BMP + Over Control	0.4				
Swales	6.1	0.54	0.43	55.2%	44.0%
Perm. Pavement	0.8	0.10	0.88	10.4%	89.7%
LID 1	0.2	0.04	0.94	3.7%	96.5%
LID 2	0.2	0.06	0.91	6.2%	92.7%
		10-Year Storm = 1.71 in.			
Pre Dev.	2.4	0.28	1.43	16.5%	83.3%
Trad. C&G:					
Uncontrolled	21.6	1.25	0.45	73.2%	26.4%
BMP + Normal Control	3.6				
BMP + Over Control	0.7				
Swales	15.3	1.23	0.47	72.1%	27.4%
Perm. Pavement	3.0	0.39	1.32	22.7%	77.4%
LID 1	2.1	0.32	1.40	18.4%	81.7%
LID 2	1.8	0.39	1.31	22.9%	76.5%
		100-Year Storm = 3.67in.			
Pre Dev.	11.9	1.98	1.69	53.9%	46.1%
Trad. C&G:					
Uncontrolled	55.8	3.19	0.47	87.0%	12.9%
BMP + Normal Control	11.9				
BMP + Over Control	1.2				
Swales	43.8	3.18	0.49	86.6%	13.2%
Perm. Pavement	13.4	1.67	2.01	45.6%	54.7%
LID 1	11.6	1.60	2.08	43.5%	56.7%
LID 2	11.0	1.85	1.81	50.4%	49.3%

13.0 APPENDIX D – Atlanta Design Storm Results

13.1 Design Storm Hydrographs – High Infiltration Soil

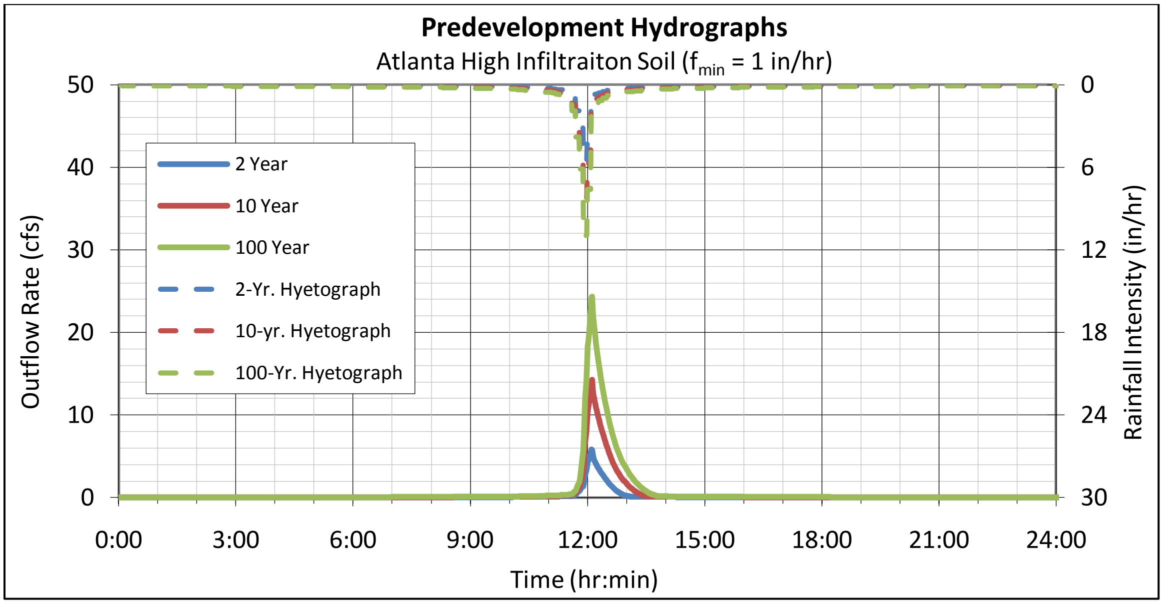


Figure 13.1-1 Predevelopment Hydrographs – Atlanta High Infiltration Soil

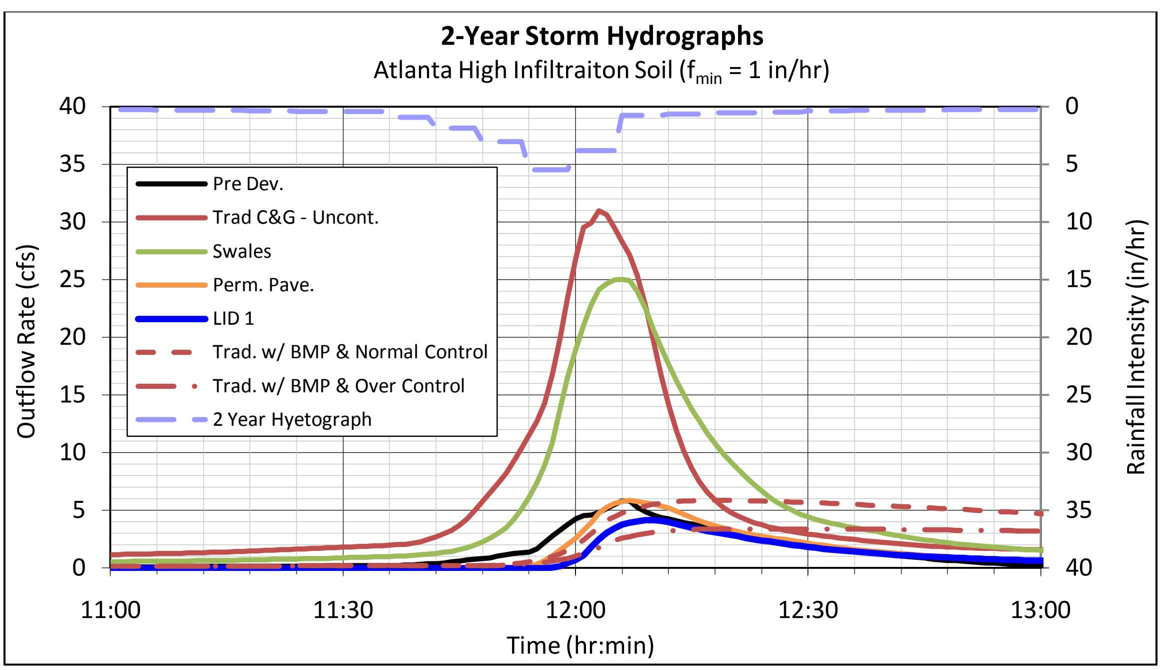


Figure 13.1-2 2-Year Storm Hydrographs – Atlanta High Infiltration Soil

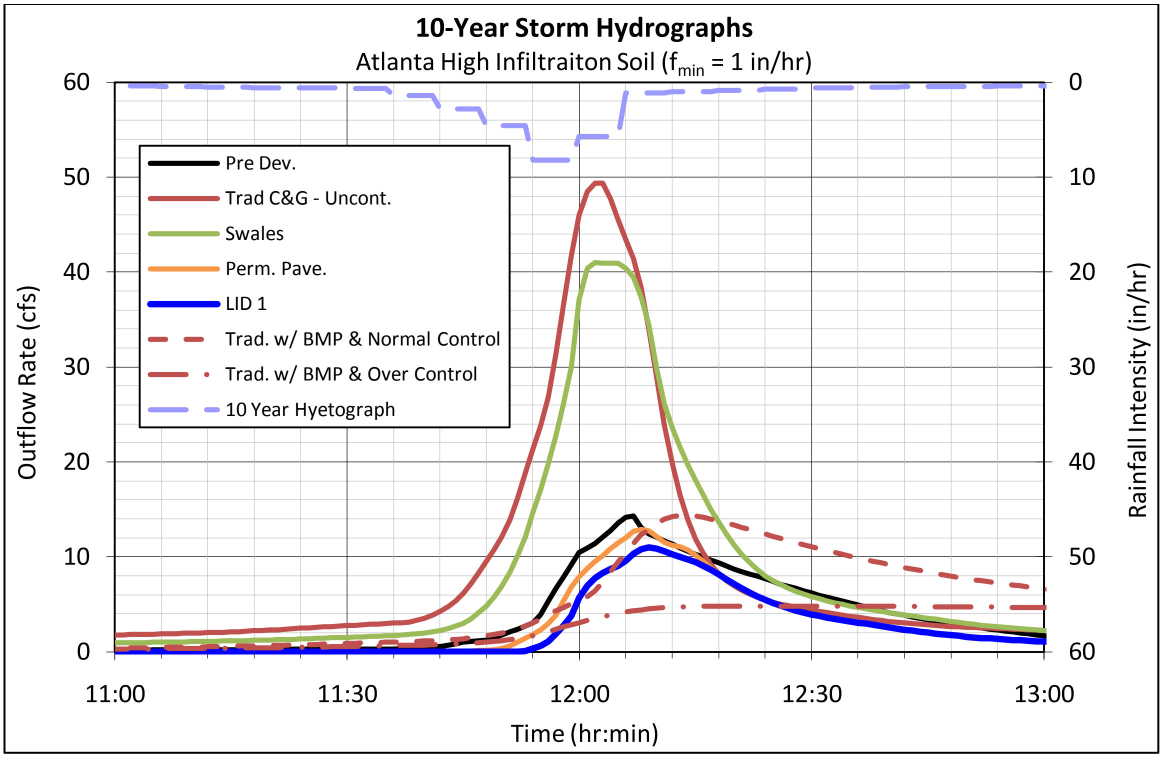


Figure 13.1-3 10-Year Storm Hydrographs – Atlanta High Infiltration Soil

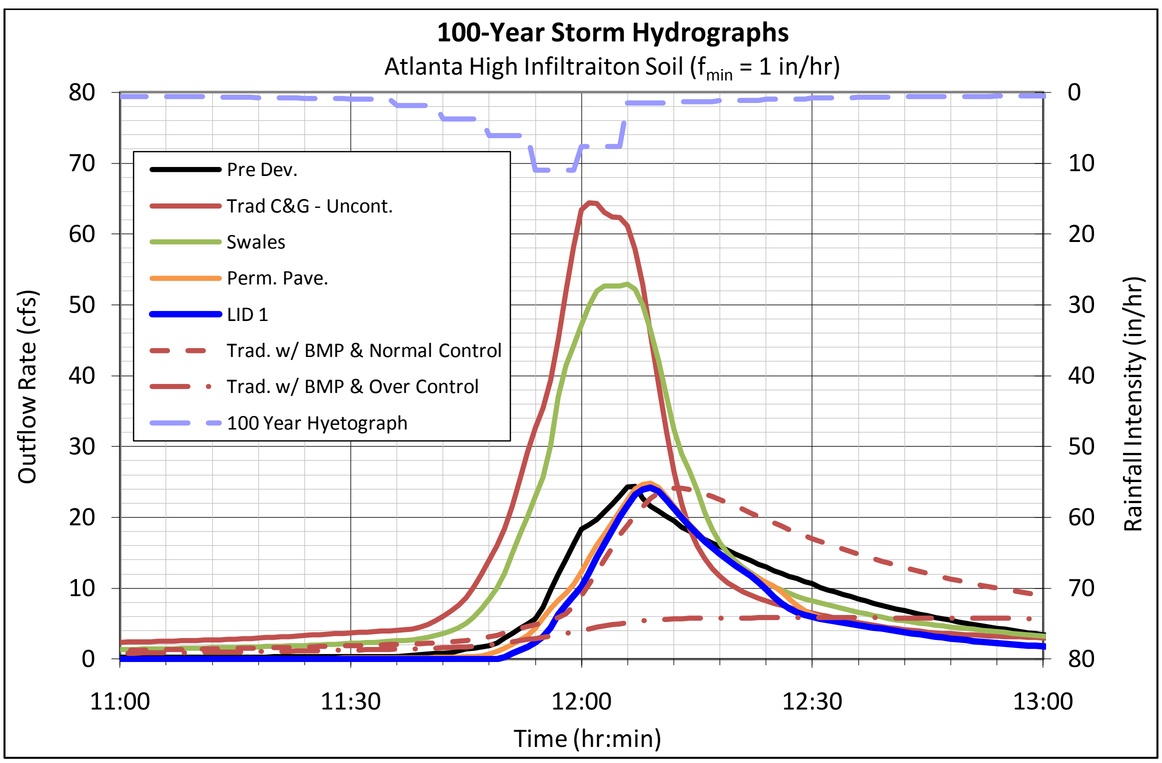


Figure 13.1-4 100-Year Storm Hydrographs – Atlanta High Infiltration Soil

13.2 Design Storm Hydrographs – Low Infiltration Soil

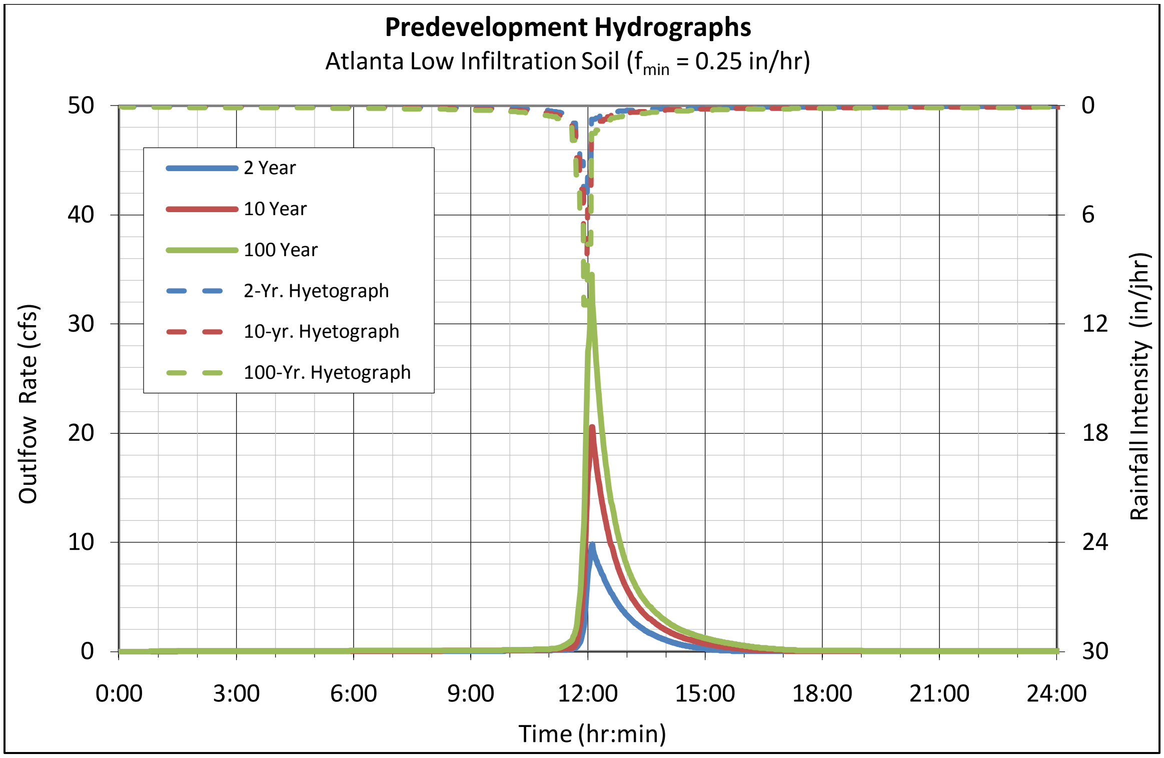


Figure 13.2-1 Predevelopment Hydrographs – Atlanta Low Infiltration Soil

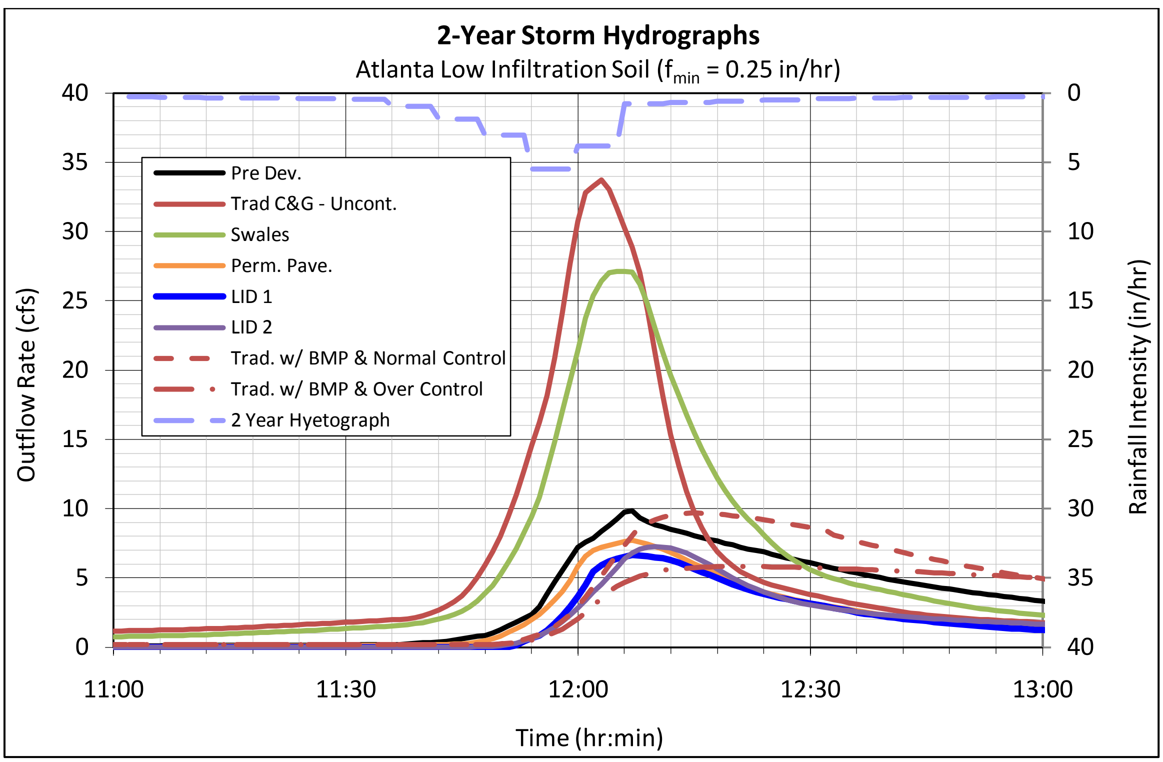


Figure 13.2-2 2-Year Storm Hydrographs – Atlanta Low Infiltration Soil

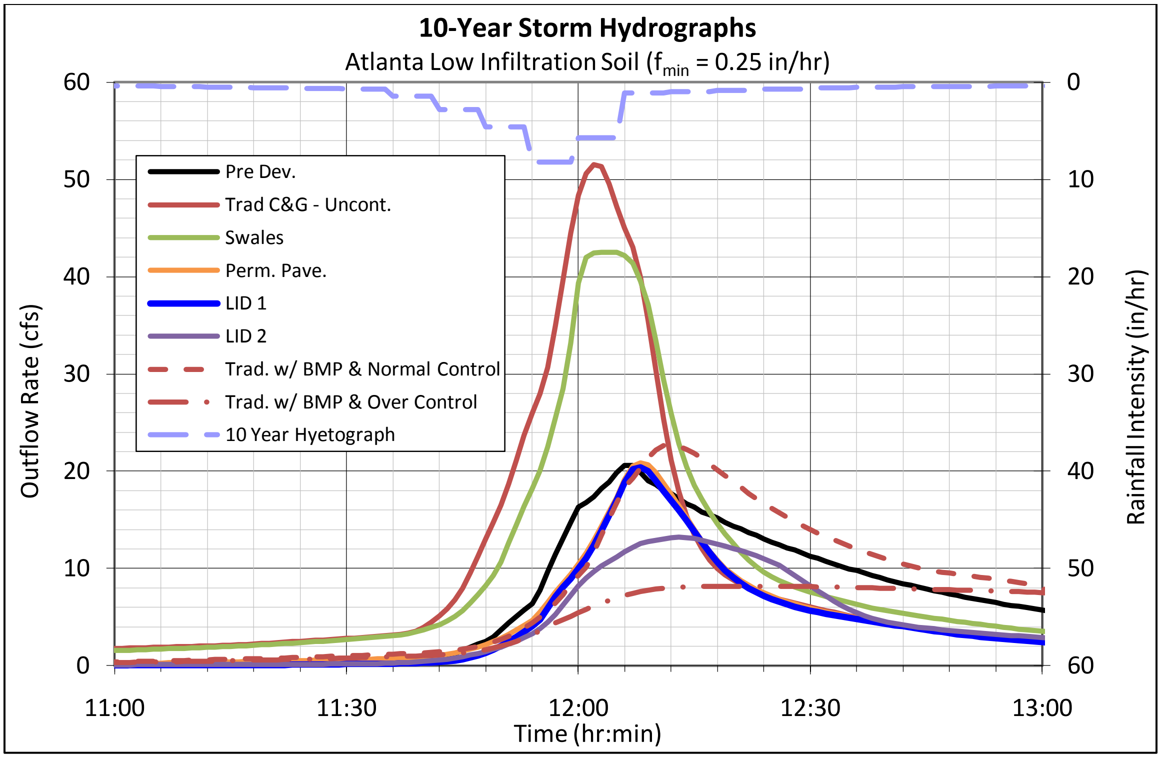


Figure 13.2-3 10-Year Storm Hydrographs – Atlanta Low Infiltration Soil

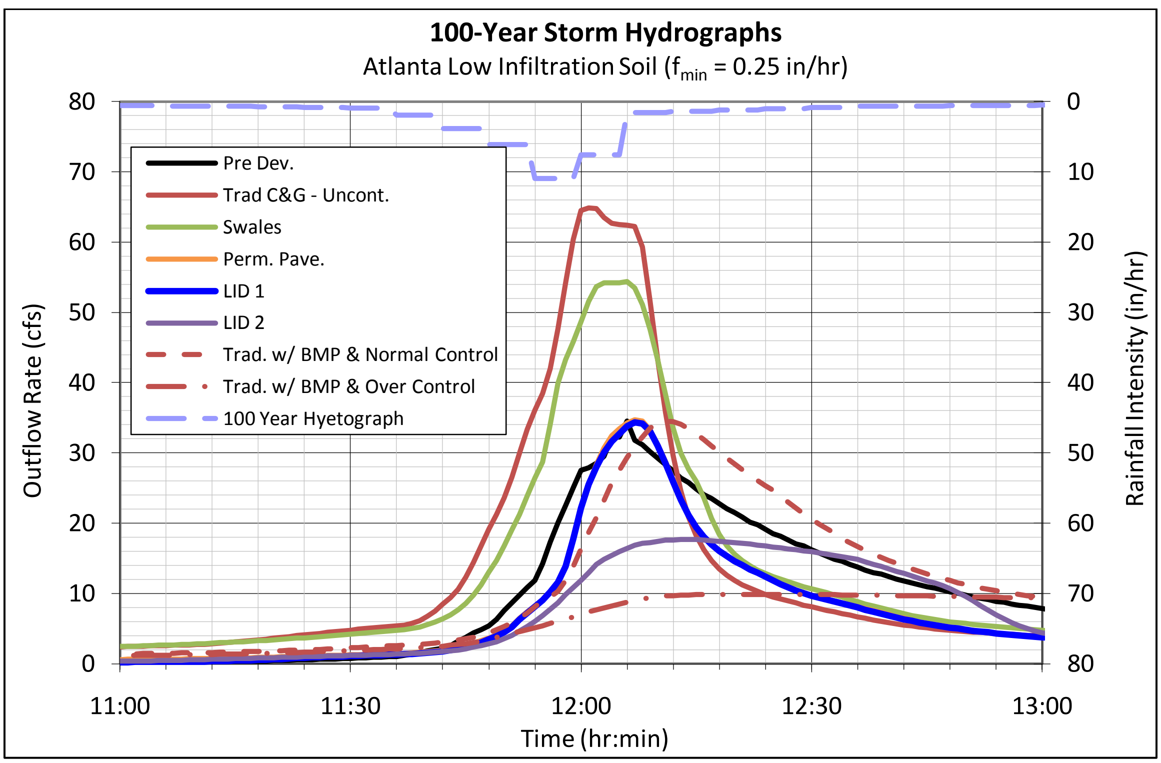
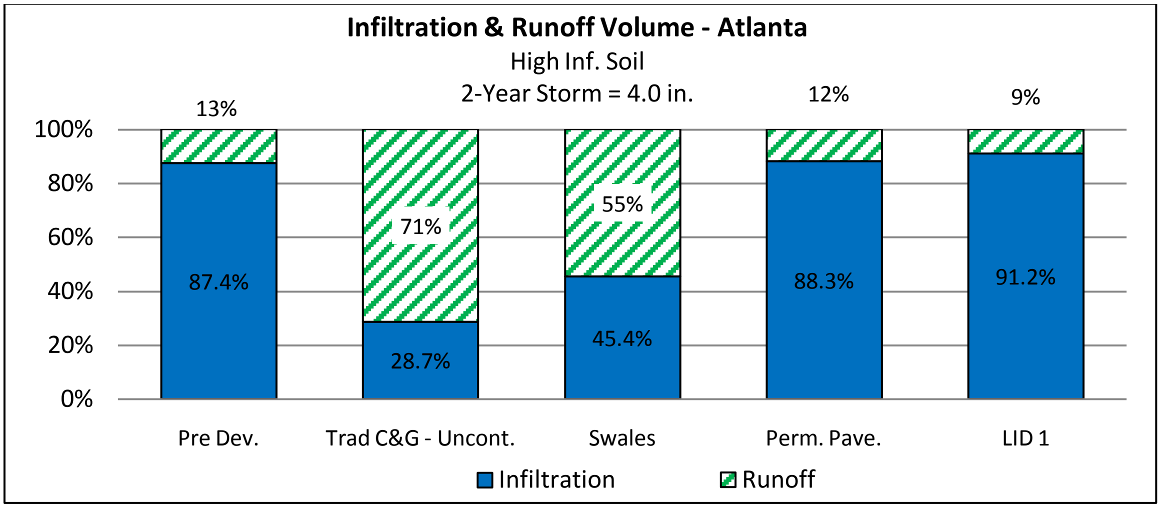
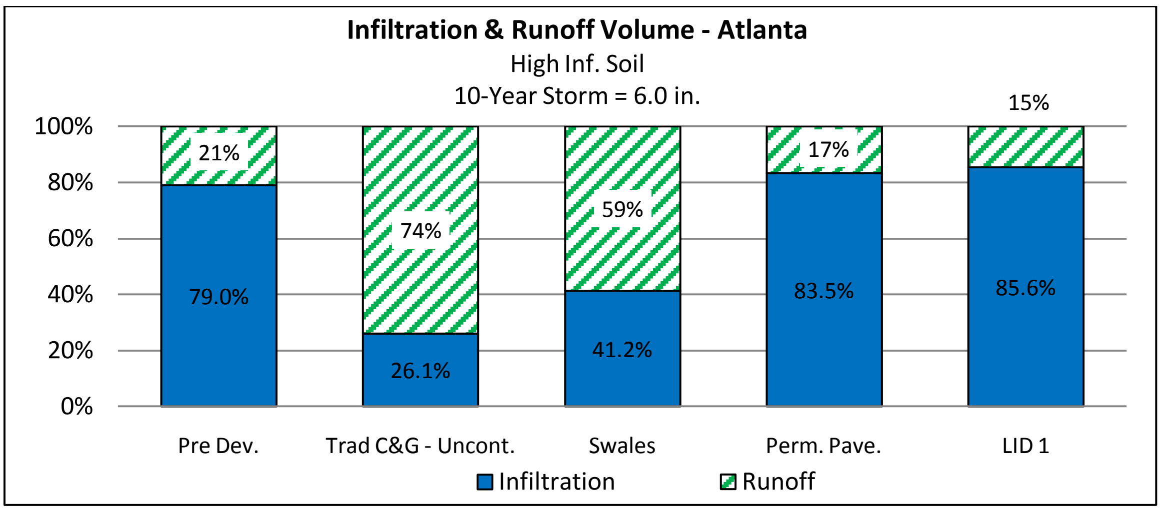


Figure 13.2-4 100-Year Storm Hydrographs – Atlanta Low Infiltration Soil

13.3 Design Storm Water Balance





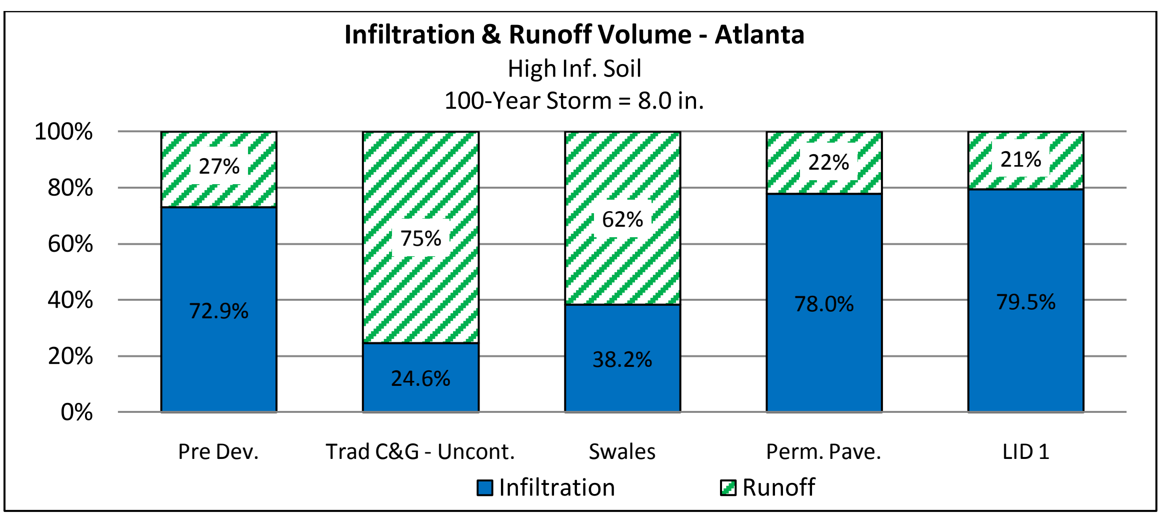
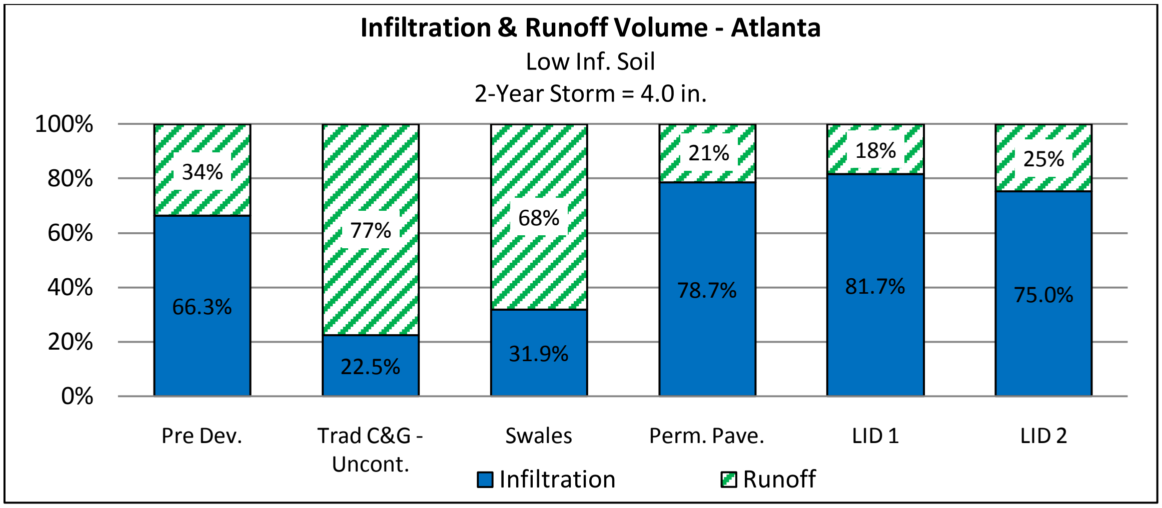
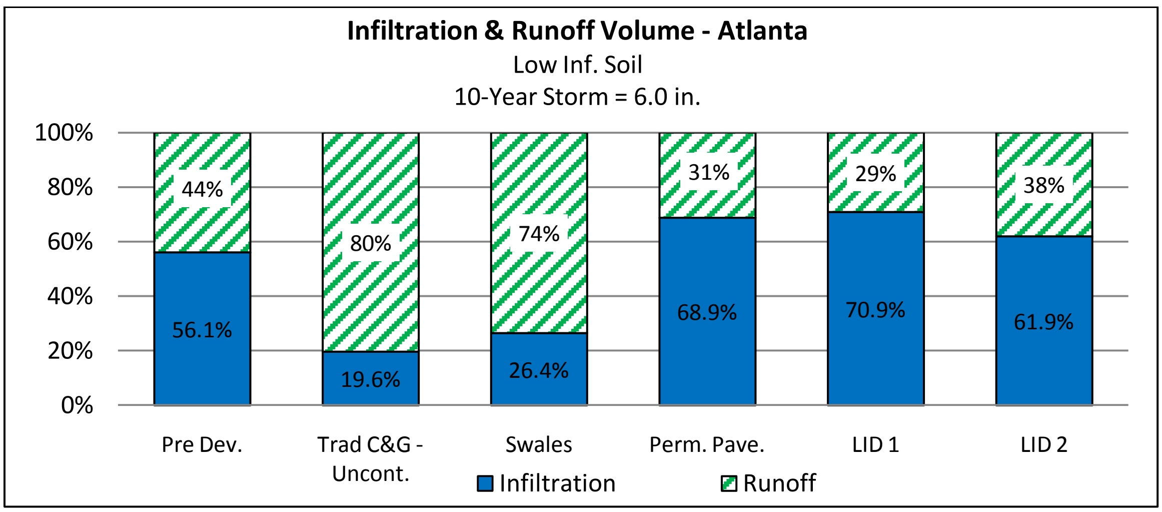


Figure 13.3-1 Water Balance Graphs – Atlanta High Infiltration Soil





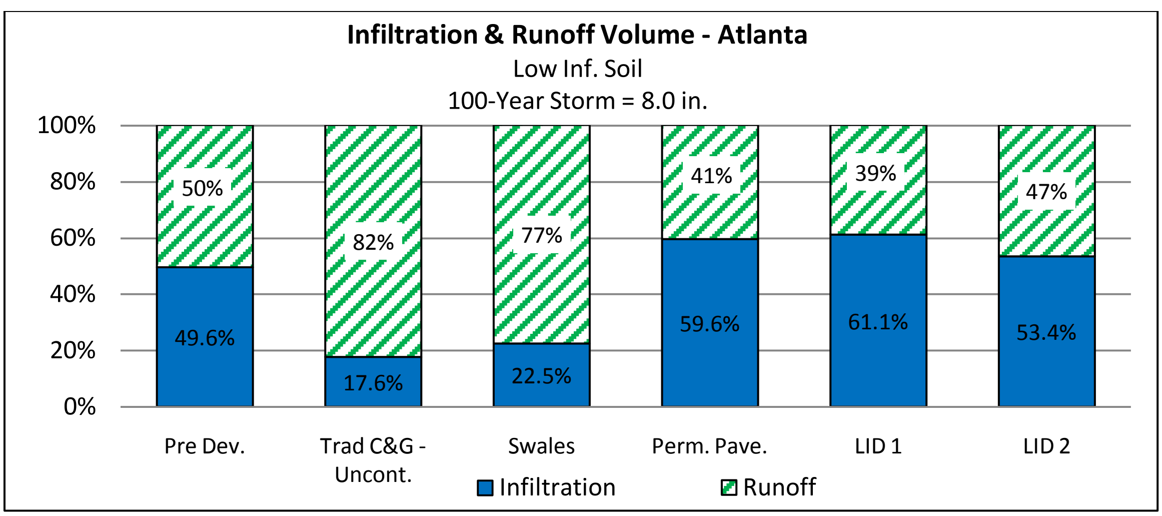


Figure 13.3-2 Water Balance Graphs – Atlanta Low Infiltration Soil

13.4 Tabular Design Storm Results

Table 13.4-1 Atlanta Design Storm Results – High Infiltration Soil

Models	Peak Discharge *cfs*	Runoff Volume *in.*	Infiltration Volume *in.*	% Runoff	% Infiltrate
		2-Year Storm = 4.00 in.			
Pre Dev.	5.9	0.50	3.50	12.6%	87.4%
Trad. C&G:					
Uncontrolled	31.0	2.85	1.15	71.2%	28.7%
BMP + Normal Control	5.9				
BMP + Over Control	3.4				
Swales	25.1	2.18	1.81	54.5%	45.4%
Perm. Pavement	5.8	0.47	3.53	11.8%	88.3%
LID 1	4.1	0.36	3.65	9.0%	91.2%
		10-Year Storm = 6.00 in.			
Pre Dev.	14.1	1.26	4.74	21.1%	79.0%
Trad. C&G:					
Uncontrolled	49.6	4.43	1.57	73.8%	26.1%
BMP + Normal Control	14.4				
BMP + Over Control	4.8				
Swales	41.0	3.53	2.47	58.8%	41.2%
Perm. Pavement	12.9	1.00	5.01	16.6%	83.5%
LID 1	11.0	0.88	5.14	14.6%	85.6%
		100-Year Storm = 8.00 in.			
Pre Dev.	24.3	2.17	5.83	27.1%	72.9%
Trad. C&G:					
Uncontrolled	64.4	6.03	1.97	75.3%	24.6%
BMP + Normal Control	24.1				
BMP + Over Control	5.8				
Swales	53.0	4.94	3.06	61.7%	38.2%
Perm. Pavement	24.9	1.78	6.24	22.2%	78.0%
LID 1	24.2	1.65	6.36	20.7%	79.5%

Table 13.4-2 Atlanta Design Storm Results – Low Infiltration Soil

Models	Peak Discharge *cfs*	Runoff Volume *in.*	Infiltration Volume *in.*	% Runoff	% Infiltrate
			2-Year Storm = 4.00 in.		
Pre Dev.	9.8	1.35	2.65	33.7%	66.3%
Trad. C&G:					
Uncontrolled	33.7	3.09	0.90	77.4%	22.5%
BMP + Normal Control	9.7				
BMP + Over Control	5.8				
Swales	27.2	2.72	1.28	68.0%	31.9%
Perm. Pavement	7.7	0.86	3.15	21.5%	78.7%
LID 1	6.7	0.74	3.27	18.4%	81.7%
LID 2	7.3	0.99	3.00	24.8%	75.0%
			10-Year Storm = 6.00 in.		
Pre Dev.	20.7	2.63	3.37	43.9%	56.1%
Trad. C&G:					
Uncontrolled	51.7	4.82	1.18	80.3%	19.6%
BMP + Normal Control	22.7				
BMP + Over Control	8.2				
Swales	42.5	4.41	1.58	73.6%	26.4%
Perm. Pavement	20.9	1.88	4.13	31.3%	68.9%
LID 1	20.4	1.75	4.25	29.2%	70.9%
LID 2	13.2	2.28	3.71	38.0%	61.9%
			100-Year Storm = 8.00 in.		
Pre Dev.	34.7	4.03	3.97	50.4%	49.6%
Trad. C&G:					
Uncontrolled	64.9	6.59	1.41	82.3%	17.6%
BMP + Normal Control	34.6				
BMP + Over Control	9.8				
Swales	54.4	6.20	1.80	77.5%	22.5%
Perm. Pavement	34.8	3.24	4.77	40.5%	59.6%
LID 1	34.4	3.12	4.89	39.0%	61.1%
LID 2	17.7	3.72	4.27	46.5%	53.4%

14.0 APPENDIX E – Continuous Simulation Graph Procedures

14.1 Peak Flow Frequency Procedure:

1. Using the statistics function in SWMM, get a list of **<u>peak</u>** flows:
    a. Pick 'inter-event' time. 6 hours is good choice; depends on regions precipitation patterns.
    b. Cut off low flows, 0.005 cfs is good.
2. Paste list into Excel
3. Order by Magnitude
4. Rank them: 1 (largest) -> n_r (smallest), m = rank.
5. Probability of Occurrence in 1-Year:
    a. $P_{year} = \dfrac{m}{n_{years} + 1}$
    b. $Time_{\mathrm{Re}turn} = \dfrac{n_{years+1}}{m}$
6. Graph Exceedances per Year (Probability) on x-axis and Peak Flow Rate on y-axis. Log-log plots are useful for looking at high-frequency events.

14.2 Flow Duration Procedure:

1. Using the table function in SWMM, get a list of all outflow rates for the period of record. Save to a .txt file.
2. Number of records = N_r
3. Find min and max flow
4. Determine bin spacing:
   a. Since this will be plotted on a log-log chart, the bins need to be set up logarithmically.
   b. If the bins are too large, the low flows will not be plotted accurately.
   c. If the bins are too small, the high flows plot jaggedly and the graph is difficult to read or understand.
   d. Bin spacing algorithm:
      i. $y = 0.0001 \cdot e^{\left[0.075 \cdot x\right]}$

         y = bin center, cfs

         x = bin number
      ii. Add one bin for the very low flows: 0.0000001 cfs.
   e. Make sure that the largest flow is in a bin.
5. Count number that fall into each bin, n_i.
   a. If the data set is more than 1,048,576 values, Excel cannot handle it. Matlab (or another program) will need to be used to do this. See next page.
6. Probability of that bin is: $P_{bin} = \dfrac{n_i}{N_r}$
7. Cumulative Probability is : $P_{cum} = P_{bin} + P_{cum\_previous\_bin}$
   a. Sum from the Highest flow rate to the lowest, so that the bin with the lowest flow rate has an exceedance probability of 100%.

8. Plot Discharge (y-axis) vs. Cum. Probability (x-axis). Use a log-log plot.

14.2.1 How to sort flow data into bins with Matlab:

1. A .txt file can be loaded as data by:
    a. Import Data (button in Workspace window, or under file menu)
    b. Select .txt file with outflows.
    c. Import Wizard Dialog Box:
        i. Select "Tab" Column Separator
        ii. Next
        iii. Uncheck "textdata". Make sure that only "data" is checked.
        iv. Finish
    d. Rename the variable from "data" to a more specific name to avoid confusion.
2. Click "New Variable" button.
    a. Name the new variable "bins"
3. The maximum value in a data set can be found by:
    a. max(*variable name*)
4. Using Excel, create the list of bins and copy this list into Matlab.
    a. Double click on "bins" variable.
    b. Click in the "Variable Editor" window.
    c. Right click and paste the bin list into Matlab.
5. To sort the data into bins:
    a. Click the 'New' button
    b. Type:

```
[sort]=hist(variable name,bins);
[sort]= sort';
```

6. Click Run
7. A new variable should appear in the Workspace window, with the name "sort". It should have the same dimensions as the bins variable.
8. Copy this list back to Excel.
    a. Hint: if there is Excel data in the clipboard the Matlab data will not be able to be copied. Go to Excel and double click on any cell to clear the clipboard.

15.0 APPENDIX F – Cost Comparison Data

15.1 Cost Summary

Table 15.1-1 Cost Comparison Summary – Fort Collins

	LID 1 High Inf.	LID 1 Low Inf.	LID 2 Low Inf.	Traditional
Model Components				
Storm Drain System	$46,790	$46,790	$46,790	$92,020
Curbs and Swales	$63,150	$63,150	$63,150	$73,650
Infiltration Trench	$4,640	$4,640	$4,640	-
Rain Garden	$8,880	$8,880	$8,880	-
Underdrain System	-	-	$122,280	-
Detention Pond	-	-	-	$36,680
Subtotal:	**$123,460**	**$123,460**	**$245,740**	**$202,350**
Paving (Pick One)				
Traditional Paving	-	-	-	$248,150
Permeable Pavers	$823,770	$810,630	$810,630	-
Pervious Concrete	$697,960	$684,820	$684,820	-
TOTALS:				
Traditional Paving	**-**	**-**	**-**	**$450,500**
Pavers	**$947,300**	**$934,100**	**$1,056,400**	
Pervious Concrete	**$821,500**	**$808,300**	**$930,600**	**-**

See Cost Calculation Section for Details

Table 15.1-2 Cost Comparison Summary – Atlanta

	LID 1 High Inf.	LID 1 Low Inf.	LID 2 Low Inf.	Traditional
Model Components				
Storm Drain System	$46,790	$46,790	$46,790	$92,020
Curbs and Swales	$63,150	$63,150	$63,150	$73,650
Infiltration Trench	$6,550	$6,550	$6,550	-
Rain Garden	$14,440	$14,440	$14,440	-
Underdrain System	-	-	$122,280	-
Detention Pond	-	-	-	$40,310
Subtotal:	**$130,930**	**$130,930**	**$253,210**	**$205,980**
Paving (Pick One)				
Traditional Paving	-	-	-	$248,150
Permeable Pavers	$823,770	$846,520	$846,520	-
Pervious Concrete	$697,960	$720,710	$720,710	-
TOTALS:				
Traditional Paving	**-**	**-**	**-**	**$454,200**
Pavers	**$954,700**	**$977,500**	**$1,099,800**	**-**
Pervious Concrete	**$828,900**	**$851,700**	**$974,000**	**-**

See Cost Calculation Section for Details

15.2 Cost Calculations

Table 15.2-1 Storm Drain System Cost Detail

Item	Units	Unit Cost	Quantity		Extended Cost	
			LID 1&2	Traditional	LID 1&2	Traditional
Curb Inlet	*ea.*	$2,500.00	3	6	$7,500	$15,000
Grate Inlet	*ea.*	$2,400.00	0	1	$0	$2,400
Area Inlet	*ea.*	$2,200.00	2	2	$4,400	$4,400
Pond Outlet Structure	*ea.*	$3,700.00	0	1	$0	$3,700
Manhole – 5 ft.	*ea.*	$2,400.00	3	3	$7,200	$7,200
Storm Pipe:						
12 in	*LF*	$23.25	34	0	$791	$0
15 in	*LF*	$36.75	165	165	$6,064	$6,064
18 in	*LF*	$41.25	246	894	$10,148	$36,878
24 in	*LF*	$44.75	180	301	$8,055	$13,470
FES:						
12 in	*ea.*	$342.00	1	0	$342	$0
15 in	*ea.*	$450.00	1	1	$450	$450
24 in	*ea.*	$613.00	3	4	$1,839	$2,452
				LID 1 & 2 Models Subtotal:	**$46,788**	
				Traditional Model Subtotal:		**$92,013**

Table 15.2-2 Curbs and Swales Cost Detail

Item	Units	Unit Cost	Quantity		Extended Cost	
			LID 1&2	Traditional	LID 1&2	Traditional
Flat Curb	*LF*	$8.00	5095	0	$40,760	$0
Rollover Curb	*LF*	$10.00	1755	1755	$17,550	$17,550
Vertical Curb	*LF*	$10.50	0	5095	$0	$53,498
Street Side Swale	*LF*	$1.22	1510	0	$1,842	$0
Large Swale	*LF*	$4.44	675	585	$2,997	$2,597
				LID 1 & 2 Models Subtotal:	**$63,149**	
				Traditional Model Subtotal:		**$73,645**

Table 15.2-3 Pavement Cost Detail - *Fort Collins Models*

Item	*Units*	Unit Cost	Quantity	Extended Cost
Traditional Pavement (Traditional C&G Model)				
Asphalt (5” HMA/ 8” Base)	*SY*	$20.68	4100	$84,829
Willox – Asphalt (9” HMA/ 6” Base)	*SY*	$30.93	2360	$72,995
Concrete (6”)	*SY*	$31.58	2860	$90,319
			Subtotal	**$248,143**
				$27 /SY
Permeable Pavement – LID 1 High Infiltration Model				
Pavers – Option 1	*SY*	$63.00	9319	$587,097
or				
Pervious Concrete – Option 2	*SY*	$49.50	9319	$461,291
Base	*CY*	$35.00	5594	$195,778
Horizontal Filter Fabric	*SY*	$1.99	9319	$18,545
Vertical Filter Fabric	*SY*	$5.04	1833	$9,238
Impermeable Cut-off Liners	*SY*	$22.14	592	$13,107
			Pavers Subtotal	**$823,765**
				$88 /SY
			Pervious Concrete Subtotal	**$697,959**
				$75 /SY
Permeable Pavement – LID 1 & LID 2 Low Infiltration Models				
Pavers – Option 1	*SY*	$63.00	9319	$587,097
or				
Pervious Concrete – Option 2	*SY*	$49.50	9319	$461,291
Base	*CY*	$35.00	5239	$183,365
Horizontal Filter Fabric	*SY*	$1.99	9319	$18,545
Vertical Filter Fabric	*SY*	$5.04	1689	$8,513
Impermeable Cut-off Liners	*SY*	$22.14	592	$13,107
			Pavers Subtotal	**$810,626**
				$87 /SY
			Pervious Concrete Subtotal	**$684,820**
				$73 /SY

Table 15.2-4 Pavement Cost Detail - ***Atlanta Models***

Item	*Units*	Unit Cost	Quantity	Extended Cost
Traditional Pavement (Traditional C&G Model)				
Asphalt (5" HMA/ 8" Base)	*SY*	$20.68	4100	$84,829
Willox – Asphalt (9" HMA/ 6" Base)	*SY*	$30.93	2360	$72,995
Concrete (6")	*SY*	$31.58	2860	$90,319
			Subtotal	**$248,143**
				$27 /SY
Permeable Pavement – LID 1 High Infiltration Model				
Pavers – Option 1	*SY*	$63.00	9319	$587,097
or				
Pervious Concrete – Option 2	*SY*	$49.50	9319	$461,291
Base	*CY*	$35.00	5594	$195,778
Horizontal Filter Fabric	*SY*	$1.99	9319	$18,545
Vertical Filter Fabric	*SY*	$5.04	1833	$9,238
Impermeable Cut-off Liners	*SY*	$22.14	592	$13,107
			Pavers Subtotal	**$823,765**
				$88 /SY
			Pervious Concrete Subtotal	**$697,959**
				$75 /SY
Permeable Pavement – LID 1 & LID 2 Low Infiltration Models				
Pavers – Option 1	*SY*	$63.00	9319	$587,097
or				
Pervious Concrete – Option 2	*SY*	$49.50	9319	$461,291
Base	*CY*	$35.00	6175	$216,113
Horizontal Filter Fabric	*SY*	$1.99	9319	$18,545
Vertical Filter Fabric	*SY*	$5.04	2018	$10,171
Impermeable Cut-off Liners	*SY*	$22.14	659	$14,590
			Pavers Subtotal	**$846,516**
				$91 /SY
			Pervious Concrete Subtotal	**$720,710**
				$77 /SY

Table 15.2-5 Infiltration Trench Cost Detail – *LID 1 & LID 2 Models*

	Units	Quantity	Cost	Quantity	Cost
		Fort Collins		Atlanta	
Infiltration Trench 1					
2" Pea Gravel	*CF*	33.3	$41.98	33.3	$41.98
Base Volume	*CF*	466.7	$570.37	806.7	$985.93
Filter Fabric - Side	*SF*	163	$30.46	274	$51.18
Filter Fabric - Bottom	*SF*	200	$14.74	200	$14.74
Excavation	*CF*	500	$125.00	840	$210.00
Subtotal			**$782.55**		**$1,303.83**
Infiltration Trench 2					
2" Pea Gravel	*CF*	83.3	$104.94	83.3	$104.94
Base Volume	*CF*	916.7	$1,120.37	1583.2	$1,934.98
Filter Fabric - Side	*SF*	206	$38.53	344	$64.22
Filter Fabric - Bottom	*SF*	500	$36.85	500	$36.85
Excavation	*CF*	1000	$250.00	1666.5	$416.63
Subtotal			**$1,550.69**		**$2,557.62**
Infiltration Trench 3					
2" Pea Gravel	*CF*	108.3	$136.42	108.3	$136.42
Base Volume	*CF*	866.7	$1,059.26	866.7	$1,059.26
Filter Fabric - Side	*SF*	177	$32.95	177	$32.95
Filter Fabric - Bottom	*SF*	650	$47.91	650	$47.91
Excavation	*CF*	975	$243.75	975	$243.75
Subtotal			**$1,520.29**		**$1,520.29**
Infiltration Trench 4					
2" Pea Gravel	*CF*	33.3	$41.98	33.3	$41.98
Base Volume	*CF*	466.7	$570.37	716.7	$875.93
Filter Fabric - Side	*SF*	163	$30.46	245	$45.70
Filter Fabric - Bottom	*SF*	200	$14.74	200	$14.74
Excavation	*CF*	500	$125.00	750	$187.50
Subtotal			**$782.55**		**$1,165.85**
Infiltration Trench Subtotals:		**Fort Collins**	**$4,636**	**Atlanta**	**$6,548**

For item unit costs see Table 15.3-1

Table 15.2-6 Rain Garden Cost Detail – *LID 1 & LID 2 Models*

Item	*Units*	Unit Cost	Quantity	Extended Cost
Price per 1 SY of Rain Garden *(RG depth is assumed to be 4' deep, 1 SY = 1.33 CY)*				
Sand (80%)/ Compost (20%) Fill	*CY*	$31.19	1.22	$38.05
Sand	*CY*	$30.44		
Compost/ Mulch Mix	*CY*	$34.16		
Wood Mulch (4")	*SY*	$3.00	1	$3.00
Plants	*SF*	$2.00	9	$18.00
Planter Excavation	*CY*	$11.05	1.33	$14.70
			Price per SY =	**$73.75**
		(assuming 4' depth, 1 SY = 1.33 CY)	**Price per CY =**	**$55.33**
Fort Collins Rain Gardens	*CY*	$55.33	160	**$8,873**
Atlanta Rain Gardens	*CY*	$55.33	261	**$14,437**

Table 15.2-7 Underdrain System Cost Detail – *LID 2 Model*

Item	*Units*	Unit Cost	Quantity	Extended Cost
4" Perf. Pipe	*LF*	$10.95	6500	$71,175
6" Main	*LF*	$6.05	3400	$20,570
12" Concrete Junction Box	*ea*	$710.00	43	$30,530
		Underdrain System Subtotal - *LID 2 Only*:		**$122,275**

Table 15.2-8 Detention Pond Cost Detail – *Traditional Model*

Item	*Units*	Unit Cost	Quantity		Extended Cost	
			Fort Collins	Atlanta	Fort Collins	Atlanta
Pond Grading	*SY*	$3.24	5830	6050	$18,889	$19,602
Pond Excavation	*CY*	$4.06	4380	5100	$17,783	$20,706
			Fort Collins Detention Pond Subtotal:		**$36,672**	
			Atlanta Detention Pond Subtotal:			**$40,308**

Assumes that entire pond volume is excavated and placed elsewhere on site

15.3 Item Unit Costs

Table 15.3-1 Item Unit Costs

Item	Thesis Cost	Unit	Source
Storm Drain System:			
Curb Inlet	$2,500.00	ea.	Union Place Bid
Grate Inlet	$2,400.00	ea.	Manhole with grated lid
Area Inlet	$2,200.00	ea.	Union Place Bid
Pond Outlet Structure	$3,700.00	ea.	Union Place Bid
Storm Pipe:			
12 in.	$23.25	LF	Union Place Bid + RSMeans for Exc. And Backfill
15 in.	$36.75	LF	Union Place Bid + RSMeans for Exc. And Backfill
18 in.	$41.25	LF	Union Place Bid + RSMeans for Exc. And Backfill
24 in.	$44.75	LF	Union Place Bid + RSMeans for Exc. And Backfill
Trench Excavation	$6.75	CY	RS Means
Backfill	$2.54	CY	RS Means
Flared End Section:			
12 in	$342.00	ea	RSMeans
15 in	$450.00	ea	Union Place Bid
18 in	$525.00	ea	Union Place Bid
24 in	$613.00	ea	Union Place Bid
Manhole (5' I.D.)	$2,400.00	ea	RS Means
UD System:			
4" Perf. Pipe	$10.95	LF	RS Means – Bedding is part of Perm. Pave. Base
6" Main	$6.05	LF	RS Means – Bedding is part of Perm. Pave. Base
Concrete Junction Box	$710.00	ea	RS Means for light duty Hand Hole Box + $100 for parts in the box
Curbs and Swales:			
Flat Curb	$8.00	LF	Union Place Bid
Rollover Curb	$10.00	LF	Union Place Bid
Vertical Curb	$10.50	LF	Union Place Bid
Street Side Swale (5.5’ wide)	$1.22	LF	$2/ SY -RS Mean Irreg. Grading. No Seeding
Large Swale	$4.44	LF	$2/ SY -RS Mean Irreg. Grading. No Seeding
Regular Paving:			
Asphalt – Residential (5" Asph., 8" Base)	$20.69	SY	Union Place Bid + RS Means for Exc.
Asphalt - Major Road (9" Asph., 6" Base)	$30.93	SY	Union Place Bid + RS Means for Exc.
6" Concrete	$31.58	SY	Union Place Bid + RS Means for Exc.

Item	Thesis Cost	Unit	Source
Permeable Paving:			
Pavers	$63.00	SY	Union Place Bid ($7/ SF)
Pervious Concrete	$49.50	SY	Colorado Hardscapes ($5.50/ SF)
Base:			
3/4" Base - compacted	$21.50	CY	RS Means
Open Graded Base (Installed and Compacted)	$35.00	CY	Based off of RS Means cost of base plus difference between quarry costs of regular base and open graded 57/67 stone. Compares well with the cost of open graded base + hauling + excav. + backfill.
Filter Fabric:			
Horizontal Surface	$1.99	SY	RS Means
Vertical Surface	$5.04	SY	RS Means
Impermeable Liner (6 mil)	$22.14	SY	RS Means x 2 for labor intensive installation
Scraper Excavation	$4.06	CY	RS Means
Infiltration Trench:			
Excavation	$6.75	CY	RS Means
Pea Gravel	$34.00	CY	RS Means. Compares with costs from LaFarge
3/4" Gravel	$33.00	CY	RS Means. Compares with costs from LaFarge
Rain Gardens			
Peat/ Sand Fill *(not used)*	$53.25	CY	RS Means
Sand/ Compost Fill	$31.19	CY	80% Sand, 20% Compost/ Mulch Mix *(used instead of Peat mix)*
Sand	$30.44	CY	Hageman's + Hauling + RSMeans Backfill.
Compost/ Mulch Mix	$34.16	CY	Hageman's + Hauling + RSMeans Backfill.
Wood Mulch (4")	$3.00	SY	Hageman's + $1/ SY to spread
Plants	$2.00	SF	Rain Garden Design Guide
Planter Excavation	$11.05	CY	RS Means
Pond Grading			
Lagoon Grading	$3.24	SY	RS Means. No seeding
Scraper Excavation	$4.06	CY	RS Means. No seeding.

RSMeans costs were adjusted with the RSMeans City Cost Indexes to the City of Fort Collins

Sources: RSMeans (Spencer, 2008)

Hagaman (Hageman, 2009)

Colo. Hardscapes Inc. (Buteyn, 2009)

The Union Place Bid was made available by Nolte Associates.

15.4 Quantities

Table 15.4-1 Fort Collins Models Quantities

		Fort Collins Models			
Quantities	**Units**	**LID 1 High Inf.**	**LID 1 Low Inf.**	**LID 2 Low Inf.**	**Traditional**
Inlets:					
Curb Inlet	ea.	3	3	3	6
Grate Inlet	ea.	0	0	0	1
Area Inlet	ea.	2	2	2	2
Pond Outlet Structure	ea.	0	0	0	1
Storm Pipe:					
12 in RCP	LF	34	34	34	
15 in RCP	LF	165	165	165	165
18 in RCP	LF	246	246	246	894
24 in RCP	LF	180	180	180	301
Flared End Section	ea.	5	5	5	5
Man Hole	ea.	3	3	3	3
Underdrain System:					
Perforated UD Pipe	LF			6500	
6" UD Main	LF			3400	
UD Manhole	ea.			43	
Regular Paving:					
Asphalt	SY				4100
Willox - Asphalt	SY				2360
Concrete	SY				2860
Permeable Paving:					
18" Base	SY	7096	8160	8160	
24" Base	SY	0	0	0	
30" Base	SY	1064	0	0	
36" Base	SY	1159	1159	1159	
Impermeable Liner Cut-off Walls	SY	592	592	592	
Vertical Filter Fabric	SY	1833	1689	1689	

Fort Collins Models					
Quantities	Units	LID 1 High Inf.	LID 1 Low Inf.	LID 2 Low Inf.	Traditional
Curbs and Swales:					
Flat Curb	LF	5095	5095	5095	0
Rollover Curb	LF	1755	1755	1755	1755
Vertical Curb	LF	0	0	0	5095
Street Side Swale	LF	1510	1510	1510	
Large Swale	LF	675	675	675	585
Infiltration Trenches:					
Trench 1					
Area	SF	200	200	200	
Depth	ft	2.5	2.5	2.5	
Volume	CF	500	500	500	
Trench 2					
Area	SF	500	500	500	
Depth	ft	2	2	2	
Volume	CF	1000	1000	1000	
Trench 3					
Area	SF	650	650	650	
Depth	ft	1.5	1.5	1.5	
Volume	CF	975	975	975	
Trench 4					
Area	SF	200	200	200	
Depth	ft	2.5	2.5	2.5	
Volume	CF	500	500	500	
Rain Gardens:					
Number	ea.	18	18	18	
Total Volume	CF	4330	4330	4330	
Detention Pond:					
Detention Pond Area	ac.				1.20
Pond Grading	SY				5830
Pond Excavation	CY				4380

Table 15.4-2 Atlanta Models Quantities

		Atlanta Models			
Quantities	**Units**	**LID 1 High Inf.**	**LID 1 Low Inf.**	**LID 2 Low Inf.**	**Traditional**
Inlets:					
Curb Inlet	ea.	3	3	3	6
Grate Inlet	ea.	0	0	0	1
Area Inlet	ea.	2	2	2	2
Pond Outlet Structure	ea.	0	0	0	1
Storm Pipe:					
12 in RCP	LF	34	34	34	
15 in RCP	LF	165	165	165	165
18 in RCP	LF	246	246	246	894
24 in RCP	LF	180	180	180	301
Flared End Section	ea.	5	5	5	5
Man Hole	ea.	3	3	3	3
Underdrain System:					
Perforated UD Pipe	LF			6500	
6" UD Main	LF			3400	
UD Manhole	ea.			43	
Regular Paving:					
Asphalt	SY				4100
Willox - Asphalt	SY				2360
Concrete	SY				2860
Permeable Paving:					
18" Base	SY	7096	4674	4674	
24" Base	SY	0	2422	2422	
30" Base	SY	1064	0	0	
36" Base	SY	1159	2223	2223	
Impermeable Liner Cut-off Walls	SY	592	659	659	
Vertical Filter Fabric	SY	1833	2018	2018	

Quantities	Units	Atlanta Models LID 1 High Inf.	LID 1 Low Inf.	LID 2 Low Inf.	Traditional
Curbs and Swales:					
Flat Curb	LF	5095	5095	5095	0
Rollover Curb	LF	1755	1755	1755	1755
Vertical Curb	LF	0	0	0	5095
Street Side Swale	LF	1510	1510	1510	
Large Swale	LF	675	675	675	585
Infiltration Trenches:					
Trench 1					
Area	SF	200	200	200	
Depth	ft	4.2	4.2	4.2	
Volume	CF	840	840	840	
Trench 2					
Area	SF	500	500	500	
Depth	ft	3.3	3.3	3.3	
Volume	CF	1667	1667	1667	
Trench 3					
Area	SF	650	650	650	
Depth	ft	1.5	1.5	1.5	
Volume	CF	975	975	975	
Trench 4					
Area	SF	200	200	200	
Depth	ft	3.75	3.75	3.75	
Volume	CF	750	750	750	
Rain Gardens:					
Number	ea.	18	18	18	
Total Volume	CF	7045	7045	7045	
Detention Pond:					
Detention Pond Area	ac.				1.25
Pond Grading	SY				6050
Pond Excavation	CY				5100