THESIS

EVALUATION OF SPATIALLY DEPENDENT ON-SITE DETENTION BASIN POLICIES

Submitted by

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ABSTRACT

EVALUATION OF SPATIALLY DEPENDENT ON-SITE DETENTION BASIN POLICIES

Stormwater detention basins are typically used for stormwater control in many communities across the United States. They are commonly constructed downstream of every new development to control post-development runoff, and are called “on-site” detention basins. It has been shown by multiple authors in the literature that the design of on-site detention basins with no consideration of their location (non-spatially dependent policies, or Non-SD) in the watershed can actually increase peak flows above post-development peaks that would occur in the absence of on-site detention basins. This is caused by on-site detention basins delaying the peak release of a particular subwatershed and combining with other peak flows in the watershed (McCuen 1974; McCuen 1979; Emerson et al. 2005). Strategies to combat this problem have been reported, but metrics used to judge their success are limited to the main channel of the watershed or the watershed outlet only, leaving its impact in the remaining other watershed locations unknown. In addition, some strategies have recommended increasing the storage of on-site detention basins, but this approach would increase construction and maintenance costs and reduce the amount of land available to developers.

Validation of increased peak flows throughout the watershed when Non-SD policies are used to design on-site detention basins compared to no on-site detention in the watershed was investigated first. The Non-SD policies used in this study controlled the post-development 10 and 100-year peak flows to flows at or below their respective pre-development peak flows (Non-SD 1), and controlled the post-development 100-year peak flow to flows at or below the 2-year pre-development peak flow (Non-SD 2). Next, spatially dependent policies (SD policies) were created by altering the peak flow release from on-site detention basins that would have occurred under a Non-SD policy based on its location in the watershed. These peak flows were altered using a linear model and a piece-wise linear model. Results from SD policies were compared to those from Non-SD policies. Metrics used to evaluate the effectiveness of the on-site detention basin policies (both SD and Non-SD) were peak flows throughout the watershed and total watershed storage. All policies were tested on a watershed in Fort Collins, Colorado using the Urban Morpho-climatic Instantaneous Unit Hydrograph model.

Results indicate that Non-SD polices effectively reduce peak flows throughout the watershed, and do not increase peak flows compared to a policy that uses no on-site detention. When compared against Non-SD 1, SD policies derived from the linear equation were successful at reducing peak flows at some 2^{nd} and 3^{rd} order channel and pipe intersections in the upper half of the watershed, while increasing peak flows at 2^{nd} order channel and pipe intersections in the lower half of the watershed. The remaining intersections were not effected by this SD policy, and the total watershed storage was shown to increase. SD policies derived from the piece-wise linear model increased peak flows at 2^{nd} order channel and pipe intersections in the lower half of the watershed. The remaining intersections were not affected by this SD policy, and watershed storage was shown to slightly decrease. When compared to Non-SD 2, SD policies had little to no effect on peak flows at any location in the watershed or on the watershed storage.

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DEDICATION

To Whitney, Mom, Dad, Meagan, Vic and Sandy.

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INTRODUCTION

Stormwater runoff from urban watersheds has been identified as a leading detriment to rivers and streams in the United States (United States Environment Protection Agency (EPA) 2004). Urbanization increases a watershed's imperviousness and the hydraulic efficiency of runoff. As a consequence, the volume of stormwater runoff is increased, and its time to peak flow is decreased. Left unmanaged, these consequences of urban runoff can cause adverse effects on receiving waters including stream erosion, flooding, and the degradation or destruction of in-stream aquatic habitats (Roesner et al. 2001).

In an effort to control these negative consequences of urban development, stormwater management policies have been adopted by many communities across the United States. A common policy is to require on-site detention basins in new developments to mitigate the detrimental effects of urbanization. Typical on-site detention basin polices control post-development flows by reducing flows to levels less than or equal to a pre-development peak flow for a specified design storm. On-site detention basins can be designed to control the post-development peak flow of one design storm (e.g. 100-year event), two design storms (e.g. 10 and 100-year events), or more. A different, but rare on-site detention basin policy is "over control." An over control policy reduces post-development flows of a specified design storm to levels equal to or below a relatively small design storm (i.e. control the 100-year post-development peak flow to levels equal to or below the 2-year pre-development peak flow). These policies are designed with no consideration of on-site detention basin location within the watershed, and are hereafter referred to as non-spatially dependent on-site detention basin (Non-SD) policies. It is assumed that the on-site detention basins will better control stormwater flows throughout the overall watershed (Lakatos and Kropp 1982; Goff and Gentry 2006).

However, there has been concern regarding the validity of this assumption. The design of on-site detention basins with no consideration of their locations in the watershed has been shown to increase post-development flows above post-development peaks without on-site detention in the watershed at the watershed outlet (McCuen 1974; Emerson et al. 2005) and in the main watershed channel (McCuen 1979; Sonnenberg 1986). This situation results when on-site detention basins delay their peak release which then combine with other peak flows within the watershed (McCuen 1974; McCuen 1979; Emerson et al. 2005). In addition, it has been shown that the effectiveness of on-site detention to control post-development peak flows decreases as the distance from the on-site detention basin increases (Mein 1980; James et al. 1987; Debo and Reese 1992). This suggests that only flows immediately downstream of an on-site detention basin would be controlled.

In response to this problem, alternative stormwater policies have been proposed, including increasing the size of detention basins (James et al. 1987), capturing stormwater volumes based on pre-development and post-development hydrographs (Wulliman and Urbonas 2007), or developing a watershed according to a specific pattern (Goff and Gentry 2006). Although some of these policies have shown promising results, the metrics used to quantify their success were often limited to peak flows in the main channel or the watershed outlet only. It is unknown if these policies would be successful at other locations within the watershed, or if they actually increase the potential for localized flooding. Other studies recommend increasing the volume of each on-site detention basin, but considering the high cost of land, this solution is not economically feasible. In addition, developing a watershed according to a specific plan or pattern is not practical because land is bought and developed based on economic factors.

This study evaluates policies that consider the locations of on-site detention basins within a watershed in their design. Hereafter these are referred to as spatially dependent on-site detention basin (SD) policies. Metrics used to evaluate the effectiveness of these policies are based on peak flow measurements throughout the watershed. In addition they are intended to have a net zero increase in total watershed storage, or a reduction in total watershed storage when compared to Non-SD policies.

LITERATURE REVIEW

Beginning in the 1970's, many authors have documented the potential ineffectiveness of Non-SD policies. Primarily through modeling, these studies have demonstrated how Non-SD policies can be detrimental to downstream receiving waters.

McCuen (1974) was one of the first to recognize the possible negative effects of an "individual-site" (i.e. on-site) detention policy. He modeled an existing watershed consisting principally of residential developments with no on-site detention. To study the effects of on-site detention, he compared and contrasted three scenarios: the existing watershed, on-site detention installed at the outlet of each subwatershed in the lower half of the watershed, and on-site detention installed at every subwatershed outlet in the watershed. The metric used to compare the three scenarios was runoff at the watershed outlet. The on-site detention basins were designed to reduce the 2-year post-development peak to the 2-year pre-development peak. When on-site detention was installed in the lower half of the watershed, peak flow at the watershed outlet was 4% cubic feet per second per acre (cfs/acre) greater than if no on-site detention was used. Based on this result, McCuen (1974) concluded that on-site detention basins in the lower portions of a watershed can delay their peak release, combine with runoff from the upper portions of the watershed, and result in a greater peak flow at the watershed outlet.

McCuen (1979) demonstrated how an on-site detention basin can increase post-development peak flows in the main channel of a watershed. He modeled a generally undeveloped 5.49 km^2 watershed in Montgomery County, Maryland using TR-20 methods. The watershed was synthetically developed with one on-site detention basin at the outlet designed to control the post-development peak flow of a 10-year design storm to levels equal to or below the pre-development 10-year peak flow. Just downstream of the synthetically developed watershed was a confluence with a larger undeveloped watershed. Results showed that due to the on-site detention basin, the post-development peak arrived at the confluence with the undeveloped watershed 36 minutes later than that associated with pre-development conditions. This delay caused the developed watershed's peak flow to occur closer in time to the undeveloped watershed's peak, consequently increasing the flow downstream of the confluence. This increase in flow was greater than post-development peaks without on-site detention for roughly one mile downstream. Thus, detention basins change the timing of peak flow releases which may not be equal to the pre-development peak release, and depending on the timing characteristics of other areas in the watershed, could increase downstream flows. Similar results were found by Lakatos and Kropp (1982) and Ahmed and Morgan (1995).

After the concept of on-site detention basins potentially increasing post-development peak flows beyond those without on-site detention was realized, various solutions have been proposed. A tool developed by Hawley et al. (1981) was one of the first methods used at the planning level to predict whether a proposed detention basin will have "detrimental downstream effects" at a location immediately downstream of a proposed on-site detention basin. Their method estimates both pre and post-development hydrographs for a watershed with a proposed detention basin. Both hydrograph peaks are calculated using TR-55 methods. Timing characteristics of the hydrographs are calculated with empirical equations based on the subwatershed's physical characteristics (area, slope), hydrological characteristics (curve number, time of concentration, storage), precipitation, and the pre-development peak to post-development peak ratio. If the post-development peak is greater than the pre-development peak, then the proposed detention basin would produce a "detrimental downstream effect" immediately downstream of the on-site detention basin. Although this method can be useful for the channel immediately downstream of the on-site detention basin, it cannot evaluate peak flows further down stream. If a particular on-site detention basin is shown to cause "detrimental downstream effects," the user is left to wonder if that result is valid for the entire channel. In addition, this method is limited to one on-site detention basin and cannot determine the downstream effects of a system of on-site detention basins.

Urbonas and Glidden (1983) studied the effects of several on-site detention polices on an urban watershed in Denver, Colorado using the Stormwater Management Model (SWMM) (Metcalf and Eddy 1971). They randomly placed 28 on-site detention basins in the watershed to test various on-site detention basin policies on their effectiveness of reducing post-development peak flows at several locations along the watershed's main channel. Policies tested included: 1) controlling the post-development peak flow to levels equal to or below the pre-development peak flow of one design storm (the 2, 10 and 100-year storms were each tested separately), 2) controlling the post-development peak flow to levels equal to or below the pre-development peak flow for two design storms (10 and 100-year), and 3) designing on-site detention basins using empirical equations based on contributing area and imperviousness. All policies were tested with the 2 hour 2-year, 10-year and 100-year Denver design storms. In general, the deficiency of this method was that a system of on-site detention basins designed to control a particular design storm was only effective at controlling that design storm (i.e. on-site detention basins designed to control the 100-year storm were not effective at controlling the 2 or 10-year storm), as seen in their graphic (Figure 1). The exception to this rule was for the 2-year storm. That policy worked well at locations immediately downstream of on-site detention basins in the main channel, however, at further distances, the basin's effectiveness diminished rapidly. The inability of on-site detention basins to reduce peak discharges further downstream has been demonstrated by others, including Mein (1980), James et al. (1987), Traver and Chadderton (1992) and Debo and Reese (1992). Debo and Reese (1992) also showed that on-site detention basins are only effective at reducing post-development flows until the point downstream where the tributary area is roughly 10 times larger than the tributary area of the on-site detention basin. Urbonas and Glidden (1983) continued by stating the on-site detention basin policy controlling the 10 and 100-year design storms worked relatively well by controlling post-development peak flows to levels at or slightly larger than pre-development peak flows at locations throughout the watershed's main channel for all three storm events studied. In addition, they concluded that the design of on-site detention basins based on empirical equations performed adequately enough that they recommend additional research on that method. However, their study did not measure peak flow reductions in other areas of the watershed, and thus their models' effectiveness in other parts of the watershed remained unclear.

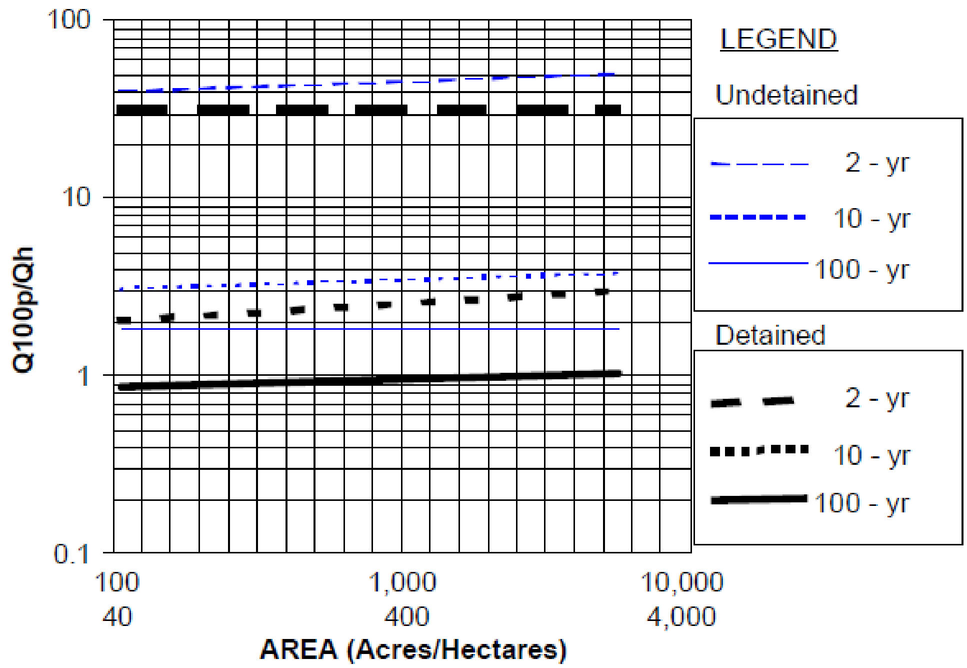

Figure 1: Ratio of post-development peak flow (Q100P) to pre-development peak flow (Qh) versus contributing area along the main channel of the watershed for three design storms (2, 10 and 100-year). Blue lines represent no on-site detention basins in the post-developed watershed, black lines represent on –site detention basins in the watershed designed to reduce the 100-year post-development peak flow to levels less thank or equal to the pre-developed 100-year peak flow (Urbonas and Glidden 1983).

Sonnenberg (1986) modeled a synthetic watershed based on the hydrological characteristics of Greenville, South Carolina using the Modified Rational Method and a dimensionless unit hydrograph. He showed that post-development flows at four locations along the main channel with on-site detention could be greater than post-development flows without on-site detention. Two methods of changing the on-site detention basin design to reduce post-development peak flows to levels equal to or below the pre-development peak flow were tested at four locations in the main channel: 1) design all on-site detention basins identically such that the time it takes to release its peak flow is equal to the watershed time of concentration, and 2) design each on-site detention basin differently such that the time it takes to release its peak flow is equal to the "flow time" from the on-site detention basin to the watershed outlet. Method two was one of the first attempts to design an on-site detention basin based on its location in the watershed. Results of both methods were that peak flows remained greater than pre-development peak flows in the main watershed channel; although they were an improvement over Greenville's then current on-site detention basin policy. Similar to the results of Urbonas and Glidden (1983), Sonnenberg's (1986) metrics to quantify the various on-site detention basin policies were based on peak flows in the main channel of the watershed. It is unclear how these policies affected the remaining portions of the watershed. In addition, method two requires the calculation of a "flow time" from each on-site detention basin to the watershed outlet. Depending on the amount of on-site detention basins in the watershed, this can be a lengthy and tedious task. Furthermore, Sonnenberg (1983) did not define "flow time," and the reader is left to wonder how it is calculated.

James et al. (1987) studied a system of detention basins to develop general size and location guidelines to control post-development peak flows to levels at or below pre-development peak flows at desired locations in the watershed. A synthetic watershed was constructed to resemble a watershed in Brazos County, Texas. Five scenarios were tested. First, detention basins were placed only on all 1st order streams; second, detention basins were placed only on 2nd order streams, and so on through 5th order streams. Each detention basin was designed to control the 2, 10 and 100-year design storms to levels equal to or below their respective pre-development peak flows, while a 24 hour, 10-year design storm was used as the precipitation in the model. Storm hydrographs were computed using a two-parameter gamma function unit hydrograph. They demonstrated that the ability of detention basins to reduce peak discharges is reduced as the distance downstream of the detention basin increases. To fix this problem, they developed a non-dimensional graphic that determines the required detention basin storage increase for post-development flows to be under a pre-development peak flow at a desired location downstream. This was accomplished by systematically increasing each detention basin's storage until the post-development peak flow was equal to the pre-development peak flow at the desired downstream location.

This process was repeated for each of the five scenarios tested. Their result showed a linear relationship between where the on-site detention basins were located and the required storage increase. For example, detention basins on all 1st ordered streams required the largest storage increase, while detention basins on all 5th ordered streams required the least. Although this method will obtain a desired post-development peak flow at a specific location in the watershed, increasing storage is not always a viable option. In addition, considering the cost of land, increasing the storage of detention basins is not economically feasible.

Shea (1996) proposed five alternative on-site detention basin policies that would "maintain stormwater peaks throughout the drainage basin at predevelopment levels." The first is a single on-site detention basin optimization "plan-as-you-go" method that optimally designs a proposed detention basin for each new development such that any downstream post-development flows are kept at or below a pre-development peak flow. This process is repeated for each new development that is constructed. A similar policy was developed by Ravazzani et al. (2014) for a system of detention basins north of Lozza, Italy. A major limitation to this method is that the size requirements for a particular on-site detention basin can vary depending on the order of development and the sequence and location of previous developments (Shea 1996). As a consequence, developers cannot anticipate detention costs for a particular subwatershed until it is time for the on-site detention basin to be constructed. The second policy is a basin-wide optimization method based on the work of Mays and Bedient (1982). That method uses the best estimation of a watershed's post-development hydrological characteristics to size on-site detention basins at candidate locations. Any errors in predicting the post-development hydrological conditions for the watershed will cause this method to fail. In addition, the method will not be effective until the entire watershed is developed and all on-site detention basins are constructed (Shea 1996). Another consideration is that optimization techniques are often difficult to use. The third policy uses a peak partitioning method. Pre-development hydrographs from each subwatershed are generated and routed to the watershed outlet, where they are summed to generate the watershed's outlet hydrograph.

The contribution from each subwatershed's hydrograph to the watershed's peak flow is noted, which in turn becomes that subwatershed's on-site detention basin's maximum allowable release rate in the post-development condition. Lakatos and Kropp (1982) recommended a similar procedure. However, the method has the potential to assign high reductions in peak flow from particular subwatersheds, which developers may find unfair (Shea 1996). Also, this method does not address the situation of on-site detention basins in parallel. The fourth policy is similar to the third where after one or two developments are constructed with their associated on-site detention basins, peak flows from the remaining developments are re-evaluated for a different post-development peak flow release. Thus, this method has the same limitations as method three. The fifth policy calls for a uniform reduction in post-development peak release from every on-site detention basin in the watershed. This policy was found to be an effective and easy method to reduce post-development flows to levels equal to or below a pre-development peak flow. But again, it increases the size of every on-site detention basin in the watershed, which can be economically infeasible.

Emerson et al. (2005) demonstrated that an existing system of approximately 100 on-site detention basins designed to control the 2 and 100-year peak flows achieved little to no reduction in peak flow at the outlet of a watershed near Philadelphia, Pennsylvania. This was accomplished by modeling the existing watershed with and without on-site detention basins using the United States Army Corps of Engineers Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS) (United States Army Corps of Engineers Hydrologic Engineering Center 2000). Results showed that on average, on-site detention basins reduced outlet peak flows by only 0.3% compared to the watershed with no on-site detention basins. Some simulations showed peak flows at the watershed outlet were actually greater with on-site detention basins in place compared to peak flows with no on-site detention basins. Similar to McCuen (1979), they concluded that the ineffectiveness of on-site detention basins was due to the detained peak release combining with peak flows from other parts of the watershed. Two different on-site detention basin policies were tested to mitigate this problem. The first reduced all existing low-flow orifices in the detention basins to 10 centimeters (cm). The reasoning behind this policy was to hold runoff in on-site detention basins for longer periods of time so that their peak flows would not combine with peak flows not detained by on-site detention basins. Results from implementing this first policy showed that, on average, peak flows at the outlet of the watershed were reduced by 4%. The second policy changed the design of an on-site detention basin based on its location in the watershed. In the upper half of the watershed on-site detention basins were drained by low-flow 10 cm orifices, while the remaining on-site detention basins in the lower half of the watershed were removed. The idea behind this policy was to delay the peak releases in the upper half of the watershed with on-site detention so their peak flows will not combine with peak flows in the lower half of the watershed. This policy reduced the peak flow at the watershed outlet by only 0.8% on average. The limitations of this study were similar to designs presented by McCuen (1974) in that peak flows were only measured at the outlet of the watershed. It is possible these policies could reduce peak flows at other watershed locations or reduce total watershed storage. But because peak flows were only measured at the watershed outlet, the benefits of this policy in other parts of the watershed are unclear.

Many of the previous studies examined changing the design of on-site detention basins to control post-development peak flows. In contrast, Goff and Gentry (2006) investigated which watershed and development characteristics contribute to the ineffectiveness of on-site detention basin policies at several locations along the watershed's main channel. They developed a synthetic watershed in HEC-HMS, where on-site detention basins were installed on all 1st ordered streams to control the post-development 10-year storm runoff to levels at or below the 10-year pre-development peak flow. Watershed and development characteristics were then systematically changed and resultant peak flows along the main channel were observed and compared to pre-development peak flows. Watershed characteristics that were tested included: shape (classic dendritic and elongated) and slope (5% and 15%). The various development characteristics that were tested included: development size (0.08 km^2 and 0.32 km^2), development intensity (0.004 km^2 lots, 0.001 km^2 lots, and commercial development), development stage (development starts in the downstream portion of the watershed and progresses upstream), and development sequence (same as development stage, but in reverse). In general, their results showed that elongated watersheds with on-site detention were less effective than dendritic watersheds, on-site detention is most effective when development occurs in the upstream portion of the watershed, and least effective when development occurs in the downstream portion of the watershed. Ahmed and Morgan (1995) and Su et al. (2010) found similar results. Results of the Goff and Gentry (2006) study suggest that the development of a watershed should adhere to a specific plan or pattern. However, a watershed's shape cannot be controlled, and asking developers to develop the watershed in a particular pattern is not practical.

Wulliman and Urbonas (2007) introduced the full spectrum detention policy. On-site detention basins designed by the full spectrum detention policy captures the water quality capture volume, the excess runoff volume, and the 100-year detention volume. The water quality capture volume is the quantity of runoff equivalent to the runoff generated from an 80th percentile storm in the Denver area (Urban Drainage and Flood Control District (UDFCD)). The excess runoff volume is defined as the difference between the post-development and pre-development runoff volumes. The design of full spectrum detention basins does not consider their location in the watershed. Experiments using three Natural Resources Conservation Service (NRCS) hydrologic soil groups (A, B, C/D) showed that within each soil group the excess runoff volume per unit impervious area was similar for a variety of storms. Therefore, an average value of unit excess runoff was used for each soil group, regardless of the storm, to calculate the excess runoff volume for each on-site detention basin. They chose to detain the excess runoff volume because after that amount of runoff is detained, the subsequent runoff will resemble that of pre-development flows. Full spectrum detention was tested on a synthetic 20 km^2 watershed comprised of 50 identical subwatersheds for which hydrographs were calculated using the Colorado Unit Hydrograph Procedure (UDFCD 2001). Two small storms and six design storms were used to test full spectrum detention. Results showed this policy is successful at reproducing pre-development peak flows for a variety of storms immediately downstream of an on-site detention basin, and at the outlet of the watershed. Similar to the results of McCuen (1974) and Emerson (2005), peak flows were measured at the outlet of the watershed to quantify the performance of full spectrum detention. Although the authors did provide peak flow information directly downstream of one detention basin, performance of this policy in the remaining portions of the watershed is unclear, and the likelihood of watershed having 50 identical subwatersheds is doubtful. Therefore, the performance of full spectrum detention on a watershed with heterogeneous subwatersheds is unknown.

Del Giudice et al. (2014) developed a planning level tool to evaluate the effectiveness of a system of detention basins at a particular downstream point of interest. The tool is based on the assumption that watershed response, river routing, and reservoir routing can be approximated by linear models. The efficiency of a system of detention basins was estimated by convolution. Watershed response was simulated by the instantaneous unit hydrograph (IUH), stream routing was simulated by a linear channel, and reservoir routing was simulated by a linear reservoir. This method was tested on a watershed in Scafati, Italy, where 15 combinations of detention basins at nine potential locations were modeled. They concluded that peak flows were lowered most when detention basins were placed in parallel and close to the point of interest. Although this method is an improvement over the planning level tool presented by Hawley et al. (1981), it still has limitations. Foremost, if a user wants to analyze the effects of a system of detention basins at several locations, many time consuming and tedious calculations would be needed, and the suggested placement of detention basins at specific locations in the watershed may not be practical.

In summary, the propensity of on-site detention basins to increase peak flows over that which would have occurred if no detention basins were installed has been well documented. The mechanism for this unexpected behavior is that on-site detention basins delay their peak release, which may combine with other peak flows within the watershed (McCuen 1974; McCuen 1979; Emerson et al. 2005). To solve this problem, many solutions have been proposed in the literature. Some proposed changing the design of on-site detention basins with no consideration of their location in the watershed (Urbonas and Glidden 1983; Shea 1996; Wulliman and Urbonas 2007). Alternately, some strategically changed on-site detention basin design based on their location in the watershed (Sonnenberg 1986; James et al. 1987; Shea 1996; Emerson et al. 2005). Others developed planning level tools (Hawley et al. 1981; Del Giudice et al. 2014), while some suggested that a watershed should be developed by a specific plan or pattern (Goff and Gentry 2006). However, there are deficiencies as well. Many of these studies used only measured peak flows along the main channel of the watershed or the watershed outlet as the primary metric to evaluate new on-site detention basin policies. By reducing peak flows in the main watershed channel or the watershed outlet, it is unclear whether the potential for localized flooding in the remaining watershed locations would increase or decrease after implementation of the new policies. Some polices suggest increasing the size of on-site detention basins compared to on-site detention basins that would have been constructed under existing Non-SD policies. But the high cost of land makes that solution economically unfeasible. The suggestion by some that watersheds should be developed according to a certain plan or pattern is not practical because it is unfair to developers. Planning level tools would require many tedious and time consuming calculations in order to quantify the impacts of proposed on-site detention basins at multiple watershed locations. Finally, computed optimum placement of on-site detention basins likely may not be practical because they are difficult to use and they depend on an accurate prediction of the built-out watershed. In summary, an on-site detention basin policy that has been evaluated by peak flows throughout the watershed, does not increase watershed storage, and is simple to use is still lacking.

STUDY OBJECTIVES

The first objective of this study was to test the results of previous studies that found Non-SD policies increase peak flows in a watershed compared to a watershed with no on-site detention. Three peak flow metrics were used: 1) the relative change in peak flow at the watershed outlet, 2) the relative change in average peak flow at the intersection of all 2nd through 5th order channel and pipe intersections, and 3) the relative change in peak flow at the intersection of all 2nd through 5th order channel and pipe intersections. The second objective was to develop SD policies that are simple to use, reduce or maintain watershed storage, and reduce peak flows throughout the watershed relative to Non-SD policies. Four metrics were used to quantify the difference between SD policies and Non-SD policies: 1) relative change in total watershed storage, 2) the relative change in peak flow at the watershed outlet, 3) the relative change in average peak flow at the intersection of all 2nd through 5th order channel and pipe intersections, and 4) the relative change in peak flow at the intersection of all 2nd through 5th order channel and pipe intersections. It is our hypothesis that an SD policy will produce lower peak flows throughout a watershed and require less watershed storage compared to a Non-SD policy.

APPROACH

For a baseline, Non-SD policies were first compared to a policy that uses no on-site site detention to control post-development runoff. These comparisons were done to evaluate the effectiveness of Non-SD policies at reducing peak flows throughout the watershed. Several studies cited in the literature review indicated that Non-SD policies increased peak flows throughout the watershed, and this comparison tested that conclusion. Results from SD policies developed in this paper were then compared to Non-SD policy results to evaluate any benefits these policies have over Non-SD policies.

Two Non-SD policies were used in this study. The first controls post-development 10 and 100-year design storm peak flows and releases at or below their respective pre-development peak flows. This policy is hereafter referred to as Non-SD 1. It was chosen because many communities in Colorado, such as Denver (City and County of Denver Storm Drainage Design and Technical Criteria, 2006), Aurora (City of Aurora Storm Drainage Design and Technical Criteria, 2010), Boulder (City of Boulder Design and Construction Standards, 2000) and Loveland (City of Loveland Storm Drainage Criteria (Addendum to the Urban Storm Drainage Criteria Manuals Volumes 1, 2 and 3), 2002) use this policy. The second Non-SD policy controls the post-development 100-year design storm peak flow and release at or below the pre-development 2-year peak flow. This policy, hereafter referred to as Non-SD 2, is used by the City of Fort Collins (City of Fort Collins Stormwater Criteria Manual, 2011).

The ineffectiveness of Non-SD policies can be attributed to on-site detention basins delaying peak releases that then combine with other peak flows within the watershed (McCuen 1974; McCuen 1979; Emerson et al. 2005). Therefore, if an SD policy can release peak flows from on-site detention basins in such a way as to avoid combining with other watershed peak flows, then peak flows would be reduced throughout a watershed compared to those of a Non-SD policy. SD policies were created by altering the peak flow release from a particular on-site detention basin from that which would have occurred under a Non-SD policy. The property that distinguishes SD from Non-SD policies is that the released peak flows from an on-site detention basin for an SD policy is based on its location in the watershed. The first method to alter on-site detention basin peak flows was a linear model (Equation 1):

$$\frac{Q_{P,SDi}}{Q_{P,NonSDi}} = \varepsilon\left(\frac{\ell_i}{L}\right) + \omega \qquad \textbf{Equation 1}$$

where $Q_{P,SDi}$ is the peak release of on-site detention basin i under an SD policy in liters per second (l/s), $Q_{P,NonSDi}$ is the peak release of the same on-site detention basin under a Non-SD policy (l/s), ℓ_i is the flow distance from on-site detention basin i to the outlet of the watershed in kilometers (km), L is the flow distance from the most remote location in the watershed to the watershed outlet (km), and ε and ω are the slope and the intercept of the linear model, respectively. An SD policy described by Equation 1 is graphically displayed in Figure 2.

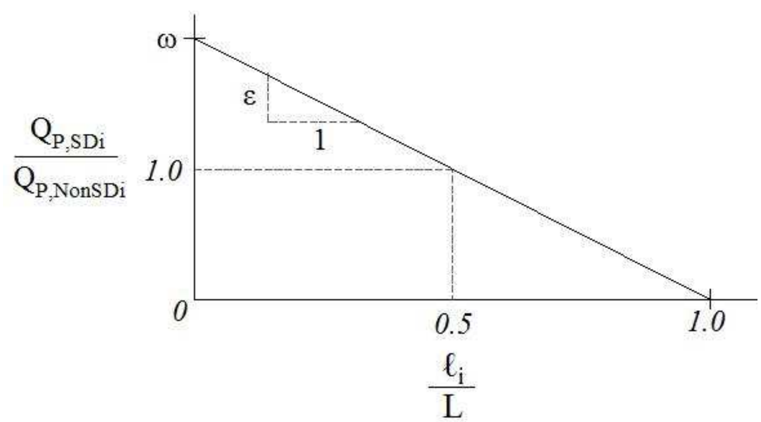

Figure 2: SD policy defined by Equation 1.

Released peak flows from on-site detention basins in the upper half of the watershed (ℓ_i /L > 0.5) decrease as a function of their flow distance to the watershed outlet. Released peak flows from on-site detention basins in the lower half of the watershed (ℓ_i /L ≤ 0.5) increase as a function of their flow distance to the watershed outlet. A consequence of decreasing the released peak flow is that the storage volume and timing of the released peak both increase. The opposite occurs when released peak flows are increased. Equation 1 was designed so that on-site detention basins in the upper half of the watershed would have an increase in storage volume, while the other half would have a reduction. Thus, under SD policies derived from Equation 1, watershed storage should be approximately equal to the watershed storage under a Non-SD policy. SD policy scenario permutations were based on varying ε and ω.

The second method to alter the released peak flow from on-site detention basins is a piece-wise linear model, described by Equation 2.

$$\frac{Q_{P,SDi}}{Q_{P,NonSDi}} = \begin{cases} \varepsilon\left(\frac{\ell_i}{L}\right) + \omega & 0 \leq \frac{\ell_i}{L} \leq 0.5 \\ 1.0 & 0.5 < \frac{\ell_i}{L} \leq 1.0 \end{cases} \qquad \textbf{Equation 2}$$

For the lower half of the watershed ($\ell_i/L \leq 0.5$), $Q_{P,SDi}$ / $Q_{P,NonSDi}$ varies by the linear model in Equation 1. In the upper half of the watershed ($\ell_i/L > 0.5$), $Q_{P,SDi}$ / $Q_{P,NonSDi}$ is held constant and equal to 1.0 ($Q_{P,SDi}$ = $Q_{P,NonSDi}$). An SD policy defined by Equation 2 is graphically displayed in Figure 3.

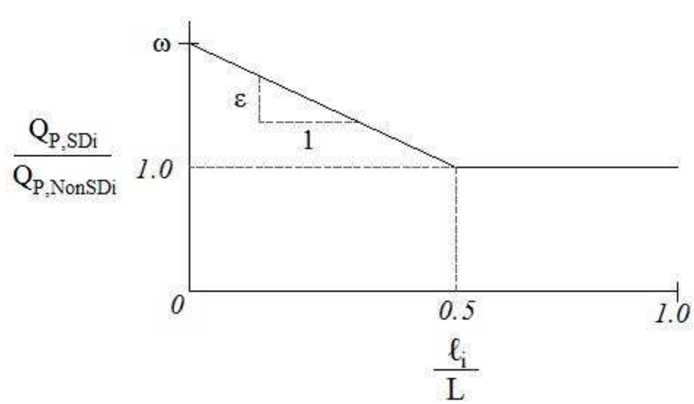

Figure 3: SD policy defined by Equation 2.

Released peak flows for on-site detention basins in the upper half of the watershed (ℓ_i /L > 0.5) are unchanged under SD policies derived from Equation 2, whereas released peak flows for on-site detention basins in the lower half of the watershed (ℓ_i /L ≤ 0.5) increase as a function of their flow distance to the watershed outlet. By increasing the released peak flows from on-site detention basins only in the lower half of the watershed, watershed storage should be less than watershed storage under a Non-SD policy. Permutations to evaluate the SD policy described by Equation 2 were also based on varying ε and ω.

Eight SD policies were created by systematically varying ε and ω in Equation 1 and Equation 2. Organization of the SD policies is displayed in Figure 4, and Table 1 defines values of ε and ω used in each SD policy.

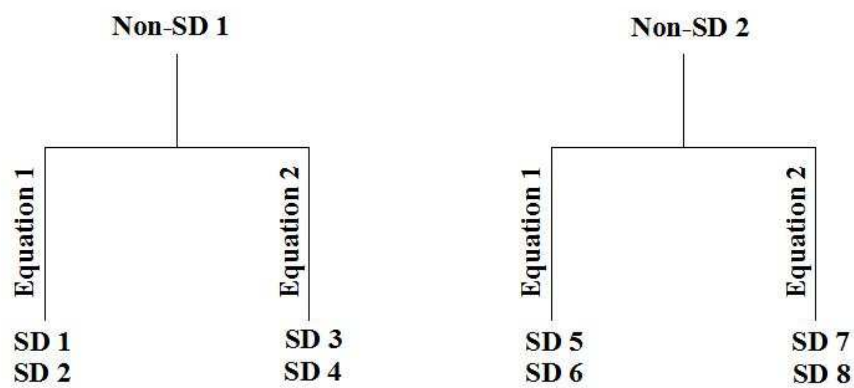

Figure 4: SD policy organization.

Table 1: Values of ε and ω used in SD policies.

SD Policy	ε	ω
SD 1	-0.6	1.3
SD 2	-2.0	2.0
SD 3	-0.6	1.3
SD 4	-2.0	2.0
SD 5	-0.6	1.3
SD 6	-2.0	2.0
SD 7	-0.6	1.3
SD 8	-2.0	2.0

As an example, consider SD 2. If $Q_{p,NonSDi}$ of Non-SD 1 is 100 l/s and has a ℓ_i/L ratio of 0.8, $Q_{p,SDi}$ for on-site detention basin i would be 40 l/s. Results from applying the values in Table 1 to Equation 1 and Equation 2 are given in Appendix I.

SD policies derived from Equation 1 and Equation 2 can easily be applied because they only require the distance form the on-site detention basin to the watershed outlet, and the distance from the most remote location in the watershed to the watershed outlet. These parameters can be easily obtained from aerial maps, a GIS, or a hydrologic model.

Three metrics were used to quantify the difference between a Non-SD policy and a policy that uses no on-site detention basins: 1) the relative change in peak flow at the watershed outlet, 2) the relative change in average peak flows at the intersection of all 2nd through 5th order channels and pipes separately, and 3) the relative change in peak flow at the intersection of all 2nd through 5th order channels and pipes. The relative change in peak flow at the watershed outlet was chosen because it can be used to evaluate the flooding and erosion potential for downstream communities. Relative change in average peak flow at all 2nd through 5th order channels and pipes evaluates how peak flows change as the channel and pipe intersection order increases on an average basis (i.e. the effectiveness of on-site detention basins to reduce peak flows as the downstream distance from the on-site detention basin increases). The relative change in peak flow at the intersection of all 2nd through 5th order channels and pipes was chosen to measure the relative change in peak flows throughout the watershed. Four metrics were used to quantify the differences between Non-SD policies and SD policies. They are the same three metrics listed above, plus the relative change in watershed storage, which can be used as an indirect method to estimate construction costs. Watershed storage was not used as a metric to compare Non-SD policies to a policy that uses no on-site detention because the latter has zero watershed storage.

STUDY SITE

The study site is the Mail Creek watershed (Figure 5) in Fort Collins, Colorado. Located in the rain shadow of the Rocky Mountains, Fort Collins receives an average annual rainfall of 381 mm (15 inches) (Western Regional Climate Center), and experiences a semi-arid climate. The Mail Creek watershed is approximately 1,520 meters (m) above sea level, has a surface area of 7.1 km^2, an average imperviousness of 50.6%, and an average slope of 2.3%. The watershed consists of mostly residential land use, with a commercial area in its eastern portion. Runoff drains from west to east, eventually flowing into Mail Creek. Mail Creek is a tributary to Fossil Creek, which is a tributary to the Cache la Poudre River.

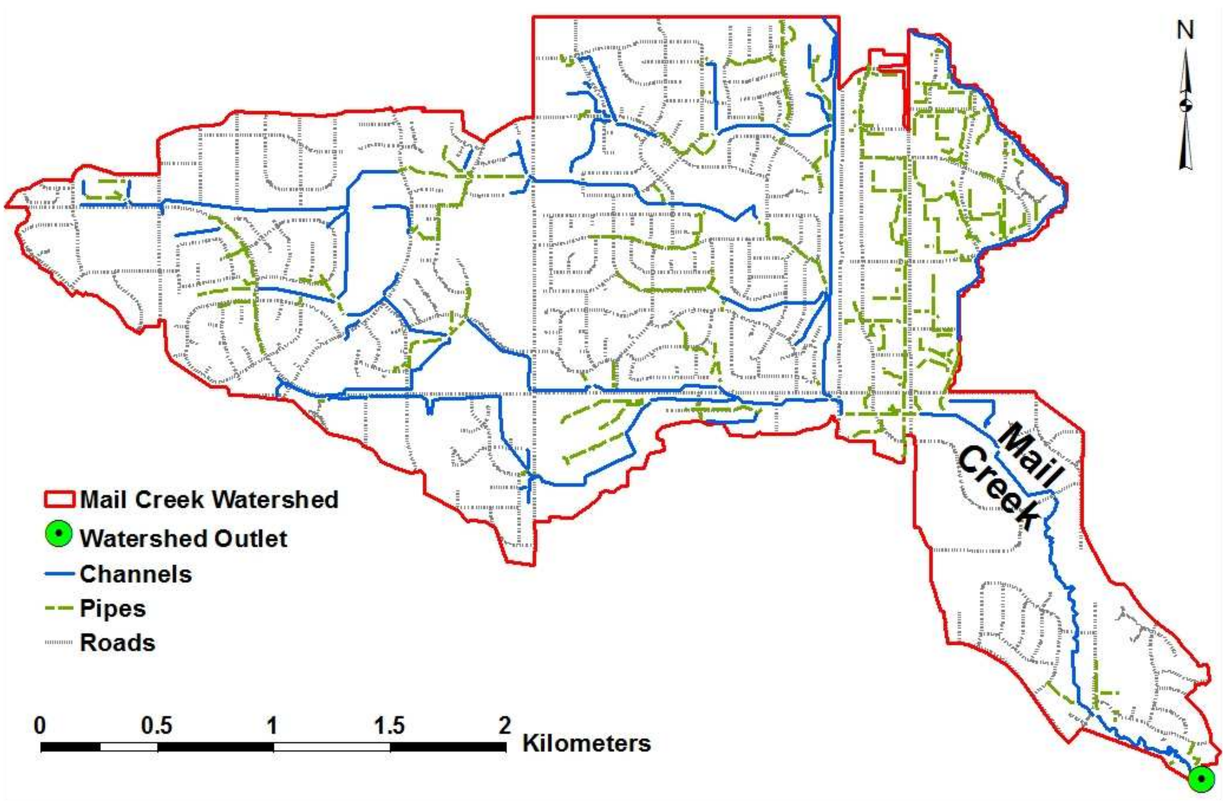

Figure 5: Mail Creek watershed.

MODEL SELECTION AND IMPLEMENTATION OF ON-SITE DETENTION BASIN FUNCTIONALITY

The Mail Creek watershed was modeled using the Urban Morpho-climactic Instantaneous Unit Hydrograph model (U-McIUH) developed by Gironás et al. (2009). The U-McIUH model was chosen for this study because flow information can be defined at a high spatial resolution. This is important because peak flows can be obtained at virtually any location in the watershed. The U-McIUH discretizes a watershed into cells of a particular resolution, which are categorized as hillslopes, channels, pipes or streets. Flow paths are derived from the watershed digital elevation model (DEM) and are used to calculate flow times from every cell in the watershed to the watershed outlet based on the kinematic wave approximation. Using a probability density function of travel times, an outlet Instantaneous Unit Hydrograph (IUH) is constructed. The IUH is transformed into a unit hydrograph (UH), which is used to construct the outlet hydrograph by convolution.

In the original version of the U-McIUH, hillslopes, channels, pipes and streets were modeled, but detention basins were not defined. For this study, the capability to incorporate a detention basin was added to the model. The approach chosen was to disaggregate the watershed into two subwatersheds, one upstream (S_B) and one downstream (S_A) of the detention basin (DB), as displayed in Figure 6a. This is the same method used to incorporate detention basins in HEC-HMS models (HEC-HMS Technical Reference Manual 2000). S_B was isolated from S_A (Figure 6b), and flow from S_B was routed through the detention basin, and subsequently through each downstream channel and pipe cell to the outlet of the watershed, or to the next detention basin. Flow was not routed through street cells because it was assumed that on-site detention basins discharge their flow into channels or pipes only. The hydrograph at the outlet of the watershed (O) is a sum of the hydrograph routed through the downstream channel and pipe cells and the hydrograph from S_A. Subwatershed hydrographs were generated using the original U-McIUH model, while the functionality of routing flow through a detention basin and downstream channels and pipes was added to the model.

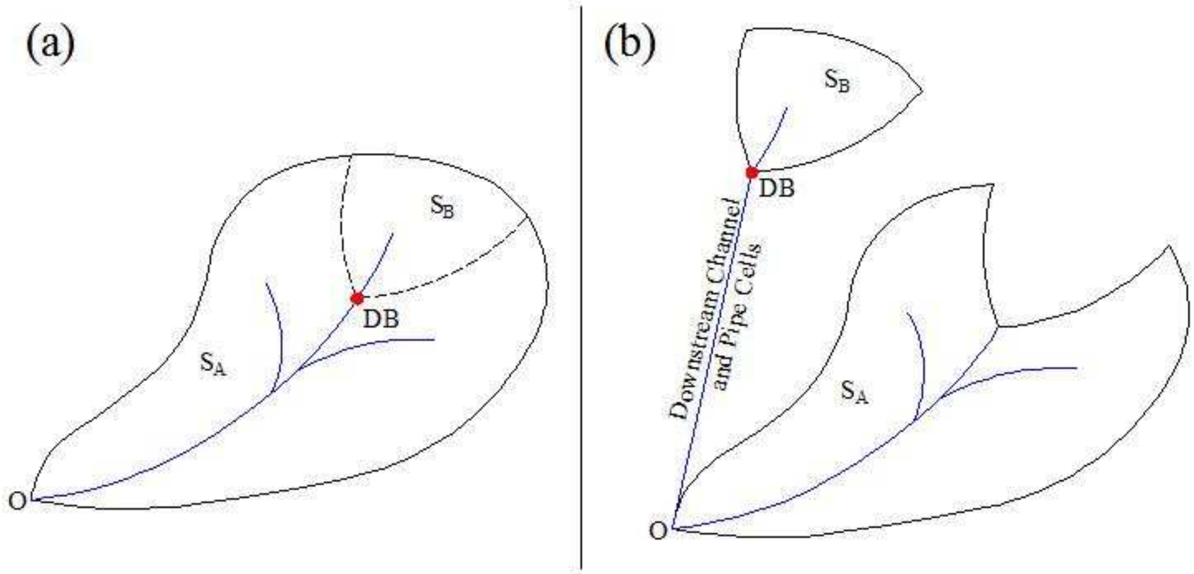

Figure 6: Illustration of watershed disaggregation.

Level pool routing (Equation 3), as described by Chow et al. (1988), was chosen to route hydrographs through on-site detention basins:

$$S_{j+1} - S_j = \frac{I_j + I_{j+1}}{2}\Delta t_{LP} - \frac{Q_j + Q_{j+1}}{2}\Delta t_{LP} \qquad \textbf{Equation 3}$$

where *S* is storage (l), *j* is the time interval, *I* is inflow (l/s), Δt_{LP} is the level pool routing time step, and *Q* is discharge from the on-site detention basin (l/s). Δt_{LP} was set to five minutes.

Two methods of routing the on-site detention basin's discharge hydrograph downstream through channel and pipe cells were tested: kinematic routing and simple routing. Kinematic routing was tested because the U-McIUH model uses the kinematic wave approximation when calculating travel times to the watershed outlet. To be consistent, the channel and pipe routing was based on kinematic theory as well.

The discharge hydrograph from an on-site detention basin was kinematically routed through each downstream channel or pipe cell as described by Chow et al. (1988) using Equation 4 with no lateral inflow:

$$Q_{i+1}^{j+1} = \frac{\frac{\Delta t}{\Delta x} Q_i^{j+1} + \alpha_i \beta Q_{i+1}^{j} \left(\frac{Q_{i+1}^{j} + Q_i^{j+1}}{2}\right)^{\beta-1}}{\frac{\Delta t}{\Delta x} + \alpha_i \beta \left(\frac{Q_{i+1}^{j} + Q_i^{j+1}}{2}\right)^{\beta-1}} \qquad \textbf{Equation 4}$$

where Q is volumetric flow rate (l/s), j is the time index, i is the distance index, Δt is the kinematic routing time step, and Δx is the distance over which the routing is performed in meters (m). α and β are kinematic wave parameters defined by Chow et al. (1988) as:

$$\alpha_i = \left(\frac{n_i P_i^{2/3}}{\sqrt{S_i}}\right)^{0.6} \qquad \textbf{Equation 5}$$

$$\beta = 0.6 \qquad \textbf{Equation 6}$$

where n_i is Manning's roughness of channel or pipe in cell i, P_i is the wetted perimeter of channel or pipe cell i (m), and S_i is the slope of channel or pipe cell i (m/m). Δt is not the same time step used in the original U-McIUH model, and was uniquely defined when each on-site detention basin's discharge hydrograph was routed downstream so that it satisfied the Courant condition, defined by Chow et al. (1988) as:

$$\Delta t \leq \frac{\Delta x_i}{c_{k,i}} \qquad \textbf{Equation 7}$$

where $c_{k,i}$ is the kinematic wave celerity in channel or pipe cell i in meters per second (m/s). For channel cells, $c_{k,i}$ was calculated using Equation 8 as described by Chow et al. (1988):

$$c_{k,i} = \left(\frac{\sqrt{S_i}}{n_i}\right)\left(\frac{5}{3}\right)y_i^{2/3} \qquad \textbf{Equation 8}$$

where y_i is the depth of flow in channel cell i (m) calculated by Manning's equation (Equation 9):

$$y_i = \left(\frac{n_i Q_i^j}{B_i\sqrt{S_{O,i}}}\right)^{3/5} \qquad \textbf{Equation 9}$$

where B_i is the width of the channel in cell i (m). Equation 8 and Equation 9 both assume a wide channel (wetted perimeter ≈ channel width). This was an appropriate assumption because a wide channel is assumed in the original U-McIUH model to generate hydrographs.

Equation 8 was not applicable for pipe cells because the depth of flow is not constant over the entirety of the free surface. Chow et al. (1988) expressed the kinematic wave celerity in an alternate formulation (Equation 10), which was used to calculate the kinematic wave celerity in pipe cells.

$$c_{k,i} = \frac{1}{\alpha_i \beta Q_i^{\beta-1}} \qquad \textbf{Equation 10}$$

P_i for pipe cells in Equation 5 was calculated using Equation 11, as described by Wurbs and James (2002):

$$P_i = D_i \frac{\theta_i}{2}$$ **Equation 11**

where D_i is the diameter of pipe cell i (m), and θ_i is the angle of the free surface in pipe cell i in radians (rad), as shown in Figure 7.

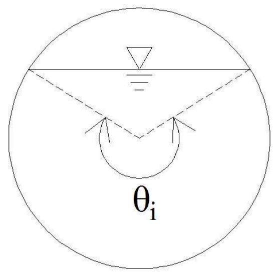

Figure 7: Cross section of pipe flow.

θ_i was calculated by solving two equations of cross sectional area of flow in a pipe simultaneously. Wurbs and James (2002) and Chow et al. (1988) each described an equation to calculate the cross sectional area of pipe flow in Equation 12 and Equation 13, respectively.

$$A_i = \frac{D_i^2}{4}\left[\frac{\theta_i}{2} + sin\left(\pi - \frac{\theta_i}{2}\right) cos\left(\pi - \frac{\theta_i}{2}\right)\right]$$ **Equation 12**

$$A_i = \left(\frac{n{P_i}^{2/3}}{\sqrt{S_{O,i}}}\right)^{3/5} Q_i^{3/5}$$ **Equation 13**

Equation 11 was substituted into Equation 13, which was set equal to Equation 12 to solve for θ_i.

For the numerical stability of Equation 4, the Courant condition must be satisfied in every channel and pipe cell downstrem of the on-site detention basin. For this to be true, Δt must be smaller than the ratio of the cell's length to its kinematic wave celerity in every downstream cell (Equation 7). The peak flow of the detention basin's discharge hydrograph was used to calculate the kinematic wave celerity in every downstream cell. The largest of these wave celerities was divided into the smallest Δx to calculate a stable Δt.

The kinematic routing code developed for this study was checked against an example presented by Chow et al. (1988) of a triangular hydrograph routed 4,572 m downstream in an open channel. Figure 8 shows the results. The similarity between the kinematic routing code's hydrograph and that of the hydrograph by Chow et al. (1988) indicates the code was programmed correctly.

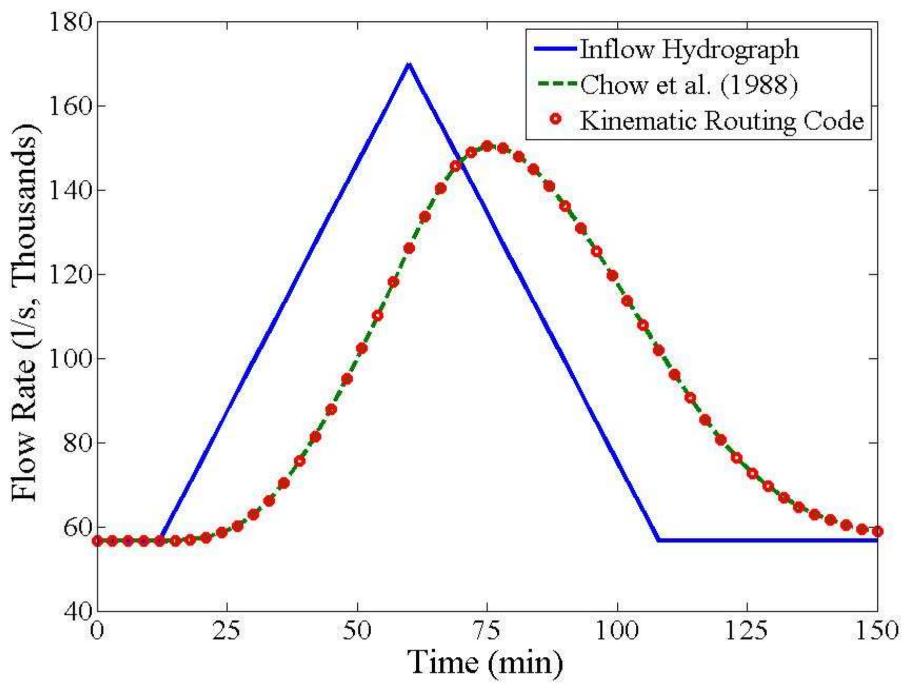

Figure 8: Comparison of kinematic routing code with Chow et al. (1988).

The second hydrograph routing method tested was simple routing, which transfers the upstream hydrograph to the outlet of the watershed or to the next downstream detention basin with no attenuation or delay. This method was tested because, compared to kinematic routing, it is much simpler and thus, less computationally expensive.

Prior to this study, the effects of disaggregating a watershed and routing hydrographs downstream (kinematic or simple) using the U-McIUH model had not been investigated. Therefore, to render confidence in our method, that investigation was included as part of this study and can be found in Appendix II. Results show that disaggregation and routing hydrographs kinematically is an appropriate method to model on-site detention basins in the U-McIUH model, and thus was chosen as the channel routing method for this study.

METHODS

On-site detention in the Mail Creek watershed was simulated by placing on-site detention basins at the downstream end of all 1st order channels and pipes. A similar approach was used by James et al. (1987) and Goff and Gentry (2006). 115 simulated on-site detention basins were placed in the Mail Creek watershed (Figure 9), with an average contributing area of 3.25 hectares (ha) (8.03 acres), ranging from 0.04 ha (0.10 acres) to 33.48 ha (82.73 acres). 54% of the total area in the Mail Creek watershed drains to a simulated on-site detention basin. This fraction is not 100% because not all locations in the watershed drain to a 1st order channel or pipe.

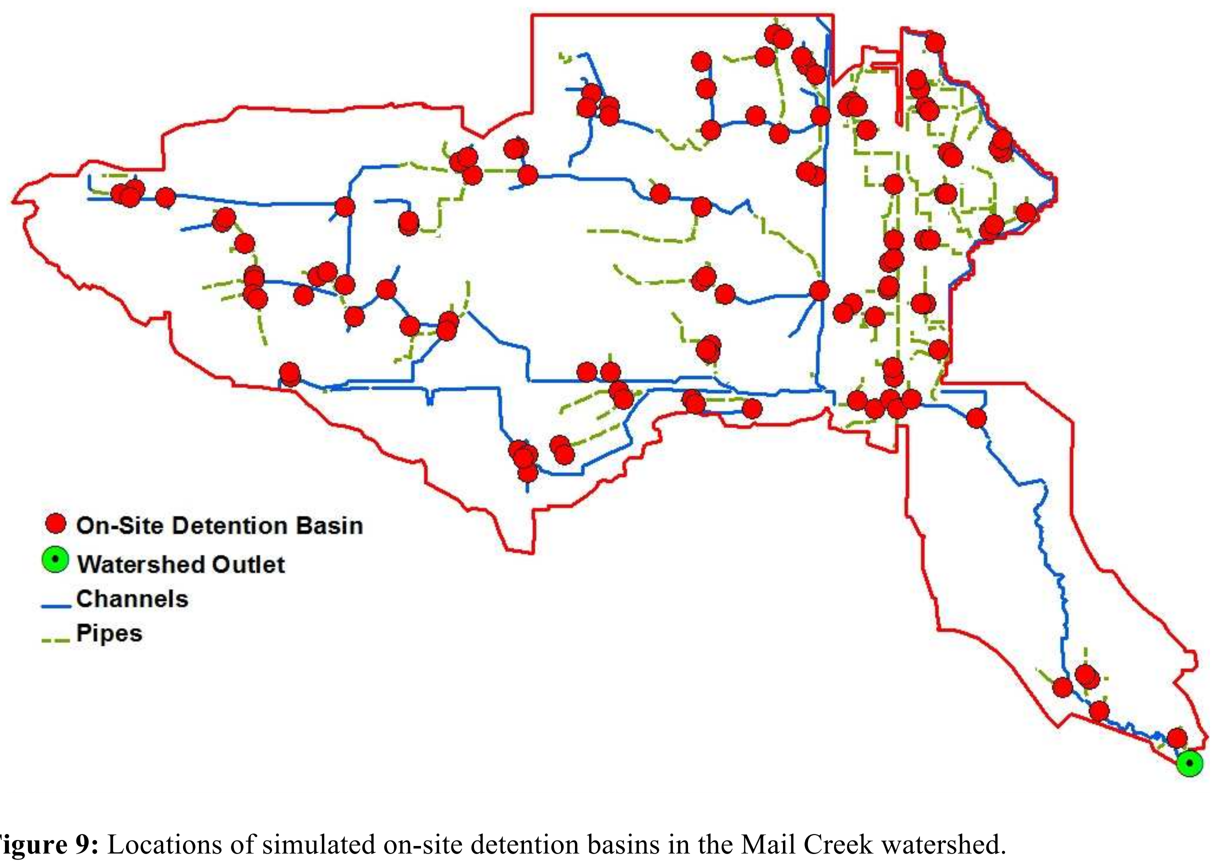

Figure 9: Locations of simulated on-site detention basins in the Mail Creek watershed.

Recall that level pool routing was chosen to route hydrographs through on-site detention basins. Level pool routing requires a storage-discharge curve for each on-site detention basin. Because we are simulating on-site detention, each on-site detention basin's storage-discharge was estimated using a method developed by Glidden (1981). That method incorporates two dimensionless storage discharge curves, one that specifies the storage discharge curve as a function of peak flows in and out of the on-site detention basin and its storage for one design storm (Figure 10), and another that does the same for two design storms (Figure 11). For a one design storm storage discharge curve, flow from the on-site detention basin is controlled by orifice flow until the detention basin's storage volume is filled, at which point any runoff spills over an emergency spillway uncontrolled. For a two design storm storage discharge curve, flow from the on-site detention basin is controlled by orifice flow until the first design storm's volume is met, and then flow is controlled by a weir until the second design storm's volume is met. Any additional runoff spills over an emergency spillway uncontrolled. The curve in Figure 10 was used to construct storage discharge curves for SD policies SD 5 – SD 8, and that in Figure 11 was used to construct storage discharge curves for SD policies SD 1 – SD 4. Q_h is the post-development peak flow release from the on-site detention basin (l/s), Q_d is the post-developed peak flow from the subwatershed's hydrograph flowing into the on-site detention basin (l/s), V_t is the on-site detention basin's storage (l), Q_i is the post-development peak flow release from the on-site detention basin for the smaller design storm (l/s) and V_i is the on-site detention basin's storage for the smaller design storm (l).

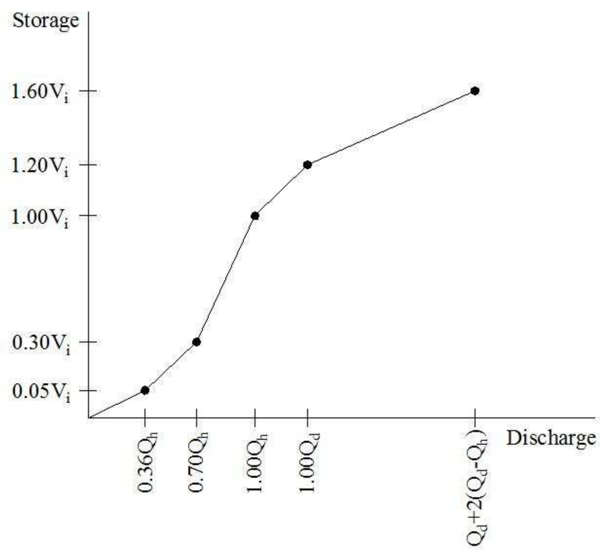

Figure 10: Dimensionless storage discharge curve to control the peak flow of one design storm (Glidden 1981). Q_h is the post-development peak flow release from the on-site detention basin (l/s), Q_d is the post-developed peak flow from the subwatershed's hydrograph flowing into the on-site detention basin (l/s), and V_i is the on-site detention basin's storage for the design storm (l).

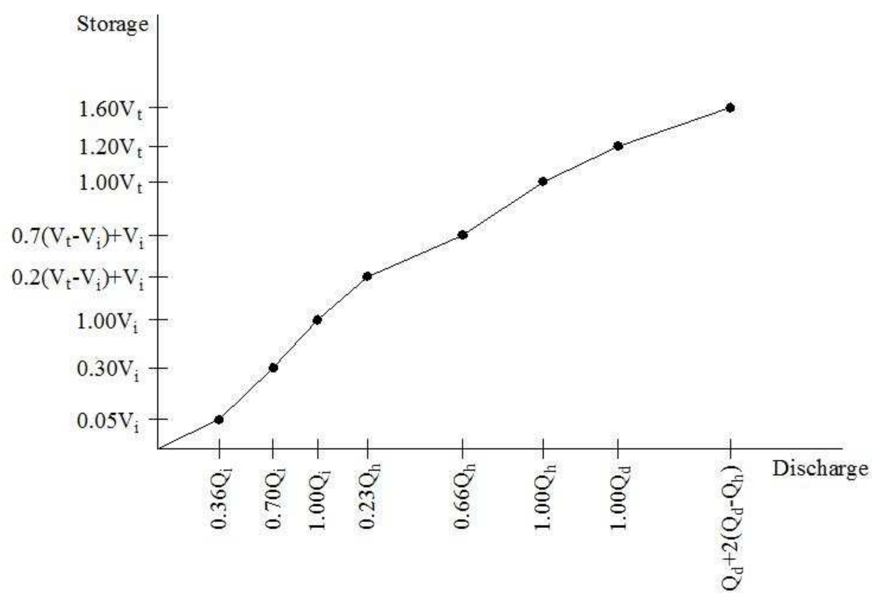

Figure 11: Dimensionless storage discharge curve to control the peak flow of two design storms (Glidden 1981). Q_i is the post-development peak flow release from the on-site detention basin for the smaller design storm (l/s), Q_h is the post-development peak flow release from the on-site detention basin (l/s), Q_d is the post-developed peak flow from the subwatershed's hydrograph flowing into the on-site detention basin (l/s), V_t is the on-site detention basin's storage (l), and V_i is the on-site detention basin's storage for the smaller design storm (l).

Q_h and Q_i were set equal to the pre-development peak runoff rate, and estimated using UDFCD unit peak release rates, which are shown in Table 2. The Mail Creek watershed contains a mixture of Type B (3.9 km^2) and Type C (3.2 km^2) soils (NRCS Web Soil Survey). As recommended by UDFCD for watersheds with a mixture of soil types, the maximum unit release rate was area weighted to 240 $l/s/km^2$, 1930 $l/s/km^2$ and 6420 $l/s/km^2$ for the 2, 10 and 100-year design storms, respectively.

Table 2: UDFCD maximum unit release rates ($l/s/km^2$) (UDFCD 2008).

Design Storm (Years)	NRCS Hydrologic Soil Group		
	A	**B**	**C & D**
2	140	210	280
10	910	1,610	2,100
100	3,500	5,950	7,000

Storage terms V_i and V_t were estimated by the same method used by Glidden (1981), which is illustrated in Figure 12. Knowing the inflow hydrograph into the detention basin (abcd), the post-development peak release (Q_h or Q_i), and assuming a linear rising portion of the on-site detention basin's discharge hydrograph (ac), the storage requirements are estimated by area (abc). The area (abc) was approximated using the rectangular rule.

The dimensionless storage discharge curves created by Glidden (1981) used design storms from Denver, Colorado. It is assumed they are applicable in Fort Collins, which is approximately 96 km north of Denver.

When storage discharge curves from Figure 11 were constructed, the value of $0.2(V_t-V_i)+V_i$ was smaller than $1.00V_i$ for each on-site detention basin's storage discharge curve. This problem was fixed by recomputing $0.2(V_t-V_i)+V_i$, by assuming linearity between coordinates ($1.00Q_i$, $1.00V_i$) and ($0.66Q_h$, $0.7(V_t-V_i)+V_i$) in Figure 11.

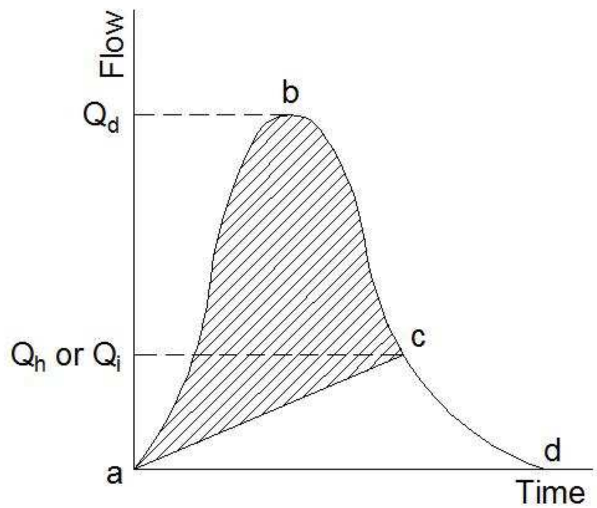

Figure 12: On-site detention basin **s**torage estimation. Q_h is the post-development peak flow release from the on-site detention basin (l/s), Q_i is the post-development peak flow release from the on-site detention basin for the smaller design storm (l/s), and Q_d is the post-developed peak flow from the subwatershed's hydrograph flowing into the on-site detention basin (l/s). Hatched area (abc) is the estimated on-site detention basin storage.

Peak flows for the various Non-SD and SD policies were computed at the intersection of every channel and pipe in the watershed. These intersections are categorized as 2nd, 3rd, 4th, or 5th order channel and pipe intersections. When two 1st order channel or pipes intersected, that intersection is considered a 2nd order intersection. When two 2nd order channel or pipes intersected, that intersection is a 3rd order intersection, etc.

The City of Fort Collins provided much of the data used to construct the Mail Creek watershed in the U-McIUH model. Two-foot elevation contours of the Mail Creek watershed were transformed into a DEM at a resolution of 20 m. That resolution was chosen because it was used by Gironás et al. (2009) when the U-McIUH model was developed. The watershed boundary and location of all channels and pipes were provided in an AutoCAD file, and street locations were obtained from the City of Fort Collins' Geographic Information Services website in the form of a geographic information system (GIS) shape file (City of Fort Collins Geographic Information Services).

Pipe diameters and channel widths were uniformly assigned a value of 4 m (with exception of channels and pipes near the outlet of the watershed, which were assigned a value of 7 m). This was done because the U-McIUH is based on kinematic wave theory where pipe cells cannot be pressurized. Such large channel width and pipe diameters were used to accommodate the 100-year design storm, which was selected as the rain event for this study. Channel and pipe slopes were obtained from a previous model of the Mail Creek watershed constructed in the Urban Drainage Storm Water Management Model (UDSWM) (Urban Drainage and Flood Control District 2001). Not all channels and pipes in the Mail Creek watershed that are included in the U-McIUH model were represented in the UDSWM model. Slopes of those channels and pipes were assumed to be equal to the slope of the DEM cell it was located in.

Manning's roughness values for channels, pipes, and streets were assigned values of 0.033, 0.014, and 0.02, respectively, which are from the UDSWM model. Manning's roughness values used for impervious and pervious surfaces were 0.016 and 0.250, respectively, and were obtained from the UDSWM model as well.

The U-McIUH model simulates infiltration using a simplified version of the Horton model, which sets the initial and final infiltration rates equal to a single value. In the UDSWM model, the Horton initial and final infiltration rates are 12.954 mm/hr and 12.700 mm/hr, respectively. In the U-McIUH model, the average of the two, i.e. 12.827 mm/hr, was used.

Table 3 defines Fort Collins' zoning and its respective imperviousness, and Figure 13 displays the zoning within the Mail Creek watershed. Percent imperviousness for each cell in the watershed was derived from this information. Each zoned area has an assigned percent imperviousness, and each watershed cell within a particular zone was assigned that zone's imperviousness.

Table 3: Zoning and its respective imperviousness in the Mail Creek watershed (City of Fort Collins Stormwater Criteria Manual 2011).

Zone	Zone Name	Imperviousness (%)
CG	General Commercial District	90
E	Employment District	80
HC	Harmony Corridor District	90
LMN	Low Density Mixed Use Neighborhood District	50
MMN	Medium Density Mixed Use Neighborhood District	70
NC	Neighborhood Commercial District	90
POL	Public Open Lands District	10
RL	Low Density Residential District	45
UE	Urban Estate District	30

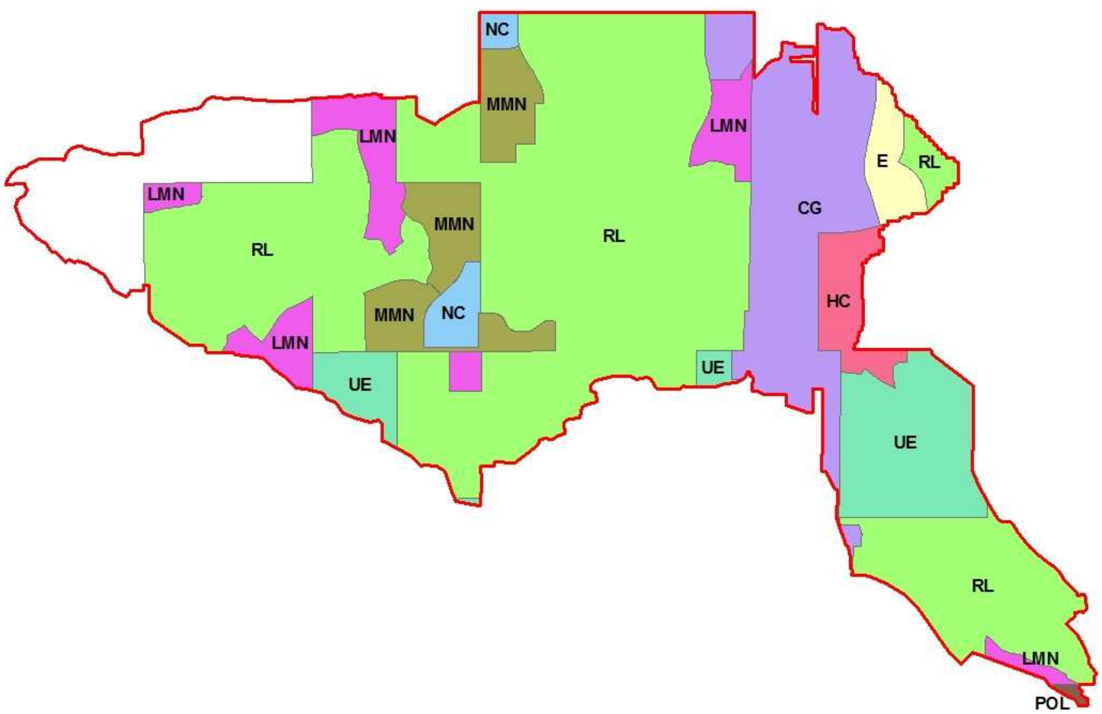

Figure 13: Mail Creek watershed zoning (City of Fort Collins Geographic Information Services).

A small area in the western portion of the Mail Creek watershed (white area in Figure 13) is not incorporated into the City of Fort Collins, and therefore zoning and impervious information for that area were not provided. That area was compared to similar areas in Fort Collins using aerial maps and assigned an imperviousness of 30%.

The UDSWM model provided depression storage for impervious (2.54 mm) and pervious (7.62 mm) surfaces. However, the U-McIUH model uses only one value of depression storage for the entire watershed. A single depression storage value of 5.051 mm was computed as a weighted average of the pervious and impervious areas within the watershed.

The 2 hour 100-year (93.19 mm) and 2 hour 10-year (43.43 mm) design storms for the City of Fort Collins were used to construct storage discharge curves, and the 2 hour 100-year design storm was used as the model's precipitation event. This precipitation event was chosen because the City of Fort Collins requires all on-site detention basins to control the post-development 100-year design storm peak flow. Both storms' precipitation time series were discretized into five-minute intervals, and are displayed in Figure 14.

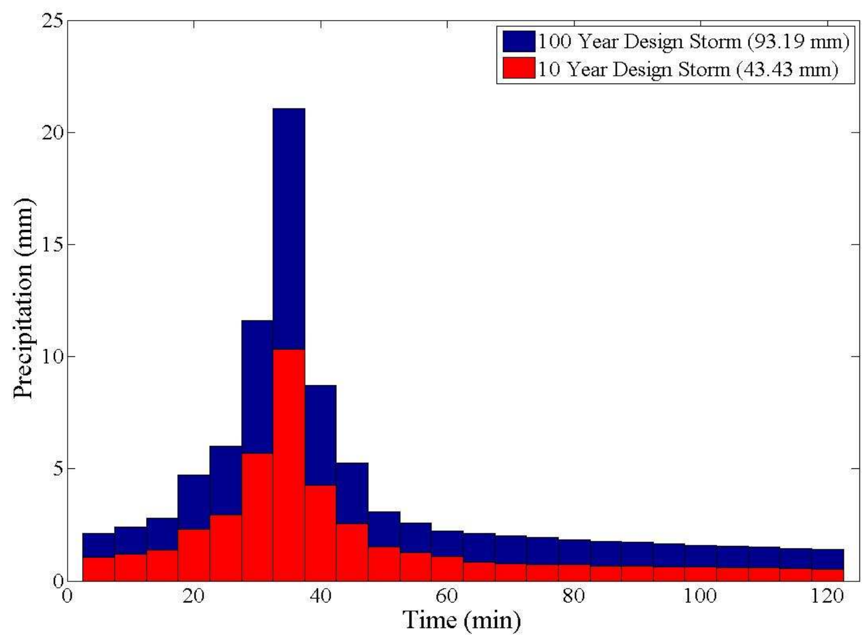

Figure 14: 2 hour 100-year and 2 hour 10-year City of Fort Collins design storms (City of Fort Collins Stormwater Criteria Manual 2011).

A principle part of the Mail Creek watershed is irrigation ditches. They are used to transport water into and out of the watershed for flood irrigation. These flows, if used in model simulations, would reduce the stormwater capacity of the ditches, increase flow velocities, and would alter the IUH calculated by the U-McIUH model. To isolate the effects of Non-SD and SD policies, irrigation flows were not modeled in this study.

The U-McIUH model of the Mail Creek watershed was not calibrated for two reasons. First, there is no stream flow data at the outlet of Mail Creek, and second, we are only interested in relative differences between scenarios and not their absolute values.

RESULTS AND DISCUSSION

Analysis of Non-SD Policies

Table 4 lists the relative reduction of peak flow at the watershed outlet when using Non-SD 1 and Non-SD 2 policies to design on-site detention basins compared to a policy that uses no on-site detention.

Table 4: Percent reduction of peak flow at the watershed outlet compared to a policy that uses no on-site detention.

Non-SD Policy	Peak Flow Reduction
Non-SD 1	50.9%
Non-SD 2	54.4%

Model results show that Non-SD 1 and Non-SD 2 policies reduced peak flows at the watershed outlet by 50.9% and 54.4% respectively when compared to a policy that uses no on-site detention basins. These results suggest if Non-SD 1 or Non-SD 2 are used to design on-site detention basins, the peak flow at the watershed outlet would be reduced. This is in contrast to the findings of McCuen (1974) and Emerson et al. (2005). For this study Non-SD 1 and Non-SD 2 policies altered the timed release of peak flows in a manner that reduced the peak flow at the watershed outlet, as well as the potential for flooding and stream erosion downstream of the watershed.

Table 5 presents the relative reduction of average peak flows at 2nd through 5th ordered channel and pipe intersections compared to a policy with no on-site detention basins.

Table 5: Percent reduction of average peak flows at 2nd through 5th ordered channel and pipe intersections compared to a policy with no on-site detention basins.

Detention Basin Policy	2nd Order	3rd Order	4th Order	5th Order
Non-SD 1	80.1%	58.2%	55.6%	52.2%
Non-SD 2	87.0%	61.0%	58.4%	55.2%

These results show Non-SD 1 and Non-SD 2 reduce average peak flows at 2nd through 5th order channel and pipe intersections compared to a policy that specifies no on-site detention. In general, peak flow reductions at all ordered channel and pipe intersections were similar for the two Non-SD policies. Both reduced average peak flows at 2nd order channel and pipe intersections most, while the reductions in 3rd, 4th and 5th ordered intersections were smaller. The difference between the 2nd to 3rd order reduction of average peak flows was greater than the difference between 3rd to 4th, and 4th to 5th order channel and pipe intersections.

These results suggest that Non-SD 1 and Non-SD 2 policies can successfully reduce average peak flows at 2nd through 5th order channel and pipe intersections. 2nd order intersections would benefit the most under Non-SD 1 or Non-SD 2 because their peak flows will, on average, be reduced by 80.1% and 87.0%, respectively, compared to no mitigation. These policies will also reduce the potential for localized flooding and erosion. Down stream of 2nd order intersections, the ability of Non-SD 1 and Non-SD 2 to reduce average peak flows in channel and pipe intersections decreases as the order of the channel and pipe intersection increases. The same observation was made by Mein (1980), James et al. (1987), and Debo and Reese (1992). These intersections will have a reduced potential for localized flooding and erosion also, but not as much as 2nd order intersections.

Figure 15 and Figure 16 present the relative reduction of peak flow at the intersection of every channel and pipe in the watershed compared to a policy with no on-site detention basins for Non-SD 1 and Non-SD 2 policies, respectively.

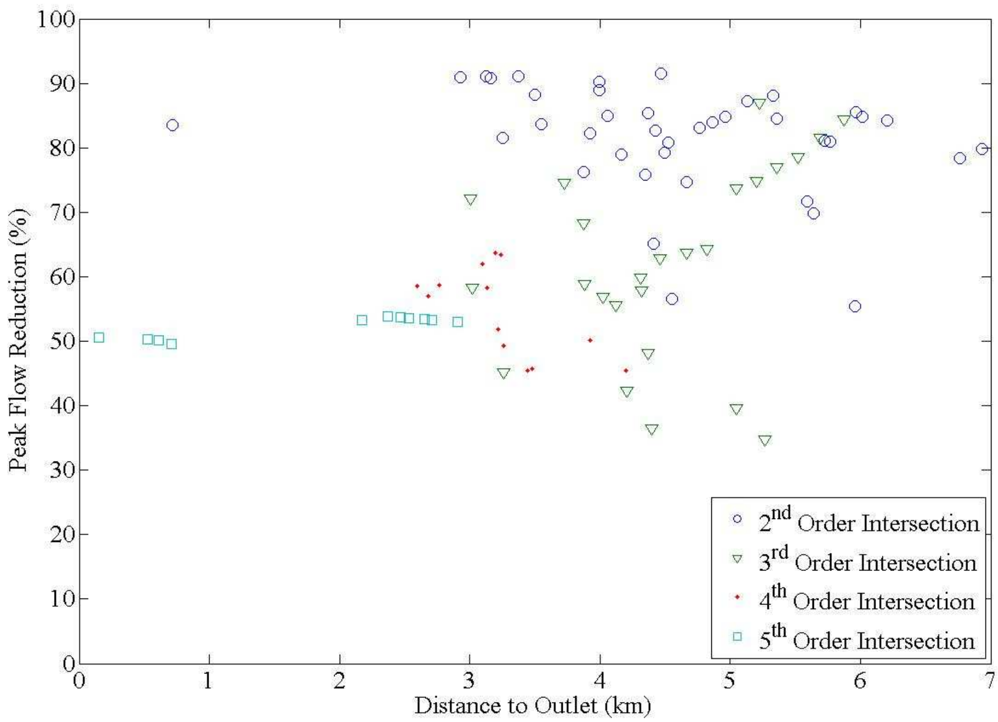

Figure 15: Percent peak flow reduction at 2nd through 5th ordered channel and pipe intersections for Non-SD 1 compared to a policy with no on-site detention basins.

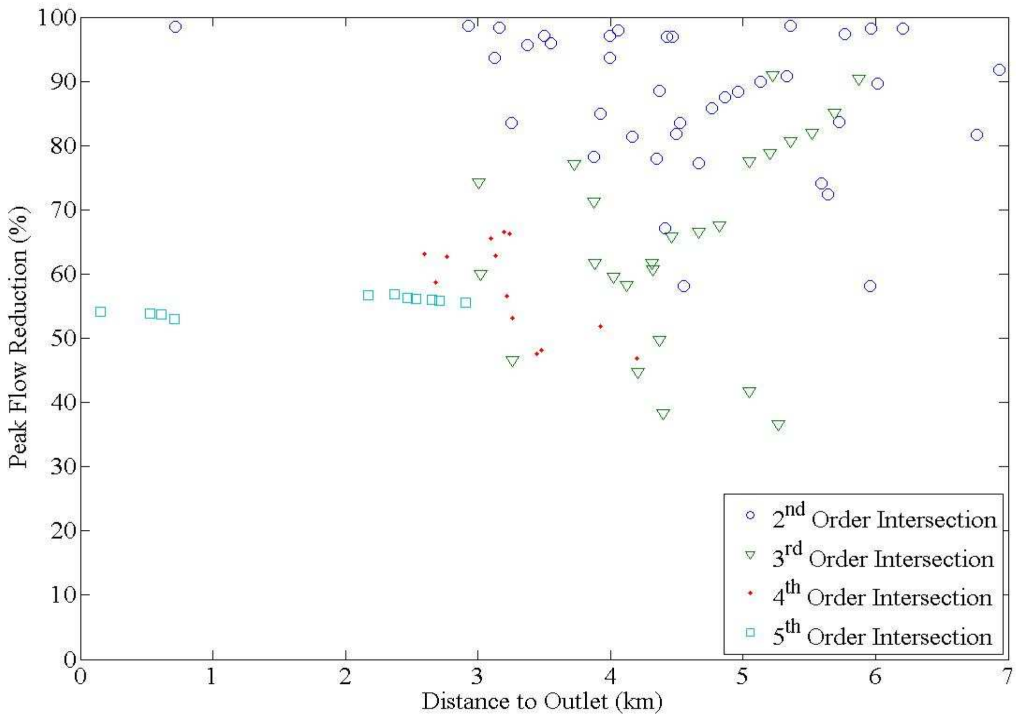

Figure 16: Percent peak flow reduction at 2nd through 5th ordered channel and pipe intersections for Non-SD 2 compared to a policy with no on-site detention basins.

In general, the Non-SD 2 policy was able to reduce peak flows only slightly more than Non-SD 1 at all intersections. This suggests the lower peak flow released from Non-SD 2 on-site detention basins has only a slight effect on peak flows at channel and pipe intersections throughout the watershed. Because all intersections in the watershed had positive peak flow reductions for both Non-SD policies, it suggests that peak flows released from on-site detention basins did not combine with other peak flows in the watershed to increase the peak flow above that which would have occurred under a policy that used no on-site detention basins. Similar to the behavior of average peak flow reductions in 2nd through 5th order intersections (Table 5), relative peak flow reductions, in general, diminished as the order of the intersection increased. Both 2nd and 3rd ordered intersections demonstrated a large variability in peak flow reductions compared to 4th and 5th ordered intersections. This indicates peak flow reductions at 2nd and 3rd order intersections can be expected to vary significantly; whereas a more consistent peak flow reduction can be expected at 4th and 5th order intersections.

Relative Change in Watershed Storage and Peak Flow at the Watershed Outlet of SD Policies Compared to Non-SD Policies

The effectiveness of SD policies to reduce watershed storage and the watershed outlet peak flow compared to a Non-SD policy are presented in Table 6 and Table 7. Table 6 compares the results of SD 1 - SD 4 policies to those of Non-SD 1. Table 7 compares the results of SD 5 – SD 8 policies to those of Non-SD 2.

Table 6: Relative reduction of watershed storage and peak flow at the watershed outlet for SD 1 - 4 compared to Non-SD 1.

SD Policy	Non-SD 1 to SD Transformation	ε	ω	Watershed Storage Reduction	Peak Flow Reduction
SD 1	Equation 1	-0.6	1.3	-1.2%	-0.1%
SD 2	Equation 1	-2.0	2.0	-7.4%	-1.0%
SD 3	Equation 2	-0.6	1.3	1.2%	-0.7%
SD 4	Equation 2	-2.0	2.0	3.5%	-2.7%

Table 7: Relative reduction of watershed storage and peak flow at the watershed outlet for SD 5 – 8 compared to Non-SD 2.

SD Policy	Non-SD 2 to SD Transformation	ε	ω	Watershed Storage Reduction	Peak Flow Reduction
SD 5	Equation 1	-0.6	1.3	-0.1%	0.0%
SD 6	Equation 1	-2.0	2.0	-0.3%	0.0%
SD 7	Equation 2	-0.6	1.3	0.1%	-0.0%
SD 8	Equation 2	-2.0	2.0	0.3%	-0.1%

Compared to Non-SD 1, SD policy scenarios derived from Equation 1 increased watershed storage, while those derived from Equation 2 decreased watershed storage. SD 2 increased watershed storage the most (7.4%), while SD 4 decreased watershed storage the most (3.5%). As ε decreased and ω increased, the relative watershed storage increased for SD policies derived from Equation 1, while it decreased for SD policies derived from Equation 2. All SD scenarios derived from Equation 1 and Equation 2 increased the peak flow at the watershed outlet relative to the Non-SD 1 policy. As ε decreased and ω increased, the relative change in the watershed outlet peak flow increased for SD policies derived from Equation 1 and Equation 2, when compared to Non-SD 1. SD 4 increased the relative watershed outlet peak flow the most (2.7%), while SD 1 increased it the least (0.1%). Compared to Non-SD 2, no SD policy had a significant effect on the relative watershed storage or relative peak flow at the watershed outlet, which did not support our hypothesis.

These results suggest that SD policies do not affect the peak flow at the watershed outlet with any significance when compared to Non-SD policies. Changing the time at which peak flows were released was not enough to prevent peak flows from combining with other flows in the watershed to alter the watershed outlet peak flow with any significance. Consequently, areas downstream of the watershed have the same potential for flooding and erosive flows regardless of the stormwater policy that is used. Although SD policies derived from Equation 1 are designed to have no change in watershed storage compared to a Non-SD policy, results showed policies SD 1 and SD 2 actually increased watershed storage. A possible explanation is the disproportionate number of on-site detention basins in the upper half of the Mail Creek watershed. Out of the 115 on-site detention basins simulated in the watershed, 77 are located in the upper half of the watershed (ℓ_i /L > 0.5), while the remaining 38 are in the lower half (ℓ_i /L ≤ 0.5). SD 1 and SD 2 increased the storage of the 77 on-site detention basins in the upper half of the watershed while reducing storage in only 38 on-site detention basins in the lower half. In contrast, SD 3 and SD 4 only decreased the storage of on-site detention basins in the lower half of the watershed and showed no change in storage in the upper half (as SD policies derived from Equation 2 are designed to do). Therefore, the only possible change in watershed storage was a decrease.

Relative Change in Average Peak Flow for 2nd Through 5th Order Channel and Pipe Intersections For SD Versus Non-SD Policies

Table 8 and Table 9 present the percent reduction of average peak flows at 2nd through 5th ordered channel and pipe intersections for SD policies compared to Non-SD policies. Table 8 compares those results from policies SD 1 - SD 4 to those of Non-SD 1, and Table 9 compares results from policies SD 5 - SD 8 to those of Non-SD 2.

Table 8: Percent reduction of average peak flows at 2nd through 5th ordered channel and pipe intersections for SD 1 - SD 4 compared to Non-SD 1.

Detention Basin Policy	Non-SD 1 to SD Transformation	ε	ω	2nd Order	3rd Order	4th Order	5th Order
SD 1	Equation 1	-0.6	1.3	4.8%	1.4%	0.4%	0.3%
SD 2	Equation 1	-2.0	2.0	10.5%	3.9%	1.2%	0.8%
SD 3	Equation 2	-0.6	1.3	-1.3%	-0.0%	0.1%	0.3%
SD 4	Equation 2	-2.0	2.0	-5.0%	-0.0%	-0.4%	-1.1%

Table 9: Percent reduction of average peak flows at 2nd through 5th ordered channel and pipe intersections for SD 5 - SD 8 compared to Non-SD 2.

Detention Basin Policy	Non-SD 2 to SD Transformation	ε	ω	2nd Order	3rd Order	4th Order	5th Order
SD 5	Equation 1	-0.6	1.3	0.2%	0.1%	0.0%	0.0%
SD 6	Equation 1	-2.0	2.0	0.6%	0.2%	0.1%	0.1%
SD 7	Equation 2	-0.6	1.3	-0.0%	0.0%	0.0%	-0.0%
SD 8	Equation 2	-2.0	2.0	-0.0%	0.0%	-0.0%	-0.0%

SD policies had varying effects on average peak flows at 2nd order channel and pipe intersections. SD policies derived from Equation 1 showed positive reductions, while SD policies derived from Equation 2 showed negative reductions. Results in Table 8 show that SD 4 and SD 2 had the least (-5.0%) and greatest (10.5%) reduction of average peak flows in 2nd order channel and pipe intersections, respectively, when compared to Non-SD 1. Results in Table 9 show that SD 7 an SD 8 had the least (-0.0%), and SD 6 had the greatest (0.6%) reduction of average peak flows at 2nd order channel and pipe intersections when compared to Non-SD 2. The percent reduction of peak flows at 2nd order channel and pipe intersections also varied with ε and ω. For SD policies derived from Equation 1, as ε decreased and ω increased, the percent reduction of the average peak flow at 2nd order channel and pipe intersections increased. For SD policy scenarios derived from Equation 2, as ε decreased and ω increased, the percent reduction of the average peak flow at 2nd order channel and pipe intersections decreased. The difference in the reduction of average peak flows between 2nd and 3rd order channel and pipe intersections was greater than the differences between 3rd and 4th, and 4th and 5th ordered intersections.

When compared to Non-SD 1, SD policies derived from Equation 1 reduce the average peak flow at 2nd order intersections more than any other ordered intersection in the watershed. As the order of the intersection increases, the average reduction in peak flow decreases. This indicates that the ability of an SD policy derived from Equation 1 to reduce average peak flows decreases as the order of a channel and pipe intersection increases. Altering the timed release of peak flows from on-site detention basins according to Equation 1 cannot maintain the reduction in average peak flow at higher order intersections downstream as that observed at 2nd order intersections. Consequently, on average, 2nd order intersections have a decreased potential for localized flooding and erosion, and that benefit diminishes as the intersection order increases.

In contrast, SD policies derived from Equation 2 increase the average peak flow at 2nd order intersections, and had little to no effect on the remaining ordered intersections. Reducing the time at which the peak flow is released from on-site detention basins only in the lower half of the watershed will increase the average peak flow of 2nd order intersections and have little to no effect on the remaining intersections. Thus, under SD policies derived by Equation 2, 2nd order intersections will have an increased potential of localized flooding and erosion while the higher ordered intersections will have the same potential for localized flooding and erosion as Non-SD 1.

When compared to Non-SD 2, SD policies had little to no effect on the ordered intersections' average peak flow. The ability of an SD policy to alter the average peak flow at any channel and pipe intersections over that of Non-SD 2 policies is limited, and does not change as the order of the intersection changes. Therefore, on average, channel and pipe intersections throughout the watershed will have the same potential for localized flooding and erosion as if the on-site detention basins were designed using Non-SD 2.

Relative Change in the Peak Flow at all 2nd Through 5th Order Channel and Pipe Intersections For SD Versus Non-SD Policies

Figure 17 and Figure 18 present the relative changes in peak flow at 2nd through 5th order intersections for SD 1 and SD 2 policies, respectively, compared to Non SD 1.

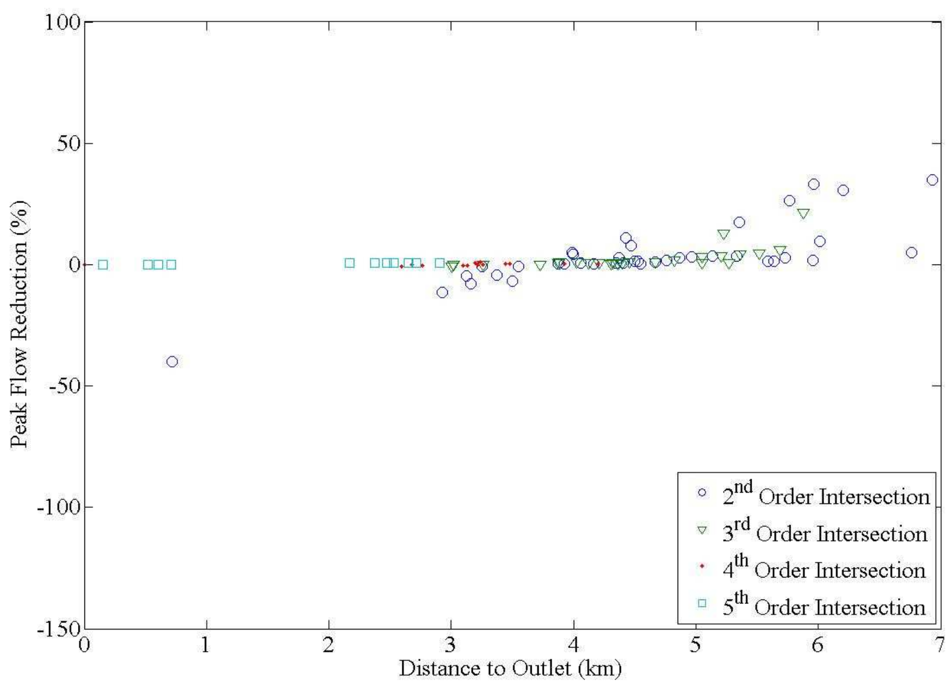

Figure 17: Percent peak flow reduction at 2nd through 5th order intersections of channel and pipes for SD 1 (ε = -0.6, ω = 1.3) compared to Non-SD 1 as a function of their distance to the watershed outlet.

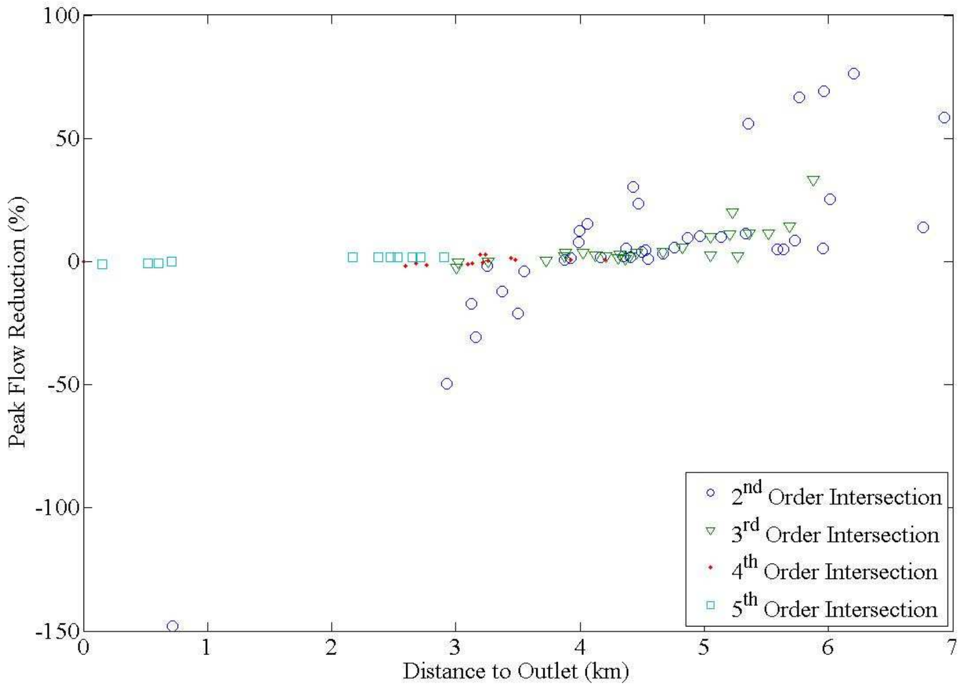

Figure 18: Percent peak flow reduction at 2nd through 5th order intersections of channels and pipes for SD 2 (ε = -2.0, ω = 2.0) compared to Non-SD 1 as a function of their distance to the watershed outlet.

SD 1 and SD 2 had both positive and negative peak flow reductions at 2nd order channel and pipe intersections when compared to Non-SD 1. 3rd order peak flow reductions were either positive or changed very little, while the effect of SD 1 and SD 2 on 4th and 5th order peak flow reductions was negligible. 2nd and 3rd order positive peak flow reductions occurred in the upper half of the watershed, and 2nd order negative peak flow reductions (peak flow increases) generally occurred in the lower half of the watershed.

All 2nd and 3rd order intersections in the upper half of the watershed had positive peak flow reductions. This suggests by delaying the peak flow release from these on-site detention basins allows peaks from other parts of the watershed to flow downstream before the peak from an on-site detention basin arrives. However, the magnitudes of the peak flow reductions at these intersections were mixed. In some instances, intersections with roughly the same flow distance to the watershed outlet (and thus roughly the same $Q_{P,SDi}$ / $Q_{P,NonSDi}$ ratio) had completely different peak flow reductions. The variability of peak flow reductions in 2nd and 3rd order intersections suggests that SD policies governed by Equation 1 cannot reliably reduce peak flows at 2nd and 3rd order intersections. For example, in Figure 18, at approximately 6 km from the watershed outlet, a 2nd order intersection has a peak flow reduction of 5%, but another 2nd order intersection with approximately the same flow distance to the watershed outlet has a peak flow reduction of 68%. A similar result for 3rd order intersections can be seen at approximately 5.2 km from the watershed outlet. The only pattern of peak flow reductions is that the range in peak flow reduction at 2nd and 3rd order intersections reduces as the distance from the watershed outlet becomes smaller. In addition, the range of 2nd and 3rd order peak flow reductions increases as ε decreases and ω increases. The unpredictability of peak flow reductions in 2nd and 3rd order intersections in the upper half of the watershed indicates that reducing the peak flow released from on-site detention basins based on its distance from the watershed outlet will not guarantee a predictable reduction in peak flows at 2nd or 3rd order channel and pipe intersections in the upper half of the watershed, and thus cannot ensure its ability to reduce the potential for localized flooding or erosion.

In the lower half of the watershed, all 2nd order channel and pipe intersections had a negative peak flow reduction (peak flow increase). In general, negative flow reduction increased as the flow distance decreased, however the variability was high and similar to that of the 2nd and 3rd intersections in the upper half of the watershed. Releasing the peak flow more quickly in on-site detention basins in the lower half of the watershed did not prevent their peak flows from combining with other peak flows in the watershed; consequently increasing the potential for localized flooding and erosion at those intersections. SD 1 and SD 2 had little effect on 3rd order intersections in the lower half of the watershed. Increasing the peak flow release from on-site detention basins in the lower half of the watershed had little effect on these peak flows combining with other peak flows in the watershed. These intersections can expect to have the same peak flows that would have occurred under Non-SD 1, and thus have the same potential for localized flooding and erosion.

SD 1 and SD 2 had little to no affect on 4th and 5th order intersections. Delaying the peak flow release from on-site detention basins in the upper half of the watershed and hastening the peak flow release of on-site detention basins in the lower half of the watershed did not significantly change the potential for combining peak flows at these intersections. Thus, under SD policies, 4th and 5th order intersections will experience virtually no change in peak flow, and would have the same potential for localized flooding and erosion as Non-SD 1.

Figure 19 and Figure 20 present the relative change in peak flow at 2nd through 5th order intersections for SD 3 and SD 4 compared to Non SD 1.

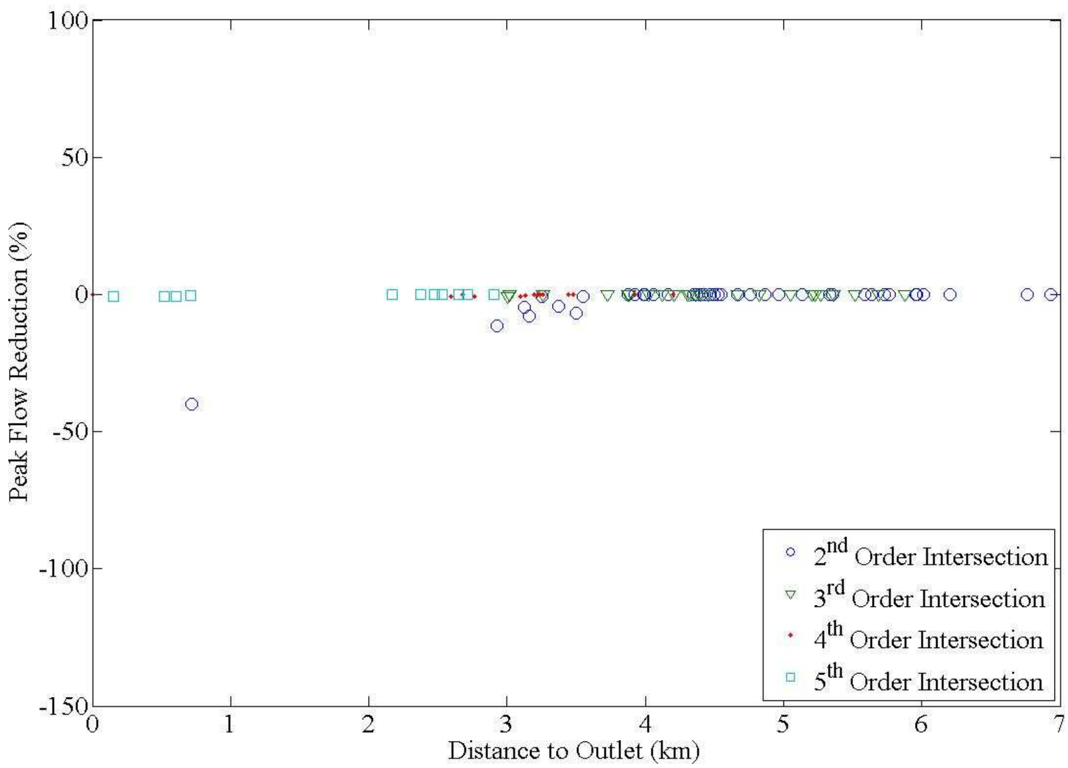

Figure 19: Percent peak flow reduction at 2nd through 5th order intersections of channels and pipes for SD 3 (ε = -0.6, ω = 1.3) compared to Non-SD 1 as a function of their distance to the watershed outlet.

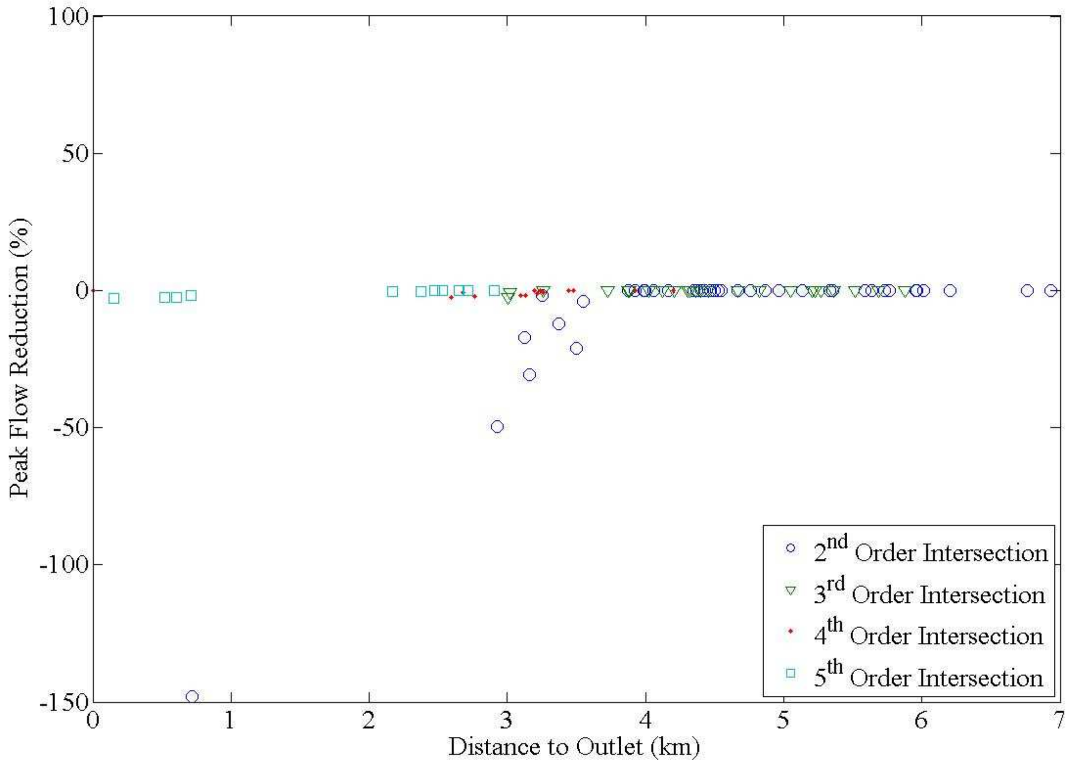

Figure 20: Percent peak flow reduction at 2nd through 5th order intersections of channel and pipes for SD 4 (ε = -2.0, ω = 2.0) compared to Non-SD 1 as a function of their distance to the watershed outlet.

The relative peak flow reduction from SD 3 and SD 4 compared to Non-SD 1 showed similar results. Peak flows were affected by those SD policies only at 2nd order intersections in the lower half of the watershed. In Figure 19 and Figure 20, those reductions are negative, signifying that the flow actually increased. The only difference between the results of SD 3 and SD 4 was in the magnitude of the 2nd order peak flow reductions, which was greater for SD 4.

Results from Figure 19 and Figure 20 indicate releasing peak flows sooner from on-site detention basins in the lower half of the watershed will have virtually no impact on 3rd, 4th or 5th ordered intersections, while increasing peak flows at 2nd order intersections in the lower half of the watershed. This suggests that SD 3 and SD 4 did not alter the timing of the peak flow releases enough to avoid combining with other peak flows form the watershed. As a result, peak flows at 2nd order intersections in the lower half of the watershed will have an increased potential for flooding and erosion, while the remaining intersections will have the same potential for flooding and erosion as under policy Non-SD 1.

Figure 21 through Figure 24 present the relative change in peak flow at 2nd through 5th order intersections for policies SD 5 through SD 8, compared to Non SD 2.

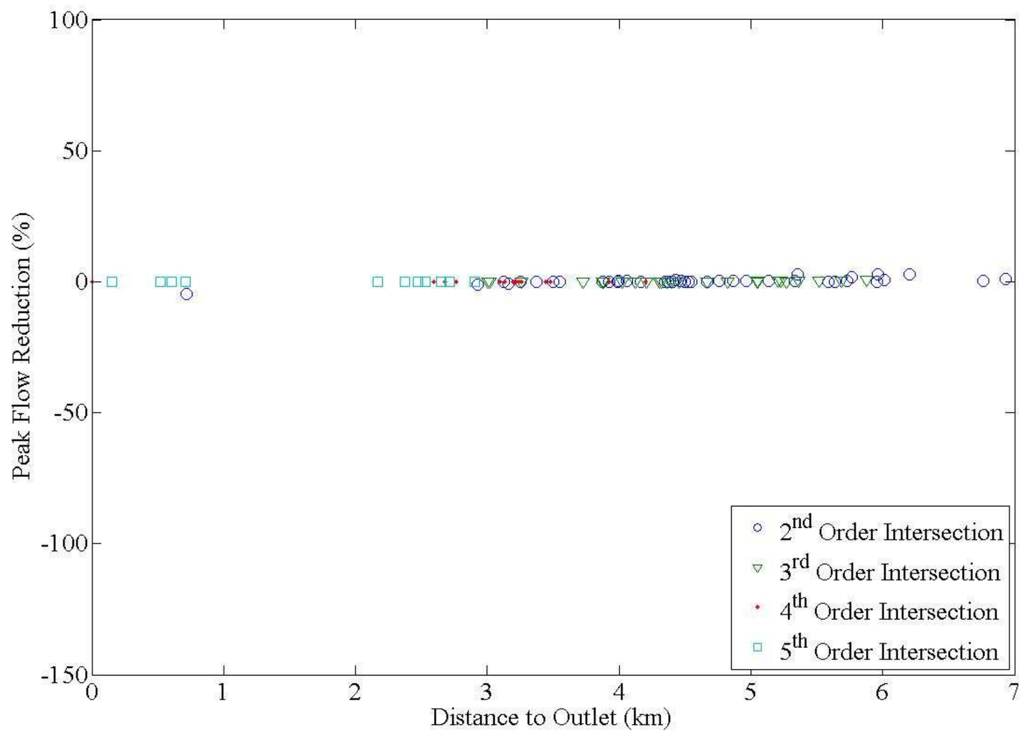

Figure 21: Percent peak flow reduction at 2nd through 5th order intersections of channels and pipes for SD 5 (ε = -0.6, ω = 1.3) compared to Non-SD 2 as a function of their distance to the watershed outlet.

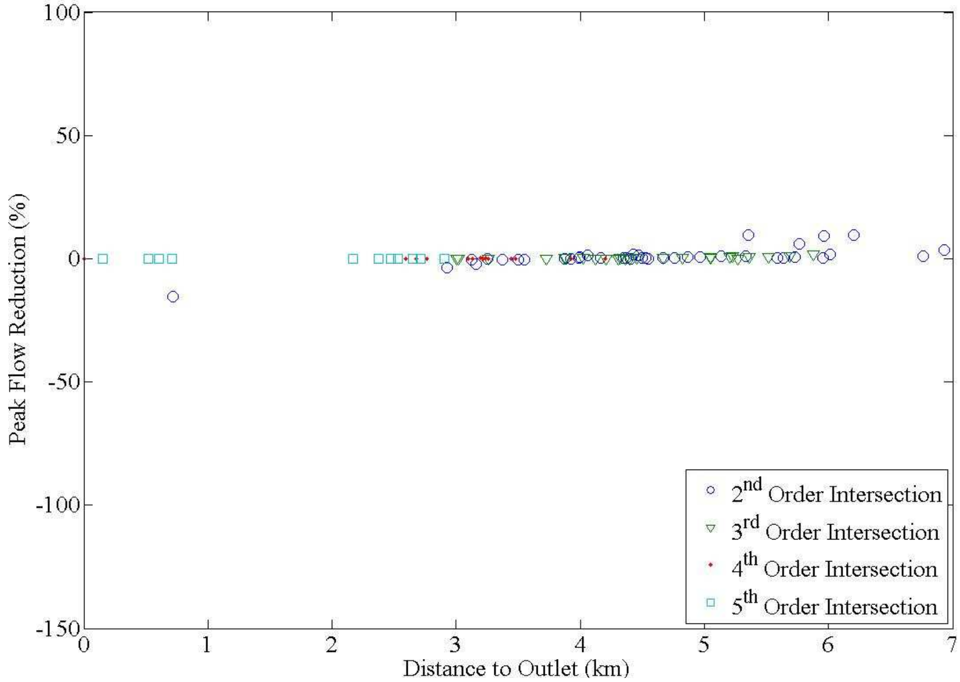

Figure 22: Percent peak flow reduction at 2nd through 5th order intersections of channels and pipes for SD 6 (ε = -2.0, ω = 2.0) compared to Non-SD 2 as a function of their distance to the watershed outlet.

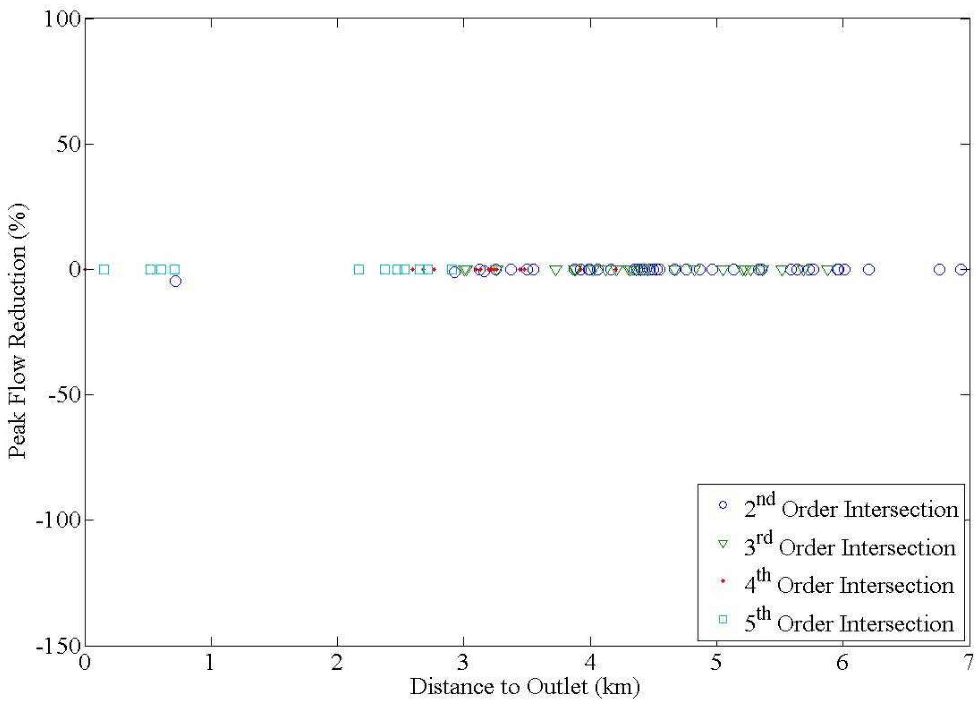

Figure 23: Percent peak flow reduction at 2nd through 5th order intersections of channels and pipes for SD 7 (ε = -0.6, ω = 1.3) compared to Non-SD 2 as a function of their distance to the watershed outlet.

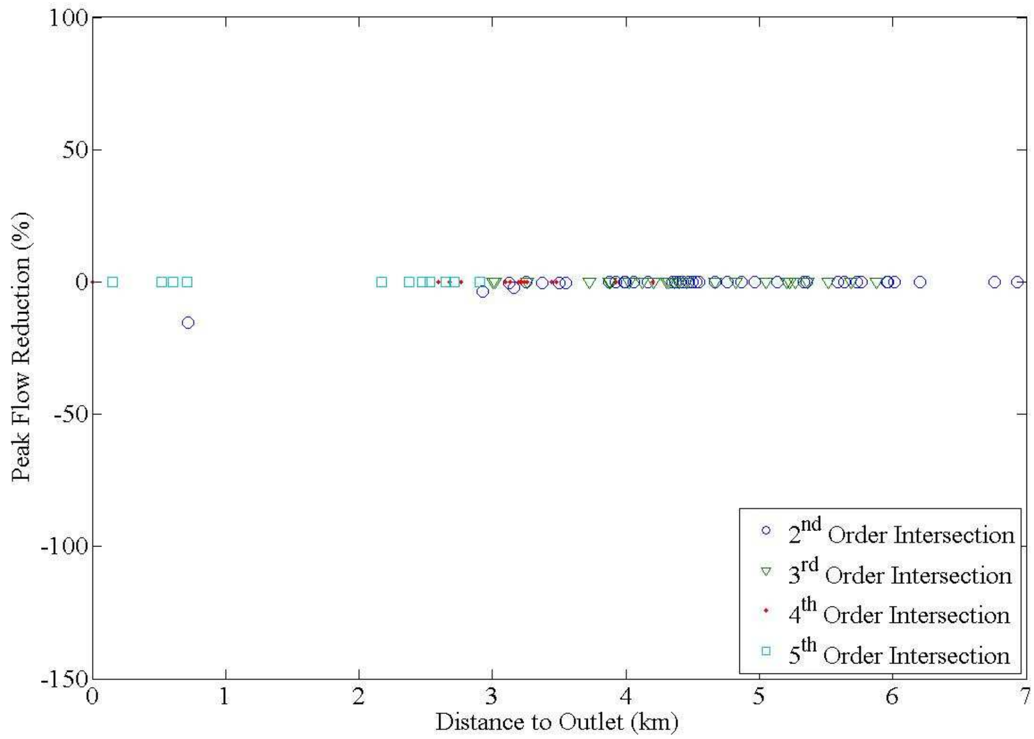

Figure 24: Percent peak flow reduction at 2nd through 5th order intersections of channels and pipes for SD 8 (ε = -2.0, ω = 2.0) compared to Non-SD 2 as a function of their distance to the watershed outlet.

The relative changes in peak flow for SD 5 - SD 8 compared to Non-SD 2 were similar to the SD 1 - SD 4 comparisons to Non-SD 1, however, the magnitudes of the changes were significantly smaller. In most cases, the peak flow reduction at channel and pipe intersections did not change with any significance regardless of the values of ε and ω in the SD governing equations. SD 6 slightly effected 2nd order peak flows in the upper half of the watershed, and SD 6 and SD 8 had slight effects on 2nd order intersections in the lower half of the watershed. Peak flows at all other intersections were virtually unchanged relative to Non-SD 2.

It is clear that altering the peak flow that would have occurred under Non-SD 2 by Equation 1 or Equation 2 did not change the interaction between the peak flows released from on-site detention basins and peak flows from other portions of the watershed with any significance. Recall that peak flow exiting on-site detention basins under Non-SD 2 are determined by the pre-developed 2-year storm peak flow. Altering the timing and quantity of each on-site detention basin's released peak by applying SD policies does not prevent them from combining with other peak flows in the watershed. The likely cause of this behavior is that any change to the 2-year pre-developed peak flow released by on-site detention basins are small relative to flows from other parts of the watershed, which are generated by the 100-year storm in the post-developed watershed.

SUMMARY AND CONCLUSIONS

In many communities across the United States, on-site detention basins are constructed at the outlets of subwatersheds without considering their integrated effect on the watershed as a whole. This practice is referred to as a Non-SD policy. Non-SD policies have been shown to be ineffective at reducing peak flows downstream of an on-site detention basin, and can actually increase peak flows. Our hypothesis was that a policy that regulates peak flow releases from an on-site detention basin as a function of its location within a watershed, referred to as an SD policy, will better reduce post-development peak flows and reduce overall detention volume requirements. To test this hypothesis, several on-site detention basin scenarios in the Mail Creek watershed, in Fort Collins, Colorado, were simulated using the U-McIUH model. The original version of the U-McIUH model could not simulate on-site detention basins. For this research the model was amended to include that capability by disaggregating the watershed at the location of simulated on-site detention basins and routing hydrographs downstream kinematically. A system of on-site detention basins was simulated by placing one at the downstream end of every 1st order channel or pipe in the watershed.

Two Non-SD policies were considered to gauge the SD policies. The first (Non-SD 1) controls the post-development 10 and 100-year design storm peak flows and releases at or below their respective pre-development peak flows. The second (Non-SD 2) controls the post-development 100-year design storm peak flow and releases at or below the pre-development 2-year storm peak flow. To establish a baseline for comparison, these two Non-SD policies were compared to a policy that uses no on-site detention basins. The two Non-SD policies were then transformed into eight easy to use SD policies using several variations of two general equations that govern peak flow release from on-site detention basins as a function of the flow distance of the on-site detention basin to the watershed outlet. The first SD equation was a linear model that decreased peak outflow from on-site detention basins in the upper half of the watershed and increased released peak flows in the lower half of the watershed. The second equation was a piece-wise linear model that operated similarly to the first in the lower half of the watershed, and as a Non-SD policy in the upper half. To evaluate SD policy performance, results from four variations of each of the two SD policies were compared to results of the two Non-SD policies.

The following general conclusions were reached:

1. Non-SD 1 and Non-SD 2 policies do not increase peak flows above that which would have occurred under a policy that does not use on-site detention basins. Non-SD 2 reduced peak flows throughout the watershed slightly more than Non-SD 1. Under both Non-SD policies, peak flows at the watershed outlet were reduced, and the reduction in peak flows at 2nd and 3rd ordered intersections exhibited large variability, while the peak flow reductions at 4th and 5th ordered intersections were more consistent and predictable.

2. SD policies can reduce peak flows in a watershed compared to a Non-SD policy. However, achieving those reductions requires a tradeoff between increasing peak flows in other locations of the watershed or increasing watershed storage. SD policies derived from Equation 1 primarily reduce peak flows at 2nd and 3rd order intersections in the upper half of the watershed, increase peak flows at 2nd order intersections in the lower half of the watershed, and increase watershed storage. Peak flows at 4th and 5th ordered intersections and the watershed outlet show little to no change. SD policies derived from Equation 2 slightly reduce watershed storage but increase peak flows at 2nd order intersections in the lower half of the watershed. 2nd order intersections in the upper half of the watershed and 3rd, 4th, and 5th order intersections show little to no change in peak flow. In the SD models, as the slope (ε) decreased and intercept (ω) increased, the relative change in magnitude of peak flow and watershed storage of the SD policy increased.

3. The ability of an SD policy to reduce the average peak flow relative to Non-SD 1 at channel and pipe intersections decreases as the order of the intersections increases. SD policies derived from Equation 2 produce larger relative reductions of average peak flows at all ordered channel and pipe intersections when compared to SD polices derived from Equation 1.

4. Compared to Non-SD 1, SD policies produce a large variability in peak flow reduction at 2nd and 3rd intersections in the upper half of the watershed, and at 2nd order intersections in the lower half of the watershed. That indicates that two 2nd order intersections that are roughly the same distance from the watershed outlet could show drastically different reductions in peak flow. Therefore, SD policies are not reliable for predicting peak flow reductions at a particular intersection.

5. When compared to Non-SD 2, SD policies have little to no effect on peak flows or storage.

RECOMMENDATIONS AND FUTURE WORK

For communities using Non-SD 2 as a stormwater control policy, the results of this study indicate that switching to an SD policy will not have a significant impact on peak flow or watershed storage. Of the four metrics used to evaluate SD policies, all four showed little to no difference between the SD policies and Non-SD 2, regardless of the properties of the equation that were used to define the SD policy. Therefore, we can conclude that switching to an SD policy that regulates peak flow based on the flow distance of an on-site detention basin to the watershed outlet will provide no benefit, and is not recommended. For communities that use Non-SD 1 as their stormwater control policy, switching to an SD policy will have both positive and negative consequences. Knowing the tradeoffs between them is important for watershed managers and decision makers contemplating the use of an SD policy. For SD policies based on Equation 1, reduced peak flows in the upper half of the watershed will lessen the potential of localized flooding and erosion in those areas. However, the resultant increase in peak flow at 2^{nd} order intersections in the lower half of the watershed and an increase in watershed storage might negate those positive aspects. For example, an increase in watershed storage will raise construction costs, and imposing a policy that requires on-site detention basins in the upper half of a watershed to be larger compared to those in the lower half might be considered unfair to developers. For SD policies based on Equation 2, watershed storage will be reduced and construction costs will be less. However, 2^{nd} order peak flows will increase in the lower half of the watershed, which will increase the potential for localized flooding and erosion. It is recommended that watershed managers and decision makers understand all aspects of an SD policy before considering its use.

Future research into the effectiveness of SD policies should test and evaluate the assumptions and limitations of this research. For instance, in this research, to make SD policies applicable to any watershed, locations of on-site detention basins were defined by a dimensionless parameter (ℓ_i/L) that defines the flow distance of a detention basin to the watershed outlet relative to the longest flow distance in the watershed. This ratio can be defined for any on-site detention basin in any watershed. But the location of a watershed's outlet is subjective, and its placement has direct consequences on ℓ_i/L ratios. For example, the outlet of the Mail Creek watershed is defined at the confluence of Mail Creek and Fossil Creek. If the watershed's outlet was instead defined at the intersection of College Avenue and Harmony Road (approximately 2.6 km upstream from the watershed's current outlet), the spatial dimensionality of the system of on-site detention basins would change. Each on-site detention basin's ℓ_i/L ratio would change because the distance to the watershed outlet and the distance from the most remote location in the watershed will have changed. How these changes would affect the results is unknown. Learning more about the sensitivity of the watershed outlet location, or using a method that does not have subjectivity in the spatial dimensionality of on-site detention basins in a watershed would benefit future research on SD policies.

On-site detention basins were assumed to be located at the most downstream end of every 1st order channel or pipe in the watershed. A consequence of this assumption is that only 54% of the watershed area drained to an on-site detention basin. Thus, 46% of the watershed had no on-site detention basins to contribute to the reduction of 100-year peak flows. Runoff from these unmanaged parts of the watershed could dwarf any effect SD polices have on peak flows at channel and pipe intersections, and could mask any beneficial effects of the SD policies. Future research should address the effects that SD policies have on watersheds that capture a greater percentage of the runoff. Rather than placing on-site detention basins at the most downstream end of all 1st ordered channels and pipes, they could be placed at all 2nd or 3rd ordered channels and pipes (similar to the approach of James et al. (1987)). Future research should also investigate the effects that SD policies have on storms other than the 100-year storm, which on average has a 1% chance of occurring annually. The more frequent storms that have a better chance of occurrence should also be used in testing SD policies (i.e. the 2-year storm).

Lastly, evidence was presented in Figure 17 through Figure 20 that an SD policy based on the flow distance from an on-site detention basin to the watershed outlet would be effective at some locations, but not at others. Investigation into why this occurs would be beneficial to SD policies.

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APPENDIX I

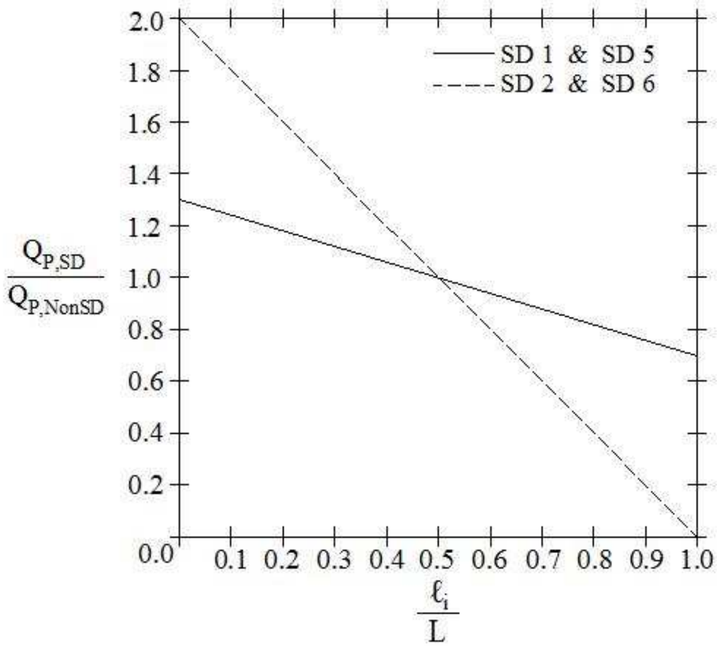

Figure 25: Alteration of the peak flow release from an on-site detention basin under a Non-SD policy to an SD policy for SD policies SD 1, SD 2, SD 5 and SD 6.

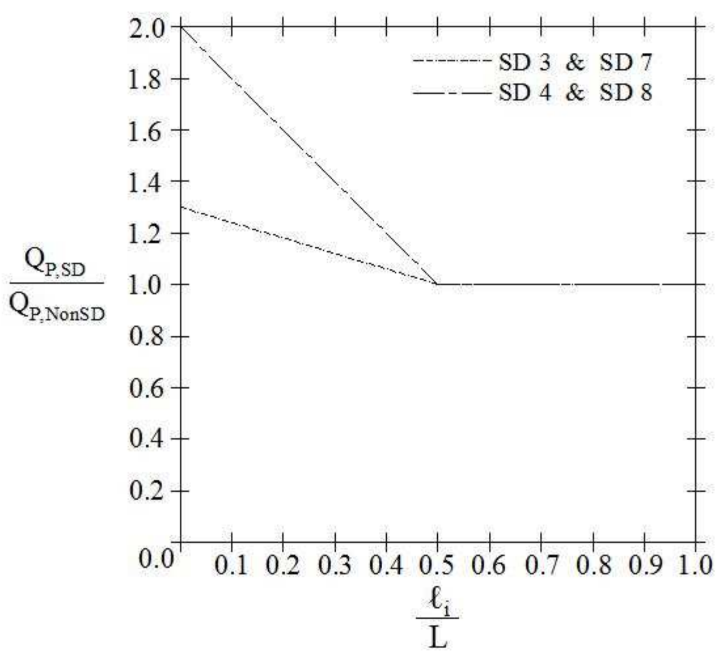

Figure 26: Alteration of the peak flow release from an on-site detention basin under a Non-SD policy to an SD policy for SD policies SD 3, SD 4, SD 7 and SD 8.

APPENDIX II

The Aubinière watershed (Figure 27) was chosen to study the effects of disaggregation and routing on the U-McIUH model because it was previously calibrated by Gironás et al. (2009). It has an area of 10.9 km^2 and an overall imperviousness of 31.7% (Gironás et al. 2009). The watershed was disaggregated at various locations along its channels and pipes. The points of disaggregation represent locations where detention basins could be installed. However, to isolate the effects of disaggregation and routing, no detention basins were represented at those locations. Subwatershed hydrographs generated by the U-McIUH were routed downstream through channels and pipes to the next location of disaggregation or the watershed outlet. Four scenarios for each routing method (kinematic and simple) were tested by disaggregating the watershed at two, five, 10 and 20 locations, displayed in Figure 28 through Figure 31, respectively. The hydrographs from the disaggregated scenarios were compared to the hydrograph derived from the non-disaggregated watershed.

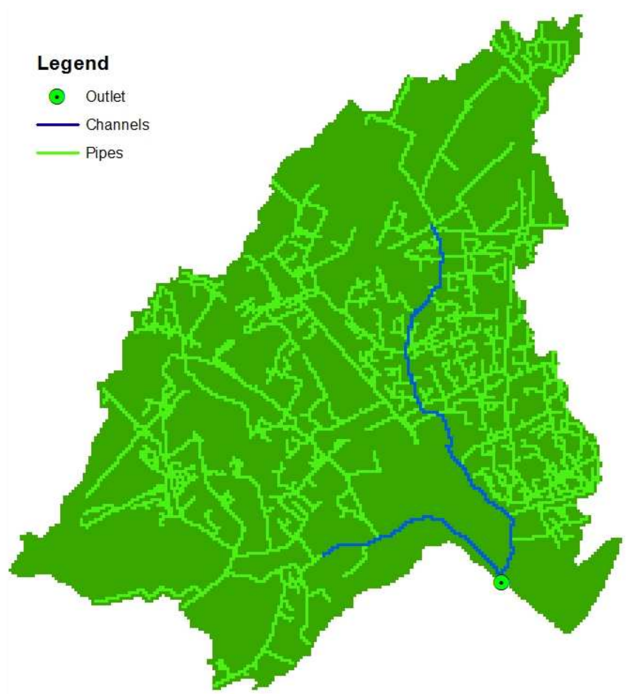

Figure 27: Aubinière Watershed.

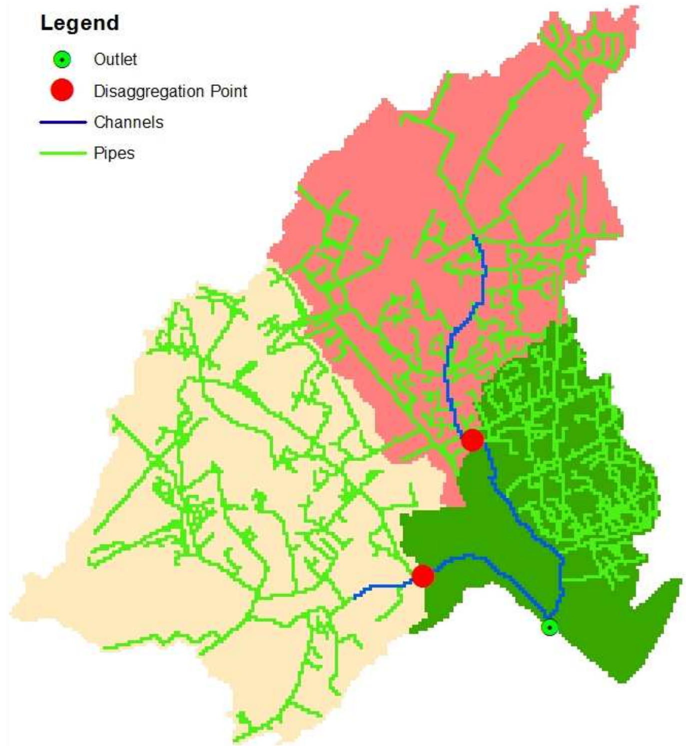

Figure 28: Two points of disaggregation in the Aubinière watershed.

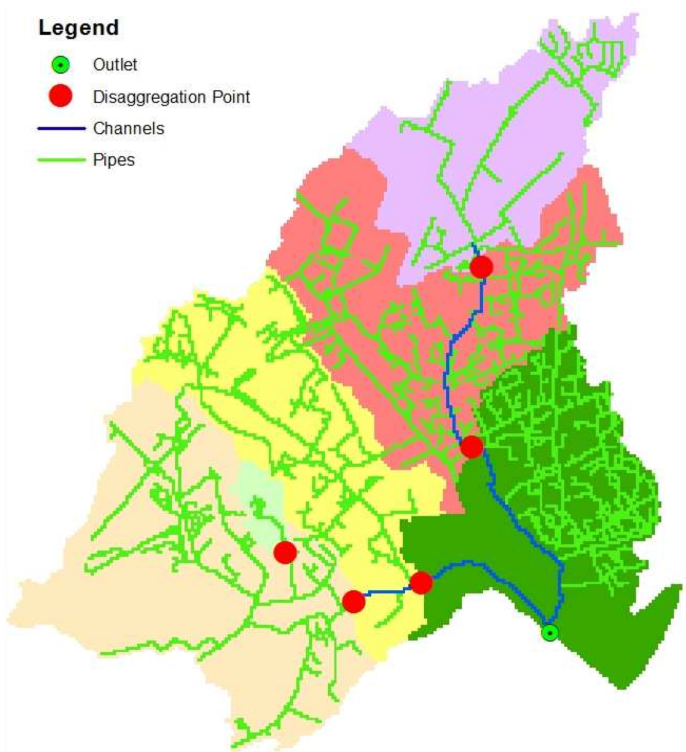

Figure 29: Five points of disaggregation in the Aubinière watershed.

Figure 30: 10 points of disaggregation in the Aubinière watershed.

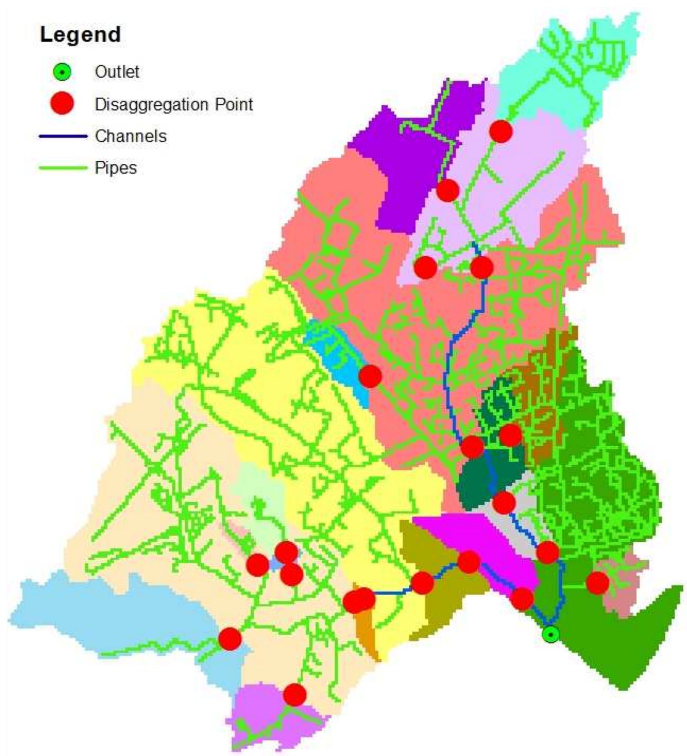

Figure 31: 20 points of disaggregation in the Aubinière watershed.

Four statistical measures were used to quantify the differences between the disaggregated and non-disaggregated outlet hydrographs: 1) the root mean square error (RMSE) (Equation 14), 2) the relative error in peak flow (REPF) (Equation 15), 3) the Nash-Sutcliffe coefficient of efficiency (NSCE) (Equation 16), and 4) the difference in time to peaks (Δt_p) of the disaggregated and non-disaggregated watershed outlet hydrographs (Equation 17).

$$RMSE = \sqrt{\frac{1}{N}\sum_{i=1}^{N}\left(Q_{ND,i} - Q_{D,i}\right)^2} \qquad \textbf{Equation 14}$$

$$REPF = \frac{\left|Q_{Peak,ND} - Q_{Peak,D}\right|}{Q_{Peak,ND}} \qquad \textbf{Equation 15}$$

$$NSCE = 1 - \frac{\sum_{i=1}^{N}\left(Q_{ND,i} - Q_{D,i}\right)^2}{\sum_{i=1}^{N}\left(Q_{ND,i} - \overline{Q_{ND}}\right)^2} \qquad \textbf{Equation 16}$$

$$\Delta t_p = t_{p,D} - t_{p,ND} \qquad \textbf{Equation 17}$$

N is the number of data points, i is the data point index, $Q_{ND,i}$ is the non-disaggregated flow of index i (m^3/s), $Q_{D,i}$ is the disaggregated flow of index i (m^3/s), $Q_{Peak,ND}$ is the non-disaggregated peak flow (m^3/s), $Q_{Peak,D}$ is the disaggregated peak flow (m^3/s), $\overline{Q_{ND}}$ is the average of all non-disaggregated flows (m^3/s), $t_{p,D}$ is the time to peak of the disaggregated watershed's hydrograph measured in hours (hr), and $t_{p,ND}$ is the time to peak of the non-disaggregated watershed's hydrograph (hr). RMSE and REPF values range from zero to infinity, zero being the best. NSCE values can range from negative infinity to one. An

NSCE value of one is a perfect fit between observed and modeled values, zero indicates the modeled values are as accurate as the average of observed values, and anything less than zero is not acceptable.

The U-McIUH assumes all upstream flow instantly contributes to a cell when its travel time is calculated (Gironás et al. 2009). A small storm will challenge this assumption more than a large storm and will have a greater influence on subwatershed hydrograph construction. Therefore, a precipitation event which fell on the Aubinière watershed on September 3, 2001 with a total depth of 2.24 mm was used to compare the hydrographs.

Figure 32 and Figure 33 compares the hydrographs of each level of disaggregation using simple and kinematic routing, respectively. Table 10 displays the statistical summary.

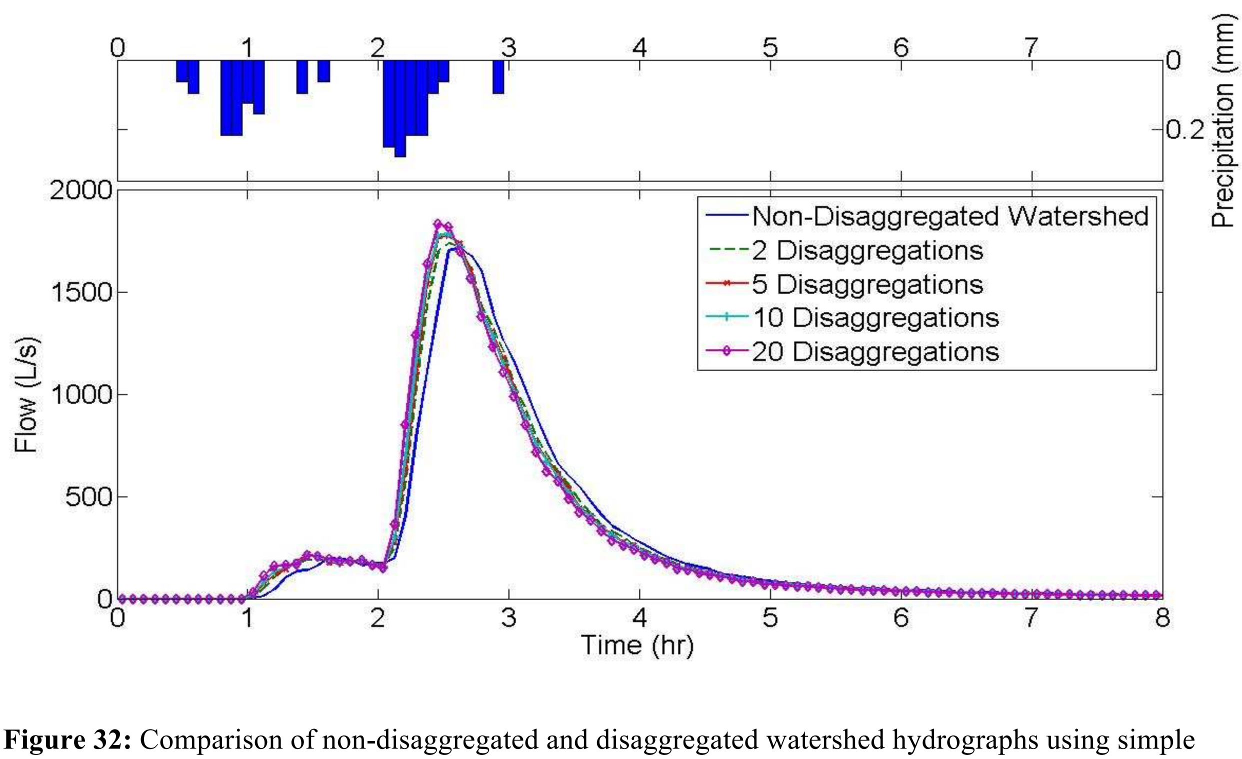

Figure 32: Comparison of non-disaggregated and disaggregated watershed hydrographs using simple routing to route disaggregated subwatershed hydrographs downstream.

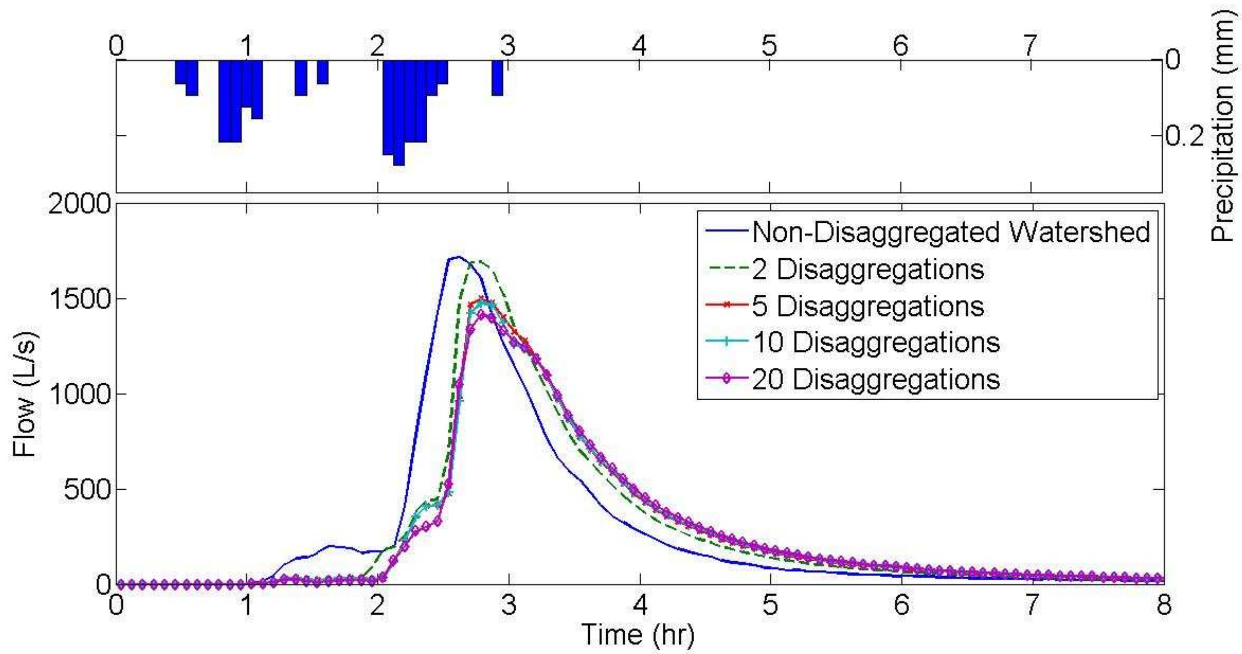

Figure 33: Comparison of non-disaggregated and disaggregated watershed hydrographs using kinematic routing to route disaggregated subwatershed hydrographs downstream.

Table 10: Statistical comparison of non-disaggregated and disaggregated hydrographs.

	2 Disaggregations				5 Disaggregations			
	RMSE (L/s)	**REPF**	**NSCE**	**Δt_p (hr)**	**RMSE (L/s)**	**REPF**	**NSCE**	**Δt_p (hr)**
Simple Routing	15.914	0.012	0.985	-0.08	20.788	0.033	0.974	-0.08
Kinematic Routing	48.329	0.015	0.858	0.08	57.954	0.127	0.796	0.17

	10 Disaggregations				20 Disaggregations			
	RMSE (L/s)	**REPF**	**NSCE**	**Δt_p (hr)**	**RMSE (L/s)**	**REPF**	**NSCE**	**Δt_p (hr)**
Simple Routing	23.032	0.039	0.967	-0.08	29.014	0.068	0.949	-0.17
Kinematic Routing	58.221	0.141	0.794	0.17	60.520	0.176	0.777	0.17

Regardless of the routing method, statistical performance of the disaggregated watershed hydrographs compared to the non-disaggregated watershed hydrograph decreased as the number of disaggregations increased. Although this is true for the simple routing method, it statistically outperformed kinematic routing. The RMSE and REPF of the simple method were routinely on an order of magnitude 2-3 times better than the kinematic routing method. This suggests simple routing is the best method for generating an outlet hydrograph from a disaggregated watershed in the U-McIUH model. But simple routing ignores any changes in the hydrograph's timing. An important aspect of this study is the timing characteristics of on-site detention basin peak flow releases and their interaction with other peak flows in the watershed. Therefore, simple routing was not chosen as the routing method. The statistical measures of performance for kinematic routing were judged to be acceptable, and was chosen as the method to route hydrographs downstream

LIST OF ABBREVIATIONS

DEM	Digital Elevation Model
EPA	Environmental Protection Agency
GIS	Geographic Information System
HEC-HMS	Hydrologic Engineering Center’s Hydrologic Modeling System
IUH	Instantaneous Unit Hydrograph
Non-SD	Non-Spatially Dependent On-Site Detention Basin
NRCS	Natural Resources Conservation Service
SD	Spatially Dependent On-Site Detention Basin
SWMM	Storm Water Management Model
UDFCD	Urban Drainage and Flood Control District
UDSWM	Urban Drainage Storm Water Management Model
UH	Unit Hydrograph
U-McIUH	Urban Morpho-climactic Instantaneous Unit Hydrograph