5-2003

A Sensitivity Analysis of the Influence of Watershed and Development Characteristics on the Cumulative Impacts of Stormwater Detention Ponds

Karen Marie Goff

University of Tennessee - Knoxville

To the Graduate Council:

I am submitting herewith a thesis written by Karen Marie Goff entitled "A Sensitivity Analysis of the Influence of Watershed and Development Characteristics on the Cumulative Impacts of Stormwater Detention Ponds." I have examined the final electronic copy of this thesis for form and content and recommend that it be accepted in partial fulfillment of the requirements for the degree of Master of Science, with a major in Environmental Engineering.

Randall W. Gentry, Major Professor

We have read this thesis and recommend its acceptance:

Gregory D. Reed, Terry L. Miller

Accepted for the Council:
Carolyn R. Hodges

Vice Provost and Dean of the Graduate School

(Original signatures are on file with official student records.)

To the Graduate Council:

I am submitting herewith a thesis written by Karen Marie Goff entitled "A Sensitivity Analysis of the Influence of Watershed and Development Characteristics on the Cumulative Impacts of Stormwater Detention Ponds." I have examined the final electronic copy of this thesis for form and content and recommend that it be accepted in partial fulfillment of the requirements for the degree of Master of Science, with a major in Environmental Engineering.

Randall W. Gentry
Major Professor

We have read this thesis
and recommend its acceptance:

Gregory D. Reed

Terry L. Miller

Accepted for the Council:

Anne Mayhew
Vice Provost and
Dean of Graduate Studies

(Original signatures are on file with official student records.)

A SENSITIVITY ANALYSIS OF THE INFLUENCE OF WATERSHED AND DEVELOPMENT CHARACTERISTICS ON THE CUMULATIVE IMPACTS OF STORMWATER DETENTION PONDS

A Thesis
Presented for the
Master of Science
Degree
The University of Tennessee, Knoxville

Karen Marie Goff

May, 2003

ACKNOWLEDGMENTS

I would like to express my appreciation to Dr. Gentry for serving as my major professor. His help was invaluable in completing this work. I would also like to thank Dr. Reed and Dr. Miller for taking the time to serve on my committee.

I would like to thank Chris Granju with the Knox County Stormwater Engineering Department and Nathan Silvus with AMEC Earth and Environmental Inc. for providing information needed to complete this work.

Finally, I would like to thank my family and friends for their encouragement. Most of all, I would like to thank my husband, Gary, for his patience and support which made this work possible.

ABSTRACT

Stormwater detention ponds are a popular stormwater management practice in many communities and city ordinances often require the uniform use of detention ponds on all new developments. Stormwater detention ponds are an effective method of controlling the peak flow rate immediately downstream from a development, but a number of detention ponds scattered at random locations throughout a watershed may not effectively control peak flows throughout the watershed.

A sensitivity analysis was conducted to determine correlations between watershed and development characteristics and the response of the watershed to the uniform use of detention ponds on all developments. The cumulative impacts of detention ponds were analyzed for sensitivity to two watershed characteristics: watershed shape and watershed slope. Peak flow impacts were also analyzed for sensitivity to four development characteristics: development size, development intensity, development stage, and development sequence. The sensitivity analysis was conducted by modeling the cumulative effects of detention in watersheds with various combinations of these characteristics.

Synthetic watersheds were used for the sensitivity analysis in order to produce general results and conclusions that can help evaluate the potential for adverse peak flow impacts in any watershed, rather than being specific to a particular watershed. The use of synthetic watersheds also provides a controlled environment that allows the effects of specific variables to be pinpointed. The synthetic watersheds for this analysis were developed using network topology. Following the sensitivity analysis using the synthetic watersheds, a “real-world” test watershed was modeled as a means of evaluating the applicability of the findings of the sensitivity analysis to an actual watershed. The Ten Mile Creek watershed in Knox County, Tennessee was used as the test watershed.

Of the six watershed and development characteristics considered in the sensitivity analysis, watershed shape, the percent of the watershed that was developed, and the location of the developed areas within the watershed had the

greatest effect on the cumulative impacts of detention ponds in the watershed. These three factors determined the pattern of impacts that occurred within a watershed. Development intensity, development size, and watershed slope contributed to the magnitude of the impacts which were created, but they were not the overriding factors that determined the pattern of impacts.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF TABLES - CONTINUED

LIST OF FIGURES

CHAPTER 1
INTRODUCTION

Stormwater detention ponds are a popular stormwater management practice in many communities. According to a survey conducted in 1997 of representatives from communities across Tennessee, 75 percent of Tennessee communities use detention ponds as a stormwater management tool (Gangaware et al. 1997). City ordinances often require the uniform use of detention ponds on all new developments. The uniform use of onsite detention is an attractive stormwater management option for many communities because it is easy to implement and enforce, and there is a sense of fairness among developers (Debo 1982). Furthermore, a uniform on-site detention policy means that the costs of stormwater management are paid by the developers rather than by the community, and the policy is flexible enough to accommodate unplanned development (Shea 1995). On-site detention policies typically require that detention ponds be designed so that the peak flow rate exiting each development is no greater than the peak flow rate that would have occurred at the development outlet before the development took place, for one or more design storms. Stormwater detention ponds designed in accordance with such policies can be an effective method of controlling the peak flow rate immediately downstream from a development.

There is a perception that if all developments release only pre-development peak flows, then the peak flows in the downstream reaches of the watershed will be held to pre-development levels as well. However, on-site detention ponds are typically designed without regard to their potential impact on downstream areas and in reality, a number of detention ponds scattered at random locations throughout a watershed may not effectively control peak flows throughout the watershed. The timing of the outflow from a detention pond is very different than the timing of the pre-development hydrograph. The peak discharge from a detention pond typically occurs later than the pre-development peak and because of the increased volume of runoff caused by development, higher flows extend for a longer duration. Because of these changes in timing, the outflow hydrographs from detention ponds may combine

differently than the pre-development hydrographs. As a result, the combined effects of detention ponds randomly located throughout a watershed can actually cause peak flows to increase at some points in the watershed, aggravating the flooding problems they intend to prevent.

The recognition that the combined effects of detention ponds can have adverse impacts downstream has led a number of researchers to suggest a reevaluation of the criteria used to evaluate the effectiveness of detention. Effectiveness should consider overall watershed impacts, rather than only considering the effectiveness at the outlet of the development (McCuen 1979; Duru 1981; Lakatos and Kropp 1982; Traver and Chadderton 1992a). While it is generally accepted that watershed level planning will control runoff more effectively than a site-by-site approach, it can be impractical for communities to evaluate the potential cumulative impacts of detention ponds in every watershed because development often occurs in an unplanned manner (Shea 1995), because of the costs involved in conducting a complete hydrologic analysis of a watershed (Whipple et al. 1987), because many municipal and county agencies lack the technical expertise required for watershed level planning (Whipple et al. 1987), and because of the coordination required between local governments (Nix and Jolley 1995). As a result, communities are often forced to hope for the best when implementing an ordinance that requires the uniform use of detention ponds on all new developments. For example, the Land Development Manual for the City of Knoxville, Tennessee (2002) states:

> “It should be noted that the conglomerate effect of dozens of detention basins in a watershed may or may not reduce peak flows at a downstream location. This uncertainty is caused by factors such as the infinite types and variety of actual rainfall distributions, spacing and sizes of the detention basins, discharge characteristics for the detention basins, maintenance and conveyance of major drainage channels."

A number of studies have investigated some of the factors that affect the overall efficiency of detention in controlling the peak flows throughout a watershed, such as the number and location of detention ponds, watershed shape, stage of development in the watershed, development intensity, development sequence,

detention design characteristics, and nonstandard rainfall distributions (Shea 1995; Ahmed 1995; Traver and Chadderton 1994; James et al. 1987; Sonnenberg and Fiuzat 1986; Duru 1981; Malcolm 1980; and others). However, most of these studies consider only two or three factors and all of the studies are specific to different watersheds. The watersheds all have different characteristics and each study was conducted with different underlying assumptions, so the results are difficult to compare. There is not a systematic understanding of how the interaction between these factors will affect the cumulative impacts of detention ponds in any given watershed.

Additional study is needed to help communities be able to predict the cumulative effects that detention ponds may have in a particular watershed. By gaining a better understanding of how the interaction between watershed and development characteristics will affect the cumulative impacts of detention ponds in a watershed, communities will be better able to identify combinations of watershed and development characteristics that may produce adverse peak flow impacts at some watershed locations.

Purpose and Scope

The purpose of this thesis is to determine correlations between watershed and development characteristics and the response of the watershed to the uniform use of detention ponds on all developments. A sensitivity analysis was conducted to evaluate the influence of six watershed and development characteristics on the cumulative impacts of detention ponds. Peak flow impacts were analyzed for sensitivity to two watershed characteristics: watershed shape and watershed slope. Peak flow impacts were also analyzed for sensitivity to four development characteristics: development size, development intensity, development stage, and development sequence. The sensitivity analysis was conducted by modeling the cumulative effects of detention in watersheds with various combinations of these characteristics.

Synthetic watersheds were used for the sensitivity analysis in order to produce general results and conclusions that can help evaluate the potential for adverse peak flow impacts in any watershed, rather than being specific to a particular watershed. The use of synthetic watersheds also provides a controlled environment that allows the effects of specific variables to be pinpointed. This approach has been used by previous researchers (James et al. 1987; Sonnenberg and Fiuzat 1986; Bedient et al. 1982). The synthetic watersheds for this analysis were developed using network topology. Network topology is based on geometric similarity between watersheds, allowing conclusions drawn from one watershed to be applied to other similar watersheds. As a result, the conclusions drawn from the synthetic watersheds are applicable to real-world watersheds.

Following the sensitivity analysis using the synthetic watersheds, a "real-world" test watershed was modeled as a means of evaluating the applicability of the findings of the sensitivity analysis to an actual watershed. The Ten Mile Creek watershed in Knox County, Tennessee was used as the test watershed.

The specific objectives of this study were:

1) To develop synthetic watersheds using network topology.

2) To model the cumulative effects of detention ponds on peak flows at selected locations in synthetic watersheds with different physical and development characteristics.

3) To evaluate which watershed and development characteristics have the greatest impact on the overall effectiveness of detention ponds at the watershed level in a watershed where detention ponds are required on all developments.

4) To identify combinations of watershed and development characteristics that may be likely to produce adverse peak flow impacts at some watershed locations in a watershed in which detention is required on all developments.

5) To evaluate the applicability of the findings of the sensitivity analysis to a "real-world" test watershed, the Ten Mile Creek watershed in Knox County, Tennessee.

CHAPTER 2
LITERATURE REVIEW

The effectiveness of detention as a means of managing urban stormwater is discussed extensively in the literature. McCuen (1974) was one of the first to demonstrate that the use of detention to control the discharge from individual subwatersheds can actually create flooding problems. When numerous small detention basins were located on tributaries near the watershed outlet, the peak flow at the watershed outlet increased because the runoff from the lower portions of the watershed was delayed so that it combined with the runoff from upper portions of the watershed. A subsequent study (McCuen 1979) examined the effects of a single detention basin on peak flows at downstream locations in the watershed. McCuen again found that because detention changes the timing characteristics of the runoff, peak flows increased at some downstream points in the watershed. He summarized his findings by stating that the detention basin "causes increases in downstream flooding and does not mitigate the effect of development".

Based on his study findings, McCuen (1979) questioned whether detention basins were effectively meeting the intent of stormwater management:

> "There is little doubt that SWM basins will appear to be effective when effectiveness is measured using a very limited criterion, such as the on-site control of peak discharge of a single return period. …a SWM alternative that adequately controls flow and sediment rates at the site of development cannot be considered effective when either flow rates or sediment volumes increase at sites downstream. …the effectiveness of SWM must be measured on a regional basis rather than using an on-site criterion alone."

Since McCuen's studies in the 1970's, numerous additional studies have addressed the effectiveness of detention ponds at a watershed level. A number of researchers have investigated some of the factors that play a role in determining the effect detention will have on peak flows at downstream watershed locations. Other researchers have taken a watershed-level approach to planning detention systems by developing computer programs to optimize the size and location of detention in a

watershed. Still others have proposed alternative policies for uniform on-site detention that are intended to minimize or eliminate adverse impacts downstream. Studies that have been conducted in each of these areas are discussed in the following sections.

Effects of Detention on Downstream Peak Flows

Malcolm (1980) studied the effects of detention on discharge at downstream locations by modeling two North Carolina watersheds, one 360 acres in size and one 18.5 acres in size. Malcolm also considered a 320-acre hypothetical watershed to examine how the location and amount of storage in relation to the size of the watershed influences how effective the detention will be in reducing peak flows. A general conclusion reached by Malcolm is that depending on the location of detention, "the interaction of storage facilities with one another can be such that they are mutually supportive, damaging or that they counteract each other so as to have no discernable effect". The study found that in general, large detention facilities on the main stream near the watershed outlet are more effective than small detention facilities distributed throughout the watershed. The study also found that "the effect on peak flow of including storage distributed broadly at a small scale in the watershed is lost a short distance downstream from each detention facility".

Duru (1981) showed that the effects of on-site detention are governed by the physical features of the watershed. He modeled a 44 square kilometer watershed in Maryland and analyzed the hydrographs at several stream cross sections. The results of the study showed that on-site detention can "either aggravate or mitigate flood conditions… each outcome was determined by the interaction of detention basins and the physical characteristics of the watershed". Duru found that detention was most effective at reducing watershed peak flows when the detention basins were sited in only the upper half of the watershed because the peak from the upper portion of the watershed arrived after the peak from the downstream portion of the watershed had passed on, producing two distinct hydrograph peaks. Duru explained the two peaks

that were produced by the fact that the bottom portion of the watershed has considerably steeper slopes than the upstream portion of the watershed. When detention was used in the lower half of the watershed, the peak flows from the upper and lower halves of the watershed coincided to produce one large peak. Duru suggested that for a watershed in which the upper portion had steeper slopes than the lower portion, the watershed would likely produce a single peak hydrograph and detention would likely be more effective because it would separate the peaks from the two portions of the watershed. According to Duru, however, the results depended on the watershed characteristics "with no possible general rule as to where on-site detention should be located on every watershed". Duru concluded by stating that "only through careful hydrologic analysis can any intelligent decision on the use and location of SWM detention basins be made".

Lakatos and Kropp (1982) discussed the need for "real data" to "document the occurrence of downstream impacts of detention basins". Lakatos and Kropp conducted a search for such data, but no actual gaging data could be found that was collected "to identify the potential for adverse downstream impacts resulting from the use of upstream detention". They stressed the need for actual monitoring in order to clearly identify the problem. In the absence of such data, they pointed to the usefulness of simulation studies to identify potential impacts and provided an overview of a study which they conducted to document the problem. Two of the conclusions reached by Lakatos and Kropp are that "on-site detention of runoff, while possibly being an effective means of runoff control for an individual site, may cause an increase in the watershed peak runoff rate that is directed toward downstream properties" and that detention in the lower portion of the study watershed "only serves to detain downstream flows to combine with upstream flows". Lakatos and Kropp also described an alternative detention policy, the "Release Rate Percentage Concept", which is discussed below.

Bedient et al. (1982) used a detention basin optimization model (DBOPT) to determine the optimal size and location of detention storage within three 10 square mile hypothetical watersheds. The DBOPT model is discussed further below. The

three 10 square mile watersheds were developed by combining 10 one square mile unit catchments into three different watershed shapes: concentrated, medium, and elongated. The study found that detention storage was more effective when planned at the watershed level rather than the catchment level. For all watershed shapes it was determined that detention storage was not needed at the most downstream locations and that "detention storage should be located in the upper 80% portion of the watershed". The study also found that for the one square mile catchments, the maximum hydrologic impact occurred when the catchments were between 50% and 75% developed.

Traver and Chadderton (1983) modeled a 5.98 square mile watershed in Pennsylvania to examine the effects of urbanization on peak flows at the watershed outlet. Traver and Chadderton concluded that:

> "Storm Water Management detention basins only are effective in eliminating increased urbanization flow at the point of design. Under current design procedures, the downstream effects of these detention basins are variable, either increasing or decreasing the post-construction flows. Due to changes in time to peak, extenuated peaks, raised recession limbs, and increased total runoff, significant increases in peak flow can occur downstream from the controlled areas."

Traver and Chadderton also concluded that effectiveness is improved by selective basin placement, but acknowledge that "placement criteria were determined only by complete modeling of the watershed".

Lee (1985) considered the effects of temporal and spatial rainfall variations on a 650 acre hypothetical watershed. Lee found that both temporal and spatial rainfall variations can cause increased peak flow rates at the watershed outlet.

Sonnenberg and Fiuzat (1986) used a simple, idealized, 6,000 acre rectangular watershed to study how the detention design criteria can affect the peak discharges for the watershed. The idealized watershed was developed using average data from eleven watersheds in the Greenville, South Carolina area. Sonnenberg and Fiuzat found that "the effects of using detention facilities is dependent upon the criteria used to design the individual facilities and the location within the watershed". They noted that the Design Holding Time, or the length of time that detention facilities hold

water, can be a good indicator of the influence a detention basin will have on the watershed. They suggested that the impact a detention basin has on the rest of the watershed can be minimized by setting the Design Holding Time or the Design Time of Concentration equal to the total watershed time of concentration or the flow time from the bottom of the watershed. Sonnenberg and Fiuzat concluded that "the use of detention facilities can reduce peak watershed discharges for some watersheds, but depending on the design criteria, location, timing relationship to the watershed, and the size of the watershed, they can also increase peak discharges".

James et al. (1987) developed a 6,400-acre hypothetical fifth-order watershed based on network topology equations relating stream length, drainage area, stream slope, and stream order. They noted that "the relative size and shape of the sub-basins within the watershed and the relative length and slope of the stream channels will influence the effectiveness of detention ponds for reducing the flood peaks". Five alternative combinations of detention ponds were considered. Detention was located on either all first-order channels (256 ponds), on all second-order channels (64 ponds), on all third-order channels (16 ponds), on all fourth-order channels (4 ponds), or on the main channel (1 pond). They also considered whether the channels were natural or hydraulically improved.

For each combination of detention ponds, the effectiveness in reducing peak discharges on the fifth-order channel was determined for natural, partially improved, and improved channels (James et al. 1987). The results showed that for each combination of detention ponds, the improved channels were the most effective. For each channel type, effectiveness increased as the stream order increased and the number of ponds decreased. The study also found that detention ponds are effective for further distances downstream if the channels are hydraulically improved. The authors also noted that channel improvements "will affect the amount and location of detention storage" and that "the amount of detention storage can be significantly reduced by selective location of detention facilities within the watershed". In watersheds with hydraulically improved channels, the use of several smaller detention ponds in the upstream portions of the watershed are favored over fewer larger

downstream ponds, because the channel improvements below the detention ponds will increase the allowable release rate and reduce the total amount of storage required. One important conclusion reached by the authors is that:

> "The concept of reducing the outflow from a detention pond to the predevelopment peak discharge has little merit in sizing most detention facilities. To be effective in reducing the peak discharge on the larger channels, when small detention ponds are used, it will usually be required that the peak outflow be reduced to less than the predevelopment peak discharge."

Claycomb (1988) investigated the effectiveness of basin wide on-site detention in reducing peak discharges in downstream portions of a large drainage basin. The study used a 4.5 square mile watershed in Colorado which was "reproduced and connected in various configurations" to form a theoretical study area of 18.18 square miles containing 124 sub-basins, each with its own detention facility. Claycomb stated that "two primary basin configurations were utilized to minimize the probability that conclusions were related specifically to the basin configuration". Claycomb also evaluated three different storage-outflow relationships to determine if the study conclusions were dependent on that relationship. The results of the study show that "the reduction of peak runoff rates in downstream portions of a large drainage basin can be expected to be significantly less than that for individual sub-basins". Claycomb went on to say that "in all cases analyzed, the effectiveness of uniform on-site detention with regard to reducing peak flows in the main channel becomes progressively less as one proceeds downstream".

Sloat and Hwang (1989) modeled a 2.21 square mile watershed in California to examine how maintaining the stream channel as a greenbelt versus lining the channel with concrete influences the effectiveness of detention. The results of the study showed that maintaining the channel as a greenbelt helped to control peak flows at the watershed outlet. Because the greenbelt condition helped keep the flow velocity near the predevelopment velocity, travel times through the channel were longer allowing the peak runoff rates from downstream subareas to enter the channel before the upstream peak arrived. The results of their analysis also showed that "the combined effect of the several subarea hydrographs in the study area generally

produced a peak flow rate at the watershed collection point that was greater than the predevelopment peak flow rate". This occurred despite the fact that each individual detention basin was found to effectively control the post development peak flow rates at the detention basin outlet. Sloat and Hwang reported that the reason for this was "the increase in volume of direct runoff resulting from the land development and the timing of flow rates throughout the watershed". Sloat and Hwang also noted that the effectiveness of detention was dependent on its location in the watershed, with detention being least effective when sited in the lower portion of the watershed.

Traver and Chadderton (1992a) modeled a 5.98 square mile watershed in Pennsylvania to examine the accumulation effects of detention basins. A decrease in detention basin efficiency was observed as the routing proceeded downstream. The results of the study indicated an apparent linear relationship between the watershed contributing area and the decrease in effectiveness. A subsequent study (Traver and Chadderton 1992b) confirmed these results. The second study continued the investigation by examining the effects of developing six remaining subareas that were not developed in the first study. The results showed that "as additional developed subareas are included in the contributing area (whether by urbanization or by incorporating more area by moving downstream), the overall efficiency of the basins decreases and the peak flow increases". A second part of the study modeled the effects of developing only upstream subareas, leaving lower subareas undeveloped. The results indicated that "with substantial travel downstream, the upper watershed accumulation effects dissipate if no added peaks coincide with the major stream peak".

Traver and Chadderton (1994) investigated the effects of nonstandard rainfall distributions on the effectiveness of detention basins. An early peaking rainfall pattern and a late peaking rainfall pattern were developed based on the central peaking SCS type II distribution. Each of the three rainfall distributions was modeled for pre-development, post-development, and post-development conditions with detention basins designed for the central peaking event. The results show that

nonstandard rainfall patterns do affect the efficiencies of detention basins in controlling peak flows at the watershed outlet.

Shea (1995) conducted an in-depth study of the impacts generated by detention, specifically detention basins designed so that the peak flow from a developed site does not exceed the peak flow that would have occurred before development took place. Shea referred to this design policy as the "Subbasin Pre-post Match". Shea categorized the types of adverse peak impacts that can occur and explained how each type is generated. The impacts were categorized as follows:

1. "A junction hydrograph impact occurs at a junction point in the channel network where total hydrographs from two subbasins are combined;"
2. "A total hydrograph impact occurs in an interior subbasin where the subbasin hydrograph is combined with an upstream hydrograph that has been routed through the subbasin; and"
3. "A downstream hydrograph impact occurs at a junction point one or more channel links downstream of a developed subbasin."

Shea noted that a fourth type of impact, a cascade, occurs when "a peak increase propagates downstream through the channel network from the location where an impact is first generated".

A model developed specifically for the study was then used to project the location and distribution of the impacts caused by development of a single subbasin using a detention basin designed with the "Subbasin Pre-Post Match" policy (Shea 1995). The study watershed was located in Florida and consisted of 285 subbasins. Shea stated that "in the majority of cases adverse peak impacts were created by development of a single subbasin". However, increases over the predevelopment peak were generally small. The results also showed that the development of an interior subbasin, or a subbasin that has upstream subbasins, "is more likely to create an adverse peak impact than development of exterior subbasins".

Shea (1995) investigated the impacts of detention further by looking at the effects of fully developing a watershed using the "Subbasin Pre-post Match" policy for each individual subbasin. The analysis findings included the following:

1. "Full development of the drainage basin produces both reductions and increases in peak discharges."

2. “Reductions in peak discharges occur at the junction points between two exterior subbasins and are caused by differential delays in the time to peak of detention hydrographs.”
3. “Increases in peak discharges are generated by increased runoff volume and accumulate in the downstream direction. The largest increases are generally found at or near the drainage basin outlet.”
4. “The size of increases in peak is related to the amount of time between the subbasin hydrograph peak and the local or total hydrograph peak.”

Shea (1995) also looked at the effects of whether the watershed is fully or partially developed. For the fully developed scenario, peak flow impacts were analyzed for sensitivity to development intensity, the design return period, the detention outlet type, and the hydrograph method. Development intensities of 0, 20, 38, 55, 72, and 85 percent were considered. The results showed that “peak increases grow larger with increasing development intensity”. Variation in the design return period was not found to have a significant effect. For the outlet type, orifice outlets were found to produce significantly greater increases in peak flows than weir outlets. The SCS Dimensionless Unit Hydrograph method was found to be more conservative than the Santa Barbara Unit Hydrograph method because it generated greater increases in peak flows.

For the partial development scenario, Shea (1995) considered the effects of development sequence on peak flow impacts. When development proceeded from upstream to downstream, the results were found to strongly resemble those of full drainage basin development, with increases in peak flow growing larger as development proceeds. When development proceeded from downstream to upstream, a very different pattern of impacts was generated. As each subbasin was developed it was found that “an impact is created at the point of development and downstream impacts are aggravated”. Finally, random development was considered, evaluating the effects of all possible combinations of subbasin development for drainage basins of magnitude 3, 4, and 5. The results show that “there is a steady increase in the percent of scenarios that generate peak increases as the magnitude of the drainage basin increases… the size of the largest peak increase also increases with drainage basin magnitude”. An important observation made by Shea is that:

> "there is a significant percent of the possible scenarios that generate peak increases greater than the peak increase associated with full development… this means that alternative stormwater management regulations must be evaluated against the full range of possible development sequences… in order to ensure that they are effective."

Ahmed (1995) conducted an examination of available detention alternatives in order to select an appropriate detention policy for the city of Bettendorf, Iowa. Two representative watersheds within the city were used to evaluate the effects of different policies. The study found that the watershed shape has a significant effect on the overall effectiveness of detention. In a watershed with a classic dendritic shape, the pre-development peak flows were maintained at some locations in the watershed. The effectiveness of the detention decreased as the watershed shape changed to long and narrow. Another finding was that the use of on-site detention "yields greater than desired flow in the receiving channel, as discharge from individual detention basins accumulates in a downstream progression". The study also found that the use of on-site detention in only the upper half of the watershed could "produce peak flow attenuation comparable to that realized by detention over the entire watershed".

Optimization Models

Mays and Bedient (1982) developed the DBOPT model for determining the minimum cost sizes and locations of detention basins within a watershed. By placing a constraint on the channel capacity linking the detention basins, the model maintains undeveloped peak flows at all watershed locations. Ormsbee et al. (1987) took the idea a step further by including water quality considerations. Shea (1995) built on the initial work of Mays and Bedient and the subsequent modifications by Ormsbee et al. to develop the Basin Wide Optimization Model (BWOP). An interesting aspect of Shea's work was an analysis of how errors in development projections and development sequence affect the success of BWOP solutions. The analysis indicated that if development of the watershed does not occur in the exact way that was projected when the modeling was done, then adverse impacts may occur. Particularly

important to the success of BWOP solutions are the sequence of development (downstream to upstream, for example) and whether development actually occurs in all subbasins that were predicted to develop. More recently, Yeh and Labadie (1997) applied successive reaching dynamic programming (SDRP) and a multiobjective genetic algorithm (MOGA) to the "integrated, watershed-level planning of storm water detention systems under multiple objectives."

Although these are very powerful methods, a number of drawbacks have been identified. Shea (1995) noted that "they require that the sizes and locations of all development to be known before any development occurs… they are not able to identify optimal individual detention basin designs for development that occurs sporadically or in an unplanned manner." Yeh and Labadie (1997) noted that watershed level approaches to planning detention systems "require the existence of regional regulatory agencies with authority to promote the implementation of watershed-level planning." They went on to cite political and legal hindrances and technical difficulties. Nix and Jolley (1995) stated that the implementation of watershed-level planning of detention can be difficult because it requires a coordinated effort between local governments. Whipple et al. (1987) agreed that watershed level planning will control runoff more effectively than the "conventional site-by-site approach", but noted that such planning "is not often used because of the large up-front cost of preparing watershed plans in advance of development." They also acknowledged that many municipal and county agencies do not have the technical expertise required for watershed level planning.

Alternative Policies for Uniform On-Site Detention

Lakatos and Kropp (1982) developed the Release Rate Percentage Concept. This concept was developed as part of a pilot watershed-level stormwater management plan for Allegheny County, Pennsylvania. The concept was developed "as a practical solution to the problem of determining the amount that can safely be released from a subbasin within a watershed in order not to cause adverse

downstream runoff impacts." The key to the Release Rate Percentage Concept is determining, under pre-development conditions, how much runoff a particular subbasin is contributing to the overall watershed peak. After development, the detention release rate must be limited to that contributing rate, rather than to the total pre-development discharge. Lakatos and Kropp illustrated the concept with an example. Prior to development, the peak runoff rate from a given subbasin is 500 cfs. However, due to the timing of the watershed, only 400 cfs is contributed to the peak at the watershed outlet. Therefore, after development the detention release rate must be limited to 400 cfs, rather than 500 cfs as would be normally be required.

Shea (1995) developed five alternative detention basin sizing policies. Each policy has the intent of maintaining stormwater peaks throughout the watershed at predevelopment levels. These policies were developed based on a detailed "examination of the mechanisms by which the Subbasin Pre-post Match creates downstream impacts" including "identification of the characteristics of development sites that are conducive to generating impacts". According to Shea (1996), an improved detention basin design policy should satisfy the following criteria:

1. "Prevent increases in peak discharges over entire drainage basin"
2. "Produce detention basin designs that are independent of sequence or pattern of development"
3. "Have the capability to be implemented on a site-by-site basis"
4. "Minimize the total drainage basin costs involved with construction of individual detention basins"
5. "Design standards for potential development locations should be established prior to any new development and they should be easily understood by the development community"
6. "The policy should be equitable. If variable detention basin design requirements are used, then the size of individual detention basins should relate directly to the impacts generated by individual development sites."

Shea (1996) compared each proposed policy "in terms of practicality, effectiveness, and cost efficiency". The two most promising methods were found to be the Peak Partitioning method and the Percent Reduction in Developed Peak method. The Peak Partitioning method is similar to the Release Rate Percentage Concept developed by Lakatos and Kropp (1982). When Peak Partitioning is used,

“the contribution from a development to post development peaks at all downstream locations are limited to the contributions made to downstream peaks under original (predevelopment) conditions.” Using a peak partitioning algorithm developed by Shea (1995), individual developments are assigned a peak reduction factor for post development conditions. The benefits of the Peak Partitioning method are that it prevents impacts regardless of the development sequence and it prevents impacts from occurring while the watershed is being developed. Also, the method is equitable, requiring larger detention basins for developments with the largest potential to cause impacts. However, the method does “result in high costs for some individual developments” (Shea 1996).

The Percent Reduction in Developed Peak method simply requires all developments to limit the post development peak to some percentage of the predevelopment peak (Shea 1996). The percent reduction appropriate for a particular watershed must be determined through a hydrologic and hydraulic analysis of the watershed. The benefit of the method is its simplicity. Drawbacks to the method are that it requires an initial watershed analysis and it does not prevent impacts effectively while the watershed is still developing (Shea 1996)

Another approach to minimizing the potential for downstream impacts is to develop a policy that requires the impacts from a detention pond to be analyzed for some specified distance downstream. In a study conducted for Greenville, South Carolina, Debo and Reese (1992) determined downstream limits for the analysis of detention ponds. These limits represent the location where the effects of the detention facility on the downstream drainage system stabilize. Studying a 640-acre watershed in the Greenville area, Debo and Reese found that:

> “Where the proposed development represents 10 percent of the total drainage area to the downstream analysis location, the effects of the development and the detention facility stabilize and remain relatively constant as the development becomes an increasing smaller part of the total drainage area.”

The analysis was then extended to six different watersheds at various locations across the United States. The watersheds represented a range of sizes, shapes, and

topography. In each case, the effects of the detention facility stabilized at or before the 10 percent location. The actual distance required to reach the 10 percent location depended on the size of the development and its location within the watershed (Debo and Reese 1992). The results of the study were used to develop the following ordinance provision (Debo and Reese 1992):

> "In determining downstream effects from stormwater management structures and the development, hydrologic-hydraulic engineering studies shall extend downstream to a point where the proposed development represents less than ten (10) percent of the total watershed to this point."

This provision was incorporated into the Greenville, SC stormwater ordinance and ordinances in several other cities.

Summary

A number of studies have evaluated the effectiveness of detention in controlling peak flows at downstream points in the watershed. The results of these studies demonstrate that the indiscriminate use of on-site detention throughout a watershed can cause increased downstream peak flows. Researchers agree that the true effectiveness of detention must consider the overall watershed impacts, even though detention may appear to be effective immediately downstream from a development.

Several studies have sought to determine some of the factors that affect the overall efficiency of detention in controlling the peak flows throughout a watershed. However, most of these studies consider only 2 or 3 factors and all of the studies look at different watersheds. The watersheds all have different characteristics and each study was conducted with different underlying assumptions, so the results are hard to compare. There does not seem to be a systematic understanding of exactly what factors determine the effect a detention pond will have on the rest of the watershed. Currently, a complete hydrologic analysis of a watershed is necessary in order to determine what effect detention will have in that watershed.

Several models have been developed that optimize the size and location of detention so that adverse impacts are avoided. These models make it possible to design a system of detention basins that will not create adverse impacts anywhere in the watershed. However, there are a number of drawbacks to these methods that may make their use impractical in most cases, including the need for watershed level planning before development occurs, the costs involved in conducting a complete hydrologic analysis of a watershed, the technical expertise required for watershed level planning, and the coordination required between local governments. Other research has focused on the development of alternative detention policies that minimize adverse impacts while remaining practical for a community with limited resources.

CHAPTER 3
METHODS

Organization of the Sensitivity Analysis

Based on the literature review, six watershed and development characteristics were selected for the sensitivity analysis. The watershed characteristics that were included in the analysis are watershed shape and watershed slope. The development characteristics that were included in the analysis are development size, development intensity, development stage, and development sequence.

Watershed shape and watershed slope are both important characteristics because they affect the timing of flows through the watershed. Two watershed shapes were considered in this analysis, a classic dendritic shape and an elongated shape. For each watershed shape, two watershed slopes will be considered, one steep and one mild, resulting in four different watersheds that were used to evaluate the development characteristics. In order to produce results that are as general as possible, synthetic watersheds were used for the analysis. The synthetic watersheds were developed from network topology equations relating stream length, stream slope, and number of stream segments to the stream order and relating stream length to watershed area. The characteristics of the synthetic watersheds are described in detail in the following section.

The first development characteristic that was evaluated is development size. Development size is important because in a watershed that requires detention on all new developments, the number of detention ponds in the watershed is directly related to the size of the developments. The size of the developments is also a factor in the size and design of the detention ponds. The number of detention ponds in a watershed and their design in turn impact the timing of flows through the watershed. In this analysis, each of the four watersheds described above were divided into subbasins that represent developments. Two development sizes were considered, so each watershed was divided into two subbasin sizes, resulting in eight watershed/subbasin

combinations that were used to evaluate the remaining three development characteristics.

The development sizes selected for this analysis were 20 acres and 80 acres. The City of Knoxville, Tennessee requires stormwater detention on residential developments with a total disturbed area greater than five acres, or greater than ½ acre of impervious area, and on commercial developments with a total disturbed area greater than 1 acre, or greater than ½ acre of impervious area (City of Knoxville, TN 2002). The 20 and 80-acre development sizes were selected in part based on consideration of subbasin sizes used in previous studies. The studies cited in the literature review used subbasin sizes ranging from 20 acres to one square mile. Consideration was also given to the practicality of modeling the synthetic watersheds, particularly the number of subbasins and detention ponds that would result from the selected development sizes.

The second development characteristic that was evaluated is development intensity. The development intensity, together with the development size, determines the increase in the volume of runoff from each subbasin over pre-development conditions and determines the size of the required detention ponds, which in turn affects the timing of outflows from the detention ponds. The development intensity was reflected in the sensitivity analysis through the Soil Conservation Service (SCS), currently known as the Natural Resource Conservation Service (NRCS), curve number for each subbasin.

Three levels of development intensity were selected for the analysis. Low intensity development corresponds to a residential area with 1-acre lots. Using values obtained from the SCS Technical Release 55 (SCS TR-55), Urban Hydrology for Small Watersheds (SCS 1986), developments of this intensity have an average of 20% impervious area. The SCS curve number is 68, assuming hydrologic soil group B. Residential developments with ¼-acre lots were selected to represent a medium intensity development. These developments have an average of 38% impervious area and a SCS curve number of 75 (SCS 1986). The third level of development is high

intensity, corresponding to commercial developments with an average of 85% impervious area and a SCS curve number of 92 (SCS 1986).

The third development characteristic that was evaluated is development stage. The development stage is important because it works in conjunction with development size to determine the number of detention ponds in the watershed. Development stage was represented as a percentage of the watershed that is developed. Four percentages were considered in this analysis: 100% (full development), 75%, 50% and 25%. In order to evaluate this factor, each watershed was divided into four areas and each area was then developed progressively one at a time. Development was assumed to begin at the most downstream area in the watershed and proceed upstream.

The fourth development characteristic that was evaluated is development sequence. Development sequence is an important factor because it affects how peak flows from the developed areas combine. Development sequence was evaluated in part using the results from the analysis for development stage. The analysis was then repeated for the 25%, 50% and 75% development stages assuming that development begins at the most upstream area in the watershed and proceeds downstream. By looking at only the portion of the watershed that is developed, watershed size was also indirectly evaluated.

The organization of the sensitivity analysis is illustrated in Table 1. Development intensity, development stage, and development sequence were evaluated for every combination of watershed shape, watershed slope, and development size. However, development intensity was not evaluated for each development stage and development sequence scenario, nor was development stage and development sequence evaluated for each development intensity. The analysis of development intensity assumed a fully developed watershed. The analysis of development stage and development sequence assumed a medium development intensity.

Table 1. Organization of the Sensitivity Analysis

Factor	Values							
Watershed Shape	classic				elongated			
Watershed Slope	steep		mild		steep		mild	
Development Size	20 acres	80 acres	20 acres	80 acres	20 acres	80 acres	20 acres	80 acres
Development Intensity	low medium high	low medium high	low medium high	low medium high	low medium high	low medium high	low medium high	low medium high
Development Stage	25 % 50 % 75 % 100 %	25 % 50 % 75 % 100 %	25 % 50 % 75 % 100 %	25 % 50 % 75 % 100 %	25 % 50 % 75 % 100 %	25 % 50 % 75 % 100 %	25 % 50 % 75 % 100 %	25 % 50 % 75 % 100 %
Development Sequence	25 % 50 % 75 % 100 %	25 % 50 % 75 % 100 %	25 % 50 % 75 % 100 %	25 % 50 % 75 % 100 %	25 % 50 % 75 % 100 %	25 % 50 % 75 % 100 %	25 % 50 % 75 % 100 %	25 % 50 % 75 % 100 %

Development of Synthetic Watersheds

Synthetic watersheds were used for the analysis in order to produce results that are as universally applicable as possible, without being tied to a specific watershed. The use of synthetic watersheds also provides a controlled environment that allows the effects of specific variables to be pinpointed. The synthetic watersheds for this analysis were developed using network topology. Network topology is based on the premise that watersheds of different sizes are geometrically similar and provides relationships between watershed characteristics such as stream length, drainage area, stream slope, and stream order. Geometric similarity between watersheds allows conclusions drawn from one watershed to be applied to other similar watersheds. As a result, the conclusions drawn from the synthetic watersheds are applicable to real-world watersheds.

Overview of Network Topology

Central to network topology is "a system of stream-ordering which recognizes the existence of a hierarchy among the separate branches of a treelike network" (Eagleson 1970). This system is attributed to Horton (1945). On a topographic map, the smallest tributaries are designated as first order streams. A second order stream is formed where two first order streams join. Similarly, a third order stream is formed at the junction of two second order streams, and so on. The practicality of the stream order system is based on the hypothesis that, on average, "order number is directly proportional to size of the contributing watershed, to channel dimensions, and to stream discharge," provided that a large enough sample is considered (Strahler 1964).

The ratio of the number of stream segments of a given order, N_u, to the number of segments of the next higher order, N_{u+1}, is referred to as the *bifurcation ratio*, R_b (Strahler 1964):

$$R_b = \frac{N_u}{N_{u+1}} \qquad \text{...(1)}$$

According to Strahler (1964), bifurcation ratios tend to be a constant throughout a series and typically range between 3.0 and 5.0. High bifurcation ratios are associated with elongated watersheds, while more rounded watersheds have lower bifurcation ratios. Strahler (1964) notes that "long narrow basins with high bifurcation ratios would be expected to have attenuated flood-discharge periods, whereas rotund basins of low bifurcation ratio would be expected to have sharply peaked flood discharges."

The shape of a watershed can be further described by the use of a form factor, R_f, which relates the basin area, A, to the square of the basin length, L_b (Strahler 1964):

$$R_f = \frac{A}{L_b{}^2} \qquad \text{...(2)}$$

The *law of stream lengths,* attributed to Horton (1945), shows that the mean lengths of stream segments, L, are related to stream order, n:

$$R_l = \frac{L_{n+1}}{L_n} \qquad \text{...(3)}$$

The ratio R_l tends to have a value close to 2 (Eagleson 1970). R_l also tends to be constant throughout successive orders of a watershed (Strahler 1964).

The *law of stream areas* shows that the area, A, of a watershed is also related to stream order, n (Eagleson 1970):

$$R_a = \frac{A_{n+1}}{A_n} \qquad \text{...(4)}$$

Studies have found values of R_a ranging from 3 to 5, but it is noted that R_a tends to be constant throughout a given watershed and also tends to be constant within regions of similar climate and geology (Eagleson 1970).

The area of a watershed is related to the length of the main stream by the equation:

$$L = 1.4A^{0.6} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(5)$$

L is defined as the "stream length in miles measured to a point on the drainage divide" and A is area is square miles (Strahler 1964). Gray (1961) notes that the length of the main stream, L, "represents a composite of stream lengths of different order."

The *law of stream slopes,* attributed to Horton (1945), shows that the average stream channel slope, S, is related to stream order, n:

$$R_s = \frac{S_{n+1}}{S_n} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(6)$$

Channel slope generally decreases with increasing stream order. The ratio R_s has been found to have a typical value of 0.55 for mature streams in a humid climate (Eagleson 1970).

Strahler (1964) provides a relationship between overland slopes and stream channel slopes. He observed that the average maximum valley-wall slope, θ_g, was related to the average gradient of second order streams, θ_c, by the equation:

$$\log\theta_g = 0.6 + 0.8\log\theta_c \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(7)$$

Watershed Development

Two watershed shapes were developed using network topology, a classic dendritic shape and an elongated shape. Each shape was combined with both steep slopes and mild slopes, resulting in four watersheds that will be used in the sensitivity analysis. All four watersheds are 10 square miles in area. This size was selected

based on its use in previous work (James et al. 1987; Bedient et al. 1982). It is also large enough to represent a small urban watershed, while being small enough that it is realistic to model the use of detention ponds on small developments throughout the entire watershed.

The development of the classic watershed follows work done by James et al. (1987) in which a 10 square mile, fifth order watershed was developed using network topology. Using Equation 5, the length of the classic 10 square mile watershed was found to be 5.57 miles. The bifurcation ratio was assumed to be an average value of 4. The ratio of area from one stream order to the next was also assumed to be an average value of 4. The stream length for each stream order was then determined using Equation 5. The resulting watershed is a third order watershed with the characteristics shown in Table 2. The watershed could have been further subdivided, resulting in a higher stream order, but it was determined that a greater level of detail was not needed for this analysis.

Bedient et al. (1982) used a shape factor, *SF*, to define three 10 square mile idealized rectangular watersheds. This shape factor is equivalent to the form factor described in Equation 2. Bedient et al. (1982) analyzed the shape of 12 watersheds in the Houston, TX area and found the average watershed shape factor to be 0.43, with extreme values of 0.15 for elongated watersheds and 1.00 for concentrated shapes. The three idealized watersheds developed in that study had shape factors of 0.92, 0.48, and 0.18.

The shape factor of the classic watershed developed above is 0.32. Following the work of Bedient et al. (1982), the elongated watershed was developed based on a shape factor of 0.18. Using a shape factor of 0.18 and a watershed area of 10 square

Table 2. Characteristics of the Classic Watershed

Stream Order	# of Stream Segments	Area, sq. miles	Stream Length, miles
1	16	0.625	1.06
2	4	2.5	2.43
3	1	10	5.57

miles, application of Equation 2 results in a basin length for the elongated watershed of 7.45 miles. It should be noted that Equation 5 will result in only one watershed length for a given area, illustrating a tendency for larger watersheds to become more elongated (Eagleson 1970). However, Gray (1961) analyzed the relationship between length and area for 47 watersheds in various states and fit Equation 5 to that data. The combination of a 10 square mile area and a 7.45 mile basin length falls within the 95% confidence limits that Gray established, so the elongated watershed developed for this analysis is a reasonable shape.

A third order drainage system was developed for the elongated watershed, as was done for the classic watershed. Since high bifurcation ratios are associated with more elongated watersheds (Strahler 1964), a bifurcation ratio of 5 was assumed for the development of the elongated watershed for this analysis. According to Strahler (1964), a bifurcation ratio of 5 is at the high end of the normal range of values. The ratio of area from one stream order to the next was also assumed to be 5 for practical purposes so that geometric similarity could be maintained between all of the stream segments. A value of 5 is within the range of reported values (Eagleson 1970). The stream lengths for the first and second order streams were then determined using Equation 5. The resulting watershed is a third order watershed with the characteristics shown in Table 3.

Each watershed shape was combined with both steep slopes and mild slopes. For the steep case, an overland slope of 15% was assumed. For the mild case, an overland slope of 5% was assumed. Given these overland slopes, Equation 7 was used to determine the corresponding slopes of the second order streams for each case. Equation 6 was then used to determine corresponding slopes for the first and third

Table 3. Characteristics of the Elongated Watershed

Stream Order	# of Stream Segments	Area, sq. miles	Stream Length, miles
1	25	0.4	0.808
2	5	2	2.12
3	1	10	7.45

Table 4. Watershed Slopes

Case	Overland Slope, %	1st Order Channel Slope, %	2nd Order Channel Slope, %	3rd Order Channel Slope, %
Mild	5	2.2	1.2	0.66
Steep	15	8.2	4.5	2.5

order streams. These slopes are summarized in Table 4. For simplicity, it was assumed that the overland slopes are the same throughout the watersheds.

Watershed slopes can of course vary dramatically depending on location and steep and mild are relative terms. While it is not possible to encompass the range of slopes that may be encountered by considering only two cases, the slopes selected for this analysis are representative of watershed slopes that may be encountered in many areas. For example, overland slopes in five urban watersheds in the Knoxville, Tennessee area range primarily between 0 and 12%, with some steeper areas of 12 to 25%, and a few small areas greater than 25% (Kung 1980). One of these watersheds, Fourth Creek, is described as having "gentle slopes", with an average basin slope of 8% (Kung 1980). Similarly, the Maryland watershed studied by Duru (1981) has slopes ranging from 1 to 15% and a Colorado watershed studied by Glidden (1981) has overland slopes ranging from 1 to 30%.

Both the classic watershed and the elongated watershed are illustrated in Figure 1. For the classic watershed shape, the first order streams each have a drainage area of 400 acres. In order to model the development of the classic watershed, each first order drainage area was broken down into five 80-acre tracts and then further subdivided into twenty 20-acre tracts. This results in a total of 80 developments in the watershed for the 80-acre development size and 320 developments in the watershed for the 20-acre development size. The runoff from these developments was added to the stream network at the junctions shown in Figure 1.

For the elongated watershed shape, the first order streams each have a drainage area of 256 acres. To model development of the elongated watershed, each

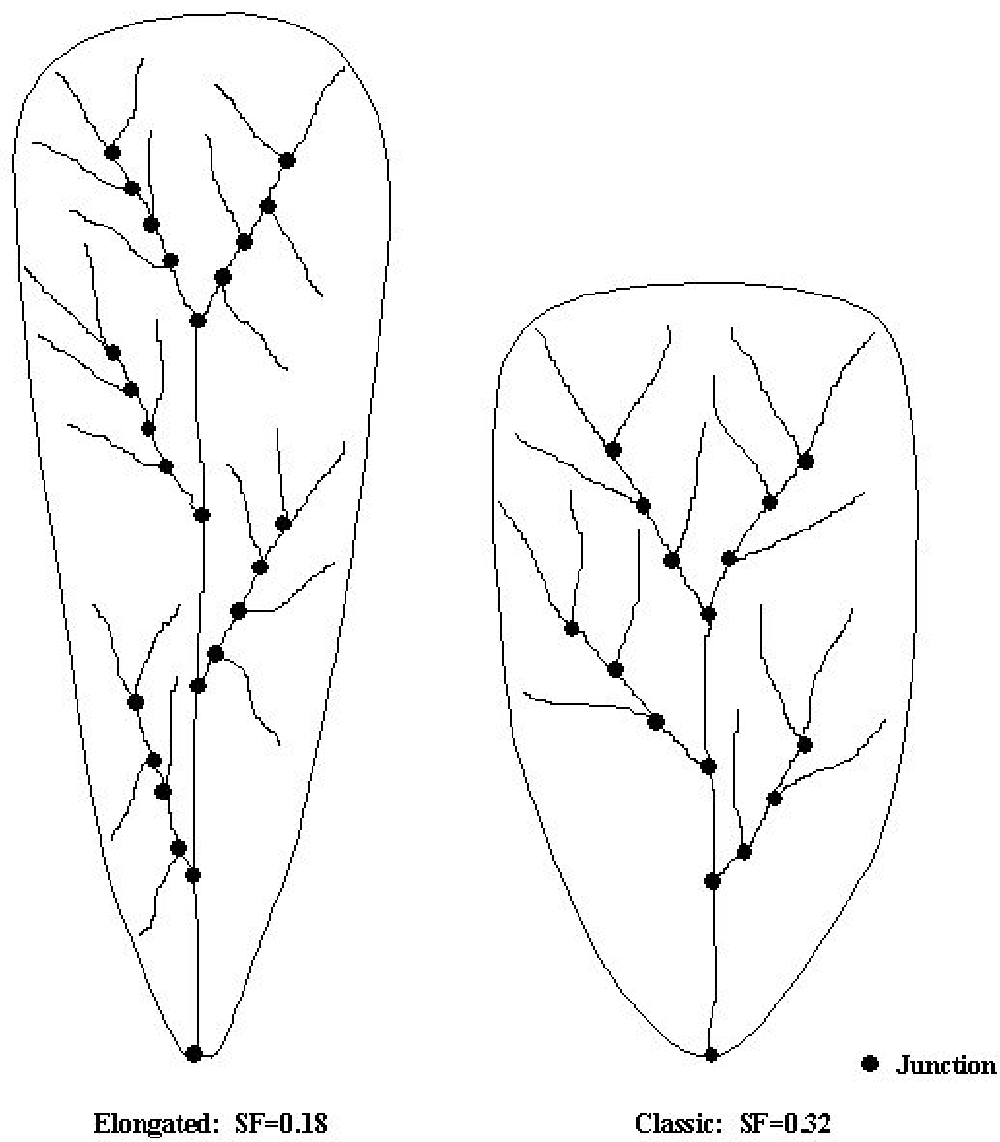

Figure 1. Watershed Shapes

first order drainage area was again divided into 80-acre tracts and 20-acre tracts as was done for the classic watershed. Since 256 acres is not exactly divisible by either 20 acres or 80 acres, the area was approximated by dividing it into three 80-acre tracts and into thirteen 20-acre tracts. Although using the same development sizes for the elongated watershed as were used for the classic watershed required some approximation, it was desirable to do so in order to make the results from the two watershed shapes comparable. This results in a total of 75 developments in the watershed for the 80-acre development size and 325 developments in the watershed for the 20-acre development size. The runoff from these developments was added to the stream network at the junctions shown in Figure 1.

Modeling of the Synthetic Watersheds

The steps required to model the synthetic watersheds were as follows:

1) Model the pre-development conditions for the watersheds for each combination of watershed shape and slope.

2) Determine the post-development peak flow, without detention, for subbasins representing each combination of development size, development intensity, and watershed slope.

3) Design detention ponds to limit the post-development peak flow from each developed subbasin to the pre-development peak flow.

4) Complete the modeling runs for the sensitivity analysis. This step required modeling the post-development conditions without detention and the post-development conditions with detention for the synthetic watersheds for each combination of watershed shape, watershed slope, development size, development intensity, development stage, and development sequence.

The modeling was completed using the U. S. Army Corps of Engineers Hydrologic Engineering Center's (HEC) Hydrologic Modeling System (HEC-HMS). A description of the methods that were used within the model and the model input requirements is given in the following section. Subsequent sections describe

modeling of pre-development conditions, modeling of post-development conditions, detention pond design, and the sensitivity analysis modeling runs.

Overview of Modeling with HEC-HMS

HEC-HMS version 2.2.1 was used to model the synthetic watersheds. HEC-HMS is a computer program that simulates precipitation-runoff and routing processes of dendritic watershed systems and is the successor to the HEC-1 Flood Hydrograph Package (HEC 2001). In order to develop a model in HEC-HMS, three basic individual model components are required, a basin model, a meteorologic model, and a control specifications file. These components are contained within a project. A project may contain multiple basin models, meteorologic models, and control specifications files. Modeling runs are defined by selecting one of each of the three components contained within a project.

Basin Model

The basin model in HEC-HMS consists of hydrologic elements that may include any combination of subbasins, reaches, reservoirs, junctions, diversions, sources, or sinks. These elements are connected to form a hydrologic element network (HEC 2001). All of these types of elements were used in this analysis, except diversions and sinks, and are explained in more detail below.

Subbasins. Within each subbasin element a loss model is selected to compute losses from precipitation, a transform method is selected to compute direct runoff from precipitation, and a baseflow method may be selected to model groundwater contributions to channel flow (HEC 2001).

HEC-HMS computes the volume of runoff from a watershed by computing losses due to infiltration, interception, surface storage, evaporation, and transpiration, and then subtracting the losses from the precipitation (HEC 2000). There are seven

methods available in HEC-HMS to estimate losses: deficit and constant, initial and constant, Green and Ampt, SCS curve number, gridded SCS curve number, soil moisture accounting, and gridded soil moisture accounting (HEC 2001).

The SCS curve number method was selected for use in this analysis because it is a familiar method and the only variable which must be determined is the curve number. The curve number is a function of land use, soil type, and antecedent moisture conditions (SCS 1986). This method lends itself well to this sensitivity analysis for the evaluation of development intensity because assuming that the soil type and antecedent moisture conditions are constant throughout the watersheds being modeled, the curve number is then a function of land use only. SCS curve numbers for various land uses and soil types are published in TR-55 (SCS 1986).

The only input that HEC-HMS requires for the SCS curve number method is a value for the intial loss, or initial abstraction, I_a.. I_a represents all losses that occur before runoff begins (SCS 1986). No runoff will occur until "the accumulated rainfall exceeds the initial abstraction" (HEC 2000). I_a can be estimated by an empirical equation developed by the SCS:

$$I_a = 0.2S \qquad \text{(8)}$$

where S is the potential maximum retention after runoff begins (SCS 1986). S is "a measure of the ability of a watershed to abstract and retain storm precipitation (HEC 2000) and is related to the curve number, CN, by the following equation (SCS 1986):

$$S = \frac{1000}{CN} - 10 \qquad \text{(9)}$$

where S is in inches.

Runoff can then be calculated using the SCS runoff equation (SCS 1986):

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \qquad \text{..(10)}$$

where Q is runoff in inches, and P rainfall in inches. HEC-HMS uses this equation to calculate the incremental precipitation excess for each time interval (HEC 2000).

Six methods are available in HEC-HMS to transform precipitation excess into direct runoff: Snyder unit hydrograph, Clark unit hydrograph, SCS unit hydrograph, user-specified unit hydrograph, ModClark model, and Kinematic-wave model (HEC 2001). The SCS unit hydrograph method was selected for use in this analysis because it is simple to apply and because it is commonly used in practice. For example, the stormwater ordinance for the City of Knoxville, TN requires the use of SCS unit hydrograph procedures (City of Knoxville, TN 2002).

The SCS unit hydrograph is a dimensionless, single-peaked, synthetic unit hydrograph which "expresses the unit hydrograph discharge, U_t, as a ratio to the unit hydrograph peak discharge, U_p, for any time t, a fraction of T_p, the time to unit hydrograph peak" (HEC 2000). The unit hydrograph peak and the time of peak are related by the following equation:

$$U_p = C \frac{A}{T_p} \qquad \text{...(11)}$$

where A is the watershed area, and C is a conversion constant that is equal to 484 in the foot-pound system (HEC 2000).

The time of peak, T_p, is given by the following equation:

$$T_p = \frac{\Delta t}{2} + t_{lag} \qquad \text{...(12)}$$

where Δt is the duration of excess precipitation and t_{lag} is the basin lag, or “the time difference between the center of mass of rainfall excess and the peak of the unit hydrograph” (HEC 2000).

The basin lag is the only input that HEC-HMS requires for the SCS unit hydrograph method. The basin lag can be estimated based on the time of concentration, t_c (HEC 2000):

$$t_{lag} = 0.6 t_c \qquad \text{(13)}$$

Four options are available in HEC-HMS for modeling baseflow. The three baseflow methods that are available are constant monthly, linear reservoir, and recession (HEC 2001). The fourth option that may be selected is that of no baseflow. Baseflow was not modeled for this analysis.

Reaches. Reach elements are used to represent open channel flow and a routing method must be selected for each reach (HEC 2001). There are six routing methods available in HEC-HMS: kinematic wave, lag, modified Puls, Muskingum, Muskingum-Cunge standard section, and Muskingum-Cunge 8-point section (HEC 2001). The Muskingum-Cunge standard section method was used for this analysis. The Muskingum-Cunge method is appropriate when no observed hydrograph data is available for calibration because the required parameters are physically based (HEC 2000) and is “one of the most recommended techniques for general use” (Viessman et al. 1989).

The Muskingum-Cunge method “blends the accuracy of the diffusion method with the simplicity of the Muskingum method… It is classified as a hydrologic method, yet it gives results comparable with hydraulic methods” (Viessman et al. 1989). The method is based on the diffusion form of the momentum equation and a simplified form of the continuity equation.

The momentum equation "accounts for forces that act on a body of water in an open channel" (HEC 2000). The diffusion wave approximation of the momentum equation is:

$$S_f = S_o - \frac{\partial y}{\partial x} \quad \text{.....(14)}$$

where S_f is the energy gradient, or the friction slope, S_o is the bottom slope, and $\frac{\partial y}{\partial x}$ is the pressure gradient (HEC 2000).

The continuity equation "accounts for the volume of water in a reach of an open channel, including that flowing into the reach, that flowing out of the reach, and that stored in the reach" (HEC 2000). The Muskingum-Cunge method uses the following form of the continuity equation:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q_L \quad \text{.....(15)}$$

where q_L is the lateral inflow per unit length of channel (HEC 2000).

Combination of these two equations, a finite difference approximation of the partial derivatives, and combination with the Muskingum model results in the following equation which is solved by HEC-HMS (HEC 2000):

$$O_t = C_1 I_{t-1} + C_2 I_t + C_3 O_{t-1} + C_4 (q_L \Delta x) \quad \text{.....(16)}$$

where the coefficients are (HEC 2000):

$$C_1 = \frac{\frac{\Delta t}{K} + 2X}{\frac{\Delta t}{K} + 2(1 - X)} \quad \text{.....(17)}$$

$$C_2 = \frac{\frac{\Delta t}{K} - 2X}{\frac{\Delta t}{K} + 2(1-X)} \quad \text{...(18)}$$

$$C_3 = \frac{2(1-X) - \frac{\Delta t}{K}}{\frac{\Delta t}{K} + 2(1-X)} \quad \text{...(19)}$$

$$C_4 = \frac{2\frac{\Delta t}{K}}{\frac{\Delta t}{K} + 2(1-X)} \quad \text{...(20)}$$

and the parameters *K* and *X* are (HEC 2000):

$$K = \frac{\Delta x}{c} \quad \text{...(21)}$$

$$X = \frac{1}{2}\left(1 - \frac{Q}{BS_o c\Delta x}\right) \quad \text{...(22)}$$

where *c* is wave celerity and *B* is the top width of the water surface.

The input required by HEC-HMS for the Muskingum-Cunge routing method is channel shape, reach length, Manning's n roughness coefficient, and channel energy slope. For the Muskingum-Cunge standard section method the channel shape is specified as either circular or prismatic, including triangular, rectangular, or prismatic. The prismatic shapes are defined by channel bottom width and side slope (HEC 2001).

Reservoirs. Detention ponds can be modeled in HEC-HMS using reservoir elements. Outflow from the reservoir is computed using the Modified-Puls routing model, also known as storage routing or level-pool routing (HEC 2000). The Modified-Puls method is based on a finite difference approximation of the continuity equation:

$$I_{avg} - O_{avg} = \frac{\Delta S}{\Delta t} \qquad \text{(23)}$$

where I_{avg} is the average inflow during a time interval, O_{avg} is the average outflow during the time interval, and ΔS is the change in storage (HEC 2000). This equation can be written in the following form used by HEC-HMS to compute the outflow hydrograph from a reservoir (HEC 2000):

$$\left(\frac{2S_{t+1}}{\Delta t} + O_{t+1}\right) = (I_t + I_{t+1}) + \left(\frac{2S_t}{\Delta t} - O_t\right) \qquad \text{(24)}$$

The only input required by HEC-HMS for reservoir elements is a storage-outflow relationship and an initial condition for each reservoir.

Junctions. Junction elements are used to model stream confluences. The use of junctions is optional as "all elements in the network automatically combine all upstream inflows before performing their own computations"; however, the use of junctions "does control what results are available" (HEC 2001). No input is required for junctions.

Sources. Source elements can be used to represent "a point discharge into the stream network" and the outflow from a source may be specified as either a constant flow or as a gage hydrograph (HEC 2001). The only input required for a source set to

the gage hydrograph option is the name of a gage. A discharge gage simply contains a user-provided hydrograph.

Sources with the gage hydrograph option were used in this analysis in order to simplify development of the basin models. In order to model the development of the synthetic watersheds, runoff from each first order stream was added to the stream network at the junctions shown in Figure 1. For each combination of watershed and development characteristics, the runoff hydrograph from each first order stream was the same. Therefore, it was only necessary to develop a detailed model for one first order drainage area for each combination of watershed and development characteristics. The runoff hydrograph from that first order drainage area could then be input into HEC-HMS as a discharge gage. Each first order drainage area could then be represented as a source, with the source set to add the corresponding gage hydrograph to the stream network.

Meteorologic Model

The meteorologic model in HEC-HMS contains precipitation data and optionally, evapotranspiration data. The evapotranspiration option was not used for this analysis. Seven precipitation methods are available in HEC-HMS: user hyetograph, user gage weighting, inverse-distance gage weights, gridded precipitation, frequency storm, SCS hypothetical storm, and standard project storm (HEC 2001). The SCS hypothetical storm method was used for this analysis with an SCS type II rainfall distribution. This method was selected because it is commonly used for stormwater design. For example, the stormwater ordinance for the City of Knoxville, TN requires the use of SCS type II rainfall distribution (City of Knoxville, TN 2002).

The SCS developed four synthetic rainfall distributions; type I, IA, II, and II; corresponding to different geographic regions of the United States. These four distributions each have a duration of 24 hours and “include maximum rainfall intensities for the selected design frequency arranged in a sequence that is critical for

producing peak runoff" (SCS 1986). The SCS type II distribution is applicable to most of the country, except for the Pacific and Atlantic coastal areas and the Gulf of Mexico (SCS 1986).

A 10-year frequency storm was used for this analysis. The rationale behind the use of this frequency is discussed further in the section on detention pond design. The only input that HEC-HMS requires for the SCS hypothetical storm method is the storm type, in this case type II, and the storm depth. The National Weather Service published 24-hour rainfall data for the eastern part of the country in Technical Paper 40 (TP-40) and rainfall maps from TP-40 are included in SCS TR-55 (SCS 1986). The 24-hour rainfall depth for a given frequency can be read off of these maps, or may be generated by the SCS TR-55 computer program. The storm depth for a 10-year, 24-hour storm is 4.93 inches (SCS 1986).

Control Specifications

The control specifications file in HEC-HMS contains the starting and ending date and time for a run as well as time interval used for computations (HEC 2001). For this analysis, the starting and ending date and time were set to encompass a 24-hour period, corresponding with the length of the SCS type II rainfall distribution. The time interval "determines the resolution of model results computed during a run" and can be set to intervals ranging from 1 minute to 24 hours (HEC 2001). The time interval was set to 1 minute for this analysis. This time interval was required in order to adequately define the rising limb of the hydrograph for some of the subbasins used in the analysis that have very short times of concentration. HEC recommends that "for adequate definition of the ordinates on the rising limb of the SCS unit hydrograph, a computation interval, Δt, that is less than 29% of t_{lag} must be used" (HEC 2000).

Pre-Development Conditions

Pre-development conditions were modeled for each watershed shape and slope combination, resulting in four pre-development watershed models: a classic shape with mild slopes, a classic shape with steep slopes, an elongated shape with mild slopes, and an elongated shape with steep slopes. In HEC-HMS, pre-development conditions for each case were first modeled for a single subbasin representing the drainage area of one first order stream. For each case, the subbasins were then combined into basin models of the entire 10-square mile watersheds.

The characteristics of the pre-development subbasins required for input into HEC-HMS are shown in Table 5, including the subbasin area, curve number, initial abstraction, time of concentration, and basin lag time. In order to keep track of the numerous subbasins, basin models, and modeling runs necessary for the sensitivity analysis, a naming convention was derived. The pre-development subbasins in Table 5 are designated by either "C" or "E", referring to either the classic watershed shape or the elongated watershed shape, respectively, and by either "M" or "S", referring to either mild slopes or steep slopes, respectively. The basin models developed from each subbasin share this naming convention, as do the modeling runs completed for each basin model. The same naming convention will be used for the post-development conditions, with some additional designations.

A curve number of 55 was assumed for the pre-development conditions. This curve number corresponds to a land use of deciduous forest in good hydrologic condition with soils in hydrologic soil group B (SCS 1986). The initial abstraction

Table 5. Characteristics of the Pre-Development Subbasins

Subbasin	Area, sq mi	CN	I_a	T_c, hrs	T_{lag}, hrs
CMPre	0.625	55	1.64	0.88	0.528
CSPre	0.625	55	1.64	0.51	0.306
EMPre	0.4	55	1.64	0.79	0.474
ESPre	0.4	55	1.64	0.46	0.276

was calculated from the curve number and the basin lag was calculated from the time of concentration, using equations given previously.

The time of concentration for each subbasin was calculated using the SCS TR-55 computer program. The time of concentration is computed as the sum of the travel time in sheet flow segments, the travel time in shallow concentrated flow segments, and the travel time in open channel flow segments (SCS 1986). Sheet flow is flow over plane surfaces and is usually less than 300 feet (SCS 1986). For the pre-development subbasins, 100 feet of sheet flow through dense woods was assumed for input into TR-55. Shallow concentrated flow is overland flow in shallow rill and rivulets or down streets and gutters in a developed area (HEC 2000). A shallow concentrated flow length of 900 feet over an unpaved surface was assumed for input into TR-55. Open channel flow is assumed to begin "where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where blue lines appear on United States Geological Survey (USGS) quadrangle sheets" (SCS 1986). For the pre-development subbasins, channel flow occurs in the first order stream channels. The first order stream lengths for the classic and elongated watershed subbasins are 5,597 feet and 4,266 feet, respectively. TR-55 also requires a value for Manning's roughness coefficient, *n*, and channel cross-section dimensions. A value of 0.045 was assumed for Manning's *n*. This is an average value for a minor, winding stream (Chow 1959). The first order stream channels were assumed to be 5 feet wide and 1 foot deep.

The peak flows for each pre-development subbasin resulting from the 10-year storm are given in Table 6. These peak flows were used for the design of the detention ponds for the post-development conditions.

Table 6. 10-Year Peak Flows for the Pre-Development Subbasins

Subbasin	10-Year Peak Flow, cfs
CMPre	178.8
CSPre	267.4
EMPre	123.6
ESPre	181.6

Post-Development Conditions

The first step in modeling the post-development conditions was to model subbasins representing individual developments. Detention ponds were then designed to limit the peak flow from these developments to the pre-development peak flow. These subbasins and detention ponds were then used to create basin models of the entire 10-square mile watersheds that were used to complete the modeling runs for the sensitivity analysis.

Subbasins

Of the factors considered in the sensitivity analysis, development size, development intensity, and watershed slope are the factors that will affect the peak flow from individual developments. Therefore, modeling of the post-development conditions required modeling twelve post-development subbasins to encompass each combination of these three factors. Six of the post-development subbasins represent 20 acre developments and six subbasins represent 80 acre developments. Each development size was combined with mild and steep watershed slopes, and with low, medium, and high development intensities.

The characteristics of the post-development subbasins required for input into HEC-HMS are shown in Table 7, including the subbasin area, curve number, initial abstraction, time of concentration, and basin lag time. The naming convention used for the subbasins is like that used for the pre-development subbasin. The post-development subbasins in Table 7 are designated by either “M” or “S”, referring to either mild slopes or steep slopes, respectively; by either “80” or “20”, referring to the development size; and by either “L”, “Med”, or “H”, referring to low, medium, or high development intensity, respectively.

As for the pre-development subbasins, the time of concentration for each post-development subbasin was calculated using the SCS TR-55 computer program. For the post-development subbasins, sheet flow was assumed to occur on dense grass.

Table 7. Characteristics of the Post-Development Subbasins

Subbasin	Area, sq mi	CN	I_a	T_c, hrs	T_{lag}, hrs
M80L	0.125	68	0.941	0.34	0.204
S80L	0.125	68	0.941	0.22	0.132
M20L	0.03125	68	0.941	0.19	0.114
S20L	0.03125	68	0.941	0.12	0.072
M80Med	0.125	75	0.667	0.34	0.204
S80Med	0.125	75	0.667	0.22	0.132
M20Med	0.03125	75	0.667	0.19	0.114
S20Med	0.03125	75	0.667	0.12	0.072
M80H	0.125	92	0.174	0.34	0.204
S80H	0.125	92	0.174	0.22	0.132
M20H	0.03125	92	0.174	0.19	0.114
S20H	0.03125	92	0.174	0.12	0.072

Shallow concentrated flow was assumed to be conveyed as quickly as possible to concrete ditches where channel flow will occur. These ditches will then drain to the first order streams. To estimate the lengths of the flow paths it was assumed that the developments are roughly square, resulting in a longest flow path of 2,000 feet for the 80 acre developments and a longest flow path of 1,000 feet for the 20 acre developments. For the 80 acre developments, a sheet flow length of 200 feet, a shallow concentrated flow length of 800 feet, and a channel flow length of 1,000 feet were input into TR-55. For the 20 acre developments, a sheet flow length of 100 feet, a shallow concentrated flow length of 400 feet, and a channel flow length of 500 feet were input into TR-55. A value of 0.013 was assumed for Manning's *n*. This is an average value for finished concrete (Chow 1959). The ditches were assumed to 3 feet wide and 1 foot deep.

The peak flows for each post-development subbasin resulting from the 10-year storm are given in Table 8.

Table 8. 10-Year Peak Flows for the Post-Development Subbasins

Subbasin	10-Year Peak Flow, cfs
M80L	163.3
S80L	195.4
M20L	51.1
S20L	57.3
M80Med	218.5
S80Med	259.2
M20Med	67.7
S20Med	76.5
M80H	350.5
S80H	410.3
M20H	106.7
S20H	119.7

Detention Pond Design

Detention ponds were designed so that the post-development peak flow from each subbasin described above is no greater than the pre-development peak flow from the subbasin for the specified design storm. The sensitivity analysis assumes that detention is used on all developments in the watershed. An important assumption in the analysis is that all of the detention ponds are properly maintained and functioning as intended, which in a real world situation might not be the case.

Many communities require that detention ponds control the peak flows from multiple return period storms. For example, the stormwater ordinance for the City of Knoxville, TN requires that detention ponds be designed to control the runoff from the 1-year, 2-year, 5-year, and 10-year frequency 24-hour duration storms (City of Knoxville, TN 2002). To simplify the detention pond design process for this analysis, the ponds were designed to control only the 10-year storm. Recall that the 10-year frequency, 24-hour duration storm was used for the meteorological model in HEC-HMS, so the detention ponds were designed to control this storm.

A simple 45^{o} V-notch weir was chosen for the outlet configuration for all of the ponds. The selection of an outlet for the ponds was an important consideration

because the outlet type affects the timing of the outflow from the detention ponds and could ultimately have a significant affect on the results of the sensitivity analysis. Shea (1995) found that orifice outlets produced significantly greater increases in peak flows at points in the watershed than weir outlets. Therefore, the use of a weir outlet was desirable so that the sensitivity analysis would not be biased towards producing greater increases in peak flows. So that the results from all of the modeling runs in the sensitivity analysis would be comparable, it was necessary to use the same outlet configuration for all of the detention ponds. From a practical design standpoint, the 45° V-notch weir was selected because it was effective in controlling the runoff from each subbasin in this analysis. The 45° V-notch weir also resulted in reasonable requirements for the surface area of the detention ponds and for the depth of the ponds. The BMP manual for the City of Knoxville, TN recommends that depths over 4 feet be avoided when possible (City of Knoxville, TN 2001). The use of the 45° V-notch weir produced a maximum depth of 5 feet, and a maximum surface area of 4 acres.

The flow over a V-notch weir can be calculated from the following equation:

$$Q = C_d \frac{8}{15}\sqrt{2g}\tan\frac{\theta}{2}H^{5/2} \qquad \text{(25)}$$

where Q is the flow in cfs, H is the head in feet, θ is the vertex angle of the weir, and C_d is a coefficient of discharge (Daugherty et al. 1985). C_d is a function of the vertex angle and the head. The minimum value of C_d for all vertex angles is 0.581 and for a vertex angle of 45° C_d is approaching the minimum value at a head of one foot (Daugherty et al. 1985).

In order to design each detention pond, the desired peak outflow was set equal to the pre-development peak flow from the development. The maximum head over the weir was then calculated from equation 25. An initial guess of the required storage volume was then made, and area-capacity data for the pond was calculated using a spreadsheet based on the geometry of the pond. All of the detention ponds

were assumed to have a simple rectangular shape with 3:1 side slopes. The elevation-outflow relationship for each pond was calculated using equation 25. For each detention pond, the elevation-storage-outflow relationship was entered into HEC-HMS and the runoff from the post-development subbasin was routed through the detention pond. The storage capacity of the pond was then adjusted until the routing yielded acceptable results. In all cases, the ponds were designed so that the peak detention outflow was slightly less than the pre-development peak. However, in order to ensure that the results from all of the modeling runs in the sensitivity analysis would be comparable, it was necessary to set a criterion to define how much reduction below the pre-development peak was acceptable so that none of the ponds were over-designed in comparison to the others. The criterion which was used was that the percent reduction of the peak detention outflow below the pre-development peak should be between 0.5% and 1%. It was not possible to meet this criterion exactly in all cases, but it was met as closely as possible.

It was necessary to design a total of 24 detention ponds. Two detention ponds were designed for each of the 12 post-development subbasins, one for the classic watershed and one for the elongated watershed. This was necessary because the pre-development peak flows were different for the two watershed shapes. Recall that the pre-development peak flows shown in Table 6 are the peak flows from the entire first order stream drainage area. In order to determine the pre-development peak flow from each 80-acre and 20-acre subbasin, the pre-development peak flows shown in Table 6 were simply divided by the number of 80-acre and 20-acre subbasins in the first order watershed. The characteristics of the 24 detention ponds are given in Table 9. The elevation-area-storage-outflow relationships for each detention pond are provided in appendix Table A-1.

Sensitivity Analysis

Execution of the sensitivity analysis required a total of 148 modeling runs in HEC-HMS. Four of these runs were for pre-development conditions, 48 runs were

Table 9. Detention Pond Characteristics

Detention Pond	Pre-Dev Peak, cfs	Post-Dev Peak, cfs	Storage, AF	Surface Area, ac	Peak Elev., ft	Peak Detention Outflow, cfs	Time of Peak	% Reduction below Pre-Dev Peak
CM80L	35.8	163.3	4.91	1.2	4.11	35.5	1234	0.84
CS80L	53.5	195.4	4.45	0.92	4.83	53.0	1217	0.93
CM20L	8.9	51.1	1.25	0.53	2.34	8.83	1224	0.79
CS20L	13.4	57.3	1.16	0.42	2.76	13.3	1209	0.75
CM80Med	35.8	218.5	7.61	1.8	4.11	35.5	1239	0.84
CS80Med	53.5	259.2	7.01	1.4	4.83	53.0	1222	0.93
CM20Med	8.9	67.7	1.92	0.81	2.34	8.83	1230	0.79
CS20Med	13.4	76.5	1.78	0.64	2.76	13.3	1210	0.75
CM80H	35.8	350.5	16.50	3.99	4.12	35.5	1249	0.84
CS80H	53.5	410.3	15.57	3.2	4.83	53.0	1229	0.93
CM20H	8.9	106.7	4.15	1.8	2.34	8.82	1239	0.89
CS20H	13.4	119.7	3.91	1.4	2.76	13.3	1221	0.75
EM80L	41.2	163.3	4.70	1.1	4.35	40.9	1230	0.73
ES80L	60.5	195.4	4.29	0.85	5.08	60.0	1215	0.83
EM20L	9.5	51.1	1.22	0.50	2.41	9.40	1222	1.05
ES20L	14.0	57.3	1.15	0.41	2.81	13.83	1209	1.20
EM80Med	41.2	218.5	7.36	1.7	4.35	40.9	1235	0.73
ES80Med	60.5	259.2	6.80	1.3	5.08	60.0	1219	0.83
EM20Med	9.5	67.7	1.89	0.78	2.41	9.40	1228	1.05
ES20Med	14.0	76.5	1.77	0.62	2.81	13.9	1209	0.71
EM80H	41.2	350.5	16.16	3.7	4.36	41.0	1244	0.5
ES80H	60.5	410.3	15.26	3.0	5.08	60.0	1226	0.83
EM20H	9.5	106.7	4.10	1.7	2.41	9.40	1237	1.05
ES20H	14.0	119.7	3.88	1.4	2.81	13.9	1220	0.71

for the evaluation of development intensity, 48 runs were for the evaluation of development stage, and 48 runs were for the evaluation of development sequence. Each of these runs required the development of an individual basin model in HEC-HMS which was then paired with the same meteorologic model and control specifications file for each run. Development intensity, development stage, and development sequence were all evaluated for each combination of watershed shape, watershed slope, and development size. This allowed watershed shape, watershed slope, and development size to be evaluated using the results from the evaluation of the other three factors.

The four pre-development modeling runs encompassed each combination of watershed shape and watershed slope. Four basin models of the 10-square mile watersheds were developed, one using each pre-development subbasin shown in Table 5. These basin models and modeling runs were designated CMPre, CSPre, EMPre, and ESPre, corresponding with their respective subbasins. The results from these four modeling runs were used to evaluate the results from all of the post-development modeling runs.

Low, medium, and high development intensities were evaluated for each combination of watershed shape, watershed slope, and development size. Each case was evaluated both with detention and without detention. Basin models of the 10-square mile watersheds were developed for each case, using the appropriate post-development subbasin from Table 7 and the appropriate detention pond from Table 9. The evaluation of development intensity assumed that the watersheds were fully developed with detention used on each individual development. The modeling runs for the evaluation of development intensity are summarized in appendix Table A-2. The naming convention is the same as for the subbasins.

The layout of the basin models in HEC-HMS for the classic watersheds and the elongated watersheds are shown in Figures 2 and 3, respectively. As explained earlier, each "source" in the HEC-HMS basin models contains the total outflow hydrograph from a first order drainage area, including subbasins and reservoirs. Figure 4 shows an example of the basin elements represented in a single source. The

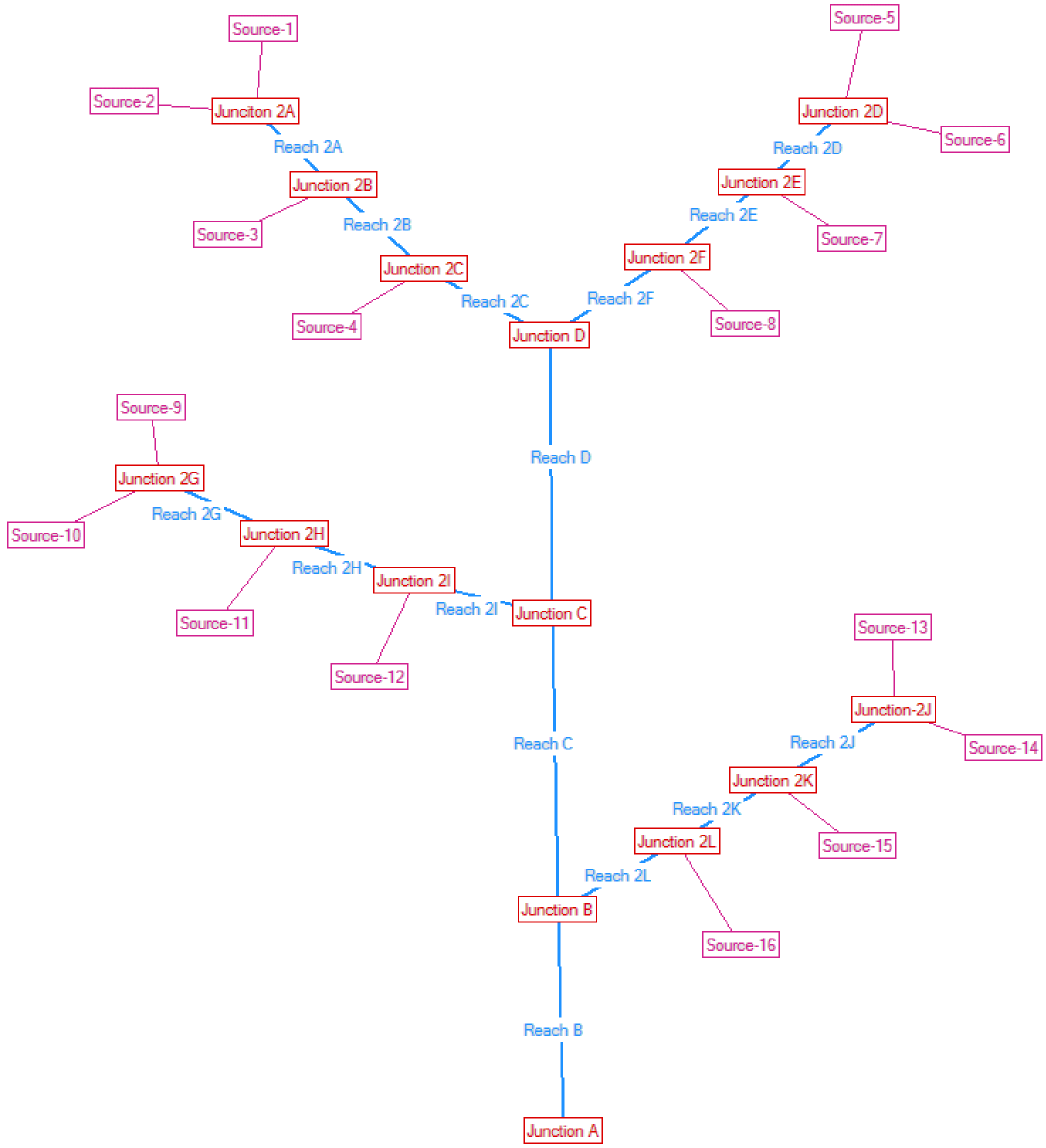

Figure 2. HEC-HMS Layout of the Classic Watersheds

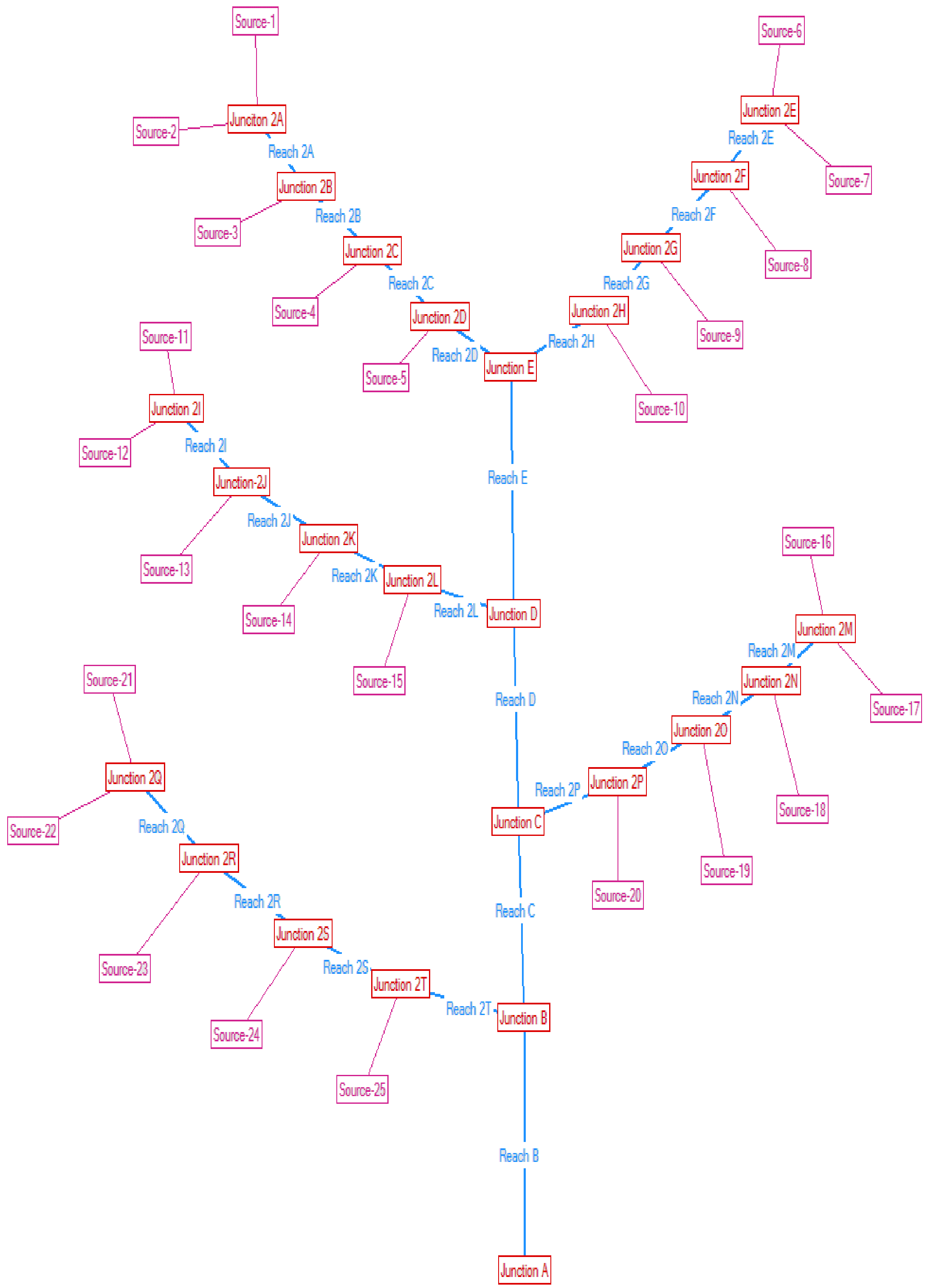

Figure 3. HEC-HMS Layout of the Elongated Watersheds

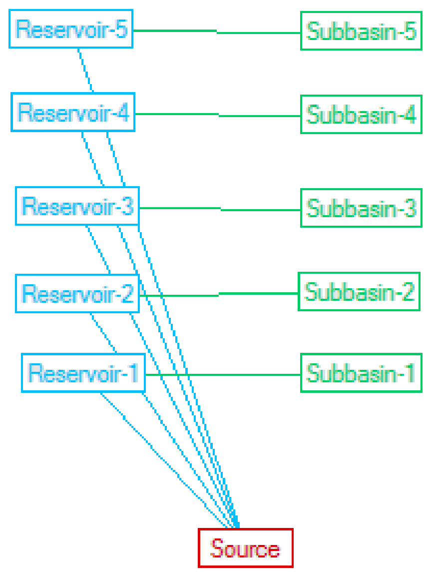

Figure 4. Example of Basin Elements Represented in a Source

example shown illustrates a first order drainage area for a classic watershed containing five 80-acre developments with detention ponds. For the routing reaches in the basin models, a value of 0.045 was assumed for Manning's *n*. The stream channels were assumed to be prismatic in shape with 3:1 side slopes. The second order stream channels were assumed to have a channel bottom width of 10 feet and the third order stream channels were assumed to have a channel bottom width of 15 feet.

To evaluate development stage, basin models were developed with 25%, 50%, and 75% of the watershed developed. Full development was evaluated using results from the analysis of development intensity. Basin models for each partial development stage were developed for each combination of watershed shape, watershed slope, and development size. The basin models were developed using the appropriate post-development subbasin from Table 7 and the appropriate detention pond from Table 9. Each case was evaluated with detention and without detention. Development was assumed to begin at the most downstream area in the watershed and proceed upstream. The evaluation of development stage assumed a medium

Table 10. Sources Developed for Each % Developed for Analysis of Development Stage

Watershed	% Developed	Developed Sources
Classic	25	13-16
	50	9-16
	75	5-16
Elongated	25	20-25
	50	14-25
	75	8-25

development intensity. The modeling runs for the evaluation of development stage are summarized in appendix Table A-3. The naming convention is the same as for the subbasins, with the addition of “25”, “50”, or “75” to designate the development stage. To create the basin models for each partial development stage, the appropriate sources were developed and the remaining sources were set to pre-development conditions. Table 10 lists the sources which were developed for each stage of development for the classic and elongated watersheds.

The evaluation of development sequence was the same as for development stage except development was assumed to begin at the most upstream area in the watershed and proceed downstream. The modeling runs for the evaluation of development sequence are summarized in appendix Table A-4. The naming convention is the same as for development stage, with the addition of “U” to designate that the development began in the upstream portion of the watershed. Table 11 lists the sources which were developed for each stage of development for the classic and elongated watersheds.

Modeling of the Ten Mile Creek Watershed

The Ten Mile Creek watershed is located in Knox County, Tennessee on the western edge of the city limits of Knoxville. A portion of the watershed lies within the city limits. The Ten Mile Creek watershed was selected as the “real-world” test

Table 11. Sources Developed for Each % Developed for Analysis of Development Sequence

Watershed	% Developed	Developed Sources
Classic	25	1-4
	50	1-8
	75	1-12
Elongated	25	1-6
	50	1-12
	75	1-18

watershed for this study due to the availability of data necessary to model the watershed. An HEC-1 model of the watershed was developed for Knox County in 2000 by Ogden Environmental and Energy Services (now AMEC Earth and Environmental Inc.). A copy of the HEC-1 model was obtained from the Knox County Stormwater Engineering Department. ArcView files containing subbasin delineations for the HEC-1 model and land use information for the watershed were obtained from AMEC.

The Ten Mile Creek watershed encompasses a total of 15.7 square miles. The watershed is shown in Figure 5. Drainage from the entire watershed flows into an active sinkhole system (Kung 1980), as indicated in Figure 5. For the purposes of this analysis it was not desirable to include the lower portion of the watershed due to the irregular drainage pattern. Therefore, only the portion of the watershed which drains to Junction 05060D, as shown in Figure 5, was considered. The Ebenezer Branch basin was not included. The total watershed area to Junction 05060D is 12.9 square miles.

The length of the main stream to Junction 05060D is approximately 5.6 miles, giving the portion of the watershed being considered a shape factor of 0.41. Most of the Ten Mile Creek watershed has overland slopes ranging primarily from 0 to 12% (Kung 1980). Some small areas of moderate slopes of 12 to 25% and steep slopes of greater than 25% do occur on ridges in the watershed (Kung 1980). The average

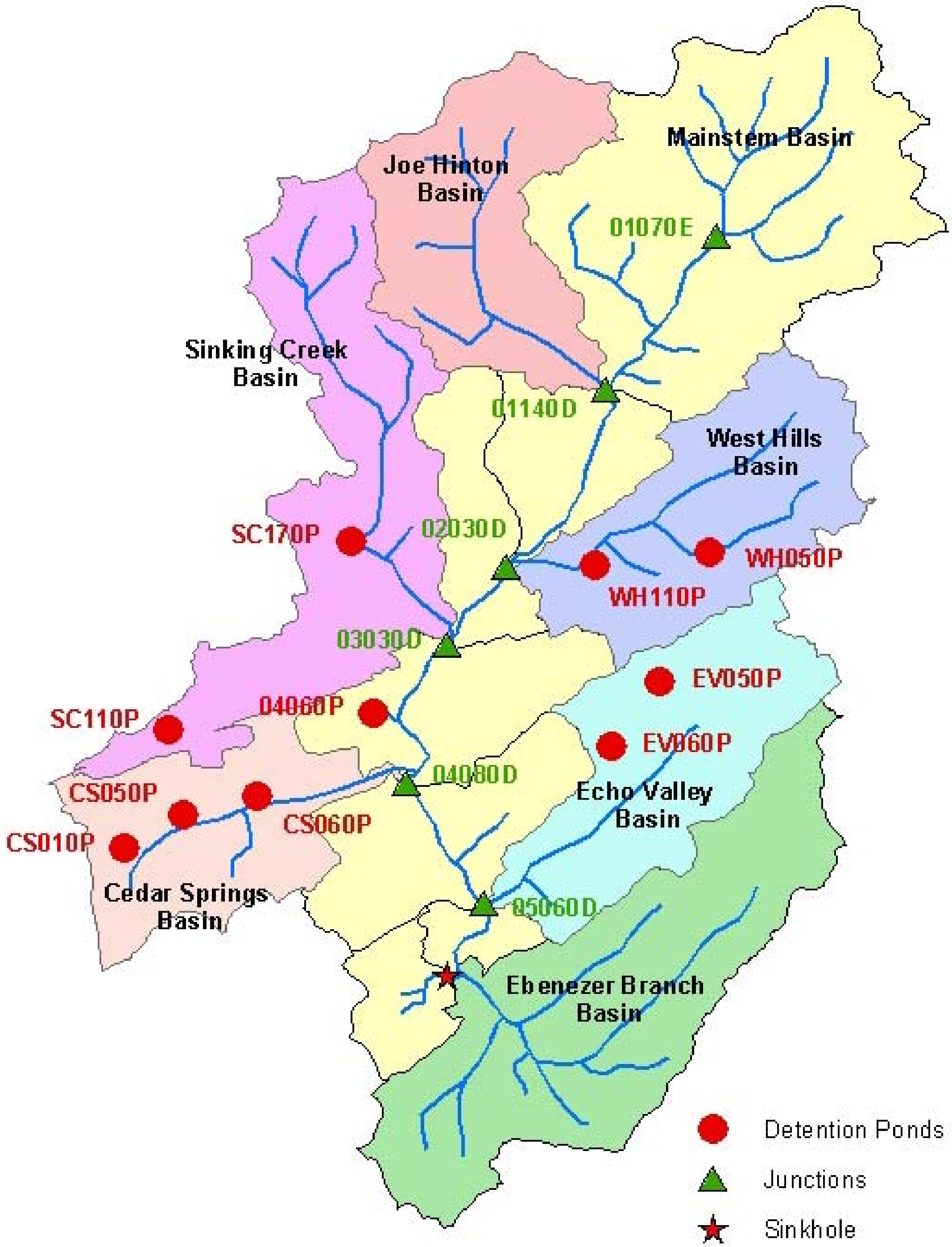

Figure 5. Ten Mile Creek Watershed (Created from AMEC 2000, 2003)

main channel slope was calculated from the USGS topographic map of the area and was found to be 2.1%.

The existing land use within the Ten Mile Creek watershed is broken down in Table 12 and is shown in Figure 6. Residential areas, wooded areas, and open land are the predominant land uses in the upper portion of the watershed. More intense land uses are concentrated in the central portion of the watershed along Interstate 40 and Kingston Pike. Approximately 80% of the watershed is developed, while the remaining 20% consists of wooded or open areas. The area weighted curve number of the entire watershed is 75.8. The weighted curve number of each basin within the watershed is shown in Table 13.

The portion of the Ten Mile Creek watershed that was considered in this analysis is divided into 84 subbasins in the HEC-1 model with an average area of 92 acres. The smallest subbasin has an area of 28 acres and the largest has an area of 204 acres. The minimum curve number of an individual subbasin is 63 and the maximum curve number of an individual subbasin is 93. The HEC-1 model of this portion of the Ten Mile Creek watershed includes ten detention ponds, as shown in Figure 5. These detention ponds are listed in Table 14 with the drainage area that contributes to each pond and the curve number of the contributing area. The total

Table 12. Existing Land Use in the Ten Mile Creek Watershed (AMEC 2003)

Land Use	% of Area
Commercial	13.7
Disturbed/Transitional	1.8
Impervious	0.75
Industrial	0.03
Meadow	2.6
Open Land	7.3
Residential (High Density)	11.5
Residential (Medium Density)	42.9
Residential (Low Density)	9.6
Water	0.17
Woods (Thick Cover)	7.2
Woods (Thin Cover)	2.4

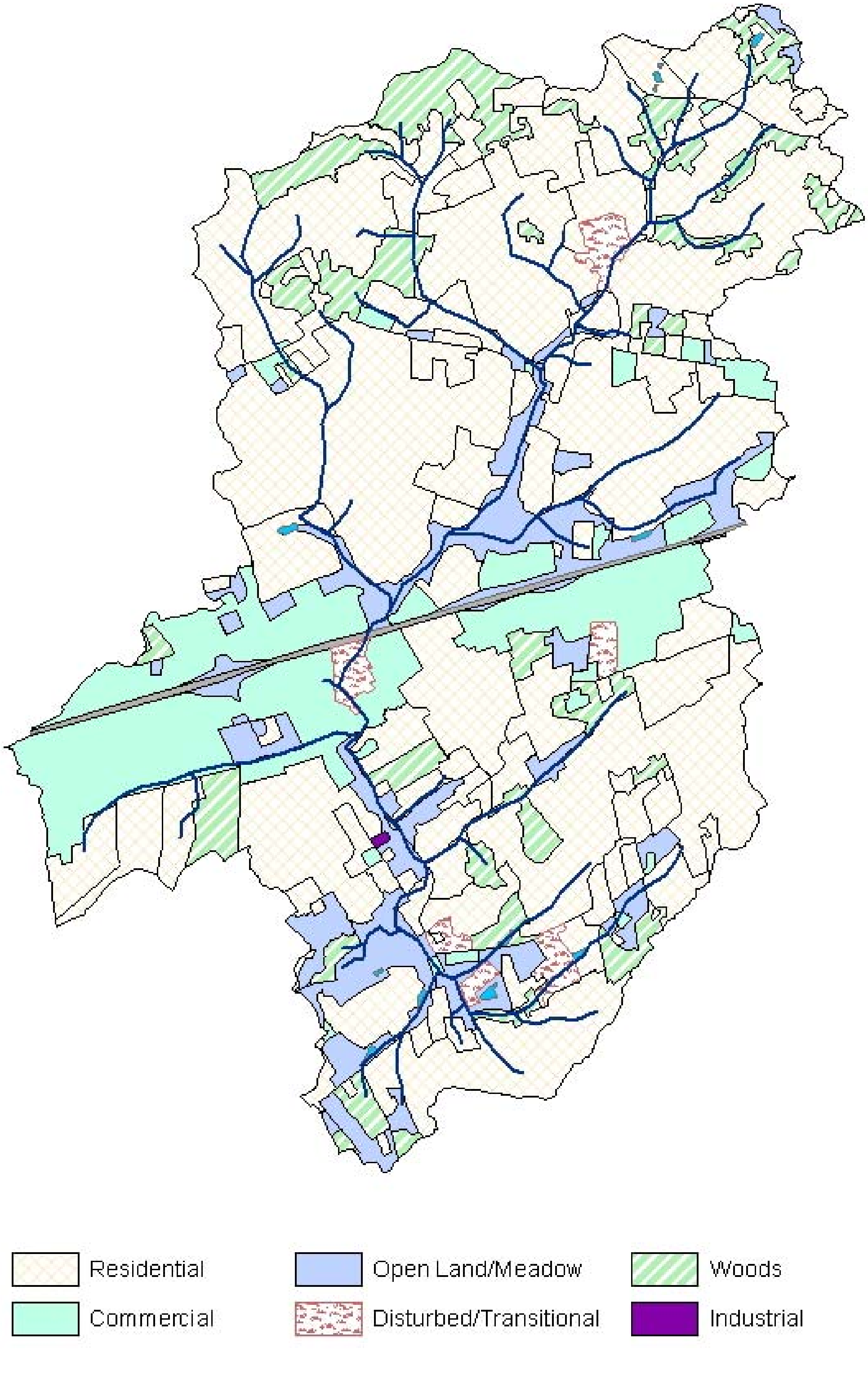

Figure 6. Existing Land Use within the Ten Mile Creek Watershed (Created from AMEC 2003)

Table 13. Weighted Curve Numbers of Basins in Ten Mile Creek Watershed (AMEC 2003)

Basin	**Weighted CN**
Joe Hinton	69.9
West Hills	77.4
Sinking Creek	78.1
Cedar Springs	79.9
Echo Valley	79.1
Mainstem	74.1

Table 14. Detention Ponds in the Ten Mile Creek Watershed (AMEC 2000)

Detention Pond	**Drainage Area, acres**	**CN**
WH050P	274.56	76
WH110P	62.72	84
SC170P	84.48	82
SC110P	83.2	93
04060P	95.36	92
CS010P	140.16	92
CS050P	43.52	93
CS060P	56.96	93
EV050P	156.16	88
EV060P	273.28	83

drainage area that is controlled by these ten detention ponds represents approximately 15% of the portion of the Ten Mile Creek watershed that was considered. The weighted curve number of the area controlled by detention is 85, which is significantly higher than the weighted curve number of the entire watershed. The names of the detention ponds are the names used in the HEC-1 model and include an abbreviation of the basin name, the subbasin number, and a letter indicating the HEC-1 computational operation.

For the purposes of this analysis, the HEC-1 model was imported into HEC-HMS. The resulting basin model of the Ten Mile Creek watershed was then modeled using the same meteorologic model and control specifications file that were used for the modeling of the synthetic watersheds. The HEC-1 model used the SCS curve number method to calculate losses and the Clark unit hydrograph method to transform precipitation excess into direct runoff. A portion of the channel routing was accomplished using the Muskingum-Cunge method. The remainder of the channel routing was accomplished using the Modified-Puls, or storage routing, method. All of this was imported directly into HEC-HMS with no changes.

The HEC-1 model, and resulting HEC-HMS model, represented existing conditions within the Ten Mile Creek watershed. In order to evaluate the effects of the detention ponds in the watershed in the same manner that was done for the synthetic watersheds, it was also desirable to model the developed watershed without detention and to model “pre-development” conditions within the watershed. In order to model developed conditions without detention, the ten detention ponds listed above were simply deleted from the HEC-HMS model. Pre-development conditions were modeled simply by changing the curve number of each subbasin to an average curve number representative of pre-development conditions.

No previous modeling of pre-development conditions had been conducted in the watershed, so estimating pre-development conditions within the watershed required making some assumptions based on limited available information. Two different approaches were considered. For the first approach, it was assumed that the pre-development land use within the basin consisted of 50% woods in fair hydrologic

condition and 50% meadow. Based on the information obtained from AMEC, the soils in the Ten Mile Creek watershed consist of 79.9% in hydrologic soil group B, 13.5% in hydrologic soil group C, and 6.6% in hydrologic soil group D. This yielded a weighted curve number for the watershed of 62 (SCS 1986).

Rather than trying to guess at what might be representative of pre-development conditions in the Ten Mile Creek watershed, the idea behind the second approach was to generate "pre-development" conditions that matched as closely as possible the peak outflow from the ten detention ponds in the watershed. This approach was based on the assumption that the peak outflow from each detention pond was designed to be less than or equal to the pre-development peak flow from the subbasin. Using a trial and error, a representative curve number of 73 was found to generate "pre-development" peak flows for which eight of the ten detention ponds could be considered as successfully limiting the peak flow from that subbasin to the pre-development peak.

Modeling results for the Ten Mile Creek watershed were tabulated at each of the junctions shown in Figure 5. A junction was located at the confluence of each major tributary to Ten Mile Creek. The junction names are the names from the HEC-1 model.

CHAPTER 4
RESULTS AND DISCUSSION

Synthetic Watersheds

For each modeling run in the sensitivity analysis, the peak flow and time of peak were compiled at each junction shown in Figures 2 and 3 for the classic and elongated watersheds, respectively. These results are provided in their entirety in the appendix. Results from the analysis of development intensity are given in appendix Tables A-5 and A-6, results from the analysis of development stage are given in appendix Tables A-7 and A-8, and results from the analysis of development sequence are given in appendix Tables A-9 and A-10.

For each modeling run, the effectiveness of the detention ponds in maintaining pre-development peak flows throughout the watershed was determined by comparing the post-development peak flows with detention to the pre-development peak flows. Using the results given in appendix Tables A-5 through A-10, the percent increase in the post-development peak flow with detention over the pre-development peak flow was calculated at each junction shown in Figures 2 and 3. These results are presented in the following section along with a general discussion of the trends that were observed. The subsequent section then evaluates in more detail the influence of development intensity, development stage, development sequence, development size, watershed shape, and watershed slope.

Results of Modeling Runs and General Trends

As noted above, the percent increase in the post-development peak flow with detention over the pre-development peak flow was calculated at each junction shown in Figures 2 and 3 for each modeling run. These results are presented below. The results are organized into three sections: evaluation of development intensity,

evaluation of development stage, and evaluation of development sequence, in accordance with the way the modeling runs were organized.

Evaluation of Development Intensity

Three levels of development intensity were considered in the sensitivity analysis: low, medium, and high. Each development intensity was evaluated for each combination of watershed shape, watershed slope, and development size. The analysis of development intensity assumed a fully developed watershed. Table 15 shows the percent increase in post-development peak flows with detention over the pre-development peak flows at the junctions located throughout the classic watersheds. The percent increase is shown for each development intensity and for each combination of watershed slope and development size. The percent increases in peak flows in the elongated watersheds are shown in Table 16. Note that negative numbers indicate a reduction below the pre-development peak flow and a value of zero indicates that the post-development peak flow with detention is exactly the same as the pre-development peak flow.

The values in Tables 15 and 16 show that for the fully developed watersheds, the detention ponds for low, medium, and high development intensities were fairly successful in maintaining pre-development peak flows on the second-order streams. This was true regardless of development size, watershed shape, or watershed slope. For the most part, the peak flows on the second order streams with detention in place were slightly below the pre-development peak flows. Some very slight increases in peak flows did begin to appear at the more downstream points on the second order streams.

Pre-development peak flows were not maintained on the main stream channel in any case. The peak flows at the most upstream points on the main channels, Junction D in the classic watersheds and Junction E in the elongated watersheds, were only slightly greater than the pre-development peak flows, but the peak flow impacts

Table 15. % Increase in Peak Flows over Pre-Dev. Conditions in Classic Watersheds - Evaluation of Dev. Intensity

		Junction											
Watershed	**Run**	**2A**	**2B**	**2C**	**2D**	**2E**	**2F**	**2G**	**2H**	**2I**	**2J**	**2K**	**2L**
Classic Mild	CM80LD	-0.8	-0.4	0.3	-0.8	-0.4	0.3	-0.8	-0.4	0.3	-0.8	-0.4	0.3
	CM80MedD	-0.7	-0.2	0.8	-0.7	-0.2	0.8	-0.7	-0.2	0.8	-0.7	-0.2	0.8
	CM80HD	-0.6	0.1	1.2	-0.6	0.1	1.2	-0.6	0.1	1.2	-0.6	0.1	1.2
	CM20LD	-1.3	-0.8	-0.1	-1.3	-0.8	-0.1	-1.3	-0.8	-0.1	-1.3	-0.8	-0.1
	CM20MedD	-1.2	-0.6	0.3	-1.2	-0.6	0.3	-1.2	-0.6	0.3	-1.2	-0.6	0.3
	CM20HD	-1.4	-0.7	0.5	-1.4	-0.7	0.5	-1.4	-0.7	0.5	-1.4	-0.7	0.5
Classic Steep	CS80LD	-0.9	-0.3	0.5	-0.9	-0.3	0.5	-0.9	-0.3	0.5	-0.9	-0.3	0.5
	CS80MedD	-0.8	-0.2	0.8	-0.8	-0.2	0.8	-0.8	-0.2	0.8	-0.8	-0.2	0.8
	CS80HD	-0.9	-0.2	1.0	-0.9	-0.2	1.0	-0.9	-0.2	1.0	-0.9	-0.2	1.0
	CS20LD	-0.9	-0.7	-0.2	-0.9	-0.7	-0.2	-0.9	-0.7	-0.2	-0.9	-0.7	-0.2
	CS20MedD	-0.8	-0.3	0.7	-0.8	-0.3	0.7	-0.8	-0.3	0.7	-0.8	-0.3	0.7
	CS20HD	-0.6	0.1	1.2	-0.6	0.1	1.2	-0.6	0.1	1.2	-0.6	0.1	1.2
Watershed	**Run**	**D**	**C**	**B**	**A**								
Classic Mild	CM80LD	0.4	2.3	5.8	6.4								
	CM80MedD	0.9	3.5	7.9	8.9								
	CM80HD	1.4	4.6	10.0	11.1								
	CM20LD	0.0	2.1	5.4	5.9								
	CM20MedD	0.5	3.2	7.7	8.7								
	CM20HD	0.6	3.9	9.3	10.5								
Classic Steep	CS80LD	0.6	2.5	5.5	5.7								
	CS80MedD	1.0	3.2	7.0	7.3								
	CS80HD	1.2	3.7	8.1	8.4								
	CS20LD	0.0	1.3	4.0	4.1								
	CS20MedD	0.9	3.1	7.0	7.2								
	CS20HD	1.4	4.0	8.4	8.7								

Table 16. % Increase in Peak Flows over Pre-Dev. Conditions in Elongated Watersheds - Evaluation of Dev. Intensity

		Junction												
Watershed	**Run**	**2A**	**2B**	**2C**	**2D**	**2E**	**2F**	**2G**	**2H**	**2I**	**2J**	**2K**	**2L**	**2M**
Elongated Mild	EM80LD	-0.8	-0.4	0.1	0.8	-0.8	-0.4	0.1	0.8	-0.8	-0.4	0.1	0.8	-0.8
	EM80MedD	-0.8	-0.3	0.4	1.3	-0.8	-0.3	0.4	1.3	-0.8	-0.3	0.4	1.3	-0.8
	EM80HD	-0.4	0.2	1.1	2.2	-0.4	0.2	1.1	2.2	-0.4	0.2	1.1	2.2	-0.4
	EM20LD	-0.6	-0.2	0.4	1.3	-0.6	-0.2	0.4	1.3	-0.6	-0.2	0.4	1.3	-0.6
	EM20MedD	-1.2	-0.7	0.1	1.1	-1.2	-0.7	0.1	1.1	-1.2	-0.7	0.1	1.1	-1.2
	EM20HD	-1.1	-0.6	0.4	1.6	-1.1	-0.6	0.4	1.6	-1.1	-0.6	0.4	1.6	-1.1
Elongated Steep	ES80LD	-0.9	-0.4	0.1	0.8	-0.9	-0.4	0.1	0.8	-0.9	-0.4	0.1	0.8	-0.9
	ES80MedD	-0.8	-0.2	0.5	1.5	-0.8	-0.2	0.5	1.5	-0.8	-0.2	0.5	1.5	-0.8
	ES80HD	-0.8	-0.2	0.7	1.8	-0.8	-0.2	0.7	1.8	-0.8	-0.2	0.7	1.8	-0.8
	ES20LD	-1.0	-0.7	-0.4	0.2	-1.0	-0.7	-0.4	0.2	-1.0	-0.7	-0.4	0.2	-1.0
	ES20MedD	-0.5	0.1	0.8	1.7	-0.5	0.1	0.8	1.7	-0.5	0.1	0.8	1.7	-0.5
	ES20HD	-0.5	0.2	1.0	2.2	-0.5	0.2	1.0	2.2	-0.5	0.2	1.0	2.2	-0.5
Watershed	**Run**	**2N**	**2O**	**2P**	**2Q**	**2R**	**2S**	**2T**	**E**	**D**	**C**	**B**	**A**	
Elongated Mild	EM80LD	-0.4	0.1	0.8	-0.8	-0.4	0.1	0.8	0.9	4.8	10.4	16.3	17.1	
	EM80MedD	-0.3	0.4	1.3	-0.8	-0.3	0.4	1.3	1.5	6.7	14.6	23.0	24.4	
	EM80HD	0.2	1.1	2.2	-0.4	0.2	1.1	2.2	2.4	8.8	18.7	30.0	31.9	
	EM20LD	-0.2	0.4	1.3	-0.6	-0.2	0.4	1.3	1.4	5.8	12.1	18.5	19.3	
	EM20MedD	-0.7	0.1	1.1	-1.2	-0.7	0.1	1.1	1.2	6.8	15.3	24.4	25.8	
	EM20HD	-0.6	0.4	1.6	-1.1	-0.6	0.4	1.6	1.7	8.3	18.5	30.2	32.2	
Elongated Steep	ES80LD	-0.4	0.1	0.8	-0.9	-0.4	0.1	0.8	0.9	4.2	9.4	14.8	15.0	
	ES80MedD	-0.2	0.5	1.5	-0.8	-0.2	0.5	1.5	1.6	5.8	12.5	19.9	20.3	
	ES80HD	-0.2	0.7	1.8	-0.8	-0.2	0.7	1.8	1.9	6.9	14.8	23.9	24.3	
	ES20LD	-0.7	-0.4	0.2	-1.0	-0.7	-0.4	0.2	0.2	3.3	8.6	14.2	14.4	
	ES20MedD	0.1	0.8	1.7	-0.5	0.1	0.8	1.7	1.8	6.2	13.2	20.8	21.1	
	ES20HD	0.2	1.0	2.2	-0.5	0.2	1.0	2.2	2.3	7.4	15.7	25.2	25.7	

continued to increase at points further downstream on the main channel and worsened with increasing development intensity. The post-development peak flows with detention were significantly greater than the pre-development peak flows at the watershed outlets, designated as Junction A. The percent increase in peak flows over pre-development conditions at Junction A ranged from 4.1% for the classic watershed with steep slopes, 20 acre developments and a low development intensity to 32.2% for the elongated watershed with mild slopes, 20 acre developments, and a high development intensity.

Evaluation of Development Stage

Four levels of development were considered in the evaluation of development stage: 25%, 50%, 75%, and 100%. These percentages refer to the percent of the watershed that was developed. Development was assumed to begin at the most downstream area in the watershed and proceed upstream. Each development stage was evaluated for each combination of watershed shape, watershed slope, and development size. Full development was evaluated using results from the analysis of development intensity, assuming medium development intensity. Table 17 shows the percent increase in post-development peak flows with detention over the pre-development peak flows at the junctions located throughout the classic watersheds. The percent increase is shown for each development stage and for each combination of watershed slope and development size. The percent increases in peak flows in the elongated watersheds are shown in Table 18.

The values in Tables 17 and 18 show that only 25% development of the downstream portion of the watersheds was enough to cause substantial increases in peak flows over pre-development conditions on the main channel. This was true for every combination of development size, watershed shape, and watershed slope. As development moved upstream, the impacts also moved upstream and impacts at points downstream were further aggravated.

Table 17. % Increase in Peak Flows over Pre-Development Conditions in Classic Watersheds - Evaluation of Development Stage

		Junction											
Watershed	**Run**	**2A**	**2B**	**2C**	**2D**	**2E**	**2F**	**2G**	**2H**	**2I**	**2J**	**2K**	**2L**
Classic Mild	CM8025D	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-0.7	-0.2	0.8
	CM8050D	0.0	0.0	0.0	0.0	0.0	0.0	-0.7	-0.2	0.8	-0.7	-0.2	0.8
	CM8075D	0.0	0.0	0.0	-0.7	-0.2	0.8	-0.7	-0.2	0.8	-0.7	-0.2	0.8
	CM80MedD	-0.7	-0.2	0.8	-0.7	-0.2	0.8	-0.7	-0.2	0.8	-0.7	-0.2	0.8
	CM2025D	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-1.2	-0.6	0.3
	CM2050D	0.0	0.0	0.0	0.0	0.0	0.0	-1.2	-0.6	0.3	-1.2	-0.6	0.3
	CM2075D	0.0	0.0	0.0	-1.2	-0.6	0.3	-1.2	-0.6	0.3	-1.2	-0.6	0.3
	CM20MedD	-1.2	-0.6	0.3	-1.2	-0.6	0.3	-1.2	-0.6	0.3	-1.2	-0.6	0.3
Classic Steep	CS8025D	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-0.8	-0.2	0.8
	CS8050D	0.0	0.0	0.0	0.0	0.0	0.0	-0.8	-0.2	0.8	-0.8	-0.2	0.8
	CS8075D	0.0	0.0	0.0	-0.8	-0.2	0.8	-0.8	-0.2	0.8	-0.8	-0.2	0.8
	CS80MedD	-0.8	-0.2	0.8	-0.8	-0.2	0.8	-0.8	-0.2	0.8	-0.8	-0.2	0.8
	CS2025D	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-0.8	-0.3	0.7
	CS2050D	0.0	0.0	0.0	0.0	0.0	0.0	-0.8	-0.3	0.7	-0.8	-0.3	0.7
	CS2075D	0.0	0.0	0.0	-0.8	-0.3	0.7	-0.8	-0.3	0.7	-0.8	-0.3	0.7
	CS20MedD	-0.8	-0.3	0.7	-0.8	-0.3	0.7	-0.8	-0.3	0.7	-0.8	-0.3	0.7

Table 17. Continued

		Junction			
Watershed	**Run**	**D**	**C**	**B**	**A**
Classic Mild	CM8025D	0.0	0.0	5.5	5.7
	CM8050D	0.0	3.0	7.8	8.3
	CM8075D	-0.8	2.6	7.5	8.2
	CM80MedD	0.9	3.5	7.9	8.9
	CM2025D	0.0	0.0	4.4	4.6
	CM2050D	0.0	2.2	6.2	6.7
	CM2075D	0.2	2.6	6.9	7.6
	CM20MedD	0.5	3.2	7.7	8.7
Classic Steep	CS8025D	0.0	0.0	5.2	5.3
	CS8050D	0.0	2.7	7.2	7.4
	CS8075D	-1.8	1.7	6.4	6.6
	CS80MedD	1.0	3.2	7.0	7.3
	CS2025D	0.0	0.0	4.1	4.1
	CS2050D	0.0	2.1	5.6	5.7
	CS2075D	0.3	2.4	6.0	6.2
	CS20MedD	0.9	3.1	7.0	7.2

Table 18. % Increase in Peak Flows over Pre-Development Conditions in Elongated Watersheds - Evaluation of Development Stage

		Junction												
Watershed	**Run**	**2A**	**2B**	**2C**	**2D**	**2E**	**2F**	**2G**	**2H**	**2I**	**2J**	**2K**	**2L**	**2M**
Elongated Mild	EM8025D	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	EM8050D	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.7	1.9	-0.8
	EM8075D	0.0	0.0	0.0	0.0	0.0	-0.1	0.7	1.9	-0.8	-0.3	0.4	1.3	-0.8
	EM80MedD	-0.8	-0.3	0.4	1.3	-0.8	-0.3	0.4	1.3	-0.8	-0.3	0.4	1.3	-0.8
	EM2025D	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	EM2050D	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.6	1.5	-1.2
	EM2075D	0.0	0.0	0.0	0.0	0.0	0.0	0.6	1.4	-1.2	-0.7	0.1	1.1	-1.2
	EM20MedD	-1.2	-0.7	0.1	1.1	-1.2	-0.7	0.1	1.1	-1.2	-0.7	0.1	1.1	-1.2
Elongated Steep	ES8025D	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	ES8050D	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.5	1.6	-0.8
	ES8075D	0.0	0.0	0.0	0.0	0.0	-0.5	0.3	1.6	-0.8	-0.2	0.5	1.5	-0.8
	ES80MedD	-0.8	-0.2	0.5	1.5	-0.8	-0.2	0.5	1.5	-0.8	-0.2	0.5	1.5	-0.8
	ES2025D	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	ES2050D	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.5	1.4	-0.5
	ES2075D	0.0	0.0	0.0	0.0	0.0	0.1	0.5	1.3	-0.5	0.1	0.8	1.7	-0.5
	ES20MedD	-0.5	0.1	0.8	1.7	-0.5	0.1	0.8	1.7	-0.5	0.1	0.8	1.7	-0.5

Table 18. Continued

Watershed	Run	Junction											
		2N	2O	2P	2Q	2R	2S	2T	E	D	C	B	A
Elongated Mild	EM8025D	0.0	0.0	1.3	-0.8	-0.3	0.4	1.3	0.0	0.0	2.2	12.2	12.5
	EM8050D	-0.3	0.4	1.3	-0.8	-0.3	0.4	1.3	0.0	3.5	12.3	21.2	21.7
	EM8075D	-0.3	0.4	1.3	-0.8	-0.3	0.4	1.3	0.5	6.4	14.7	23.2	24.1
	EM80MedD	-0.3	0.4	1.3	-0.8	-0.3	0.4	1.3	1.5	6.7	14.6	23.0	24.4
	EM2025D	0.0	0.0	1.0	-1.2	-0.7	0.1	1.1	0.0	0.0	2.1	11.7	12.0
	EM2050D	-0.7	0.1	1.1	-1.2	-0.7	0.1	1.1	0.0	3.0	11.1	19.8	20.3
	EM2075D	-0.7	0.1	1.1	-1.2	-0.7	0.1	1.1	0.6	5.7	13.6	22.3	23.3
	EM20MedD	-0.7	0.1	1.1	-1.2	-0.7	0.1	1.1	1.2	6.8	15.3	24.4	25.8
Elongated Steep	ES8025D	0.0	0.0	1.1	-0.8	-0.2	0.5	1.5	0.0	0.0	2.3	12.3	12.3
	ES8050D	-0.2	0.5	1.5	-0.8	-0.2	0.5	1.5	0.0	3.3	11.7	20.2	20.3
	ES8075D	-0.2	0.5	1.5	-0.8	-0.2	0.5	1.5	0.1	5.5	13.2	21.2	21.4
	ES80MedD	-0.2	0.5	1.5	-0.8	-0.2	0.5	1.5	1.6	5.8	12.5	19.9	20.3
	ES2025D	0.0	0.0	0.9	-0.5	0.1	0.8	1.7	0.0	0.0	2.0	10.9	10.9
	ES2050D	0.1	0.8	1.7	-0.5	0.1	0.8	1.7	0.0	2.8	10.0	17.6	17.7
	ES2075D	0.1	0.8	1.7	-0.5	0.1	0.8	1.7	0.5	4.8	11.5	18.9	19.1
	ES20MedD	0.1	0.8	1.7	-0.5	0.1	0.8	1.7	1.8	6.2	13.2	20.8	21.1

The peak flow impacts were most severe in the elongated watersheds. At Junctions D and C in the elongated watersheds, the post-development peak flows with detention were actually slightly greater than the post-development peak flows without detention for the 25% and 50% development levels. The percent increases in peak flows with detention over peak flows without detention at these two junctions were in the range of 1.3% to 3.6% and occurred for every combination of development size, watershed shape, and watershed slope.

Evaluation of Development Sequence

Like the evaluation of development stage, four levels of development were considered for the evaluation of development sequence: 25%, 50%, 75%, and 100%. However, development was assumed to begin at the most upstream area in the watershed and proceed downstream. Each level of development was again evaluated for each combination of watershed shape, watershed slope, and development size. Full development was evaluated using results from the analysis of development intensity, assuming medium development intensity. Table 19 shows the percent increase in post-development peak flows with detention over the pre-development peak flows at the junctions located throughout the classic watersheds. The percent increase is shown for each level of development and for each combination of watershed slope and development size. The percent increases in peak flows in the elongated watersheds are shown in Table 20.

The values in Tables 19 and 20 show that when development occurred in the upstream portions of the watersheds, the effects on the rest of the watershed were very different than the pattern of impacts that was generated by development in the downstream portions of the watersheds. For the most part, 25% and 50% development of the upstream portion of the watersheds resulted in peak flows below pre-development levels on the main channel for all combinations of watershed shape and slope with 80-acre developments. For the 75% level of development in the watersheds with 80-acre developments, pre-development peak flows were maintained

Table 19. % Increase in Peak Flows over Pre-Development Conditions in Classic Watersheds - Evaluation of Development Sequence

		Junction											
Watershed	**Run**	**2A**	**2B**	**2C**	**2D**	**2E**	**2F**	**2G**	**2H**	**2I**	**2J**	**2K**	**2L**
Classic Mild	CM8025UD	-0.7	-0.2	0.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	CM8050UD	-0.7	-0.2	0.8	-0.7	-0.2	0.8	0.0	0.0	0.0	0.0	0.0	0.0
	CM8075UD	-0.7	-0.2	0.8	-0.7	-0.2	0.8	-0.7	-0.2	0.8	0.0	0.0	0.0
	CM80MedD	-0.7	-0.2	0.8	-0.7	-0.2	0.8	-0.7	-0.2	0.8	-0.7	-0.2	0.8
	CM2025UD	-1.2	-0.6	0.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	CM2050UD	-1.2	-0.6	0.3	-1.2	-0.6	0.3	0.0	0.0	0.0	0.0	0.0	0.0
	CM2075UD	-1.2	-0.6	0.3	-1.2	-0.6	0.3	-1.2	-0.6	0.3	0.0	0.0	0.0
	CM20MedD	-1.2	-0.6	0.3	-1.2	-0.6	0.3	-1.2	-0.6	0.3	-1.2	-0.6	0.3
Classic Steep	CS8025UD	-0.8	-0.2	0.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	CS8050UD	-0.8	-0.2	0.8	-0.8	-0.2	0.8	0.0	0.0	0.0	0.0	0.0	0.0
	CS8075UD	-0.8	-0.2	0.8	-0.8	-0.2	0.8	-0.8	-0.2	0.8	0.0	0.0	0.0
	CS80MedD	-0.8	-0.2	0.8	-0.8	-0.2	0.8	-0.8	-0.2	0.8	-0.8	-0.2	0.8
	CS2025UD	-0.8	-0.3	0.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	CS2050UD	-0.8	-0.3	0.7	-0.8	-0.3	0.7	0.0	0.0	0.0	0.0	0.0	0.0
	CS2075UD	-0.8	-0.3	0.7	-0.8	-0.3	0.7	-0.8	-0.3	0.7	0.0	0.0	0.0
	CS20MedD	-0.8	-0.3	0.7	-0.8	-0.3	0.7	-0.8	-0.3	0.7	-0.8	-0.3	0.7

Table 19. Continued

		Junction			
Watershed	**Run**	**D**	**C**	**B**	**A**
Classic Mild	CM8025UD	-0.8	-1.2	-1.2	-1.0
	CM8050UD	0.9	-2.1	-2.2	-1.9
	CM8075UD	0.9	3.5	0.3	1.0
	CM80MedD	0.9	3.5	7.9	8.9
	CM2025UD	0.2	0.8	1.6	1.8
	CM2050UD	0.5	2.1	4.1	4.3
	CM2075UD	0.5	3.2	4.5	5.0
	CM20MedD	0.5	3.2	7.7	8.7
Classic Steep	CS8025UD	-1.8	-2.5	-2.8	-2.7
	CS8050UD	1.0	-4.0	-5.1	-4.9
	CS8075UD	1.0	3.2	-1.9	-1.7
	CS80MedD	1.0	3.2	7.0	7.3
	CS2025UD	0.3	0.8	1.5	1.6
	CS2050UD	0.9	2.6	4.2	4.2
	CS2075UD	0.9	3.1	4.4	4.5
	CS20MedD	0.9	3.1	7.0	7.2

Table 20. % Increase in Peak Flows over Pre-Development Conditions in Elongated Watersheds - Evaluation of Development Sequence

		Junction												
Watershed	**Run**	**2A**	**2B**	**2C**	**2D**	**2E**	**2F**	**2G**	**2H**	**2I**	**2J**	**2K**	**2L**	**2M**
Elongated Mild	EM8025UD	-0.8	-0.3	0.4	1.3	-1.9	-1.9	-1.8	-1.7	0.0	0.0	0.0	0.0	0.0
	EM8050UD	-0.8	-0.3	0.4	1.3	-0.8	-0.3	0.4	1.3	-0.8	-3.1	-3.3	-3.3	0.0
	EM8075UD	-0.8	-0.3	0.4	1.3	-0.8	-0.3	0.4	1.3	-0.8	-0.3	0.4	1.3	-0.8
	EM80MedD	-0.8	-0.3	0.4	1.3	-0.8	-0.3	0.4	1.3	-0.8	-0.3	0.4	1.3	-0.8
	EM2025UD	-1.2	-0.7	0.1	1.1	-0.7	-0.4	-0.1	0.2	0.0	0.0	0.0	0.0	0.0
	EM2050UD	-1.2	-0.7	0.1	1.1	-1.2	-0.7	0.1	1.1	-1.2	-0.7	-0.1	0.5	0.0
	EM2075UD	-1.2	-0.7	0.1	1.1	-1.2	-0.7	0.1	1.1	-1.2	-0.7	0.1	1.1	-1.2
	EM20MedD	-1.2	-0.7	0.1	1.1	-1.2	-0.7	0.1	1.1	-1.2	-0.7	0.1	1.1	-1.2
Elongated Steep	ES8025UD	-0.8	-0.2	0.5	1.5	-2.8	-2.7	-2.6	-2.5	0.0	0.0	0.0	0.0	0.0
	ES8050UD	-0.8	-0.2	0.5	1.5	-0.8	-0.2	0.5	1.5	-0.8	-4.3	-4.7	-4.7	0.0
	ES8075UD	-0.8	-0.2	0.5	1.5	-0.8	-0.2	0.5	1.5	-0.8	-0.2	0.5	1.5	-0.8
	ES80MedD	-0.8	-0.2	0.5	1.5	-0.8	-0.2	0.5	1.5	-0.8	-0.2	0.5	1.5	-0.8
	ES2025UD	-0.5	0.1	0.8	1.7	-0.5	0.0	0.3	0.6	0.0	0.0	0.0	0.0	0.0
	ES2050UD	-0.5	0.1	0.8	1.7	-0.5	0.1	0.8	1.7	-0.5	0.4	1.0	1.5	0.0
	ES2075UD	-0.5	0.1	0.8	1.7	-0.5	0.1	0.8	1.7	-0.5	0.1	0.8	1.7	-0.5
	ES20MedD	-0.5	0.1	0.8	1.7	-0.5	0.1	0.8	1.7	-0.5	0.1	0.8	1.7	-0.5

Table 20. Continued

		Junction											
Watershed	**Run**	**2N**	**2O**	**2P**	**2Q**	**2R**	**2S**	**2T**	**E**	**D**	**C**	**B**	**A**
Elongated Mild	EM8025UD	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-1.4	-2.3	-2.3	-2.2	-1.9
	EM8050UD	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.5	-0.2	-1.4	-1.6	-0.9
	EM8075UD	-0.3	-3.2	-3.8	0.0	0.0	0.0	0.0	1.5	6.7	9.7	8.0	9.0
	EM80MedD	-0.3	0.4	1.3	-0.8	-0.3	0.4	1.3	1.5	6.7	14.6	23.0	24.4
	EM2025UD	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.6	1.8	2.7	3.2	3.3
	EM2050UD	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.2	3.1	4.6	5.5	5.9
	EM2075UD	-0.7	-0.4	0.4	0.0	0.0	0.0	0.0	1.2	6.8	11.1	11.0	12.0
	EM20MedD	-0.7	0.1	1.1	-1.2	-0.7	0.1	1.1	1.2	6.8	15.3	24.4	25.8
Elongated Steep	ES8025UD	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-2.5	-4.1	-4.5	-4.6	-4.6
	ES8050UD	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.6	-1.9	-4.5	-5.5	-5.3
	ES8075UD	-0.2	-4.3	-5.3	0.0	0.0	0.0	0.0	1.6	5.8	7.4	4.3	4.5
	ES80MedD	-0.2	0.5	1.5	-0.8	-0.2	0.5	1.5	1.6	5.8	12.5	19.9	20.3
	ES2025UD	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.2	2.1	2.9	3.2	3.2
	ES2050UD	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.8	3.5	4.7	5.3	5.3
	ES2075UD	0.1	1.0	1.8	0.0	0.0	0.0	0.0	1.8	6.2	9.4	8.8	9.0
	ES20MedD	0.1	0.8	1.7	-0.5	0.1	0.8	1.7	1.8	6.2	13.2	20.8	21.1

at most points on the main channel in the classic watersheds, but were not maintained on the main channel in the elongated watersheds.

When development occurred in the upstream portions of the watersheds, detention was not nearly as effective in maintaining pre-development peak flows on the main channel with the 20-acre developments as it was with the 80-acre developments. This was true for all combinations of watershed shape and slope. With the 20-acre developments, 25% development of the upper portion of the watershed produced relatively minor increases in peak flows over pre-development levels at points on the main channel in the classic watersheds. In the elongated watersheds, 25% development of the upper portion of the watershed produced relatively substantial increases in peak flows over pre-development levels on the main channel. In all cases, these impacts worsened with increasing levels of development.

Influence of Watershed and Development Characteristics

The results presented in the previous section show that the effectiveness of detention ponds in any given watershed was the result of a combination of factors. All of the watershed and development characteristics considered in the sensitivity analysis influenced the results to various degrees. The influence of each watershed and development characteristic on the effectiveness of the detention ponds in maintaining pre-development peak flows throughout a watershed is evaluated in detail below.

Influence of Development Intensity

In the fully developed watersheds, peak flow impacts at each point on the main channel increased with increasing development intensity for each combination of watershed shape, watershed slope, and development size. This occurred even though the peak detention outflow from each developed subbasin was the same regardless of the development intensity. For a given combination of watershed shape,

watershed slope, and development size, the detention ponds were all designed to maintain the peak outflow from each post-development subbasin at the same pre-development level. However, the higher development intensities produce a greater volume of runoff. In order to maintain the pre-development peak flow, the higher development intensities require a greater detention storage volume, which in turn causes the peak flow from the detention pond to occur later than the pre-development peak. For each progressively higher development intensity, the peak flows occur progressively later. The higher flows also occur over a longer period of time than for pre-development conditions. These changes in timing are illustrated in Figure 7, which shows the hydrograph at Junction 2A for pre-development conditions and the detention outflow hydrographs for low, medium, and high development intensities. The example shown is for the elongated watershed with steep slopes and 80-acre developments.

Figure 8 provides an example of how these changes in timing result in peak flows that are higher than the pre-development peaks at points on the main channel and how the peak flow impacts are affected by changing the development intensity. Figure 8 illustrates the creation of the hydrographs at Junction B in the elongated watershed with steep slopes and 80-acre developments. The hydrographs at Junction B are created by combining the outflow hydrographs from Reach C and Reach 2T. The outflow hydrograph from Reach C represents the flow from the entire watershed to that point, while the outflow hydrograph from Reach 2T represents the flow from one second order drainage area at its junction with the main channel. Figure 8 shows the pre-development hydrographs and the low, medium, and high intensity detention outflow hydrographs for Junction B, Reach C, and Reach 2T.

Under pre-development conditions, the peak flow at Junction B occurred approximately midway through the 24-hour storm event at 12:36, exactly the same time as the peak flow from Reach C. The peak flow from reach 2T occurred earlier at 12:18, and the relatively small flow from Reach 2T that was occurring at 12:36 added little to the peak flow at Junction B. For the developed conditions the peak flows at

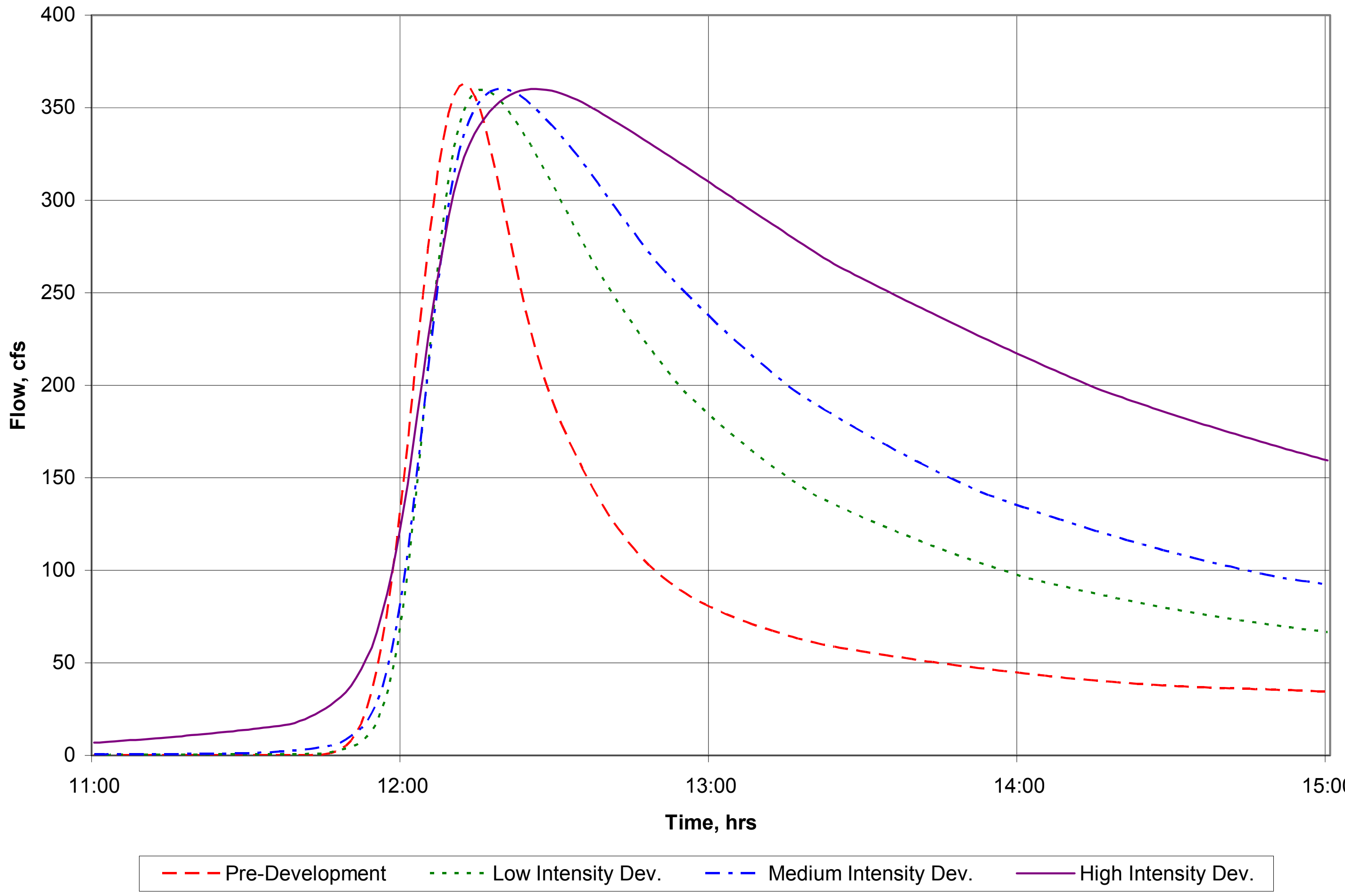

Figure 7. Changes in Timing of Detention Outflow Hydrographs for Varying Development Intensities

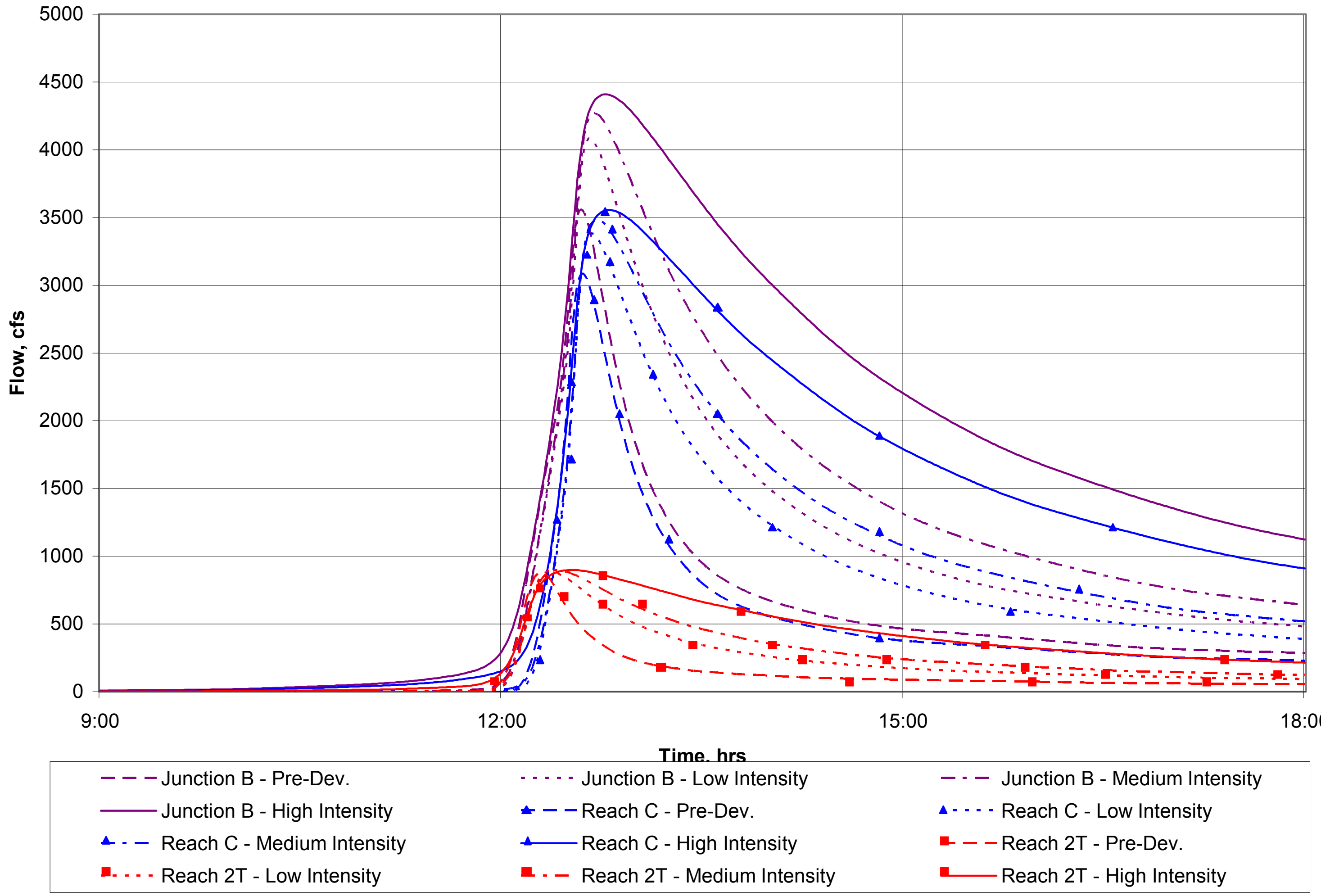

Figure 8. Effect of Development Intensity on Creation of Peak Flow Impacts

Junction B still occur at approximately the same time as the peak flows from Reach C, but the extended peak flows from Reach 2T caused by detention have a greater impact on the peak flows at Junction B. Under pre-development conditions, the flow from Reach 2T represented 13.3% of the peak at Junction B. This percentage increases with increasing development intensity: 17.2% for low intensity, 18.4% for medium intensity, and 19.4% for high intensity.

Influence of Development Stage

When development began in the lower portion of the watersheds and proceeded upstream, the percentage of the watershed that was developed had a significant effect on how well pre-development peak flows were maintained on the main stream channel. The results presented in the previous section show that for every combination of development size, watershed shape, and watershed slope, development of only 25% of the downstream portion of the watersheds was enough to cause substantial increases in peak flows over pre-development conditions on the main channel.

An example of how development of 25% of the lower portion of a watershed can cause peak flows to increase over pre-development conditions at points on the main channel is shown in Figure 9. Figure 9 illustrates how the hydrograph at Junction B is created when 25% development occurs in the lower portion of the classic watershed with mild slopes and 20 acre developments. In the classic watersheds, the 25% development level was represented by the development of the most downstream second order drainage area. The outflow hydrograph from Reach 2L represents the runoff from this second order drainage area. The hydrograph at Junction B is a combination of the outflow hydrograph from Reach 2L and the outflow hydrograph from Reach C, which represents the runoff from the entire watershed to that point. In this case, the outflow hydrograph from Reach C is the same for both the post-development and pre-development conditions because with

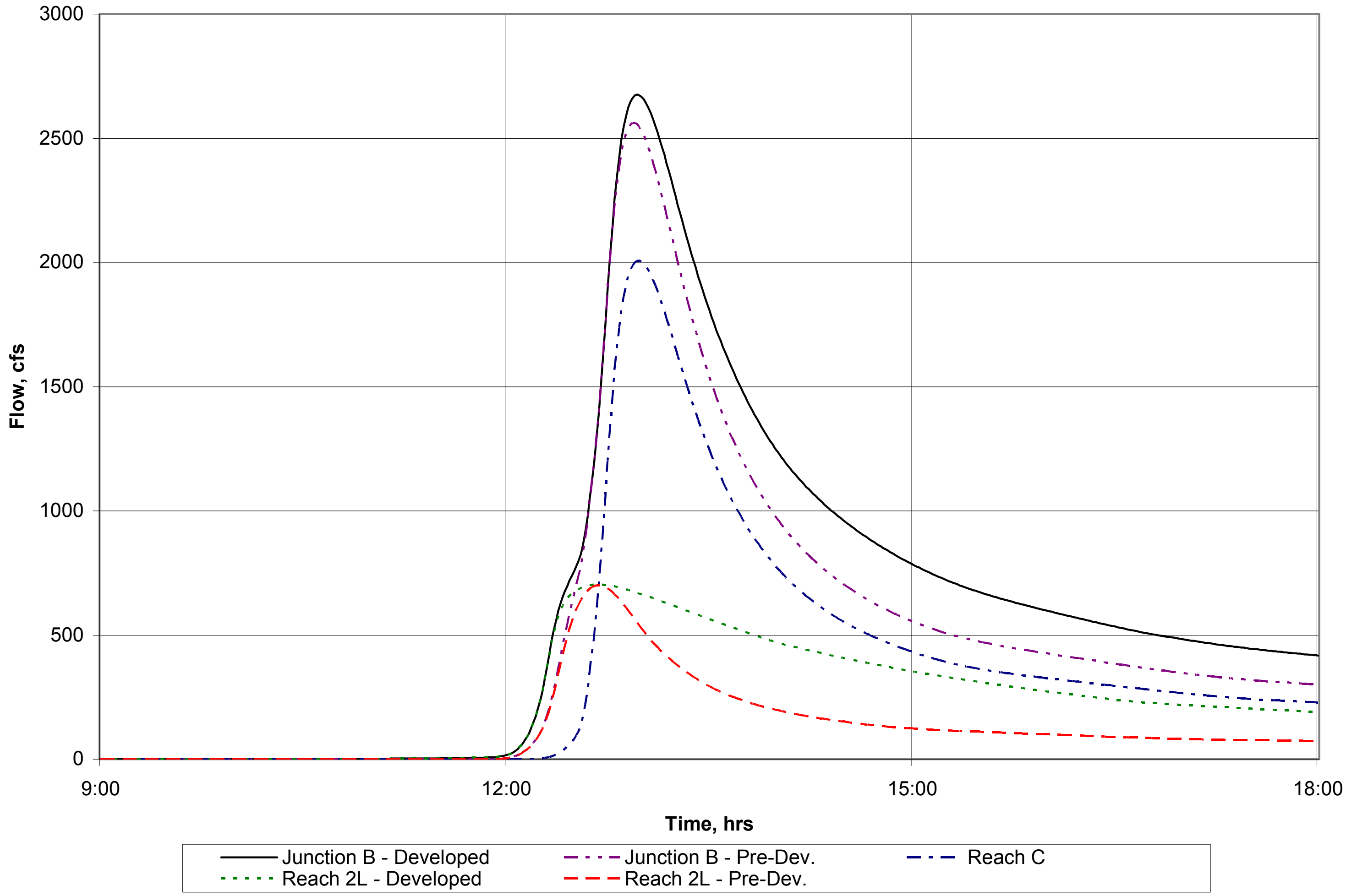

Figure 9. Creation of Peak Flow Impacts on Main Channel by Development of 25% of Watershed

only 25% of the watershed developed, no development has occurred upstream of Reach C.

As expected, the peak outflow from Reach 2L for the post-development with detention case is approximately the same as the pre-development peak. In this case, it happens that the two peaks occur at approximately the same time. However, the detention outflow hydrograph has higher flows for a longer period of time than the pre-development hydrograph. As was explained in the discussion of development intensity, it is this change in timing that again produces higher than pre-development peak flows on the main channel. Figure 9 shows that for both the post-development and pre-development cases, the peak flow from Reach C occurs after the peak from Reach 2L. However, detention causes a higher flow to coincide with the peak from Reach C, resulting in a higher peak flow at Junction B. Under pre-development conditions, the contribution from Reach 2L represents 22.4% of the peak at Junction B. Under the developed conditions with detention, the contribution from Reach 2L increases to 25% of the peak at Junction B.

As the level of development in the watershed increased and development moved upstream, impacts were also created at points further upstream. The process described above caused peak flows on the main channel to increase over the pre-development peak wherever the extended detention outflow hydrograph from a second order drainage basin combined with the pre-development hydrograph from the rest of the watershed. The increased peaks at points further upstream then propagated downstream, aggravating the impacts at points downstream in the watershed. Figure 10 shows how the peak flows on the main channel propagate downstream from Junction D to Junction B, for each level of development. This example again uses the classic watershed with mild slopes and 20 acre developments.

Influence of Development Sequence

The results presented in the previous section show that when development occurred in the upstream portions of the watersheds, the effects on the rest of the

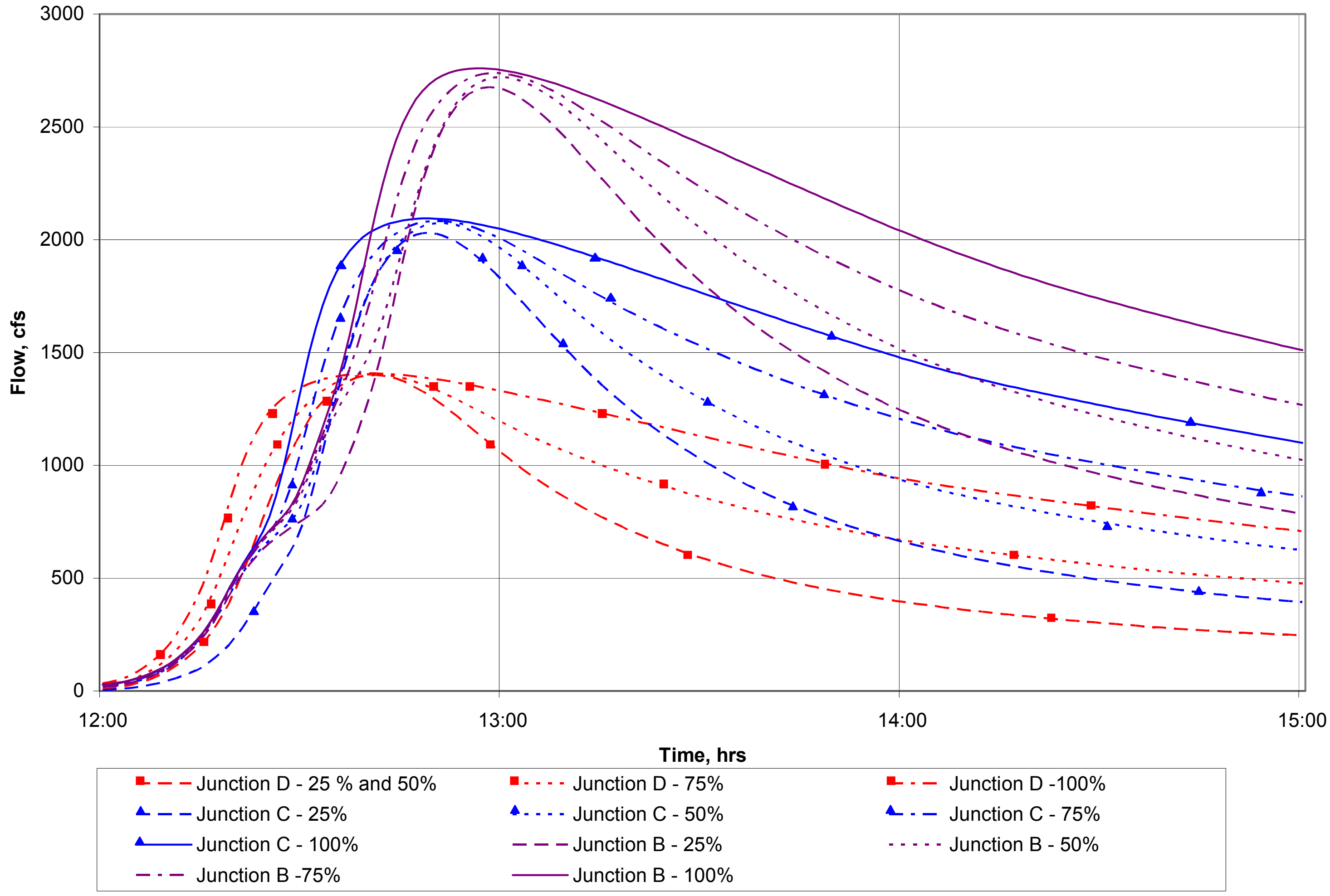

Figure 10. Propagation of Peak Flow Impacts for Varying Levels of Development

watershed were very different than the pattern of impacts that was generated by development in the downstream portions of the watersheds. In some cases, pre-development peak flows were maintained on the main channel, while in other cases they were not. The exact result for a given level of development varied particularly depending on the development size. However, in all cases, any increases in peak flows on the main channel over pre-development levels were significantly smaller when development occurred in the upstream portion of the watershed than when the downstream portion of the watershed was developed.

Figure 11 shows an example of the difference in peak flows generated at the watershed outlet, Junction A, for varying levels of development when development occurred in the upstream portion of the watershed versus the downstream portion. The example shown is for the elongated watershed with mild slopes and 20 acre developments. Figure 11 illustrates that for each level of development, peak flows at the watershed outlet were significantly higher when the development occurred in the downstream portions of the watershed than when it occurred in the upstream portion. It is interesting to note that only 25% development of the downstream portion of the watershed generated approximately the same peak flow at the watershed outlet as 75% development of the upstream portion of the watershed. Also note that the peak flow generated by 75% of the downstream portion of the watershed is nearly as high as the peak generated by 100% development, while the peak flow generated by 75% development of the upper portion of the watershed is significantly less than the fully developed peak.

When development occurred in the downstream portion of the watershed, runoff from the lower portion of the watershed was detained so that it contributed more to the peak from the rest of the watershed. When the upstream portion of the watershed is developed, the opposite effect occurs. Delaying the runoff from the developed areas through the use of detention allows the peak flows from the rest of the watershed to pass before the peak from the developed area arrives. An example of this is shown in Figure 12. Figure 12 illustrates the development of the peak at Junction D for pre-development conditions and for the case of 25% development in

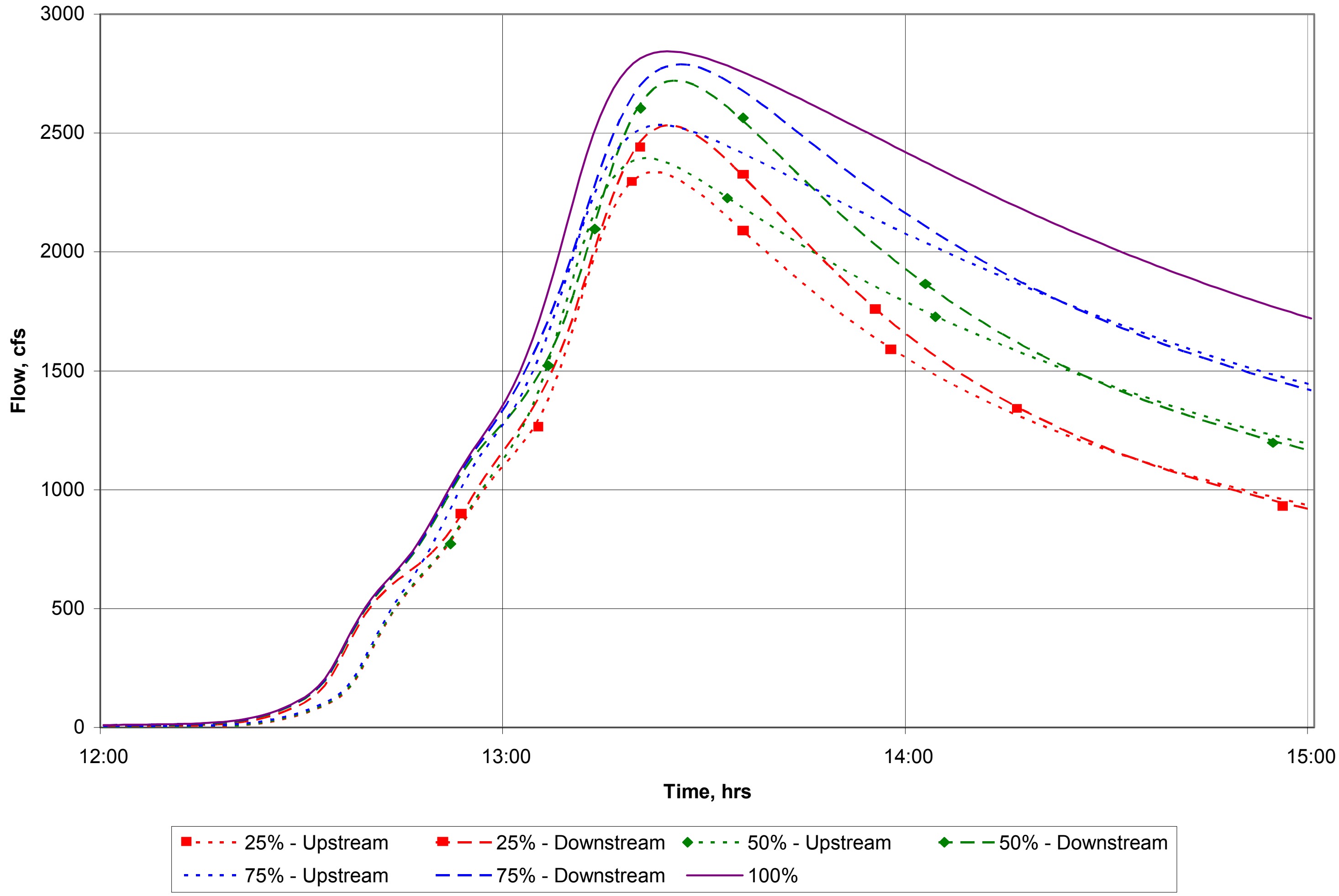

Figure 11. Effect of Development Location on Peak Flows at the Watershed Outlet

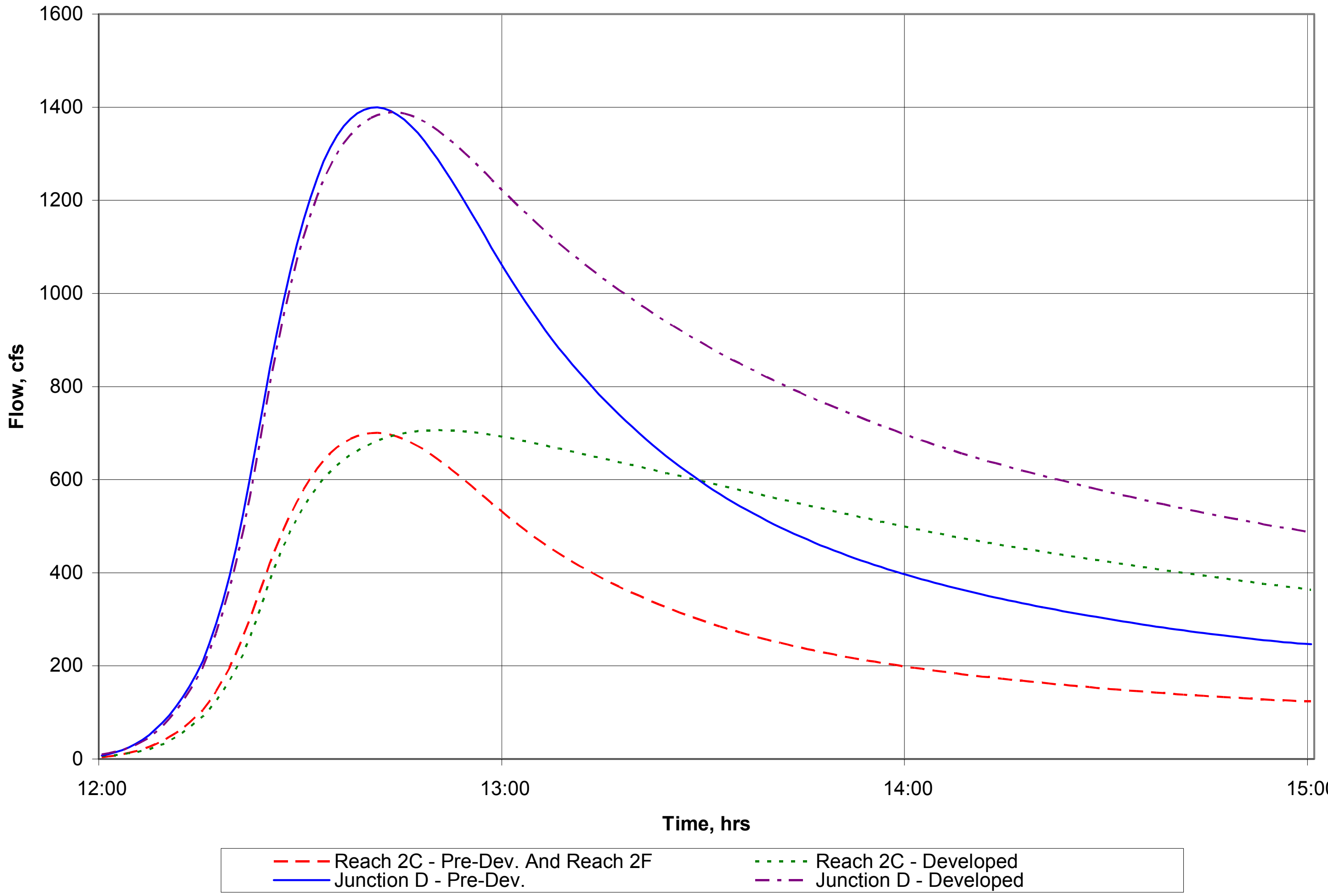

Figure 12. Delay of Peak Flows due to Development in the Upstream Portion of the Watershed

the upper portion of the watershed. The example shown is for the classic watershed with mild slopes and 80 acre developments. Recall from Table 19 that in this watershed, peak flows on the main channel were slightly lower than the pre-development conditions when 25% of the upper watershed was developed.

Junction D is located at the confluence of the two most upstream second order streams. For the 25% development scenario, one of these second order drainage areas was developed and the hydrograph for Reach 2C represents the runoff from this drainage area. The hydrograph for Reach 2F represents the runoff from the other second order drainage area. For the 25% development scenario, no development occurs in that drainage area, so the hydrograph for Reach 2F is the same for pre-development and 25% development conditions. It is also the same as the pre-development hydrograph for Reach 2C.

Figure 12 shows that the pre-development peaks from Reach 2C and Reach 2F occur simultaneously and combine to create the peak at Junction D. When the second order drainage area that contributes to Reach 2C is developed with detention, the peak occurs later than the peak from Reach 2F. As a result, Reach 2C contributes less to the peak at Junction D, and the peak at Junction D is slightly reduced. This same effect results in lower peaks at points further downstream on the main channel.

Influence of Development Size

The size of a development is a factor in the size and design of the detention pond needed to control the runoff from that development. Other factors being equal, a larger development will produce a larger volume of runoff than a smaller development, requiring a greater detention storage volume in order to limit the peak flow from the development to the pre-development level. The greater storage volume required by the larger development will also cause the peak outflow from the detention pond to occur later than the peak outflow from a smaller development. These characteristics are evidenced by the detention pond design data shown in Table 9. Figure 13 illustrates the hydrographs at Junction 2A for 80-acre developments and

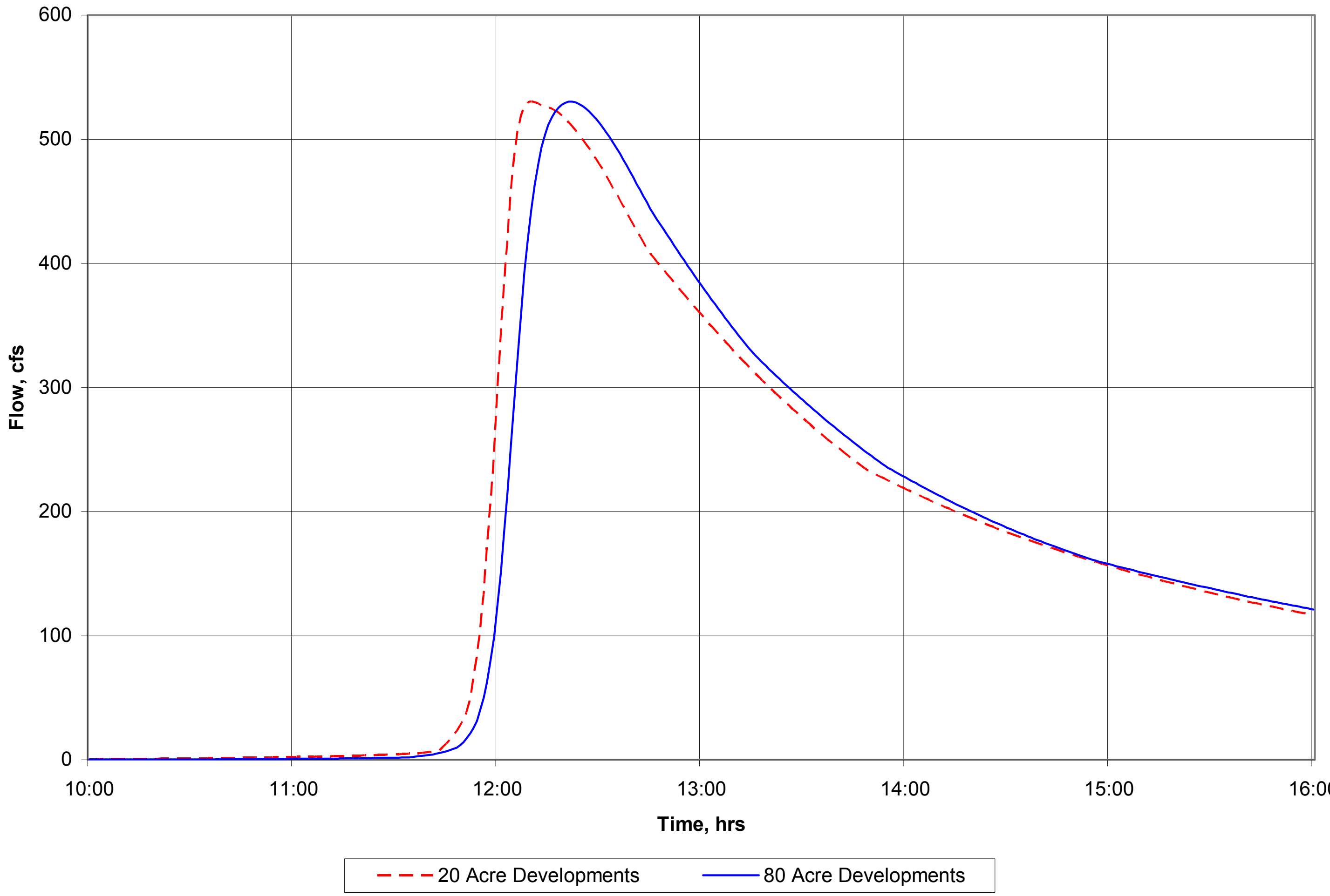

Figure 13. Effect of Development Size on Timing of Peak Flows for a Fixed Drainage Area

20-acre developments in the classic watershed with steep slopes and medium development intensity. This example shows that the outflow hydrograph from a first order drainage area peaks later when that drainage area is divided into a smaller number of larger developments than when it is divided into a larger number of smaller developments. The total runoff volume at Junction 2A is the same for both the 20-acre and 80-acre developments because the contributing area is the same in both cases, only divided into a different number of developments.

Table 21 provides a summary comparison of the effectiveness of detention in controlling peak flows on the main channel with 20-acre developments versus 80-acre developments, using the results from the evaluations of development stage and development sequence. The values in Table 21 were calculated using the percent increases in peak flows over pre-development conditions that were tabulated at each junction for each combination of factors in Tables 17 to 20. The values in Table 21 are the difference between the percent increase for the 20-acre developments and the percent increase for the 80-acre developments, for the same combination of all other factors. A positive number means that the 20-acre developments caused a greater increase in peak flow over pre-development conditions. A negative number means that the 80-acre developments caused a greater increase in peak flow over pre-development conditions.

As noted in the discussion of the results from the evaluation of development sequence, when development occurred in the upstream portions of the watersheds, detention was not nearly as effective in maintaining pre-development peak flows on the main channel with the 20-acre developments as it was with the 80-acre developments. This was true for all combinations of watershed shape and slope. Table 21 also shows that when development occurred in the downstream portions of the watersheds the 20-acre developments were actually slightly more effective, even though substantial impacts were generated in all cases.

These results are easily explained by the fact that the detention outflow from the 80-acre developments peaks later than the detention outflow from the 20-acre developments. Recall that when the upstream portion of the watershed was

Table 21. Comparison of Peak Flow Impacts for 20-acre and 80-acre Developments

Evaluation of Development Stage					
Watershed	**E**	**D**	**C**	**B**	**A**
CM2025D	---	0.00	0.00	-1.03	-1.03
CM2050D	---	0.00	-0.86	-1.68	-1.66
CM2075D	---	1.02	0.01	-0.68	-0.64
CM20MedD	---	-0.41	-0.31	-0.20	-0.21
CS2025D	---	0.00	0.00	-1.14	-1.15
CS2050D	---	0.00	-0.59	-1.69	-1.76
CS2075D	---	2.10	0.72	-0.34	-0.32
CS20MedD	---	-0.05	-0.07	-0.05	-0.11
EM2025D	0.00	0.00	-0.14	-0.54	-0.54
EM2050D	0.00	-0.44	-1.14	-1.38	-1.35
EM2075D	0.11	-0.72	-1.10	-0.97	-0.81
EM20MedD	-0.25	0.12	0.68	1.35	1.38
ES2025D	0.00	0.00	-0.26	-1.38	-1.39
ES2050D	0.00	-0.52	-1.71	-2.59	-2.60
ES2075D	0.45	-0.69	-1.64	-2.29	-2.25
ES20MedD	0.23	0.37	0.63	0.87	0.85
Evaluation of Development Sequence					
Watershed	**E**	**D**	**C**	**B**	**A**
CM2025UD	---	1.02	2.01	2.78	2.85
CM2050UD	---	-0.41	4.17	6.32	6.17
CM2075UD	---	-0.41	-0.31	4.18	3.97
CM20MedD	---	-0.41	-0.31	-0.20	-0.21
CS2025UD	---	2.10	3.36	4.31	4.30
CS2050UD	---	-0.05	6.57	9.26	9.17
CS2075UD	---	-0.05	-0.07	6.31	6.20
CS20MedD	---	-0.05	-0.07	-0.05	-0.11
EM2025UD	2.09	4.13	5.05	5.34	5.19
EM2050UD	-0.25	3.30	6.07	7.05	6.75
EM2075UD	-0.25	0.12	1.40	3.03	2.99
EM20MedD	-0.25	0.12	0.68	1.35	1.38
ES2025UD	3.75	6.18	7.45	7.89	7.79
ES2050UD	0.23	5.36	9.22	10.78	10.59
ES2075UD	0.23	0.37	2.03	4.56	4.53
ES20MedD	0.23	0.37	0.63	0.87	0.85

developed, delaying the runoff from the developed areas through the use of detention allowed the peak flows from the rest of the watershed to pass before the peak from the developed area arrived. The greater delay caused by the larger developments enhances this effect, further decreasing peak flows on the main channel. When development occurred in the downstream portion of the watershed, runoff from the lower portion of the watershed was detained so that it contributed more to the peak from the rest of the watershed. In this case, the additional delay in the peak flow from the larger developments is a detriment, causing the runoff from the developments to further coincide with the peak from the rest of the watershed.

Influence of Watershed Shape

In almost all cases, the elongated watersheds generated greater increases in peak flows over pre-development conditions on the main channel than the classic watersheds. Table 22 provides a summary comparison of the effectiveness of detention in controlling peak flows at the watershed outlet between the elongated watersheds versus the classic watersheds, using the results from the evaluations of development intensity, development stage, and development sequence. The values in Table 22 are the difference between the percent increases in peak flows over pre-development conditions at Junction A in the elongated watersheds and the percent increases at Junction A in the classic watersheds, for the same combination of all other factors. The values in Table 22 were calculated using the increases in peak flows over pre-development conditions that were tabulated at Junction A for each combination of factors in Tables 15 to 20. The larger the value in Table 22, the greater the impact at Junction A was in the elongated watershed compared to the classic watershed. A negative number means that the impact at Junction A was greater in the classic watershed.

In general under pre-development conditions, the classic watershed shape produces a sharper, higher peak at the watershed outlet than the elongated watershed shape and peaks earlier than the elongated watershed shape. Runoff from the

Table 22. Comparison of Peak Flow Impacts for Elongated and Classic Shapes

Eval. of Dev. Intensity			Eval. of Dev. Stage			Eval. of Dev. Sequence	
Watershed	**Difference**		**Watershed**	**Difference**		**Watershed**	**Difference**
EM80LD	10.67		EM8025D	6.84		EM8025UD	-0.86
EM80MedD	15.53		EM8050D	13.36		EM8050UD	1.01
EM80HD	20.75		EM8075D	15.92		EM8075UD	8.01
EM20LD	13.39		EM80MedD	15.53		EM80MedD	15.53
EM20MedD	17.12		EM2025D	7.33		EM2025UD	1.48
EM20HD	21.71		EM2050D	13.66		EM2050UD	1.60
ES80LD	9.26		EM2075D	15.74		EM2075UD	7.03
ES80MedD	12.95		EM20MedD	17.12		EM20MedD	17.12
ES80HD	15.94		ES8025D	6.98		ES8025UD	-1.88
ES20LD	10.28		ES8050D	12.91		ES8050UD	-0.33
ES20MedD	13.91		ES8075D	14.83		ES8075UD	6.16
ES20HD	16.95		ES80MedD	12.95		ES80MedD	12.95
			ES2025D	6.74		ES2025UD	1.62
			ES2050D	12.07		ES2050UD	1.09
			ES2075D	12.90		ES2075UD	4.49
			ES20MedD	13.91		ES20MedD	13.91

elongated watershed shape is more extended, producing a lower, later peak at the watershed outlet. Figure 14 illustrates the pre-development hydrographs generated at Junction A for both the classic and elongated watershed shapes, using the watersheds with mild slopes as an example. Figure 14 also shows the hydrographs generated at Junction A when both watershed shapes were developed with detention. The example shown is for fully developed watersheds with mild slopes, 20 acre developments, and high development intensity. Figure 14 shows that with detention, the elongated watershed still peaked later than the classic watershed, but the characteristics of the elongated watershed were such that when combined with the effects of detention they created a peak flow at the watershed outlet that was higher than in the classic watershed.

Comparison of the peak flows at the watershed outlets to the peak flows in the upstream portions of the watersheds provides some insight into why the elongated watersheds generated greater impacts at the watershed outlets than the classic watersheds. Table 23 summarizes the peak flows and time of peak at Junction A for

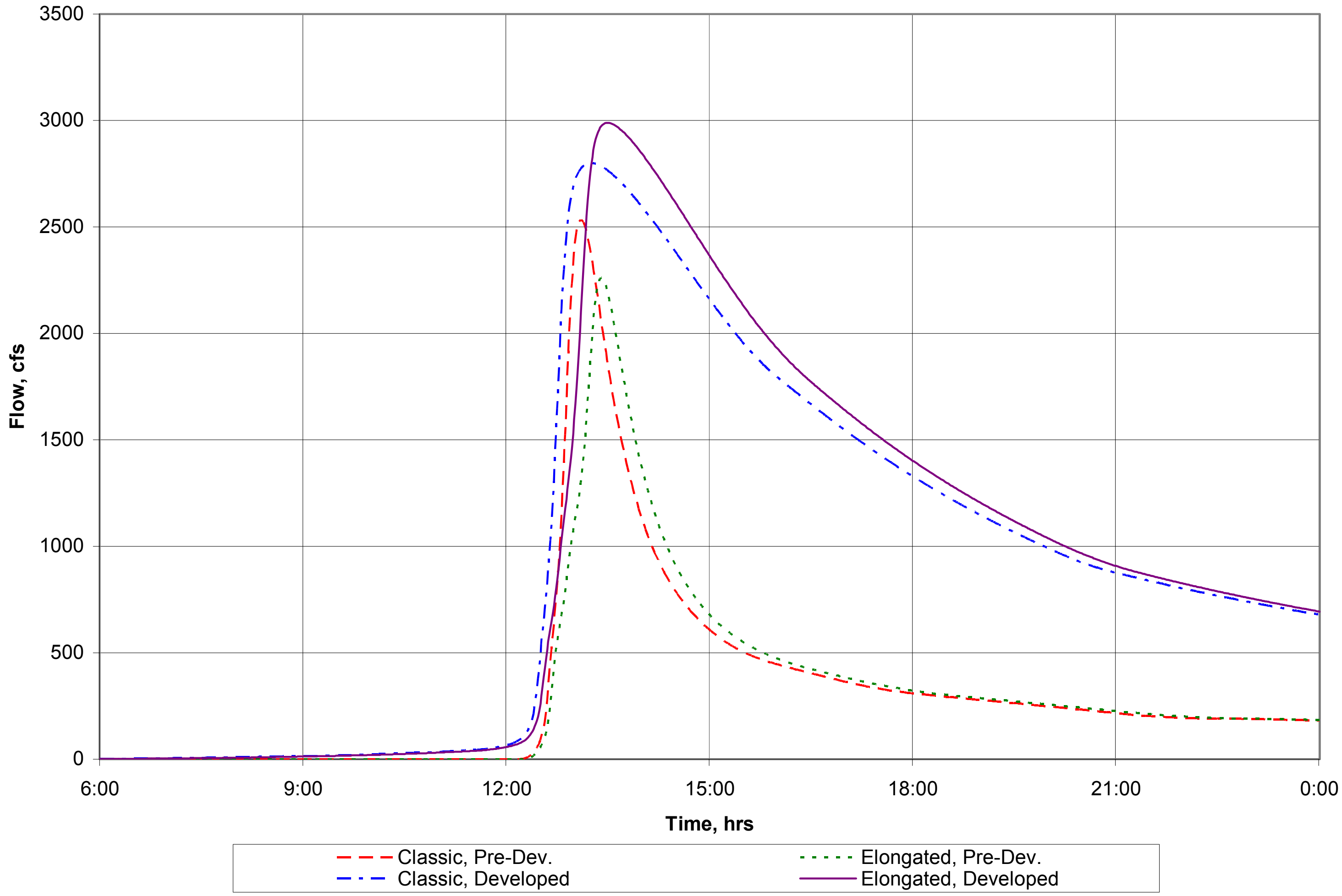

Figure 14. Comparison of Hydrographs at Watershed Outlets for Different Watershed Shapes

Table 23. Comparison of Peak Flows at Watershed Outlets to Upstream Junctions for Elongated and Classic Shapes

Condition	Factor	Classic Junction A	Classic Junction D	Elongated Junction A	Elongated Junction E
Pre-Development	Peak Flow, cfs	2533.7	1399.7	2261.1	1198.8
	Time of Peak	1306	1241	1324	1236
Developed	Peak Flow, cfs	2799.4	1408.5	2989.0	1219.7
	Time of Peak	1315	1251	1330	1247

pre-development conditions and developed conditions with detention, for both the classic and elongated watersheds. Table 23 also summarizes the peak flows and time of peak for pre-development conditions and developed conditions with detention at the most upstream junctions in the watersheds, Junction D in the classic watershed and Junction E in the elongated watershed. The example again considers the watersheds with mild slopes, 20 acre developments, and high development intensity.

Table 23 shows that at the most upstream junctions in the watersheds, the pre-development peak in the classic watershed was higher than in the elongated watershed, as was the case at the watershed outlets. However, unlike at the watershed outlets, the pre-development peak at the most upstream junction in the classic watershed occurred later than in the elongated watershed. These characteristics at Junctions D and E also held true when the watersheds were developed with detention. For both pre-development conditions and developed conditions, the classic watershed peaks at the upstream junction were higher and occurred later than the elongated watershed peaks due to the larger drainage areas of the first order streams in the classic watershed.

Under both the pre-development conditions and developed conditions with detention, the peak flow at the upstream junction in the elongated watershed occurred earlier than the peak in the classic watershed, but arrived at the watershed outlet later than the classic watershed peak. This delay was a result of the longer length of the main channel in the elongated watershed which increased the travel time to the

watershed outlet. Under pre-development conditions, the travel time of the peak flow from the most upstream junction to the watershed outlet was 48 minutes in the elongated watershed compared to 25 minutes in the classic watershed. Under developed conditions, the travel time of the peak flow from the most upstream junction to the watershed outlet was 43 minutes in the elongated watershed compared to 24 minutes in the classic watershed.

Under both the pre-development conditions and developed conditions with detention, the peak flow at the upstream junction in the elongated watershed was lower than the peak in the classic watershed. Under pre-development conditions, the peak flow in the elongated watershed remained lower than the peak flow in the classic watershed at every point along the main channel. Under developed conditions with detention, however, the characteristics of the elongated watershed were such that by the time the peak propagated downstream to the watershed outlet it had become larger than the classic watershed peak. This occurred because detention delayed the peak flow from the tributaries in the downstream portion of the watershed so that they coincided to a greater degree with the peak flow from the upstream portion of the watershed. This change in timing had a greater impact on peak flows in the elongated watershed because of the greater travel time from the upstream portion of the watershed to the watershed outlet.

Figure 15 illustrates the timing of peak flows from the second order streams and the timing of peak flows on the main channel in the elongated watershed, for both pre-development and developed with detention conditions. The figure shows the outflow hydrographs from the second order drainage areas, and the outflow hydrographs from the main channel reaches. In the elongated watershed, the hydrograph for Reach 2L would combine with the hydrograph for Reach E to create the hydrograph at Junction D, the hydrograph for Reach 2P would combine with the hydrograph for Reach D to create the hydrograph at Junction C, and the hydrograph for Reach 2T would combine with the hydrograph for Reach C to create the hydrograph at Junction B. The timing of peak flows in the classic watershed is similarly illustrated in Figure 16. In the classic watershed, the hydrograph for Reach

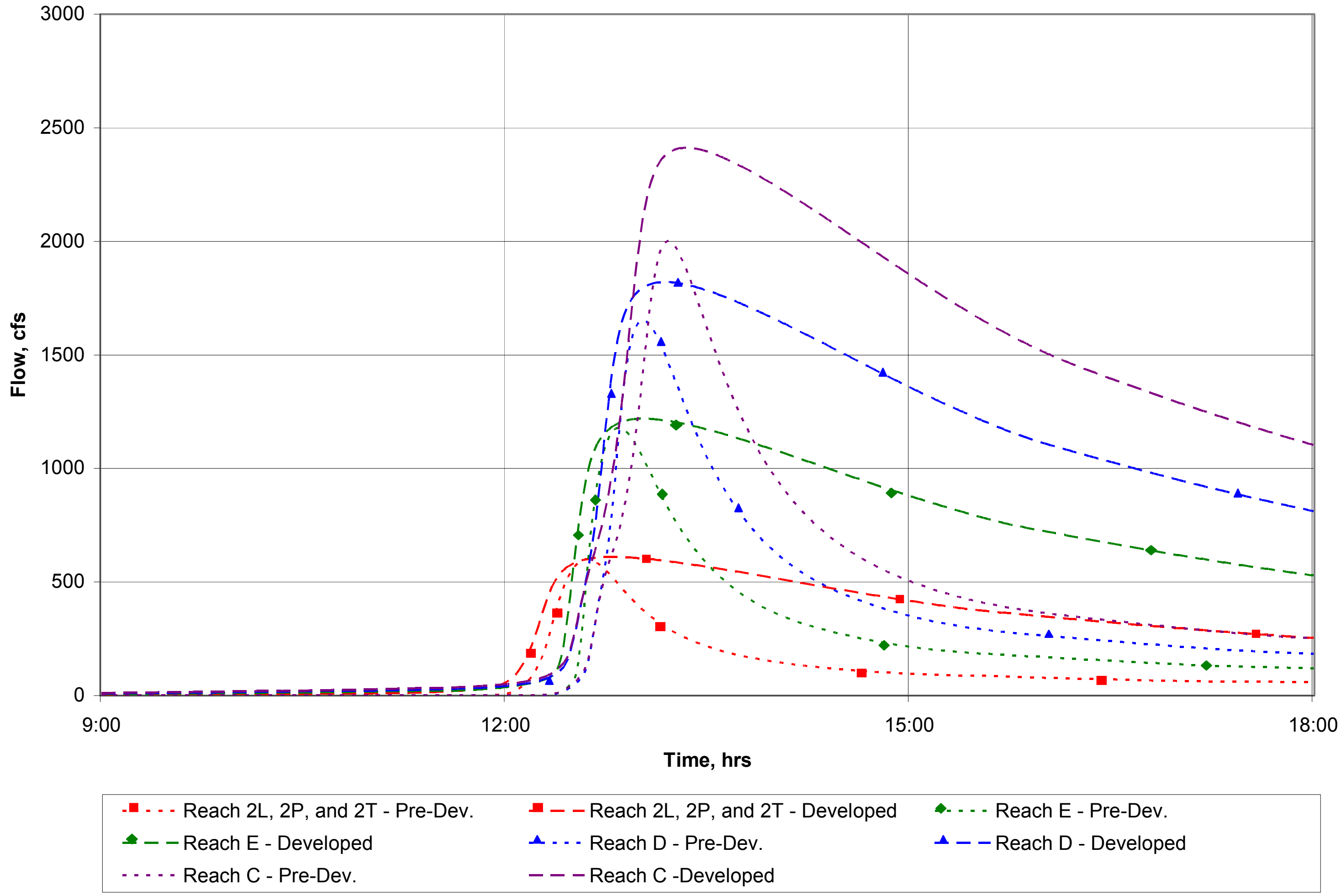

Figure 15. Timing of Peak Flows in the Elongated Watershed

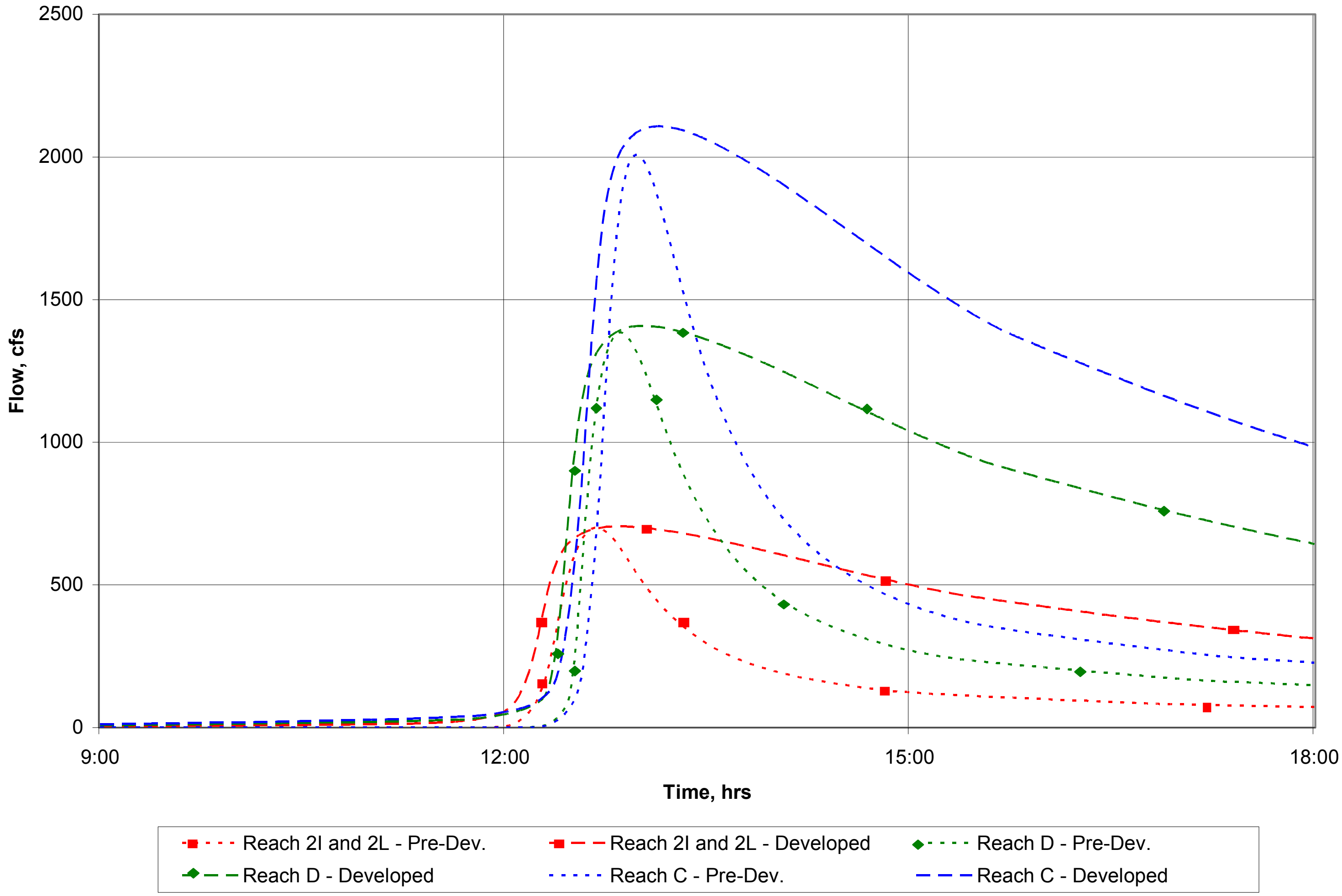

Figure 16. Timing of Peak Flows in the Classic Watershed

2I would combine with the hydrograph for Reach D to create the hydrograph at Junction C, and the hydrograph for Reach 2L would combine with the hydrograph for Reach C to create the hydrograph at Junction B.

Under pre-development conditions in the elongated watershed, longer travel times on the main channel allowed the peak flows from the downstream portions of the watershed to pass before the peak from the upper portion of the watershed arrived,as illustrated in Figure 15. For example, note how the pre-development peak from Reach C on the main channel completely misses the pre-development peak from the second order stream, Reach 2T. The peak from Reach C actually occurred 37 minutes after the peak from Reach 2T. In the classic watershed, the pre-development peak flows on the main channel coincide to a greater degree with the pre-development peak flows from the second order tributaries, as shown in Figure 16. For example, the pre-development peak from Reach C occurred only 18 minutes later than the pre-development peak from Reach 2L.

Table 24 shows the percent of the peak flow at the junctions on the main channel that was contributed by the second order tributaries and the percent that was contributed by the flow in the main channel from the upstream portion of the watershed. This is shown for each watershed shape and for both pre-development and developed with detention conditions. As shown in Figures 15 and 16, detention greatly extended the runoff from the second order tributaries in both the classic and elongated watersheds. However, Table 24 shows that this delay in the runoff from the second order streams had a more significant impact on the peak flows on the main channel in the elongated watershed. In the classic watershed, the contribution from the second order streams at each junction on the main channel was approximately the same for both the pre-development and developed conditions. In the elongated watershed, however, detention increased the contribution from the second order streams to the peak flows on the main channel.

Table 24. % of Peak Flow at Main Channel Junctions Contributed by Second Order Streams and by Upstream Portions of Watershed

		Classic		Elongated	
Junction	Reach	Pre-Dev.	Developed	Pre-Dev.	Developed
D	E	---	---	69.3	66.8
	2L	---	---	30.7	33.2
C	D	67.8	66.7	80.9	75.3
	2P	32.2	33.3	19.1	24.7
B	C	77.6	75.2	87.0	80.5
	2T	22.4	24.8	13.0	19.5

Influence of Watershed Slope

In almost all cases, the watersheds with mild slopes generated greater increases in peak flows over pre-development conditions on the main channel than the watersheds with steep slopes. Table 25 provides a summary comparison of the effectiveness of detention in controlling peak flows on the main channel in watersheds with steep slopes versus mild slopes, using the results from the evaluations of development intensity, development stage, and development sequence. The values in Table 25 are the difference between the percent increases in peak flows over pre-development conditions in the watersheds with steep slopes and the watersheds with mild slopes, for the same combination of all other factors. The values in Table 25 were calculated using the increases in peak flows over pre-development conditions that were tabulated at each junction on the main channel for each combination of factors in Tables 15 to 20. A negative number means that a greater increase in peak flow over pre-development conditions was generated in the watershed with mild slopes.

Watershed slope is very similar to watershed shape in how it affects the peak flows on the main channel. Like watershed shape, watershed slope affects the timing of flows through the watershed. Table 26 summarizes the peak flow and time of peak at Junctions A and D for pre-development conditions and developed conditions with detention, for watersheds with mild slopes and steep slopes. The example shown is

Table 25. Comparison of Peak Flow Impacts for Steep and Mild Slopes

Evaluation of Development Intensity					
Watershed	**E**	**D**	**C**	**B**	**A**
CS80LD	---	0.28	0.14	-0.24	-0.68
CS80MedD	---	0.07	-0.31	-0.92	-1.56
CS80HD	---	-0.22	-0.91	-1.95	-2.75
CS20LD	---	-0.03	-0.78	-1.44	-1.83
CS20MedD	---	0.44	-0.07	-0.77	-1.45
CS20HD	---	0.78	0.07	-0.96	-1.77
ES80LD	-0.05	-0.57	-1.00	-1.50	-2.08
ES80MedD	0.10	-0.88	-2.04	-3.09	-4.13
ES80HD	-0.48	-1.95	-3.92	-6.07	-7.57
ES20LD	-1.19	-2.52	-3.48	-4.26	-4.94
ES20MedD	0.58	-0.64	-2.09	-3.57	-4.66
ES20HD	0.58	-0.90	-2.84	-4.98	-6.53
Evaluation of Development Stage					
Watershed	**E**	**D**	**C**	**B**	**A**
CS8025D	---	0.00	0.00	-0.26	-0.39
CS8050D	---	0.00	-0.34	-0.61	-0.89
CS8075D	---	-0.98	-0.96	-1.18	-1.67
CS80MedD	---	0.07	-0.31	-0.92	-1.56
CS2025D	---	0.00	0.00	-0.38	-0.52
CS2050D	---	0.00	-0.07	-0.62	-0.99
CS2075D	---	0.10	-0.26	-0.84	-1.35
CS20MedD	---	0.44	-0.07	-0.77	-1.45
ES8025D	0.00	0.00	0.04	0.01	-0.25
ES8050D	0.00	-0.17	-0.57	-0.92	-1.34
ES8075D	-0.39	-0.87	-1.50	-2.06	-2.75
ES80MedD	0.10	-0.88	-2.04	-3.09	-4.13
ES2025D	0.00	0.00	-0.08	-0.82	-1.10
ES2050D	0.00	-0.26	-1.14	-2.13	-2.58
ES2075D	-0.05	-0.84	-2.03	-3.38	-4.19
ES20MedD	0.58	-0.64	-2.09	-3.57	-4.66

Table 25. Continued

Evaluation of Development Sequence					
Watershed	**E**	**D**	**C**	**B**	**A**
CS8025UD	---	-0.98	-1.29	-1.61	-1.72
CS8050UD	---	0.07	-1.92	-2.82	-3.05
CS8075UD	---	0.07	-0.31	-2.22	-2.73
CS80MedD	---	0.07	-0.31	-0.92	-1.56
CS2025UD	---	0.10	0.06	-0.08	-0.27
CS2050UD	---	0.44	0.47	0.13	-0.05
CS2075UD	---	0.44	-0.07	-0.09	-0.49
CS20MedD	---	0.44	-0.07	-0.77	-1.45
ES8025UD	-1.06	-1.80	-2.21	-2.48	-2.74
ES8050UD	0.10	-1.69	-3.08	-3.86	-4.39
ES8075UD	0.10	-0.88	-2.35	-3.73	-4.58
ES80MedD	0.10	-0.88	-2.04	-3.09	-4.13
ES2025UD	0.60	0.25	0.19	0.07	-0.13
ES2050UD	0.58	0.37	0.07	-0.13	-0.56
ES2075UD	0.58	-0.64	-1.73	-2.20	-3.03
ES20MedD	0.58	-0.64	-2.09	-3.57	-4.66

Table 26. Comparison of Peak Flows at Watershed Outlets and Upstream Junctions for Mild and Steep Slopes

		Mild		**Steep**	
Condition	**Factor**	**Junction A**	**Junction D**	**Junction A**	**Junction D**
Pre-Development	Peak Flow, cfs	2533.7	1399.7	3891	2093.9
	Time of Peak	1306	1241	1233	1220
Developed	Peak Flow, cfs	2815.9	1419	4217.3	2118.2
	Time of Peak	1324	1300	1247	1235

for the fully developed classic watersheds with 80 acre developments and high development intensity. Table 26 shows that both at Junction D in the upstream portion of the watersheds and at the watershed outlet, the steep watershed peaks much earlier than the mild watershed and has a significantly higher peak than the mild watershed. These characteristics are illustrated for Junction A in Figure 17.

Table 26 also shows that the travel time from the most upstream junction in the watershed to the watershed outlet is much longer in the watershed with mild slopes. Under pre-development conditions, the travel time of the peak flow from the most upstream junction to the watershed outlet was 25 minutes in the watershed with mild slopes compared to 13 minutes in the watershed with steep slopes. Under developed conditions, the travel time of the peak flow from the most upstream junction to the watershed outlet was 24 minutes in the watershed with mild slopes compared to 12 minutes in the watershed with steep slopes.

Although the effect is less pronounced, the increased travel time in the watershed with mild slopes had the same effect that the increased travel time had in the elongated watersheds. Under pre-development conditions, longer travel times on the main channel allowed the peak flows from the downstream portions of the watershed to pass before the peak from the upper portion of the watershed arrived. Under developed conditions with detention, however, the detention ponds extended the peak flows from the second order tributaries in the downstream portion of the watershed so that they contributed more to the overall watershed peak.

Ten Mile Creek Watershed

The shape of the Ten Mile Creek watershed is comparable to the classic watershed shape considered in the synthetic watershed analysis. The slopes in the Ten Mile Creek are slightly gentler than the slopes which were considered for the steep watersheds in the synthetic watershed analysis. The average weighted curve number in the watershed is comparable to the medium development intensity considered in the synthetic watershed analysis, although the weighted curve number

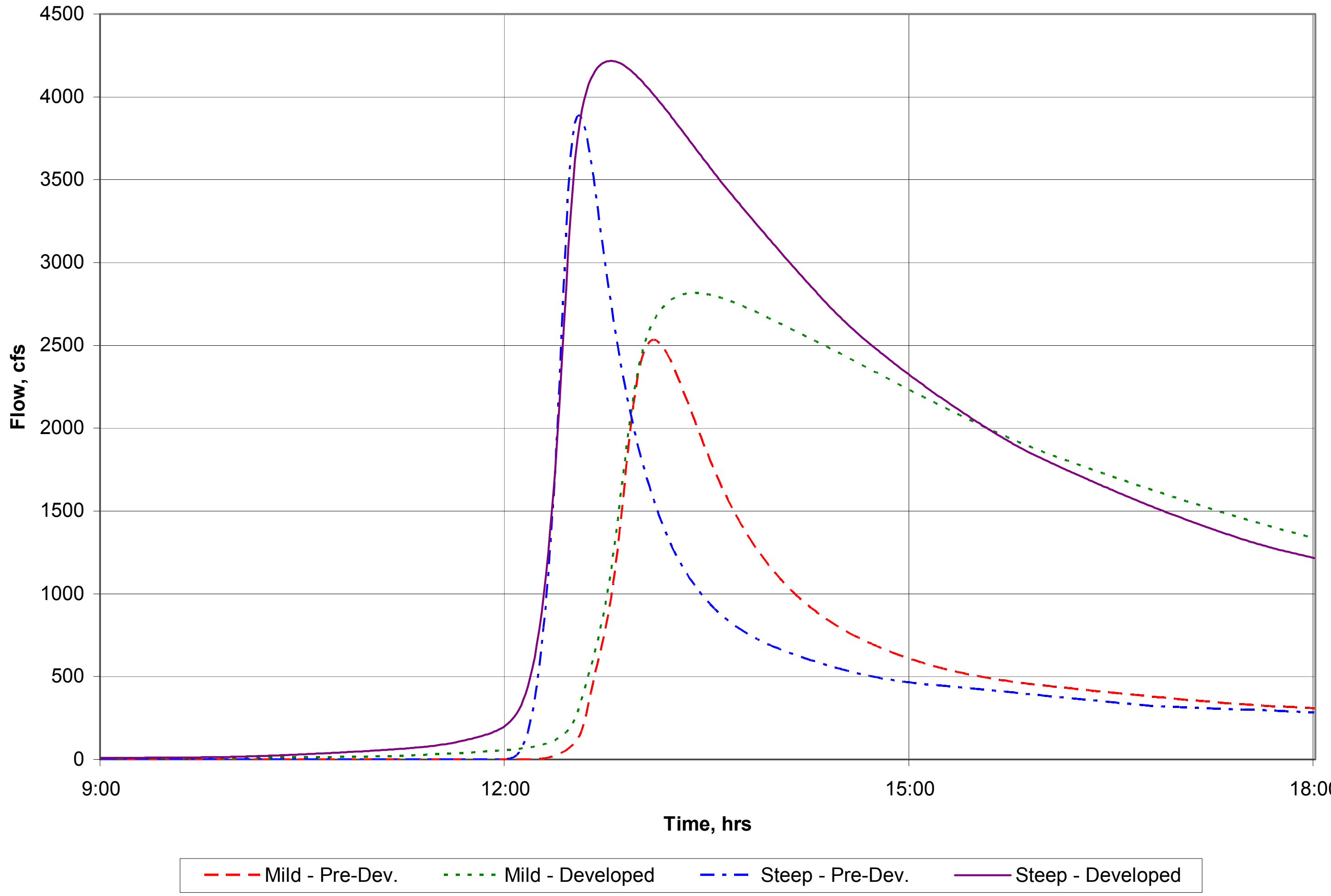

Figure 17. Comparison of Hydrographs at Watershed Outlets for Different Watershed Slopes

in the Joe Hinton basin is comparable to the low development intensity and some of the individual subbasins in the watershed have high development intensities. Although approximately 80% of the watershed is developed, only about 15% of the in the Joe Hinton basin is comparable to the low development intensity and some of the individual subbasins in the watershed have high development intensities. Although approximately 80% of the watershed is developed, only about 15% of the watershed is controlled by detention. However, the portion of the watershed that is controlled by detention includes some of the more intensely developed subbasins in the watershed. The ten detention ponds in the Ten Mile Creek watershed are located in approximately the lower half of the watershed. The majority of the detention ponds have contributing drainage areas that are larger than the 80-acre development size considered in the synthetic watershed analysis.

Based on the results of the synthetic watershed analysis, it was expected that peak flows on the lower portion of Ten Mile Creek would be greater than "pre-development" peak flows, even if the detention ponds in the watershed successfully limited the runoff from their respective subbasins to the "pre-development" peaks. It was also expected that any impacts would grow larger in the downstream direction. The results from the synthetic watersheds with comparable characteristics indicated that at the watershed outlet, an increase in peak flows over pre-development conditions on the order of 7.5% could be expected.

The modeling results for the Ten Mile Creek watershed are given in Table 27. Results are shown for existing conditions with detention, developed conditions without detention, and "pre-development" conditions, assuming a curve number of 62 and assuming a curve number of 73. The peak flow and time of peak are tabulated at each junction shown in Figure 5.

The percent increase in the peak flows for the existing conditions with detention over the "pre-development" peak flows were calculated at each junction and are tabulated in Table 28. From the values in Table 28 it is obvious that the assumptions made regarding the pre-development conditions had a tremendous impact on the results. The assumption of a pre-development curve number of 62

Table 27. Results from Modeling of Ten Mile Creek Watershed

	Existing Condtions With Detention		Developed Conditions Without Detention		Undeveloped, CN=62		Undeveloped, CN=73	
Junction	Peak Flow, cfs	Time	Peak Flow, cfs	Time	Peak Flow, cfs	Time	Peak Flow, cfs	Time
01070E	844.54	1236	844.54	1236	559.94	1236	991.16	1236
01140D	1898.3	1248	1898.3	1248	1193.9	1254	2093.6	1251
02030D	2512.9	1306	2510.9	1303	1453.2	1315	2596.9	1309
03030D	2428.1	1406	2416.6	1400	1321.6	1421	2378.5	1412
04080D	2542.4	1451	2547.6	1445	1365.4	1515	2428.9	1457
05060D	2930.8	1251	3434.9	1248	1507.2	1515	2643.4	1500

Table 28. % Increase in Peak Flows over Pre-Development Conditions in Ten Mile Creek Watershed

Junction	CN=62	CN=73
01070E	50.8	-14.8
01140D	59.0	-9.3
02030D	72.9	-3.2
03030D	83.7	2.1
04080D	86.2	4.7
05060D	94.5	10.9

resulted in huge increases in peak flows over the assumed pre-development conditions. While a curve number of 62 seemed to be a reasonable estimation of what may have actually existed in the Ten Mile Creek watershed before it was developed, none of the detention ponds in the watershed limited the runoff from their subbasins to the pre-development peak flows associated with a curve number of 62, as shown in Table 29. Therefore, it was determined that these results had little meaning for this analysis and were not given any further consideration.

Table 29 shows that when a pre-development curve number of 73 was assumed, all but two of the detention ponds were fairly successful in limiting the peak detention outflow to at or below the pre-development peak flow from their respective subbasins. A curve number of 73 seems high for "pre-development" conditions, but whether or not it is representative of conditions that may have existed in the watershed is not pertinent to this analysis. What is important is that it represents conditions in which the detention ponds are effective at maintaining pre-development peak flows at the subbasin outlets. This allows the cumulative impacts of detention in the watershed as a whole to be evaluated.

The values in Table 28 for a curve number of 73 show that peak flows at the junctions in the upper portion of the basin for the existing conditions with detention were less than the peak flows for the "pre-development" conditions. This occurred

Table 29. Comparison of Peak Detention Outflows to Pre-Development Peaks

		Pre-Development Subbasin Peak, cfs	
Detention Pond	**Peak Outflow, cfs**	**CN=62**	**CN=73**
WH050P	333.9	182.0	324.2
WH110P	98.2	49.4	88.1
SC170P	163.6	72.9	129.3
SC110P	105.1	59.4	106.1
04060P	195.0	81.0	143.5
CS010P	8.53	110.3	196.6
CS050P	94.5	70.4	120.6
CS060P	47.2	92.2	157.9
EV050P	113.4	91.9	164.8
EV060P	179.9	114.4	202.4

because the average curve number for the existing conditions in the upper portion of the watershed is less than 73. The values in Table 28 also show that even though the detention ponds effectively controlled the peak flows at the subbasin outlets, the "pre-development" peak flows were not maintained at the watershed outlet.

In the case of the Ten Mile Creek watershed, it is important to remember that while approximately 80% of the watershed is developed, only about 15% of the watershed is controlled by detention, so detention is not used on all developments as was the assumption in the analysis of the synthetic watersheds. However, recall that the weighted curve number of the entire Ten Mile Creek watershed under existing conditions is 75.8. Since the curve number being used to represent the "pre-development" conditions is nearly as high as the average existing curve number, the fact that all of the developed areas in the watershed are not controlled by detention has little effect on the results.

The results shown in Table 28 for the curve number of 73 closely resemble the results from the analysis of the synthetic watersheds. The results in Table 28 can be compared to the results from the analysis of development stage for the classic watershed shape with steep slopes, as shown in Table 17. This comparison, presented in Table 30, shows that in both the Ten Mile Creek watershed and the synthetic

Table 30. Comparison of Results from Ten Mile Creek to Results from Synthetic Watersheds

Ten Mile Creek Watershed		Synthetic Watersheds				
			CS8025	CS8050	CS2025	CS2050
Junction	% Increase	Junction	% Increase	% Increase	% Increase	% Increase
01070E	-14.8	D	0.0	0.0	0.0	0.0
01140D	-9.3	C	0.0	2.7	0.0	2.1
02030D	-3.2	B	5.2	7.2	4.1	5.6
03030D	2.1	A	5.3	7.4	4.1	5.7
04080D	4.7					
05060D	10.9					

watersheds, the use of detention in the downstream portion of the watershed caused peak flows to increase over pre-development conditions on the main channel and those impacts increased in the downstream direction.

The impacts in the Ten Mile Creek watershed were generated in the same manner as described for the synthetic watersheds. Figure 18 illustrates the creation of the peak flow at Junction 03030D, the first junction at which there was an increase over the "pre-development" peak. The figure shows, for both "pre-development" conditions and existing conditions with detention, the total hydrograph at Junction 03030D, the hydrograph from the main stem of Ten Mile Creek representing the runoff from the entire watershed to that point, and the hydrograph representing the runoff from the Sinking Creek basin, which joins Ten Mile Creek at Junction 03030D. Figure 18 shows that the peak from the main stem of Ten Mile Creek at that point was actually slightly less for the existing conditions with detention than for the "pre-development" conditions. However, the Sinking Creek basin produced a higher, slightly more extended peak under the existing conditions with detention. This caused a slightly higher flow to coincide with the peak from the main stem of Ten Mile Creek, resulting in a higher peak flow at Junction 03030D. Under "pre-development conditions", the contribution from Sinking Creek represented 17.8% of the peak at Junction 03030D. Under the existing conditions with detention, the contribution from Sinking Creek increased to 22.3% of the peak at Junction 03030D.

Figure 19 illustrates the creation of the hydrograph at the outlet of the Ten Mile Creek watershed, Junction 05060D. The figure shows, for both "pre-development" conditions and existing conditions with detention, the total hydrograph at Junction 05060D, the hydrograph from the main stem of Ten Mile Creek representing the runoff from the entire watershed to that point, and the hydrograph representing the runoff from the Echo Valley basin which joins Ten Mile Creek at Junction 05060D. There are two peaks at Junction 05060D. The second peak coincides with the peak from the main channel hydrograph, which occured much later than the Echo Valley peak. However, the main channel hydrograph has such a slow rising limb that another peak was created that coincides with the Echo Valley peak.

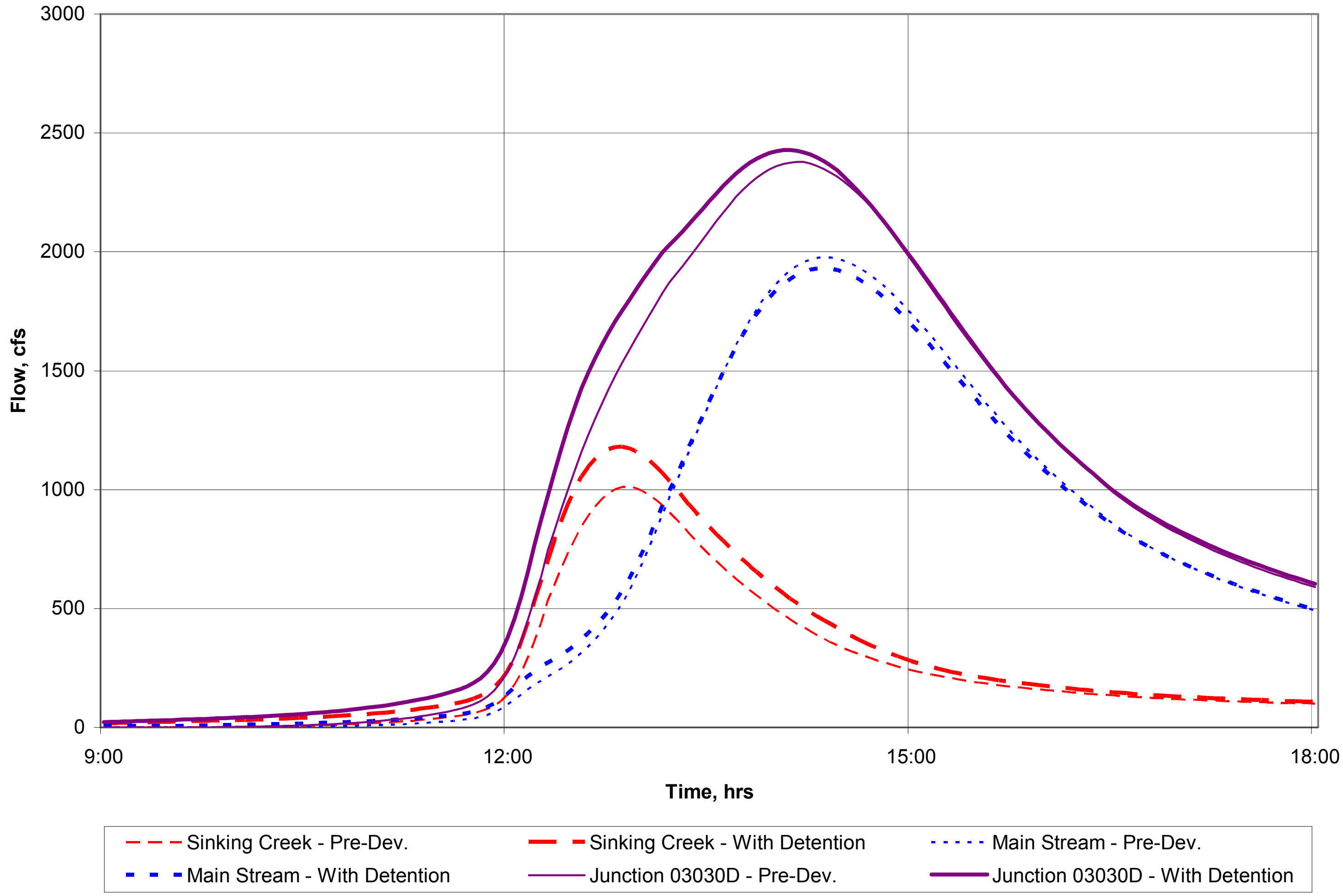

Figure 18. Creation of Peak Flow Impacts on the Main Stem of Ten Mile Creek

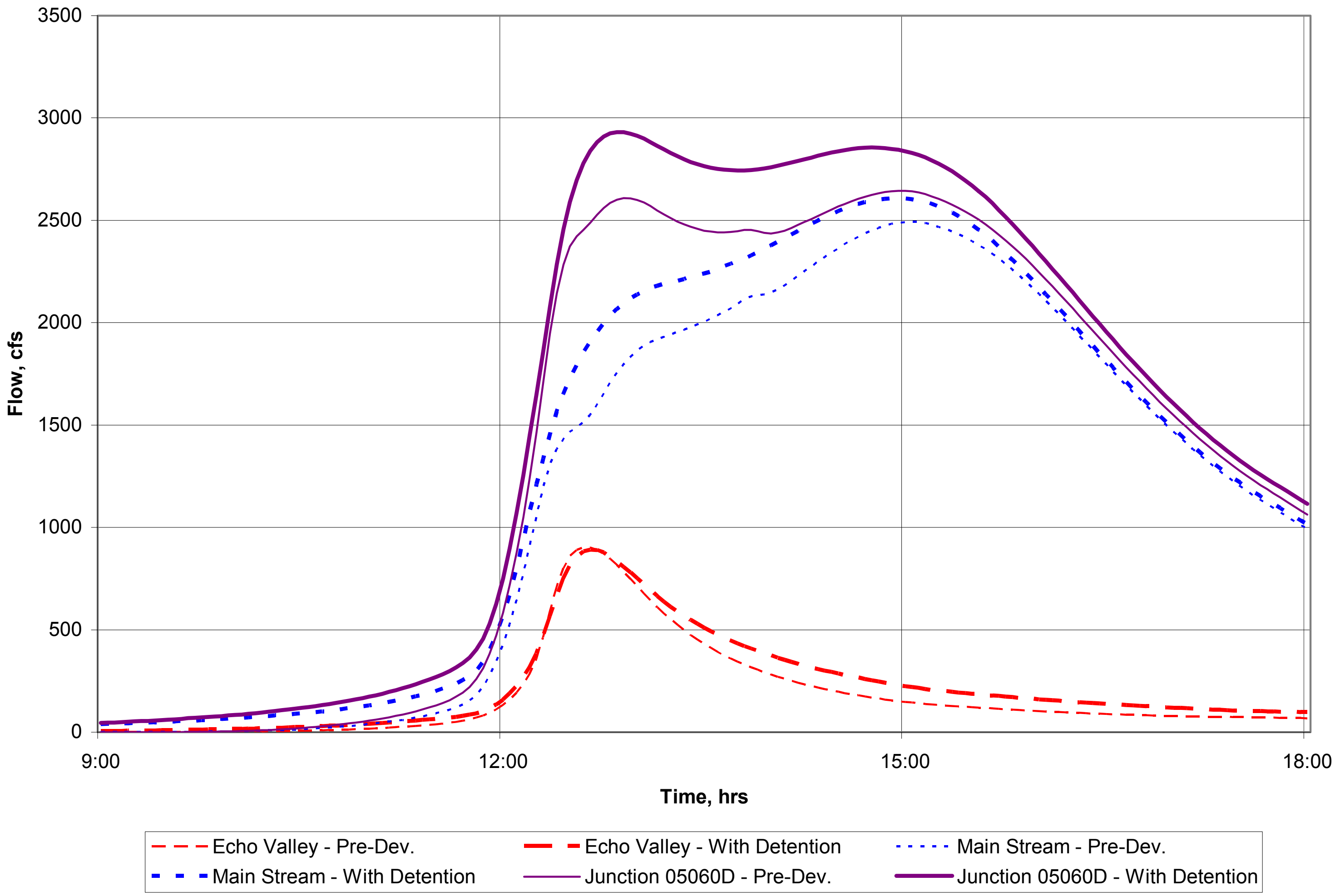

Figure 19. Hydrograph at the Ten Mile Creek Watershed Outlet

As was the case in the synthetic watersheds, the increased peak flows from points further upstream propagated downstream, aggravating the impacts at points further downstream in the watershed. Figure 19 shows that the increase in peak flows over “pre-development” conditions at the watershed outlet was primarily due to the increased contribution from the main stem, which began much further upstream at Junction 03030D. The runoff from the Echo Valley basin was also slightly extended, contributing a small amount to the second peak at the watershed outlet.

CHAPTER 5
CONCLUSIONS AND RECOMMENDATIONS

Of the six watershed and development characteristics considered in the sensitivity analysis, watershed shape, the percent of the watershed that was developed, and the location of the developed areas within the watershed had the greatest effect on the cumulative impacts of detention ponds in the watershed. These three factors determined the pattern of impacts that occurred within a watershed. Development intensity, development size, and watershed slope contributed to the magnitude of the impacts which were created, but they were not the overriding factors that determined the pattern of impacts. The results from the sensitivity analysis are summarized in Table 31. Table 31 lists the percent increase in peak flow over pre-development conditions at the watershed outlet that resulted from each combination of watershed and development characteristics.

Full development of the synthetic watersheds with detention used on all developments caused peak flows to increase over pre-development levels on the main channel in all cases, regardless of the other watershed and development characteristics. When the watershed was partially developed, the results depended on the percent of the watershed that was developed and the location of the developed area.

Development in the downstream portion of the watershed caused peak flows on the main channel to increase over pre-development levels in all cases. The use of detention in the downstream portion of the watershed detained runoff from the downstream tributaries so that it added to the peak from the rest of the watershed. The percentage of the watershed that was developed had a significant effect on how well pre-development peak flows were maintained on the main stream channel. For every combination of development size, watershed shape, and watershed slope, development of only 25% of the downstream portion of the watersheds was enough to cause substantial increases in peak flows over pre-development conditions on the

Table 31. Summary of Sensitivity Analysis Results

Factor		% Increase in Peak Flow over Pre-Development Conditions at the Watershed Outlet							
Watershed Shape		classic				elongated			
Watershed Slope		mild		steep		mild		steep	
Development Size		20 acres	80 acres	20 acres	80 acres	20 acres	80 acres	20 acres	80 acres
Development Intensity	low	5.9	6.4	4.1	5.7	19.3	17.1	14.4	15.0
	medium	8.7	8.9	7.2	7.3	25.8	24.4	21.1	20.3
	high	10.5	11.1	8.7	8.4	32.2	31.9	25.7	24.3
Development Stage *(development began in downstream portion of watershed)*	25 %	4.6	5.7	4.1	5.3	12.0	12.5	10.9	12.3
	50 %	6.7	8.3	5.7	7.4	20.3	21.7	17.7	20.3
	75 %	7.6	8.2	6.2	6.6	23.3	24.1	19.1	21.4
	100 %	8.7	8.9	7.2	7.3	25.8	24.4	21.1	20.3
Development Sequence *(development began in upstream portion of watershed)*	25 %	1.8	-1.0	1.6	-2.7	3.3	-1.9	3.2	-4.6
	50 %	4.3	-1.9	4.2	-4.9	5.9	-0.9	5.3	-5.3
	75 %	5.0	1.0	4.5	-1.7	12.0	9.0	9.0	4.5
	100 %	8.7	8.9	7.2	7.3	25.8	24.4	21.1	20.3

main channel. As the level of development in the watershed increased and development moved upstream, impacts were also created at points further upstream. The increased peaks at points further upstream then propagated downstream, aggravating the impacts at points downstream in the watershed.

When development occurred in the upstream portions of the watersheds, the effects on the rest of the watershed were very different than the pattern of impacts that was generated by development in the downstream portions of the watersheds. The use of detention in the upstream portion of the watershed delayed the runoff from the upstream area so that the peak from the rest of the watershed could pass. In all cases, any increases in peak flows on the main channel over pre-development levels were significantly smaller when development occurred in the upstream portion of the watershed than when the downstream portion of the watershed was developed. With 25% and 50% development of the upstream portion of the watersheds, peak flows were maintained at pre-development levels on the main channel in some cases, but the exact effect of development in the upstream portion of a watershed was dependent on the other watershed and development characteristics. In all cases, impacts worsened as a greater percentage of the watershed was developed and development proceeded downstream.

In almost all cases, the elongated watersheds generated significantly more severe increases in peak flows over pre-development conditions on the main channel than the classic watersheds. In some cases, the use of detention in the downstream portion of the elongated watersheds generated peak flows at some points on the main channel that were greater than if detention was not used at all. In general under pre-development conditions, runoff from the elongated watershed shape is more extended than runoff from the classic watershed shape, producing a lower peak at the watershed outlet. The longer length of the main channel in the elongated watershed increases the travel time to the watershed outlet, allowing the peak flows from the downstream portions of the watershed to pass before the peak from the upper portion of the watershed arrives. When detention is used in an elongated watershed, however, the detention ponds extended the peak flows from the second order

tributaries in the downstream portion of the watershed so that they contributed more to the overall watershed peak.

Development intensity played a role in determining the magnitude of the impacts which were created in a watershed. In the fully developed watersheds, peak flow impacts at each point on the main channel increased with increasing development intensity for each combination of watershed shape, watershed slope, and development size. Higher development intensities caused greater increases in peak flows on the main channel over pre-development conditions because of the greater volume of runoff produced and the resulting changes in timing of the detention outflow hydrograph.

Development size also played a role in determining the magnitude of the impacts which were created in a watershed. Larger developments required a greater detention storage volume which caused the peak outflow from the detention pond to occur later than the peak outflow from a smaller development. When detention was used in the upstream portions of a watershed, delaying the runoff from the developed areas through the use of detention allowed the peak flows from the rest of the watershed to pass before the peak from the developed area arrived. The greater delay caused by the larger developments enhanced this effect, further decreasing peak flows on the main channel. When development occurred in the downstream portion of the watershed, runoff from the lower portion of the watershed was detained so that it contributed more to the peak from the rest of the watershed. In this case, the additional delay in the peak flow from the larger developments was a detriment, causing the runoff from the developments to further coincide with the peak from the rest of the watershed.

The magnitude of the impacts which were created in a watershed were further dependent on the watershed slope. In almost all cases, the watersheds with mild slopes generated greater increases in peak flows over pre-development conditions on the main channel than the watersheds with steep slopes. Watershed slope affected peak flows on the main channel in a manner that was very similar to the effect of watershed shape, although the effect of watershed slope was much less pronounced.

In the watersheds with mild slopes, longer travel times on the main channel allowed the pre-development peak flows from the downstream portions of the watershed to pass before the pre-development peak from the upper portion of the watershed arrived. The use of detention, however, extended the peak flows from the second order tributaries in the downstream portion of the watershed so that they contributed more to the overall watershed peak.

The Ten Mile Creek watershed can be compared to the synthetic watersheds with a classic watershed shape and steep slopes. The results from the Ten Mile Creek watershed were very similar to the results from the synthetic watersheds. Even though the detention ponds in the Ten Mile Creek watershed effectively controlled the peak flows at the subbasin outlets to the representative "pre-development" peak flows, the "pre-development" peak flows were not maintained at the watershed outlet. As was the case in the synthetic watersheds, the use of detention in the downstream portion of the Ten Mile Creek watershed caused peak flows to increase over pre-development conditions on the main channel and those impacts increased in the downstream direction.

One of the objectives of this study was to identify combinations of watershed and development characteristics that may be likely to produce adverse peak flow impacts at some watershed locations in a watershed in which detention is required on all developments. The results of the sensitivity analysis can be summarized by the following key points:

- Detention is most effective at controlling peak flows throughout a watershed when development occurs in the upstream portion of the watershed.

- Detention is less effective when development occurs in the downstream portion of a watershed.

- The effectiveness of detention decreases as a greater percentage of the watershed is developed.

- When a watershed is fully developed with detention on all developments, pre-development peak flows will not be maintained at

all points on the main channel, regardless of the other watershed and detention characteristics.

- Detention is less effective in watersheds with an elongated shape than in watersheds with a more rounded shape.

- Detention is least effective when development occurs in the downstream portion of an elongated watershed.

The results of this study indicate that policies which require the uniform use of onsite detention ponds should be used with caution. Onsite detention can be an effective method of controlling the peak flow rate immediately downstream from a development, and the uniform use of onsite detention throughout a watershed will, in almost all cases, greatly reduce peak flows throughout the watershed below what they would be if detention were not used. However, the goal of requiring the use of onsite detention on all new developments is to maintain pre-development peak flows throughout the watershed and it must be recognized that the uniform use of onsite detention does not achieve this goal.

In order to maintain pre-development peak flows throughout a watershed, more strategic placement of detention ponds is required. Emphasis should also be placed on the use of stormwater management practices that control the volume of runoff from a development, rather than controlling only the rate of runoff, in order to more closely reproduce the pre-development hydrograph. More work needs to be done to develop and implement alternative detention policies which more effectively achieve the goal of maintaining pre-development peak flows throughout the watershed, while remaining practical for communities with limited resources.

LIST OF REFERENCES

Ahmed, A. R. (1995). "Watershed Approach to Stormwater Detention Policy Development." *Water Resources Engineering: Proceedings of the First International Conference,* American Society of Civil Engineers, New York, NY, 1208-1212.

AMEC Earth and Environmental, Inc. (2000). HEC-1 model of Ten Mile Creek watershed, developed for Knox County, TN.

AMEC Earth and Environmental, Inc. (2003). ArcView computer files.

Bedient, P. B., et al. (1982). *Evaluation of Effects of Stormwater Detention in Urban Areas*, Research Project Completion Report, Project No. B-244-TEX, Department of Environmental Science and Engineering, Rice University, Houston, TX.

Chow, V. T. (1959). *Open-Channel Hydraulics*, McGraw-Hill, New York, NY, 110-113.

City of Knoxville, TN. (2001). *Knoxville BMP Manual*, Stormwater Engineering Division, Knoxville, TN.

City of Knoxville, TN. (2002). *Land Development Manual,* Stormwater Engineering Division, Knoxville, TN.

Claycomb, E. L. (1988). "Effects of Basin-Wide On-Site Stormwater Detention." *Hydraulic Engineering: Proceedings of the 1988 National Conference on Hydraulic Engineering*, American Society of Civil Engineers, New York, NY, 195-200.

Daugherty, R. L., Franzini, J. B., and Finnemore, E. J. (1985). *Fluid Mechanics with Engineering Applications,* Eighth Edition, McGraw-Hill, New York, NY, 432-433.

Debo, T. N. (1982). "Detention Ordinances - Solving or Causing Problems?" *Proceedings, Engineering Foundation Conference on Stormwater Detention*, Hennicker, NH, 332-341.

Debo, T. N. and Reese, A. J. (1992). "Determining Downstream Analysis Limits for Detention Facilities." *Proceedings, NOVATECH 92*, Lyon, France, 15-22.

Duru, J. O. (1981). "On-Site Detention: A Stormwater Management or

Mismanagement Technique?" *Proceedings, 1981 International Symposium on Urban Hydrology, Hydraulics, and Sediment Control*, University of Kentucky, Lexington, KY, 297-302.

Eagleson, P. S. (1970). *Dynamic Hydrology*, McGraw-Hill, New York, NY, 369-394.

Gangaware, T. R., Smoot, J. L., Thomason, K. M., Tschantz, B. A., and Woodiel, B. J. (1997). *Stormwater Management Practices in Tennessee: A Survey of Local Government Representatives,* Tennessee Nonpoint Source Pollution Program, Nashville, TN.

Glidden, M. W. (1981). "The Effects of Stormwater Detention Policies on Peak Flows in Major Drainageways," M.S. thesis, University of Colorado.

Gray, D. M. (1961). "Interrelationships of Watershed Characteristics." *Journal of Geophysical Research*, 66(4), 1215-1223.

Horton, R. E. (1945). "Erosional Development of Streams and their Drainage Basins: Hydrophysical Approach to Quantitative Morphology." *Bull. Geol. Soc. Am.*, 56, 275-370.

Hydrologic Engineering Center (2000). *HEC-HMS Hydrologic Modeling System: Technical Reference Manual*, U. S. Army Corps of Engineers, Davis, CA.

Hydrologic Engineering Center (2001). *HEC-HMS Hydrologic Modeling System: User's Manual*, U. S. Army Corps of Engineers, Davis, CA.

James, W. P., Bell J. F., and Leslie D. L. (1987). "Size and Location of Detention Storage." *Journal of Water Resources Planning and Management*, American Society of Civil Engineers, 113(1), 15-28.

Kung, H.-T. (1980). "Geographic Aspects of the Urban Hydrology of Knoxville, Tennessee," Ph.D. dissertation, University of Tennessee, Knoxville, TN.

Lakatos, D. F. and Kropp, R. H. (1982). "Stormwater Detention: Downstream Effects on Peak Flow Rates." *Proceedings, Conference on Stormwater Detention Facilities: Planning, Design, and Maintenance*, American Society of Civil Engineers, New York, NY, 105-120.

Lee, B.-Y. (1985). "Downstream Effects on the Flood Hydrograph Caused by

Upstream Detention Storage," M.S. thesis, Texas Tech University.

Malcolm, H. R. (1980). *A Study of Detention in Urban Stormwater Management*, Water Resources Research Institute of The University of North Carolina, Report No. 156, North Carolina State University, Raleigh, NC.

Mays, L. W. and Bedient, P. B. (1982). "Model for Optimal Size and Location of Detention." *Journal of Water Resources Planning and Management*, American Society of Civil Engineers, 108(3), 270-285.

McCuen, R. H. (1974). "A Regional Approach to Urban Storm Water Detention." *Geophysical Research Letters*, 1(7), 321-322.

McCuen, R. H. (1979). "Downstream Effects of Stormwater Management Basins." *Journal of the Hydraulics Division,* American Society of Civil Engineers, 105(11), 1343-1356.

Nix, S. J. and Jolley, S. B. (1995). "A Re-evaluation of Artificial Detention for Stormwater Management in Urban Areas." *Proceedings, Symposium on Water Management in Urban Areas*, American Water Resources Association, Houston, TX, 213-223.

Ormsbee, L. E., Houck, M. H., and Delleur, J. W. (1987). "Design of Dual-Purpose Detention Systems Using Dynamic Programming." *Journal of Water Resources Planning and Management*, American Society of Civil Engineers, 113(4), 471-484.

Shea, C. C. (1995). "Alternative Strategies for Stormwater Management Using Individual Detention Basins," Ph.D. dissertation, Johns Hopkins University, Baltimore, MD.

Shea, C. C. (1996). "Reduction of Downstream Impacts Through Use of Variable Detention Basin Volume Requirements." *North American Water and Environment Congress*, American Society of Civil Engineers, New York, NY, 1858-1863.

Sloat, M. S. and Hwang, R. B. (1989). "Sensitivity Study of Detention Basins in Urbanized Watershed." *Journal of Urban Planning and Development*, American Society of Civil Engineers, 115(3), 135-155.

Soil Conservation Service (1986). *Urban Hydrology for Small Watersheds, Technical Release 55,* Second Edition, United States Department of Agriculture, Washington, DC.

Sonnenberg, S. E. and Fiuzat, A. A. (1986). *Effect of Detention Basins on Peak Watershed Discharges*, Engineering Report, Clemson University Department of Civil Engineering, Clemson, SC.

Strahler, A. N. (1964). "Quantitative Geomorphology of Drainage Basins and Channel Networks," in Section 4-II of *Handbook of Applied Hydrology*, V. T. Chow, Ed., McGraw-Hill, New York, NY, 4-39 to 4-76.

Traver, R. G. and Chadderton, R. A. (1983). "The Downstream Effects of Storm Water Detention Basins." *Proceedings, 1983 International Symposium on Urban Hydrology, Hydraulics, and Sediment Control*, University of Kentucky, Lexington, KY, 455-460.

Traver, R. G. and Chadderton, R. A. (1992a). "Accumulation Effects of Stormwater Management Detention Basins." *Proceedings of the Water Resources Sessions at Water Forum '92*, *Baltimore, MD,* American Society of Civil Engineers, New York, NY, 925-930.

Traver, R. G. and Chadderton, R. A. (1992b). "Modeling Stormwater Basin Effects." *Proceedings of the Water Resources Sessions at Water Forum '92, Baltimore, MD*, American Society of Civil Engineers, New York, NY, 855-860.

Traver, R. G. and Chadderton, R. A. (1994). "Effect of Nonstandard Rainfall Distributions on Stormwater Management Detention Basins." *Proceedings of the 21st Annual Conference on Water Policy and Management: Solving the Problems, Denver, CO*, American Society of Civil Engineers, New York, NY, 612-615.

Viessman, W. Jr., Lewis, G. L., and Knapp, J. W. (1989). *Introduction to Hydrology*, Third Edition, Harper Collins, New York, NY, 243-246.

Whipple, W. Jr., Kropp, R., and Burke, S. (1987). "Implementing Dual-Purpose

Stormwater Detention Programs." *Journal of Water Resources Planning and Management*, American Society of Civil Engineers, 113(6), 779-792.

Yeh, C.-H. and Labadie, J. W. (1997). "Multiobjective Watershed-Level Planning of Storm Water Detention Systems." *Journal of Water Resources Planning and Management*, American Society of Civil Engineers, 123(6), 336-343.

APPENDIX

Table A-1. Detention Pond Design Data

Pond	Depth, ft	Area, ac	Area, sq ft	Volume, AF	Outflow, cfs
CM80L	0	1.187	51692.792	0.000	0.000
	0.5	1.187	51695.042	0.593	0.182
	1	1.187	51701.792	1.187	1.030
	1.5	1.187	51713.042	1.781	2.838
	2	1.188	51728.792	2.375	5.827
	2.5	1.188	51749.042	2.969	10.179
	3	1.189	51773.792	3.565	16.056
	3.5	1.189	51803.042	4.161	23.605
	4	1.190	51836.792	4.758	32.960
	4.5	1.191	51875.042	5.357	44.246
	5	1.192	51917.792	5.957	57.579
	5.5	1.193	51965.042	6.558	73.072
CS80L	0	0.915	39871.804	0.000	0.000
	0.5	0.915	39874.054	0.458	0.182
	1	0.916	39880.804	0.915	1.030
	1.5	0.916	39892.054	1.373	2.838
	2	0.916	39907.804	1.832	5.827
	2.5	0.917	39928.054	2.291	10.179
	3	0.917	39952.804	2.751	16.056
	3.5	0.918	39982.054	3.211	23.605
	4	0.919	40015.804	3.673	32.960
	4.5	0.920	40054.054	4.136	44.246
	5	0.920	40096.804	4.600	57.579
	5.5	0.922	40144.054	5.066	73.072
CM20L	0	0.527	22968.888	0.000	0.000
	0.5	0.527	22971.138	0.264	0.182
	1	0.527	22977.888	0.527	1.030
	1.5	0.528	22989.138	0.791	2.838
	2	0.528	23004.888	1.056	5.827
	2.5	0.529	23025.138	1.321	10.179
	3	0.529	23049.888	1.587	16.056
	3.5	0.530	23079.138	1.853	23.605
	4	0.531	23112.888	2.121	32.960
	4.5	0.531	23151.138	2.390	44.246
	5	0.532	23193.888	2.660	57.579
	5.5	0.534	23241.138	2.932	73.072

Table A-1. Continued

Pond	Depth, ft	Area, ac	Area, sq ft	Volume, AF	Outflow, cfs
CS20L	0	0.415	18089.239	0.000	0.000
	0.5	0.415	18091.489	0.208	0.182
	1	0.415	18098.239	0.415	1.030
	1.5	0.416	18109.489	0.623	2.838
	2	0.416	18125.239	0.832	5.827
	2.5	0.417	18145.489	1.041	10.179
	3	0.417	18170.239	1.251	16.056
	3.5	0.418	18199.489	1.461	23.605
	4	0.419	18233.239	1.673	32.960
	4.5	0.419	18271.489	1.886	44.246
	5	0.420	18314.239	2.100	57.579
	5.5	0.422	18361.489	2.315	73.072
CM80Med	0	1.840	80139.713	0.000	0.000
	0.5	1.840	80141.963	0.920	0.182
	1	1.840	80148.713	1.840	1.030
	1.5	1.840	80159.963	2.760	2.838
	2	1.841	80175.713	3.681	5.827
	2.5	1.841	80195.963	4.602	10.179
	3	1.842	80220.713	5.524	16.056
	3.5	1.842	80249.963	6.447	23.605
	4	1.843	80283.713	7.371	32.960
	4.5	1.844	80321.963	8.296	44.246
	5	1.845	80364.713	9.222	57.579
	5.5	1.846	80411.963	10.150	73.072
CS80Med	0	1.443	62839.803	0.000	0.000
	0.5	1.443	62842.053	0.721	0.182
	1	1.443	62848.803	1.443	1.030
	1.5	1.443	62860.053	2.164	2.838
	2	1.443	62875.803	2.886	5.827
	2.5	1.444	62896.053	3.609	10.179
	3	1.444	62920.803	4.333	16.056
	3.5	1.445	62950.053	5.057	23.605
	4	1.446	62983.803	5.782	32.960
	4.5	1.447	63022.053	6.509	44.246
	5	1.448	63064.803	7.236	57.579
	5.5	1.449	63112.053	7.966	73.072

Table A-1. Continued

Pond	Depth, ft	Area, ac	Area, sq ft	Volume, AF	Outflow, cfs
CM20Med	0	0.810	35286.982	0.000	0.000
	0.5	0.810	35289.232	0.405	0.182
	1	0.810	35295.982	0.810	1.030
	1.5	0.811	35307.232	1.216	2.838
	2	0.811	35322.982	1.621	5.827
	2.5	0.811	35343.232	2.028	10.179
	3	0.812	35367.982	2.435	16.056
	3.5	0.813	35397.232	2.843	23.605
	4	0.813	35430.982	3.252	32.960
	4.5	0.814	35469.232	3.662	44.246
	5	0.815	35511.982	4.074	57.579
	5.5	0.816	35559.232	4.487	73.072
CS20Med	0	0.637	27766.991	0.000	0.000
	0.5	0.637	27769.241	0.319	0.182
	1	0.638	27775.991	0.638	1.030
	1.5	0.638	27787.241	0.957	2.838
	2	0.638	27802.991	1.276	5.827
	2.5	0.639	27823.241	1.596	10.179
	3	0.639	27847.991	1.917	16.056
	3.5	0.640	27877.241	2.239	23.605
	4	0.641	27910.991	2.561	32.960
	4.5	0.642	27949.241	2.885	44.246
	5	0.643	27991.991	3.211	57.579
	5.5	0.644	28039.241	3.537	73.072
CM80H	0	3.990	173803.835	0.000	0.000
	0.5	3.990	173806.085	1.995	0.182
	1	3.990	173812.835	3.990	1.030
	1.5	3.990	173824.085	5.985	2.838
	2	3.991	173839.835	7.981	5.827
	2.5	3.991	173860.085	9.978	10.179
	3	3.992	173884.835	11.975	16.056
	3.5	3.993	173914.085	13.973	23.605
	4	3.993	173947.835	15.972	32.960
	4.5	3.994	173986.085	17.972	44.246
	5	3.995	174028.835	19.973	57.579
	5.5	3.996	174076.085	21.976	73.072

Table A-1. Continued

Pond	Depth, ft	Area, ac	Area, sq ft	Volume, AF	Outflow, cfs
CS80H	0	3.206	139639.051	0.000	0.000
	0.5	3.206	139641.301	1.603	0.182
	1	3.206	139648.051	3.206	1.030
	1.5	3.206	139659.301	4.809	2.838
	2	3.206	139675.051	6.413	5.827
	2.5	3.207	139695.301	8.017	10.179
	3	3.208	139720.051	9.622	16.056
	3.5	3.208	139749.301	11.228	23.605
	4	3.209	139783.051	12.834	32.960
	4.5	3.210	139821.301	14.442	44.246
	5	3.211	139864.051	16.052	57.579
	5.5	3.212	139911.301	17.663	73.072
CM20H	0	1.751	76286.012	0.000	0.000
	0.5	1.751	76288.262	0.876	0.182
	1	1.751	76295.012	1.751	1.030
	1.5	1.752	76306.262	2.627	2.838
	2	1.752	76322.012	3.504	5.827
	2.5	1.753	76342.262	4.381	10.179
	3	1.753	76367.012	5.259	16.056
	3.5	1.754	76396.262	6.137	23.605
	4	1.755	76430.012	7.017	32.960
	4.5	1.755	76468.262	7.898	44.246
	5	1.756	76511.012	8.780	57.579
	5.5	1.758	76558.262	9.663	73.072
CS20H	0	1.401	61014.752	0.000	0.000
	0.5	1.401	61017.002	0.700	0.182
	1	1.401	61023.752	1.401	1.030
	1.5	1.401	61035.002	2.102	2.838
	2	1.402	61050.752	2.803	5.827
	2.5	1.402	61071.002	3.504	10.179
	3	1.403	61095.752	4.207	16.056
	3.5	1.403	61125.002	4.910	23.605
	4	1.404	61158.752	5.614	32.960
	4.5	1.405	61197.002	6.320	44.246
	5	1.406	61239.752	7.027	57.579
	5.5	1.407	61287.002	7.735	73.072

Table A-1. Continued

Pond	Depth, ft	Area, ac	Area, sq ft	Volume, AF	Outflow, cfs
EM80L	0	1.074	46769.664	0.000	0.000
	0.5	1.074	46771.914	0.537	0.182
	1	1.074	46778.664	1.074	1.030
	1.5	1.074	46789.914	1.611	2.838
	2	1.075	46805.664	2.149	5.827
	2.5	1.075	46825.914	2.687	10.179
	3	1.076	46850.664	3.226	16.056
	3.5	1.076	46879.914	3.766	23.605
	4	1.077	46913.664	4.306	32.960
	4.5	1.078	46951.914	4.848	44.246
	5	1.079	46994.664	5.392	57.579
	5.5	1.080	47041.914	5.937	73.072
ES80L	0	0.840	36583.537	0.000	0.000
	0.5	0.840	36585.787	0.420	0.182
	1	0.840	36592.537	0.840	1.030
	1.5	0.840	36603.787	1.260	2.838
	2	0.841	36619.537	1.681	5.827
	2.5	0.841	36639.787	2.102	10.179
	3	0.842	36664.537	2.524	16.056
	3.5	0.842	36693.787	2.947	23.605
	4	0.843	36727.537	3.371	32.960
	4.5	0.844	36765.787	3.796	44.246
	5	0.845	36808.537	4.223	57.579
	5.5	0.846	36855.787	4.651	73.072
EM20L	0	0.501	21838.890	0.000	0.000
	0.5	0.501	21841.140	0.251	0.182
	1	0.502	21847.890	0.501	1.030
	1.5	0.502	21859.140	0.753	2.838
	2	0.502	21874.890	1.004	5.827
	2.5	0.503	21895.140	1.256	10.179
	3	0.503	21919.890	1.509	16.056
	3.5	0.504	21949.140	1.762	23.605
	4	0.505	21982.890	2.017	32.960
	4.5	0.506	22021.140	2.273	44.246
	5	0.507	22063.890	2.530	57.579
	5.5	0.508	22111.140	2.789	73.072

Table A-1. Continued

Pond	Depth, ft	Area, ac	Area, sq ft	Volume, AF	Outflow, cfs
ES20L	0	0.405	17620.746	0.000	0.000
	0.5	0.405	17622.996	0.202	0.182
	1	0.405	17629.746	0.405	1.030
	1.5	0.405	17640.996	0.607	2.838
	2	0.405	17656.746	0.810	5.827
	2.5	0.406	17676.996	1.014	10.179
	3	0.406	17701.746	1.218	16.056
	3.5	0.407	17730.996	1.423	23.605
	4	0.408	17764.746	1.630	32.960
	4.5	0.409	17802.996	1.837	44.246
	5	0.410	17845.746	2.046	57.579
	5.5	0.411	17892.996	2.256	73.072
EM80Med	0	1.682	73263.660	0.000	0.000
	0.5	1.682	73265.910	0.841	0.182
	1	1.682	73272.660	1.682	1.030
	1.5	1.682	73283.910	2.523	2.838
	2	1.683	73299.660	3.365	5.827
	2.5	1.683	73319.910	4.207	10.179
	3	1.684	73344.660	5.050	16.056
	3.5	1.684	73373.910	5.894	23.605
	4	1.685	73407.660	6.739	32.960
	4.5	1.686	73445.910	7.585	44.246
	5	1.687	73488.660	8.433	57.579
	5.5	1.688	73535.910	9.282	73.072
ES80Med	0	1.332	58022.130	0.000	0.000
	0.5	1.332	58024.380	0.666	0.182
	1	1.332	58031.130	1.332	1.030
	1.5	1.332	58042.380	1.999	2.838
	2	1.333	58058.130	2.665	5.827
	2.5	1.333	58078.380	3.333	10.179
	3	1.334	58103.130	4.001	16.056
	3.5	1.335	58132.380	4.670	23.605
	4	1.335	58166.130	5.340	32.960
	4.5	1.336	58204.380	6.011	44.246
	5	1.337	58247.130	6.683	57.579
	5.5	1.338	58294.380	7.357	73.072

Table A-1. Continued

Pond	Depth, ft	Area, ac	Area, sq ft	Volume, AF	Outflow, cfs
EM20Med	0	0.779	33929.244	0.000	0.000
	0.5	0.779	33931.494	0.389	0.182
	1	0.779	33938.244	0.779	1.030
	1.5	0.779	33949.494	1.169	2.838
	2	0.780	33965.244	1.559	5.827
	2.5	0.780	33985.494	1.950	10.179
	3	0.781	34010.244	2.341	16.056
	3.5	0.781	34039.494	2.734	23.605
	4	0.782	34073.244	3.127	32.960
	4.5	0.783	34111.494	3.522	44.246
	5	0.784	34154.244	3.918	57.579
	5.5	0.785	34201.494	4.315	73.072
ES20Med	0	0.623	27130.411	0.000	0.000
	0.5	0.623	27132.661	0.311	0.182
	1	0.623	27139.411	0.623	1.030
	1.5	0.623	27150.661	0.935	2.838
	2	0.624	27166.411	1.247	5.827
	2.5	0.624	27186.661	1.560	10.179
	3	0.625	27211.411	1.873	16.056
	3.5	0.625	27240.661	2.188	23.605
	4	0.626	27274.411	2.503	32.960
	4.5	0.627	27312.661	2.820	44.246
	5	0.628	27355.411	3.138	57.579
	5.5	0.629	27402.661	3.457	73.072
EM80H	0	3.694	160912.970	0.000	0.000
	0.5	3.694	160915.220	1.847	0.182
	1	3.694	160921.970	3.694	1.030
	1.5	3.695	160933.220	5.542	2.838
	2	3.695	160948.970	7.389	5.827
	2.5	3.695	160969.220	9.238	10.179
	3	3.696	160993.970	11.087	16.056
	3.5	3.697	161023.220	12.937	23.605
	4	3.697	161056.970	14.788	32.960
	4.5	3.698	161095.220	16.640	44.246
	5	3.699	161137.970	18.494	57.579
	5.5	3.700	161185.220	20.349	73.072

Table A-1. Continued

Pond	Depth, ft	Area, ac	Area, sq ft	Volume, AF	Outflow, cfs
ES80H	0	2.991	130281.293	0.000	0.000
	0.5	2.991	130283.543	1.495	0.182
	1	2.991	130290.293	2.991	1.030
	1.5	2.991	130301.543	4.487	2.838
	2	2.992	130317.293	5.983	5.827
	2.5	2.992	130337.543	7.480	10.179
	3	2.993	130362.293	8.977	16.056
	3.5	2.993	130391.543	10.476	23.605
	4	2.994	130425.293	11.975	32.960
	4.5	2.995	130463.543	13.476	44.246
	5	2.996	130506.293	14.978	57.579
	5.5	2.997	130553.543	16.481	73.072
EM20H	0	1.686	73424.403	0.000	0.000
	0.5	1.686	73426.653	0.843	0.182
	1	1.686	73433.403	1.686	1.030
	1.5	1.686	73444.653	2.529	2.838
	2	1.686	73460.403	3.372	5.827
	2.5	1.687	73480.653	4.217	10.179
	3	1.687	73505.403	5.062	16.056
	3.5	1.688	73534.653	5.907	23.605
	4	1.689	73568.403	6.754	32.960
	4.5	1.690	73606.653	7.602	44.246
	5	1.691	73649.403	8.451	57.579
	5.5	1.692	73696.653	9.302	73.072
ES20H	0	1.366	59493.946	0.000	0.000
	0.5	1.366	59496.196	0.683	0.182
	1	1.366	59502.946	1.366	1.030
	1.5	1.366	59514.196	2.049	2.838
	2	1.367	59529.946	2.733	5.827
	2.5	1.367	59550.196	3.417	10.179
	3	1.368	59574.946	4.102	16.056
	3.5	1.368	59604.196	4.788	23.605
	4	1.369	59637.946	5.475	32.960
	4.5	1.370	59676.196	6.163	44.246
	5	1.371	59718.946	6.852	57.579
	5.5	1.372	59766.196	7.543	73.072

Table A-2. Modeling Runs for Evaluation of Development Intensity

Watershed: CM80- Classic, Mild, 80 ac Devs.				
Development Intensity	**Condition**	**Model**	**Subbasin**	**Detention Pond**
Low	without detention	CM80L	M80L	none
	with detention	CM80LD	M80L	M80L
Medium	without detention	CM80Med	M80Med	none
	with detention	CM80MedD	M80Med	M80Med
High	without detention	CM80H	M80H	none
	with detention	CM80HD	M80H	M80H
Watershed: CS80- Classic, Steep, 80 ac Devs.				
Development Intensity	**Condition**	**Model**	**Subbasin**	**Detention Pond**
Low	without detention	CS80L	S80L	none
	with detention	CS80LD	S80L	S80L
Medium	without detention	CS80Med	S80Med	none
	with detention	CS80MedD	S80Med	S80Med
High	without detention	CS80H	S80H	none
	with detention	CS80HD	S80H	S80H
Watershed: CM20- Classic, Mild, 20 ac Devs.				
Development Intensity	**Condition**	**Model**	**Subbasin**	**Detention Pond**
Low	without detention	CM20L	M20L	none
	with detention	CM20LD	M20L	M20L
Medium	without detention	CM20Med	M20Med	none
	with detention	CM20MedD	M20Med	M20Med
High	without detention	CM20H	M20H	none
	with detention	CM20HD	M20H	M20H
Watershed: CS20- Classic, Steep, 20 ac Devs.				
Development Intensity	**Condition**	**Model**	**Subbasin**	**Detention Pond**
Low	without detention	CS20L	S20L	none
	with detention	CS20LD	S20L	S20L
Medium	without detention	CS20Med	S20Med	none
	with detention	CS20MedD	S20Med	S20Med
High	without detention	CS20H	S20H	none
	with detention	CS20HD	S20H	S20H

Table A-2. Continued

Watershed: EM80- Elongated, Mild, 80 ac Devs.				
Development Intensity	**Condition**	**Model**	**Subbasin**	**Detention Pond**
Low	without detention	EM80L	M80L	none
	with detention	EM80LD	M80L	M80L
Medium	without detention	EM80Med	M80Med	none
	with detention	EM80MedD	M80Med	M80Med
High	without detention	EM80H	M80H	none
	with detention	EM80HD	M80H	M80H
Watershed: ES80- Elongated, Steep, 80 ac Devs.				
Development Intensity	**Condition**	**Model**	**Subbasin**	**Detention Pond**
Low	without detention	ES80L	S80L	none
	with detention	ES80LD	S80L	S80L
Medium	without detention	ES80Med	S80Med	none
	with detention	ES80MedD	S80Med	S80Med
High	without detention	ES80H	S80H	none
	with detention	ES80HD	S80H	S80H
Watershed: EM20- Elongated, Mild, 20 ac Devs.				
Development Intensity	**Condition**	**Model**	**Subbasin**	**Detention Pond**
Low	without detention	EM20L	M20L	none
	with detention	EM20LD	M20L	M20L
Medium	without detention	EM20Med	M20Med	none
	with detention	EM20MedD	M20Med	M20Med
High	without detention	EM20H	M20H	none
	with detention	EM20HD	M20H	M20H
Watershed: ES20- Elongated, Steep, 20 ac Devs.				
Development Intensity	**Condition**	**Model**	**Subbasin**	**Detention Pond**
Low	without detention	ES20L	S20L	none
	with detention	ES20LD	S20L	S20L
Medium	without detention	ES20Med	S20Med	none
	with detention	ES20MedD	S20Med	S20Med
High	without detention	ES20H	S20H	none
	with detention	ES20HD	S20H	S20H

Table A-3. Modeling Runs for Evaluation of Development Stage

Watershed: CM80- Classic, Mild, 80 ac Devs.				
Development Stage	**Condition**	**Model**	**Subbasin**	**Detention Pond**
25%	without detention	CM8025	M80Med	none
	with detention	CM8025D	M80Med	M80Med
50%	without detention	CM8050	M80Med	none
	with detention	CM8050D	M80Med	M80Med
75%	without detention	CM8075	M80Med	none
	with detention	CM8075D	M80Med	M80Med
100%	without detention	CM80Med	M80Med	none
	with detention	CM80MedD	M80Med	M80Med
Watershed: CS80- Classic, Steep, 80 ac Devs.				
Development Stage	**Condition**	**Model**	**Subbasin**	**Detention Pond**
25%	without detention	CS8025	S80Med	none
	with detention	CS8025D	S80Med	S80Med
50%	without detention	CS8050	S80Med	none
	with detention	CS8050D	S80Med	S80Med
75%	without detention	CS8075	S80Med	none
	with detention	CS8075D	S80Med	S80Med
100%	without detention	CS80Med	S80Med	none
	with detention	CS80MedD	S80Med	S80Med
Watershed: CM20- Classic, Mild, 20 ac Devs.				
Development Stage	**Condition**	**Model**	**Subbasin**	**Detention Pond**
25%	without detention	CM2025	M20Med	none
	with detention	CM2025D	M20Med	M20Med
50%	without detention	CM2050	M20Med	none
	with detention	CM2050D	M20Med	M20Med
75%	without detention	CM2075	M20Med	none
	with detention	CM2075D	M20Med	M20Med
100%	without detention	CM20Med	M20Med	none
	with detention	CM20MedD	M20Med	M20Med

Table A-3. Continued

Watershed: CS20- Classic, Steep, 20 ac Devs.				
Development Stage	**Condition**	**Model**	**Subbasin**	**Detention Pond**
25%	without detention	CS2025	S20Med	none
	with detention	CS2025D	S20Med	S20Med
50%	without detention	CS2050	S20Med	none
	with detention	CS2050D	S20Med	S20Med
75%	without detention	CS2075	S20Med	none
	with detention	CS2075D	S20Med	S20Med
100%	without detention	CS20Med	S20Med	none
	with detention	CS20MedD	S20Med	S20Med
Watershed: EM80- Elongated, Mild, 80 ac Devs.				
Development Stage	**Condition**	**Model**	**Subbasin**	**Detention Pond**
25%	without detention	EM8025	M80Med	none
	with detention	EM8025D	M80Med	M80Med
50%	without detention	EM8050	M80Med	none
	with detention	EM8050D	M80Med	M80Med
75%	without detention	EM8075	M80Med	none
	with detention	EM8075D	M80Med	M80Med
100%	without detention	EM80Med	M80Med	none
	with detention	EM80MedD	M80Med	M80Med
Watershed: ES80- Elongated, Steep, 80 ac Devs.				
Development Stage	**Condition**	**Model**	**Subbasin**	**Detention Pond**
25%	without detention	ES8025	S80Med	none
	with detention	ES8025D	S80Med	S80Med
50%	without detention	ES8050	S80Med	none
	with detention	ES8050D	S80Med	S80Med
75%	without detention	ES8075	S80Med	none
	with detention	ES8075D	S80Med	S80Med
100%	without detention	ES80Med	S80Med	none
	with detention	ES80MedD	S80Med	S80Med

Table A-3. Continued

Watershed: EM20- Elongated, Mild, 20 ac Devs.				
Development Stage	**Condition**	**Model**	**Subbasin**	**Detention Pond**
25%	without detention	EM2025	M20Med	none
	with detention	EM2025D	M20Med	M20Med
50%	without detention	EM2050	M20Med	none
	with detention	EM2050D	M20Med	M20Med
75%	without detention	EM2075	M20Med	none
	with detention	EM2075D	M20Med	M20Med
100%	without detention	EM20Med	M20Med	none
	with detention	EM20MedD	M20Med	M20Med
Watershed: ES20- Elongated, Steep, 20 ac Devs.				
Development Stage	**Condition**	**Model**	**Subbasin**	**Detention Pond**
25%	without detention	ES2025	S20Med	none
	with detention	ES2025D	S20Med	S20Med
50%	without detention	ES2050	S20Med	none
	with detention	ES2050D	S20Med	S20Med
75%	without detention	ES2075	S20Med	none
	with detention	ES2075D	S20Med	S20Med
100%	without detention	ES20Med	S20Med	none
	with detention	ES20MedD	S20Med	S20Med

Table A-4. Modeling Runs for Evaluation of Development Sequence

Watershed: CM80- Classic, Mild, 80 ac Devs.				
Development Sequence	**Condition**	**Model**	**Subbasin**	**Detention Pond**
25%	without detention	CM8025U	M80Med	none
	with detention	CM8025UD	M80Med	M80Med
50%	without detention	CM8050U	M80Med	none
	with detention	CM8050UD	M80Med	M80Med
75%	without detention	CM8075U	M80Med	none
	with detention	CM8075UD	M80Med	M80Med
100%	without detention	CM80Med	M80Med	none
	with detention	CM80MedD	M80Med	M80Med
Watershed: CS80- Classic, Steep, 80 ac Devs.				
Development Stage	**Condition**	**Model**	**Subbasin**	**Detention Pond**
25%	without detention	CS8025U	S80Med	none
	with detention	CS8025UD	S80Med	S80Med
50%	without detention	CS8050U	S80Med	none
	with detention	CS8050UD	S80Med	S80Med
75%	without detention	CS8075U	S80Med	none
	with detention	CS8075UD	S80Med	S80Med
100%	without detention	CS80Med	S80Med	none
	with detention	CS80MedD	S80Med	S80Med
Watershed: CM20- Classic, Mild, 20 ac Devs.				
Development Stage	**Condition**	**Model**	**Subbasin**	**Detention Pond**
25%	without detention	CM2025U	M20Med	none
	with detention	CM2025UD	M20Med	M20Med
50%	without detention	CM2050U	M20Med	none
	with detention	CM2050UD	M20Med	M20Med
75%	without detention	CM2075U	M20Med	none
	with detention	CM2075UD	M20Med	M20Med
100%	without detention	CM20Med	M20Med	none
	with detention	CM20MedD	M20Med	M20Med

Table A-4. Continued

Watershed: CS20- Classic, Steep, 20 ac Devs.				
Development Stage	**Condition**	**Model**	**Subbasin**	**Detention Pond**
25%	without detention	CS2025U	S20Med	none
	with detention	CS2025UD	S20Med	S20Med
50%	without detention	CS2050U	S20Med	none
	with detention	CS2050UD	S20Med	S20Med
75%	without detention	CS2075U	S20Med	none
	with detention	CS2075UD	S20Med	S20Med
100%	without detention	CS20Med	S20Med	none
	with detention	CS20MedD	S20Med	S20Med
Watershed: EM80- Elongated, Mild, 80 ac Devs.				
Development Stage	**Condition**	**Model**	**Subbasin**	**Detention Pond**
25%	without detention	EM8025U	M80Med	none
	with detention	EM8025UD	M80Med	M80Med
50%	without detention	EM8050U	M80Med	none
	with detention	EM8050UD	M80Med	M80Med
75%	without detention	EM8075U	M80Med	none
	with detention	EM8075UD	M80Med	M80Med
100%	without detention	EM80Med	M80Med	none
	with detention	EM80MedD	M80Med	M80Med
Watershed: ES80- Elongated, Steep, 80 ac Devs.				
Development Stage	**Condition**	**Model**	**Subbasin**	**Detention Pond**
25%	without detention	ES8025U	S80Med	none
	with detention	ES8025UD	S80Med	S80Med
50%	without detention	ES8050U	S80Med	none
	with detention	ES8050UD	S80Med	S80Med
75%	without detention	ES8075U	S80Med	none
	with detention	ES8075UD	S80Med	S80Med
100%	without detention	ES80Med	S80Med	none
	with detention	ES80MedD	S80Med	S80Med

Table A-4. Continued

Watershed: EM20- Elongated, Mild, 20 ac Devs.				
Development Stage	**Condition**	**Model**	**Subbasin**	**Detention Pond**
25%	without detention	EM2025U	M20Med	none
	with detention	EM2025UD	M20Med	M20Med
50%	without detention	EM2050U	M20Med	none
	with detention	EM2050UD	M20Med	M20Med
75%	without detention	EM2075U	M20Med	none
	with detention	EM2075UD	M20Med	M20Med
100%	without detention	EM20Med	M20Med	none
	with detention	EM20MedD	M20Med	M20Med
Watershed: ES20- Elongated, Steep, 20 ac Devs.				
Development Stage	**Condition**	**Model**	**Subbasin**	**Detention Pond**
25%	without detention	ES2025U	S20Med	none
	with detention	ES2025UD	S20Med	S20Med
50%	without detention	ES2050U	S20Med	none
	with detention	ES2050UD	S20Med	S20Med
75%	without detention	ES2075U	S20Med	none
	with detention	ES2075UD	S20Med	S20Med
100%	without detention	ES20Med	S20Med	none
	with detention	ES20MedD	S20Med	S20Med

Table A-5. Results from Evaluation of Development Intensity for the Classic Watersheds

		Junction											
Run	**Result**	**2A**	**2B**	**2C**	**2D**	**2E**	**2F**	**2G**	**2H**	**2I**	**2J**	**2K**	**2L**
CMPre	Peak Flow, cfs	357.66	532.4	701.03	357.66	532.4	701.03	357.66	532.4	701.03	357.66	532.4	701.03
	Time of Peak	1230	1234	1236	1230	1234	1236	1230	1234	1236	1230	1234	1236
CM80L	Peak Flow, cfs	1633.2	2386.2	3057.6	1633.2	2386.2	3057.6	1633.2	2386.2	3057.6	1633.2	2386.2	3057.6
	Time of Peak	1206	1208	1210	1206	1208	1210	1206	1208	1210	1206	1208	1210
CM80LD	Peak Flow, cfs	354.68	530.2	702.89	354.68	530.2	702.89	354.68	530.2	702.89	354.68	530.2	702.89
	Time of Peak	1234	1237	1241	1234	1237	1241	1234	1237	1241	1234	1237	1241
CM80Med	Peak Flow, cfs	2184.7	3203.7	4128.8	2184.7	3203.7	4128.8	2184.7	3203.7	4128.8	2184.7	3203.7	4128.8
	Time of Peak	1205	1208	1209	1205	1208	1209	1205	1208	1209	1205	1208	1209
CM80MedD	Peak Flow, cfs	355	531.44	706.3	355	531.44	706.3	355	531.44	706.3	355	531.44	706.3
	Time of Peak	1239	1243	1246	1239	1243	1246	1239	1243	1246	1239	1243	1246
CM80H	Peak Flow, cfs	3505	5163.6	6716.3	3505	5163.6	6716.3	3505	5163.6	6716.3	3505	5163.6	6716.3
	Time of Peak	1205	1207	1208	1205	1207	1208	1205	1207	1208	1205	1207	1208
CM80HD	Peak Flow, cfs	355.36	532.72	709.58	355.36	532.72	709.58	355.36	532.72	709.58	355.36	532.72	709.58
	Time of Peak	1249	1253	1256	1249	1253	1256	1249	1253	1256	1249	1253	1256
CM20L	Peak Flow, cfs	2044.3	2952.9	3717.9	2044.3	2952.9	3717.9	2044.3	2952.9	3717.9	2044.3	2952.9	3717.9
	Time of Peak	1201	1203	1205	1201	1203	1205	1201	1203	1205	1201	1203	1205
CM20LD	Peak Flow, cfs	353.04	528.02	700.66	353.04	528.02	700.66	353.04	528.02	700.66	353.04	528.02	700.66
	Time of Peak	1224	1228	1231	1224	1228	1231	1224	1228	1231	1224	1228	1231
CM20Med	Peak Flow, cfs	2708	3937.4	4995.7	2708	3937.4	4995.7	2708	3937.4	4995.7	2708	3937.4	4995.7
	Time of Peak	1201	1203	1204	1201	1203	1204	1201	1203	1204	1201	1203	1204
CM20MedD	Peak Flow, cfs	353.28	529.02	703.41	353.28	529.02	703.41	353.28	529.02	703.41	353.28	529.02	703.41
	Time of Peak	1230	1233	1236	1230	1233	1236	1230	1233	1236	1230	1233	1236
CM20H	Peak Flow, cfs	4269	6247.7	8027.9	4269	6247.7	8027.9	4269	6247.7	8027.9	4269	6247.7	8027.9
	Time of Peak	1200	1202	1203	1200	1202	1203	1200	1202	1203	1200	1202	1203
CM20HD	Peak Flow, cfs	352.62	528.67	704.33	352.62	528.67	704.33	352.62	528.67	704.33	352.62	528.67	704.33
	Time of Peak	1239	1243	1246	1239	1243	1246	1239	1243	1246	1239	1243	1246

Table A-5. Continued

		Junction			
Run	**Result**	**D**	**C**	**B**	**A**
CMPre	Peak Flow, cfs	1399.7	2029.3	2561.9	2533.7
	Time of Peak	1241	1249	1256	1306
CM80L	Peak Flow, cfs	6063.2	8223.7	9650	9357.8
	Time of Peak	1213	1219	1225	1232
CM80LD	Peak Flow, cfs	1404.8	2076.5	2709.5	2696.1
	Time of Peak	1245	1253	1301	1311
CM80Med	Peak Flow, cfs	8201.1	11248	13406	13015
	Time of Peak	1212	1217	1222	1228
CM80MedD	Peak Flow, cfs	1412	2100.3	2765.3	2758.3
	Time of Peak	1250	1258	1306	1315
CM80H	Peak Flow, cfs	13316	18521	22686	22132
	Time of Peak	1211	1215	1219	1224
CM80HD	Peak Flow, cfs	1419	2122.7	2818.3	2815.9
	Time of Peak	1300	1308	1315	1324
CM20L	Peak Flow, cfs	7348.3	9552.3	10765	10367
	Time of Peak	1208	1213	1219	1225
CM20LD	Peak Flow, cfs	1400.03	2072	2700.1	2683.3
	Time of Peak	1235	1244	1252	1302
CM20Med	Peak Flow, cfs	9886.2	13113	14968	14445
	Time of Peak	1207	1212	1217	1222
CM20MedD	Peak Flow, cfs	1406.2	2094.1	2760.1	2752.9
	Time of Peak	1241	1249	1256	1306
CM20H	Peak Flow, cfs	15860	21593	25540	24704
	Time of Peak	1206	1210	1213	1218
CM20HD	Peak Flow, cfs	1408.5	2108.2	2801.4	2799.4
	Time of Peak	1251	1259	1306	1315

Table A-5. Continued

		Junction											
Run	**Result**	**2A**	**2B**	**2C**	**2D**	**2E**	**2F**	**2G**	**2H**	**2I**	**2J**	**2K**	**2L**
CSPre	Peak Flow, cfs	534.75	796.14	1049	534.75	796.14	1049	534.75	796.14	1049	534.75	796.14	1049
	Time of Peak	1214	1216	1217	1214	1216	1217	1214	1216	1217	1214	1216	1217
CS80L	Peak Flow, cfs	1953.8	2892.2	3783.6	1953.8	2892.2	3783.6	1953.8	2892.2	3783.6	1953.8	2892.2	3783.6
	Time of Peak	1202	1203	1204	1202	1203	1204	1202	1203	1204	1202	1203	1204
CS80LD	Peak Flow, cfs	530.18	793.53	1054.3	530.18	793.53	1054.3	530.18	793.53	1054.3	530.18	793.53	1054.3
	Time of Peak	1217	1219	1221	1217	1219	1221	1217	1219	1221	1217	1219	1221
CS80Med	Peak Flow, cfs	2591.6	3854	5055.6	2591.6	3854	5055.6	2591.6	3854	5055.6	2591.6	3854	5055.6
	Time of Peak	1202	1203	1204	1202	1203	1204	1202	1203	1204	1202	1203	1204
CS80MedD	Peak Flow, cfs	530.24	794.4	1057.2	530.24	794.4	1057.2	530.24	794.4	1057.2	530.24	794.4	1057.2
	Time of Peak	1222	1224	1225	1222	1224	1225	1222	1224	1225	1222	1224	1225
CS80H	Peak Flow, cfs	4103	6113.5	8050.6	4103	6113.5	8050.6	4103	6113.5	8050.6	4103	6113.5	8050.6
	Time of Peak	1201	1202	1203	1201	1202	1203	1201	1202	1203	1201	1202	1203
CS80HD	Peak Flow, cfs	530.18	794.94	1059.2	530.18	794.94	1059.2	530.18	794.94	1059.2	530.18	794.94	1059.2
	Time of Peak	1229	1231	1232	1229	1231	1232	1229	1231	1232	1229	1231	1232
CS20L	Peak Flow, cfs	2292.8	3395	4430.6	2292.8	3395	4430.6	2292.8	3395	4430.6	2292.8	3395	4430.6
	Time of Peak	1158	1200	1201	1158	1200	1201	1158	1200	1201	1158	1200	1201
CS20LD	Peak Flow, cfs	530.2	790.78	1047.2	530.2	790.78	1047.2	530.2	790.78	1047.2	530.2	790.78	1047.2
	Time of Peak	1209	1211	1214	1209	1211	1214	1209	1211	1214	1209	1211	1214
CS20Med	Peak Flow, cfs	3061	4544.6	5941.9	3061	4544.6	5941.9	3061	4544.6	5941.9	3061	4544.6	5941.9
	Time of Peak	1158	1159	1200	1158	1159	1200	1158	1159	1200	1158	1159	1200
CS20MedD	Peak Flow, cfs	530.34	793.93	1056.7	530.34	793.93	1056.7	530.34	793.93	1056.7	530.34	793.93	1056.7
	Time of Peak	1210	1212	1215	1210	1212	1215	1210	1212	1215	1210	1212	1215
CS20H	Peak Flow, cfs	4788.8	7117	9342.4	4788.8	7117	9342.4	4788.8	7117	9342.4	4788.8	7117	9342.4
	Time of Peak	1157	1158	1159	1157	1158	1159	1157	1158	1159	1157	1158	1159
CS20HD	Peak Flow, cfs	531.36	796.75	1061.7	531.36	796.75	1061.7	531.36	796.75	1061.7	531.36	796.75	1061.7
	Time of Peak	1221	1223	1225	1221	1223	1225	1221	1223	1225	1221	1223	1225

Table A-5. Continued

		Junction			
Run	**Result**	**D**	**C**	**B**	**A**
CSPre	Peak Flow, cfs	2093.9	3059.4	3903.3	3891
	Time of Peak	1220	1224	1228	1233
CS80L	Peak Flow, cfs	7564.7	10885	13549	13445
	Time of Peak	1206	1209	1212	1215
CS80LD	Peak Flow, cfs	2107.3	3134.9	4118.7	4114
	Time of Peak	1224	1228	1232	1237
CS80Med	Peak Flow, cfs	10066	14598	18346	18258
	Time of Peak	1206	1208	1211	1214
CS80MedD	Peak Flow, cfs	2113.8	3157.1	4177.2	4175.4
	Time of Peak	1227	1232	1235	1240
CS80H	Peak Flow, cfs	16047	23441	29898	29727
	Time of Peak	1204	1207	1209	1212
CS80HD	Peak Flow, cfs	2118.2	3172.3	4218	4217.3
	Time of Peak	1235	1239	1242	1247
CS20L	Peak Flow, cfs	8834.5	12586	15391	15259
	Time of Peak	1202	1205	1208	1211
CS20LD	Peak Flow, cfs	2093.7	3099.9	4057.5	4049.4
	Time of Peak	1216	1221	1226	1231
CS20Med	Peak Flow, cfs	11824	17000	21015	20807
	Time of Peak	1201	1204	1206	1209
CS20MedD	Peak Flow, cfs	2112.8	3155	4175.1	4171.3
	Time of Peak	1217	1222	1227	1232
CS20H	Peak Flow, cfs	18623	26930	33953	33757
	Time of Peak	1200	1203	1204	1207
CS20HD	Peak Flow, cfs	2123.3	3180.5	4230.6	4230.1
	Time of Peak	1227	1231	1234	1239

Table A-6. Results from Evaluation of Development Intensity for the Elongated Watersheds

		Junction											
Run	**Result**	**2A**	**2B**	**2C**	**2D**	**2E**	**2F**	**2G**	**2H**	**2I**	**2J**	**2K**	**2L**
EMPre	Peak Flow, cfs	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54
	Time of Peak	1226	1229	1231	1233	1226	1229	1231	1233	1226	1229	1231	1233
EM80L	Peak Flow, cfs	979.9	1442.7	1871.4	2261.4	979.9	1442.7	1871.4	2261.4	979.9	1442.7	1871.4	2261.4
	Time of Peak	1206	1208	1209	1211	1206	1208	1209	1211	1206	1208	1209	1211
EM80LD	Peak Flow, cfs	245.3	366.97	487.08	605.48	245.3	366.97	487.08	605.48	245.3	366.97	487.08	605.48
	Time of Peak	1230	1233	1236	1238	1230	1233	1236	1238	1230	1233	1236	1238
EM80Med	Peak Flow, cfs	1310.8	1937.1	2520.5	3064.5	1310.8	1937.1	2520.5	3064.5	1310.8	1937.1	2520.5	3064.5
	Time of Peak	1205	1207	1209	1210	1205	1207	1209	1210	1205	1207	1209	1210
EM80MedD	Peak Flow, cfs	245.26	367.31	488.45	608.6	245.26	367.31	488.45	608.6	245.26	367.31	488.45	608.6
	Time of Peak	1235	1238	1241	1243	1235	1238	1241	1243	1235	1238	1241	1243
EM80H	Peak Flow, cfs	2103	3117.7	4081	4989.8	2103	3117.7	4081	4989.8	2103	3117.7	4081	4989.8
	Time of Peak	1205	1206	1207	1208	1205	1206	1207	1208	1205	1206	1207	1208
EM80HD	Peak Flow, cfs	246.24	369.15	491.74	613.94	246.24	369.15	491.74	613.94	246.24	369.15	491.74	613.94
	Time of Peak	1244	1247	1249	1251	1244	1247	1249	1251	1244	1247	1249	1251
EM20L	Peak Flow, cfs	1328.8	1944.1	2492.9	2970.4	1328.8	1944.1	2492.9	2970.4	1328.8	1944.1	2492.9	2970.4
	Time of Peak	1201	1203	1204	1205	1201	1203	1204	1205	1201	1203	1204	1205
EM20LD	Peak Flow, cfs	245.68	367.73	488.61	608.21	245.68	367.73	488.61	608.21	245.68	367.73	488.61	608.21
	Time of Peak	1222	1225	1228	1230	1222	1225	1228	1230	1222	1225	1228	1230
EM20Med	Peak Flow, cfs	1759.2	2580.6	3333.4	4009.4	1759.2	2580.6	3333.4	4009.4	1759.2	2580.6	3333.4	4009.4
	Time of Peak	1201	1202	1204	1205	1201	1202	1204	1205	1201	1202	1204	1205
EM20MedD	Peak Flow, cfs	244.16	365.82	486.86	607.09	244.16	365.82	486.86	607.09	244.16	365.82	486.86	607.09
	Time of Peak	1228	1231	1233	1235	1228	1231	1233	1235	1228	1231	1233	1235
EM20H	Peak Flow, cfs	2774.8	4084.5	5315	6449.7	2774.8	4084.5	5315	6449.7	2774.8	4084.5	5315	6449.7
	Time of Peak	1200	1201	1203	1203	1200	1201	1203	1203	1200	1201	1203	1203
EM20HD	Peak Flow, cfs	244.46	366.52	488.36	609.94	244.46	366.52	488.36	609.94	244.46	366.52	488.36	609.94
	Time of Peak	1237	1240	1242	1244	1237	1240	1242	1244	1237	1240	1242	1244

Table A-6. Continued

		Junction												
Run	**Result**	**2M**	**2N**	**2O**	**2P**	**2Q**	**2R**	**2S**	**2T**	**E**	**D**	**C**	**B**	**A**
EMPre	Peak Flow, cfs	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54	1198.8	1682.1	2036.3	2298.5	2261.1
	Time of Peak	1226	1229	1231	1233	1226	1229	1231	1233	1236	1248	1300	1312	1324
EM80L	Peak Flow, cfs	979.9	1442.7	1871.4	2261.4	979.9	1442.7	1871.4	2261.4	4495.1	5775.5	6455.8	6872.2	6687.6
	Time of Peak	1206	1208	1209	1211	1206	1208	1209	1211	1213	1222	1230	1239	1248
EM80LD	Peak Flow, cfs	245.3	366.97	487.08	605.48	245.3	366.97	487.08	605.48	1210.1	1762.9	2248.3	2672.5	2647.2
	Time of Peak	1230	1233	1236	1238	1230	1233	1236	1238	1241	1253	1304	1316	1328
EM80Med	Peak Flow, cfs	1310.8	1937.1	2520.5	3064.5	1310.8	1937.1	2520.5	3064.5	6093.1	7955.9	9015.7	9655.9	9415.5
	Time of Peak	1205	1207	1209	1210	1205	1207	1209	1210	1212	1220	1227	1235	1243
EM80MedD	Peak Flow, cfs	245.26	367.31	488.45	608.6	245.26	367.31	488.45	608.6	1216.7	1795.1	2333.2	2827.8	2812.6
	Time of Peak	1235	1238	1241	1243	1235	1238	1241	1243	1246	1257	1308	1319	1331
EM80H	Peak Flow, cfs	2103	3117.7	4081	4989.8	2103	3117.7	4081	4989.8	9930.9	13308	15414	16766	16433
	Time of Peak	1205	1206	1207	1208	1205	1206	1207	1208	1210	1217	1223	1229	1236
EM80HD	Peak Flow, cfs	246.24	369.15	491.74	613.94	246.24	369.15	491.74	613.94	1227.7	1830.5	2417.8	2987.3	2982.2
	Time of Peak	1244	1247	1249	1251	1244	1247	1249	1251	1254	1305	1315	1325	1336
EM20L	Peak Flow, cfs	1328.8	1944.1	2492.9	2970.4	1328.8	1944.1	2492.9	2970.4	5887	7154.6	7656.3	7956.5	7715.5
	Time of Peak	1201	1203	1204	1205	1201	1203	1204	1205	1208	1216	1224	1233	1241
EM20LD	Peak Flow, cfs	245.68	367.73	488.61	608.21	245.68	367.73	488.61	608.21	1215.7	1780.3	2281.9	2723.3	2697.3
	Time of Peak	1222	1225	1228	1230	1222	1225	1228	1230	1233	1245	1258	1309	1322
EM20Med	Peak Flow, cfs	1759.2	2580.6	3333.4	4009.4	1759.2	2580.6	3333.4	4009.4	7945.2	9884.5	10708	11179	10877
	Time of Peak	1201	1202	1204	1205	1201	1202	1204	1205	1207	1214	1221	1229	1237
EM20MedD	Peak Flow, cfs	244.16	365.82	486.86	607.09	244.16	365.82	486.86	607.09	1213.7	1797.1	2347.1	2858.9	2843.9
	Time of Peak	1228	1231	1233	1235	1228	1231	1233	1235	1238	1250	1301	1312	1324
EM20H	Peak Flow, cfs	2774.8	4084.5	5315	6449.7	2774.8	4084.5	5315	6449.7	12806	16558	18455	19509	19090
	Time of Peak	1200	1201	1203	1203	1200	1201	1203	1203	1205	1211	1216	1223	1229
EM20HD	Peak Flow, cfs	244.46	366.52	488.36	609.94	244.46	366.52	488.36	609.94	1219.7	1822.1	2413.9	2992.9	2989
	Time of Peak	1237	1240	1242	1244	1237	1240	1242	1244	1247	1259	1308	1318	1330

Table A-6. Continued

		Junction											
Run	**Result**	**2A**	**2B**	**2C**	**2D**	**2E**	**2F**	**2G**	**2H**	**2I**	**2J**	**2K**	**2L**
ESPre	Peak Flow, cfs	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62
	Time of Peak	1212	1214	1215	1216	1212	1214	1215	1216	1212	1214	1215	1216
ES80L	Peak Flow, cfs	1172.3	1744.5	2296	2821.7	1172.3	1744.5	2296	2821.7	1172.3	1744.5	2296	2821.7
	Time of Peak	1202	1203	1204	1205	1202	1203	1204	1205	1202	1203	1204	1205
ES80LD	Peak Flow, cfs	359.7	538.58	715.15	889.95	359.7	538.58	715.15	889.95	359.7	538.58	715.15	889.95
	Time of Peak	1215	1217	1219	1220	1215	1217	1219	1220	1215	1217	1219	1220
ES80Med	Peak Flow, cfs	1555	2317.2	3050.8	3769.1	1555	2317.2	3050.8	3769.1	1555	2317.2	3050.8	3769.1
	Time of Peak	1202	1203	1203	1204	1202	1203	1203	1204	1202	1203	1203	1204
ES80MedD	Peak Flow, cfs	360.1	539.59	718.21	895.71	360.1	539.59	718.21	895.71	360.1	539.59	718.21	895.71
	Time of Peak	1219	1221	1222	1223	1219	1221	1222	1223	1219	1221	1222	1223
ES80H	Peak Flow, cfs	2461.8	3671.7	4847.3	6007.4	2461.8	3671.7	4847.3	6007.4	2461.8	3671.7	4847.3	6007.4
	Time of Peak	1201	1202	1203	1203	1201	1202	1203	1203	1201	1202	1203	1203
ES80HD	Peak Flow, cfs	360.1	539.93	719.43	898.57	360.1	539.93	719.43	898.57	360.1	539.93	719.43	898.57
	Time of Peak	1226	1227	1229	1230	1226	1227	1229	1230	1226	1227	1229	1230
ES20L	Peak Flow, cfs	1490.3	2213.6	2911.6	3574.6	1490.3	2213.6	2911.6	3574.6	1490.3	2213.6	2911.6	3574.6
	Time of Peak	1158	1159	1200	1201	1158	1159	1200	1201	1158	1159	1200	1201
ES20LD	Peak Flow, cfs	359.5	536.86	711.94	884.29	359.5	536.86	711.94	884.29	359.5	536.86	711.94	884.29
	Time of Peak	1209	1211	1212	1214	1209	1211	1212	1214	1209	1211	1212	1214
ES20Med	Peak Flow, cfs	1989.7	2958.7	3895.2	4799.7	1989.7	2958.7	3895.2	4799.7	1989.7	2958.7	3895.2	4799.7
	Time of Peak	1158	1159	1159	1200	1158	1159	1159	1200	1158	1159	1159	1200
ES20MedD	Peak Flow, cfs	361.4	541.23	720.1	897.9	361.4	541.23	720.1	897.9	361.4	541.23	720.1	897.9
	Time of Peak	1209	1211	1213	1215	1209	1211	1213	1215	1209	1211	1213	1215
ES20H	Peak Flow, cfs	3110.8	4632.3	6102.3	7549.7	3110.8	4632.3	6102.3	7549.7	3110.8	4632.3	6102.3	7549.7
	Time of Peak	1157	1158	1158	1159	1157	1158	1158	1159	1157	1158	1158	1159
ES20HD	Peak Flow, cfs	361.3	541.78	722.03	902.03	361.3	541.78	722.03	902.03	361.3	541.78	722.03	902.03
	Time of Peak	1220	1221	1223	1224	1220	1221	1223	1224	1220	1221	1223	1224

Table A-6. Continued

		Junction												
Run	**Result**	**2M**	**2N**	**2O**	**2P**	**2Q**	**2R**	**2S**	**2T**	**E**	**D**	**C**	**B**	**A**
ESPre	Peak Flow, cfs	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62	1762.9	2512.7	3097.9	3558.5	3545.3
	Time of Peak	1212	1214	1215	1216	1212	1214	1215	1216	1218	1224	1230	1236	1242
ES80L	Peak Flow, cfs	1172.3	1744.5	2296	2821.7	1172.3	1744.5	2296	2821.7	5628.9	7856.2	9350.5	10356	10286
	Time of Peak	1202	1203	1204	1205	1202	1203	1204	1205	1206	1210	1215	1219	1224
ES80LD	Peak Flow, cfs	359.7	538.58	715.15	889.95	359.7	538.58	715.15	889.95	1778.6	2619	3389.3	4084.3	4076.8
	Time of Peak	1215	1217	1219	1220	1215	1217	1219	1220	1222	1228	1234	1240	1246
ES80Med	Peak Flow, cfs	1555	2317.2	3050.8	3769.1	1555	2317.2	3050.8	3769.1	7508.3	10602	12811	14300	14228
	Time of Peak	1202	1203	1203	1204	1202	1203	1203	1204	1205	1209	1213	1217	1221
ES80MedD	Peak Flow, cfs	360.1	539.59	718.21	895.71	360.1	539.59	718.21	895.71	1791	2659.3	3486.5	4267.9	4263.6
	Time of Peak	1219	1221	1222	1223	1219	1221	1222	1223	1225	1231	1236	1242	1248
ES80H	Peak Flow, cfs	2461.8	3671.7	4847.3	6007.4	2461.8	3671.7	4847.3	6007.4	11975	17129	21170	24146	24034
	Time of Peak	1201	1202	1203	1203	1201	1202	1203	1203	1204	1208	1210	1213	1217
ES80HD	Peak Flow, cfs	360.1	539.93	719.43	898.57	360.1	539.93	719.43	898.57	1797	2685.4	3556.8	4409	4407.7
	Time of Peak	1226	1227	1229	1230	1226	1227	1229	1230	1231	1237	1242	1247	1253
ES20L	Peak Flow, cfs	1490.3	2213.6	2911.6	3574.6	1490.3	2213.6	2911.6	3574.6	7122.4	9792.9	11348	12137	12060
	Time of Peak	1158	1159	1200	1201	1158	1159	1200	1201	1202	1206	1210	1214	1219
ES20LD	Peak Flow, cfs	359.5	536.86	711.94	884.29	359.5	536.86	711.94	884.29	1766.7	2596.1	3363.8	4064.4	4054.1
	Time of Peak	1209	1211	1212	1214	1209	1211	1212	1214	1216	1222	1229	1235	1241
ES20Med	Peak Flow, cfs	1989.7	2958.7	3895.2	4799.7	1989.7	2958.7	3895.2	4799.7	9565.7	13311	15656	16945	16838
	Time of Peak	1158	1159	1159	1200	1158	1159	1159	1200	1201	1205	1208	1212	1216
ES20MedD	Peak Flow, cfs	361.4	541.23	720.1	897.9	361.4	541.23	720.1	897.9	1795	2668.5	3506.1	4299	4293.8
	Time of Peak	1209	1211	1213	1215	1209	1211	1213	1215	1217	1224	1230	1236	1242
ES20H	Peak Flow, cfs	3110.8	4632.3	6102.3	7549.7	3110.8	4632.3	6102.3	7549.7	15042	21303	25835	28719	28525
	Time of Peak	1157	1158	1158	1159	1157	1158	1158	1159	1200	1203	1206	1209	1212
ES20HD	Peak Flow, cfs	361.3	541.78	722.03	902.03	361.3	541.78	722.03	902.03	1803.9	2699.2	3584.3	4456.4	4455.1
	Time of Peak	1220	1221	1223	1224	1220	1221	1223	1224	1226	1231	1235	1239	1245

Table A-7. Results from Evaluation of Development Stage for the Classic Watersheds

		Junction											
Run	**Result**	**2A**	**2B**	**2C**	**2D**	**2E**	**2F**	**2G**	**2H**	**2I**	**2J**	**2K**	**2L**
CMPre	Peak Flow, cfs	357.66	532.4	701.03	357.66	532.4	701.03	357.66	532.4	701.03	357.66	532.4	701.03
	Time of Peak	1230	1234	1236	1230	1234	1236	1230	1234	1236	1230	1234	1236
CM8025	Peak Flow, cfs	357.66	532.39	701.03	357.66	532.39	701.03	357.66	532.39	701.03	2184.6	3203.7	4128.8
	Time of Peak	1230	1234	1236	1230	1234	1236	1230	1234	1236	1205	1208	1209
CM8025D	Peak Flow, cfs	357.66	532.39	701.03	357.66	532.39	701.03	357.66	532.39	701.03	355	531.44	706.3
	Time of Peak	1230	1234	1236	1230	1234	1236	1230	1234	1236	1239	1243	1246
CM8050	Peak Flow, cfs	357.66	532.39	701.03	357.66	532.39	701.03	2184.6	3203.7	4128.8	2184.6	3203.7	4128.8
	Time of Peak	1230	1234	1236	1230	1234	1236	1205	1208	1209	1205	1208	1209
CM8050D	Peak Flow, cfs	357.66	532.39	701.03	357.66	532.39	701.03	355	531.44	706.3	355	531.44	706.3
	Time of Peak	1230	1234	1236	1230	1234	1236	1239	1243	1246	1239	1243	1246
CM8075	Peak Flow, cfs	357.66	532.39	701.03	2184.6	3203.7	4128.8	2184.6	3203.7	4128.8	2184.6	3203.7	4128.8
	Time of Peak	1230	1234	1236	1205	1208	1209	1205	1208	1209	1205	1208	1209
CM8075D	Peak Flow, cfs	357.66	532.39	701.03	355	531.44	706.3	355	531.44	706.3	355	531.44	706.3
	Time of Peak	1230	1234	1236	1239	1243	1246	1239	1243	1246	1239	1243	1246
CM2025	Peak Flow, cfs	357.66	532.39	701.03	357.66	532.39	701.03	357.66	532.39	701.03	2708	3937.4	4995.7
	Time of Peak	1230	1234	1236	1230	1234	1236	1230	1234	1236	1201	1203	1204
CM2025D	Peak Flow, cfs	357.66	532.39	701.03	357.66	532.39	701.03	357.66	532.39	701.03	353.28	529.02	703.41
	Time of Peak	1230	1234	1236	1230	1234	1236	1230	1234	1236	1230	1233	1236
CM2050	Peak Flow, cfs	357.66	532.39	701.03	357.66	532.39	701.03	2708	3937.4	4995.7	2708	3937.4	4995.7
	Time of Peak	1230	1234	1236	1230	1234	1236	1201	1203	1204	1201	1203	1204
CM2050D	Peak Flow, cfs	357.66	532.39	701.03	357.66	532.39	701.03	353.28	529.02	703.41	353.28	529.02	703.41
	Time of Peak	1230	1234	1236	1230	1234	1236	1230	1233	1236	1230	1233	1236
CM2075	Peak Flow, cfs	357.66	532.39	701.03	2708	3937.4	4995.7	2708	3937.4	4995.7	2708	3937.4	4995.7
	Time of Peak	1230	1234	1236	1201	1203	1204	1201	1203	1204	1201	1203	1204
CM2075D	Peak Flow, cfs	357.66	532.39	701.03	353.28	529.02	703.41	353.28	529.02	703.41	353.28	529.02	703.41
	Time of Peak	1230	1234	1236	1230	1233	1236	1230	1233	1236	1230	1233	1236

Table A-7. Continued

		Junction			
Run	**Result**	**D**	**C**	**B**	**A**
CMPre	Peak Flow, cfs	1399.7	2029.3	2561.9	2533.7
	Time of Peak	1241	1249	1256	1306
CM8025	Peak Flow, cfs	1399.7	2029.3	4101.1	3903.8
	Time of Peak	1241	1249	1212	1220
CM8025D	Peak Flow, cfs	1399.7	2029.3	2702	2677.2
	Time of Peak	1241	1249	1258	1307
CM8050	Peak Flow, cfs	1399.7	4101.2	7246	7005.2
	Time of Peak	1241	1212	1217	1224
CM8050D	Peak Flow, cfs	1399.7	2090.4	2763	2744.3
	Time of Peak	1241	1251	1300	1310
CM8075	Peak Flow, cfs	4166.6	7291.4	9602.1	9378.5
	Time of Peak	1212	1217	1222	1228
CM8075D	Peak Flow, cfs	1388.7	2082.7	2755.1	2742
	Time of Peak	1243	1253	1302	1311
CM2025	Peak Flow, cfs	1399.7	2029.3	4943.4	4600.6
	Time of Peak	1241	1249	1207	1215
CM2025D	Peak Flow, cfs	1399.7	2029.3	2675.7	2651.2
	Time of Peak	1241	1249	1258	1307
CM2050	Peak Flow, cfs	1399.7	4943.5	8305.7	7940.7
	Time of Peak	1241	1207	1211	1218
CM2050D	Peak Flow, cfs	1399.7	2073	2719.9	2702.3
	Time of Peak	1241	1251	1300	1309
CM2075	Peak Flow, cfs	4969	8332.5	10572	10307
	Time of Peak	1207	1212	1216	1222
CM2075D	Peak Flow, cfs	1403	2083	2737.7	2725.9
	Time of Peak	1241	1250	1259	1308

Table A-7. Continued

		Junction											
Run	**Result**	**2A**	**2B**	**2C**	**2D**	**2E**	**2F**	**2G**	**2H**	**2I**	**2J**	**2K**	**2L**
CSPre	Peak Flow, cfs	534.75	796.14	1049	534.75	796.14	1049	534.75	796.14	1049	534.75	796.14	1049
	Time of Peak	1214	1216	1217	1214	1216	1217	1214	1216	1217	1214	1216	1217
CS8025	Peak Flow, cfs	534.75	796.14	1049	534.75	796.14	1049	534.75	796.14	1049	2591.6	3854	5055.6
	Time of Peak	1214	1216	1217	1214	1216	1217	1214	1216	1217	1202	1203	1204
CS8025D	Peak Flow, cfs	534.75	796.14	1049	534.75	796.14	1049	534.75	796.14	1049	530.24	794.4	1057.2
	Time of Peak	1214	1216	1217	1214	1216	1217	1214	1216	1217	1222	1224	1225
CS8050	Peak Flow, cfs	534.75	796.14	1049	534.75	796.14	1049	2591.6	3854	5055.6	2591.6	3854	5055.6
	Time of Peak	1214	1216	1217	1214	1216	1217	1202	1203	1204	1202	1203	1204
CS8050D	Peak Flow, cfs	534.75	796.14	1049	534.75	796.14	1049	530.24	794.4	1057.2	530.24	794.4	1057.2
	Time of Peak	1214	1216	1217	1214	1216	1217	1222	1224	1225	1222	1224	1225
CS8075	Peak Flow, cfs	534.75	796.14	1049	2591.6	3854	5055.6	2591.6	3854	5055.6	2591.6	3854	5055.6
	Time of Peak	1214	1216	1217	1202	1203	1204	1202	1203	1204	1202	1203	1204
CS8075D	Peak Flow, cfs	534.75	796.14	1049	530.24	794.4	1057.2	530.24	794.4	1057.2	530.24	794.4	1057.2
	Time of Peak	1214	1216	1217	1222	1224	1225	1222	1224	1225	1222	1224	1225
CS2025	Peak Flow, cfs	534.75	796.14	1049	534.75	796.14	1049	534.75	796.14	1049	3061	4544.6	5941.9
	Time of Peak	1214	1216	1217	1214	1216	1217	1214	1216	1217	1158	1159	1200
CS2025D	Peak Flow, cfs	534.75	796.14	1049	534.75	796.14	1049	534.75	796.14	1049	530.34	793.93	1056.7
	Time of Peak	1214	1216	1217	1214	1216	1217	1214	1216	1217	1210	1212	1215
CS2050	Peak Flow, cfs	534.75	796.14	1049	534.75	796.14	1049	3061	4544.6	5941.9	3061	4544.6	5941.9
	Time of Peak	1214	1216	1217	1214	1216	1217	1158	1159	1200	1158	1159	1200
CS2050D	Peak Flow, cfs	534.75	796.14	1049	534.75	796.14	1049	530.34	793.93	1056.7	530.34	793.93	1056.7
	Time of Peak	1214	1216	1217	1214	1216	1217	1210	1212	1215	1210	1212	1215
CS2075	Peak Flow, cfs	534.75	796.14	1049	3061	4544.6	5941.9	3061	4544.6	5941.9	3061	4544.6	5941.9
	Time of Peak	1214	1216	1217	1158	1159	1200	1158	1159	1200	1158	1159	1200
CS2075D	Peak Flow, cfs	534.75	796.14	1049	530.34	793.93	1056.7	530.34	793.93	1056.7	530.34	793.93	1056.7
	Time of Peak	1214	1216	1217	1210	1212	1215	1210	1212	1215	1210	1212	1215

Table A-7. Continued

		Junction			
Run	**Result**	**D**	**C**	**B**	**A**
CSPre	Peak Flow, cfs	2093.9	3059.4	3903.3	3891
	Time of Peak	1220	1224	1228	1233
CS8025	Peak Flow, cfs	2093.9	3059.4	5068.3	5037.6
	Time of Peak	1220	1224	1206	1210
CS8025D	Peak Flow, cfs	2093.9	3059.4	4106.6	4096.1
	Time of Peak	1220	1224	1229	1234
CS8050	Peak Flow, cfs	2093.9	5093.5	9573.5	9516.9
	Time of Peak	1220	1206	1208	1212
CS8050D	Peak Flow, cfs	2093.9	3141	4185.8	4179.9
	Time of Peak	1220	1225	1230	1235
CS8075	Peak Flow, cfs	5344.2	9745.2	13532	13460
	Time of Peak	1206	1208	1210	1214
CS8075D	Peak Flow, cfs	2057	3110.5	4151.7	4146.1
	Time of Peak	1221	1227	1232	1237
CS2025	Peak Flow, cfs	2093.9	3059.4	5914.6	5860.1
	Time of Peak	1220	1224	1201	1206
CS2025D	Peak Flow, cfs	2093.9	3059.4	4062	4051.4
	Time of Peak	1220	1224	1229	1234
CS2050	Peak Flow, cfs	2093.9	5915.6	11060	10935
	Time of Peak	1220	1201	1204	1208
CS2050D	Peak Flow, cfs	2093.9	3123.1	4120	4111.4
	Time of Peak	1220	1225	1230	1235
CS2075	Peak Flow, cfs	6013.3	11124	15250	15119
	Time of Peak	1202	1204	1206	1209
CS2075D	Peak Flow, cfs	2101	3132.4	4138.3	4133.5
	Time of Peak	1219	1224	1229	1234

Table A-8. Results from Evaluation of Development Stage for the Elongated Watersheds

		Junction											
Run	**Result**	**2A**	**2B**	**2C**	**2D**	**2E**	**2F**	**2G**	**2H**	**2I**	**2J**	**2K**	**2L**
EMPre	Peak Flow, cfs	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54
	Time of Peak	1226	1229	1231	1233	1226	1229	1231	1233	1226	1229	1231	1233
EM8025	Peak Flow, cfs	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54
	Time of Peak	1226	1229	1231	1233	1226	1229	1231	1233	1226	1229	1231	1233
EM8025D	Peak Flow, cfs	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54
	Time of Peak	1226	1229	1231	1233	1226	1229	1231	1233	1226	1229	1231	1233
EM8050	Peak Flow, cfs	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54	247.16	368.55	683.44	1303.1
	Time of Peak	1226	1229	1231	1233	1226	1229	1231	1233	1226	1229	1206	1207
EM8050D	Peak Flow, cfs	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54	247.16	368.55	490.11	612.13
	Time of Peak	1226	1229	1231	1233	1226	1229	1231	1233	1226	1229	1233	1236
EM8075	Peak Flow, cfs	247.16	368.55	486.58	600.54	247.16	690.4	1309	1895	1310.8	1937.1	2520.5	3064.5
	Time of Peak	1226	1229	1231	1233	1226	1206	1207	1208	1205	1207	1209	1210
EM8075D	Peak Flow, cfs	247.16	368.55	486.58	600.54	247.16	368.08	490.11	612.05	245.26	367.31	488.45	608.6
	Time of Peak	1226	1229	1231	1233	1226	1231	1235	1238	1235	1238	1241	1243
EM2025	Peak Flow, cfs	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54
	Time of Peak	1226	1229	1231	1233	1226	1229	1231	1233	1226	1229	1231	1233
EM2025D	Peak Flow, cfs	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54
	Time of Peak	1226	1229	1231	1233	1226	1229	1231	1233	1226	1229	1231	1233
EM2050	Peak Flow, cfs	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54	247.16	368.55	886.84	1715.4
	Time of Peak	1226	1229	1231	1233	1226	1229	1231	1233	1226	1229	1201	1202
EM2050D	Peak Flow, cfs	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54	247.16	368.55	489.47	609.29
	Time of Peak	1226	1229	1231	1233	1226	1229	1231	1233	1226	1229	1232	1235
EM2075	Peak Flow, cfs	247.16	368.55	486.58	600.54	247.16	890.36	1717.5	2482	1759.2	2580.6	3333.4	4009.4
	Time of Peak	1226	1229	1231	1233	1226	1201	1202	1203	1201	1202	1204	1205
EM2075D	Peak Flow, cfs	247.16	368.55	486.58	600.54	247.16	368.69	489.49	609.17	244.16	365.82	486.86	607.09
	Time of Peak	1226	1229	1231	1233	1226	1230	1233	1236	1228	1231	1233	1235

Table A-8. Continued

		Junction												
Run	**Result**	**2M**	**2N**	**2O**	**2P**	**2Q**	**2R**	**2S**	**2T**	**E**	**D**	**C**	**B**	**A**
EMPre	Peak Flow, cfs	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54	1198.8	1682.1	2036.3	2298.5	2261.1
	Time of Peak	1226	1229	1231	1233	1226	1229	1231	1233	1236	1248	1300	1312	1324
EM8025	Peak Flow, cfs	247.16	368.55	486.58	682.63	1310.8	1937.1	2520.5	3064.5	1198.8	1682.1	2040.4	3149.6	3017.7
	Time of Peak	1226	1229	1231	1206	1205	1207	1209	1210	1236	1248	1300	1212	1224
EM8025D	Peak Flow, cfs	247.16	368.55	486.58	608.17	245.26	367.31	488.45	608.6	1198.8	1682.1	2081.8	2580	2543.9
	Time of Peak	1226	1229	1231	1234	1235	1238	1241	1243	1236	1248	1300	1312	1324
EM8050	Peak Flow, cfs	1310.8	1937.1	2520.5	3064.5	1310.8	1937.1	2520.5	3064.5	1198.8	1687.6	3486.6	5146.4	5033.3
	Time of Peak	1205	1207	1209	1210	1205	1207	1209	1210	1236	1248	1217	1221	1231
EM8050D	Peak Flow, cfs	245.26	367.31	488.45	608.6	245.26	367.31	488.45	608.6	1198.8	1740.6	2286.3	2784.7	2751.1
	Time of Peak	1235	1238	1241	1243	1235	1238	1241	1243	1236	1249	1301	1313	1325
EM8075	Peak Flow, cfs	1310.8	1937.1	2520.5	3064.5	1310.8	1937.1	2520.5	3064.5	1960.3	3987.5	5380.2	6441.1	6368.1
	Time of Peak	1205	1207	1209	1210	1205	1207	1209	1210	1211	1218	1224	1229	1238
EM8075D	Peak Flow, cfs	245.26	367.31	488.45	608.6	245.26	367.31	488.45	608.6	1204.2	1789.4	2334.9	2832.4	2806.9
	Time of Peak	1235	1238	1241	1243	1235	1238	1241	1243	1238	1252	1304	1315	1327
EM2025	Peak Flow, cfs	247.16	368.55	486.58	887.32	1759.2	2580.6	3333.4	4009.4	1198.8	1682.1	2038.8	4080	3793.7
	Time of Peak	1226	1229	1231	1201	1201	1202	1204	1205	1236	1248	1300	1207	1218
EM2025D	Peak Flow, cfs	247.16	368.55	486.58	606.64	244.16	365.82	486.86	607.09	1198.8	1682.1	2079	2567.5	2531.6
	Time of Peak	1226	1229	1231	1234	1228	1231	1233	1235	1236	1248	1300	1312	1324
EM2050	Peak Flow, cfs	1759.2	2580.6	3333.4	4009.4	1759.2	2580.6	3333.4	4009.4	1198.8	1694.4	4310.5	6192.9	6012.3
	Time of Peak	1201	1202	1204	1205	1201	1202	1204	1205	1236	1205	1209	1214	1224
EM2050D	Peak Flow, cfs	244.16	365.82	486.86	607.09	244.16	365.82	486.86	607.09	1198.8	1733.2	2263	2752.9	2720.5
	Time of Peak	1228	1231	1233	1235	1228	1231	1233	1235	1236	1249	1301	1313	1325
EM2075	Peak Flow, cfs	1759.2	2580.6	3333.4	4009.4	1759.2	2580.6	3333.4	4009.4	2485.7	4806.4	6391.6	7430	7344
	Time of Peak	1201	1202	1204	1205	1201	1202	1204	1205	1206	1212	1216	1222	1230
EM2075D	Peak Flow, cfs	244.16	365.82	486.86	607.09	244.16	365.82	486.86	607.09	1205.5	1777.3	2312.4	2810.1	2788.5
	Time of Peak	1228	1231	1233	1235	1228	1231	1233	1235	1237	1251	1303	1314	1326

Table A-8. Continued

		Junction											
Run	**Result**	**2A**	**2B**	**2C**	**2D**	**2E**	**2F**	**2G**	**2H**	**2I**	**2J**	**2K**	**2L**
ESPre	Peak Flow, cfs	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62
	Time of Peak	1212	1214	1215	1216	1212	1214	1215	1216	1212	1214	1215	1216
ES8025	Peak Flow, cfs	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62
	Time of Peak	1212	1214	1215	1216	1212	1214	1215	1216	1212	1214	1215	1216
ES8025D	Peak Flow, cfs	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62
	Time of Peak	1212	1214	1215	1216	1212	1214	1215	1216	1212	1214	1215	1216
ES8050	Peak Flow, cfs	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62	363.1	540.84	941	1653.9
	Time of Peak	1212	1214	1215	1216	1212	1214	1215	1216	1212	1214	1205	1204
ES8050D	Peak Flow, cfs	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62	363.1	540.84	718.35	897.13
	Time of Peak	1212	1214	1215	1216	1212	1214	1215	1216	1212	1214	1216	1218
ES8075	Peak Flow, cfs	363.1	540.84	714.71	882.62	363.1	915.74	1645.5	2371.2	1555	2317.2	3050.8	3769.1
	Time of Peak	1212	1214	1215	1216	1212	1204	1203	1204	1202	1203	1203	1204
ES8075D	Peak Flow, cfs	363.1	540.84	714.71	882.62	363.1	538.17	716.88	896.39	360.1	539.59	718.21	895.71
	Time of Peak	1212	1214	1215	1216	1212	1215	1217	1219	1219	1221	1222	1223
ES2025	Peak Flow, cfs	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62
	Time of Peak	1212	1214	1215	1216	1212	1214	1215	1216	1212	1214	1215	1216
ES2025D	Peak Flow, cfs	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62
	Time of Peak	1212	1214	1215	1216	1212	1214	1215	1216	1212	1214	1215	1216
ES2050	Peak Flow, cfs	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62	363.1	540.84	1038.5	2000.8
	Time of Peak	1212	1214	1215	1216	1212	1214	1215	1216	1212	1214	1158	1159
ES2050D	Peak Flow, cfs	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62	363.1	540.84	718.16	894.65
	Time of Peak	1212	1214	1215	1216	1212	1214	1215	1216	1212	1214	1215	1217
ES2075	Peak Flow, cfs	363.1	540.84	714.71	882.62	363.1	1041.6	2002.3	2938.4	1989.7	2958.7	3895.2	4799.7
	Time of Peak	1212	1214	1215	1216	1212	1158	1159	1159	1158	1159	1159	1200
ES2075D	Peak Flow, cfs	363.1	540.84	714.71	882.62	363.1	541.15	718.19	893.76	361.4	541.23	720.1	897.9
	Time of Peak	1212	1214	1215	1216	1212	1214	1216	1218	1209	1211	1213	1215

Table A-8. Continued

		Junction												
Run	**Result**	**2M**	**2N**	**2O**	**2P**	**2Q**	**2R**	**2S**	**2T**	**E**	**D**	**C**	**B**	**A**
ESPre	Peak Flow, cfs	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62	1762.9	2512.7	3097.9	3558.5	3545.3
	Time of Peak	1212	1214	1215	1216	1212	1214	1215	1216	1218	1224	1230	1236	1242
ES8025	Peak Flow, cfs	363.1	540.84	714.71	961.08	1555	2317.2	3050.8	3769.1	1762.9	2512.7	3118	4262	4231.2
	Time of Peak	1212	1214	1215	1208	1202	1203	1203	1204	1218	1224	1230	1206	1213
ES8025D	Peak Flow, cfs	363.1	540.84	714.71	892.61	360.1	539.59	718.21	895.71	1762.9	2512.7	3168.4	3994.7	3979.7
	Time of Peak	1212	1214	1215	1217	1219	1221	1222	1223	1218	1224	1230	1236	1242
ES8050	Peak Flow, cfs	1555	2317.2	3050.8	3769.1	1555	2317.2	3050.8	3769.1	1762.9	2561.7	4842	7485.3	7449.5
	Time of Peak	1202	1203	1203	1204	1202	1203	1203	1204	1218	1224	1208	1210	1215
ES8050D	Peak Flow, cfs	360.1	539.59	718.21	895.71	360.1	539.59	718.21	895.71	1762.9	2595.7	3460.6	4278.5	4266.2
	Time of Peak	1219	1221	1222	1223	1219	1221	1222	1223	1218	1224	1231	1237	1243
ES8075	Peak Flow, cfs	1555	2317.2	3050.8	3769.1	1555	2317.2	3050.8	3769.1	2688.2	5632.2	8001.8	9895	9864.8
	Time of Peak	1202	1203	1203	1204	1202	1203	1203	1204	1207	1209	1211	1214	1218
ES8075D	Peak Flow, cfs	360.1	539.59	718.21	895.71	360.1	539.59	718.21	895.71	1763.9	2651	3505.8	4311.7	4303.6
	Time of Peak	1219	1221	1222	1223	1219	1221	1222	1223	1219	1226	1233	1239	1245
ES2025	Peak Flow, cfs	363.1	540.84	714.71	1037.1	1989.7	2958.7	3895.2	4799.7	1762.9	2512.7	3104.2	5303	5250.7
	Time of Peak	1212	1214	1215	1158	1158	1159	1159	1200	1218	1224	1230	1202	1208
ES2025D	Peak Flow, cfs	363.1	540.84	714.71	890.26	361.4	541.23	720.1	897.9	1762.9	2512.7	3160.3	3945.7	3930.3
	Time of Peak	1212	1214	1215	1217	1209	1211	1213	1215	1218	1224	1230	1236	1242
ES2050	Peak Flow, cfs	1989.7	2958.7	3895.2	4799.7	1989.7	2958.7	3895.2	4799.7	1762.9	2492.6	5988.8	9186.1	9119.4
	Time of Peak	1158	1159	1159	1200	1158	1159	1159	1200	1218	1224	1203	1205	1210
ES2050D	Peak Flow, cfs	361.4	541.23	720.1	897.9	361.4	541.23	720.1	897.9	1762.9	2582.6	3407.6	4186.3	4174.1
	Time of Peak	1209	1211	1213	1215	1209	1211	1213	1215	1218	1224	1231	1237	1243
ES2075	Peak Flow, cfs	1989.7	2958.7	3895.2	4799.7	1989.7	2958.7	3895.2	4799.7	3035.9	6834	9672.7	11864	11808
	Time of Peak	1158	1159	1159	1200	1158	1159	1159	1200	1201	1204	1206	1208	1213
ES2075D	Peak Flow, cfs	361.4	541.23	720.1	897.9	361.4	541.23	720.1	897.9	1771.9	2633.7	3455.1	4230.1	4223.7
	Time of Peak	1209	1211	1213	1215	1209	1211	1213	1215	1218	1225	1232	1237	1244

Table A-9. Results from Evaluation of Development Sequence for the Classic Watersheds

		Junction											
Run	**Result**	**2A**	**2B**	**2C**	**2D**	**2E**	**2F**	**2G**	**2H**	**2I**	**2J**	**2K**	**2L**
CMPre	Peak Flow, cfs	357.66	532.4	701.03	357.66	532.4	701.03	357.66	532.4	701.03	357.66	532.4	701.03
	Time of Peak	1230	1234	1236	1230	1234	1236	1230	1234	1236	1230	1234	1236
CM8025U	Peak Flow, cfs	2184.7	3203.7	4128.8	357.66	532.4	701.03	357.66	532.4	701.03	357.66	532.4	701.03
	Time of Peak	1205	1208	1209	1230	1234	1236	1230	1234	1236	1230	1234	1236
CM8025UD	Peak Flow, cfs	355	531.44	706.3	357.66	532.4	701.03	357.66	532.4	701.03	357.66	532.4	701.03
	Time of Peak	1239	1243	1246	1230	1234	1236	1230	1234	1236	1230	1234	1236
CM8050U	Peak Flow, cfs	2184.7	3203.7	4128.8	2184.7	3203.7	4128.8	357.66	532.4	701.03	357.66	532.4	701.03
	Time of Peak	1205	1208	1209	1205	1208	1209	1230	1234	1236	1230	1234	1236
CM8050UD	Peak Flow, cfs	355	531.44	706.3	355	531.44	706.3	357.66	532.4	701.03	357.66	532.4	701.03
	Time of Peak	1239	1243	1246	1239	1243	1246	1230	1234	1236	1230	1234	1236
CM8075U	Peak Flow, cfs	2184.7	3203.7	4128.8	2184.7	3203.7	4128.8	2184.7	3203.7	4128.8	357.66	532.4	701.03
	Time of Peak	1205	1208	1209	1205	1208	1209	1205	1208	1209	1230	1234	1236
CM8075UD	Peak Flow, cfs	355	531.44	706.3	355	531.44	706.3	355	531.44	706.3	357.66	532.4	701.03
	Time of Peak	1239	1243	1246	1239	1243	1246	1239	1243	1246	1230	1234	1236
CM2025U	Peak Flow, cfs	2708	3937.4	4995.7	357.66	532.4	701.03	357.66	532.4	701.03	357.66	532.4	701.03
	Time of Peak	1201	1203	1204	1230	1234	1236	1230	1234	1236	1230	1234	1236
CM2025UD	Peak Flow, cfs	353.28	529.02	703.41	357.66	532.4	701.03	357.66	532.4	701.03	357.66	532.4	701.03
	Time of Peak	1230	1233	1236	1230	1234	1236	1230	1234	1236	1230	1234	1236
CM2050U	Peak Flow, cfs	2708	3937.4	4995.7	2708	3937.4	4995.7	357.66	532.4	701.03	357.66	532.4	701.03
	Time of Peak	1201	1203	1204	1201	1203	1204	1230	1234	1236	1230	1234	1236
CM2050UD	Peak Flow, cfs	353.28	529.02	703.41	353.28	529.02	703.41	357.66	532.4	701.03	357.66	532.4	701.03
	Time of Peak	1230	1233	1236	1230	1233	1236	1230	1234	1236	1230	1234	1236
CM2075U	Peak Flow, cfs	2708	3937.4	4995.7	2708	3937.4	4995.7	2708	3937.4	4995.7	357.66	532.4	701.03
	Time of Peak	1201	1203	1204	1201	1203	1204	1201	1203	1204	1230	1234	1236
CM2075UD	Peak Flow, cfs	353.28	529.02	703.41	353.28	529.02	703.41	353.28	529.02	703.41	357.66	532.4	701.03
	Time of Peak	1230	1233	1236	1230	1233	1236	1230	1233	1236	1230	1234	1236

Table A-9. Continued

		Junction			
Run	**Result**	**D**	**C**	**B**	**A**
CMPre	Peak Flow, cfs	1399.7	2029.3	2561.9	2533.7
	Time of Peak	1241	1249	1256	1306
CM8025U	Peak Flow, cfs	4166.6	4256.7	4730.8	4625.6
	Time of Peak	1212	1222	1231	1240
CM8025UD	Peak Flow, cfs	1388.7	2004.6	2531.8	2507.7
	Time of Peak	1243	1250	1257	1307
CM8050U	Peak Flow, cfs	8201	8036.2	8236	7980.2
	Time of Peak	1212	1220	1227	1235
CM8050UD	Peak Flow, cfs	1412	1987.3	2504.4	2485.7
	Time of Peak	1250	1252	1258	1308
CM8075U	Peak Flow, cfs	8201	11248	11244	10907
	Time of Peak	1212	1217	1224	1231
CM8075UD	Peak Flow, cfs	1412	2100.3	2569.8	2560.1
	Time of Peak	1250	1258	1302	1311
CM2025U	Peak Flow, cfs	4969	4745.6	4876.8	4688.9
	Time of Peak	1207	1215	1224	1233
CM2025UD	Peak Flow, cfs	1403	2045.4	2603.1	2580
	Time of Peak	1241	1248	1255	1304
CM2050U	Peak Flow, cfs	9886.2	9360.2	9106.9	8685.1
	Time of Peak	1207	1214	1221	1228
CM2050UD	Peak Flow, cfs	1406.2	2072	2666.2	2642
	Time of Peak	1241	1245	1252	1302
CM2075U	Peak Flow, cfs	9886.2	13113	12629	12120
	Time of Peak	1207	1212	1218	1225
CM2075UD	Peak Flow, cfs	1406.2	2094.1	2676.9	2660.6
	Time of Peak	1241	1249	1252	1302

Table A-9. Continued

		Junction											
Run	**Result**	**2A**	**2B**	**2C**	**2D**	**2E**	**2F**	**2G**	**2H**	**2I**	**2J**	**2K**	**2L**
CSPre	Peak Flow, cfs	534.75	796.14	1049	534.75	796.14	1049	534.75	796.14	1049	534.75	796.14	1049
	Time of Peak	1214	1216	1217	1214	1216	1217	1214	1216	1217	1214	1216	1217
CS8025U	Peak Flow, cfs	2591.6	3854	5055.6	534.75	796.14	1049	534.75	796.14	1049	534.75	796.14	1049
	Time of Peak	1202	1203	1204	1214	1216	1217	1214	1216	1217	1214	1216	1217
CS8025UD	Peak Flow, cfs	530.24	794.4	1057.2	534.75	796.14	1049	534.75	796.14	1049	534.75	796.14	1049
	Time of Peak	1222	1224	1225	1214	1216	1217	1214	1216	1217	1214	1216	1217
CS8050U	Peak Flow, cfs	2591.6	3854	5055.6	2591.6	3854	5055.6	534.75	796.14	1049	534.75	796.14	1049
	Time of Peak	1202	1203	1204	1202	1203	1204	1214	1216	1217	1214	1216	1217
CS8050UD	Peak Flow, cfs	530.24	794.4	1057.2	530.24	794.4	1057.2	534.75	796.14	1049	534.75	796.14	1049
	Time of Peak	1222	1224	1225	1222	1224	1225	1214	1216	1217	1214	1216	1217
CS8075U	Peak Flow, cfs	2591.6	3854	5055.6	2591.6	3854	5055.6	2591.6	3854	5055.6	534.75	796.14	1049
	Time of Peak	1202	1203	1204	1202	1203	1204	1202	1203	1204	1214	1216	1217
CS8075UD	Peak Flow, cfs	530.24	794.4	1057.2	530.24	794.4	1057.2	530.24	794.4	1057.2	534.75	796.14	1049
	Time of Peak	1222	1224	1225	1222	1224	1225	1222	1224	1225	1214	1216	1217
CS2025U	Peak Flow, cfs	3061	4544.6	5941.9	534.75	796.14	1049	534.75	796.14	1049	534.75	796.14	1049
	Time of Peak	1158	1159	1200	1214	1216	1217	1214	1216	1217	1214	1216	1217
CS2025UD	Peak Flow, cfs	530.34	793.93	1056.7	534.75	796.14	1049	534.75	796.14	1049	534.75	796.14	1049
	Time of Peak	1210	1212	1215	1214	1216	1217	1214	1216	1217	1214	1216	1217
CS2050U	Peak Flow, cfs	3061	4544.6	5941.9	3061	4544.6	5941.9	534.75	796.14	1049	534.75	796.14	1049
	Time of Peak	1158	1159	1200	1158	1159	1200	1214	1216	1217	1214	1216	1217
CS2050UD	Peak Flow, cfs	530.34	793.93	1056.7	530.34	793.93	1056.7	534.75	796.14	1049	534.75	796.14	1049
	Time of Peak	1210	1212	1215	1210	1212	1215	1214	1216	1217	1214	1216	1217
CS2075U	Peak Flow, cfs	3061	4544.6	5941.9	3061	4544.6	5941.9	3061	4544.6	5941.9	534.75	796.14	1049
	Time of Peak	1158	1159	1200	1158	1159	1200	1158	1159	1200	1214	1216	1217
CS2075UD	Peak Flow, cfs	530.34	793.93	1056.7	530.34	793.93	1056.7	530.34	793.93	1056.7	534.75	796.14	1049
	Time of Peak	1210	1212	1215	1210	1212	1215	1210	1212	1215	1214	1216	1217

Table A-9. Continued

		Junction			
Run	**Result**	**D**	**C**	**B**	**A**
CSPre	Peak Flow, cfs	2093.9	3059.4	3903.3	3891
	Time of Peak	1220	1224	1228	1233
CS8025U	Peak Flow, cfs	5344.2	6007	6972.6	6949.2
	Time of Peak	1206	1212	1217	1221
CS8025UD	Peak Flow, cfs	2057	2982.7	3794.5	3784.2
	Time of Peak	1221	1225	1229	1234
CS8050U	Peak Flow, cfs	10066	10582	11404	11331
	Time of Peak	1206	1210	1214	1218
CS8050UD	Peak Flow, cfs	2113.8	2937.3	3705.8	3698.8
	Time of Peak	1227	1227	1230	1235
CS8075U	Peak Flow, cfs	10066	14598	15269	15149
	Time of Peak	1206	1208	1212	1216
CS8075UD	Peak Flow, cfs	2113.8	3157.1	3828.7	3825.3
	Time of Peak	1227	1232	1232	1237
CS2025U	Peak Flow, cfs	6013.3	6281.5	6927.1	6896.4
	Time of Peak	1202	1206	1212	1216
CS2025UD	Peak Flow, cfs	2101	3085.4	3962.9	3951.6
	Time of Peak	1219	1223	1227	1232
CS2050U	Peak Flow, cfs	11824	11991	12416	12327
	Time of Peak	1201	1205	1209	1213
CS2050UD	Peak Flow, cfs	2112.8	3138.2	4067.1	4055.5
	Time of Peak	1217	1221	1225	1230
CS2075U	Peak Flow, cfs	11824	17000	17212	17105
	Time of Peak	1201	1204	1208	1211
CS2075UD	Peak Flow, cfs	2112.8	3155	4075.1	4066.7
	Time of Peak	1217	1222	1225	1230

Table A-10. Results from Evaluation of Development Sequence for the Elongated Watersheds

		Junction											
Run	**Result**	**2A**	**2B**	**2C**	**2D**	**2E**	**2F**	**2G**	**2H**	**2I**	**2J**	**2K**	**2L**
EMPre	Peak Flow, cfs	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54
	Time of Peak	1226	1229	1231	1233	1226	1229	1231	1233	1226	1229	1231	1233
EM8025U	Peak Flow, cfs	1310.8	1937.1	2520.5	3064.5	700.6	762.26	847.09	949.93	247.16	368.55	486.58	600.54
	Time of Peak	1205	1207	1209	1210	1206	1210	1214	1217	1226	1229	1231	1233
EM8025UD	Peak Flow, cfs	245.26	367.31	488.45	608.6	242.58	361.69	477.96	590.45	247.16	368.55	486.58	600.54
	Time of Peak	1235	1238	1241	1243	1229	1230	1232	1234	1226	1229	1231	1233
EM8050U	Peak Flow, cfs	1310.8	1937.1	2520.5	3064.5	1310.8	1937.1	2520.5	3064.5	1310.8	1360.3	1428.2	1513.7
	Time of Peak	1205	1207	1209	1210	1205	1207	1209	1210	1205	1208	1211	1214
EM8050UD	Peak Flow, cfs	245.26	367.31	488.45	608.6	245.26	367.31	488.45	608.6	245.26	357.06	470.31	580.93
	Time of Peak	1235	1238	1241	1243	1235	1238	1241	1243	1235	1232	1233	1234
EM8075U	Peak Flow, cfs	1310.8	1937.1	2520.5	3064.5	1310.8	1937.1	2520.5	3064.5	1310.8	1937.1	2520.5	3064.5
	Time of Peak	1205	1207	1209	1210	1205	1207	1209	1210	1205	1207	1209	1210
EM8075UD	Peak Flow, cfs	245.26	367.31	488.45	608.6	245.26	367.31	488.45	608.6	245.26	367.31	488.45	608.6
	Time of Peak	1235	1238	1241	1243	1235	1238	1241	1243	1235	1238	1241	1243
EM2025U	Peak Flow, cfs	1759.2	2580.6	3333.4	4009.4	901.51	926.13	968.72	1033.2	247.16	368.55	486.58	600.54
	Time of Peak	1201	1202	1204	1205	1201	1204	1207	1211	1226	1229	1231	1233
EM2025UD	Peak Flow, cfs	244.16	365.82	486.86	607.09	245.53	367.08	485.96	601.47	247.16	368.55	486.58	600.54
	Time of Peak	1228	1231	1233	1235	1226	1228	1230	1232	1226	1229	1231	1233
EM2050U	Peak Flow, cfs	1759.2	2580.6	3333.4	4009.4	1759.2	2580.6	3333.4	4009.4	1759.2	1769.9	1796.9	1835.9
	Time of Peak	1201	1202	1204	1205	1201	1202	1204	1205	1201	1203	1206	1209
EM2050UD	Peak Flow, cfs	244.16	365.82	486.86	607.09	244.16	365.82	486.86	607.09	244.16	365.92	486.08	603.69
	Time of Peak	1228	1231	1233	1235	1228	1231	1233	1235	1228	1228	1230	1231
EM2075U	Peak Flow, cfs	1759.2	2580.6	3333.4	4009.4	1759.2	2580.6	3333.4	4009.4	1759.2	2580.6	3333.4	4009.4
	Time of Peak	1201	1202	1204	1205	1201	1202	1204	1205	1201	1202	1204	1205
EM2075UD	Peak Flow, cfs	244.16	365.82	486.86	607.09	244.16	365.82	486.86	607.09	244.16	365.82	486.86	607.09
	Time of Peak	1228	1231	1233	1235	1228	1231	1233	1235	1228	1231	1233	1235

Table A-10. Continued

		Junction												
Run	**Result**	**2M**	**2N**	**2O**	**2P**	**2Q**	**2R**	**2S**	**2T**	**E**	**D**	**C**	**B**	**A**
EMPre	Peak Flow, cfs	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54	1198.8	1682.1	2036.3	2298.5	2261.1
	Time of Peak	1226	1229	1231	1233	1226	1229	1231	1233	1236	1248	1300	1312	1324
EM8025U	Peak Flow, cfs	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54	3753.5	4019.3	4465.7	4837.7	4673.6
	Time of Peak	1226	1229	1231	1233	1226	1229	1231	1233	1214	1226	1237	1247	1258
EM8025UD	Peak Flow, cfs	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54	1181.5	1643.4	1989.3	2248.8	2218.5
	Time of Peak	1226	1229	1231	1233	1226	1229	1231	1233	1240	1250	1301	1313	1326
EM8050U	Peak Flow, cfs	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54	6093.1	7147.6	7370.7	7662.2	7364.9
	Time of Peak	1226	1229	1231	1233	1226	1229	1231	1233	1212	1221	1231	1240	1250
EM8050UD	Peak Flow, cfs	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54	1216.7	1679.3	2006.9	2261.8	2241.2
	Time of Peak	1226	1229	1231	1233	1226	1229	1231	1233	1246	1255	1305	1316	1329
EM8075U	Peak Flow, cfs	1310.8	1937.1	1991.2	2059.7	247.16	368.55	486.58	600.54	6093.1	7955.9	8836.6	9145.7	8883.9
	Time of Peak	1205	1207	1210	1212	1226	1229	1231	1233	1212	1220	1228	1236	1245
EM8075UD	Peak Flow, cfs	245.26	367.31	471.22	577.57	247.16	368.55	486.58	600.54	1216.7	1795.1	2234.4	2482.2	2465.7
	Time of Peak	1235	1238	1235	1236	1226	1229	1231	1233	1246	1257	1308	1318	1331
EM2025U	Peak Flow, cfs	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54	4670	4569.5	4885.4	5249.6	5024.8
	Time of Peak	1226	1229	1231	1233	1226	1229	1231	1233	1208	1219	1230	1241	1252
EM2025UD	Peak Flow, cfs	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54	1206.5	1712.9	2092.1	2371.5	2335.9
	Time of Peak	1226	1229	1231	1233	1226	1229	1231	1233	1236	1246	1258	1309	1322
EM2050U	Peak Flow, cfs	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54	7945.2	8941.5	8767.6	8865.2	8404.3
	Time of Peak	1226	1229	1231	1233	1226	1229	1231	1233	1207	1215	1224	1234	1243
EM2050UD	Peak Flow, cfs	247.16	368.55	486.58	600.54	247.16	368.55	486.58	600.54	1213.7	1734.8	2130.6	2423.9	2393.9
	Time of Peak	1226	1229	1231	1233	1226	1229	1231	1233	1238	1246	1256	1308	1321
EM2075U	Peak Flow, cfs	1759.2	2580.6	2591.4	2619.2	247.16	368.55	486.58	600.54	7945.2	9884.5	10545	10699	10338
	Time of Peak	1201	1202	1205	1207	1226	1229	1231	1233	1207	1214	1221	1230	1238
EM2075UD	Peak Flow, cfs	244.16	365.82	484.67	602.7	247.16	368.55	486.58	600.54	1213.7	1797.1	2263	2551.8	2533.2
	Time of Peak	1228	1231	1230	1231	1226	1229	1231	1233	1238	1250	1300	1310	1323

Table A-10. Continued

		Junction											
Run	**Result**	**2A**	**2B**	**2C**	**2D**	**2E**	**2F**	**2G**	**2H**	**2I**	**2J**	**2K**	**2L**
ESPre	Peak Flow, cfs	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62
	Time of Peak	1212	1214	1215	1216	1212	1214	1215	1216	1212	1214	1215	1216
ES8025U	Peak Flow, cfs	1555	2317.2	3050.8	3769.1	876.04	1008.7	1164.7	1338.2	363.1	540.84	714.71	882.62
	Time of Peak	1202	1203	1203	1204	1203	1205	1207	1209	1212	1214	1215	1216
ES8025UD	Peak Flow, cfs	360.1	539.59	718.21	895.71	352.76	526.32	696.1	860.52	363.1	540.84	714.71	882.62
	Time of Peak	1219	1221	1222	1223	1214	1214	1215	1216	1212	1214	1215	1216
ES8050U	Peak Flow, cfs	1555	2317.2	3050.8	3769.1	1555	2317.2	3050.8	3769.1	1555	1666.2	1802.4	1960.4
	Time of Peak	1202	1203	1203	1204	1202	1203	1203	1204	1202	1204	1206	1207
ES8050UD	Peak Flow, cfs	360.1	539.59	718.21	895.71	360.1	539.59	718.21	895.71	360.1	517.51	680.78	841.38
	Time of Peak	1219	1221	1222	1223	1219	1221	1222	1223	1219	1216	1216	1217
ES8075U	Peak Flow, cfs	1555	2317.2	3050.8	3769.1	1555	2317.2	3050.8	3769.1	1555	2317.2	3050.8	3769.1
	Time of Peak	1202	1203	1203	1204	1202	1203	1203	1204	1202	1203	1203	1204
ES8075UD	Peak Flow, cfs	360.1	539.59	718.21	895.71	360.1	539.59	718.21	895.71	360.1	539.59	718.21	895.71
	Time of Peak	1219	1221	1222	1223	1219	1221	1222	1223	1219	1221	1222	1223
ES2025U	Peak Flow, cfs	1989.7	2958.7	3895.2	4799.7	1042.9	1111.5	1209.2	1339.2	363.1	540.84	714.71	882.62
	Time of Peak	1158	1159	1159	1200	1158	1200	1203	1205	1212	1214	1215	1216
ES2025UD	Peak Flow, cfs	361.4	541.23	720.1	897.9	361.25	541.1	716.92	887.95	363.1	540.84	714.71	882.62
	Time of Peak	1209	1211	1213	1215	1212	1213	1214	1215	1212	1214	1215	1216
ES2050U	Peak Flow, cfs	1989.7	2958.7	3895.2	4799.7	1989.7	2958.7	3895.2	4799.7	1989.7	2045.3	2122.8	2219.2
	Time of Peak	1158	1159	1159	1200	1158	1159	1159	1200	1158	1159	1201	1203
ES2050UD	Peak Flow, cfs	361.4	541.23	720.1	897.9	361.4	541.23	720.1	897.9	361.4	542.89	721.92	895.95
	Time of Peak	1209	1211	1213	1215	1209	1211	1213	1215	1209	1212	1213	1214
ES2075U	Peak Flow, cfs	1989.7	2958.7	3895.2	4799.7	1989.7	2958.7	3895.2	4799.7	1989.7	2958.7	3895.2	4799.7
	Time of Peak	1158	1159	1159	1200	1158	1159	1159	1200	1158	1159	1159	1200
ES2075UD	Peak Flow, cfs	361.4	541.23	720.1	897.9	361.4	541.23	720.1	897.9	361.4	541.23	720.1	897.9
	Time of Peak	1209	1211	1213	1215	1209	1211	1213	1215	1209	1211	1213	1215

Table A-10. Continued

		Junction												
Run	**Result**	**2M**	**2N**	**2O**	**2P**	**2Q**	**2R**	**2S**	**2T**	**E**	**D**	**C**	**B**	**A**
ESPre	Peak Flow, cfs	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62	1762.9	2512.7	3097.9	3558.5	3545.3
	Time of Peak	1212	1214	1215	1216	1212	1214	1215	1216	1218	1224	1230	1236	1242
ES8025U	Peak Flow, cfs	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62	4856.4	5593.6	6430.1	7161.3	7109
	Time of Peak	1212	1214	1215	1216	1212	1214	1215	1216	1207	1213	1219	1224	1229
ES8025UD	Peak Flow, cfs	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62	1718.7	2409.6	2958	3393.2	3381.5
	Time of Peak	1212	1214	1215	1216	1212	1214	1215	1216	1220	1225	1231	1237	1244
ES8050U	Peak Flow, cfs	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62	7508.3	9356.4	10131	10923	10851
	Time of Peak	1212	1214	1215	1216	1212	1214	1215	1216	1205	1210	1215	1220	1225
ES8050UD	Peak Flow, cfs	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62	1791	2466	2957.8	3364.4	3358.4
	Time of Peak	1212	1214	1215	1216	1212	1214	1215	1216	1225	1229	1234	1239	1246
ES8075U	Peak Flow, cfs	1555	2317.2	2436.8	2579.8	363.1	540.84	714.71	882.62	7508.3	10602	12444	13248	13143
	Time of Peak	1202	1203	1204	1206	1212	1214	1215	1216	1205	1209	1213	1218	1223
ES8075UD	Peak Flow, cfs	360.1	539.59	683.93	835.49	363.1	540.84	714.71	882.62	1791	2659.3	3326.4	3710.1	3703.9
	Time of Peak	1219	1221	1218	1218	1212	1214	1215	1216	1225	1231	1236	1241	1247
ES2025U	Peak Flow, cfs	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62	5846	6231.5	7000.1	7802.3	7758.1
	Time of Peak	1212	1214	1215	1216	1212	1214	1215	1216	1202	1208	1214	1219	1225
ES2025UD	Peak Flow, cfs	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62	1784.8	2564.9	3188.7	3674	3657.8
	Time of Peak	1212	1214	1215	1216	1212	1214	1215	1216	1217	1223	1228	1234	1241
ES2050U	Peak Flow, cfs	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62	9565.7	11599	12127	12869	12742
	Time of Peak	1212	1214	1215	1216	1212	1214	1215	1216	1201	1206	1211	1216	1220
ES2050UD	Peak Flow, cfs	363.1	540.84	714.71	882.62	363.1	540.84	714.71	882.62	1795	2600.6	3243.4	3747.9	3733.7
	Time of Peak	1212	1214	1215	1216	1212	1214	1215	1216	1217	1222	1227	1233	1240
ES2075U	Peak Flow, cfs	1989.7	2958.7	3024.2	3097.3	363.1	540.84	714.71	882.62	9565.7	13311	15293	15932	15784
	Time of Peak	1158	1159	1200	1202	1212	1214	1215	1216	1201	1205	1209	1213	1218
ES2075UD	Peak Flow, cfs	361.4	541.23	721.65	898.46	363.1	540.84	714.71	882.62	1795	2668.5	3389.3	3872.2	3864.6
	Time of Peak	1209	1211	1213	1214	1212	1214	1215	1216	1217	1224	1229	1234	1241

VITA

Karen Marie (Thomason) Goff was born on October 26, 1973 in Davenport, Iowa. She grew up primarily in Colorado, attending elementary school and middle school there. She graduated from Heritage High School in Maryville, Tennessee in 1992.

Karen earned a Bachelor of Science degree in Civil Engineering from the University of Tennessee, Knoxville in May, 1997. While working toward her Bachelor of Science degree, she worked for the Federal Highway Administration in Sterling, Virginia through the cooperative engineering program. Karen entered the graduate program at the University of Tennessee, Knoxville in the summer of 1997. She will receive her Master of Science degree in Environmental Engineering in May, 2003.

Karen and her husband, Gary, moved to Bismarck, North Dakota in 1998. For the past four years she has been employed as a water resource engineer for the North Dakota State Water Commission, Water Appropriations Division.