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# Improving the accuracy of simple runoff estimates : the role of parameter resolution and data collection

Steven Reed Potter

To the Graduate Council:

I am submitting herewith a thesis written by Steven Reed Potter entitled "Improving the accuracy of simple runoff estimates : the role of parameter resolution and data collection." I have examined the final electronic copy of this thesis for form and content and recommend that it be accepted in partial fulfillment of the requirements for the degree of Master of Science, with a major in Biosystems Engineering Technology.

Daniel C. Yoder, Major Professor

We have read this thesis and recommend its acceptance:

Ronald E. Yoder, Roger B. Clapp

Accepted for the Council:
Carolyn R. Hodges

Vice Provost and Dean of the Graduate School

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

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# Improving the Accuracy of Simple Runoff Estimates:

# The Role of Parameter Resolution and Data Collection

A

Thesis

Presented for the

Master of Science Degree

The University of Tennessee, Knoxville

Steven Reed Potter

December, 1999

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## Abstract

This research project collected, compared, and analyzed rainfall and runoff data from two similar small watersheds within Knoxville's Second Creek drainage area. The primary reason for the study was to determine the most efficient way to increase the accuracy of two well known and simple runoff estimation models: the Rational and SCS Curve Number Methods. This was accomplished by investigating the impact that incorporating different amounts and types of information had on the accuracy of the models. An important component of this investigation was to examine the relative cost-benefit ratios of the different techniques that were attempted.

To optimize the models, the investigation took a three-pronged approach. First, the a priori model parameters and the parameter selection methods were optimized by using increasingly higher-resolution data to characterize the watersheds. The second step of the research was to see by what degree collecting rain and/or runoff data improved model estimations on that watershed. The process began by using the least possible data and incrementally increasing it. The final approach was to investigate whether the measured rainfall-runoff records from one watershed enhanced the estimates on another similar, nearby watershed. Once those steps had been accomplished, a benefit-cost analysis was performed to determine which techniques and data most efficiently improved the models.

For the peak flow estimates, the results showed that fine-tuning the parameters with high-resolution data did not result in better estimates. In fact, for these watersheds the highest resolution parameters produced some of the poorest estimates. Using observed runoff data, however, substantially decreased the estimate errors. Errors were reduced up to 90% using data collected within the watershed. Using data collected from a similar watershed to cross-calibrate the model reduced errors by up to 70%. In addition, the results indicated that more data further improved the estimates. However, the larger amounts of data tended to have a lower benefit to cost ratio.

The results from the volume estimates were not as clear-cut. While all the techniques appeared to work, the evaluation was hampered by the limited observations of storm events. Thus, the first two techniques, the a priori and the calibrated estimates were unreliable. Even so, the third technique, which used data from the similar watershed to calibrate the model, reduced errors by approximately 60 %.

This research provides engineers, hydrologists, and others needing quick and simple runoff estimates with techniques that increase the models' accuracy. This should aid in the sizing of stormwater conveyances, determining mass contaminant loads, making land management decisions, and other actions requiring accurate runoff volumes and peak flows. Because these techniques allow more accurate estimations while maintaining the simplicity and cost effectiveness of the models, it is expected to primarily benefit those in smaller communities, suburban, and rural areas. However, anyone who uses the Rational and SCS Curve Number Methods should find the techniques applicable.

**Keywords:** Surface Runoff Estimations, Rational Method, SCS Curve Number Method, Representative Watershed, Paired Watershed, Data Collection and Accuracy, Calibration Techniques, A Priori Parameter Resolution, Rainfall and Runoff Data, Calibration Data from a "Similar Watershed", Peak Flow, Volumes, Cross-watershed Calibrations, Time of Concentration, Benefit-Cost Ratio

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## Chapter 1 Nomenclature

(chapter where term is introduced)

| | | |
|---|---|---|
| NPS | Nonpoint source | |

## Chapter 2 Nomenclature

(chapter where term is introduced)

| | | |
|---|---|---|
| $q$ | Peak flow rate | cfs |
| cfs | Cubic feet per second | $ft^3/s$ |
| $k$ | Fitting coefficient used in empirical formulas | |
| $x$ | Fitting exponent used in empirical formulas | |
| $C_f$ | Runoff factor used in early Rational Method | |
| $A$ | Area | acres |
| $I_{24}$ | 24-hour rainfall depth | inches |
| $C$ | Rational Method C coefficient | |
| $i$ | Rainfall intensity | inches/hour |
| $T_c$ | Time of concentration | minutes |
| SCS | Soil Conservation Service | |
| CN | Curve number | |
| $Q$ | Runoff depth | inches |
| $P$ | Rainfall depth for storm event | inches |
| $S$ | Maximum watershed retention of rainfall once runoff begins | inches |
| USGS | United States Geological Survey | |
| $q_2$ | 2-year return period peak flow rate | cfs |
| IA | Impervious area | acres |
| P2_24 | 2-year 24-hour design rainfall depth | inches |
| SWM | Stormwater Management Model | |
| HSPF | Hydraulic Simulation Program--Fortran | |
| HEC | Hydraulic Engineering Center | |

## Chapter 4 Nomenclature

(chapter where term is introduced)

| | | |
|---|---|---|
| HSG | Hydrologic soil group | |
| h | Stage height | feet |
| TFLI | Tennessee Fluid Level Indicator | |
| $L_o$ | Overland flow length | feet |
| $S_o$ | Mean slope of overland flow | feet / feet |
| $L_c$ | Channel flow length | feet |
| $S_c$ | Mean slope of channel flow | feet / feet |
| n | Manning's roughness coefficient | |
| $L_s$ | Sheet flow length | feet |
| $S_s$ | Mean slope of sheet flow | feet / feet |
| $P_w$ | Wetted perimeter | feet |
| r | Hydraulic radius | feet |
| v | Velocity | feet /second |
| l | Length | feet |
| AMC | Antecedent moisture condition | |
| NRCS | Natural Resource Conservation Service | |
| AE | Absolute error | cfs |
| AAE | Average absolute error | cfs |
| N | Number of events | |
| B-C | Benefit to cost ratio | cfs / $1,000 |

# CHAPTER 1

## INTRODUCTION

### THE IMPORTANCE OF ACCURATE HYDROLOGICAL INFORMATION

A glance through history shows many examples of once-glorious civilizations ending in ruin because of poor water-management decisions (Biswas, 1970). Today, because of our knowledge of hydrology, we are not doomed to repeat past mistakes. Nonetheless, long-term sustainability of our water resources requires accurate information regarding various components of the hydrologic cycle (Ward and Elliot, 1995).

Within the hydrologic cycle, there are a variety of components that can be measured or estimated at many different time scales. This leads to a potentially huge database of hydrologic information. To be useful, it is essential to relate the hydrologic analysis to the purpose of analysis or issue (Haan et al., 1993). Locally, government and community leaders require hydrologic data for wise decision making. Their concerns include:

- Nonpoint Source (NPS) pollution effects on water quality
- Land use and management decisions
- Flood plain / flood way delineation
- Streambank, habitat, and riparian zone reconstruction
- Stormwater design and management

Specific issues of concern in Knoxville, Tennessee include flooding along First Creek, rapid development and land use changes along Beaver Creek, cleanup and restoration in the Second Creek Watershed, and high pollutant loads in all creeks (Ganju, 1998). Managing these problems requires information about surface runoff.

## SURFACE RUNOFF

Runoff is defined as that surface flow resulting from precipitation or storm events (Haan et al., 1993), and it is a major factor in each of the concerns mentioned previously. For example, runoff is the major force driving NPS pollution, and contaminant mass loads are directly linked to runoff volumes. Flooding also illustrates the importance of surface runoff because extreme peak runoffs from precipitation events are the major cause of flooding. Solving these problems requires runoff volume and peak flow information. This information can be determined either by actual field measurements or by estimations. Since actual field measurements are expensive, engineers, hydrologists, and planners frequently use estimation techniques.

## FIELD MEASUREMENTS VS. ESTIMATIONS

As mentioned, two types of runoff information are needed—runoff volumes and peak discharge rates. This information can be determined either by actual field measurements or by estimations. To measure runoff, each area of interest would need rain gages, flow recorders, and personnel to collect and analyze the data. Depending on the measurement scale, field measurement usually results in an extensive and expensive monitoring network. In addition, field measurements only provide information on past events, and often it is necessary to predict future events

## ESTIMATION TECHNIQUE REQUIREMENTS

Ward (1995) surveyed engineers and hydrologists concerning runoff estimation methods. Ease of use was the primary reason they selected an estimation technique. Furthermore, Ward found that for an estimation technique to be used on a consistent and reliable basis, the method must be:

- Fairly inexpensive
- Easy to use
- Consistently accurate

However, most survey respondents reported that while the frequently used methods are easy to apply, they are also insufficiently accurate. In addition, respondents reported that some of the more accurate techniques are extremely complex.

Pilgrim and Condery (1993) state that there is a need for "simple and unambiguous methods" to predict runoff. They further state that many estimations on small watersheds are performed by people with little hydrologic experience and therefore simple, reproducible methods that are hard to misuse work best. In addition, Loague and Freeze (1985) report in a study of three types of estimations that the more complex methods were of similar or lower accuracy than were the simpler methods. This may be due to the "unmeasurable spatial variability of rainfall and soil hydrologic properties", which rendered the complex data intensive models ineffective (Loague and Freeze, 1985).

## COSTS OF INACCURATE ESTIMATES

The importance of accurate runoff estimates is often overlooked. When using runoff estimates to evaluate or solve problems, accurate estimates will save time, money, and other resources. Conversely, the costs of inaccurate estimates are large and of a diverse nature. Often these are hidden costs and are easily overlooked.

The following issues illustrate some of the problems related to poor runoff estimations. Although they have been separated for ease of discussion, they are interrelated. What affects one issue tends to affect others.

### Streambank, Habitat, and Riparian Zone Reconstruction

In this concern, two basic problems arise from poor runoff estimations. First, reconstruction projects fail to perform as intended. Specific failures include 1) allowing continued sedimentation to damage aquatic life, and 2) streambank collapse or creep that reduces riparian land, which subsequently undermines structures and roads. The U.S. Army Corps of Engineers estimated that streambank erosion alone cost $295 million (1985 dollars) a year in direct and indirect costs (L.R. Johnson Associates, 1992).

The second problem is over-design of hydraulic structures, which results in unnecessary straightening and channeling of streams, and/or larger structures than necessary. Linsley (1986) noted that actual costs of over-designing structures are difficult to obtain or estimate. However, he observed that roughly one fourth of the cost of highways pays for drainage. He further noted that highways and infrastructure spending was approximately $66 billion dollars per year (1986 dollars) and that even a small percentage savings would amount to a large sum.

Pilgrim (1989) also commented that over-designed structures in small and medium drainage projects may waste many public dollars. Since each structure on these projects is relatively inexpensive, only basic design estimations usually are made. Although the individual costs may be small, the total cost for all of the projects is substantial. Pilgrim (1989) reported the following annual expenditures in Australia (1988 Australian dollars):

| | |
|---|---|
| Rural Roads - waterway crossings | $240 million |
| Railways - waterway crossings | $30 million |
| Urban drainage | $180 million |
| Flood mitigation and stream improvement works | $30 million |
| Farm dams | $50 million |

These expenditures comprised about 0.25% of the Australian gross domestic product.

Besides financial costs, Linsley (1986) stated that over-designed projects were likely to have greater environmental impacts. He believed that many environmental objections to projects were because of unnecessary over-design.

### Floodplain / Floodway Delineation

Inaccurate runoff estimates can cause regulators to incorrectly delineate flood zones. This may result in either unclassified flood-prone land, or land incorrectly classified as flood prone. Either situation is unsound.

Development on unclassified land within the flood zone can result in obvious problems, such as flood damage to buildings, streets, and utilities. A study prepared for the Federal Emergency Management Agency estimated that there are about 22,000 flood-prone communities in the U.S.. In Tennessee alone, there are at least 160,000 households worth an estimated $5.6 billion (1987 dollars) located in over 2,000 square miles of flood prone land (L.R Johnson and Assoc., 1992).

These developments on improperly delineated floodplains can also cause unanticipated and higher flood levels downstream. Other costs which may result from poor peak flow estimations include increased insurance rates, lost business and tax revenues, and the inconvenience of flooded streets and utility outages (L.R. Johnson and Assoc., 1992)

## NPS Effects on Water Quality

NPS pollution is pollution that moves into the waterways from dispersed or diffuse sources (USEPA, 1992). Typical urban contaminants and sources include:

- Various hydrocarbons from parking lots and roads
- Nutrients and pesticides from lawns and gardens
- Bacteria and viruses from animal feces or leaking sewer mains
- Sedimentation from building sites, lawns, or uncovered areas

The common factor of the above list is that the contaminants are widely dispersed, often at low concentrations, over the entire landscape. Storm event runoff is the most common mechanism transporting the contaminants from the land into waterways. Identifying and characterizing the contaminants and quantities entering the waterways is the first step in controlling NPS pollution. Contaminant quantities or loads can be calculated using the general formula (Huber, 1992):

$$\text{pollutant mass (m)} = \text{pollutant concentration (m/v)} \times \text{runoff volume (v)}$$

Concentration is measured by collecting runoff samples and using standard laboratory techniques. Then, if the runoff volume is known, a contaminant mass load can be determined. Both estimated and actual volumes can be used to assess pollutant loads. Since widespread measurement of runoff volumes is expensive, volume estimation methods are often used (Smoley, 1993). Accurate estimations are essential in that poor runoff estimations can lead to inaccurate pollutant load calculations, which in turn frustrates practical and workable solutions for NPS problems. Problems associated with NPS pollution include loss of habitat and aquatic life, channel sedimentation, and streams posted as bacterial hazards. It is not unusual for the stream to become a community eyesore or hazard, rather than an asset.

Although usually considered a point source, combined sewer outflows (CSO) result from large runoff volumes in combined stormwater-sewer systems. During intense or prolonged storm events, these combined systems overflow directly into waterways. In a similar manner, the wastewater plant can receive large inputs, which are then shunted into receiving waters with minimum treatment. While poor runoff estimates don't actually cause pollution, they may very well lead to insufficient or inefficient containment or reduction measures (Smoley, 1993).

A recent concern in many communities is development of Total Maximum Daily Loads, or TMDLs, a concept which is stipulated in The Clean Water Act of 1987. According to the EPA (1992), ***"a TMDL calculates allowable loadings from the contributing point and nonpoint sources to a given waterbody and provides the quantitative basis for pollution reduction necessary to meet water quality standards."*** This means that mass loads, not just concentrations, must be measured or calculated. Therefore, knowledge of flow volumes is necessary. Lack of compliance with The Clean Water Act has led to a number of lawsuits against local, state, and federal governments.

### Land Use and Management Decisions

Regulators can make all of the previous problems worse with poor land-use decisions. In addition, inaccurate runoff projections lead to poor projections about the effects of growth, which can lead to building the wrong thing in the wrong place.

## SUMMARY

In the subjects discussed above, inaccurate runoff estimates contributed to wasted time and money, reduced social value of creeks and rivers, and added inconvenience and headaches for everyone. While accurate runoff estimates won't correct all the problems, good numbers will certainly help reduce these problems.

# CHAPTER 2

## BACKGROUND

### EARLY ESTIMATION METHODS

Trying to estimate or predict runoff is not simply a modern endeavor. For years, scientists and engineers have estimated runoff with limited success. Pierre Parrault (1608-1680) estimated that 1/6 of the rainfall appeared as stream flow (Biswas, 1970). The method by which Parrault calculated the runoff was simple and theoretically correct. Parrault first estimated the rainfall in the Seine River Watershed above Paris on an annualized basis. He then estimated the river flow volume for the same period. By dividing rainfall by stream flow, Parrault concluded that 1/6 of the rainfall equaled the stream flow volume. He classified this amount as runoff and the remainder as waste or loss. Terms such as evapotranspiration, baseflow, and infiltration were as yet unknown. Amazingly, three hundred years, later Parrault's technique is still used in some forms. Sheridan (1997) notes the simplest method of estimating runoff is to use a coefficient (0.0-1.0) to determine the fraction of rainfall that becomes runoff. He further states that using annual rainfall and annual stream flow is a fair method to determine the coefficient.

Certainly, Parrualt's estimate that 1/6 of rainfall becomes runoff is an adequate first approximation. This technique, however, is not usable in this study for several reasons. First, the estimations are made over a long time-period, and not on a storm-event basis. Secondly, in Parrault's method, no distinction is made between runoff and baseflow. This distinction is critical for pollutant load calculations and floodplain delineation. Therefore, such a generalized method is insufficient for our needs.

### EARLY EMPIRICAL FORMULAS

These methods use drainage area along with a coefficient to estimate peak flow. The coefficients are not related to any basin characteristics, but instead are based on observed flows (Wigham, 1970). The formula took several forms and a general example is: $q = k A^x$ Where: $q$ = peak flow rate (cfs)

$k$ = coefficient

$x$ = exponent

$A$ = area (acres)

The coefficient and exponent are simply fitting parameters. The major advantage of these formulas is that they are simple to apply—only drainage area is needed. Wigham shows 35 different empirical equations that were developed for various basins. Usually these formulas were used within climatic or physiographic regions (Wigham, 1970).

Horner and Jens (1942) state the main reason the empirical methods were used was due to the lack of adequate hydrological data such as rainfall information. Today, the methods are not considered adequate for general use because other engineering methods have supplanted them (Jens and McPherson, 1964).

### RATIONAL METHOD

According to Biswas (1970), the first attempt to predict or estimate peak discharge on a rational basis was carried out during the years 1842-1847 by a group of Irish engineers. The assumption was that rainfall infiltrated, evaporated or became stream flow. The engineers reasoned that if they could estimate amounts on an annual basis, the same percentages would generally hold true on an event basis. The formula appeared to be the first relating catchment characteristics to runoff. In its original form, the method was:

$q = 2.52\ C_f\ I_{24}\ A$ Where: $q$ = peak flow (cfs)

$C_f$ = runoff factor

$I_{24}$ = 24-hour rainfall depth (in)

$A$ = area (acres)

In 1852, T. J. Mulvaney improved on this method by theorizing that a maximum peak flow rate occurs when runoff from all areas of the catchment contributes to stream flow (Biswas, 1970). Thus, Mulvaney incorporated time of concentration ($T_c$) into the formula. This method is very similar to the Rational Method used today. The current Rational Method relates rainfall to peak flow on a proportional basis. The equation is:

$q = C\,i\,A$ Where: $q$ = peak flow (cfs)

$C$ = coefficient representing watershed characteristics

$i$ = rain intensity (in/hr) for the $T_c$

$A$ = watershed area (acres)

The formula's C coefficient accounts for the watershed variables such as land use, antecedent moisture conditions, soil or surface type, vegetation, and geology. More specifically, this single coefficient reflects rainfall interception, surface storage, surface infiltration, and evapotranspiration (Haan et al., 1993). Typical values of the C coefficient are shown in Tables 2.1 and 2.2. Other tables may have different parameters or values. Some common controlling parameters include land use, slope steepness, and soil hydrologic condition.

Sometimes, more than one C coefficient is chosen for a watershed (Haan et al, 1993). In this case, the area is divided into supposedly homogenous units. Each of these units is then assigned a C coefficient. The individual C coefficients are weighted by area and are added to determine the composite C. An area-weighted composite C coefficient is thus represented by: $C = A_1/A_T * C_1 + A_2/A_T * C_2 + \ldots + A_i/A_T * C_i$

While the Rational Method is over 100 years old, it is still frequently used in determining peak flows. One researcher states that billions of dollars have been spent building structures designed using the Rational Method (Shaake et al.,1967). According to Haan et al. (1993), the popularity of the method continues primarily because it is simple, entrenched, and because it receives much text coverage. Furthermore, Haan states that there is a lack of a comparable alternative.

**Table 2.1. C coefficients for different surface characteristics.**

| Surface Characteristic | C Coefficient |
|---|---|
| Pavement | 0.70 to 0.95 |
| Roofs | 0.75 to 0.95 |
| Lawns, Sandy Soil | |
| Flat (2% slope) | .05 to 0.10 |
| Average (2 to 7 % slope) | 0.10 to 0.15 |
| Steep (>7% slope) | 0.15 to 0.20 |
| Lawns, Clay Soil | |
| Flat (2% slope) | 0.13 to 0.17 |
| Average (2 to 7 % slope) | 0.18 to 0.22 |
| Steep (>7% slope) | 0.25 to 0.35 |

(adapted from American Soc. of Civil Engrs. and Water Environ. Fed., 1992)

**Table 2.2. C coefficients for various land uses.**

| Land Use | C Coefficient |
|---|---|
| Business | |
| Downtown | 0.70 to 0.95 |
| Neighborhood | 0.50 to 0.70 |
| Residential | |
| Single Family | 0.30 to 0.50 |
| Multi-units, detached | 0.40 to 0.60 |
| Multi-units, attached | 0.60 to 0.75 |
| Suburban | 0.25 to 0.40 |
| Apartments | 0.50 to 0.7 |
| Industrial | |
| Light | 0.50 to 0.80 |
| Heavy | 0.60 to 0.90 |
| Parks | 0.10 to 0.25 |
| Unimproved | 0.10 to 0.30 |

(adapted from American Soc. of Civil Engrs. and Water Environ. Fed., 1992)

## Limitations of Rational Method

Although the Rational Method is widely used, certain underlying assumptions limit the usability of the method. Generally, errors increase as actual conditions vary from the simplifying assumptions on which the method is based (Haan et al., 1993). Chow (1964) outlines the following assumptions inherent in the Rational Method:

1. The runoff rate is at maximum when the duration of an intense rainfall equals, or exceeds, the $T_c$
2. The maximum runoff rate is a straight-line fraction of rainfall intensity
3. The peak discharge frequency is the same as rainfall intensity at a given $T_c$. For example, a five-year 30-minute intensity yields a five-year peak discharge for a watershed where the $T_c = 30$ minutes.
4. The coefficient of runoff is the same for all storms

Investigators report that these assumptions rarely are valid for real storm events (Ben-Zvi, 1989; Pilgrim and Condery, 1993; Schaake et al. 1967). These and other limitations of the method, such as difficulty in assigning a proper C coefficient, assigning an true $T_c$ value, and improper use of the method beyond peak flow estimates are detailed in the following section. These limitations combine to make estimation errors common, and result in a tendency to overestimate peak flows by as much as 200% to 300% (Pilgrim & Condery, 1993).

### C Coefficient

Generally, when using the Rational Method, the C coefficient is assumed to be constant over time. However, studies show that the parameters making up the C coefficient can vary with the storm event, runoff event, or antecedent moisture (Chow, 1964; Haan et al., 1993; McPherson, 1969). For example, an extended dry spell may cause the soil surface to seal, causing less water to infiltrate and more to run off. In the following week or month, the runoff for a given location could change dramatically.

Besides temporal variations, the factors influencing the C coefficient such as soil properties vary spatially (Loague & Freeze, 1985). These variations can be significant even when the area of interest is divided into calculation units that appear homogenous.

Furthermore, the Rational Method presupposes a linear relationship between the rainfall rate and the peak flow rate, and this assumption appears contrary to the evidence (Gregory & Arnold, 1932; Schaake et al., 1967; Ben-Zvi, 1989;).

### Time of Concentration

The standard definition of the Tc can be summarized as the time it takes for rain falling on the most hydrologically-remote point on the watershed to reach the discharge point (Chow, 1964). This means that for a storm event lasting this length of time the entire watershed would contribute flow to the discharge or measurement point. Since the entire watershed would be contributing runoff, stage height at the discharge point should be at a peak. While conceptually this makes sense, field observations indicate that the $T_c$ is not a simple number to find.

Researchers have used many different techniques to determine the "true" $T_c$. Chow (1964) noted that C. E. Ramser assigned the time necessary for the observed flow to rise from the minimum to the maximum stage height as that time.

Horner and Flynt (1936) used observed lag time to approximate the $T_c$. Lag time was determined by measuring the time difference between the salient features of the rainfall hyetograph and the runoff hydrograph. Their investigation found large variations in lag times between events on the same watershed. In addition, Horner and Flynt measured the period from the center of mass of the entire rain event to the center of mass of the runoff as another indicator of the $T_c$.

Other investigators reported using center of mass techniques to determine the true $T_c$. McCuen et al. (1984) stated that the $T_c$ is that period from the center of mass of the excess rainfall to the point of inflection on the hydrograph's recession limb. Yet, in a study to evaluate the accuracy of different $T_c$ estimation formulas, the investigators compared the formula results to a "real" $T_c$ that was set by calculating times of travel along the flow-path. This was done by identifying the slopes, roughness, hydraulic radii, and lengths along the path from the most hydrologically remote point and then using velocity equations to calculate the time of travel. When compared to the calculated time of travel, none of the $T_c$ formulas produced very accurate estimates, and no two formulas produced results consistent with one another.

In a study of peak discharge rates, Hotchkiss and McCallum (1995) designated the period from the end of the excess rainfall to the inflection point on the recession limb as the "true" $T_c$. They noted that the periods varied widely between storm events.

Pilgrim and Cordery (1993) noted that there are typically two inflection points the recession curve and that the point used by researchers appeared to be a matter of preference. In a different report, Pilgrim (1976) attempted to measure the $T_c$ using radioactive tracers, but could not determine a single time for a given watershed.

Apparently, there is no “true” $T_c$. Overton and Meadows (1975) referred to the time period as the “alleged” $T_c$. Schakke et. al. (1967) stated that there was no known way to measure this time either in the field or with rainfall and runoff records. Unfortunately, there does not appear to be any standard method of estimating $T_c$ that works in a wide variety of circumstances.

### Improper Use of Rational Method

Although the Rational Method has many limitations, it continues to be used because of its simplicity and ease of use. Even though this is a method for estimating peak flows, the C coefficients are often incorrectly used to estimate runoff volumes and even to derive hydrographs (Ward, 1995).

## SCS CURVE NUMBER METHOD

While somewhat more complex than the Rational Method, the SCS Curve Number (SCS CN) Method is commonly used to estimate runoff volume. The SCS CN Method estimates total runoff depth from rainfall depth within a watershed. The equation is:

$Q = (P - 0.2S)^2 / (P + 0.8S)$ where $P > 0.2S$ and

$Q$ = Runoff depth (in)

$P$ = Rainfall depth (in)

S = Maximum watershed retention of rainfall once runoff begins (in)

S is mainly a function of infiltration that occurs after runoff begins and is calculated by the formula: $S = (1000 / CN) - 10$ where CN = Curve number

The CN indicates runoff potential from an area, and the value accounts for variables such as land use, antecedent moisture conditions, soil types and vegetative cover (Mockus, 1972a). This number is between zero and 100, with higher values indicating increased runoff potential; e.g., an impervious surface would have a CN of 100. The first curve numbers were derived using runoff records from field plots scattered across the U.S. (Haan et al., 1993). Those records were correlated to land use and hydrologic soil group. Researchers then plotted the data and extended the curves as needed (Mockus, 1972a).

The equation also accounts for the amount of rainfall that is intercepted, stored, or infiltrated prior to any runoff. This amount is known as the initial abstraction and was empirically derived from small experimental watersheds (Mockus, 1972a). In the SCS method, the initial abstraction is set to 0.2S. Thus, rainfall must exceed this amount before any runoff is estimated

To choose a curve number, the first step is to evaluate the land use, soil hydrologic group, and antecedent moisture condition. Then, using charts and tables found in hydrology references, the CN is tabulated. Trained users often employ an area-weighted CN for better accuracy in accounting for spatial variability (Huggins et al., 1982).

### Advantages of SCS CN Method

There are several advantages of the method. One is that the curve number incorporates a slight nonlinear increase in runoff as rainfall depth increases. Runoff approaches rainfall as the event continues for an indefinite time (Huggins and Burney, 1982), which matches actual field conditions. A second advantage is that the initial abstraction attempts to approximate the hydrological characteristics of a watershed. A third benefit is that the method accounts for antecedent moisture conditions along with the physical characteristics of the watershed.

### Limitations of SCS CN Method

The SCS method has limitations. One is the lack of accounting for rainfall intensity and its corresponding effect on the runoff volume (Pilgrim et al., 1993). According to Mockus (1972b), the difficulty in choosing the proper CN is a major source of error when estimating runoff from ungaged watersheds. Although the exact value was not stated, correlation of estimated runoff to actual runoff was judged to be moderately low. Even so, Mockus (1972a) states that using similar, gaged watersheds might be useful as "guides in judgement" when selecting a CN for an ungaged watershed.

Some investigators have questioned the accuracy of the method. For example, in a study evaluating the Curve Number method, Wood and Blackburn (1984) reported differences between observed and estimated values to be greater than 50% in 67% of the events.

## USGS Regional Equations

The U.S.G.S. Regional Regression Equations are the principle statistical methods that are used to estimate runoff peak flows and volumes from ungaged watersheds. The equations can be used to transfer the runoff characteristics from gaged to ungaged sites (Jennings et al., 1993). The approach is to develop a regression equation relating flow to watershed characteristics for gaged watersheds that are within a geographic region (Helsel and Hirsch, 1992). First, an analysis is made of basin characteristics to determine which variables have a significant impact on runoff (Choquette, 1988). These significant variables are further analyzed to remove any multicollinearity effects. The remaining variables are then fit to existing regional flow records using least squares multiple regression techniques (Choquette, 1988). Jennings et al. (1993) gives the following equation, which is of a representative form.

$Q_2 = 1.76 * Ac^{0.74} * IA^{0.43} * P2\_24^{3.01}$ Where Ac = contributing drainage area, (acres)

IA = Impervious area (acres)

P2_24 = 2-year-24 hour rainfall (in)

$Q_2$ = 2-year peak discharge (cfs)

To apply the equation, the needed characteristics are measured on the ungaged watershed. Those characteristics are inserted into the regression equation and a result is produced.

### Advantages of Regional Regression Equations

The primary advantage of the regression equations is that they are easy to use (Jennings et al. 1993). In addition, equations have been developed for many different return periods, such as peak flows for 5-year, 10-year, 25-year, 50-year, 100-year, and 500-year return periods. Other similar equations are used to estimate runoff volumes, contaminant concentrations and contaminant mass loads (Driver and Tasker, 1990).

### Limitations of Regional Regression Equations

Generally, the greatest limitation to accurately using the regional regression techniques is based on how well the ungaged stream and basin matches those used in the regression equations (Jenning et al. 1993). If the streams used in the equation have similar rainfall-runoff regimes to the watershed in question, then the result may be acceptable. If the stream in question falls outside of the characterization range of the streams used in forming the equation, the error will increase (Driver and Tasker, 1990).

Another limitation is that specific land use parameters are missing from the equations. Although some equations attempt to account for basin development, the parameter is general. For example, if a certain land parcel was changed from agricultural to urban use, the equation might not pick up any effect. Many equations do not have any watershed parameters except for drainage area (Driver and Tasker, 1990).

A third limitation of regression equations is that long-term records are rarely available for extreme events. To get estimations for the extreme events, the existing records are statistically extrapolated. The equations predicting 50-year, 100-year and 500-year events are probably not very accurate (Haan et al., 1993).

### Other Statistical Methods

Other statistical methods such as simple linear regression between rainfall and runoff are also used on watersheds (Sheridan, 1997). Generally, the y intercept of runoff is negative to allow for some amount of rainfall to occur before any runoff occurs. Simple linear regression assumes that runoff is directly related to, and some constant fraction of, rainfall (Sheridan, 1997). However, like in the Rational Method, this assumption is often incorrect and limits the utility of the method (Chow, 1964). Other promising statistical techniques include multivariate analysis, time-series analysis, and stochastic methods. Nonetheless, using these methods requires complex skills and they cannot be classified as "easy to use."

## OTHER METHODS

### Full-scale Hydrological Modeling

Other types of models attempt to estimate runoff by using parameters that describe the physical attributes of the watershed (Grayson et al., 1992). While the number and type of parameters vary between models, some common parameters include drainage area, land use, soil type, vegetation, evapotranspiration, and topography (WEF/ASCE, 1992). The parameters are then mathematically manipulated to simulate the hydrologic processes that occur within the watershed such as infiltration, storage, interception, evapotranspiration and surface runoff.

There are many full-scale models, including SWMM, HSPF, and HEC-1 (WEF/ASCE, 1992). Often, these types of models are quite complicated and require extensive amounts of calibration data. For example, it took Fontaine (1995) 4 full months to use 3 years of observed data in calibrating HSPF to estimate yearly peak flows on the Kickapoo River in Wisconsin. Even so, the model overestimated peak flows by 12—68%. These data and time requirements are well beyond the resources of most engineers, hydrologists, and planners.

## Representative Watershed Data Transfer

During the 1960s and 70s, researcher advocated the use of representative watersheds to estimate or predict hydrological parameters. The theory was that a particular basin could be representative of a predefined geologic, geographic, or climatic region (Liebscher, 1986). Those basins could then be monitored and the results would transfer well to other watersheds.

Although this method would be simple to use and relatively inexpensive, the approach was later found to give inaccurate results because of the huge variation between watersheds. Studies indicate that factors other than geology, geography, and climate influence runoff (Liebscher, 1986). Since these factors may be unknown, the question is raised whether the target watershed would respond the same as the similar, representative watershed. The inability to transfer data directly to other watersheds led to the limited use of the method.

## Paired Watershed Approach

Although the representative watershed concept is currently out of fashion, the paired watershed approach is similar and has been successfully implemented for over forty years (Clausen et al., 1996). The paired watershed approach uses two watersheds near one another with similar characteristics. Both watersheds are instrumented and monitored. The data records from each watershed are then correlated and fitted with regression equations. After correlation, a treatment is applied to one watershed and no treatment to the other. The treatment effects are determined by comparing the observed discharge in the treated watershed to the estimated discharge calculated from the regression model combined with the measurements on the untreated watershed. This approach has proven useful when evaluating water quality, quantity, and best management practices (Clausen et al., 1996; Cooke et al., 1995).

The paired approach requires monitoring and correlating two watersheds for an extended time, frequently at least one year (Clausen and Spooner, 1993). This extensive monitoring makes the method too expensive and complex for local needs. However, studies indicate that correlations between similar watersheds do exist and that these correlations can be exploited (Clausen and Spooner, 1993).

## SUMMARY

Runoff volume and peak flow information is needed for assessing stormwater related problems and implementing solutions in Knoxville. Some of these problems include flooding, NPS pollution, land use management, and stream restoration.

Acquiring the needed data requires either direct measurement or accurate estimations. Actual field measurement is expensive, therefore, use of estimation techniques is needed. Furthermore, field measurements must be extrapolated for extreme rainfall because runoff records are rarely long enough. Therefore, one can never avoid using estimation techniques for design storms.

There are many runoff estimation and prediction methods, ranging from the relatively simple to the highly complex. Our need is for a relatively uncomplicated and efficient method to use in small watersheds. While there are quick and easy techniques, none are consistently accurate. In essence, there are four means of acquiring the needed information:

- Simple models such as the Rational, SCS CN, or Regional Methods
- Full-scale monitoring
- Full-scale modeling
- Transfer of data from a representative basin

Unfortunately, each has limitations making it unsuitable for our needs. Table 2.3 summarizes the main advantages and disadvantages of the overall methods. In the table, each of the methods is not a distinct category, but instead is better viewed as a continuum. This continuum ranges from the simplest and least accurate models to the most expensive, complex, and highly accurate monitoring scheme.

**Table 2.3. Matrix showing the advantages and disadvantages of the different methods used to acquire hydrological data.**

| Method | Advantages | Disadvantages |
|---|---|---|
| **Simple Estimation Methods** | Simple to use<br><br>Low data requirements | Considered inaccurate<br><br>Low reproducibility between different users |
| **Representative Basin Transfer** | Simple to use<br><br>Accuracy varies with similarity of basins | May be inaccurate<br><br>More data required than for simple models |
| **Full-scale Modeling** | Assess land use changes and other manipulations<br><br>Considered accurate if sufficient calibration data used | Expensive and complex<br><br>Large data requirements |
| **Full-scale Monitoring** | Accurate for measured storms | Expensive and complex<br><br>May not extrapolate well to low probability design storms |

# CHAPTER 3

## THE PROBLEM STATEMENT AND RESEARCH OBJECTIVES

### THE PROBLEM STATEMENT

The simple, inexpensive, and commonly used runoff estimation models are inherently inaccurate. Therefore, this researcher wanted to improve the accuracy of those methods without overly complicating them. In addition, I thought it was necessary to keep the methods inexpensive. The question, therefore, was how to most efficiently improve the accuracy of simple models like the Rational and SCS Curve Number Methods.

### DATA COLLECTION AND ACCURACY

This led me to examine the problem while considering the amount of data collection involved in the methods. From this perspective, accuracy appeared related to the quantity of data used in the model. For example, I assumed two extreme cases--a very simple estimation method and a full-scale monitoring effort. The low-accuracy, simple method used minimal data, such as watershed area and an empirical fitting parameter. Conversely, a highly accurate, full-scale monitoring scheme required an extensive database. During the process, a relationship between data collection and cost was also revealed. Table 3.1 denotes the assumed relationship between accuracy, data collection, and cost for the four general methods. It is important to note that the accuracy, data collection, and cost factors in the table were not in distinct divisions, but were thought of as a continuous series from least to greatest. Figure 3.1 illustrates this continuum.

### THE EFFECT OF CALIBRATION DATA ON THE ACCURACY OF A MODEL

As previously noted in the text, Fontaine (1995) spent considerable time and effort calibrating the HSPF model with 3 years of data to get marginal predictions. While Fontaine noted that more calibration data might have resulted in better predictions, he

**Table 3.1 Accuracy-Data Collection-Cost Factors**

| Method | Accuracy | Data Collection | Cost |
|---|---|---|---|
| **Simple Models** | Typically low | Little | Low |
| **Representative Basin Transfer** | Fairly low but depends on the representative basins | Some | Fairly low to moderate |
| **Full-scale Modeling** | Low to high, depending on the amount, resolution, and accuracy of calibration data | Can be extensive | Moderate to fairly high |
| **Full-scale Monitoring** | High for measured events, varies for prediction of probable events | Extensive | High |

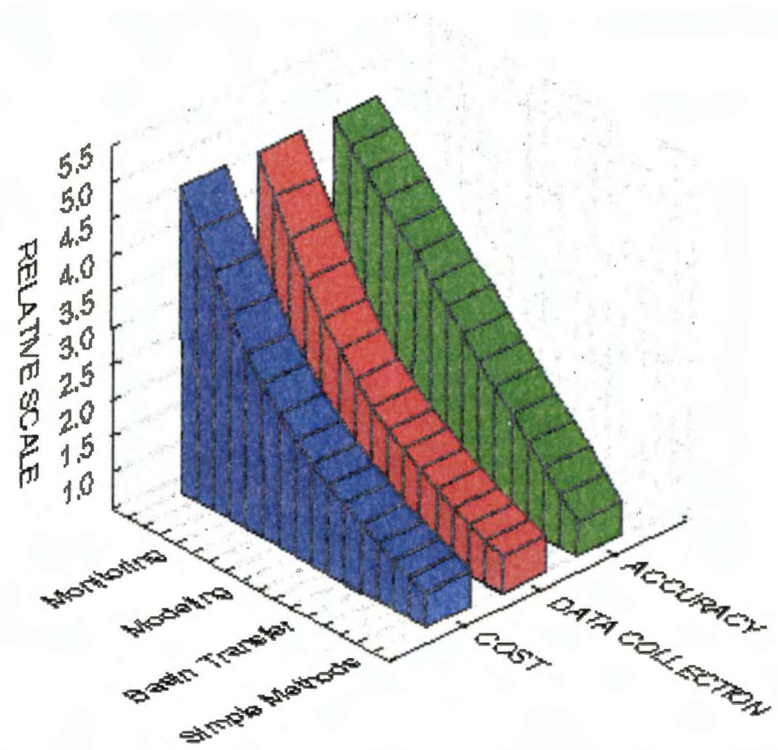



**Figure 3.1 Illustrative relationship between accuracy, data collection and cost.**

also stated that the cost would have been higher also. Apparently, at some point further data collection becomes pointless because of increasing costs and lessening benefits.

A number of investigators noted that improvements to runoff estimations are possible with even small amounts of data. Mockus (1972b) reported that actual flow records could serve as guides in choosing a curve number on an ungaged watershed. Haan et al. (1993) observed that even a short stream flow record could be a "great aid" when checking calibrations and procedures. Furthermore, using a short flow record is relatively inexpensive and could easily pay for itself through reduced costs and improvement to the drainage system (Haan et al., 1993).

In a similar vein, Pilgrim and Condery (1992) stated that estimation accuracy is only partly aided by using parameters based on physical watershed characteristics. They further argue that some observed data should be used to calibrate or check the model, even if the records are only from a nearby watershed. However, one notable limitation when using calibration data on multiple parameters is that interactions of the parameters may occur. This may lead to different parameter values giving similar results for the observed data, but very different results for unobserved predicted events (Pilgrim and Condery, 1992).

## OBJECTIVE

The question that prompted the research project was: can the accuracy of simple models be improved while keeping the methods inexpensive and easy to use? The background research indicated that higher-resolution parameters and increased data collection might increase the accuracy of the simple models. This led to the hypothesis that increasing the resolution of the model parameters, and/or calibrating the models with measured hydrologic data would increase the accuracy of the estimations. The primary objectives of the research were:

1. Optimize the parameter resolutions and the method of selecting the parameters.
2. Develop calibration techniques to improve the accuracy of the Rational and SCS CN Methods.
3. Identify the most efficient techniques.

# CHAPTER 4

## APPROACH

To test the hypothesis and fulfill the research objectives, the investigation took a three-pronged approach. The first approach was to evaluate a priori parameter selection techniques for the Rational and SCS CN Methods; techniques which varied from the quick and simple to the time-consuming and complex. The second approach used direct measurements of flow at the target watershed to improve the runoff estimates. Different quantities of data were employed and evaluated in this procedure. The third approach used data from a similar, nearby watershed to assist in the calibration of the target watershed. Again, different quantities of data were used and evaluated.

### TEST WATERSHEDS

Attacking the problem from those three angles required collection of rainfall and runoff data and assessment of the physical characteristics from two similar watersheds. Two subwatersheds located in the upper reaches of Second Creek in Knoxville, TN were selected for the project. One watershed, the Sanford Watershed, had been used in a previous study, and rainfall and runoff data collection equipment were in place. The second area, the Church Watershed, was selected because of its visible similarity to the Sanford Watershed.

#### Watershed Descriptions

Both watersheds are located in the headwaters of Second Creek, with the Church watershed being located the in northwestern corner and the Sanford watershed in the most northeast corner (see Figure 4.1). The Church watershed encompasses 107 acres (43 ha) and the Sanford watershed is 179 acres (72 ha). The watersheds were delimited by drawing in the probable watershed divides on the Fountain City 7.5-minute (1:24,000 scale) quadrangle obtained from the USGS (1978). The boundaries were then field checked and adjustments were made as necessary. As the research progressed, a drainage map (1:6,000 scale) was obtained from the City of Knoxville Records and Mapping

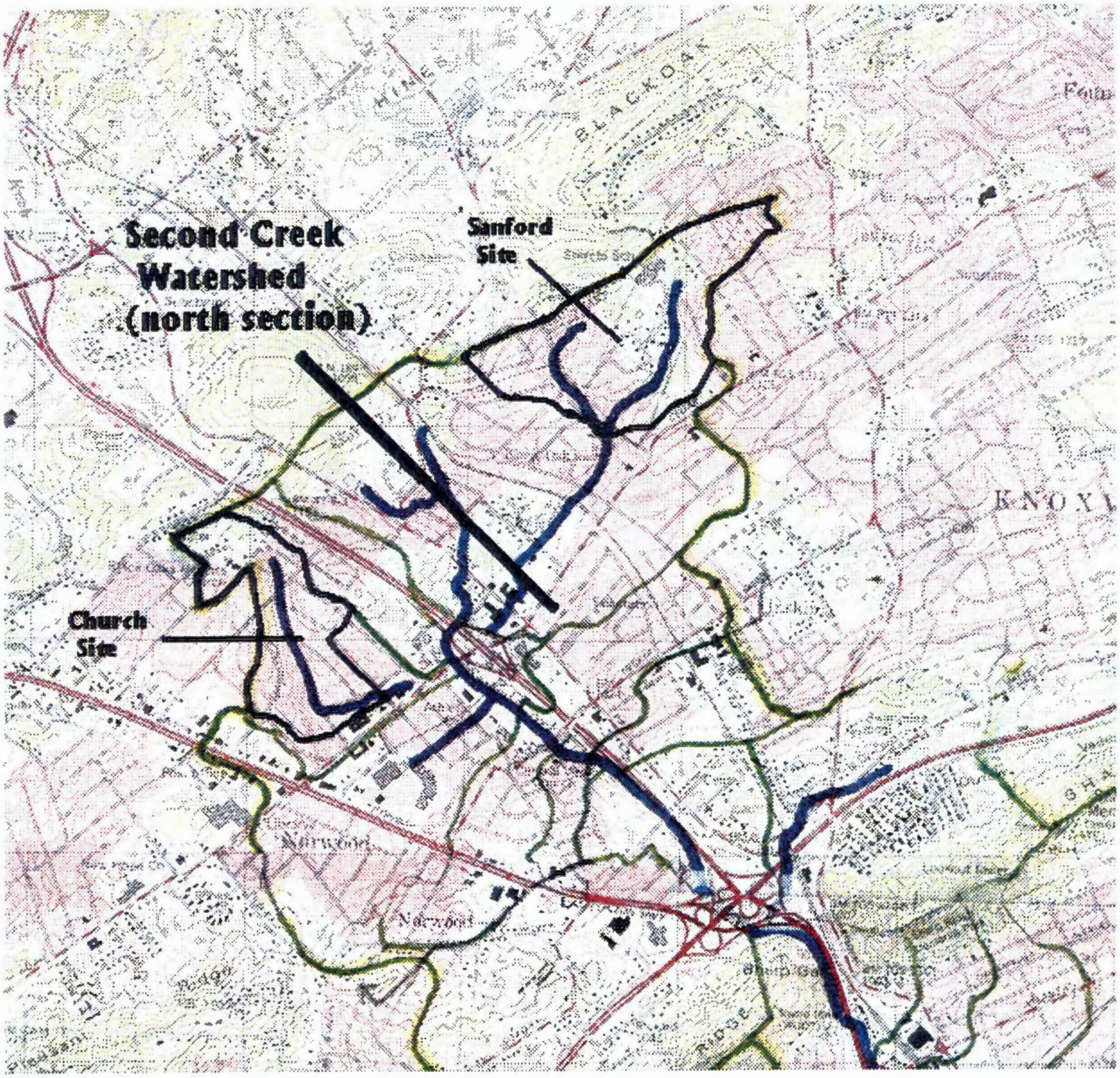



**Figure 4.1. Location of the Second Creek research watersheds. Source: USGS Fountain City 7.5-minute Quadrangle (1:24,000 scale).**

Department. The watershed boundaries from the drainage map agreed with those delineated earlier.

Both watersheds are positioned on the slopes of Black Oak Ridge. According to USGS maps, the primary geologic formation along the ridge is a Longview dolomite, and the lower areas are usually Nolichuchky shale and the areas may exhibit some karst features. The ridge runs from southeast to northwest and has moderate slopes (10-20%) near the top, which become less steep towards the bottom of the slope. Soils along the upper ridges tend to be a Fullerton or Clarksville, and the lower, flatter areas tend to be Decatur or Dewey. Generally, the soils are class B hydrologic soil group (HSG).

Weather patterns tend to follow the ridge, which allows both areas to receive similar precipitation. The average annual precipitation is 48 inches, with the majority occurring from midwinter to midsummer. Fall tends to be the driest season although the maximum recorded 24-hour storm occurred in September, 1944 and had a depth of 5.08 inches.

According to previous studies in the area, land use and land cover are similar at both locations (Kung, 1980). Each watershed has similar land use--mostly suburban or rural-residential--and both have small areas of multifamily housing. Typical land cover is grass lawns with many large deciduous trees.

The stormwater hydrology of the area is complex. Stormwater drainage includes overland flow, concrete culverts, earth channels, and underground pipes. There may be sinkholes and secondary solution channels that drain surface water in the karst areas. Other surface water may drain through fractures and between bedding planes of the shale formations. Both watersheds have only ephemeral storm runoff, which eliminated the need to adjust for base flow and removed an added source of uncertainty during the study.

Figure 4.2 shows representative runoff hydrographs and rainfall hyetographs for the two watersheds. The hydrographs illustrates relatively similar watershed responses to rainfall events. The rising limb of the Sanford hydrograph shows a time to peak of approximately one hour and the recession limb was about one and half hour. The time to peak on the Church hydrograph was approximately one and a half hours, but this included about twenty minutes where the rainfall and runoff where at an equilibrium The recession limb was less steep on the Church hydrograph, and may indicate more responsiveness to the rainfall that occurred after the peak flow. Although the peak flow rates where the same, the rainfall depth and intensity was greater on the Church watershed (See Table 4.1 Storm 29 and Table 4.2 Storm 37). Both hydrographs show no long-term runoff.

## Instrumentation

### Sanford Watershed

The Sanford Watershed discharge was located near the junction of Haynes-Sterchi and Sanford Roads. During a previous study, the site had been equipped with rain and flow measurement instruments. The watershed discharge was measured using a 120° v-notch sharp-crested weir (Figure 4.3), along with a Tennessee Fluid Level Indicator (TFLI) to measure stage height (Yoder et al., 1999). The following empirical formula related stage height to discharge:

$$q = 4.33\ h^{2.5}$$

Where: $q$ = discharge (cfs)
$h$ = stage height (feet)

Data were recorded on a Campbell Scientific CR-10 datalogger. The data were initially recorded at five-minute intervals. In February 1998, an attempt was made to change the recording interval to one-minute increments, but several problems surfaced. One problem involved an automatic water quality sampler that was installed at the site. The sampler was controlled by the CR-10 and the data logger program was changed several times during this period. In June 1998, the sampler was disconnected and the data-recording interval was successfully changed to one-minute increments.

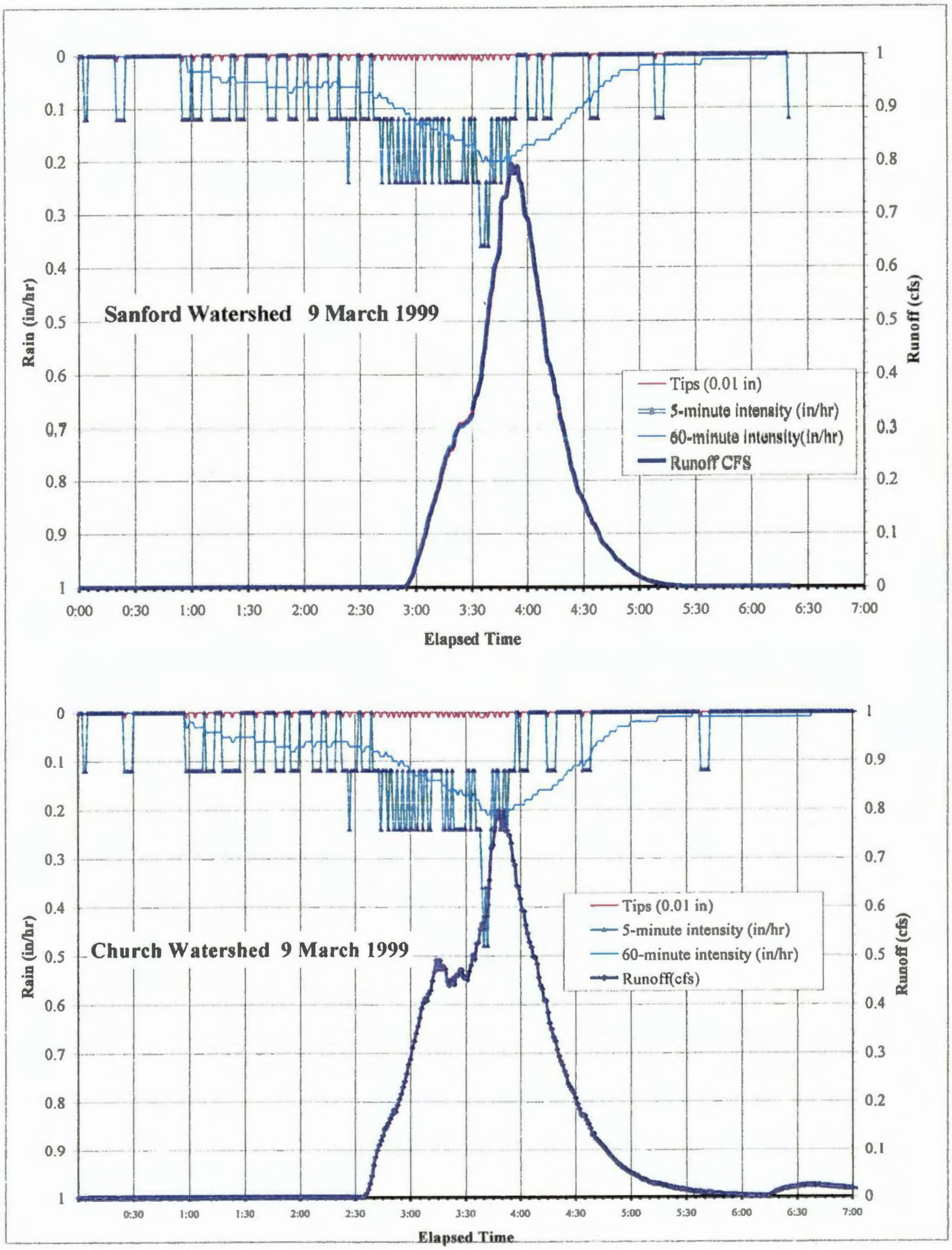



**Figure 4.2. Representative hydrographs and hyetographs for the two watersheds. Total rainfall depth was 0.43 inches on the Sanford watershed and 0.56 inches on the Church watershed.**

**Table 4.1. Summary of the rainfall and runoff data collected on the Sanford watershed.**

| | | | AMC (prior days) | | | Rainfall | | | Intensity for these times (min) | | | | | | | | Runoff | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 2 | 5 | 8 | Duration | Depth | Intensity | 5 | 20 | 25 | 30 | 40 | 45 | 50 | 60 | Peak | Volume |
| Storm | Date | Season | (in) | (in) | (in) | (hr) | (in) | (in/hr) | (in/hr) | (in/hr) | (in/hr) | (in/hr) | (in/hr) | (in/hr) | (in/hr) | (in/hr) | (cfs) | (cf) |
| 1 | 10/25/97 23:30 | 3 | 0.22 | 0.34 | 0.35 | 17.42 | 0.98 | 0.06 | 0.600 | 0.390 | 0.360 | 0.320 | 0.285 | 0.267 | 0.254 | 0.230 | 17.29 | 2.95E+05 |
| 2 | 10/26/97 17:00 | 3 | 1.02 | 1.27 | 1.33 | 11.58 | 0.35 | 0.03 | 0.720 | 0.540 | 0.456 | 0.380 | 0.300 | 0.280 | 0.263 | 0.230 | 20.38 | 1.94E+05 |
| 3 | 11/1/97 16:23 | 3 | 0.02 | 0.02 | 1.4 | 8.07 | 0.62 | 0.08 | 0.72 | 0.48 | 0.408 | 0.36 | 0.315 | 0.293 | 0.286 | 0.27 | 3.01 | 1.57E+04 |
| 4 | 11/13/97 22:10 | 3 | 0.07 | 0.09 | 0.26 | 12.08 | 0.39 | 0.03 | 0.360 | 0.180 | 0.168 | 0.140 | 0.120 | 0.107 | 0.101 | 0.090 | 2.75 | 4.10E+04 |
| 5 | 11/21/97 8:40 | 3 | 0.1 | 0.11 | 0.5 | 33.83 | 0.60 | 0.02 | 0.456 | 0.342 | 0.319 | 0.285 | 0.242 | 0.215 | 0.2 | 0.171 | 4.75 | 4.16E+04 |
| 6 | 1/16/98 6:50 | 4 | 0.26 | 0.36 | 0.44 | 6.58 | 0.43 | 0.07 | 0.360 | 0.210 | 0.192 | 0.180 | 0.165 | 0.160 | 0.15 | 0.130 | 2.59 | 1.41E+04 |
| 7 | 1/18/98 20:35 | 4 | 0.01 | 0.7 | 0.3 | 4.58 | 0.16 | 0.03 | 0.120 | 0.090 | 0.010 | 0.010 | 0.010 | 0.010 | 0.009 | 0.007 | 0.26 | 8.59E+02 |
| 8 | 1/27/98 5:55 | 4 | 0.01 | 0.59 | 0.64 | 15.42 | 1.27 | 0.08 | 0.240 | 0.270 | 0.240 | 0.240 | 0.225 | 0.227 | 0.221 | 0.210 | 3.53 | 7.01E+04 |
| 9 | 2/2/98 23:40 | 4 | 0 | 0 | 0.25 | 50.58 | 1.66 | 0.03 | 0.240 | 0.270 | 0.264 | 0.260 | 0.240 | 0.240 | 0.23 | 0.210 | 2.40 | 1.19E+05 |
| 10 | 2/11/98 10:20 | 4 | 0 | 0.03 | 1.1 | 2.37 | 0.21 | 0.09 | 0.240 | 0.210 | 0.216 | 0.220 | 0.210 | 0.200 | 0.19 | 0.170 | 0.95 | 2.70E+03 |
| 11 | 2/17/98 3:00 | 4 | 0.17 | 0.17 | 0.45 | 2.00 | 0.13 | 0.07 | 0.240 | 0.120 | 0.096 | 0.100 | 0.090 | 0.093 | 0.086 | 0.070 | 0.11 | 3.62E+02 |
| 12 | 2/17/98 10:12 | 4 | 0.3 | 0.3 | 0.59 | 5.67 | 0.31 | 0.05 | 0.600 | 0.360 | 0.336 | 0.300 | 0.300 | 0.280 | 0.27 | 0.250 | 3.07 | 1.07E+04 |
| 13 | 4/3/98 16:31 | 1 | 0.48 | 1.2 | 1.21 | 7.30 | 0.79 | 0.11 | 2.760 | 0.990 | 0.816 | 0.680 | 0.540 | 0.493 | 0.459 | 0.390 | 19.72 | 7.15E+04 |
| 14 | 4/8/98 22:52 | 1 | 0.09 | 0.1 | 2.07 | 4.97 | 1.38 | 0.28 | 1.920 | 1.200 | 1.032 | 0.920 | 0.780 | 0.707 | 0.668 | 0.590 | 26.34 | 1.75E+05 |
| 15 | 4/16/98 11:33 | 1 | 0 | 0.31 | 1.7 | 15.30 | 3.34 | 0.22 | 4.800 | 2.400 | 2.040 | 1.780 | 1.425 | 1.320 | 1.237 | 1.070 | 53.66 | 5.71E+05 |
| 16 | 4/18/98 21:36 | 1 | 2.05 | 4.06 | 4.06 | 18.55 | 3.22 | 0.17 | 1.680 | 0.870 | 0.744 | 0.700 | 0.630 | 0.640 | 0.613 | 0.560 | 41.81 | 1.24E+06 |
| 17 | 5/1/98 2:13 | 1 | 0.29 | 0.39 | 0.52 | 0.85 | 0.36 | 0.42 | 3.000 | 0.840 | 0.696 | 0.660 | 0.510 | 0.467 | 0.311 | 0.000 | 7.02 | 1.34E+04 |
| 18 | 5/21/98 17:57 | 1 | 0 | 0 | 0.31 | 1.37 | 1.76 | 1.29 | 5.280 | 3.750 | 3.384 | 2.920 | 2.310 | 2.067 | 1.911 | 1.600 | 54.30 | 1.47E+05 |
| 19 | 7/9/98 16:56 | 2 | 0.05 | 0.05 | 0.07 | 0.32 | 0.39 | 1.23 | 1.920 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0 | 0.000 | 11.44 | 1.02E+04 |
| 20 | 7/31/98 2:08 | 2 | 0.15 | 0.16 | 0.56 | 2.75 | 0.25 | 0.09 | 1.080 | 0.330 | 0.288 | 0.240 | 0.195 | 0.200 | 0.193 | 0.180 | 0.74 | 1.78E+03 |
| 21 | 7/31/98 8:19 | 2 | 0.4 | 0.41 | 0.72 | 4.38 | 0.41 | 0.09 | 1.080 | 0.480 | 0.384 | 0.320 | 0.240 | 0.267 | 0.254 | 0.230 | 6.36 | 1.36E+04 |
| 22 | 8/10/98 12:45 | 2 | 0.04 | 0.04 | 0.04 | 0.48 | 0.58 | 1.20 | 3.480 | 1.680 | 1.368 | 0.000 | 0.000 | 0.000 | 0 | 0.000 | 6.40 | 7.91E+03 |
| 23 | 1/31/99 14:24 | 4 | 0.06 | 0.32 | xxx | 19.10 | 0.54 | 0.03 | 0.480 | 0.270 | 0.240 | 0.200 | 0.180 | 0.173 | 0.169 | 0.160 | 1.48 | 9.58E+03 |
| 24 | 2/17/99 2:43 | 4 | 0 | 0 | 0.33 | 6.30 | 0.36 | 0.06 | 0.360 | 0.210 | 0.216 | 0.220 | 0.210 | 0.187 | 0.178 | 0.160 | 0.05 | 4.28E+01 |
| 25 | 2/17/99 13:39 | 4 | 0.37 | 0.37 | 0.69 | 5.88 | 0.26 | 0.04 | 0.480 | 0.210 | 0.168 | 0.140 | 0.120 | 0.107 | 0.101 | 0.090 | 0.04 | 3.17E+01 |
| 26 | 2/19/99 9:00 | 4 | 0.32 | 0.62 | 0.8 | 5.88 | 0.22 | 0.04 | 0.240 | 0.120 | 0.120 | 0.120 | 0.105 | 0.107 | 0.104 | 0.100 | 0.40 | 1.67E+03 |
| 27 | 2/27/99 15:49 | 4 | 0.04 | 0.16 | 0.17 | 10.12 | 0.87 | 0.09 | 0.480 | 0.300 | 0.264 | 0.260 | 0.240 | 0.240 | 0.233 | 0.220 | 2.77 | 2.02E+04 |
| 28 | 3/2/99 23:09 | 1 | 0 | 0.92 | 1.03 | 15.47 | 1.44 | 0.09 | 1.320 | 0.630 | 0.528 | 0.480 | 0.390 | 0.360 | 0.347 | 0.320 | 4.41 | 2.86E+04 |
| 29 | 3/9/99 0:21 | 1 | 0 | 0.15 | 1.59 | 6.33 | 0.43 | 0.07 | 0.360 | 0.240 | 0.240 | 0.240 | 0.210 | 0.213 | 0.209 | 0.200 | 0.79 | 2.32E+03 |
| 30 | 5/5/99 3:33 | 1 | 0 | 0 | 1.24 | 3.48 | 0.34 | 0.10 | 0.480 | 0.240 | 0.216 | 0.180 | 0.150 | 0.133 | 0.122 | 0.100 | 0.56 | 1.24E+03 |
| 31 | 5/5/99 23:48 | 1 | 0.35 | 0.35 | 1.29 | 7.15 | 2.32 | 0.32 | 5.880 | 2.580 | 2.136 | 1.840 | 1.455 | 1.333 | 1.239 | 1.050 | 35.19 | 2.11E+05 |
| 32 | 5/18/99 14:44 | 1 | 0 | 0 | 0.51 | 2.05 | 0.36 | 0.18 | 1.440 | 0.630 | 0.528 | 0.480 | 0.390 | 0.373 | 0.356 | 0.320 | 2.97 | 4.04E+03 |

Note: xxx denotes data collection error for the 8-day AMC for storm # 23.

Table 4.2. Summary of the rainfall and runoff data collected on the Church watershed.

| | | | AMC (prior days) | | | Rainfall | | | Intensity for these times (min) | | | | | | | | Runoff | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 2 | 5 | 8 | Duration | Depth | Intensity | 5 | 20 | 25 | 30 | 40 | 45 | 50 | 60 | Peak | Volume |
| Storm | Date | Season | (in) | (in) | (in) | (hr) | (in) | (in/hr) | (in/hr) | (in/hr) | (in/hr) | (in/hr) | (in/hr) | (in/hr) | (in/hr) | (in/hr) | (cfs) | (cf) |
| 1 | 10/24/97 | 3 | 0.03 | 0.18 | 0.29 | 3.1 | 0.24 | 0.08 | 0.240 | 0.120 | 0.120 | 0.120 | 0.120 | 0.120 | 0.117 | 0.110 | 0.77 | 6.5E+03 |
| 2 | 10/25/97 | 3 | 0.29 | 0.29 | 0.47 | 14.0 | 1.06 | 0.08 | 0.480 | 0.360 | 0.312 | 0.320 | 0.285 | 0.280 | 0.263 | 0.230 | 2.50 | 3.2E+04 |
| 3 | 10/26/97 | 3 | 1.11 | 1.45 | 1.53 | 3.9 | 0.36 | 0.09 | 0.600 | 0.450 | 0.408 | 0.380 | 0.315 | 0.280 | 0.263 | 0.230 | 2.89 | 1.2E+04 |
| 4 | 11/1/97 | 3 | 0.03 | 0.03 | 0.42 | 8.1 | 0.62 | 0.08 | 0.480 | 0.330 | 0.288 | 0.260 | 0.240 | 0.253 | 0.246 | 0.230 | 1.99 | 2.0E+04 |
| 5 | 11/6/97 | 3 | 0.03 | 0.83 | 0.84 | 2.2 | 0.11 | 0.05 | 0.120 | 0.060 | 0.072 | 0.060 | 0.060 | 0.053 | 0.052 | 0.050 | 0.44 | 2.1E+03 |
| 6 | 11/13/97 | 3 | 0.1 | 0.12 | 0.35 | 13.9 | 0.49 | 0.04 | 0.480 | 0.180 | 0.168 | 0.160 | 0.150 | 0.147 | 0.141 | 0.130 | 1.15 | 2.1E+04 |
| 7 | 11/21/97 | 3 | 0.04 | 0.04 | 0.57 | 14.2 | 0.55 | 0.04 | 0.360 | 0.240 | 0.192 | 0.160 | 0.150 | 0.133 | 0.129 | 0.120 | 0.03 | 1.5E+02 |
| 8 | 1/7/98 | 4 | 0.09 | 0.09 | 0.35 | 15.1 | 0.91 | 0.06 | 0.360 | 0.180 | 0.168 | 0.160 | 0.165 | 0.160 | 0.160 | 0.160 | 1.74 | 4.8E+04 |
| 9 | 1/15/98 | 4 | 0.02 | 0.016 | 1.16 | 5.5 | 0.17 | 0.03 | 0.240 | 0.120 | 0.096 | 0.080 | 0.060 | 0.067 | 0.061 | 0.050 | 0.73 | 8.5E+03 |
| 10 | 1/15/98 | 4 | 0.19 | 0.32 | 0.79 | 3.8 | 0.18 | 0.05 | 0.120 | 0.090 | 0.096 | 0.080 | 0.075 | 0.067 | 0.064 | 0.060 | 1.00 | 1.1E+04 |
| 11 | 1/16/98 | 4 | 0.37 | 0.5 | 0.55 | 5.4 | 0.54 | 0.10 | 0.360 | 0.210 | 0.192 | 0.200 | 0.180 | 0.173 | 0.169 | 0.160 | 2.15 | 2.9E+04 |
| 12 | 1/18/98 | 4 | 0.02 | 0.92 | 1.05 | 3.9 | 0.2 | 0.05 | 0.240 | 0.120 | 0.120 | 0.100 | 0.090 | 0.080 | 0.080 | 0.080 | 0.93 | 1.0E+04 |
| 13 | 1/27/98 | 4 | 0.01 | 0.76 | 0.76 | 14.6 | 1.21 | 0.08 | 0.240 | 0.210 | 0.192 | 0.200 | 0.180 | 0.187 | 0.181 | 0.170 | 1.95 | 2.8E+04 |
| 14 | 2/2/98 | 4 | 0.01 | 0.02 | 1.23 | 43.6 | 1.83 | 0.04 | 0.240 | 0.240 | 0.240 | 0.220 | 0.210 | 0.200 | 0.197 | 0.190 | 2.58 | 1.3E+05 |
| 15 | 2/11/98 | 4 | 0.01 | 0.04 | 1.6 | 2.4 | 0.23 | 0.10 | 0.360 | 0.240 | 0.240 | 0.240 | 0.225 | 0.227 | 0.211 | 0.180 | 1.97 | 1.1E+04 |
| 16 | 2/11/98 | 4 | 0.27 | 0.31 | 1.09 | 3.5 | 0.06 | 0.02 | 0.120 | 0.060 | 0.072 | 0.060 | 0.045 | 0.040 | 0.037 | 0.030 | 0.57 | 3.3E+03 |
| 17 | 2/16/98 | 4 | 0.01 | 0.33 | 0.33 | 6.0 | 0.17 | 0.03 | 0.240 | 0.090 | 0.096 | 0.080 | 0.060 | 0.067 | 0.061 | 0.050 | 0.97 | 7.4E+03 |
| 18 | 2/17/98 | 4 | 0.19 | 0.19 | 0.51 | 2.0 | 0.16 | 0.08 | 0.240 | 0.120 | 0.120 | 0.120 | 0.105 | 0.107 | 0.101 | 0.090 | 1.25 | 8.1E+03 |
| 19 | 2/17/98 | 4 | 0.35 | 0.35 | 0.67 | 4.7 | 0.28 | 0.06 | 0.360 | 0.300 | 0.288 | 0.280 | 0.270 | 0.267 | 0.254 | 0.230 | 2.89 | 1.5E+04 |
| 20 | 5/21/98 | 1 | 0 | 0.16 | 0.16 | 1.2 | 1.64 | 1.33 | 5.160 | 3.750 | 3.456 | 3.160 | 2.430 | 2.160 | 1.980 | 1.620 | 19.84 | 5.9E+04 |
| 21 | 6/30/98 | 2 | 0.01 | 0.01 | 0.02 | 1.5 | 0.31 | 0.20 | 1.560 | 0.540 | 0.432 | 0.360 | 0.270 | 0.240 | 0.220 | 0.180 | 2.28 | 3.4E+03 |
| 22 | 7/9/98 | 2 | 0.04 | 0.04 | 0.04 | 2.2 | 0.18 | 0.08 | 0.720 | 0.420 | 0.336 | 0.280 | 0.210 | 0.187 | 0.171 | 0.140 | 0.22 | 3.6E+02 |
| 23 | 7/23/98 | 2 | 0.59 | 1.25 | 2.11 | 2.1 | 0.29 | 0.14 | 1.200 | 0.510 | 0.432 | 0.360 | 0.285 | 0.267 | 0.254 | 0.230 | 2.32 | 5.1E+03 |
| 24 | 7/31/98 | 2 | 0.45 | 0.49 | 1.25 | 5.1 | 0.26 | 0.05 | 0.600 | 0.300 | 0.264 | 0.220 | 0.195 | 0.213 | 0.209 | 0.200 | 1.26 | 3.4E+03 |
| 25 | 8/10/98 | 2 | 0.06 | 0.06 | 0.06 | 0.4 | 0.6 | 1.44 | 4.200 | 1.770 | 1.416 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 8.16 | 9.5E+03 |
| 26 | 8/11/98 | 2 | 0.66 | 0.66 | 0.66 | 2.6 | 0.21 | 0.08 | 1.320 | 0.570 | 0.456 | 0.380 | 0.285 | 0.253 | 0.232 | 0.190 | 0.02 | 2.3E+01 |
| 27 | 8/13/98 | 2 | 0.02 | 0.02 | 0.88 | 1.3 | 0.59 | 0.46 | 3.000 | 1.710 | 1.368 | 1.140 | 0.855 | 0.760 | 0.697 | 0.570 | 1.96 | 2.3E+03 |
| 28 | 8/16/98 | 2 | 0.45 | 1.13 | 1.97 | 6.1 | 0.44 | 0.07 | 1.200 | 0.420 | 0.336 | 0.280 | 0.210 | 0.187 | 0.171 | 0.140 | 0.09 | 1.0E+02 |
| 29 | 8/16/98 | 2 | 0.59 | 1.25 | 2.11 | 3.7 | 0.3 | 0.08 | 0.960 | 0.480 | 0.384 | 0.320 | 0.240 | 0.213 | 0.212 | 0.210 | 0.16 | 4.1E+02 |
| 30 | 10/3/98 | 3 | 0 | 0.3 | 0.3 | 4.7 | 0.46 | 0.10 | 0.600 | 0.510 | 0.504 | 0.500 | 0.480 | 0.467 | 0.444 | 0.400 | 0.77 | 1.4E+03 |
| 31 | 1/31/99 | 4 | 0.31 | 0.41 | 0.7 | 20.8 | 0.6 | 0.03 | 0.360 | 0.240 | 0.216 | 0.200 | 0.165 | 0.173 | 0.169 | 0.160 | 0.21 | 2.5E+03 |
| 32 | 2/17/99 | 4 | 0 | 0.29 | 0.46 | 7.4 | 0.39 | 0.05 | 0.360 | 0.240 | 0.216 | 0.240 | 0.225 | 0.213 | 0.199 | 0.170 | 0.50 | 1.5E+03 |
| 33 | 2/17/99 | 4 | 0.4 | 0.43 | 0.85 | 5.8 | 0.27 | 0.05 | 0.360 | 0.180 | 0.168 | 0.140 | 0.105 | 0.107 | 0.101 | 0.090 | 0.19 | 9.4E+02 |
| 34 | 2/19/99 | 4 | 0.42 | 0.7 | 0.98 | 6.7 | 0.25 | 0.04 | 0.240 | 0.120 | 0.120 | 0.120 | 0.120 | 0.120 | 0.113 | 0.100 | 0.20 | 8.4E+02 |
| 35 | 2/27/99 | 4 | 0.05 | 0.18 | 0.19 | 10.2 | 0.88 | 0.09 | 0.600 | 0.300 | 0.264 | 0.240 | 0.225 | 0.213 | 0.209 | 0.200 | 1.19 | 6.7E+03 |
| 36 | 3/2/99 | 1 | 0.02 | 0.92 | 107 | 16.1 | 1.45 | 0.09 | 0.600 | 0.450 | 0.408 | 0.380 | 0.315 | 0.280 | 0.273 | 0.260 | 2.10 | 1.2E+04 |
| 37 | 3/9/99 | 1 | 0 | 0.19 | 1.72 | 8.0 | 0.56 | 0.07 | 0.480 | 0.270 | 0.264 | 0.240 | 0.225 | 0.213 | 0.212 | 0.210 | 0.80 | 3.2E+03 |
| 38 | 5/5/99 | 1 | 0.01 | 0.01 | 1.3 | 9.0 | 0.41 | 0.05 | 1.080 | 0.420 | 0.336 | 0.300 | 0.255 | 0.227 | 0.214 | 0.190 | 0.17 | 1.8E+02 |
| 39 | 5/5/99 | 1 | 0.38 | 0.38 | 1.38 | 7.1 | 2.01 | 0.28 | 3.960 | 2.070 | 1.728 | 1.500 | 1.215 | 1.107 | 1.044 | 0.920 | 5.73 | 2.0E+04 |
| 40 | 5/18/99 | 1 | 0 | 0.17 | 0.37 | 2.2 | 0.39 | 0.17 | 1.320 | 0.660 | 0.552 | 0.480 | 0.405 | 0.387 | 0.371 | 0.340 | 1.09 | 1.2E+03 |

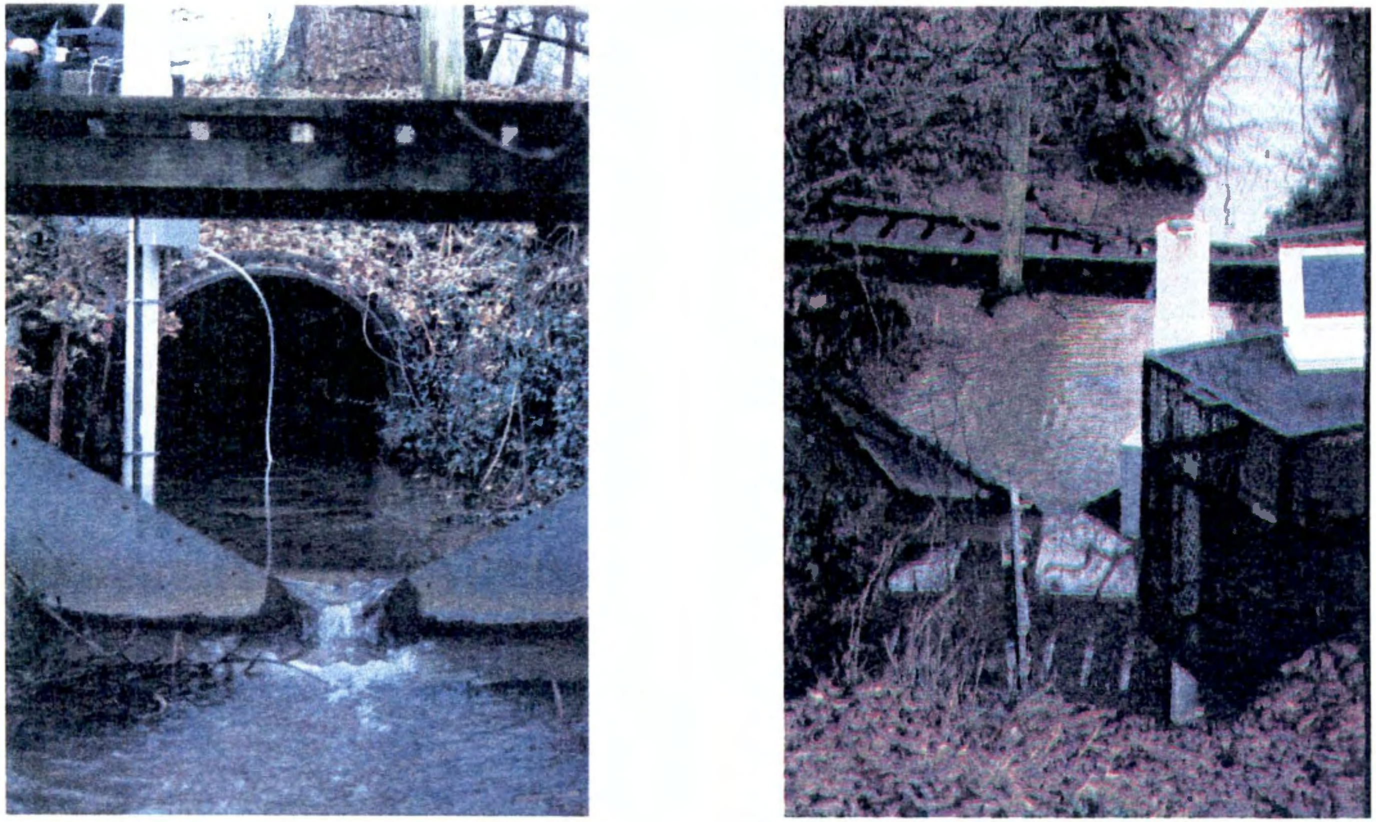


**Figure 4.3. Upstream (right) and downstream views at the Sanford weir.**

In October 1998, a crest staff gage similar to that described by Mockus (1972a) was installed. The peak flow recorded by the crest staff gage was compared to the stage height measured by the TFLI. The two devices recorded similar peak flow stage heights. In addition, real-time comparisons of direct measurements and electronic measurements were made during storm events.

A Qualimetrics, Inc. tipping bucket rain gage (0.01 in per tip) to record rainfall data for the Sanford Watershed was located just outside of the watershed. Initially, the data were recorded on a CR-10 datalogger at five-minute intervals. In February 1998, the CR-10 logger was removed and a HOBO event logger was installed. The HOBO logger recorded the date and time each tip occurred. A loose wire caused intermittent errors between November 1998 through January 1999 and most of those rainfall data were incorrect. In May 1999, the rain gage was rechecked and found to be accurate.

### Church Watershed

The Church Watershed discharge point was located near the Merchants Road Home Federal Bank in a box culvert parallel to Davida Road (Figure 4.4). The watershed discharge was measured using a $120^0$ v-notch sharp-crested weir, and a pressure transducer measured stage height. The pressure transducer was calibrated prior to installation. During installation, the elevation difference between the weir crest and the pressure transducer was measured. The elevation difference was then accounted for as an offset in the datalogger program. The calibration and offset were periodically field-checked during runoff events. This was done by measuring the height of flow above the crest and comparing it to the recorded data. Discharge was determined using the same empirical relationship noted earlier. Data were recorded in one-minute increments on a Campbell Scientific CR-10 datalogger.

In March 1998, and again during June 1998, the equipment was submerged during runoff events and data were lost. In August 1998, the datalogger was moved to a higher elevation.

**Figure 4.4. Looking downstream at the Church site. Weir is located within a box culvert, which is approximately six feet downstream from the oval pipe opening.**

A Davis Instruments tipping bucket rain gage (0.01 in per tip) measured rainfall data for the Church watershed. Again, tips were recorded using a HOBO event logger. In May 1999, the rain gage was rechecked for accuracy using a method recommended by the manufacturer. The gage was found to be within the specified tolerances (Davis Instruments, 1999).

### Rainfall and Runoff Data

The collected data were processed using a spreadsheet software program to summarize rainfall depth and intensities for various time periods, peak flow rate, and total volume for each event. Antecedent moisture conditions (AMC) were determined using a standard NRCS method (NRCS, 1986). This method considered the previous five-day rainfall and a seasonal adjustment to account for evapotranspiration (NRCS, 1986). Tables 4.1 and 4.2 contain the derived information.

## A PRIORI PARAMETER SELECTION METHODS

As noted in the background section on the Rational Method, the C coefficient and $T_c$ are the major sources of error in the peak flow estimates. In a similar manner, the CN is a major source of uncertainty in the SCS Curve Number Method. To examine whether the errors are due to the resolution of the model parameters, different characterizations were performed on the watersheds.

The first step in applying the Rational and SCS Methods required both watersheds to be quantified for area, land use, topography, and soil characteristics. Low, moderate, and high-resolution characterizations were performed so that different resolution parameters could be selected for the models.

### Low-resolution Characterization

Characterization was cursory. Only the watershed area and a single land use were determined (Table 4.3). Area was determined by measuring the watershed perimeter on the USGS Fountain City Quadrangle map using a Planix Model 5 digital planimeter manufactured by Tamaya Technics Inc. Land use was determined by the author's general knowledge of the area. The time expended was about one hour.

**Table 4.3. Watershed land use from the low-resolution characterization.**

| | **Sanford Watershed** | **Church Watershed** |
|---|---|---|
| **Dominant Land Use** | Suburban Residential | Suburban Residential |
| **Watershed Area (acres)** | 179 | 107 |
| **Watershed Area (ha)** | 72 | 43 |

## Moderate-resolution Characterization

Additional characterization was performed using easily obtained and relatively large-scale data sources. Land use and soil family areas were determined using a dot-grid overlay technique. In this technique, a transparency with evenly spaced dots was placed over a paper map. The number of dots in each subarea were counted and divided by the total number of dots within the entire watershed. From this ratio, area extents of the land use and soils were determined. Land use was determined using USGS 1:100,000-scale land use/cover map, and soil series using USDA NRCS STATSGO soil maps. Tables 4.4 and 4.5 show the results for the two watersheds. Measuring the intersecting sets of land use and soil types was important. To do this, a modified dot-grid method was used. The different land uses were first traced onto a gridded transparency with a marker. This grid was then placed over the soil map. The dots that intersected the different land use and soils were then counted and the ratios for each group were determined. Table 4.6 shows the results for the two watersheds. The topographic features were determined by examining the USGS Fountain City Quadrangle. Table 4.7 contains that information.

The time and effort involved in this characterization was more intense than that of the low-resolution characterization. Obtaining the maps, available from government sources, map resellers, or digital versions available on the worldwide web, took approximately two hours. The remaining steps in the moderate-resolution characterization including using the dot-grid overlays and tabulating the results, and required one hour total for the combined watersheds.

**Table 4.4. Moderate-resolution land use characterization. The areas were determined using the dot-grid method and a USGS 1:100,000-scale land use/cover map.**

| | Sanford Watershed | | | Church Watershed | | |
|---|---|---|---|---|---|---|
| **Land Use** | **Percent** | **Acres** | **ha** | **Percent** | **Acres** | **ha** |
| Mixed Forest | 5 | 8.9 | 36 | 14 | 15.3 | 62 |
| Residential | 95 | 170.1 | 68 | 86 | 91.7 | 37 |

**Table 4.5. Moderate-resolution soil family characterization. Areas were determined using the dot-grid method and USDA-NRCS STATSGO maps. It should be noted that the hydrologic soil group (HSG) is the same for both families.**

| | | Sanford Watershed | | | Church Watershed | | |
|---|---|---|---|---|---|---|---|
| **Soil Family** | **HSG** | **Percent** | **Acres** | **ha** | **Percent** | **Acres** | **ha** |
| Fullerton-Bodine-Clarksville | B | 100 | 179 | 72 | 53.3 | 57.1 | 23 |
| Dewey-Decatur-Emory | B | - | - | - | 46.7 | 49.9 | 20 |

**Table 4.6. Intersecting groups of land uses and soil families determined using a modified version of the dot-grid method during the moderate-resolution characterization.**

| Land Use | Soil Family | Sanford Watershed | | | Church Watershed | | |
|---|---|---|---|---|---|---|---|
| | | Pe<br>rcent | /<br>cres | <br>a | Pe<br>rcent | /<br>cres | <br>a |
| Mixed<br>Forest | F-B-C | 5 | ç | | 13 | 1<br>4.3 | |
| Mixed<br>Forest | D-D-E | - | - | | - | - | |
| Residenti<br>al | F-B-C | 95 | 1<br>70 | <br>8 | 40 | 4<br>2.8 | <br>7 |
| Residenti<br>al | D-D-E | - | - | | 46<br>.4 | 4<br>9.9 | <br>0 |

**Table 4.7. Topographic features of the watersheds.**

| Feature | Sanford Watershed | Church Watershed |
|---|---|---|
| Watershed Shape | **Triangular** | **Rectangular** |
| Highest Elevation | **1300 ft (396 m)** | **1195 ft (364 m)** |
| Lowest Elevation | **1070 ft (326 m)** | **1033 ft (315 m)** |
| Change in Elevation | **230 ft (70 m)** | **162 ft (49 m)** |
| Watershed Length | **4380 ft (1335 m)** | **4170 ft (1270 m)** |
| Watershed Width | **2590 ft (790 m)** | **2160 ft (660 m)** |
| Overall Slope | **6.0%** | **3.5%** |

Note: Data source USGS Fountain City Quadrangle (1:24,000 scale)

### High-resolution Characterization

This characterization was made using the best available data. The following data sources were used in the characterization:

- Land use data (1:6,000 scale)
- Soil data (1:24,000 scale)
- Additional data from field survey

The land use data were obtained from the City of Knoxville Mapping and Records department. The information was then field checked and appropriate corrections were made. Similar information could be assembled strictly from field surveys. The soils data were obtained from the Knox County Soil Survey (SCS, 1953).

The land use and soil areas were measured using a different method for each watershed. The Church watershed was evaluated using the dot-grid overlay method. However, the different map scales were a complicating issue since the size of the soil map was 25% of the size of the land use map. To overcome this, the soils map was enlarged 400% and photocopied onto a grid-dot transparency. During the copying process, the contrast was adjusted such that only the soil type boundaries were visible. This transparency was then placed over the land use map and aligned using common reference points. The dot-grid method was used to measure the intersecting groups of soil types and land uses (Table 4.8). It should be noted that photocopying could introduce some small distortions because of the imprecision in scanning and the shrinkage or enlargement of the paper map and transparency. These errors were judged to be inconsequential. The procedure required about two hours to evaluate the Church watershed.

The same information sources were used in evaluating the Sanford watershed (Table 4.9). This time however, the information was analyzed using a Geographic Information System (GIS) software package (Potter, et al. 1999). This required the paper maps to be converted into a digital format. Once the information was in the GIS, the land

**Table 4.8. Intersecting soil types and land use areas in the Church watershed determined using the dot-grid method during the high-resolution characterization.**

| Soil Type | Single Fam. Residential Acres | Single Fam. Residential Percent | Multi-Fam. Residential Acres | Multi-Fam. Residential Percent | Commercial Acres | Commercial Percent | Churches/ Schools Acres | Churches/ Schools Percent | Roads Acres | Roads Percent | Forest Acres | Forest Percent | Soil Totals Acres | Soil Totals Percent |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| *Cd* | 3.0 | 2.8 | 0.8 | 0.7 | 0.0 | 0.0 | 0.0 | 0.0 | 0.8 | 0.7 | 0.8 | 0.7 | 5.3 | 5.0 |
| *Cg* | 1.5 | 1.4 | 0.8 | 0.7 | 0.0 | 0.0 | 1.5 | 1.4 | 0.4 | 0.4 | 0.8 | 0.7 | 4.9 | 4.6 |
| *Dn* | 0.8 | 0.7 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.8 | 0.7 |
| *Do* | 6.8 | 6.4 | 0.8 | 0.7 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 7.6 | 7.1 |
| *Dp* | 0.8 | 0.7 | 0.0 | 0.0 | 0.0 | 0.0 | 0.8 | 0.7 | 0.0 | 0.0 | 0.0 | 0.0 | 1.5 | 1.4 |
| *Du* | 11.4 | 10.6 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.8 | 0.7 | 0.0 | 0.0 | 12.1 | 11.3 |
| *Dx* | 3.0 | 2.8 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.4 | 0.4 | 0.8 | 0.7 | 4.2 | 3.9 |
| *Ea* | 3.8 | 3.5 | 0.8 | 0.7 | 0.8 | 0.7 | 4.6 | 4.3 | 3.8 | 3.5 | 0.0 | 0.0 | 13.7 | 12.8 |
| *Ec* | 6.1 | 5.7 | 0.0 | 0.0 | 0.0 | 0.0 | 0.4 | 0.4 | 0.8 | 0.7 | 0.8 | 0.7 | 8.0 | 7.4 |
| *Fg* | 0.8 | 0.7 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.8 | 0.7 | 1.5 | 1.4 |
| *Fs* | 4.6 | 4.3 | 0.8 | 0.7 | 0.0 | 0.0 | 0.0 | 0.0 | 0.4 | 0.4 | 0.0 | 0.0 | 5.7 | 5.3 |
| *Ft* | 5.3 | 5.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.8 | 0.7 | 0.4 | 0.4 | 0.0 | 0.0 | 6.5 | 6.0 |
| *Fv* | 6.1 | 5.7 | 0.0 | 0.0 | 0.0 | 0.0 | 0.8 | 0.7 | 0.2 | 0.2 | 0.8 | 0.7 | 7.8 | 7.3 |
| *Fw* | 11.4 | 10.6 | 0.0 | 0.0 | 0.0 | 0.0 | 1.5 | 1.4 | 1.5 | 1.4 | 0.0 | 0.0 | 14.4 | 13.5 |
| *Gd* | 3.8 | 3.5 | 0.0 | 0.0 | 0.0 | 0.0 | 1.5 | 1.4 | 0.2 | 0.2 | 1.5 | 1.4 | 7.0 | 6.6 |
| *Gf* | 6.1 | 5.7 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 6.1 | 5.7 |
| **Acres** | 75.1 | | 3.8 | | 0.8 | | 11.8 | | 9.5 | | 6.1 | | 107.0 | |
| **Percentage** | | 70.2 | | 3.5 | | 0.7 | | 11.0 | | 8.9 | | 5.7 | | 100.0 |



**Table 4.9. Intersecting soil types and land use areas in the Sanford watershed determined using GIS software during the high-resolution characterization.**

| Soil Type | Rural Residential Acres | Rural Residential Percent | Single Fam. Residential Acres | Single Fam. Residential Percent | Multi-Fam. Residential Acres | Multi-Fam. Residential Percent | Churches/ Schools Acres | Churches/ Schools Percent | Roads Acres | Roads Percent | Forest Acres | Forest Percent | Soil Totals Acres | Soil Totals Percent |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| *Cc* | 0.0 | 0.0 | 2.5 | 1.4 | 0.0 | 0.0 | 1.8 | 1.0 | 0.2 | 0.1 | 0.0 | 0.0 | 4.4 | 2.5 |
| *Cd* | 0.0 | 0.0 | 4.3 | 2.4 | 0.0 | 0.0 | 3.7 | 2.1 | 0.7 | 0.4 | 0.5 | 0.3 | 9.2 | 5.1 |
| *Cf* | 0.0 | 0.0 | 17.3 | 9.6 | 0.0 | 0.0 | 0.0 | 0.0 | 2.5 | 1.4 | 0.3 | 0.1 | 20.1 | 11.2 |
| *Cg* | 0.0 | 0.0 | 9.1 | 5.1 | 0.0 | 0.0 | 0.1 | 0.0 | 1.3 | 0.7 | 0.4 | 0.2 | 10.9 | 6.1 |
| *Dt* | 1.7 | 1.0 | 0.2 | 0.1 | 1.5 | 0.8 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 3.4 | 1.9 |
| *Du* | 2.1 | 1.1 | 0.0 | 0.0 | 3.5 | 2.0 | 0.0 | 0.0 | 0.1 | 0.0 | 0.0 | 0.0 | 5.7 | 3.2 |
| *Ec* | 0.1 | 0.0 | 0.0 | 0.0 | 0.2 | 0.1 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.3 | 0.2 |
| *Fd* | 3.8 | 2.1 | 18.4 | 10.2 | 0.0 | 0.0 | 0.7 | 0.4 | 3.1 | 1.7 | 0.6 | 0.3 | 26.5 | 14.8 |
| *Fe* | 4.9 | 2.7 | 13.0 | 7.2 | 0.0 | 0.0 | 0.7 | 0.4 | 1.9 | 1.1 | 0.9 | 0.5 | 21.5 | 12.0 |
| *Fh* | 0.0 | 0.0 | 5.2 | 2.9 | 0.0 | 0.0 | 0.0 | 0.0 | 0.5 | 0.3 | 0.0 | 0.0 | 5.7 | 3.2 |
| *Fm* | 0.0 | 0.0 | 1.4 | 0.8 | 0.0 | 0.0 | 0.0 | 0.0 | 0.5 | 0.3 | 0.0 | 0.0 | 2.0 | 1.1 |
| *Fn* | 0.0 | 0.0 | 0.5 | 0.3 | 0.0 | 0.0 | 0.0 | 0.0 | 0.1 | 0.0 | 0.0 | 0.0 | 0.5 | 0.3 |
| *Fs* | 1.5 | 0.8 | 6.7 | 3.7 | 1.5 | 0.9 | 3.6 | 2.0 | 1.7 | 0.9 | 1.2 | 0.7 | 16.3 | 9.1 |
| *Ft* | 3.0 | 1.7 | 5.2 | 2.9 | 5.0 | 2.8 | 2.7 | 1.5 | 0.6 | 0.3 | 1.3 | 0.7 | 17.7 | 9.9 |
| *Fy* | 0.9 | 0.5 | 3.1 | 1.8 | 0.0 | 0.0 | 0.1 | 0.1 | 0.4 | 0.2 | 0.6 | 0.3 | 5.2 | 2.9 |
| *Gc* | 0.0 | 0.0 | 5.1 | 2.8 | 0.0 | 0.0 | 2.5 | 1.4 | 0.7 | 0.4 | 0.4 | 0.2 | 8.6 | 4.8 |
| *Gd* | 4.2 | 2.3 | 12.7 | 7.1 | 1.9 | 1.1 | 0.0 | 0.0 | 1.5 | 0.9 | 1.1 | 0.6 | 21.5 | 12.0 |
| **Acres** | 22.2 | | 104.8 | | 13.7 | | 15.8 | | 15.7 | | 7.3 | | 179.5 | |
| **Percent** | | 12.4 | | 58.4 | | 7.6 | | 8.8 | | 8.8 | | 4.1 | | 100.0 |

use and soil type information were combined into a single map. The divisions in the map were a soil series--land use composite. The software then calculated the area of each division.

Digitizing the paper maps was complex and time consuming, but once the information was in the correct format the analysis required only a small amount of time and little effort. The total procedure of digitizing and analysis would take about eight hours to repeat for an experienced GIS user. Since this method was so time intensive, it should only be useful if the digital watershed data were going to be reused.

Using either method, the high-resolution characterization required considerably more time and effort than the low or moderate-resolution characterizations. Acquiring the county soil survey information was uncomplicated and required less than ½ hour at the library. The 1:6,000 scale land use information had already been developed by the City of Knoxville. Since the map was several years old, it was updated it by a "windshield" survey of each watershed that took roughly four hours.

Measuring the land use or soil type areas individually was routine using the dot-grid overlay. It took about twice the time of the moderate-resolution characterization simply because of the increased number of soil and land use groups. For example, the Church watershed had 16 soil types and six land uses in the high-resolution characterization, as compared to two soil types and two land uses in the moderate-resolution characterization. Once the maps and overlays were assembled, measuring the land use and soil type areas for both watersheds took about two hours.

In all, the time required to gather, correct and analyze the information for each of the three characterizations is as follows:

| | |
|---|---|
| Low-resolution Characterization | about one hour per watershed |
| Moderate-resolution Characterization | about two hours per watershed |
| High-resolution Characterization | about six hours per watershed |

## RATIONAL METHOD

To evaluate whether increasing the parameter resolution increased estimation accuracy, nine peak flows were estimated for each observed event. These estimations were based on three C coefficients and three $T_{c}$.

### C Coefficient

Three progressively higher resolution C coefficients were established for each watershed using the three watershed characterizations. Table 4.10 summarizes the selected C coefficients.

#### Low-resolution C Coefficient

These C coefficients were simply and quickly selected and were based on the dominant land use determined in the low-resolution watershed characterizations. A chart produced jointly by the Water Environment Federation (WEF) and the American Society of Civil Engineers (ASCE) was used to relate land use to a C Coefficient (ASCE/WEF, 1992).

**Table 4.10. A priori C coefficient selections for the watersheds.**

| | C Coefficient Resolution | | |
|---|---|---|---|
| **Watershed** | **Low** | **Moderate** | **High** |
| ***Sanford*** | 0.30 | 0.29 | 0.32 |
| ***Church*** | 0.30 | 0.28 | 0.33 |

### Moderate-resolution C Coefficient

These coefficients were more complex and time-consuming than the low-resolution selections. The land use and soil hydrologic group (HSG) information located in Table 4.4 was reviewed in selecting a C coefficient for each watershed. The selection process was simplified because there was only a single soil hydrologic condition for each watershed, but since there were multiple land uses a composite land-use coefficient was calculated for each watershed on an area-weighted basis. The C coefficient for each land use was taken from the WEF/ASCE chart. The calculations and results are as follows:

Sanford Watershed

$$C = 9/179 \times 0.15 + 170/179 \times 0.30$$
$$C = 0.29$$

Church Watershed

$$C = 14.3/107 \times 0.15 + 92.7/107 \times 0.30$$
$$C = 0.28$$

### High-resolution C Coefficient

The third C coefficient used the land use, slope, and runoff potential information from the high-resolution watershed characterizations. Using those data, each watershed was divided into heterogeneous plots. For each plot, the corresponding area, percent of total area, land use, soil series, hydrologic group, and slope were recorded. A C coefficient table that considered land use, soil hydrologic condition, and slope was then used to assign a coefficient to each plot (Ward, 1995). Each coefficient was then multiplied by the percent of total area to get an area-weighted coefficient. Next, the weighted coefficients were summed to get the overall coefficient. Tables 4.11 and 4.12 show the characterization data, the assigned C coefficients and the resultant area-weighted C coefficients for the Sanford and Church watersheds respectively.

Table 4.11. The Sanford watershed high-resolution C coefficient estimation. The watershed was divided into 48 homogeneous plots assigned C coefficients based on land use, HSG, and slope, and the plots were then area-weighted. The products were summed and yielded a composite C = 0.32.

| *Plot* | *Acres* | *Area–weight* | *Land Use* | *Soil Type* | *HSG* | *Slope (%)* | *C Coefficient* | *$Ai/A_T \times Ci$* |
|---|---|---|---|---|---|---|---|---|
| 1 | 1.11 | 0.006 | Forest | Gd | A | 2 | 0.05 | 0.000 |
| 2 | 0.46 | 0.003 | Forest | Cd | A | 5 | 0.1 | 0.000 |
| 3 | 0.38 | 0.002 | Forest | Cg | A | 5 | 0.1 | 0.000 |
| 4 | 0.94 | 0.005 | Forest | Fe | A | 5 | 0.1 | 0.001 |
| 5 | 1.32 | 0.007 | Forest | Ft | B | 5 | 0.13 | 0.001 |
| 6 | 0.36 | 0.002 | Forest | Gc | B | 5 | 0.13 | 0.000 |
| 7 | 0.26 | 0.001 | Forest | Cf | B | 12 | 0.19 | 0.000 |
| 8 | 0.55 | 0.003 | Forest | Fd | B | 12 | 0.19 | 0.001 |
| 9 | 0.03 | 0.000 | Forest | Fm | C | 12 | 0.24 | 0.000 |
| 10 | 1.24 | 0.007 | Forest | Fs | C | 12 | 0.24 | 0.002 |
| 11 | 0.61 | 0.003 | Forest | Fy | D | 12 | 0.28 | 0.001 |
| 12 | 0.25 | 0.001 | Multi-family | Ec | A | 2 | 0.47 | 0.001 |
| 13 | 1.89 | 0.011 | Multi-family | Gd | A | 2 | 0.47 | 0.005 |
| 14 | 3.55 | 0.020 | Multi-family | Du | B | 5 | 0.5 | 0.010 |
| 15 | 4.96 | 0.028 | Multi-family | Ft | B | 5 | 0.5 | 0.014 |
| 16 | 0.03 | 0.000 | Multi-family | Fd | B | 12 | 0.52 | 0.000 |
| 17 | 1.46 | 0.008 | Multi-family | Dt | C | 12 | 0.54 | 0.004 |
| 18 | 1.54 | 0.009 | Multi-family | Fs | C | 12 | 0.54 | 0.005 |
| 19 | 0.01 | 0.000 | Multi-family | Fy | D | 12 | 0.56 | 0.000 |
| 20 | 1.53 | 0.009 | Roads | Gd | A | 2 | 0.57 | 0.005 |
| 21 | 0.68 | 0.004 | Roads | Cd | A | 5 | 0.57 | 0.002 |
| 22 | 1.33 | 0.007 | Roads | Cg | A | 5 | 0.57 | 0.004 |
| 23 | 1.93 | 0.011 | Roads | Fe | A | 5 | 0.57 | 0.006 |
| 24 | 0.46 | 0.003 | Roads | Fh | A | 5 | 0.57 | 0.001 |
| 25 | 0.08 | 0.000 | Roads | Du | B | 5 | 0.6 | 0.000 |
| 26 | 0.07 | 0.000 | Roads | Fn | B | 5 | 0.6 | 0.000 |
| 27 | 0.57 | 0.003 | Roads | Ft | B | 5 | 0.6 | 0.002 |
| 28 | 0.67 | 0.004 | Roads | Gc | B | 5 | 0.6 | 0.002 |
| 29 | 0.17 | 0.001 | Roads | Cc | B | 12 | 0.61 | 0.001 |
| 30 | 2.53 | 0.014 | Roads | Cf | B | 12 | 0.61 | 0.009 |
| 31 | 3.11 | 0.017 | Roads | Fd | B | 12 | 0.61 | 0.011 |
| 32 | 0.51 | 0.003 | Roads | Fm | C | 12 | 0.63 | 0.002 |
| 33 | 1.68 | 0.009 | Roads | Fs | C | 12 | 0.63 | 0.006 |
| 34 | 0.39 | 0.002 | Roads | Fy | D | 12 | 0.78 | 0.002 |
| 35 | 0.07 | 0.000 | Rural residential | Ec | A | 2 | 0.14 | 0.000 |
| 36 | 4.21 | 0.023 | Rural residential | Gd | A | 2 | 0.14 | 0.003 |
| 37 | 4.93 | 0.028 | Rural residential | Fe | A | 5 | 0.19 | 0.005 |
| 38 | 2.06 | 0.012 | Rural residential | Du | B | 5 | 0.21 | 0.002 |
| 39 | 3.02 | 0.017 | Rural residential | Ft | B | 5 | 0.21 | 0.004 |
| 40 | 3.75 | 0.021 | Rural residential | Fd | B | 12 | 0.26 | 0.005 |
| 41 | 1.73 | 0.010 | Rural residential | Dt | C | 12 | 0.31 | 0.003 |
| 42 | 1.48 | 0.008 | Rural residential | Fs | C | 12 | 0.31 | 0.003 |
| 43 | 0.93 | 0.005 | Rural residential | Fy | D | 12 | 0.35 | 0.002 |
| 44 | 0.02 | 0.000 | Schools/church | Gd | A | 2 | 0.67 | 0.000 |
| 45 | 3.69 | 0.021 | Schools/church | Cd | A | 5 | 0.68 | 0.014 |
| 46 | 0.06 | 0.000 | Schools/church | Cg | A | 5 | 0.68 | 0.000 |
| 47 | 0.70 | 0.004 | Schools/church | Fe | A | 5 | 0.68 | 0.003 |
| 48 | 2.66 | 0.015 | Schools/church | Ft | B | 5 | 0.68 | 0.010 |
| 49 | 2.45 | 0.014 | Schools/church | Gc | B | 5 | 0.68 | 0.009 |

| Plot | Acres | Area–weight | Land Use | Soil Type | HSG | Slope (%) | C Coefficient | $Ai/A_T \times Ci$ |
|---|---|---|---|---|---|---|---|---|
| 50 | 1.76 | 0.010 | Schools/church | Cc | B | 12 | 0.69 | 0.007 |
| 51 | 0.74 | 0.004 | Schools/church | Fd | B | 12 | 0.69 | 0.003 |
| 52 | 3.62 | 0.020 | Schools/church | Fs | C | 12 | 0.69 | 0.014 |
| 53 | 0.11 | 0.001 | Schools/church | Fy | D | 12 | 0.7 | 0.000 |
| 54 | 12.72 | 0.071 | Single family | Gd | A | 2 | 0.14 | 0.010 |
| 55 | 4.35 | 0.024 | Single family | Cd | A | 5 | 0.19 | 0.005 |
| 56 | 9.11 | 0.051 | Single family | Cg | A | 5 | 0.19 | 0.010 |
| 57 | 13.00 | 0.073 | Single family | Fe | A | 5 | 0.19 | 0.014 |
| 58 | 5.19 | 0.029 | Single family | Fh | A | 5 | 0.19 | 0.006 |
| 59 | 0.04 | 0.000 | Single family | Du | B | 5 | 0.21 | 0.000 |
| 60 | 0.46 | 0.003 | Single family | Fn | B | 5 | 0.21 | 0.001 |
| 61 | 5.18 | 0.029 | Single family | Ft | B | 5 | 0.21 | 0.006 |
| 62 | 5.10 | 0.029 | Single family | Gc | B | 5 | 0.21 | 0.006 |
| 63 | 2.52 | 0.014 | Single family | Cc | B | 12 | 0.26 | 0.004 |
| 64 | 17.29 | 0.097 | Single family | Cf | B | 12 | 0.26 | 0.025 |
| 65 | 18.36 | 0.103 | Single family | Fd | B | 12 | 0.26 | 0.027 |
| 66 | 0.21 | 0.001 | Single family | Dt | C | 12 | 0.31 | 0.000 |
| 67 | 1.45 | 0.008 | Single family | Fm | C | 12 | 0.31 | 0.003 |
| 68 | 6.70 | 0.037 | Single family | Fs | C | 12 | 0.31 | 0.012 |
| 69 | 3.15 | 0.018 | Single family | Fy | D | 12 | 0.35 | 0.006 |
| **Area-weighted coefficient based on land use, soil hydrologic group, and slope** | | | | | | | | **0.32** |

**Table 4.12. The Church watershed high-resolution C coefficient estimation. The watershed was divided into 48 homogeneous plots and assigned C coefficients based on land use, HSG, and slope, and the plots were then area-weighted. The products were summed and yielded a composite C = 0.33.**

| *Plot* | *Area Acres* | *Area–weight* | *Land Use* | *Soil Type* | *HSG* | *Slope (%)* | *C Coefficient* | *$Ai/A_T \times Ci$* |
|---|---|---|---|---|---|---|---|---|
| 1 | 0.76 | 0.007 | Commercial | Ea | A | 0 | 0.71 | 0.005 |
| 2 | 0.76 | 0.007 | Forest | Ec | A | 2 | 0.05 | 0.000 |
| 3 | 1.52 | 0.014 | Forest | Gd | A | 2 | 0.05 | 0.001 |
| 4 | 0.76 | 0.007 | Forest | Cg | A | 5 | 0.05 | 0.000 |
| 5 | 0.76 | 0.007 | Forest | Cd | B | 5 | 0.08 | 0.001 |
| 6 | 0.76 | 0.007 | Forest | Fg | B | 12 | 0.19 | 0.001 |
| 7 | 0.76 | 0.007 | Forest | Dx | D | 12 | 0.28 | 0.002 |
| 8 | 0.76 | 0.007 | Forest | Fv | D | 12 | 0.28 | 0.002 |
| 9 | 0.76 | 0.007 | Multi-family | Ea | A | 0 | 0.47 | 0.003 |
| 10 | 0.76 | 0.007 | Multi-family | Cg | A | 5 | 0.49 | 0.003 |
| 11 | 0.76 | 0.007 | Multi-family | Cd | B | 5 | 0.5 | 0.004 |
| 12 | 0.76 | 0.007 | Multi-family | Fs | C | 12 | 0.54 | 0.004 |
| 13 | 0.76 | 0.007 | Multi-family | Do | D | 12 | 0.56 | 0.004 |
| 14 | 3.79 | 0.035 | Roads | Ea | A | 0 | 0.57 | 0.020 |
| 15 | 0.76 | 0.007 | Roads | Ec | A | 2 | 0.57 | 0.004 |
| 16 | 0.19 | 0.002 | Roads | Gd | A | 2 | 0.57 | 0.001 |
| 17 | 0.38 | 0.004 | Roads | Cg | A | 5 | 0.59 | 0.002 |
| 18 | 0.76 | 0.007 | Roads | Cd | B | 5 | 0.6 | 0.004 |
| 19 | 0.76 | 0.007 | Roads | Du | B | 5 | 0.6 | 0.004 |
| 20 | 0.38 | 0.004 | Roads | Ft | B | 5 | 0.6 | 0.002 |
| 21 | 1.52 | 0.014 | Roads | Fw | B | 5 | 0.6 | 0.009 |
| 22 | 0.38 | 0.004 | Roads | Fs | C | 12 | 0.63 | 0.002 |
| 23 | 0.38 | 0.004 | Roads | Dx | D | 12 | 0.64 | 0.002 |
| 24 | 0.19 | 0.002 | Roads | Fv | D | 12 | 0.64 | 0.001 |
| 25 | 4.55 | 0.043 | Schools/church | Ea | A | 0 | 0.67 | 0.029 |
| 26 | 0.38 | 0.004 | Schools/church | Ec | A | 2 | 0.67 | 0.002 |
| 27 | 1.52 | 0.014 | Schools/church | Gd | A | 2 | 0.67 | 0.010 |
| 28 | 1.52 | 0.014 | Schools/church | Cg | A | 5 | 0.68 | 0.010 |
| 29 | 0.76 | 0.007 | Schools/church | Ft | B | 5 | 0.68 | 0.005 |
| 30 | 1.52 | 0.014 | Schools/church | Fw | B | 5 | 0.68 | 0.010 |
| 31 | 0.76 | 0.007 | Schools/church | Dp | C | 5 | 0.69 | 0.005 |
| 32 | 0.76 | 0.007 | Schools/church | Fv | D | 12 | 0.7 | 0.005 |
| 33 | 3.79 | 0.035 | Single family | Ea | A | 0 | 0.14 | 0.005 |
| 34 | 0.76 | 0.007 | Single family | Dn | A | 2 | 0.14 | 0.001 |
| 35 | 6.07 | 0.057 | Single family | Ec | A | 2 | 0.14 | 0.008 |
| 36 | 3.79 | 0.035 | Single family | Gd | A | 2 | 0.14 | 0.005 |
| 37 | 1.52 | 0.014 | Single family | Cg | A | 5 | 0.19 | 0.003 |
| 38 | 3.04 | 0.028 | Single family | Cd | B | 5 | 0.21 | 0.006 |
| 39 | 11.38 | 0.106 | Single family | Du | B | 5 | 0.21 | 0.022 |
| 40 | 5.31 | 0.050 | Single family | Ft | B | 5 | 0.21 | 0.010 |
| 41 | 11.38 | 0.106 | Single family | Fw | B | 5 | 0.21 | 0.022 |
| 42 | 0.76 | 0.007 | Single family | Fg | B | 12 | 0.26 | 0.002 |
| 43 | 0.76 | 0.007 | Single family | Dp | C | 5 | 0.25 | 0.002 |
| 44 | 4.55 | 0.043 | Single family | Fs | C | 12 | 0.31 | 0.013 |
| 45 | 6.83 | 0.064 | Single family | Do | D | 12 | 0.35 | 0.022 |
| 46 | 3.04 | 0.028 | Single family | Dx | D | 12 | 0.35 | 0.010 |
| 47 | 6.07 | 0.057 | Single family | Fv | D | 12 | 0.35 | 0.020 |
| 48 | 6.07 | 0.057 | Single family | Gf | D | 15 | 0.35 | 0.020 |
| **Area-weighted coefficient based on land use, HSG, and slope** | | | | | | | | **0.33** |

## Times of Concentration

Once the C coefficients were established, three times of concentration (crude, intermediate, and detailed) were estimated for each watershed. The aim was for the successive estimates to represent progressively more detailed assessments of the factors that determine the $T_c$. Each succeeding estimate also represented an increase in time, effort and complexity. Choosing the point from which to calculate the $T_c$ was somewhat arbitrary. Basically, the point on each watershed that was the furthest physical distance and where the runoff had to flow overland was designated as the "most hydrologically remote" point.

### Crude Time of Concentration

This first method estimation was the most simple to perform. In this method the runoff flow was divided into two flow sections: overland flow and channel flow. The hydraulic travel times for the sections were estimated using Kerby's formula for overland flow and Kirpich's formula for channel flow. The estimated times were then added to get the total Tc. Kerby's formula for overland flow is:

$$T_c = 0.83\ L_o\ 2\ n^{0.467}\ S_o^{-0.235}$$

Where:
$T_c$ = Time (min.)
n = Manning's roughness coefficient
$L_o$ = Overland flow length (ft)
$S_o$ = Slope length (ft/ft)

Kirpich's Tennessee formula for channel flow is:

$$T_c = 0.0078\ L_c^{0.77}\ S_c^{-0.385}$$

Where:
$T_c$ = Time (min)
$L_c$ = Hydraulic length (ft)
$S_c$ = Mean slope (ft/ft)

To derive the length and slope information the USGS Fountain City 7.5-minute 1:24,000 quadrangle map was used. After determining the most hydrologically remote point, the most probable flow path was drawn on the map. The overland flow length was set at a standard 300 feet for both watersheds. The slope for the overland flow section was determined by measuring the elevation drop over the first 300 feet of the flow path. The roughness coefficient used was for average grass.

Channel flow was dealt with similarly. The channel flow path was measured in about three segments. The elevation change over the entire distance was then noted and used to calculate the channel slope. Table 4.13 shows the crude $T_c$ variables and results for each watershed. Rainfall intensity for the $T_c$ periods were interpolated from the observed rainfall data for each storm event.

This technique required only a 7.5-minute quadrangle map and about ½ hour to estimate the $T_c$ for both watersheds.

### Intermediate Time of Concentration

The Fountain City quadrangle was the primary information source for estimating the intermediate $T_c$. Drawing the assumed flow path on the map was the first step. Next, the flow paths were segmented by major changes in slope or flow type. After that, the segments' length and slope were measured and recorded. Then, each segment was designated as sheet, shallow-concentrated, or open-channel flow.

Once the flow length had been divided and measured, Tc was calculated for each segment. The open-channel flow portions were estimated using Kirpich's equation. Sheet flow travel time was estimated using the kinematic wave formula (Overton and Meadows, 1976): $T_c = 0.928\ (n\ l_s)^{0.8} / (i^{0.4}\ S_s^{0.3})$

Where: $T_c$ = Time (min)
$n$ = Manning's roughness
$l_s$ = Sheet flow length (ft)
$i$ = Rainfall intensity (in/hr)
$S_s$ = Mean slope (ft/ft)

Shallow concentrated flow travel time was estimated by calculating the sheet flow velocity using Manning's formula and then dividing then this figure into the segment length. A standard (USDA, 1986) roughness coefficient ($n = 0.05$) and hydraulic radius ($r = 0.4$) for shallow concentrated flow were used in Manning's formula.

**Table 4.13. The watershed characteristics used to estimate the crude $T_c$ for the Sanford and Church watersheds. The time estimated from Kerby's overland flow formula were added to the results from Kirpich's channel flow formula to determine the crude $T_c$ values.**

| *Overland Flow* | Sanford | Church |
|---|---|---|
| **Flow length (l)** | 91m (300 ft) | 91m (300 ft) |
| **Starting elevation** | 396 m (1300 ft) | 363 m (1190 ft) |
| **Ending elevation** | 378 m (1240 ft) | 354m (1160 ft) |
| **Elevation** | 18 m (60 ft) | 9 m (30 ft) |
| **Slope(s) (m/m)** | 0.20 | 0.10 |
| **Roughness (n)** | 0.25 | 0.25 |
| **Overland $T_c$ (min)** | 9.1 | 10.7 |
| *Channel Flow* | | |
| **Flow length (l)** | 1372 m (4500 ft) | 1280 m (4200 ft) |
| **Starting elevation** | 378 m (1240 ft) | 354 m (1160 ft) |
| **Ending elevation** | 326.14 m (1070 ft) | 314 m (1030 ft) |
| **Elevation** | 51.82 m (170 ft) | 40 m (130 ft) |
| **Slope(s) (m/m)** | 0.038 | .031 |
| **Channel $T_c$ (min)** | 17.9 | 18.3 |
| **Total $T_c$ (min.)** | **27** | **29** |

Variable $T_C$ and Rainfall Intensity

It should be noted that the $T_c$ varies inversely with the rainfall intensity in a nonlinear manner, and that Tables 4.14 and 4.15 show the $T_c$ when the rainfall intensity is one inch per hour. Since the Rational Method estimates require average rainfall intensity for a period equal to the $T_c$, a polynomial regression equation relating the rainfall intensity to the $T_c$ was calculated for each watershed. This was needed in order to use an appropriate observed intensity when making Rational Method estimates. The following equations relate intensity to the intermediate $T_c$ for the watersheds (see Appendix A).

Sanford intermediate $T_c = 49.10\ i^{-0.168}$ $(R^2 = 0.97)$

Church intermediate $T_c = 43.02\ i^{-0.149}$ $(R^2 = 0.97)$

The equations were then used in an iterative approach to establish the a posteriori rainfall intensity for use in the Rational Method. The iterative approach was as follows:

1. Using the rainfall intensities in Tables 4.1 or 4.2, make a "best guess" of i (typically the 60-minute intensity worked well as an initial approximation).

2. Insert i into the appropriate regression formula.

3. Iterate the process using the observed i for the Tc indicated in step 2. In all cases, two iterations were sufficient to converge the $T_c$ and intensity.

Estimating the intermediate $T_c$ required a 7.5-minute quadrangle map, general knowledge of the land cover, and approximately one hour of time for the combined watersheds.

Detailed Time of Concentration

The detailed $T_c$ was estimated using information gathered from maps and field surveys. After delineating the flow path on a map, a field survey was made by walking the $T_c$ path. During the survey the channel types, sizes, lengths, and the high and low elevations of each segment were noted. Figures 4.5 and 4.6 show the flow paths and data collected for each watershed.

**Table 4.14. The flow-path segments used to estimate the Sanford intermediate Tc.**

| *Segment #* | *Length (ft)* | *Δ elevation (ft)* | *Slope (ft/ft)* | *Primary Flow (type)* | *$T_c$ (min)* |
|---|---|---|---|---|---|
| 1 | 200 | 60 | 0.3 | Sheet | 18.46 |
| 2 | 200 | 40 | 0.2 | Concentrated | 0.46 |
| 3 | 200 | 19 | 0.095 | Open Channel | 1.14 |
| 4 | 200 | 10 | 0.05 | Open Channel | 1.46 |
| 5 | 200 | 11 | 0.055 | Open Channel | 1.41 |
| 6 | 200 | 15 | 0.075 | Open Channel | 1.25 |
| 7 | 200 | 5 | 0.025 | Open Channel | 1.91 |
| 8 | 200 | 5 | 0.025 | Open Channel | 1.91 |
| 9 | 200 | 5 | 0.025 | Open Channel | 1.91 |
| 10 | 200 | 5 | 0.025 | Open Channel | 1.91 |
| 11 | 200 | 5 | 0.025 | Open Channel | 1.91 |
| 12 | 200 | 20 | 0.1 | Open Channel | 1.12 |
| 13 | 200 | 5 | 0.025 | Open Channel | 1.91 |
| 14 | 200 | 5 | 0.025 | Open Channel | 1.91 |
| 15 | 200 | 5 | 0.025 | Open Channel | 1.91 |
| 16 | 200 | 5 | 0.025 | Open Channel | 1.91 |
| 17 | 200 | 5 | 0.025 | Open Channel | 1.91 |
| 18 | 200 | 5 | 0.025 | Open Channel | 1.91 |
| Totals | 3600 | 230 | 0.06 | | 46 |

Data source: USGS Fountain City 7.5 minute Quadrangle (1:24,000 scale)

Notes:

Segment 1 calculated using kinematic wave formula with roughness = 0.4 and i = 1.0 in/hr.

Segment 2 calculated using Manning's formula with roughness = 0.05 and hydraulic radius = 0.4 (NRCS standards).

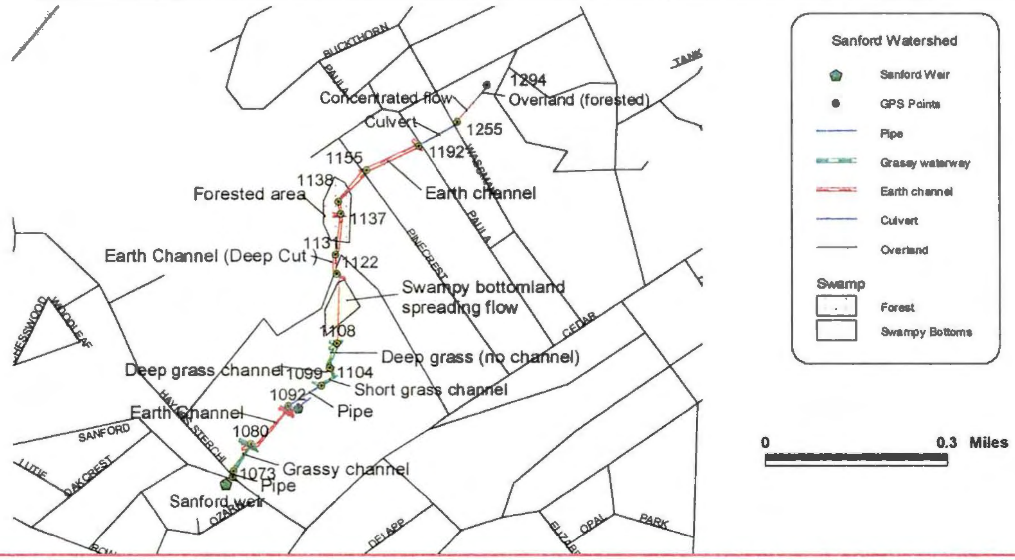
Open channel segments calculated using Kirpich's formula.

**Table 4.15. The flow-path segments used to estimate the Church intermediate Tc.**

| *Segment* # | *Length (ft)* | *Δ elevation (ft)* | *Slope (ft/ft)* | *Primary Flow (type)* | *$T_c$ (min)* |
|---|---|---|---|---|---|
| 1 | 300 | 30 | 0.100 | Sheet | 14.25 |
| 2 | 600 | 30 | 0.050 | Concentrated | 2.77 |
| 3 | 600 | 30 | 0.050 | Open Channel | 3.41 |
| 4 | 600 | 10 | 0.017 | Open Channel | 5.20 |
| 5 | 600 | 30 | 0.050 | Open Channel | 3.41 |
| 6 | 600 | 20 | 0.033 | Open Channel | 3.98 |
| 7 | 600 | 8 | 0.013 | Open Channel | 5.66 |
| 8 | 150 | 2 | 0.013 | Open Channel | 1.95 |
| **Totals** | 4050 | 160 | 0.040 | | 41 |

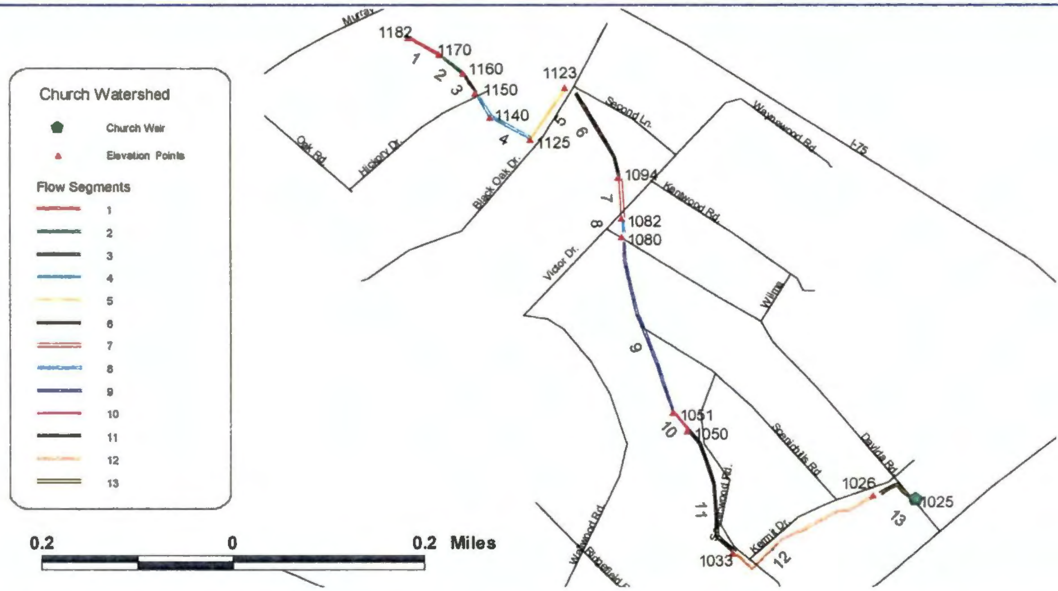
Data source: USGS Fountain City 7.5 minute Quadrangle (1:24,000 scale)

Notes:
Segment 1 calculated using kinematic wave formula with roughness = 0.1 and i = 1.0 in/hr.
Segment 2 calculated using Manning's formula with roughness = 0.05 and hydraulic radius = 0.4 (NRCS standards).
Open channel segments calculated using Kirpich's formula.




| Segment # | Primary Flow (type) | Depth (ft) | Width (ft) | Channel shape | Side slope | Area ($ft^2$) | $P_w$ (ft) | r (ft) | Channel Material | n | Length (ft) | Up elev. (ft) | Low elev. (ft) | Δ elev. (ft) | Slope (ft/ft) | Velocity (ft/s) | Time (min) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Overland | | | | | | | | litter | 0.250 | 200 | 1294 | 1275 | 19 | 0.095 | 1.0 | 15.6 |
| 2 | Concentrated | | | | | | | | litter | 0.250 | 215 | 1275 | 1255 | 20 | 0.093 | 5.0 | 0.7 |
| 3 | Culvert | 0.67 | 2.00 | parab | na | 0.89 | 2.59 | 0.34 | concrete | 0.015 | 400 | 1255 | 1192 | 63 | 0.158 | 19.3 | 0.3 |
| 4 | Earth channel | 1.50 | 1.50 | straight | 1 | 4.50 | 5.74 | 0.78 | packed earth | 0.040 | 500 | 1192 | 1155 | 37 | 0.074 | 8.6 | 1.0 |
| 5 | Earth channel | 1.50 | 1.50 | straight | 1 | 4.50 | 5.74 | 0.78 | packed earth | 0.040 | 365 | 1155 | 1138 | 17 | 0.047 | 6.8 | 0.9 |
| 6 | Earth channel | 1.67 | 1.67 | straight | 1 | 5.56 | 6.38 | 0.87 | packed earth | 0.040 | 115 | 1138 | 1137 | 1 | 0.009 | 3.2 | 0.6 |
| 7 | Earth channel | 2.50 | 2.50 | straight | 1 | 12.50 | 9.57 | 1.31 | packed earth | 0.040 | 365 | 1137 | 1131 | 6 | 0.016 | 5.7 | 1.1 |
| 8 | Earth channel | 5.83 | 5.83 | straight | 1 | 68.06 | 22.33 | 3.05 | packed earth | 0.037 | 175 | 1131 | 1122 | 9 | 0.051 | 19.2 | 0.2 |
| 9 | Earth channel | 0.67 | 1.25 | meander | 1 | 1.28 | 3.14 | 0.41 | packed earth | 0.100 | 650 | 1122 | 1108 | 14 | 0.022 | 1.2 | 9.0 |
| 10 | Grass channel | 1.17 | 0.75 | trap | 1 | 2.24 | 4.05 | 0.55 | long | 0.100 | 230 | 1108 | 1105 | 3 | 0.013 | 1.1 | 3.3 |
| 11 | Grass channel | 1.00 | 0.50 | trap | 1 | 1.50 | 3.33 | 0.45 | med length | 0.060 | 25 | 1105 | 1104 | 1 | 0.040 | 2.9 | 0.1 |
| 12 | Grass channel | 1.83 | 1.67 | trap | 1 | 6.42 | 6.85 | 0.94 | short | 0.045 | 220 | 1104 | 1099 | 5 | 0.023 | 4.8 | 0.8 |
| 13 | Pipe | 2.00 | 2.00 | round | na | 3.14 | 2.09 | 1.50 | Corr metal | 0.025 | 350 | 1099 | 1092 | 7 | 0.020 | 11.0 | 0.5 |
| 14 | Earth channel | 2.08 | 1.33 | straight | 1 | 7.12 | 7.23 | 0.99 | loose earth | 0.040 | 575 | 1092 | 1080 | 12 | 0.021 | 5.3 | 1.8 |
| 15 | Grass channel | 3.50 | 3.83 | trap | 1 | 25.67 | 13.73 | 1.87 | med length | 0.055 | 275 | 1080 | 1073 | 7 | 0.025 | 6.6 | 0.7 |
| 16 | Pipe | 3.83 | 3.83 | round | na | 11.54 | 12.34 | 0.93 | Concrete | 0.014 | 160 | 1073 | 1070 | 3 | 0.019 | 13.9 | 0.2 |
| Totals | | | | | | | | | | | 4820 | | | 224 | | | 37 |

**Figure 4.5. Map and associated data evaluated for the flow-path segments during the detailed $T_c$ estimate on the Sanford watershed. Segment 1 $T_c$ was calculated using the kinematic wave equation ($T_c$ shown assumes i =1 in/hr). Travel times for the remaining segments were calculated using Manning's velocity equation. Full or very nearly full flows were assumed in all calculations.**




| Segment # | Primary Flow (type) | Depth (ft) | Width (ft) | Channel shape | Side slope | Area (ft2) | Pw (ft) | r (ft) | Channel material | n | Length (ft) | High elev. (ft) | Low elev. (ft) | Δ elev. (ft) | Slope (ft/ft) | Velocity (ft/s) | Time (min) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | Overland | na | na | na | na | na | na | na | thick grass | 0.125 | 250 | 1182 | 1170 | 12.0 | 0.048 | na | 13.5 |
| 2 | Concentrated | 0.40 | na | na | na | na | na | 0.4 | litter | 0.050 | 170 | 1170 | 1160 | 10.0 | 0.059 | 3.9 | 0.7 |
| 3 | Culvert | 1.00 | 1.00 | rectangular | na | 1.0 | 3.0 | 0.3 | earth | 0.032 | 125 | 1160 | 1150 | 10.0 | 0.080 | 6.2 | 0.3 |
| 4 | Pipe | 1.00 | na | round | na | 0.8 | 3.1 | 0.2 | concrete | 0.014 | 400 | 1150 | 1125 | 25.0 | 0.063 | 10.3 | 0.6 |
| 5 | Pipe | 1.50 | na | round | na | 1.7 | 4.7 | 0.4 | concrete | 0.014 | 350 | 1125 | 1123 | 2.0 | 0.006 | 4.1 | 1.4 |
| 6 | Earth channel | 1.50 | 2.00 | trapizodial | 1:1 | 5.3 | 6.2 | 0.8 | packed earth | 0.040 | 550 | 1123 | 1094 | 29.0 | 0.053 | 7.6 | 1.2 |
| 7 | Brick culvert | 3.00 | 3.00 | parabola | na | 6.0 | 11.0 | 0.5 | brick | 0.025 | 225 | 1094 | 1082 | 12.0 | 0.053 | 9.2 | 0.4 |
| 8 | Concentrated | 1.00 | na | spreading | na | na | na | 1.3 | short grass | 0.100 | 100 | 1082 | 1080 | 2.0 | 0.020 | 2.5 | 0.7 |
| 9 | Earth channel | 1.50 | 3.00 | trapizodial | 3:1 | 11.3 | 9.0 | 1.3 | packed earth | 0.037 | 1000 | 1080 | 1050 | 30.0 | 0.030 | 8.1 | 2.1 |
| 10 | Concentrated | 0.40 | na | spreading | na | na | na | 0.4 | high grass | 0.200 | 175 | 1050 | 1050 | 0.5 | 0.003 | 0.2 | 13.5 |
| 11 | Earth channel | 2.00 | 4.00 | trapizodial | 2:1 | 16.0 | 10.9 | 1.5 | packed earth | 0.040 | 775 | 1050 | 1033 | 17.0 | 0.022 | 7.1 | 1.8 |
| 12 | Pipe | 4.25 | na | round | na | 14.2 | 13.3 | 1.1 | corr metal | 0.014 | 950 | 1033 | 1026 | 6.5 | 0.007 | 9.2 | 1.7 |
| 13 | Earth channel | 2.25 | 14.50 | trapizodial | 2:1 | 217.0 | 40.3 | 1.6 | weeds/stones | 0.022 | 210 | 1026 | 1025 | 1.0 | 0.005 | 6.5 | 0.5 |
| *Totals* | | | | | | | | | | | ***5280*** | | | ***157*** | ***0.03*** | | ***38*** |

**Figure 4.6. Map and associated data evaluated for the flow-path segments during the detailed $T_c$ estimate on the Church watershed. Segment 1 $T_c$ was calculated using the kinematic wave equation ($T_c$ shown assumes i =1 in/hr). Travel times for the remaining segments were calculated using Manning's velocity equation. Full or very nearly full flows were assumed in all calculations.**

Once the channel information had been collected, the time estimates were calculated using the kinematic wave equation for overland flow and Manning's formula for concentrated, open-channel, and pipe flow. Each segment was assumed to be flowing full or nearly full. The individual times were then summed to get the total $T_c$. Estimates using several rainfall intensities were determined and a polynomial regression equation was determined for each watershed.

Sanford detailed $T_c = 39.08\ i^{-0.177}$ $(R^2 = 0.98)$

Church detailed $T_c = 40.71\ i^{-0.149}$ $(R^2 = 0.97)$

Appendix A contains the regression data for the watersheds. As in the intermediate $T_c$ section, an iterative approach was used to determine rainfall intensities from the regression equations.

Completing this estimation technique took considerably more time and effort than the previous methods. The actual field survey took roughly three hours per watershed. This time included walking the flow path and evaluating the channel characteristics. Determining the channel lengths and elevations took an additional two—three hours per watershed. Applying the formulas and calculating the $T_c$ required approximately one hour per watershed. Total time required for the detailed $T_c$ estimates was approximately six hours per watershed.

### Estimating Peak Flow

The final step in the a priori Rational Methods estimations was to apply the parameters to the equation. The three estimated C coefficients were cross-classified with recorded rainfall intensities observed for the periods estimated by the three $T_c$ parameters. This yielded nine peak flow estimates for each recorded storm event. Figure 4.7 illustrates the cross-classification process.

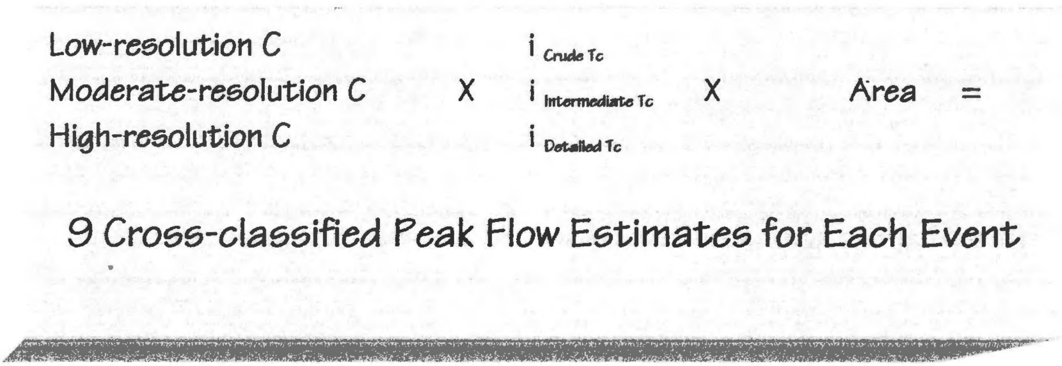



**Figure 4.7. For each storm event, the a priori parameters, measured rainfall intensities, and watershed areas were inserted into the Rational Method to estimate peak flow rates on both watersheds. The multiple estimations allowed the parameter resolutions and selection techniques to be tested and optimized.**

## SCS CN METHOD

To evaluate whether increasing the parameter resolution increased estimation accuracy, six volume estimates were estimated for each observed event. These estimations were based on varying-resolution CN parameters that were estimated with and without considering antecedent moisture.

### Curve Numbers

Three progressively higher-resolution curve numbers were established for each watershed using previous watershed characterizations.

Low-resolution Curve Number

The first curve numbers were based on the single dominant land use determined in the low-resolution characterizations. A chart from the SCS (1985) was used to relate land use to a curve number. Table 4.16 shows the results.

**Table 4.16. Low-resolution CN.**

| | AMC I | AMC II | AMC III |
|---|---|---|---|
| **Sanford** | 53 | 72 | 86 |
| **Church** | 53 | 72 | 86 |

### Moderate-resolution Curve Number

The land use and soils information contained in Table 4.6 was reviewed in selecting a curve number for each watershed. Since there were multiple land uses, a composite curve number was calculated for each watershed on an area-weighted basis. The curve number for each land use was selected using the charts from the SCS (1985) for AMC II. Once this was accomplished, another chart was used to obtain the curve numbers for AMC I and III (SCS, 1985). The CN parameters for AMC II are as follow:

Sanford Watershed

$CN_{AMCII} = 9/179 \times 60 + 170/179 \times 70$

$CN_{AMCII} = 66$

Church Watershed

$CN_{AMCII} = 14.3/107 \times 60 + 92.7/107 \times 70$

$CN_{AMCII} = 69$

### High-resolution Curve Number

The third Curve number used the land use, slope, and runoff potential information from the high-resolution characterizations. Using those data, each watershed was divided into separate plots. For each plot, the corresponding area, percent of total area, land use, soil series, and hydrologic group were recorded. A curve number was then assigned to each plot based on AMC II. Each curve number was then multiplied by the percent of total area to get an area-weighted CN. These were summed to get the curve number for each watershed. The SCS table was used to obtain the CN for AMC I and III. Table 4.17 and 4.18 shows the resultant data for the Sanford and Church watersheds respectively.

Table 4.17. The Sanford watershed high-resolution CN.

| Plot | Area (A) | Area–Weight | Land Use | Soil Type | HSG | CN AMC I | CN AMC II | CN AMC III | Ai/AT x CNi AMC I | Ai/AT x CNi AMC II | Ai/AT x CNi AMC III |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 1.11 | 0.006 | Forest | Gd | A | 19 | 36 | 56 | 0.118 | 0.224 | 0.348 |
| 2 | 0.46 | 0.003 | Forest | Cd | A | 19 | 36 | 56 | 0.049 | 0.093 | 0.145 |
| 3 | 0.38 | 0.002 | Forest | Cg | A | 19 | 36 | 56 | 0.041 | 0.077 | 0.120 |
| 4 | 0.94 | 0.005 | Forest | Fe | A | 19 | 36 | 56 | 0.100 | 0.190 | 0.295 |
| 5 | 1.32 | 0.007 | Forest | Ft | B | 40 | 60 | 78 | 0.296 | 0.444 | 0.577 |
| 6 | 0.36 | 0.002 | Forest | Gc | B | 40 | 60 | 78 | 0.079 | 0.119 | 0.155 |
| 7 | 0.26 | 0.001 | Forest | Cf | B | 40 | 60 | 78 | 0.058 | 0.086 | 0.112 |
| 8 | 0.55 | 0.003 | Forest | Fd | B | 40 | 60 | 78 | 0.123 | 0.185 | 0.240 |
| 9 | 0.03 | 0.000 | Forest | Fm | C | 54 | 73 | 87 | 0.008 | 0.011 | 0.013 |
| 10 | 1.24 | 0.007 | Forest | Fs | C | 54 | 73 | 87 | 0.376 | 0.508 | 0.605 |
| 11 | 0.61 | 0.003 | Forest | Fy | D | 62 | 79 | 91 | 0.213 | 0.271 | 0.312 |
| 12 | 0.25 | 0.001 | Multi-family | Ec | A | 64 | 81 | 92 | 0.089 | 0.113 | 0.128 |
| 13 | 1.89 | 0.011 | Multi-family | Gd | A | 64 | 81 | 92 | 0.677 | 0.857 | 0.973 |
| 14 | 3.55 | 0.020 | Multi-family | Du | B | 75 | 88 | 95 | 1.487 | 1.744 | 1.883 |
| 15 | 4.96 | 0.028 | Multi-family | Ft | B | 75 | 88 | 95 | 2.079 | 2.439 | 2.633 |
| 16 | 0.03 | 0.000 | Multi-family | Fd | B | 75 | 88 | 95 | 0.011 | 0.013 | 0.014 |
| 17 | 1.46 | 0.008 | Multi-family | Dt | C | 80 | 91 | 97 | 0.652 | 0.741 | 0.790 |
| 18 | 1.54 | 0.009 | Multi-family | Fs | C | 80 | 91 | 97 | 0.688 | 0.782 | 0.834 |
| 19 | 0.01 | 0.000 | Multi-family | Fy | D | 83 | 93 | 98 | 0.004 | 0.005 | 0.005 |
| 20 | 1.53 | 0.009 | Roads | Gd | A | 94 | 98 | 99 | 0.803 | 0.837 | 0.846 |
| 21 | 0.68 | 0.004 | Roads | Cd | A | 94 | 98 | 99 | 0.355 | 0.370 | 0.374 |
| 22 | 1.33 | 0.007 | Roads | Cg | A | 94 | 98 | 99 | 0.700 | 0.730 | 0.738 |
| 23 | 1.93 | 0.011 | Roads | Fe | A | 94 | 98 | 99 | 1.013 | 1.056 | 1.067 |
| 24 | 0.46 | 0.003 | Roads | Fh | A | 94 | 98 | 99 | 0.243 | 0.253 | 0.256 |
| 25 | 0.08 | 0.000 | Roads | Du | B | 94 | 98 | 99 | 0.042 | 0.044 | 0.044 |
| 26 | 0.07 | 0.000 | Roads | Fn | B | 94 | 98 | 99 | 0.037 | 0.039 | 0.039 |
| 27 | 0.57 | 0.003 | Roads | Ft | B | 94 | 98 | 99 | 0.299 | 0.312 | 0.315 |
| 28 | 0.67 | 0.004 | Roads | Gc | B | 94 | 98 | 99 | 0.350 | 0.365 | 0.369 |
| 29 | 0.17 | 0.001 | Roads | Cc | B | 94 | 98 | 99 | 0.089 | 0.092 | 0.093 |
| 30 | 2.53 | 0.014 | Roads | Cf | B | 94 | 98 | 99 | 1.331 | 1.387 | 1.402 |
| 31 | 3.11 | 0.017 | Roads | Fd | B | 94 | 98 | 99 | 1.634 | 1.704 | 1.721 |
| 32 | 0.51 | 0.003 | Roads | Fm | C | 94 | 98 | 99 | 0.266 | 0.277 | 0.280 |
| 33 | 1.68 | 0.009 | Roads | Fs | C | 94 | 98 | 99 | 0.883 | 0.920 | 0.929 |
| 34 | 0.39 | 0.002 | Roads | Fy | D | 94 | 98 | 99 | 0.205 | 0.214 | 0.216 |
| 35 | 0.07 | 0.000 | Rural Residential | Ec | A | 31 | 51 | 70 | 0.012 | 0.020 | 0.028 |
| 36 | 4.21 | 0.023 | Rural Residential | Gd | A | 31 | 51 | 70 | 0.728 | 1.198 | 1.645 |
| 37 | 4.93 | 0.028 | Rural Residential | Fe | A | 31 | 51 | 70 | 0.855 | 1.406 | 1.930 |
| 38 | 2.06 | 0.012 | Rural Residential | Du | B | 48 | 68 | 84 | 0.553 | 0.784 | 0.968 |
| 39 | 3.02 | 0.017 | Rural Residential | Ft | B | 48 | 68 | 84 | 0.811 | 1.148 | 1.419 |
| 40 | 3.75 | 0.021 | Rural Residential | Fd | B | 48 | 68 | 84 | 1.006 | 1.425 | 1.761 |
| 41 | 1.73 | 0.010 | Rural Residential | Dt | C | 62 | 79 | 91 | 0.597 | 0.761 | 0.877 |
| 42 | 1.48 | 0.008 | Rural Residential | Fs | C | 62 | 79 | 91 | 0.511 | 0.651 | 0.750 |
| 43 | 0.93 | 0.005 | Rural Residential | Fy | D | 68 | 84 | 93 | 0.355 | 0.438 | 0.485 |
| 44 | 0.02 | 0.000 | School/Church | Gd | A | 64 | 81 | 92 | 0.006 | 0.008 | 0.009 |
| 45 | 3.69 | 0.021 | School/Church | Cd | A | 64 | 81 | 92 | 1.319 | 1.670 | 1.897 |
| 46 | 0.06 | 0.000 | School/Church | Cg | A | 64 | 81 | 92 | 0.022 | 0.028 | 0.032 |
| 47 | 0.70 | 0.004 | School/Church | Fe | A | 64 | 81 | 92 | 0.251 | 0.318 | 0.361 |
| 48 | 2.66 | 0.015 | School/Church | Ft | B | 75 | 88 | 95 | 1.114 | 1.307 | 1.411 |
| 49 | 2.45 | 0.014 | School/Church | Gc | B | 75 | 88 | 95 | 1.028 | 1.207 | 1.302 |
| 50 | 1.76 | 0.010 | School/Church | Cc | B | 75 | 88 | 95 | 0.738 | 0.866 | 0.934 |
| 51 | 0.74 | 0.004 | School/Church | Fd | B | 75 | 88 | 95 | 0.309 | 0.363 | 0.392 |
| 52 | 3.62 | 0.020 | School/Church | Fs | C | 80 | 91 | 97 | 1.617 | 1.840 | 1.961 |
| 53 | 0.11 | 0.001 | School/Church | Fy | D | 83 | 93 | 98 | 0.049 | 0.055 | 0.058 |
| 54 | 12.72 | 0.071 | Single Family | Gd | A | 37 | 57 | 75 | 2.630 | 4.052 | 5.331 |
| 55 | 4.35 | 0.024 | Single Family | Cd | A | 37 | 57 | 75 | 0.899 | 1.385 | 1.822 |
| 56 | 9.11 | 0.051 | Single Family | Cg | A | 37 | 57 | 75 | 1.884 | 2.902 | 3.819 |
| 57 | 13.00 | 0.073 | Single Family | Fe | A | 37 | 57 | 75 | 2.687 | 4.140 | 5.447 |
| 58 | 5.19 | 0.029 | Single Family | Fh | A | 37 | 57 | 75 | 1.073 | 1.654 | 2.176 |
| 59 | 0.04 | 0.000 | Single Family | Du | B | 53 | 72 | 86 | 0.011 | 0.014 | 0.017 |
| 60 | 0.46 | 0.003 | Single Family | Fn | B | 53 | 72 | 86 | 0.137 | 0.186 | 0.222 |
| 61 | 5.18 | 0.029 | Single Family | Ft | B | 53 | 72 | 86 | 1.535 | 2.085 | 2.491 |
| 62 | 5.10 | 0.029 | Single Family | Gc | B | 53 | 72 | 86 | 1.511 | 2.053 | 2.452 |
| 63 | 2.52 | 0.014 | Single Family | Cc | B | 53 | 72 | 86 | 0.745 | 1.012 | 1.209 |
| 64 | 17.29 | 0.097 | Single Family | Cf | B | 53 | 72 | 86 | 5.121 | 6.957 | 8.309 |
| 65 | 18.36 | 0.103 | Single Family | Fd | B | 53 | 72 | 86 | 5.437 | 7.386 | 8.822 |
| 66 | 0.21 | 0.001 | Single Family | Dt | C | 64 | 81 | 92 | 0.076 | 0.097 | 0.110 |
| 67 | 1.45 | 0.008 | Single Family | Fm | C | 64 | 81 | 92 | 0.518 | 0.656 | 0.745 |
| 68 | 6.70 | 0.037 | Single Family | Fs | C | 64 | 81 | 92 | 2.394 | 3.030 | 3.441 |
| 69 | 3.15 | 0.018 | Single Family | Fy | D | 72 | 86 | 94 | 1.266 | 1.512 | 1.653 |
| **Area-weighted curve number based on land use and soil hydrologic group** | | | | | | | | | **55** | **72** | **85** |

**Table 4.18. The Church watershed high-resolution CN.**

| Plot | Area (A) | Area–Weight | Land Use | Soil Type | HSG | CN | | | Ai/AT x CNi | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | AMC I | AMC II | AMC III | AMC I | AMC II | AMC III |
| 1 | 0.8 | 0.007 | Commercial | Ea | A | 81 | 92 | 97 | 0.575 | 0.653 | 0.689 |
| 2 | 0.8 | 0.007 | Forest | Ec | A | 19 | 36 | 56 | 0.135 | 0.255 | 0.397 |
| 3 | 1.5 | 0.014 | Forest | Gd | A | 19 | 36 | 56 | 0.270 | 0.511 | 0.794 |
| 4 | 0.8 | 0.007 | Forest | Cg | A | 19 | 36 | 56 | 0.135 | 0.255 | 0.397 |
| 5 | 0.8 | 0.007 | Forest | Cd | B | 40 | 60 | 78 | 0.284 | 0.426 | 0.553 |
| 6 | 0.8 | 0.007 | Forest | Fg | B | 40 | 60 | 78 | 0.284 | 0.426 | 0.553 |
| 7 | 0.8 | 0.007 | Forest | Dx | D | 63 | 80 | 91 | 0.447 | 0.567 | 0.645 |
| 8 | 0.8 | 0.007 | Forest | Fv | D | 63 | 80 | 91 | 0.447 | 0.567 | 0.645 |
| 9 | 0.8 | 0.007 | Multi-family | Ea | A | 64 | 81 | 92 | 0.454 | 0.574 | 0.653 |
| 10 | 0.8 | 0.007 | Multi-family | Cg | A | 64 | 81 | 92 | 0.454 | 0.574 | 0.653 |
| 11 | 0.8 | 0.007 | Multi-family | Cd | B | 75 | 88 | 95 | 0.532 | 0.624 | 0.674 |
| 12 | 0.8 | 0.007 | Multi-family | Fs | C | 80 | 91 | 97 | 0.567 | 0.645 | 0.688 |
| 13 | 0.8 | 0.007 | Multi-family | Do | D | 83 | 93 | 98 | 0.589 | 0.660 | 0.695 |
| 14 | 3.8 | 0.035 | Roads | Ea | A | 94 | 98 | 99 | 3.333 | 3.475 | 3.511 |
| 15 | 0.8 | 0.007 | Roads | Ec | A | 94 | 98 | 99 | 0.667 | 0.695 | 0.702 |
| 16 | 0.2 | 0.002 | Roads | Gd | A | 94 | 98 | 99 | 0.167 | 0.174 | 0.176 |
| 17 | 0.4 | 0.004 | Roads | Cg | A | 94 | 98 | 99 | 0.333 | 0.348 | 0.351 |
| 18 | 0.8 | 0.007 | Roads | Cd | B | 94 | 98 | 99 | 0.667 | 0.695 | 0.702 |
| 19 | 0.8 | 0.007 | Roads | Du | B | 94 | 98 | 99 | 0.667 | 0.695 | 0.702 |
| 20 | 0.4 | 0.004 | Roads | Ft | B | 94 | 98 | 99 | 0.333 | 0.348 | 0.351 |
| 21 | 1.5 | 0.014 | Roads | Fw | B | 94 | 98 | 99 | 1.333 | 1.390 | 1.404 |
| 22 | 0.4 | 0.004 | Roads | Fs | C | 94 | 98 | 99 | 0.333 | 0.348 | 0.351 |
| 23 | 0.4 | 0.004 | Roads | Dx | D | 94 | 98 | 99 | 0.333 | 0.348 | 0.351 |
| 24 | 0.2 | 0.002 | Roads | Fv | D | 94 | 98 | 99 | 0.167 | 0.174 | 0.176 |
| 25 | 4.6 | 0.043 | School/Church | Ea | A | 64 | 81 | 92 | 2.723 | 3.447 | 3.915 |
| 26 | 0.4 | 0.004 | School/Church | Ec | A | 64 | 81 | 92 | 0.227 | 0.287 | 0.326 |
| 27 | 1.5 | 0.014 | School/Church | Gd | A | 64 | 81 | 92 | 0.908 | 1.149 | 1.305 |
| 28 | 1.5 | 0.014 | School/Church | Cg | A | 64 | 81 | 92 | 0.908 | 1.149 | 1.305 |
| 29 | 0.8 | 0.007 | School/Church | Ft | B | 75 | 88 | 95 | 0.532 | 0.624 | 0.674 |
| 30 | 1.5 | 0.014 | School/Church | Fw | B | 75 | 88 | 95 | 1.064 | 1.248 | 1.348 |
| 31 | 0.8 | 0.007 | School/Church | Dp | C | 80 | 91 | 97 | 0.567 | 0.645 | 0.688 |
| 32 | 0.8 | 0.007 | School/Church | Fv | D | 83 | 93 | 98 | 0.589 | 0.660 | 0.695 |
| 33 | 3.8 | 0.035 | Single Family | Ea | A | 37 | 57 | 75 | 1.312 | 2.021 | 2.660 |
| 34 | 0.8 | 0.007 | Single Family | Dn | A | 37 | 57 | 75 | 0.262 | 0.404 | 0.532 |
| 35 | 6.1 | 0.057 | Single Family | Ec | A | 37 | 57 | 75 | 2.099 | 3.234 | 4.255 |
| 36 | 3.8 | 0.035 | Single Family | Gd | A | 37 | 57 | 75 | 1.312 | 2.021 | 2.660 |
| 37 | 1.5 | 0.014 | Single Family | Cg | A | 37 | 57 | 75 | 0.525 | 0.809 | 1.064 |
| 38 | 3.0 | 0.028 | Single Family | Cd | B | 53 | 72 | 86 | 1.504 | 2.043 | 2.440 |
| 39 | 11.4 | 0.106 | Single Family | Du | B | 53 | 72 | 86 | 5.638 | 7.660 | 9.149 |
| 40 | 5.3 | 0.050 | Single Family | Ft | B | 53 | 72 | 86 | 2.631 | 3.575 | 4.270 |
| 41 | 11.4 | 0.106 | Single Family | Fw | B | 53 | 72 | 86 | 5.638 | 7.660 | 9.149 |
| 42 | 0.8 | 0.007 | Single Family | Fg | B | 53 | 72 | 86 | 0.376 | 0.511 | 0.610 |
| 43 | 0.8 | 0.007 | Single Family | Dp | C | 64 | 81 | 92 | 0.454 | 0.574 | 0.653 |
| 44 | 4.6 | 0.043 | Single Family | Fs | C | 64 | 81 | 92 | 2.723 | 3.447 | 3.915 |
| 45 | 6.8 | 0.064 | Single Family | Do | D | 72 | 86 | 94 | 4.596 | 5.490 | 6.000 |
| 46 | 3.0 | 0.028 | Single Family | Dx | D | 72 | 86 | 94 | 2.043 | 2.440 | 2.667 |
| 47 | 6.1 | 0.057 | Single Family | Fv | D | 72 | 86 | 94 | 4.085 | 4.880 | 5.334 |
| 48 | 6.1 | 0.057 | Single Family | Gf | D | 72 | 86 | 94 | 4.085 | 4.880 | 5.334 |
| **Area-weighted curve number based on land use and HSG.** | | | | | | | | | 60 | 76 | 88 |

### Volume Estimates

The final step in the a priori volume estimates was to apply the rainfall depth from Tables 4.1 or 4.2 and the CN parameters to equation. There were two ways of applying the CN.

1) Unadjusted: The low, moderate, and high-resolution AMC II CN parameters were used in the equation for each recorded storm event.

2) Adjusted: The low, moderate, and high-resolution CN parameters that were adjusted for AMC were used in the equation for each recorded storm event

Table 4.19 summarizes the CN parameters used in the equation. The process yielded six volume estimates for each recorded storm event.

**Table 4.19. The unadjusted CN were the low, moderate and high-resolution AMC II values. The adjusted CN were selected from either AMC I, II, or III, which were determined by the previous 5-day rainfall depth.**

| | AMC I | | | AMC II | | | AMC III | | |
|---|---|---|---|---|---|---|---|---|---|
| **Watershed** | **Low** | **Moderate** | **High** | **Low** | **Moderate** | **High** | **Low** | **Moderate** | **High** |
| Sanford | 53 | 46 | 55 | 72 | 66 | 72 | 86 | 82 | 85 |
| Church | 53 | 50 | 60 | 72 | 69 | 76 | 86 | 84 | 88 |

## DIRECT FLOW MEASUREMENTS

In this technique, runoff data collected within a watershed were used to calibrate the Rational Method estimates. In general, the approach was to 1) select the events, 2) mathematically fit (usually using least-squares regression) the observed peak flow rates and the estimated peak flow rates to derive a calibration equation, and 3) apply the calibration equation to the remaining peak flow estimates to form a calibrated estimate. It is important to note that calibrated estimates were not derived from those events used to develop the calibration equations. Another important point is that only three of the original nine estimates in the a-priori analysis were brought forward for investigation using the calibration techniques in order to limit the number of permutations being analyzed. Calibration equations were developed for each watershed using the following combinations:

- One random event
- Five random events
- Ten random events
- Five nonrandom events (Sanford only)

The procedure began by using the least amount of data. In the simplest calibration, one storm event was randomly selected from each watershed to derive a calibration equation for that watershed. Event 27 from the Sanford watershed and event 17 from the Church watershed were those selected. Next, the selected peak flow rate was divided by the moderate-resolution C-intermediate $T_c$ estimate to derive a ratio. The following are the calibration ratios developed for each watershed.

Sanford Watershed Ratio

0.25 = 2.77cfs / 10.94 cfs

Church Watershed Ratio

0.81 = 0.97cfs / 1.19cfs

These ratios were then multiplied into the moderate-resolution estimates for all remaining storms on the respective watersheds. For example, the Sanford calibrated estimates were determined by the following:

ratio calibrated peak flow estimate = 0.25 x CiA

It should be note that the peak flow estimates developed from the low-resolution and high-resolution C coefficients were not used in any calibrations.

The next step up in complexity was to use multiple events to calibrate the estimations. For these events, the observed peak flows were fit to the estimated peak flows using linear and polynomial least squares regression. Appendix B shows the calibration equations and the information used to develop them. The calibration equations were then applied to the remaining estimates that were not used in the calibration. The calibrated estimates from the polynomial equations were dropped because of several extremely large errors. The events for the five-nonrandom calibration were selected to best represent runoff events on the watershed, and it was assumed that twenty events were needed to represent the range of storm events.

## DATA FROM A "SIMILAR WATERSHED"

### Peak Flow

The final technique to improve the accuracy of the models was to use the calibration equations developed for one watershed to adjust the estimates on the other watershed. This procedure followed the same pattern as the previous calibration technique; it started with a ratio from a single event and progressively applied equations developed using more data. The final step was to calibrate each event on one watershed and apply those individual equations to the matching event on the other watershed.

The first calibration equations applied were those developed in the previous technique, although this time the equations were used to modify the Rational Method estimates on the other watersheds. To begin, the ratio from the one-random event on the Church watershed was applied to all of the Sanford moderate-resolution estimates to produce a cross-calibrated estimate. This same technique was applied on the Church watershed using the ratio derived from the Sanford data.

Next, the five-random and ten-random equations developed on one watershed were applied to the other watershed. An additional calibration equation was derived using all of the Church watershed events and was applied to the Sanford estimates.

The final peak flow calibration was to develop an individual adjustment for each event and apply it to the matched event on the other watershed. Thus, only the paired events were used in this procedure. The calibrations were developed in the same way as in the single event ratio adjustment earlier, except this time there were twenty-six ratios.

The ratios were then applied to the matching moderate-resolution estimates on the other watershed.

### Volumes

Since the runoff volume calibration equations performed poorly within the watersheds, they were not applied between the watersheds and the research moved directly to using matched events. Also, since the a priori estimates yielded zero runoff so often, it was decided not to use those estimates. However, since the actual precipitation and runoff were known, the actual CN was backed-calculated for each event (Appendix C). The first procedure was to use the observed CN from one watershed to make a new volume estimate on the other watershed.

The second technique was to fit a portion of the observed curve numbers on both watersheds to one another. To do this, ten paired events were randomly chosen and a least squares regression was used to find the fitting equations (Appendix C). Then the regression equation was applied to the observed CN on the Church site. The calibrated CN was then used on the Sanford watershed to make a volume estimate. The final step was to apply this same method to the Church estimates.

## COMPARING THE ESTIMATES

Each estimated value was compared to the observed value using an absolute error (AE) term. This error term was calculated as:

$$AE = |\text{ observed value} - \text{estimated value }|$$

An average absolute error (AAE) was calculated using the following modification:

$$AAE = \Sigma\, |\text{ observed value} - \text{estimated value }| \,/\, n \qquad \text{Where: } n = \text{number of events}$$

For peak flow the error is in cfs, while volume errors are in inches. A percent error equation also was used to evaluate estimation errors and was calculated by:

$$\text{percent error} = [(\text{observed} - \text{estimated}) / \text{observed}] * 100\%$$

## Benefit to Cost Analysis

A benefit to cost analysis was performed to identify the most efficient improvement techniques. Because of the wider variety of observed storm events, this analysis was carried out on the Sanford watershed.

### Costs

To do this, it was first necessary to establish the "costs". Since these cost were primarily for comparing the calibration techniques, they were simplified. The simplifications were 1) time was allocated at $20.00 per hour, 2) no opportunity costs or time-value of money was accounted for, 3) initial setup for collecting observed data was set to $1,000 per watershed, 4) the per event costs were set at $50.00, and 6) determining the AMC was set at $20.00. Some notes and assumptions about the costs are as follows:

- Costs for the estimations were figured using the times to complete the relevant tasks as noted in the previous sections.
- It was assumed that twenty observations ($1,000) were necessary to collect the twenty representative events for the 5-nonrandom calibration.
- Forty events ($2,000) were used in the all-events calibration.
- Twenty-six events ($1,300) were used in the matched-events calibration.
- A multi-watershed reduction factor was used to reflect the benefits gained on two watersheds while collecting data on a single watershed.

### Benefits

Next, it was necessary to determine the "benefits". For peak flows, the AAE of each technique was subtracted from the smallest a priori AAE, which was 10.6 cfs. Thus, higher numbers indicated larger benefits. To determine the volume benefits, the AAE of each technique was subtracted from the lowest unadjusted a priori estimate, which was 0.106 inches.

The final step was to divide the benefits by the cost in thousands of dollars. This produced a benefit to cost ratio with the units being cfs/ $1000/watershed for the peak flow rate analysis and inches/ $1000/ watershed for the volume estimates. In this analysis the larger ratios indicated the more efficient techniques.

# CHAPTER 5

## RESULTS AND DISCUSSION

### A PRIORI ESTIMATION

#### Rational Method

The Rational Method was used to estimate nine peak flow rates for each storm event by multiplying the three C coefficients into each of the recorded rainfall intensities observed for the periods determined by the three $T_c$ parameters. This resulted in a total of 288 peak flow rate estimates for the thirty-two observed events on the Sanford watershed and 360 estimates for the 40 observed events on the Church watershed. The tables showing all of the parameters and the estimated peak flow rates are contained in Appendix E.

Average absolute errors ranged from 10.6 to 22.5 cfs, with an observed average peak flow rate of 10.55 cfs for the Sanford watershed. Errors on the Church watershed ranged from 8.5 to 11.6 cfs, with an observed average of 1.93 cfs (Tables 5.1 and 5.2 column 3). The observed peak flow rates on the Sanford watershed ranged from 0.03 to 54.0 cfs, while the peak flow rates ranged from 0.002 to 19 cfs on the Church watershed. On both watersheds, the moderate-resolution C coefficient and the intermediate $T_c$ parameter combination yielded the estimates with the lowest AAE. This is illustrated in Figure 5.1.

Furthermore, there appeared to be more variability in the errors produced by the different $T_c$ parameters than resulted from the different resolution C coefficients. This was most likely due to the relative homogeneity of the watersheds and the slight differences in C coefficients (Table 4.10) that were estimated. The largest difference was on the Church watershed, which had about a 15% difference between the moderate and high-resolution estimates. The Sanford watershed had approximately an 8% difference.

**Table 5.1. Summary of the a priori, directly calibrated, and the cross-watershed calibrated peak flow rate estimation errors on the Sanford watershed.**

| A Priori Parameters | | | 1-R-ww[i] | | 5-R-ww | | 10-R-ww | | 5-NR-ww | | 1-R-cw | | 5-R-cw | | 10-R-cw | | All-cw | | P-P-cw | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| C Coefficient Resolution | Tc Method | AAE[ii] (cfs) | AAE (cfs) | ER[iii] (%) | AAE (cfs) | ER (%) | AAE (cfs) | ER (%) | AAE (cfs) | ER (%) | AAE (cfs) | ER (%) | AAE (cfs) | ER (%) | AAE (cfs) | ER (%) | AAE (cfs) | ER (%) | AAE (cfs) | ER (%) |
| Low | Crude | 20.1 | | | | | | | | | | | | | | | | | | |
| Medium | Crude | 18.2 | 5.7 | 68.7 | 7.3 | 59.9 | 5.5 | 69.8 | 5.0 | 72.5 | 13.8 | 24.5 | 5.9 | 67.5 | 6.8 | 62.7 | 6.2 | 65.9 | 4.3 | 76.4 |
| High | Crude | 22.5 | | | | | | | | | | | | | | | | | | |
| Low | Intermed. | 11.1 | | | | | | | | | | | | | | | | | | |
| Medium | Intermed. | 10.6 | 6.5 | 38.7 | 8.1 | 23.6 | 3.9 | 63.2 | 3.5 | 67.0 | 7.8 | 26.4 | 6.7 | 37.1 | 7.6 | 28.4 | 7.0 | 33.8 | 4.4 | 58.5 |
| High | Intermed. | 12.1 | | | | | | | | | | | | | | | | | | |
| Low | Detailed | 13.5 | | | | | | | | | | | | | | | | | | |
| Medium | Detailed | 12.9 | 6.1 | 52.7 | 7.8 | 39.5 | 4.6 | 64.3 | 4.0 | 69.0 | 9.5 | 26.7 | 6.4 | 50.7 | 7.3 | 43.3 | 6.7 | 48.1 | 4.2 | 67.4 |
| High | Detailed | 14.6 | | | | | | | | | | | | | | | | | | |

Notes: Observed peak flow ranged from 0.03 to 54 cfs.

[i] Technique codes: $1^a$-$R^b$-$ww^c$

a. Number of events used to calibrate

b. Selection of events

R = Random

NR = Non-random

All = All events

P-P = Paired events

c. Source of calibration data

ww = within the watershed

cw = other watershed

[ii] AAE = average absolute error

AAE = Σ | Observed - Estimated | / number of events

[iii] ER: The percent difference between AAE of technique and the corresponding a priori AAE.

**Table 5.2. Summary of the a priori, directly calibrated, and the cross-watershed calibrated peak flow rate estimation errors on the Church watershed.**

| A Priori Parameters | | | 1-R-ww[i] | | 5-R-ww | | 10-R-ww | | 1-R-cw | | 5-R-cw | | 10-R-cw | | P-P-cw | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| C Coefficient Resolution | Tc Method | AAE[ii] (cfs) | AAE (cfs) | ER[iii] (%) | AAE (cfs) | ER (%) | AAE (cfs) | ER (%) | AAE (cfs) | ER[3] (%) | AAE (cfs) | ER (%) | AAE (cfs) | ER (%) | AAE (cfs) | ER (%) |
| Low | Crude | 11.6 | | | | | | | | | | | | | | |
| Medium | Crude | 10.7 | 7.6 | 28.8 | 1.0 | 90.6 | 1.1 | 89.6 | 1.3 | 88.1 | 1.1 | 90.0 | 4.2 | 60.5 | | |
| High | Crude | 12.9 | | | | | | | | | | | | | | |
| Low | Intermed. | 9.3 | | | | | | | | | | | | | | |
| Medium | Intermed. | 8.5 | 5.9 | 44.2 | 0.9 | 91.3 | 1.0 | 90.8 | 1.0 | 90.6 | 1.1 | 90.1 | 3.2 | 70.2 | 2.7 | 74.6 |
| High | Intermed. | 10.3 | | | | | | | | | | | | | | |
| Low | Detailed | 9.7 | | | | | | | | | | | | | | |
| Medium | Detailed | 8.9 | 6.2 | 41.5 | 1.0 | 91.0 | 1.0 | 90.8 | 1.1 | 90.1 | 1.0 | 90.3 | 3.3 | 68.5 | | |
| High | Detailed | 10.8 | | | | | | | | | | | | | | |

Notes: Observed peak flow ranged from 0.002 to 19 cfs.

[i] Technique codes: 1a-Rb-wwc

[i] Technique codes: $1^a$-$R^b$-$ww^c$

a. Number of events used to calibrate

b. Selection of events

R = Random

NR = Non-random

All = All events

P-P = Paired events

c. Source of calibration data

ww = within the watershed

cw = other watershed

[ii] AAE = average absolute error

AAE = Σ | Observed - Estimated | / number of events

[iii] ER: The percent difference between AAE of technique and the corresponding a priori AAE.

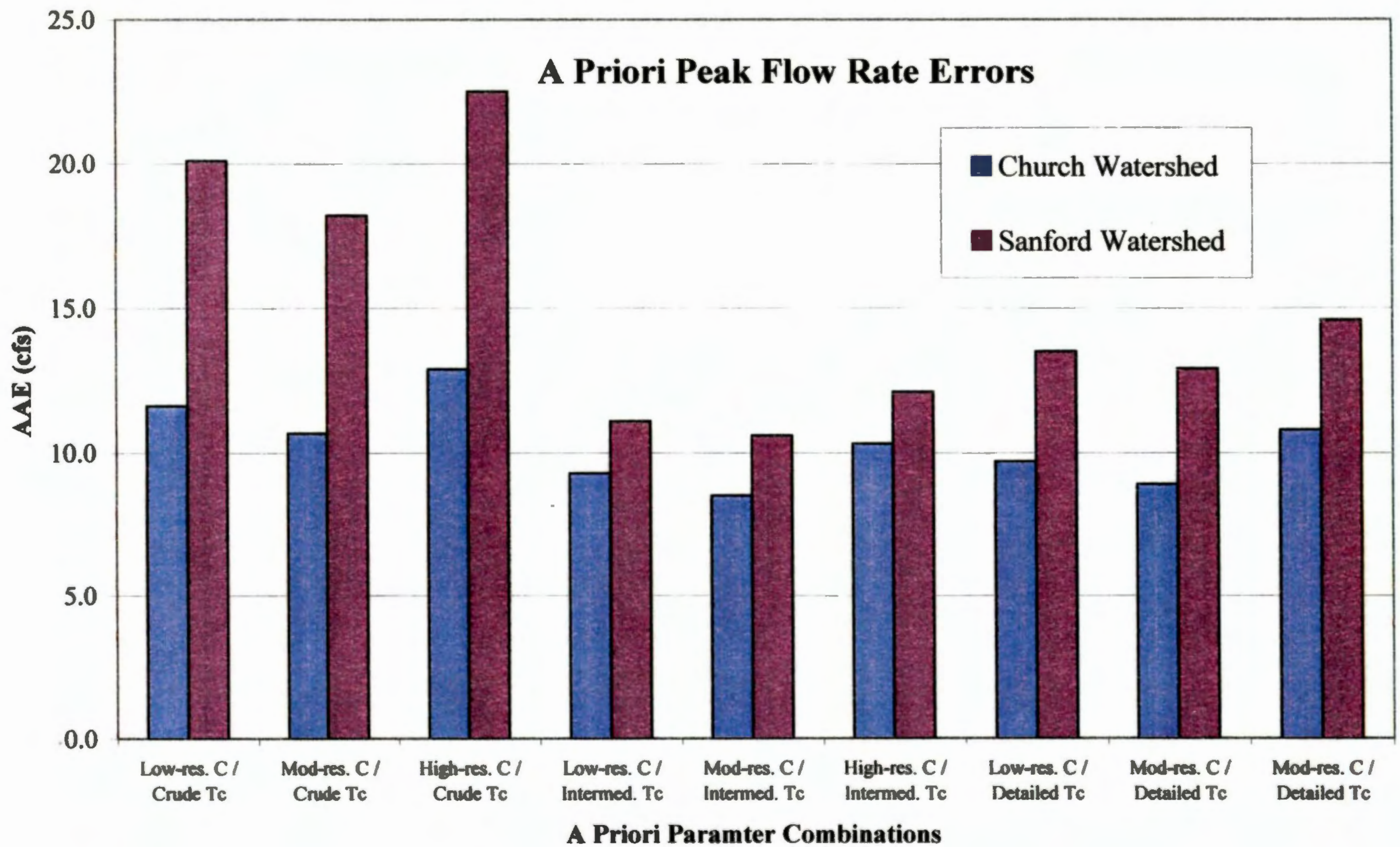



**Figure 5.1. The average absolute errors produced by the nine a priori parameter combinations for the Rational Method.**

Another important point to note is that the intermediate and detailed $T_c$ parameters produced more accurate estimates than did the crude $T_c$. Apparently, this improvement in accuracy was due to the longer times estimated by those techniques, and appeared primarily attributable to the use of the kinematic wave formula.

### SCS CN Method

Two sets of runoff volume estimates were made for each observed event from the low, moderate, and high-resolution CN parameters. The first set did not account for AMC, and so the CN for AMC II was used to make the estimates. The second set of estimates accounted for AMC. All of the estimated volumes are contained in Appendix F.

On the Sanford watershed, AAE values were 0.106 inches for the low-and high-resolution unadjusted CN estimates and 0.147 inches for the moderate-resolution CN estimates. The average observed volume was 0.161 inches and ranged from 0.0013 to 1.91 inches. The Church watershed had errors of 0.063, 0.147, and 0.106 inches, respectively, for the low, moderate, and high-resolution unadjusted CN estimates. The average runoff volume for the Church watershed was 0.034 inches and ranged from 0.004 to 0.33 inches. See Tables 5.3 and 5.4, column 2.

The second set of volume estimates was made to incorporate the antecedent moisture conditions. Errors on the Sanford watershed were reduced to 0.099, 0.138, and 0.88 inches for the low, moderate, and high-resolution estimates. The Church watershed had average errors of 0.037, 0.038, and 0.035 inches. All of the AAE from the a priori estimated volumes are represented in Figure 5.2. At first glance, it appears that accounting for AMC reduced the errors, particularly for the Church watershed, but it should be noted that when rainfall depths were less than the initial abstraction, the runoff volume was estimated as zero. Thus, many events were estimated as zero runoff, so the maximum errors were the actual runoff amounts. On the Church watershed, this was particularly noticeable due to the small flow volumes.

Table 5.3. Summary of the a priori and cross-watershed calibrated volume estimation errors on the Sanford watershed.

| A Priori Parameters | AMC II[i] | 5-day AMC[ii] | | Cross CN Substitution | | Cross CN Sub. W/ calibration | |
|---|---|---|---|---|---|---|---|
| CN Resolution | AAE[iii] (in) | AAE (in) | ER[iiii] (%) | AAE (in) | ER (%) | AAE (in) | ER (%) |
| Low | 0.106 | 0.099 | 6.6 | 0.039 | 63.7 | 0.036 | 66.3 |
| Moderate | 0.147 | 0.138 | 6.1 | | | | |
| High | 0.106 | 0.088 | 17.0 | | | | |

Observed volume ranged from 0.0013 to 1.9 inches.

Notes:

[i] No CN adjustments made for AMC.

[ii] Estimates were adjusted for the preceding 5-day AMC.

[iii] AAE = average absolute error
AAE = $\Sigma$ | Observed - Estimated | / number of events

[iiii] ER: The percent difference between AAE of technique and the corresponding a priori AAE.



Table 5.4. Summary of the a priori and cross-watershed calibrated volume estimation errors on the Church watershed.

| A Priori Parameters | AMC II[i] | 5-day AMC[ii] | | Cross CN Substitution | | Cross CN Sub. W/ calibration | |
|---|---|---|---|---|---|---|---|
| CN Resolution | AAE[iii] (in) | AAE (in) | ER[iiii] (%) | AAE (in) | ER (%) | AAE (in) | ER (%) |
| Low | 0.063 | 0.037 | 41.6 | 0.031 | 50.2 | 0.021 | 66.8 |
| Moderate | 0.147 | 0.038 | 74.4 | | | | |
| High | 0.106 | 0.035 | 67.1 | | | | |

Observed volumes ranged from 0.003 to 0.33 inches.

Notes:

[i] No CN adjustments made for AMC.

[ii] Estimates were adjusted for the preceding 5-day AMC.

[iii] AAE = average absolute error
AAE = $\Sigma$ | Observed - Estimated | / number of events

[iiii] ER: The percent difference between AAE of technique and the corresponding a priori AAE.

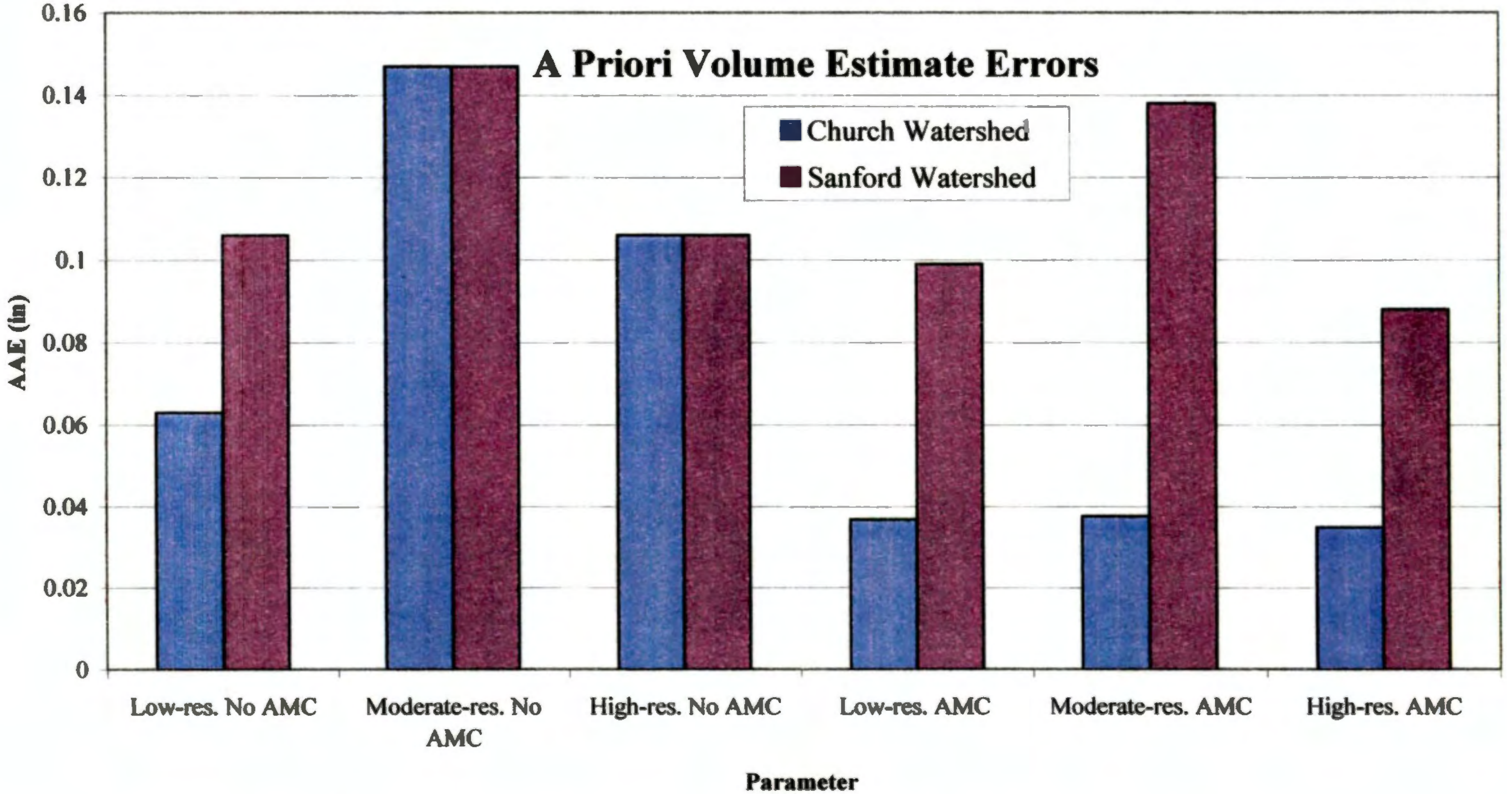



**Figure 5.2. The average absolute errors produced by the AMC adjusted and unadjusted CN parameters.**

The AMC adjusted estimates indicated two issues. The first issue is that the initial abstraction built into the SCS CN equation is probably set too high for suburban watersheds. Likely, this is because the equation was primarily developed using data gathered from agricultural plots, which tend to have higher infiltration rates and thus, require higher rainfall amounts before runoff can begin. The second issue is that the 5-day AMC adjustment might not be appropriate on this type of watershed. Again, the watershed conditions from which the model was developed were substantially different from the Sanford and Church watersheds.

## DIRECT FLOW MEASUREMENTS FROM THE WATERSHED

### Single Random Event (Ratio Technique)

The first calibration technique used an estimated to observed ratio determined from a single randomly selected event, which was then applied to the remaining estimates. The two ratios were 0.25 for the Sanford watershed and 0.81 for the Church watershed. The difference in these ratios was quite large and depended upon how accurate the randomly selected event happened to be.

The average absolute errors were 5.7, 6.5, and 6.1 cfs for the crude, intermediate, and detailed $T_c$-moderate resolution estimates on the Sanford watershed. The Church watershed had errors of 7.55, 5.91, and 6.20 cfs. The AAE for the single random calibration are summarized in Tables 5.1 and 5.2 column four (calibration technique coded as 1-R-ww). Column five (ER) of the same tables show the percent difference between the a priori estimate AAE and the AAE of the calibrated estimate. The reduction in AAE ranged from 39 to 69 % on the Sanford watershed and the Church watershed had reductions of 29 to 44%. The reductions in error are a function of the similarity of the ratio determined from the randomly selected event to the expected value of the ratio for all events. The Sanford watershed had an expected ratio for all events of 0.40 and the Church watershed had an expected ratio for all events of 0.27. This explains why the Sanford watershed had a higher reduction in errors than did the Church watershed. It is notable that the Church watershed had substantial reductions in error although the calibration ratio did not well represent the expected ratio for all events.

### Multiple Events

The next step up in complexity was to use multiple events to calibrate the estimations using linear least squares regression. Again, the events were randomly selected except for one specific calibration. The AAE for the watersheds are illustrated in Figure 5.3 and are summarized in Table 5.1 and 5.2 using the following codes:

| | |
|---|---|
| Five-random events | 5-R-ww |
| Ten-random events | 10-R-ww |
| Five-nonrandom events | 5-NR-ww |

The most noteworthy issue in these calibrations is the dramatic improvement in the estimations on the Church watershed, where the errors where reduced 90%. This reduction seems to be the result of combination of two factors. The first is that the storms selected to derive the calibration equations included a large runoff event, which led to a moderately accurate regression equation (see Appendix C). The second factor is that the largest storm events that occurred on this watershed flooded the datalogger and thereby eliminated some of the extreme events from consideration.

However, while the largest event used in the ten-random calibration had a peak flow rate of 8 cfs, the calibration equation reduced the error on the largest event recorded, which had a peak flow rate of 19.8 cfs. On this event, the a priori estimate absolute error of approximately 300 % was reduced to about 30%. This indicates 1) that large or low probability events need to be included in the calibration, and 2) even if they are not included in developing the calibration equations, then the estimates for extreme events may still be improved. There is a caveat, however; the calibrated value was an underestimation. Thus, a safety factor may need to be included in the calibration, but in this case doubling or even tripling the calibrated estimate still results in a substantially improved estimate (Table 5.5).

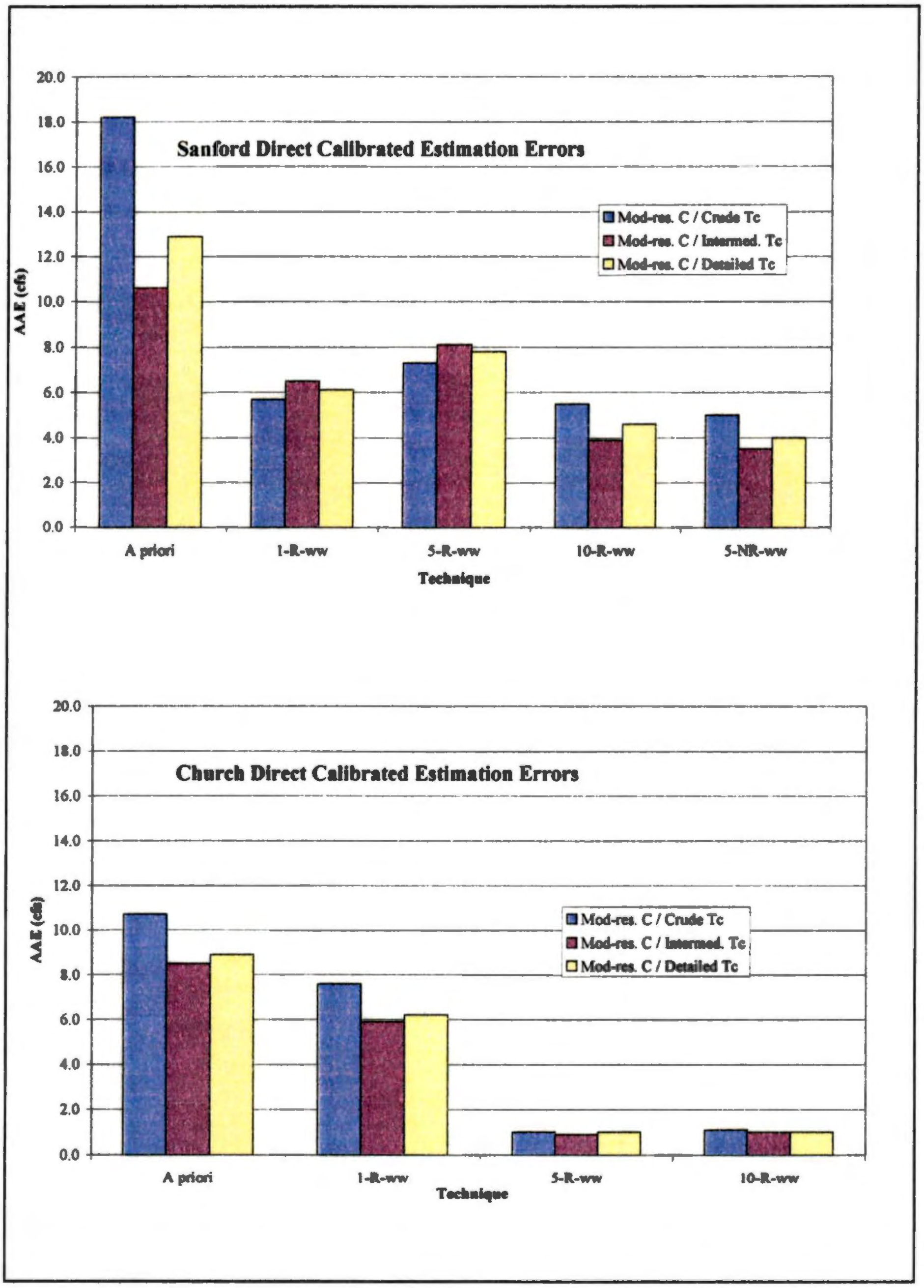



**Figure 5.3. The average absolute errors produced by the directly calibrated estimates.**

**Table 5.5. The effect of the ten-random event calibration on the most extreme observed peak flow rate from the Church watershed.**

| Event Date | Observed Peak Flow | Best A Priori Estimate | Calibrated Estimate |
|---|---|---|---|
| 5-21-98 | 19.8 cfs | 79 cfs | 13 cfs |

On the Sanford watershed, the five-random event calibration did not reduce errors as much as did the single random event. This is best explained by the calibration equation for the five-random events, which had an $R^2 = 0.44$. In this case, the linear regression equation provided only a weak fit to the data. The polynomial equation fit the data better, but in some of the calibrations the errors increased exponentially, so I decided to not use polynomial equations on any calibrations. Another factor that could have improved the regression equation was if I had not set the intercept = 0. Setting the intercept = 0 worked well in most cases and was a standard procedure.

The ten-random event calibration on the Sanford watershed reduced errors from 63 to 70% and had an AAE of 3.9 to 5.5 cfs. The regression equation had an $R^2 = 0.94$. The errors for the five-nonrandom calibration were slightly lower, with an AAE ranging from 3.5 to 5.0 cfs. These were from 67 to 73% lower than the a priori estimates and the regression equation had an $R^2 = 0.83$. Since I assumed it required twenty observed events to acquire the data to select the five-nonrandom events, this calibration was a step up in the quantity of data needed. Thus, the improvement of 0.5 cfs between the ten-random and the five-nonrandom calibrations required twice as much data.

### Volumes

Calibrating the volume estimates using runoff data collected in the watershed was attempted using a number of techniques. Besides a single exception, all estimates were less accurate than the a priori estimates. This appeared to be due to the variability of the observed CN between storm events, and will be discussed in more detail in a later section.

## DIRECT FLOW MEASUREMENTS FROM A SIMILAR WATERSHED

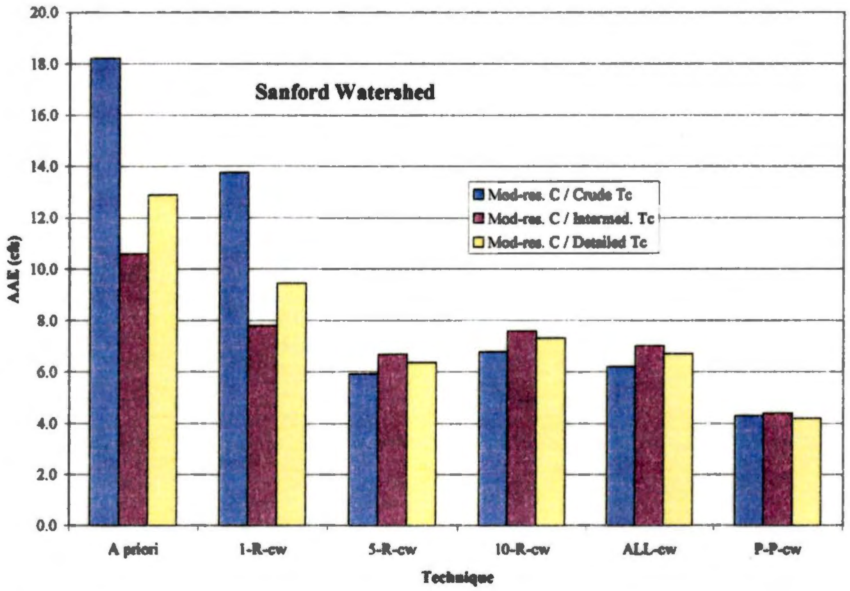
### Peak Flows

The final peak flow calibration techniques used data from the other watershed to calibrate the estimations on the target watershed. The calibrations that were performed and the corresponding codes used in the tables are as follows (cw indicates "cross-watershed"):

| | |
|---|---|
| One-random event | 1-R-cw |
| Five-random events | 5-R-cw |
| Ten-random events | 10-R-cw |
| Five-nonrandom events (Sanford only) | 5-NR-cw |
| All events (Sanford only) | All-cw |
| Peak by peak | P-P-cw |

#### One-random Event

The errors from the one-random event cross-watershed calibrations are summarized in Tables 5.1 and 5.2 and are illustrated in Figure 5.4. The AAE ranged from 7.8 to 13.6 cfs for the Sanford watershed. This was about 25% lower than the AAE from the a priori estimates. More noteworthy was the Church watershed, in which one AAE was reduced from 8.5 cfs to 1.0 cfs, which was about a 90% reduction. The other AAE values were reduced similarly. These reductions were not surprising considering the former discussion about the expected ratio values for all events. The expected ratio for all events on the Church watershed was 0.27 and the ratio used calibrate the Church estimations was 0.25, which led to highly accurate estimations. This occurred by random chance, and this randomness is illustrated by the moderate improvements noted in the Sanford estimates. In this case, the expected ratio for all events was 0.40, but the ratio used to calibrate the estimates was 0.81 and the result was a moderate improvement in accuracy.



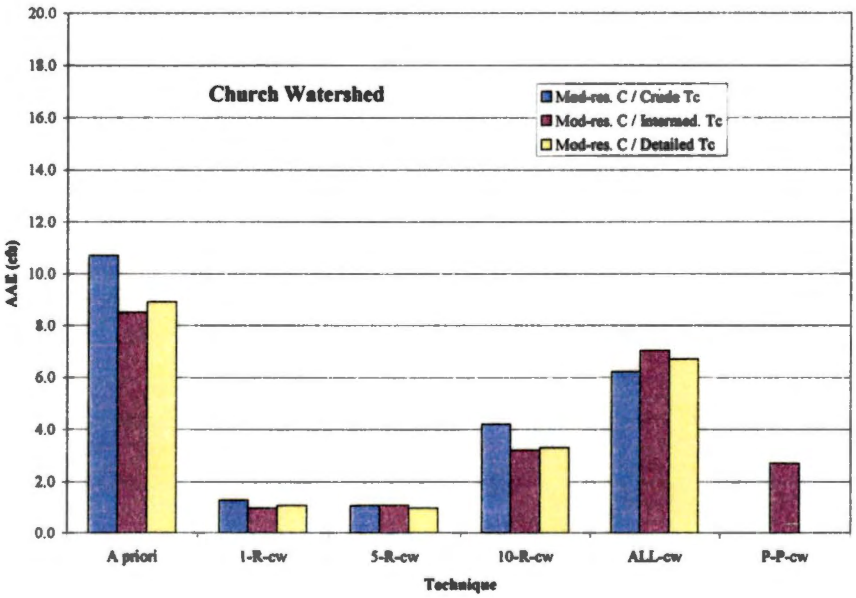




**Figure 5.4. The peak flow rate average absolute errors produced by the estimates calibrated using data from a similar watershed.**

### Multiple Events

Next, the five-random and ten-random equations developed on one watershed were used to calibrate the estimations on the other watershed. Again the results are summarized in Tables 5.1 and 5.2 and are illustrated in Figure 5.4. The five-random event calibration AAE ranged from 5.91 cfs to 6.67 cfs (37-44% reduction) for the Sanford watershed and 1.03 to 1.06 cfs (87-88% reduction) on the Church watershed. Again, the low AAE on the Church watershed does not mean much. It is explained by the equation used to calibrate the estimates on the Church watershed, which in effect multiplied 0.13 into the estimated peak flows. This resulted in substantially lowering all of the estimates, and since many of the large events were lost, the net effect was a low AAE. Conversely, the Sanford watershed provides a more realistic view of the affects of the calibrations.

For the ten-random event calibration, the Sanford watershed had errors of 6.78 to 7.59 cfs (28-36% reduction). An additional calibration equation was derived using all of the Church watershed events and applied to the Sanford estimates. Average absolute errors of 6.2 to 7.02 were the results of this calibration.

These results imply 1) more data tends to increase the estimation accuracy, and 2) less data can be used if a broad range of runoff data including lower-probability events are used to develop the calibration. The first implication is indicated by the general trend that is shown by the graph in Figure 5.5. While the regression equations may not mean much, they do indicate a slope that could be conservatively approximated as negative one, which implies an inverse relationship between the quantity of data used in the calibration equation and the error values. The second implication is supported by the five-random AAE, which is lower than both the ten-random calibrations and the all event calibrations. Again, a look at the Church calibration equations located in Appendix C show that the five-random calibration comes the closest to the expected observed / estimated value of 0.40.

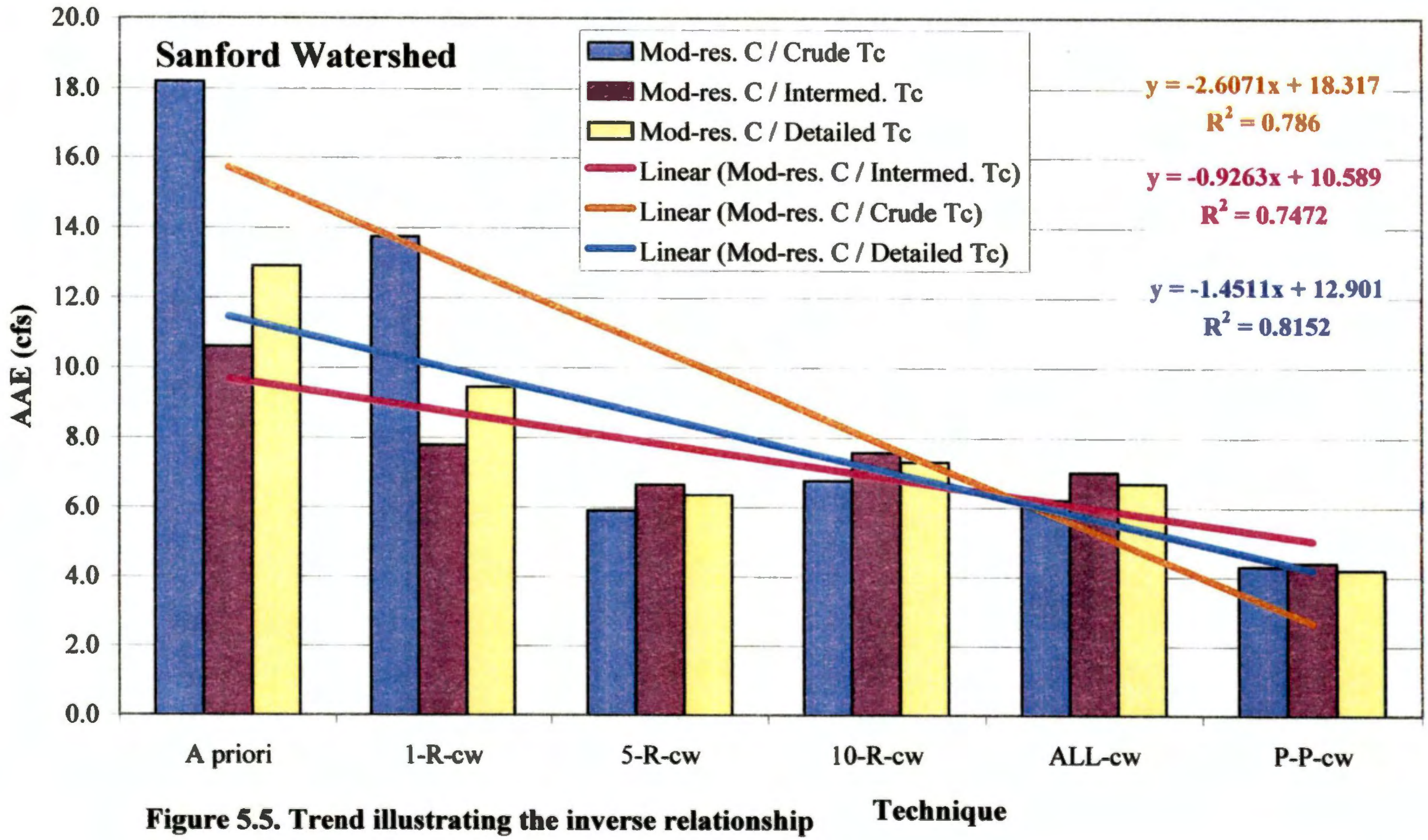



**Figure 5.5. Trend illustrating the inverse relationship between the quantity of calibration data and the AAE.**

### Matched Events

The final peak flow calibration was to develop an individual adjustment for each event and apply it to the matched event on the other watershed. The resulting AAE ranged from 4.2 to 4.4 cfs on the Sanford site. These estimates had the lowest errors of any of the cross-watershed calibrated estimates and were more accurate than all but three of the calibration techniques that used data collected from within the watershed. This accuracy was unexpected and indicates that there is a parallel relationship between the observed and estimated values on both watersheds. In effect, this suggests that the estimation error on one watershed was matched by a similar error on the other watershed. Since this technique was nearly as accurate as the best calibration techniques, this could imply that evaluating the between-events temporal variability might be as important as accounting for the other characteristics that influence runoff. Considering that rainfall intensity was a model parameter, a question is raised regarding what other variables change between events and whether there are uncomplicated procedures to adjust the Rational Method.

While the peak by peak technique controlled some of the between-events variability, it required real-time or post hoc estimates rather than predictive estimates. Thus, this technique would be limited to applications where real-time estimations are useful, such as flood-warning systems, contaminant transport estimations, or large watershed models in which results from a small area could be used to calibrate estimates on several subwatersheds.

## Volume

The runoff volumes were estimated by back-calculating the CN from one watershed and applying that CN to the other watershed. The AAE for the Sanford watershed was reduced from 0.106 inches to 0.039 inches, which was a 63% reduction (Table 5.3). The AAE on the Church watershed was reduced from 0.063 inches to 0.034 inches, which was a 50% reduction (Table 5.4) A second technique mathematically fit ten back-calculated CN from the source watershed to ten back-calculated CN from the target watershed. Then for the remaining events, the calibration equation was applied to the back-calculated CN from the source water, and used to estimate runoff volumes on the target watershed. This resulted in a slight decrease in the AAE to 0.036 inches on the Sanford watershed and to 0.021 inches on the Church watershed.

These results suggest that 1) the two watersheds behave similarly during a storm event, which implies that the spatial characteristics are similar, and 2) the physical characteristics that control runoff on the watersheds change temporally, which indicates that a single CN, no matter how well estimated, may not work.

This technique required estimates to be calibrated during or after the events rather than before the events, which limits the predictive aspects of the model. Therefore, it would be confined to those applications where prediction is not the primary need, and where real-time or post hoc estimates would suffice.

## BENEFIT TO COST ANALYSIS

### Peak Flows

During the a priori estimations, the moderate-resolution C coefficient and intermediate $T_c$ parameter combination provided the lowest AAE, which was 10.6 cfs. Therefore, 10.6 cfs was used as the standard to judge the benefits of the other estimation methods. The remaining a priori estimates were dropped from the benefit to cost (B-C) analysis because each would have a negative benefit value. Table 5.6 contains the estimation costs, observation costs, benefits, and the benefit to cost ratio for each of the calibration techniques, while Tables 5.7 and 5.8 present the data sorted by descending benefits and also by descending B-C ratio. The data are graphically represented in Figure 5.6

Examination of the data raises several important points that warrant some discussion. The first issue is that the techniques with the three highest benefits all used direct flow measurement calibrations, and the only cross-watershed calibration techniques that had benefits in the top ten were the peak by peak techniques. This confirms the expectation that using calibration data collected from within a watershed

**Table 5.6. Per watershed analysis of the benefit to cost ratio of the peak flow calibration techniques.**

| Technique[i] | AAE[ii] (cfs) | Estimation costs[iii] ($) C coefficient | $T_c$ | Subtotal | Observation Costs[iiii] ($) Initial Setup | Per Event | Multi-watershed Reduction Factor[iiii] | Subtotal | Total Costs ($) | Benefit[iiiii] (10.6 - AAE) | Benefit/Cost[iiiiii] (cfs/$1000/watershed) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 1-R-ww-M-C | 5.7 | 40.00 | 5.00 | 45.00 | 1,000.00 | 50.00 | 1 | 1,050.00 | 1,095.00 | 4.9 | 4.5 |
| 1-R-ww-M-I | 6.5 | 40.00 | 10.00 | 50.00 | 1,000.00 | 50.00 | 1 | 1,050.00 | 1,100.00 | 4.1 | 3.7 |
| 1-R-ww-M-D | 6.1 | 40.00 | 120.00 | 160.00 | 1,000.00 | 50.00 | 1 | 1,050.00 | 1,210.00 | 4.5 | 3.7 |
| 5-R-ww-M-C | 7.3 | 40.00 | 5.00 | 45.00 | 1,000.00 | 250.00 | 1 | 1,250.00 | 1,295.00 | 3.3 | 2.5 |
| 5-R-ww-M-I | 8.1 | 40.00 | 10.00 | 50.00 | 1,000.00 | 250.00 | 1 | 1,250.00 | 1,300.00 | 2.5 | 1.9 |
| 5-R-ww-M-D | 7.8 | 40.00 | 120.00 | 160.00 | 1,000.00 | 250.00 | 1 | 1,250.00 | 1,410.00 | 2.8 | 2.0 |
| 10-R-ww-M-C | 5.5 | 40.00 | 5.00 | 45.00 | 1,000.00 | 500.00 | 1 | 1,500.00 | 1,545.00 | 5.1 | 3.3 |
| 10-R-ww-M-I | 3.9 | 40.00 | 10.00 | 50.00 | 1,000.00 | 500.00 | 1 | 1,500.00 | 1,550.00 | 6.7 | 4.3 |
| 10-R-ww-M-D | 4.6 | 40.00 | 120.00 | 160.00 | 1,000.00 | 500.00 | 1 | 1,500.00 | 1,660.00 | 6.0 | 3.6 |
| 5-NR-ww-M-C | 5 | 40.00 | 5.00 | 45.00 | 1,000.00 | 1,000.00 | 1 | 2,000.00 | 2,045.00 | 5.6 | 2.7 |
| 5-NR-ww-M-I | 3.5 | 40.00 | 10.00 | 50.00 | 1,000.00 | 1,000.00 | 1 | 2,000.00 | 2,050.00 | 7.1 | 3.5 |
| 5-NR-ww-M-D | 4 | 40.00 | 120.00 | 160.00 | 1,000.00 | 1,000.00 | 1 | 2,000.00 | 2,160.00 | 6.6 | 3.1 |
| 1-R-cw-M-C | 13.75 | 80.00 | 10.00 | 90.00 | 1,000.00 | 50.00 | 2 | 525.00 | 615.00 | -3.2 | -5.1 |
| 1-R-cw-M-I | 7.8 | 80.00 | 20.00 | 100.00 | 1,000.00 | 50.00 | 2 | 525.00 | 625.00 | 2.8 | 4.5 |
| 1-R-cw-M-D | 9.45 | 80.00 | 240.00 | 320.00 | 1,000.00 | 50.00 | 2 | 525.00 | 845.00 | 1.2 | 1.4 |
| 5-R-cw-M-C | 5.91 | 80.00 | 10.00 | 90.00 | 1,000.00 | 250.00 | 2 | 625.00 | 715.00 | 4.7 | 6.6 |
| 5-R-cw-M-I | 6.67 | 80.00 | 20.00 | 100.00 | 1,000.00 | 250.00 | 2 | 625.00 | 725.00 | 3.9 | 5.4 |
| 5-R-cw-M-D | 6.36 | 80.00 | 240.00 | 320.00 | 1,000.00 | 250.00 | 2 | 625.00 | 945.00 | 4.2 | 4.5 |
| 10-R-cw-M-C | 6.78 | 80.00 | 10.00 | 90.00 | 1,000.00 | 500.00 | 2 | 750.00 | 840.00 | 3.8 | 4.5 |
| 10-R-cw-M-I | 7.59 | 80.00 | 20.00 | 100.00 | 1,000.00 | 500.00 | 2 | 750.00 | 850.00 | 3.0 | 3.5 |
| 10 R-cw-M-D | 7.32 | 80.00 | 24.00 | 104.00 | 1,000.00 | 500.00 | 2 | 750.00 | 854.00 | 3.3 | 3.8 |
| ALL-cw-M-C | 6.21 | 80.00 | 10.00 | 90.00 | 1,000.00 | 2,000.00 | 2 | 1,500.00 | 1,590.00 | 4.4 | 2.8 |
| ALL-cw-M-I | 7.02 | 80.00 | 20.00 | 100.00 | 1,000.00 | 2,000.00 | 2 | 1,500.00 | 1,600.00 | 3.6 | 2.2 |
| ALL-cw-M-D | 6.7 | 80.00 | 240.00 | 320.00 | 1,000.00 | 2,000.00 | 2 | 1,500.00 | 1,820.00 | 3.9 | 2.1 |
| P-P-cw-M-C | 4.3 | 80.00 | 10.00 | 90.00 | 1,000.00 | 1,300.00 | 2 | 1,150.00 | 1,240.00 | 6.3 | 5.1 |
| P-P-cw-M-I | 4.4 | 80.00 | 20.00 | 100.00 | 1,000.00 | 1,300.00 | 2 | 1,150.00 | 1,250.00 | 6.2 | 5.0 |
| P-P-cw-M-D | 4.2 | 80.00 | 240.00 | 320.00 | 1,000.00 | 1,300.00 | 2 | 1,150.00 | 1,470.00 | 6.4 | 4.4 |

Notes:

[i] Technique codes: $1^a$-$R^b$-$ww^c$-$M^d$-$C^e$

a. Number of events used to calibrate

b. Selection of events
- R = Random
- NR = Non-random
- All = All events
- P-P = Paired events

c. Source of calibration data
- ww = within the watershed
- cw = other watershed

d. C coefficient resolution
- L = Low
- M = Moderate
- H = High

d. $T_c$ model parameter
- C = Crude
- I = Intermediate
- D = Detailed

[ii] AAE = average absolute error
AAE = Σ | Observed - Estimated | / number of events

[iii] Costs to asses model parameters
In this series only the moderate-resolution C coefficient was used.
Techniques using cw data required estimations on both watersheds

[iiii] Assumed costs to collect rainfall-runoff data
Per event cost is $50 * number of observed events
5-NR techniques assumed 20 observed events.

[iiii] Costs reflect estimation improvements in two watersheds while collecting data in a single watershed.

[iiiii] The benefit is the reduction in AAE a technique yielded.
The standard of comparison was the best a priori AAE, which was 10.6 cfs.

[iiiiii] Cost is in thousands ($) per watershed

**Table 5.7. The calibration techniques listed in order of greatest to least benefit.**

| Technique | Costs ($) | Benefit | Benefit/Cost |
|---|---|---|---|
| 5-NR-ww-M-I | 2,050.00 | 7.1 | 3.5 |
| 10-R-ww-M-I | 1,550.00 | 6.7 | 4.3 |
| 5-NR-ww-M-D | 2,160.00 | 6.6 | 3.1 |
| P-P-cw-M-D | 1,470.00 | 6.4 | 4.4 |
| P-P-cw-M-C | 1,240.00 | 6.3 | 5.1 |
| P-P-cw-M-I | 1,250.00 | 6.2 | 5.0 |
| 10-R-ww-M-D | 1,660.00 | 6.0 | 3.6 |
| 5-NR-ww-M-C | 2,045.00 | 5.6 | 2.7 |
| 10-R-ww-M-C | 1,545.00 | 5.1 | 3.3 |
| 1-R-ww-M-C | 1,095.00 | 4.9 | 4.5 |
| 5-R-cw-M-C | 715.00 | 4.7 | 6.6 |
| 1-R-ww-M-D | 1,210.00 | 4.5 | 3.7 |
| ALL-cw-M-C | 1,590.00 | 4.4 | 2.8 |
| 5-R-cw-M-D | 945.00 | 4.2 | 4.5 |
| 1-R-ww-M-I | 1,100.00 | 4.1 | 3.7 |
| 5-R-cw-M-I | 725.00 | 3.9 | 5.4 |
| ALL-cw-M-D | 1,820.00 | 3.9 | 2.1 |
| 10-R-cw-M-C | 840.00 | 3.8 | 4.5 |
| ALL-cw-M-I | 1,600.00 | 3.6 | 2.2 |
| 5-R-ww-M-C | 1,295.00 | 3.3 | 2.5 |
| 10 R-cw-M-D | 854.00 | 3.3 | 3.8 |
| 10-R-cw-M-I | 850.00 | 3.0 | 3.5 |
| 5-R-ww-M-D | 1,410.00 | 2.8 | 2.0 |
| 1-R-cw-M-I | 625.00 | 2.8 | 4.5 |
| 5-R-ww-M-I | 1,300.00 | 2.5 | 1.9 |
| 1-R-cw-M-D | 845.00 | 1.2 | 1.4 |
| 1-R-cw-M-C | 615.00 | -3.2 | -5.1 |

Note: See Table 5.6 for technique codes and units.

**Table 5.8. The calibration techniques listed in order of greatest to least benefit to cost ratio.**

| Technique | Costs ($) | Benefit | Benefit/Cost |
|---|---|---|---|
| 5-R-cw-M-C | 715.00 | 4.7 | 6.6 |
| 5-R-cw-M-I | 725.00 | 3.9 | 5.4 |
| P-P-cw-M-C | 1,240.00 | 6.3 | 5.1 |
| P-P-cw-M-I | 1,250.00 | 6.2 | 5.0 |
| 10-R-cw-M-C | 840.00 | 3.8 | 4.5 |
| 5-R-cw-M-D | 945.00 | 4.2 | 4.5 |
| 1-R-cw-M-I | 625.00 | 2.8 | 4.5 |
| 1-R-ww-M-C | 1,095.00 | 4.9 | 4.5 |
| P-P-cw-M-D | 1,470.00 | 6.4 | 4.4 |
| 10-R-ww-M-I | 1,550.00 | 6.7 | 4.3 |
| 10 R-cw-M-D | 854.00 | 3.3 | 3.8 |
| 1-R-ww-M-I | 1,100.00 | 4.1 | 3.7 |
| 1-R-ww-M-D | 1,210.00 | 4.5 | 3.7 |
| 10-R-ww-M-D | 1,660.00 | 6.0 | 3.6 |
| 10-R-cw-M-I | 850.00 | 3.0 | 3.5 |
| 5-NR-ww-M-I | 2,050.00 | 7.1 | 3.5 |
| 10-R-ww-M-C | 1,545.00 | 5.1 | 3.3 |
| 5-NR-ww-M-D | 2,160.00 | 6.6 | 3.1 |
| ALL-cw-M-C | 1,590.00 | 4.4 | 2.8 |
| 5-NR-ww-M-C | 2,045.00 | 5.6 | 2.7 |
| 5-R-ww-M-C | 1,295.00 | 3.3 | 2.5 |
| ALL-cw-M-I | 1,600.00 | 3.6 | 2.2 |
| ALL-cw-M-D | 1,820.00 | 3.9 | 2.1 |
| 5-R-ww-M-D | 1,410.00 | 2.8 | 2.0 |
| 5-R-ww-M-I | 1,300.00 | 2.5 | 1.9 |
| 1-R-cw-M-D | 845.00 | 1.2 | 1.4 |
| 1-R-cw-M-C | 615.00 | -3.2 | -5.1 |

Note: See Table 5.6 for technique codes and units.

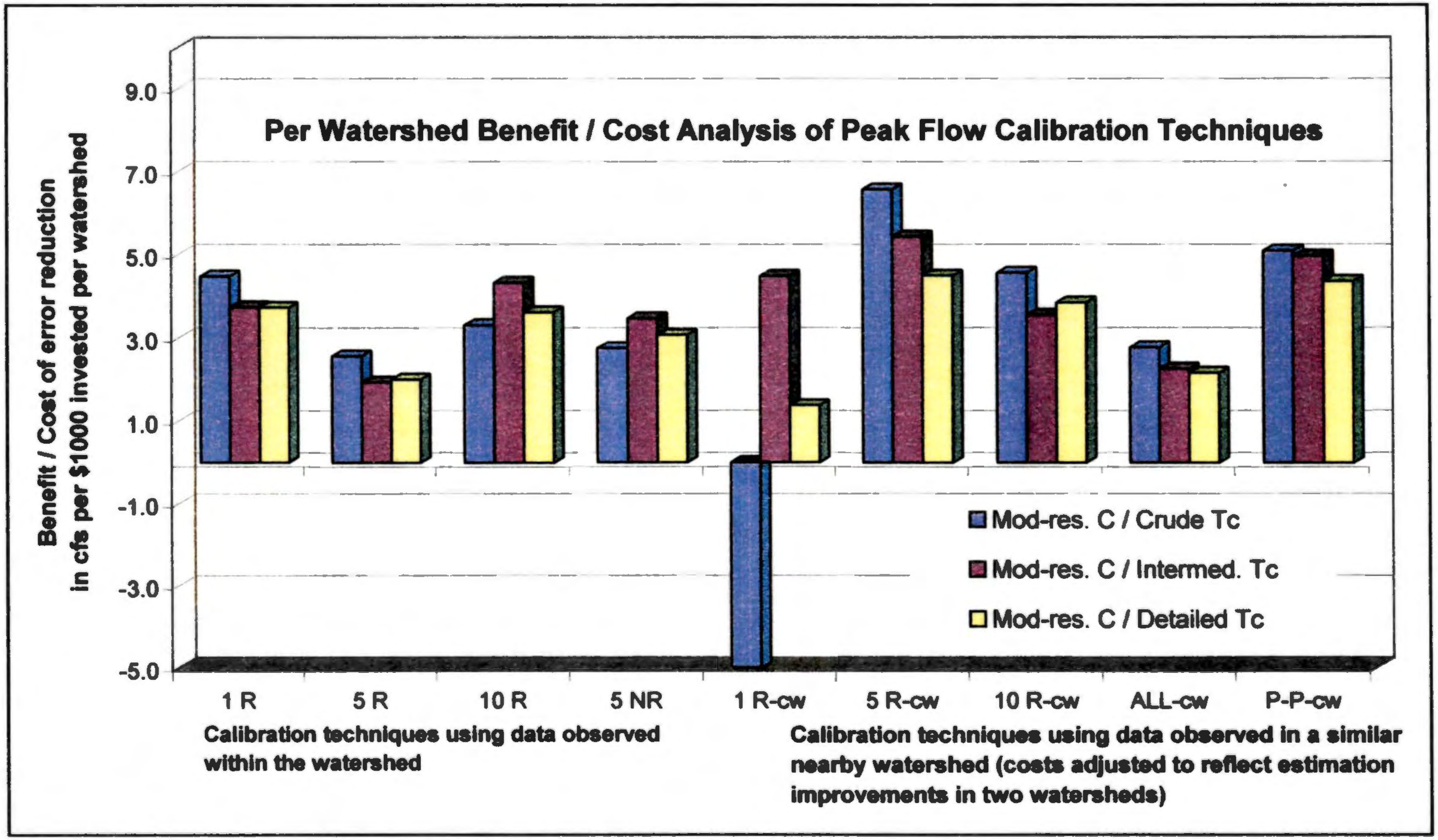



**Figure 5.6 The benefit to cost analysis of the peak flow rate calibration techniques.**

leads to more accurate estimates than does the cross-watershed use of data. The high accuracy of the three peak by peak results was discussed above.

A second point is that eight of the top ten B-C ratios are techniques that used cross-watershed calibration data. This appears due to the multiple watershed factor and the assigned costs. In other B-C analyses, the cost and accuracy could be weighted to reflect different assessments of their relative values.

### Volumes

As previously noted, the AAE for the unadjusted a priori volume estimates ranged from 0.106 inches to 0.138 inches. Although the high and low-resolution estimates had the same error value, the low-resolution estimates was less expensive, and therefore was more cost effective for all of the volume B-C information (see Table 5.9 and Figure 5.7). The other two estimates were regarded as less efficient and so the comparison standard for the remaining estimation was the low-resolution AAE at 0.106.

The benefit derived from adding the 5-day AMC data were 0.01 inches for the low-resolution estimate and 0.02 inches for the high-resolution data, while the moderate resolution estimate had a negative benefit and was dropped. Although slightly less accurate, the low-resolution estimate had better B-C ratio due to the differences in cost.

The CN substitution increased the benefit to 0.07 inches at a cost of $1,150 per watershed to yield a B-C ratio of 0.06. The final volume method had only a slight increase in benefit. However, the cost increased substantially, which led to a decreased B-C ratio of 0.04.

**Table 5.9. Per watershed benefit to cost analysis of the volume estimates.**

| Technique | AAE | Estimation Costs ($) | | | Observation Costs ($) | | Multi-watershed Reduction | | Total | Benefit | Benefit/Cost |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Undajusted CN | (in) | CN | AMC | Subtotal | Setup | Per Event | Factor | Subtotal | Costs ($) | (0.106 - est) | (in/$1000/watershed) |
| Low-res. | 0.106 | 20 | 0 | 20 | | | | 0 | 20 | 0.00 | |
| Moderate-res. | 0.147 | 40 | 0 | 40 | | | 1 | 0 | 40 | -0.04 | -1.03 |
| High-res. | 0.106 | 120 | 0 | 120 | | | 1 | 0 | 120 | 0.00 | |
| **CN Resolution (5-day AMC)** | | | | | | | | | | | |
| Low-res. | 0.099 | 20 | 20 | 40 | | | 1 | 0 | 40 | 0.01 | 0.18 |
| Moderate-res. | 0.138 | 40 | 20 | 60 | | | 1 | 0 | 60 | -0.03 | -0.53 |
| High-res. | 0.088 | 120 | 20 | 140 | | | 1 | 0 | 140 | 0.02 | 0.13 |
| **Cross-Calibration** | | | | | | | | | | | |
| CN Substitution | 0.039 | 0 | 0 | 0 | 1,000 | 1,300 | 2 | 1,150 | 1,150 | 0.07 | 0.06 |
| CN Sub. w/ 10-R | 0.036 | 0 | 0 | 0 | 2,000 | 1,800 | 2 | 1,900 | 1,900 | 0.07 | 0.04 |

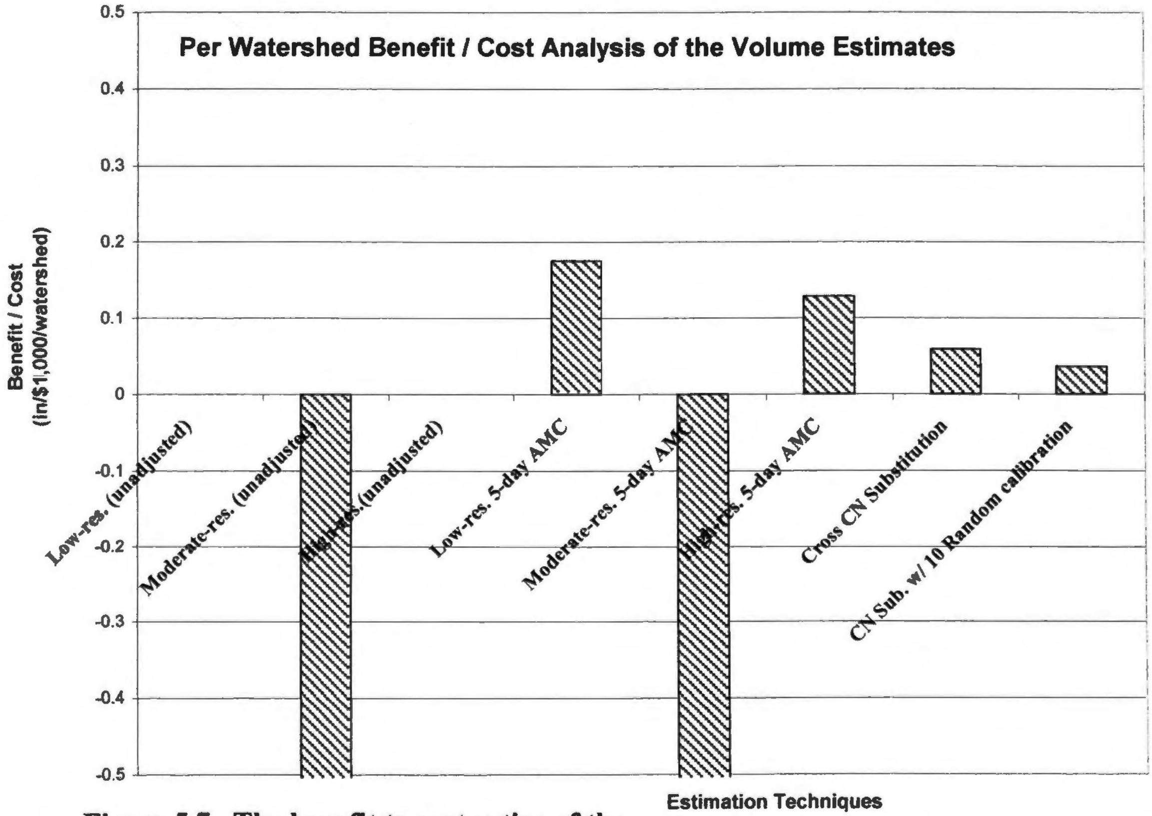



**Figure 5.7. The benefit to cost ratios of the volume calibration techniques.**

# CHAPTER 6

## CONCLUSIONS AND RECOMMENDATIONS

### RESEARCH SUMMARY

This research project was aimed at improving the accuracy of estimations from two simple runoff models: the Rational Method and the SCS CN Method. The research question was whether the accuracy of simple models can be improved while keeping the methods inexpensive and easy to use. The background research indicated that higher-resolution parameters and increased data collection might increase the accuracy of the simple models. This led to the hypothesis that increasing the resolution of the model parameters, and/or calibrating the models with measured hydrologic data, would increase the accuracy of the estimations. Thus, the primary objectives of the research were:

4. Optimize the parameter resolutions and the method of selecting the parameters.

5. Develop calibration techniques to improve the accuracy of the Rational and SCS CN Methods.

6. Identify the most efficient techniques.

To test the hypothesis and fulfill the research objectives, the investigation took a three-pronged approach. The first approach was to evaluate the a priori parameter selection techniques for the Rational and SCS CN Methods. The parameter selection techniques varied from the quick and simple to the time-consuming and complex. The second approach used direct measurements of flow at the target watershed to improve the runoff estimates. Different quantities of data were employed and evaluated in this procedure. The third approach used data from a similar, nearby watershed to assist in the calibration of the target watershed. Again, different quantities of data were used and evaluated.

Attacking the problem from those three angles meant that I had to collect rainfall and runoff data, and assess the physical characteristics from two similar watersheds. Two subwatersheds located in the upper reaches of Second Creek in Knoxville, TN were selected for the project.

### SUMMARY OF PEAK FLOW RESULTS

The data indicated that a priori high-resolution C coefficients did not improve estimation accuracy in this project. In all cases, the low and moderate-resolution C coefficients produced estimates that were more accurate than were the high-resolution estimates. This could have been due to unknown watershed variables that affected the peak flow rate, or perhaps the measured characteristics could not be linearly added because of interactions between the variables. At this time, only a moderate amount of effort appears to be warranted in choosing a C coefficient. Any further effort seems to wastes resources and could lead to less accurate estimates.

The crude $T_c$ estimates produced less accurate estimates than did the intermediate or detailed $T_c$ parameters. Apparently, this method underestimated $T_c$, which led to using a higher than necessary a posteriori rainfall intensity in the Rational Method. The intermediate $T_c$ had lower errors than did the detailed $T_c$, which could be explained by interactions when the separate components were added, or by poor choice of flow paths. However, in both the intermediate and detailed methods, using the kinematic wave formula to account for the overland flow portion of the $T_c$ work worked remarkably well. While it is doubtful that the formula accounted for the same flow section that it was mapped to, it did allow for unknown areas where there were detention or ponding. In addition, the kinematic formula changed the standard linear relationship between rainfall and runoff on the Rational Method into a nonlinear relationship, which seemed more fitting of the true physical character that exists. Thus, a moderate effort that includes the kinematic wave formula is the preferable method to determine the $T_c$.

The calibrated estimates revealed that any observed data reduced the estimation errors, and with a single exception, increased data further decreased the errors. The cost of collecting data, however, rose nonlinearly with respect to the benefits. This resulted in decreased benefit-cost (B-C) ratios after collecting about 10 events. While it appeared that increased data collection quickly reached the point of diminishing returns, it is important to note that in this study, the benefit was simply the amount a technique reduced the calibrated estimate from the best a priori estimate. How these reductions affect the real-life costs of flooding, overdesign of structures, and similar expenses are difficult to assess.

Data collected on a single watershed could perhaps be used for calibrating several watersheds, which would lead to decreased costs per watershed, and thus increasing the B-C ratio. Also, it is extremely likely that accuracy improvements for low-probability events will require a longer period of observed data. Therefore, it is important to collect longer periods of observed data, but at the same time, costs must be reduced. A way this may be done is to further study the crest staff gage that Mokus described (1972a). One of these was used at the Sanford site to confirm the peaks measured by the TFLI, and while it seemed to work fine an in-depth study wasn't performed. Regardless of how the data are collected, the observed runoff information clearly improves peak runoff estimates.

In lieu of observed data on a watershed, local data from a similar watershed apparently is an acceptable substitute. The cross-calibrated estimates plainly show that data from one watershed can improve the estimates on a similar watershed, although the cross-watershed data are not as good as data collected within the watershed. Again, more observations tend to result in lower errors, while the same diminishing returns as previously noted were seen. The matched peak by peak method was the most accurate cross-watershed calibrated method and was fairly cost effective. If accurate estimations on an entire watershed were needed, this technique holds enormous potential. For example, to accurately estimate peak flow rates for the entire Second Creek watershed (1860.4 ha), several representative monitoring areas could be measured and the results applied to the remaining areas. The caveat however, is that this technique is not a predictive model and is only applicable to real-time or post hoc estimations.

## Summary of Volume Estimates

The effect of different parameter resolutions gave mixed results when the estimates were compared to the observed runoff volumes. While it seemed that on both watersheds the detailed CN with AMC adjustments allowed better estimates, a closer examination showed that this was misleading and was probably due to the large number of estimated zero volumes combined with relatively small runoff volumes. This same pattern led to the inability to calibrate the equations by adjusting the estimated values. Thus, for the range of volumes observed in this study, use of any CN based on the characteristics of a watershed is not recommended.

One area of potential for improving the SCS CN estimates lie in the cross-watershed CN substitution method, which allowed volume estimations using the back-calculated CN from the matched events on the other watershed. This simple technique resulted in an error reduction of 63% and had a moderate B-C ratio, so this technique has a great deal of potential. Why did the CN substitution work well and the other techniques work so poorly? While it is only speculative, is seems as though using the cross-watershed CN substitution accounted for much of the temporal and spatial variation of the watersheds, but that the curve number itself did not really characterize those watershed variables that control runoff. Since the watersheds have a karst topography, that may explain some of the difficulty. Further study in this area is necessary to derive the full benefits of using these techniques with the SCS CN method.

## Conclusions

### A Priori Parameters

- Increasing the resolution of the a priori parameters did not improve the estimations.
- The $T_c$ parameter caused more estimation variability than did the C coefficient. This was likely due to the relative homogeneity of the watersheds.
- The kinematic wave equation improved the $T_c$ estimations and added a nonlinear component to the linear Rational Method

- The a priori CN estimates that were adjusted for AMC frequently estimated zero runoff.
- The 5-day AMC did not seem appropriate for these watersheds, and a 2-day AMC might work better.
- The SCS CN method initial abstraction of 20% seems high for these watersheds, and a lower initial abstract might improve the estimation accuracy.

## Direct Flow Measurement Calibrations

- The SCS CN method was unable to be calibrated successfully, which implied that the CN did not represent the runoff characteristics of the watershed very well.
- For peak flow rates, any amount of data improved the accuracy of the estimations, and more data led to further increases in accuracy.
- In the single random event calibration, the amount of improvement in accuracy was dependent upon how close the calibration ratio matched the expected ratio for all events. Since the expected ratio for all events is typically an unknown, the amount of improvement was due to random chance.
- It was important to have at least one large runoff event in the calibration data, but even calibrations that were developed from moderately large events substantially improved estimations on the largest event.
- A broad nonrandom selection of data provided the optimum increases in accuracy, but also had the highest costs.
- A point of diminishing returns was reached regarding the amounts of calibration data, which indicates a nonlinear relationship between cost and accuracy. The biggest "bang for the buck" came from the first few calibration data.

### Cross-watershed Calibrations

- Calibration data from a similar nearby watershed can improve the accuracy for estimating peak flows, but does not work quite as well as data from within the watershed.
- Peak flow calibrations individually developed for each event improved the estimations nearly as well as any of the directly calibrated techniques. The limitation is that these techniques could only be used for real-time or post hoc applications.
- Runoff volumes estimated by substituting the back-calculated CN from one watershed to another substantially reduced estimation errors.
- The previous two bullets imply that there are some variables that changed the runoff characteristics between storm events and that the two watersheds changed in a similar manner.

## RECOMMENDATIONS

The research project answered the three original objectives and in the process developed a methodology that can be applied to systematically improve runoff estimates. However, the project led to unanswered questions that require further investigation to develop the advances already achieved.

The obvious question is whether the techniques can be applied to another local watershed. Similarly, it would be helpful to learn whether these techniques can be applied to a more heterogeneous watershed or different types of watersheds. In addition, further data are needed to check the accuracy for lower probability events. Since there isn't a convenient replacement for observed runoff data, there is need to reduce the cost of collecting and managing data. While peak flows can be observed using the crest staff gage, there is no comparable alternative for observing volumes. At the same time, substituting personnel costs for equipment costs is not a viable option in the U.S. Such research would certainly be beneficial.

A second line of investigation that should prove worthwhile is to examine why the techniques were more readily applied to the Rational Method than to the SCS CN method. Further research needs to be conducted to fully elucidate the failings of the curve number approach. A starting point for such research would surely examine the initial abstraction and different methods of adjusting the AMC. Along similar lines, research into the variation of runoff between storm events is needed. A project to investigate whether a simple parameter could account for this variability is sorely needed.

In summation, this research provides engineers, hydrologists, and others needing uncomplicated and efficient runoff estimates with techniques to increase the models' accuracy. This aids in the sizing of stormwater conveyances, determining mass contaminant loads, making land management decisions, and other actions requiring accurate runoff volumes and peak flows. Because these techniques allow more accurate estimations while maintaining the simplicity and cost effectiveness of the models, it will primarily benefit those in smaller communities, suburban, and rural areas. However, anyone who uses the Rational and SCS Curve Number Methods should find the techniques applicable.

# REFERENCES

# REFERENCES


American Society of Civil Engineers and Water Environment Federation. 1992. Design and Construction of Urban Stormwater Management Systems. ASCE Manuals and Reports of Engineering Practice no. 77 and WEF Manual of Practice FD-20.

Ben-Zvi, A. 1989. Toward a new rational method. *J. Hydraulic Engrg.* 115(9):1241-1255.

Biswas, A.K. 1970. History of Hydrology. Amsterdam: North-Holland.

City of Knoxville, Records and Mapping Department. 1996. GIS drainage map. Map Series 651600. Knoxville, TN.

City of Knoxville, Records and Mapping Department. Undated land use map of the Second Creek watershed. Knoxville, TN.

Choquette, A.F. 1988. Regionalization of Peak Discharges for Streams in Kentucky. Louisville, KY: U.S.G.S. 87-4209.

Chow, V.T. 1964. Runoff. In V. T. Chow (Ed.), *Handbook of Applied Hydrology*. (pp. 14.1-14.54). N.Y.: McGraw Hill.

Clausen J.C., J. Spooner. 1993. Paired watershed study design. Environmental Protection Agency. Office of Water. Washington, D.C.

Clausen, J.C., W.E. Jokela, F.I. Potter, and J.W. Williams. 1996. Paired watershed comparison of tillage effects on runoff, sediment, and pesticide losses. *J. Environ. Qual.* 25(5), 1000-1007.

Cooke. R.A., S. Mostaghimi, P.W. McClellen. 1995. Application of robust regression to the analysis of BMP effects in paired watersheds. *Trans. Amer. Soc. Agric. Engrs.* 38(1), 93-101.

Ganju, C. 1998. Personal conversation about Beaver Creek modeling. Sept., 12, 1998.

Grayson, R.B., I.D. Moore, and T.A. McMahon. 1992. Physically based hydrologic modeling 2. Is the concept realistic? *Water Resources Res.* 26, 2659-2666.

Gregory, R.L. and C.E. Arnold. 1932. Runoff—Rational Runoff Formula. *Trans. Amer. Soc. Civil Engrs.* 96, 1038-1177.

Fontaine, T.A. 1995. Rainfall-runoff model accuracy for an extreme flood. *J. Hydraulic Engrg.* 121:365-374.

Haan, C.T., B.J.Barfield, and J.C.Hayes. 1994. *Design Hydrology and Sedimentology for Small Catchments.* New York: Academic Press.

Horner, W.W and S.W. Jens. 1942. Surface runoff determination from rainfall without using coefficients. *Trans. Amer. Soc.Civil. Engrs.* 107, 1039-1102.

Huggins, L.F., and J.R.Burney. 1982. Surface Runoff, Storage and Routing. In C. T. Haan, H.P.Johnson, and D.L.Brakensink (Eds.), *Hydrologic Modeling of Small Watersheds.* (169-225). St. Joseph: Amer. Soc. Agric. Engrs.

Huber, W.C. 1993. Contaminant transport in surface water. In D. R. Maidment (Ed.), *Handbook of Hydrology.* N.Y.: McGraw Hill.

Jennings, M.E., W.O. Thomas Jr., H.C. Riggs. 1993. Nationwide Summary of U.S. Geological Survey regional regression equations for estimating magnitude and frequency of floods for ungaged sites. Reston, VA.:U.S.G.S.

Jens, S.W. and M.B. McPherson. 1964. Hydrology of Urban Areas. In V. T. Chow (Ed.), *Handbook of Applied Hydrology*. (pp. 20.1-20.44). N.Y.: McGraw Hill.

Johnson, L.R. and Associates. 1992. Floodplain Management in the United States: An Assessment Report. TV-72105A. Washington, D. C. FEMA.

Kung, Hsiang-te. 1980. *Geographic Aspects of the Urban Hydrology of Knoxville*, Tennessee. Doctoral dissertation for the Department of Geography, Univ. of Tenn., Knoxville.

Liebscher, H. 1986. Minimum Requirements for Evaluation of Measurement Results from Small Basins. In *Integrated Design of Hydrological Networks*. Budapest: IAHS.

Linsley, R.K. 1986. Flood estimates: How good are they? *Water Resource Res*. 22, 159-164.

Loague, K.M. and R.A. Freeze. 1985. A comparision of rainfall—runoff modeling techniques on small upland catchments. *Water Resource Res*. 21, 229-248.

McCuen, R.H., S.L.Wong, and W.L.Rawls. 1984. Estimating urban times of concentration. *J. Hydraulic Engrg.* 110 :887-904.

McPherson, M.B. 1969. Some Notes on the Rational Method of Storm Drain Design. Technical Memorandum No.6 Amer. Soc. Civil Engrs. Urban Water Resources Research Program. N.Y.: ASCE.

Mockus, V. 1972a. Hydrologic Soil-Cover Complexes. In V. Mockus (Ed.), *Hydrology*. (9.1-9.11) Washington, D.C.: Soil Conservation Service. U.S.D.A.

Mockus, V. 1972b. Estimation of Direct Runoff from Storm Rainfall. In V. Mockus (Ed.), *Hydrology*. (10.1-10.24) Washington, D.C.: Soil Conservation Service. U.S.D.A.

Overton, D.E. and M.E. Meadows. 1976. *Stormwater Modeling.* New York: Academic Press.

Pilgrim, D.H. 1989. Regional methods for estimation of design floods for small to medium sized drainage areas in Australia. *New Directions for Surface Water Modeling,* (Proceedings of the Baltimore Symposium, May 1989). IAHS publ. no. 181.

Pilgrim, D.H.,and I.Condery. 1993. Flood Runoff. In D. R. Maidment (Ed.), *Handbook of Hydrology*. N.Y.: McGraw Hill.

Potter, S.R., J.C. King, and D.C. Yoder. 1999. Using GIS to evaluate the heterougeneity of an urban watershed for a water quality study. Proceedings of the Ninth Annual Tennessee Water Resources Symposium, April 12-14. Nashville, TN. 2C-43--2C-48.

Schaake, J.G., J.C. Geyer and J.W. Knapp. 1967. Experimental examination of the Rational Method. *J. Hydraulics Div.* Amer. Soc. Civil Engrs. 93, 353-370.

Sheridan, J.M. 1997. Rainfall-Streamflow Relations for Coastal Plain Watersheds. *Applied Engrg. Agriculture*. 13(3):333-334.

Smoley,C.K. 1993. NPDES-Storm Water Sampling Guidance Manual. USEPA Office of Water. CRC Press. San Diego.

Swindel, B.F., and J.E. Douglass. 1984. Describing and testing nonlinear treatment effects in paired watershed experiments. *For. Sci.* 30(2):305-313.

U.S.D.A. National Resource Conservation Service. 1985.Urban Hydrology for Small Watersheds. Technical Release No.55. Washington, D.C.

U.S.D.A. Soil Conservation Service. 1953. Knox County, TN Soil Survey.

U.S. Environmental Protection Agency (USEPA). 1992. The Quality of Our Nation's Water:1992. The 1992 Report to Congress. URL: http://www.epa.gov/owowwtr1/305b/sum1.html.

U.S. Geological Survey (USGS). 1974. Fountain City 7.5-minute Quadrangle.

Ward A.D. 1995. Surface Runoff and Subsurface Flow. In A.D. Ward and W.J. Elliot (Eds.), *Environmental Hydrology*. N.Y.: CRC.

Ward A.D. and W.J Elloit. 1995. The hydrologic cycle, water resources and society. In A.D. Ward and W.J. Elliot (Eds.), *Environmental Hydrology*. N.Y.: CRC.

Wigham, J.S. (1970). Peak flow—Rainfall. In D.M. Gray (Ed.).*Handbook on the Principles of Hydrology*. National Research Council of Canada.

Yoder, D.C., J.R. Buchanan, G.S. Honea, B.F. Staley, J.B. Wilkerson, and R.E. Yoder. 1999. The Tennessee Fluid Level Indicator. *Applied Engrg. Agriculture*. 15(1):49-52.

# APPENDICES

# APPENDIX A.

# $T_C$ REGRESSION EQUATIONS

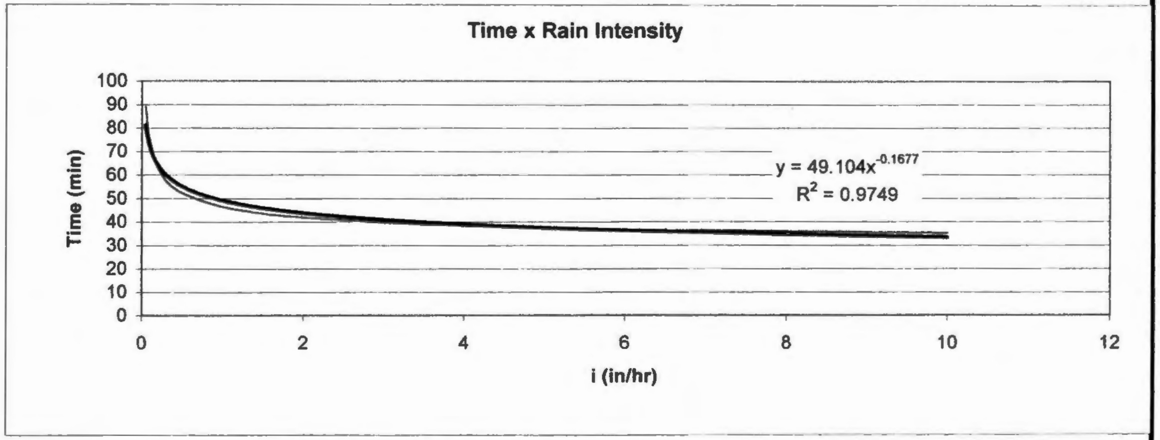
**The Sanford watershed intermediate $T_c$ data and regression equation to determine rainfall intensity.**

*Data source: USGS Fountain City Quadrangle (1:24000 scale)*
*Starting elevation 1300 feet--20 foot contours*

| Segment | Length (ft) | End Elevation (ft) | Δ Elev. (ft) | Slope (ft/ft) | Primary Flow | Roughness (n) | Intensity (in/hr) | Tc |
|---|---|---|---|---|---|---|---|---|
| 1 | 200 | 1240 | 60 | 0.3 | Sheet | 0.4 | 2 | 13.99 |
| 2 | 200 | 1200 | 40 | 0.2 | Concentrated | 0.05 | | 0.46 |
| 3 | 200 | 1181 | 19 | 0.095 | Open Channel | | | 1.14 |
| 4 | 200 | 1171 | 10 | 0.05 | Open Channel | | | 1.46 |
| 5 | 200 | 1160 | 11 | 0.055 | Open Channel | | | 1.41 |
| 6 | 200 | 1145 | 15 | 0.075 | Open Channel | | | 1.25 |
| 7 | 200 | 1140 | 5 | 0.025 | Open Channel | | | 1.91 |
| 8 | 200 | 1135 | 5 | 0.025 | Open Channel | | | 1.91 |
| 9 | 200 | 1130 | 5 | 0.025 | Open Channel | | | 1.91 |
| 10 | 200 | 1125 | 5 | 0.025 | Open Channel | | | 1.91 |
| 11 | 200 | 1120 | 5 | 0.025 | Open Channel | | | 1.91 |
| 12 | 200 | 1100 | 20 | 0.1 | Open Channel | | | 1.12 |
| 13 | 200 | 1095 | 5 | 0.025 | Open Channel | | | 1.91 |
| 14 | 200 | 1090 | 5 | 0.025 | Open Channel | | | 1.91 |
| 15 | 200 | 1085 | 5 | 0.025 | Open Channel | | | 1.91 |
| 16 | 200 | 1080 | 5 | 0.025 | Open Channel | | | 1.91 |
| 17 | 200 | 1075 | 5 | 0.025 | Open Channel | | | 1.91 |
| 18 | 200 | 1070 | 5 | 0.025 | Open Channel | | | 1.91 |
| | | | | | | | Total min. | 42 |

*Tc at various Intensities*

| i (in/hr) | Tc (min) |
|---|---|
| 0.05 | 89 |
| 0.1 | 74.2 |
| 0.25 | 60 |
| 0.5 | 52.2 |
| 1 | 46.3 |
| 1.5 | 43.5 |
| 2 | 41.8 |
| 3 | 39.7 |
| 4 | 38.4 |
| 5 | 37.5 |
| 6 | 36.8 |
| 7 | 36.3 |
| 8 | 35.9 |
| 9 | 35.5 |
| 10 | 35.1 |



**Church Watershed Time of Concentration (Intermediate Method)**

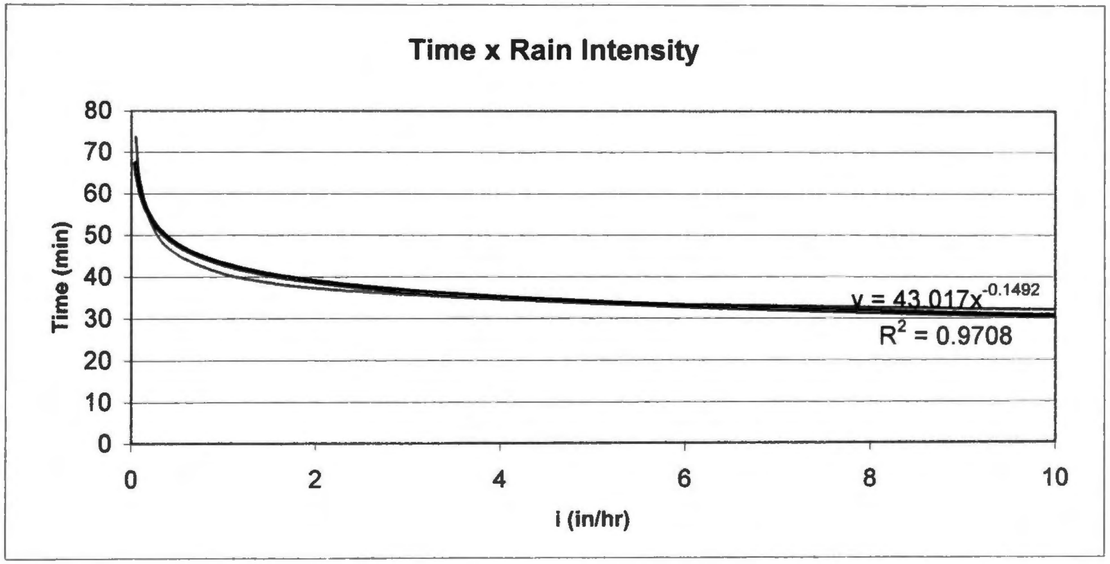
Data source: USGS Fountain City Quadrangle (1:24000 scale)

Starting elevation 1190 feet--20 foot contours

| segment | length (ft) | end elevation (ft | Δ el. (ft) | slope (ft/ft) | primary flow | n | $i_e$ (in/hr) | $T_c$ |
|---|---|---|---|---|---|---|---|---|
| 1 | 300 | 1160 | 30 | 0.1 | Sheet | 0.1 | 2 | 10.7996264 |
| 2 | 600 | 1130 | 30 | 0.05 | Concentrated | 0.05 | | 2.77 |
| 3 | 600 | 1100 | 30 | 0.05 | Open Channel | | | 3.40539594 |
| 4 | 600 | 1090 | 10 | 0.01666667 | Open Channel | | | 5.19827555 |
| 5 | 600 | 1060 | 30 | 0.05 | Open Channel | | | 3.40539594 |
| 6 | 600 | 1040 | 20 | 0.03333333 | Open Channel | | | 3.98073025 |
| 7 | 600 | 1032 | 8 | 0.01333333 | Open Channel | | | 5.66460527 |
| 8 | 150 | 1030 | 2 | 0.01333333 | pipe | | | 1.94797536 |
| | | | | | | | Total min. | 37.172 |

$T_c$ at various $i_e$

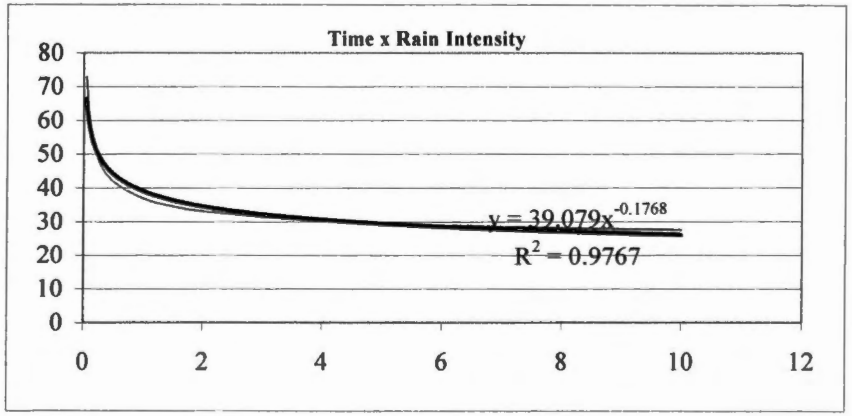
| $i_{e\,(in/hr)}$ | $T_{c\,(min)}$ |
|---|---|
| 0.05 | 73.6 |
| 0.1 | 62.2 |
| 0.25 | 51.2 |
| 0.5 | 45.2 |
| 1 | 40.6 |
| 1.5 | 38.5 |
| 2 | 37.2 |
| 3 | 35.6 |
| 4 | 34.6 |
| 5 | 33.9 |
| 6 | 33.3 |
| 7 | 32.9 |
| 8 | 32.6 |
| 9 | 32.3 |
| 10 | 32 |



## Sanford High Resolution Tc

| GPS_Date | Feat_Name | Depth (ft) | Width (ft) | Shape | Side slope | Area ($ft^2$) | $P_w$ (ft) | R (ft) | material | n | length (ft) | up el. (ft) | low el. (ft) | Δ el (ft) | slope (ft/ft) | Vel. (ft/s) | Time (min) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 4/29/99 16:02 | pipe | 3.83 | 3.83 | round | na | 11.54 | 12.34 | 0.93 | Concrete | 0.014 | 160 | 1073 | 1070 | 3 | 0.019 | 13.93 | 0.19140468 |
| 4/29/99 16:07 | grassy w'way | 3.50 | 3.83 | trap | 1 | 25.67 | 13.73 | 1.87 | med length | 0.055 | 275 | 1080 | 1073 | 7 | 0.025 | 6.56 | 0.69873366 |
| 4/29/99 16:13 | Earth channel | 2.08 | 1.33 | straight | 1 | 7.12 | 7.23 | 0.99 | loose earth | 0.040 | 575 | 1092 | 1080 | 12 | 0.021 | 5.33 | 1.79882578 |
| 4/29/99 16:58 | pipe | 2.00 | 2.00 | round | na | 3.14 | 2.09 | 1.50 | Corr metal | 0.025 | 350 | 1099 | 1092 | 7 | 0.020 | 11.05 | 0.52808358 |
| 4/29/99 17:03 | grassy w'way | 1.83 | 1.67 | trap | 1 | 6.42 | 6.85 | 0.94 | short | 0.045 | 220 | 1104 | 1099 | 5 | 0.023 | 4.78 | 0.76743869 |
| 4/29/99 17:15 | grassy w'way | 1.00 | 0.50 | trap | 1 | 1.50 | 3.33 | 0.45 | med length | 0.060 | 25 | 1105 | 1104 | 1 | 0.040 | 2.92 | 0.14275927 |
| 4/29/99 17:07 | grassy w'way | 1.17 | 0.75 | trap | 1 | 2.24 | 4.05 | 0.55 | long | 0.100 | 230 | 1108 | 1105 | 3 | 0.013 | 1.15 | 3.34766493 |
| 4/29/99 17:16 | Earth channel | 0.67 | 1.25 | meander | 1 | 1.28 | 3.14 | 0.41 | packed earth | 0.100 | 650 | 1122 | 1108 | 14 | 0.022 | 1.20 | 9.01592604 |
| 4/29/99 17:16 | Earth channel | 5.83 | 5.83 | straight | 1 | 68.06 | 22.33 | 3.05 | packed earth | 0.037 | 175 | 1131 | 1122 | 9 | 0.051 | 19.20 | 0.15188718 |
| 4/29/99 17:16 | Earth channel | 2.50 | 2.50 | straight | 1 | 12.50 | 9.57 | 1.31 | packed earth | 0.040 | 365 | 1137 | 1131 | 6 | 0.016 | 5.71 | 1.06597679 |
| 4/29/99 17:16 | Earth channel | 1.67 | 1.67 | straight | 1 | 5.56 | 6.38 | 0.87 | packed earth | 0.040 | 115 | 1138 | 1137 | 1 | 0.009 | 3.17 | 0.6051788 |
| 4/29/99 17:16 | Earth channel | 1.50 | 1.50 | straight | 1 | 4.50 | 5.74 | 0.78 | packed earth | 0.040 | 365 | 1155 | 1138 | 17 | 0.047 | 6.83 | 0.89037309 |
| 4/29/99 17:16 | Earth channel | 1.50 | 1.50 | straight | 1 | 4.50 | 5.74 | 0.78 | packed earth | 0.040 | 500 | 1192 | 1155 | 37 | 0.074 | 8.61 | 0.96763448 |
| 4/29/99 17:16 | culvert | 0.67 | 2.00 | parab | na | 0.89 | 2.59 | 0.34 | concrete | 0.015 | 400 | 1255 | 1192 | 63 | 0.158 | 19.31 | 0.3452297 |
| 4/29/99 17:18 | Concentrated flow | | | | | | | | litter | 0.250 | 215 | 1275 | 1255 | 20 | 0.093 | 5.00 | 0.71666667 |
| | | | | | | | | | | | | | | | | $i_e$ (in/hr) | |
| 4/29/99 17:19 | overland flow | | | | | | | | litter | 0.250 | 200 | 1294 | 1275 | 19 | 0.095 | 1.00 | 15.5711336 |
| | Total time of concentration (min.) | | | | | | | | | | 4820 | | | 224 | 0.045 | | 36.8049169 |

| $i_e$ (in/hr) | 'ime (min) |
|---|---|
| 0.05 | 72.84 |
| 0.1 | 60.35 |
| 0.25 | 48.34 |
| 0.5 | 41.78 |
| 1 | 36.8 |
| 1.5 | 34.47 |
| 2 | 33.03 |
| 3 | 31.27 |
| 4 | 30.18 |
| 5 | 29.41 |
| 6 | 28.83 |
| 7 | 28.38 |
| 8 | 28.01 |
| 9 | 27.7 |
| 10 | 27.43 |



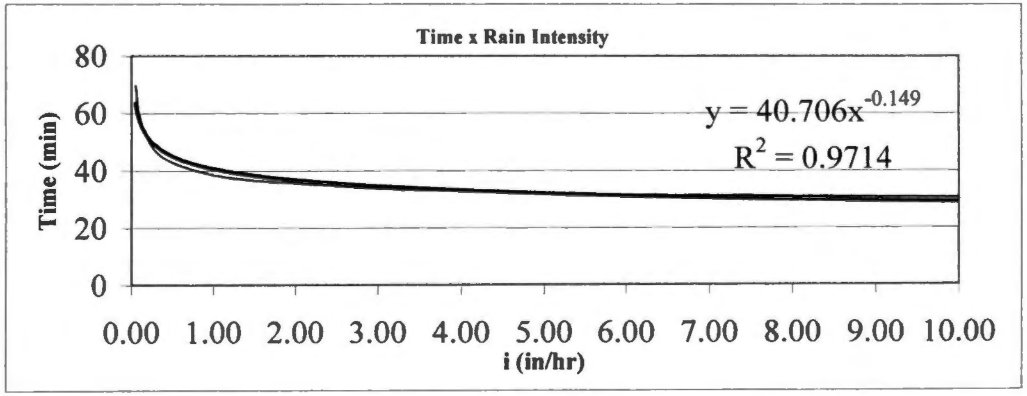
## Church Watershed High Resolution Tc

Channel Characteristics

| *Primary Flow (type)* | *Depth (ft)* | *Width (ft)* | *Shape* | *Side slope* | *Area ($ft^2$)* | *$P_w$ (ft)* | *r (ft)* | *Material* | *n* | *Length (ft)* | *Up elev. (ft)* | *Low elev. (ft)* | *D elev. (ft)* | *Slope (ft/ft)* | *Velocity (ft/s)* | *Time (min)* |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Overland | na | | | na | na | na | na | thick grass | 0.125 | 250.0 | 1182.0 | 1170.0 | 12.0 | 0.048 | | 13.46 |
| Concentrated | 0.40 | | | na | na | na | 0.4 | litter | 0.050 | 170.0 | 1170.0 | 1160.0 | 10.0 | 0.059 | 3.92 | 0.72 |
| Culvert | 1.00 | 1.00 | rectan | na | 1.0 | 3.0 | 0.3 | earth | 0.032 | 125.0 | 1160.0 | 1150.0 | 10.0 | 0.080 | 6.20 | 0.34 |
| Pipe | 1.00 | | round | na | 0.8 | 3.1 | 0.2 | concrete | 0.014 | 400.0 | 1150.0 | 1125.0 | 25.0 | 0.063 | 10.27 | 0.65 |
| Pipe | 1.50 | | round | na | 1.7 | 4.7 | 0.4 | concrete | 0.014 | 350.0 | 1125.0 | 1123.0 | 2.0 | 0.006 | 4.11 | 1.42 |
| Earth channel | 1.50 | 2.00 | straight-trap | 1 | 5.3 | 6.2 | 0.8 | packed earth | 0.040 | 550.0 | 1123.0 | 1094.0 | 29.0 | 0.053 | 7.64 | 1.20 |
| Brick culvert | 3.00 | 3.00 | straight-parab | na | 6.0 | 11.0 | 0.5 | brick | 0.025 | 225.0 | 1094.0 | 1082.0 | 12.0 | 0.053 | 9.22 | 0.41 |
| Concentrated | 1.00 | | spreading | na | na | na | 1.3 | short grass | 0.100 | 100.0 | 1082.0 | 1080.0 | 2.0 | 0.020 | 2.51 | 0.66 |
| Earth channel | 1.50 | 3.00 | straight-trap | 3 | 11.3 | 9.0 | 1.3 | packed earth | 0.037 | 1000.0 | 1080.0 | 1050.0 | 30.0 | 0.030 | 8.12 | 2.05 |
| Concentrated | 0.40 | | spreading | na | na | na | 0.4 | high grass | 0.200 | 175.0 | 1050.0 | 1049.5 | 0.5 | 0.003 | 0.22 | 13.50 |
| Earth channel | 2.00 | 4.00 | trap | 2 | 16.0 | 10.9 | 1.5 | packed earth | 0.040 | 775.0 | 1049.5 | 1032.5 | 17.0 | 0.022 | 7.13 | 1.81 |
| Pipe | 4.25 | | round | na | 14.2 | 13.3 | 1.1 | Corr metal | 0.014 | 950.0 | 1032.5 | 1026.0 | 6.5 | 0.007 | 9.17 | 1.73 |
| Earth channel | 2.25 | 14.50 | straight-trap | 2 | 217.0 | 40.3 | 1.6 | weeds/stones | 0.022 | 210.0 | 1026.0 | 1025.0 | 1.0 | 0.005 | 6.50 | 0.54 |
| | | | | | | | | | | 5280.0 | | | 157.0 | 0.034 | | 38 |

Total time of concentration (min.)

| $i_e$ (in/hr) | Time (min) |
|---|---|
| 0.05 | 70 |
| 0.10 | 59 |
| 0.25 | 48 |
| 0.50 | 43 |
| 1.00 | 38 |
| 1.50 | 36 |
| 2.00 | 36 |
| 3.00 | 34 |
| 4.00 | 33 |
| 5.00 | 32 |
| 6.00 | 32 |
| 7.00 | 31 |
| 8.00 | 31 |
| 9.00 | 31 |
| 10.00 | 30 |



## APPENDIX B.

## PEAK FLOW CALIBRATION EQUATIONS

**Peak by peak ratios for the matched events.**

**Church watershed**

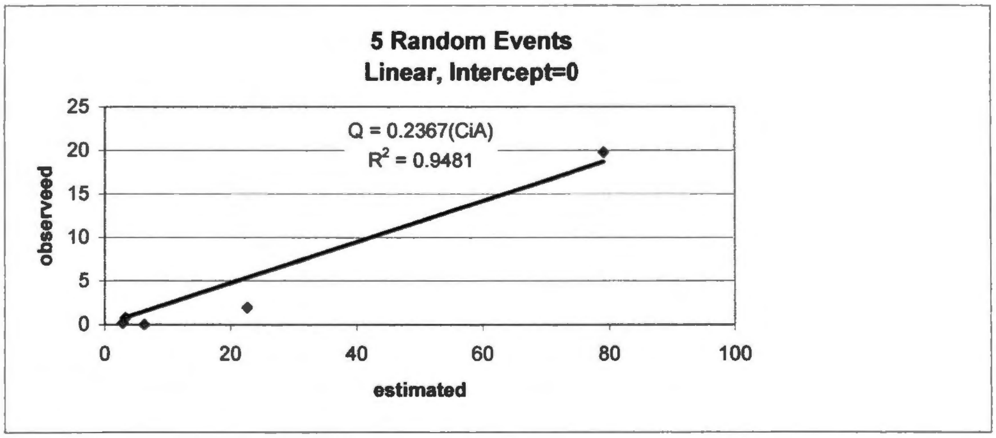
| Event | Date | Observed Peak Flow (cfs) | Moderate C Crude Tc Est (cfs) | Moderate C Crude Tc Obs/est Ratio | Moderate C Intermed. Tc Est (cfs) | Moderate C Intermed. Tc obs/est Ratio | Moderate C Detailed Tc Est (cfs) | Moderate C Detailed Tc obs/est Ratio |
|---|---|---|---|---|---|---|---|---|
| 2 | 10/25/97 | 2.502 | 8.478 | 0.295 | 7.613 | 0.329 | 7.851 | 0.319 |
| 3 | 10/26/97 | 2.893 | 11.511 | 0.251 | 7.463 | 0.388 | 7.851 | 0.368 |
| 4 | 11/1/97 | 1.993 | 8.120 | 0.245 | 7.463 | 0.267 | 7.344 | 0.271 |
| 6 | 11/13/97 | 1.150 | 4.776 | 0.241 | 3.881 | 0.296 | 4.030 | 0.285 |
| 7 | 11/21/97 | 0.032 | 5.493 | 0.006 | 3.582 | 0.009 | 3.732 | 0.009 |
| 11 | 1/16/98 | 0.734 | 5.254 | 0.140 | 4.896 | 0.150 | 4.956 | 0.148 |
| 12 | 1/18/98 | 0.995 | 3.344 | 0.298 | 2.239 | 0.444 | 2.388 | 0.417 |
| 13 | 1/27/98 | 2.161 | 5.254 | 0.409 | 5.224 | 0.412 | 5.284 | 0.407 |
| 14 | 2/2/98 | 0.926 | 6.926 | 0.134 | 5.762 | 0.161 | 5.821 | 0.159 |
| 15 | 2/11/98 | 1.952 | 6.806 | 0.287 | 5.971 | 0.327 | 5.971 | 0.327 |
| 18 | 2/17/98 | 1.246 | 3.224 | 0.387 | 2.687 | 0.464 | 2.746 | 0.454 |
| 19 | 2/17/98 | 2.885 | 8.359 | 0.345 | 7.165 | 0.403 | 7.583 | 0.380 |
| 20 | 5/21/98 | 19.837 | 85.738 | 0.231 | 79.110 | 0.251 | 83.439 | 0.238 |
| 22 | 7/9/98 | 0.224 | 8.359 | 0.027 | 4.627 | 0.048 | 4.627 | 0.048 |
| 24 | 7/31/98 | 1.264 | 7.284 | 0.174 | 6.090 | 0.208 | 6.180 | 0.205 |
| 25 | 8/10/98 | 8.164 | 42.272 | 0.193 | 50.750 | 0.161 | 50.750 | 0.161 |
| 31 | 1/31/99 | 0.212 | 5.612 | 0.038 | 4.776 | 0.044 | 4.926 | 0.043 |
| 32 | 2/17/99 | 0.501 | 6.090 | 0.082 | 5.374 | 0.093 | 5.672 | 0.088 |
| 33 | 2/17/99 | 0.188 | 4.179 | 0.045 | 2.567 | 0.073 | 2.764 | 0.068 |
| 34 | 2/19/99 | 0.196 | 3.582 | 0.055 | 2.836 | 0.069 | 3.105 | 0.063 |
| 35 | 2/27/99 | 1.189 | 7.523 | 0.158 | 6.120 | 0.194 | 6.209 | 0.191 |
| 36 | 3/2/99 | 2.102 | 10.628 | 0.198 | 8.060 | 0.261 | 8.150 | 0.258 |
| 37 | 3/9/99 | 0.797 | 7.523 | 0.106 | 6.299 | 0.127 | 6.329 | 0.126 |
| 38 | 5/5/99 | 0.170 | 8.956 | 0.019 | 5.971 | 0.028 | 6.269 | 0.027 |
| 39 | 5/5/99 | 5.732 | 44.780 | 0.128 | 34.391 | 0.167 | 36.271 | 0.158 |
| 40 | 5/18/99 | 1.092 | 14.688 | 0.074 | 11.640 | 0.094 | 12.240 | 0.089 |

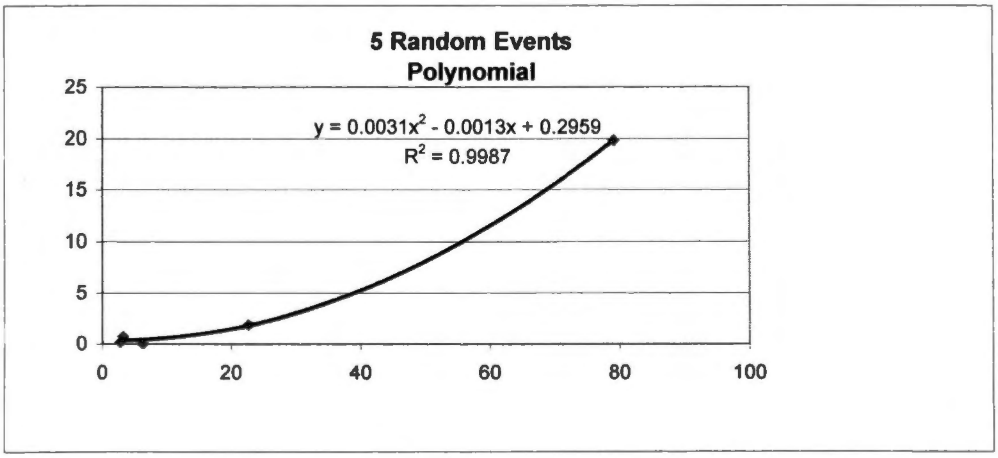
**Sanford Watershed**

| Storm | Date | Observed Peak Flow (cfs) | Moderate C Intermed. Tc Est (cfs) | Church Intermediate Moderate C Intermed. Tc obs/est Ratio |
|---|---|---|---|---|
| 1 | 10/25/97 23:30 | 17.29 | 12.03 | 1.44 |
| 2 | 10/26/97 17:00 | 20.38 | 11.98 | 1.70 |
| 3 | 11/1/97 16:23 | 3.01 | 13.02 | 0.23 |
| 4 | 11/13/97 22:10 | 2.75 | 2.34 | 1.17 |
| 5 | 11/21/97 8:40 | 4.75 | 9.12 | 0.52 |
| 6 | 1/16/98 6:50 | 2.59 | 6.25 | 0.41 |
| 7 | 1/18/98 20:35 | 0.26 | 0.21 | 1.25 |
| 8 | 1/27/98 5:55 | 3.53 | 10.68 | 0.33 |
| 9 | 2/2/98 23:40 | 2.40 | 10.63 | 0.23 |
| 10 | 2/11/98 10:20 | 0.95 | 8.33 | 0.11 |
| 11 | 2/17/98 3:00 | 0.11 | 3.65 | 0.03 |
| 12 | 2/17/98 10:12 | 3.07 | 12.71 | 0.24 |
| 18 | 5/21/98 17:57 | 54.30 | 115.12 | 0.47 |
| 19 | 7/9/98 16:56 | 11.44 | 21.09 | 0.54 |
| 21 | 7/31/98 8:19 | 6.36 | 12.03 | 0.53 |
| 22 | 8/10/98 12:45 | 6.40 | 37.76 | 0.17 |
| 23 | 1/31/99 14:24 | 1.48 | 8.75 | 0.17 |
| 24 | 2/17/99 2:43 | 0.05 | 8.75 | 0.01 |
| 25 | 2/17/99 13:39 | 0.04 | 4.17 | 0.01 |
| 26 | 2/19/99 9:00 | 0.40 | 3.91 | 0.10 |
| 27 | 2/27/99 15:49 | 2.77 | 10.94 | 0.25 |
| 28 | 3/2/99 23:09 | 4.41 | 16.67 | 0.26 |
| 29 | 3/9/99 0:21 | 0.79 | 9.90 | 0.08 |
| 30 | 5/5/99 3:33 | 0.56 | 5.21 | 0.11 |
| 31 | 5/5/99 23:48 | 35.19 | 67.98 | 0.52 |
| 32 | 5/18/99 14:44 | 2.97 | 11.17 | 0.27 |

**Peak flow calibration equations using five random Church events.**

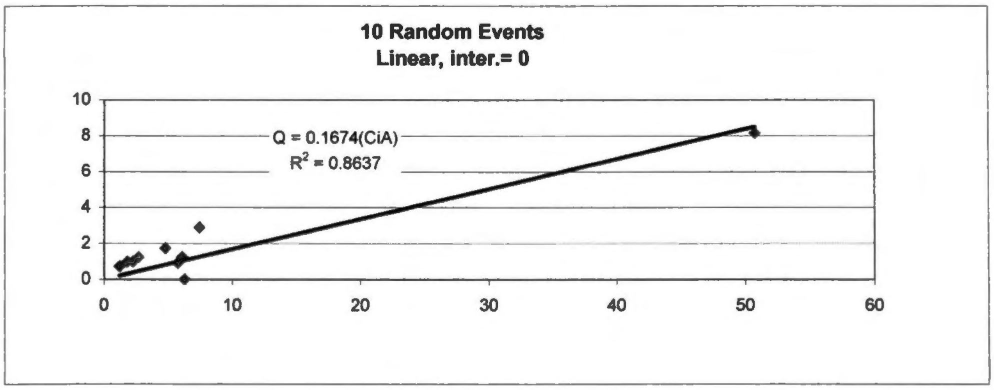
| Storm Event | Runoff Observed (cfs) | Runoff Est. Medium-resolution, Intermediate Tc (cfs) |
|---|---|---|
| 1 | 0.770 | 3.2838 |
| 20 | 19.837 | 79.1104 |
| 26 | 0.018 | 6.2990 |
| 27 | 1.958 | 22.6883 |
| 34 | 0.196 | 2.8360 |

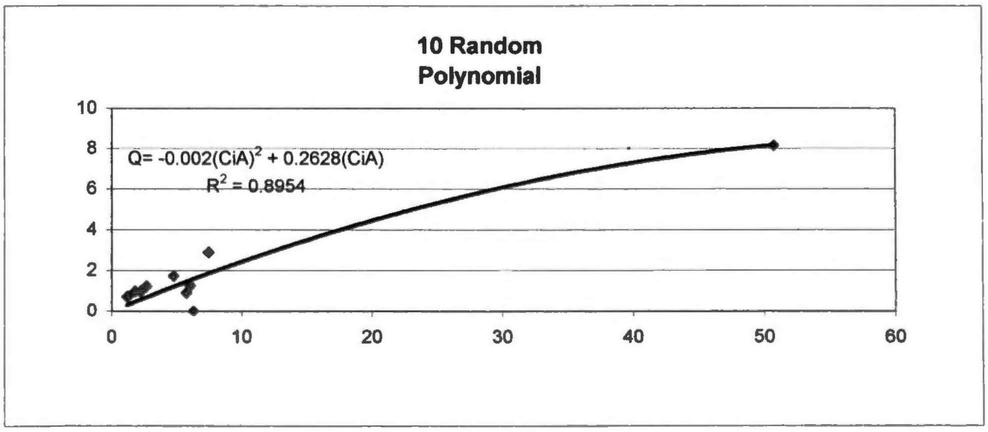


**Peak flow calibration equations using ten random Church events.**

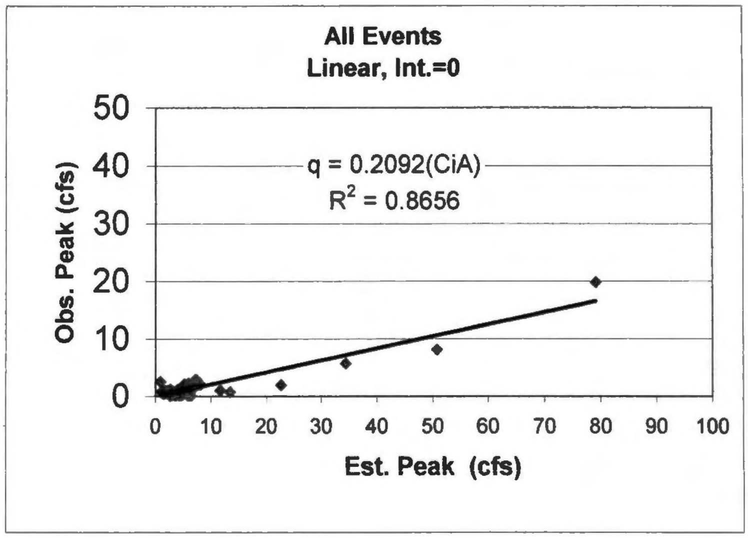
| Storm Event | Runoff Observed (cfs) | Runoff Est. Medium-res C; intermediate $T_c$ (cfs) |
|---|---|---|
| 3 | 2.893 | 7.4633 |
| 8 | 1.743 | 4.7765 |
| 9 | 0.734 | 1.1941 |
| 10 | 0.995 | 1.7912 |
| 12 | 0.995 | 2.2390 |
| 14 | 0.926 | 5.7616 |
| 18 | 1.246 | 2.6868 |
| 24 | 1.264 | 6.0900 |
| 25 | 8.164 | 50.7501 |
| 26 | 0.018 | 6.2990 |

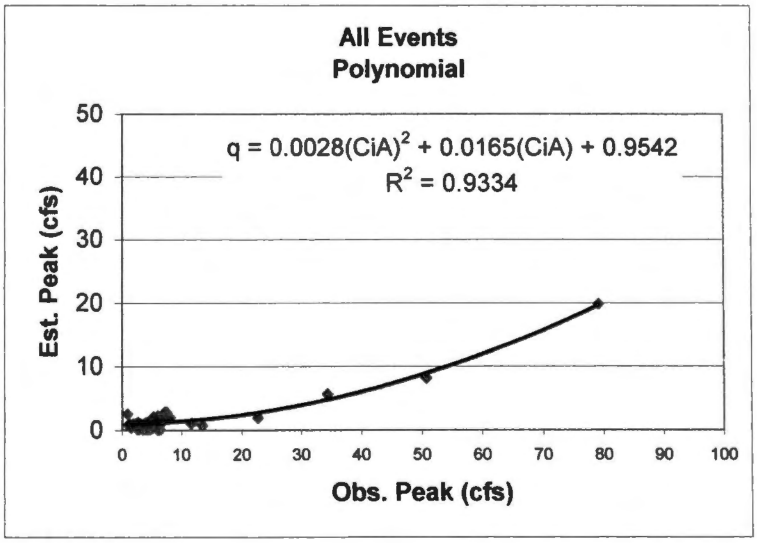


**Calibrating the all of the Church peak flow events for cross-watershed use.**

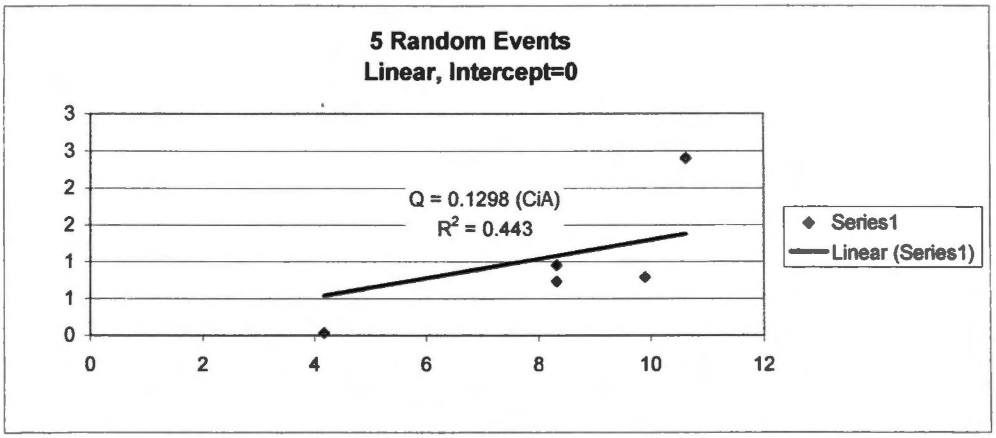
| Storm Event | Church Est | Church Obs. |
|---|---|---|
| 1 | 3.284 | 0.770 |
| 2 | 7.613 | 2.502 |
| 3 | 7.463 | 2.893 |
| 4 | 7.463 | 1.993 |
| 5 | 1.493 | 0.441 |
| 6 | 3.881 | 1.150 |
| 7 | 3.582 | 0.032 |
| 8 | 4.776 | 1.743 |
| 9 | 1.194 | 0.734 |
| 10 | 1.791 | 0.995 |
| 11 | 4.896 | 0.734 |
| 12 | 2.239 | 0.995 |
| 13 | 5.224 | 2.151 |
| 14 | 5.762 | 0.926 |
| 15 | 5.971 | 1.952 |
| 16 | 0.896 | 2.584 |
| 17 | 1.194 | 0.972 |
| 18 | 2.687 | 1.246 |
| 19 | 7.165 | 2.885 |
| 20 | 79.110 | 19.837 |
| 21 | 5.971 | 2.277 |
| 22 | 4.627 | 0.224 |
| 23 | 7.165 | 2.323 |
| 24 | 6.090 | 1.264 |
| 25 | 50.750 | 8.164 |
| 26 | 6.299 | 0.018 |
| 27 | 22.688 | 1.958 |
| 28 | 4.239 | 0.094 |
| 29 | 6.299 | 0.156 |
| 30 | 13.434 | 0.767 |
| 31 | 4.776 | 0.212 |
| 32 | 5.374 | 0.501 |
| 33 | 2.567 | 0.188 |
| 34 | 2.836 | 0.196 |
| 35 | 6.120 | 1.189 |
| 36 | 8.060 | 2.102 |
| 37 | 6.299 | 0.797 |
| 38 | 5.971 | 0.170 |
| 39 | 34.391 | 5.732 |
| 40 | 11.640 | 1.092 |

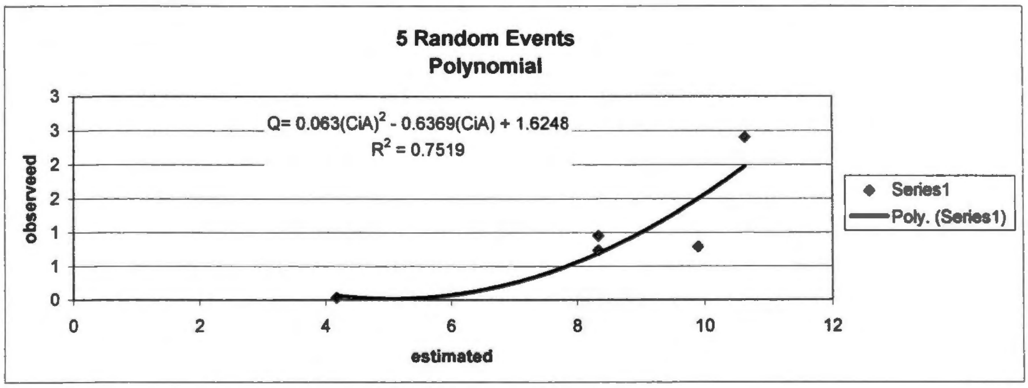


## Sanford 5 random events peak flow calibration equations

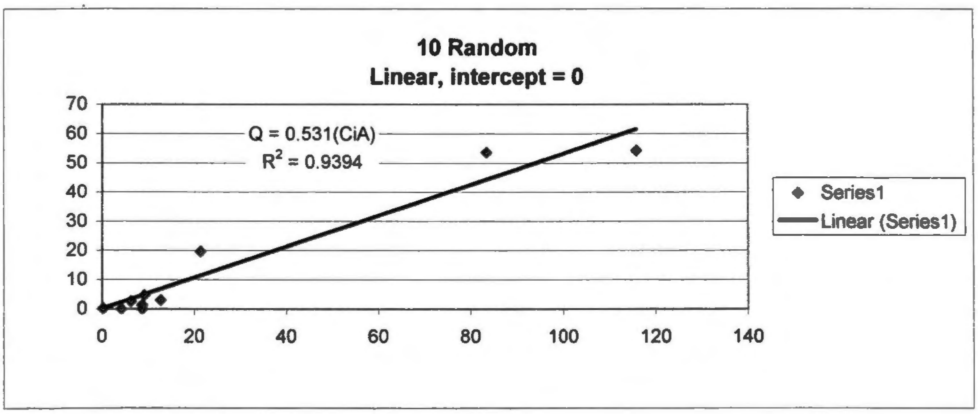
| | Runoff | Runoff Est. |
|---|---|---|
| Storm Event | Observed Peak (cfs) | Medium-resolution, Intermediate Tc (cfs) |
| 9 | 2.402 | 10.6300 |
| 10 | 0.954 | 8.3300 |
| 20 | 0.739 | 8.3300 |
| 25 | 0.035 | 4.1700 |
| 29 | 0.794 | 9.9000 |

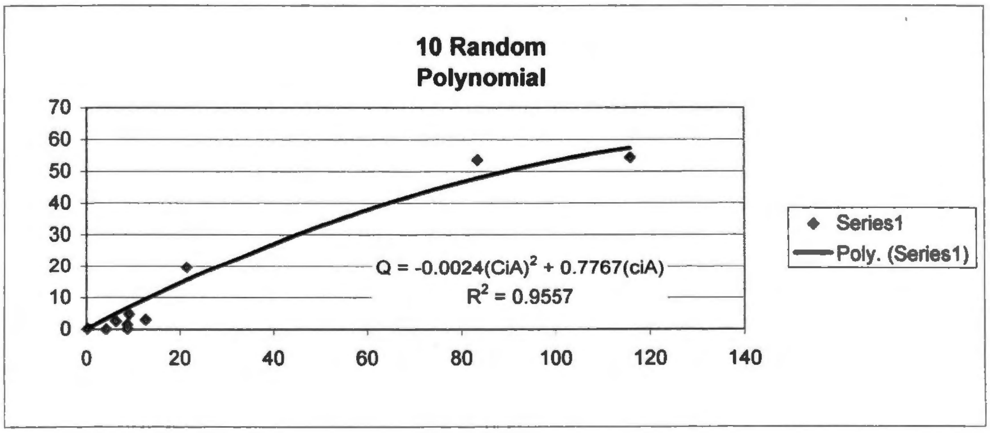


**Sanford 10 random events peak flow calibration equations**

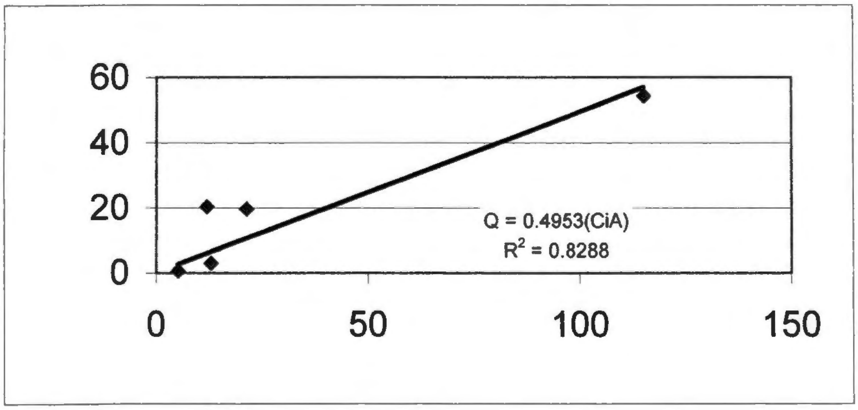
| Storm Event | Runoff Observed Peak (cfs) | Runoff Est. Medium-resolution, Intermediate Tc 0.279 |
|---|---|---|
| 5 | 4.753 | 9.1200 |
| 6 | 2.589 | 6.2500 |
| 7 | 0.260 | 0.2100 |
| 12 | 3.075 | 12.7100 |
| 13 | 19.724 | 21.3600 |
| 15 | 53.663 | 83.3400 |
| 18 | 54.302 | 115.8200 |
| 23 | 1.481 | 8.7500 |
| 24 | 0.051 | 8.7500 |
| 25 | 0.035 | 4.1700 |

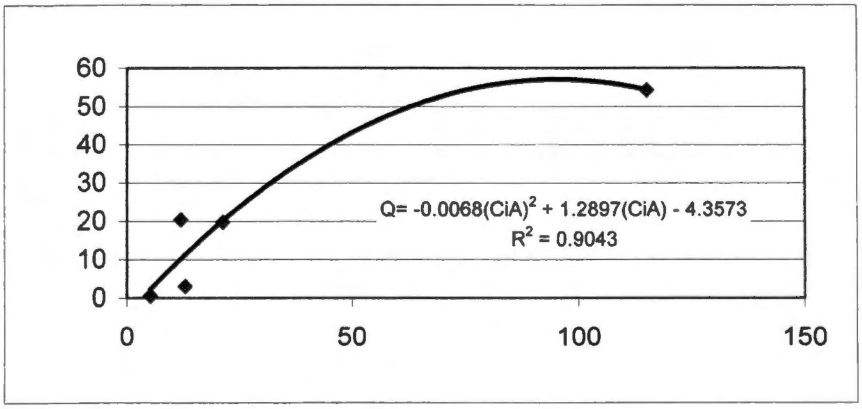


**Sanford 5 nonrandom events peak flow calibration equations.**

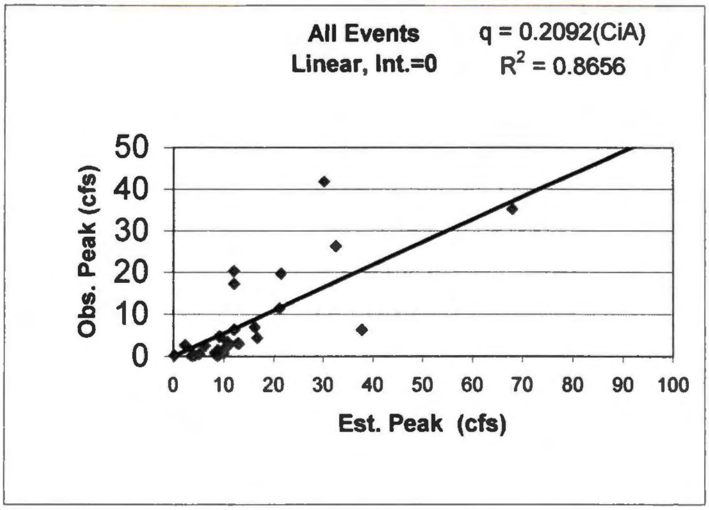
| Storm event | estimated (cfs) | observed (cfs) |
|---|---|---|
| 2 | 11.98 | 20.375 |
| 3 | 13.02 | 3.015 |
| 13 | 21.36 | 19.724 |
| 18 | 115.117 | 54.302 |
| 30 | 5.21 | 0.556 |

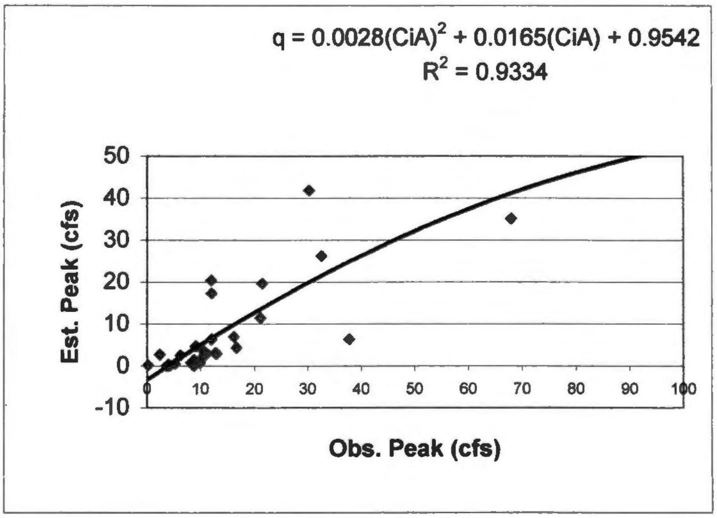


**Calibrating the all of the Sanford peak flow events for cross-watershed use.**

| Storm Event | Sanford Est | Sanford Obs. |
|---|---|---|
| 1 | 12.03 | 17.29 |
| 2 | 11.98 | 20.38 |
| 3 | 13.02 | 3.01 |
| 4 | 2.34 | 2.75 |
| 5 | 9.12 | 4.75 |
| 6 | 6.25 | 2.59 |
| 7 | 0.21 | 0.26 |
| 8 | 10.68 | 3.53 |
| 9 | 10.63 | 2.40 |
| 10 | 8.33 | 0.95 |
| 11 | 3.65 | 0.11 |
| 12 | 12.71 | 3.07 |
| 13 | 21.36 | 19.72 |
| 14 | 32.56 | 26.34 |
| 15 | 83.34 | 53.66 |
| 16 | 30.21 | 41.81 |
| 17 | 16.15 | 7.02 |
| 18 | 115.12 | 54.30 |
| 19 | 21.09 | 11.44 |
| 20 | 8.33 | 0.74 |
| 21 | 12.03 | 6.36 |
| 22 | 37.76 | 6.40 |
| 23 | 8.75 | 1.48 |
| 24 | 8.75 | 0.05 |
| 25 | 4.17 | 0.04 |
| 26 | 3.91 | 0.40 |
| 27 | 10.94 | 2.77 |
| 28 | 16.67 | 4.41 |
| 29 | 9.90 | 0.79 |
| 30 | 5.21 | 0.56 |
| 31 | 67.98 | 35.19 |
| 32 | 11.17 | 2.97 |



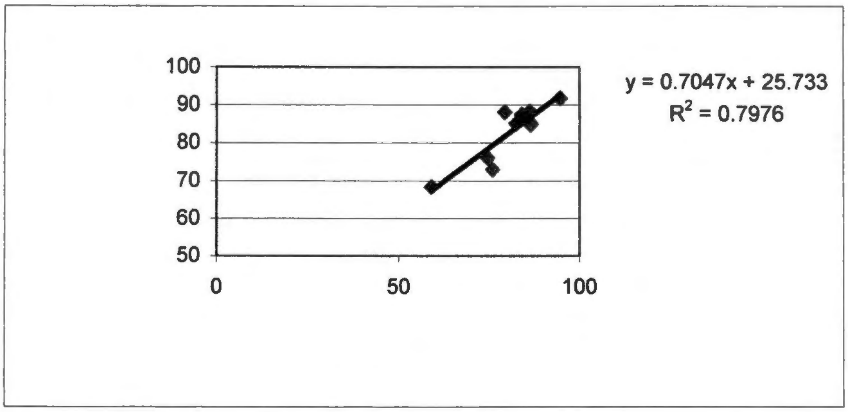
# APPENDIX C.

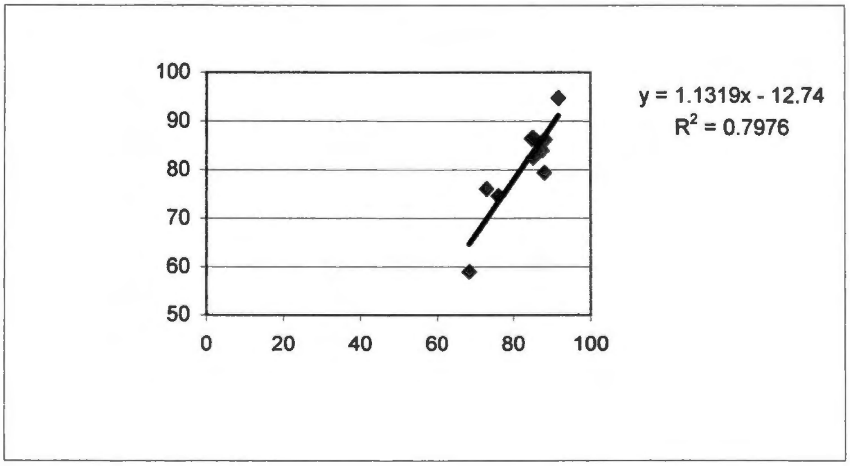
# VOLUME CALIBRATION EQUATIONS

## Cross-watershed volume calibration.

**10 Random Paired Events**

| Storm Event Sanford # | Church # | Date | Sanford Observed CN | Church Observed CN | Sanford Observed CN |
|---|---|---|---|---|---|
| 3 | 4 | 11/1/97 | 84.76 | 86.53 | 84.76 |
| 5 | 7 | 11/21/97 | 88.1 | 79.42 | 88.1 |
| 8 | 13 | 1/27/98 | 76.04 | 74.65 | 76.04 |
| 9 | 14 | 2/2/98 | 72.98 | 76.09 | 72.98 |
| 12 | 19 | 2/17/98 | 91.75 | 94.75 | 91.75 |
| 24 | 32 | 2/17/99 | 85.13 | 86.59 | 85.13 |
| 29 | 37 | 3/9/99 | 85.13 | 82.51 | 85.13 |
| 30 | 38 | 5/5/99 | 87.46 | 84.03 | 87.46 |
| 31 | 39 | 5/5/99 | 68.41 | 59 | 68.41 |
| 32 | 40 | 5/18/99 | 88.26 | 86.27 | 88.26 |



## Sanford paired events volume calibration.

| Sanford Estimates | | | | | | Simple cross CN estimate | | | | Cross CN calibrated w/ 10 random paired events | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Storm Event | | | Sanford | Sanford | Sanford | Church | Sanford | | | Calibrated | (CN = 0.7047$CN_{obs}$ + 25.733) | | |
| Sanford | Church | | Precip | Obs. Vol. | Obs. | Obs. | Adj Est | Abs error | Difference | Church | Adj Est | Abs error | Difference |
| # | # | Date | (in) | (in) | CN | CN | (in) | (in) | % | CN | | | % |
| 2 | 3 | 10/26/97 | 0.350 | 0.29933 | 99.53 | 91.88 | 0.02840 | 0.2709 | 90.5134 | 90.48 | 0.01635 | 0.2830 | 94.5374 |
| 3 | 4 | 11/1/97 | 0.620 | 0.02410 | 84.76 | 86.53 | 0.05108 | 0.0270 | 111.9540 | 86.71 | 0.05323 | | 0.0000 |
| 4 | 6 | 11/13/97 | 0.390 | 0.06304 | 93.31 | 90.08 | 0.02267 | 0.0404 | 64.0348 | 89.21 | 0.01617 | 0.0469 | 74.3459 |
| 5 | 7 | 11/21/97 | 0.598 | 0.06401 | 88.1 | 79.42 | 0.00238 | 0.0616 | 96.2814 | 81.70 | 0.00942 | | 0.0000 |
| 6 | 11 | 1/16/98 | 0.430 | 0.02165 | 88.76 | 90.17 | 0.03451 | 0.0129 | 59.3489 | 89.28 | 0.02588 | 0.0042 | 19.5389 |
| 7 | 12 | 1/18/98 | 0.160 | 0.00132 | 93.9 | 96.04 | 0.01227 | 0.0109 | 828.1755 | 93.41 | 0.00050 | 0.0008 | 62.4697 |
| 8 | 13 | 1/27/98 | 1.270 | 0.10793 | 76.04 | 74.65 | 0.08756 | 0.0204 | 18.8743 | 78.34 | 0.14764 | | 0.0000 |
| 9 | 14 | 2/2/98 | 1.660 | 0.18304 | 72.98 | 76.09 | 0.25493 | 0.0719 | 39.2816 | 79.35 | 0.34713 | | 0.0000 |
| 10 | 15 | 2/11/98 | 0.210 | 0.00416 | 92.95 | 95.27 | 0.02018 | 0.0160 | 384.9628 | 92.87 | 0.00387 | 0.0003 | 7.1147 |
| 11 | 18 | 2/17/98 | 0.130 | 0.00056 | 94.7 | 96.8 | 0.01035 | 0.0098 | 1754.8127 | 93.95 | 0.00000 | 0.0006 | 99.6250 |
| 12 | 19 | 2/17/98 | 0.310 | 0.01647 | 91.75 | 94.75 | 0.05267 | 0.0362 | 219.8186 | 92.50 | 0.02283 | | 0.0000 |
| 18 | 20 | 5/21/98 | 1.760 | 0.22558 | 73.21 | 71.73 | 0.19221 | 0.0334 | 14.7903 | 76.28 | 0.30496 | 0.0794 | 35.1905 |
| 19 | 22 | 7/9/98 | 0.390 | 0.01574 | 89.15 | 92.89 | 0.05600 | 0.0403 | 255.8226 | 91.19 | 0.03333 | 0.0176 | 111.7563 |
| 21 | 24 | 7/31/98 | 0.410 | 0.02092 | 89.26 | 92.19 | 0.05320 | 0.0323 | 154.3233 | 90.70 | 0.03413 | 0.0132 | 63.1304 |
| 22 | 25 | 8/10/98 | 0.580 | 0.01217 | 82.82 | 84.27 | 0.02060 | 0.0084 | 69.2577 | 85.12 | 0.02681 | 0.0146 | 120.2521 |
| 23 | 31 | 1/31/99 | 0.540 | 0.01474 | 84.47 | 80.87 | 0.00184 | 0.0129 | 87.5189 | 82.72 | 0.00676 | 0.0080 | 54.1289 |
| 24 | 32 | 2/17/99 | 0.360 | 0.00007 | 85.13 | 86.56 | 0.00153 | 0.0015 | 2220.6497 | 86.73 | 0.00184 | | 0.0000 |
| 25 | 33 | 2/17/99 | 0.260 | 0.00005 | 88.8 | 90.19 | 0.00160 | 0.0015 | 3170.7847 | 89.29 | 0.00033 | 0.0003 | 579.5770 |
| 26 | 34 | 2/19/99 | 0.220 | 0.00257 | 92.09 | 90.82 | 0.00031 | 0.0023 | 87.9717 | 89.73 | 0.00007 | 0.0025 | 97.3407 |
| 27 | 35 | 2/27/99 | 0.870 | 0.03113 | 78.15 | 75.84 | 0.01586 | 0.0153 | 49.0490 | 79.18 | 0.03980 | 0.0087 | 27.8295 |
| 28 | 36 | 3/2/99 | 1.440 | 0.04401 | 67.61 | 66.01 | 0.03026 | 0.0137 | 31.2410 | 72.25 | 0.10002 | 0.0560 | 127.2815 |
| 29 | 37 | 3/9/99 | 0.430 | 0.00358 | 85.14 | 82.51 | 0.00002 | 0.0036 | 99.5182 | 83.88 | 0.00106 | | 0.0000 |
| 30 | 38 | 5/5/99 | 0.340 | 0.00191 | 87.46 | 84.03 | 0.00086 | 0.0010 | 54.7122 | 84.95 | 0.00012 | | 0.0000 |
| 31 | 39 | 5/5/99 | 2.320 | 0.32410 | 68.41 | 59 | 0.10981 | 0.2143 | 66.1194 | 67.31 | 0.29313 | | 0.0000 |
| 32 | 40 | 5/18/99 | 0.360 | 0.00621 | 88.26 | 86.27 | 0.00106 | 0.0051 | 82.8682 | 86.53 | 0.00147 | | 0.0000 |
| **Averages** | | | | **0.04651** | | | **0.0332** | 0.0133 | 28.6367 | | **0.0406** | 0.0059 | 12.7144 |
| **Average Absolute Error** | | | | | | | | **0.0385** | | | | **0.0357** | |

## Church paired events volume calibration.

| **Church Estimates** | | | | | | Simple cross CN estimate | | | | Cross CN calibrated w/ 10 random paired events | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Storm Event | | | Church | Church | Church | Sanford | Church | | | Calibrated | (CN = 1.1319$CN_{obs}$ - 12.74) | | |
| Sanford | Church | | Precip. | Obs. Vol. | Obs. | Obs. | Adj Est | Abs error | Difference | Sanford | Adj Est | Abs error | Difference |
| # | # | Date | (in) | (in) | CN | CN | (in) | (in) | % | CN | (in) | (in) | % |
| 2 | 3 | 10/26/97 | 0.36 | 0.018798 | 91.88 | 99.53 | 0.30894 | 0.2901 | 1543.44 | 99.92 | 0.35034 | 0.3315 | 1763.65 |
| 3 | 4 | 11/1/97 | 0.62 | 0.030557 | 86.53 | 84.76 | 0.05108 | 0.0205 | 67.15 | 83.20 | 0.02090 | | 0.00 |
| 4 | 6 | 11/13/97 | 0.49 | 0.031747 | 90.08 | 93.31 | 0.02267 | 0.0091 | 28.59 | 92.88 | 0.10269 | 0.0709 | 223.46 |
| 5 | 7 | 11/21/97 | 0.55 | 0.000228 | 79.42 | 88.1 | 0.00238 | 0.0022 | 945.91 | 86.98 | 0.03595 | | 0.00 |
| 6 | 11 | 1/16/98 | 0.54 | 0.043889 | 90.17 | 88.76 | 0.03451 | 0.0094 | 21.38 | 87.73 | 0.04081 | 0.0031 | 7.02 |
| 7 | 12 | 1/18/98 | 0.2 | 0.015576 | 96.04 | 93.9 | 0.01227 | 0.0033 | 21.21 | 93.55 | 0.00511 | 0.0105 | 67.18 |
| 8 | 13 | 1/27/98 | 1.21 | 0.042904 | 74.65 | 76.04 | 0.08756 | 0.0447 | 104.09 | 73.33 | 0.05653 | | 0.00 |
| 9 | 14 | 2/2/98 | 1.83 | 0.198714 | 76.09 | 72.98 | 0.25493 | 0.0562 | 28.29 | 69.87 | 0.17722 | | 0.00 |
| 10 | 15 | 2/11/98 | 0.23 | 0.016302 | 95.27 | 92.95 | 0.02018 | 0.0039 | 23.81 | 92.47 | 0.00511 | 0.0112 | 68.63 |
| 11 | 18 | 2/17/98 | 0.16 | 0.012445 | 96.8 | 94.7 | 0.01035 | 0.0021 | 16.86 | 94.45 | 0.00287 | 0.0096 | 76.96 |
| 12 | 19 | 2/17/98 | 0.28 | 0.023677 | 94.75 | 91.75 | 0.05267 | 0.0290 | 122.44 | 91.11 | 0.00680 | | 0.00 |
| 18 | 20 | 5/21/98 | 1.64 | 0.090535 | 71.73 | 73.21 | 0.19221 | 0.1017 | 112.31 | 70.13 | 0.12301 | 0.0325 | 35.87 |
| 19 | 22 | 7/9/98 | 0.18 | 0.000547 | 92.89 | 89.15 | 0.05600 | 0.0555 | 10128.64 | 88.17 | 0.00623 | 0.0057 | 1038.07 |
| 21 | 24 | 7/31/98 | 0.26 | 0.005232 | 92.19 | 89.26 | 0.05320 | 0.0480 | 916.97 | 88.29 | 0.00002 | 0.0052 | 99.61 |
| 22 | 25 | 8/10/98 | 0.6 | 0.0245 | 84.27 | 82.82 | 0.02060 | 0.0039 | 15.91 | 81.00 | 0.00693 | 0.0176 | 71.72 |
| 23 | 31 | 1/31/99 | 0.6 | 0.003862 | 80.87 | 84.47 | 0.00184 | 0.0020 | 52.36 | 82.87 | 0.01546 | 0.0116 | 300.24 |
| 24 | 32 | 2/17/99 | 0.39 | 0.002364 | 86.56 | 85.13 | 0.00153 | 0.0008 | 35.39 | 83.62 | 0.00000 | | 0.00 |
| 25 | 33 | 2/17/99 | 0.27 | 0.001441 | 90.19 | 88.8 | 0.00160 | 0.0002 | 10.71 | 87.77 | 0.00005 | 0.0014 | 96.28 |
| 26 | 34 | 2/19/99 | 0.25 | 0.001291 | 90.82 | 92.09 | 0.00031 | 0.0010 | 76.02 | 91.50 | 0.00414 | 0.0028 | 220.76 |
| 27 | 35 | 2/27/99 | 0.88 | 0.010288 | 75.84 | 78.15 | 0.01586 | 0.0056 | 54.19 | 75.72 | 0.01653 | 0.0062 | 60.63 |
| 28 | 36 | 3/2/99 | 1.45 | 0.018962 | 66.01 | 67.61 | 0.03026 | 0.0113 | 59.58 | 63.79 | 0.01652 | 0.0024 | 12.88 |
| 29 | 37 | 3/9/99 | 0.56 | 0.004916 | 82.51 | 85.14 | 0.00002 | 0.0049 | 99.65 | 83.63 | 0.01336 | | 0.00 |
| 30 | 38 | 5/5/99 | 0.41 | 0.000279 | 84.03 | 87.46 | 0.00086 | 0.0006 | 210.24 | 86.26 | 0.00495 | | 0.00 |
| 31 | 39 | 5/5/99 | 2.01 | 0.030344 | 59 | 68.41 | 0.10981 | 0.0795 | 261.88 | 64.69 | 0.13231 | | 0.00 |
| 32 | 40 | 5/18/99 | 0.39 | 0.001851 | 86.27 | 88.26 | 0.00106 | 0.0008 | 42.48 | 87.16 | 0.00580 | | 0.00 |
| **Averages** | | | | **0.02525** | | | **0.05371** | 0.0285 | 112.71 | | **0.0464** | 0.0211 | 83.71 |
| **Average Absolute Error** | | | | | | | | **0.0314** | | | | **0.0209** | |

# APPENDIX D.

# PEAK FLOW ESTIMATES FROM THE A PRIORI PARAMETERS

**Sanford estimated peak flows using the a priori paramets. For each observed event, a crude, intermediate and detailed Tc was estimated. Rainfall intensities for each Tc were derived from the measured rain data. These intensities were cross-multiplied to the low, moderate, and high-resolution C coefficients.**

| Storm Event | Date | $T_c$ Method | $T_c$ (min) | Rainfall Intensity (in/hr) | Observed Peak Flow (cfs) | C Coefficient Resolution Low | Moderate | High |
|---|---|---|---|---|---|---|---|---|
| | | | | | | Estimated Peak Runoff (cfs) | | |
| 1 | 10/25/97 | Crude | 27.0 | 0.34 | 17.3 | 18.3 | 17.0 | 20.0 |
| 1 | 10/25/97 | Intermediate | 62.8 | 0.23 | 17.3 | 12.4 | 12.0 | 13.1 |
| 1 | 10/25/97 | Detailed | 49.8 | 0.26 | 17.3 | 13.7 | 13.3 | 14.5 |
| 2 | 10/26/97 | Crude | 27.0 | 0.42 | 20.4 | 22.6 | 21.9 | 23.8 |
| 2 | 10/26/97 | Intermediate | 62.8 | 0.23 | 20.4 | 12.4 | 12.0 | 13.1 |
| 2 | 10/26/97 | Detailed | 49.3 | 0.27 | 20.4 | 14.5 | 14.1 | 15.3 |
| 3 | 11/1/97 | Crude | 27.0 | 0.39 | 3.0 | 20.9 | 20.3 | 22.1 |
| 3 | 11/1/97 | Intermediate | 62.0 | 0.25 | 3.0 | 13.4 | 13.0 | 14.2 |
| 3 | 11/1/97 | Detailed | 48.6 | 0.29 | 3.0 | 15.6 | 15.1 | 16.5 |
| 4 | 11/13/97 | Crude | 27.0 | 0.16 | 2.8 | 8.6 | 8.3 | 9.1 |
| 4 | 11/13/97 | Intermediate | 82.6 | 0.05 | 2.8 | 2.4 | 2.3 | 2.6 |
| 4 | 11/13/97 | Detailed | 59.8 | 0.09 | 2.8 | 4.8 | 4.7 | 5.1 |
| 5 | 11/21/97 | Crude | 27.0 | 0.30 | 4.8 | 16.2 | 15.7 | 17.1 |
| 5 | 11/21/97 | Intermediate | 65.8 | 0.18 | 4.8 | 9.4 | 9.1 | 9.9 |
| 5 | 11/21/97 | Detailed | 52.4 | 0.19 | 4.8 | 10.2 | 9.9 | 10.8 |
| 6 | 1/16/98 | Crude | 27.0 | 0.19 | 2.6 | 10.0 | 9.7 | 10.6 |
| 6 | 1/16/98 | Intermediate | 70.1 | 0.12 | 2.6 | 6.4 | 6.3 | 6.8 |
| 6 | 1/16/98 | Detailed | 55.3 | 0.14 | 2.6 | 7.5 | 7.3 | 7.9 |
| 7 | 1/18/98 | Crude | 27.0 | 0.01 | 0.3 | 0.5 | 0.5 | 0.6 |
| 7 | 1/18/98 | Intermediate | 124.0 | 0.00 | 0.3 | 0.2 | 0.2 | 0.2 |
| 7 | 1/18/98 | Detailed | 101.6 | 0.00 | 0.3 | 0.2 | 0.2 | 0.3 |
| 8 | 1/27/98 | Crude | 27.0 | 0.23 | 3.5 | 12.6 | 12.2 | 13.3 |
| 8 | 1/27/98 | Intermediate | 64.1 | 0.21 | 3.5 | 11.0 | 10.7 | 10.7 |
| 8 | 1/27/98 | Detailed | 51.1 | 0.22 | 3.5 | 11.8 | 11.5 | 12.5 |
| 9 | 2/2/98 | Crude | 27.0 | 0.26 | 2.4 | 13.7 | 13.3 | 14.5 |
| 9 | 2/2/98 | Intermediate | 64.1 | 0.20 | 2.4 | 11.0 | 10.6 | 11.6 |
| 9 | 2/2/98 | Detailed | 50.7 | 0.23 | 2.4 | 12.4 | 12.0 | 13.1 |
| 10 | 2/11/98 | Crude | 27.0 | 0.21 | 1.0 | 11.4 | 11.0 | 12.0 |
| 10 | 2/11/98 | Intermediate | 66.8 | 0.16 | 1.0 | 8.6 | 8.3 | 9.1 |
| 10 | 2/11/98 | Detailed | 52.7 | 0.19 | 1.0 | 9.9 | 9.6 | 10.5 |
| 11 | 2/17/98 | Crude | 27.0 | 0.09 | 0.1 | 4.9 | 4.8 | 5.2 |
| 11 | 2/17/98 | Intermediate | 76.7 | 0.07 | 0.1 | 3.8 | 3.6 | 4.0 |
| 11 | 2/17/98 | Detailed | 62.2 | 0.07 | 0.1 | 3.9 | 3.8 | 4.1 |
| 12 | 2/17/98 | Crude | 27.0 | 0.34 | 3.1 | 18.0 | 17.5 | 19.1 |
| 12 | 2/17/98 | Intermediate | 62.2 | 0.24 | 3.1 | 13.1 | 12.7 | 13.8 |
| 12 | 2/17/98 | Detailed | 49.3 | 0.27 | 3.1 | 14.5 | 14.1 | 15.3 |
| 13 | 4/3/98 | Crude | 27.0 | 0.76 | 19.7 | 40.8 | 39.6 | 43.1 |
| 13 | 4/3/98 | Intermediate | 57.0 | 0.41 | 19.7 | 22.0 | 21.4 | 23.3 |
| 13 | 4/3/98 | Detailed | 44.3 | 0.50 | 19.7 | 26.6 | 25.8 | 28.1 |
| 14 | 4/8/98 | Crude | 27.0 | 0.98 | 26.3 | 52.4 | 50.8 | 55.4 |
| 14 | 4/8/98 | Intermediate | 53.1 | 0.63 | 26.3 | 33.6 | 32.6 | 35.5 |
| 14 | 4/8/98 | Detailed | 40.8 | 0.78 | 26.3 | 41.9 | 40.6 | 44.3 |
| 15 | 4/16/98 | Crude | 27.0 | 1.90 | 53.7 | 101.9 | 98.9 | 107.7 |
| 15 | 4/16/98 | Intermediate | 47.0 | 1.30 | 53.7 | 69.8 | 67.7 | 73.8 |
| 15 | 4/16/98 | Detailed | 36.0 | 1.60 | 53.7 | 85.9 | 83.3 | 90.8 |
| 16 | 4/18/98 | Crude | 27.0 | 0.72 | 41.8 | 38.4 | 37.3 | 40.6 |
| 16 | 4/18/98 | Intermediate | 53.8 | 0.58 | 41.8 | 31.1 | 30.2 | 32.9 |
| 16 | 4/18/98 | Detailed | 42.3 | 0.64 | 41.8 | 34.1 | 33.1 | 36.0 |
| 17 | 5/1/98 | Crude | 27.0 | 0.64 | 7.0 | 34.2 | 33.1 | 36.1 |
| 17 | 5/1/98 | Intermediate | 59.8 | 0.31 | 7.0 | 16.6 | 16.1 | 17.6 |
| 17 | 5/1/98 | Detailed | 48.3 | 0.30 | 7.0 | 16.1 | 15.6 | 17.0 |
| 18 | 5/21/98 | Crude | 27.0 | 3.14 | 54.3 | 168.6 | 163.6 | 178.2 |
| 18 | 5/21/98 | Intermediate | 43.0 | 2.21 | 54.3 | 118.7 | 115.1 | 125.4 |
| 18 | 5/21/98 | Detailed | 32.7 | 2.72 | 54.3 | 146.1 | 141.7 | 154.3 |
| 19 | 7/9/98 | Crude | 27.0 | 0.00 | 11.4 | 0.0 | 0.0 | 0.0 |
| 19 | 7/9/98 | Intermediate | 47.4 | 1.23 | 11.4 | 66.1 | 64.1 | 69.8 |
| 19 | 7/9/98 | Detailed | 37.7 | 1.23 | 11.4 | 66.1 | 64.1 | 69.8 |

| Storm Event | Date | $T_c$ Method | $T_c$ (min) | Rainfall Intensity (in/hr) | Observed Peak Flow (cfs) | C Coefficient Resolution Low | Moderate | High |
|---|---|---|---|---|---|---|---|---|
| | | | | | | Estimated Peak Runoff (cfs) | | |
| 20 | 7/31/98 | Crude | 27.0 | 0.27 | 0.7 | 14.5 | 14.1 | 15.3 |
| 20 | 7/31/98 | Intermediate | 66.8 | 0.16 | 0.7 | 8.6 | 8.3 | 9.1 |
| 20 | 7/31/98 | Detailed | 52.1 | 0.20 | 0.7 | 10.6 | 10.3 | 11.2 |
| 21 | 7/31/98 | Crude | 27.0 | 0.35 | 6.4 | 18.9 | 18.3 | 20.0 |
| 21 | 7/31/98 | Intermediate | 62.8 | 0.23 | 6.4 | 12.4 | 12.0 | 13.1 |
| 21 | 7/31/98 | Detailed | 49.8 | 0.25 | 6.4 | 13.7 | 13.3 | 14.4 |
| 22 | 8/10/98 | Crude | 27.0 | 1.37 | 6.4 | 73.5 | 71.3 | 77.6 |
| 22 | 8/10/98 | Intermediate | 47.6 | 1.20 | 6.4 | 64.4 | 62.5 | 68.1 |
| 22 | 8/10/98 | Detailed | 37.6 | 1.20 | 6.4 | 64.4 | 62.5 | 68.1 |
| 23 | 1/31/99 | Crude | 27.0 | 0.23 | 1.5 | 12.5 | 12.1 | 13.2 |
| 23 | 1/31/99 | Intermediate | 66.2 | 0.17 | 1.5 | 9.0 | 8.8 | 9.5 |
| 23 | 1/31/99 | Detailed | 53.7 | 0.17 | 1.5 | 8.9 | 8.6 | 9.4 |
| 24 | 2/17/99 | Crude | 27.0 | 0.21 | 0.1 | 11.4 | 11.0 | 12.0 |
| 24 | 2/17/99 | Intermediate | 66.2 | 0.17 | 0.1 | 9.0 | 8.8 | 9.5 |
| 24 | 2/17/99 | Detailed | 53.5 | 0.17 | 0.1 | 9.1 | 8.9 | 9.6 |
| 25 | 2/17/99 | Crude | 27.0 | 0.16 | 0.0 | 8.6 | 8.3 | 9.1 |
| 25 | 2/17/99 | Intermediate | 75.0 | 0.08 | 0.0 | 4.3 | 4.2 | 4.5 |
| 25 | 2/17/99 | Detailed | 59.8 | 0.09 | 0.0 | 4.8 | 4.7 | 5.1 |
| 26 | 2/19/99 | Crude | 27.0 | 0.11 | 0.4 | 6.1 | 5.9 | 6.5 |
| 26 | 2/19/99 | Intermediate | 75.8 | 0.08 | 0.4 | 4.0 | 3.9 | 4.3 |
| 26 | 2/19/99 | Detailed | 58.6 | 0.10 | 0.4 | 5.4 | 5.3 | 5.7 |
| 27 | 2/27/99 | Crude | 27.0 | 0.26 | 2.8 | 13.7 | 13.3 | 14.5 |
| 27 | 2/27/99 | Intermediate | 63.8 | 0.21 | 2.8 | 11.3 | 10.9 | 11.9 |
| 27 | 2/27/99 | Detailed | 50.6 | 0.23 | 2.8 | 12.5 | 12.1 | 13.2 |
| 28 | 3/2/99 | Crude | 27.0 | 0.49 | 4.4 | 26.4 | 25.6 | 27.9 |
| 28 | 3/2/99 | Intermediate | 59.4 | 0.32 | 4.4 | 17.2 | 16.7 | 18.2 |
| 28 | 3/2/99 | Detailed | 46.8 | 0.36 | 4.4 | 19.4 | 18.9 | 20.5 |
| 29 | 3/9/99 | Crude | 27.0 | 0.23 | 0.8 | 12.2 | 11.9 | 12.9 |
| 29 | 3/9/99 | Intermediate | 64.9 | 0.19 | 0.8 | 10.2 | 9.9 | 10.8 |
| 29 | 3/9/99 | Detailed | 51.5 | 0.21 | 0.8 | 11.2 | 10.9 | 11.9 |
| 30 | 5/5/99 | Crude | 27.0 | 0.20 | 0.6 | 11.0 | 10.6 | 11.6 |
| 30 | 5/5/99 | Intermediate | 72.2 | 0.10 | 0.6 | 5.4 | 5.2 | 5.7 |
| 30 | 5/5/99 | Detailed | 57.7 | 0.11 | 0.6 | 5.9 | 5.7 | 6.2 |
| 31 | 5/5/99 | Crude | 27.0 | 1.98 | 35.2 | 106.4 | 103.2 | 112.5 |
| 31 | 5/5/99 | Intermediate | 47.0 | 1.31 | 35.2 | 70.1 | 68.0 | 74.0 |
| 31 | 5/5/99 | Detailed | 35.8 | 1.65 | 35.2 | 88.6 | 85.9 | 93.6 |
| 32 | 5/18/99 | Crude | 27.0 | 0.49 | 3.0 | 26.4 | 25.6 | 27.9 |
| 32 | 5/18/99 | Intermediate | 63.6 | 0.21 | 3.0 | 11.5 | 11.2 | 12.2 |
| 32 | 5/18/99 | Detailed | 50.5 | 0.24 | 3.0 | 12.7 | 12.3 | 13.4 |

**Church estimated peak flows using the a priori parameters. For each observed event, a crude, intermediate and detailed Tc was estimated. Rainfall intensities for each Tc were derived from the measured rain data. These intensities were cross-multiplied to the low, moderate, and high-resolution C coefficients.**

| Storm Event | Date | $T_c$ Method | $T_c$ (min) | Rainfall Intensity (in/hr) | Observed Peak Flow (cfs) | C Coefficient Resolution: Low | Moderate | High |
|---|---|---|---|---|---|---|---|---|
| | | | | | | Estimated Peak Runoff (cfs) | | |
| 1 | 10/24/97 | Crude | 29.0 | 0.12 | 0.77 | 3.9 | 3.6 | 4.2 |
| 1 | 10/24/97 | Intermediate | 59.8 | 0.11 | 0.77 | 3.5 | 3.3 | 3.9 |
| 1 | 10/24/97 | Detailed | 56.6 | 0.11 | 0.77 | 3.5 | 3.3 | 3.9 |
| 2 | 10/25/97 | Crude | 29.0 | 0.28 | 2.50 | 9.1 | 8.5 | 10.0 |
| 2 | 10/25/97 | Intermediate | 52.7 | 0.26 | 2.50 | 8.2 | 7.6 | 8.9 |
| 2 | 10/25/97 | Detailed | 49.7 | 0.26 | 2.50 | 8.4 | 7.9 | 9.2 |
| 3 | 10/26/97 | Crude | 29.0 | 0.39 | 2.89 | 12.4 | 11.5 | 13.5 |
| 3 | 10/26/97 | Intermediate | 52.9 | 0.25 | 2.89 | 8.0 | 7.5 | 8.8 |
| 3 | 10/26/97 | Detailed | 49.7 | 0.26 | 2.89 | 8.4 | 7.9 | 9.2 |
| 4 | 11/1/97 | Crude | 29.0 | 0.27 | 1.99 | 8.7 | 8.1 | 9.5 |
| 4 | 11/1/97 | Intermediate | 52.9 | 0.25 | 1.99 | 8.0 | 7.5 | 8.8 |
| 4 | 11/1/97 | Detailed | 50.2 | 0.25 | 1.99 | 7.9 | 7.3 | 8.6 |
| 5 | 11/6/97 | Crude | 29.0 | 0.07 | 0.44 | 2.3 | 2.1 | 2.5 |
| 5 | 11/6/97 | Intermediate | 67.3 | 0.05 | 0.44 | 1.6 | 1.5 | 1.8 |
| 5 | 11/6/97 | Detailed | 63.6 | 0.05 | 0.44 | 1.6 | 1.5 | 1.8 |
| 6 | 11/13/97 | Crude | 29.0 | 0.16 | 1.15 | 5.1 | 4.8 | 5.6 |
| 6 | 11/13/97 | Intermediate | 58.3 | 0.13 | 1.15 | 4.2 | 3.9 | 4.6 |
| 6 | 11/13/97 | Detailed | 54.9 | 0.14 | 1.15 | 4.3 | 4.0 | 4.7 |
| 7 | 11/21/97 | Crude | 29.0 | 0.18 | 0.03 | 5.9 | 5.5 | 6.5 |
| 7 | 11/21/97 | Intermediate | 59.0 | 0.12 | 0.03 | 3.9 | 3.6 | 4.2 |
| 7 | 11/21/97 | Detailed | 55.5 | 0.13 | 0.03 | 4.0 | 3.7 | 4.4 |
| 8 | 1/7/98 | Crude | 29.0 | 0.17 | 1.74 | 5.5 | 5.1 | 6.0 |
| 8 | 1/7/98 | Intermediate | 56.5 | 0.16 | 1.74 | 5.1 | 4.8 | 5.6 |
| 8 | 1/7/98 | Detailed | 53.5 | 0.16 | 1.74 | 5.1 | 4.8 | 5.6 |
| 9 | 1/15/98 | Crude | 29.0 | 0.08 | 0.73 | 2.6 | 2.4 | 2.8 |
| 9 | 1/15/98 | Intermediate | 69.5 | 0.04 | 0.73 | 1.3 | 1.2 | 1.4 |
| 9 | 1/15/98 | Detailed | 64.6 | 0.05 | 0.73 | 1.4 | 1.3 | 1.6 |
| 10 | 1/15/98 | Crude | 29.0 | 0.09 | 1.00 | 3.0 | 2.7 | 3.2 |
| 10 | 1/15/98 | Intermediate | 65.5 | 0.06 | 1.00 | 1.9 | 1.8 | 2.1 |
| 10 | 1/15/98 | Detailed | 61.9 | 0.06 | 1.00 | 1.9 | 1.8 | 2.1 |
| 11 | 1/16/98 | Crude | 29.0 | 0.18 | 0.73 | 5.6 | 5.3 | 6.2 |
| 11 | 1/16/98 | Intermediate | 56.3 | 0.16 | 0.73 | 5.3 | 4.9 | 5.8 |
| 11 | 1/16/98 | Detailed | 53.2 | 0.17 | 0.73 | 5.3 | 5.0 | 5.8 |
| 12 | 1/18/98 | Crude | 29.0 | 0.11 | 1.00 | 3.6 | 3.3 | 3.9 |
| 12 | 1/18/98 | Intermediate | 63.3 | 0.07 | 1.00 | 2.4 | 2.2 | 2.6 |
| 12 | 1/18/98 | Detailed | 59.3 | 0.08 | 1.00 | 2.6 | 2.4 | 2.8 |
| 13 | 1/27/98 | Crude | 29.0 | 0.18 | 2.15 | 5.6 | 5.3 | 6.2 |
| 13 | 1/27/98 | Intermediate | 55.8 | 0.18 | 2.15 | 5.6 | 5.2 | 6.1 |
| 13 | 1/27/98 | Detailed | 52.7 | 0.18 | 2.15 | 5.7 | 5.3 | 6.2 |
| 14 | 2/2/98 | Crude | 29.0 | 0.23 | 0.93 | 7.4 | 6.9 | 8.1 |
| 14 | 2/2/98 | Intermediate | 55.0 | 0.19 | 0.93 | 6.2 | 5.8 | 6.8 |
| 14 | 2/2/98 | Detailed | 51.9 | 0.20 | 0.93 | 6.3 | 5.8 | 6.8 |
| 15 | 2/11/98 | Crude | 29.0 | 0.23 | 1.95 | 7.3 | 6.8 | 8.0 |
| 15 | 2/11/98 | Intermediate | 54.7 | 0.20 | 1.95 | 6.4 | 6.0 | 7.0 |
| 15 | 2/11/98 | Detailed | 51.7 | 0.20 | 1.95 | 6.4 | 6.0 | 7.0 |
| 16 | 2/11/98 | Crude | 29.0 | 0.06 | 2.58 | 1.9 | 1.8 | 2.1 |
| 16 | 2/11/98 | Intermediate | 72.6 | 0.03 | 2.58 | 1.0 | 0.9 | 1.1 |
| 16 | 2/11/98 | Detailed | 68.6 | 0.03 | 2.58 | 1.0 | 0.9 | 1.1 |
| 17 | 2/16/98 | Crude | 29.0 | 0.08 | 0.97 | 2.6 | 2.4 | 2.8 |
| 17 | 2/16/98 | Intermediate | 69.5 | 0.04 | 0.97 | 1.3 | 1.2 | 1.4 |
| 17 | 2/16/98 | Detailed | 64.6 | 0.05 | 0.97 | 1.4 | 1.3 | 1.6 |
| 18 | 2/17/98 | Crude | 29.0 | 0.11 | 1.25 | 3.5 | 3.2 | 3.8 |
| 18 | 2/17/98 | Intermediate | 61.6 | 0.09 | 1.25 | 2.9 | 2.7 | 3.2 |
| 18 | 2/17/98 | Detailed | 58.1 | 0.09 | 1.25 | 3.0 | 2.7 | 3.2 |
| 19 | 2/17/98 | Crude | 29.0 | 0.28 | 2.89 | 9.0 | 8.4 | 9.8 |
| 19 | 2/17/98 | Intermediate | 53.2 | 0.24 | 2.89 | 7.7 | 7.2 | 8.4 |
| 19 | 2/17/98 | Detailed | 49.9 | 0.25 | 2.89 | 8.2 | 7.6 | 8.9 |
| 20 | 5/21/98 | Crude | 29.0 | 2.87 | 19.84 | 92.2 | 85.7 | 100.8 |
| 20 | 5/21/98 | Intermediate | 37.2 | 2.65 | 19.84 | 85.1 | 79.1 | 93.0 |
| 20 | 5/21/98 | Detailed | 34.9 | 2.80 | 19.84 | 89.7 | 83.4 | 98.1 |

| Storm Event | Date | $T_c$ Method | $T_c$ (min) | Rainfall Intensity (in/hr) | Observed Peak Flow (cfs) | C Coefficient Resolution Low | Moderate | High |
|---|---|---|---|---|---|---|---|---|
| | | | | | | Estimated Peak Runoff (cfs) | | |
| 21 | 6/30/98 | Crude | 29.0 | 0.36 | 2.28 | 11.6 | 10.7 | 12.6 |
| 21 | 6/30/98 | Intermediate | 54.7 | 0.20 | 2.28 | 6.4 | 6.0 | 7.0 |
| 21 | 6/30/98 | Detailed | 51.0 | 0.22 | 2.28 | 7.1 | 6.6 | 7.7 |
| 22 | 7/9/98 | Crude | 29.0 | 0.28 | 0.22 | 9.0 | 8.4 | 9.8 |
| 22 | 7/9/98 | Intermediate | 56.8 | 0.16 | 0.22 | 5.0 | 4.6 | 5.4 |
| 22 | 7/9/98 | Detailed | 53.7 | 0.16 | 0.22 | 5.0 | 4.6 | 5.4 |
| 23 | 7/23/98 | Crude | 29.0 | 0.37 | 2.32 | 11.9 | 11.1 | 13.1 |
| 23 | 7/23/98 | Intermediate | 53.2 | 0.24 | 2.32 | 7.7 | 7.2 | 8.4 |
| 23 | 7/23/98 | Detailed | 49.9 | 0.25 | 2.32 | 8.2 | 7.6 | 8.9 |
| 24 | 7/31/98 | Crude | 29.0 | 0.24 | 1.26 | 7.8 | 7.3 | 8.6 |
| 24 | 7/31/98 | Intermediate | 54.5 | 0.20 | 1.26 | 6.5 | 6.1 | 7.2 |
| 24 | 7/31/98 | Detailed | 51.5 | 0.21 | 1.26 | 6.6 | 6.2 | 7.3 |
| 25 | 8/10/98 | Crude | 29.0 | 1.42 | 8.16 | 45.5 | 42.3 | 49.7 |
| 25 | 8/10/98 | Intermediate | 39.7 | 1.70 | 8.16 | 54.6 | 50.8 | 59.7 |
| 25 | 8/10/98 | Detailed | 37.6 | 1.70 | 8.16 | 54.6 | 50.8 | 59.7 |
| 26 | 8/11/98 | Crude | 29.0 | 0.38 | 0.02 | 12.2 | 11.3 | 13.3 |
| 26 | 8/11/98 | Intermediate | 54.3 | 0.21 | 0.02 | 6.8 | 6.3 | 7.4 |
| 26 | 8/11/98 | Detailed | 50.9 | 0.22 | 0.02 | 7.1 | 6.6 | 7.8 |
| 27 | 8/13/98 | Crude | 29.0 | 1.14 | 1.96 | 36.6 | 34.0 | 40.0 |
| 27 | 8/13/98 | Intermediate | 44.8 | 0.76 | 1.96 | 24.4 | 22.7 | 26.7 |
| 27 | 8/13/98 | Detailed | 42.1 | 0.80 | 1.96 | 25.6 | 23.8 | 28.0 |
| 28 | 8/16/98 | Crude | 29.0 | 0.28 | 0.09 | 9.0 | 8.4 | 9.8 |
| 28 | 8/16/98 | Intermediate | 57.6 | 0.14 | 0.09 | 4.6 | 4.2 | 5.0 |
| 28 | 8/16/98 | Detailed | 53.7 | 0.16 | 0.09 | 5.0 | 4.6 | 5.4 |
| 29 | 8/16/98 | Crude | 29.0 | 0.32 | 0.16 | 10.3 | 9.6 | 11.2 |
| 29 | 8/16/98 | Intermediate | 54.3 | 0.21 | 0.16 | 6.8 | 6.3 | 7.4 |
| 29 | 8/16/98 | Detailed | 51.3 | 0.21 | 0.16 | 6.8 | 6.3 | 7.4 |
| 30 | 10/3/98 | Crude | 29.0 | 0.49 | 0.77 | 15.7 | 14.6 | 17.1 |
| 30 | 10/3/98 | Intermediate | 48.5 | 0.45 | 0.77 | 14.4 | 13.4 | 15.8 |
| 30 | 10/3/98 | Detailed | 45.6 | 0.47 | 0.77 | 15.1 | 14.0 | 16.5 |
| 31 | 1/31/99 | Crude | 29.0 | 0.19 | 0.21 | 6.0 | 5.6 | 6.6 |
| 31 | 1/31/99 | Intermediate | 56.5 | 0.16 | 0.21 | 5.1 | 4.8 | 5.6 |
| 31 | 1/31/99 | Detailed | 53.2 | 0.17 | 0.21 | 5.3 | 4.9 | 5.8 |
| 32 | 2/17/99 | Crude | 29.0 | 0.20 | 0.50 | 6.5 | 6.1 | 7.2 |
| 32 | 2/17/99 | Intermediate | 55.6 | 0.18 | 0.50 | 5.8 | 5.4 | 6.3 |
| 32 | 2/17/99 | Detailed | 52.1 | 0.19 | 0.50 | 6.1 | 5.7 | 6.7 |
| 33 | 2/17/99 | Crude | 29.0 | 0.14 | 0.19 | 4.5 | 4.2 | 4.9 |
| 33 | 2/17/99 | Intermediate | 62.0 | 0.09 | 0.19 | 2.8 | 2.6 | 3.0 |
| 33 | 2/17/99 | Detailed | 58.0 | 0.09 | 0.19 | 3.0 | 2.8 | 3.2 |
| 34 | 2/19/99 | Crude | 29.0 | 0.12 | 0.20 | 3.9 | 3.6 | 4.2 |
| 34 | 2/19/99 | Intermediate | 61.1 | 0.10 | 0.20 | 3.0 | 2.8 | 3.3 |
| 34 | 2/19/99 | Detailed | 57.0 | 0.10 | 0.20 | 3.3 | 3.1 | 3.6 |
| 35 | 2/27/99 | Crude | 29.0 | 0.25 | 1.19 | 8.1 | 7.5 | 8.8 |
| 35 | 2/27/99 | Intermediate | 54.5 | 0.21 | 1.19 | 6.6 | 6.1 | 7.2 |
| 35 | 2/27/99 | Detailed | 51.4 | 0.21 | 1.19 | 6.7 | 6.2 | 7.3 |
| 36 | 3/2/99 | Crude | 29.0 | 0.36 | 2.10 | 11.4 | 10.6 | 12.5 |
| 36 | 3/2/99 | Intermediate | 52.3 | 0.27 | 2.10 | 8.7 | 8.1 | 9.5 |
| 36 | 3/2/99 | Detailed | 49.4 | 0.27 | 2.10 | 8.8 | 8.1 | 9.6 |
| 37 | 3/9/99 | Crude | 29.0 | 0.25 | 0.80 | 8.1 | 7.5 | 8.8 |
| 37 | 3/9/99 | Intermediate | 54.3 | 0.21 | 0.80 | 6.8 | 6.3 | 7.4 |
| 37 | 3/9/99 | Detailed | 51.3 | 0.21 | 0.80 | 6.8 | 6.3 | 7.4 |
| 38 | 5/5/99 | Crude | 29.0 | 0.30 | 0.17 | 9.6 | 9.0 | 10.5 |
| 38 | 5/5/99 | Intermediate | 54.7 | 0.20 | 0.17 | 6.4 | 6.0 | 7.0 |
| 38 | 5/5/99 | Detailed | 51.4 | 0.21 | 0.17 | 6.7 | 6.3 | 7.4 |
| 39 | 5/5/99 | Crude | 29.0 | 1.50 | 5.73 | 48.2 | 44.8 | 52.6 |
| 39 | 5/5/99 | Intermediate | 42.1 | 1.15 | 5.73 | 37.0 | 34.4 | 40.4 |
| 39 | 5/5/99 | Detailed | 39.5 | 1.22 | 5.73 | 39.0 | 36.3 | 42.6 |
| 40 | 5/18/99 | Crude | 29.0 | 0.49 | 1.09 | 15.8 | 14.7 | 17.3 |
| 40 | 5/18/99 | Intermediate | 42.1 | 0.39 | 1.09 | 12.5 | 11.6 | 13.7 |
| 40 | 5/18/99 | Detailed | 39.5 | 0.41 | 1.09 | 13.2 | 12.2 | 14.4 |

# APPENDIX E.

# VOLUME ESTIMATES FROM THE A PRIORI PARAMETERS

**The a priori volume estimates produced by the CN paramters unadjusted for AMC.**

| | | | | Low-resolution CN (72) | | | Moderate-resolution CN (66) | | | High-resolution CN (72) | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Event | Storm Date | Precip. Depth (in) | Runoff Observed (in) | Estimated Runoff (in) | AAE (in) | Difference % | Estimated Runoff (in) | AAE (in) | Difference % | Estimated Runoff (in) | AAE (in) | Difference % |
| 1 | 10/25/97 | 0.98 | 0.45 | 0.01 | 0.44 | 98 | 0.00 | 0.45 | 100 | 0.01 | 0.44 | 98 |
| 2 | 10/26/97 | 0.35 | 0.30 | 0.05 | 0.25 | 82 | 0.10 | 0.20 | 65 | 0.05 | 0.25 | 82 |
| 3 | 11/1/97 | 0.58 | 0.02 | 0.01 | 0.01 | 56 | 0.04 | 0.02 | 79 | 0.01 | 0.01 | 56 |
| 4 | 11/13/97 | 0.39 | 0.06 | 0.04 | 0.02 | 32 | 0.09 | 0.03 | 44 | 0.04 | 0.02 | 32 |
| 5 | 11/21/97 | 0.60 | 0.06 | 0.01 | 0.06 | 86 | 0.04 | 0.02 | 38 | 0.01 | 0.06 | 86 |
| 6 | 1/16/98 | 0.43 | 0.02 | 0.03 | 0.01 | 58 | 0.08 | 0.06 | 266 | 0.03 | 0.01 | 58 |
| 7 | 1/18/98 | 0.16 | 0.00 | 0.12 | 0.12 | 8724 | 0.18 | 0.18 | 13281 | 0.12 | 0.12 | 8724 |
| 8 | 1/27/98 | 1.27 | 0.11 | 0.06 | 0.05 | 49 | 0.01 | 0.10 | 90 | 0.06 | 0.05 | 49 |
| 9 | 2/2/98 | 1.66 | 0.18 | 0.16 | 0.02 | 11 | 0.07 | 0.11 | 63 | 0.16 | 0.02 | 11 |
| 10 | 2/11/98 | 0.21 | 0.00 | 0.10 | 0.09 | 2232 | 0.16 | 0.15 | 3633 | 0.10 | 0.09 | 2232 |
| 11 | 2/17/98 | 0.13 | 0.00 | 0.13 | 0.13 | 23110 | 0.19 | 0.19 | 34081 | 0.13 | 0.13 | 23110 |
| 12 | 2/17/98 | 0.31 | 0.02 | 0.06 | 0.05 | 288 | 0.12 | 0.10 | 611 | 0.06 | 0.05 | 288 |
| 13 | 4/3/98 | 0.79 | 0.11 | 0.00 | 0.11 | 100 | 0.01 | 0.10 | 89 | 0.00 | 0.11 | 100 |
| 14 | 4/8/98 | 1.38 | 0.27 | 0.08 | 0.19 | 70 | 0.02 | 0.25 | 92 | 0.08 | 0.19 | 70 |
| 15 | 4/16/98 | 3.34 | 0.88 | 1.02 | 0.14 | 16 | 0.71 | 0.16 | 19 | 1.02 | 0.14 | 16 |
| 16 | 4/18/98 | 3.22 | 1.91 | 0.94 | 0.97 | 51 | 0.65 | 1.26 | 66 | 0.94 | 0.97 | 51 |
| 17 | 5/1/98 | 0.36 | 0.02 | 0.05 | 0.03 | 145 | 0.10 | 0.08 | 388 | 0.05 | 0.03 | 145 |
| 18 | 5/21/98 | 1.76 | 0.23 | 0.20 | 0.03 | 12 | 0.09 | 0.14 | 60 | 0.20 | 0.03 | 12 |
| 19 | 7/9/98 | 0.39 | 0.02 | 0.04 | 0.03 | 173 | 0.09 | 0.08 | 477 | 0.04 | 0.03 | 173 |
| 20 | 7/31/98 | 0.25 | 0.00 | 0.08 | 0.08 | 2931 | 0.14 | 0.14 | 4994 | 0.08 | 0.08 | 2931 |
| 21 | 7/31/98 | 0.41 | 0.02 | 0.04 | 0.02 | 84 | 0.08 | 0.06 | 306 | 0.04 | 0.02 | 84 |
| 22 | 8/10/98 | 0.58 | 0.01 | 0.01 | 0.00 | 13 | 0.04 | 0.03 | 254 | 0.01 | 0.00 | 13 |
| 23 | 1/31/99 | 0.54 | 0.01 | 0.02 | 0.00 | 5 | 0.05 | 0.04 | 250 | 0.02 | 0.00 | 5 |
| 24 | 2/17/99 | 0.36 | 0.00 | 0.05 | 0.05 | 76311 | 0.10 | 0.10 | 152263 | 0.05 | 0.05 | 76311 |
| 25 | 2/17/99 | 0.26 | 0.00 | 0.08 | 0.08 | 162966 | 0.14 | 0.14 | 277601 | 0.08 | 0.08 | 162966 |
| 26 | 2/19/99 | 0.22 | 0.00 | 0.09 | 0.09 | 3530 | 0.15 | 0.15 | 5779 | 0.09 | 0.09 | 3530 |
| 27 | 2/27/99 | 0.87 | 0.03 | 0.00 | 0.03 | 93 | 0.01 | 0.03 | 83 | 0.00 | 0.03 | 93 |
| 28 | 3/2/99 | 1.44 | 0.04 | 0.10 | 0.05 | 119 | 0.03 | 0.01 | 31 | 0.10 | 0.05 | 119 |
| 29 | 3/9/99 | 0.43 | 0.00 | 0.03 | 0.03 | 855 | 0.08 | 0.08 | 2115 | 0.03 | 0.03 | 855 |
| 30 | 5/5/99 | 0.34 | 0.00 | 0.06 | 0.05 | 2809 | 0.11 | 0.10 | 5496 | 0.06 | 0.05 | 2809 |
| 31 | 5/5/99 | 2.32 | 0.32 | 0.44 | 0.11 | 35 | 0.26 | 0.07 | 20 | 0.44 | 0.11 | 35 |
| 32 | 5/18/99 | 0.36 | 0.01 | 0.05 | 0.04 | 709 | 0.10 | 0.09 | 1514 | 0.05 | 0.04 | 709 |

The Church watershed a priori volume estimates produced by the unadjusted CN paramters.

| Event | Storm Date | Rainfall Depth (in) | Observed Runoff (in) | Low-resolution CN (72) Estimated Runoff (in) | Moderate-resolution CN (69) Estimated Runoff (in) | High-resolution CN (76) Estimated Runoff (in) |
|---|---|---|---|---|---|---|
| 1 | 10/24/97 | 0.24 | 0.0167 | 0.0863 | 0.1131 | 0.0554 |
| 2 | 10/25/97 | 1.06 | 0.0813 | 0.0191 | 0.0056 | 0.0512 |
| 3 | 10/26/97 | 0.36 | 0.0314 | 0.0503 | 0.0733 | 0.0256 |
| 4 | 11/1/97 | 0.62 | 0.0511 | 0.0067 | 0.0184 | 0.0000 |
| 5 | 11/6/97 | 0.11 | 0.0054 | 0.1384 | 0.1679 | 0.1032 |
| 6 | 11/13/97 | 0.49 | 0.0531 | 0.0230 | 0.0409 | 0.0066 |
| 7 | 11/21/97 | 0.55 | 0.0004 | 0.0142 | 0.0293 | 0.0022 |
| 8 | 1/7/98 | 0.91 | 0.1233 | 0.0043 | 0.0000 | 0.0226 |
| 9 | 1/15/98 | 0.17 | 0.0219 | 0.1126 | 0.1410 | 0.0790 |
| 10 | 1/15/98 | 0.18 | 0.0281 | 0.1086 | 0.1368 | 0.0754 |
| 11 | 1/16/98 | 0.54 | 0.0734 | 0.0155 | 0.0311 | 0.0027 |
| 12 | 1/18/98 | 0.2 | 0.0261 | 0.1008 | 0.1286 | 0.0683 |
| 13 | 1/27/98 | 1.21 | 0.0718 | 0.0432 | 0.0202 | 0.0895 |
| 14 | 2/2/98 | 1.83 | 0.3324 | 0.2241 | 0.1599 | 0.3297 |
| 15 | 2/11/98 | 0.23 | 0.0273 | 0.0898 | 0.1169 | 0.0585 |
| 16 | 2/11/98 | 0.06 | 0.0085 | 0.1625 | 0.1924 | 0.1263 |
| 17 | 2/16/98 | 0.17 | 0.0190 | 0.1126 | 0.1410 | 0.0790 |
| 18 | 2/17/98 | 0.16 | 0.0208 | 0.1167 | 0.1453 | 0.0828 |
| 19 | 2/17/98 | 0.28 | 0.0396 | 0.0731 | 0.0988 | 0.0440 |
| 20 | 5/21/98 | 1.64 | 0.1515 | 0.1565 | 0.1050 | 0.2441 |
| 21 | 6/30/98 | 0.31 | 0.0088 | 0.0640 | 0.0887 | 0.0365 |
| 22 | 7/9/98 | 0.18 | 0.0009 | 0.1086 | 0.1368 | 0.0754 |
| 23 | 7/23/98 | 0.29 | 0.0133 | 0.0700 | 0.0953 | 0.0414 |
| 24 | 7/31/98 | 0.26 | 0.0088 | 0.0795 | 0.1058 | 0.0496 |
| 25 | 8/10/98 | 0.6 | 0.0245 | 0.0085 | 0.0213 | 0.0003 |
| 26 | 8/11/98 | 0.21 | 0.0001 | 0.0971 | 0.1246 | 0.0650 |
| 27 | 8/13/98 | 0.59 | 0.0060 | 0.0095 | 0.0228 | 0.0006 |
| 28 | 8/16/98 | 0.44 | 0.0003 | 0.0321 | 0.0521 | 0.0124 |
| 29 | 8/16/98 | 0.3 | 0.0011 | 0.0669 | 0.0920 | 0.0389 |
| 30 | 10/3/98 | 0.46 | 0.0035 | 0.0283 | 0.0474 | 0.0099 |
| 31 | 1/31/99 | 0.6 | 0.0065 | 0.0085 | 0.0213 | 0.0003 |
| 32 | 2/17/99 | 0.39 | 0.0040 | 0.0429 | 0.0649 | 0.0200 |
| 33 | 2/17/99 | 0.27 | 0.0024 | 0.0763 | 0.1022 | 0.0468 |
| 34 | 2/19/99 | 0.25 | 0.0022 | 0.0829 | 0.1094 | 0.0524 |
| 35 | 2/27/99 | 0.88 | 0.0172 | 0.0026 | 0.0001 | 0.0181 |
| 36 | 3/2/99 | 1.45 | 0.0317 | 0.0991 | 0.0603 | 0.1685 |
| 37 | 3/9/99 | 0.56 | 0.0082 | 0.0129 | 0.0276 | 0.0017 |
| 38 | 5/5/99 | 0.41 | 0.0005 | 0.0384 | 0.0596 | 0.0167 |
| 39 | 5/5/99 | 2.01 | 0.0508 | 0.2965 | 0.2204 | 0.4189 |
| 40 | 5/18/99 | 0.39 | 0.0031 | 0.0429 | 0.0649 | 0.0200 |

The Sanford watershed a priori volume estimates produced by the AMC adjusted CN paramters.

| Storm Event | Date | Observed AMC | Observed Runoff (in) | Low-resolution CN Estimated Runoff (in) | | | Moderate-resolution CN Estimated Runoff (in) | | | High-resolution CN Estimated Runoff (in) | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | AMC I (53) | AMC II (72) | AMC III (86) | AMC I (46) | AMC II (66) | AMC III (82) | AMC I (55) | AMC II (72) | AMC III (85) |
| 1 | 10/25/97 | 1 | 0.454 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 2 | 10/26/97 | 3 | 0.299 | - | - | 0.000 | - | - | 0.000 | - | - | 0.000 |
| 3 | 11/1/97 | 1 | 0.024 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 4 | 11/13/97 | 1 | 0.063 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 5 | 11/21/97 | 1 | 0.064 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 6 | 1/16/98 | 1 | 0.022 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 7 | 1/18/98 | 2 | 0.001 | - | 0.117 | - | - | 0.177 | - | - | 0.117 | - |
| 8 | 1/27/98 | 2 | 0.108 | - | 0.055 | - | - | 0.011 | - | - | 0.055 | - |
| 9 | 2/2/98 | 1 | 0.183 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 10 | 2/11/98 | 1 | 0.004 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 11 | 2/17/98 | 1 | 0.001 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 12 | 2/17/98 | 1 | 0.016 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 13 | 4/3/98 | 1 | 0.110 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 14 | 4/8/98 | 1 | 0.270 | 0.000 | - | - | 0.000 | - | - | 0.008 | - | - |
| 15 | 4/16/98 | 1 | 0.878 | 0.235 | - | - | 0.077 | - | - | 0.294 | - | - |
| 16 | 4/18/98 | 3 | 1.914 | - | - | 1.853 | - | - | 1.554 | - | - | 1.775 |
| 17 | 5/1/98 | 1 | 0.021 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 18 | 5/21/98 | 1 | 0.226 | 0.000 | - | - | 0.000 | - | - | 0.002 | - | - |
| 19 | 7/9/98 | 1 | 0.016 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 20 | 7/31/98 | 1 | 0.003 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 21 | 7/31/98 | 1 | 0.021 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 22 | 8/10/98 | 1 | 0.012 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 23 | 1/31/99 | 1 | 0.015 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 24 | 2/17/99 | 1 | 0.000 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 25 | 2/17/99 | 1 | 0.000 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 26 | 2/19/99 | 2 | 0.003 | - | 0.093 | - | - | 0.151 | - | - | 0.093 | - |
| 27 | 2/27/99 | 1 | 0.031 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 28 | 3/2/99 | 2 | 0.044 | - | 0.096 | - | - | 0.030 | - | 0.005 | 0.096 | - |
| 29 | 3/9/99 | 1 | 0.004 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 30 | 5/5/99 | 1 | 0.002 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 31 | 5/5/99 | 1 | 0.324 | 0.032 | - | - | 0.000 | - | - | 0.053 | - | - |
| 32 | 5/18/99 | 1 | 0.006 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |

The Church watershed apriori volume estimates produced by the AMC adjusted CN parameters .

| Storm Event | Date | Observed AMC | Observed Runoff (in) | Low-resolution CN Estimated Runoff (in) AMC I (53) | AMC II (72) | AMC III (86) | Moderate-resolution CN Estimated Runoff (in) AMC I (50) | AMC II (69) | AMC III (84) | High-resolution CN Estimated Runoff (in) AMC I (60) | AMC II (76) | AMC III (88) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 10/24/97 | 1 | 0.017 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 2 | 10/25/97 | 1 | 0.081 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 3 | 10/26/97 | 3 | 0.031 | - | - | 0.001 | - | - | 0.000 | 0.000 | - | 0.005 |
| 4 | 11/1/97 | 1 | 0.051 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 5 | 11/6/97 | 3 | 0.005 | - | - | 0.000 | - | - | 0.000 | 0.000 | - | 0.000 |
| 6 | 11/13/97 | 1 | 0.053 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 7 | 11/21/97 | 1 | 0.000 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 8 | 1/7/98 | 1 | 0.123 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 9 | 1/15/98 | 1 | 0.022 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 10 | 1/15/98 | 1 | 0.028 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 11 | 1/16/98 | 2 | 0.073 | - | 0.015 | - | - | 0.031 | - | 0.000 | 0.003 | - |
| 12 | 1/18/98 | 2 | 0.026 | - | 0.101 | - | - | 0.129 | - | 0.000 | 0.068 | - |
| 13 | 1/27/98 | 2 | 0.072 | - | 0.043 | - | - | 0.020 | - | 0.000 | 0.090 | - |
| 14 | 2/2/98 | 1 | 0.332 | 0.000 | - | - | 0.000 | - | - | 0.034 | - | - |
| 15 | 2/11/98 | 1 | 0.027 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 16 | 2/11/98 | 1 | 0.008 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 17 | 2/16/98 | 1 | 0.019 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 18 | 2/17/98 | 1 | 0.021 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 19 | 2/17/98 | 1 | 0.040 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 20 | 5/21/98 | 1 | 0.151 | 0.000 | - | - | 0.000 | - | - | 0.013 | - | - |
| 21 | 6/30/98 | 1 | 0.009 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 22 | 7/9/98 | 1 | 0.001 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 23 | 7/23/98 | 1 | 0.013 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 24 | 7/31/98 | 1 | 0.009 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 25 | 8/10/98 | 1 | 0.025 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 26 | 8/11/98 | 1 | 0.000 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 27 | 8/13/98 | 1 | 0.006 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 28 | 8/16/98 | 1 | 0.000 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 29 | 8/16/98 | 1 | 0.001 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 30 | 10/3/98 | 1 | 0.003 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 31 | 1/31/99 | 1 | 0.006 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 32 | 2/17/99 | 1 | 0.004 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 33 | 2/17/99 | 1 | 0.002 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 34 | 2/19/99 | 2 | 0.002 | - | 0.083 | - | - | 0.109 | - | 0.000 | 0.052 | - |
| 35 | 2/27/99 | 1 | 0.017 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 36 | 3/2/99 | 2 | 0.032 | - | 0.099 | - | - | 0.060 | - | 0.002 | 0.168 | - |
| 37 | 3/9/99 | 1 | 0.008 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 38 | 5/5/99 | 1 | 0.000 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |
| 39 | 5/5/99 | 1 | 0.051 | 0.006 | - | - | 0.000 | - | - | 0.062 | - | - |
| 40 | 5/18/99 | 1 | 0.003 | 0.000 | - | - | 0.000 | - | - | 0.000 | - | - |

# VITA

Steven R. Potter was born in Washington, D. C. and attended schools in Paris, France and Austin, Texas. Following high school, Steve enlisted in the U.S. Navy as an Aviation Electronics Technician and served in several stateside locations and on the flight deck aboard the U.S.S. Eisenhower. During his military service, he earned several awards, including the Navy Expeditionary Medal for duty in the Lebanon theater of operations.

Following his military service, Steve was employed by Dow Jones & Co., Inc. and performed many computer-related jobs in the company's Atlanta offices. In September of 1991, he was promoted and moved to Tennessee to manage the Knoxville office.

After several years, Steve decided to further his education and entered Pellisippi State Community College, where he was on the Dean's list each semester. In August 1995, he transferred to the University of Tennessee, and received a Bachelors of Science with a major in Plant and Soil Science in May 1997 graduating Magna Cum Laude. In June of 1997, he received a graduate assistantship from the Agricultural and Biosystems Engineering Department and thus, entered the Graduate School at The University of Tennessee. In December 1999, he completed the requirements for a Masters of Science in Agricultural and Biosystems Engineering Technology.