THESIS

COMPARING MULTI-LEVEL AND FULL SPECTRUM DETENTION DESIGN FOR URBAN STORMWATER DETENTION FACILITIES

Submitted by

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In partial fulfillment of the requirements

For the Degree of Master of Science

Colorado State University

Fort Collins, Colorado

Summer 2010

COLORADO STATE UNIVERSITY

June 23, 2010

WE HEREBY RECOMMEND THAT THE THESIS PREPARED UNDER OUR SUPERVISION BY XIAOJU ZHANG ENTITLED COMPARING MULTI-LEVEL AND FULL SPECTRUM DETENTION DESIGN FOR URBAN STORMWATER DETENTION FACILITIES BE ACCEPTED AS FULFILLING IN PART REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE.

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ABSTRACT OF THESIS

COMPARING MULTI-LEVEL AND FULL SPECTRUM DETENTION DESIGN FOR URBAN STORMWATER DETENTION FACILITIES

Peak flow attenuation and water quality control are widely used in urban stormwater systems. Standard practice typically involves peak shaving of post-development flows to pre-development peak flow levels to control flood flows and best management practices (BMPs) for removing pollutants from runoff. Usually both practices are integrated by using *Multi-level Detention* ponds. Recently, Wulliman and Urbonas (2005 and 2007) have proposed the so-called *Full Spectrum Detention* approach to design detention facilities able to control runoff events. This method is based on the concept of capturing the *Excess Urban Runoff Volume* (EURV) that results from urbanization and releasing it over a period of 72 hours. This method has been tested successfully for the Denver region and excellent matching of post-development peak flows to pre-development peak flows has been achieved. However, these results have been obtained using discrete design storms and the model has not been studied using a continuous simulation approach. Continuous simulations are useful because they provide information about the long-term performance through peak flow exceedance frequency and flow duration curves. Moreover, these results can be used to define the stream erosion potential, a metric that characterizes the geomorphic stability of urban streams. Continuous simulation *has* been successfully used to characterize the performance of

Multi-level Detention method, which uses combined peak shaving and extended detention practices, and protocols to reduce urbanization impacts in different locations have been demonstrated with it.

This study compares the effectiveness and differences of the *Multi-level Detention* design approach with that of the *Full Spectrum Detention* approach through the use of design storms and 60-year continuous precipitation records in a conceptual watershed for two different climate regions in the United States. The US EPA Stormwater Management Model (SWMM) is used to simulate the response of a conceptual watershed using both design approaches. Sensitivity analysis of the land-use properties is performed in order to validate the assumptions of the *Full Spectrum Detention* method by using both Colorado Unit Hydrograph Procedure (CUHP) and SWMM models. The performances of these two design approaches are tested initially by comparing the post-development peak flows for different design storms with the pre-development conditions. Additionally, 60 years of hourly rainfall records are used to run continuous simulations and compute peak flow frequency exceedance curves, flow duration curves, the hydrologic metrics $T_{0.5}$, and average boundary shear stress curves, which are used to compute the stream erosion potential. The differences of both design methods are assessed by comparing the post-development results with those obtained for the pre-development conditions.

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ACKNOWLEDGEMENTS

I am very grateful for the support, guidance and encouragement provided by Dr. Larry Roesner, my advisor. Thank you, Dr. Roesner, for inviting me to the Harold Short Laboratory, coaching me through the thesis, and teaching me to be a real engineer step by step. I also appreciate the input provided by the other members of my graduate committee, Dr. Neil Grigg, Dr. Stephanie Kampf and Dr. Ellen Wohl.

Thanks to Christine Pomeroy for her timely assistance in the peak flow frequency curve and flow duration curve in this thesis.

Thanks to the graduate students in the HHS Lab who have helped me over the years: Adam Jorkest, Daeyong Park, Chris Orson, and especially Jorge Gironas who is always eager to lend a hand.

Thank you to Qing Xu, my best friend in the same department. Her patience and encouragement helped me to get through the difficult time in the whole graduate study.

I thank Xiangjiang Huang for his help and encouragement. I also appreciate the input he has given on my research, and his willingness to answer questions night and day.

Thanks to Qian Liang for his assisted with average boundary shear stress curves calculation.

Thanks to my family for their love and patience throughout my life.

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1 INTRODUCTION

When a watershed is developed, the land is covered with impervious surfaces such as roads, parking lots, roofs and sidewalks of concrete or asphalt. These manmade changes to the land significantly restrict the amount of water that is allowed to infiltrate into the ground and increase both the amount and the rate of surface runoff flowing into drainage channels and receiving water bodies. Increases in the magnitude of stormwater runoff that accompany uncontrolled development result in serious implications for the long-term sustainability and health of stream systems, such as deeper and more frequent downstream flooding, aquatic habitat degradation, accelerating stream bank erosion and downstream pollutant loading.

Many municipalities and agencies are required to control stormwater to some level when development or redevelopment takes place within a jurisdiction. The stormwater control practices range from flood prevention to water quality protection. Peak flow attenuation and water quality control are widely used in urban stormwater systems. Standard practice typically involves peak shaving of post-development flows to pre-development peak flow levels plus best management practices (BMPs) for removing pollutants from runoff for given return interval storms (Nehrke and Roesner, 2004). Usually the peak-shaving detention and the BMP facilities are integrated by using the multi-level stormwater control facilities. Recently, Wulliman and Urbonas (2005 and 2007) have proposed the *Full Spectrum Detention* approach to design detention facilities able to control runoff events. This method is based on the concept of capturing the *Excess Urban Runoff Volume* (EURV) that results from urbanization to attenuate the peak discharges, and releasing it over a period of 72 hours for water quality control. The *Full Spectrum Detention* approach has been tested successfully by using discrete design storms for the Denver region using the Colorado Unit Hydrograph Procedure (CUHP) model and excellent matching of pre-development peak flows has been achieved. But the approach has not been tested with respect to its ability to control the post-development flow duration curve to pre-development levels, nor to control erosion potential or hydrologic metrics related to stream ecologic health. Researchers from Colorado (Nehrke and Roesner, Rohrer, et. al, 2004) showed that the *Multi-level Detention* method provides an excellent match of the post-development peak flow frequency curve to pre-development conditions.

By using design storms and continuous precipitation modeling, this study examines the performance of the two detention design approaches that best match the peak flow frequency curve, flow duration curve, average boundary shear stress curve and $T_{0.5}$ with the pre-development conditions in a conceptual watershed with two different climate regions: Fort Collins, Colorado and Atlanta, Georgia. The frequency, intensity, and duration of precipitation vary greatly in these watersheds, which generate different runoff responses. In the semiarid climate of Fort Collins, precipitation is highly-variable, and often occurs at high intensities, for a limited duration, with an average precipitation of 18 inches/year. The humid subtropical climate of Atlanta, Georgia, has more frequent precipitation of variable intensity and long duration, with an average precipitation of 50.2 inches/year. In both climates, after urbanization takes place runoff rates respond much more directly to rainfall intensities, in time, magnitude, and duration (Roesner and Bledsoe, 2003). The varied frequencies, magnitudes, and durations of stormwater runoff produced by different climates make climate an important factor in deciding which stormwater controls are more effective for minimizing effects due to urbanization. Studies were conducted of the peak flow frequency curves, flow-duration curves to evaluate the flow regime; average boundary shear stress curves to estimate stream erosion potential; and stream metrics $T_{0.5}$ which are used to evaluate the Benthic Macroinvertebrate Index (B-IBI) in a receiving stream.

The study serves four goals: (1) test the assumptions of *Full Spectrum Detention* approach through a conceptual watershed by using both SWMM and CUHP models; (2) examine the applicability of *Full Spectrum Detention* approach in the regions other than Denver area; (3) compare the performance of *Full Spectrum Detention* approach by using different *Excess Urban Runoff Volume* (EURV) for pond design; and (4) assess the differences of the *Multi-level* and *Full Spectrum* stormwater detention ponds by comparing the peak flow frequency curves, flow duration curves, average boundary shear stress curves and $T_{0.5}$ of the watershed in its developed condition with each other

Chapter Two of this document presents a review of relevant literature. In Chapter Three, the study watershed is presented, a validation process of the *Full Spectrum Detention* approach is performed, and detention ponds used for analysis are designed for each climate region with the two design approaches. In Chapter Four, results of the analysis are presented. Chapter Five includes conclusions and recommendations for future research.

2 LITERATURE REVIEW

2.1 Problem statement

Greater runoff volume, increased peak discharges, more rapid response times, and variations in sediment production often occur during urbanization (Bledsoe and Watson, 2001; White and Greer, 2006; Rose and Peters, 2001), causing great stress on ecology, morphology and flood control for river managers (Gregory, 2002). The earliest documentation of increased runoff from urban areas was in the late 1800s (Kuichling 1889), and urban runoff continues to be a leading cause of impairments in waterways (US EPA 2002).

Typically, urbanization refers to urban development and is characterized by increasing land surface imperviousness, such as driveways, roofs, and parking lots, etc. The change of land use and increase of impervious surfaces raise the flood magnitude; shorten the lag time from the center of the rainfall volume to the center of the runoff volume; and increase the temporal and spatial variation in streamflow conditions (Espey et al. 1965; Seaburn1969; Hirsch et al. 1990; Arnold and Gibbons 1996; Beighley and Moglen 2002; Randolph 2004; Dougherty et al. 2006). Stream flood hydrology changes along with the rainfall–runoff relationship in response to the expansion of the impervious surface in the watershed.

Many studies have examined the combined effects of land use and climate changes on hydrologic regimes. Claessens et al. (2006) studied the long-term effects of changes in land use and climate in suburban watersheds and other unique aspects of urban hydrology. Reynard et al. (2001) used a continuous flow simulation model to assess future impacts of climate and land use changes on floods in large watersheds in the United Kingdom. They found that urbanization has a large effect on flood regimes, increasing both the frequency and magnitude of floods, significantly beyond the changes due to climate change alone. By studying the urbanizing watersheds in northeastern Illinois, Hejazi et al. (2009) indicated that both the increasing precipitation and urbanization in the watersheds appear to be major contributors to the increasing peaks, but the average contribution of urbanization to increases in flood peaks was 34 percent higher than that of the increase in precipitation.

Urbanization brought increases from two- to more than 50-fold typify the changes of peak-flow on flood peaks (Hollis, 1975; Roesner, et. al., 2001). Applying fifty-year continuously hourly precipitation to a conceptual watershed, Nehrke and Roesner (2004) demonstrated that peak flow exceedance frequencies increased dramatically when development of a watershed was left uncontrolled.

2.1.1 Flooding and water quality deterioration

Floods result from a combination of meteorological and hydrological extremes as indicated in the Table 2-1 (Urban Flood Risk Management, 2008). The flood frequencies and magnitudes rise due to the occupation of land with impervious surfaces and runoff conduit systems. Urban floods have large impacts particularly in terms of economic losses both direct and indirect.

Table 2-1 Factors contributing to flooding

Meteorological Factors	**Hydrological factors**	**Human factors aggravating natural flood hazards**
• Rainfall • Cyclonic storms • Small-scale storms • Temperature • Snowfall and snowmelt	• Soil moisture level • Groundwater level prior to storm • Natural surface infiltration rate • Presence of impervious cover • Channel cross-sectional shape and roughness • Presence or absence of over bank flow, channel network • Synchronization of run-offs from various parts of watershed • High tide impeding drainage	• Land-use changes (e.g. surface sealing due to urbanization, deforestation) increase run-off and may be sedimentation • Occupation of the flood plain obstructing flows • Inefficiency or non-maintenance of infrastructure • Too efficient drainage of upstream areas increases flood peaks • Climate change affects magnitude and frequency of precipitations and floods • Urban microclimate may enforce precipitation events

Source: Urban Flood Risk Management – A Tool for Integrated Flood Management Version 1.0

The stormwater runoff picks up and carries away natural and human-made pollutants, finally depositing them into receiving waters. Human activities can be sources of pollutants to receiving waters through various pathways, including atmospheric deposition, solid and liquid waste disposal and a combination of diffuse and point-source distribution (Peters and Meybeck, 2000). The pollutants include fertilizers, herbicides, and insecticides, oil, grease, and toxic chemicals, salt, bacteria and sediment. Many pollutants in stormwater runoff associate with the particulate fraction, as well as cause receiving water degradation themselves. Therefore, removing a substantial amount of the solids such as all particles above a critical particle size can reduce the concentrations of many pollutants (USEPA, 1994).

2.1.2 Effects on channel morphology

The morphology of a stream results from its response to hydrological, hydraulic, climatic and geologic conditions over generally long time periods. Wolman (1967) initially categorized stages of stream channel change in response to urbanization. The first stage is equilibrium and river channel stability. As development and construction begin in the second stage, the increase of the sediment delivery rates cause channel aggradations. The third stage is an urban landscape with increased areas of impervious surfaces leading to decreased sediment inputs and channel degradation due to flash discharges with low sediment yield (Wolman 1967). When a watershed cannot supply the stream with the volume of sediment it has the capacity to carry, channel degradation may occur in the form of incision, lateral migration or a combination of both (Bledsoe, 2002). The higher frequency of the peak flows causes the stream to cut a deeper and wider channel, degrading or destroying the in-stream aquatic habitat (Roesner and Bledsoe, 2001). Another contribution to the degradation process is digging of the gravel bed material and leveling of the bottom surface (Rosgen, 2006). These causes are usually accompanied by degradation by bank erosion. The effects of this degradation can be visualized by a change in channel geometry – shape of cross-section and longitudinal profile – which is the consequence of reciprocal relations among such characteristics as: water flow, water depth, channel width and hydraulic slope (Rosgen, 2006). Wolman and Miller (1960) observe that the frequency of geomorphically effective events is inversely proportional to the threshold of erosion. Given the high threshold conditions that characterize mixed bedrock–alluvial rivers, sediment transport and bedrock erosion is typically episodic and restricted to infrequent, high magnitude floods.

Excess shear stress and erosion potential indices are geomorphic metrics that can be used to evaluate the impacts of urbanization on stream channels. The potential for erosion in a channel can be quantitatively evaluated as the difference between calculated shear stress and critical shear stress (Rohrer and Roesner, 2005). Rivers incise bedrock when the mean shear stress exceeds a critical threshold to initiate erosion.

2.1.3 Effects on channel ecology and biotic integrity

A stable river channel with hydrodynamic balance preserves ecological continuity (Michalik, 2009). Urbanization and the hydrologic changes have degraded aquatic life to the extent that this pattern has been dubbed the “urban stream syndrome” (Walsh et al., 2005; Meyer et al., 2005). The effect of these hydrologic changes on stream geomorphology and stream ecology can be severe, especially in headwater streams. Macroinvertebrate abundance and diversity both generally declined in response to alteration in flow magnitude, whether an increase or a decline (Leroy and Zimmerman, 2001). During low flow periods, on-going water resources abstraction results in gradual reduction of flow available for instream uses, which, in turn, trigger a number of environmental effects, including increased sedimentation. The associated elevated stream sediment concentrations are harmful to water quality and aquatic biota (Smakhtin, 2001;

Simon and Rinaldi, 2006). During high flow periods, intense rainfall increases river runoff and pollutant load, posing more challenges on aquatic biota protection. More seriously, the global warming and related climate changes are predicted to occur over the next century, which will significantly increase the weather-related risks (Muller, 2007). These conditions cause ecological discontinuity of the stream (WiŚniewolski, 2002; Michalik, 2009).

As implementation of the Clean Water Act and associated pollution control measures continue to influence improvements in the quality of lakes and streams, emphasis is becoming more focused on physical conditions such as flow regime and habitat that also limit aquatic life diversity. This trend is particularly important in urban watersheds developed during a time when the stomwater management focus was on moving water away as quickly as possible rather than maximizing infiltration (Rohrer, 2004). However, even very large expenditures may not be adequate to reduce watershed sediment yield and the associated harmful to water quality and aquatic biota if peak discharges and channel energy slopes are not reduced (Douglas, 2009).

Hydrologic metrics that demonstrate altered stream flow regimes can in some cases provide a direct mechanistic link between the changes associated with urban development and degraded stream ecosystems (Booth et al. 2004). The work of Booth et.

al. (2004) relating the stream metrics T_{Qmean} and $T_{0.5}$ to the Benthic Macroinvertebrate Index (B-IBI), appears promising as the basis of an algorithm for relating B-IBI to land use development patterns and different runoff control methods. T_{Qmean} is the fraction of a year that the flow rate exceeds the mean flow rate of the entire flow time series. $T_{0.5}$ was defined as the fraction of the time, over a multi-year record, during which the flow in the stream equals or exceeds the peak flow of the 0.5-year storm generated by existing conditions in each watershed (Egderly and Roesner, 2006).

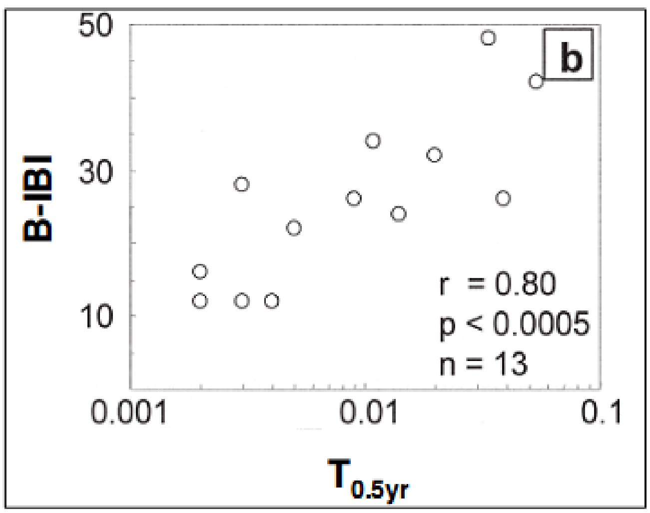

Figure 2-1 An empirical relationship generated by an urban gradient study
Source: Booth et al. (2004)

Figure 2-1 reproduced from Booth et. al. (2004) shows very good correlation between the B-IBI and the stream metric $T_{0.5}$ for streams in urbanized areas of Seattle, Washington, which is $T_{0.5}$ values are lower for watersheds with higher percentages of imperviousness. Pomeroy (2007) validates the conclusion by computing $T_{0.5}$ for various locations within the Morgan Creek watershed, North Carolina Piedmont.

2.2 Approaches for stormwater runoff controls

Based on the problems mentioned in the previous section, stormwater runoff is a major driver behind many processes that ultimately determine the state of the stream ecosystems (Roesner and Bledsoe 2003). The five aspects of a flow regime identified in the natural flow paradigm as most important to the geomorphology, physical habitat and ultimately stream biota are: magnitude, frequency, duration, timing, and rate of change (Poff et al. 1997). Effective management of stormwater runoff offers a multitude of possible benefits, including protection aquatic ecosystems, improved quality of receiving water bodies, conservation of water resources, protection of public health, and flood control.

2.2.1 Peak shaving

Traditional flood control measures that rely on the storage of the peak flow referred to as peak shaving, which sought to reduce flooding and mitigate erosive flows, have been characteristic of many stormwater management approaches. A common control strategy designed to reduce peak runoff from urban areas is detention storage. Storage ponds are typically designed to reduce the peak runoff from a moderately sized design event (usually 5- to 20-year return period) (Walesh, 1989) to maintain an estimated pre-development peak flow rate for the same design event (Chow et al., 1988, McCuen, 1989: ASCE, 1992).

Theoretically, it is well known that the peak shaving approach does little to reduce runoff volume and may exacerbate stream erosion problems by sustaining relatively high flows for longer periods. Moreover, the peak shaving method has generally not targeted pollutant reduction and in many cases has exacerbated the problems associated with changes in hydrology and hydraulics. An approach that integrates the control of storm water peak flows and the protection of natural channels to sustain the physical and chemical properties of aquatic habitat should be considered. While still an effective control measure for hydrograph peak shaving, detention does not appear to be the sole solution to mitigate urban impacts, but rather one component of a stormwater management plan.

2.2.2 Best management practices

With increased understanding of nonpoint source pollution, which has traditionally included stormwater sources, a holistic design of urban stormwater management systems needs to incorporate the multiple purposes of controlling major and minor floods, as well as stormwater pollution (Sample et al, 2003). The concept of best management practices (BMPs) encompasses a wide variety of appropriate technologies and activities intended to minimize the effect of watershed development on water quality of the runoff receiving water bodies. BMPs can be classified into two groups: structural and non-structural BMPs (Novotny, 2003). Structural BMPs include engineered and built systems designed to provide for water quality control; these are based on stromwater filtration, extended detention time, or infiltration into the soil. Non-structural BMPs include a range of pollution prevention, education, management and development practices designed to limit the conversion of rainfall into runoff. Traditional engineering approaches to stormwater management have tended to focus on structural BMPs.

Structural BMPs can be classified into three major types: infiltration BMPs, filtration BMPs, and extended detention time BMPs. Infiltration systems recharge the groundwater, helping to mitigate the impacts of development on the hydrologic cycle. In addition, they use the soil as a filter, treating polluted runoff as it percolates into the ground. Filtration BMPs are densely vegetated and uniformly graded areas that treat sheet flow adjacent impervious surfaces. Filtration BMPs function by slowing runoff velocities, trapping sediment and other pollutants and providing some filtration, have been shown to be very effective at removing a wide range of pollutants from stormwater runoff.

Extended detention BMPs provide for entrapment of pollutants via sedimentation (settlement) by capturing the high-frequency water quality storm and detaining it for a specified period of time, such as 40 or 72 hours. Over the past several decades, flood extended detention/retention BMPs have become the most common engineering approach to controlling the impacts of storm water runoff (Yeh and Labadie 1997). Ponds are popular among developers for a number of reasons including the fact that they provide open space and wildlife habitat, provide fill material, can be aesthetically pleasing, and require little maintenance.

2.2.3 On-site controls

The on-site control has been recommended as an alternative to traditional stormwater BMPs, such as low impact development (LID) approach. LID allows for greater development potential with less environmental impacts using on-site distributed stormwater controls that achieve a good balance among conservation, growth, ecosystem protection, and public safety (Guo, 2009). Research on individual LID practices such as bioretention, pervious pavements, and grassed swales and filter strips has increased in recent years (ASCE 1992; Dietz, 2007). The preservation of the pre-development hydrology of a site is the overall goal of LID. Cluster layouts, grass swales, rain gardens/bioretention areas, and pervious pavements all reduce the “effective impervious area” (Booth and Jackson 1997) of a watershed, or the area that is directly connected to the stormwater system.

In general, LID techniques are aimed at the entire land use management with emphasis on the controls of micro events such as 3-month to 2-year events (Roesner et al. 1996). Rather than the conventional approach for stormwater detention, which is to focus on the control of the design flow release, the focus of LID detention design has shifted to control the runoff volume over a specified drain time (EPA Report 2006). For instance, the flush volume and the water quality control volume were developed as a response to stormwater volume and quality control (Guo and Urbonas 1996).

2.3 Stormwater management facilities

The conventional approach for stormwater detention is to focus on the control of the design flow release that is defined by the local drainage criteria or the downstream existing drainage capacity (Guo, 2009). This section discusses two design approaches of the stormwater detention ponds: 1) *Multi-level Detention* pond and 2) *Full Spectrum Detention* pond.

Multi-level Detention is standard practice and typically involves peak shaving of post-development flows to pre-development peak flow levels for various design storms plus best management practices (BMPs) for removing pollutants from runoff for given return interval. Usually the peak shaving facilities are integrated with the water quality facilities For example, the large storms are controlled so that the post development peak discharge for a given return interval storm, such as 100-year, 50-year, 25-year, 10-year, or 2-year, does not exceed the pre-development peak discharge for the same storm. Meanwhile, the stormwater BMPs are designed to capture small storms that carry most of the sediments and solids which cause water quality problems in the receiving water bodies.

Full Spectrum Detention is a new stormwater detention approach developed by Wulliman and Urbonas (2005 and 2007) for controlling stormwater peak flow rates along the modeled stream from the small event up to the 100-year major flood: it is based on the following conditions:

1) The difference between post-development and pre-development runoff volume, the *Excess Urban Runoff Volume* (EURV) per impervious unit area, was found to be fairly constant for a wide range of design storm sizes and watershed imperviousness for given NRCS hydrologic soil groups. It was shown that the EURV became a constant

value once 20 percent imperviousness was reached and there was little difference between the various design storms, with the exception of very small storms in Denver region, as is shown in Figure 2-2.

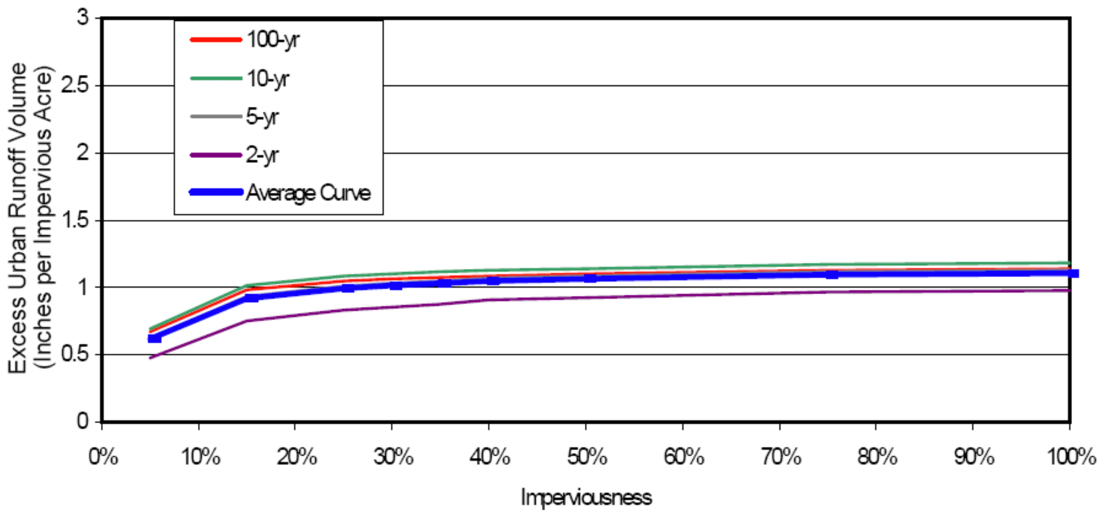

Figure 2-2 Excess Urban Runoff Volume for Hydrologic Soil Group C/D in Denver
Source: Wulliman and Urbonas, 2005

2) The first stage of a two-stage *Full Spectrum Detention* basin captures approximately the volume equal to the excess urban runoff volume (EURV), and it is released over a 72 hour period.

3) The upper stage of a *Full Spectrum Detention* basin is sized to control the 100-year peak flow rate from the watershed to the pre-development peak flow rate.

Previous studies at the Colorado State University Urban Water Center have showed that if the design storms for peak discharge control are properly chosen and used in conjunction with the appropriate sized volumetric BMPs, it is possible to preserve the pre-development peak flow frequency curve and to minimize geomorphic instability in an urbanizing watershed (Roesner and Rohrer, 2004). However, several authors (Maxted and Shaver, 1997; Roesner, 1999; Schueler, 1999) have investigated the design practices and effectiveness of extended detention BMPs in protecting small urban water courses and concluded that BMPs as were implemented do not adequately protect the downstream aquatic environment. Roesner et al. (2001) discussed the state of practice in the performance-based BMP design that is currently used in the United States and pointed out that there is a lack of agreement in the scientific and engineering community about what constitutes a properly designed BMP with respect to real protection of receiving waters. Pomeroy and Roesner (2008) showed that if the hydrologic regime of the runoff from an urbanized area is controlled so that it reproduces the pre-development runoff regime of the watershed, then the geomorphic and biotic stream degradation will be minimized.

Wong and Somes (1997) showed that in Melbourne, Australia, control of the peak discharge for the 1.5-year and 100-year storms resulted in lowering the entire flow frequency curve between these return intervals to the pre-development curves. Similarly, by applying 50 years of hourly continuous rainfall record on two climatically diverse locales in the United States, Nerke and Roesner (2004) showed that control of the 2-year and 100-year storms with a sized BMP result in a match of the flow frequency curve to pre-development flows between the two intervals, but the 2-year control is not sufficient for high frequency events. Roesner (2004) subsequently discovered that in Fort Collins, controlling the post development 100-yr, and 1.5-yr storms to pre-development levels plus an extended detention BMP sized with a drawdown time of 40 hours, the entire peak-flow frequency curve could be reproduced from events that occur multiple times per year, to the 100-yr storm.

Due in large part to a lack of flow data in watersheds, it is difficult to compute the differences between hydrologic variations caused by watershed characteristics and those actually caused by urbanization by applying the empirical urban gradient and paired watershed approaches (Roesner and Bledsoe 2003). Since flow regime analyses require long-term, continuous flow data (i.e. greater than 10 to 20 years of record) (Konrad and Booth 2002; Richter et al. 1997), it is proposed that continuous simulation with mathematical models can provide an approximate quantification of the flow regime alteration expected for various scenarios of development. The event-based design storm techniques tend to ignore or approximate certain site conditions such as the antecedent moisture conditions and the storage needs attributable to snow melt (Wang, 2004).

Continuous simulation can reflect fluctuations in the moisture content of the ground, which can have a significant impact on runoff flow rates.

Different types of urban streams are likely to exhibit varying degrees of instability (depending on enhancement), relative erodibility of bed and banks, riparian condition, mode of sediment transport (bedload versus suspended load) and proximity to geomorphic thresholds (Bledsoe and Watson, 2001). Shear stress duration curves can be used to help evaluate the full range of shear stress forces exerted on a stream channel. The duration curve is generated by calculating average boundary shear stress in the stream channel for each time step in the measured or modeled flow record. Inability to match the full duration curve does not mean that erosion potential due to changes in flow in the stream has to increase, however, so long as the critical portion of the flow duration curve is matched (Nardi and Roesner, 2004).

3 STUDY APPROACH

3.1 Method and objectives

This study evaluates the performance of two stormwater detention design approaches （*Multi-level Detention* and *Full Spectrum Detention*） in minimizing the urbanization effects on peak flow frequency curve, flow duration curve, average boundary shear stress curve and hydrologic metrics $T_{0.5}$.

A 29.15-acre subarea within a proposed subdivision was used as the test watershed for this study (see Figure 3-1 below). This watershed was examined with respect to two hydrologically distinct climatic areas: Fort Collins, Colorado, and Atlanta, Georgia. Fort Collins represents an arid climate receiving relatively low amounts of annual rainfall, which is 18 inches per year. The humid climate of Atlanta, Georgia which has frequent precipitation of variable intensity and duration produces the average rainfall amount of 50.2 inches per year.

As stated, the same watershed was used for both Fort Collins and Atlanta analysis and the only variable input of the two regions is the precipitation data. The runoff drainage system and receiving stream were identical and the watershed hydrogeometric properties were kept the same for these two study areas, such as area, width, and slope. The watershed variable values for each study area were also held the same, such as land use (pervious and impervious values), site layout, Manning's n values, and Horton infiltration values. These values were held constant so that the sensitivity of flow regime changes and aquatic habitat to different climatically regions could be examined without the influence of changes in the other variables. It is obvious that under different climates and geographic regions, the hydrogeometric properties, state variables and receiving stream hydrogeomorphic characteristics are different.

The study includes two major stages:

1) Validate the EURV conclusion of *Full Spectrum Detention* method by using both the SWMM and CUHP models and comparing the performance of the two models in the EURV estimation.

2) Compare the two detention pond design approaches in terms of computed peak flow frequency curves, flow duration curves, average boundary shear stress curves and $T_{0.5}$.

For the first stage, two rainfall-runoff models SWMM (Version 5.0.013) and CUHP (Version 1.3.1) were first employed in the climate region Fort Collins, with a variable imperviousness values ranging from 20 percent to 100 percent, to validate the assumption of the *Full Spectrum Detention* method, which is, when the imperviousness is larger than 20 percent, EURV is approaching fairly constant for a wide range of design storm sizes. Then, based on the validation results, the model which performs better for the watershed of Fort Collins is used to compute the EURV in Atlanta for a variety of imperviousness values. Furthermore, the model is used to estimate the EURV for the proposed post-development conditions in both Fort Collins and Atlanta with a certain land use for each subcatchment. If the EURV constant assumption could not be applied in Fort Collins and Atlanta, the EURV value varies for each design storm.

In the second stage, four scenarios were evaluated for both Fort Collins and Atlanta in minimizing the urbanization impacts. These scenarios included pre-developed conditions, proposed developed conditions without stormwater control, developed conditions with stormwater controlled by multi-level design approach, and *Full Spectrum Design* approach. The stormwater control methods examined use: (a) the City of Fort Collins flood control standard, (b) the City of Fort Collins flood control standard and water quality capture volume (WQCV) criteria, and (c) a new concept of practice in the

Denver region: control of the 100 year storm to historic peak discharge rate and control of the EURV.

3.2 Data

3.2.1 Precipitation Data

Sixty years of hourly continuous precipitation data were obtained from the National Climatic Data Center (NCDC, 2008) for Fort Collins, Colorado (NCDC COOP ID: 053005) and Atlanta, Georgia(NCDC COOP ID: 090451). The record ranged from August 1st 1948 to March 31th 2008. RAINMASTER, the software used to process and analyze continuous precipitation data (Heineman, 2001), was applied to calculate precipitation depths for various rainfall intervals based on the 60-year records.

To be consistent with work conducted with Fort Collins precipitation data in a previous study (Nehrke and Roesner, 2004; Rohrer, 2004;), the error that showed the precipitation data in April 30 and May 1st of 1973 has an intensity of 250in/hr was changed to 0.25in/hr for the continuous simulation. Moreover, the error in the data set on September 20, 1980 with a value of 6.5 inches was changed to 0.65 inches.

Design storms were used to size the stormwater conveyance facilities and the stormwater detention facilities. In this study, three design storms were used, which are

1.5-year, 10-year and 100-year storms. Design storms of 10-year and 100-year hyetographs for Fort Collins were obtained from the City of Fort Collins Storm Water Utility Storm Drainage Design Criteria and Construction Standards, 1999. Design storm hyetographs of 1.5 year for Fort Collins was generated by applying Urban Drainage and Flood Control District (UDFCD, 2007) distribution criteria to 1.5 year, 1-hour total storm depths generated by the RAINMASTER analysis. The same method was used to generate the 1.5-year, 10-year and 100-year design storms for the Atlanta. The design storm hyetographs for Fort Collins and Atlanta are presented in Table A-1 and A-2 of Appendix A.

Total depths of the 1.5-year, 10-year and 100-year return interval storms for Fort Collins and Atlanta are summarized in Table 3-1. The depths of the record were generated from the RAINMASTER software, and the design storm depths are calculated based on the 2-hour distribution criteria mentioned in UDFCD, 2007.

Table 3-1 Two-hour design storm depths

Location	Precipitation Depth (in)					
	1.5-year		10-year		100-year	
	60-year Record	Design Storm	60- year Record	Design Storm	60- year Record	Design Storm
Atlanta, GA	1.75	1.60	2.74	2.51	4.69	4.14
Fort Collins, CO	1.03	1.02	2.41	1.71	3.10	3.67

3.2.2 Watershed Characteristics

The test watershed used to compare the two approaches for stormwater runoff control is shown in Figure 3-1. It has an area of 29.1 acres and is approximately rectangular (1126 ft x 1126 ft). Since a maximum overland-flow length of 500 ft is recommended for undeveloped areas (UDFCD, 2007), the watershed was divided into an upper portion and a lower portion, each 500 ft. by 1260 ft.. Since the watershed slope is approximately the same for the upper and lower watersheds, they can be considered as a single watershed with a overland flow length of 500 ft and a width of 2520 ft (2 x 1260). It is assumed that the undeveloped site is NRCS hydrologic soil type B and is approximately 5 percent impervious.

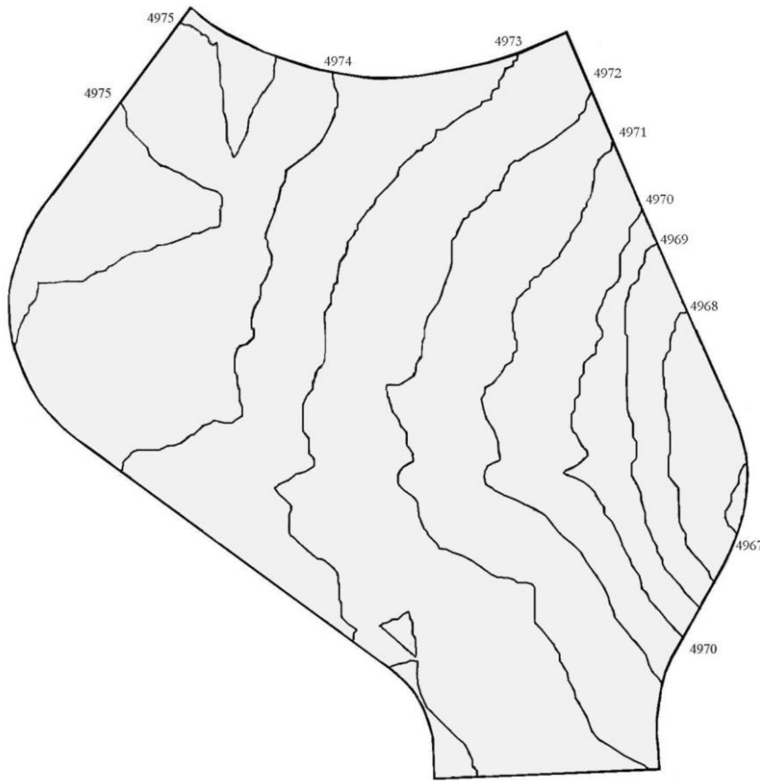

Figure 3-1 Predevelopment watershed

The properties assigned to the single predevelopment subcatchment S1 are summarized as follows:

- Area: 29.15 acres
- Length: 503.88ft
- Width: 2520ft
- Slope: 0.5 percent
- Imperviousness: 5 percent

The subdivision layout was then superimposed on the pre-development subbasin as shown in Figure 3.2. The increase in impervious surface and reduction of overland flow length are the main factors affecting the hydrologic response of a watershed when it becomes urbanized. These factors create additional surface runoff as well as higher and faster peak discharges. Subarea boundaries within the subcatchment were developed to coincide with the topography of the regarded subdivision. The overland flow routes, subcatchment location and the slope and length of overland flow were taken into account when setting subarea boundaries. Seventeen subareas were determined, which are shown in Figure 3-2, with an average imperviousness of 57 percent. The land use in the study area consists of low density residential development, high density commercial use, and open spaces and natural parks.

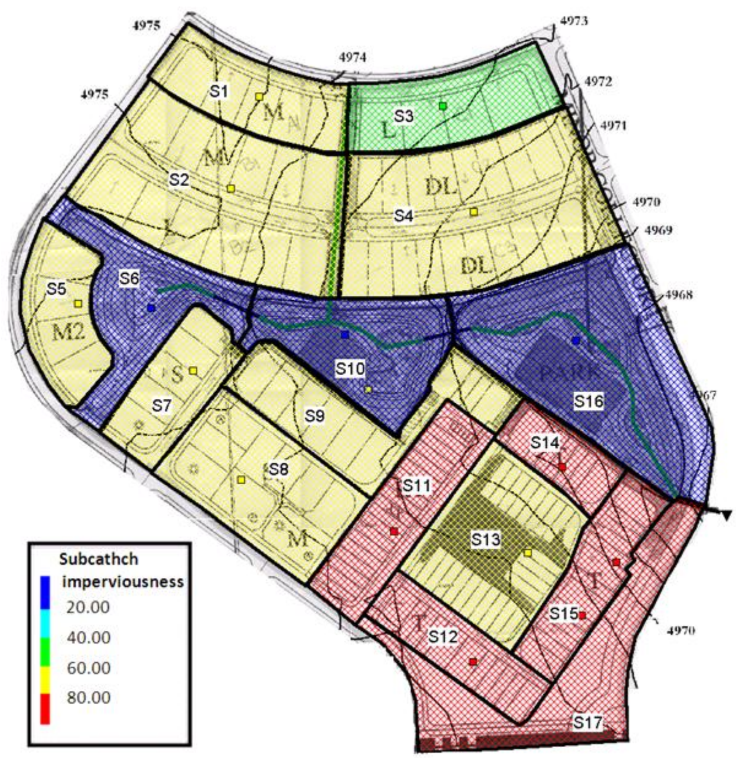

Figure 3-2 Developed watershed layout

Imperviousness of each subbasin was calculated by measuring directly connected impervious areas in each subcatchment (roads and driveways) and dividing by the total area of each subcatchment. Percent impervious values for subbasins that contained different developed uses were calculated by using the area weighted average of the runoff coefficients for all of the land uses in the subcatchment. A runoff coefficient is an empirical-constant value that represents the percentage of rainfall that becomes runoff. Table 3-2 displays the various land use categories that appear in the developed site along with their runoff coefficients.

Table 3-2 Land use categories in the developed watershed

Id	Land Use	Runoff coefficient (C)
M	Medium density	0.65
L	Low density	0.45
DL	Duplex	0.7
M2	Medium density	0.65
S	Apartment, high density	0.7
RT	Commercial	0.95
T	Commercial	0.95
P	Natral (park)	0

Table 3-3 summarizes the hydrogeometric input parameters of the proposed developed watershed.

Table 3-3 The hydrogeometric input parameters of the developed watershed

Subcatchment	Area, acre	Length, ft	Width, ft	Slope, %	Imperviousness, %
S1	1.21	124.60	423.00	2	65.00
S2	3.36	116.92	1251.80	2	53.81
S3	1.31	124.59	458.00	2	45.00
S4	3.4	105.32	1406.22	2	70.00
S5	1.01	145.62	302.12	2	65.00
S6	1.49	229.18	283.20	4.7	0.00
S7	1.01	132.05	333.17	2	70.00
S8	1.88	101.48	806.98	2	68.30
S9	1.56	122.77	553.50	2	70.00
S10	1.72	199.14	376.23	5	0.00
S11	1.32	124.00	463.70	2	95.00
S12	0.96	84.12	497.12	2	95.00
S13	1.96	85.69	996.35	0.5	78.80
S14	0.63	102.47	267.81	2	95.00
S15	1.07	110.70	421.04	2	95.00
S16	3.28	281.33	507.86	3.1	0.00
S17	1.98	124.64	692.00	2	95.00

In addition to data describing watershed size, shape, slope and imperviousness, SWMM model requires input of watershed characteristics related to infiltration, roughness, evaporation, and depression storage.

Infiltration in each subcatchment was modeled via the Horton Equation:

$$f = f_{min} + (f_{max} - f_{min}) * e^{-t*k} \quad \text{Equation 3-1}$$

wherein:

f = infiltration capacity of the soil

f_{min} = initial infiltration capacity

f_{max} = final (constant) infiltration capacity

t = elapsed time from start of rainfall

k = decay time constant

Horton equation parameters are soil type dependent, which are given in the UDFCD, 2007 manual for each soil type and listed below in Table 3-4.

Table 3-4 Horton infiltration parameters for various soil types

NRCS Hydrologic Soil Group	Infiltration (inches per hour)		Decay Coefficient-a
	Initial-f_i	Final-f_0	
A	5.0	0.1	0.0007
B	4.5	0.6	0.0018
C	3.0	0.5	0.0018
D	3.0	0.5	0.0018

Soil type B was assumed for the study watershed, so the input parameters for infiltration used in both the Atlanta and Fort Collins simulations were:

- maximum or ultimate value of fmin: 4.5 in/hr,
- minimum or initial value of fmax: 0.6 in/hr, and
- decay coefficient for regeneration of infiltration capacity: 6.5 hr^{-1}

Manning's roughness values *n* for pervious overland flow and depression storage parameters depend on land use/vegetation type. The values of Manning's roughness and depression storage for each vegetation cover found respectively in the SWMM Users' Manual and the UDFCD Manual, 2007 are listed in Table 3-5. In this study, the pervious development Manning's roughness was 0.24, which represented dense grass land cover and the depression storage was set at 0.3-in.

Table 3-5 Manning's roughness and depression storage values

Land Cover	Manning's Roughness	Depression Storage, in
Grass	0.2	0.2
Dense Forest	0.8	0.3
Light Forest	0.4	0.3

Source: McCuen, 1996, UDFCD Manual, 2007

Impervious Manning's roughness and depression storage remained unchanged between subcatchments. An impervious Manning's roughness value of 0.015 was used, which corresponds to the value estimated from the SWMM Users' Manual for dense gravel (roads and driveways appeared and were assumed gravel). The impervious depression storage was set at 0.06-in, which is again given in the SWMM Users' Manual for impervious surfaces.

Other parameters corresponding to the watershed characteristics for all simulations were used in the SWMM model:

- Snowmelt was not simulated
- Evaporation from channels was not allowed
- Dynamic wave routing method was used
- All internal routing in each subcatchment was to the outlet and no pervious to impervious routing or vice versa was specified.

Pre-development and developed condition runoff hydrographs were generated by SWMM using the input data described above. The design storms of 1.5-year, 10-year and 100-year were modeled in addition to the 60-year continuous simulations.

3.2.3 Drainage System Hydraulics

The drainage design consisted primarily of street gutters, culverts, and storm pipes. Figure 3-3 shows the layout of the runoff conveyance system that will be added to the developed watershed. It consisted of 8 grass swales, 3 culverts, and 17 street gutters, 1 detention pond, and 1 receiving channel. The drainage system was sized to allow full conveyance of the discharges for the three design storms without flooding, and the receiving stream is also designed to transfer the 100-year storm without bank overflow. The property of the drainage system is shown in the appendix. In Figure 3-3, blue links represent culverts, green links represent connector swales, and black links represent gutters. The drainage system properties were summarized in Appendix A, Table A-3.

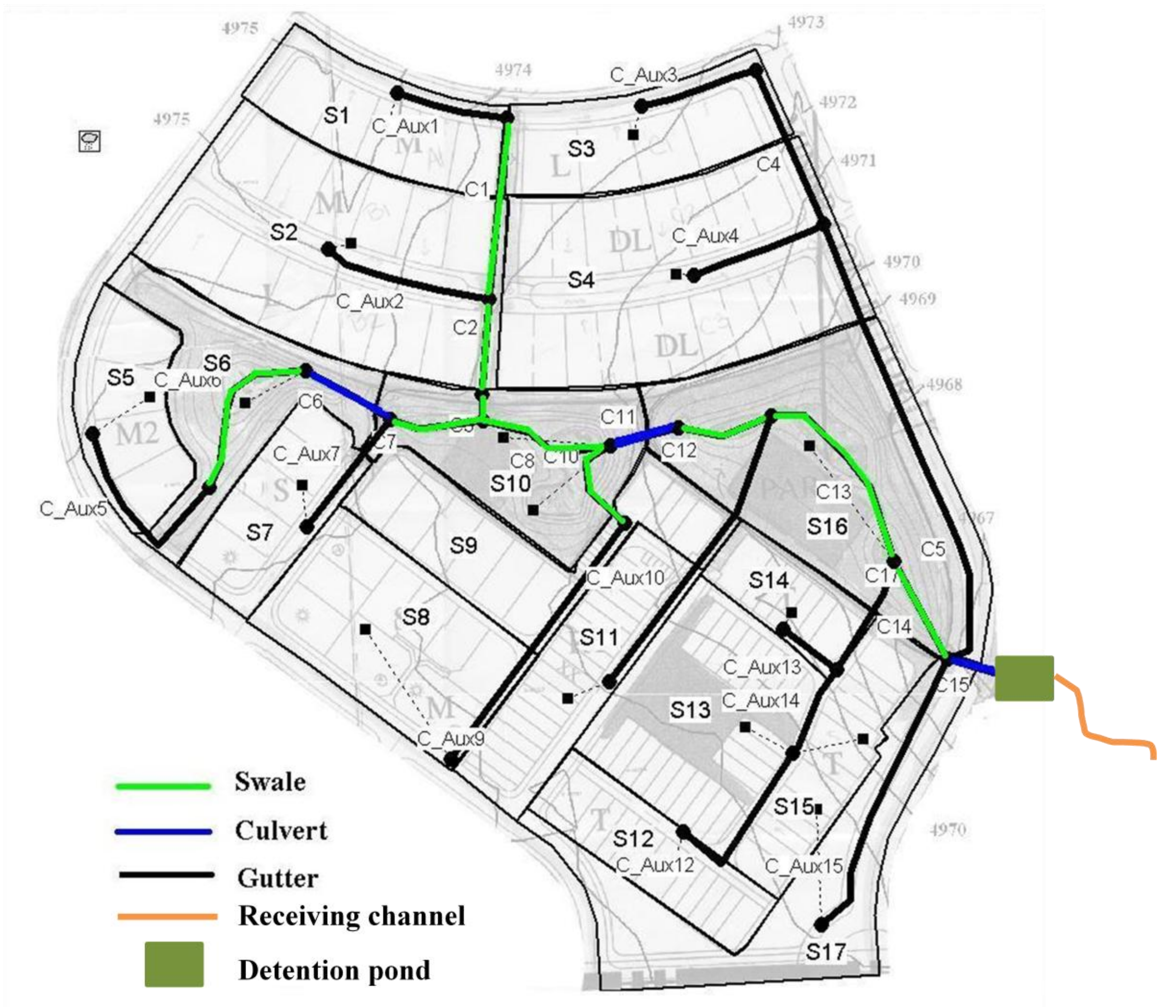

Figure 3-3 Layout of the runoff conveyance system

3.3 Validation of Full Spectrum Detention Method

The *Full Spectrum Detention Design Approach* has been tested successfully for the Denver region by using the CUHP model and excellent matching of pre-development peak flows has been achieved. This section evaluates the application of this concept from the aspects as:

1) Whether this design approach can be widely applied to the other regions in the United States.

2) Whether this design approach can be applied by using the SWMM model.

Since the design approach is based on the concept that the EURV became a constant value once 20 percent imperviousness was reached and there was little difference between the various design storms, with the exception of very small storms in Denver region, the process in this section includes by applying CUHP model and SWMM model to a watershed rather than Denver region, to verify the EURV constant concept.

3.3.1 Model description

SWMM and CUHP were used to simulate the runoff to yield 1.5-year, 10-year, and 100-year pre-development and post-development discharges for NRCS Hydrologic Soil Group B, and C/D in Fort Collins and Atlanta respectively. The EURV was calculated by subtracting pre-developed from the post-developed runoff volumes.

SWMM model is a dynamic rainfall-runoff model using the linear reservoir runoff model, capable of performing continuous or event simulation of surface runoff and conveyance in open channel and pipe systems. The runoff component of SWMM operates on a collection of subcatchment areas that receive precipitation and generate runoff and pollutant loads. The routing portion of SWMM transports this runoff through a system of pipes, channels, storage/treatment devices, pumps, and regulators. SWMM tracks the quantity and quality of runoff generated within each subcatchment, and the flow rate, flow depth, and quality of water in each pipe and channel during a simulation period comprised of multiple time steps (EPA Storm Water Management Model User's Manual, 2004).

The CUHP model is an evolution of the Snyder unit hydrograph rainfall-runoff model. It has been calibrated to the Colorado Front Range using of data collected by the U.S. Geological Survey beginning in 1969. The concept "effective precipitation" is used that accounts for volume losses, and a unit hydrograph that accounts for routing and basin size. The given parameters, particularly imperviousness, are approximately uniform over the basin (uniform land use). CUHP doesn't have the flow routing function, so basins with different land-use zones should be broken into multiple smaller basins and routed using EPA SWMM or other flow routing software. Figure 3-4 shows the workbook of CUHP mode (CUHP User Manual, 2005).

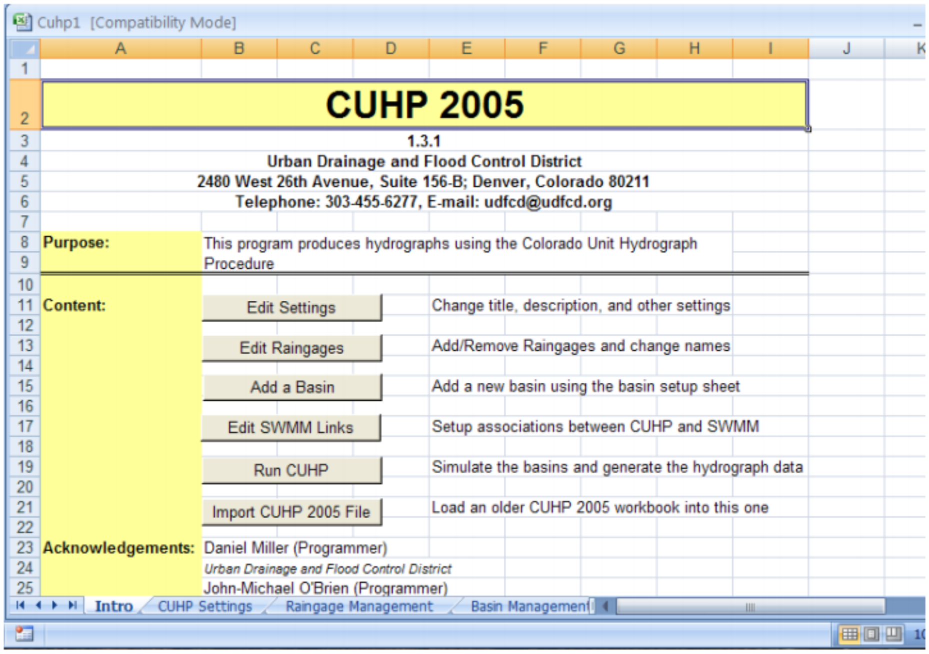

Figure 3-4 CUHP 2005 Intro Page

3.3.2 EURV study

The EURV was first calculated in Fort Collins, which is located 57 miles (92 km) north of Denver, Colorado, and thus has the similar climate with Denver region area. Meanwhile, Fort Collins is in the region of Colorado Front Range, which meets the requirement of the CUHP application. All the input parameters of the two models were set the same. The catchment and subcatchment parameters and drainage systems were kept the same as mentioned in section 3.2. To compare the EURV results with Wulliman and Urbonas (2005, 2007), the soil type C/D was first tested, and then soil type B. The only difference of these two soil types was the infiltration rate. Two rainfall intervals were adopted, which are 5-minute and 15-minute rainfall interval, and the corresponding simulation time steps are 1 minute and 5 minute separately. The 15-minute rainfall interval was computed from the 5-minute rainfall interval, shown in Appendix A, Table A-4 and A-5 for Fort Collins and Atlanta respectively. Table 3-6 shows representative watershed and channel routing parameters for the pre-development.

Table 3-6 Predevelopment input parameters

Parameter		Value
Subcatchment property	Area, acre	29.15
	Imperv, %	5
	Width,ft	2540
	Slope, %	0.5
Manning's n	N-Imperv	0.015
	N-Perv	0.24
	S-Imperv	0.06
	S-Perv	0.3
Infiltration	MaxRate, in/hr	3
	MinRate, in/hr	0.5
	Decay, s^{-1}	6.5
	Dry Time	0

In the post-development, there are 17 subcatchments, and the properties of each subcatchment are summarized in Table 3-7, the imperiousness of was kept identical for each subcatchment, ranging from 20 percent to 100 percent. The Manning’s n and infiltration parameters were the same as pre-development.

Table 3-7 Postdevelopment input parameters

Subcatchment	Area	Width	Slope, %
S1	1.21	423.00	2
S2	3.36	1251.80	2
S3	1.31	458.00	2
S4	3.4	1406.00	2
S5	1.01	302.12	2
S6	1.49	283.20	4.7
S7	1.01	333.17	2
S8	1.88	806.98	2
S9	1.56	553.50	2
S10	1.72	376.23	5
S11	1.32	463.70	2
S12	0.96	497.12	2
S13	1.96	996.35	0.5
S14	0.63	267.81	2
S15	1.07	421.04	2
S16	3.28	507.86	3.1
S17	1.98	692.0	2

The pre-developed and post-developed runoff volumes were first calculated by using the 5-minute rainfall interval, and 1-minute time step. Table 3-8 shows the pre-development runoff volume calculated by SWMM and CUHP.

Table 3-8 Pre-development runoff volume

Design Storm, year	Pre-development runoff volume, ft^3	
	SWMM	CUHP
1.5	5387	5290
10	16710	59298
100	157609	231801

Table 3-8 indicates that SWMM and CUHP produced similar runoff volume for the 1.5 year design storms. However, CUHP generates much more runoff for 10 year and

100 year design storms. Figure 3-5, 3-6 and 3-7 show the pre-development hydrographs of different design storms generated by these two models.

For the 1.5 year design storm, both of the two models produce very similar amount of total runoff volume. . However, Figure 3-6 indicates that the 10-year design storm runoff volume generated by CUHP model is about 3.5 times of the one simulated by SWMM model. The pre-development peak discharge of 6.97cfs for the 10 year design storm is more reasonable according to the SWMM Application Manual, (2008). Detailed examination of the algorithms in SWMM and CUHP revealed that the difference in the runoff volumes is due to the way in which infiltration is computed in the two models. While both models use the Horton method to compute infiltration, CUHP reduces infiltration capacity as a function of time since start of storm, irrespective of the how much rainfall has occurred, while SWMM uses an integrated form of Horton's equation that relates infiltration capacity to the amount of rainfall that has infiltrated since the start of the storm. This allows SWMM to infiltrate significantly more rainfall than CUHP. Thus, it is concluded that SWMM gives a better representation of runoff for the 10 year design storm simulation than CUHP. Similar as Figure 3-6, Figure 3-7 illustrates that CUHP has a higher simulated runoff volume than the SWMM for the 100 year design storm, which is to be expected due to the way infiltration is computed.

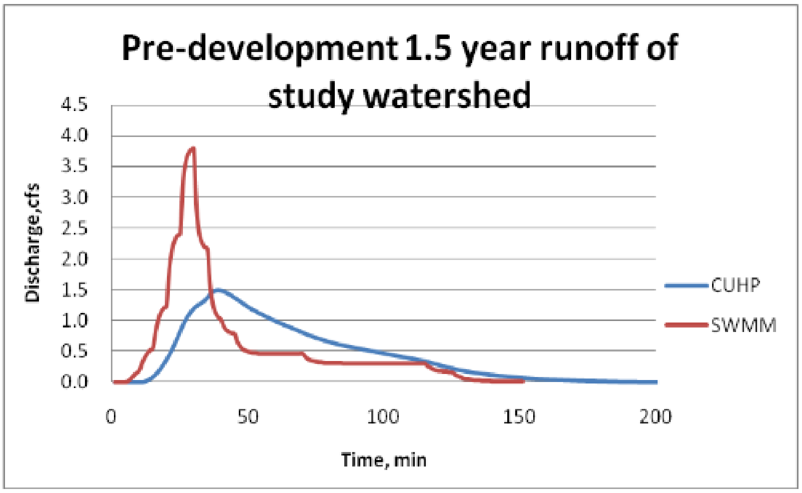

Figure 3-5 Predevelopment 1.5-year hydrograph

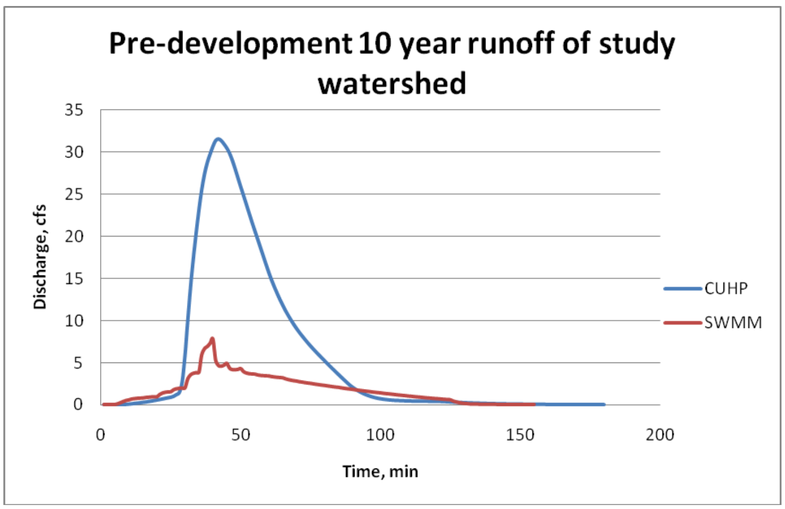

Figure 3-6 Predevelopment 10-year hydrograph

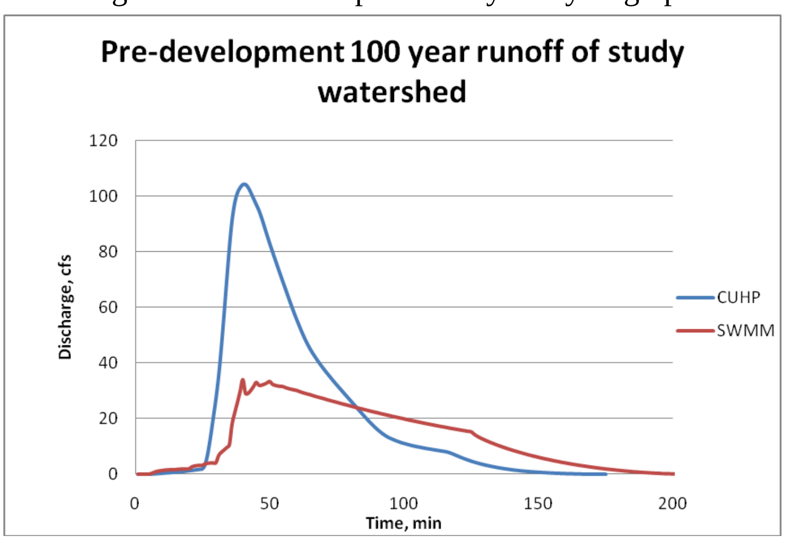

Figure 3-7 Predevelopment 100-year hydrograph

In the post-development condition, there are 9 different hydrographs for each design storm and each subcatchment simulated by the two models due to the changes of imperviousness from 20 percent to 100 percent. Comparing the total discharges volume in the whole watershed generated by various imperviousness values and design storms is one simplified way. In this process, SWMM model could route the discharges, and the total runoff volumes were computed by summing the volumes for each time step. But for CUHP model without routing, the total volumes were calculated as following: 1) The runoff hydrograph of each subcatchment is computed by CUHP model; 2) Each runoff hydrograph is imported to SWMM model as inflow of the corresponding subcathment and routed by SWMM model. The total runoff computed for each imperviousness value is summarized in Table 3-9.

Table 3-9 Post-development total discharges volume

Imperv, %	1.5yr		10yr		100yr	
	SWMM, ft^3	CUHP, ft^3	SWMM, ft^3	CUHP, ft^3	SWMM, ft^3	CUHP, ft^3
20	20587	48413	53873	125441	239155	321484
30	30880	54090	70316	130295	258806	326133
40	41307	59714	86759	135055	278189	330572
50	51601	65609	102934	140421	297306	336491
60	64701	71695	118976	146146	316021	343222
70	75530	78019	134884	152275	334602	350821
80	86358	84183	150524	158023	352917	357468
90	97052	90129	165898	163332	370830	363087
100	107880	96502	180870	169376	388342	370302

Table 3-9 shows that, most of the time, CUHP model overestimated the total discharge for each design storm in the post-development compared to SWMM in Fort Collins.

Then, EURV was calculated by subtracting the pre-development total runoff volume from the developed total runoff volumes. Table 3-10 shows the EURV for the three design storms computed by SWMM and CUHP models.

Table 3-10 EURV of Fort Collins

	1.5yr EURV, ft^3		10yr EURV, ft^3		100yr EURV, ft^3	
Imperv, %	SWMM	CUHP	SWMM	CUHP	SWMM	CUHP
20	15199	43123	37163	66143	81545	89683
30	25493	48800	53606	70997	101196	94332
40	35920	54424	70049	75757	120580	98771
50	46213	60319	86224	81123	139696	104690
60	59314	66405	102266	86848	158411	111421
70	70142	72729	118174	92977	176993	119020
80	80970	78893	133814	98725	195307	125667
90	91665	84839	149188	104034	213220	131286
100	102493	91212	164160	110078	230733	138501

Table 3-10 reveals that with the increase of imperviousness, EURV increases. So the EURV constant conclusion does not work in Fort Collins either by SWMM or CUHP when the 5-mintue rainfall interval and 1-minute time steps was applied. To further test the conclusion, keeping all the other input parameters constant, a 15-minute rainfall interval and 5-minute simulation time step was applied in the both models. The same procedure was taken as in the previous section. The 15-minute design storm hyetographs of 1.5-, 10- and 100-year for Fort Collins were generated based on the 5-minute time steps. It is well known that, the 15-minute interval might change the shape of the hydrographs, but would not change the total volume of the discharges. To evaluate the performances of these two models, the total discharge volumes using the two different time intervals for each model were studied.

Table 3-11 Pre-development runoff volume estimated by SWMM model

Design Storm, yr	Runoff volume, ft^3		*Difference,
	5-min interval	15-min interval	%
1.5	5387.4	5213.54	3.23
10	16710.07	15373.26	8.00
100	157609.4	156807.29	0.51

*Difference= $\mid$ Runoff volume$_{15\text{-min}}$- Runoff volume$_{5\text{-min}}$ $\mid$ / Runoff volume$_{5\text{-min}}$*100%

Table 3-11 illustrates that the difference between two rainfall intervals ranged from 0.51 percent to 8 percent in the SWMM model, while Table 3-12 shows that the difference generated by CUHP model varied from 13.23 percent to 15.18 percent. So SWMM model is again more consistent in estimating the discharge volumes compared to CUHP.

Table 3-12 Predevelopment runoff volume estimated by CUHP model

Design Storm, yr	Runoff volume, ft^3		*Difference,
	5-min interval	15-min interval	%
1.5	5290	4590	13.23
10	59298	49248	16.95
100	231801	196608	15.18

*Difference= $\mid$ Runoff volume$_{15\text{-min}}$- Runoff volume$_{5\text{-min}}$ $\mid$ / Runoff volume$_{5\text{-min}}$*100%

For the post-development, the percent of imperviousness varies from 20 percent to 100 percent, which results 9 different discharges for each model. Table 3-14 and 3-15 show the EURV in volume for the three design storms, and compare the total discharge volume differences between the 5-minute and 15-minute hyetographs. Table 3-14 illustrates that the differences range from 0 to 2.66 percent by the SWMM model estimation. Table 3-14 indicates that the differences vary from 2.46 percent to 75.71 percent when CUHP model was used.

Table 3-13 Post-development total discharges volume estimated by SWMM model

Imperv, %	1.5yr, ft^3			10yr, ft^3			100yr, ft^3		
	5-min	15-min	Differ, %	5-min	15-min	Differ, %	5-min	15-min	Differ, %
20	20587	20586	0.00	53873	52938	1.74	239155	238219	0.39
30	30880	30880	0.00	70316	68845	2.09	258806	257335	0.57
40	41307	41307	0.00	86759	84753	2.31	278189	276318	0.67
50	51601	51601	0.00	102934	100528	2.34	297306	294899	0.81
60	64701	61894	4.34	118976	116168	2.36	316021	313214	0.89
70	75530	72188	4.42	134884	131542	2.48	334602	331260	1.00
80	86358	82481	4.49	150524	146781	2.49	352917	349040	1.10
90	97052	92774	4.41	165898	161620	2.58	370830	366552	1.15
100	107880	103068	4.46	180870	176057	2.66	388342	383663	1.20

Table 3-11 and Table 3-13 shows that EURV values computed by SWMM in 5-minute and 15 minute rainfall intervals are almost the same. However, Table 3-12, Table 3-14 and Table 3-15 indicate the EURV values calculated by CUHP in 5-minute and 15 minute rainfall intervals have significant differences, with a variation of 2.47 percent to 75.71 percent.

Table 3-14 Post-development total discharges volume estimated by CUHP model

Imperv, %	1.5yr, ft^3			10yr, ft^3			100yr, ft^3		
	5-min	15-min	Differ, %	5-min	15-min	Differ, %	5-min	15-min	Differ, %
20	48413	11760	75.71	125441	58311	53.52	321484	209343	34.88
30	54090	20073	62.89	130295	67482	48.21	326133	218310	33.06
40	59714	31296	47.59	135055	81879	39.37	330572	239160	27.65
50	65609	40944	37.59	140421	94437	32.75	336491	257871	23.36
60	71695	51084	28.75	146146	107331	26.56	343222	276216	19.52
70	78019	61851	20.72	152275	121110	20.47	350821	296070	15.61
80	84183	74121	11.95	158023	137505	12.98	357468	321714	10
90	90129	84042	6.75	163332	149772	8.3	363087	338145	6.87
100	96502	94119	2.47	169376	162159	4.26	370302	354525	4.26

Table 3-15 EURV comparison

Imperv, %	1.5yr EURV, ft^3			10yr EURV, ft^3			100yr EURV, ft^3		
	5-min	15-min	Differ, %	5-min	15-min	Differ, %	5-min	15-min	Differ, %
20	43123	7170	83.4	66143	9063	86.3	89683	12735	85.8
30	48800	15483	68.3	70997	18234	74.3	94332	21702	77.0
40	54424	26706	50.9	75757	32631	56.9	98771	42552	56.9
50	60319	36354	39.7	81123	45189	44.3	104690	61263	41.5
60	66405	46494	30.0	86848	58083	33.1	111421	79608	28.6
70	72729	57261	21.3	92977	71862	22.7	119020	99462	16.4
80	78893	69531	11.9	98725	88257	10.6	125667	125106	0.4
90	84839	79452	6.3	104034	100524	3.4	131286	141537	7.8
100	91212	89529	1.8	110078	112911	2.6	138501	157917	14.0

In this section, the conclusion can be drawn as follows:

1) SWMM performs better than CUHP in simulating the total runoff volume in predevelopment and post-development condition, especially in large design storms, such as 10- and 100-year design storm in Fort Collins.
2) The EURV is not constant when the imperviousness of the watershed is higher than 20 percent either by using SWMM model or CUHP model for the soil type C/D. This concept does not work well for the study watershed, Fort Collins, which is near Denver region.

The EURV constant conclusion is tested in the watershed of Fort Collins, Colorado, and the results show that it cannot be applied to the area outside of Denver region by either SWMM model or CUHP model in soil type C/D. To further test this assumption, the EURV is calculated by SWMM for the watershed Atlanta, Georgia, which represents a wetter climate, in both soil type B and C/D by applying 5-minute rainfall interval. Furthermore, the soil type B EURV test is also performed in Fort Collins. The EURV of Fort Collins and Atlanta of soil type C/D and B was calculated and shown in shown in Appendix B, Table B-1 to B-12.

Finally, by comparing all the EURV curves in Fort Collins and Atlanta with the Denver region as is shown in Figure 2-2, the conclusion that the EURV is constant for all imperviousness and all storms cannot be applied either in Fort Collins or Atlanta by using

SWMM model or CUHP model in any soil type. This concept cannot be applied to the region other than Denver region area. For the further detention design, SWMM model is used and three different EURV values for each design storms were calculated in the developed conditions.

3.4 Stormwater detention pond design

Storage is widely used in urban runoff quantity and quality control, providing both peak flow reduction and suspended solids removal. The design criteria for storage structures have changed over time due to an improved understanding of the effects that urban runoff have on the environment. Facilities control not only the extreme runoff events to prevent flooding, but also the more common smaller events that produce a "first flush" pollution phenomenon and thereby impact the quality of receiving water bodies. This section shows two design approaches to achieve the both quantity and quality control, which are: *Multi-Level Design* method and *Full-Spectrum Design* method.

3.4.1 Multi-level design approach

The *Multi-level Detention* pond and its outlets were designed to control a Water Quality Capture Volume (WQCV) and to peak shave the 1.5-year, 10-year and 100-year discharges to the pre-development historic rates. Figure 3-8 shows the schematic of the

Multi-level Detention pond. In this study the minor storms' (WQCV and 1.5-yr) and the major storms' (10- and 100-yr) runoff will be detained in separate sections of the detention pond. Table 3-16 and 3-17 show the discharges to be controlled by the pond in Fort Collins and Atlanta.

Table 3-16 Precipitation magnitude of Fort Collins, CO

Return Period, year	Rainfall Magnitude, in	Pre Peak Discharge, cfs	Post Peak Discharge, cfs
1.5	1.02	1.55	21.65
10	1.71	3.31	40.16
100	3.67	25.95	119.06

Table 3-17 Precipitation magnitude of Atlanta, GA

Return Period, yr	Rainfall Magnitude, in	Pre Peak Discharge, cfs	Post Peak Discharge, cfs
1.5	1.6	2.99	37.5
10	2.51	6.48	65.74
100	4.14	45.5	120.03

The WQCV is a suitable and critical runoff volume in inches that is detained for a long enough period of time to achieve a targeted level of pollutant removal. The required volume and drawdown time vary under different stormwater policies (Akan and Houghtalen, 2003).

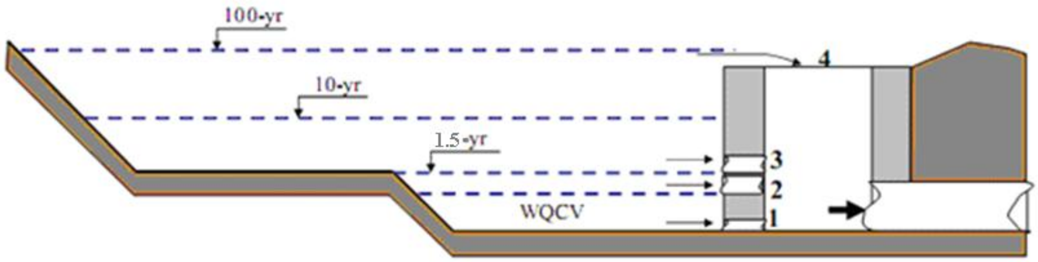

Figure 3-8 Schematic of a *Multi-level Detention* pond

Where 1 is the orifice of WQCV, 2 is the orifice of 1.5 year discharge, 3 is the orifice to release the 10-year discharge, and 4 is the weir used to control the 100 year flow.

Source: SWMM Application Manual 2008

According to the design criteria of UDFCD 2007, the WQCV was computed as a function of the tributary catchment's total imperviousness and the drain time of the capture volume, as is shown in Figure 3-9. The 40-hour draw down time was chosen in this study, during which a significant portion of particulate pollutants found in urban stormwater runoff are removed. Figure 3-9 is appropriate for use in Fort Collins, while for Atlanta, the different portions of Colorado, the WQCV obtained from this figure could be adjusted using

$WQCV_0=d_6$ Equation 3-2

in which,

WQCV = Water quality capture volume taken from Figure 3-9

$WQCV_0$ = Water quality capture volume outside Denver region

d_6 = Depth of average runoff producing storm from Figure 3-10 (watershed inches)

The required storage volume for WQCV in acre-ft is computed as follows:

$$\text{Required storage} = \frac{WQCV}{12} * (\text{Area}) \qquad \text{Equation 3-3}$$

in which,

Required storage = Required storage volume in acre-feet

Area = The tributary catchment's area upstream in acres

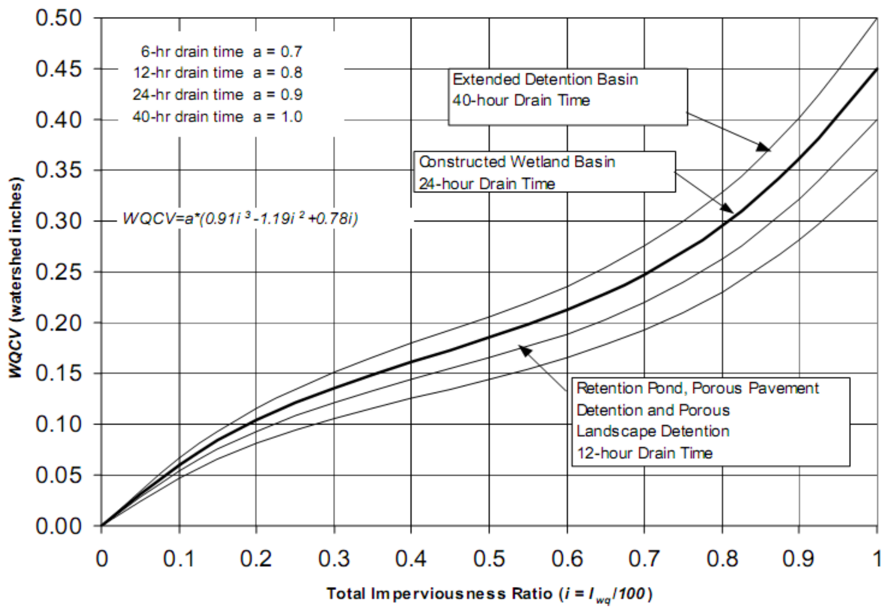

Figure 3-9 WQCV as a function of imperviousness and draw down time
Source: UDFCD, 2007

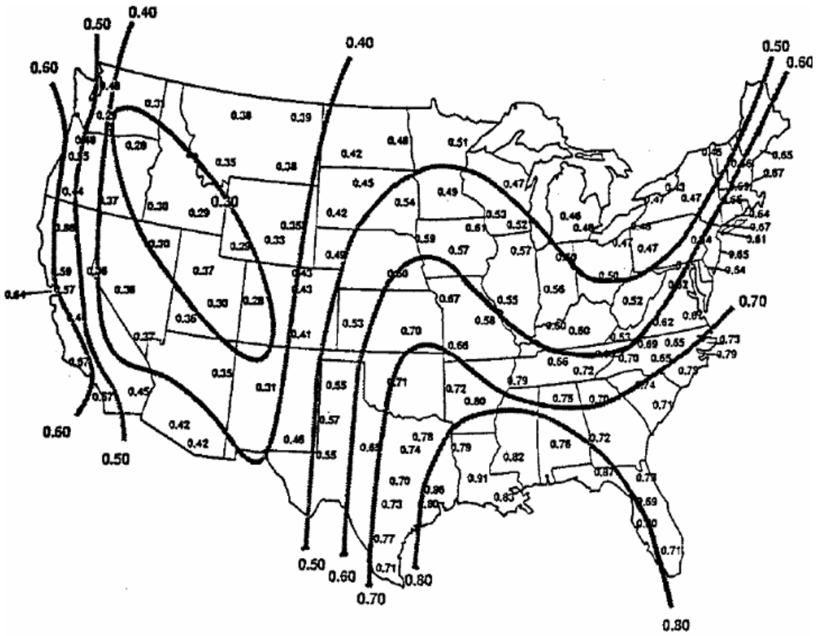

Figure 3-10 Map of the average runoff producing storm's precipitation depth in US, Ref: Driscoll et. Al., 1989

The calculated WQCV for Fort Collins was 0.23 inches, and for Atlanta, 0.7 inches. The geometry of the WQCV storages were chosen as trapezoid with length to width ratio of 2:1 and a side slope of 4:1. Orifices outlets for both WQCV 40-hour draw down time control were sized using an iterative procedure in Excel. Dimensions of the 1.5-year, 10-year, and 100-year peak shaving ponds are sized the same length to width ratio, and side slope for each scenario. The outlets orifices are computed based on the orifice discharge Equation 3-4. The 100-year weirs for both cities were calculated according to the Equation 3-5.

$$Q=C_0\sqrt{2gh} \qquad \text{Equation 3-4}$$

where: C_0 = orifice coefficient (0.65), A = area of orifice, g = gravitational constant (9.81 m/s), and h = hydraulic head on the orifice.

$$Q=CLh^{3/2} \qquad \text{Equation 3-5}$$

where C=weir coefficient (3.3), h=hydraulic head on the weir.

A summary of the designed detention ponds geometry and orifices dimensions in Fort Collins and Atlanta are presented in Table 3-18, 3-19, 3-20, and 3-21.

Table 3-18 Characteristics for multi-purpose detention pond in Fort Collins

Height, ft	Area, ft^2	Event Controlled
0	22973.55	WQCV
1.9	28091.84	
2	42137.75	1.5-yr
2.72	44679.15	10-yr
4.8	52393.68	100-yr

Table 3-19 Characteristics for multi-purpose detention pond in Atlanta

Height, ft	Area, ft^2	Event Controlled
0.55	37861.5	WQCV
1.86	44216.3	
2	48637.93	1.5-yr
2.76	51519.34	10-yr
4.7	59209.75	100-yr

Table 3-20 Outlet structure of the multi-purpose detention pond in Fort Collins

Type of element	Event control	Type	Shape	Height, ft	Width, ft	Inlet offset, ft	Discharge coefficient
ORIFICE 1	WQCV	Side	Rec-closed	0.35	0.35	0	0.65
ORIFICE 2	1.5-yr	Side	Rec-closed	0.4	0.4	1	0.65
ORIFICE 3	10-yr	Side	Rec-closed	0.65	0.5	1.9	0.65
Weir 1	100-yr	Side	Rec-closed	2	2.15	2.72	3.33

Table 3-21 Outlet structure of the multi-purpose detention pond in Atlanta

Type of element	Event control	Type	Shape	Height, ft	Width, ft	Inlet offset, ft	Discharge coefficient
ORIFICE 1	WQCV	Side	Rec-closed	0.55	0.5	0	0.65
ORIFICE 2	1.5-yr	Side	Rec-closed	0.55	0.54	1	0.65
ORIFICE 3	10-yr	Side	Rec-closed	0.6	0.7	1.86	0.65
Weir 1	100-yr	Side	Rec-closed	1.54	4.7	3.02	3.3

3.4.2 Full Spectrum Detention design approach

Based on the conclusions from the section 3.3, the EURV was not approaching to a constant value for different design storms when the imperviousness was higher than 20 percent. The SWMM model was used to compute the EURVs for the studied watershed, the post-development with an average imperviousness of 57 percent and was compositing with residential area, commercial area, etc. Two different climatically rainfall of Fort Collins and Atlanta were used for soil type B. The EURV values for Fort Collins and Atlanta are summarized in Table 3-22.

Table 3-22 EURV values for three design storms

Design	Fort Collins, CO	Atlanta, GA

Storm	EURV, ft^3	EURV, ft^3
1.5yr	54736.8	87042.6
10yr	94772.4	136612.8
100yr	165032.4	156516.6

The *Full Spectrum Detention* pond and its outlets were sized to capture the EURV, release it in 72 hours for water quality control, and peak shave the 100-year post-development discharges to the historic rates. Figure 3-11 shows the schematic of the *Full Spectrum Detention* pond. The geometry of the different stages of the detention pond were designed similarly as the multi-level approach, such as trapezoid shape with length to width ratio of 2:1 and a side slope of 4:1, same equations for orifice and weir dimensions. A summary of the designed detention ponds geometry and orifices dimensions in Fort Collins and Atlanta are presented in Table 3-23, 3-24, 3-25, and 3-26.

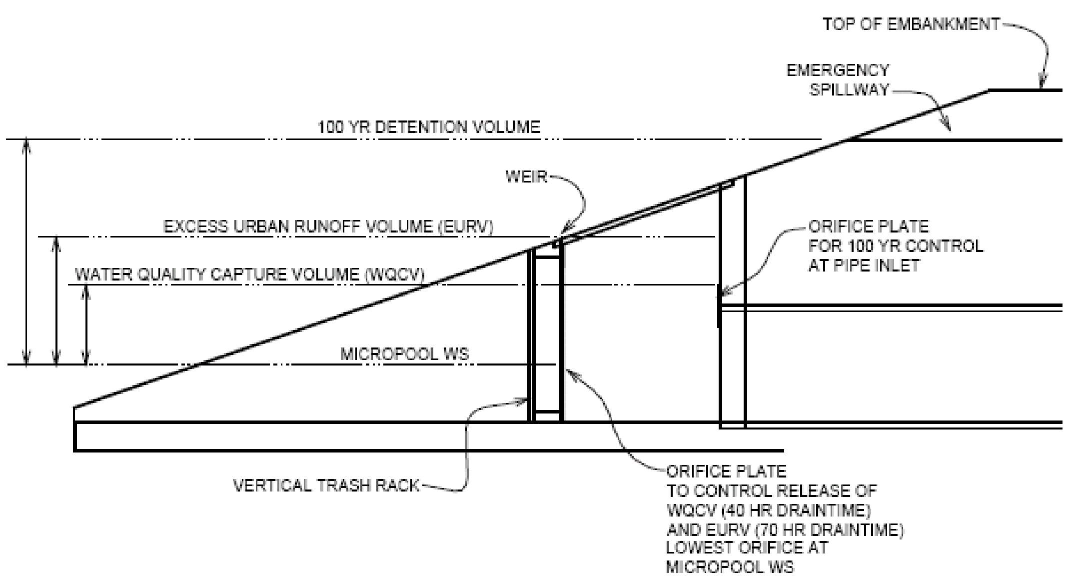

Figure 3-11 Schematic of a *Full Spectrum Detention* pond
Source: Urbonas and Wulliman, 2007

Table 3-23 Characteristics for *Full Spectrum Detention* ponds in Fort Collins

Scenario	Height, ft	Area, ft^2	Event Controlled
1.5-yr EURV	0	24579.77	WQ &
	2	30157.03	1.5-yr EURV
	2.1	60314.07	Backflow
	4.5	70685.4	100-yr
10-yr EURV	0	43710.17	WQ&
	2	51062.23	10-yr EURV
	2.1	56168.45	Backflow
	3.5	61924.7	100-yr
100-yr EURV	0	77658.95	WQ&
	2	87373.45	100-yr EURV
	2.1	96110.79	Backflow
	2.5	98225.5	100-yr

Table 3-24 Characteristics for *Full Spectrum Detention* ponds in Atlanta

Scenario	Height, ft	Area, ft^2	Event Controlled
1.5-yr EURV	0	39999.22	WQ
	2	47043.38	1.5-yr EURV
	2.1	94086.76	Backflow
	3.5	101499.9	100-yr
10-yr EURV	0	63888.88	WQ
	2	72723.93	10-yr EURV
	2.1	74178.41	Backflow
	3.5	154634.32	100-yr
100-yr EURV	0	73528.54	WQ
	2	82988.06	100-yr EURV
	2.1	165976.13	Backflow
	3	172250.42	100-yr

Table 3-25 Outlet structure of *Full Spectrum Detention* ponds in Fort Collins

Scenario	Type of element	Event control	Type	Shape	Height, ft	Width, ft	Inlet offset, ft	Discharge coefficient

1.5-yr EURV	Orifice 1	WQ	Side	circle	0.25	-	0	0.65
	Orifice 2	1.5-yr EURV	Side	circle	0.27	-	0.4	0.65
	Weir 1	100-yr	Side	Transverse	3	1.4	3	3.33
10-yr EURV	Orifice 1	WQ	Side	circle	0.35	-	0	0.65
	Orifice 2	10-yr EURV	Side	circle	0.45	-	0.4	0.65
	Weir 1	100-yr	Side	Transverse	2	2.6	2	3.33
100-yr EURV	Orifice 1	WQ	Side	circle	0.5	-	0	0.65
	Orifice 2	100-yr EURV	Side	circle	0.65	-	0.6	0.65
	Weir 1	100-yr	Side	Transverse	1.3	5	1.2	3.33

Table 3-26 Outlet structure of *Full Spectrum Detention* ponds in Atlanta

Scenario	Type of element	Event control	Type	Shape	Height, ft	Width, ft	Inlet offset, ft	Discharge coefficient
1.5-yr EURV	Orifice 1	WQ	Side	Rec-closed	0.35	0.35	0	0.65
	Orifice 2	1.5-yr EURV	Side	Rec-closed	0.4	0.5	0.5	0.65
	Weir 1	100-yr	Side	Transverse	1.5	5.7	1.86	3.33
10-yr EURV	Orifice 1	WQ	Side	Rec-closed	0.5	0.57	0	0.65
	Orifice 2	10-yr EURV	Side	Rec-closed	0.5	0.55	0.6	0.65
	Weir 1	100-yr	Side	Transverse	2	6.8	1.5	3.33
100-yr EURV	Orifice 1	WQ	Side	Rec-closed	0.5	0.72	0	0.65
	Orifice 2	100-yr EURV	Side	Rec-closed	0.5	0.75	0.6	0.65
	Weir 1	100-yr	Side	Transverse	1.5	6	1.5	3.33

3.5 Peak flow frequency analysis

Urban runoff peak frequency curves, which serve for the design of drainage systems, are one of the most important evaluation indices of urban hydrology. One of the tools for determination of runoff peak frequency curves is simulation of rainfall runoff processes for selected rainfall inputs and catchment conditions. Ideally, a continuous simulation model should be used to produce a simulated runoff record which would be then subject to frequency analysis to derive the runoff peak frequency curves (Marsalek, 1984). The SWMM Statistics tool was used to compute the frequency of peak flows generated by individual events during the 60-year hourly precipitation records. A six-hour inter-event time and minimum threshold of 0.05cfs was specified to separate the flow data into individual events. The Cunnane (1978) formula was used to calculate the frequency of an event peak flow, T, as:

$$\mathrm{T(i)}=\frac{N+\beta-2\alpha}{i-\alpha}$$

where, T= return interval (years), N=number of years of record, i= rank of the event in descending order of magnitude, α is plotting position coefficient with a value of 0.4, and β is another coefficient which equals to 1. The return interval was converted to exceedances per year, E, using:

$$E=\frac{1}{T}$$

3.6 Flow duration analysis

Cumulative frequency curves, called flow duration curves, show the average percentage of time that specific flows are equaled or exceeded at sites where continuous records of flow are available. The duration of each flow value was determined by grouping the discharge values into classes, and the frequency distribution is represented as a histogram. The size of the class interval is determined by (Yevjevich, 1972)

$$\Delta x = \frac{x_{\max} - x_{\min}}{N_c - 1}$$

The number of observation n_i that fall into the ith class interval is called the absolute frequency of that class, and then f(i), the ith relative frequency, is calculated as

$$f(i)=\frac{n_i}{N}, \ i=1,2,\ldots,Nc$$

The histogram is transformed into an empirical density function by

$$\hat{f}(i) = \frac{f(i)}{\Delta x} = \frac{n_i}{N\Delta x}$$

Finally, the flow duration of each discharge was calculated by

$$\hat{F}(j) = \sum_{i=1}^{j} \hat{f}(i)(\Delta x), \ j=1, 2,\ldots, Nc$$

where, x_{max} is the maximum flow; x_{min} is the minimum flow, Nc is the number of class intervals Nc=1000 was chosen for the analysis.

3.7 Average boundary shear stress and $T_{0.5}$ analysis

The discharges simulated by the 60-year continuous rainfall were used to compute the boundary shear stress, an important parameter associated with sediment transport and stream bank erosion potential. For a stream bank with noncohesive bed particles, the motion happens when the shear stress τ applied to the bed material exceeds the critical shear stress τ_c. The threshold conditions for movement occur when the fluid flow around a sediment particle exerts a force that is balanced with the resisting force of the particle weight, which condition is called incipient motion. The comparison of the shear stress? to a critical shear stress τ_c could be used to evaluate the incipient motion. Meanwhile, the boundary shear stress could also be used to estimate erosion rates, and then compute the erosion potential. Boundary shear stress is an important indicator of the stream geomorphology.

The average boundary shear stress is calculated as:

$$\tau = \gamma R_h S_f$$

Where γ is the specific weight of the fluid mixture (assumed to be same as water), R_h is the hydraulic radius of channel, and S_f is the friction slope of channel.

For each discharge, the hydraulic radius is calculated from the Manning's equation as:

$$Q = \frac{1}{n} A R^{2/3} S_f^{\ 1/2}$$

In this study, the medium gravel (particle diameter>8mm) was chosen as the bed materials and a rectangular channel with a bottom width of 15ft, and length of 656ft, friction slope S_f=0.02, and Manning's n=0.01 was designed as the receiving river channel. Table 3-27 summarizes the critical Shield's Parameters used for this study.

Table 3-27 Threshold Conditions at 20˚C

Class Name	Particle Diameter(mm)	ω_0 (mm/s)	τ^*_c	τ_c (Pa)
Medium Sand	>0.25	36	0.048	0.194
Medium Gravel	>8	338	0.044	5.7

Source: Julien, 1995; Rohrer, 2004

The $T_{0.5}$ hydrologic metric was defined as the fraction of the time, over a multi-year record, during which the flow in the stream equals or exceeds the peak flow of the 0.5-year storm generated by existing conditions in each watershed. The 0.5-year storm value was obtained from the flow frequency curve, and then the exceedance probability of the 0.5-year storm was gained from the flow duration curve.

4 RESULTS

4.1 Peak-shaving in design storms

For the *Multi-level Detention* approach, detention ponds and outlets were sized to release the post-development flow rate to the historic values in both Fort Collins and Atlanta as mentioned in section 3.4. The *Full Spectrum Detention* approach aims to control all the peak discharges through controlling the EURVs generated by design storms for post-development. Table 4-1 and Table 4-2 summarize the peak discharges in the developed condition controlled through the two kinds of detention ponds in each watershed. For Fort Collins, the control of 10-year and 100-year EURVs detention ponds perform well in peak shaving the three design storms, however the 1.5-year EURV control detention pond fails to restrict the 10-year peak discharge to the pre-developed values. Table 4-2 demonstrates that in Atlanta, the 100-year EURV control pond results in the 1.5-year, 10-year, and 100-year peak discharge values well below the pre-development conditions. However, both the 1.5-year and 10-year and 100-year

EURVs detention ponds could not provide fully control on the 10-year peak discharges.

Table 4-1 indicates that the 10-year EURV scenario is the best one in the peak shaving compared with the other EURV scenario for Fort Collins while the 100-year EURV scenario the best in Atlanta, as is shown in Table 4-2. The peak discharges of different scenarios were demonstrated in Appendix C.

Table 4-1 Comparison of peak discharge control in Fort Collins

Design Storm	Pre-development (cfs)	Post-development (cfs)	Post-development controlled			
			Multi-level (cfs)	1.5-yr EURV (cfs)	10-yr EURV (cfs)	100-yr EURV (cfs)
1.5-year	1.55	21.65	1.55	1.06	1.02	1.11
10-year	3.31	40.16	3.26	4.59	1.49	2.91
100-year	25.95	119.06	25.85	25.96	25.61	25.55

Table 4-2 Comparison of peak discharge control in Atlanta

Design Storm	Pre-development (cfs)	Post-development (cfs)	Post-development controlled			
			Multi-level (cfs)	1.5-yr EURV (cfs)	10-yr EURV (cfs)	100-yr EURV (cfs)
1.5-year	2.99	37.5	2.98	1.96	2.29	2.48
10-year	6.48	65.74	6.48	10.18	9.39	7.10
100-year	45.5	120.03	37.25	37.20	38.48	26.13

4.2 Peak flow frequency analysis

The 60-year 1-hour continuous rainfall data were applied to the watershed of Fort Collins and Atlanta respectively, and simulated by SWMM model. Figure 4-1 shows the peak discharge frequency-exceedance curves resulting from the continuous simulation of the pre-development, post-development and developed controlled by *Multi-level Detention* pond in Fort Collins. Similarly, Figure 4-2 shows peak flow frequency curve for the pre-, post-development and developed controlled by *Full Spectrum Detention* ponds, and Figure 4-3 compares the effects of the two design approaches on the flow frequency curves.

The design storms of City of Fort Collins (1999) were originally used to design the *Multi-level Detention* pond and size the outlets to release the peak discharges to pre-development peak discharge levels. Differences between undeveloped condition peak discharges generated by the continuous simulations and the design storm simulations are significant for the Fort Collins analysis. As is shown in Figure 4-1, the *Multi-level Detention* method fails to regulate the 1.5-, 10-, and 100-year storm events in Fort Collins. The reason is that the pre-development peak flow values, obtained using traditional design storms methods, were greater than the historical values obtained by continuous records. The validity of using single-event design storms to size flow controls has been questioned for some time by researchers (Rohrer, 2004, and Nehrke, 2004). The continuous simulation peak discharge values were used as the target peak discharge rate for the design storms in the final *Multi-level Detention* design. The final design allowes the *Multi-level Detention* scenario flow frequency curve to better match the curve for undeveloped conditions, as demonstrated in Figure 4-1.

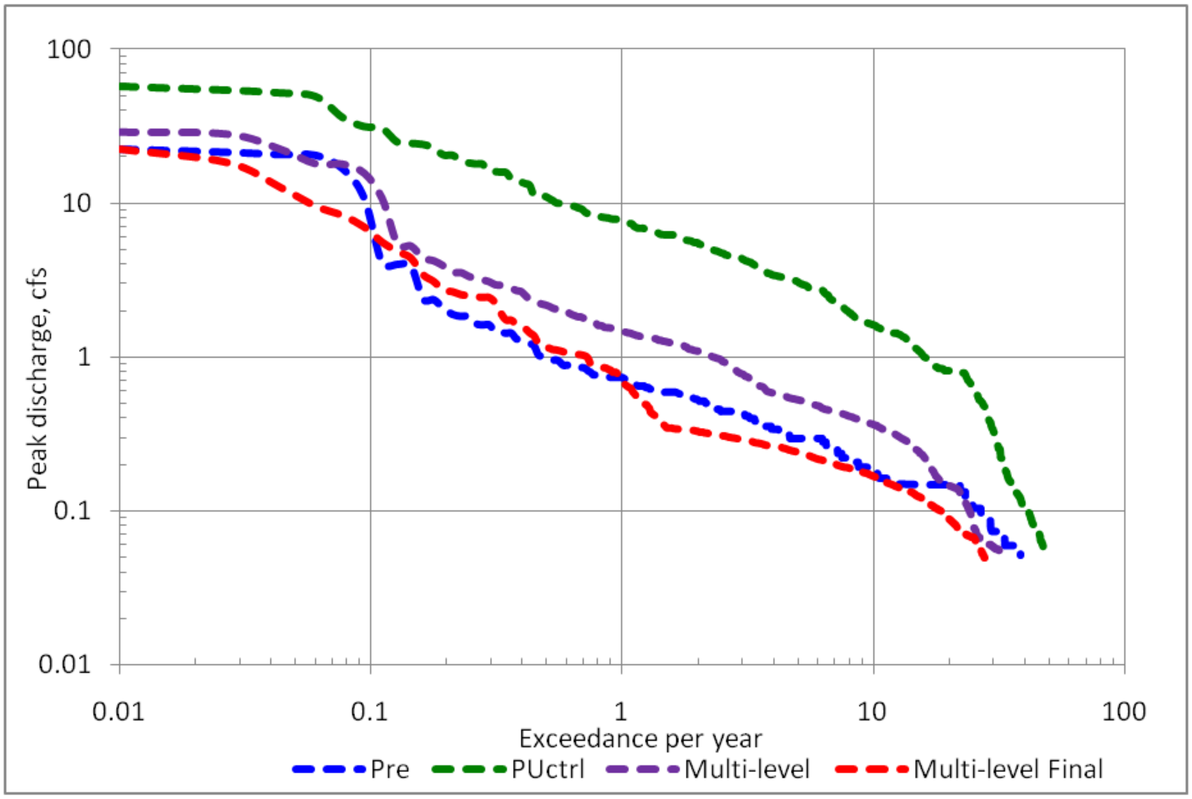

Figure 4-1 Peak flow frequency curve in Fort Collins: *Multi-level Detention* pond

Figure 4-2 demonstrates three scenarios of the *Full Spectrum Detention* approach with various EURV values. Apparently, the 1.5-year EURV scenario does not match the pre-development condition well. For the 100-year EURV scenario, the peak discharges from 10-year to 25-year are controlled under the pre-development flow rate. Nevertheless, the frequent and small peak discharges that are exceeded from 8.71-year storm to the 11.1times a year are left uncontrolled in this situation. The 10-year EURV scenario fits the pre-development peak discharge for the 3-year, 6.7-year and 100-year storm events, and also provides a better control for the small storms peak discharges between 1-year and 10 times per year, but 1.5-year and 10-year storm peak discharge is not well controlled. Among the three scenarios, the 10-year EURV works the best, which was chosen to compare with the *Multi-level Detention* scenario. For the large storm between 50-year and 100-year peak discharge, the 10-year EURV has a similar curve with the *Multi-level Detention*, as is shown in Figure 4-3.

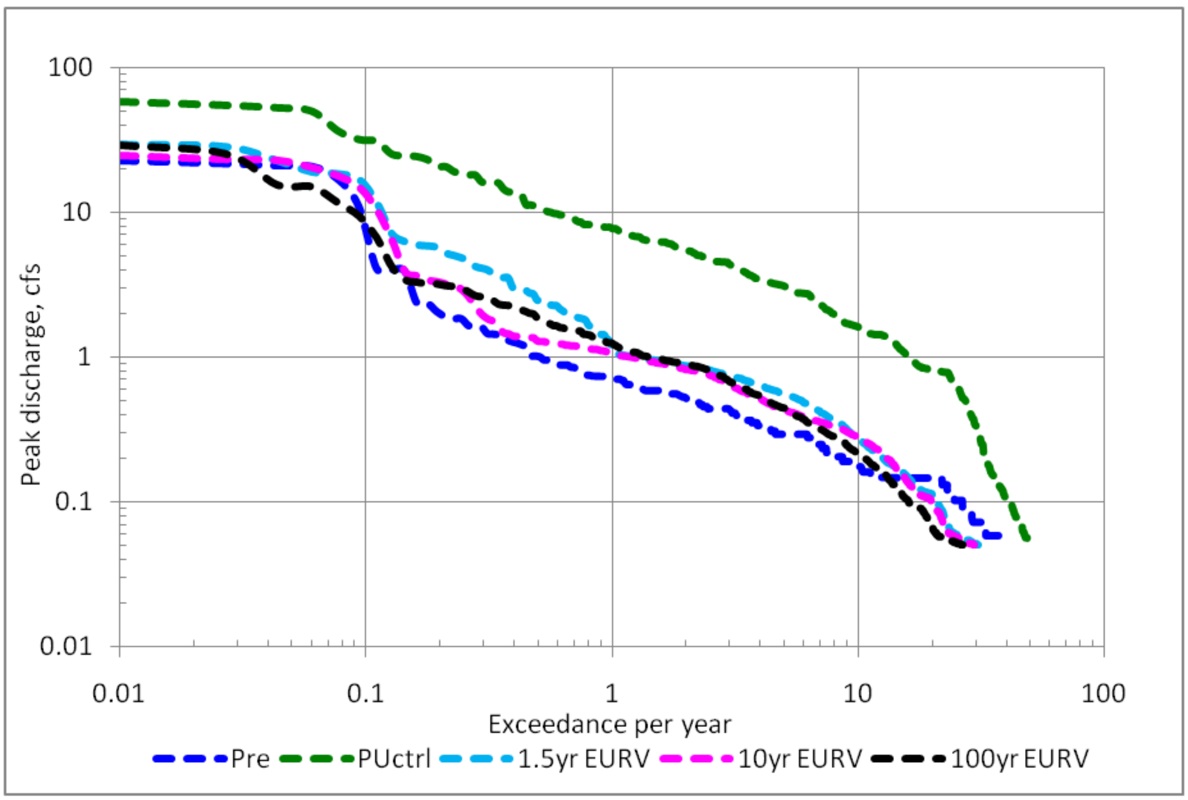

Figure 4-2 Peak flow frequency curve in Fort Collins: *Full Spectrum Detention*

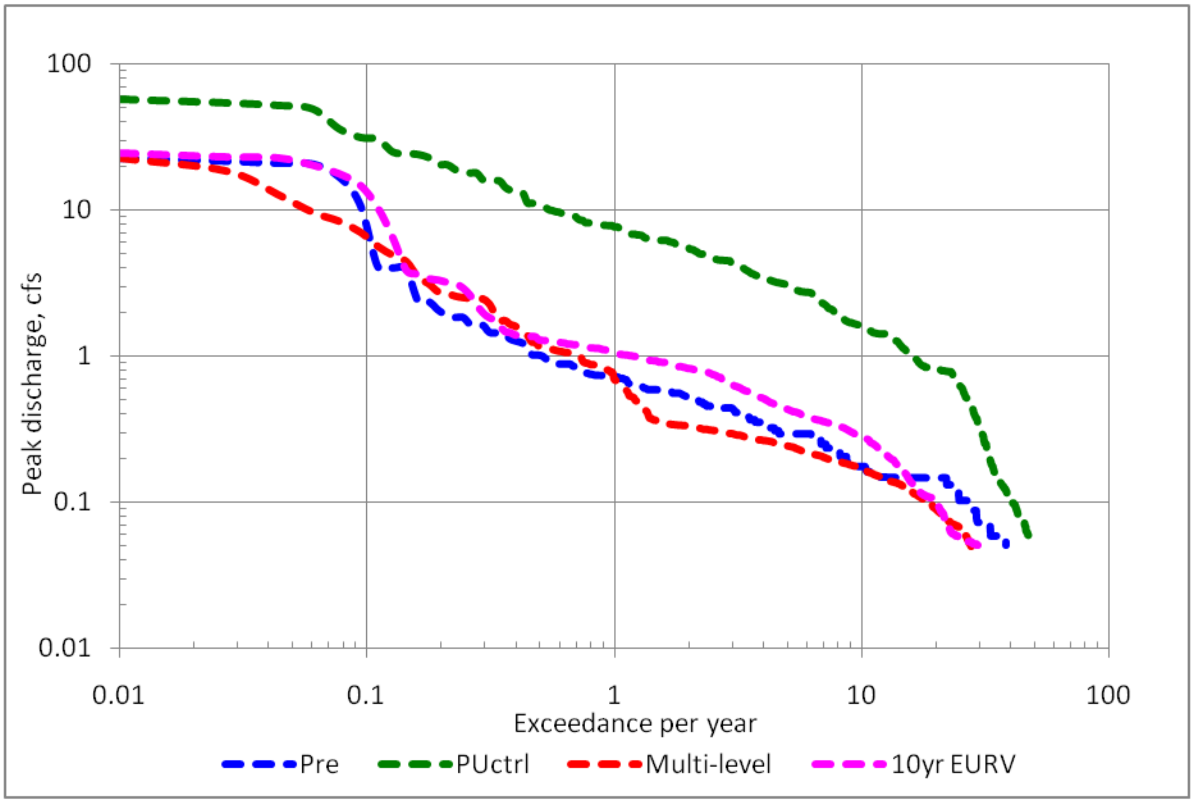

Figure 4-3 Peak flow frequency curve in Fort Collins

It is apparent that *Multi-level Detention* methods can easily regulate the 1.5-year, 10-year and 100-year peak discharges to the historic peak discharge levels as is demonstrated in Figure 4-3, and *Full Spectrum Detention* method performs less efficiently than the *Multi-level* method. Besides the three storm events, *Multi-level Detention* method provides a better control for smaller and more frequent storms in the range from 1-year to 30 times a year.

Figure 4-4, 4-5 and 4-6 show the peak flow frequency curve in the watershed of Atlanta. The design storms for Atlanta were computed from the actual precipitation record used for continuous simulations rather than using design storms from a drainage manual as was done if Fort Collins. In Figure 4-4, the peak flow frequency curve of the *Multi-level Detention* scenario matches closely to the pre-development curve, indicating that the *Multi-level Detention* was effective for storms from 1.5-year to 100-year. However, peak discharges from 1.25-year to 30 times a year storm are not as well fitted to the pre-development condition. Peak discharges that occur 31.1 times a year are restricted less by the *Multi-level Detention* than the pre-development condition.

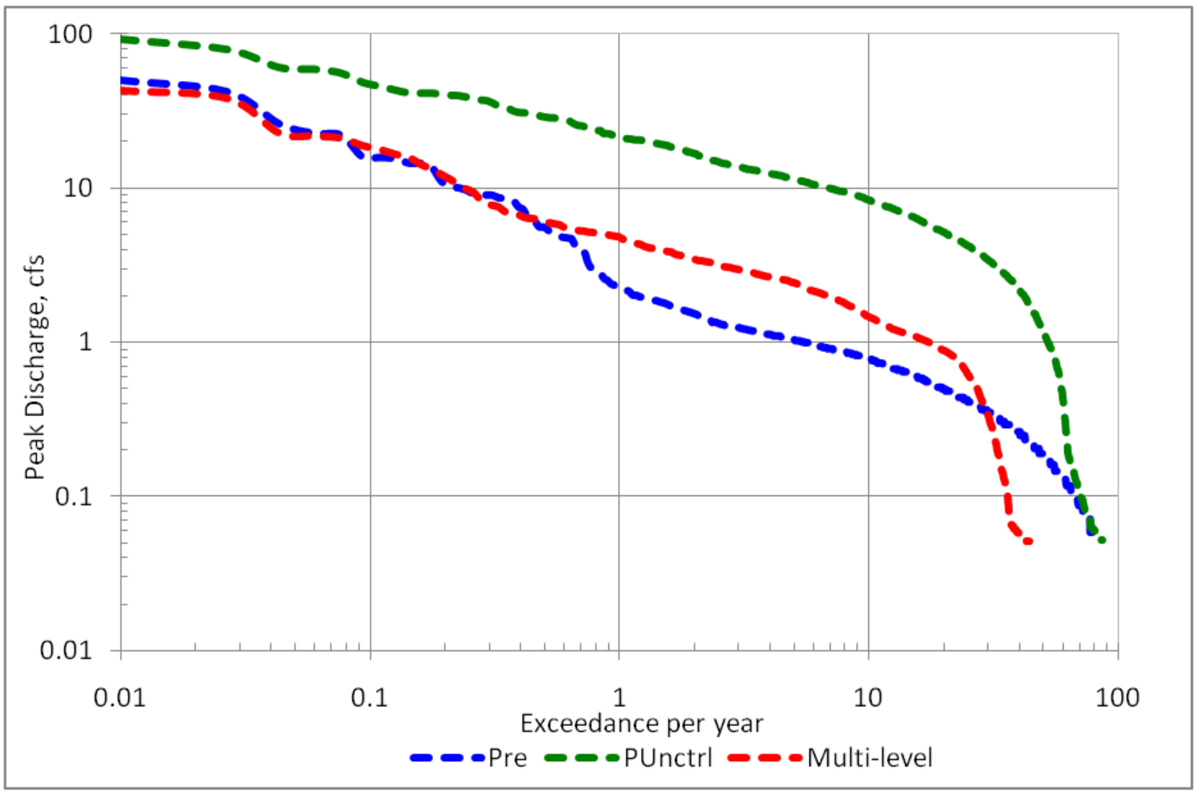

Figure 4-4 Peak flow frequency curve in Atlanta: *Multi-level Detention*

The comparison of three *Full Spectrum Detention* ponds is summarized in Figure 4-5. The peak discharges from 1.25-year to 100-year are restricted to the pre-development values by the 100-EURV scenario while the 1.5- and 10-year EURVs could only control post-development discharges from 12.5-year to 100-year. The three detention ponds are almost the same for the 3 to 21.2 times a year storm events and the peak discharges exceed the pre-developed conditions. Peak discharges that occur more than 21.2 times a year controlled by the three scenarios are less than those occurring under pre-development simulation. The 100-year EURV curve fits closest to the pre-development curve especially for storms with exceedance frequencies less than 0.5 per year compared with the other two scenarios.

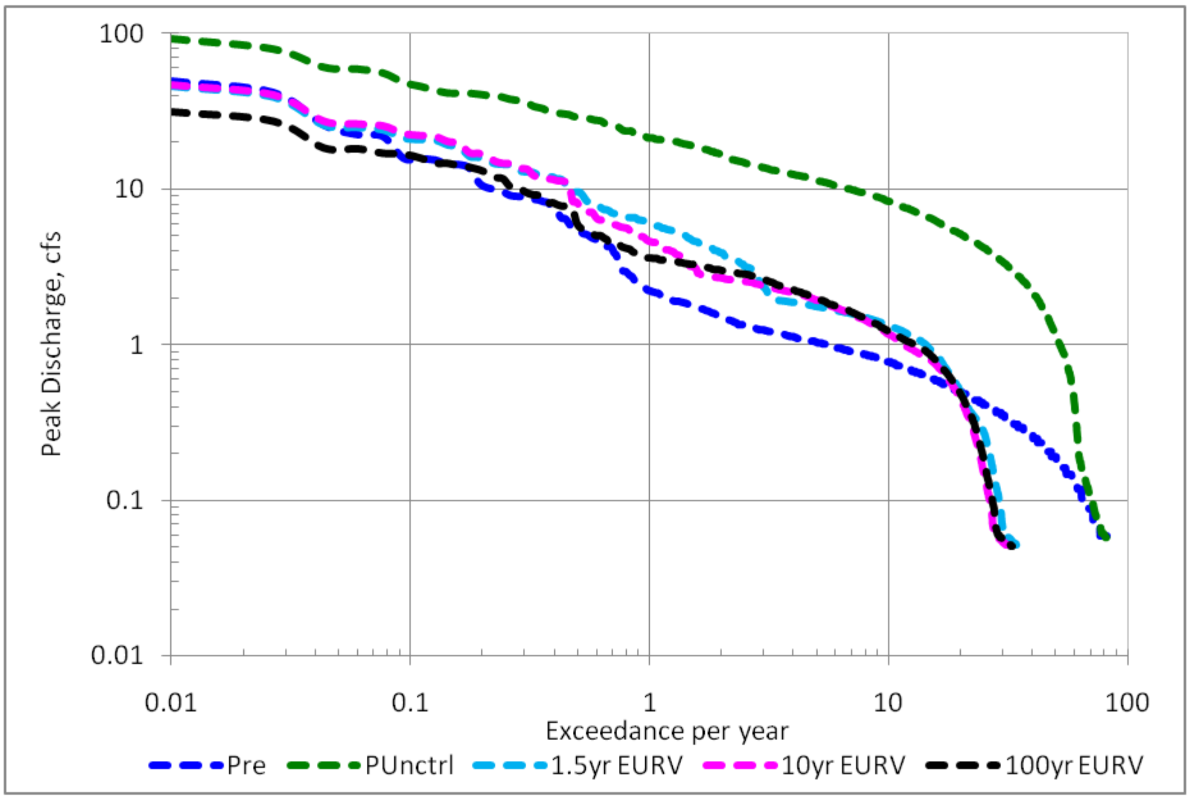

Figure 4-5 Peak flow frequency curve in Atlanta: *Full Spectrum Detention*

Figure 4-6 demonstrates that the peak flow frequency curves of both of the *Multi-level Detention* and 100-year EURV control scenarios have close fits to the pre-development curve from 1.25-year to 100-year storms. For the storms less than the 1.25-year, the 100-year EURV control scenario is a little better than the other one. Both of these two kinds of detention methods perform a good control at the peak flow frequency control in the watershed of Atlanta, the *Multi-level Detention* pond is good at peak shaving large storms and the *Full Spectrum Detention* pond is slightly more efficient in control of frequent small storms.

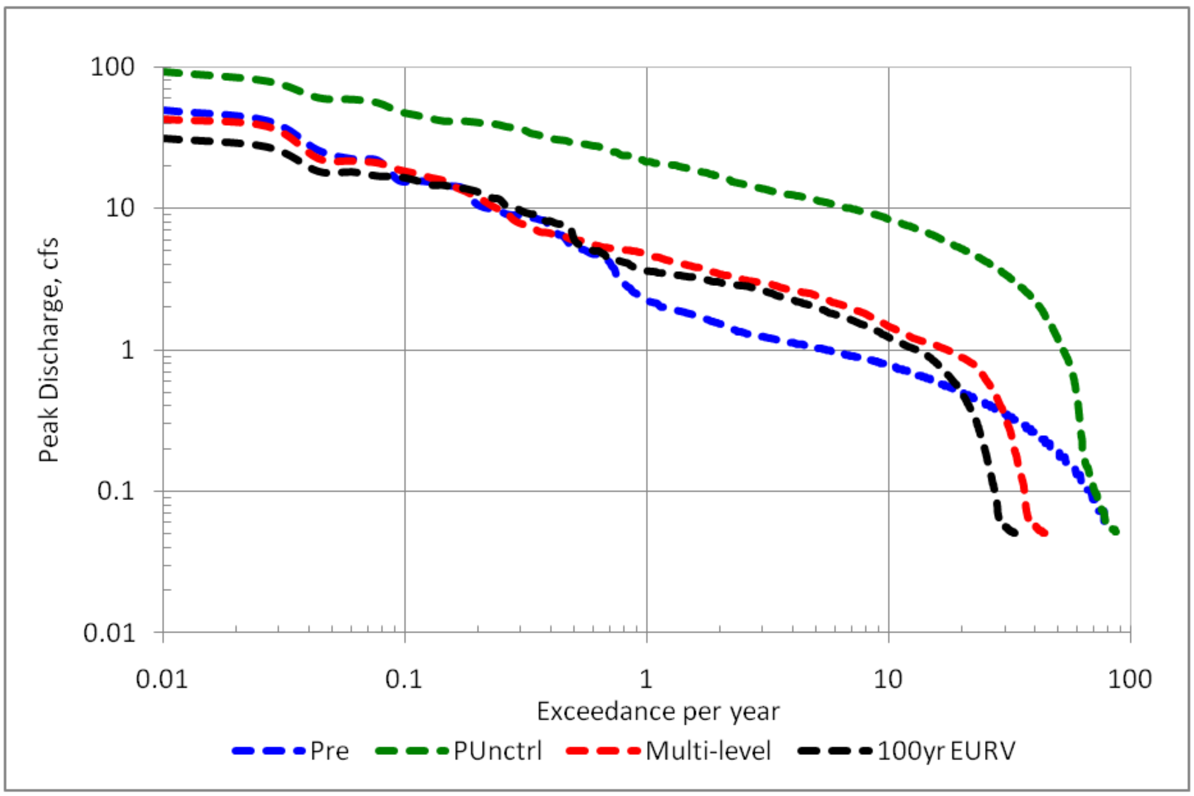

Figure 4-6 Peak flow frequency curve in Atlanta

4.3 Flow duration analysis

Flow duration curves, show the average percentage of time that specific flows are equal or exceeded at a watershed where continuous records are available. Since the study is focusing on the differences between the two different detention methods, the model is not calibrated to match the pre-development condition. The pre-development and post-development conditions in Fort Collins and Atlanta are not shown in the following two sections.

Figure 4-7, 4-8 and 4-9 show the duration that the downstream flows are equaled or exceeded over the 60-year continuous simulation for post-developed controlled conditions in Fort Collins; Figure 4-10, 4-11 and 4-12 show the results in Atlanta. In the following figures, one percent represents one percent of the total flow events during the 60-year simulation period. Figure 4-7 and Figure 4-10 show the flow duration curves from the *Multi-level Detention* ponds in stormwater control scenarios, while Figure 4-8 and Figure 4-11 show the *Full Spectrum Detention* ponds stormwater control in Fort Collins and Atlanta. Figure 4-9 and Figure 4-12 compare the two kinds of detention ponds' effect on the flow duration curves. Figure 4-9 demonstrates that, in Fort Collins, flows are greater than baseflow 20 percent of the total flow events for the post-development *Multi-level Detention* scenario, and around 25 percent of the flow events for the post-development 10-year EURV control scenario. The flow duration curves show that *Multi-level Detention* method provides a better control for the flows in the duration range of 0.1-2 percent in contrast to *Full Spectrum Detention* method which works better in the range of 0.003-0.05 percent.

Figure 4-12 shows that, in Atlanta, flows are greater than baseflow 30 percent of the total flow events for the post-development *Multi-level Detention* scenario, and around 60 percent of the total flow events for the post-development *Full-Spectrum* 100-year EURV scenario. The flow duration curves from both stormwater control detention ponds scenarios are fairly close to each other, and the 100-year EURV control regulates the flows in the duration range of 0.02-0.1 percent a little better than the *Multi-level Detention* method.

This phenomenon is because that the detention ponds capture the runoff, and release it for longer time period than the pre- and post-development discharge conditions. Quantitative stream boundary shear stress curves due to the flow duration changes are examined in section 4.3.

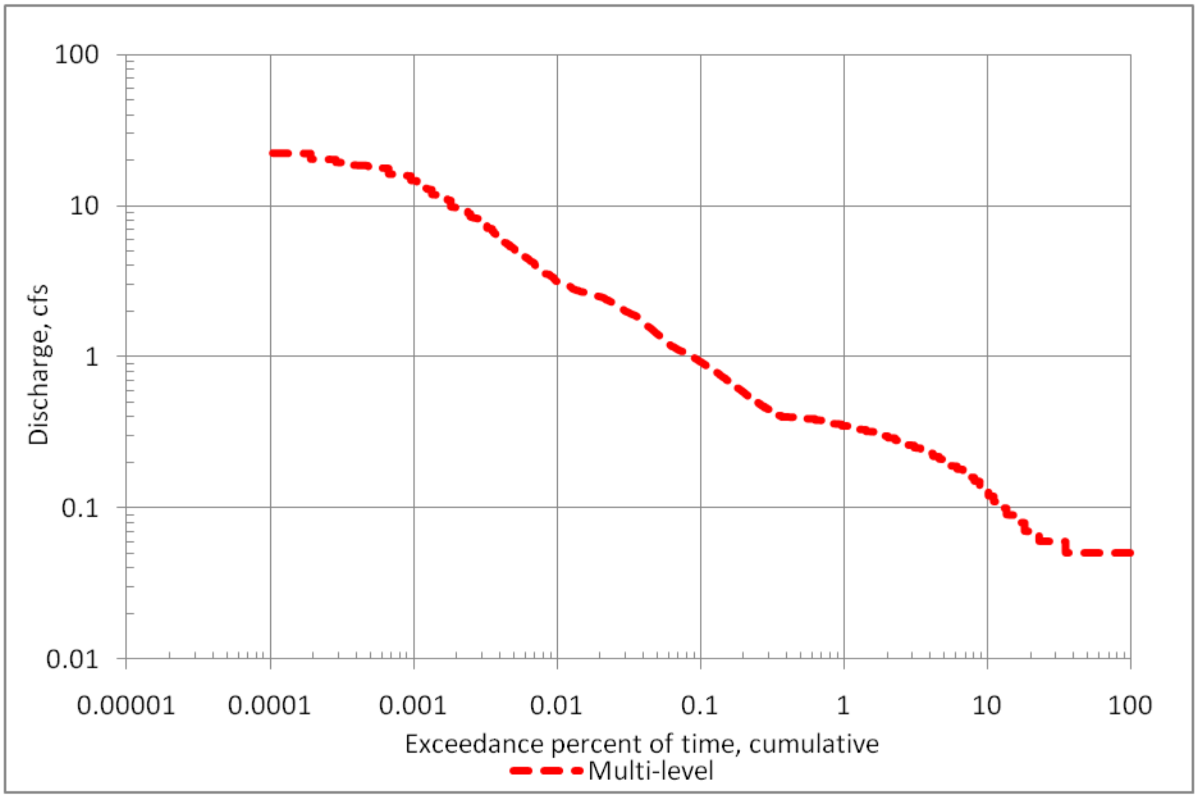

Figure 4-7 Flow duration curve in Fort Collins: *Multi-level Detention*

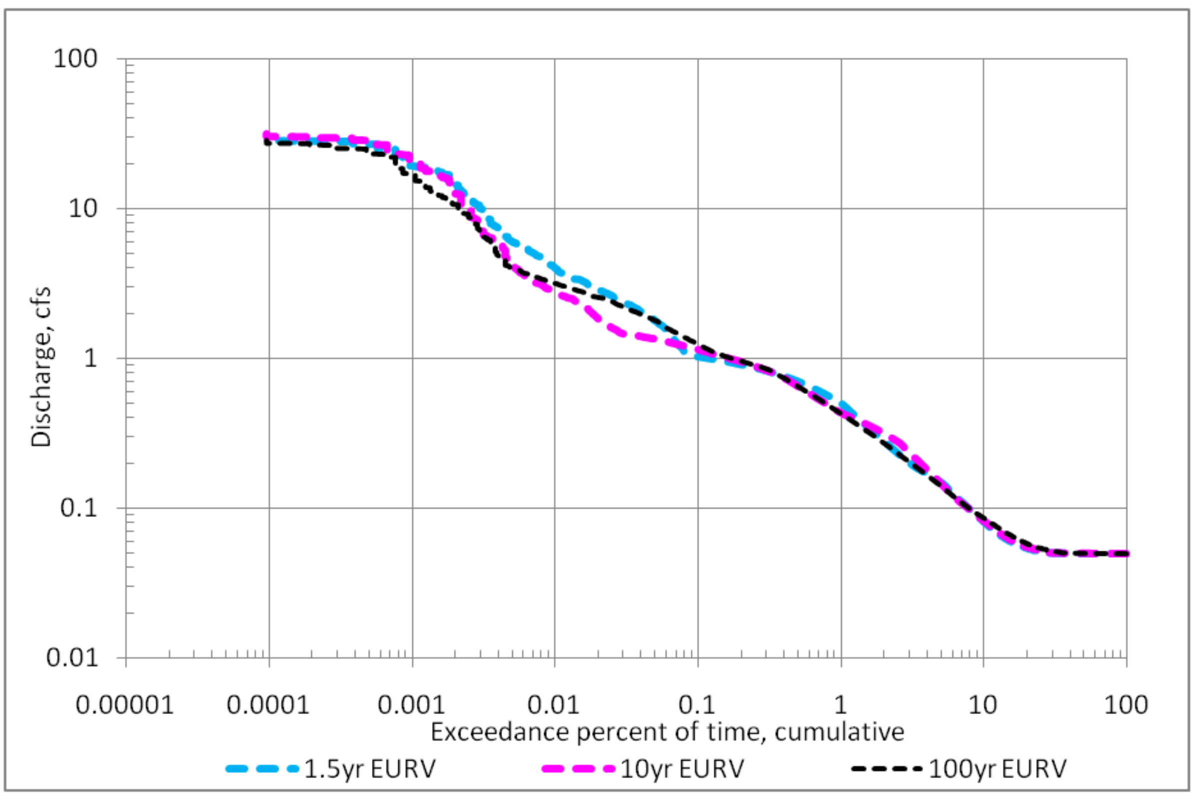

Figure 4-8 Flow duration curve in Fort Collins: *Full Spectrum Detention*

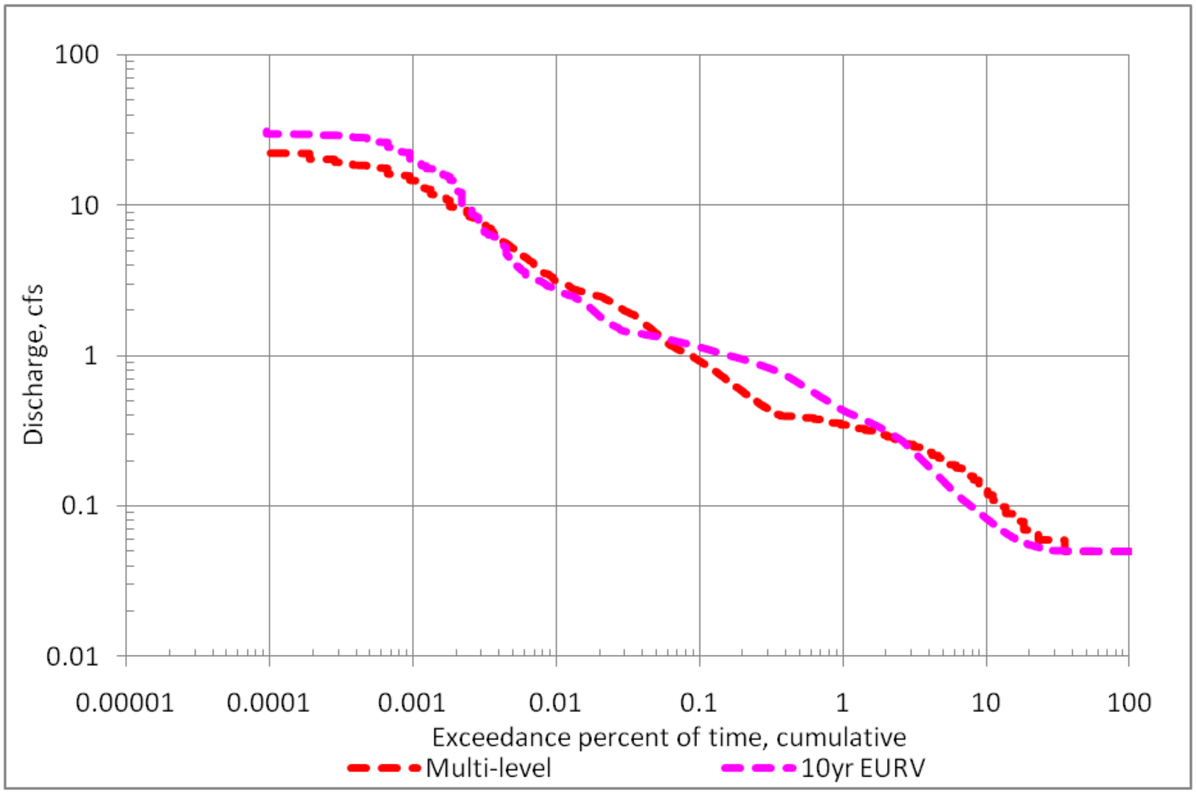

Figure 4-9 Flow duration curve in Fort Collins

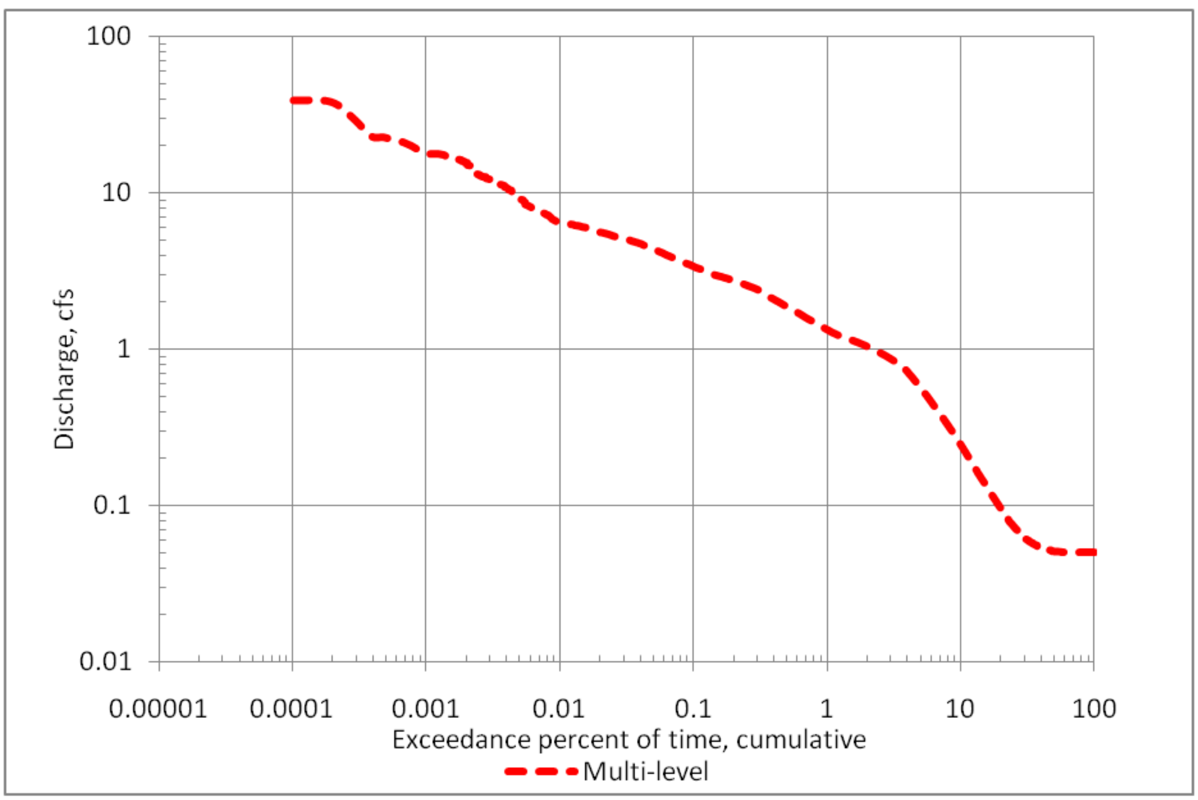

Figure 4-10 Flow duration curve in Atlanta: *Multi-level Detention*

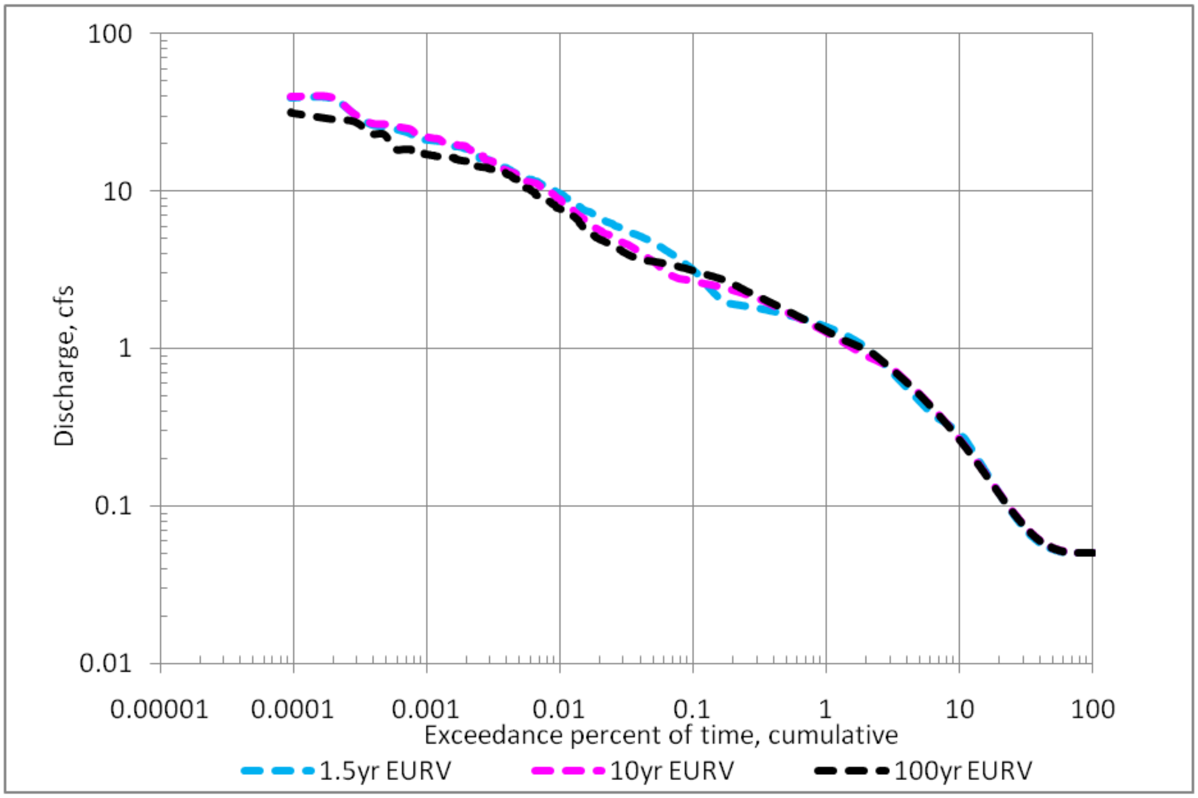

Figure 4-11 Flow duration curve in Atlanta: *Full Spectrum Detention*

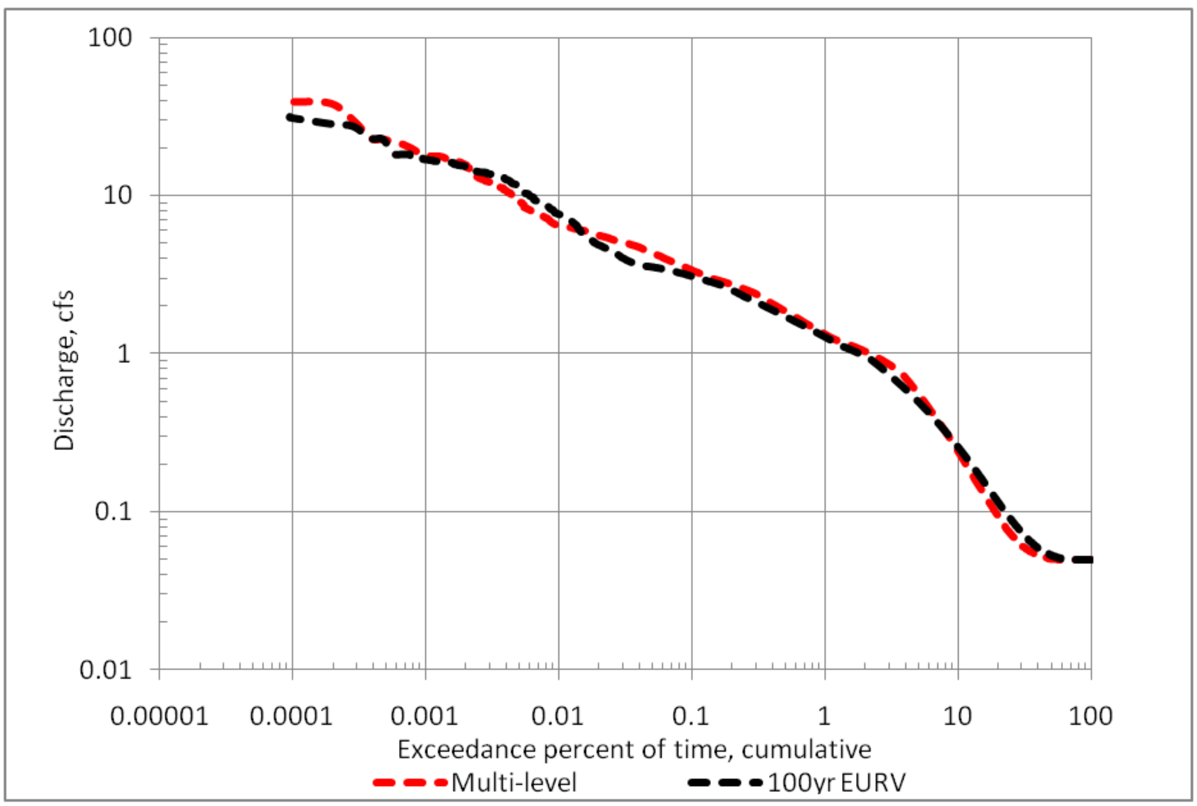

Figure 4-12 Flow duration curve in Atlanta

4.4 Average Boundary shear stress analysis

Based on the results from the previous two sections, for the *Full Spectrum Detention* methods, it is apparent that the 10-year EURV scenario performs the best compared with the other 1.5- and 100-year EURV in Fort Collins, and 100-year EURV the best in Atlanta. In this section, these two spectrum detention ponds were selected to compare with the *Multi-level Detention* ponds in each watershed.

Average boundary shear stress has a direct relationship with the sediment transport and stream erosion potential. To study the efficiency of the two stromwater design approaches on the stream morphology, the comparison of average boundary shear stress curves from the *Multi-level Detention* and *Full Spectrum Detention* methods is a simplified way. Figure 4-13 and Figure 4-14 demonstrate the channel shear stress curves in Fort Collins and Atlanta under the conditions of post-development controlled by two kinds of detention ponds.

The critical shear stress value for medium gravel is 5.7 Pa. From Figures 4-13 and 4-14, it can be seen that the incipient motion thresholds for medium gravel are exceeded 0.002– 0.02 percent of the total events time in Fort Collins and 0.004 – 0.2 percent of the total discharge events in Atlanta. The *Multi-level Detention* scenarios result in average boundary shear stresses that are well controlled for around 0.001 percent of time in Fort

Collins and 0.0015 percent of time in Atlanta. Shear stresses from the *Full Spectrum Detention* stormwater control scenario are greater than the *Multi-level Detention* scenarios in Fort Collins for 0.004 percent of time, and fit close to the *Multi-level Detention* condition in Atlanta.

In conclusion, shear stresses for the *Multi-level Detention* scenario provide a slightly better control than the *Full Spectrum Detention* scenario in Fort Collins, especially in the range of shear stresses that are above the critical shear stress for medium gravel. In Atlanta, the two detention ponds provide almost the same average boundary shear stresses.

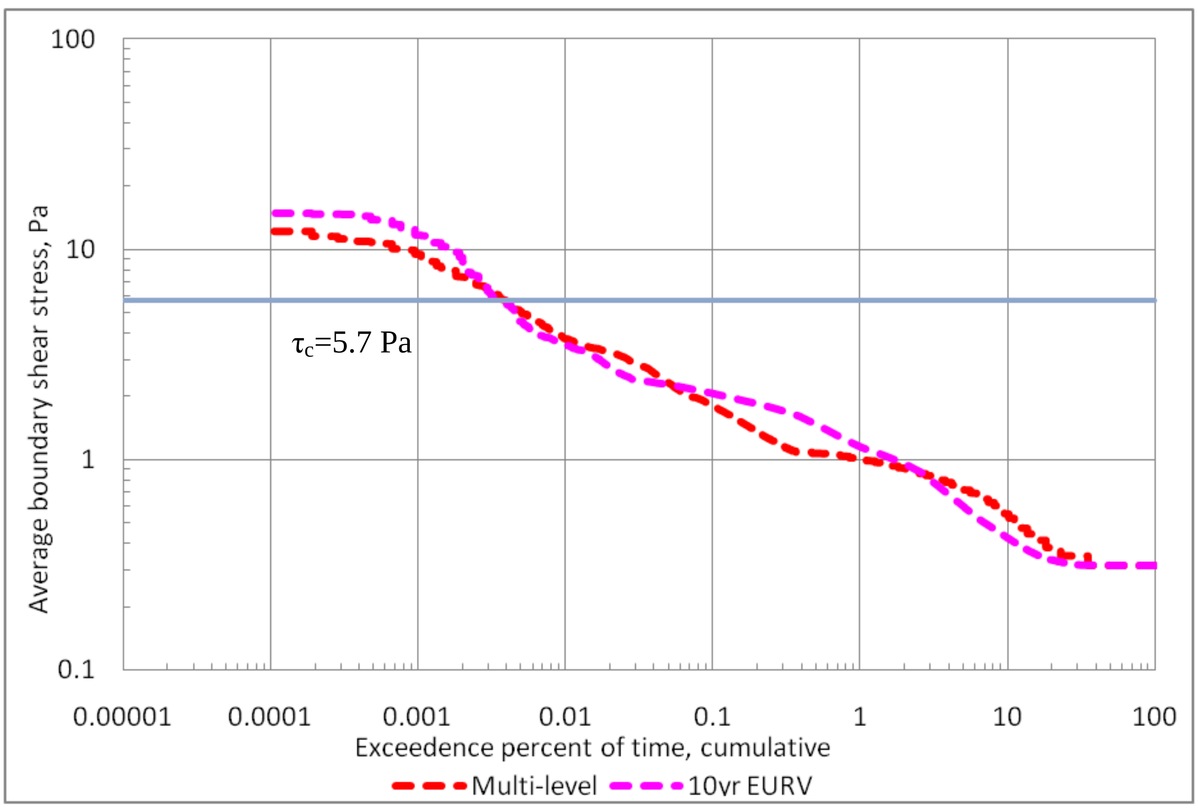

Figure 4-13 Average boundary shear stress curves in Fort Collins

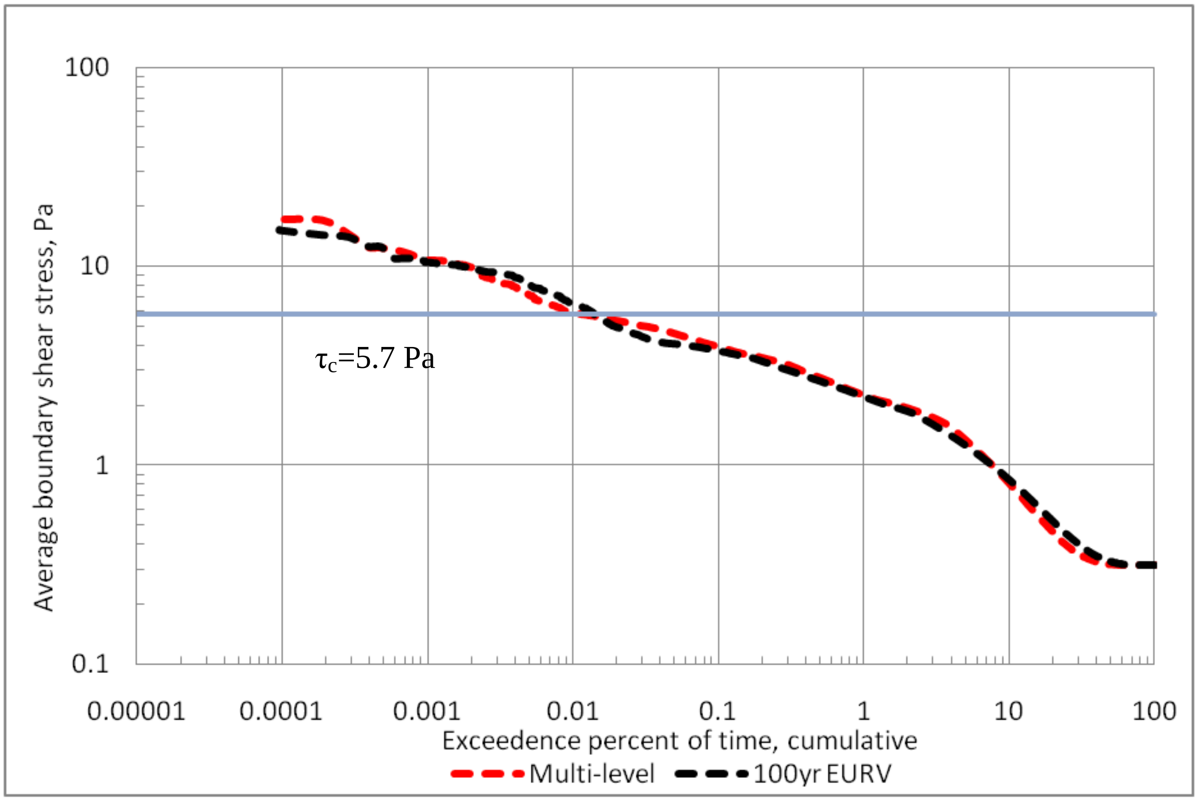

Figure 4-14 Average boundary shear stress curves in Atlanta

4.5 $T_{0.5}$ analysis

In this section, the undeveloped and developed without control scenario were used as the common base scenario. The effects development with stormwater controls were examined by comparing with the common scenarios. Similarly as the previous section, the 10-year EURV was chosen to compare with the *Multi-level Detention* in Fort Collins and 100-year EURV was chosen in Atlanta.

The trends observed in urban gradient studies in the Pacific Northwest (Booth et al. 2004) found that $T_{0.5}$ values were lower for watersheds with higher percentages of imperviousness, suggesting that the metrics are inversely proportional to degree of watershed development. Similarly as the conclusion, the developed uncontrolled scenario has a smaller $T_{0.5}$ value than the pre-development condition; however, this trend was not obvious. Figure 4-15 and Figure 4-16 demonstrate a positive trend for the post-development stormwater control scenarios. $T_{0.5}$ values of developed uncontrolled scenarios have the lowest $T_{0.5}$ values. In the developed controlled scenarios, *Multi-level Detention* method has the highest value in Fort Collins, while *Full Spectrum Detention* approach has the highest values for Atlanta. However, both of these two detention approaches produce higher $T_{0.5}$ values than the pre-development condition, which were not expected. The effect was observed in this analysis because the flow time series for the undeveloped scenarios were extremely flashy to begin with (Egderly et. al, 2004). An integration of engineering and ecological approaches is needed to facilitate a more complete assessment of the consequences associated with various alternatives facing decision makers at the planning and design stage of development.

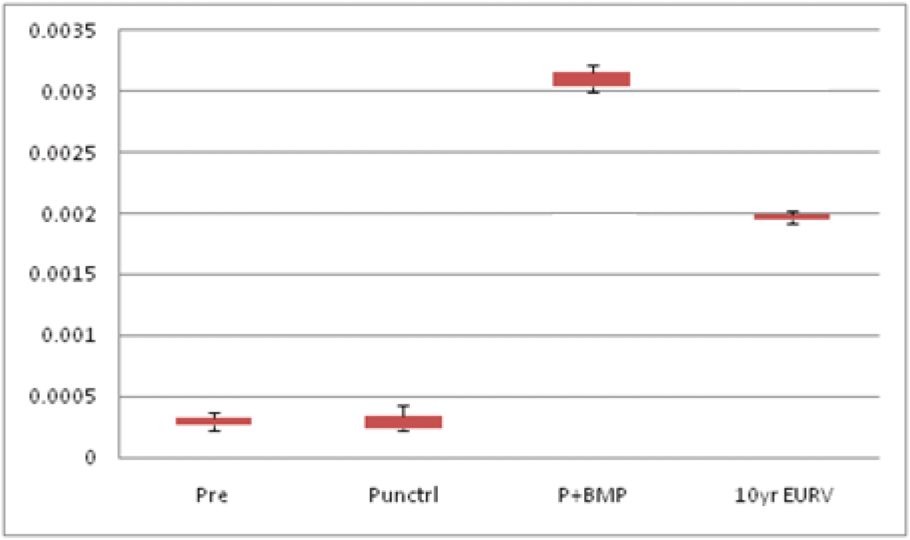

Figure 4-15 $T_{0.5}$ of Fort Collins

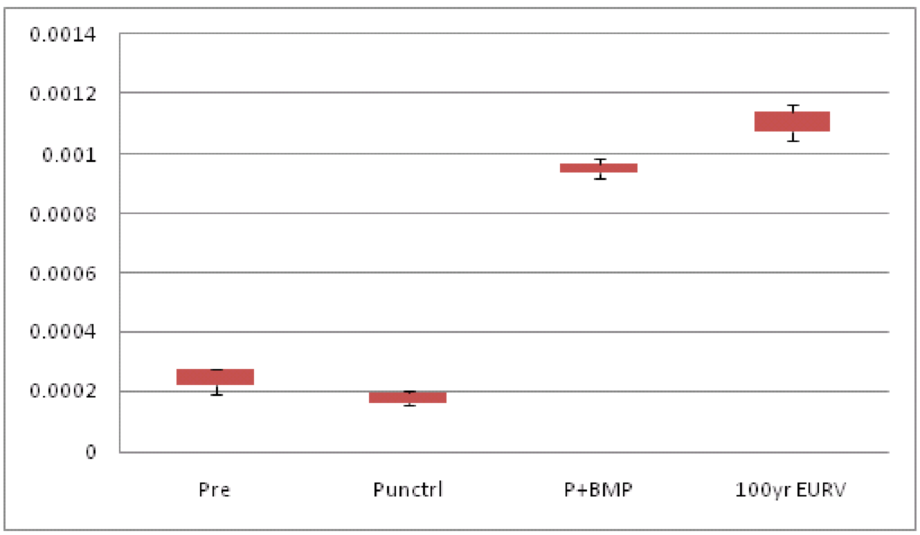

Figure 4-16 $T_{0.5}$ of Atlanta

5 CONCLUSIONS AND RECOMMENDATIONS

The work of Wulliman and Urbonas (2005, 2007) in Denver region shows that EURV became a constant value once the watershed imperviousness is higher than 20 percent for various design storms. The validation results in Section 3.3 indicate that the EURV constant conclusion could not be applied in either Fort Collins, located 57 miles (92 km) north of Denver, or in Atlanta by using the SWMM model and CUHP model. The SWMM model provides a better and more reasonable performance in computing the EURV in both Fort Collins and Atlanta by applying both 5-minute and 15-minute rainfall intervals for the 1.5- 10- and 100-year design storms. The CUHP model overestimated the total runoff volumes during the validation process, probably due to the limitations of the CUHP model for modeling infiltration. Moreover, the CUHP model has been developed and calibrated using basins between 0.15 and 3.08 square miles, so the appropriate basin size for CUHP simulation is between 0.003 and 5 square miles (2 to 3200 acres). However, the areas of most subcatchments are less than 2 acres in the developed condition in this study, which violates the limitation of CUHP.

Even though EURV values were not constant for Fort Collins and Atlanta, the *Full Spectrum Detention* ponds were designed by applying separate EURV values for each design storm. It turns out that the 10-year EURV control performs better than the other two EURV controls in the watershed of Fort Collins, the semi-arid climate area, which receives relatively low amounts of annual rainfall. For the humid climate of Atlanta which has frequent precipitation of variable intensity and long duration, the 100-year EURV control has the best results compared with the 1.5- and 10-year EURVs control in peak shaving, peak flow frequency, flow duration control, average boundary shear stress and $T_{0.5}$.

Chapter four shows that the *Multi-level Detention* performs better than the *Full Spectrum Detention* design approach in the evaluation indices in Fort Collins, and both detention methods have similar performances in Atlanta. In the peak shaving in design storms, the 10-year EURV control has very close peak shaved values for the 1.5-year and 100-year storms using the *Multi-level Detention* method in Fort Collins. For the 10-year storm event, the 10-year EURV control restricts the 10-year peak discharge to a smaller value, which is lower than the pre-development peak discharges. Similarly in Atlanta, the 100-year EURV control releases the 100-year discharge at a rate which is also smaller than the pre-developed condition. The efficiency of the 1.5-year peak discharge control was almost the same for both detention ponds, while the 100-year EURV control could not restrict the 10-year peak discharge values to the historic rate. Section 4.1 demonstrates that the *Multi-level* control could successfully peak shave the post-development discharge to the historic values, while the *Full Spectrum Detention* method could achieve the results for most of the storms events, but not all.

Results from the peak flow frequency analysis show that in the watershed of Fort Collins, where the design storms are from the City of Fort Collins Design Criteria (1997), the initially *Multi-level Detention* method fails to provide full control for most of the storm events. The continuous simulation peak discharge values were used as the target peak discharge rates for the design storms to resize the *Multi-level Detention* design, which allows flow frequency curve to better match the curve for undeveloped conditions. The *Multi-level Detention* methods could easily regulate the 1.5-year, 10-year and 100-year peak discharges to the historic peak discharge levels, and the *Full Spectrum Detention* method performs less efficiently than the *Multi-level Detention* method. Besides the three storm events, the *Multi-level Detention* method provides a better control for smaller and more frequent storms in the range from 1-year to 30 times a year. In Atlanta, both of the *Multi-level Detention* and the 100-year EURV control scenarios have close fits to the pre-development curve from 1.25-year to 100-year storms. For the storms less than the 1.25-year, the 100-year EURV control scenario is performing better than the *Multi-level Detention* method. The *Multi-level Detention* pond is good at peak shaving large storms and the *Full Spectrum Detention* pond is more efficient in the frequent small storms control.

The flow duration curves show that the *Multi-level Detention* method provides a better control for the flows in the duration range of 0.1-2 percent in contrast to *Full Spectrum Detention* method which works better in the range of 0.003-0.05 percent in the watershed of Fort Collins. The results in Atlanta are quite different from Fort Collins, and the flow duration curves from both stormwater control detention ponds scenarios are fairly close to each other. Also, the 100-year EURV control regulates the flows in the duration range of 0.02-0.1 percent a little better than the *Multi-level Detention* method.

Due to the effects of the flow duration curves, shear stresses for the *Multi-level Detention* scenario shows better control than the *Full Spectrum Detention* scenario in Fort Collins, especially in the range of shear stresses that are above the critical shear stress for medium gravel. In Atlanta, the two detention ponds provide almost the same average boundary shear stresses.

The $T_{0.5}$ analysis results in both Fort Collins and Atlanta in this study partially come with the conclusions from Booth et al (2004). The results from Booth et. al. (2004)

showed that $T_{0.5}$ values were lower for watersheds with higher percentages of imperviousness, suggesting that the metrics are inversely proportional to degree of watershed development. The trends observed in this study showed that the developed uncontrolled scenarios have the lowest $T_{0.5}$ values, *Multi-level Detention* method has the highest value in Fort Collins; *Full Spectrum Detention* method works best in Atlanta. However, the $T_{0.5}$ values of these two stormwater management detention methods are higher than the pre-developed condition, which is not expected. It may because of the flow time series for the undeveloped scenarios were extremely flashy to begin with, and further study is required.

The research demonstrated in this thesis indicates that further study is required. The study chose a conceptual watershed of 29.15 acres to do a series of analyses, such as validate the *Full Spectrum Detention* assumption by using SWMM and CUHP and evaluate the efficiency of two design approaches. In the future, real watersheds with differently hydrologic characteristics including varied size, shape, slope, infiltration parameters, and evaporation rates with should be used to calibrate both of the models, test the EURV constant assumption, and evaluate the performance of stormwater management practices. Additional types of stormwater controls such as low impact development (LID), infiltration basins, and peak shaving without control of the WQCV should also be evaluated. More stream morphology parameters needs to be studied: stream erosion potential, sediment load, T_{Qmean}. In the process of pond design, the design storms calculated from the continuous precipitation records are recommended for use.

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APPENDIX A

Table A-1 City of Fort Collins Design Storms for using SWMM

Time (hr:min)	1.5-year intensity, in/hr	10-year intensity, in/hr	100-year intensity, in/hr
0:05	0.21	0.49	1
0:10	0.42	0.56	1.14
0:15	0.89	0.65	1.33
0:20	1.69	1.09	2.23
0:25	2.64	1.39	2.84
0:30	1.48	2.69	5.49
0:35	0.67	4.87	9.95
0:40	0.53	2.02	4.12
0:45	0.32	1.21	2.48
0:50	0.32	0.71	1.46
0:55	0.32	0.6	1.22
1:00	0.32	0.52	1.06
1:05	0.32	0.39	1
1:10	0.21	0.37	0.95
1:15	0.21	0.35	0.91
1:20	0.21	0.34	0.87
1:25	0.21	0.32	0.84
1:30	0.21	0.31	0.81
1:35	0.21	0.3	0.78
1:40	0.21	0.29	0.75
1:45	0.21	0.28	0.73
1:50	0.21	0.27	0.71
1:55	0.11	0.26	0.69
2:00	0.11	0.25	0.67

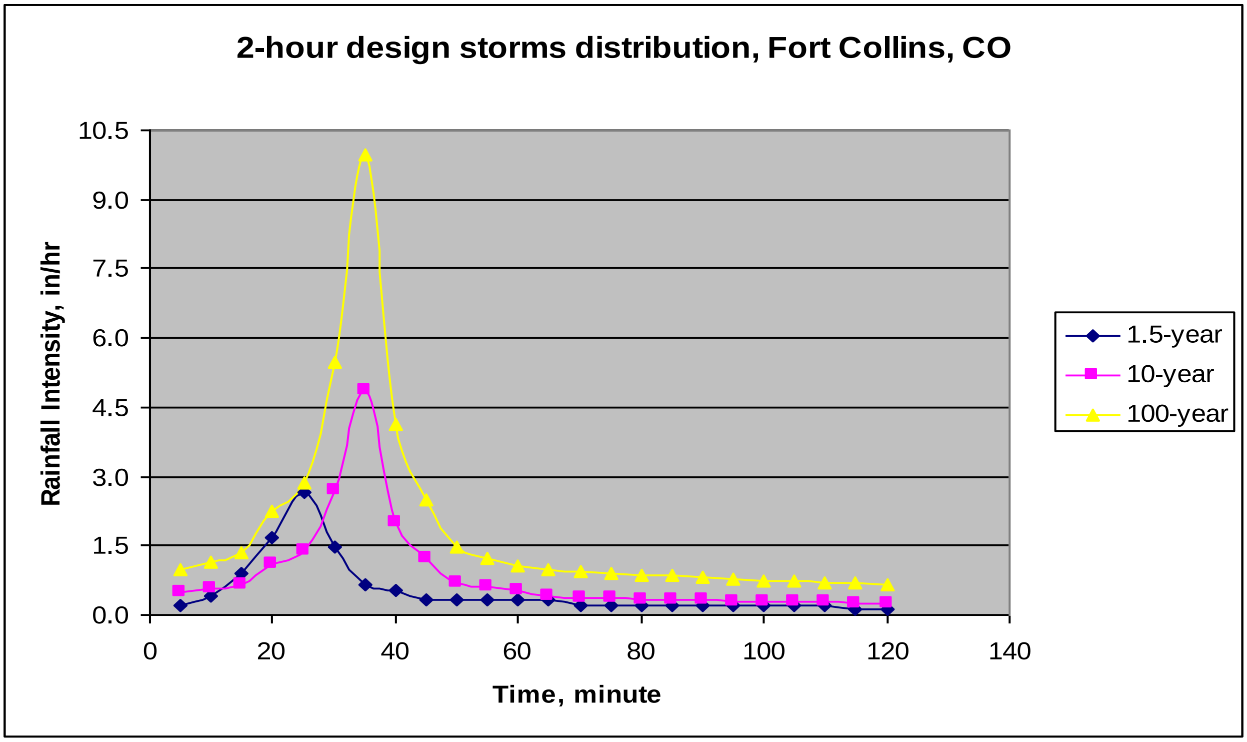

Figure A-1 Two-hour design storm distribution, Fort Collins CO

Table A-2 Atlanta Design Storms for using SWMM

Time (hr:min)	1.5-year intensity, in/hr	10-year intensity, in/hr	100-year intensity, in/hr
0:05	0.33	0.52	0.43
0:10	0.66	0.96	1.29
0:15	1.39	2.14	1.98
0:20	2.65	3.91	3.44
0:25	4.14	6.51	6.01
0:30	2.32	3.12	10.74
0:35	1.04	1.46	6.01
0:40	0.83	1.12	3.44
0:45	0.50	0.99	2.66
0:50	0.50	0.83	2.15
0:55	0.50	0.83	1.72
1:00	0.50	0.83	1.72
1:05	0.50	0.83	1.72
1:10	0.33	0.83	0.86
1:15	0.33	0.83	0.86
1:20	0.33	0.65	0.52
1:25	0.33	0.49	0.52
1:30	0.33	0.49	0.52
1:35	0.33	0.49	0.52
1:40	0.33	0.49	0.52
1:45	0.33	0.49	0.52
1:50	0.33	0.49	0.52
1:55	0.17	0.44	0.52
2:00	0.17	0.34	0.52

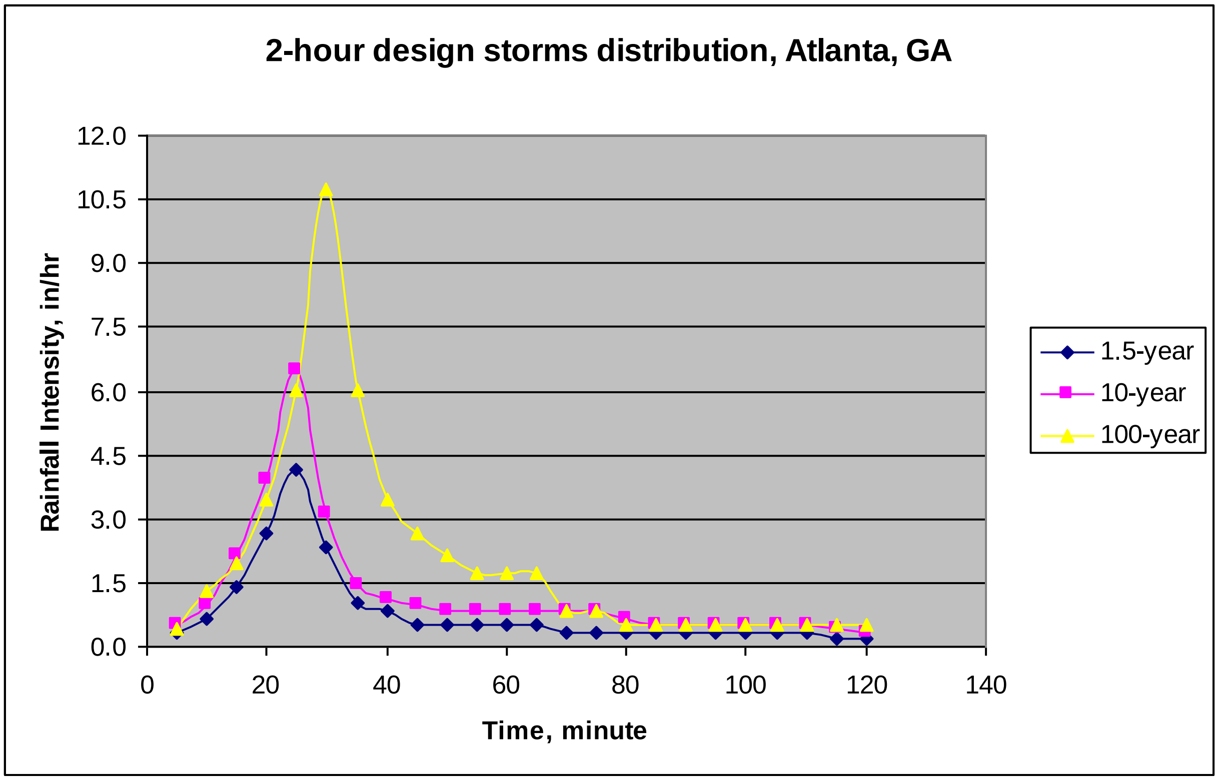

Figure A-2 Two-hour design storm distribution, Atlanta, GA

Table A-3 Conveyance Channel Characteristics (Runoff): Developed Conditions.

Name	Inlet Node	Outlet Node	Length, ft	Manning N
C1	J1	J2	271.52	0.05
C2	J2	J3	142.42	0.05
C3	J3	J8	49.11	0.05
C4	J4	J5	251.84	0.016
C5	J5	J15	714.7	0.016
C6	J6	J7	146.25	0.016
C7	J7	J8	141.99	0.05
C8	J8	J11	205.37	0.05
C10	J10	J11	162.12	0.05
C11	J11	J12	104.01	0.016
C12	J12	J13	145.64	0.05
C13	J13	J14	313	0.05
C14	J14	J15	166.11	0.05
C15	J15	O1	95.3	0.016
C17	J17	J14	184.68	0.016
C_Aux1	Aux1	J1	176.46	0.016
C_Aux2	Aux2	J2	254.05	0.016
C_Aux3	Aux3	J4	179.08	0.016
C_Aux4	Aux4	J5	229.58	0.016
C_Aux5	Aux5	Aux6	311.72	0.016
C_Aux6	Aux6	J6	267.19	0.05
C_Aux7	Aux7	J7	202.9	0.016
C_Aux9	Aux8	J10	437.5	0.016
C_Aux10	Aux10	J13	469.2	0.016
C_Aux12	Aux12	Aux13	271.81	0.016
C_Aux13	Aux14	J17	100.77	0.016
C_Aux14	Aux13	J17	141.47	0.016
C_Aux15	Aux15	J15	449.46	0.016

Table A-4 Conveyance Channel Characteristics (Runoff): Developed Conditions

Link	Shape	Geom1	Geom2	Geom3	Geom4	Barrels
C1	TRAPEZOIDAL	1.5	3	4	4	1
C2	TRAPEZOIDAL	2	3	4	4	1
C3	TRAPEZOIDAL	1.5	3	3	3	1
C4	TRAPEZOIDAL	1	0	0.0001	25	1
C5	TRAPEZOIDAL	1	0	0.0001	25	1
C6	CIRCULAR	3	0	0	0	1
C7	TRAPEZOIDAL	1.5	3	3	3	1
C8	TRAPEZOIDAL	2	3	3.5	3.5	1
C10	TRAPEZOIDAL	1	0	0.0001	25	1
C11	CIRCULAR	3	0	0	0	1
C12	TRAPEZOIDAL	2	3	3.5	3.5	1
C13	TRAPEZOIDAL	2.5	3	3.5	3.5	1
C14	TRAPEZOIDAL	2.5	4	4	4	1
C15	CIRCULAR	5.5	0	0	0	1
C17	TRAPEZOIDAL	1	0	0.0001	25	1
C_Aux1	TRAPEZOIDAL	1	0	0.0001	25	1
C_Aux2	TRAPEZOIDAL	1	0	0.0001	25	1
C_Aux3	TRAPEZOIDAL	1	0	0.0001	25	1
C_Aux4	TRAPEZOIDAL	1	0	0.0001	25	1
C_Aux5	TRAPEZOIDAL	1	0	0.0001	25	1
C_Aux6	TRAPEZOIDAL	1	3	2.5	2.5	1
C_Aux7	TRAPEZOIDAL	1	0	0.0001	25	1
C_Aux9	TRAPEZOIDAL	1	0	0.0001	25	1
C_Aux10	TRAPEZOIDAL	1	0	0.0001	25	1
C_Aux12	TRAPEZOIDAL	1	0	0.0001	25	1
C_Aux13	TRAPEZOIDAL	1	0	0.0001	25	1
C_Aux14	TRAPEZOIDAL	1	0	0.0001	25	1
C_Aux15	TRAPEZOIDAL	1	0	0.0001	25	1

Table A-5 City of Fort Collins Design Storms for using SWMM15-minute interval

Time (hr:min)	1.5-year, in/hr	10-year, in/hr	100-year, in/hr
0:15	0.51	0.57	1.16
0:30	1.94	1.72	3.52
0:45	0.50	2.70	5.52
1:00	0.32	0.61	1.25
1:15	0.25	0.37	0.95
1:30	0.21	0.32	0.84
1:45	0.21	0.29	0.75
2:00	0.14	0.26	0.69

Table A-6 Atlanta Design Storms for using SWMM15-minute interval

Time (hr:min)	1.5-year, in/hr	10-year, in/hr	100-year, in/hr
0:15	0.79	1.21	1.23
0:30	3.04	4.51	6.73
0:45	0.79	1.19	4.04
1:00	0.50	0.83	1.86
1:15	0.39	0.83	1.15
1:30	0.33	0.55	0.52
1:45	0.33	0.49	0.52
2:00	0.22	0.43	0.52

APPENDIX B

Table B-1 EURV of 1.5 year storm for soil type C/D in Atlanta

Imperv, %	Pre, ft^3	Post, ft^3	EURV, ft^3
20	11056.8	42362.4	31305.6
30	11056.8	58066.2	47009.4
40	11056.8	73657.8	62601
50	11056.8	89148.6	78091.8
60	11056.8	104533.8	93477
70	11056.8	119772.6	108715.8
80	11056.8	134858.4	123801.6
90	11056.8	149732.4	138675.6
100	11056.8	164328.6	153271.8

Table B-2 EURV of 10 year storm for soil type C/D in Atlanta

Imperv, %	Pre, ft^3	Post, ft^3	EURV, ft^3
20	59862.6	121767	61904.4
30	59862.6	139902	80039.4
40	59862.6	157821.6	97959
50	59862.6	175514.4	115651.8
60	59862.6	193002	133139.4
70	59862.6	210273.6	150411
80	59862.6	227341.8	167479.2
90	59862.6	244208.4	184345.8
100	59862.6	260869.2	201006.6

Table B-3 EURV of 100 year storm for soil type C/D in Atlanta

Imperv, %	Pre, ft^3	Post, ft^3	EURV, ft^3
20	206298.6	291522.6	85224
30	206298.6	310236.6	103938
40	206298.6	328650.6	122352
50	206298.6	346783.8	140485.2
60	206298.6	364647.6	158349
70	206298.6	382251	175952.4
80	206298.6	399575.4	193276.8
90	206298.6	416674.2	210375.6
100	206298.6	433550.4	227251.8

Table B-4 EURV of 1.5 year storm for soil type B in Atlanta

Imperv, %	Pre, ft^3	Post, ft^3	EURV, ft^3
20	8184	32965.2	24781.2
30	8184	49438.8	41254.8
40	8184	65868	57684
50	8184	82294.2	74110.2
60	8184	98711.4	90527.4
70	8184	115119	106935
80	8184	131524.2	123340.2
90	8184	147924.6	139740.6
100	8184	164328.6	156144.6

Table B-5 EURV of 10 year storm for soil type B in Atlanta

Imperv, %	Pre, ft^3	Post, ft^3	EURV, ft^3
20	36671.4	94725.6	58054.2
30	36671.4	116181.6	79510.2
40	36671.4	137461.8	100790.4
50	36671.4	158536.8	121865.4
60	36671.4	179424	142752.6
70	36671.4	200114.4	163443
80	36671.4	220601.4	183930
90	36671.4	240869.4	204198
100	36671.4	260869.2	224197.8

Table B-6 EURV of 100 year storm for soil type B in Atlanta

Imperv, %	Pre, ft^3	Post, ft^3	EURV, ft^3
20	172649.4	263078.4	90429
30	172649.4	285364.2	112714.8
40	172649.4	307339.2	134689.8
50	172649.4	329036.4	156387
60	172649.4	350459.4	177810
70	172649.4	371641.2	198991.8
80	172649.4	392548.2	219898.8
90	172649.4	413214.6	240565.2
100	8184	269085	260901

Table B-7 EURV of 1.5 year storm for soil type C/D in Fort Collins

Imperv, %	Pre, ft^3	Post, ft^3	EURV, ft^3
20	5387.4	20586.81	15199.41
30	5387.4	30880.21	25492.81
40	5387.4	41307.29	35919.89
50	5387.4	51600.69	46213.29
60	5387.4	64701.39	59313.99
70	5387.4	75529.51	70142.11
80	5387.4	86357.64	80970.24
90	5387.4	97052.08	91664.68
100	5387.4	107880.2	102492.8

Table B-8 EURV of 10 year storm for soil type C/D in Fort Collins

Imperv, %	Pre, ft^3	Post, ft^3	EURV, ft^3
20	16710.07	53873.26	37163.19
30	16710.07	70315.97	53605.9
40	16710.07	86758.68	70048.61
50	16710.07	102934	86223.96
60	16710.07	118975.7	102265.6
70	16710.07	134883.7	118173.6
80	16710.07	150524.3	133814.2
90	16710.07	165897.6	149187.5
100	16710.07	180869.8	164159.7

Table B-9 EURV of 100 year storm for soil type C/D in Fort Collins

Imperv, %	Pre, ft^3	Post, ft^3	EURV, ft^3
20	157609.4	239154.5	81545.14
30	157609.4	258805.6	101196.2
40	157609.4	278189.2	120579.9
50	157609.4	297305.6	139696.2
60	157609.4	316020.8	158411.5
70	157609.4	334602.4	176993.1
80	157609.4	352916.7	195307.3
90	157609.4	370829.9	213220.5
100	157609.4	388342	230732.6

Table B-10 EURV of 1.5 year storm for soil type B in Fort Collins

Imperv, %	Pre, ft^3	Post, ft^3	EURV, ft^3
20	5105.4	20669.4	15564
30	5105.4	31030.8	25925.4
40	5105.4	41385.6	36280.2
50	5105.4	51706.2	46600.8
60	5105.4	62022	56916.6
70	5105.4	72333.6	67228.2
80	5105.4	82633.8	77528.4
90	5105.4	92943.6	87838.2
100	5105.4	103239.6	98134.2

Table B-11 EURV of 10 year storm for soil type B in Fort Collins

Imperv, %	Pre, ft^3	Post, ft^3	EURV, ft^3
20	9160.8	37521	28360.2
30	9160.8	55066.2	45905.4
40	9160.8	72566.4	63405.6
50	9160.8	90035.4	80874.6
60	9160.8	107473.8	98313
70	9160.8	124860	115699.2
80	9160.8	142156.2	132995.4
90	9160.8	159319.8	150159
100	9160.8	176201.4	167040.6

Table B-12 EURV of 100 year storm for soil type B in Fort Collins

Imperv, %	Pre, ft^3	Post, ft^3	EURV, ft^3
20	123808.2	208518.6	84710.4
30	123808.2	231354.6	107546.4
40	123808.2	253911	130102.8
50	123808.2	276197.4	152389.2
60	123808.2	298218.6	174410.4
70	123808.2	319966.2	196158
80	123808.2	341448	217639.8
90	123808.2	362659.8	238851.6
100	123808.2	383544	259735.8

APPENDIX C

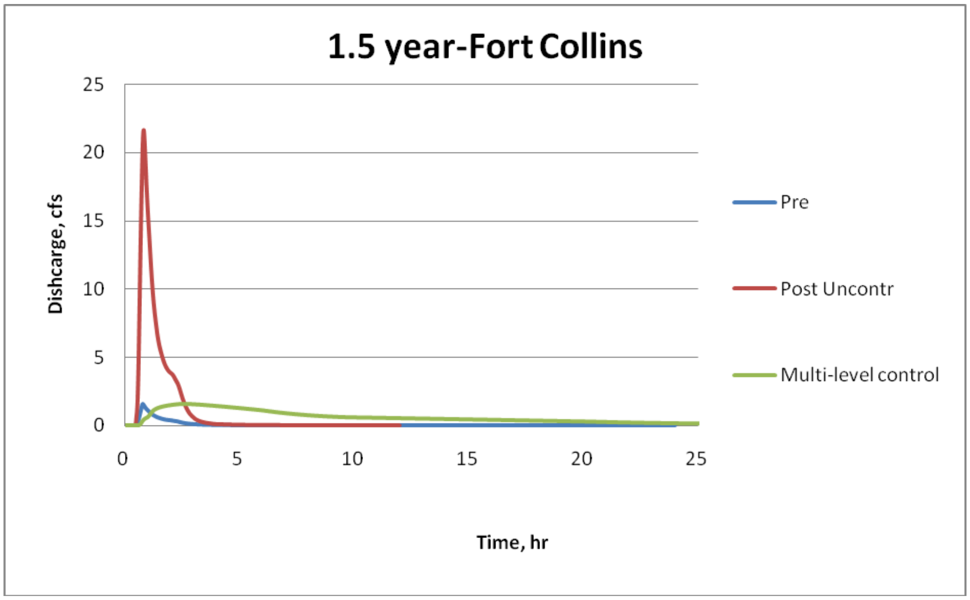

Figure C-1 1.5-year hydrograph, Fort Collins, CO

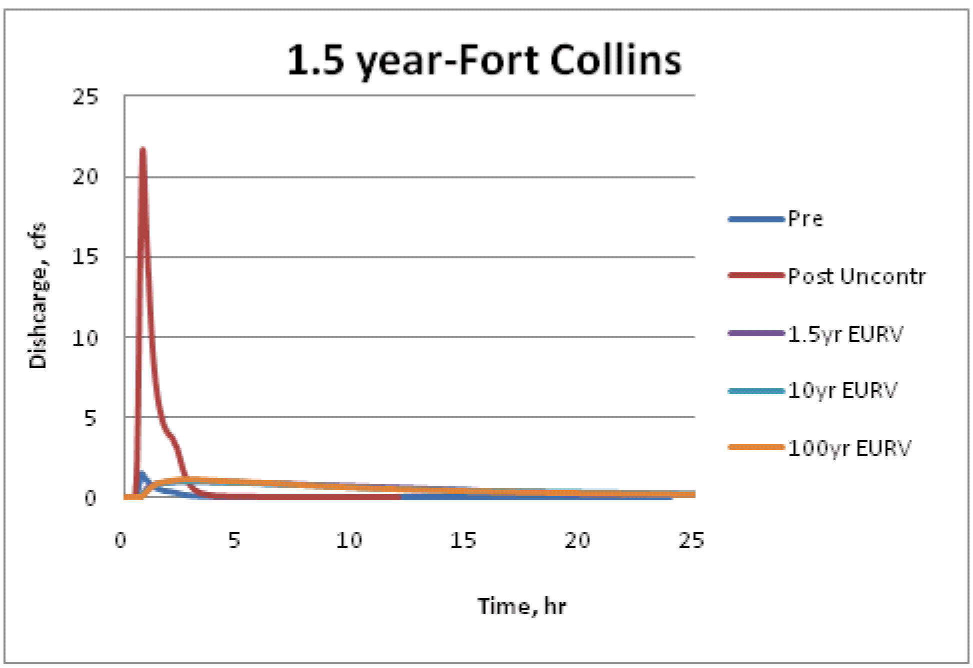

Figure C-2 1.5-year hydrograph, Fort Collins, CO

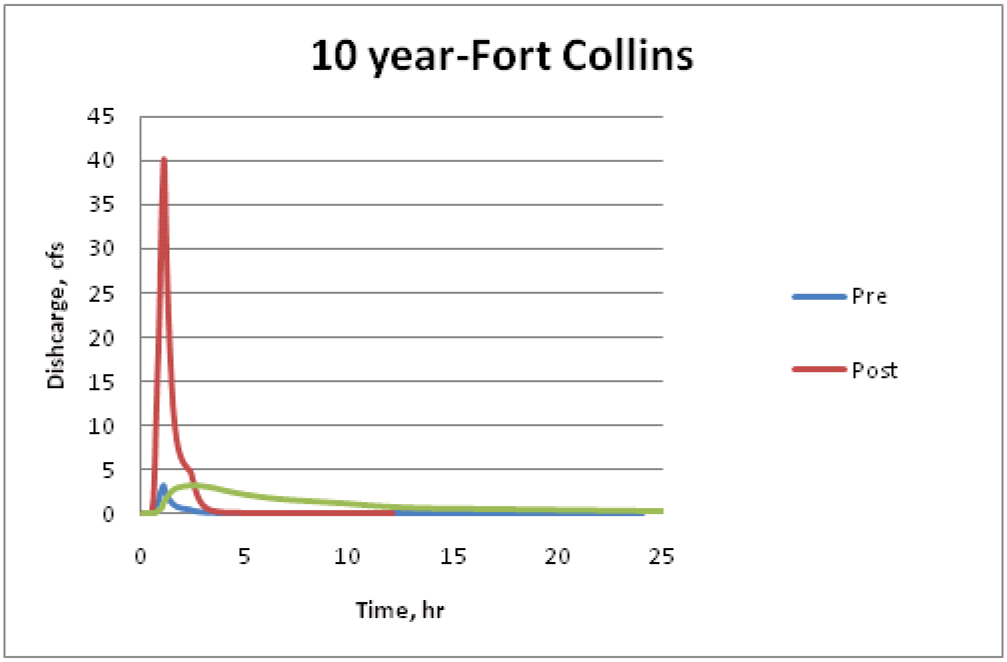

Figure C-3 10-year hydrograph, Fort Collins, CO

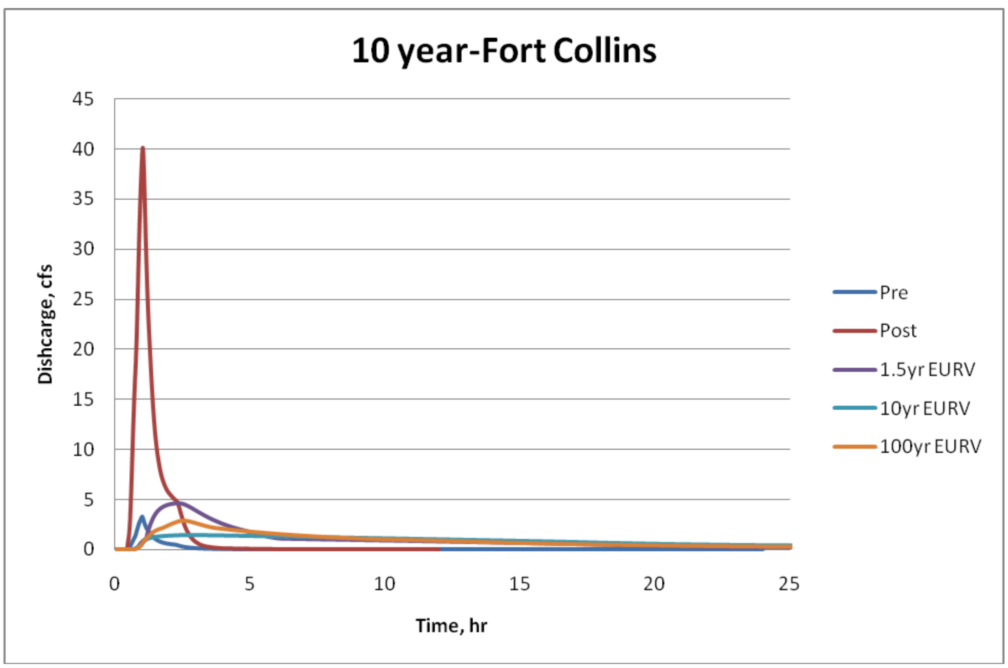

Figure C-4 10-year hydrograph, Fort Collins, CO

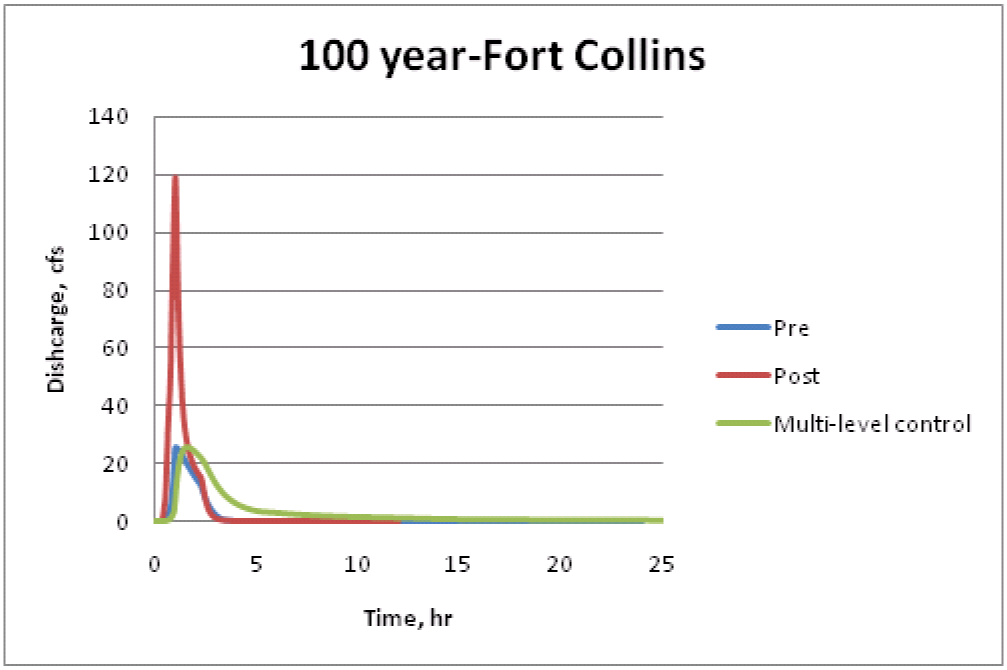

Figure C-5 100-year hydrograph, Fort Collins, CO

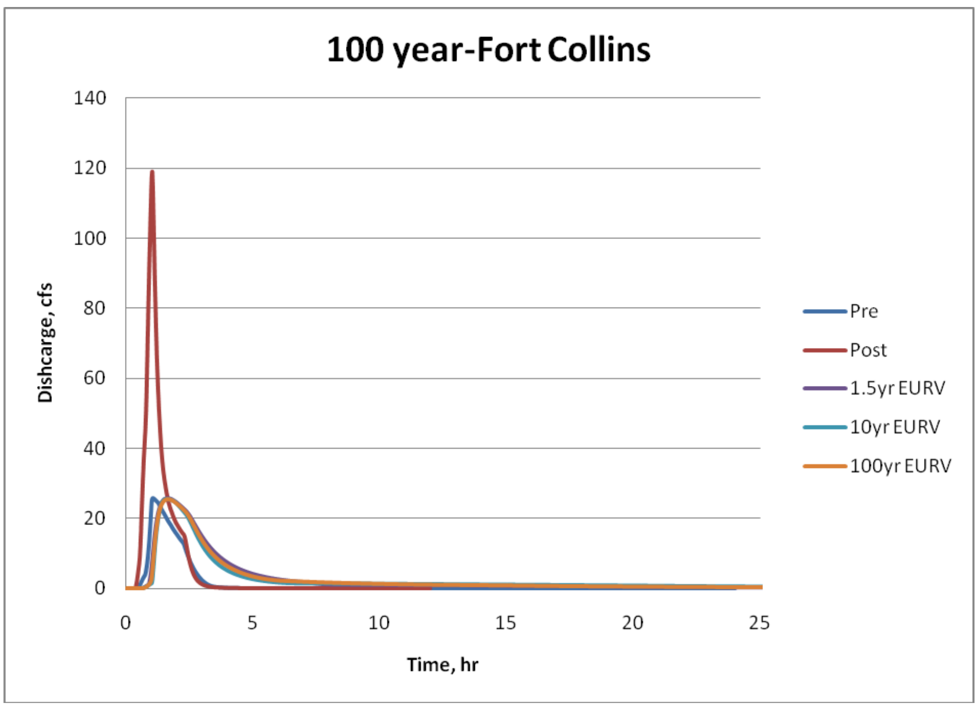

Figure C-6 100-year hydrograph, Fort Collins, CO

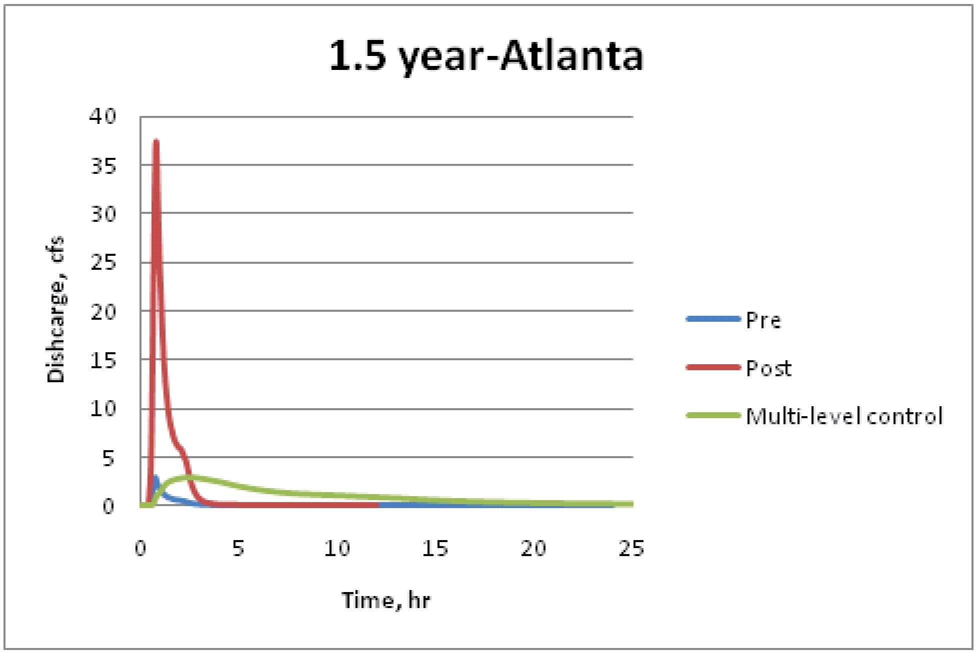

Figure C-7 1.5-year hydrograph, Atlanta, GA

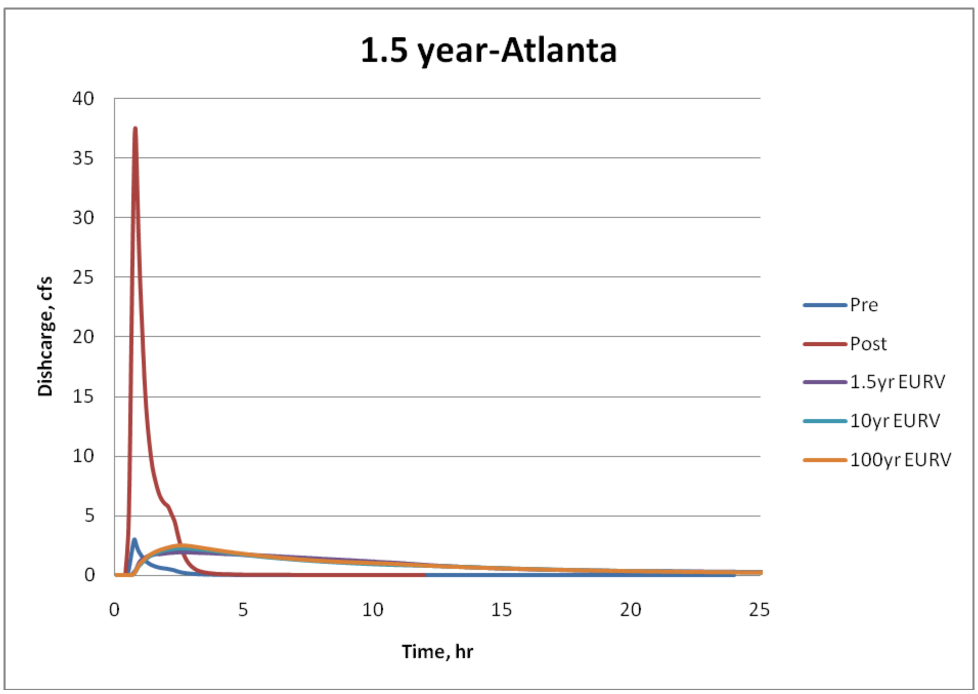

Figure C-8 1.5-year hydrograph, Atlanta, GA

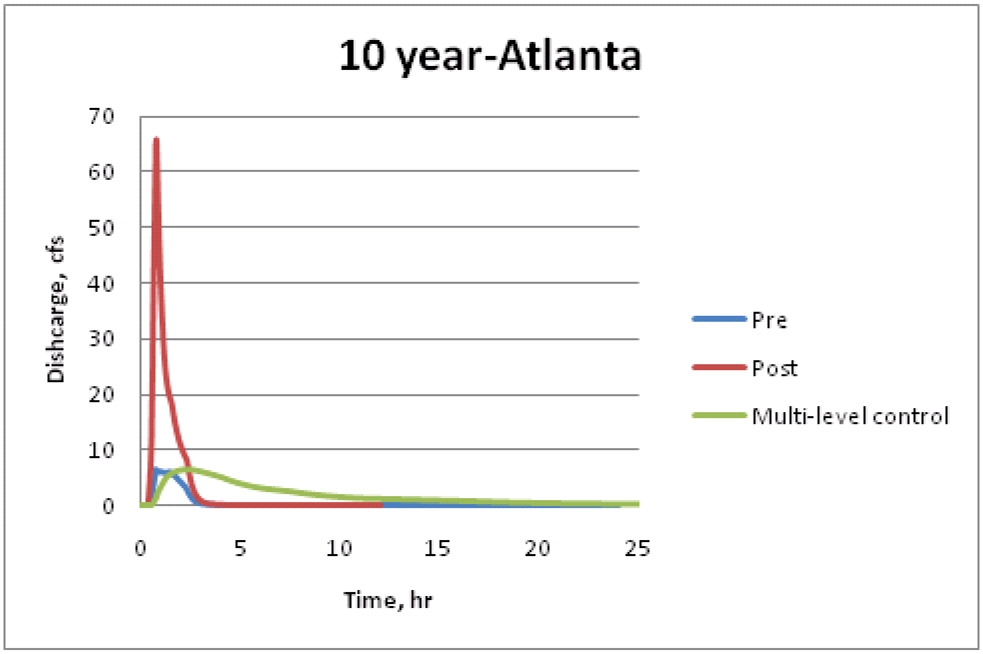

Figure C-9 10-year hydrograph, Atlanta, GA

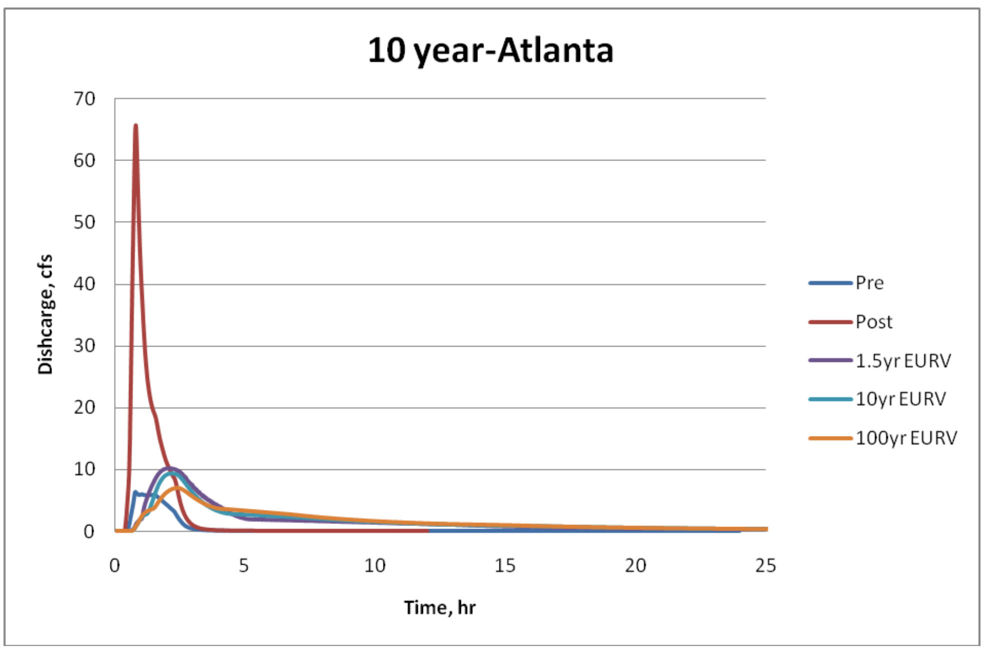

Figure C-10 10-year hydrograph, Atlanta, GA

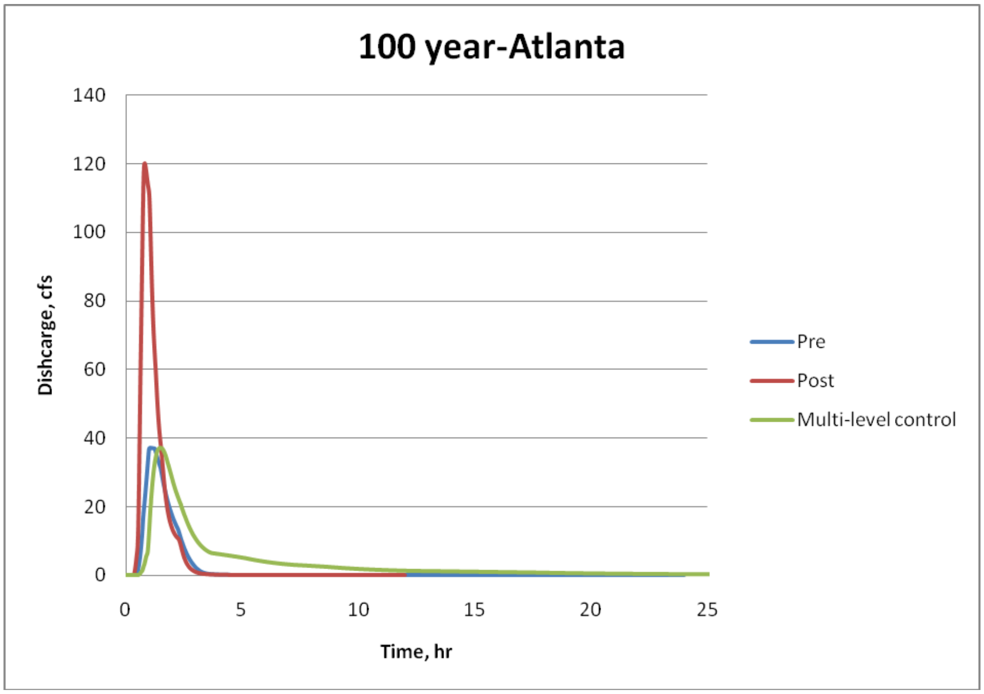

Figure C-11 100-year hydrograph, Atlanta, GA

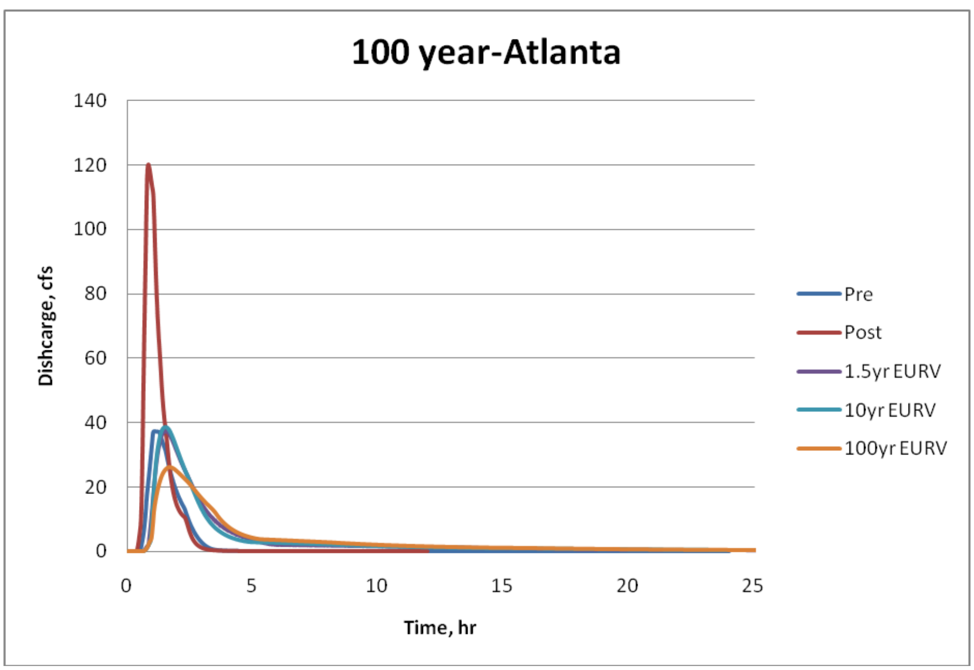

Figure C-12 100-year hydrograph, Atlanta, GA