RAINFALL-RUNOFF MODELING OF SMALL URBAN WATERSHEDS IN MILWAUKEE, WISCONSIN

by

Aaron C. Volkening

A Thesis Submitted in

Partial Fulfillment of the

Requirements for the Degree of

Master of Science

in Engineering

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ABSTRACT

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The University of Wisconsin-Milwaukee, 2004
Under the supervision of Dr. Hector R. Bravo

Hydrologic models are used by engineers, scientists and planners to simulate the rainfall-runoff process for small urban watersheds. These rainfall-runoff models are commonly not calibrated, primarily because of a lack of data on observed rainfall and watershed runoff. This study evaluates the performance of uncalibrated and calibrated rainfall-runoff models. Three commonly used rainfall-runoff models were applied to two small urban watersheds in Milwaukee, Wisconsin – the Lyons Park Creek watershed and the Eighteenth Street Storm Sewer watershed. The United States Geological Survey measured rainfall and runoff in each of these watersheds during 2002 and 2003. The three models applied to the watersheds were the Rational Method, the Storm Water Management Model (SWMM), and the Soil Conservation Service (SCS) model. First, uncalibrated models were constructed using default model parameters or parameters estimated from literature review. The uncalibrated models were used to simulate ten recorded rainfall events in each watershed. The peak flows and runoff volumes simulated by the uncalibrated models were compared to observed data, and the error associated with each model assessed. The models were then calibrated to the observed runoff data. The SWMM models appeared to perform the best, while the SCS models were generally the

poorest performers. Based on the results of this study, as well as results reported by other modelers, recommendations are made on the selection and use of uncalibrated rainfall-runoff models.

Major Professor Date

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LIST OF ABBREVIATIONS

CASC2D:	Cascade 2-Dimensional runoff model
cfs:	cubic feet per second
CUHP:	Colorado Unit Hydrograph Procedure
DR3M:	Distributed Rainfall Routing Runoff Model
GIS:	Geographic Information System
HEC-1:	Hydrologic Engineering Center – 1 model
HSPF:	Hydrologic Simulation Program – Fortran
MMSD:	Milwaukee Metropolitan Sewerage District
NRCS:	Natural Resources Conservation Service
PRMS:	Precipitation Runoff Modeling System
PSRM:	Penn State Runoff Model
SCS:	Soil Conservation Service
SEWRPC:	Southeastern Wisconsin Regional Planning Commission
SWMM:	Storm Water Management Model
USGS:	United States Geological Survey

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I would also like to thank my mentors and colleagues at Earth Tech for their encouragement and patience while I pursued my graduate education.

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CHAPTER 1: INTRODUCTION

The Problem of Uncalibrated Rainfall-Runoff Models

Rainfall-runoff models are used by engineers, scientists and planners to answer the question: if a certain rainfall occurred, how much stormwater runoff would be generated? Rainfall-runoff models attempt to answer this question by simulating important components of the hydrologic cycle such as precipitation, evapotranspiration, infiltration and runoff.

There are many uses for rainfall-runoff models. Perhaps the most traditional use of rainfall-runoff models is to assist in the planning and design of stormwater drainage infrastructure such as storm sewers, surface inlets, culverts and manmade channels. Rainfall-runoff models are used to predict flows in manmade channels and natural streams and rivers during floods, so that floodplains can be mapped and managed. The hydrologic effects of land development, such as increases in runoff caused by additional rooftops and pavement, are often analyzed using rainfall-runoff models. If runoff is collected by combined sewers, which also carry sanitary sewage to a wastewater treatment plant, then estimating runoff rates is important in minimizing sewer overflows and designing the downstream treatment plant. In recent years, engineers, scientists, and the general public have become increasingly concerned about the poor quality of stormwater runoff, and its impacts on natural water bodies. Rainfall-runoff models play a key role in analyzing this problem. As humans work towards managing all of our water resources as one connected system, rainfall-runoff models can help us understand the hydrologic cycle as a whole. They can help us estimate the amount of groundwater recharge occurring, the amount of surface water available to ecological communities such as fisheries and wetlands, and the amount of surface water that could be stored and used by humans for purposes such as irrigation.

To be useful, a model must adequately represent the real phenomena it is used to simulate. Models are simplifications of reality, and so no model can be expected to be 100% accurate. However, models should be able to reproduce real data and observations, with error that is acceptable to the users of the model. Rainfall-runoff models are no different. How does a modeler determine if a model adequately represents reality? One way is to compare model results to observed data, and if needed adjust the model until the two match. This process of adjusting model inputs, and sometimes the underlying model algorithms and structure, to match observed data is called model calibration.

The most straightforward way of calibrating a rainfall-runoff model would be to measure rainfall occurring over a watershed, and runoff discharging from the watershed. The watershed would be simulated using a rainfall-runoff model, and the runoff predicted by the model would be compared to the runoff actually measured. The model would be adjusted in various ways until a good match between observed and simulated runoff discharge was achieved.

Perhaps surprisingly, many rainfall-runoff models of watersheds are not calibrated in this manner. This is not because rainfall-runoff modelers are lazy. The primary reason for lack of calibration is that real data is seldom available to compare model results to. Rainfall-runoff models are most commonly applied to small watersheds, several square miles or less in area. Although the United States Geological Survey continuously monitors and records water stages and flows for most of the major rivers in Wisconsin (and throughout the nation), only a few small streams are monitored. The vast majority of small watersheds are ungauged. Few government agencies and private companies have the money and staff expertise to conduct stream gauging of their own. Also, as mentioned earlier, rainfall-runoff models are often used to estimate the impact that a land development will have on stormwater runoff. The model must usually be constructed before land development occurs; therefore it would be impossible to measure ahead of time the behavior of runoff after development.

Data collection also requires time. Monitoring a single storm event and calibrating to that event is of little value, because a watershed responds differently to different amounts and patterns of rainfall. Many runoff events should be recorded and used for calibration, ideally over multiple years. Even if money were available, time would often prohibit the installation and monitoring of a gage over a period of several years. Most engineering studies and design projects vary from several months to 2 years in length. It is rare for a hydrologic analysis effort to extend for a longer period. However, two years seems to be about the minimum length of time required to design a monitoring system, install it, and record enough data for use.

Observed rainfall data is also required to calibrate hydrologic models. Rain gauges are often more common than stream gauges, but there are still large distances between rain gauges. Because rainfall is unevenly distributed, especially the convective thunderstorms that usually cause the highest flows on small watersheds in Wisconsin, the rainfall pattern recorded by one rain gauge may be quite different from rainfall occurring just a few miles away. Radar and satellite measurements are increasingly being used to try to fill in the gaps, but that science is still developing.

In many cases, model calibration for a specific watershed is not warranted. Many engineering designs have a large factor of safety included, which is assumed to compensate for the uncertainty in the hydrologic modeling. Even when no real data is available for calibration, most engineers compare their model results to their past experience and the work of others, to see if the results seem “in the ballpark”. Relative to the construction cost of drainage infrastructure, and the level of damage that would occur if the capacity of the infrastructure was exceeded, it is often uneconomical to do detailed hydrologic modeling and calibration.

For these reasons, calibrated rainfall-runoff modeling of small watersheds is rarely done. Without large technological or economic changes, we will never be able to collect accurate rainfall and flow data for every watershed, and use this data to develop calibrated hydrologic models. For reasons discussed above, that is probably not needed.

However, models should not be used blindly. A model user must be able to assess whether the model is providing reasonable results. If no watershed runoff data is available for comparison, assessing model output can be difficult. This problem is compounded by the fact that, like many other fields, models of the rainfall-runoff process appear to be proliferating rapidly. Every year journal articles are published about new theoretical models, and software vendors are releasing new model applications. These problems often lead people involved in rainfall-runoff modeling to ask questions such as the following:

- I'm beginning a modeling study and numerous potential models are available to me. Which model should be selected for use?

- I applied several different models to the same watershed and got different results. Which model is right? Which set of results should I use?

- Although no quantitative data is available for calibration, the model results seem unrealistic. Perhaps the model is predicting a depth of flooding or a rate of flow that, based on anecdotal evidence or someone's gut feeling, doesn't sound right. Now what?

- Model results are being used to plan and design expensive public works or environmental projects. If the model was not calibrated to watershed conditions, how do the project owners know the project will work? How do they know they're not wasting money on a problem that exists in the model but not in real life? Conversely, how can owners be assured they're not spending scarce resources to implement a project that still won't solve the flooding or other water management problem?

Without model calibration, it can be difficult to answer the above questions. If the model cannot be calibrated, the model builder selects input parameters and model structure based on past experience or engineering/scientific literature, and hopes that his or her model reflects reality. Many engineers believe that commonly used rainfall-runoff models usually overestimate peak flows and runoff volumes. Several research studies on rainfall-runoff modeling tentatively support this conclusion, as will be discussed later. Therefore, typical use of rainfall-runoff models probably results in a conservative design and provides a factor of safety, most of the time.

But as the budgets of municipalities, counties and states get tighter, it becomes more important that the cost of excessive overdesign of infrastructure be avoided. There are environmental consequences of unnecessary overdesign also. In infrastructure design, more land must be devoted to concrete pipes and structures. Overestimating runoff rates for a stream restoration might lead to heavy rip-rap or concrete blocks being used to armor the stream, while in reality more ecologically friendly methods such as natural vegetation would have worked. And what if runoff peaks and volumes are significantly underestimated rather than overestimated? Infrastructure will be insufficient, and floodwaters may damage property and threaten human safety.

The above discussion is not intended to suggest that conservatism in the estimation of runoff peaks and volumes is never needed. Certainly, a lot depends on the use of the model output. Overall, what appears to be needed is more information on model uncertainty, and on the envelope of runoff that is likely to be experienced. Academic modeling studies usually report on the uncertainty of the modeling, the expected error and imprecision. In contrast, engineering designs and plans rarely discuss model uncertainty – numbers are given as an absolute fact. If a factor of safety is being applied, wouldn't it be nice to know what that factor of safety is?

Contribution of this Study

It is very unlikely that model calibration could ever be done for all small watersheds. But model users can learn from real data measured by others, and model calibration done by others. In this study, rainfall-runoff models were applied to two small urban watersheds in the City of Milwaukee. For a period of two years, the United States Geological Survey (USGS) collected continuous discharge and rainfall data for these watersheds. Thus, results from rainfall-runoff models could be compared to observed runoff for these watersheds, and the models then calibrated. The performance of each model, both before and after calibration, was assessed. Hopefully this effort will contribute to a better understanding of how rainfall-runoff models should be applied to small urban watersheds, and assist modelers in answering the questions posed above.

A detailed literature review of previous modeling efforts for small urban watersheds was conducted as part of this study. Unfortunately, most of the documentation typically read by model users (the users manuals and technical references produced by the developers of the model) contains few or no examples of application of the model to real watersheds. For example, the official manual for application of the SCS rainfall-runoff model to urban watersheds (Soil Conservation Service, 1986) contains no examples of the comparison of model output to real data, or model calibration. Many studies have been done comparing observed data to results of uncalibrated models, or reporting on model calibration. But these studies are typically reported in journals and conference proceedings that are often not consulted in the offices of government agencies and engineering firms.

The literature review performed as part of this study is certainly not all-inclusive. The amount of literature on rainfall-runoff modeling is large and continues to grow. Assembling a comprehensive bibliography would be very difficult and time-consuming. However, this study summarizes many of the available references which report on the performance of uncalibrated and calibrated rainfall-runoff models. Even if many modelers do not have the opportunity to perform calibrated modeling on their own watersheds, much could be learned from what others have observed.

Overview of the Study

In this study, three commonly used rainfall-runoff models were applied to two small urban watersheds on the south side of the City of Milwaukee. These watersheds are:

- Lyons Park Creek (0.46 square mile drainage area)
- Eighteenth Street storm sewer (0.10 square mile drainage area)

For each of these watersheds, the USGS collected continuous discharge and rainfall records during the non-winter periods of 2002 and 2003. During wet weather flows, flow and rainfall were recorded every 60 seconds. Thus, highly detailed data is available for model calibration. Furthermore, the period of data collection included several large runoff events. During 2002, a rainfall event occurred in the Lyons Park Creek watershed that had an estimated average recurrence interval of 50 years.

This event offered an unusual opportunity to observe model performance and perform calibration for the type of runoff event often used to design stormwater management and flood management infrastructure. Most of this infrastructure is designed for flood flows with recurrence intervals of ten to 100 years. However, flow monitoring of small urban watersheds, already rare to begin with, is typically done for only a few years. It is uncommon to measure rainfall and runoff data for major flood events on small watersheds. For example, if a flow monitoring study lasts two years, there is a nineteen percent chance that a flood flow with a recurrence interval of 10 years will occur during the monitoring period. Therefore, even if a rainfall-runoff model can be calibrated, often the calibration is only for smaller rainfall events, and the extrapolation of model performance to large flood events typically used for engineering design is questionable. Therefore, this study offers a somewhat rare opportunity to see how models represent a large observed runoff event.

Three of the modeling methods most commonly used for engineering design were applied in this study. The rainfall-runoff modeling methods applied to these watersheds are the:

- Storm Water Management Model (SWMM)
- Soil Conservation Service (SCS) method
- Rational Method

Using each of these methods, models were developed for the two watersheds. The models were constructed using procedures recommended in model users manuals and commonly available engineering manuals, so that these models represent typical engineering practice. The largest rainfall and runoff events occurring during 2002 and 2003 on each watershed were then selected, resulting in approximately ten storm events for each watershed. The recorded rainfall data from these historic storm events were input into the models, and the rainfall-runoff process simulated. The observed flow data was not used in any way to construct the models, so the models could be considered uncalibrated. The output from the uncalibrated models (particularly peak flows and total runoff volumes for each storm event) was then compared to observed flows. At this stage, the following questions could be addressed:

- How closely did the uncalibrated models match the observed flow data, particularly the peak flows and total volumes? Were any biases or patterns noticeable?
- How similar were the results obtained from the different models?

The answers to these questions provide some insight on the accuracy and precision of uncalibrated rainfall-runoff models, which constitute the vast majority of rainfall-runoff models that are used in engineering practice.

After the uncalibrated models were evaluated, the models were calibrated to the observed data in an attempt to obtain better matches to observed peak flows and runoff volumes. The storm events which occurred in 2002 were used for calibration. Following calibration, the 2003 storm events were simulated, to see if the calibrated models could reproduce additional storm events better than the uncalibrated models. This process of simulating additional storms not included in the calibration effort is called model validation. After model calibration and validation, the following questions were addressed:

- How well could be models be calibrated? Could some models be better calibrated than others? Even after calibration, how much uncertainty exists in model output?
- What can be learned from the required adjustments to model input parameters during calibration? Are any biases apparent in default or typical values for input parameters obtained from software user manuals and engineering handbooks (for example, the use of a certain value recommended by the model documentation consistently overpredicts or underpredicts peak flows)? Could calibrated model parameters be used for modeling of ungaged watersheds also?

Hopefully, addressing the above questions will benefit users of rainfall-runoff models, by providing some insight on how well models can replicate real stormwater flow.

CHAPTER 2: BACKGROUND AND LITERATURE REVIEW

Rainfall-Runoff Models

The primary purpose of a rainfall-runoff model is to predict the stormwater runoff that would be generated by a certain rainfall. As discussed in the Introduction, there are many uses for rainfall-runoff models: design of drainage infrastructure, analysis of runoff quality problems, planning and design of wastewater collection systems, floodplain management, and water budgeting, to name a few. All of these types of problems require estimates of stormwater runoff. And rainfall is what drives stormwater runoff.

For example, one of the primary reasons for predicting runoff amounts is to predict flows in natural streams and manmade conduits during floods. Floods can be caused by numerous factors – often more than one of these factors work together. Heavy rainfall is one of the primary causes of flooding. One intense burst of rainfall can cause flooding, especially in small watersheds. Floods on large watersheds are more likely to be caused by long periods of heavy rainfall, such as the floods experienced in southeastern Wisconsin in May 2004. Floods can also be caused by the melting of large amounts of snow, and by rain falling on snow or frozen ground. Sometimes floods are caused or exacerbated by deficiencies in the drainage system: sewers, culverts, manmade channels or natural watercourses that cannot convey the runoff, or drainage system components that become plugged or blocked.

In small urban areas, floods are most often caused by intense, relatively short rainfalls. This type of rainfall is generated by convective thunderstorms, usually during the summer months. This type of event can be simulated using a rainfall-runoff model.

Rainfall-runoff modeling is not the only way to predict flood flows. Statistical hydrology offers an alternative. By analyzing historical data on peak flows, it is possible to statistically predict how often certain flows are likely to occur, without considering the rainfall or other processes that generate those flood flows. For watersheds where flows have been measured for a long period, this may be the best method to estimate flood flows, as it is based on observed data for that particular watershed. Observed flood flows may be considered in combination with watershed characteristics, and regression equations developed which relate the two. These regression equations can be extended to ungauged watersheds with similar characteristics to the gauged watersheds used to develop the equations. Regression equations are a popular and useful way to estimate flood flows. However, as references such as Hagen (1994) document, rainfall-runoff modeling is the most popular way to estimate flood flows for small watersheds.

What does a rainfall-runoff model do? Basically, it transforms a rainfall signal into a runoff signal. The signals can be single values, such as a total depth of rainfall or peak intensity of rainfall, which could be transformed into a total volume of runoff or a peak flow of runoff. The signal can be more complex. A pattern of unsteady rainfall over time (known as a hyetograph) can be transformed into a record of runoff flow over time (a hydrograph).

Different rainfall-runoff models do this transformation in different ways. Models range from simple to complex, and empirical to theoretical. Most engineering or scientific hydrology textbooks discuss rainfall-runoff modeling. (for example, see Gupta (1995), Linsley et al. (1975), and McCuen (1998)).

For this investigation, three rainfall-runoff models were used:

- Storm Water Management Model (SWMM)
- Soil Conservation Service (SCS) curve number / unit hydrograph method
- Rational Method

These models were chosen primarily because they are some of the most widely-used rainfall-runoff models in the United States at the present time. Hagen (1994) summarized a survey by the federal government on models being used to estimate flood flows in small urban watersheds. The SCS method (as implemented in two computer programs, TR-55 and TR-20) was used approximately 60% of the time. The Rational method was the second most popular choice, used 20% of the time. SWMM does not appear in this survey. However, this writer's personal experience with consulting and municipal engineering indicates that use of SWMM is becoming quite popular, along with the SCS and Rational methods.

Other models were considered for this study. For example, the Hydrological Simulation Program – Fortran (HSPF) has been used by several agencies to simulate the rainfall-runoff process for larger watersheds in southeastern Wisconsin. HSPF is a fairly complex model that simulates many components of the hydrologic cycle, not just the rainfall-runoff process. The Milwaukee Metropolitan Sewerage District has developed HSPF models for watersheds in their jurisdiction (Camp Dresser & McKee, 2000 a,b,c,d). The Southeastern Wisconsin Regional Planning Commission (SEWRPC) has used HSPF to simulate numerous watersheds in their planning area (for example, see Southeastern Wisconsin Regional Planning Commission, 2003). The Precipitation Runoff Modeling System (PRMS) is similar to HSPF. PRMS was used to simulate runoff from observed rainfall events for an 18 square mile watershed near Madison (Steuer and Hunt, 2001), and to simulate seven watersheds in southeastern Wisconsin ranging from approximately 20 to 190 square miles (Cherkauer, 2004).

HSPF and PRMS are sometimes included in a class of runoff models known as “physically-based” models. Physically-based models attempt to simulate the phenomena that generate runoff by applying physics-based equations that describe the movement of water, rather than relying on an empirical or statistical model. This type of model tends to be much more complex than the Rational Method and the SCS model. SWMM could potentially be considered a physically-based model. The use of a model such as HSPF or PRMS was considered for this study. However, these models are not widely used by consulting or municipal engineers for small watersheds, probably because of the complexity and data needs associated with them.

Rational Method

The Rational Method is the simplest of the models used in this study. Development of the Rational Method in the United States is often credited to Kuichling (1889). The Irish engineer Mulvaney developed a similar procedure in 1850. Developed over 150 years ago, the Rational Method is still in use today. The Rational Method uses the equation:

$$Q = C \cdot I \cdot A \qquad \text{(eqn. 2-1)}$$

where Q = peak flow, cubic feet per second

C = rational method coefficient or runoff coefficient, dimensionless

I = rainfall intensity, inches per hour

A = watershed area, acres

The Water Pollution Control Federation (WPCF) (1969) provides a comprehensive overview of the Rational Method, as do many other hydrology textbooks and engineering manuals. The WPCF manual also includes a table of runoff coefficients that is probably the most commonly used reference for these coefficients. This table has been reprinted in many manuals and texts, such as Viessman and Hammer (1998) and McCuen (1998). SEWRPC (Bauer, 1965) developed runoff coefficients for southeastern Wisconsin that are also used in Wisconsin Department of Transportation (1997) design guidelines. The

SEWRPC documentation provides no indication that these runoff coefficients were calibrated to observed peak flow data.

Despite the long history and widespread use of the Rational Methods, critical evaluations of it are uncommon. Schaake et al. (1967) provide one of the few published evaluations of the Rational Method. Measured rainfall and runoff data from twenty small urban watersheds, ranging in size from 0.2 to 150 acres, were used to apply and calibrate the Rational Method. They found that the Rational runoff coefficient C varied widely from storm to storm, and recommended against using the Rational Method to predict peak flows for actual storms. Rather, they concluded that the Rational Method should be calibrated and used as a statistical method, as a mathematical operation to transform a rainfall rate of a certain frequency into a peak runoff flow with the same frequency. They also found that when experienced modelers were asked to perform uncalibrated Rational Methods applications, by using textbook values and computations for input parameters, the modeled peak flows for a five year recurrence interval varied widely from the peak flow calculated from observed runoff frequencies.

Other researchers have also argued that the Rational Method and other rainfall-runoff models should be viewed as a type of statistical transform, giving a proper frequency distribution of flows, rather than as a deterministic model for correctly simulating individual storms. Their opinion is that, because of the high uncertainty associated with the rainfall-runoff process, it is useless in most cases to attempt to simulate individual real storm events with models. Rather, rainfall-runoff models should adequately simulate the probability distribution of flood flows from a watershed. This view recognizes that most engineering designs are based on probabilities, rather than actual historic events.

A group of researchers in Australia are at the forefront of this approach. Titmarsh et al. (1995) and Pilgrim and Cordery (1993) provide details on a probabilistic approach to using rainfall-runoff models. In particular, they consider the Rational Method and SCS method. These rainfall-runoff model are viewed as statistical transforms. Given rainfall frequency distributions, the model is calibrated to observed flood frequency distributions. Titmarsh et al. (1995) describe how they calibrated Rational Method runoff coefficients and SCS curve numbers for 105 small agricultural catchments in Australia. Textbook values for these model parameters varied widely from those calibrated using the flood frequency distributions. Wong (2002) also called for the application of the Rational Method using a probabilistic approach.

Several studies have been published comparing the Rational Method to other rainfall-runoff models. These studies will be discussed later in this chapter.

SCS Method

The SCS rainfall-runoff method was developed by the Soil Conservation Service (SCS), a branch of the United States Department of Agriculture. (The Soil Conservation Service is now called the Natural Resources Conservation Service, or NRCS). There are two major components to the SCS method: a model for separating precipitation into runoff and infiltration, and a model for transforming the runoff into a downstream hydrograph once it is generated. The SCS rainfall-runoff model is described in detail in their National Engineering Handbook (Natural Resources Conservation Service, various years). Its application to urban watersheds is presented in Technical Release 55 (Soil Conservation Service, 1975,1986), commonly referred to as TR-55. Many hydrology and water resources engineering textbooks also cover the SCS rainfall-runoff model.

For a given rainfall depth, the corresponding runoff depth is computed using the equation:

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

Where Q = Runoff depth, inches

P = Precipitation depth, inches

I_a = Initial abstraction, inches

S = Potential maximum retention, inches

The potential maximum retention S depends on factors such as soil conditions, land use, vegetative cover and antecedent moisture. S is related to another parameter called the curve number, or CN, by the following equation:

$$S = \frac{1000}{CN} - 10 \qquad \text{(eqn. 2-3)}$$

A constant curve number is often assumed for a watershed. In reality, the curve number varies with each storm, because of antecedent moisture and other conditions. References such as TR-55 contain recommended curve number values for a wide range of land uses and soil conditions.

Once the volume of runoff is computed using a curve number, this runoff is transformed to a downstream hydrograph using the unit hydrograph approach. Unit hydrographs are discussed in most hydrology textbooks, such as Linsley et al. (1975) and McCuen (1998). The SCS method uses a standard dimensionless unit hydrograph. The standard unit hydrograph is a composite of recorded hydrographs from many small watersheds. However, the watersheds used to develop the unit hydrograph were rural, not urban.

The SCS rainfall-runoff model was developed internally by a federal government agency, and initially was not subject to much outside peer review. Thus, documentation on how the model was developed is not complete. In more recent years, numerous researchers have published evaluations and reviews of the SCS rainfall-runoff model, such as Hawkins (1975, 1978, 1993), Hawkins et al. (1985), Hjelmfelt (1980, 1991), Mishra and Singh (1999), Muzik (2003) and Ponce and Hawkins (1996). Many of these papers provide some history and background on the development of this method. The above papers focus almost entirely on modeling of rural watersheds.

Richard McCuen is one of the few researchers to study the application of the SCS rainfall-runoff model to urban areas. He has published a series of papers on modeling both urban and rural watersheds using the SCS procedure, such as McCuen (2002), McCuen and Bondelid (1983), and McCuen and Okunola (2002). McCuen et al. (1984) applied the model to 51 small urban watersheds where historical flood frequency data was available. (Rawls et al. (1982) also report results from this study). Two variations of the SCS method, both found in the original release of TR-55 (Soil Conservation Service, 1975), were applied: the graphical method and the chart method. The graphical method is still used in later versions of TR-55, though the chart method is not. The focus of McCuen et al. (1984) is in evaluating the chart method, which will not be discussed here because it is obsolete. The authors do report that the graphical method is essentially unbiased. However, the accuracy, though the authors do not call attention to it, appears to be very poor.

As mentioned earlier, Titmarsh et al. (1995) have developed a probabilistic application of the SCS rainfall-runoff method, calibrating curve numbers to observed flood frequency curves. The SCS model was also included in several studies comparing different models, which will be discussed later in this chapter.

Storm Water Management Model (SWMM)

The Storm Water Management Model, or SWMM, is a widely used computer program for simulating the hydrology, hydraulics and quality of urban runoff. SWMM consists of a number of different modules which simulate different parts of the urban runoff process. SWMM was originally developed by contractors working for the United States

Environmental Protection Agency in the 1960s. Many additions and updates have been released over the years. The original SWMM software, also called EPA-SWMM, remains freely available over the Internet. Several private software vendors sell enhanced versions of SWMM. One of the commercial versions, XP-SWMM, was used for this study.

Two modules of SWMM were primarily used in this study: the Runoff and Extran modules. The Runoff module simulates the hydrologic processes which generate runoff from urban land surfaces. The Extran module allows for detailed hydraulic routing of this runoff through the stormwater conveyance system. The concepts behind each of these modules are discussed below. This discussion is a summary of concepts and equations explained in SWMM documentation such as James et al. (2003), and Huber and Dickinson (1992).

The Runoff module simulates the processes that generate runoff from rainfall. Parameters that describe drainage catchments are input into the model. A time series of precipitation is used, in conjunction with the catchment input parameters, to generate a time series of runoff.

SWMM simulates the catchment surface as a non-linear reservoir. All the inflows and outflows from this reservoir are represented mathematically. The reservoir is divided into an area of impervious surface (pavement, rooftops, etc.) and an area of pervious surface (lawns, woods, fields, landscaped areas). Certain losses occur from rain that falls on each of these surfaces. Before runoff occurs, rain must initially fill a "depression storage" depth. Depression storage represents precipitation that becomes trapped in puddles, cracks, vegetation, etc., and is not available to become runoff. Depression storage can occur on both impervious and pervious surfaces, though it is usually larger for pervious surfaces. On pervious surfaces, water also infiltrates into the ground. SWMM can use either one of two known infiltration models, the Horton model and the Green-Ampt model, to simulate infiltration.

When the depression storage fills and the rate of precipitation exceeds the rate of infiltration, there is a rainfall excess, i_*. The rate of runoff is determined by combining the continuity equation with Manning's equation for free-surface uniform flow.

For each surface, the continuity equation is written as:

$$\frac{dV}{dt} = A\frac{dd}{dt} = A \cdot i^* - Q \qquad \text{(eqn. 2-4)}$$

where $V = A \bullet d$ = volume of water on the surface, ft^3

d = water depth, ft

t = time, seconds

A = surface area, ft^2

i* = rate of rainfall excess, ft/second (rainfall rate minus infiltration & evaporation rate)

Q = outflow rate, cubic feet per second

For a given depth of water on the surface, the outflow rate is calculated using Manning's equation. Because the surface is very wide compared to the depth of water, the hydraulic radius can be represented by the depth of water on the surface. Manning's equation for this case can be written as:

$$Q = W \cdot \frac{1.49}{n}\left(d - d_p\right)^{5/3} S^{1/2} \qquad \text{(eqn. 2-5)}$$

where W = subcatchment width, ft

n = Manning's roughness coefficient

d_p = depth of depression storage, ft

S = subcatchment slope (ft/ft)

By solving each equation for Q and setting the equations equal to each other, the non-linear reservoir equation for the catchment is written as:

$$\frac{dd}{dt} = i * -WCON \cdot \left(d - d_p\right)^{5/3} \qquad \text{(eqn. 2-6)}$$

WCON is a combination of catchment characteristics which remain constant, and is equal to:

$$WCON = \frac{1.49 \cdot W \cdot S^{1/2}}{A \cdot n} \qquad \text{(eqn. 2-7)}$$

By representing d as $(d_1 + d_2)/2$, where d_1 and d_2 represent the depths at the beginning and end of a time step, the non-linear reservoir equation can be rewritten in finite difference form and solved for each time step. SWMM documentation such as Huber and Dickinson (1992) provides additional information on the algorithms used in the Runoff module. Using these procedures, the Runoff module generates a hydrograph for each catchment. Many catchments can be simulated in the same SWMM file.

The other SWMM module used is the Extran module. In the Extran module (called Hydraulics mode in XP-SWMM), runoff is routed through the stormwater drainage system – through sewers, streets, manmade channels and natural streams. Routing is performed by numerically solving the St. Venant equations, which describe the conservation of mass and momentum for unsteady free-surface flow.

Publications on the application and calibration of SWMM for urban watersheds are more common than those for the Rational and SCS methods, perhaps because SWMM was specifically developed for urban watersheds (unlike the SCS method) and is more much complex than the Rational Method. For example, Jewell et al. (1978) reported calibrating SWMM to water quantity and quality data for a 1,000 acre urban watershed in Massachusetts. Zaghloul (1981) tested SWMM on several small urban watersheds with observed rainfall and runoff data. Zaghloul was particularly interested in the effect that different levels of model discretization had on simulation results. His results will be discussed in more detail elsewhere in this report. Baffaut and Delleur (1989) report on an expert system approach for building and calibrating SWMM models. Maalel and Huber (1984) used continuous simulation to calibrate SWMM for four small urban watersheds in Florida.

Two additional sources of information of rainfall-runoff modeling, especially the use of SWMM, should be mentioned. Every year, a conference on modeling of urban water systems is held in Ontario. This conference evolved out of annual meetings of the SWMM users group, and the papers presented at this conference still tend to focus on SWMM. Conference proceedings are published in book form. Recent published proceedings include *Practical Modeling of Urban Water Systems* (2003) and Best *Modeling Practices for Urban Water Systems* (2002). Numerous papers on the application of SWMM, most on specific case studies and model examples, can be found in these proceedings.

Also, every couple years an International Conference on Urban Drainage is held. The proceedings include a wide variety of papers on stormwater runoff, on topics such as hydrology, hydraulics, modeling, water quality, management, design and maintenance. The Ninth International Conference on Urban Drainage was held in September 2002 in Portland, Oregon. The Eighth International Conference on Urban Storm Drainage was held in Sydney, Australia in August 1999. Several papers from this series of conferences are referenced in this report; the reader may find other relevant publications on rainfall-runoff modeling in this series also.

Performance of Uncalibrated Runoff Models

As discussed earlier in this report, calibration of rainfall-runoff models for specific watersheds is relatively uncommon, because of the scarcity of runoff data. Therefore, information is needed about the error and bias that is likely to be present in uncalibrated runoff models. Several researchers have investigated this problem by applying uncalibrated rainfall-runoff models to watersheds where runoff discharges have been measured, and comparing the model results to the observed data. Several of these studies are summarized below.

Trommer et al. (1996) applied five modeling techniques to 14 small watersheds in west-central Florida. The watersheds ranged in size from 0.14 to 15.2 square miles, with a median size of approximately 1.2 square miles. Six of the watersheds contained predominantly urban land use, five were predominantly natural/rural, and three were classified as mixed. Observed rainfall and runoff data from 62 storm events were available. The models used were the Rational Method, USGS regional regression equations, TR-20, HEC-1, and SWMM. The TR-20 and HEC-1 models both used the SCS curve number/unit hydrograph runoff method. Within the SWMM model, two different infiltration models were used – the Horton model and the Green-Ampt model.

Initially, models were constructed using accepted procedures for each method – no use was made of the recorded data. Then, rainfall from each historical storm was input into the models, to generate a simulated hydrograph. Runoff volumes and peak flows from these hydrographs were compared to observed runoff measured at USGS gauges. The following table summarizes how these uncalibrated models performed when applied to the six urban Florida watersheds.

Table 2-1
Performance of Uncalibrated Models in Trommer et al. (1996)

Model	Average percent error for:	
	Peak Flow	Runoff Volume
Rational Method	70%	Not simulated
USGS Regression Equations	-25%	-32%
TR-20	12%	31%
HEC-1	75%	25%
SWMM (Green-Ampt infiltration)	20%	-28%
SWMM (Horton infiltration)	20%	-21%

The TR-20 appears to predict peak flow the most accurately for these watersheds. The SWMM models and regression equations also are fairly good at predicting peak flows, while the Rational Method and HEC-1 overpredict peak flows by a large amount. It is interesting to note that both the TR-20 and HEC-1 models used the SCS curve number/unit hydrograph approach. In fact, the same curve numbers were entered into both models. The time parameters were also determined using the same procedure. However, the SCS procedure also includes a parameter which adjusts the shape of the unit hydrograph. In TR-20, the shape of the hydrograph was adjusted to decrease the dimensionless peak, based on literature review and the authors' experience with other Florida watersheds (not based on any model calibration using the observed data). The HEC-1 program does not allow the shape of the unit hydrograph to be adjusted, so the default shape must be used. Although the authors do not explicitly state this, the difference in peak flows between the two models is likely due to this shape adjustment. Many software packages using the SCS hydrologic method do not allow the shape of the unit hydrograph to be changed. Thus, the good performance of the TR-20 model in this study may be somewhat unusual.

The models were then calibrated. With the exception of the regression equations, calibration reduced the average errors in urban peak flow and runoff volume to less than 10% for all models.

Zarriello (1998) summarized a study where nine uncalibrated runoff models were applied to two small urban watersheds in Washington and Colorado. A number of interesting observations can be made about this study, so it will be discussed in some detail here. The Colorado watershed, located near Denver, had an area of 3.1 square miles. The Washington watershed, located near Seattle, had an area of 0.14 square miles. The models used included CASC2D, CUHP, CUHP/SWMM, DR3M, HEC-1, HSPF, PSRM, SWMM, and TR-20. Each model package was applied separately by a team of experienced modelers, often by the agency or personnel responsible for developing the model software. Each team was provided with the same set of input data, including base, topographic, soil and drainage system maps. This set of data represented the data that would be available for a typical municipal or consulting modeling project. When the models were assembled, the observed rainfall records were input into the models. Simulated runoff was then compared to observed runoff measured at USGS gauges. No model calibration was performed.

Six storms were simulated for the Colorado watershed, and five storms were simulated for the Washington watershed. For each storm event and each model, the percent difference between observed and simulated values of peak flow and runoff volume was calculated. Simulated peak flows differed from observed peak flows by –100 percent to 260 percent. A root mean squared (RMS) peak flow error, based on results from all storms, was computed for each model at each watershed.

Table 2-2
Performance of Uncalibrated Models in Zarriello (1998)

	Colorado watershed		Washington watershed	
Model	Peak RMSE	Volume RMSE	Peak RMSE	Volume RMSE
CUHP / SWMM	19%	32%	--	--
CUHP	21%	39%	42%	55%
DR3M	19%	79%	42%	15%
HEC-1	171%	101%	162%	142%
HSPF	33%	17%	52%	35%
PSRM	47%	44%	40%	42%
SWMM	44%	45%	67%	16%
TR-20	89%	89%	74%	73%

All values are Root Mean Square Error (RMSE) for
percent difference between observed and simulated values

For the Colorado watershed, RMS square peak flow error ranged from 19% (CUHP/SWMM combination and DR3M model) to 171% (HEC-1 model). SCS curve numbers and unit hydrographs were used in HEC-1, so that model was really an application of the SCS procedure. The RMS peak flow error for SWMM was 44%, and the error for TR-20 (another application of SCS methods) was 89%. HSPF, another model commonly applied in southeastern Wisconsin, had an error of 33%.

For the Washington watershed, the PSRM (Penn State Runoff Model) has the lowest RMS peak flow error at 40%. The highest error again was achieved by HEC-1. SWMM had a peak flow error of 67%, TR-20 had an error of 74%, and HSPF had an error of 52%.

Errors in runoff volumes were also computed. Simulated runoff volumes (total volume for each storm event) differed from observed volumes by –100 percent to 240 percent. At the Colorado watershed, HSPF had the lowest RMS error for volume, at 17%. HEC-1 had the highest error, at 101%. The RMS errors for SWMM and TR-20 were 45% and 89% respectively. For the Washington watershed, DR3M had the lowest error at 15%, with SWMM a close second at 16%. HEC-1 once again had the highest error at 142%. TR-20 had an error of 73%, and HSPF had an error of 35%.

One of the main conclusions that can be drawn from the Zarriello paper is how difficult it is to accurately predict runoff peaks and volumes, and how much error and scatter can be present in the results. Different models can give widely different results. Even the best models had RMS errors of 15%-20%.

Zarriello concluded that models based on the SCS curve number (HEC-1 and TR-20) approach performed the poorest. It is also noteworthy that the HEC-1 and TR-20 models gave very different results. At the Colorado watershed, the arithmetic mean error (not the RMS error) in peak flows predicted by HEC-1 was 150%, indicating a trend to overpredict peaks. The arithmetic mean error in peak flows predicted by TR-20 was –87 percent, so peaks were drastically underpredicted. The algorithms behind both models are very similar and the same types of input parameters are used, so in theory the models should have produced similar results. However, each model was applied by a different group of modelers. The two different modelers apparently used significantly different input parameters. The discrepancy shows how even expert modelers can apply the same model in different ways and get drastically different results. It should be noted that the PSRM, which performed fairly well, computes runoff volume partially based on SCS curve numbers. However, PSRM also does some soil moisture accounting, and runoff routing is performed without using the SCS unit hydrograph.

The SCS models are representive of simple, lumped-parameter runoff models.   HSPF, SWMM, DR3M and PSRM are more complex models that represent input parameters in a more distributed manner. These “distributed” models tended to perform better. The CUHP (Colorado Urban Hydrograph Procedure) can be considered a simple, lumped-parameter model. It predicted peak flows well, particularly in the Colorado watershed.

This makes sense, as it was developed based on observed hydrographs on watersheds in the Denver, Colorado area. This points out the importance of developing or calibrating rainfall-runoff models based on local or regional conditions, a point that will be discussed again later.

It should be noted that the rainfall events in the Zarriello study were fairly small. For example, at the Washington watershed, the largest rainfall included in the study had a depth of 0.58”. The highest rainfall intensity included in the study was only 0.34 inches per hour. For the Colorado watershed, the largest storm had a volume of 1.04 inches, with a maximum intensity of 1.56 inches per hour. These depths and intensities of rainfall are smaller than the “design-magnitude” events often used for engineering design, such as events with a recurrence interval of 5 to 100 years. It is possible that models that don’t simulate runoff well during small rainfalls may work well for large storms, or that models which work well for smaller storms may not simulate large events well.

Yu et al. (1997) took a different approach to evaluating the hydrologic modeling of ungauged watershed. They investigated how well models reproduced flood quantiles, rather than specific storm events. Flood quantiles are discharges that have a certain annual exceedance probability or average recurrence interval. The Ward Creek watershed in Baton Rouge, Louisiana was selected for study. This urban watershed has an area of 4.63 square miles (2,960 acres). 41 years of observed annual flood peaks were available for this watershed. A statistical analysis was performed on the observed flood peaks to estimate flood quantiles (the 2-year flood, 10-year flood, 100-year flood, etc.).

Nine models for predicting flood quantiles were then applied to the Ward Creek watershed. These models were applied as if the watershed was ungauged, without using any of the observed data for calibration. The models included:

- Three sets of regression equations developed for Louisiana watersheds
- Two sets of nationwide regression equations developed by the USGS
- Two Louisiana “Regional Generalized Extreme Value” models. Although not explained in detail, these models appeared to be based on regional probability distributions and regression analyses, similar to the use of regression equations
- Rational Method
- SCS Method

Because these models are being used to compute flood quantiles, rather than simulate observed storms, “design storms” (synthetic rainfall patterns) must be used as the precipitation input for the Rational and SCS methods. The other methods are statistically based and do not attempt to simulate the rainfall-runoff process, so no corresponding rainfall input is required for each flood quantile.

Flood quantiles produced by each model were compared to flood quantiles from the observed record. The regression equations generally predicted flood quantiles with the least error. The Rational and SCS methods were two of the three worst performers. (The other poor performing model was a set of regression equations developed for larger watersheds, which the authors said may have been outside its range of application for this small watershed). Both the Rational and SCS methods overpredicted flood quantiles, for the entire range of flood flows considered. The following table shows the error in these methods, for selected flood quantiles.

Table 2-3
Performance of Uncalibrated Models in Yu et al. (1997)

	Observed flow (cfs)	SCS simulated flow (cfs)	% error in SCS simulation	Rational Method simulated flow (cfs)	% error in Rational Method
2-year flood flow	1,146	2,270	98%	1,791	56%
10-year flood	1,804	4,232	135%	2,819	56%
100-year flood	2,970	8,067	172%	4,461	50%

Data from Yu et al. (1997)

The authors felt that the difficulty of estimating the watershed time of concentration, which is required for the both the Rational and SCS methods, was a likely source of error for those models. They had sixteen students each estimate the time of concentration, and received estimates ranging from 1.5 to 5 hours. The authors arrived at the following conclusions regarding the estimation of flood flows for ungauged urban watersheds in Louisiana, which are worth quoting directly:

"1. Simple regression models developed by using local watershed and climatic conditions predict flood quantiles better than other more complicated models for ungauged urban basins.

"2. Models with few parameters that can be accurately estimated perform better than models with a large number of parameters that may not be easily estimated.

"3. When a model's assumptions are violated or a model is applied outside its boundary, large prediction error should be expected.

"4. Regionalization models perform reasonably well if they are properly applied.

"5. When estimated model parameters are subject to large variation or a model has many assumptions that are difficult to check, substantial prediction error may result."

- Yu et al. (1997)

Interestingly, the conclusion that simple models perform better than more complex models somewhat contradicts the results of the Zarriello (1998) study. However, input parameters for the simple models had been previously calibrated with observations from other watersheds, usually in the same region. This regional calibration may be more important than the level of complexity of a model.

Rawls et al. (1982) applied rainfall-runoff models to 51 small urban watersheds in the United States under 4,000 acres in area. A Log-Pearson frequency analysis was applied to the measured annual flood peak series for each watershed, to develop peak discharge estimates for various recurrence intervals. Peak discharge estimates from the Rational

Method, TR-55 (SCS method) and several sets of USGS regression equations were then compared to the peak flows from the flood frequency analysis, which were considered to be the true values. Rawls et al. found that the USGS regression equations performed the best, followed by the Rational Method. The SCS method was the worst performer. However, the authors felt that some of the test watersheds violated some of the conditions for using the SCS method, such as very flat slopes or different rainfall distributions. The SCS method tended to overpredict peak flows, while the Rational Method and regression equations tended to underpredict peak flows.

Calibration of Rainfall-Runoff Models

Calibration of rainfall-runoff models requires the adjustment of model input parameters until simulation results match observed data, within a certain tolerance for error. Often rainfall-runoff models are calibrated to peak flows for individual storm events, and/or runoff volumes for storm events or longer periods of time. Sometimes calibration to the shape of a observed hydrograph, or to a range of points on the observed hydrograph, will also be done, though this adds complexity to the calibration. The choice of measurements to use in calibration usually depends on the intended use of the model. If the calibrated model will be used to predict peak flows for design, then the model should be calibrated primarily to observed peak flows, and runoff volumes may be of only secondary importance. If the model is used to predict water budgets, then calibration of runoff volumes is probably more important than specific values of peak flow.

Methods of calibration will be discussed in the Procedures chapter of this report. In the current section, several publications on model calibration are reviewed. These particular studies are of interest because they provide information on what adjustments from textbook/default parameter values were necessary to achieve calibration, and what ranges of model parameters were found to provide the best fit.

The study by Trommer et al. (1996) has been summarized earlier. After the researchers evaluated the performance of the uncalibrated models, model calibration was performed. Calibration was able to increase the accuracy of the models in predicting peak flows and runoff volumes. The Rational Method was calibrated to produce an average C coefficient for each watershed. After calibration, average errors in predicting peak flow were in the range of 1%-3%. Similarly, the SCS curve number and time of concentration were calibrated to match observed peak flows and runoff volumes. Compared to initial uncalibrated values, curve numbers had to be decreased an average of 3%. Times of concentration were increased an average of 59%. Also, the unit hydrograph shape factor was calibrated. Final hydrograph shape factors ranged from 162 to 454, compared to the default value of 484. The SWMM models were also calibrated, and errors in peak flow and volume generally reduced.

Williams (1980) calibrated rainfall-runoff models for three urban watersheds in Oklahoma City, Oklahoma. The three watersheds had areas of 1.6 square miles, 3.0 square miles, and 28.2 square miles. Impervious cover in the watersheds ranged from 35% to 45%. Six models were applied: the Rational Method, TR-20 (an SCS model), HEC-1 (using the Clark unit hydrograph, not SCS procedures), SWMM, and two other less-common rainfall-runoff models. Twenty-three rainfall events were simulated, with rainfall depths ranging from 0.98 to 4.9 inches. The models were calibrated by comparing simulated peak flows and runoff volumes to observed peaks and volumes. No information on pre-calibration model performance was reported.

The following table provides some information on watershed characteristics and calibrated model parameters.

Table 2-4
Calibrated Model Parameters from Williams (1980)

	Deep Fork Creek at Portland Ave.	Deep Fork Creek at Eastern Ave.	Bluff Creek at Northwest Highway
Watershed characteristics			
Watershed area (sq. mi.)	2.98	28.2	1.64
Length of main watercourse (mi)	2.88	11.4	2.18
Slope of main watercourse (ft/mi)	44	20	66
Impervious cover, %	45	35	42
Watershed upstream of ponds, %	0	15	30
Calibrated model parameters			
Rational Method Runoff Coefficient C	0.38	0.38	0.22
SCS and Rational Method time of concentration, hours	0.70	2.83	0.46
SCS unit hydrograph shape factor	484	484	205
SCS Curve number	88	86	85
SWMM directly connected impervious area, %	45	25	25
SWMM impervious roughness	0.03	0.03	0.013
SWMM pervious roughness	0.4	0.25	0.05
Ratio of actual subcatchment width (measured on map) to SWMM catchment width	0.95	1.3	1.0
SWMM maximum infiltration rate (in/hr)	3	1	5
SWMM minimum infiltration rate (in/hr)	0.15	0.10	0.55

Data from Williams (1980)

In order to achieve a good calibration of the TR-20 model for one of the watersheds, Williams found it was necessary to adjust the shape of the unit hydrograph by increasing the volume of the descending limb relative to the rising limb. This may have been due to runoff detention in ponds in the watershed. Similar adjustments to the unit hydrograph shape were reported necessary for urban or rural watersheds by Sorrell (2003), Trommer et al. (1996), and McCuen and Bondelid (1983).

Williams also reported that antecedent moisture conditions did not have a statistically significant influence on runoff volume. For example, when using the SCS (TR-20) model, the observed curve number for each rain event was independent of the antecedent rainfall. Williams hypothesized that errors in rainfall may mask the effect of antecedent moisture. Rainfall for each watershed was collected at one rain gauge, located at the outlet of the watershed. This rainfall record was applied uniformly to the entire watershed in each model, but may not represent the average rainfall over the watershed. It was also possible that lawn watering during dry periods may have affected antecedent moisture in watershed soils.

Modeling of Small Rural Watersheds

The focus of this study is on small urban watersheds. The two watersheds that were modeled were entirely urban, and most of the literature reviewed was related to urban watersheds. However, the rainfall-runoff modeling methods used in this study can also be applied to rural watersheds, especially the SCS method, which was originally developed by a branch of the U.S. Department of Agriculture for application to rural watersheds. Many users of these models apply them to both urban and rural watersheds, depending on the projects. Therefore, it was useful to review some references on the performance and calibration of rural rainfall-runoff models. The literature record on rainfall-runoff modeling of rural areas is also quite large, and because of the focus of this study, the literature review on this topic is not as comprehensive as that on urban watersheds. However, some selected papers that provided interesting examples of the application of rainfall-runoff models to rural areas are summarized below.

One of the few published reports on rainfall-runoff model calibration in the Midwest or Great Lakes region was published by the Michigan state government. Sorrell (2003) reported that in Michigan, use of the standard SCS dimensionless hydrograph consistently overestimated discharges when compared to recorded gage flows. A study was done to evaluate whether the shape of the standard SCS dimensionless unit hydrograph is applicable to Michigan streams. The study involved 24 gaged streams with drainage areas less than 50 square miles. These streams were apparently in predominantly rural areas. The results from this study demonstrated that the recorded floods are best reproduced if the SCS unit hydrograph has 28.5 percent of the volume under the rising limb of the hydrograph, rather than the default value of 37.5%. Making this change will result in lower simulated peak discharges.

Hotchkiss and McCallum (1995) compared recorded peak discharges to modeled peaks for four small rural Nebraska watersheds. This study, sponsored by the Nebraska Department of Roads (NDOR), was intended to determine which method should be used to calculate peak discharge for small ungaged agricultural Nebraska watersheds, for use in culvert design. The study concluded that various regression equations provided the best estimate of large flood quantiles used in engineering design, such as the 25-year flood flow used in Nebraska to design rural culverts. The study found that both the Rational Method and the SCS method significantly underestimated the 25-year flood flow. The study also examined numerous formulas for computing the watershed time of concentration. The actual time of concentration for each watershed and each storm event was estimated as the time between the end of runoff-producing rainfall and the inflection point on the falling limb of the hydrograph. A nomograph developed by the NDOR, based on a modification of the Kirpich method, was found to reproduce the observed times of concentration best. The SCS average velocity method also performed well.

Fontaine (1995) modeled a historic flood event on a river in southwestern Wisconsin. In July 1978, the Kickapoo River experienced a flood resulting from an average depth of seven inches of rain falling on the watershed in two days. The resulting flood peak was estimated to have a recurrence interval of slightly more than 100 years. At the gaging station used in Fontaine's study, the Kickapoo River has a drainage area of 270 square miles, which is several orders of magnitude larger than other watersheds modeled in this study or included in the literature review. However, it is still interesting to consider the results of the rainfall-runoff simulation of this very large flood.

Two models were used to simulate this event: an HSPF model and a HEC-1 model. The HEC-1 model used the SCS curve number to calculate runoff volume, and a river-specific unit hydrograph to route runoff. The HSPF model was calibrated to several previous years of rainfall and runoff data. Its application to the extreme July 1978 event could be considered a model validation, though the calibration was done for smaller flood events.

The HEC-1 model was not explicitly calibrated, though by using a unit hydrograph derived from previous Kickapoo River hydrographs, some watershed-specific information is reflected in the model. Both models significantly overestimated peak discharge and runoff volume. The HSPF models (three different HSPF models were actually used based on three different sets of calibrated parameters) overestimated the peak flow by an average of 40%, even though the models had been previously calibrated to other observed hydrographs. HEC-1 overestimated the peak by 79%. HSPF overestimated the total flood volume by an average of 20%, while HEC-1 overestimated the volume by 29%. Fontaine summarizes these results by saying “These are significant errors for most conceivable runoff model applications. The magnitudes of the oversimulated peak discharge and runoff volumes would result in significant overdesign of water resources structures and in overly conservative risk assessments for safety reviews of dams and other high hazard facilities.” (Fontaine, 1995). Fontaine believed that the biggest source of error was probably inadequate precipitation data. Detailed data from four rain gages were available, but given the size of the watershed, this was probably not enough.

Summary of Literature Review

One of the dominant features of the rainfall-runoff process seems to be its uncertainty and variability. Runoff from real watersheds is unpredictable. Rainfall, the primary driver of runoff, is highly variable in time and space. Even for known rainfall, the response of runoff to rainfall is highly complex. Runoff is influenced by factors that are difficult to measure and predict, such as antecedent moisture conditions, soil characteristics, and almost infinite flow paths, each with a unique geometry and hydraulic resistance. Therefore, any attempt to model this process, especially without model calibration, seems bound to include a great deal of uncertainty. As the literature shows, uncalibrated runoff models can have a high degree of error. Even when models are calibrated, their predictive capabilities are often poor.

What can be said about the individual models? Uncalibrated SCS models seem to be consistently poor performers. The Rational Method is somewhat better. Most published reports on SWMM describe calibrated models, but the few reports on uncalibrated SWMM models indicate they performed well compared to other rainfall-runoff models. For all models, calibration can increase their accuracy.

Application of these models to the Milwaukee watersheds will be described next.

CHAPTER 3: PROCEDURES

Selection of Study Watersheds

One of the first steps in this study was the selection of the watersheds that would be modeled. The list of current or past USGS gauging stations in southern Wisconsin was reviewed. From this list, possible study watersheds were screened and evaluated according to the following criteria.

Size

In this study, it was desired to focus on small watersheds. The definition of a "small watershed" is subjective. Relative to the size of the Missouri-Mississippi River watershed, even the largest watersheds in the Milwaukee area would be considered small. For this study, a small watershed is defined as less than five square miles in areas. Most of the studies described in the literature review were done for watersheds less than five square miles. Most engineering design is done for small watersheds of this magnitude, because watersheds of this size are much more numerous than larger watersheds (due to the simple fact that larger watersheds are made up of combinations of smaller watersheds). In fact, much engineering design is done for drainage basins that are even smaller than one square mile (640 acres). Typical developments and subdivisions, for which detailed hydrologic design is required, are on the order of 40 acres to 160 acres in area. Many storm sewers are designed to drain areas of only a few acres.

Urban land use

In addition to being focused on small watersheds, this study also focuses on urban watersheds. The rainfall-runoff characteristics of urban and rural watersheds can be quite different. Urban watersheds contain a large amount of impervious surfaces – streets, rooftops, parking lots, driveways and other hard surfaces which allow very little water to infiltrate through them into the soil. Therefore, runoff volumes in urban watersheds tend to be larger than volumes in rural watersheds. Peak flows in urban watersheds also tend to be much higher. Water flows quickly off pavement and rooftops, and manmade drainage channels such as gutters and storm sewers rapidly convey runoff to the outlet of the watershed. Data from rural watersheds is of limited use in studying the runoff response of urban watersheds.

Therefore, watersheds were sought which represent typical urban land use conditions in southeastern Wisconsin. Because one of the expected benefits of this study is to improve the modeling of ungauged watersheds, the study watershed conditions must be typical enough so that results can be extrapolated to other watersheds. A few small urban watersheds with USGS gauges were rejected for study because they appeared to represent unusual watershed conditions. For example, the USGS operates several gauges that measure flows from areas of General Mitchell International Airport in Milwaukee. These gauges were not selected for study, because the land conditions associated with a large commercial airport are different from the mixes of residential, commercial or industrial land uses commonly found in urban watersheds. Also, obtaining data on infrastructure within the airport would have been difficult in this age of tightening security, let alone obtaining physical access to the airport to conduct field observations. The USGS also operates several gauges in storm sewers that drain Milwaukee freeways. Although studying runoff from freeway surfaces is important, especially in the study of water quality, freeway right-of-way represents a small percentage of the total urban area in southeastern Wisconsin.

Location

The USGS operates numerous gauging stations in urban and suburban watersheds around the Madison metropolitan area. However, preference was given to watersheds in the Milwaukee area. This permitted easy visits to the watersheds to observe surface conditions and the drainage system. The Milwaukee sites could even be visited on short notice during runoff events, to observe runoff patterns and develop a deeper appreciation and understanding of the recorded data. Visiting a study area in person provides insight that cannot be gained from maps, pictures or recorded data alone. Because suitable sites were found in the Milwaukee area, the Madison sites were not used. However, they could be suitable for future study.

Availability of hydrologic data for model calibration

The study watersheds must have continuously recorded flow and rainfall data, at small time increments (one to five minutes). In small watersheds, streamflow quickly responds to rainfall, and small time steps are needed to adequately reflect how hydrologic conditions change. The data must be continuously recorded. There are some small watersheds for which the USGS collects data on peak flows, typically using crest gauges which record only the maximum stage of water. However, these watersheds were not used in this study. Without continuous recording of flow, no data is available on runoff volume and hydrograph shape, which is needed for the calibration of some models.

Watersheds which had a range of recorded runoff events, including some very high runoff events, were sought. Many USGS gauges are only operational for two to three years, because of funding limitations. In two years of measuring flow, the probability of experiencing a 100-year runoff event or even a ten-year runoff event is low. However, calibrating a model to large infrequent runoff events is desirable, because those are the magnitude of events usually used for design. Obviously, a long period of measurement is desirable. It proved difficult to find gauging sites with long periods of record which met the other criteria for study.

Preference was also given to watersheds where the data was collected in the recent past. Runoff was monitored in several small watersheds in the Milwaukee and Madison areas in the late 1970s and early 1980s. These watersheds meet many of the criteria described in this section. However, they were rejected because of the length of time between the data collection and the present. There is more uncertainty about the condition of the watershed when the data was collected. Major changes in land use and/or drainage infrastructure may have occurred in the intervening period, and there may be little available mapping or records to document those changes. Also, technology for flow measurement is continually improving. The equipment used to measure and record flows 25 years ago may be less accurate or precise. Also, this study required communication with staff who operated and maintained the data collection stations, and with public officials who operate and maintain the drainage systems in the watersheds. The people who performed these duties 25 years ago may be unavailable, or may not remember much about the system.

Availability of Data/Mapping to Develop Model Input

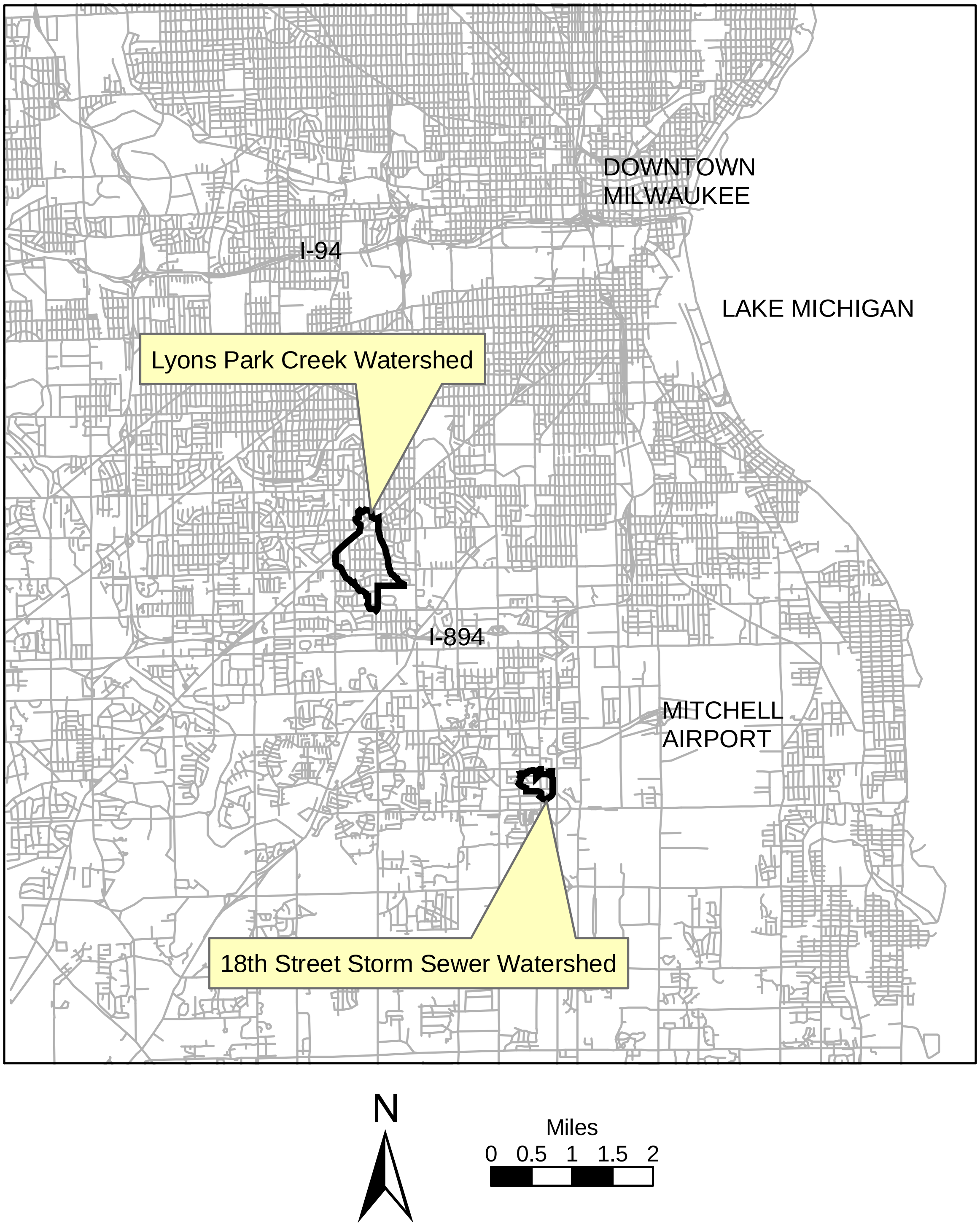
Hydrologic models require input which describes the surface conditions of the watershed, such as topography, land use, soil types, vegetation, and the nature of the drainage system. The study sites must have this information available. In southeastern Wisconsin, the Southeastern Wisconsin Regional Planning Commission (SEWRPC) produces high-quality mapping of topography and land use. Soil information is available for most areas from the County Soil Surveys. Information on the drainage system is typically collected from a variety of sources, including topographic maps, aerial photographs, construction plans, utility/system maps, field surveys and site visits. The selected watersheds must be located in municipalities where local officials are willing to provide this information.

After evaluating potential study watersheds according to these criteria, the following watersheds were selected for study.

- Lyons Park Creek, USGS gauge 040871465, 0.5 mi^2 drainage area
- 18th Street storm sewer, USGS gauge 04087193, 0.1 mi^2 drainage area

These two watersheds are located on the south side of the City of Milwaukee, allowing easy access. Figure 3-1 shows the location of these watersheds. Rainfall and flow data was recorded by the USGS for both watersheds during 2002 and 2003. Both watersheds contain predominantly residential land use typical of urban areas in southeastern Wisconsin. A variety of mapping and data for these watersheds could be obtained from the City of Milwaukee and the UWM/American Geographical Society library. The watersheds are small; watersheds of similar size are often studied as part of engineering planning and design projects. Perhaps the main disadvantage of these sites is the short two-year period of data collection. However, within those two years, the Lyons Park Creek watershed experienced a heavy rainfall event, with an estimated recurrence interval of fifty years. The 18th Street storm sewer also experienced several intense rainfall-runoff events. Thus, these watersheds presented opportunities to model infrequent, extreme runoff events of the magnitude used for engineering design.

Figure 3-1
Location of Study Watersheds



Data Collection and Analysis

Flow and Rainfall Data

Discharge and rainfall data for both watersheds were provided by the USGS in digital format. Discharge at both gauging stations was measured using Isco flow meters, which utilized ultrasonic velocity probes and pressure transducers. The instantaneous flow was recorded every minute during runoff events. During dry weather, flow was recorded less frequently, typically every hour. Rainfall was measured using recording rain gauges, which were placed on light poles near each flow meter. During rain events, rainfall was recorded every minute, or whenever a 0.01” increment of rain occurred.

Raw flow and rainfall data were recorded from April 2002 through the end of 2003. However, the USGS indicated that during freezing weather, the data was less reliable, because of possible operational problems with the measuring equipment. Fortunately, the major runoff events occurred during late spring, summer and early fall. Therefore, data from the winter months were not used in this study.

Typically the USGS reviews the recorded raw data as part of a quality control process. Erroneous measurements caused by equipment malfunctions are corrected or deleted. The USGS (P. Hughes, personal communication, 2003) indicated that the data recorded in 2002 had been reviewed and should be considered final. However, the data recorded in 2003 was not reviewed by the USGS for quality control purposes, because of budgetary constraints. The raw data was provided for use in this study, with the caveat that it may contain errors. During data review and analysis for this study, some data points were detected that appeared to be erroneous. These data points were eliminated. Even though an attempt was made to review the 2003 data for errors, it may be less reliable than the 2002 data, because it was not finalized by the USGS. Therefore, only the 2002 data was used for model calibration. The 2003 data were used to attempt model validation, but it should be remembered that the 2003 data may be less accurate than the 2002 data. This issue will be discussed later in this report.

Flows in Lyons Park Creek were measured in one barrel of a twin-barrel (each barrel is 7 ft. x 5 ft.) concrete box culvert under 55th Street. The USGS assumed that flow was approximately equal in each culvert barrel, so the measured flow was multiplied by two to obtain the total flow in the creek. This appears to be a reasonable assumption. Both culvert barrels have the same cross-sectional area and the same invert elevations. Flow in the culvert was observed during several rain events, and flow appeared to be split evenly between the culvert barrels.

The 18th Street flow gauge was located in a 42” storm sewer under Eighteenth Street. The USGS refers to this gauge as the “Ramsey Street Storm Sewer” gauge. However, since the gauge is actually located in 18th Street, and its tributary area does not include Ramsey Street, the gauge and watershed will be referred to as the 18th Street site in this study.

Lyons Park Creek normally contains a small amount of baseflow. To more accurately estimate the volume of stormwater runoff, hydrograph separation was performed to estimate the amount of baseflow included in the measured hydrographs. Baseflow separation was performed similar to a method described in Linsley et al. (1975). Initially, an attempt was made to estimate a baseflow recession slope during the beginning of each storm. However, it was difficult to establish baseflow recession rates – The baseflow in Lyons Park Creek is very low – usually around 0.1 cfs. There appeared to be a lot of noise in the recorded streamflow during dry weather periods, and trends were generally not apparent. Therefore, a constant baseflow rate was assumed during the rising limb of the storm, with baseflow assumed to equal the flow in the stream right before the hydrograph began to rise. This constant baseflow was extended to the time of peak flow.

Next, the time when direct runoff ended was estimated. Linsley et al. (1975) give an equation for estimating the time between the hydrograph peak and the end of runoff, based on watershed area. This analysis is complicated by runoff events with multiple peaks in flow. When multi-peak storms occurred, the last distinct peak was usually chosen as the time to apply the equation to. In some cases, flow in the channel fell below the original baseflow value before the equation-predicted time for end of runoff. It does not seem reasonable that baseflow after a significant rain event would be much less than baseflow before the event occurred. Therefore, in this case the ending time of direct runoff was chosen as the time when flow in the channel reached the same value as the baseflow before the storm.

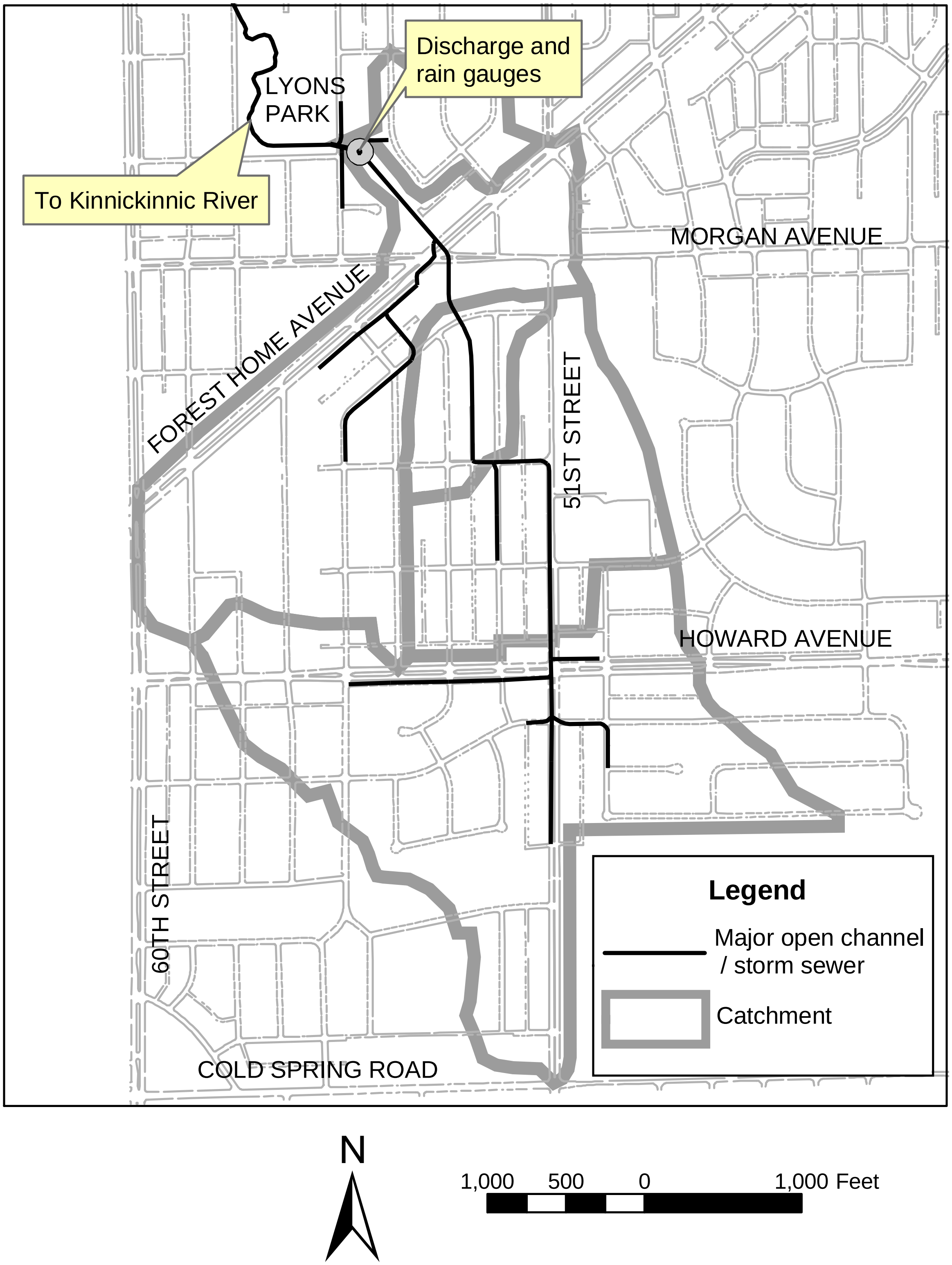
It should be recognized that baseflow separation is an imprecise process. Assumptions and approximations must often be made, as described above. In Lyons Park Creek, the estimated baseflow volumes during runoff events were less than 5% of the estimated runoff volumes. Therefore, study results are not very sensitive to the exact method of baseflow separation used.

During most dry weather periods, no flow was measured in the Eighteenth Street storm sewer. This is not surprising, as it is quite possible that the water table lies below the storm sewer. Therefore, all flow measured in the Eighteenth Street storm sewer was assumed to be runoff, and no baseflow separation was needed.

Watershed Characteristics

Both watersheds are located on the south side of the city of Milwaukee, Wisconsin. Lyons Park Creek is a tributary to the Kinnickinnic River, which drains to the Milwaukee Harbor estuary and Lake Michigan. The watershed area upstream from the gauging station is 297 acres, or 0.46 square miles. The gauging station is not located at the watershed outlet; there is a downstream portion of the Lyons Park Creek watershed which is not tributary to the gauge. Figure 3-2 shows the Lyons Park Creek watershed.

Figure 3-2
Lyons Park Creek Watershed



As mentioned earlier, the Lyons Park Creek gauge was located in a concrete box culvert (twin barrels, each 7' x 5') under 55th Street. Upstream from this culvert, Lyons Park Creek is alternately contained in open channels and underground storm sewers. The open channels are of manmade origin; very little of the natural creek bed and valley remains.

The Lyons Park Creek watershed was delineated from detailed topographic maps developed by the Southeastern Wisconsin Regional Planning Commission (SEWRPC), and storm sewer system maps and plans provided by the City of Milwaukee. Storm sewer routes generally follow the surface topography. Watershed topography could be described as gently rolling. Elevations in the watershed range from 730 feet above sea level (the creek bottom at the gauging station) to 785 feet above sea level at the watershed divide. Most watershed data and mapping was stored and analyzed in a digital Geographic Information System (GIS), using ESRI ArcGIS software.

Land use plays an important role in the hydrologic response of a watershed. Therefore, a watershed land use map was developed in the GIS. Land use was determined from aerial photography and site visits. The Lyons Park Creek watershed contains predominantly residential land use. The residential neighborhoods are typical of developments built in the city of Milwaukee and neighboring suburbs during the middle of the 20th century, with moderate lot sizes and relatively small houses. The houses are not as tightly packed as they are in neighborhoods closer to downtown, but the lots and houses are not as big as some neighborhoods in newer or wealthier areas of Milwaukee County. The watershed also contains some commercial land use along Forest Home Avenue and other arterial streets, as well as a school site and some open space.

The types of soil found in a watershed also have a major influence on its hydrologic response. Sandy soils usually allow a large amount of rainfall to infiltrate into the soil, and runoff rates are relatively low. Conversely, clay soils do not allow much infiltration, and more runoff occurs. Impervious surfaces such as pavement and roofs almost totally block infiltration into the soil.

The Natural Resources Conservation Service (NRCS), formerly the Soil Conservation Service, publishes soil surveys for each county in Wisconsin. These surveys contain maps of soil units with the county, and descriptions of the soil units that are mapped. The Lyons Park Creek watershed was mostly developed when the Milwaukee County Soil Survey was written. Therefore, rather than reporting natural soil units within the watershed, the soil survey classifies watershed soils as “Urban Land”. Detailed soil properties such as infiltration rates are not available for this category, since the soil conditions can be highly variable and impacted by development. However, some inferences about the watershed soils can be made from nearby mapped soil units, which are clayey soils. Most soil near the surface in Milwaukee County is clayey. In the absence of site-specific information, watershed soils will be assumed to be claylike.

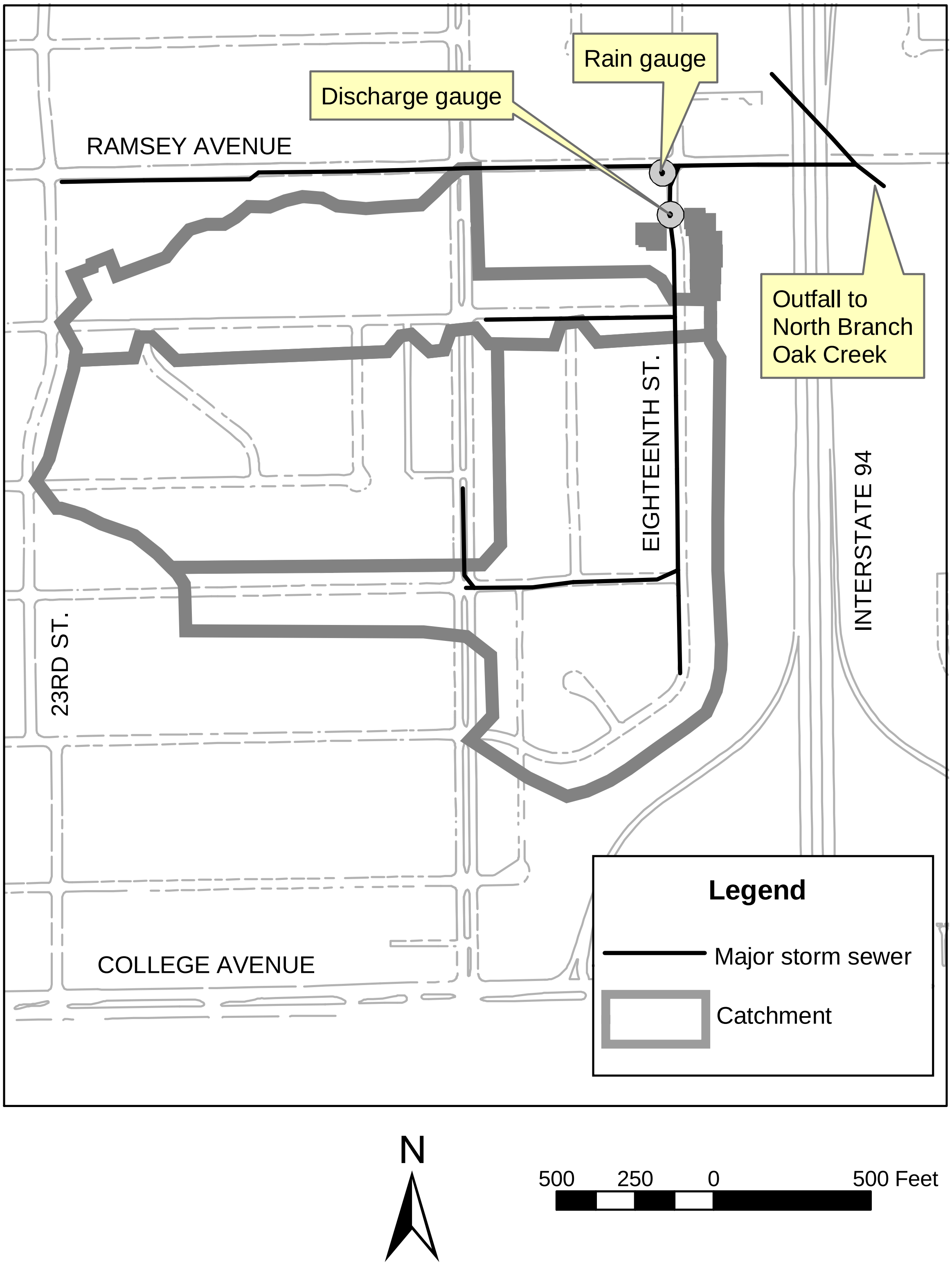
Data was collected and analyzed in a similar fashion for the Eighteenth Street Storm Sewer watershed. This storm sewer joins a storm sewer in Ramsey Street, which in turn discharges into the North Branch of Oak Creek a short distance to the east. Oak Creek is a tributary of Lake Michigan. The watershed is drained entirely by storm sewer.

Once again, the watershed was delineated using topographic maps and storm sewer plans. The watershed has an area of 64 acres, or 0.10 square miles. The topography is hilly in the western half of the watershed, and fairly flat in the eastern half. Elevations in the watershed range from approximately 750 feet above sea level (the low point in Eighteenth Street near the gauging station) to 810 feet above sea level at the watershed divide.

Land use in this watershed is almost entirely residential, with the exception of one school site. Like the Lyons Park Creek watershed, house and lot sizes are modest. The Milwaukee County Soil Survey (Soil Conservation Service, 1971) did map the soil units for this watershed. Soils are generally clay or silt, with low infiltration potential.

Figure 3-3 shows the Eighteenth Street Storm Sewer watershed.

Figure 3-3
Eighteenth Street Storm Sewer watershed



Model Development

Once watershed data and mapping was collected, model development could begin. The first step in model development was to decide on the spatial detail of each model. The level of spatial detail or discretization in a model is an important characteristic that must be determined by the model builder. The catchment (also called drainage basin, subbasin or subwatershed) is the building block of many rainfall-runoff models. One watershed can be divided into many catchments. The more catchments used, the finer the level of spatial discretization.

Numerous researchers, such as Zaghloul (1981), Mazion and Yen (1994) and Stephenson (1989) have investigated the effects of spatial discretization of rainfall-runoff models. Models of the same watershed with different levels of discretization can give quite different results. Peak flow is usually more sensitive to discretization than runoff volume. One might expect that model results will become more accurate as the level of model discretization increases. However, research indicates that this is not necessarily the case. Stephenson (1989) found that models with coarser spatial discretization were able to better predict peak flows than models with finer spatial discretization, though the models with finer discretization predicted runoff volume better. It is possible that the uncertainty associated with rainfall-runoff modeling masks the benefits of building a more detailed model. Without calibration, it may be just as difficult to select the parameters for a finely discretized model as for a coarse model.

Ideally, model builders should probably try several different levels of spatial discretization, and investigate the sensitivity of model results to discretization. However, in practice often a certain spatial scale is selected at the beginning of the modeling process. Building additional models of different discretization can be costly in terms of time or money, so often this is not done.

Model calibration can make the issue of spatial discretization less important. As Zaghloul (1981) suggests, a coarse model can be calibrated just as well as a fine model. But as this study discusses, uncalibrated rainfall-runoff modeling is prevalent. One of the aims of this study is to investigate the accuracy of uncalibrated rainfall-runoff models, as used in typical engineering practice. Since modelers often select a certain level of discretization, without investigating alternate levels, it is useful to investigate the sensitivity of model results to different levels of discretization.

Therefore, multiple SWMM models were constructed for each watershed, with different levels of discretization. For the Lyons Park Creek watershed, two different SWMM models were constructed. These divided the watershed into one and five catchments respectively. The detail of the hydraulic routing model also varied. The detailed model represented approximately 4,000 feet of storm sewer or open channels within the watershed, divided into 10 segments. In the simple model, with only one catchment, no hydraulic routing through the drainage system was simulated. Similarly, two different models were assembled using the SCS methodology. One model contains 5 catchments and included hydraulic routing through the drainage system, while the other model represented the entire watershed with one catchment with no hydraulic routing. Because the Rational Method is intended to calculate peak flows and is not well suited for routing hydrographs, no investigation of discretization levels was made for the Rational Method.

A similar investigation was also constructed for the Eighteenth Street storm sewer watershed. Three SWMM models were constructed and two SCS models were constructed, with varying levels of discretization The simplest models contain only one catchment and no hydraulic routing through the storm sewer system. The most detailed model divides the watershed into four catchments, and includes hydraulic routing through approximately 2,300 feet of storm sewer upstream from the gauging station. Results from all levels of discretization will be discussed in the following chapter.

Each of the rainfall-runoff models requires its own set of input parameters. The following sections describe how the input parameters were developed for each model.

Rational Method input parameters

The Rational Method is the simplest rainfall-runoff model used, and requires the fewest number of input parameters: the catchment area, a Rational Method runoff coefficient, and a rainfall intensity. Once the watershed was delineated using contour and storm sewer maps, the catchment area was easily calculated in the GIS. The runoff coefficient was calculated based on the amount of “directly connected” impervious surface in the watershed. Actually, all three models use input parameters related to the amount of imperviousness in the watershed, so a detailed analysis of imperviousness was conducted.

The Geographic Information Systems built for the watersheds included topographic maps and aerial photographs which showed all of the impervious surfaces in the watershed in detail. Even though it would have been possible to trace each piece of impervious surface separately and calculate its area, that task would have been extremely time consuming. Therefore, representative sections of each watershed were selected, and the imperviousness within each of those sections measured. For example, the street and house sizes on most blocks in the Lyons Park Creek watershed appeared very similar. Therefore, impervious area measurements were made for a few of the blocks, and the results extrapolated to the entire watershed. Some areas had more unique land use characteristics, such as school sites, commercial areas or parks. In these areas, all impervious surfaces were measured in the GIS, instead of extrapolating from other areas.

A distinction was made between "directly connected" impervious surfaces and disconnected impervious surfaces. A directly connected impervious surface is one that drains directly into drainage system without passing over any pervious surfaces. Most street pavement is directly connected, because it drains to the street gutters, which in turn drain to storm sewer inlets. Many rooftops are directly connected, because the roof downspouts are connected to storm sewer laterals. However, some rooftops are disconnected, because the downspouts drain to lawns or gardens, where the runoff can infiltrate into the ground. Driveways and sidewalks, the other major impervious surfaces, can be either connected or disconnected, depending on whether they drain towards the street or towards grassed areas. Field visits to the watershed helped classify impervious areas as being connected or disconnected.

40% of the Lyons Park Creek watershed was estimated to consist of directly connected impervious surfaces, and another 8% consists of disconnected imperviousness. Therefore, the total watershed imperviousness was estimated to be 48%. The Eighteenth Street Storm Sewer watershed was estimated to contain 35% directly connected imperviousness, and 4% disconnected imperviousness, for a total imperviousness of 39%.

Returning to the Rational Method, one method for estimating the runoff coefficient uses the percentage of imperviousness. The WPCF/ASCE sewer design manual (Water Pollution Control Federation, 1969) suggests ranges of runoff coefficients for different types of impervious and pervious surfaces. An average value for impervious surfaces (rooftops and pavement) is 0.85. Rational Method documentation is not clear on whether the total impervious area should be used to calculate the runoff coefficient, or just the directly connected impervious area. In this study, only the directly connected impervious area was used. An average runoff coefficient for the predominant pervious surface found in the watersheds (lawns, heavy soil, 0-2% slopes) is about 0.15. Using these values, and the estimated percentages of impervious and pervious area, a weighted runoff coefficient was calculated for each watershed. The estimated Rational Method runoff coefficient for Lyons Park Creek is 0.43. For the Eighteenth Street watershed, the estimated runoff coefficient is 0.40.

In reality, the runoff coefficient tends to vary with storm intensity (Schaake et al., 1967; Titmarsh et al., 1995). For the same watershed, very large infrequent events (such as the 100-year rainfall) often have higher runoff coefficients than relatively more frequent events, such as the 2-year rainfall. Also, runoff coefficients will also vary with antecedent moisture conditions. However, in this author's experience, modelers and designers often calculate a single value for the watershed runoff coefficient, and use it for all hydrologic analyses. Therefore, the uncalibrated runoff coefficient was held constant in this study also.

The last input parameter required for the Rational Method is the rainfall intensity. Rainfall intensity varies with time during a storm – therefore, rainfall intensity will depend on the length of time used to calculate it. According to the theory of the Rational Method, the time used to calculate the rainfall intensity should be equal to the "time of concentration" of the catchment. Time of concentration is defined as the time it takes water to flow from the most hydraulically remote part of a watershed (the point furthest upstream in time, not necessarily in distance) to the watershed outlet. The reasoning behind its use in the Rational Method is that if the time used to estimate rainfall intensity is less than the time of concentration, not all of the rain falling at that intensity will have time to travel to the watershed outlet and contribute to the peak flow. And because longer rainfall periods have lower average rainfall intensities (heavy downpours are usually not sustained for very long), using a time longer than the time of concentration probably underestimates the peak intensity of rainfall contributing to the peak flow.

Despite the seemingly simple definition, the time of concentration is a difficult hydrological parameter to measure or estimate. Like other parameters, it actually varies with rainfall intensity and antecedent conditions, though it is often considered to be constant. Measuring time of concentration in the field is difficult, though there is a method to estimate it from a hydrograph, which will be discussed later. Therefore, empirical or theoretical equations are usually used to estimate time of concentration. Many of these equations make use of the definition as the time required for water to travel from the watershed divide to the outlet. If the velocity of runoff can be calculated, and its length of travel known, then a time of travel can be calculated. However, sources differ as to whether the actual velocity of a particle should be used, or the wave velocity, which is faster. In summary, estimating the watershed time of concentration is difficult. McCuen (1998) says “While the time of concentration is an important input to hydrologic design, it is neither a highly accurate input nor highly reproducible…every designer should recognize that a single correct method for estimating time of concentration is not possible, and, therefore, the true value can never be determined.”

Despite this difficulty and uncertainty, many rainfall-runoff models use the time of concentration, and it must be estimated somehow. One of the most popular methods is the SCS “velocity” method, described in references such as Soil Conservation Service (1986). This method was used to estimate times of concentration for the Lyons Park Creek and Eighteenth Street watersheds. The initial estimated time of concentration was 50 minutes for Lyons Creek and 29 minutes for Eighteenth Street.

For each storm event, the most intense period of rainfall for a duration equal to the time of concentration was found, using a search routine applied to the recorded rainfall. The peak flow for that storm event was then estimated using the Rational Method equation.

Some references on the Rational Method (for example, Wisconsin Department of Transportation, 1997) describe how applying the Rational Method to only the impervious area of a watershed can produce a higher peak flow estimate than applying the equation to the entire watershed. The reason is that the impervious area usually has a much shorter time of concentration, resulting in a much higher rainfall intensity to be used. Sometimes this higher rainfall intensity more than offsets the smaller drainage area used in the calculation, and the calculated peak flow is higher. Therefore, many modelers will compute peak flows with the Rational Method using both the entire catchment and only the impervious area, and use the higher peak flow estimate. In this study, peak flows were calculated using both the total catchment area and the impervious-only area. Results from both methods are presented in the following chapter.

SCS input parameters

The second rainfall-runoff model used was the SCS model. This method, described in the previous chapter, uses curve numbers to calculate runoff volume, and a unit hydrograph to transform excess rainfall into a hydrograph at the catchment outlet. The Soil Conservation Service (1986) manual gives standard curve numbers for both pervious and impervious areas. Similar to the Rational Method coefficient, a weighted curve number was calculated for each watershed. The SCS documentation is not entirely clear whether the directly connected impervious area or total impervious area should be used to calculate the curve number, but seems to imply that the total impervious area should be used. The estimated curve number for the Lyons Park Creek watershed was 86; for Eighteenth Street it was 83. For the more detailed models, where the watersheds were divided into smaller catchments, a separate curve number was calculated for each catchment. The other parameter used to calculate runoff volume in the SCS method is the initial abstraction. Usually a default ratio between initial abstraction and curve number is used. This default value was used for the uncalibrated modeling, but it was adjusted during the calibration process.

Once the runoff volume is calculated, it is transferred to the catchment outlet using a unit hydrograph. The shape and coordinates of the unit hydrograph are related to the time of concentration. The same time of concentration calculated for the Rational Method was also used for the SCS method. For finer levels of discretization, a unique time of concentration was calculated for each catchment.

SWMM input parameters

SWMM is the most complex of the rainfall-runoff models used in this study, and requires the most input parameters. Manuals such as James et al. (2003) explain each input parameter in detail. The percent of impervious area (directly connected, in this case) is a very important input parameter, and was calculated as described earlier. Catchment width is another sensitive parameter, particularly for peak flows. In this study, the width was initially estimated as the catchment area divided by the length of the longest flow path (the same flow path used to calculate the time of concentration). Width is one of the main parameters adjusted during calibration.

Many SWMM input parameters were estimated using default or recommended values from the model documentation, or typical values reported in the literature by other modelers. The SWMM model can use one of two different infiltration models – the Horton model or the Green-Ampt model. The Horton model uses exponential decay curves to represent the decrease in infiltration rate during a rainfall event. Parameters describing the shape of the curve have been empirically derived from measured infiltration data. The Green-Ampt model is a more physically-based method that applies Darcy's Law to simulate the vertical movement of water from the surface through the soil. To compare the two methods, both were applied to the study watersheds.

The following table summarizes the initial estimates of SWMM input parameters, as well as parameters for other models. These values are for the models with the simple discretization, where the entire watershed was modeled as one catchment. For detailed levels of discretization, separate parameters were calculated for each catchment.

Table 3-1
Initial Model Input Parameters

Model input parameter	Lyons Park Creek model	Eighteenth Street Storm Sewer model
Watershed area, acres	297	64
SWMM input parameters:		
Impervious percentage	40	35
Watershed width, ft	1840	770
Slope, ft/ft	0.008	0.017
Impervious surface Manning's roughness coefficient	0.015	0.015
Pervious surface Manning's roughness coefficient	0.24	0.24
Impervious surface depression storage, inches	0.02	0.02
Pervious surface depression storage, inches	0.10	0.10
Horton initial infiltration rate, in/hr	1.5	1.5
Horton final infiltration rate, in/hr	0.10	0.10
Horton infiltration decay, 1/sec	0.00115	0.00115
Green-Ampt capillary suction, inches	11.5	11.5
Green–Ampt saturated hydraulic conductivity, in/hr	0.04	0.04
Green-Ampt nitial moisture deficit	0.09	0.09
Rational Method input parameters:		
Runoff coefficient	0.43	0.40
Time of concentration, minutes	50	29
SCS input parameters:		
Curve number	86	83
Time of concentration, minutes	50	29
Initial abstraction, inches	0.33	0.41
Unit hydrograph shape factor	484	484

Running the Models

Once the rainfall-runoff models were constructed, they were used to simulate ten recorded rainfall events in each watershed. The precipitation recorded at each rain gauge was used as input to the models. No attempt was made to adjust model input parameters to match the observed runoff data. Instead, as described above, model input parameters were selected using model documentation and references. The model results from the historical storms could then be compared to the recorded flow data, as the next chapter shows. This round of uncalibrated modeling offers a test of how rainfall-runoff models are likely to perform when applied to ungauged watersheds.

The Rational Method and SCS method can simulate single events only. There is no "memory" of model state from previous events. SWMM can be run for single events or as a continuous, long-term simulation. In continuous simulation, some of the parameters, particularly the infiltration parameters, vary through time depending on moisture conditions. In this study, to compare the effects of single event model runs vs. continuous simulation, the historical storms were simulated in both modes. In continous simulation, the entire 2002 monitoring period (aside from the winter months) constituted one model run, as did the 2003 period.

At this point, the results of the uncalibrated models were recorded and reviewed. Results from each rainfall-runoff model were compared to the observed flow data and to the other models. Analysis of the error and bias in model results was made. These results are discussed in the following chapter.

Sensitivity Analysis

Following the review of the results of the uncalibrated modeling, calibration could begin. A sensitivity analysis of the model input parameters should be conducted as part of any model calibration effort. A sensitivity analysis requires changing one of the model input parameters by a specified amount, and observing how the model output changes. This shows which input parameters affect the model results more than others. More effort should be spent estimating and calibrating those parameters which the model is most sensitive to. Even for uncalibrated modeling, performing a sensitivity analysis is a good idea. Such an analysis will suggest which input parameters should be estimated most carefully, and provides insight on how much uncertainty is present in the model output.

A detailed sensitivity analysis was performed using the Lyons Park Creek SWMM model; because the watershed characteristics are similar, parameters should show similar sensitivity in the Eighteenth Street model. Sensitivity analyses were performed for two observed storm events: a medium-sized event with approximately an inch of rainfall, and the largest recorded event, with over five inches of rain. Two different storms were used because different hydrologic processes may predominate for these events. Most of the runoff from small and medium rainfalls is from impervious areas, so parameters describing the impervious areas will show the most sensitivity. For larger events, parameters describing the pervious areas, such as infiltration parameters, will become more sensitive.

The XPSWMM software includes an automatic sensitivity analysis routine where all catchment Runoff module parameters are adjusted by the same amount. This routine was used to adjust parameters by +/- 25%. Some manual sensitivity analyses were also performed for maximum reasonable ranges of parameter values (some parameters could feasibly vary by an order of magnitude or more, whereas a 25% change is unrealistically high for some parameters). XPSWMM also does not allow an automatic sensitivity analysis on parameters in the hydraulic routing module. Therefore, some manual sensitivity analysis was performed on certain hydraulic parameters such as storm sewer roughness, open channel roughness, and culvert head loss coefficients. Other drainage system hydraulic parameters such as sewer diameters and slopes can be estimated with a high degree of confidence from maps and plans, so a sensitivity analyses was not performed on some of the hydraulic routing input. In general, peak flows and runoff volumes were more sensitive to hydrologic catchment parameters, as opposed to hydraulic routing parameters. There was some sensitivity in peak flows to open channel and storm sewer roughness. More results of the sensitivity analysis are reported in the following chapter, and a detailed table of sensitivity results is included in the Appendix.

Calibration

Calibration is the process of adjusting model input until results simulated by the model are acceptably close to measured data. In order to calibrate, target values for model output must be selected. For this study, observed peak flows and runoff volumes from major storms occurring in 2002 in each watershed were used as target values. This resulted in seven storms being used for calibration for each watershed. Only the 2002 events were used because runoff data from that year was finalized by the USGS. In contrast, runoff data from 2003 was not reviewed and finalized by the USGS; it was provided for use in its raw recorded form. Therefore, the 2003 data may contain more errors than the 2002 data, and probably should be used with a lower degree of confidence. The 2003 data will be used for model validation, the process of testing the calibrated model with additional data. However, the lower degree of confidence in the 2003 data should be remembered when reviewing the model validation results.

When calibrating a model, one or more methods must be selected for calculating how closely model results match measured data. These measures are often called the “goodness of fit” of a model. For this study, differences between the measured peak flow and simulated peak flow, and between the measured runoff volume and simulated runoff volume, were used to assess goodness of fit. These differences were calculated both on an absolute basis and as a percentage difference. The “average” differences for each model and each watershed were then calculated in several ways, by taking a simple arithmetic mean, or by calculating the root mean square of the differences. No one single measure was used to determine the goodness of fit; rather, some judgment was used to weigh the results of different fit measures for each calibration effort. Chapter Four describes these error measures in more detail.

For many rainfall-runoff models, the recommended calibration procedure is to first calibrate to runoff volumes (mass balance), and then calibrate to peak flows (James and Burges, 1982). The reason for this is that peak flows are somewhat dependent on runoff volume, but volume totals are not very dependent on peak flows. Therefore, this order of calibration was used for this study.

Baffaut and Delleur (1989) provide background information on the calibration of rainfall-runoff models such as SWMM. Model calibration (for any type of model) can be done manually, by adjusting parameters in a trial and error fashion. Or calibration can be done with a computer-based automatic routine. Both approaches were used in this study. Most of the SWMM model calibration was done using software called PEST, which uses mathematical optimization to find appropriate model input parameters (Doherty, 2002). The SCS and Rational Methods are simpler models, and it appeared to be more efficient to calibrate those models by hand. Results of the calibration process are discussed in Chapter Four.

Validation

Validation (sometimes called verification) of a calibrated model is often done by simulating additional recorded events that were not included in the data set used to calibrate the model. Simulated results from the validation events are then compared to measured data. This offers a test of how well the calibrated model can be expected to reproduce additional events.

Three storm events from 2003 were used to try to validate each watershed model. Results of model validation are discussed in Chapter Four.

CHAPTER 4: RESULTS AND DISCUSSION

In this chapter, results from the various models are presented and discussed. First, results from the uncalibrated models are presented. Model results will be compared to the observed data, and the accuracy of the uncalibrated models is discussed. Sensitivity analyses on model input parameters are briefly reviewed. The process and results of model calibration are then presented.

Results of Uncalibrated Modeling

Because many rainfall-runoff models are never calibrated, the accuracy of uncalibrated models was investigated as a part of this study. The models were first created using typical modeling guidelines and procedures for estimating model inputs. Observed rainfall records were then input into each model, and the runoff process simulated. Model performance and results were reviewed to ensure good model stability and low continuity error, but at this stage no attempt was made to adjust the model to match the observed runoff flows.

Lyons Park Creek: Observed Data

Ten rainfall events in the Lyons Park Creek watershed were simulated. Table 4-1 summarizes the observed data from these events.

Table 4-1
Lyons Park Creek events

Date of Storm	Observed Rainfall (in)	Observed Runoff (in)	Runoff / Rainfall Ratio	Observed Peak Flow (cfs)
6/3/2002	2.01	0.97	0.48	81
6/10/2002	0.83	0.31	0.37	153
7/8/2002	0.93	0.31	0.33	85
7/26/2002	1.50	0.52	0.35	155
8/12/2002	5.49	2.80	0.51	147
8/13/2002				503
8/21/2002 - 8/22/02	1.48	0.74	0.50	86
9/2/2002	1.29	0.53	0.41	74
4/30/2003	1.62	0.74	0.46	54
5/9/2003	0.85	0.60	0.71	70
8/3/2003	1.83	0.65	0.36	300
		Mean:	0.45	

Heavy rainfall occurred on both August 12 and 13, 2002. Rainfall and runoff totals from these days were combined and considered as one event. However, a separate distinct hydrograph peak occurred on each day, and is reported separately.

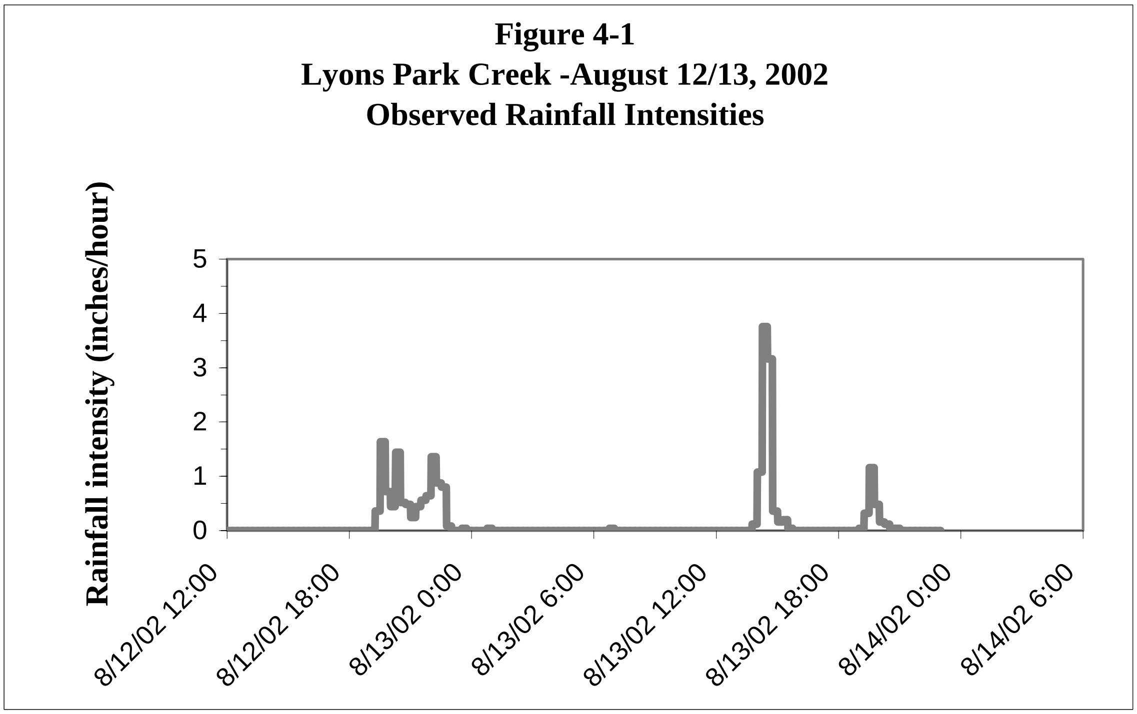
These storms represent the ten largest events, in terms of both rainfall totals and flows, that occurred during the 2002-2003 period of monitoring. The largest event, in terms of rainfall depth, runoff volume and peak flow, was the August 12/13, 2002 event. For each event, a runoff depth was calculated by converting the measured runoff volume (in cubic feet) to an equivalent depth spread over the watershed area. A runoff/rainfall ratio was then calculated by dividing the runoff depth by the rainfall depth. This ratio indicates the fraction of rainfall that became runoff. The mean runoff ratio was 0.45. This value seems reasonable, based on the estimate that 40% of the watershed consists of directly connected impervious area. Most of the rain falling over directly connected impervious areas should become runoff. A small amount infiltrates into the ground through pavement cracks, or is trapped in tree canopies and puddles and eventually evaporates. In all but the heaviest rainfalls, most of the rainfall falling on pervious areas infiltrates into the ground or is held on the surface until evaporated.

The August 12/13, 2002 storm event is notable because the rainfall was very heavy, and the recorded rainfall intensities have a very low probability of occurrence. Table 4-2 shows the highest rainfall totals recorded during this event for certain lengths of time. Figure 4-1 on the following page shows the pattern of rainfall intensity for this event.

Table 4-2
Rainfall during August 12/13, 2002 Event
At Lyons Park Creek

Rainfall duration	Maximum Rain Depth (in)	Estimated Return Period
5 minutes	0.56	10 yr
10 minutes	1.03	25 yr - 50 yr
15 minutes	1.28	25 yr - 50 yr
30 minutes	1.75	25 yr - 50 yr
1 hour	2.10	10 yr- 25 yr
3 hours	2.41	10 yr
6 hours	2.70	5 yr- 10 yr
12 hours	2.81	5 yr
24 hours	4.94	~ 50 yr
48 hours	5.49	50 yr

The estimated return period (also called the average recurrence interval)l is how often, over a long time period, a rainfall of a certain duration and depth is likely to occur. Return periods are estimated based on statistical analyses of long rainfall records. Rainfall probability data reported in Loucks et al. (2000) were used to estimate return periods for the rainfall depths occurring during this storm event. For both short and long periods of rainfall, the recorded depths had a return period approaching 50 years. This rare, large event had a magnitude in the range that is often used by engineers and planners to design drainage infrastructure. Therefore, rainfall-runoff models are often used to simulate storms of this magnitude. It is fortunate that a runoff event this large is included in the data available for this project.



Overview of Uncalibrated Model Results for Lyons Park Creek

The following graphs compare the observed and simulated runoff volumes and peak flows for several of the rainfall-runoff models used. Because the clustering of data points for the smaller storms can obscure some data, a portion of each graph is repeated at a finer scale, to show results from the smaller storms in more detail.

Figure 4-2
Runoff Volumes From Uncalibrated Lyons Park Creek Models

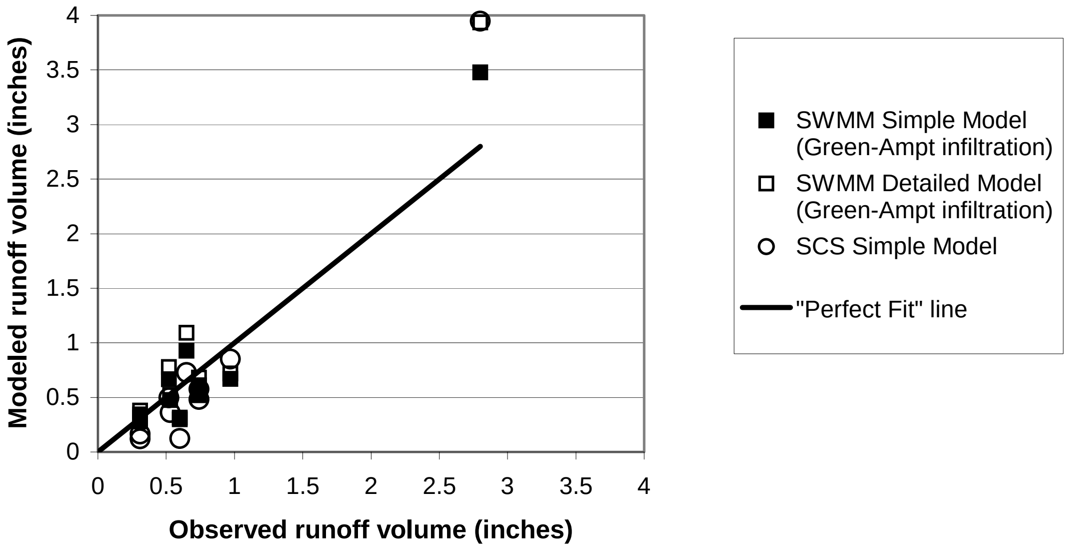


Figure 4-3
Runoff Volumes from Uncalibrated Lyons Park Creek Models
Inset for Volumes Under 1.5"

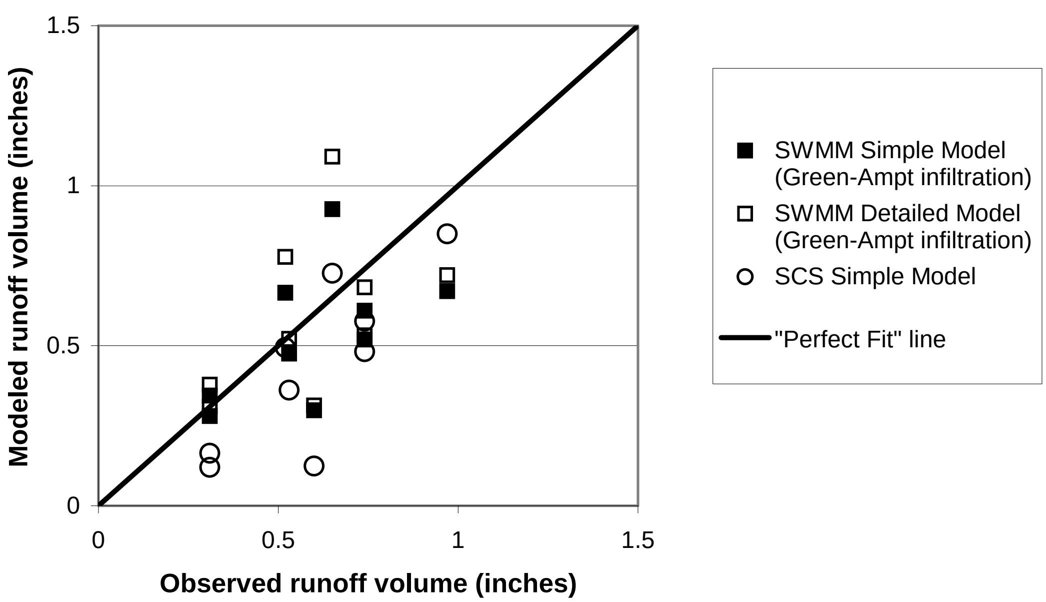


Figure 4-4
Peak Flows from Uncalibrated Lyons Park Creek Models

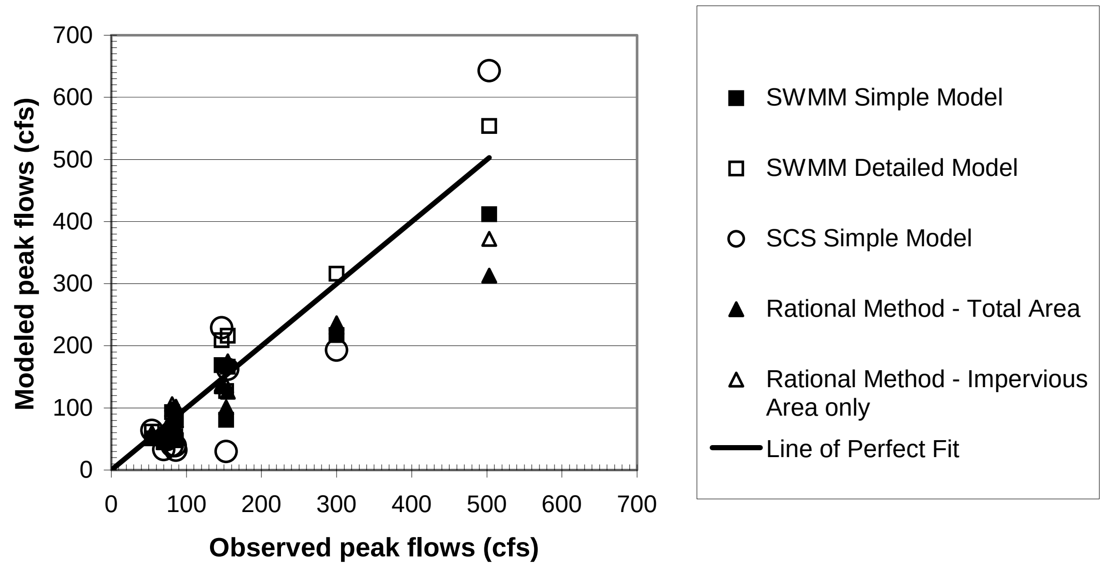
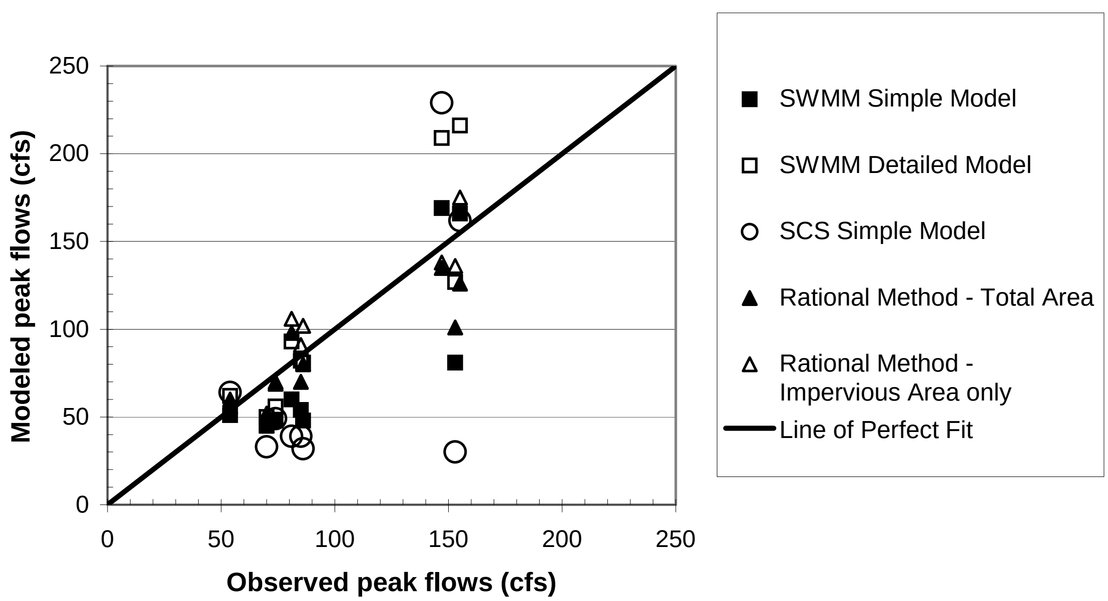


Figure 4-5
Peak Flows from Uncalibrated Lyons Park Creek Models
Inset for Flows Under 250 cfs



Measurement of Errors and Goodness of Fit

Graphs provide a visual representation of how the model results compare to measured values. It is also useful to quantify how well the models fit the observed data, by computing various measures of error or goodness of fit. Many equations and methods for quantifying model error exist. For this study, the following simple error measures were chosen:

For each event, the absolute errors in peak flow and runoff volume were calculated as follows.

Absolute peak flow error = (Simulated peak flow) – (Observed peak flow)

(Eqn 4-1)

Absolute runoff volume error = (Simulated runoff volume) – (Observed runoff volume)

(Eqn 4-2)

Similarly, a percent error was calculated as follows.

$$Percent\ error\ in\ peak\ flow = \frac{Simulated\ peak - Observed\ peak}{Observed\ peak} * 100 \qquad \text{(Eqn 4-3)}$$

The percent error in runoff volume was calculated using an equation similar to (4-3).

The above equations provide measures of error for individual storm events. Two measures of error were computed for each model for the entire range of modeled events: mean error and root mean square error.

$$Mean\ error = \frac{\sum(error\ \ for\ each\ event)}{Number\ of\ \ events} \qquad \text{(Eqn 4-4)}$$

$$Root\ Mean\ Square\ Error\ (RMSE) = \sqrt{\frac{\sum(error\ \ for\ each\ event)^2}{(number\ of\ \ events)}} \qquad \text{(Eqn 4-5)}$$

The equations for mean error and RMSE were applied to both the absolute errors and percent errors for peak flows and runoff volumes.

The reason for using multiple measures of error is that no one measure seemed adequate to describe model performance. For instance, consider the difference between absolute error and percentage error. Using absolute error gives more weight to errors from large storm events. For models designed to simulate large runoff events for drainage design purposes, this may be appropriate; errors in the simulation of smaller runoff events may not be as important. But for hydrologic models where accuracy across a wide range of event sizes is important, measuring error on a percentage basis may be more appropriate.

Consider also the difference between the calculation of mean error and root mean square error (RMSE). The mean error provides information on the overall accuracy of the model and consistent overprediction or underprediction. Underpredictions for some events can be masked by overpredictions for other events. In contrast, the RMSE provides information on the variance of the errors. For these reasons, multiple measures of the error are presented for each model.

Table 4-3 presents the errors in predicting runoff volumes for each of the models.

Table 4-3
Errors in Runoff Volumes
for Uncalibrated Lyons Park Creek Models

Model	Mean Absolute Error (in)	Absolute RMSE (in)	Mean Percentage Error	Percentage RMSE
SWMM - Simple	-0.05	0.21	-10%	26%
SWMM - Detailed	0.02	0.31	-3%	29%
SCS - Simple	-0.03	0.42	-24%	41%
SCS- Detailed	-0.04	0.41	-25%	41%

SWMM models used continuous simulation, Green-Ampt infiltration procedure

Errors in runoff volume prediction are presented for the SWMM and SCS models. The Rational Method does not predict runoff volume. The SWMM models perform better than the SCS models. If one considers the volume errors from individual storm events (by looking at Figures 4-2 and 4-3, or reviewing the storm-specific information in the Appendix), there is an apparent trend for the SCS model to underpredict volumes for small storm events and overpredict volumes for large storm events.

The following table summarizes the performance of the various uncalibrated rainfall-runoff models of Lyons Park Creek in predicting peak flows.

Table 4-4
Errors in Peak Flows
for Uncalibrated Lyons Park Creek Models

Model	Mean Absolute Error (cfs)	Absolute RMSE (cfs)	Mean Percentage Error	Percentage RMSE
SWMM - Simple	-35	49	-25%	31%
SWMM - Detailed	8	28	0%	20%
SCS - Simple	-18	75	-24%	48%
SCS- Detailed	-3	76	-14%	48%
Rational Method - Total Area	-34	64	-13%	22%
Rational Method - Impervious Area	-17	46	-3%	19%

SWMM models used continuous simulation, Green-Ampt infiltration procedure

Mean percentage errors in predicting peak flows ranged from 0% to –25%. Given the uncertainties involved in rainfall-runoff modeling, and the fact that no model calibration was performed, that range of error percentages is not unexpected, perhaps even better than one would normally expect. Errors for individual storm events are presented in the Appendix. Percentage errors for individual storm events ranged from –80% to +75%. There is much more uncertainty in the prediction of peak flows from individual storm events.

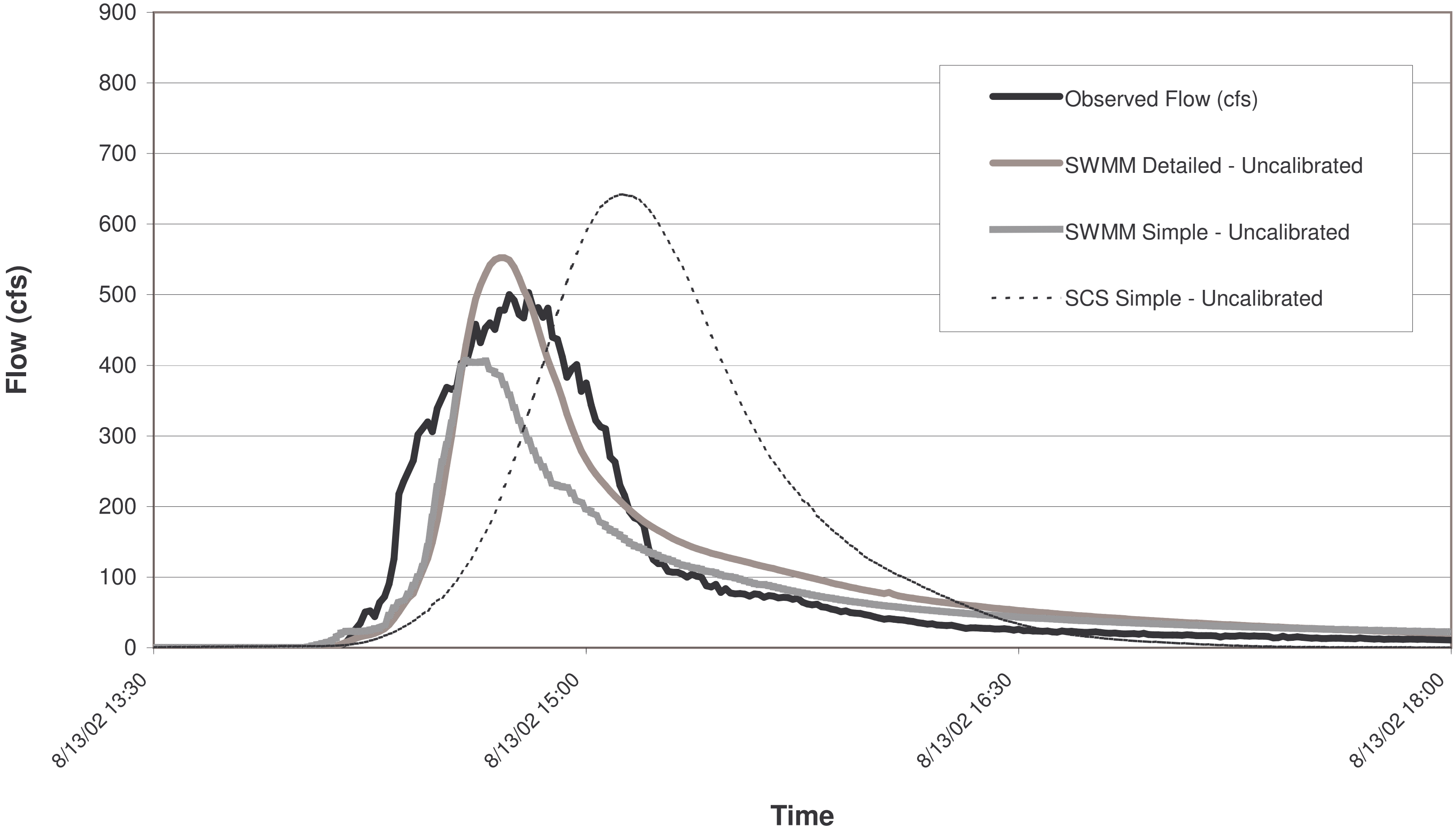
With the exception of the detailed SWMM model, the mean absolute errors and percentage errors are less than zero, indicating a tendency to underpredict peak flows. However, as Figures 4-4 and 4-5 show, there is considerable scatter in the data, and many individual storm events are overpredicted. The detailed SWMM model appears to be the best overall performer. Its mean error and RMSE, on both an absolute and percentage basis, are consistently among the lowest of any models. This is visually apparent in Figure 4-4. The detailed SWMM model seems to predict peak flows the most accurately, with less variability and bias than the other models.

The detailed SCS model does have the lowest mean error on an absolute basis. However, the RMSEs of both the simple and detailed SCS models are the worst of any of the models. This indicates high variability in the results of the SCS models. Figures 4-4 and 4-5 show that the SCS model predictions are consistently farthest from the true values.

The Rational Method, particularly when applied to the impervious area of the watershed only, also performs well. The impervious area Rational Method model has the lowest RMSE on a percentage basis, and its mean percentage error and absolute RMSE are second only to the detailed SWMM model. Figures 4-4 and 4-5 indicate graphically that both of the Rational Method models tend to predict peak flows fairly well.

Another way to assess model performance is to compare simulated hydrographs with the observed hydrograph. Following is a sample of such a comparison. This graph compares the observed August 13, 2002 hydrograph (the largest event recorded on Lyons Park Creek) with simulated hydrographs from the uncalibrated models. Additional hydrographs from other storm events are included in the Appendix.

Figure 4-6
Lyons Park Creek - August 13, 2002
Uncalibrated Models



SWMM Infiltration Routines and Continuous vs. Event Simulation

SWMM offers the choice of two different routines to model infiltration into pervious soil: the Horton model and the Green-Ampt model. Models were run using both methods, to see if any conclusion could be drawn about the relative performance of the two methods.

There are also two different scales of simulation length for which SWMM can be run. In single-event simulation (sometimes called simply event simulation), each storm event is simulated with a separate model run. The model run begins shortly before the beginning of rainfall for that event. The model user must estimate the antecedent conditions for the event, such as the current saturation of the soil. These antecedent conditions can have a significant impact on the generation of runoff. For example, if the soil is nearly saturated from previous rainfall, runoff from pervious areas will occur much more quickly. At the end of a single-event simulation, the model run ends, and no information is directly transferred to simulations of later events.

In continuous simulation, multiple rainfall events, along with the intervening dry periods are simulated. Continuous simulation runs can cover months or years of time. Model algorithms represent how the watershed "recovers" after each storm, such as how soil moisture changes, or how water in surface depressions evaporates. In theory, continuous simulation is more accurate than event modeling, because the user does not need to estimate antecedent conditions at the start of each rainfall event.

Despite the advantages of continuous simulation, much rainfall-runoff modeling is still done on an event basis. One reason is that continuous simulation requires more data, and often more effort to process and analyze the results. Also, rainfall-runoff models for drainage design often simulate hypothetical "design storms", rather than observed rainfall patterns. Finally, certain rainfall-runoff models, such as the SCS and Rational models, can only be used for event simulation – the model algorithms are not designed for continuous simulation.

Because SWMM can be applied to either single events or continuous simulation, a comparison of the two methods was made as part of this study. The infiltration routines are most affected by the simulation length, because they are designed to account for soil moisture and infiltration potential. Therefore, the following tables show how both the choice of infiltration routine and the time scale of the simulation influence model results.

Table 4-5
Effect of Infiltration Method and Continuous Simulation on Runoff Volumes
Lyons Park Creek Models

Model	Infiltration method	Single-event or continuous simulation?	Mean Absolute Error (in)	Absolute RMSE (in)	Mean Percentage Error	Percentage RMSE
SWMM Simple	Horton	Single-event	-0.02	0.23	-7%	28%
SWMM Simple	Green-Ampt	Single-event	0.01	0.28	-4%	29%
SWMM Detailed	Horton	Single-event	0.07	0.35	3%	34%
SWMM Detailed	Green-Ampt	Single-event	0.11	0.42	7%	36%
SWMM Simple	Horton	Continuous	-0.02	0.23	-7%	27%
SWMM Simple	Green-Ampt	Continuous	-0.05	0.21	-10%	26%
SWMM Detailed	Horton	Continuous	0.07	0.35	4%	34%
SWMM Detailed	Green-Ampt	Continuous	0.02	0.31	-3%	29%

Table 4-6
Effect of Infiltration Method and Continuous Simulation on Peak Flows
Lyons Park Creek Models

Model	Infiltration method	Single-event or continuous simulation?	Mean Absolute Error (cfs)	Absolute RMSE (cfs)	Mean Percentage Error	Percentage RMSE
SWMM Simple	Horton	Single-event	-34	49	-24%	31%
SWMM Simple	Green-Ampt	Single-event	-32	48	-23%	30%
SWMM Detailed	Horton	Single-event	10	32	3%	22%
SWMM Detailed	Green-Ampt	Single-event	13	33	4%	23%
SWMM Simple	Horton	Continuous	-34	49	-24%	31%
SWMM Simple	Green-Ampt	Continuous	-35	49	-25%	31%
SWMM Detailed	Horton	Continuous	10	31	3%	22%
SWMM Detailed	Green-Ampt	Continuous	8	28	0%	20%

The differences in results between the two infiltration routines are minor. When single events are modeled, the Horton method seems to provide slightly more accurate results. When continuous simulation is used, the Green-Ampt method is slightly more accurate.

For individual storm events, the simulated peak flows and volumes from the two infiltration methods are usually very close. This may be because most of the runoff is originating from impervious surfaces, which are modeled separately from the infiltration routines.

When using the Horton infiltration method, there is little difference in results between continuous simulation and single-event simulation. When using the Green-Ampt method, continuous simulation offers some improvement over single-event simulation.

In summary, the type of infiltration method used and the time scale of the simulation (continuous vs. event) do not appear to influence the model results as much as other factors such as the overall rainfall-runoff model and the level of model discretization. Therefore, in most comparisons and evaluations of SWMM, results from continuous simulation, Green-Ampt infiltration runs will be reported, to minimize the information presented. Detailed results from all SWMM modeling methods are available in the Appendix.

Levels of Discretization

As discussed in the previous chapter, models with different levels of discretization were assembled for both the SWMM and SCS methods. In the “simple” models, the watershed was simulated as one catchment, with no routing through the drainage system. In the detailed models, the watershed was broken up into five smaller catchments, and routing through the drainage system of pipes and channels was also simulated. Tables 4-3 and 4-4 show results from both levels of discretization.

Table 4-3 shows that mean error in runoff volume for the detailed SWMM model is lower than the simple model, but RMSE is higher. Volumes from the SCS models are nearly identical for the simple and detailed models. From Table 4-4, it is apparent that the detailed SWMM model predicts peak flows better than the simple SWMM model. Both the mean errors and RMSEs are smaller for the more detailed models. The detailed SCS model also has lower mean errors than the simple SCS model, though the RMSE for the two levels of discretization is nearly identical.

Eighteenth Street Storm Sewer: Observed Data

Table 4-7 shows the recorded events for the Eighteenth Street Storm Sewer watershed that were modeled. Again, the ten largest events during the monitoring period in 2002 and 2003 were selected.

Table 4-7
Eighteenth Street Storm Sewer events

Date of Storm	Observed Rainfall (in)	Observed Runoff (in)	Runoff / Rainfall Ratio	Observed Peak Flow (cfs)
6/3/2002	2.26	0.48	0.21	28
7/8/2002	0.97	0.16	0.16	26
7/26/2002	1.25	0.21	0.17	19
8/12/2002	3.55	0.71	0.20	47
8/13/2002				27
8/21/2002	1.92	0.36	0.19	50
9/2/2002	1.2	0.21	0.18	13
4/30/2003	1.34	0.18	0.13	5
5/9/2003	0.84	0.11	0.13	5
7/6/2003	0.90	0.10	0.11	29
8/6/2003	0.75	0.08	0.11	31
		Mean:	0.16	

The most noticeable feature of the data from the Eighteenth Street watershed is the low runoff/rainfall ratio. Based on detailed topographic mapping and aerial photographs, as well as several site visits, the percentage of directly connected impervious area in the watershed was estimated at 35%. However, the mean runoff/rainfall ratio is 0.16, or 16%. The highest runoff/rainfall ration is 0.21. Therefore, much less runoff was measured than was expected, based on the estimated imperviousness in the watershed.

One possible reason for the low observed runoff volumes is measurement error. The USGS was contacted regarding this possibility. USGS staff were confident that the recorded flows were reasonably accurate, though they agreed that the runoff volume did seem low. If the measurements are accurate, then it appears that over half of the rainfall over directly connected impervious areas did not travel to the storm sewer, which is unusual. The results from the Lyons Park Creek watershed were more typical, with average runoff ratios being slightly higher than the amount of connected impervious area in the watershed.

The low recorded runoff volumes had a major influence on how well the uncalibrated models of the Eighteenth Street watershed performed, and how the models were calibrated. Therefore, this phenomena is worth considering in detail. First, it should be noted that the estimate of 35% imperviousness includes only “connected” impervious areas that have a direct path to the street gutters or storm sewer networks. Sometimes impervious areas are “disconnected” from the drainage system. Examples are roof downspouts that drain onto grassed areas away from any pavement or pipe inlets. Based on site visits to the watershed, impervious surfaces which appeared to be disconnected were identified, and are not included in the 35% estimate.

Other researchers have also observed runoff volumes that are much lower than the percentage of connected impervious area in a watershed. One of the small Florida watersheds studied by Trommer et al. (1996) was estimated to contain 85% effective impervious surfaces, but on average watershed runoff volume was only 66% of rainfall volume. However, runoff volume from most of the other watersheds in the Trommer study equaled or exceeded the amount of impervious area.

Jones et al. (2003) reported rainfall and runoff volumes from several small urban catchments in British Columbia. One of the catchments consisted entirely of a single flat tar and gravel rooftop on an office building. The entire roof drained to one drainage sump, where flow was measured with a weir. Because of the assumed watertight nature of the roofing material, the connected impervious of the catchment was initially assumed to be 100%. However, the total runoff volume was only 82% of the rainfall volume. The authors hypothesized that some rainfall was stored in the gravel and subsequently evaporated. Hence, even though there was no apparent disconnection of impervious area from the drainage system, some water loss from impervious area occurred.

Another watershed studied by Jones et al. (2003) was the 590-acre Upper Serpentine River watershed. The total percentage of imperviousness in the watershed was estimated to be 66%. No major patterns of impervious area disconnection (i.e. disconnected roof drains) were known in the watershed. However, the total measured runoff volume was only 52% of the rainfall volume. Therefore, significant losses were occurring from the impervious areas. The authors hypothesized that again some water was trapped on roofs, particularly flat roofs. A strong seasonal difference was also observed. During one winter month (the rainy season in this region of British Columbia), monthly runoff volume was 66% of rainfall volume, indicating that runoff was probably also occurring from saturated pervious areas. In contrast, during a summer month (a drier period), runoff volume was only 25% of rainfall volume, indicating a large loss from impervious areas.

Waschbusch (2003) reported runoff monitoring results from a study on freeway right-of-way in western Milwaukee County. Precipitation and runoff for two small catchments were measured as part of this study. One basin, the “test basin”, was estimated to be 95% impervious. The control basin was estimated to be 63% impervious. The mean runoff volume from the test basin was 62% of rainfall, while mean runoff from the control basin was 28% of rainfall. Thus, both basins exhibited a large loss of runoff from impervious areas.

Therefore, although the large loss of runoff from impervious areas in the Eighteenth Street watershed is not normal, a similar phenomena has been observed at other sites. What could cause this loss of runoff? Possible reasons include the following:

- Water ponding and then evaporating on flat roofs and pavements
- Water infiltrating into the ground through cracks in the pavement
- Water infiltrating into the ground through cracks in storm sewers
- Rainfall being intercepted and then evaporated from trees overhanging pavement and roofs

One other possible reason for the low measured runoff volume is that roof downspouts thought to be connected to the storm sewer actually drain somewhere else. Watershed visits showed that most of the roof downspouts appeared to be connected to underground pipes. City of Milwaukee staff indicated that when the neighborhood in this watershed was developed, city policy required that roof downspouts be connected to storm sewers via small laterals. Studies by the Milwaukee Metropolitan Sewerage District (Gonwa and Simmons, 2004) have suggested that some roof downspouts in the region are connected to sanitary sewers, though the overall incidence of these connections is estimated to be low. It is also possible that the downspouts are connected to underground footing drains or foundation drains that are pumped onto the lawn rather than to the storm sewer.

Implications of this low runoff volume will be discussed in later sections of this report.

Overview of Uncalibrated Model Results for Eighteenth Street Storm Sewer

As with the Lyons Park Creek watershed, the following graphs compare the observed and simulated peak flows and runoff volumes for the rainfall-runoff models used.

Figure 4-7
Runoff Volumes from Uncalibrated Eighteenth Street Models

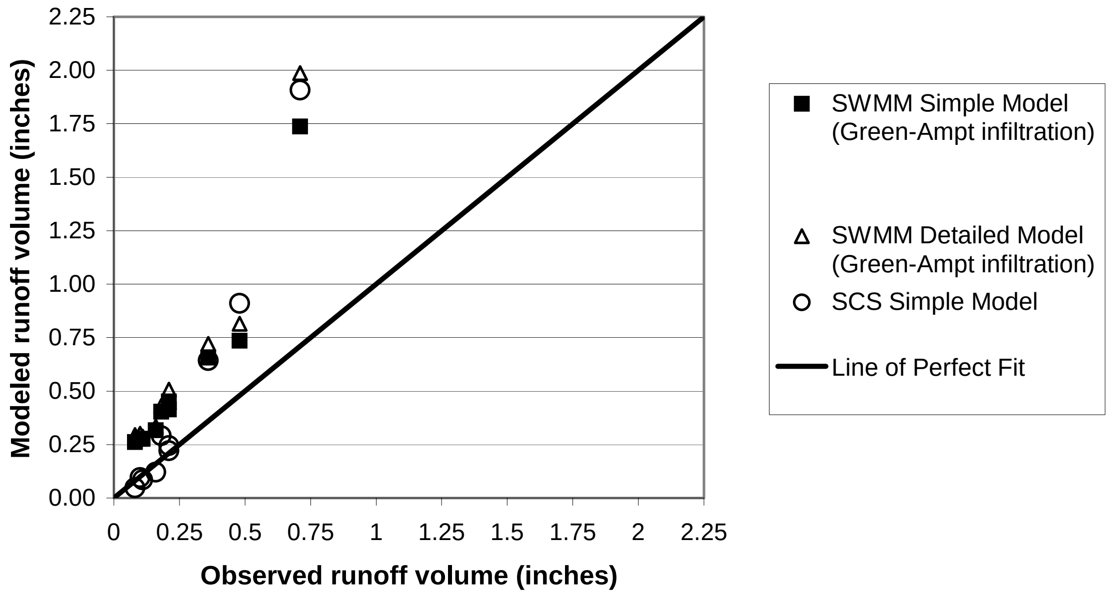


Figure 4-8
Runoff Volumes from Uncalibrated Eighteenth Street Models
Inset for Volumes Under 1"

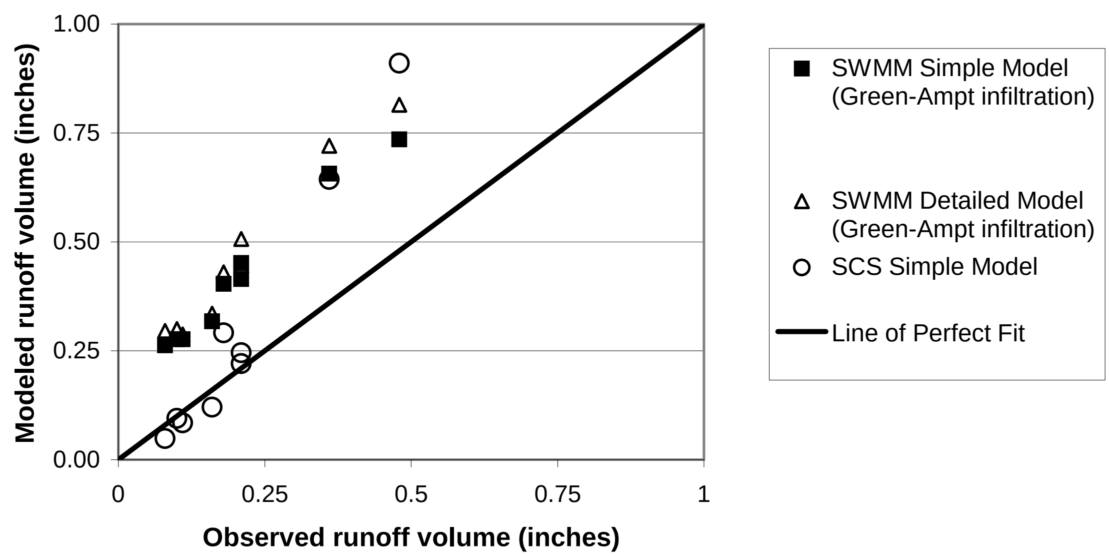


Figure 4-9
Peak Flows from Uncalibrated Eighteenth Street Models

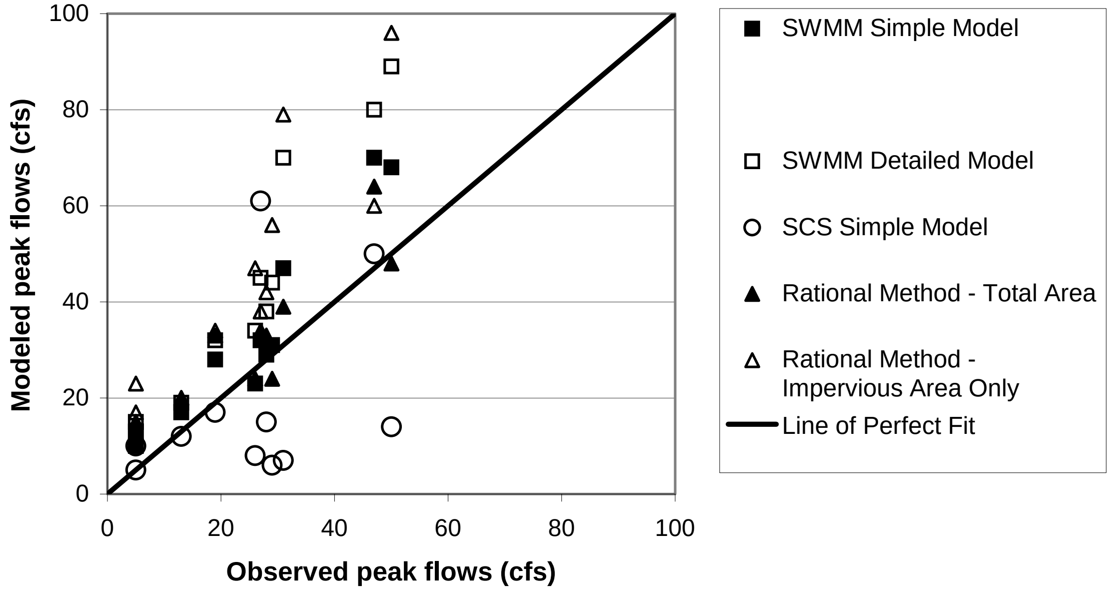
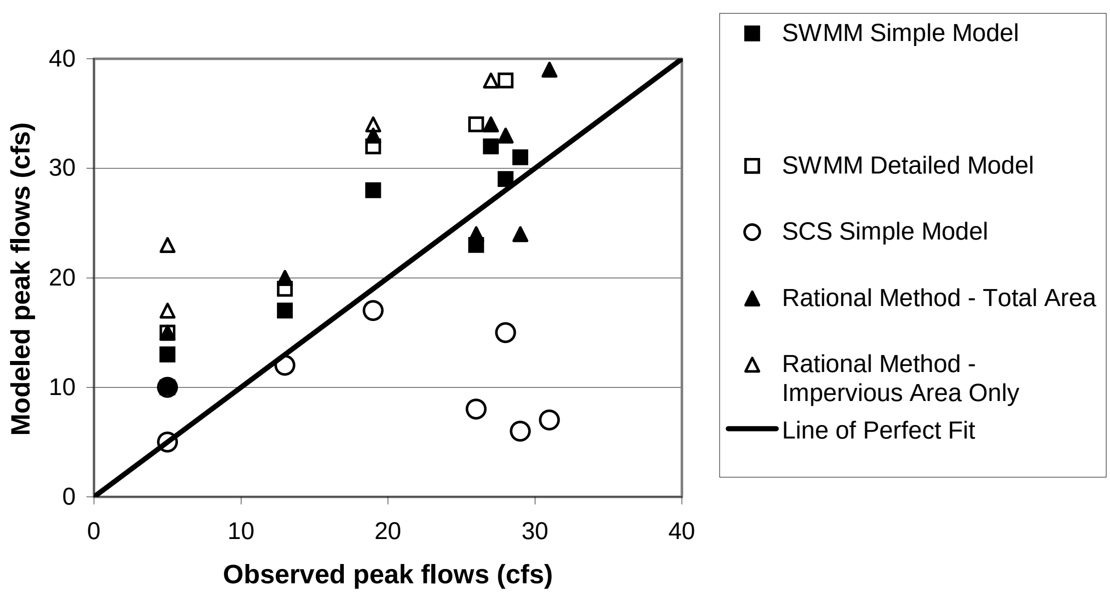


Figure 4-10
Peak Flows from Uncalibrated Eighteenth Street Models
Inset for Flows Under 40 cfs



The error measures described earlier were also applied to the Eighteenth Street Storm Sewer models. Table 4-8 presents the errors in predicting runoff volumes for each of the Eighteenth Street models.

Table 4-8
Errors in Runoff Volume
For Uncalibrated Eighteenth Street Models

Model	Mean Absolute Error (in)	Absolute RMSE (in)	Mean Percentage Error	Percentage RMSE
SWMM - Simple	0.29	0.38	127%	136%
SWMM - Detailed	0.35	0.47	148%	158%
SCS - Simple	0.20	0.41	33%	70%
SCS- Detailed	0.20	0.42	35%	71%

SWMM models used continuous simulation, Green-Ampt infiltration procedure

Runoff volume prediction is much poorer for this watershed than the Lyons Park Creek watershed. Runoff volumes are often dramatically overpredicted by 100% or more. Figure 4-8 does show that the SCS models predicts runoff volumes from small storm events well.

The following table summarizes the performance of the models when predicting peak flows from storm events.

Table 4-9
Errors in Peak Flow
For Uncalibrated Eighteenth Street Models

Model	Mean Absolute Error (cfs)	Absolute RMSE (cfs)	Mean Percentage Error	Percentage RMSE
SWMM - Simple	8	11	45%	64%
SWMM - Detailed	18	22	85%	99%
SCS - Simple	-7	19	-12%	68%
SCS- Detailed	-2	21	10%	82%
Rational Method - Total Area	6	9	55%	91%
Rational Method - Impervious Area	21	25	116%	150%

SWMM models used continuous simulation, Green-Ampt infiltration procedure

The results show that peak flow prediction for this watershed is much worse than peak flow prediction in the Lyons Park Creek watershed. The mean errors, as well as Figures 4-9 and 4-10, show that most models consistently overpredict peak flows. Figure 4-10 shows that the SCS model sometimes underpredicts peak flows by a wide margin. It is difficult to draw many conclusions about which models perform better than others. The Rational Method, applied to the impervious area only, has the worst error measures. However, the total-area Rational Method results are comparable with the other models. For this watershed, the simple SWMM model has less error than the detailed SWMM model. Errors for the Eighteenth Street SCS models are generally comparable or better than the other models, whereas the Lyons Park Creek SCS models were noticeably worse than the other models applied to that watershed.

Because the Lyons Park Creek simulation results showed little difference in SWMM results between continuous and single-event simulations, only continuous simulation was done for the Eighteenth Street SWMM models. There were only minor differences in runoff volumes between the Horton and Green-Ampt infiltration methods used in SWMM. The Green-Ampt method tended to predict somewhat higher infiltration and therefore lower runoff volumes than the Horton method. There was almost no difference in peak flows between the two infiltration methods.

The effect of the level of model discretization was significant, especially for peak flows. As with the Lyons Park Creek models, the more detailed models generated higher peak flows and runoff volumes. The simple Eighteenth Street SWMM model had lower error than the more detailed SWMM model. Although the simple SCS model predicted lower peak flows than the detailed SCS model, errors for the two models were similar.

Summary of Uncalibrated Modeling

The error measures for the uncalibrated Lyons Park Creek models were fairly good; it would be unreasonable to expect an uncalibrated model to be much better. The errors were much worse for the Eighteenth Street Storm Sewer models. The errors in the Eighteenth Street models were highly affected by the low runoff volumes measured in this watershed, which appears to have been an unusual condition.

Conclusions that can be drawn from the uncalibrated modeling will be discussed in the following chapter.

Model Calibration

Before model calibration began, sensitivity analyses were conducted on model input parameters. The procedure for these sensitivity analyses was described in the previous chapter. Peak flow results from SWMM were most sensitive to the impervious area percentage, subcatchment width and the impervious surface roughness. Because subcatchment width is more subjective than surface roughness, width will be the primary parameter adjusted to try to match observed peak flows. Runoff volume results from SWMM were most sensitive to the impervious area percentage and various infiltration parameters, such as saturated infiltration rate and initial moisture deficit.

The rainfall-runoff models were then calibrated to observed peak flows and runoff volumes from the 2002 events. The SWMM models were calibrated with the assistance of the optimization software PEST. PEST optimizes input parameters by minimizing the sum of the squared absolute errors. Therefore, the larger events, with higher absolute peaks and volumes, have more influence on the optimization. This can be balanced by assigning higher weights to the observed peaks and volumes from smaller storms. Both unweighted and weighted optimizations were performed using PEST, and the parameter sets from each optimization were averaged to obtain the final parameter values.

The The SCS and Rational Method models were simple enough that it appeared easier to calibrate them using a manual trial-and-error process. Only the simple SCS models were calibrated. Calibrating the detailed SCS model might have provided some additional information about the effects of model discretization, but that issue had already been explored in other phases of the modeling.

Tables 4-10 through 4-13 compare model errors before and after calibration for the two watersheds.

As mentioned in Chapter 4, only the 2002 storm events were used for calibration. After the models were calibrated, the 2003 storm events were then simulated, in an attempt at model validation. The results presented in this chapter include both the 2002 and 2003 storm events. The Appendix includes separate error analyses for the 2002 and 2003 events. For Lyons Park Creek, errors for the two years were fairly similar when using the calibrated models. For the calibrated Eighteenth Street models, errors for 2003 events were higher than errors for the 2002 events. As discussed earlier, one possible reason for this error is the 2003 data may be less reliable than the 2002 data. It is also possible that a better calibration of the Eighteenth Street models could be obtained by also using the 2003 data in the calibration, instead of applying them separately for model validation.

Table 4-10
Changes in Model Error with Calibration
Runoff Volumes - Lyons Park Creek

Model	Mean Absolute Error (inches)		Absolute RMSE (inches)		Mean % Error		Percentage RMSE	
	Uncalibrated	Calibrated	Uncalibrated	Calibrated	Uncalibrated	Calibrated	Uncalibrated	Calibrated
SWMM - Simple	-0.05	-0.10	0.21	0.18	-10%	-13%	26%	24%
SWMM - Detailed	0.02	-0.04	0.31	0.15	-3%	-3%	29%	23%
SCS - Simple Model, Composite Curve Number	-0.03	-0.03	0.42	0.42	-24%	-24%	41%	41%
SCS- Simple Model, Separate Impervious CN	NS	-0.03	NS	0.19	NS	-10%	NS	22%

NS = not simulated

Table 4-11
Changes in Model Error with Calibration
Peak Flows - Lyons Park Creek

Model	Mean Absolute Error (cfs)		Absolute RMSE (cfs)		Mean % Error		Percentage RMSE	
	Uncalibrated	Calibrated	Uncalibrated	Calibrated	Uncalibrated	Calibrated	Uncalibrated	Calibrated
SWMM - Simple	-35	-7	49	26	-25%	-9%	31%	19%
SWMM - Detailed	8	-4	28	21	0%	-6%	20%	18%
SCS - Simple Model, Composite Curve Number	-18	13	75	111	-24%	-9%	48%	50%
SCS- Simple Model, Separate Impervious CN	NS	-1	NS	28	NS	-3%	NS	17%
Rational Method - Total Area	-34	-3	64	38	-13%	9%	22%	23%
Rational Method - Impervious Area	-17	-8	46	40	-3%	3%	19%	20%

NS = not simulated

Table 4-12
Changes in Model Error with Calibration
Runoff Volumes - Eighteenth Street Storm Sewer

Model	Mean Absolute Error (inches)		Absolute RMSE		Mean % Error		Percentage RMSE	
	Uncalibrated	Calibrated	Uncalibrated	Calibrated	Uncalibrated	Calibrated	Uncalibrated	Calibrated
SWMM - Simple	0.29	0.02	0.38	0.04	127%	21%	136%	33%
SWMM - Detailed	0.35	0.02	0.47	0.04	148%	23%	158%	34%
SCS - Simple Model, Composite Curve Number	0.20	-0.02	0.41	0.20	33%	-48%	70%	72%
SCS- Simple Model, Separate Impervious CN	NS	0.01	NS	0.03	NS	13%	NS	23%

NS = not simulated

Table 4-13
Changes in Model Error with Calibration
Peak Flows - Eighteenth Street Storm Sewer

Model	Mean Absolute Error (cfs)		Absolute RMSE		Mean % Error		Percentage RMSE	
	Uncalibrated	Calibrated	Uncalibrated	Calibrated	Uncalibrated	Calibrated	Uncalibrated	Calibrated
SWMM - Simple	8	1	11	5	45%	13%	64%	38%
SWMM - Detailed	18	0	22	5	85%	10%	99%	33%
SCS - Simple Model, Composite Curve Number	-7	-13	19	24	-12%	-52%	68%	83%
SCS- Simple Model, Separate Impervious CN	NS	3	NS	6	NS	27%	NS	52%
Rational Method - Total Area	6	3	9	7	55%	30%	91%	60%
Rational Method - Impervious Area	21	3	25	6	116%	28%	150%	58%

NS = not simulated

As the tables show, after calibration there were some moderate improvements in the ability of the Lyons Park Creek models to predict peak flows and runoff volumes. Calibration of the Eighteenth Street models resulted in much more improvement, probably because those models had much higher errors originally. To reproduce the low measured runoff volumes in the Eighteenth Street watershed, during calibration the percentage of connected impervious area was reduced to 22%, from the original estimate of 35%. As discussed earlier, possible reasons for this loss of effective impervious area includes connections to the sanitary sewer, interception of rainfall by trees, infiltration through cracks in the pavement, and ponding on flat roofs and pavements. Without further field investigation, it is impossible to determine what the exact cause is.

Some error measures actually increased after calibration. This is because multiple error measures were used to evaluate the fit of the model, and the error measures were not always directly correlated with one another. For example, calibration almost always decreased the variation in the model results, decreasing the RMSE. However, this sometimes caused the mean error to increase somewhat, because of the results from certain individual storm events.

The SCS models which used a composite curve number (lumping impervious and pervious area together) were the most difficult to calibrate. Unique curve numbers were calibrated for each storm event, allowing a perfect match to observed runoff volumes for specific events. A single constant curve number was then selected that attempted to minimize the various error measures related to volumes. Times of concentration were also calibrated for each storm event, and a representative constant time of concentration then chosen. When this constant curve number and time of concentration was used to simulate the observed storm events, some error measures were much worse than the uncalibrated model. What appears to be occurring is that it is difficult to find a set of input parameters for this model that consistently reduced the error in both peak flows and runoff volumes. Although the model can be calibrated for individual events, using this information to calculate a representative curve number and time of concentration did little to improve the model.

When the SCS models were modified by separating the watershed into separate pervious and impervious areas, model accuracy increased dramatically. It became possible to obtain a good calibration using the SCS method, equal to or better than the other models. An uncalibrated model using separate impervious and pervious curve numbers was not assembled during the initial modeling. The intent of the uncalibrated modeling effort was to replicate typical modeling practice, such as the use of a composite curve number.

It was interesting to note that the time of concentration for Lyons Park Creek, determined by calibrating the SCS model to observed peak flows, was very close to the original estimated value. This does not necessarily mean that this value is the true time of concentration of the watershed, but it seems to be appropriate for the SCS model.

Times of concentration can also be estimated from observed runoff hydrographs. This was done for the Lyons Park Creek watershed, and these times were much shorter than those calculated using the theoretical equations and models. Two methods were used. Both methods are described in SCS model documentation such as the Natural Resources Conservation Service *National Engineering Handbook* (various dates), as well as McCuen et al. (1984). In Method 1, the time of concentration is assumed equal to the time between the end of excess rainfall, and the inflection point on the receding limb of the hydrograph. In Method 2, the "lag time" is first computed. Lag time is defined as the time between the center of mass of excess rainfall and the hydrograph peak, which can be computed from the observed hyetograph and hydrograph. The SCS has developed an empirical relationship where the time of concentration is equal to 1.67 multiplied by the lag time. The following table shows the time of concentration computed using these two methods for the Lyons Park Creek 2002 storm events. As the table shows, the average time of concentration from the observed data is approximately 20 to 25 minutes.

Table 4-14
Time of Concentration Estimates from Observed Hydrographs

Date of Storm	Estimated inflection point	Estimated end of excess rainfall	Tc by Method 1 (hour:minutes)	Estimated center of mass of rain	Time of peak flow	Lag time	Tc by Method 2 (hour:minutes)
6/3/2002	8:29	8:13	0:16	7:57	8:13	0:16	0:26
6/10/2002	21:09	20:55	0:14	20:43	20:55	0:12	0:20
7/8/2002	22:24	22:04	0:20	21:58	22:14	0:16	0:26
7/26/2002	3:38	3:26	0:12	3:12	3:29	0:17	0:28
8/12/2002	22:57	22:37	0:20	22:18	22:26	0:08	0:13
8/13/2002	15:09	14:44	0:25	14:27	14:48	0:21	0:35
8/21/2002 - 8/22/02	21:04	20:50	0:14	20:33	20:48	0:15	0:25
9/2/2002	6:37	6:20	0:17	5:45	6:20	0:35	0:58
Means			0:17				0:29

Because multiple error measures were used to evaluate each calibration attempt, and calibration was done with both PEST and by hand, the calibration process was somewhat subjective. It is possible that with further model calibration, minor improvements in the accuracy of the calibrated models could be made. With the application of numerous different rainfall-runoff models to two different watersheds, it was not possible to do an exhaustive calibration of each model.

Figures 4-11 and 4-12 compare hydrographs for uncalibrated and calibrated models for the August 13, 2002 event on Lyons Park Creek. Figure 4-11 shows that calibration of the SWMM models resulting in only minor improvements in the simulated hydrograph; the hydrographs from the uncalibrated models already matches the observed hydrographs fairly well. Figure 4-12 shows how the uncalibrated and calibrated SCS models compared to the observed hydrograph. The calibrated SCS model separating the impervious and pervious areas matches the observed hydrograph the best. The model that used a composite, mean curve number calculated from event-calibrated curve numbers appears the most different from the observed hydrograph.

Figure 4-11
Lyons Park Creek - August 13, 2002
Uncalibrated and Calibrated SWMM Models

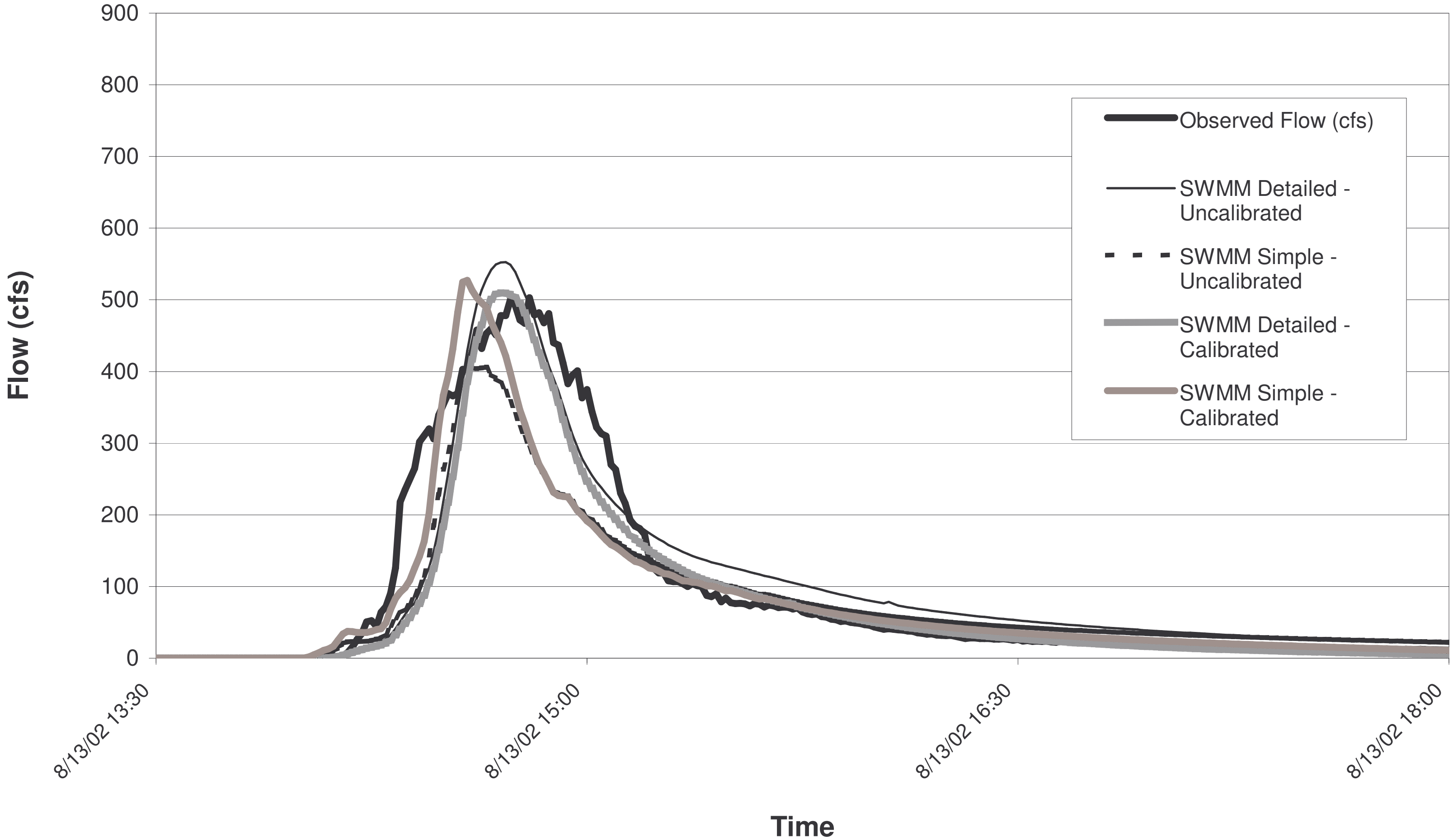
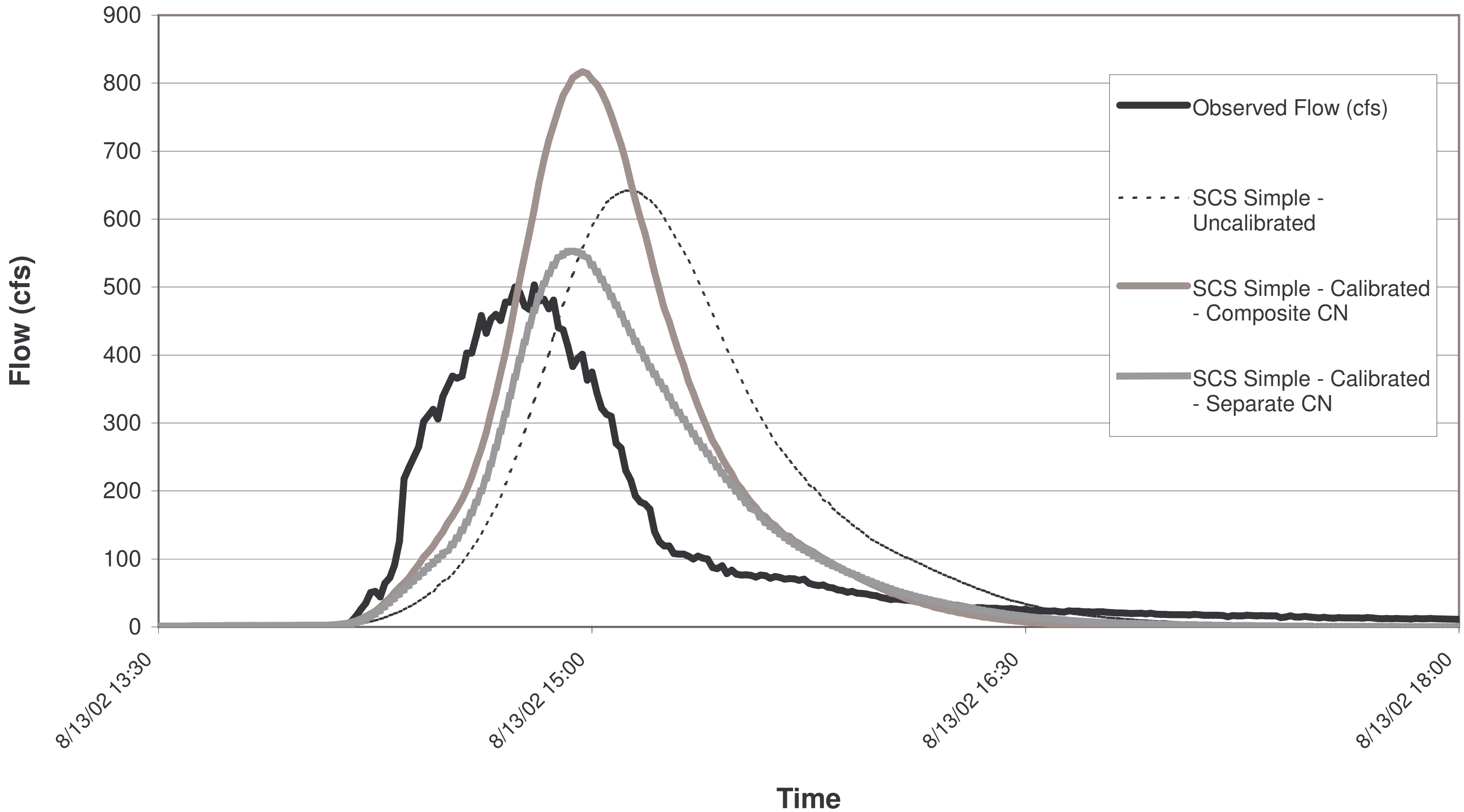


Figure 4-12
August 13, 2002
Uncalibrated and Calibrated SCS Models



CHAPTER 5: CONCLUSIONS

In the Introduction to this report, several questions regarding the performance of uncalibrated and calibrated rainfall-runoff models were presented. It is difficult to conclusively answer these questions by studying and modeling two watersheds. But by reviewing the modeling performed for this study, in conjunction with work performed by other researchers, some insights on rainfall-runoff modeling can be gained. In the following sections, the questions posed in the Introduction will be revisited and discussed.

The recorded runoff volumes in the Eighteenth Street watershed were much lower than normal, given the amount of impervious area in the watershed. As a result, most of the Eighteenth Street uncalibrated models overpredicted peak flows and runoff volumes by a large margin. Although other researchers have reported significant runoff losses from impervious areas, this is an unusual condition. Because of this, caution should be used when extrapolating observations from this watershed to others. When discussing model performance, sometimes more weight will be given to the results from Lyons Park Creek, because the observations from this watershed were more typical of urban watershed conditions. For planning and design of infrastructure, modelers should probably assume that almost all rainfall on connected impervious areas becomes runoff, unless watershed-specific observations indicate otherwise.

Conclusions on Uncalibrated Models

How well can uncalibrated models simulate observed peak flows and runoff volumes?

As discussed earlier, the majority of urban rainfall-runoff models are never calibrated, because no observed data is available. Therefore, it is useful to make some observations about how well uncalibrated models can simulate observed stormwater flow.

Mean errors in peak flow for the uncalibrated Lyons Park Creek models ranged from 0% to –25%. Mean errors in runoff volume ranged from –3% to –25%. These ranges are within the expected accuracy of the models. It is probably unreasonable to expect that uncalibrated models would perform much better, unless the modeler was simply lucky with his choice of input parameters. Other measures of average error also were generally favorable. However, the errors for individual storm events were often much higher, ranging from –80% to +75% for peak flows and –80% to +68% for runoff volumes.

Uncalibrated model performance for the Eighteenth Street Storm Sewer watershed was much worse. Mean errors in peak flow ranged from –12% to +116%. Even the SCS models, which had relatively good mean errors of –12% and +10%, had Root Mean Square Errors (RMSEs) of 68% and 82%, indicating wide variation in the model results. Mean errors in runoff volume for the uncalibrated Eighteenth Street models ranged from +33% to +148%. However, the observed runoff volumes were unusually low.

Other researchers have reported on the errors present in uncalibrated rainfall-runoff models, such as Trommer et al. (1996), Yu et al. (1997) and Zarriello (1998). A wide range of errors was reported for different models. Error measures for the Lyons Park Creek watershed models were generally about the same or better than errors reported by these other researchers. The errors in some of the Eighteenth Street uncalibrated models were probably higher than normal, but within the range of errors reported by others.

Without model calibration, it appears unreasonable to expect that individual storm events will be simulated accurately. However, if a series of storm events are simulated, the mean errors may be acceptable.

Are any bias or trends apparent in the uncalibrated models?

The negative mean errors for the Lyons Park Creek models indicate a tendency to underpredict peak flows and runoff volumes. This could be troubling to engineers and planners, who usually desire models to produce conservative results, and therefore provide a margin of safety for planning and design. However, some models, especially the SCS models, tended to underpredict peaks and volumes for smaller events and overpredict for larger events.

Most of the Eighteenth Street uncalibrated models overpredicted peak flows and runoff volumes by a large margin. Recorded runoff volumes were much lower in this watershed than the impervious area would normally generate.

Numerous studies found that rainfall-runoff models tended to overpredict peak flows and runoff volumes. Trommer et al. (1996) reported that uncalibrated SWMM, Rational Method and SCS models all tended to overpredict when applied to small Florida watersheds. Yu et al. (1997) found that the Rational Method overpredicted observed flood flows by 50% or more, and the SCS model overpredicted flood flows by 100% or more. Rawls et al. (1982) found that SCS models tended to overpredict flood flows, while the Rational Method underpredicted flood flows. Sorrell (2003) and Fontaine (1995) found that the SCS model overpredicted flood flows in rural watersheds, but Hotchkiss and McCallum (1995) reported that both the SCS and Rational Method models significantly underpredicted the 25-year flood flow for a small rural Nebraska watershed.

The most noticeable trend is perhaps the variability and unpredictability of model error. For small to medium sized rainfall events, such as most of those included in this study, and the storms studied by Zarriello (1998), runoff volume and peak flows were often underpredicted. Based on a the literature reviewed, there appears to be a trend for models to overpredict the flows associated with large, infrequent floods, but this is not a universal occurrence.

Are some uncalibrated models better than others?

Stating that one model is always better than another is usually difficult to justify. How well a model performs depends partly on the intended use of the model, and the experience of the person doing the modeling.

The Lyons Park Creek model results show that the uncalibrated SWMM and Rational Method models predicted peak flows fairly well. The literature review indicated that uncalibrated SWMM models are usually more accurate than uncalibrated Rational Method or SCS models. SWMM may be able to match observed peak flows and volumes well because it contains the most explicit representation of impervious area. Impervious area has a very strong influence on runoff in urban areas. As discussed earlier, many researchers report a trend in rainfall-runoff models overpredicting flood flows. However, the Rational Method may be a partial exception to this trend. As discussed earlier, several researchers reported that the Rational Method underpredicted flood flows, and this method underpredicted the largest observed peak flows on Lyons Park Creek.

The SCS model seems to be the least satisfactory of the models considered. The results from Lyons Park Creek and other literature show high error associated with SCS modeling. Even when the mean error is better than other models, the RMSE for SCS models is usually higher than other models, indicating greater variability in the results. There may be significant flaws in how the SCS model represents urban hydrology and how it has been applied to urban watersheds. This topic will be discussed further later in this chapter.

How well did calibration improve the models?

One of the most noticeable aspects of the model calibration was how difficult it was to calibrate the SCS models that used an average, composite curve number to represent the watershed. These models were by far the worst of any of the calibrated models. In contrast, separating the SCS models into impervious and pervious watershed areas improved the calibration dramatically.

Calibrating the SWMM and Rational Method models of the Lyons Park Creek watershed resulted in minor improvements in model error. These models had relatively low error to begin with, so there was less room for improvement. Much more improvement was seen when calibrating the Eighteenth Street Storm Sewer models, probably because the original errors were so high.

What adjustments were made to input parameters during calibration? Can any extrapolation of input parameters be made to ungaged urban watersheds in the southeastern Wisconsin area?

Sensitivity analyses showed that peak flows and volumes simulated in SWMM are very sensitive to the impervious area in a watershed. Therefore, this is an important parameter to estimate accurately. The original estimate of imperviousness in the Lyons Park Creek, made using GIS data and field observations, required little adjustment during calibration. The percentage of directly connected impervious area in the Eighteenth Street watershed was lowered from 35% to 22% during calibration, to account for the low observed runoff volumes from this watershed.

When curve numbers (for the SCS method) and runoff coefficients (Rational Method) were calibrated for individual storm events, a wide range of variation was observed in these values. For example, event-calibrated Rational Method runoff coefficients in the Lyons Park Creek watershed ranged from 0.36 to 0.69. In general, runoff coefficients for Lyons Park Creek were adjusted upward from the original estimate.

Calibrating the model parameters that describe infiltration was difficult, because most of the events generated little or no runoff from pervious areas. Therefore, one should probably not draw many conclusions about infiltration parameters from this study. It was observed that the curve number which best simulated the pervious areas in the Lyons Park Creek was much lower than the model documentation suggests, given the soil conditions in the watershed.

The SCS Model and Urban Watersheds

Of the models evaluated in this study, the SCS models generally had the highest errors (unless the watershed was represented with separate impervious and pervious SCS curve numbers). Other researchers have also reported the poor performance of this model compared to other urban rainfall-runoff models. There are several possible reasons for this.

Most applications of the SCS model use a composite curve number to represent the watershed surface. The relative percentages of impervious and pervious surface are weighted to calculate this composite curve number. Use of composite curve numbers results in an initial abstraction that is generally too high for watersheds with significant impervious area. For example, the mean curve number for the Lyons Park Creek watershed, after calibration of individual storm events, was 86. This corresponds to an initial abstraction of over 0.3 inches, using the default procedure for calculating initial abstraction. Using this value, no runoff would occur from the watershed until rainfall depths exceeded 0.3 inches. In reality, the impervious surfaces start generating runoff almost immediately. The impervious depression storage (roughly analogous to initial abstraction) had a value of only 0.01 inches for the calibrated Lyons Park Creek SWMM model.

These problems are briefly mentioned in the SCS manual for applying this method to urban areas (Soil Conservation Service, 1986). This manual states that the user must understand the assumptions behind the initial abstraction term and the SCS method for calculating runoff volume, and only apply the model when appropriate, or modify the model. It appears that this method may in fact rarely be appropriate for watersheds with high amounts of impervious area. Golding (1979) reported similar difficulties in attempting to simulate urban runoff using the SCS model. He recommended simulating the impervious and pervious areas separately, and using a lower initial abstraction. Based on the results of this study, the SCS documentation, and other literature, the SCS model should probably not be used to simulate urban runoff from actual storm events, unless the watershed is divided into separate impervious and pervious areas and the model can be calibrated.

The Soil Conservation Service (1986) does state that this method is less accurate when runoff is less than 0.5 inches. This manual also states that the curve number procedure is intended primarily for design procedures, and will be less accurate for simulating actual storm events. Therefore, testing the model with historical, small to medium size storm events may be an unfair test of its abilities. However, considering the performance of the model for the Lyons Park Creek event of August 13, 2002, as well as reports by others such as Yu et al. (1997) and Titmarsh et al. (1995), the SCS model does not appear to perform much better at simulating large storm events and the associated low flood probabilities.

In general, the extension of the SCS rainfall-runoff model to urban watersheds has not been justified very well. As discussed above, the simulation of runoff from impervious and pervious areas with the same parameter and equation seems to be flawed. Furthermore, curve numbers for pervious areas in an urban watershed (lawns, parks, golf course, etc.) were selected by assuming the same values that were derived for pastures in agricultural land. No justification is provided for why urban lawns and parks should have the same runoff performance as pastures. Given the popularity of the SCS model for urban hydrology, it is disappointing that apparently little testing, verification, and calibration has taken place.

The Probabilistic Approach To Rainfall-Runoff Models

Given the problems that some rainfall-runoff models have in simulating real rainfall events, some researchers have suggested an alternate use for these models. Researchers such as Titmarsh et al. (1995) and Pilgrim and Cordery (1993) have suggested that the Rational Method and SCS models be viewed as probabilistic models, rather than physically-based models. This argument makes sense, given that many users of these models are using them to estimate flows of a certain probability, rather than trying to reproduce historical events. To date, this approach has not seen much use in the United States, but it seems promising.

Future Research

Many possibilities exist for future research on this topic. Additional small urban watersheds could be studied in a similar fashion. The USGS has recorded rainfall and runoff for numerous small urban watersheds in Wisconsin. Some of these were considered for use in this study, but were not included because of time constraints. Uncalibrated rainfall-runoff models could be constructed for these watersheds, historical storm events simulated, and model error assessed. The models could then be calibrated.

Results from these watersheds could be compared to results from the Lyons Park Creek and Eighteenth Street watersheds, to see if the conclusions of this study are supported, or perhaps contradicted. Additional models could also be applied. HSPF would be a prime candidate. The Milwaukee Metropolitan Sewerage District uses HSPF to simulate wet weather flows to its sewer collection system and local watercourses.

The probabilistic approach to rainfall-runoff modeling should be investigated for use in Wisconsin. Rainfall-runoff models such as the SCS model and Rational Method could be applied to watersheds that have long records of recorded flood peaks. Flood flow probabilities have been developed for many of these watersheds, or could be developed. Model input parameters could be calibrated to produce the observed discharge probabilities, for precipitation design storms with the same probability. For example, the USGS has recorded peak flows and stages at a location on Honey Creek in Milwaukee County for over 40 years. The watershed tributary to this gauge has an area of approximately 3 square miles and is mostly urbanized. Although some adjustments may have to be made for changing land use within the watershed, the long record of recorded peak discharges makes this site a candidate for calibrating rainfall-runoff models on a probabilistic basis.

It appears that more research on the appropriateness of the SCS model for urban watersheds is needed. More calibration of the input parameters should be done, whether for actual storm events or to reproduce a flood probability distribution. The separation of the watershed into impervious and pervious areas could be investigated further, to see if separation improves the model, as this study suggests.

Guidance for Designers and Planners

Given the uncertainty and variability associated with rainfall-runoff modeling, how should these models be used to plan and design drainage infrastructure, one of their primary uses? Much modeling work is guided by engineering codes, standards and ordinances. For example, one reason the SCS model is so widely used is that many stormwater-related codes and ordinances require it. To meet regulatory requirements and protect themselves against liability, engineers often have no choice but to follow the accepted standards. For that reason, the SCS model will continue to be widely used. Modelers should recognize that it may tend to significantly overpredict flood flows for large events, resulting in highly conservative designs. In the long term, researchers and government agencies should continue to compare the SCS model to real watershed data and regionally calibrate the model, as discussed in the previous section.

If the modeler has some freedom to select a rainfall-runoff model, it appears that SWMM and the Rational Method give more realistic results, as compared to the SCS model. Therefore, they should be considered for use. SWMM and the Rational Method do not appear to have any strong bias to significantly overpredict flood flows. Planners and designers should therefore apply a factor of safety to the model results. Error measures reported in this study, and elsewhere in the literature, provide some ideas on what factor of safety may be appropriate. The factor of safety could be applied directly to the design flows and volumes, or it could be incorporated into the design in another manner, such as a freeboard requirement for a hydraulic structure.

Model users should also consider the purpose of their modeling, and the cost, benefits, and risks associated with the use of the model results. For example, what are the costs associated with using a higher design flow and therefore increasing the size of drainage infrastructure. What are the risks associated with designing infrastructure that is too small? This concept has not been focused on in this report, but it is very important.

Final Thoughts on Model Comparison and Selection

As discussed in the Introduction, modelers must often try to answer questions such as “which model is best?” or “which model should be used?”. Many factors go into selecting rainfall-runoff models for use, and there will probably never be one model that suits every application. In reviewing the results of this study, and the modeling literature, two related themes stood out. These themes may seem obvious, but they are worth reviewing. First, modeling methods should be developed using real data and observed runoff characteristics. Second, models should be applied to systems similar to those they were developed for, and caution must be used when applying models to systems or problems different than used in the original model development.

In reviewing the literature, it was apparent that these concepts are sometimes violated in rainfall-runoff modeling. For instance, the SCS hydrologic model was originally developed using rainfall and runoff data from rural watersheds. It was extended to urban watersheds by manipulating some of the input parameters, but there is no documentation that indicates any testing or calibration of the method was done for real urban watersheds before the release of the model to the public.

In contrast, the literature indicates that various regression equations developed by the USGS are among the most accurate methods of predicting flood flows. This is not surprising, as these equations were developed using extensive observed data. The Rational Method is often criticized as being too old and too simple, but this study found it performed fairly well. A review of its origins (Kuichling, 1889) shows that it was developed using observed rainfall and runoff data, as well as a solid investigation of the processes of urban runoff. It is still being successfully used to do what it was intended to do – predict peak flows from runoff in small urban watersheds. Similarly, SWMM was developed to predict runoff in urban watersheds, and appeared to perform the most successfully of the models reviewed in this study.

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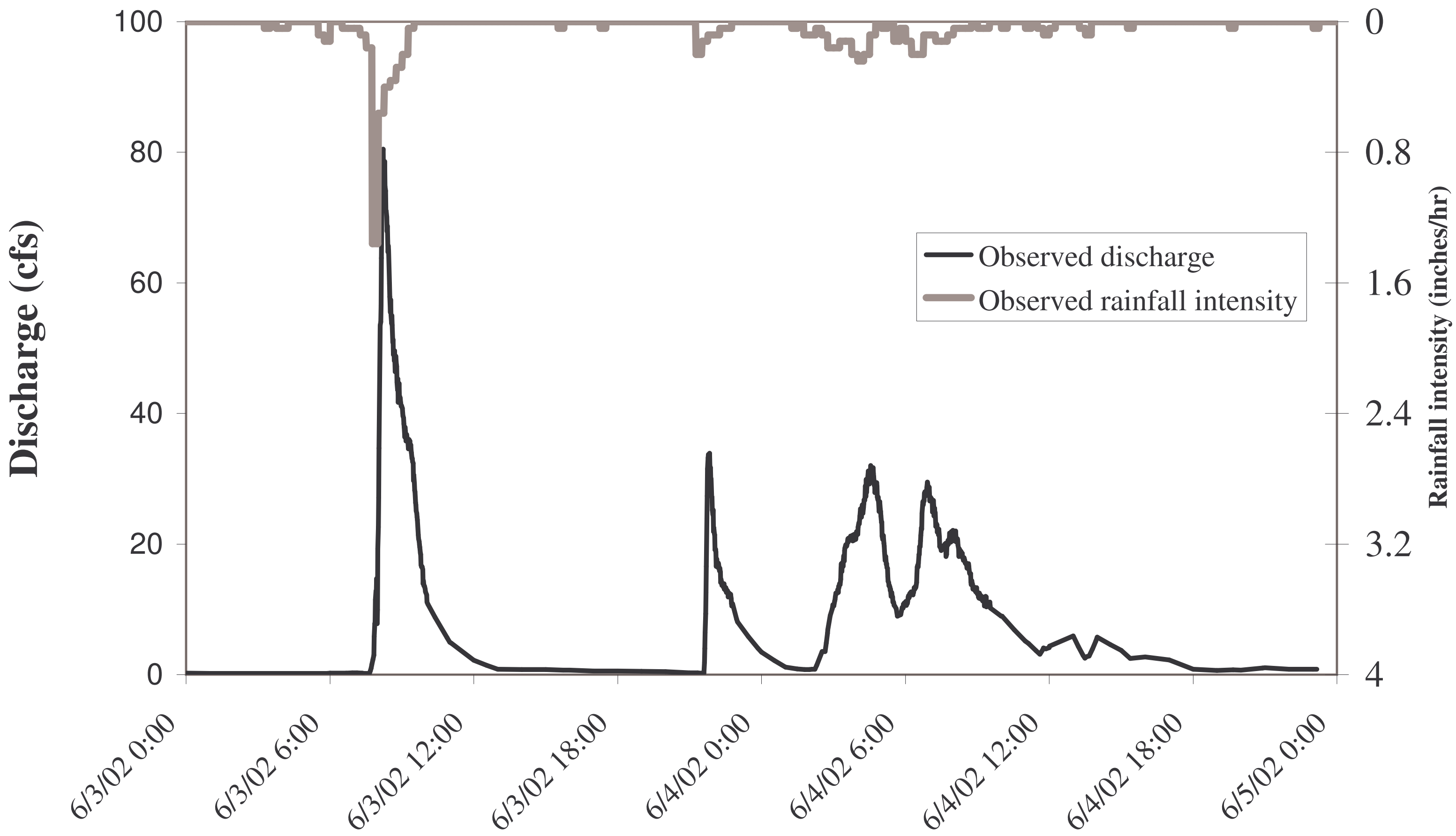
APPENDIX A

OBSERVED HYDROGRAPHS

Lyons Park Creek
June 3-4, 2002



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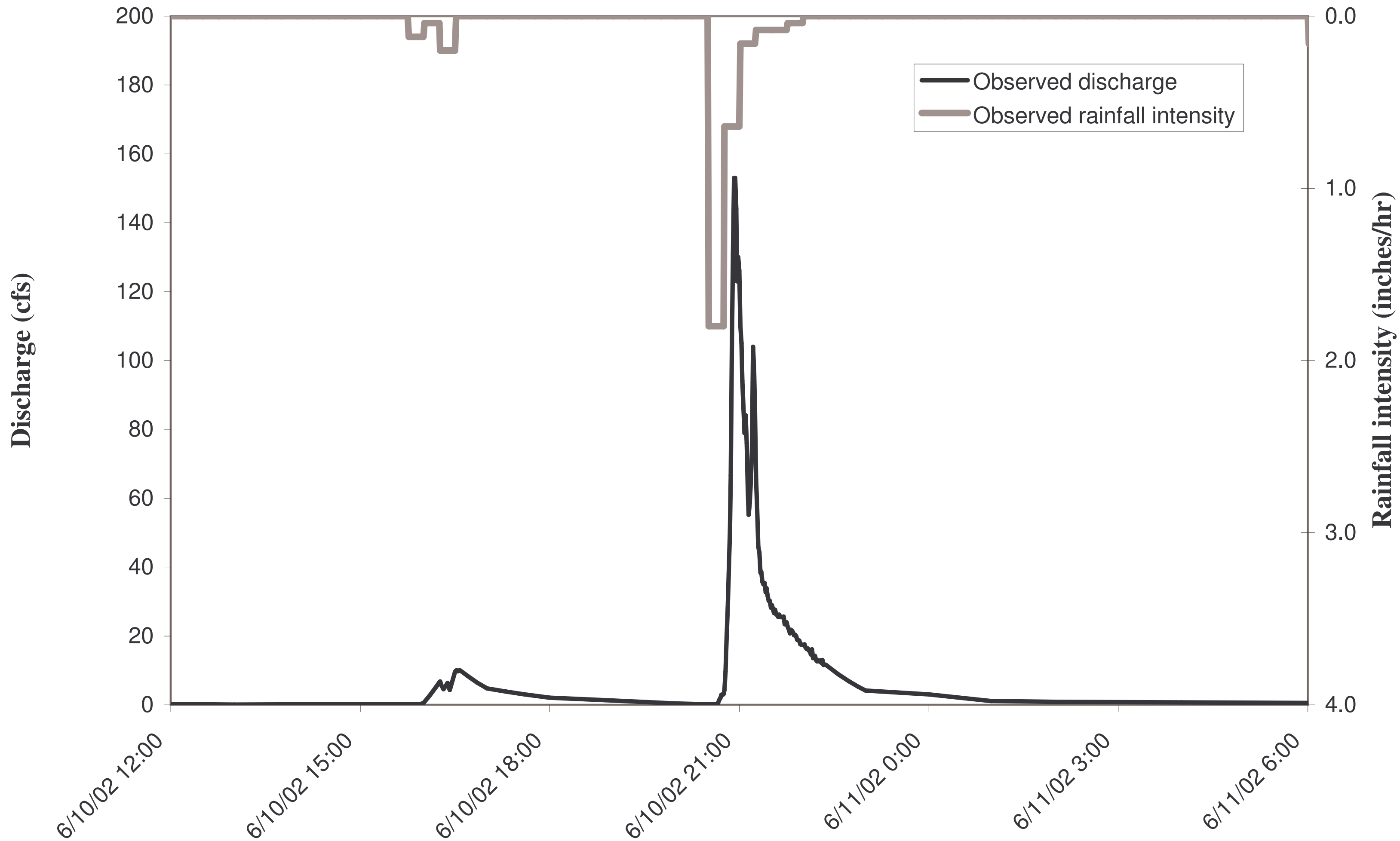
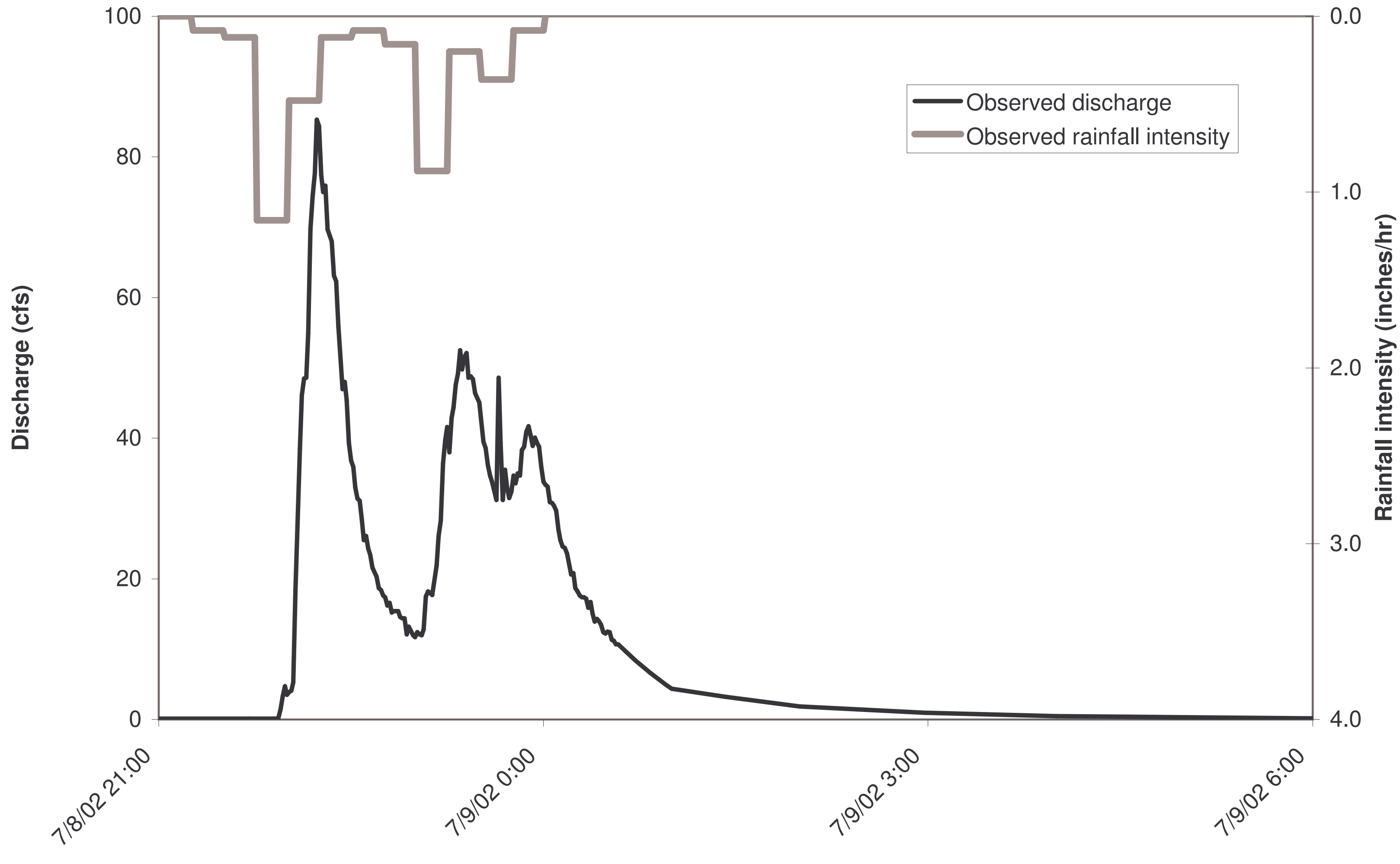
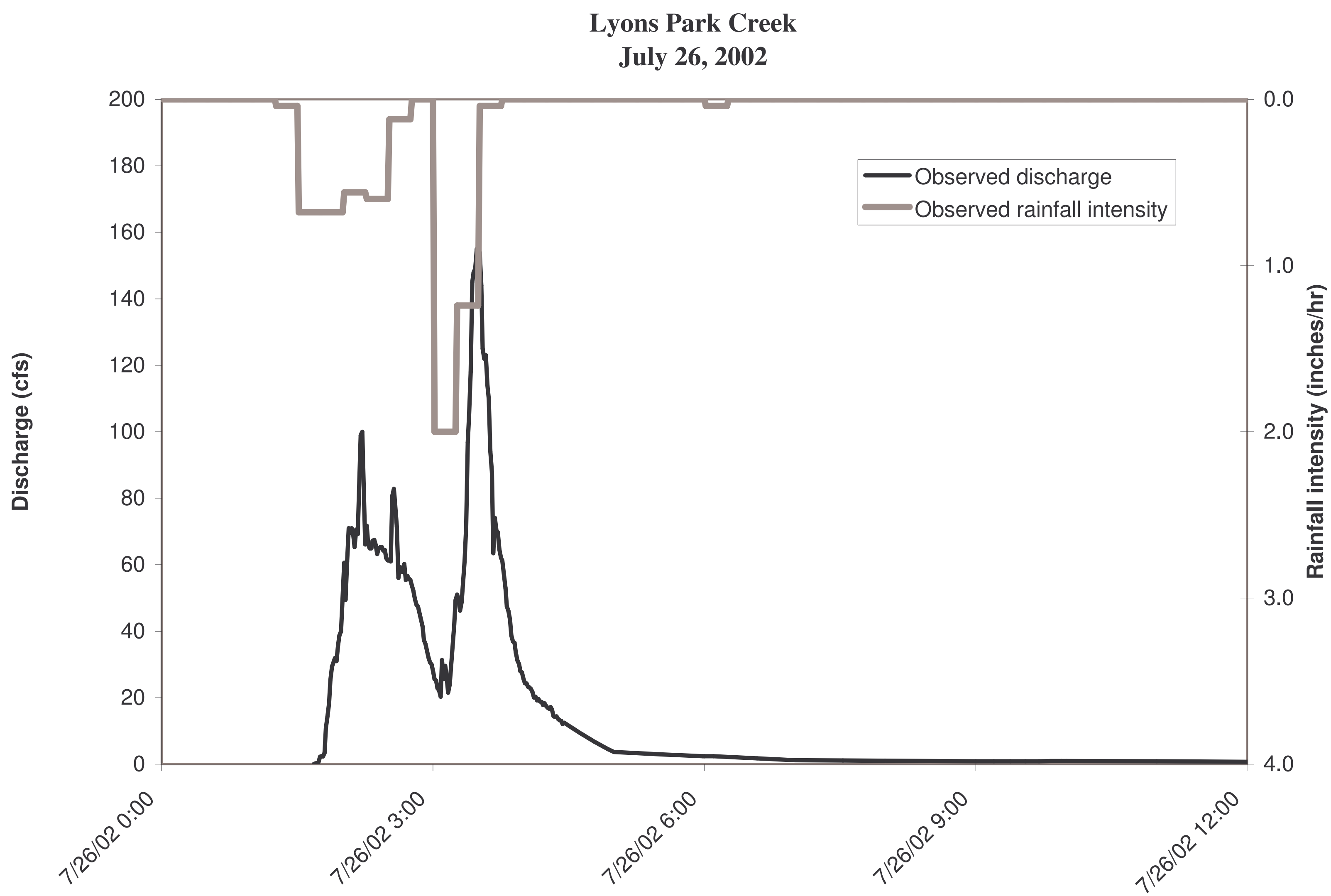
June 10, 2002



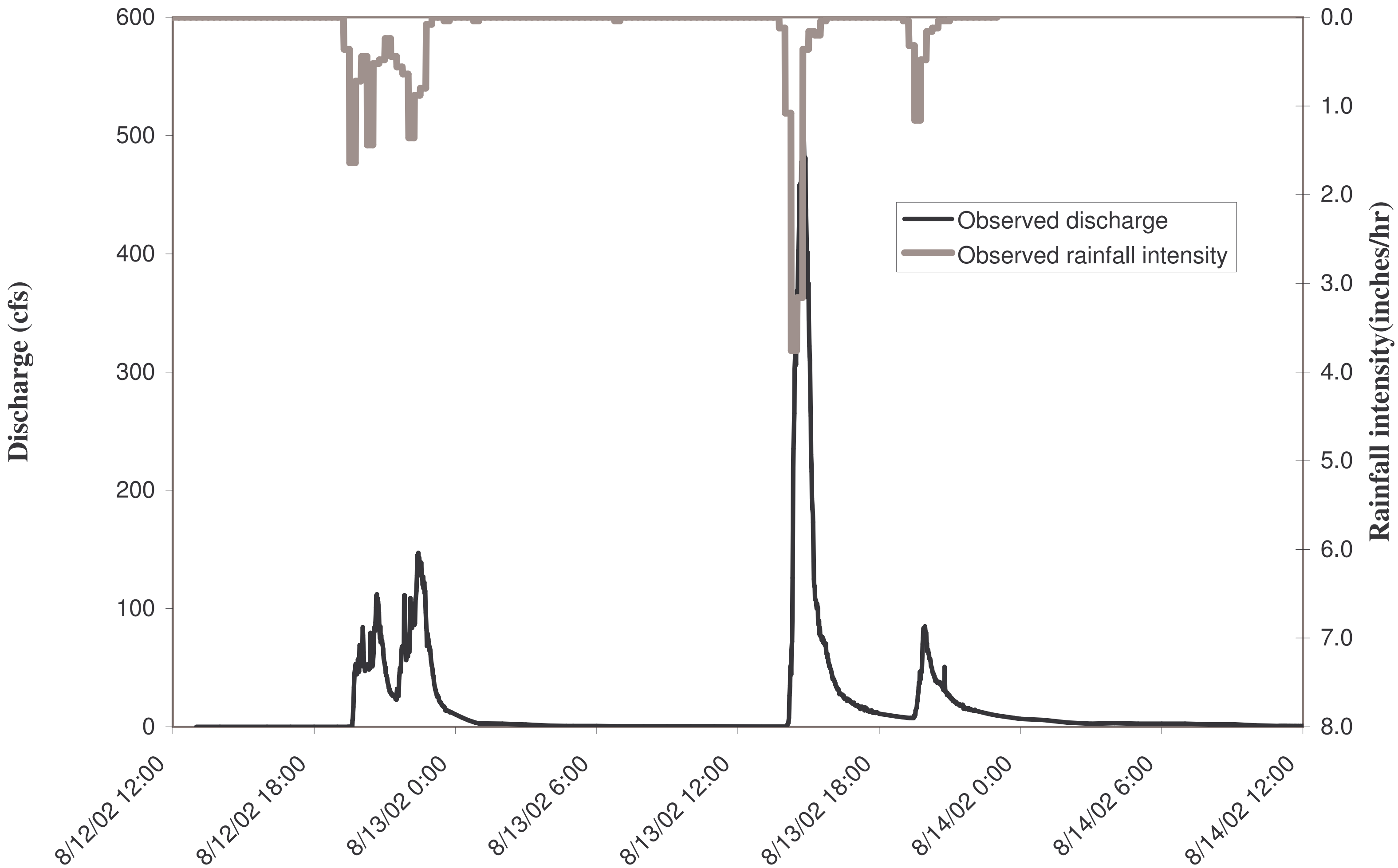
Lyons Park Creek
July 8-9, 2002



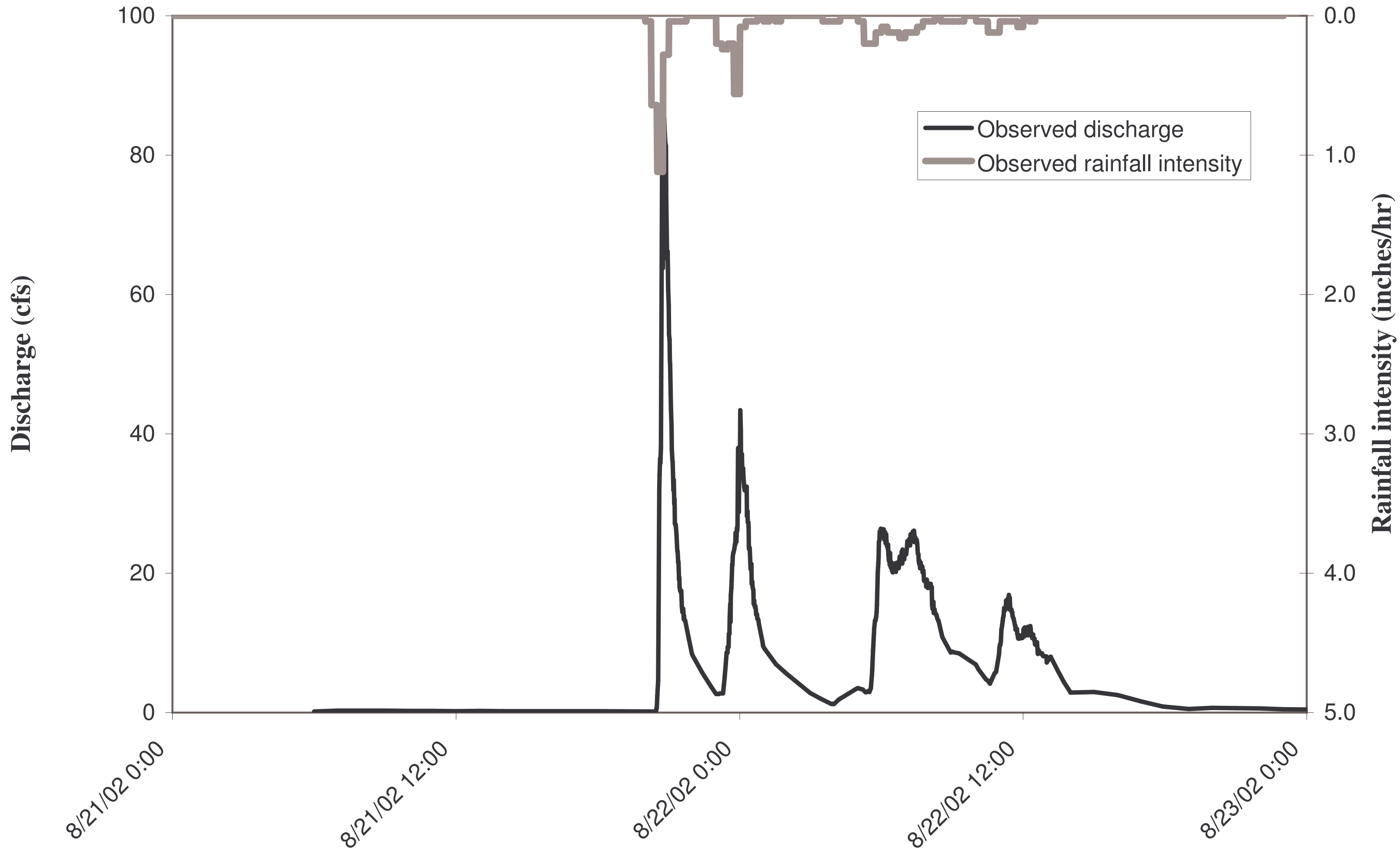


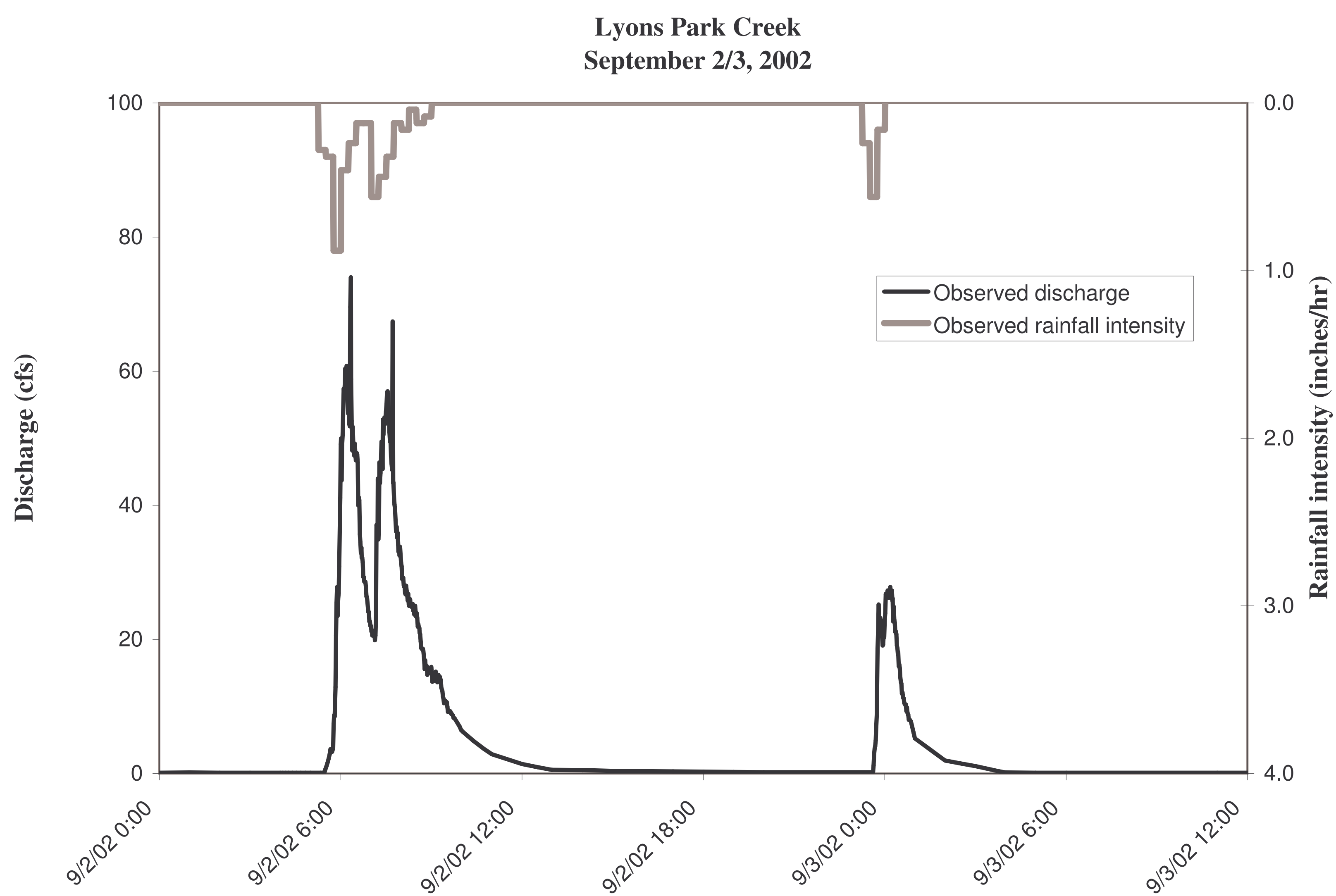
Lyons Park Creek
July 26, 2002
Discharge (cfs)
Rainfall intensity (inches/hr)
Observed discharge
Observed rainfall intensity
200
180
160
140
120
100
80
60
40
20
0
0.0
1.0
2.0
3.0
4.0
7/26/02 0:00
7/26/02 3:00
7/26/02 6:00
7/26/02 9:00
7/26/02 12:00

Lyons Park Creek
August 12/13, 2002



Lyons Park Creek
August 21/22, 2002




Lyons Park Creek
September 2/3, 2002
Discharge (cfs)
Rainfall intensity (inches/hr)
Observed discharge
Observed rainfall intensity
100
80
60
40
20
0
0.0
1.0
2.0
3.0
4.0
9/2/02 0:00
9/2/02 6:00
9/2/02 12:00
9/2/02 18:00
9/3/02 0:00
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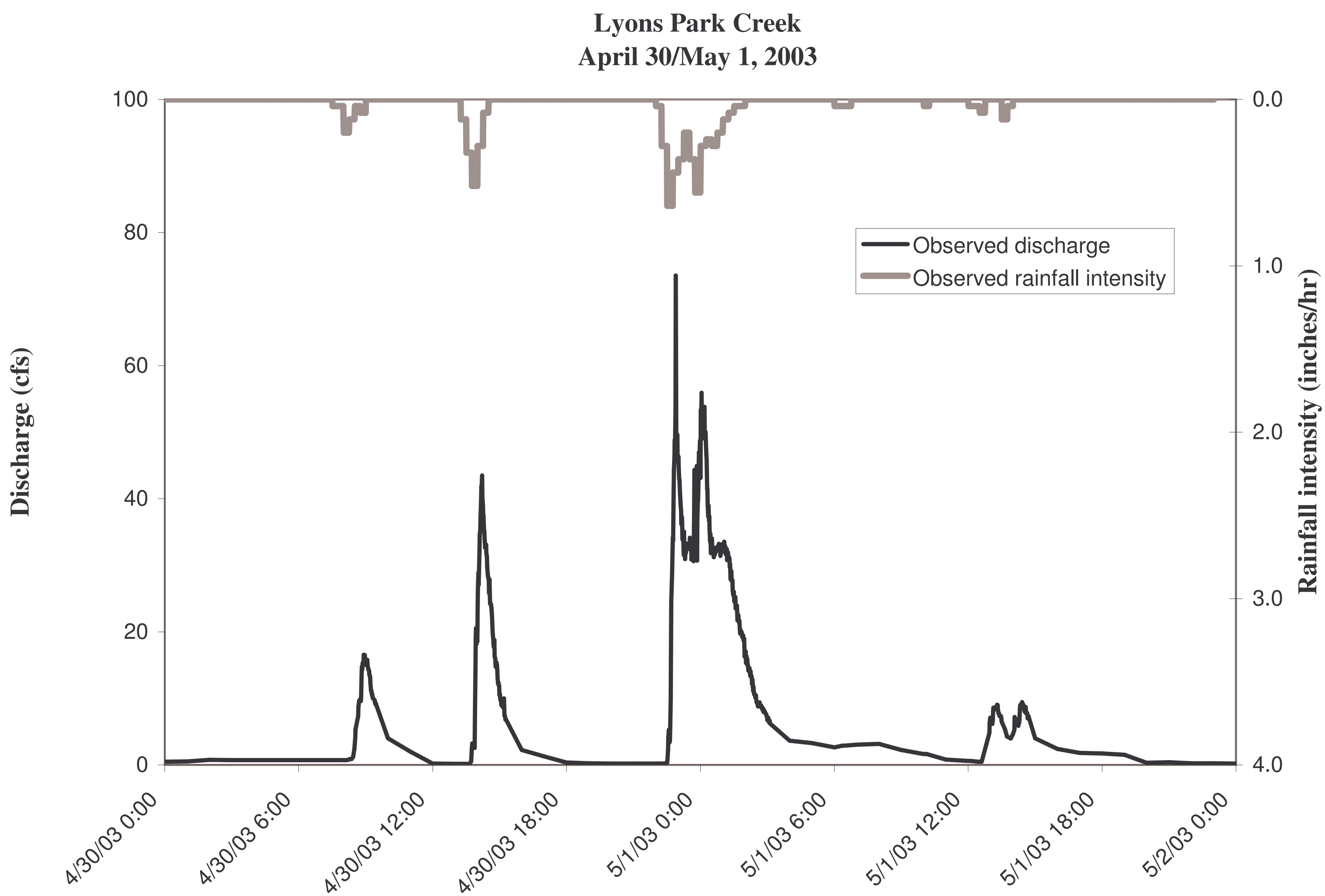
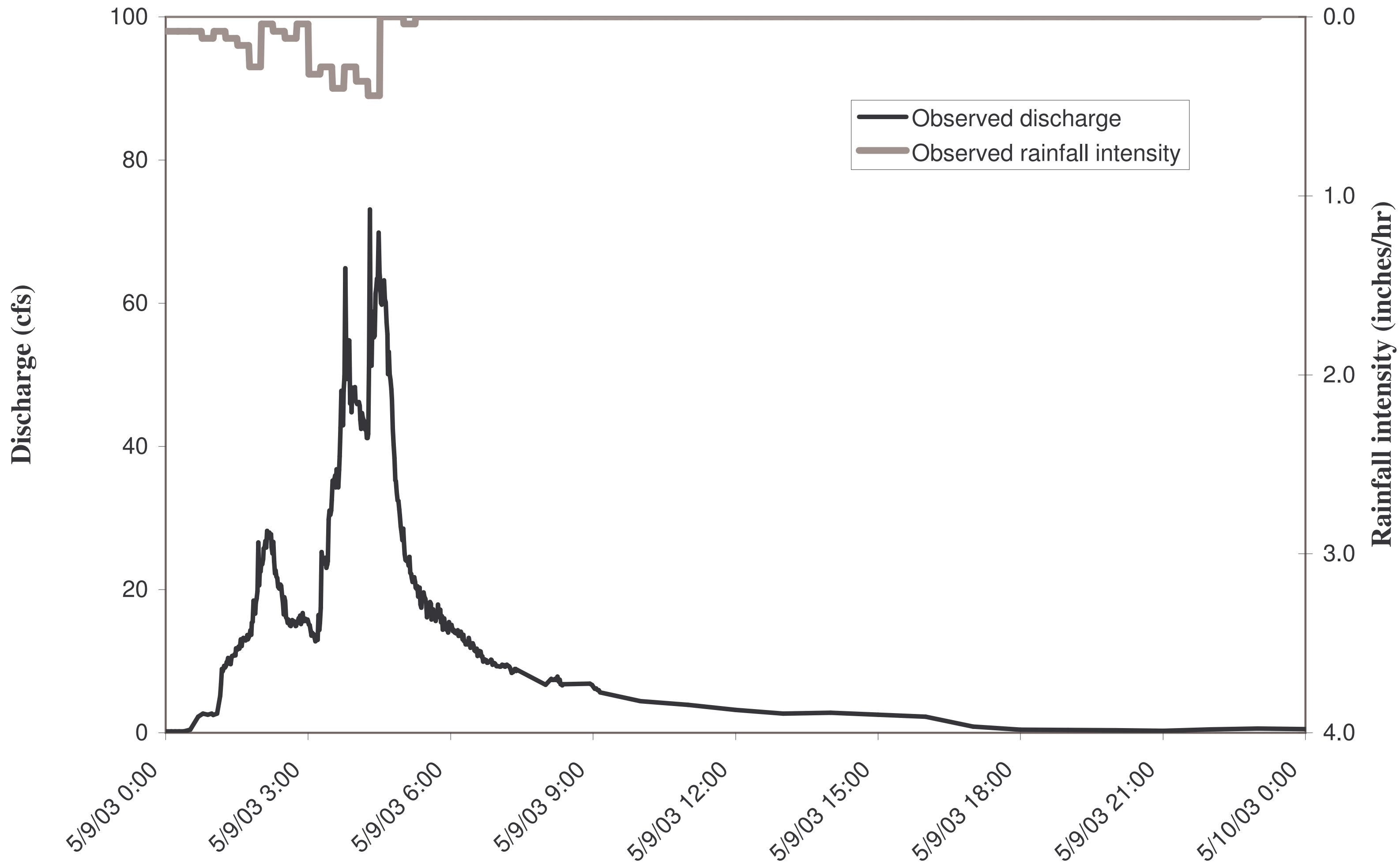
9/3/02 12:00

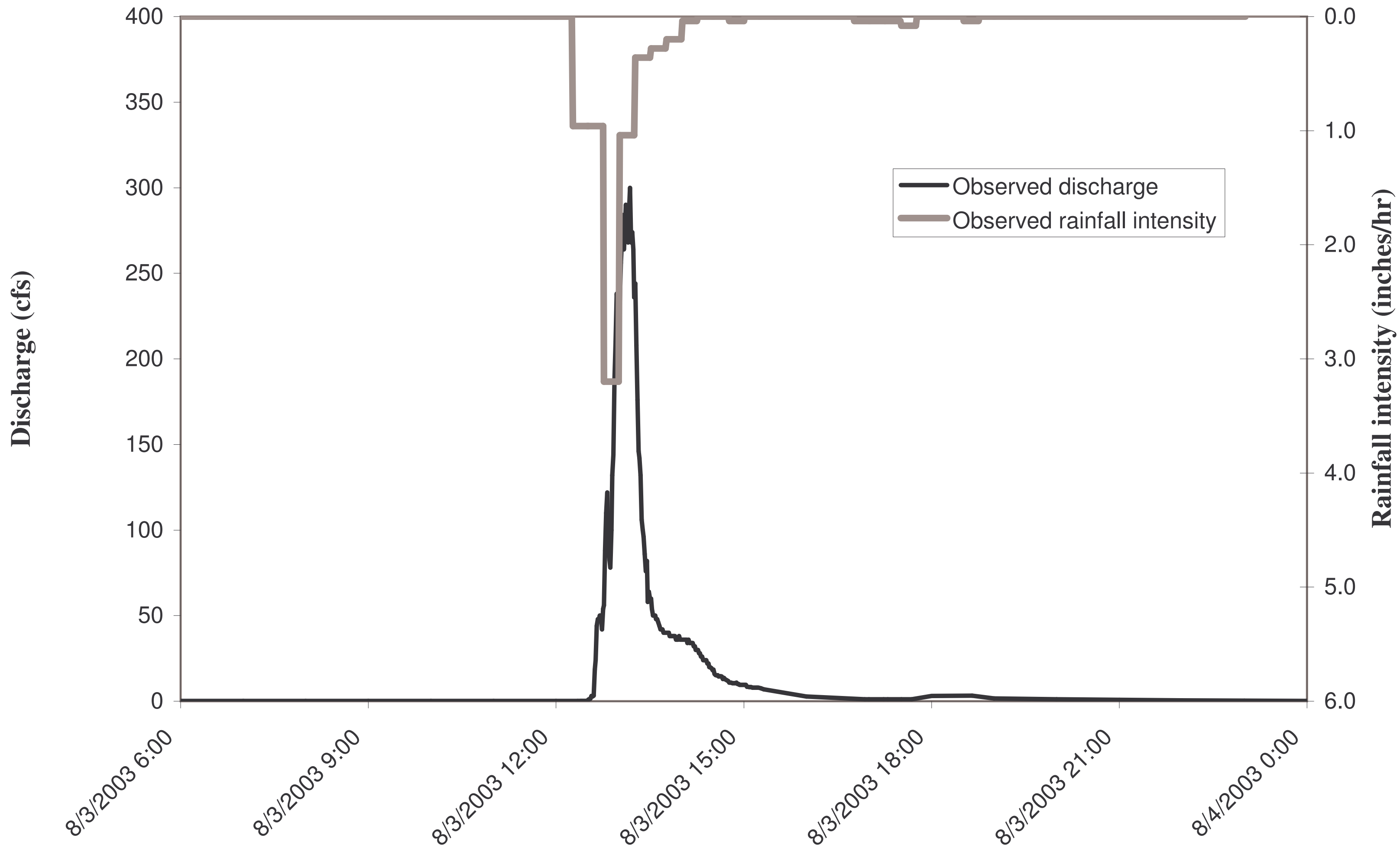


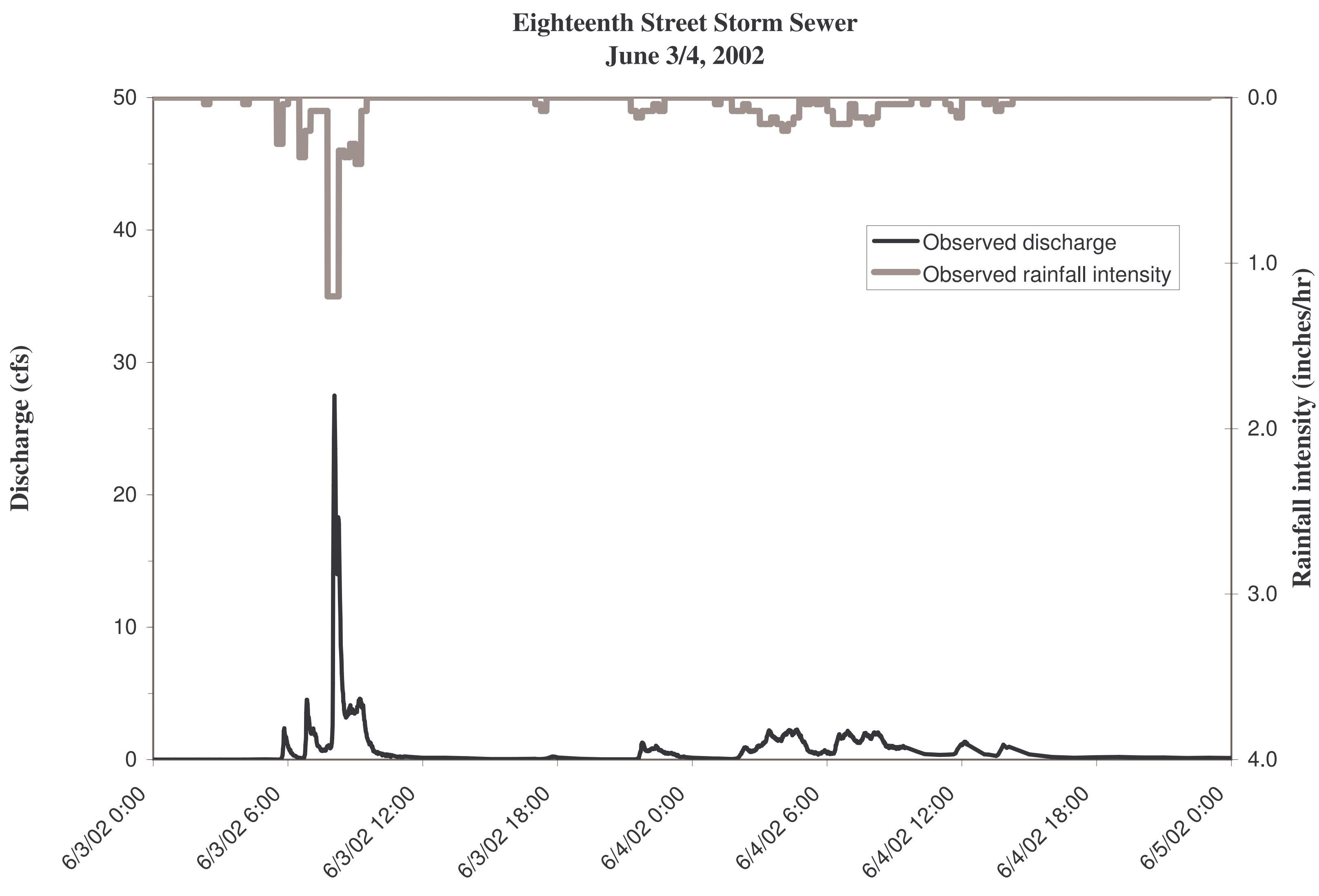
Lyons Park Creek
April 30/May 1, 2003
Discharge (cfs)
Rainfall intensity (inches/hr)
Observed discharge
Observed rainfall intensity
100
80
60
40
20
0
0.0
1.0
2.0
3.0
4.0
4/30/03 0:00
4/30/03 6:00
4/30/03 12:00
4/30/03 18:00
5/1/03 0:00
5/1/03 6:00
5/1/03 12:00
5/1/03 18:00
5/2/03 0:00

Lyons Park Creek
May 9, 2003

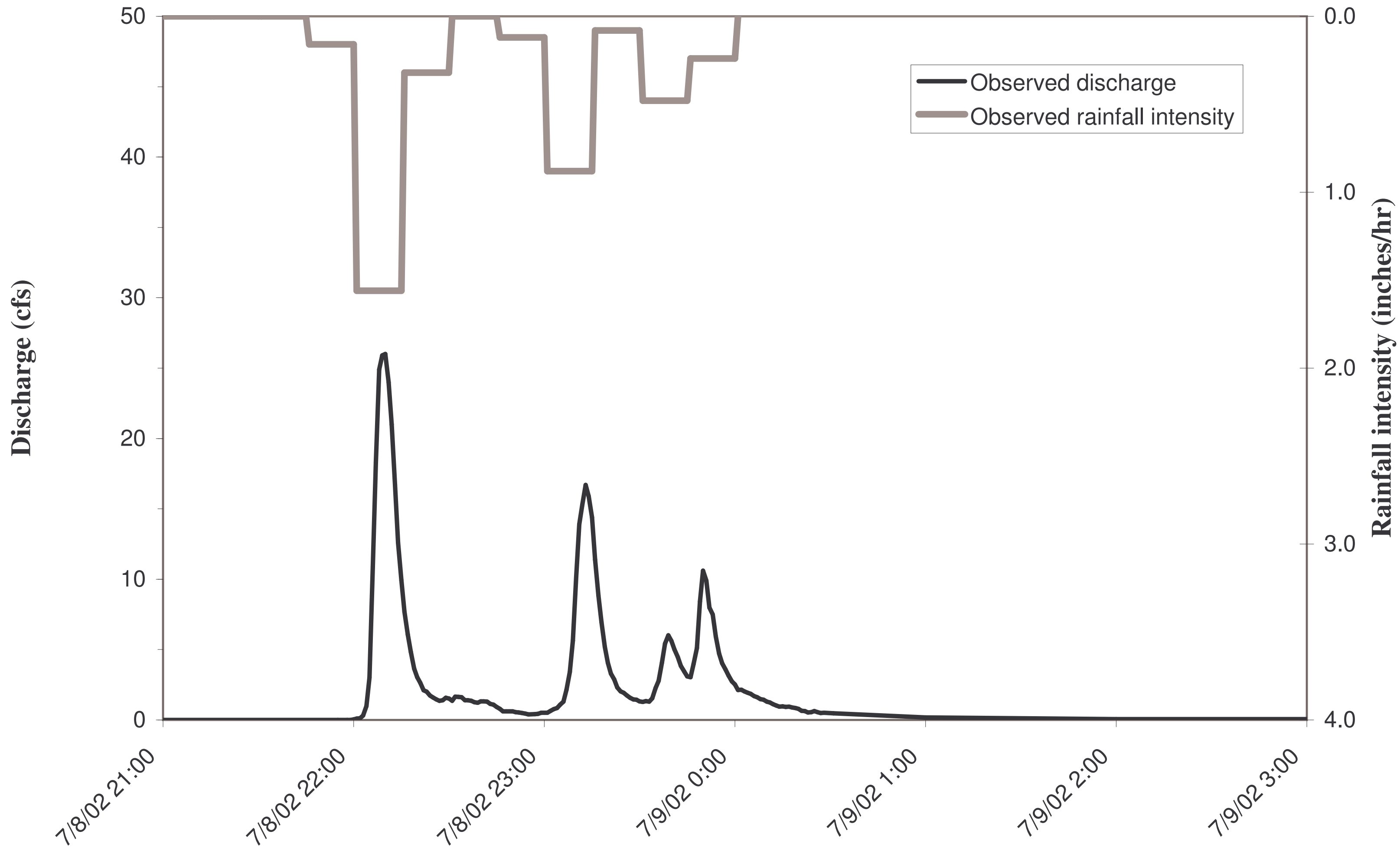


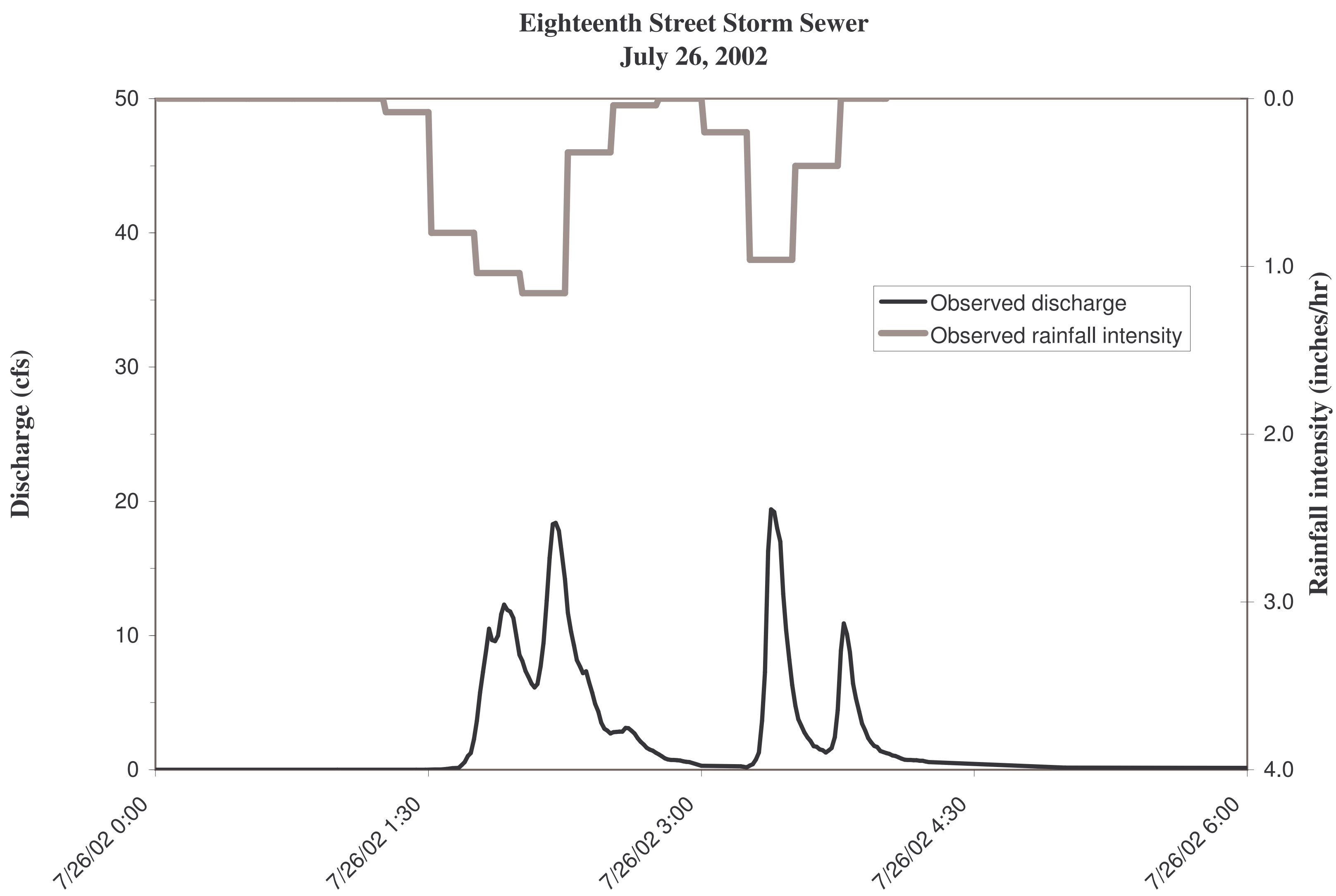
Lyons Park Creek
August 3, 2003

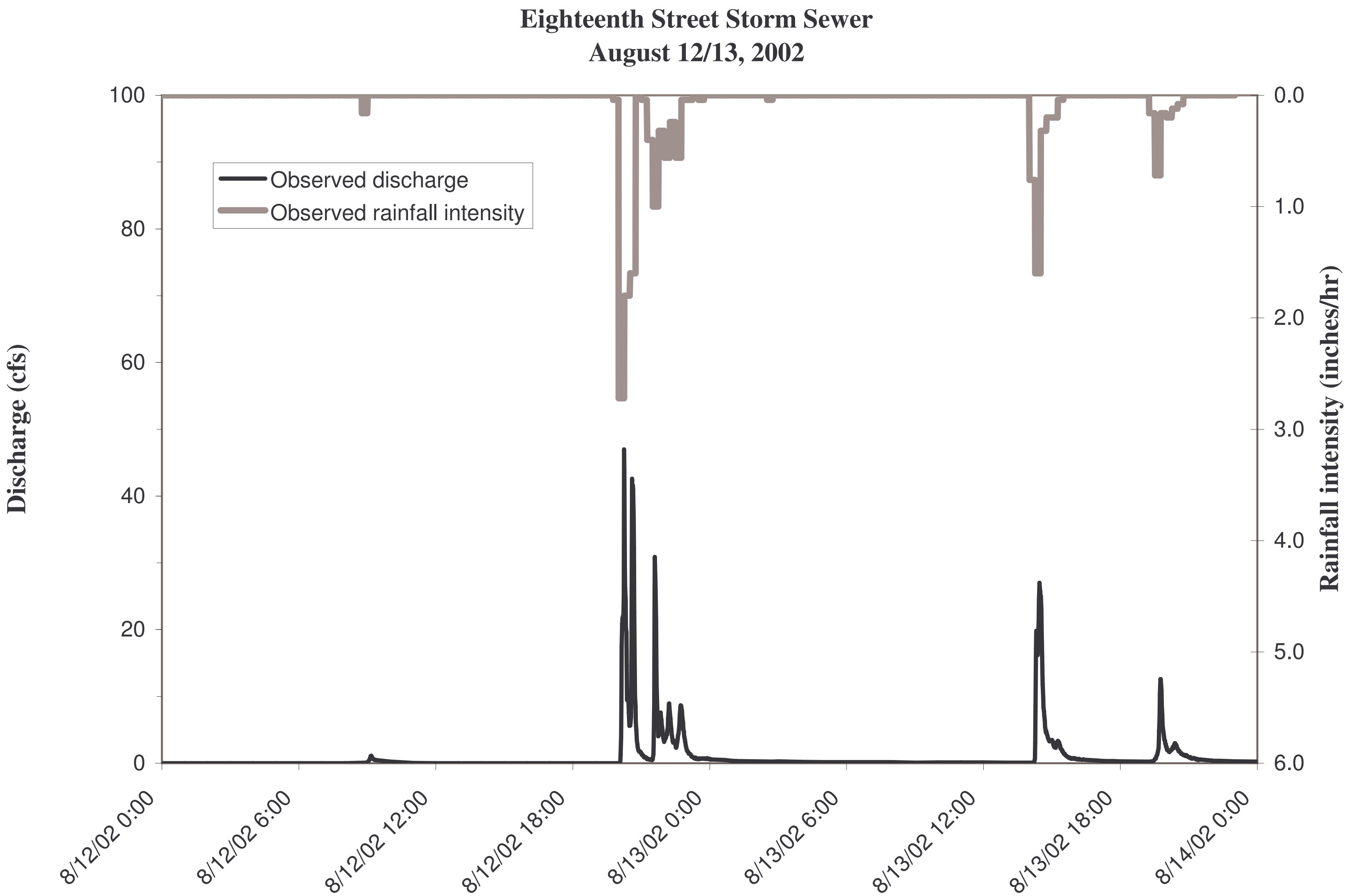



Eighteenth Street Storm Sewer
June 3/4, 2002
Discharge (cfs)
Rainfall intensity (inches/hr)
Observed discharge
Observed rainfall intensity
50
40
30
20
10
0
0.0
1.0
2.0
3.0
4.0
6/3/02 0:00
6/3/02 6:00
6/3/02 12:00
6/3/02 18:00
6/4/02 0:00
6/4/02 6:00
6/4/02 12:00
6/4/02 18:00
6/5/02 0:00

Eighteenth Street Storm Sewer
July 8, 2002




Eighteenth Street Storm Sewer
July 26, 2002
Discharge (cfs)
Rainfall intensity (inches/hr)
Observed discharge
Observed rainfall intensity
0
10
20
30
40
50
0.0
1.0
2.0
3.0
4.0
7/26/02 0:00
7/26/02 1:30
7/26/02 3:00
7/26/02 4:30
7/26/02 6:00


Eighteenth Street Storm Sewer
August 12/13, 2002
Observed discharge
Observed rainfall intensity
Discharge (cfs)
Rainfall intensity (inches/hr)
100
80
60
40
20
0
0.0
1.0
2.0
3.0
4.0
5.0
6.0
8/12/02 0:00
8/12/02 6:00
8/12/02 12:00
8/12/02 18:00
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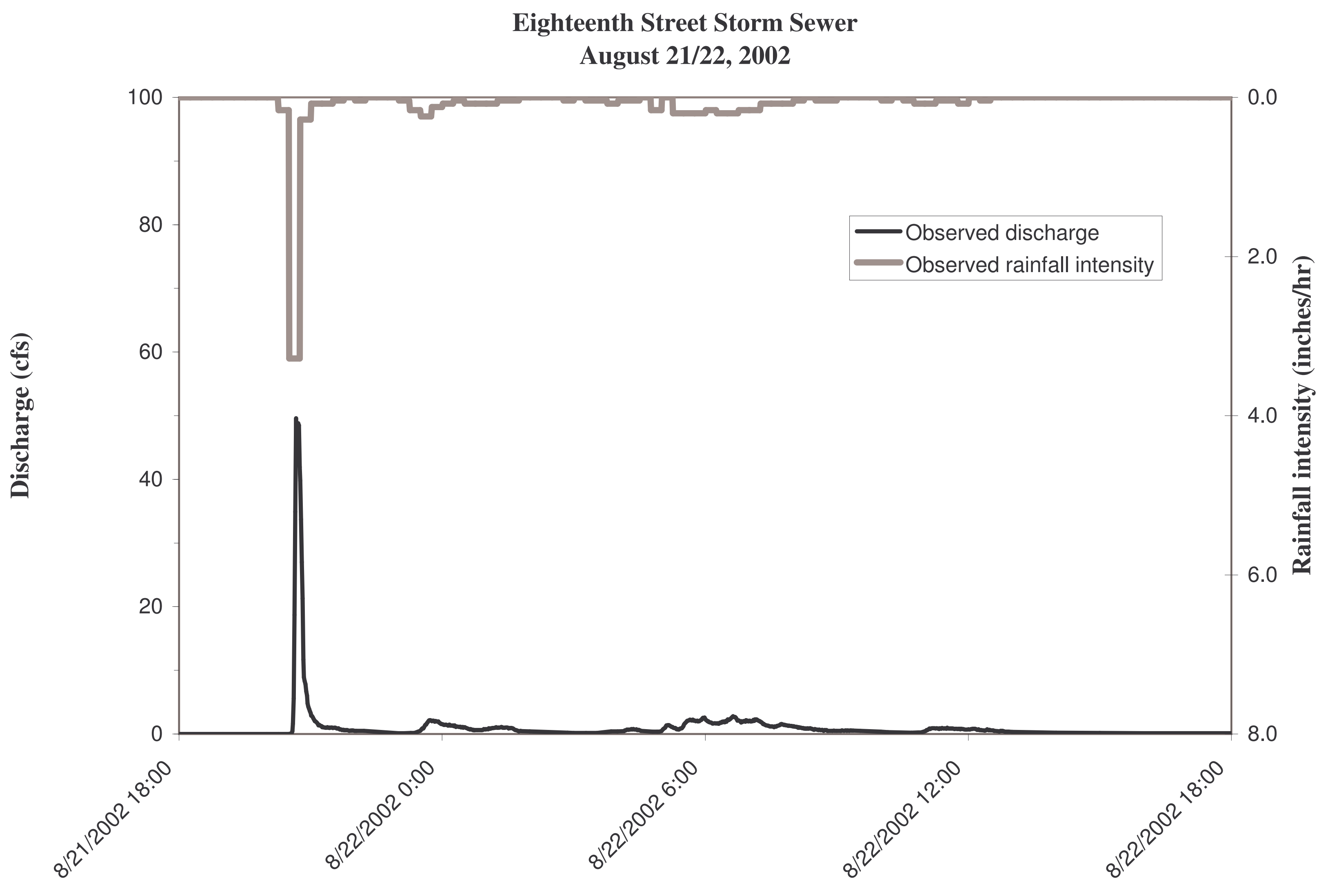
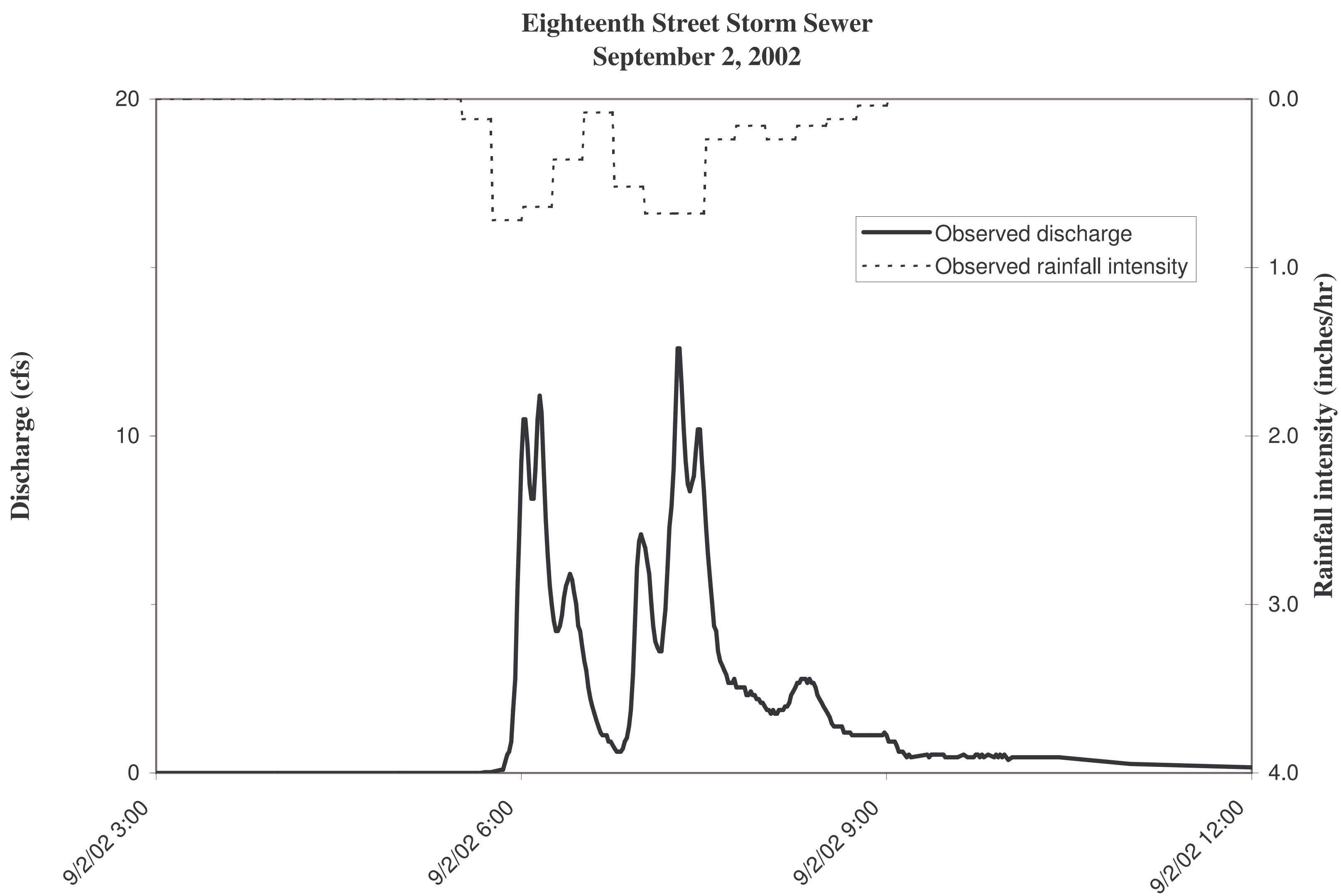
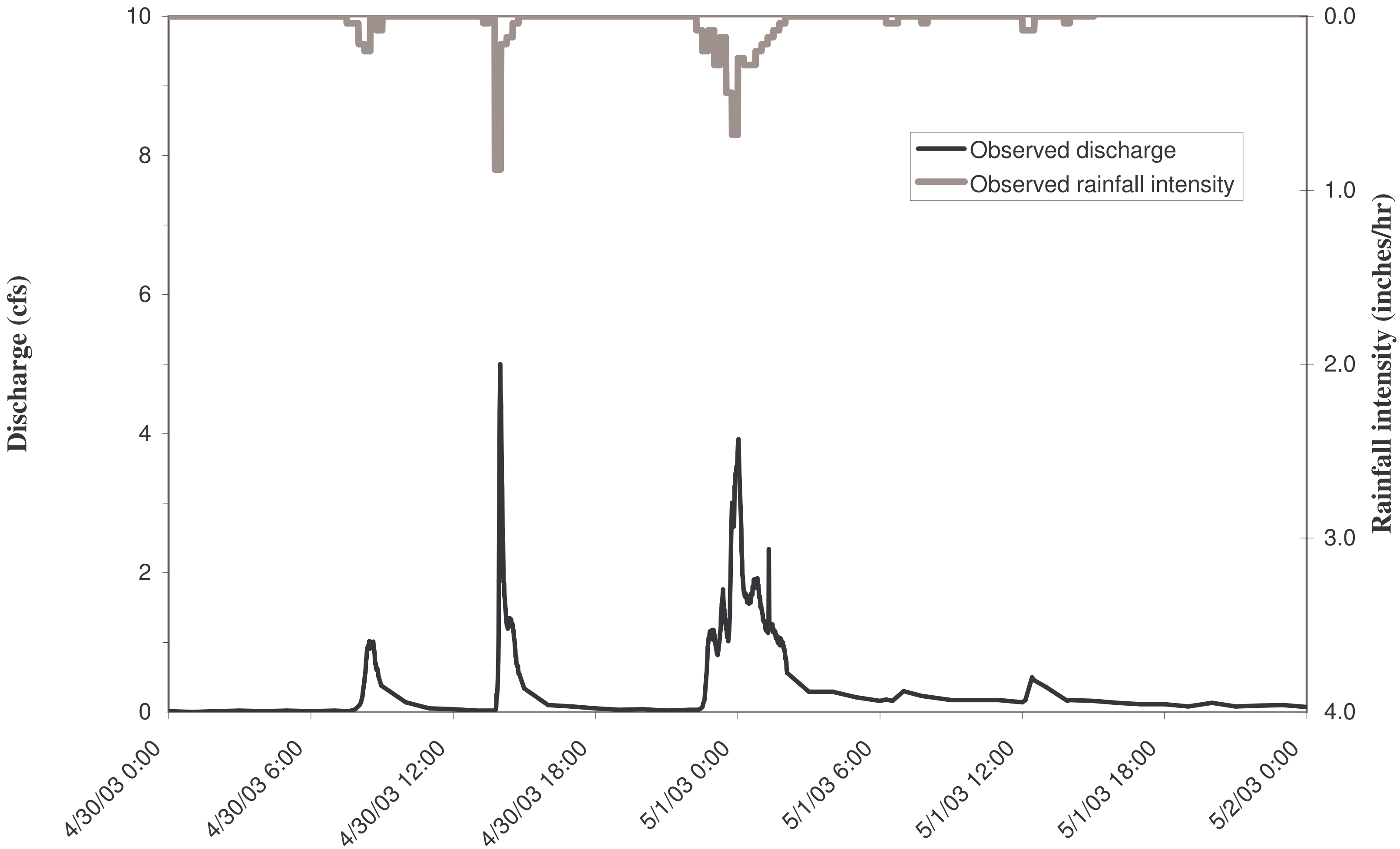
8/14/02 0:00

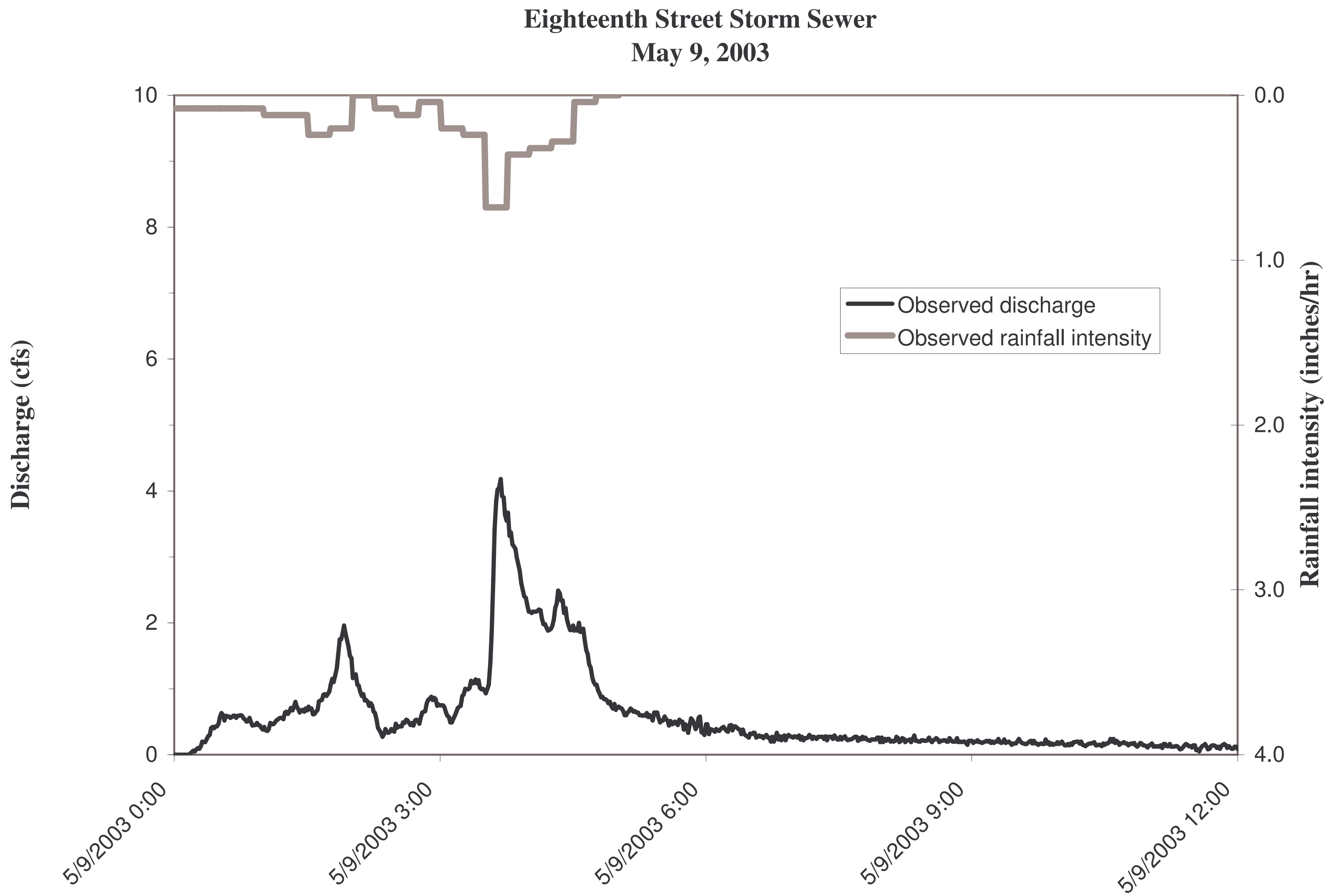


Eighteenth Street Storm Sewer
August 21/22, 2002
Discharge (cfs)
Rainfall intensity (inches/hr)
Observed discharge
Observed rainfall intensity
100
80
60
40
20
0
0.0
2.0
4.0
6.0
8.0
8/21/2002 18:00
8/22/2002 0:00
8/22/2002 6:00
8/22/2002 12:00
8/22/2002 18:00

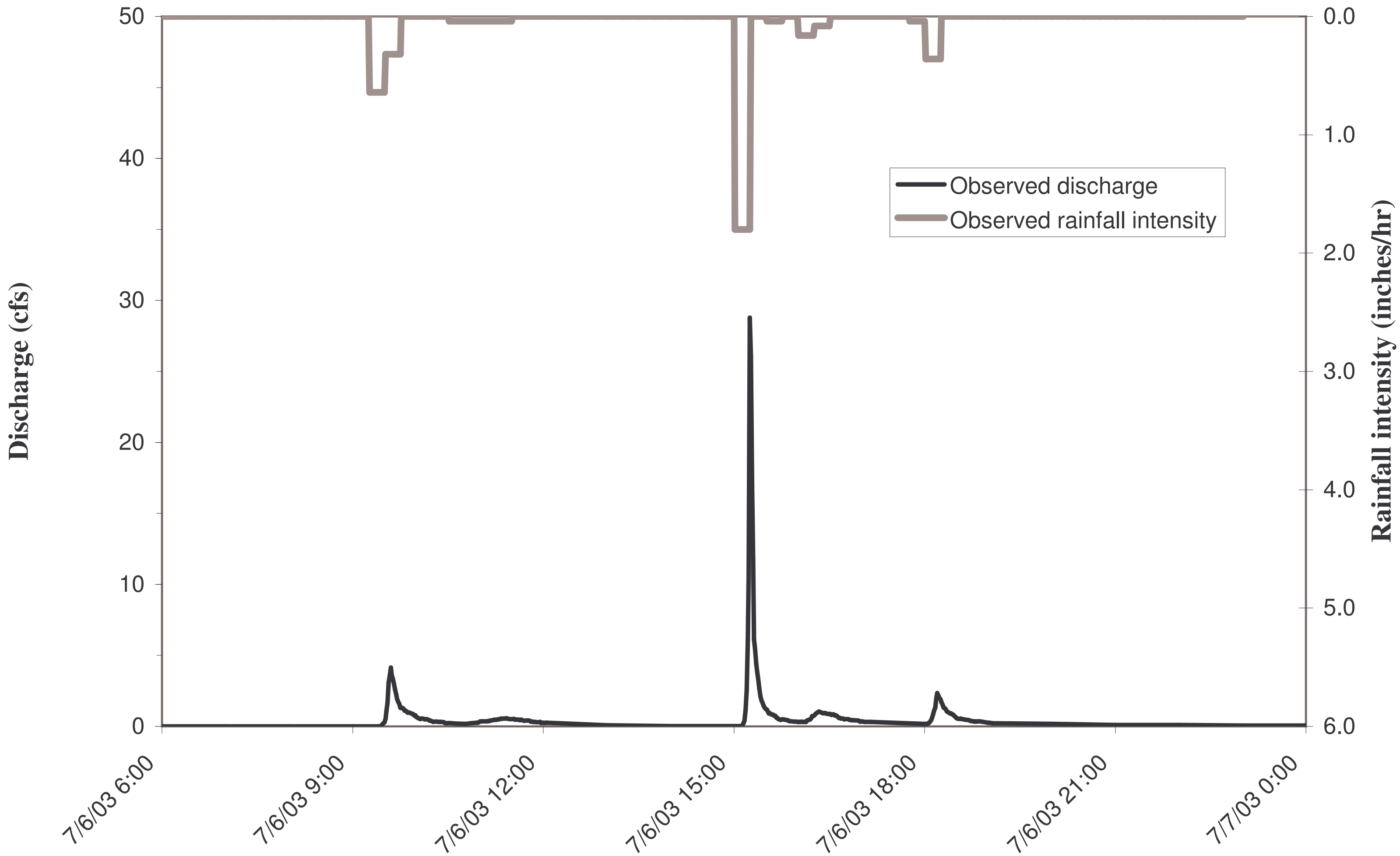

Eighteenth Street Storm Sewer
September 2, 2002
Observed discharge
Observed rainfall intensity
Discharge (cfs)
Rainfall intensity (inches/hr)
20
10
0
0.0
1.0
2.0
3.0
4.0
9/2/02 3:00
9/2/02 6:00
9/2/02 9:00
9/2/02 12:00

Eighteenth Street Storm Sewer
April 30/May 1, 2003

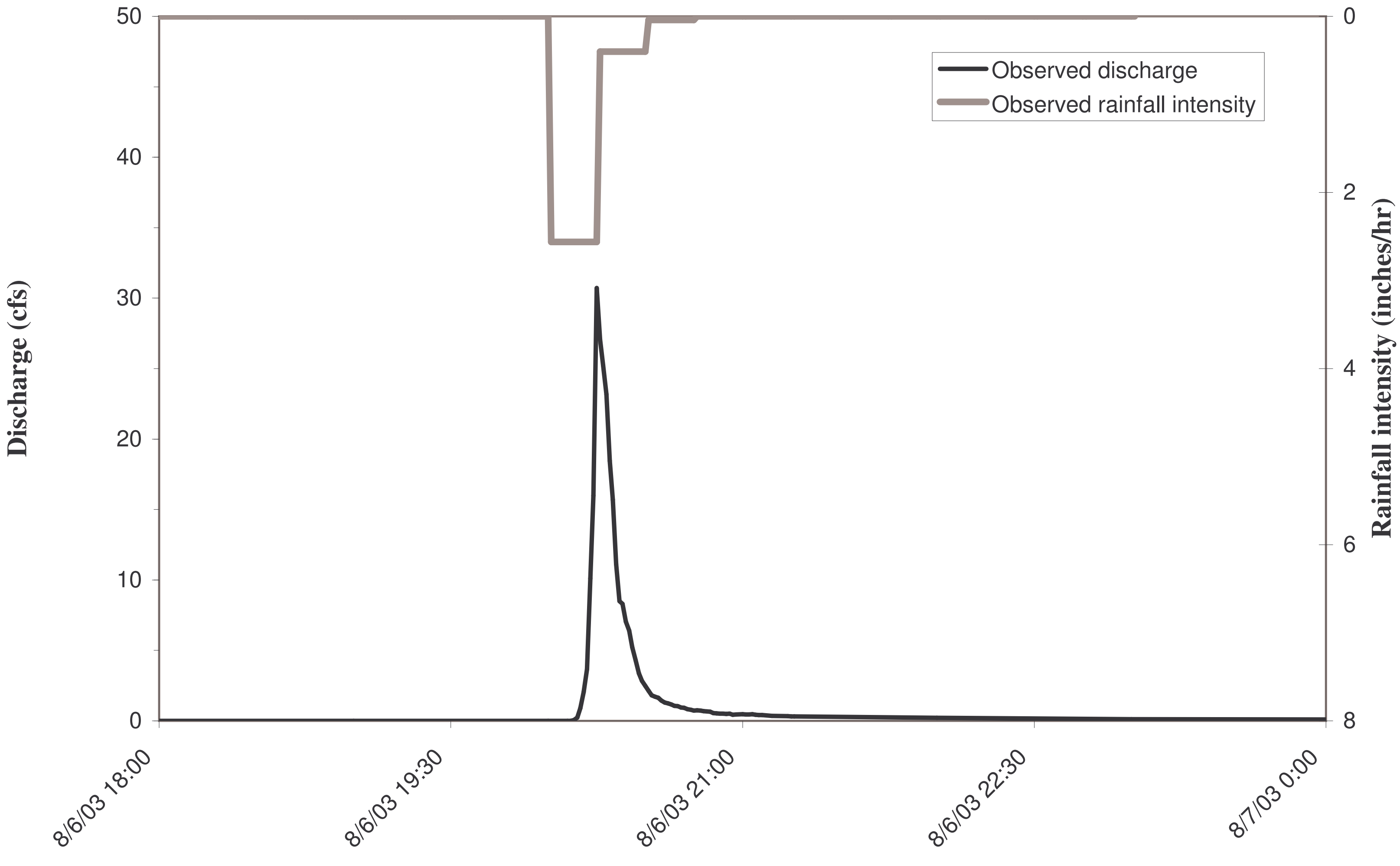



Eighteenth Street Storm Sewer
May 9, 2003
Discharge (cfs)
Rainfall intensity (inches/hr)
Observed discharge
Observed rainfall intensity
0
2
4
6
8
10
0.0
1.0
2.0
3.0
4.0
5/9/2003 0:00
5/9/2003 3:00
5/9/2003 6:00
5/9/2003 9:00
5/9/2003 12:00

Eighteenth Street Storm Sewer
July 6, 2003



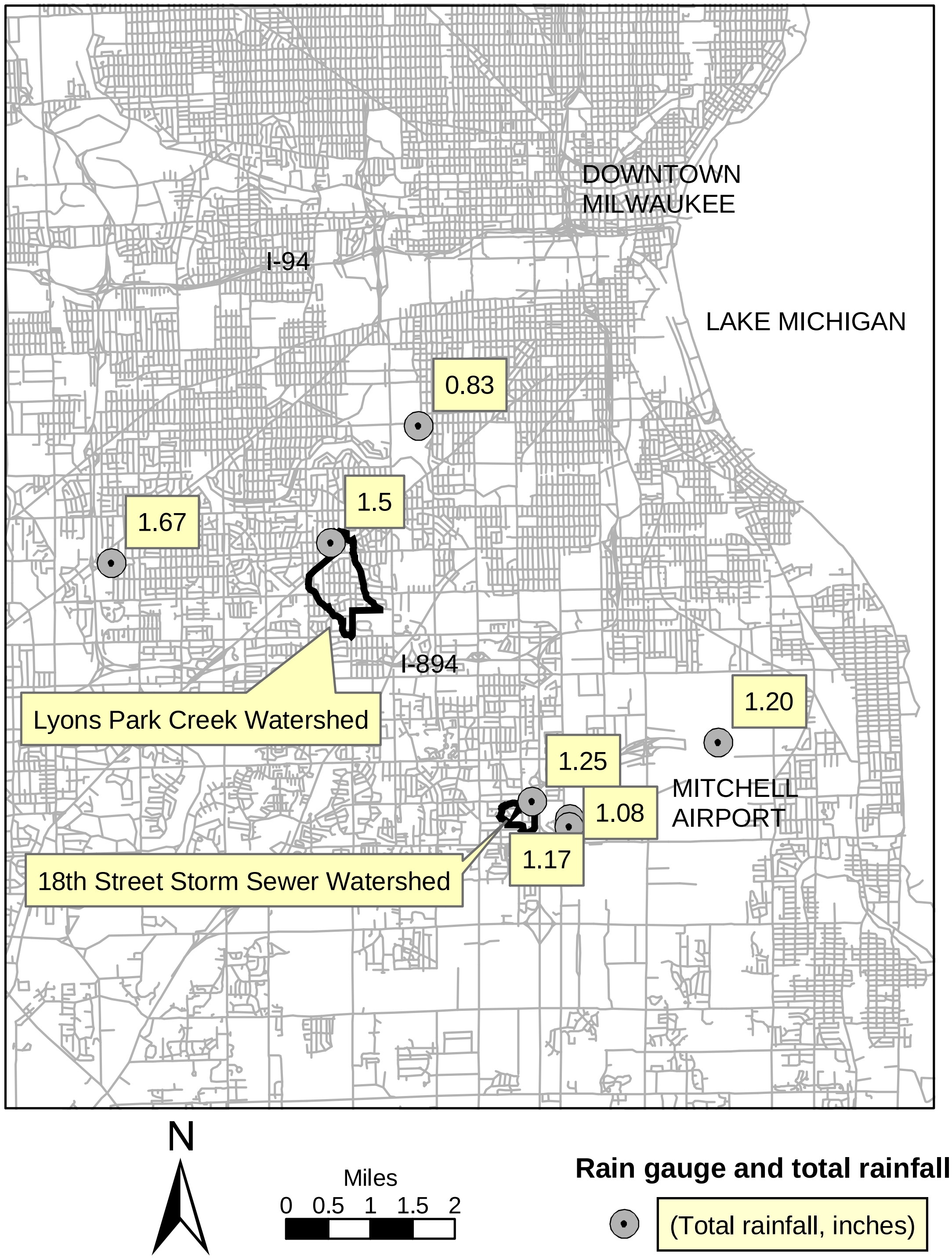
Eighteenth Street Storm Sewer
August 6, 2003



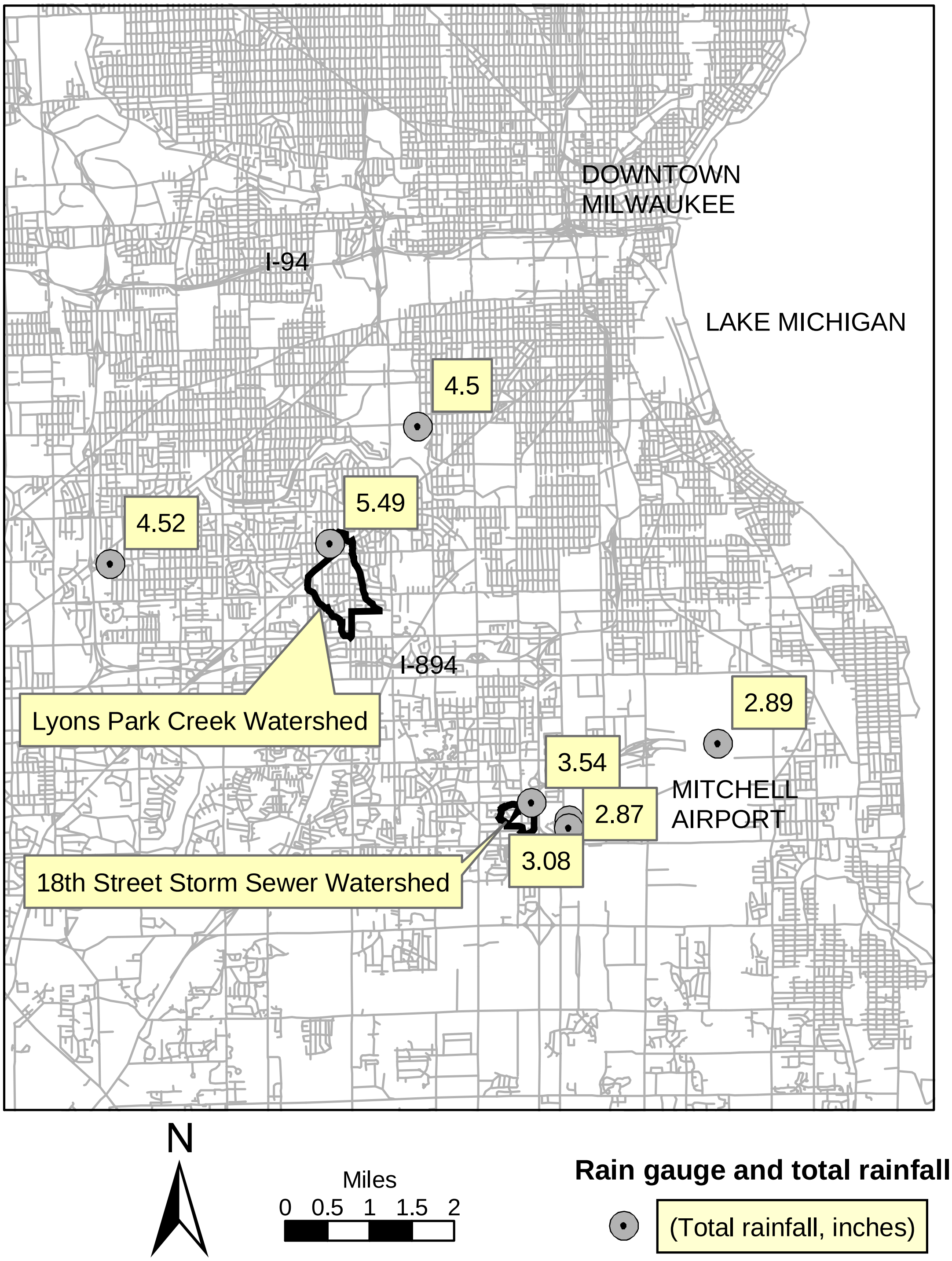
APPENDIX B

ADDITIONAL RAINFALL DATA

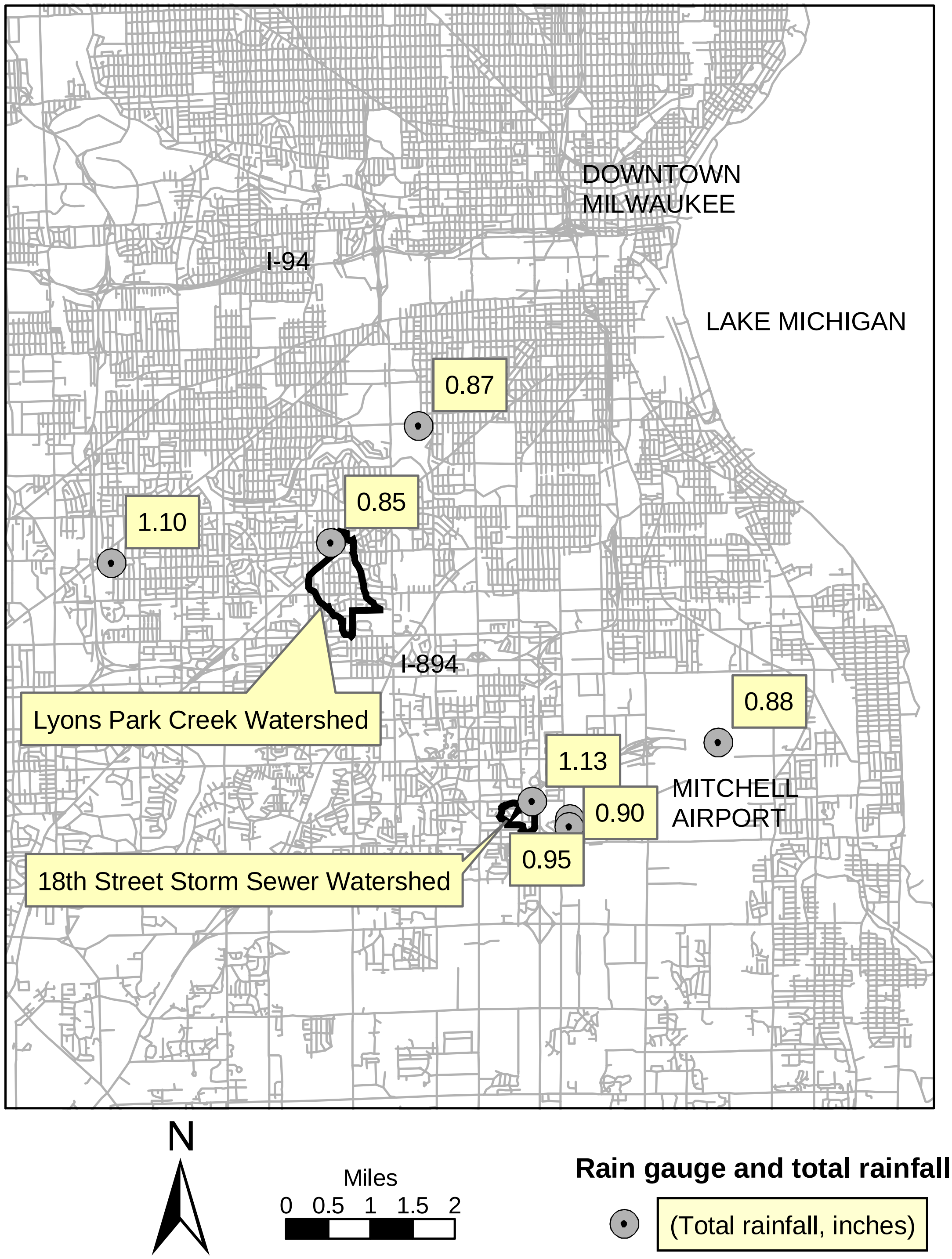
Rainfall Totals
July 26, 2002



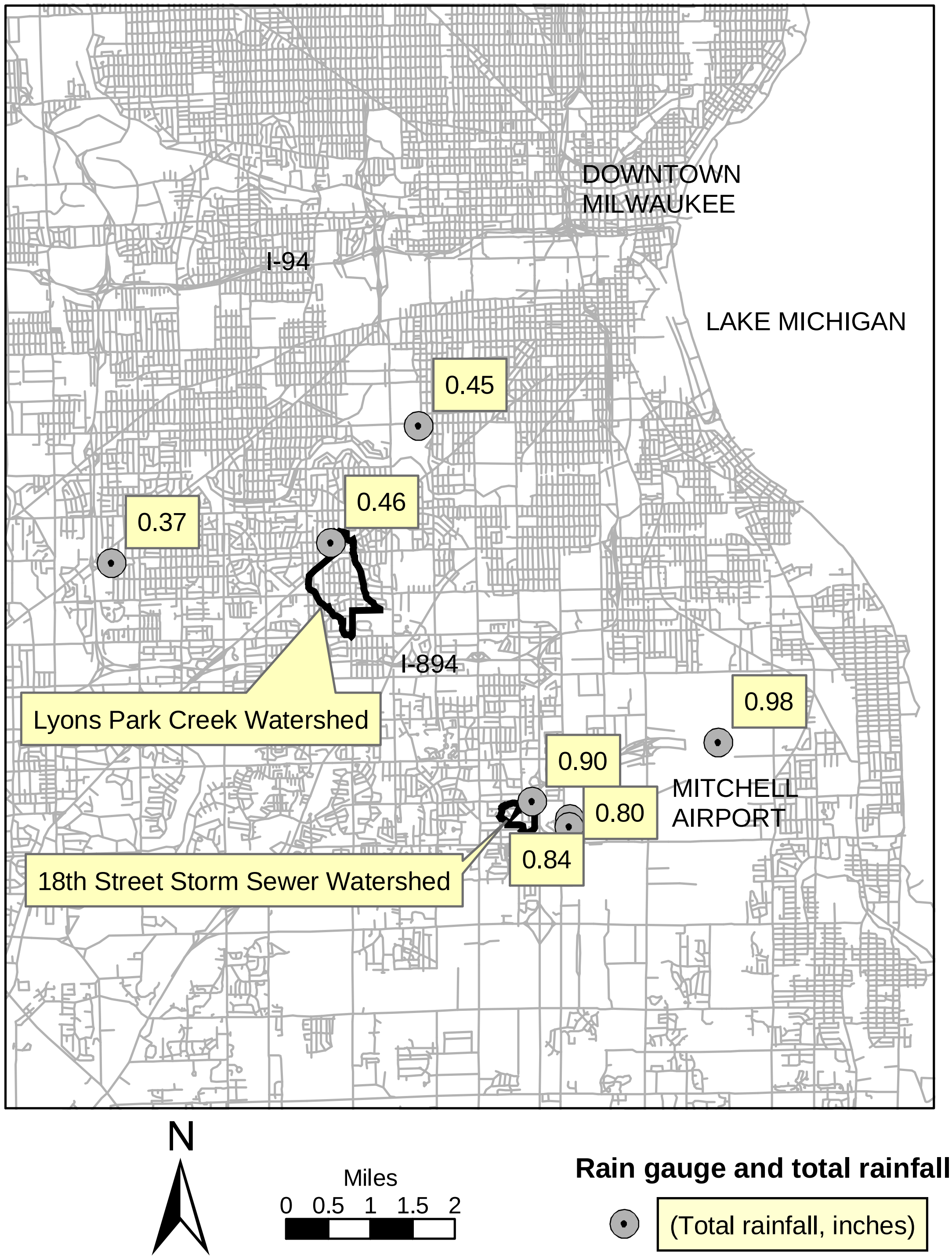
Rainfall Totals
August 12/13, 2002



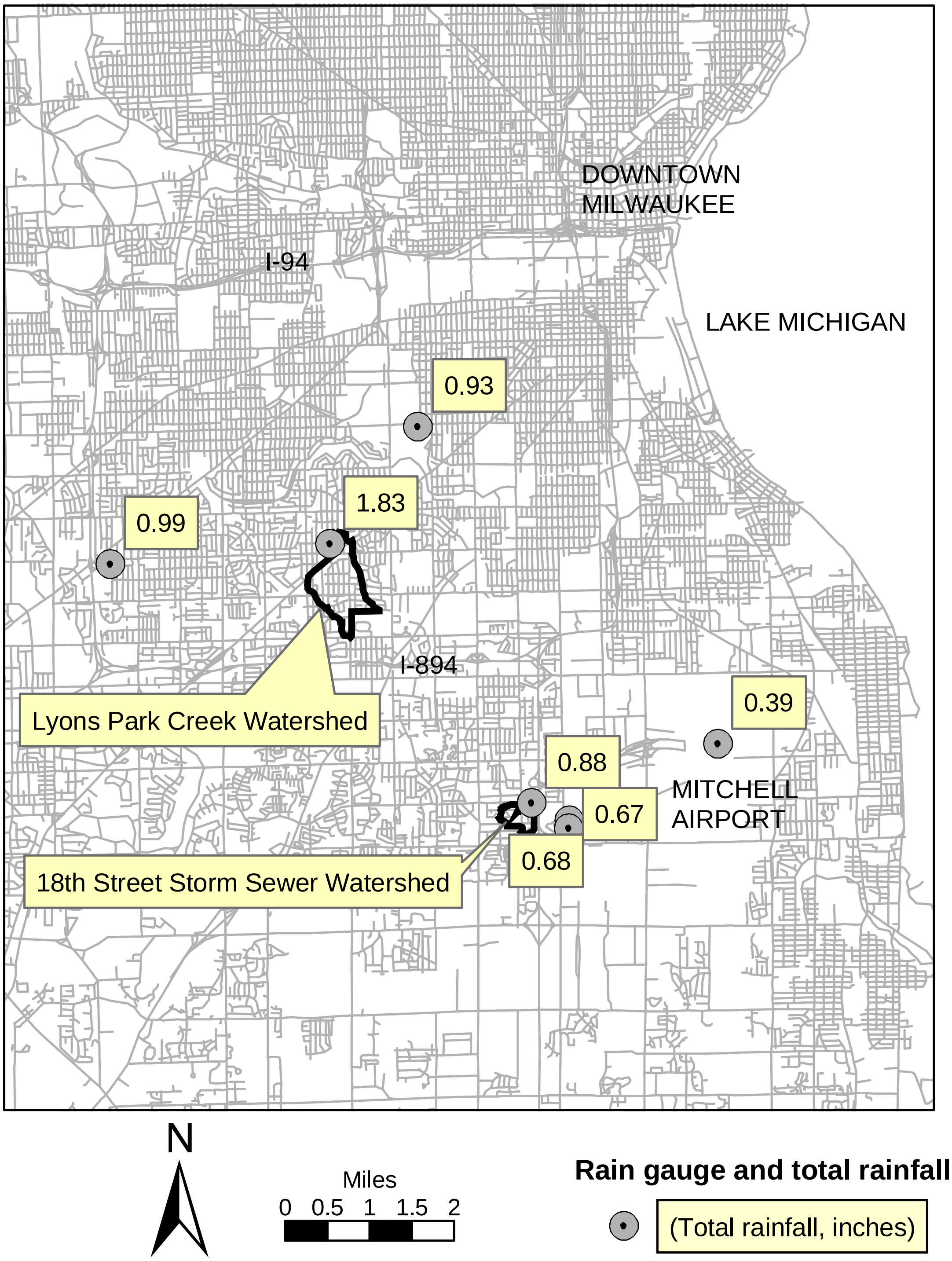
Rainfall Totals
August 21, 2002



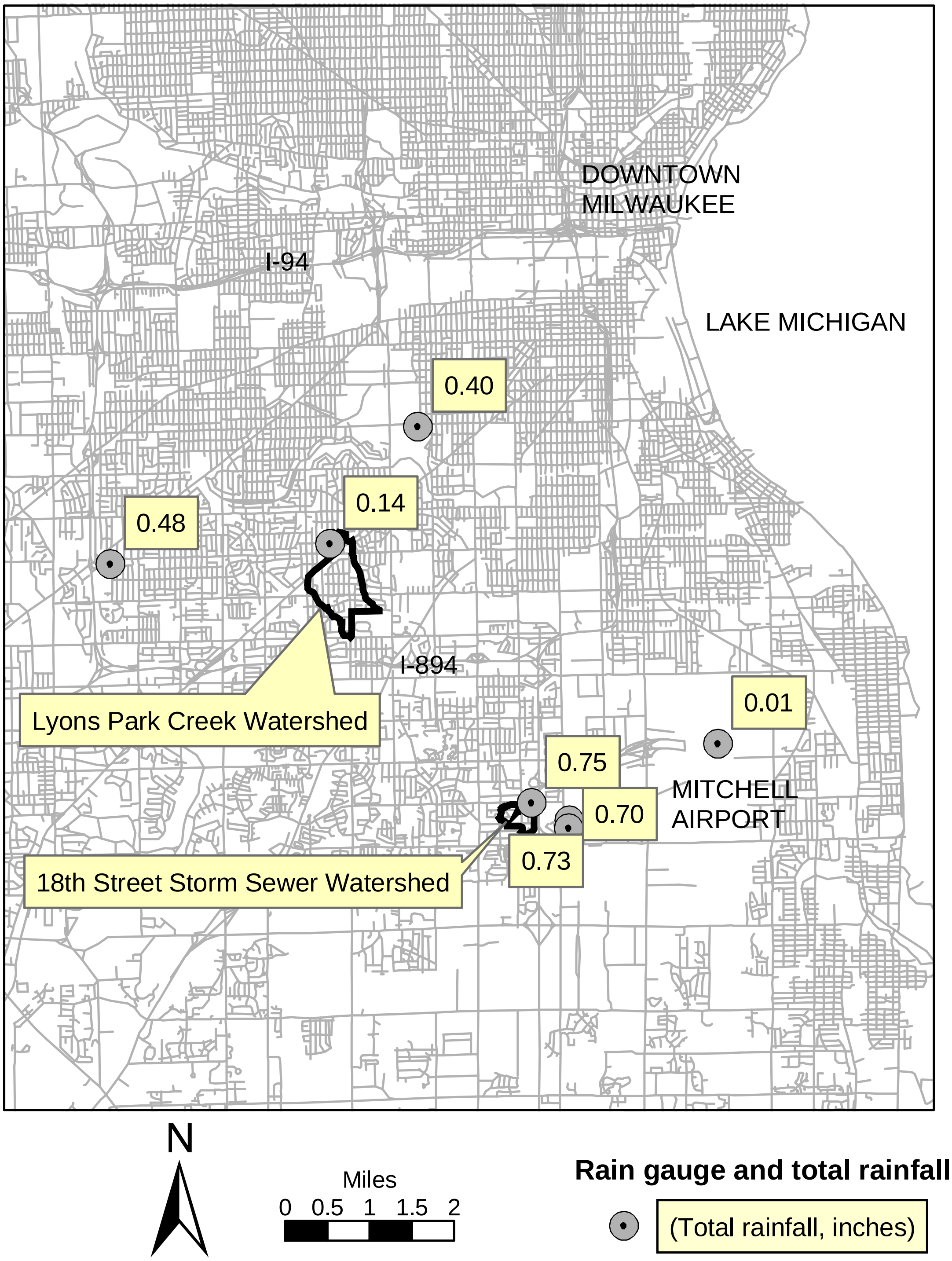
Rainfall Totals
July 6, 2003



Rainfall Totals
August 3, 2003



Rainfall Totals
August 6, 2003



APPENDIX C

MODEL RESULTS FOR

INDIVIDUAL STORM EVENTS

LYONS PARK CREEK
Runoff Volumes for Individual Storm Events

UNCALIBRATED MODELS

	Infiltration Routine	Single event or continuous simulation?	Runoff volume (inches)									
			6/3/2002	6/10/2002	7/8/2002	7/26/2002	8/12/2002 & 8/13/2002	8/21/2002	9/2/2002	4/30/2003	5/9/2003	8/3/2003
Observed runoff volume			*0.97*	*0.31*	*0.31*	*0.52*	*2.80*	*0.74*	*0.53*	*0.74*	*0.60*	*0.65*
Observed rainfall volume			*2.01*	*0.83*	*0.93*	*1.50*	*5.49*	*1.48*	*1.29*	*1.62*	*0.85*	*1.83*
SWMM - Simple	Horton	Single event	0.66	0.27	0.34	0.65	3.23	0.52	0.47	0.60	0.30	0.90
SWMM - Simple	Green-Ampt	Single event	0.67	0.28	0.34	0.67	3.48	0.52	0.48	0.61	0.30	0.93
SWMM - Detailed	Horton	Single event	0.70	0.29	0.36	0.75	3.71	0.53	0.51	0.66	0.31	1.06
SWMM - Detailed	Green-Ampt	Single event	0.72	0.31	0.38	0.78	3.93	0.53	0.52	0.68	0.31	1.09
SCS - Simple	curve number	Single event	0.85	0.12	0.16	0.49	3.95	0.48	0.36	0.58	0.12	0.73
SCS - Detailed	curve number	Single event	0.84	0.12	0.16	0.49	3.92	0.48	0.36	0.57	0.12	0.72
SWMM - Simple	Horton	Continuous	0.67	0.27	0.34	0.65	3.23	0.52	0.47	0.59	0.30	0.91
SWMM - Simple	Green-Ampt	Continuous	0.66	0.27	0.33	0.60	3.16	0.52	0.46	0.56	0.30	0.82
SWMM - Detailed	Horton	Continuous	0.71	0.29	0.36	0.75	3.71	0.53	0.51	0.65	0.32	1.07
SWMM - Detailed	Green-Ampt	Continuous	0.69	0.29	0.34	0.66	3.59	0.52	0.47	0.59	0.31	0.95

CALIBRATED MODELS

	Infiltration Routine	Single event or continuous simulation?	Runoff volume (inches)									
			6/3/2002	6/10/2002	7/8/2002	7/26/2002	8/12/2002 & 8/13/2002	8/21/2002	9/2/2002	4/30/2003	5/9/2003	8/3/2003
Observed runoff volume			*0.97*	*0.31*	*0.31*	*0.52*	*2.80*	*0.74*	*0.53*	*0.74*	*0.60*	*0.65*
Observed rainfall volume			*2.01*	*0.83*	*0.93*	*1.50*	*5.49*	*1.48*	*1.29*	*1.62*	*0.85*	*1.83*
SWMM - Simple	Horton	Continuous	0.66	0.27	0.34	0.59	2.60	0.52	0.47	0.55	0.30	0.81
SWMM - Simple	Green-Ampt	Continuous	0.65	0.27	0.34	0.58	2.69	0.52	0.47	0.55	0.30	0.76
SWMM - Detailed	Horton	Continuous	0.74	0.31	0.38	0.66	2.93	0.58	0.52	0.61	0.34	0.91
SWMM - Detailed	Green-Ampt	Continuous	0.73	0.31	0.37	0.65	2.79	0.58	0.52	0.61	0.34	0.86
SCS - Simple	composite constant curve number	Single event	0.85	0.12	0.17	0.50	3.95	0.48	0.36	0.58	0.13	0.72
SCS - Simple	separate constant curve numbers	Single event	0.83	0.26	0.31	0.57	3.22	0.56	0.47	0.63	0.27	0.74

LYONS PARK CREEK
Peak Flows for Individual Storm Events

UNCALIBRATED MODELS

	Infiltration Routine	Single event or continuous simulation?	Peak flow (cubic feet per second)										
			6/3/2002	6/10/2002	7/8/2002	7/26/2002	8/12/2002	8/13/2002	8/21/2002	9/2/2002	4/30/2003	5/9/2003	8/3/2003
Observed peak			*81*	*153*	*85*	*155*	*147*	*503*	*86*	*74*	*54*	*70*	*300*
SWMM - Simple	Horton	Single event	59	80	54	164	168	409	47	47	50	45	214
SWMM - Simple	Green-Ampt	Single event	60	81	54	166	169	412	48	48	51	45	217
SWMM - Detailed	Horton	Single event	91	125	81	211	209	549	79	56	61	49	308
SWMM - Detailed	Green-Ampt	Single event	93	127	82	216	209	554	81	56	62	50	316
SCS - Simple	curve number	Single event	39	30	39	162	229	643	32	49	64	33	193
SCS - Detailed	curve number	Single event	40	35	44	207	257	653	37	60	70	37	235
SWMM - Simple	Horton	Continuous	60	80	54	164	168	409	47	47	49	46	214
SWMM - Simple	Green-Ampt	Continuous	59	80	54	161	160	410	47	46	48	44	214
SWMM - Detailed	Horton	Continuous	92	125	81	211	209	549	79	56	59	51	309
SWMM - Detailed	Green-Ampt	Continuous	92	125	81	203	194	553	79	56	55	47	308
Rational Method (total area)	--	Single event	98	101	70	126	135	313	80	70	60	52	230
Rational Method (impervious only)	--	Single event	106	136	91	175	138	372	102	69	56	45	236

CALIBRATED MODELS

	Infiltration Routine	Single event or continuous simulation?	Peak flow (cubic feet per second)										
			6/3/2002	6/10/2002	7/8/2002	7/26/2002	8/12/2002	8/13/2002	8/21/2002	9/2/2002	4/30/2003	5/9/2003	8/3/2003
Observed peak			*81*	*153*	*85*	*155*	*147*	*503*	*86*	*74*	*54*	*70*	*300*
SWMM - Simple	Horton	Continuous	74	108	80	187	172	534	68	54	52	50	265
SWMM - Simple	Green-Ampt	Continuous	73	107	80	185	164	541	68	54	51	50	262
SWMM - Detailed	Horton	Continuous	86	113	73	195	178	518	73	55	54	49	294
SWMM - Detailed	Green-Ampt	Continuous	85	112	73	193	168	510	73	55	54	49	289
SCS - Simple	composite constant curve number	Single event	42	39	46	207	257	816	40	62	71	40	234
SCS - Simple	separate constant curve numbers	Single event	93	125	80	180	177	553	78	56	59	49	248
Rational Method (total area)	--	Single event	123	127	89	158	169	393	100	89	75	65	289
Rational Method (impervious only)	--	Single event	112	144	96	186	147	394	108	73	60	48	250

EIGHTEENTH STREET STORM SEWER
Runoff Volumes for Individual Storm Events

UNCALIBRATED MODELS

	Infiltration Routine	Single event or continuous simulation?	Runoff volume (inches)									
			6/3/2002	7/8/2002	7/26/2002	8/12/2002 & 8/13/2002	8/21/2002	9/2/2002	4/30/2003	5/9/2003	7/6/2003	8/6/2003
Observed runoff volume			*0.48*	*0.16*	*0.21*	*0.71*	*0.36*	*0.21*	*0.18*	*0.11*	*0.1*	*0.08*
Observed rainfall volume			*2.26*	*0.97*	*1.25*	*3.55*	*1.92*	*1.2*	*1.34*	*0.84*	*0.9*	*0.75*
SWMM - Simple	Horton	Continuous	0.80	0.34	0.51	1.87	0.70	0.47	0.43	0.29	0.29	0.27
SWMM - Simple	Green-Ampt	Continuous	0.74	0.32	0.45	1.74	0.66	0.41	0.40	0.28	0.28	0.26
SWMM - Medium Discretization	Horton	Continuous	0.86	0.37	0.56	2.01	0.74	0.51	0.47	0.31	0.31	0.29
SWMM - Medium Discretization	Green-Ampt	Continuous	0.78	0.33	0.48	1.88	0.70	0.43	0.42	0.29	0.29	0.28
SWMM - Detailed	Horton	Continuous	0.90	0.38	0.60	2.12	0.77	0.54	0.49	0.32	0.32	0.31
SWMM - Detailed	Green-Ampt	Continuous	0.81	0.34	0.51	1.99	0.72	0.45	0.43	0.29	0.30	0.30
SCS - Simple	curve number	Single event	0.91	0.12	0.25	1.91	0.64	0.22	0.29	0.08	0.09	0.05
SCS - Detailed	curve number	Single event	0.92	0.12	0.25	1.92	0.65	0.23	0.30	0.09	0.10	0.05

CALIBRATED MODELS

	Infiltration Routine	Single event or continuous simulation?	Runoff volume (inches)									
			6/3/2002	7/8/2002	7/26/2002	8/12/2002 & 8/13/2002	8/21/2002	9/2/2002	4/30/2003	5/9/2003	7/6/2003	8/6/2003
Observed runoff volume			*0.48*	*0.16*	*0.21*	*0.71*	*0.36*	*0.21*	*0.18*	*0.11*	*0.1*	*0.08*
Observed rainfall volume			*2.26*	*0.97*	*1.25*	*3.55*	*1.92*	*1.2*	*1.34*	*0.84*	*0.9*	*0.75*
SWMM - Simple	Green-Ampt	Continuous	0.40	0.18	0.24	0.71	0.37	0.23	0.23	0.15	0.15	0.14
SWMM - Medium Discretization	Green-Ampt	Continuous	0.40	0.18	0.24	0.71	0.37	0.23	0.23	0.15	0.15	0.14
SWMM - Detailed	Green-Ampt	Continuous	0.41	0.18	0.24	0.71	0.37	0.23	0.23	0.16	0.15	0.14
SCS - Simple	composite constant curve number	Single event	0.48	0.02	0.07	1.28	0.32	0.06	0.10	0.01	0.01	0.00
SCS - Simple	separate constant curve numbers	Single event	0.42	0.16	0.22	0.74	0.35	0.21	0.23	0.13	0.14	0.12

EIGHTEENTH STREET STORM SEWER
Peak Flows for Individual Storm Events

UNCALIBRATED MODELS

	Infiltration Routine	Single event or continuous simulation?	Peak flow (cubic feet per second)										
			6/3/2002	7/8/2002	7/26/2002	8/12/2002	8/13/2002	8/21/2002	9/2/2002	4/30/2003	5/9/2003	7/6/2003	8/6/2003
Observed peak			*28*	*26*	*19*	*47*	*27*	*50*	*13*	*5*	*5*	*29*	*31*
SWMM - Simple	Horton	Continuous	30	23	28	70	31	68	18	13	11	31	47
SWMM - Simple	Green-Ampt	Continuous	29	23	28	70	32	68	17	13	10	31	47
SWMM - Medium Discretization	Horton	Continuous	35	32	32	81	40	84	20	14	12	45	62
SWMM - Medium Discretization	Green-Ampt	Continuous	34	32	32	81	41	84	19	14	12	45	62
SWMM - Detailed	Horton	Continuous	38	34	32	80	43	89	21	15	13	44	70
SWMM - Detailed	Green-Ampt	Continuous	38	34	32	80	45	89	19	15	13	44	70
SCS - Simple	curve number	Single event	15	8	17	50	61	14	12	10	5	6	7
SCS - Detailed	curve number	Single event	19	9	24	64	76	19	14	12	6	9	9
Rational Method (total area)	--	Single event	33	24	33	64	34	48	20	15	15	24	39
Rational Method (impervious only)	--	Single event	42	47	34	60	38	96	20	23	17	56	79

CALIBRATED MODELS

	Infiltration Routine	Single event or continuous simulation?	Peak flow (cubic feet per second)										
			6/3/2002	7/8/2002	7/26/2002	8/12/2002	8/13/2002	8/21/2002	9/2/2002	4/30/2003	5/9/2003	7/6/2003	8/6/2003
Observed peak			*28*	*26*	*19*	*47*	*27*	*50*	*13*	*5*	*5*	*29*	*31*
SWMM - Simple	Green-Ampt	Continuous	23	17	20	46	25	58	12	10	8	33	38
SWMM - Medium Discretization	Green-Ampt	Continuous	23	17	19	45	24	57	11	9	8	33	37
SWMM - Detailed	Green-Ampt	Continuous	25	17	19	43	26	57	12	9	8	29	39
SCS - Simple	composite constant curve number	Single event	5	2.5	11	36	63	5	3.4	3.5	0.8	2	0.1
SCS - Simple	separate constant curve numbers	Single event	27	22	24	43	26	57	13	12	9	40	40
Rational Method (total area)	--	Single event	27	28	23	39	23	57	12	13	10	36	46
Rational Method (impervious only)	--	Single event	27	28	22	39	22	56	12	13	9	35	45

APPENDIX D

MODEL SENSITIVITY ANALYSES

Lyons Park Creek Sensitivity Analysis

Original values (uncalibrated SWMM model):

Area	297	acres
Imperviousness	40%	
Width	1840	ft
Slope	0.0081	
Imp. depression storage	0.02	in
Per. depression storage	0.1	in
Imp. roughness	0.015	
Per. roughness	0.24	
% zero detention	25	
Green-Ampt suction	11.5	in
Green-Ampt init. deficit	0.09	
Green-Ampt sat. K	0.04	in/hr
Horton max rate	1.5	in/hr
Horton final rate	0.1	in/hr
Horton decay constant	0.00115	/sec

Lyons Park Creek
Large storm (August 12/13, 2002)
Automated XPSWMM sensitivity analysis

Runoff Input parameter	% change	% change in runoff volume	% change in peak flow
Area	+25%	29	43.3*
Area	-25%	-28	-36.7*
Imperviousness	+25%	8	9
Imperviousness	-25%	-8	-10
Width	+25%	3	15
Width	-25%	-3	-16
Slope	+25%	1	7
Slope	-25%	-2	-9
Imp. depression storage	+25%	0	0
Imp. depression storage	-25%	0	0
Per. depression storage	+25%	-1	0
Per. depression storage	-25%	1	0
Imp. roughness	+25%	0	-11
Imp. roughness	-25%	0	16
Per. roughness	+25%	-3	-2
Per. roughness	-25%	3	3
Green-Ampt suction	+25%	-2	0
Green-Ampt suction	-25%	2	0
Green-Ampt init. deficit	+25%	-4	0
Green-Ampt init. deficit	-25%	6	1
Green-Ampt sat. K	+25%	-2	0
Green-Ampt sat. K	-25%	2	0
Horton max rate	+25%	-1	0
Horton max rate	-25%	1	0
Horton final rate	+25%	-5	0
Horton final rate	-25%	6	0
Horton decay constant	+25%	1	0
Horton decay constant	-25%	-2	0

* There may have been a software problem with the variation of area during the automated sensitivity analysis, leading to an inaccurate result.

Lyons Park Creek
Small storm (July 8, 2002)
Automated XPSWMM sensitivity analysis

Runoff Input parameter	% change	% change in runoff volume	% change in peak flow
Area	+25%	27	50.3*
Area	-25%	-27	-34.4*
Imperviousness	+25%	23	12
Imperviousness	-25%	-23	-5
Width	+25%	2	20
Width	-25%	-2	-13
Slope	+25%	1	10
Slope	-25%	-1	-7
Imp. depression storage	+25%	0	-1
Imp. depression storage	-25%	0	1
Per. depression storage	+25%	-1	-1
Per. depression storage	-25%	1	1
Imp. roughness	+25%	-1	-9
Imp. roughness	-25%	1	25
Per. roughness	+25%	-1	-1
Per. roughness	-25%	1	1
Green-Ampt suction	+25%	-1	-1
Green-Ampt suction	-25%	2	1
Green-Ampt init. deficit	+25%	-2	-1
Green-Ampt init. deficit	-25%	3	1
Green-Ampt sat. K	+25%	-1	-1
Green-Ampt sat. K	-25%	2	1
Horton max rate	+25%	-1	-1
Horton max rate	-25%	2	2
Horton final rate	+25%	-1	-1
Horton final rate	-25%	1	1
Horton decay constant	+25%	1	1
Horton decay constant	-25%	-2	-2

* There may have been a software problem with the variation of area during the automated sensitivity analysis, leading to an inaccurate result.

"Reasonable range" sensitivity analysis for large storm - SWMM parameters

Parameter	Value	Runoff volume (in)	% change in volume	Peak flow (cfs)	% change in peak
Area	282				
	297				
	312				
Imperviousness	See previous table				
Imp. depression storage	0.01	3.55	0.0%	423	0.0%
	0.02	3.55		423	
	0.1	3.52	-0.8%	423	0.0%
Pervious depression storage	0.05	3.59	1.1%	425	0.5%
	0.1	3.55		423	
	0.25	3.42	-3.7%	419	-0.9%
Impervious % zero detention	0	3.51	-0.3%	423	0.0%
	25	3.52		423	
	50	3.53	0.3%	424	0.2%
Green-Ampt suction	6	3.73	5.1%	426	0.7%
	11.5	3.55		423	
	14	3.49	-1.7%	423	0.0%
Green-Ampt Initial deficit	0.01	4.03	13.5%	432	2.1%
	0.09	3.55		423	
	0.17	3.35	-5.6%	423	0.0%
Green-Ampt Sat. K.	0.01	4.31	21.4%	439.0	3.8%
	0.04	3.55		423	
	0.27	2.24	-36.9%	401.0	-5.2%
Horton max rate	0.5	3.44	3.0%	423.0	0.5%
	1.5	3.34		421.0	
	5	3.00	-10.2%	415.0	-1.4%
Horton final rate	0.01	4.54	35.9%	445	5.7%
	0.1	3.34		421.0	
	0.3	2.53	-24.3%	413	-1.9%
Horton decay constant	0.0006	3.16	-5.4%	420	-0.2%
	0.00115	3.34		421.0	
	0.0015	3.34	0.0%	421	0.0%

Extran sensitivities

All models runs for Level C, hydrology pre-calibration base case, Green-Ampt infiltration

Upstream storm sewer roughness	0.010	3.99	0.0%	559	0.4%
	0.014	3.99		557	
	0.020	4.00	0.3%	507	-9.0%
Open channel roughness	0.028	3.99	0.0%	565	1.4%
	0.035	3.99		557	
	0.050	3.98	-0.3%	521	-6.5%
Culvert roughness	0.010			555	-0.4%
	0.016			557	
	0.025			549	-1.4%
Culvert entrance loss coefficient	0.20			555	-0.4%
	0.50			557	
	0.80			551	-1.1%
Culvert exit loss coefficient	0.50			551	-1.1%
	1.00			557	
	2.00			544	-2.3%
Open channel bottom width	2.00			530	-4.8%
	8.00			557	
	20.00			557	0.0%

"Reasonable range" sensitivity analysis for small storm - SWMM parameters

Parameter	Value	Runoff volume (in)	% change in volume	Peak flow (cfs)	% change in peak
Area	282				
	297				
	312				
Imperviousness	See previous table				
Imp. depression storage	0.01	0.36	0.0%	57.85	2.0%
	0.02	0.36		56.7	
	0.1	0.34	-5.6%	54.4	-4.1%
Pervious depression storage	0.05	0.367	1.9%	57.3	1.1%
	0.1	0.36		56.7	
	0.25	0.348	-3.3%	55.1	-2.8%
Impervious % zero detention	0	0.326	-3.0%	52.8	-2.9%
	25	0.336		54.4	
	50	0.346	3.0%	55.9	2.8%
Green-Ampt suction	6	0.378	5.0%	58.1	2.5%
	11.5	0.360		56.7	
	14	0.356	-1.1%	56.2	-0.9%
Green-Ampt Initial deficit	0.01	0.42	16.7%	60.3	6.3%
	0.09	0.360		56.7	
	0.17	0.348	-3.3%	55.1	-2.8%
Green-Ampt Sat. K.	0.01	0.424	17.8%	60.2	6.2%
	0.04	0.360		56.7	
	0.27	0.344	-4.4%	54.0	-4.8%
Horton max rate	0.5	0.384	8.8%	58.9	5.6%
	1.5	0.353		55.8	
	5	0.344	-2.5%	54.0	-3.2%
Horton final rate	0.01	0.405	14.7%	57.9	3.8%
	0.1	0.353		55.8	
	0.3	0.344	-2.5%	54.5	-2.3%
Horton decay constant	0.0006	0.344	-2.5%	54.3	-2.7%
	0.00115	0.353		55.8	
	0.0015	0.361	2.3%	56.7	1.6%
Extran sensitivities					
All models runs for Level C, hydrology pre-calibration base case, Green-Ampt infiltration					
Upstream storm sewer roughness	0.010	0.39		92	5.7%
	0.014	0.39		87	
	0.020	0.39		81	-6.9%
Open channel roughness	0.028			89	2.3%
	0.035			87	
	0.050			79	-9.2%
Culvert roughness	0.010			87	0.0%
	0.016			87	
	0.025			87	0.0%
Culvert entrance loss coefficient	0.20			87	0.0%
	0.50			87	
	0.80			88	1.1%
Culvert exit loss coefficient	0.50			86	-1.1%
	1.00			87	
	2.00			87	0.0%
Open channel bottom width	2.00			80	-8.0%
	8.00			87	
	20.00			83	-4.6%

SCS Model - Small Storm (July 8, 2002) Sensitivity Analysis

Parameter	Value	Runoff volume (in)	% change in volume	Peak flow (cfs)	% change in peak
curve number	74	0.01	-94.1%	5	-89.1%
	86	0.17		46	
	95	0.51	200.0%	114	147.8%
time of concentration	24	0.17	0.0%	59	28.3%
	36	0.17		46	
	50	0.17	0.0%	39	-15.2%
Hydrograph shape factor	363	0.17	0.0%	40	-13.0%
	484	0.17		46	
	605	0.17	0.0%	53	15.2%

SCS Model - Large Storm (August 12/13, 2002) Sensitivity Analysis

Parameter	Value	Runoff volume (in)	% change in volume	Peak flow (cfs)	% change in peak
curve number	74	2.77	-29.9%	649	-20.5%
	86	3.95		816	
	95	4.93	24.8%	897	9.9%
time of concentration	24	3.95	0.0%	1048	28.4%
	36	3.95		816	
	50	3.95	0.0%	643	-21.2%
Hydrograph shape factor	363	3.95	0.0%	664	-18.6%
	484	3.95		816	
	605	3.93	-0.5%	951	16.5%

Rational Method - Small Storm (July 8, 2002) Sensitivity Analysis

Parameter	Value	Peak flow (cfs)	% change in peak
Runoff coefficient	0.32	70	-25.6%
	0.43	94	
	0.54	118	25.6%
time of concentration	24	131	39.8%
	36	94	
	50	70	-24.7%

Rational Method - Large Storm (August 12/13, 2002) Sensitivity Analysis

Parameter	Value	Peak flow (cfs)	% change in peak
Runoff coefficient	0.32	293	-25.6%
	0.43	394	
	0.54	495	25.6%
time of concentration	24	527	33.8%
	36	394	
	50	313	-20.6%

APPENDIX E

MODEL ERROR STATISTICS

LYONS PARK CREEK
Error Summary for Runoff Volumes

UNCALIBRATED MODELS

			Absolute errors (inches)						Percentage errors					
		Single event	2002 and 2003		2002 only		2003 only		2002 and 2003		2002 only		2003 only	
	Infiltration Routine	or continuous simulation?	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error
SWMM - Simple	Horton	Single event	-0.02	0.23	-0.01	0.23	-0.07	0.24	-7%	28%	-5%	21%	-10%	38%
SWMM - Simple	Green-Ampt	Single event	0.01	0.28	0.04	0.30	-0.05	0.25	-4%	29%	-2%	22%	-8%	39%
SWMM - Detailed	Horton	Single event	0.07	0.35	0.10	0.38	0.01	0.29	3%	34%	4%	27%	1%	46%
SWMM - Detailed	Green-Ampt	Single event	0.11	0.42	0.14	0.46	0.03	0.31	7%	36%	8%	29%	4%	48%
SCS - Simple	curve number	Single event	-0.03	0.42	0.03	0.46	-0.19	0.29	-24%	41%	-22%	38%	-30%	48%
SCS - Detailed	curve number	Single event	-0.04	0.41	0.03	0.45	-0.19	0.30	-25%	41%	-22%	38%	-31%	48%
SWMM - Simple	Horton	Continuous	-0.02	0.23	0.00	0.22	-0.06	0.24	-7%	27%	-5%	21%	-10%	38%
SWMM - Simple	Green-Ampt	Continuous	-0.05	0.21	-0.03	0.20	-0.10	0.22	-10%	26%	-8%	20%	-16%	36%
SWMM - Detailed	Horton	Continuous	0.07	0.35	0.10	0.38	0.02	0.29	4%	34%	4%	27%	2%	46%
SWMM - Detailed	Green-Ampt	Continuous	0.02	0.31	0.06	0.33	-0.05	0.26	-3%	29%	-1%	22%	-8%	40%

CALIBRATED MODELS

			Absolute errors (inches)						Percentage errors					
		Single event	2002 and 2003		2002 only		2003 only		2002 and 2003		2002 only		2003 only	
	Infiltration Routine	or continuous simulation?	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error
SWMM - Simple	Horton	Continuous	-0.11	0.19	-0.11	0.17	-0.11	0.22	-12%	25%	-10%	19%	-17%	35%
SWMM - Simple	Green-Ampt	Continuous	-0.10	0.18	-0.10	0.16	-0.13	0.21	-13%	24%	-10%	19%	-20%	34%
SWMM - Detailed	Horton	Continuous	-0.02	0.17	-0.01	0.13	-0.04	0.23	-2%	25%	1%	18%	-7%	36%
SWMM - Detailed	Green-Ampt	Continuous	-0.04	0.15	-0.03	0.12	-0.06	0.21	-3%	23%	-1%	18%	-10%	33%
SCS - Simple	composite constant curve number	Single event	-0.03	0.42	0.04	0.46	-0.19	0.29	-24%	41%	-21%	38%	-30%	47%
SCS - Simple	separate constant curve numbers	Single event	-0.03	0.19	0.00	0.19	-0.12	0.21	-10%	22%	-6%	15%	-19%	34%

LYONS PARK CREEK
Error Summary for Peak Flows

UNCALIBRATED MODELS

			Absolute errors (cubic feet per second)						Percentage errors					
		Single event	2002 and 2003		2002 only		2003 only		2002 and 2003		2002 only		2003 only	
	Infiltration Routine	or continuous simulation?	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error
SWMM - Simple	Horton	Single event	-34	49	-32	48	-38	52	-24%	31%	-24%	32%	-24%	27%
SWMM - Simple	Green-Ampt	Single event	-32	48	-31	47	-37	50	-23%	30%	-23%	32%	-23%	26%
SWMM - Detailed	Horton	Single event	10	32	15	36	-2	14	3%	22%	6%	23%	-5%	19%
SWMM - Detailed	Green-Ampt	Single event	13	33	17	38	1	15	4%	23%	7%	24%	-3%	19%
SCS - Simple	curve number	Single event	-18	75	-8	78	-45	66	-24%	48%	-24%	51%	-23%	38%
SCS - Detailed	curve number	Single event	-3	76	6	85	-27	43	-14%	48%	-14%	53%	-13%	34%
SWMM - Simple	Horton	Continuous	-34	49	-32	48	-38	52	-24%	31%	-24%	32%	-24%	26%
SWMM - Simple	Green-Ampt	Continuous	-35	49	-33	47	-39	52	-25%	31%	-25%	32%	-26%	28%
SWMM - Detailed	Horton	Continuous	10	31	15	36	-2	12	3%	22%	6%	23%	-5%	17%
SWMM - Detailed	Green-Ampt	Continuous	8	28	12	32	-5	14	0%	20%	4%	20%	-9%	19%
Rational Method (total area)	--	Single event	-34	64	-36	71	-27	42	-13%	22%	-13%	22%	-13%	21%
Rational Method (impervious only)	--	Single event	-17	46	-12	49	-29	40	-3%	19%	2%	17%	-18%	24%

CALIBRATED MODELS

			Absolute errors (cubic feet per second)						Percentage errors					
		Single event	2002 and 2003		2002 only		2003 only		2002 and 2003		2002 only		2003 only	
	Infiltration Routine	or continuous simulation?	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error
SWMM - Simple	Horton	Continuous	-6	25	-1	26	-19	23	-8%	19%	-6%	19%	-15%	18%
SWMM - Simple	Green-Ampt	Continuous	-7	26	-2	26	-20	25	-9%	19%	-7%	19%	-16%	18%
SWMM - Detailed	Horton	Continuous	-2	22	1	25	-9	13	-5%	19%	-3%	19%	-11%	17%
SWMM - Detailed	Green-Ampt	Continuous	-4	21	-2	23	-11	14	-6%	18%	-5%	18%	-11%	17%
SCS - Simple	composite constant curve number	Single event	13	111	28	128	-26	43	-9%	50%	-8%	54%	-11%	33%
SCS - Simple	separate constant curve numbers	Single event	-1	28	7	26	-23	33	-3%	17%	0%	16%	-13%	21%
Rational Method (total area)	--	Single event	-3	38	-5	44	2	14	9%	23%	9%	23%	10%	23%
Rational Method (impervious only)	--	Single event	-8	40	-3	42	-22	32	3%	20%	9%	20%	-13%	21%

EIGHTEENTH STREET STORM SEWR
Error Summary for Runoff Volumes

UNCALIBRATED MODELS

			Absolute errors (inches)						Percentage errors					
		Single event	2002 and 2003		2002 only		2003 only		2002 and 2003		2002 only		2003 only	
	Infiltration Routine	or continuous simulation?	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error
SWMM - Simple	Horton	Continuous	0.34	0.44	0.43	0.54	0.20	0.21	144%	151%	118%	122%	183%	186%
SWMM - Simple	Green-Ampt	Continuous	0.29	0.38	0.36	0.47	0.19	0.19	127%	136%	99%	102%	170%	174%
SWMM - Medium Discretization	Horton	Continuous	0.38	0.50	0.49	0.61	0.23	0.23	162%	170%	135%	140%	203%	207%
SWMM - Medium Discretization	Green-Ampt	Continuous	0.33	0.44	0.41	0.54	0.20	0.20	140%	149%	111%	115%	185%	190%
SWMM - Detailed	Horton	Continuous	0.42	0.54	0.53	0.66	0.24	0.25	175%	183%	147%	152%	218%	222%
SWMM - Detailed	Green-Ampt	Continuous	0.35	0.47	0.45	0.58	0.21	0.21	148%	158%	119%	124%	192%	199%
SCS - Simple	curve number	Single event	0.20	0.41	0.32	0.53	0.01	0.06	33%	70%	56%	85%	-1%	38%
SCS - Detailed	curve number	Single event	0.20	0.42	0.33	0.54	0.02	0.06	35%	71%	58%	87%	2%	38%

CALIBRATED MODELS

			Absolute errors (inches)						Percentage errors					
		Single event	2002 and 2003		2002 only		2003 only		2002 and 2003		2002 only		2003 only	
	Infiltration Routine	or continuous simulation?	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error
SWMM - Simple	Green-Ampt	Continuous	0.02	0.04	0.00	0.04	0.05	0.05	21%	33%	4%	11%	47%	50%
SWMM - Medium Discretization	Green-Ampt	Continuous	0.02	0.04	0.00	0.04	0.05	0.05	21%	33%	4%	11%	48%	50%
SWMM - Detailed	Green-Ampt	Continuous	0.02	0.04	0.00	0.04	0.05	0.05	23%	34%	5%	11%	50%	52%
SCS - Simple	composite constant curve number	Single event	-0.02	0.20	0.02	0.25	-0.09	0.09	-48%	72%	-26%	63%	-81%	84%
SCS - Simple	separate constant curve numbers	Single event	0.01	0.03	0.00	0.03	0.04	0.04	13%	23%	-1%	6%	34%	36%

EIGHTEENTH STREET STORM SEWR
Error Summary for Peak Flows

UNCALIBRATED MODELS

			Absolute errors (cubic feet per second)						Percentage errors					
			2002 and 2003		2002 only		2003 only		2002 and 2003		2002 only		2003 only	
	Infiltration Routine	Single event or continuous simulation?	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error
SWMM - Simple	Horton	Continuous	8	11	8	12	8	9	47%	68%	26%	33%	85%	103%
SWMM - Simple	Green-Ampt	Continuous	8	11	8	12	8	9	45%	64%	25%	32%	80%	98%
SWMM - Medium Discretization	Horton	Continuous	16	19	16	20	16	18	76%	88%	51%	55%	119%	128%
SWMM - Medium Discretization	Green-Ampt	Continuous	16	19	16	20	16	18	75%	88%	50%	54%	119%	128%
SWMM - Detailed	Horton	Continuous	18	22	18	22	18	22	86%	100%	58%	60%	134%	145%
SWMM - Detailed	Green-Ampt	Continuous	18	22	18	22	18	22	85%	99%	57%	59%	134%	145%
SCS - Simple	curve number	Single event	-7	19	-5	21	-11	17	-12%	68%	-11%	63%	-14%	75%
SCS - Detailed	curve number	Single event	-2	21	2	24	-9	15	10%	82%	13%	79%	5%	86%
Rational Method (total area)	--	Single event	6	9	7	9	6	9	55%	91%	28%	39%	102%	142%
Rational Method (impervious only)	--	Single event	21	25	18	22	26	30	116%	150%	61%	64%	212%	234%

CALIBRATED MODELS

			Absolute errors (cubic feet per second)						Percentage errors					
			2002 and 2003		2002 only		2003 only		2002 and 2003		2002 only		2003 only	
	Infiltration Routine	Single event or continuous simulation?	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error	Mean Error	Root Mean Squared Error
SWMM - Simple	Green-Ampt	Continuous	1	5	-1	5	5	5	13%	38%	-7%	17%	49%	60%
SWMM - Medium Discretization	Green-Ampt	Continuous	0	5	-2	5	4	4	9%	34%	-10%	17%	43%	51%
SWMM - Detailed	Green-Ampt	Continuous	0	5	-2	5	4	5	10%	33%	-7%	15%	41%	52%
SCS - Simple	composite constant curve number	Single event	-13	24	-12	26	-16	21	-52%	83%	-38%	83%	-77%	81%
SCS - Simple	separate constant curve numbers	Single event	3	6	0	4	8	8	27%	52%	1%	13%	72%	84%
Rational Method (total area)	--	Single event	3	7	0	5	9	9	30%	60%	0%	13%	82%	98%
Rational Method (impervious only)	--	Single event	3	6	-1	4	8	9	28%	58%	-1%	13%	79%	95%