Evaluation of the Additive Effects of Stormwater Detention Basins at the Watershed Scale

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Abstract
Evaluation of the Additive Effects of Stormwater Detention Basins at the Watershed Scale
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Valley Creek watershed in Chester County, Pennsylvania is used as a case study in this thesis to evaluate the impacts of current design standards for stormwater management on a watershed-wide scale. Southeastern Pennsylvania has experienced dramatic population growth in the past 30 years as the urban sprawl of Philadelphia extends westward. As a result of this urbanization, Valley Creek watershed is now covered by approximately 17% impervious surfaces. Implementation of stormwater management plans during this period have left the 24 square-mile watershed dotted with more than 100 stormwater detention basins. The design objective of the majority of these stormwater facilities has been to limit on-site peak outflow rate to near predevelopment levels for a large design storm of a low return frequency. Problems with this design objective include the following. (1) Historically, designs have not provided for any attenuation of total storm runoff volume, whereas in nearly all cases the on-site volume has been greatly increased compared to predevelopment conditions. (2) In urban watersheds, flooding and in-stream erosion often result from smaller, more frequent storm events. The large design storms with low return-frequencies account for only a small percentage of the total annual precipitation and therefore the design of these detention basins has not focused on the bulk of the yearly precipitation. (3) Design of these detention basins has solely been focused on reducing on-site flow rates and has not taken into consideration the effect of other flow conditions in the watershed including neighboring detention basins on the total stream flow. The purpose of this work is to quantify the composite effect of multiple
detention basins operating in an uncoordinated fashion on the total stream flow in Valley Creek watershed. In order to assess the additive effect of stormwater detention facilities, all detention basins in Valley Creek watershed were surveyed and incorporated into a watershed-scale rainfall-runoff model. The model was created using HEC-HMS in conjunction with the HEC-GeoHMS extension in ArcView 3.3. The watershed was modeled with measured storm hyetographs from a network of recording rain gages as well as larger hypothetical storms. Model results show that the detention basins have essentially no attenuating effect on peak stream flow. The model was then modified to further explain the results and to answer several “what if” questions on watershed-wide stormwater management. This analysis points to the need for a fundamental re-evaluation of the premise of stormwater detention basin design. The results show that the benefits of a volume-based approach would far exceed those realized from a release-rate, detention-basin-based approach.
Chapter 1. Introduction

1.1 Purpose and Scope

It is well known that increased impervious area coverage exacerbates stormwater flooding by increasing runoff volumes and peak runoff rates and flow volumes. In suburban Philadelphia, this phenomenon has been identified as one of the leading causes of stream degradation (Chester County Water Resources Authority, 2003). In the spring of 2000, Drexel University undertook a large study to quantify anthropogenic influences on the ecology and hydrology of a model watershed in suburban Philadelphia. Valley Creek was chosen as the model watershed. Although evaluation of stormwater management was not part of the original scope of work of the Drexel study, it quickly became apparent that this was one of the most significant environmental problems in Valley Creek watershed. Evaluation of the additive effects of the system of detention basins on watershed hydrology was identified as an area of research interest and became the subject of this thesis.

Valley Creek watershed encompasses 24 square miles of Chester County, Pennsylvania, as shown in Figure 1.
Figure 1. Location map of Southeastern Pennsylvania showing Valley Creek watershed (green) and urban areas (beige).

The creek has both historic and ecological significance. Valley Forge National Historical Park is located at the confluence of Valley Creek and the Schuylkill River. Valley Creek is currently listed as an Exceptional Value stream in the PA Code 25 Environmental Protection, Chapter 93 Water Quality Standards. The watershed supports a reproducing population of Brown Trout and is one of only two Class A trout fisheries as designated by the Pennsylvania Fish and Boat Commission in the five-county region of Southeastern Pennsylvania (Personal Communication, Alan Everett PA DEP, July 2003). It has been noted that the creek may have recently supported a reproducing population of native Brook Trout (Kemp, 1994).
Chester County is experiencing rapid population growth as the suburban sprawl of Philadelphia extends westward. The watershed is primarily located in East Whiteland and Tredyffrin Townships as shown in Figure 2.

![Figure 2. Townships boundaries in Valley Creek watershed.](image)

These two townships have experienced a 400% population increase in the past fifty years (U.S. Census Bureau, 1715 - 2000). Tredyffrin Township saw gradual growth in the early 1900s when the railroad connected Philadelphia with suburban areas, and then more significant growth in the 1950s owing to post-World War II construction and extension of the railroad to East Whiteland Township. Figure 3 shows the population trend of the two major municipalities in the Valley Creek watershed from 1715 to 2000. The construction of Route 202, which runs an eight-mile stretch directly through the middle of the
watershed, has also brought much development. The urbanization process has gone uncontrolled with little attention given to ecological planning and design. The years of development have left the watershed covered by approximately 17% impervious surfaces as of March 2000.

![Graph showing population trends for Tredyffrin and East Whiteland Townships.](image)

Figure 3. Population trends for Tredyffrin and East Whiteland Townships.

In observing stormwater runoff from parking lots, lawns, and forested areas into Valley Creek, the Drexel University watershed research team noted that overland flow was typically clear and showed very little turbidity. However, as Valley Creek swells from storm runoff, it quickly becomes a reddish-brown color, and stream banks are eroded from the high peak flow rates. Empirical evidence points to high stream velocities generated by stormwater runoff channeled to the stream and consequent in-stream bank
erosion as the principal cause of the mud-brown color of Valley Creek during storm flows. Excessive in-stream sediment deposition can cause ecological damage to the stream system; therefore concern for in-stream channel erosion as a possible dominant source of sediment to Valley Creek is a major motivation for documenting the controls on stormwater flow. Investigation of the stormwater controls in this watershed revealed a system of 111 detention basins. A major question to be addressed by this thesis is to what degree the engineered stormwater controls affect watershed-wide in-stream flow rates.

1.2 Background

In the late 1970s, the Commonwealth of Pennsylvania began to notice the threat posed to the public from increased runoff from developed areas. The Commonwealth also documented the detrimental impact this runoff was having on the aquatic habitat. In 1978 the Commonwealth of Pennsylvania’s Department of Environmental Resources passed Act 167, better known as the “Storm Water Management Act” (PADER, 1978). To the best knowledge of the author this was the first Pennsylvania legislation that addressed stormwater management. The Act’s statement of legislative findings state:

(1) Inadequate management of accelerated runoff of storm water resulting from development throughout a watershed increases flood flows and velocities, contributes to erosion and sedimentation, overtaxes the carrying capacity of streams and storm sewers, greatly increases the cost of public facilities to carry and control storm water, undermines flood plain management and flood control efforts in downstream communities, reduces ground-water recharge, and threatens public health and safety.
(2) A comprehensive program of storm water management, including reasonable regulation of development and activities causing accelerated runoff, is fundamental to the public health, safety and welfare and the protection of the people of the Commonwealth, their resources and the environment.

In Section 3 the Purpose and Policy of the Act is outlined as follows:

(1) Encourage planning and management of storm water runoff in each watershed which is consistent with sound water and land use practices.

(2) Authorize a comprehensive program of storm water management designated to preserve and restore the flood carrying capacity of Commonwealth streams; to preserve to the maximum extent practical the natural storm water runoff regimes and natural course, current and cross-section of water of the Commonwealth; and to protect and conserve ground waters and ground-water recharge areas.

(3) Encourage local administration and management of storm water consistent with the Commonwealth’s duty as trustee of natural resources and the people’s constitutional right to the preservation of natural, economic, scenic, aesthetic, recreational and historic values of the environment.

The Storm Water Management Act defined the role of the Pennsylvania Department of Environmental Resources, the counties, and the municipalities in the creation and implementation of stormwater control plans. The act stated that the counties “in consultation with the municipalities located within each watershed” will “prepare and adopt” a stormwater management program for each watershed in that county, while the Department of Environmental Resources would regulate Act 167 stormwater management plans. This “watershed storm water management plan” was to be an extensive survey of all factors that affect stormwater runoff within the watershed. This included “a survey of existing runoff characteristics [of the watershed] in small as well as large storms...” The plan would also include identification of highly developed areas and
areas where there is significant proposed development. The Act states that the plan shall include “a designation of those areas to be served by storm water collection and control facilities within a ten-year period…”

Act 167 of 1978 outlines the Commonwealth’s new strategy for stormwater management. It is clear that the policy’s goal is to encourage a land development strategy that can continue to maintain the natural hydrologic regime. However, the only specifics or design criteria the Storm Water Management Act provides are found in Section 13, titled “Duty of persons engaged in the development of land”:

1. to assure that the maximum rate of storm water runoff is no greater after development than prior to development activities; or

2. to manage the quantity, velocity and direction of resulting stormwater runoff in a manner which otherwise adequately protects health and property from possible injury.

The first clause of Section 13 is the foundation on which the release-rate method of stormwater management in Pennsylvania is based. The detention basin quickly became the “premier” method for peak flow rate attenuation (Traver and Chadderton, 1983). The principal of the detention basin release rate method is simple. The designer has a target outflow rate. Typically this target outflow rate is determined by mathematical modeling of the predevelopment site conditions using the specified theoretical design storm. The volume of the basin and the outlet structure configuration are designed such that when the inflow rate exceeds the target outflow rate storage will develop in the basin. In essence the runoff is temporarily detained; hence the name detention basin. This design method is relatively simple and has become the standard method for stormwater management.
Figure 4 illustrates the effect of an existing detention basin design for a six-acre site located in Valley Creek watershed for a six-hour duration, 2-year-return frequency storm event. The event was simulated using HEC-HMS (Feldman, 2000).

![Graph](image)

**Figure 4.** Effect of detention on a six-acre site in Valley Creek watershed.

In this example the detention basin limited the post development peak flow rate to 80% of the predevelopment level. Therefore this detention basin meets and exceeds the requirement of Section 13 of the Storm Water Management Act. Whether or not this detention basin meets the purpose of the Storm Water Management Act set forth in Section 3 is questionable, because even though the peak flow rate has been reduced, the total flow volume has been increased by nearly an order of magnitude as indicated by the area under the hydrograph.
There are three major problems with the release rate detention basin approach to stormwater management:

1. The release rate method of stormwater management does not address the increase in total runoff volume. This increase is usually far more significant than the increase in peak flow rate. The increase in runoff volume results from impervious surfaces that prevent precipitation from infiltrating into the soil, thereby altering the predevelopment hydrologic cycle.

2. Generally only large, less frequent storm events are used in the design of detention basins. Storms of return frequencies between two and one hundred years are commonly used in design. However in urban watersheds characterized by large percentages of impervious surfaces, flooding and in-stream erosion is caused by smaller, far more frequent events. It is these small storm events that make up the majority of the annual precipitation volume.

3. Historically, the design of detention basins has solely been focused on reducing on-site flow rates and has not taken into consideration other watershed stormwater flow characteristics, including the effect of neighboring detention basins on total stream flow.

1.3 Summary

Valley Creek watershed is a model watershed undergoing rapid urbanization. The watershed is covered by approximately 17% impervious surfaces and has over 100 simultaneously operating detention basins. The purpose of this work is to quantify the composite effect of multiple detention basins operating in an uncoordinated fashion on the total stream flow in the Valley Creek watershed using a mathematical modeling approach. While very little research has been published on this topic, it has previously been recognized that the efficiency of detention basins at the watershed scale should decrease as basin density increases (Traver and Chadderton, 1983) and that a “regional” approach should be used to properly assess the effectiveness of stormwater detention basins (McCuen, 1979). In Chapter 2 the methods used in this study are outlined.
Chapter 3 provides modeling results, these results are furthered explored and discussed in Chapter 4. Finally, Chapter 5 summarizes the findings and their implications.
Chapter 2. Methods

The purpose of this chapter is to outline the specific methods used to evaluate the effect of multiple detention basins on stream flow during storm events in Valley Creek watershed. This process included field surveys, use of Geographic Information Systems (GIS), and hydrologic modeling. Each step of this process is described in the following sections.

2.1 Detention Basin Survey

As a first step in detention-basin modeling, information on basin locations, dimensions, and specifications of outlet structures was needed. Currently in Chester County there is no single entity that oversees stormwater management facilities on a watershed basis. Each detention basin is owned and required to be maintained by individual landowners. The site plans for each development are kept on record at the corresponding municipal building. These site plans include detailed information of the hydraulic characteristics of each proposed detention basin. However, the site plans are often poorly maintained, stored, and filed, some are over 25 years old, therefore finding all the plans with the required information can be a difficult if not impossible task. In addition, even if all site plans could be located, often facilities are not built as designed, and more frequently they are no longer functioning as they were originally designed and built. Acquiring the necessary hydraulic information of all the detention basins in the watershed from plans kept on record at township municipal buildings was determined to be an option not worth pursuing for the purpose of this study.
Prior to this study, the detention basins in Valley Creek watershed had never been fully inventoried. A GIS data set of the detention basin locations within the Valley Creek watershed was published in 2001 (Cahill Associates, 2001). These locations were determined from aerial photographs of the watershed. However, this dataset had not been fully ground-truthed, nor did it contain any hydraulic information on the detention basins.

In order to acquire the necessary physical and hydraulic characteristics of each detention basin, an intensive, watershed-wide survey was undertaken in the summer of 2001 by car and on foot. Both aerial photographs and the Cahill Associates data set were used as the basis of survey. By inspection, each potential basin in the Cahill data set was either verified or disqualified for its pertinence to this study. A number of detention basins were also discovered that were not depicted in the Cahill data set.

The survey involved driving every mile of roadway in the watershed, investigating each potential detention basin location, and searching for new locations. Once a stormwater management facility was found, it was first evaluated for its pertinence to this study. This study is focused solely on stormwater detention basins. Detention basins are designed to temporarily store runoff and release it at a rate controlled by the design and sizing of its outlet structure. Stormwater retention basins are those that collect runoff but have no designated outlet. Detention basins are generally dry while retention basins generally hold water during dry periods. For this reason retention basins are often called “wet ponds”. Retention basins were not included in this study. In some cases detention basins were found with a constant water level where the lowest orifice of the outlet
structure was at the water surface elevation (see Figure 5). These basins were included in the model with the assumption that the water surface at the beginning of each modeled storm was at the same elevation as it was when the basin was surveyed. However, the vast majority of the detention basins surveyed were standard dry detention basins.

![Figure 5. Detention basins with constant water surface elevation.](image)

Five detention basins were discovered where sites were still under construction. For some of these basins it was not possible to determine the final outflow structure configuration. Some of the detention basins still in the construction phase had elaborate sediment control devices in place as shown in Figure 6. One of these basins had a temporary baffle system. The baffles were built with plywood and stood about six feet tall. There were four baffles in total. Both of the detention basins pictured in Figure 6 had a floating outflow pipe. The idea is that as the sediment-laden runoff accumulates in the basin, the sediment will begin to settle out and the water near the surface will contain less suspended solids than the water near the bottom. The float will therefore only drain
the water near the surface. This also greatly restricts the outflow rate and allows the runoff an extended period of time to settle out more suspended solids.

In total, 111 detention basins were surveyed, 82 of which ultimately were included in the model. The locations of all the detention basins found in the survey and the percentage of those that were included in the model are illustrated in Figure 7. In this figure the detention basin locations are overlaid with the subwatersheds within Valley Creek watershed. As shown in Figure 7, two detention basins were unknowingly surveyed that were in fact outside of the watershed of interest. The model was created only for the portion of the Valley Creek watershed that is upstream of the USGS stream gage. The majority of the area below the stream gage is Valley Forge National Historical Park, therefore this area is not heavily developed and few detention basins exist below this point. The basin outside the watershed boundary to the southeast is located on a relatively flat ridge where it is difficult to judge the precise watershed boundary.
The detention basins of Valley Creek watershed display a wide range of designs and vintages. Some of these basins are most likely over 25 years old while others are still being built. However, through all the variations there are two styles that are most commonly found. The oldest variety is the metal standpipe. The standpipes range from one to over ten feet in diameter. Many of the standpipes have a short segment of a slightly larger-diameter pipe mounted on top of the main pipe. This larger diameter pipe has vertical plates inside that serve as an anti-vortex device during extreme storm events. The standpipes occasionally have an orifice cut out at the ground surface elevation. Most of the standpipes have small-diameter, 1/2-to-one-inch holes cut out along the length of the pipe. The pipes are usually coated with tar to protect them from rusting. Figure 8 shows two typical standpipe outlet structures.
The most commonly found outlet structure design is the more modern concrete box outlet structure. These box-type outlet structures are usually a standard 2 feet by 4 feet tower with a metal grate on top. The grate serves as an anti-vortex device and prevents large debris from entering the outflow pipe. The concrete box-type structures always have what is often referred to as a “low-flow” orifice located at the base of the structure. Many of the towers also have a secondary orifice or weir higher up the structure which only functions during larger storm events. Finally, the grate on top of the structure is usually set about a foot below the emergency spillway. The emergency spillway is simply a section of the berm that is about a foot lower than the rest of the berm and is protected with erosion control materials. The spillway is built to protect the berm from an overtopping failure during an extreme rainfall event. Figure 9 shows two typical concrete box-type outlet structures.
2.1.1 Maintenance

Stormwater management facilities are not owned or maintained by any one entity. It is not the duty of the municipalities to keep detention basins in good working order. This responsibility is left to the individual land owners. Section 401 of the Tredyffrin Township Erosion, Sedimentation, and Stormwater Control Ordinance (Tredyffrin Township, 2002) states the following:

The owner of stormwater management facilities shall be responsible for their proper maintenance during and after development. A Maintenance Plan shall be prepared for review and approval by the Township Engineer and shall be executed and signed by the Township Engineer and applicant. Where appropriate, as described below, this Maintenance Plan also must be signed by the Homeowners Association. Where appropriate, maintenance responsibilities must be included as deed restrictions on individual lots. During all subsequent real estate transactions, maintenance responsibilities shall be pointed out to new owners. All deeds shall incorporate these specified maintenance responsibilities, making explicit owners responsibilities for stormwater management measures and for the common property.
The ordinance then defines the course of action that will be followed if a given stormwater management facility is not properly maintained.

Failure of any person, individual lot owner or private entity to properly maintain any stormwater management facility shall be construed to be a violation of this Ordinance and is declared to be a public nuisance…If the Township determines at any time that stipulated permanent stormwater management facilities have been eliminated, altered, or improperly maintained, the owner shall be advised of corrective measures required within a period of time set by the Township Engineer. If such measures are not taken by the owner, the Township may cause the work to be done and lien all cost against the property.

The East Whiteland Township Ordinance, “Stormwater Drainage; Erosion and Sedimentation Control” (East Whiteland Township, 2000), does not present any specific guidelines for the maintenance of stormwater management facilities. The ordinance does state that the stormwater management plan shall include “Notation of ownership and responsibility for maintenance of stormwater management structures.” The ordinance also states that the developer shall provide easements for maintenance. The word “maintenance” is only mentioned twice in the entire document.

Although the Tredyffrin Township ordinance gives a plan of action for poorly maintained stormwater management facilities, neither ordinance sets any guidelines for the inspection of stormwater management facilities within the townships. Both plans do establish ownership and maintenance responsibilities. However, the ordinances do not give the owner incentive or establish significant penalties for improper maintenance of stormwater facilities. Figure 10 shows the distribution of the detention basins between the two main townships within Valley Creek watershed.
Based on the above discussion, it is not surprising that many of the detention basins in Valley Creek watershed are poorly maintained and in some cases completely neglected. Of the 111 detention basins surveyed, 26 had significant in-basin erosion problems. This did not include those basins characterized by outfalls that have caused significant erosion. Many of the basins had missing or dislodged grates and other appurtenances. Five of the detention basins had been neglected to the point where they no longer served their purpose. For this reason these basins were not included in the model. Some of the failures included rusted-out holes in standpipes, undermined outlet structures, and complete berm failure. Figure 11 illustrates two of the most blatant failures found in the survey.
The metal standpipe in Figure 11 is completely rusted out around what was once its low-flow orifice. This enlarged opening has lead to the formation of scoured-out flow paths from both inlets to the opening. At the time of the survey, these flow paths were 1.5 feet deep. The eroded soil has all been washed downstream to receiving waters.

Interestingly, the rest of the property was meticulously maintained, with recently cut grass and trimmed bushes as seen in the photograph. The outflow capacity of the outlet structure likely exceeds the capacity of the basin’s two inlets combined. Therefore, this detention basin will not develop any significant storage even during the most intense rainfall events.

The right photograph in Figure 11 was the most extreme failure discovered. The majority of the basin was recently cut grass. However the owner had let one small area of the basin become overgrown. The area around the outlet structure was concealed with
undergrowth, thick vines, and small trees. It was apparent that this area had been
overgrown for about five years. Upon further investigation, the outlet structure was
found with the outlet pipe and seepage plate completely exposed. The berm had
completely washed out. The specific failure mechanism was not clear. The failure could
have been an overtopping failure, or it may have been a piping failure due to an
improperly installed seepage plate. Whatever the mechanism for failure was, it had been
in this condition for a number of years. The interesting aspect about this failure was that
it appeared the owner was aware of the failure and had made a conscious decision to not
repair it, but rather to allow the area around the outlet structure to become overgrown and
hide the problem. The residents of the complex, who may also be owners, had even
disposed of many years of Christmas trees in the void where the berm once stood.

The two detention basin failures shown in Figure 11 not only demonstrate a lack of
maintenance, but also demonstrate an ignorance of the principles of stormwater
management. In both cases the property has been well cared for, yet at both sites there
were clear stormwater management maintenance concerns. The owner, who is by law
responsible for the upkeep of the detention basins, had made a conscious decision not to
maintain the basins.
2.1.2 Dataset

Before the survey was undertaken, a checklist of desired design and other characteristics to be collected was created. The list helped ensure that none of the necessary measurements or observations were overlooked during each survey. The survey form also allotted space for general comments and observations. The back of the checklist was used to sketch the basin’s shape and to detail any complicated measurements or features unique to that basin. An example detention basin checklist is provided in Appendix A.

A hand-held Garmin GPS 12Map Global Positioning System (GPS) was used to mark the location of each outlet structure. These coordinates were downloaded to a computer and an ArcView format shapefile was created from them. The GIS work conducted is explained in more detail in Section 2.4. Each detention basin was photographed with a digital camera. Between two and ten photographs were taken for each basin. This photographic record, containing over 700 digital photographs, was a key component in the preparation of the hydraulic model. After the basin was photographed and its coordinates logged onto the GPS, a survey of the hydraulic characteristics was begun. First the outlet structure was surveyed, including measurement of each orifice’s diameter and elevation. The basins’ slopes were measured using a lock level and surveyor’s tape. A transit and Philadelphia Rod were used where the lock level and tape were not adequate.

The physical observations were used to create storage versus outflow curves for each of the 82 detention basins ultimately included in the model. HEC-HMS, the mathematical
model used in this study as described in subsequent sections, (see section 2.6 for
definition; Feldman, 2000), requires this paired data set to calculate how each detention
basin performs during storm events. More detail on this aspect of the model is presented
in Section 2.6.4. This dataset was calculated using a spreadsheet, which enabled some of
the steps to be automated.

The first step in the creation of the storage versus outflow curves is the calculation of an
elevation versus storage curve. For this task, the Incremental Storage method was
employed. A plan view scaled sketch was created for each detention basin. One-foot
contour lines were added to each sketch based on the measured slopes. A digital
planimeter was used to measure the area bounded by each contour elevation. The
Incremental Storage method works by taking the average of two adjacent contour line
areas and multiplying this area by the elevation difference between the two contours of
interest. The method starts with the zero and one-foot contours and continues vertically
until the highest contour is reached. The volume is summed at each elevation to create
the elevation versus storage curve. The elevation versus storage curve is only a function
of the size, shape and slopes of the detention basin.

It was then necessary to relate outflow to elevation. Standard orifice and weir equations
were used to describe the outflow characteristics of each outlet structure. Outlet
structures found in the survey ranged from only two orifices to more elaborate designs
that had more than five orifices. Separate equations were used for each orifice depending
on its size and shape. The outflows for each orifice were summed as the elevation
increased, thereby creating the elevation versus outflow curve. The elevation versus outflow curve is solely a function of the outlet structure design and is completely independent of the size and shape of the basin itself.

The two curves both have elevation as a common ordinate. Therefore the curves can be combined to produce the storage versus elevation relationship required by HEC-HMS. Figure 12 shows an example detention basin and a close-up of the outlet structure. The corresponding storage versus outflow curve is displayed in Figure 13.

Figure 12. Outlet structure close-up and detention basin.
2.2 Precipitation Data

Accurate precipitation data is the most important part of calibrating a hydrologic rainfall-runoff model. This was identified as a priority from the beginning of the study. The two controlling factors are spatial coverage and time increment. Rainfall intensities can vary significantly over relatively short distances. Therefore, it was important to have good spatial coverage of rainfall data. The two most popular methods for rainfall measurement are rain gages and radar estimates. Although the use of radar to estimate rainfall has made many advances in recent years, currently radar rainfall records for Valley Creek watershed are not available. Therefore a network of recording rain gages was used to measure precipitation. The tipping bucket rain gage has become the standard rain gage design. However, these rain gages along with the necessary data logging capabilities are
very expensive. Due to budget constraints of this project, setting up a network of tipping bucket rain gages was not feasible. A much cheaper alternative to the tipping bucket design is one that utilizes a drip through mechanism. Spectrum Technologies, Inc. manufactures the Datalogging Digital Rain Gauge Model 120, which was used in this study (see Figure 14.)

![Figure 14. Spectrum Technologies, Inc. Model 120 Rain Gage.](image)

The Spectrum Technologies rain gage is much smaller than the tipping bucket type, with a collector diameter of approximately 1.5 inches. The gages use a funnel and wick mechanism to drip the rainfall through a sensor that counts each individual drop and relates it to a depth of rainfall. In this system, each drop is the equivalent to 0.01 in of precipitation. The recording interval can be set from one minute to two hours; for this study, a five-minute recording interval was used. Before deployment, the rain gages were tested in the laboratory to ensure that the recorded volume corresponded to the measured
volume of precipitation throughput. The rain gages were all well within the ±4% range cited by the instruction manual (Spectrum Technologies, Inc., 2001). The rain gage was also tested beside the more popular tipping bucket rain gage during a real storm event. The tipping bucket rain gage was an American Sigma Model 2149 with an American Sigma 950 data logger. The rain gages both recorded very similar rainfall intensities, with the Spectrum rain gage recording a total of 0.76 inches of rain and the American Sigma model a total of 0.75 inches, as shown in Figure 15.

Due to vandalism, theft, and maintenance problems, it was not possible to leave the rain gages in the field. Instead they were deployed on a storm-by-storm basis. Each rain gage was securely mounted to a metal post that was hammered into the ground at each location. In total, seven storm events were measured. The network of rain gages varied
from five rain gages to seven for some storms. Finding both strategic and safe locations for the rain gages was a challenging task. The locations also had to be clear of any obstructions that might interfere with the rainfall measurements. The chosen rain gage locations varied slightly among the eight storms events measured.

The distributed model option for hydrologic modeling is a new addition to the HEC-HMS software. At the time of this study it had not been widely used outside of US Army Corps of Engineers applications. The distributed model requires precipitation data to be in a gridded format. This gridded data must be recorded in the HEC Data Storage System or DSS file format, as described in more detail in Section 2.3. However the rain gage data represent only a few discrete locations within the entire watershed. It was therefore necessary to interpolate among the point values to create a grid that gives each cell over the entire watershed a precipitation value. A grid was needed for each five-minute increment contained in the rain gage data.

The U.S. Army Corps of Engineers Hydrologic Engineering Center was in the process of completing the GageInterp software at the time it was needed for this study. The software was given to the author for testing purposes and had not been officially released to the public. The GageInterp software does what its name implies and interpolates rain gage data to create gridded data in the HEC-DSS file format required by the distributed model in HEC-HMS. The software can employ various methods for interpolation, but for this application the commonly-used inverse-distance-squared method was employed. The program runs through a DOS prompt, and can be executed either manually or
automated with a control file. Example GageInterp control files are provided in Appendix B (Evans, 2002).

Using a rain gage network to measure rainfall for a watershed has some inherent limitations. The network of five to seven gages provided fairly good coverage with an average density of about one rain gauge per 3.5 mi$^2$. The National Weather Service recommends a minimum number of rain gages for flood prediction (Feldman, 2000). For a watershed the size of Valley Creek this number would be only three gages. However, clearly it is possible that the network could over- or underestimate rainfall, depending on the individual storm distributions. The HEC-HMS Technical Reference Manual stresses the significance of such a small observation being used to represent large areas. In the storm events measured for this work, the combined rain gage “catch surface” only represents $10^{-8}$ of the 20.9 square miles in the model (Feldman, 2000).

2.2.1 Hypothetical Storms

Detention basins are traditionally designed for large storms events. These large events have very low return frequencies. The Stormwater Drainage, Erosion and Sedimentation Control Ordinance of East Whiteland Township (East Whiteland Township, 2000) states that the “peak discharge rate” after development should not exceed the predevelopment rate for the 24-hour 2-, 5-, 10-, 25-, 50-, and 100-year storms. Not surprisingly, there were no events that approached these intensities during observation periods. Since the detention basins are designed for these large storm events, evaluation of their performance during these events was an important part of this study. Due to the lack of
occurrence of actual large events, synthetic storms were generated to use in evaluation of
detention basin performance for the large rainfall events.

The Pennsylvania Intensity-Duration-Frequency, or PA-IDF, charts are commonly used
in design to determine rainfall intensities for high-intensity events (Aron et al., 1986).
Different regions of the state experience different climatology and rainfall patterns
therefore the state is divided into five regions. Southeastern Pennsylvania is defined as
Region 5. The PA-IDF manual contains curves of time duration versus maximum rainfall
depth. There are seven curves each representing a different storm return-frequency.
From these curves maximum depths were read off for each of the seven return-
frequencies. These precipitation depths are summarized in Table 1.

Table 1. Depth duration data for hypothetical storms.

<table>
<thead>
<tr>
<th>Duration</th>
<th>one-year</th>
<th>two-year</th>
<th>five-year</th>
<th>ten-year</th>
<th>twenty five-year</th>
<th>fifty-year</th>
<th>one hundred-year</th>
</tr>
</thead>
<tbody>
<tr>
<td>5 minutes</td>
<td>0.34</td>
<td>0.4</td>
<td>0.45</td>
<td>0.5</td>
<td>0.56</td>
<td>0.63</td>
<td>0.7</td>
</tr>
<tr>
<td>15 minutes</td>
<td>0.61</td>
<td>0.72</td>
<td>0.84</td>
<td>1</td>
<td>1.15</td>
<td>1.3</td>
<td>1.5</td>
</tr>
<tr>
<td>1 hour</td>
<td>1.1</td>
<td>1.3</td>
<td>1.6</td>
<td>1.9</td>
<td>2.1</td>
<td>2.5</td>
<td>3.0</td>
</tr>
<tr>
<td>2 hours</td>
<td>1.4</td>
<td>1.65</td>
<td>2</td>
<td>2.4</td>
<td>2.7</td>
<td>3.1</td>
<td>3.7</td>
</tr>
<tr>
<td>3 hours</td>
<td>1.6</td>
<td>1.9</td>
<td>2.3</td>
<td>2.6</td>
<td>3.1</td>
<td>3.7</td>
<td>4.2</td>
</tr>
<tr>
<td>6 hours</td>
<td>1.9</td>
<td>2.3</td>
<td>2.7</td>
<td>3.4</td>
<td>4.0</td>
<td>4.5</td>
<td>5.3</td>
</tr>
<tr>
<td>12 hours</td>
<td>2.4</td>
<td>2.8</td>
<td>3.5</td>
<td>4.1</td>
<td>4.9</td>
<td>5.7</td>
<td>7.0</td>
</tr>
<tr>
<td>24 hours</td>
<td>2.8</td>
<td>3.45</td>
<td>4.1</td>
<td>5</td>
<td>6</td>
<td>7.1</td>
<td>8.2</td>
</tr>
</tbody>
</table>

These values were imported into a meteorologic model in HEC-HMS. The model creates
a hypothetical hyetograph based on the input depths for each specified duration. Each of
the seven hypothetical storms was set at a twenty-four hour duration, as this is common
practice in design. HEC-HMS also allows the user to specify the distribution of the
rainfall for hypothetical storms; for this study, center-peaking storms were created with the maximum precipitation depth occurring twelve hours into the event. The resulting hyetograph created by HEC-HMS for the 24-hour 100-year storm event is shown in Figure 16. Notice that the time interval is five minutes and therefore the peak rainfall depth is the five-minute depth from Table 1.

Figure 16. Hypothetical 24-hour, 100-year storm hyetograph

All seven hypothetical storm events were run with the same basic basin model. However, the hypothetical storm meteorologic model does not use a grid-based approach. Instead, the sub-basin is treated in the same manner as each grid cell in the distributed approach. Therefore the standard Clark unit hydrograph method is used in place of the ModClark
method. Section 2.6.2 on runoff transformation provides more detail on the runoff calculations chosen.

2.3 HEC Data Storage System (DSS)

The U.S. Army Corps of Engineers HEC Data Storage System or DSS file format is the only data storage format recognized by HEC-HMS. Therefore all rain gage data, rainfall grids, observed flow data, and storage versus outflow curves must be in the HEC-DSS format. The format is best described in the HEC-DSS User’s Guide as follows:

HEC-DSS uses a block of sequential data as the basic unit of storage. This concept results in a more efficient access of time series or other uniquely related data. Each block contains a series of values of a single variable over a time span appropriate for most applications. The basic concept underlying the DSS is the organization of data into records of continuous, applications-related elements, as opposed to individually addressable data items. This approach is more efficient for water resources applications than that of a conventional data base system because it avoids the processing and storage overhead required to assemble an equivalent record from a conventional system.

However the storing of data in blocks of sequential data prevents the data from being created, accessed, displayed, or edited without using the HEC-DSS software in DOS. Therefore all input and output data from the model could only be created and viewed in these DOS applications. DSSTS is the DOS application used for entering regular-interval time series data. Like GageInterp, the program can either be run manually or automated with a control file. An example DSSTS control file is provided in Appendix C. This program was used to create a DSS file for the rain gage data that is read by GageInterp which then creates precipitation grids in the DSS format. It was also used to create DSS files for the USGS stream gage data. In late 2002, the Army Corps HEC released the
Java-based application HEC-DSSVue, which allowed for much easier viewing and editing of DSS files. This application was used to create a DSS file containing the storage versus outflow curves for each detention basin.

2.4 GIS Data

The watershed-scale evaluation of a network of detention basins is an ideal application for a Geographic Information System. The GIS was an integral part of the study. Nearly all the data used in this undertaking was either part of the GIS or was directly derived from it. Some of the spatial data was obtained from external sources while other datasets were created as a part of the study.

One of the most important data sets used in this study was the Digital Elevation Model or DEM. The DEM was obtained from Cahill Associates, West Chester, PA. It was created by joining the Malvern and Valley Forge DEMs obtained from the USGS. This portion of the data manipulation was carried out by Cahill Associates. This dataset was then clipped to only include the area of interest, which allowed for much shorter computation times. There were some cells within the DEM that contained -9999, the GRID code for no data. These cells were filled with the values of those cells surrounding them. Further information on the DEM information is provided in Section 2.5 on HEC-GeoHMS. The original DEM from Cahill Associates can be seen in Figure 17.
An impervious coverage dataset was also provided by Cahill Associates. The shapefile was created by manually outlining impervious areas seen in aerial photographs. The shapefile consists of many individual polygons each classified as either pervious or impervious. The dataset obtained from Cahill Associates was made from aerial photos from 1995. This dataset was then updated by the author using the March 2000 aerial photographs. The Cahill data showed a watershed wide impervious coverage of 14.8% in 1995. After updating the shapefile with the March 2000 photographs, the impervious coverage increased to 16.1% as shown in Figure 18. These two observations show a rate of 0.39% impervious coverage increase per year, or approximately 58 acres per year.
Impervious coverage calculations pertaining to the model are provided in more detail in Section 2.5.

![Figure 18. Impervious cover in Valley Creek watershed, March 2000.](image)

The soil type data was also generously provided by Cahill Associates. The dataset is an ArcView shapefile, with polygons representing different soil types as shown in Figure 19. The shapefile was created from a Soil Survey Geographic dataset, or SSURGO. The Soil Survey for Chester County Pennsylvania was used to further classify the soil types found within the watershed. Further soils data analysis is included in Section 2.5.
Each detention basin’s coordinates were recorded with the GPS. From these coordinates an ArcView shapefile was created. The shapefile was then projected in the NAD 1983 State Plane Pennsylvania South projection. The detention basin locations could then be accurately overlaid with other GIS data. The GPS proved to be quite accurate with many of the points lying directly on the outlet structures in aerial photographs.

The aerial photographs were obtained directly from Chester County Planning Commission. There are a total of 28 separate photos, or tiles that compose the entire Valley Creek watershed. The March 2000 photos are high quality TIFF files that are georeferenced for easy use in a GIS. They were released in 2002 and are the most recent.
photos available. Figure 20 shows a close up of one of the detention basins taken from the aerial photographs.

![Figure 20. Detail of aerial photograph showing a typical detention basin.](image)

### 2.5 HEC-GeoHMS

The Geospatial Hydrologic Modeling Extension, or HEC-GeoHMS was released in 2000.

A beta version of the upcoming second release, version 1.1, was provided by the Hydrologic Engineering Center to the author for testing purposes. The extension was jointly developed by the U.S. Army Corps of Engineers Hydrologic Engineering Center, the Environmental Systems Research Institute (ESRI), and the University of Texas at Austin. GeoHMS is an extension for the ArcView 3.x series of software. It is written in Avenue and therefore will only work in the 3.x line of ArcView. For this study it was
used in the ArcView 3.3 software package. The extension is not a hydrologic model itself, but a preprocessor for the HEC-HMS model. HEC-GeoHMS can be used to create the basin model for HEC-HMS based solely on topography. It derives watersheds and river networks from the topographic information and calculates their relevant characteristics. The extension can also be used to create the grid cell file required by the distributed model in HEC-HMS.

2.5.1 Subwatershed Delineation

In order to model the effects of multiple detention basins at the watershed scale, each detention basin needed to be modeled as an individual watershed. This presented two problems to the GeoHMS extension. GeoHMS delineates watersheds based solely on the topographic data in the DEM. The original DEM was a grid of cells with each cell measuring 30 meters square. However this resolution was too coarse to be used to represent sub-watersheds with areas as small as one acre. Therefore it was necessary to resample the DEM at a higher resolution. The DEM was resampled to a resolution of 10 meters. Each 30-meter pixel was split into 9 even pixels all containing the same value, or elevation, as the original pixel. A neighborhood smoothing technique was then applied to the DEM. The results of this operation can be seen in Figure 21.
HEC-GeoHMS calculates drainage boundaries from this topographic data. However the original DEM was not accurate enough to enable HEC-GeoHMS to distinguish watersheds for each detention basin. Therefore each area draining to a detention basin was created by manually editing the DEM to create the desired flow patterns. These sub-basins are often determined by curb cuts and other small changes in elevation. In many cases topography alone does not determine small urban watershed boundaries needed to model detention basins. Storm sewers add another complexity to watershed delineation that HEC-GeoHMS cannot compute. Therefore individual watershed delineation was performed based on aerial photos and personal observations of the site. This process was performed using a single cell editor extension in ArcView3.3. The extension was written...
in the Avenue language by John Grayson of ESRI and was provided to the author at no charge.

The first step of the delineation procedure was to outline the desired watershed draining to a basin. For simplicity the watershed areas were approximated as rectangles. Once a watershed was outlined, the boundary’s high and low points were determined. The point in the detention basin survey shapefile representing the detention basin’s outlet structure was shifted to denote the exact cell where the outlet in the model would be located. Often the points were not far from this low point as detention basins are generally located at the lowest elevations on the site. The exact elevation of the highest cell in the border was recorded. Then all the cells except the low point were changed to an elevation slightly higher than the original elevation of the high point. This effectively created a wall around the area of interest that prevented off-site flows from entering and caused all flows from on-site to exit at precisely one cell. This procedure is illustrated in Figure 22. This process was completed for all 82 detention basins that were to be included in the model.
After the individual watershed delineation was completed, the DEM was ready for use in HEC-GeoHMS. The HEC-GeoHMS procedure for developing the necessary input files for HEC-HMS can be split into four steps: 1) Terrain Preprocessing; 2) Basin Processing; 3) Stream and Watershed Characteristics; and 4) Hydrologic Modeling System.

2.5.2 Terrain Preprocessing

The terrain preprocessing procedure is by far the most memory-intensive step in the creation of the HEC-HMS input files. The input to terrain preprocessing is the DEM, a raster dataset, and the ultimate outputs are shapefiles, vector datasets, representing watersheds and rivers. The entire HEC-GeoHMS process is based on the DEM. The many datasets that are produced are all derived from this single dataset. The first step is to ensure that the DEM is hydraulically correct. Many DEMs may contain depressions or pits. These pits are often considered as errors in the DEM due to re-sampling and interpolating of the grid (Doan, 2000). HEC-GeoHMS uses an “8-point pour model” to calculate flow directions for each cell. Therefore, these depressions will not have an
outlet. To prevent this, the program can perform a “Fill Sinks” operation that will increase the value of all cells within the depressions to that of the cells surrounding the depression. The next in the terrain preprocessing procedure is the “Flow Direction” operation. Each cell is analyzed and the direction of the steepest downward gradient is calculated. This is assumed to be the flow path that runoff will follow from the cell. Each cell is then given a value corresponding to the direction of the steepest gradient. Since each cell is surrounded by eight other cells, there are eight possible values. This is where the term “8-pour point” originates. The next step is the “Flow Accumulation” operation. This operation calculates and assigns each cell a value representing the number of cells that flow to it. Cells near high points will have low values while cells located in major flow paths and drainage paths will have much higher values. The cell located at the outlet of the watershed will have the largest value. This value multiplied by the area of each cell should give the watershed’s total area. The “Stream Definition” operation follows. In this step the user must define a minimum area to be defined as a river. A smaller value will define many more small tributaries, while larger numbers may only define the main reaches of a watershed. This will also dictate the size and number of corresponding watersheds that will be calculated later in the procedure. For this application a drainage area of approximately one square mile was chosen to begin stream definition. Once the major drainage paths are defined the watersheds can be delineated. The first step in watershed delineation is the “Stream Segmentation” operation which breaks the stream up at each confluence. The next step is the final “Watershed Delineation”. All the steps to this point have produced raster datasets. However, after the “Watershed Delineation” operation vectorized datasets are created. These steps
include “Watershed Polygon Processing”, “Stream Segment Processing” and “Watershed Aggregation”. Using an area of approximately 1 square mile on the 20.9 square mile area of the Valley Creek watershed upstream of the USGS gage results in the creation of nine sub-basins which range in size from 0.8 to 5.1 square miles. The results of the terrain preprocessing procedure are shown in Figure 23 (Doan, 2000). The nine major sub-basins created in this process are labeled as shown in Figure 24.

Figure 23. Result of terrain preprocessing setup.
2.5.3 Basin Processing and Stream and Watershed Characteristics

The “Basin Processing” procedure is the next step towards developing the necessary input files for HEC-HMS. In this application, the Basin Processing component is a crucial part of the HEC-GeoHMS system. However, before the Basin Processing can begin the project area needs to be determined. This is accomplished by simply snapping to the desired outlet cell in the river dataset. This operation clips out the watershed of interest and creates a new project view and folder that contain only the datasets that will be used in the remainder of the HEC-GeoHMS work. The Basin Processing procedure is one where the user can edit the sub-basin delineation to better fit the objectives of his or her work. It is this step by which all the individual watersheds for the detention basins will be created. At this point the DEM has already been edited to create the desired
drainage patterns. However, these smaller areas have not yet been delineated by HEC-GeoHMS because of the relatively high threshold area for stream delineation. The Basin Subdivide tool allows the user to delineate contributing areas for any point in the watershed. The detention basin location dataset was overlaid with the project view. The shapefile had been edited such that each point representing a detention basin was located precisely over the low point cell in the manually created wall encompassing the desired watershed. The Basin Subdivide tool was used on each point and created both the watershed for each basin and a new river segment that connected the watershed to the existing river network created in the Terrain Preprocessing procedure. The watershed and river shapefiles were automatically updated as each watershed and river segment was updated. The resulting sub-basins and river network are shown in Figure 25.

![Figure 25. Final sub-basin delineation.](image-url)
After all the required watersheds and river segments were created it was then necessary to calculate their physical characteristics. These physical characteristics are both directly used in the HEC-HMS model and facilitate the estimation of various hydrologic parameters. The steps in the “Stream and Watershed Characteristics” procedure calculate river lengths, river slopes, sub-basin centroids, longest flow paths, and centroidal flow paths. The river lengths and slopes are stored in the attribute table of the river shapefile and can be easily viewed or further analyzed in other applications.

2.5.4 Hydrologic Model Setup

The final step in HEC-GeoHMS is the “Hydrologic Model Setup”. This is the process that ultimately creates the HEC-HMS input files. The files that are created include a background map file, the basin schematic, and a grid-cell parameter file. The first step in this procedure is to name the river segments and sub-basins that have been created. HEC-GeoHMS automatically names these elements; the names can be easily changed to titles more relevant to the project. HEC-GeoHMS then conducts a series of error checks. The first check ensures that the units are consistent between the map units in ArcView and the HMS units. The second check, “HMS Data Check” examines the entire network of sub-basins and river segments. This check ensures that all the elements are properly connected to their respective downstream elements. A text file is automatically created that summarizes the results of this check. Following the data check the HMS schematic is developed. This schematic is used in the HEC-HMS model to display elements and the connections between them. The Background Map File is then created. This file is not actually used by the HEC-HMS model. It is simply a text file that is read by HEC-HMS
and used to graphically represent the watershed boundaries and stream network. For the distributed model a Grid-Cell Parameter File is developed. In this step the sub-basins are divided into a grid of cells. The “Standard Hydrologic Grid” is used to create the grid, the resolution of the grid is defined by the user. A resolution of 100 meters was chosen for this application as shown in Figure 26.

![Figure 26. 100 meter grid over study area.](image)

The Grid-Cell Parameter file is a text file that lists each grid cell contained within each sub-basin. The file contains each cell’s coordinates, area, and the distance from the cell to the outlet of the sub-basin it is contained by. This information is used by the ModClark runoff transformation. Appendix D contains an excerpt of the Grid-Cell Parameter file. The final step in HEC-GeoHMS is the creation of the Distributed-Basin Model. HEC-GeoHMS creates a text file that is read by HEC-HMS. This text file
contains a list of all the elements within the model and it describes their interconnections. This basin file will also contain various parameters describing each element as they are assigned in HEC-HMS. HEC-GeoHMS automatically writes each sub-basin’s area into this basin file. Appendix E shows an excerpt of the Distributed-Basin Model.

2.6 HEC-HMS

There are numerous rainfall-runoff hydrologic models available; this made the selection process difficult. It is imperative to select the model that will best fit the specific needs of the project at hand. There is overlap between the capabilities of many of the various models; this made the selection process even more difficult. The U.S. Army Corps of Engineers Hydrologic Engineering Center (HEC) provides a model titled “Hydrologic Modeling System”, or simply HEC-HMS (Feldman, 2000). HEC-HMS was the model of choice because it best fit the requirements of the study. The software is an event-based model that can easily include the effects of detention basins using a level-pool reservoir routing technique. The program includes many different methods of modeling losses due to infiltration, including the Green and Ampt routine. It also contains several different techniques to choose from for routing flows through stream reaches. Unlike most other hydrologic models, HEC-HMS is capable of using gridded precipitation data in a distributed-runoff transformation. Another major feature that made HEC-HMS appealing was the HEC-GeoHMS extension that coupled the hydrologic model with GIS data. Although the U.S. Army Corps of Engineers does not provide any technical assistance for the software, HEC-HMS is widely used and there is some information available outside HEC. HEC does however provide user and technical manuals for the model. The model
is a Windows-based application with a simple and attractive Graphical User Interface (GUI). Since HEC-HMS is produced by the Army Corps of Engineers it is considered public domain and there is no charge to obtain it. Currently HEC-HMS does not contain any sediment transport or pollutant loading models. These components were not included in the scope of the research. Version 2.2.0 of HEC-HMS was used for this study.

2.6.1 Loss Model
The Green and Ampt Model was chosen to model the precipitation losses due to infiltration. Unlike most commonly used loss models, the Green and Ampt method is a mechanistic approach to the infiltration process. The Green and Ampt method is a simplification of the Richards equation. The equation used by HEC-HMS is given as:

$$f_t = K \left[ \frac{1 + (\phi - \theta_i)S_f}{F_t} \right]$$

(1)

where  

- $f_t$ = precipitation loss during time step ‘t’ [ L/T ]
- $K$ = saturated hydraulic conductivity [L/T]
- $\phi$ = porosity [dimensionless]
- $\theta_i$ = initial moisture content [dimensionless]
- $S_f$ = wetting front suction [ L ]
- $F_t$ = cumulative losses at time ‘t’ [L]
HEC-HMS groups the difference between the porosity and the initial moisture content, 
\((\phi - \theta_i)\), together as one parameter called the “Volumetric Moisture Deficit”. This parameter is used to reflect the sub-basin’s initial moisture levels prior to the event. The Green and Ampt model in HEC-HMS requires an additional two parameters not included in Equation 1. The “Initial Loss” is a depth of precipitation that the model automatically designates as losses. The Green and Ampt calculations will not begin until the initial loss depth has been exceeded. This parameter is similar to the initial abstraction in other loss models (Feldman, 2000). The initial loss parameter is a function of the sub-basin’s land use and is also used to reflect the sub-basin’s antecedent moisture level. The Green and Ampt model also requires an “impervious percentage” for each sub-basin. The Green and Ampt calculations are only performed on the remaining pervious percentage of the sub-basin. This “impervious percentage” is the percent of the sub-basin that acts as directly connected impervious areas. “Directly-connected impervious” means areas where rain that falls on these areas will flow across impervious surfaces directly into the stream network, as opposed to those impervious areas that may connect first to pervious areas before reaching the stream network.

The parameter estimation procedure was first based on existing soil data and supplemented with model calibration. The Soil Survey of Chester County lists many different soil types found in Valley Creek watershed. However, these soil types can be divided roughly into two major soil types, Loam and Silt Loam as illustrated in Figure 27 (USDA, 1963).
Green and Ampt parameters were assigned to each textural soil type (Rawls, et al, 1982). The GIS was used to calculate the percentage of each soil type within each sub-basin. Then a weighted average was used to calculate the sub-basin’s average parameter values. Table 2 summarizes the results of the GIS. This method was only used for the nine major sub-basins. Each smaller sub-basin for each detention basin was simply assigned the same Green and Ampt parameters as the larger sub-basin it was enclosed by. Later the saturated hydraulic conductivity was decreased by 30% to better represent observed flow data.
Table 2. Green and Ampt parameters for Valley Creek watershed.

<table>
<thead>
<tr>
<th>Sub-Basin</th>
<th>Silt Loam %</th>
<th>Loam %</th>
<th>Suction (in)</th>
<th>K (in/hr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R130W130</td>
<td>82.3</td>
<td>17.7</td>
<td>15.29</td>
<td>0.31</td>
</tr>
<tr>
<td>R140W140</td>
<td>74.6</td>
<td>25.4</td>
<td>15.01</td>
<td>0.33</td>
</tr>
<tr>
<td>R150W150</td>
<td>98.5</td>
<td>1.5</td>
<td>15.08</td>
<td>0.27</td>
</tr>
<tr>
<td>R160W160</td>
<td>99.9</td>
<td>0.1</td>
<td>15.9</td>
<td>0.27</td>
</tr>
<tr>
<td>R170W170</td>
<td>80.5</td>
<td>19.5</td>
<td>15.22</td>
<td>0.34</td>
</tr>
<tr>
<td>R180W180</td>
<td>96</td>
<td>4</td>
<td>15.76</td>
<td>0.28</td>
</tr>
<tr>
<td>R190W190</td>
<td>72.7</td>
<td>27.3</td>
<td>14.95</td>
<td>0.34</td>
</tr>
<tr>
<td>R200W200</td>
<td>78.2</td>
<td>21.8</td>
<td>15.14</td>
<td>0.32</td>
</tr>
<tr>
<td>R300W300</td>
<td>79.7</td>
<td>20.3</td>
<td>15.19</td>
<td>0.32</td>
</tr>
</tbody>
</table>

Both the initial loss and volumetric moisture deficit were adjusted until the calculated outflow volume best matched that observed by the USGS stream gage. Together these two parameters reflect the antecedent moisture levels of the watershed. The values for these parameters were kept consistent for each sub-basin in the entire model area. The initial loss and volumetric moisture deficit parameters for each of the seven rainfall events that were measured are summarized in Table 3.

Table 3. Antecedent moisture Green and Ampt parameters.

<table>
<thead>
<tr>
<th>Storm Date</th>
<th>Rainfall (in)</th>
<th>I. Loss</th>
<th>Vol MD</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/14/2001</td>
<td>0.47</td>
<td>0.15</td>
<td>0.47</td>
<td>storm 10 days prior</td>
</tr>
<tr>
<td>9/20/2001</td>
<td>1.98</td>
<td>0.15</td>
<td>0.07</td>
<td>storm of 39cfs 6 days prior</td>
</tr>
<tr>
<td>5/12/2002</td>
<td>1.35</td>
<td>0</td>
<td>0.04</td>
<td>peak of 27cfs 8hr prior and a storm of 86cfs 4 days prior</td>
</tr>
<tr>
<td>6/6/2002</td>
<td>1.65</td>
<td>0</td>
<td>0.07</td>
<td>peak of 38cfs 27hr prior and a storm of 518cfs 30 days prior</td>
</tr>
<tr>
<td>7/23/2002</td>
<td>0.13</td>
<td>na</td>
<td>na</td>
<td>no runoff</td>
</tr>
<tr>
<td>8/24/2002</td>
<td>0.59</td>
<td>0.15</td>
<td>0.47</td>
<td>storm of 94cfs 36 days prior</td>
</tr>
<tr>
<td>8/28/2002</td>
<td>1.20</td>
<td>0.15</td>
<td>0.01</td>
<td>storm of 64cfs 4 days prior</td>
</tr>
</tbody>
</table>

The 7/23/02 event showed no increase in flow rate at the USGS stream gage. The averaged total rainfall from the network of rain gages for this event was 0.13 inches.
This provided a good estimate of the initial loss of the system. Therefore the initial loss for events with relatively dry antecedent conditions was set at 0.15 inches. Since the 7/23/02 event displayed no observable increase in flow rate this event was not simulated. On both the 5/12/02 and 6/6/02 events, the rain gages were deployed shortly after some precipitation had already fallen. Therefore, the initial loss was assumed to be zero, as the majority of the depression storage was already full. The moist antecedent conditions are also shown by the significantly smaller volumetric moisture deficits for both of these storm events.

As previously mentioned, the original impervious coverage dataset for the Valley Creek watershed was supplied by Cahill Associates. This dataset was created from 1995 aerial photographs. The coverage was then updated using the March 2000 photographs. A GIS was used to calculate the percent impervious coverage for each of the nine major sub-basins. However this represented the total area of impervious surfaces, both connected and not. For a watershed this size, it is not possible to directly determine the percentage of impervious area that acts directly connected. There has also been little research done on this topic. However, Pierce County, Washington has prepared guidelines on the estimation of impervious and directly or “hydraulically” connected impervious area (Pierce County, 2000). This was part of Pierce County’s Basin Plans program. Impervious coverage data is very difficult to attain or create, while land use data is usually readily available. Therefore the county initiated a study to relate land use to both percent impervious coverage and percent directly connected impervious coverage. The results from Pierce County’s study along with the Valley Creek land use and model
calibration were all used to arrive at a directly connected impervious percentage for the Valley Creek watershed. As of March 2000, the area upstream of the USGS stream gage was 17.4% impervious with 30% of that impervious area acting as directly connected impervious cover. The difference between the 17.4% and the entire watershed value of 16.1% is predominately due to the fact that Valley Forge National Historical Park is downstream of the stream gage. As previously mentioned, this area is still rather undeveloped.

As each detention basin’s drainage area was delineated, the directly-connected impervious percentages were estimated from the aerial photographs. Of the 82 additional sub-basins, 51 of them were industrial, commercial or other land-use types that generally have extremely high directly connected impervious percentages. The remainder of the sub-basins were residential areas, which tend to have much lower percentages. The average directly connected impervious percentage assigned for these sub-basins was 54% with a maximum of 90% and a minimum of 10%. In order to avoid counting impervious area in the detention basin sub-basins twice, these areas were subtracted from the larger sub-basins within which they were contained. For this procedure, a GIS was used to correct the directly connected impervious percentages of the nine major sub-basins for the impervious areas within the detention basin sub-basins. The results from this GIS are shown in Table 4.
Table 4. Impervious percentages for major sub-basins.

<table>
<thead>
<tr>
<th>Sub-Basin</th>
<th>Area (mi²)</th>
<th>Impervious %</th>
<th>Connected Imp. %</th>
<th>Corrected Connected Imp. %</th>
</tr>
</thead>
<tbody>
<tr>
<td>R130W130</td>
<td>2.844</td>
<td>15.1</td>
<td>6.0</td>
<td>1.1</td>
</tr>
<tr>
<td>R140W140</td>
<td>3.094</td>
<td>7.7</td>
<td>3.1</td>
<td>1.4</td>
</tr>
<tr>
<td>R150W150</td>
<td>0.763</td>
<td>29.4</td>
<td>11.8</td>
<td>5.9</td>
</tr>
<tr>
<td>R160W160</td>
<td>1.589</td>
<td>18.5</td>
<td>7.4</td>
<td>2.5</td>
</tr>
<tr>
<td>R170W170</td>
<td>5.094</td>
<td>25.9</td>
<td>10.4</td>
<td>5.8</td>
</tr>
<tr>
<td>R180W180</td>
<td>1.202</td>
<td>19</td>
<td>7.6</td>
<td>5.1</td>
</tr>
<tr>
<td>R190W190</td>
<td>2.173</td>
<td>9.7</td>
<td>3.9</td>
<td>1.2</td>
</tr>
<tr>
<td>R200W200</td>
<td>3.026</td>
<td>15.6</td>
<td>6.2</td>
<td>3.2</td>
</tr>
<tr>
<td>R70W70</td>
<td>1.128</td>
<td>20.2</td>
<td>8.1</td>
<td>4.3</td>
</tr>
</tbody>
</table>

2.6.2 Runoff Transformation

The only distributed runoff model available in HEC-HMS is the Modified Clark, or “ModClark” method (Clark, 1945). This method is based on the original Clark Unit Hydrograph method. Both the ModClark and the standard Clark Unit Hydrograph are empirical methods used to relate precipitation excess to outflow (stream flow at the basin outlet point) for a given watershed. The Clark Unit Hydrograph method is based on representing the translation and attenuation effects of the watershed on the precipitation excess. HEC-HMS only requires two parameters for this model: a watershed storage coefficient ‘R’, and time of concentration “\(T_c\)”. The translation is modeled by a “grid-based travel-time model” and the attenuation effects are simulated using a linear reservoir model (Feldman, 2000). The difference between the ModClark and the standard Clark method is that the ModClark model “accounts explicitly for variations in travel time to the watershed outlet from all regions of the watershed” (Feldman, 2000). The ModClark method uses the Grid Cell Parameter file created by HEC-GeoHMS. This file contains the coordinates of each grid cell along with their area and travel distance to the outlet of the sub-basin they are contained by. The travel time or “translation time” for each cell is
computed by equating velocities between the cell of interest and the most remote cell in the sub-basin of interest, whose velocity is indirectly supplied by the time of concentration. This is accomplished using the following equation for cell translation time supplied in the HEC-HMS Technical Reference Manual (Feldman, 2000):

\[
t_{cell} = t_c \frac{d_{cell}}{d_{max}}
\]  

(2)

where

- \( t_{cell} \) = travel time for cell of interest [T]
- \( t_c \) = specified time of concentration for sub-basin [T]
- \( d_{cell} \) = travel distance from cell of interest to outlet of containing basin [L]
- \( d_{max} \) = travel distance of most remote cell in sub-basin [L]

Both the attenuation and translation effects of a watershed on a water travel time are clearly a function of physical characteristics such as slope and type of land cover. There have been efforts to develop empirical relationships between physical watershed characteristics and the Clark parameters ‘R’ [hr], which is described as the “Proportionality constant between storage and discharge at the outlet point of a basin, a time characteristic of a basin indicative of channel storage capacity”, and ‘Tc’ [hr], described as “The time required for excess rain falling on the remotest part of a drainage area to reach the outlet or point of discharge on the stream” (Graf, 1982; Straub, 2000). These relationships however, are only valid when used on the watershed for which they were developed. The most common method for estimating these parameters is through model calibration. For this investigation the storage coefficient and time of concentration were fit using the USGS observed flow data. The parameters where kept the same for all
six of the storm events analyzed. Therefore the final values may not represent the optimal values for each event, but the optimal for all six events. This parameter estimating was done by eye. The storage coefficient and time of concentration for all the detention basin sub-basins were kept at their lowest allowed value of 0.1hr. The parameter values for the nine major sub-basins are summarized in Table 5.

Table 5. ModClark parameters for major sub-basins in Valley Creek watershed.

<table>
<thead>
<tr>
<th>Sub-Basin</th>
<th>Tc (hr)</th>
<th>R (hr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>R130W130</td>
<td>0.64</td>
<td>0.85</td>
</tr>
<tr>
<td>R140W140</td>
<td>0.85</td>
<td>1.14</td>
</tr>
<tr>
<td>R150W150</td>
<td>0.26</td>
<td>0.34</td>
</tr>
<tr>
<td>R160W160</td>
<td>0.91</td>
<td>1.21</td>
</tr>
<tr>
<td>R170W170</td>
<td>1.14</td>
<td>1.52</td>
</tr>
<tr>
<td>R180W180</td>
<td>0.31</td>
<td>0.42</td>
</tr>
<tr>
<td>R190W190</td>
<td>0.55</td>
<td>0.73</td>
</tr>
<tr>
<td>R200W200</td>
<td>0.78</td>
<td>1.04</td>
</tr>
<tr>
<td>R70W70</td>
<td>0.22</td>
<td>0.30</td>
</tr>
</tbody>
</table>

2.6.3 Channel Routing

HEC-GeoHMS creates a junction at each outlet of each sub-basin. It connects these junctions by creating river reaches based on the flow paths derived from the DEM. When two of these reaches join, another junction is formed and a new reach is begun. In total there were 91 separate sub-basins in the final model of Valley Creek watershed. HEC-GeoHMS delineated 165 reaches required to represent the interconnectivity of the system. These reaches ranged from 33 feet to 1.3 miles in length. Each one of these reaches required its own channel routing parameters based on its individual physical characteristics. For this reason one of the simplest yet most widely accepted channel routing methods was chosen, the Muskingum method (e.g., Bedient and Huber, 1992).
The Muskingum method is an empirical channel flow model that simulates the storage and attenuation that occurs in a reach. It routes flows from an upstream point to a downstream point given and inflow hydrograph. The model is based on a finite difference approximation of the continuity equation. The Muskingum technique requires only three parameters ‘K’, ‘X’, and ‘number of sub-reaches’ for each reach.

The parameters for the Muskingum model are largely empirical but do have some physical significance. The Muskingum ‘K’ represents the travel time through the reach, and therefore it has no numerical boundaries. The ‘X’ parameter “…defines the relative weighing of inflow and outflow in determining storage volume in the reach” (Bedient and Huber, 1992). However, this factor has little physical relevance. While ‘X’ can range from 0.0 to 0.5, it is generally kept at 0.2 for natural streams (Bedient and Huber, 1992). However, the Muskingum method is only numerically stable under a certain range of ‘K’ and ‘X’ values. This range is also influenced by the computation time step, which in this application was five minutes. With ‘X’ being kept at 0.2 and a five minute time step, ‘K’ can only range from 0.05 to 0.21 hours. Clearly 0.21 hours is not a long enough time period to accurately reflect the travel time of the larger reaches. To account for this HEC-HMS is able to break the reach into equal-distance sub-reaches, thereby dividing the travel time by the number of sub-reaches chosen. The routing is then calculated for the first sub-reach and its outflow is used as the subsequent sub-reach’s inflow.

Therefore, by increasing the number of sub-reaches the Muskingum calculations can remain stable for longer reaches. The ‘number of sub-reaches’ is therefore used for
numeric stability. The sub-reaches are just used during computation and do not appear in
the actual basin model.

The standard practice for the Muskingum ‘K’ estimation is calibration to observed flow
data. Initial versions of the basin model used in this study only had the nine major sub-
basins. In this model there were only five reaches compared to the 165 in the final
model. Using the optimization feature in HEC-HMS, the computed hydrograph could be
modeled to be nearly collinear with the observed hydrograph. This was accomplished by
solely optimizing the Muskingum ‘K’ of the five reaches. However some of the
“optimal” values for ‘K’ found by this procedure were unrealistic. The values would
likely fail to accurately simulate storm events besides the one they were derived from.
The problem would be further exacerbated by the addition of 160 reaches in the final
model.

Having the USGS stream gage data was an extremely useful asset in this study.
However, the purpose of this work was not to create and calibrate a model that perfectly
simulates the observed hydrograph for one or two isolated storm events. The purpose of
this work was to determine the effects of multiple detention basins on the watershed
scale. Overcalibrating channel routing parameters would mask the effects of the
detention basins. The goal was to arrive at reasonable values for Muskingum parameters
that adequately represent the watershed’s response over a range of measured storm
events. Therefore Muskingum ‘K’ estimation was based on measurable physical
characteristics that would affect travel time. These characteristics were length, slope, and
channel width. It was imperative to use an estimation procedure that was consistent among reaches with the only influence being the reach’s physical characteristics.

Reach travel time is approximated by dividing the length of the reach by an approximate flow velocity. Therefore, one could assume an average velocity to represent all the reaches in the watershed and let just the reach length determine the differences in ‘K’ for the reaches. However, Valley Creek watershed has a wide range of channel slopes. The upstream reaches that originate in the north and south hills of the valley are very steep while the main branches of Valley Creek and Little Valley Creek in the valley are rather flat. This discrepancy in slope should be accounted for in the channel routing. The GIS made this information easy to extract and use outside of the GIS. Both the channel length and slope are automatically calculated by HEC-GeoHMS and stored in the river shapefile’s attribute table.

Manning’s equation calculates velocity based on channel slope, hydraulic radius and Manning’s roughness coefficient as follows:

\[
V = \frac{1.5}{n} R_h^\frac{2}{3} S^\frac{1}{2}
\]  

(3)

where  

- \( V \) = channel velocity [ft/s]  
- \( n \) = Manning’s roughness coefficient [dimensionless]  
- \( R_h \) = hydraulic radius [ft]  
- \( S \) = channel slope [dimensionless]
As previously mentioned, reach length and slope were determined by the GIS. Hydraulic radius, however, was not. As shown in Equation 4, the hydraulic radius of a cross section is a ratio of the effective flow area to the wetted perimeter. For shallow stream beds where channel width greatly exceeds channel depth, the hydraulic radius can be estimated as the average depth of the cross section.

\[ R_h = \frac{wh}{2h+w} \approx h \]  

(4)

where  

- \( R_h \) = hydraulic radius [L]
- \( w \) = channel width [L]
- \( h \) = average channel depth [L]

During the course of this research many flow rate measurements were taken throughout Valley Creek watershed by the Drexel University watersheds research group. Each survey location’s coordinates were logged with a GPS and incorporated into a GIS dataset. From this dataset average depth versus stream distance plots could be created for both Valley Creek and Little Valley Creek as shown for the example data set from October 2001 in Figure 28.
The only parameter that still had to be estimated was the Manning’s roughness coefficient. This coefficient is used to describe how the channel’s roughness affects flow velocity. The physical characteristics of the Valley Creek watershed break the watershed into two distinct flow regimes: the steep headwaters and the main branches. Manning’s ‘n’ has been reported in the literature to be much higher in steep upland streams. In these situations “…the effective roughness is controlled by the riffle-and-pool structure and not by the size of the bed material” (Hornberger et al., 1998). This was accounted for in the model by assigning a larger ‘n’ to reaches in the upstream regions. Upstream reaches were simply defined as those with widths less than 20 feet. Since there is no quantitative data for this parameter in the Valley Creek watershed, it was estimated by calibration to the observed USGS stream gage. From the calibration, Manning’s ‘n’ values of 0.2 and 0.1 were determined for the upstream and downstream reaches, respectively. Finally the Manning’s ‘n’ coefficient, the channel slope, and the channel depth were used to estimate
the flow velocity. This velocity was then used with the reach length data to estimate travel time in the individual reaches. The travel times were input as the Muskingum ‘K’ in the HEC model and kept consistent throughout the simulations.

2.6.4 Reservoir Routing

HEC-HMS uses the “level pool routing technique”, or Modified Puls routing method to simulate the storage effect of reservoirs, or in this case detention basins. The model recursively solves a finite difference approximation of the continuity equation, which is given as:

\[
\frac{I_t + I_{t+1}}{2} - \frac{O_t + O_{t+1}}{2} = \frac{S_{t+1} - S_t}{\Delta t}
\]

(5)

where

- \( I_t \) = inflow at beginning of ‘t’ time interval \([L^3/T]\)
- \( I_{t+1} \) = inflow at end of ‘t’ time interval \([L^3/T]\)
- \( O_t \) = outflow at beginning of ‘t’ time interval \([L^3/T]\)
- \( O_{t+1} \) = outflow at end of ‘t’ time interval \([L^3/T]\)
- \( S_t \) = storage at beginning of ‘t’ time interval \([L^3]\)
- \( S_{t+1} \) = storage flow at end of ‘t’ time interval \([L^3]\)
- \( \Delta t \) = time interval \([T]\)

The inflow ordinates for all time steps are determined by the loss model and ModClark method. The Modified Puls method requires an initial condition to be specified in order to begin the iteration of the continuity equation. For this application the initial condition
was specified by setting the initial outflow to 0 cfs. Storage vs. outflow curves were used to describe how each individual detention basin’s size and outlet structure configurations attenuate incoming flows. The derivation of these relationships is described in section 2.1.2. The storage vs. outflow dataset for all the detention basins in the watershed were all stored in a single external DSS file.

2.7 Summary

Field data from onsite detention-basin surveys and rain gages were collected along with physical watershed data (DEM, soils) from outside sources. These data were then transformed and assembled using the ArcView GIS and the HEC-GeoHMS extension in ArcView. The GIS was then used as a preprocessor for the U.S. Army Corps of Engineers HEC-HMS hydrologic model. The completed HEC-HMS model was calibrated for use in simulating six measured storm events and a series of hypothetical storms. The results of this effort are presented in Chapter 3.
Chapter 3. Modeling Results

This chapter presents the initial modeling effort for the case where all 82 observed and surveyed detention basins in the Valley Creek watershed are included in a hydrologic rainfall-runoff model for the purpose of evaluating the effect of the multiple detention basins on the outlet stream hydrograph. Model choice (U.S. Army Corps of Engineers HEC-HMS model), data collection, parameter selection, and model set-up are described in Chapter 2. Results presented in this chapter include modeling of six measured precipitation events, with both calibrated and dry antecedent moisture levels, as well as modeling of larger hypothetical storms, which are more representative of the design basis for detention basins, but the magnitudes of which were not observed during the project duration.

3.1 Measured Precipitation Events

There were six storm events measured using the network of rain gages that were modeled for this investigation. The first modeling endeavor was split up into two groups. In the first group, the USGS stream gage data was used to calibrate the loss model parameters to obtain a good agreement for runoff volume. The only parameters that were varied between the storms for this effort were Initial Losses and the Volumetric Moisture Deficit as discussed in Table 3 Section 2.6.1. Each storm event was first input to the basin model without the detention basin’s storage capabilities. Then the same precipitation data were input to an identical model with the exception that the second model accounted for the detention basin’s storage characteristics as dictated by the storage vs. outflow curves.

The second group of model simulations used Green and Ampt parameters indicative of
dry antecedent moisture conditions. In these simulations all parameters were kept the same between storm events.

3.1.1 Calibrated Antecedent Moisture Conditions

Figure 30 shows the computed hydrographs for all six storm events. All of the results are displayed along with a hyetograph computed as the average of all rain gages for the event. Also shown on the figures is the observed USGS stream gage hydrograph used to calibrate the Green and Ampt loss model.

Figure 30 shows that the differences between the outflow hydrographs calculated with and without detention were minor. Some of the computed hydrographs do show ever-so-slight evidence of detention. The hydrographs with detention basins show some peak flow reduction and elevated receding limbs indicative of detention storage.

Table 6 summarizes the average rainfall depth recorded by the rain gages for each event and the peak discharges computed at the watershed outlet for both scenarios. This table shows that there was essentially no peak flow rate reduction observed in the six storms events. In two of the events the peak flow rate actually was increased with the addition of the detention basins; however, as a percentage of the total peak flow rate these reductions or in two cases additions are insignificant. The peak flow rate reduction generally is shown to increase as the total precipitation depth increases as shown in Figure 29.
Table 6. Model results with calibrated antecedent moisture conditions.

<table>
<thead>
<tr>
<th>Date</th>
<th>Avg. Rain Gage Totals (in)</th>
<th>Peak Discharge Without Detention Basins (cfs)</th>
<th>Peak Discharge With Detention Basins (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/14/2001</td>
<td>0.47</td>
<td>45</td>
<td>46</td>
</tr>
<tr>
<td>9/20/2001</td>
<td>1.97</td>
<td>307</td>
<td>304</td>
</tr>
<tr>
<td>5/12/2002</td>
<td>1.35</td>
<td>217</td>
<td>216</td>
</tr>
<tr>
<td>6/6/2002</td>
<td>1.65</td>
<td>465</td>
<td>464</td>
</tr>
<tr>
<td>8/24/2002</td>
<td>0.59</td>
<td>50</td>
<td>51</td>
</tr>
<tr>
<td>8/28/2002</td>
<td>1.20</td>
<td>115</td>
<td>112</td>
</tr>
</tbody>
</table>

Figure 29. Peak flow rate reductions with calibrated antecedent moisture conditions.
Figure 30. Simulated hydrographs for calibrated antecedent moisture conditions for six different storms in 2001-2002 in Valley Creek Watershed.
The 9/14/01 event was the smallest event modeled in the study with a watershed wide average rainfall depth of 0.47 in. In this event the Green and Ampt model did not predict any runoff. Therefore all the runoff computed in this run was from the areas considered to be directly connected impervious. Total runoff volume was over-predicted in this case as shown in Figure 30. There is evidence of detention in this event with a slight peak flow rate reduction and a slightly higher receding limb.

The second event modeled occurred four days after the first event on 9/20/01. This storm was much larger than the previous event with a watershed wide average rainfall depth of 1.97 in which made this the largest event monitored over the course of the study. The computed hydrographs are shown in Figure 30. This event was intense enough to produce runoff from the pervious areas as calculated by the Green and Ampt loss model. The parameters in the loss model provided a close estimation of the total runoff volume. However, the computed hydrographs exhibit a double peak while the USGS stream gage observed a single-peaking event. The discrepancies between the computed runs do indicate the presence of detention in the watershed, however again these differences are minor.

The third event modeled was the 5/12/02 event with a watershed-wide average rainfall depth of 1.35 in. Overall the computed hydrograph follows the observed hydrograph fairly well. The rainfall observed in this storm was drawn out over a period of almost two days. However, there were some periods of intense rain. In the southwest corner of the watershed the rain gages observed a brief period of intense rainfall with a peak five-
minute intensity of 0.13 in, or about 1.6 in/hr. This short period of rainfall is what created the large surge in the computed hydrographs during the receding limb of the observed hydrograph as shown in Figure 30. The model predicted flows of over 300 cfs in the upstream reaches. The surge was attenuated by the channel routing model to approximately 275 cfs predicted at the outlet.

The next storm event measured and modeled in this study occurred on 6/6/02. The total precipitation for the watershed in this event was 1.65 in. While this is less than the 1.97 in observed in the 9/20/01 event, this was the most intense event with the precipitation occurring over a much shorter duration. The 6/6/02 event also had the highest five-minute peak intensity recorded in all seven of the events monitored. In one five-minute interval one rain gage counted 16 drops, the equivalent of 1.9 in/hr. The USGS stream gage recorded a peak flow rate of 576 cfs, which was the highest observed flow rate of all the modeled events. The computed hydrographs are shown in Figure 30.

The final two storms measured and modeled in this study occurred in late August of 2002. The 8/24/02 event was a smaller event with an average precipitation depth of 0.59 inches. The precipitation had a bimodal distribution that produced a slight double peak as shown in Figure 30. The first peak flow rate was over-predicted by about 10 cfs and the event peak flow rate was underpredicted by about 10 cfs. As with the smaller storm on 9/14/01 all the runoff in this event came from directly connected impervious surfaces. The Green and Ampt loss model did not predict any runoff for the duration of the event.
The antecedent moisture conditions for this event were kept at their minimum as this event was the only significant precipitation event in 36 days.

The 8/28/02 event was larger with the rain gages recording an average of 1.20 inches of rainfall. However, the precipitation for this event was a very slow steady rainfall with a peak five-minute intensity of 0.72 in/hr and the majority of the five-minute readings being the minimum of 0.12 in/hr. The average hyetograph is shown in Figure 30. Since the precipitation intensity during this event rarely exceeds the hydraulic conductivity specified in the Green and Ampt loss model, the model predicts very little precipitation excess. Therefore, in this event the vast majority of the runoff is only from the directly-connected-impervious percentage of the watershed.

3.1.2 Dry Antecedent Moisture Conditions

The average directly-connected-impervious percentage for the areas draining to detention basins is 54%. This is much higher than the watershed’s average of 5% directly connected impervious coverage. Although the detention basin drainage areas only make up about 4% of the watershed area, they account for 39% of the directly connected impervious cover. In larger storms when the pervious areas governed by the Green and Ampt model contribute runoff, the effects of the detention basins are masked by runoff from the remaining 96% of the watershed. Therefore to maximize the effect of the detention basins in the system, the antecedent moisture levels should be kept at their minimum thereby minimizing the runoff from these areas without affecting the amount of runoff from the directly-connected impervious areas.
In Section 3.1.1, “Calibrated Antecedent Moisture Conditions” both the 9/14/01 and the 8/24/02 storm events were modeled with dry antecedent moisture conditions as these events occurred after long periods without significant precipitation. Therefore the computed hydrographs for these two events are the same as for the results from the previous section. The results from the dry antecedent moisture conditions are summarized in Table 7.

Table 7. Model results with dry antecedent moisture conditions.

<table>
<thead>
<tr>
<th></th>
<th>Avg. Rain Gage Totals (in)</th>
<th>Peak Discharge Without Detention Basins (cfs)</th>
<th>Peak Discharge With Detention Basins (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/14/2001</td>
<td>0.47</td>
<td>45</td>
<td>46</td>
</tr>
<tr>
<td>9/20/2001</td>
<td>1.97</td>
<td>117</td>
<td>114</td>
</tr>
<tr>
<td>5/12/2002</td>
<td>1.35</td>
<td>61</td>
<td>61</td>
</tr>
<tr>
<td>6/6/2002</td>
<td>1.65</td>
<td>136</td>
<td>136</td>
</tr>
<tr>
<td>8/24/2002</td>
<td>0.59</td>
<td>50</td>
<td>51</td>
</tr>
<tr>
<td>8/28/2002</td>
<td>1.20</td>
<td>78</td>
<td>75</td>
</tr>
</tbody>
</table>

The results again do not show a significant difference between the computed hydrographs with and without detention basins. Again in the same two cases, peak flow rate was found to increase with the detention basins in place. The 8/28/02 event again showed the highest peak flow rate reduction of just over 3cfs. The peak flow rate reductions for the dry antecedent-moisture-condition modeling are summarized in Figure 31.
The computed hydrographs with dry antecedent moisture levels for the six storm events are shown in Figure 32.
Figure 32. Simulated hydrographs for dry antecedent moisture conditions for six different storms in 2001-2002 in Valley Creek watershed.
3.2 Hypothetical Storm Events

There were no severe precipitation events observed during the precipitation monitoring period for this study. Since the amount of detention is a function of the rainfall intensity, and since detention basins are designed for more severe precipitation events, it was of great interest to this study to determine what cumulative effects the detention basins might have for more intense storms. Therefore seven hypothetical storms were created as input for the HEC-HMS model.

The results from the hypothetical large storm events are similar to those observed for the measured smaller precipitation events. The computed hydrographs from the various return frequency events show some evidence of detention with small peak flow rate reductions. The peak flow rates are summarized in Table 8. Since the areas that drain to the detention basins have high percentages of directly connected impervious surfaces, the antecedent moisture levels throughout the watershed were kept at their minimum values to make the effect of the detention basins most evident for the model runs.

<table>
<thead>
<tr>
<th>Storm Return Period</th>
<th>Precip. (in)</th>
<th>Peak Discharge Without Detention Basins (cfs)</th>
<th>Peak Discharge With Detention Basins (cfs)</th>
<th>Percent Decrease With Detention Basins (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-Year</td>
<td>2.8</td>
<td>547</td>
<td>546</td>
<td>0.18</td>
</tr>
<tr>
<td>2-Year</td>
<td>3.45</td>
<td>623</td>
<td>621</td>
<td>0.32</td>
</tr>
<tr>
<td>5-Year</td>
<td>4.1</td>
<td>1169</td>
<td>1166</td>
<td>0.26</td>
</tr>
<tr>
<td>10-Year</td>
<td>5</td>
<td>1919</td>
<td>1912</td>
<td>0.37</td>
</tr>
<tr>
<td>25-Year</td>
<td>6</td>
<td>2588</td>
<td>2577</td>
<td>0.43</td>
</tr>
<tr>
<td>50-Year</td>
<td>7.1</td>
<td>3594</td>
<td>3579</td>
<td>0.42</td>
</tr>
<tr>
<td>100-Year</td>
<td>8.2</td>
<td>4859</td>
<td>4840</td>
<td>0.39</td>
</tr>
</tbody>
</table>
The peak flow rate reductions are relatively insignificant compared to the watershed-wide peak flow rates. The one hundred-year event produced the largest peak flow rate reduction of 19 cfs. This reduction constitutes only 0.4% of the watershed wide peak flow rate of 4,859 cfs. The percent peak flow reductions are plotted vs. the total rainfall depth for each of the return frequencies in Figure 33.

![Figure 33. Percent peak flow rate reductions for various return period storm events.](image)

The detention basins do not appear to have a significant effect on the watershed-wide peak flow rates for these large hypothetical storms. However, any attenuating effect the detention basins may be having is being overshadowed by the unattenuated area which
constitutes 96% of the total watershed area. In these large storms all pervious areas are likely contributing runoff.

3.3 Summary

In this chapter, six measured precipitation events and a series of larger hypothetical storms were modeled for the study area. The results summarized in this chapter clearly indicate that the detention basins in Valley Creek watershed have a negligible impact on watershed-wide peak flow rates, with insignificant reductions, and in some cases, with increases. This illustrates that the current design basis of detention basins is significantly deficient and that current stormwater management practices based on design requirements are ineffective. In Chapter 4, the model is next utilized in a design mode to evaluate possible scenarios for reducing stormwater flow rates and volumes by modifying the existing functioning of the system of detention basins in the watershed.


Chapter 4. Analysis

In order to better describe the role of the detention basins on stormwater flow, both the model and the model results are further examined in this chapter. In Chapter 3 the model is used to represent actual field conditions. In this chapter, the model is abstracted in an attempt to better explain the function and potential functions of the detention basins in Valley Creek watershed. The first topic examined is determining what effect the detention basins have when only those areas that drain to a detention basin are modeled. The second detail of the system investigated is an insight into the design rationale of the detention basins based on an analysis of local annual rainfall patterns. Then watershed timing issues critical to detention basin effectiveness are examined. The final two sections of this chapter explore hypothetical retrofit scenarios and their effect on stormwater flow.

4.1 Analysis of the Effect of Drainage Area

One of the factors that may describe the lack of watershed wide peak flow attenuation is the relatively small total drainage area captured by the detention basins in the model. The final watershed delineation is shown in Figure 34 with the areas attenuated by detention basins shaded.
The detention basins collect stormwater from 4% of the entire 21-square-mile watershed. Clearly as this percentage increases so would the cumulative effect of the detention basins on stream flow at the watershed scale. Although the detention basins drain only 4% of the area, this area accounts for 39% of the directly-connected impervious cover within the model. This makes the mere 4% of the area a slightly misleading statistic. In smaller storms of approximately 0.50 in and less, only the impervious areas will produce runoff. This is especially true when antecedent moisture levels are low. This means that for these storms, the areas that drain to detention basins are responsible for 39% of the stormwater flow in the stream system. For example, in the 9/14/01 storm the average total rainfall depth recorded was 0.47 in. The total computed runoff was 27.4 ac-ft, not including base flow. The 82 sub-basins that drain to detention basins produced 10.9 ac-ft of that stormwater flow. However, in larger storms the infiltration capacity of pervious...
areas is often exceeded. When this once-pervious percentage of the watershed begins to contribute runoff, the effects of the detention basins will become less evident.

In order to better evaluate the cumulative effects of the detention basins, the HEC-HMS model was modified to include only the 0.78 square miles that drain to detention basins. The nine larger sub-basins delineated in Figure 34 were not represented in this version of the model. However, the spatial distributions and travel times were all kept consistent with the original model. The purpose of this modification was to illustrate how the detention basins function independent of the non-attenuated areas of the watershed. All six of the measured storms were run on this model with dry antecedent moisture conditions. A summary of the results is shown in Table 9.

<table>
<thead>
<tr>
<th>Avg. Rain Gage Totals (in)</th>
<th>Peak Discharge Without Detention Basins (cfs)</th>
<th>Peak Discharge With Detention Basins (cfs)</th>
<th>Percent Reduction in Peak Flow Rate (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/14/2001 0.47</td>
<td>8.6</td>
<td>7.6</td>
<td>13</td>
</tr>
<tr>
<td>9/20/2001 1.97</td>
<td>36.8</td>
<td>32.0</td>
<td>15</td>
</tr>
<tr>
<td>5/12/2002 1.35</td>
<td>12.7</td>
<td>11.8</td>
<td>7.6</td>
</tr>
<tr>
<td>6/6/2002 1.65</td>
<td>29.7</td>
<td>24.0</td>
<td>24</td>
</tr>
<tr>
<td>8/24/2002 0.59</td>
<td>10.6</td>
<td>8.8</td>
<td>20</td>
</tr>
<tr>
<td>8/28/2002 1.20</td>
<td>16.0</td>
<td>14.3</td>
<td>12</td>
</tr>
</tbody>
</table>

Each computed hydrograph from the modified model for the six measured storms is shown in Figure 35.
Figure 35. Simulated hydrographs for cases including solely detention basin drainage areas for six different storms in 2001-2002 in Valley Creek watershed.
This analysis makes the role of the detention basins more evident. The results from these six measured events show that the peak flow rates from the attenuated areas for each event are decreased by the detention basins. The peak flow rate reductions range from 7.6% to 24%. However, if the peak flow rate of the unattenuated areas occurs at any point where the hydrograph of the modified model with detention basins is higher than the modified model without the detention basins, there will be a peak flow rate increase for the entire watershed’s hydrograph. This phenomenon was observed in the 9/14/01 and 8/24/02 events. During storms with even temporally distributed rainfall like the 8/28/02 storm, it will be less likely that this phenomenon will occur. This topic is further discussed in the Section 4.3.

4.2 Analysis of Design Storms

Although the exact design criteria may vary from basin to basin, the detention basins in Valley Creek watershed have been designed for large storms with long return frequencies, i.e., 2- to 100-year storms. An examination of the annual rainfall patterns of southeastern Pennsylvania illustrates the shortcoming of this design approach. A cumulative plot of approximately 50 years of daily rainfall data from the Chadds Ford rain gage is summarized in Figure 36. Daily rainfall is defined here as the amount of rain that falls from midnight to midnight of the following day. This means that storms that began before midnight and continued into the next day are counted as two separate, smaller storms. Chadds Ford rain gage is located 20 miles from Valley Creek watershed.
Figure 36. Annual rainfall distribution for 50 years of rainfall in Chadds Ford, Pennsylvania.

The plot in Figure 36 was created by first sorting each daily total from the smallest value, 0.01 in, to the largest, 6.05 in. Then a running sum of these daily values was generated. The running sum was then divided by the number of days in the record and plotted versus the daily values. The plot illustrates that this area of southeastern Pennsylvania receives approximately 45 in of rain annually. The two-year, 24-hour storm is denoted in Figure 36 by a dashed line at a value of approximately 3.5 in of daily rainfall. This point in the plot corresponds with 97% of the annual rainfall. This means that 97% of the annual rainfall by volume falls as storms that are less than the two-year, 24-hour event; therefore storms of the two-year return period and greater account for only the remaining 3% of the annual rainfall. The purpose of this exercise is to illustrate that the design of the
detention basins in Valley Creek watershed has been focused on only 3% of the annual rainfall volume.

All of the storms measured in this study were well below the total depth of 3.5 in of a two-year storm event for southeastern Pennsylvania. Therefore it is not surprising that the model predicts that the surveyed detention basins have little effect on stormwater flow, since they are typically designed for storms of large return periods. The detention basins surveyed are characterized by a variety of outflow structure designs. In most storms the most crucial part of the outlet structure is the bottom-most orifice often referred to as the “low-flow orifice”. Many of the detention basins were found to have relatively large low-flow orifices. For example, Basin 018, which drains a contributing area of approximately two acres, has a six-inch-diameter low flow orifice shown in the photograph in Figure 37. Generally, detention basins that have such large low-flow orifices will not be able to significantly detain stormwater runoff. The detention basin’s detention capability is also a function of rainfall intensity and contributing drainage area.
During the 9/20/01 storm, the most significant storm volume measured during this study, the sub-basin associated with Basin 018 received 2.05 in of rainfall in an approximately 24-hour period. According to the model calculations, 1.66 in of the precipitation became precipitation excess and was routed to the detention basin. The resulting runoff produced a peak inflow of 1.34 cfs to the detention basin. The outlet structure was calculated to have attenuated this peak flow rate by only a small amount (1.33 cfs), a 0.7% reduction. Figure 38 illustrates the negligible effect of the outlet for Basin 018 on 9/20/01 storm, which was a typical frequent storm for this region.
Figure 38. Basin 018 performance during the 9/20/01 storm.

During a simulated 10-year storm (5.0 in of rainfall), the same detention basin received a peak inflow rate of 6.3 cfs which it attenuated to 4.1 cfs, a 35% reduction. This is a significant reduction compared to the 0.7% reduction observed in the 9/20/01 event. It was beyond the scope of this work to analyze the behavior of each detention basin for each measured and hypothetical storm event. However, it is clear from the survey data and an examination of the modeling results that many of the detention basins in Valley Creek watershed have been designed for extreme events and do very little to attenuate the more frequent storms.
4.3 Timing Issues

Each detention basin is designed to attenuate on-site peak flow rates. Generally these designs do not include watershed-wide considerations, including the impact of other detention basins within the larger watershed. The concept of the release-rate method of stormwater management is to limit the peak flow rate to a design outflow rate. However, since the on-site runoff volume is typically increased greatly from development, the design outflow rate will generally be sustained for a longer period of time than the predevelopment peak outflow rate. With many neighboring detention basins designed to operate simultaneously in the same fashion, it becomes clear that the additive timing and travel times of these peaks can have a significant impact on the watershed-wide effectiveness of the detention basins on limiting peak in-stream flows. The variability of natural rainfall patterns both spatially and temporally adds even more complexity to the issue of overall peak flow rate timing.

In two of the measured storm events, the peak flow rate at the outlet of the watershed was found to increase with the use of detention basins. While this may seem counterintuitive, it is simply a matter of timing. The precipitation pattern of the 9/14/01 event produced a modeled peak flow rate of 45 cfs without detention and 46 cfs with detention basins. To describe this 1 cfs increase it is necessary to exploit the additive property of hydrographs and model the areas that drain to detention basins separately from the nine larger sub-basins (that are not attenuated by detention basins). The results from this simulation of only the areas draining to detention basins are shown in Figure 39.
The model reveals that the peak flow rate of the hydrograph resulting from the unattenuated nine larger sub-basins (those not connected to detention basins), as modeled and discussed in Chapter 3, occurs during a receding limb of the hydrograph from the areas that are attenuated by the detention basins. This point is indicated with a dashed line in Figure 39. During this receding limb the hydrograph with detention is 1 cfs higher. Therefore when the two hydrographs in Figure 39 are added to the unattenuated hydrograph (which is the same both with and without detention basins), a higher peak flow is created for the model run with detention basins, as shown in Chapter 3 Figure 30.
4.4 Low Flow Modifications

Low flow orifice modification is a topic of interest when discussing the issue of detention basins. The idea behind such modifications, or retrofits, is to simply increase a basin’s detention capabilities by further restricting the lower orifice. This type of retrofit is most effective for smaller storms when the low-flow orifice is the only controlling outlet. To test the concept of watershed-wide detention retrofits, the HEC-HMS model was changed to reflect two possible retrofit detention scenarios. The first condition is a watershed-wide, low-flow orifice modification and the second is modification of only the upstream basins. This modeling was done with dry antecedent moisture levels to make the effect of the detention basins most evident.

4.4.1 Watershed-Wide Modifications

The first scenario was to simply replace the lowest outlet of all the basins with a four-inch orifice, while maintaining the basin’s original volume and remaining outflow structure characteristics. During the detention basin survey, the smallest outlet found had a four-inch diameter; only 18 of the 111 detention basins found had this size orifice. Generally these bottom-most outlets are not designed to have a smaller diameter because smaller diameters would tend to clog more easily. For this reason, the four-inch diameter was chosen. The storage versus outflow curves for all 82 detention basins were recalculated to reflect the modified outlet structures. The HEC-DSS file that contains all the storage versus outflow curves was then updated. The six measured storm events were all run with on the model with the modified outlet structures. The peak flow results from these simulations are summarized in Table 10.
Table 10. Summary of watershed-wide modified outlet structure results.

<table>
<thead>
<tr>
<th>Avg. Rain Gage Totals (in)</th>
<th>Peak Discharge Without Detention Basins (cfs)</th>
<th>Peak Discharge With Detention Basins (cfs)</th>
<th>Peak Discharge With Reduced Basin Orifice Diameters (cfs)</th>
<th>Percent Peak Discharge Reduction With Modification (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/14/2001</td>
<td>0.47</td>
<td>45</td>
<td>46</td>
<td>0</td>
</tr>
<tr>
<td>9/20/2001</td>
<td>1.97</td>
<td>117</td>
<td>114</td>
<td>108</td>
</tr>
<tr>
<td>5/12/2002</td>
<td>1.35</td>
<td>61</td>
<td>61</td>
<td>126</td>
</tr>
<tr>
<td>6/6/2002</td>
<td>1.65</td>
<td>136</td>
<td>136</td>
<td>7.4</td>
</tr>
<tr>
<td>8/24/2002</td>
<td>0.59</td>
<td>50</td>
<td>51</td>
<td>49</td>
</tr>
<tr>
<td>8/28/2002</td>
<td>1.20</td>
<td>78</td>
<td>75</td>
<td>71</td>
</tr>
</tbody>
</table>

In all but one storm, the results with the modified outlet structures show larger peak flow reductions compared to previous model runs. For the 6/6/02 storm the modifications produced a 9% reduction. This was the largest reduction observed. Detention occurs when the rainfall intensity and the corresponding inflow begins to exceed the capacity of the detention basin. Therefore it is not surprising that the largest reduction is found in the storm with a short and intense rainfall distribution.

4.4.2 Upstream Modifications

The second scenario incorporated a heuristic optimization of the detention basins with the objective of limiting stream flow rate at the outlet of the study area. For this scenario, the upstream detention basins were retrofitted with the four-inch-diameter outlet while the remaining downstream basins were relinquished of all detention capabilities. A GIS was used to determine which basins would be retrofitted and which would be removed. HEC-GeoHMS enables the user to trace the flow paths, in this case from each detention basin to the outlet of the study area, by clicking on the point of interest in the DEM. HEC-GeoHMS provides some basic statistics on flow paths including its distance to the outlet.
of the watershed. The distances from all 82 basins to the watershed outlet were calculated using this feature in HEC-GeoHMS. The detention basins were then divided in half based on their travel distances downstream to the outlet as shown in Figure 40.

![Figure 40](image.png)

Figure 40. Summary of travel distances from detention basins to the watershed outlet.

A total of 41 detention basins were modified with the four-inch outlet. Figure 41 shows the geographic distribution of the basins with the modification and those for which all detention was removed.
The six measured precipitation events were all run on the model with the upstream modifications. The peak flow results from these events are summarized in Table 11.

Table 11. Summary of model results for upstream modified outlet structures.

<table>
<thead>
<tr>
<th>Avg. Rain Gage Totals (in)</th>
<th>Peak Discharge With Reduced Basin Orifice Diameters (cfs)</th>
<th>Percent Peak Discharge Reduction With Watershed-Wide Modification (%)</th>
<th>Peak Discharge With Upstream Reduced Basin Orifice Diameters (cfs)</th>
<th>Percent Peak Discharge Reduction With Upstream Modification (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/14/2001</td>
<td>0.47</td>
<td>45</td>
<td>0</td>
<td>45</td>
</tr>
<tr>
<td>9/20/2001</td>
<td>1.97</td>
<td>108</td>
<td>7.7</td>
<td>115</td>
</tr>
<tr>
<td>5/12/2002</td>
<td>1.35</td>
<td>61</td>
<td>0</td>
<td>61</td>
</tr>
<tr>
<td>6/6/2002</td>
<td>1.65</td>
<td>126</td>
<td>7.4</td>
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</tr>
<tr>
<td>8/24/2002</td>
<td>0.59</td>
<td>49</td>
<td>2.0</td>
<td>49</td>
</tr>
<tr>
<td>8/28/2002</td>
<td>1.20</td>
<td>71</td>
<td>9.0</td>
<td>77</td>
</tr>
</tbody>
</table>

This table shows that the upstream modifications were not as effective as the watershed-wide approach for all six of the modeled storm events. However, in the smaller events,
5/12/02, 9/14/01, and 8/24/02, neither scenario produced a significant peak flow reduction. It does not appear that any practical release-rate optimization would be useful in this situation. Precipitation patterns are not consistent from storm to storm, especially for a watershed of this size. Since an optimization strategy would be focused on the detention basin’s location relative to other detention basins, the inconsistent rainfall patterns would render such a strategy useless without the use of remote sensing and automated control. Furthermore, the objective of such strategy should not be to limit the peak flow rate at one point in the watershed, but to limit the peak flow rate throughout the drainage area. Clearly the best way to manage stormwater runoff is not to create it in the first place. This topic is explored in the following section.

4.5 Infiltration Scenario

The final adaptation of the HEC-HMS model was to evaluate the effect of a network of infiltration practices on the watershed-wide stormwater flow regime. While retrofit of 82 detention basins and their drainage areas for infiltration would be expensive and logistically challenging, modification of the model was relatively simple to implement. This analysis was of key interest to the study. Infiltration is slowly replacing the standard release rate method of stormwater management. Therefore, insight into how a network of these infiltration practices would perform on a watershed scale was of great pertinence to this study.

Stormwater infiltration can be easily achieved using a variety of stormwater infiltration Best Management Practices (BMPs). Such a strategy would not use the typical design
tactics employed in standard detention basin designs. One of these goals of detention
basin design is to collect and concentrate stormwater runoff to a central location. A much
sounder approach is to keep the precipitation as close to where it falls as possible. This
can be accomplished using many smaller stormwater management BMPs, rather than one
single detention basin for the entire site.

The goal of these BMPs is to maintain predevelopment runoff volumes, therefore peak
flow rate attenuation may not be directly addressed in their design. During extreme
events detention basins may attenuate peak outflow rates, whereas infiltration BMPs
would likely have little attenuation.

For simplicity, however, the modifications to the model do not mimic multiple BMPs
within each drainage area, but rather lump their effects together. Functionally, the
modifications to the model are created such that each detention basin would have
infiltration capabilities in addition to its existing detention capabilities. This was
accomplished by connecting a diversion element between the sub-basin and its
corresponding detention basin reservoir in the HEC-HMS basin model. The diversions
were programmed to “divert” a specific volume of runoff for infiltration. When this
volume of runoff is exceeded, the diversion passes any additional incoming runoff
directly through to the detention basin. HEC-HMS allows the user to connect the
“diverted” water to other elements, but in this case the flow is not connected to any other
elements and is therefore conceptually removed from the model. There were three
different scenarios tested in this exercise: infiltration of the first 0.5 inch, 1.0 inch, and
2.0 inches of runoff. The diverted volume for each diversion was calculated by multiplying the three depths by each sub-basin’s area to yield the volume input required by the diversion element in HEC-HMS. The results from these runs are shown in Figure 42.
Figure 42. Simulated hydrographs for infiltration scenarios for six different storms in 2001-2002 in Valley Creek watershed.
Table 12 shows a summary of the computed peak flow rates for the three different infiltration scenarios and Table 13 summarizes the results as percent peak reductions.

Table 12. Summary of computed peak flow rates for infiltration scenarios.

<table>
<thead>
<tr>
<th>Date</th>
<th>Avg. Rain Gage Totals (in)</th>
<th>Peak Discharge Without Det. Basins (cfs)</th>
<th>Peak Discharge With Det. Basins (cfs)</th>
<th>Peak Discharge 0.5&quot; Infiltration (cfs)</th>
<th>Peak Discharge 1.0&quot; Infiltration (cfs)</th>
<th>Peak Discharge 2.0&quot; Infiltration (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/14/2001</td>
<td>0.47</td>
<td>45</td>
<td>46</td>
<td>42</td>
<td>42</td>
<td>42</td>
</tr>
<tr>
<td>9/20/2001</td>
<td>1.97</td>
<td>116</td>
<td>113</td>
<td>105</td>
<td>100</td>
<td>99</td>
</tr>
<tr>
<td>5/12/2002</td>
<td>1.35</td>
<td>61</td>
<td>61</td>
<td>59</td>
<td>59</td>
<td>59</td>
</tr>
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<td>6/6/2002</td>
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<td>119</td>
</tr>
<tr>
<td>8/24/2002</td>
<td>0.59</td>
<td>50</td>
<td>51</td>
<td>44</td>
<td>44</td>
<td>44</td>
</tr>
<tr>
<td>8/28/2002</td>
<td>1.20</td>
<td>78</td>
<td>75</td>
<td>68</td>
<td>64</td>
<td>64</td>
</tr>
</tbody>
</table>

Table 13. Summary of percent peak flow rate reductions for infiltration scenarios.

<table>
<thead>
<tr>
<th>Date</th>
<th>Avg. Rain Gage Totals (in)</th>
<th>Peak Discharge Without Det. Basins (cfs)</th>
<th>Peak Discharge With Det. Basins (cfs)</th>
<th>0.5&quot; Infiltration Percent Peak Reduction (%)</th>
<th>1.0&quot; Infiltration Percent Peak Reduction (%)</th>
<th>2.0&quot; Infiltration Percent Peak Reduction (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/14/2001</td>
<td>0.47</td>
<td>45</td>
<td>46</td>
<td>6.7</td>
<td>6.7</td>
<td>6.7</td>
</tr>
<tr>
<td>9/20/2001</td>
<td>1.97</td>
<td>116</td>
<td>113</td>
<td>9.5</td>
<td>14</td>
<td>15</td>
</tr>
<tr>
<td>5/12/2002</td>
<td>1.35</td>
<td>61</td>
<td>61</td>
<td>3.3</td>
<td>3.3</td>
<td>3.3</td>
</tr>
<tr>
<td>6/6/2002</td>
<td>1.65</td>
<td>136</td>
<td>136</td>
<td>5.9</td>
<td>11</td>
<td>13</td>
</tr>
<tr>
<td>8/24/2002</td>
<td>0.59</td>
<td>50</td>
<td>51</td>
<td>12</td>
<td>12</td>
<td>12</td>
</tr>
<tr>
<td>8/28/2002</td>
<td>1.20</td>
<td>78</td>
<td>75</td>
<td>13</td>
<td>18</td>
<td>18</td>
</tr>
</tbody>
</table>

The infiltration results do show peak flow rate reductions. This is expected as portions, and in some cases all of the runoff from the detention basin sub-basins is being diverted from the model. However, this is fairly representative of an actual infiltration process.

The most surprising finding of these model runs is the volume of runoff that is reduced by the theoretical infiltration practices. The area underneath each hydrograph represents a volume of runoff. The difference between the various infiltration scenarios and the red and blue, with and without detention basin hydrographs is the volume of runoff prevented by each different infiltration depth. Depending on the site specific conditions and BMPs
of choice, the runoff would be removed from the storm hydrograph by traveling through the soil to be later evapotranspirated, or become base flow or deep groundwater recharge. In order to better quantify this volume of water it was assumed that all the infiltrated water would be evenly distributed over a one-year period as base flow. Then a simple mass balance was performed using the Chadds Ford rainfall data described in Section 4.2. Each infiltration depth was subtracted from each daily rainfall depth in the 50-year data set. The remaining depth was considered runoff and the subtracted volume was assumed to contribute to the yearly base flow. The results are displayed in Table 14.

<table>
<thead>
<tr>
<th>Cubic Feet of Water Infiltrated per Year</th>
<th>Base Flow Addition (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5&quot; Infiltration</td>
<td>4.87E+07</td>
</tr>
<tr>
<td>1.0&quot; Infiltration</td>
<td>6.75E+07</td>
</tr>
<tr>
<td>2.0&quot; Infiltration</td>
<td>7.97E+07</td>
</tr>
</tbody>
</table>

Only the area that drains to a detention basin was used in these calculations. This 0.8 square miles represents only 4% of the total watershed area in the model. However, the infiltration of only the first 0.5 inch of each daily precipitation depth can theoretically account for a 1.5 cfs increase in base flow. According to the 18-year record kept by the USGS, the average annual flow rate for the watershed is approximately 32 cfs. A 2.5 cfs increase in base flow represents an 8% increase. Increasing base flow is desirable for a number of reasons, including enhancement of fish habitat, improvement of the recreational value of a stream, and possibly restoring year-round flow in tributaries which have dried out due to lowered groundwater levels. Increasing stream base flow would only be one of the advantages of this infiltration scenario. As the runoff is prevented, so
would be the pollutant loads carried by the runoff. Sediment loads from both watershed and in-stream sources (from bank erosion) would also be decreased. Finally, the in-stream temperature characteristics would not be as affected by storm flow, since storm flow generally increases stream temperatures in urbanizing streams. Any reduction in the trend of increasing stream temperatures would also improve habitat for cool-water fish (Steffy, 2003).

4.6 Summary

In this chapter, the HEC-HMS model was used to further explain the role that the detention basins of Valley Creek watershed play on stormwater flow. The first point explored was how the amount of area attenuated by detention basins affects the watershed-wide storm hydrographs. The attenuated areas of the watershed were modeled separately to better illustrate their cumulative effect. Then an analysis of historic rainfall data was used to explain the design rational of these basins. This analysis showed that the hypothetical storms the detention basins are designed for represent only 3% of the annual precipitation volume. In Chapter 3 it was shown that the detention basins can actually increase peak flow rate. The third section of this chapter explained how extended receding limbs produced by detention basins can coincide with watershed-wide peak flow rates and actually increase peak flow rates. In the final two sections of this chapter, alternate management scenarios were simulated with the model. The first hypothetical scenario was low-flow orifice modifications. There were two different scenarios explored: modification of all detention basins with a 4-inch low-flow orifice, and such modification of only the upstream detention basins. The results from this
modeling do not show either technique to be an effective means for decreasing watershed-wide peak flow rates. The second alternate management technique explored was a volume-based approach of promoting stormwater infiltration. The results from this section showed significant peak flow rate reductions from this practice. These results were then used in conjunction with historic rainfall data to quantify potential base flow increases that could be realized through an infiltration scenario over the course of a year.
Chapter 5. Summary and Conclusions

This thesis has explored the effect of a network of detention basins on the watershed-wide stormwater flow regime of Valley Creek watershed. While the release-rate method of stormwater management has become widespread, very little research has been published on the topic. Some early investigators have shown that the effectiveness of detention basins decreases at the watershed scale (Traver and Chadderton, 1983) and that a “regional”, or watershed scale approach should be used to properly manage stormwater runoff (McCuen, 1979). However, to the author’s knowledge this is the first undertaking of a real-world analysis of an existing network of detention basins.

A comprehensive survey of stormwater management facilities in the 24 square-mile Valley Creek watershed in Chester County, PA was conducted. From this survey the hydraulic characteristics of each detention basin were used to create storage vs. outflow curves for all detention basins. This paired data set was used as input by the reservoir elements in the HEC-HMS hydrologic model. The remainder of the HEC-HMS model input was assembled using GIS data in the HEC-GeoHMS V1.1 extension in ArcView 3.3. Six precipitation events were measured using a network of recording rain gages. This rainfall data along with observed flow data from the USGS stream gage, DEM, and soils data, provided a solid dataset for model calibration. The HEC-HMS model was used to simulate both measured and hypothetical storm events on the network of detention basins. The results from these simulations showed that the detention basins have little attenuating effect on the stormwater flow regime. Contrary to their design purpose, it was found that the network of detention basins can actually increase
watershed-wide peak flow rates. This predicted increase occurred in two of the six measured storm events. An analysis of local historic rainfall data was used to demonstrate that the results found in these six events applies to the majority of the annual precipitation volume. It was shown that the detention basins have been designed for only 3% of the yearly precipitation volume and therefore have little effect on the watershed wide flow regime, and in some cases, little effect on the basin’s local stormwater flow. These results indicate that the watershed-wide release-rate method of stormwater management is not an effective means of limiting peak flow rates. The only function the detention basins may serve is to attenuate peak flows from large storms, thereby protecting properties located directly downstream.

The model was then modified to further explain the initial results. The model was also changed to the answer several “what if” questions for watershed-wide stormwater management. These results demonstrated that a volume-based approach to stormwater management was the only effective method of peak flow reduction. This approach appears to be the only method that can successfully fulfill the “Purpose and Policy” goals outlined in Pennsylvania’s Act 167 of 1978. A volume based management plan obeys the site’s predevelopment hydrologic cycle and “…preserve(s) to the maximum extent practical the natural storm water runoff regimes and natural course, current and cross-section of water of the Commonwealth; and protect(s) and conserve(s) ground waters and ground-water recharge areas,” as outlined in Act 176.
List of References


Cahill Associates (2001). "Evaluation of Potential Environmental Impacts of Vanguard Site Improvements on Little Valley Creek, Schuylkill River Basin, Chester County", West Chester, PA.


East Whiteland Township (2000). Stormwater Drainage; Erosion and Sedimentation Control, Frazer, PA.


Pennsylvania Code, Title 25, Chapter 93.


Pierce County. (2000) “Impervious Surface Percentages for Different Land Use Types” Pierce County, WA.


U.S. Census Bureau (1715-2000) Washington DC.
Appendix A. Sample Detention Basin Checklist

Valley Creek Watershed Detention Basin Inspection Chart

<table>
<thead>
<tr>
<th>Inspector:</th>
<th>CHE CAUJ</th>
<th>Date:</th>
<th>07/01/00</th>
<th>GPS ID #:</th>
<th>051</th>
</tr>
</thead>
<tbody>
<tr>
<td>Time / Number of Basin of Day:</td>
<td>8</td>
<td>No of Pictures Taken:</td>
<td>7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Location:</td>
<td>SWEDENFORD RD &amp; WINTERFORD</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Receiving Waters:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Name of Development / Complex:</td>
<td>VERY LOW DENSITY HOMES</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Size:

<table>
<thead>
<tr>
<th>Length (top):</th>
<th>20</th>
<th>ft</th>
<th>Length (boi):</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Width (top):</td>
<td></td>
<td>ft</td>
<td>Width (boi):</td>
<td></td>
</tr>
<tr>
<td>Other (top):</td>
<td></td>
<td>ft</td>
<td>Other (boi):</td>
<td></td>
</tr>
<tr>
<td>Depth:</td>
<td></td>
<td>ft</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slopesc:</td>
<td>2:1:6:5</td>
<td>(see back)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Volume:</td>
<td>60</td>
<td>ft³</td>
<td>Height of Instrument:</td>
<td>ft (top of berm)</td>
</tr>
</tbody>
</table>

Vegetation: CUT GRASS

Overall Condition / Comments: WELL MAINTAINED

Inlet Structures:

<table>
<thead>
<tr>
<th>Number:</th>
<th>1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Types:</td>
<td>RCP</td>
</tr>
<tr>
<td>Condition:</td>
<td>GOOD</td>
</tr>
</tbody>
</table>

Outlet Structure:

Description: STANDARD BOX TYPE

Outlet 1:

<table>
<thead>
<tr>
<th>Type:</th>
<th>ORIFICE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elevation:</td>
<td>0</td>
</tr>
<tr>
<td>Condition:</td>
<td>GOOD, CLEAR</td>
</tr>
</tbody>
</table>

Outlet 2:

<table>
<thead>
<tr>
<th>Type:</th>
<th>ORIFICE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elevation:</td>
<td>4</td>
</tr>
<tr>
<td>Condition:</td>
<td>GOOD, CLEAR</td>
</tr>
</tbody>
</table>

Outlet 3:

<table>
<thead>
<tr>
<th>Type: WEIR (GRATE)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Elevation: 60</td>
</tr>
<tr>
<td>Condition: GOOD, CLEAR</td>
</tr>
</tbody>
</table>

Outlet 4:

<table>
<thead>
<tr>
<th>Type:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dimensions:</td>
</tr>
<tr>
<td>Condition:</td>
</tr>
<tr>
<td>Distance to stream:</td>
</tr>
<tr>
<td>Low Flow Channel:</td>
</tr>
<tr>
<td>Outlet Protection:</td>
</tr>
</tbody>
</table>

General Comments: VERY LOW DENSITY HOMES
Appendix B. Control Files for GageInterp

*GageInterp control file for Valley Creek Watershed 14SEP2001
TimeStart: 14SEP2001, 0540
TimeEnd: 14SEP2001, 1625
TimeStep: 5

OutFile: 091401GD.DSS
OutPath: /091401/RGS/PRECIP-INC///GRIDDED/

weight: id2w

range: UNLIMITED

GridType: SHG
CellSize: 100
GridBounds: -753606 400112 -752644 400523

DSSFile: 091401GA.DSS

gage: RG1, -753606, 400302
path: /091401/RG1/PRECIP-INC/14SEP2001/5MIN/GAGE/
timezone: EST, Y

gage: RG2, -753152, 400112
path: /091401/RG2/PRECIP-INC/14SEP2001/5MIN/GAGE/
timezone: EST, Y

gage: RG3, -753108, 400403
path: /091401/RG3/PRECIP-INC/14SEP2001/5MIN/GAGE/
timezone: EST, Y

gage: RG4, -752758, 400235
path: /091401/RG4/PRECIP-INC/14SEP2001/5MIN/GAGE/
timezone: EST, Y

gage: RG5, -752644, 400523
path: /091401/RG5/PRECIP-INC/14SEP2001/5MIN/GAGE/
timezone: EST, Y
*gageInterp control file for Valley Creek Watershed 20SEP2001

times however. UTC = EST + 5hr except during DST (4hr) DST is from APR-OCT
TimeStart: 20SEP2001, 0950
TimeEnd: 21SEP2001, 0920
TimeStep: 5

OutFile: 092001GD.DSS
OutPath: /092001/RGS/PRECIP-INC///GRIDDED/

weight: id2w

range: UNLIMITED

GridType: SHG
CellSize: 100
GridBounds: -753606 400112 -752644 400523

DSSFile: 092001GA.DSS

gage: RG1, -753606, 400302
path: /092001/RG1/PRECIP-INC/20SEP2001/5MIN/GAGE/
timezone: EST, Y

gage: RG2, -753152, 400112
path: /092001/RG2/PRECIP-INC/20SEP2001/5MIN/GAGE/
timezone: EST, Y

gage: RG3, -753108, 400403
path: /092001/RG3/PRECIP-INC/20SEP2001/5MIN/GAGE/
timezone: EST, Y

gage: RG4, -752758, 400235
path: /092001/RG4/PRECIP-INC/20SEP2001/5MIN/GAGE/
timezone: EST, Y

gage: RG5, -752644, 400523
path: /092001/RG5/PRECIP-INC/20SEP2001/5MIN/GAGE/
timezone: EST, Y
*gageInterp control file for Valley Creek Watershed 12MAY2002
TimeStart: 12MAY2002, 2210
TimeEnd: 14MAY2002, 2005
TimeStep: 5

OutFile: 051202GD.DSS
OutPath: /051202/RGS/PRECIP-INC///GRIDDED/

weight: id2w

range: UNLIMITED

GridType: SHG
CellSize: 100
GridBounds: -753606 400112 -752630 400523

DSSFile: 051202GA.DSS

gage: RG1, -752630, 400452
path: /051202/RG1/PRECIP-INC/12MAY2002/5MIN/GAGE/
timezone: EST, Y

gage: RG2, -752928, 400523
path: /051202/RG2/PRECIP-INC/12MAY2002/5MIN/GAGE/
timezone: EST, Y

gage: RG3, -753108, 400403
path: /051202/RG3/PRECIP-INC/12MAY2002/5MIN/GAGE/
timezone: EST, Y

gage: RG5, -753523, 400307
path: /051202/RG5/PRECIP-INC/12MAY2002/5MIN/GAGE/
timezone: EST, Y

gage: RG6, -753606, 400112
path: /051202/RG6/PRECIP-INC/12MAY2002/5MIN/GAGE/
timezone: EST, Y

gage: RG7, -753152, 400243
path: /051202/RG7/PRECIP-INC/12MAY2002/5MIN/GAGE/
timezone: EST, Y
*gageInterp control file for Valley Creek Watershed 06JUN2002
TimeStart: 06JUN2002, 1945
TimeEnd: 07JUN2002, 1325
TimeStep: 5

OutFile: 060602GD.DSS
OutPath: /060602/RGS/PRECIP-INC///GRIDDED/

weight: id2w

range: UNLIMITED

GridType: SHG
CellSize: 100
GridBounds: -753606 400112 -752630 400523

DSSFile: 060602GA.DSS

gage: RG1, -753152, 400243
path: /060602/RG1/PRECIP-INC/06JUN2002/5MIN/GAGE/
timezone: EST, Y

gage: RG2, -753606, 400112
path: /060602/RG2/PRECIP-INC/06JUN2002/5MIN/GAGE/
timezone: EST, Y

gage: RG5, -753108, 400403
path: /060602/RG5/PRECIP-INC/06JUN2002/5MIN/GAGE/
timezone: EST, Y

gage: RG6, -752928, 400523
path: /060602/RG6/PRECIP-INC/06JUN2002/5MIN/GAGE/
timezone: EST, Y

gage: RG7, -752630, 400452
path: /060602/RG7/PRECIP-INC/06JUN2002/5MIN/GAGE/
timezone: EST, Y
gageInterp control file for Valley Creek Watershed 24AUG2002

TimeStart: 24AUG2002, 1025
TimeEnd: 25AUG2002, 0035
TimeStep: 5

OutFile: 082402GD.DSS
OutPath: /082402/RGS/PRECIP-INC///GRIDDED/

weight: id2w

range: UNLIMITED

GridType: SHG
CellSize: 100
GridBounds: -753606 400112 -752630 400523

DSSFile: 082402GA.DSS

gage: RG1, -753152, 400243
path: /082402/RG1/PRECIP-INC/24AUG2002/5MIN/GAGE/
timezone: EST, Y

gage: RG2, -753606, 400112
path: /082402/RG2/PRECIP-INC/24AUG2002/5MIN/GAGE/
timezone: EST, Y

gage: RG5, -753523, 400307
path: /082402/RG5/PRECIP-INC/24AUG2002/5MIN/GAGE/
timezone: EST, Y

gage: RG6, -753108, 400403
path: /082402/RG6/PRECIP-INC/24AUG2002/5MIN/GAGE/
timezone: EST, Y

gage: RG7, -752928, 400523
path: /082402/RG7/PRECIP-INC/24AUG2002/5MIN/GAGE/
timezone: EST, Y

gage: RG8, -752630, 400452
path: /082402/RG8/PRECIP-INC/24AUG2002/5MIN/GAGE/
timezone: EST, Y
gageInterp control file for Valley Creek Watershed 28AUG2002
TimeStart: 28AUG2002, 2325
TimeEnd: 29AUG2002, 1225
TimeStep: 5

OutFile: 082802GD.DSS
OutPath: /082802/RGS/PRECIP-INC///GRIDDED/

weight: id2w

range: UNLIMITED

GridType: SHG
CellSize: 100
GridBounds: -753606 400112 -752630 400523

DSSFile: 082802GA.DSS

gage: RG1, -753152, 400243
path: /082802/RG1/PRECIP-INC/28AUG2002/5MIN/GAGE/
timezone: EST, Y

gage: RG2, -753606, 400112
path: /082802/RG2/PRECIP-INC/28AUG2002/5MIN/GAGE/
timezone: EST, Y

gage: RG5, -753523, 400307
path: /082802/RG5/PRECIP-INC/28AUG2002/5MIN/GAGE/
timezone: EST, Y

gage: RG6, -753108, 400403
path: /082802/RG6/PRECIP-INC/28AUG2002/5MIN/GAGE/
timezone: EST, Y

gage: RG7, -752928, 400523
path: /082802/RG7/PRECIP-INC/28AUG2002/5MIN/GAGE/
timezone: EST, Y

gage: RG8, -752630, 400452
path: /082802/RG8/PRECIP-INC/28AUG2002/5MIN/GAGE/
timezone: EST, Y
Appendix C. Example Control File for DSSTS

091401GA.DSS
/091401/RG1/PRECIP-INC/14SEP2001/5MIN/GAGE/
MM
PER-CUM
14SEP2001,0140
(precipitation values omitted)
END
/091401/RG2/PRECIP-INC/14SEP2001/5MIN/GAGE/
MM
PER-CUM
14SEP2001,0140
(precipitation values omitted)
END
/091401/RG3/PRECIP-INC/14SEP2001/5MIN/GAGE/
MM
PER-CUM
14SEP2001,0140
(precipitation values omitted)
END
/091401/RG4/PRECIP-INC/14SEP2001/5MIN/GAGE/
MM
PER-CUM
14SEP2001,0140
(precipitation values omitted)
END
/091401/RG5/PRECIP-INC/14SEP2001/5MIN/GAGE/
MM
PER-CUM
14SEP2001,0140
(precipitation values omitted)
END
Appendix D. Grid Cell Parameter File

PARAMETER ORDER: Xcoord Ycoord TravelLength Area ScsCn
END:
SUBBASIN: BASIN001
GRIDCELL: 17138 20765 0.472649 0.000767 0
GRIDCELL: 17138 20766 0.318108 0.004438 0
GRIDCELL: 17139 20765 0.179577 0.000158 0
GRIDCELL: 17139 20766 0.179577 0.003506 0
END:
SUBBASIN: BASIN002
GRIDCELL: 17136 20767 0.252340 0.002825 0
GRIDCELL: 17137 20767 0.098583 0.001360 0
GRIDCELL: 17136 20777 1.442292 0.000000 0
GRIDCELL: 17136 20778 1.545212 0.000439 0
GRIDCELL: 17137 20777 1.730162 0.004403 0
GRIDCELL: 17137 20778 1.442292 0.007631 0
GRIDCELL: 17138 20777 1.318964 0.002756 0
GRIDCELL: 17138 20778 1.129561 0.009484 0
GRIDCELL: 17138 20777 1.060917 0.000068 0
GRIDCELL: 17139 20777 1.442292 0.000000 0
GRIDCELL: 17139 20778 1.545212 0.000439 0
GRIDCELL: 17139 20779 0.865039 0.009996 0
GRIDCELL: 17139 20779 0.637625 0.001474 0
GRIDCELL: 17140 20778 0.577534 0.008913 0
GRIDCELL: 17140 20779 0.333662 0.004449 0
GRIDCELL: 17141 20778 0.333662 0.004449 0
GRIDCELL: 17141 20779 0.128940 0.002453 0
END:
(many lines omitted)
Appendix E. Distributed Basin Model

Basin: 091401TB
Description: 091401 BASINS THESIS
Last Modified Date: 24 March 2003
Last Modified Time: 10:21:35
Version: 2.2.0
Default DSS File Name: C:\hmsproj\THESIS\091401T\091401T.dss
Unit System: English
Map File: c:\hmsproj\THESIS\THESIS.map
Grid Cell File: c:\hmsproj\THESIS\THESIS.mod
End:

Junction: USGSOUTLET
Canvas X: 2609046.842
Canvas Y: 280019.237
Label X: 16
Label Y: 0
Observed Hydrograph Gage: USGSOUTLET
End:

Junction: BA019
Canvas X: 2586744.092
Canvas Y: 277120.862
Label X: 16
Label Y: 0
Downstream: R230
End:

(many junctions omitted)

Gridded Subbasin: R70W70
Canvas X: 2607081.842
Canvas Y: 279282.362
Label X: 16
Label Y: 0
Area: 1.096000
Downstream: USGSOUTLET

LossRate: Green and Ampt
Percent Impervious Area: 4.28
Initial Loss: 0.15
Moisture Deficit: 0.47
Wetting Front Suction: 15.19
Hydraulic Conductivity: 0.224

Transform: Modified Clark
Time of Concentration: 0.22
Storage Coefficient: 0.30

Baseflow: Monthly Constant

Monthly rate:
Monthly rate:
Monthly rate:
Monthly rate:
Monthly rate:
Monthly rate:
Monthly rate: 8
Monthly rate:
Monthly rate:
Monthly rate:
Monthly rate:

End:

(many sub-basins omitted)

Reach: R70
Canvas X: 2609046.842
Canvas Y: 280019.237
From Canvas X: 2608850.342
From Canvas Y: 278823.862
Label X: 16
Label Y: 0
Downstream: USGSOUTLET

Route: Muskingum
Muskingum K: 0.332
Muskingum X: 0.2
Muskingum Steps: 2

End:

(many reaches omitted)

Reservoir: OUTLET001
Canvas X: 2576722.406
Canvas Y: 265985.516
Label X: 16
Label Y: 0
Downstream: BA001

Route: Modified Puls
Routing Curve: Storage-Outflow
Initial Outflow: 0
Routing Table in DSS: Yes
Storage-Outflow Table: OUTLET001
End:

(many reservoirs omitted)

Default Attributes:
  Default Basin Unit System: English
  Default Meteorology Unit System: SI
  Default Loss Rate: Green and Ampt
  Default Transform: Modified Clark
  Default Baseflow: None
  Default Route: Muskingum
  Enable Flow Ratio: No
  Enable Evapotranspiration: No
  Compute Local Flow At Junctions: No
  Missing Flow To Zero: No

End: