EFFECT OF WATERSHED SUBDIVISION AND
PARAMETER SELECTION ON MODELING RESULTS

by

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ABSTRACT

During the hydrologic design process, after the watershed has been delineated, the question arises whether or not to subdivide the watershed into smaller sub-watersheds. It is assumed that by subdividing the watershed, the designer would expect improved accuracy in the hydrologic modeling results than when treating the watershed as a single, or lumped model. The objectives of this thesis are to test the theory that increased subdivision of a watershed improves estimates of runoff hydrographs at the outlet of a watershed and to determine situations where watershed subdivision would be justified.

The hydrologic parameters of five Texas watersheds, ranging between 7.1 to 46.1 square miles, were extracted using the computer software ArcGIS. The five watersheds were then subdivided into 3, 5, 7, 10, 15, and 30 sub-watersheds. The modeling software HEC-HMS generated runoff hydrographs for the lumped models and the subdivision schemes using historic USGS gage data. The Runoff hydrographs were then compared to observed runoff data in terms of runoff volume, peak flow, and time to peak.

The results show that there was no single subdivision scheme that consistently outperformed any of the others. The lumped model performed equal or better in terms of runoff volume and time to peak. Some accuracy was gained from subdividing in terms of peak flow, but the rate of increased accuracy leveled out between 5 to 10 subdivisions. The results also show that the variations in curve numbers throughout a watershed provide a better indication of subdivision locations than variations in slope.

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CHAPTER I

INTRODUCTION

Background

One of the most fundamental and important tasks of a hydrologist is to calculate storm runoff from a watershed. There are a variety of reasons this task is important, ranging from culvert and bridge design to flood analysis. If storm runoff is over-estimated, money and time are wasted on installing oversized structures. Conversely, if storm runoff is under-estimated, undersized structures are installed that could lead to flooding and property damage. The Texas Department of Transportation (TxDOT) uses a large portion of its design and construction budget each year on highway drainage facilities. Beginning in the early 2000s, TxDOT initiated several hydrologic related research projects with the goal of making the task of estimating runoff from Texas watersheds more accurate and efficient. This thesis is presented as a part of TxDOT Project 0-5822, “Subdivision of Watersheds for Modeling.”

Problem Statement

During the hydrologic design process, after the watershed is delineated, the question arises whether or not to subdivide the watershed into smaller sub-watersheds. Leaving the watershed as a lumped model, where the parameters are considered homogeneous over the entire watershed, may be easier to model in terms of input parameters, but may not accurately depict physical watershed conditions (this can be especially true for larger watersheds where changes in slope or land use could affect runoff conditions). Subdividing the watershed into smaller sub-watersheds may take into account physical changes within the watershed, but the increased time and effort used for determining model parameters may or may not produce results that are more accurate than the lumped model. The current TxDOT design manuals do not contain much guidance for watershed subdivision.

There may be sound reasons a designer would want to subdivide a watershed, such as changes in land use (developed vs. undeveloped), changes in watershed slope, or streams branching off from the main channel. Whatever the case may be, the designer would expect improved “accuracy” in runoff calculations by subdividing the watershed at these locations. While this procedure seems logical, there is limited documentation demonstrating that subdividing the watershed in this way produces more accurate runoff estimates.

Objectives

This thesis is in response to TxDOT Project 0-5822, “Subdivision of Watersheds for Modeling.” One of the main goals of this research project is to develop a set of defensible guidelines for watershed subdivision in order to improve the TxDOT design process. Within this thesis, there are three main objectives:

1. Use the extension tools Arc Hydro and HEC-GeoHMS within ArcGIS, to delineate the study watersheds and to extract model parameters for each watershed,

2. Evaluate the enhanced or diminished prediction value on watershed modeling as a function of subdivision, and

3. To determine if there is a certain percentage of a watershed that needs to be significantly different from the rest of the watershed in order to justify subdividing.

The first objective is to use the extension tools Arc Hydro and HEC-GeoHMS within ArcGIS, to delineate the study watersheds and to extract model parameters for each watershed (drainage areas, curve numbers, channel slopes, etc.). Initially, the watershed will be treated as a lumped model. ArcGIS will then be used to subdivide the watersheds into 3, 5, 7, 10, 15, and 30 sub-watersheds using a heuristic approach. The criteria used for the location of the subdivisions are as follows:

1. Subdivide where there is a distinct change in land use or land cover,

2. Where there is a noticeable change in channel or watershed slope, and

3. In areas where there are stream branches within the drainage network.

The second objective of this research is to evaluate the enhanced or diminished prediction value on watershed modeling as a function of subdivision. The hydrologic modeling software HEC-HMS (USACE 2006) is used to compute runoff hydrographs for the various subdivision schemes. The runoff hydrographs are computed for the lumped models first and from the suite of 3, 5, 7, 10, 15, and 30 sub-watersheds. The computed runoff hydrographs will be compared to observed runoff hydrographs in terms of runoff volume, peak flow, and time to peak. It is important to note that the models were intentionally left uncalibrated with respect to observed runoff behaviors. In this sense the watershed models represent the judgment of the hydrologic analyst, as would be applied at ungaged sites.

The third objective of this research is to determine if there is a certain percentage of a watershed that needs to be significantly different from the rest of the watershed in order to justify subdividing. To accomplish this, the same lumped and subdivided watershed models will be computed again, but this time one of three watershed parameters (curve number, basin transformation time, or routing time) will be substantially changed for approximately 1/5, 1/3 and 1/2 of the total watershed area. The runoff hydrographs will then be compared to the unchanged, or base, models.

The steps taken to address these research objectives are described within the remainder of this document. In Chapter 2, a review of the literature, the use of GIS in hydrology, and fundamental hydrologic definitions and concepts are provided. In Chapter 3, the GIS datasets, GIS processing techniques and the hydrologic modeling techniques are described. The results of the modeling study are described in Chapter 4. In Chapter 5, the model results are interpreted and recommendations are made for future areas of research.

CHAPTER II

LITERATURE REVIEW

Watershed Subdivision

Within this section, a selection of literature published on the topic of watershed subdivision will be examined to see what other researchers attempted and to study the results of their work.

Hromodka (1986), in the San Bernardino hydrologic design manual, states that, "Arbitrary subdivision of the watershed into subareas should generally be avoided." He goes on to say that "…an increase in the watershed subdivision does not necessarily increase the modeling 'accuracy' but rather transfers the model's reliability from the calibrated unit hydrograph and lag relationships to the unknown reliability of the several flow routing submodels used to link together the several subareas."

Wood and others (1988) conducted an experiment between watershed scale and watershed runoff on the Coweeta River Basin (6.56 mi^2) located in North Carolina. The watershed was subdivided into 3, 19, 39, and 87 smaller sub-watersheds to determine the average water fluxes at each sub-watershed. The results of Wood's research showed that below a drainage area scale of about 0.4 mi^2, sub-watershed response was highly variable. At scales greater than 0.4 mi^2, further aggregation of sub-watersheds had little impact on the simulated results.

Sasowsky and Gardner (1991) used the hydrologic model SPUR to evaluate the accuracy of model simulation on the Walnut Gulch watershed (56.4 mi^2) located in Arizona. This watershed was subdivided into 3, 37, and 66 sub-watersheds based on a stream network of 13th, 4th, and 2nd Shreve order networks, respectively. GIS programs were used to determine SPUR model parameters. The results of Sasowsky and Gardner show that the subdivision scheme of 37 sub-watersheds produce results in the SPUR simulation that are similar to the 66 sub-watershed scheme. Sasowsky and Gardner also show that these conclusions can be affected by the curve numbers used. Greater curve numbers yielded better results for the 66 sub-watersheds, while smaller curve numbers yielded better results for the 37 sub-watersheds. The results also show that both of these subdivision schemes performed better than the 3 sub-watershed scheme.

Norris and Haan (1993) used a portion of the Little Washita watershed (58.8 mi^2) in Oklahoma to demonstrate the effects of watershed subdivision. The watershed was first modeled as a single watershed and then modeled as 2, 5, 10, and 15 sub-watersheds. Norris and Haan showed that as the number of subdivisions increased the estimated peak flows also increase, but the rate of increase in the peak flow decreased rapidly after 5 subdivisions. The objective of their research was not to compare their results to observed runoff, but an intercomparison of subdivided model results to determine differences, if any, between subdivision modeling schemes.

Mamillapalli and others (1996) used the Soil Water Assessment Tool (SWAT) software to examine what effect increasing watershed subdivisions and soil/landuse combinations had on a Texas watershed (1659.1 mi^2). The watershed was subdivided into

4, 8, 14, 20, 24, 29, 35, 40, and 54 subbasins using GIS software. Mamillapalli and others show that, in general, the level of accuracy increased as the number of subdivisions and soil and landuse combinations increased. They also show that beyond 14 sub-watersheds the accuracy was not greatly improved and suggest that a more detailed simulation may not always lead to better results.

Bingner and others (1996), FitzHugh and Mackay (2000), Jha (2002), and Tripathi and others (2006) also applied the SWAT program to conduct studies to examine the relation between watershed subdivision and water-quality model results. The focus of all of these articles was on watershed sediment yield. They concluded that sediment yield is highly variable as the amount of subdivisions increase, but the annual stream flow calculated using SWAT was not seriously affected as the number of sub-watersheds increased.

GIS Applications

GIS emerged as a significant resource for hydrologic modeling during the 1990s. Use of GIS provides a consistent method for watershed and stream network delineation using digital elevation models (DEMs) of land-surface terrain (Maidment 2002). The GIS software used to extract watershed parameters for this project was ArcGIS 9.2 (ESRI 2006). According to the ESRI website, ArcGIS systems are used by more that 300,000 organizations around the world, including most U.S. federal agencies. Although the ArcGIS software is not public domain, a large number of the datasets produced by federal agencies are in the public domain.

O'Callaghan and Mark (1984) were one of the first to introduce a method for extracting drainage networks from a gridded elevation dataset. The method included algorithms for elevation smoothing, drainage direction assignment, and drainage accumulation using digital elevation models. The ideas presented in their article formed the foundation for many of today's GIS water resource applications.

Arc Hydro 1.2 is a public domain utility which works within ArcGIS. Arc Hydro was developed jointly by the Center for Research in Water Resources (CRWR) of the University of Texas and ESRI (Maidment 2002). For the research project reported herein, Arc Hydro is used for terrain processing, watershed delineation, and creation of watershed parameters. Maidment (2002) discusses the data structure framework of Arc Hydro, along with an illustrative example for a Texas watershed that demonstrates the different functions of Arc Hydro.

Hellweger (1997) gives background information about reconditioning the digital elevation models. In this terrain processing step, the AGREE method adjusts the surface elevation of the DEM to be consistent with the vector stream definition. The delineation of watersheds and stream channel networks from raster elevation models are based on the 8 point raster processing algorithms of O'Callaghan and Mark (1984).

HEC-GeoHMS 4.1 (USACE 2000), a public domain utility which works within ArcGIS, is used to process watershed data after the initial compilation and preparation of terrain data is complete. HEC-GeoHMS was developed by the U.S. Army Corps of Engineers Hydrologic Engineering Center (HEC) and ESRI. After the terrain and spatial information assembly is completed, the data are processed by HEC-GeoHMS to generate hydrologic inputs, which are used in HEC-HMS (USACE 2003). For this research, HEC-GeoHMS is used to create the curve number grids used with the National Resource Conservation Service (NRCS) curve number method. Merwade (2008a) illustrates the different functions of HEC-GeoHMS through a series of tutorials.

Hydrologic Modeling

The hydrologic modeling software used for this research is HEC-HMS 3.1.0 (USACE 2006). The U.S. Army Corps of Engineers developed the hydrologic modeling system HEC-HMS and it is available without cost to the general public. HMS is designed to simulate the precipitation-runoff processes of dendritic, or branching, watershed systems. HMS is applicable to a wide range of hydrologic problems such as large river basin water supply, flood hydrology, or small urban or natural watershed runoff (USACE 2006). The availability of HEC-HMS and the capabilities of the ArcGIS extension program, HEC-GeoHMS, make HMS an effective tool for this research's modeling needs.

National Resource Conservation Service Curve Number Method

The Natural Resources Conservation Service (NRCS), formerly the Soil Conservation Service (SCS), developed a method of computing the direct runoff resulting from a storm by studying the infiltration behavior of different types of soils (NRCS 1986). The factors affecting infiltration are: hydrologic soil group, land cover type, hydrologic condition, and antecedent moisture condition. To estimate the runoff from a storm, NRCS uses the runoff curve number (CN) method. NRCS (1986) describes the CN method in detail and is summarized here. The SCS runoff equation is:

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S}, \tag{2.1}$$

where: Q = runoff (in),

P = Rainfall (in),

S = Potential maximum retention after runoff begins (in), and

I_a = Initial Abstraction (in).

The initial abstraction (I_a) value accounts for all losses before runoff begins, such as ponding in surface depressions, interception by vegetation, evaporation, and infiltration. I_a is variable and for small agricultural watersheds is routinely approximated by:

$$I_a = 0.2S\,. \tag{2.2}$$

Substituting Equation 2.1 into Equation 2.2 gives:

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)}\,. \tag{2.3}$$

Potential maximum retention (S) is related to the physical characteristics of the landscape with the curve number (CN), as:

$$S = \frac{1000}{CN} - 10\,. \tag{2.4}$$

CN has a range of 0 to 100, with greater numbers reflecting greater runoff potential, and is calculated based on two major classifications of the landscape: the hydrologic soil group (HSG) and the land use and land cover condition (LULC). The hydrologic soil groups (which are labeled A, B, C, and D) indicate the minimum rate of infiltration obtained for bare soil after prolonged wetting. NRCS (1986) describes the soil groups in detail. Group A soils have low runoff potential and high infiltration rates even when thoroughly wetted (sand, loamy sand, or sandy loam). Group B soils have moderate infiltration rates when thoroughly wetted (silt loam or loam). Group C soils have low infiltration rates when thoroughly wetted (sandy clay or loam). Group D soils have high runoff potential. They have very low infiltration rates when thoroughly wetted (clay loam, silty clay loam, sandy clay, silty clay, or clay).

Curve numbers are estimated from the hydrologic soil groups, described above, and land use and land cover (NRCS 1986). Technical Report 55 (TR55) contains tabulated values for curve numbers based on the hydrologic soil group for different LULC types (NRCS 1986). There are different methods for determining LULC values, including field observations, aerial photos, and land use maps. The LULC dataset for this project was obtained from the 2001 National Land Cover Database (NLCD). This dataset was downloaded from the United States Geologic Survey (USGS) website and imported as a layer into ArcGIS. Unfortunately, the NLCD database's land cover descriptions do not directly correlate with the land cover descriptions in the TR55 tables. This lack of correlation requires judgment on the part of the modeler to determine an appropriate CN from the digital data.

Time Response Characteristics for Watersheds

Chow and others (1988) defines the time of concentration as the time at which the entire watershed begins to contribute to runoff. Roussel and others (2005) used 92 Texas watersheds to assess various approaches for estimating watershed characteristics necessary to estimate time of concentration. Based on their analysis, the preferable approaches for estimation of time of concentration are the Kirpich inclusive approaches and the Kerby-Kirpich approach. Their preference is based on simplicity of the approach and ease of input-data acquisition. According to Roussel and others (2005), the Kerby-Kirpich approach is straightforward to use and produces time of concentration values which mimic time to peak from analysis of observed rainfall and runoff data for the 92 watersheds.

Equation 2.5 is adapted from the method presented by Kerby (1959) to estimate travel time for overland flow (T_t),

$$T_t = K(L * N)^{0.467} S^{-0.235}, \tag{2.5}$$

where: T_t = Overland flow time of concentration (min),

K = Unit conversion coefficient (Traditional units K = 0.828, SI units K = 1.44),

L = Length of overland flow (ft),

N = Retardance coefficient based on condition of the overland flow surface, and

S = Dimensionless overland flow slope.

In Equation 2.5, the length of overland flow (L) becomes channel flow within about 1,200 feet (366 meters) (Kerby 1959). Roussel and others (2005) state that this maximum overland flow length is considered an upper limit and shorter values are generally expected in practice. Typical values for the retardance coefficient are listed Table 2.1.

Table 2.1. Retardence coefficients for the Kerby Equation (Kerby 1959).

Surface Cover Type	Retardance Coefficient (N)
Smooth, impervious surface	0.02
Smooth, bare, packed soil	0.10
Poor grass, cultivated row crops, or moderately rough packed surfaces	0.20
Pasture or average grass	0.40
Deciduous timberland	0.60
Dense grass, coniferous forest, or deciduous fores with deep litter	0.80

Kirpich (1940) provided a method to estimate travel time for channel flow (T_{ch}),

$$T_{ch} = KL^{0.770}S^{-0.385}, \tag{2.6}$$

where: T_{ch} = Channel flow time of concentration (min),

K = Unit conversion coefficient (Traditional units K = 0.0078, SI units K = 0.0195),

L = Length of channel flow (ft), and

S = Dimensionless main channel slope.

Together, the Kerby equation for overland flow and the Kirpich equation for channel flow combine to get the Kerby-Kirpich method for time of concentration as shown in Equation 2.7.

$$T_c = T_t + T_{ch} \tag{2.7}$$

where: T_c = Time of concentration (min),

T_t = Overland flow time of concentration (min), and

T_{ch} = Channel flow time of concentration (min).

NRCS Unit Hydrograph Method

The unit hydrograph is a simple linear model that can be used to derive the runoff hydrograph resulting from excess rainfall on a watershed (Chow and others 1988). A unit hydrograph is the hydrograph of runoff from a watershed in response to one unit of effective precipitation over a specific length of time over the entire watershed area. For this research, a synthetic unit hydrograph (the NRCS Unit Hydrograph) method is used to transform the excess precipitation of a watershed or sub-watershed to the outlet. The NRCS Unit Hydrograph was developed from analysis of a large number of unit hydrographs from field data of various-sized basins throughout the United States (USACE 2000). This field data was generalized into dimensionless unit hydrographs and a best-fit hydrograph was developed. The general hydrograph is scaled by the time lag to produce the unit hydrograph for use in HEC-HMS (USACE 2006).

Chow and others (1988) define the lag time as the time between the center of mass of excess rainfall and the peak flow of the resulting direct runoff hydrograph. The NRCS studied the unit hydrographs from the large and small rural watersheds and found that, in general, lag time can be approximated by:

$$t_p = 0.6T_c \, , \tag{2.8}$$

where : t_p = Lag time (min), and

T_c = Time of concentration (min).

Chow and others (1988) explain that the NRCS Unit Hydrograph model is a dimensionless, single-peaked unit hydrograph in which the discharge is expressed by the ratio of discharge to peak discharge and the time is expressed as a ratio of time to the time to peak discharge. The peak discharge, time to peak, and base time are calculated using:

$$Q_p = \frac{483.4A}{T_p} \text{, and} \tag{2.9}$$

$$T_p = \frac{D}{2} + t_p \, , \tag{2.10}$$

where: Q_p = peak discharge (cfs),

A = Watershed area (mi^2),

T_p = Time to peak (hrs),

D = Duration of effective rainfall (hrs), and

t_p = Watershed lag time (hrs).

The author notes that Equations 2.8 and 2.9 are dimensional and must be used within the stated units. The constant 483.4 is both a unit conversion (cfs/in-hr-mi^2) as well as a peaking rate factor.

Flow Routing

When the runoff from a modeled watershed or sub-watershed enters a river reach, it is moved to the next downstream element using a routing method. Chow and others (1988) describe flow routing as a procedure to determine the time and magnitude of flow at a certain point within a basin element from one or more points upstream of that point. Flow routing may be thought of as an analysis to trace runoff flow through a hydrologic system. HEC-HMS has a total of six different routing methods to choose from. Each method requires differing levels of detail and not all methods are equally applicable at representing a particular stream (USACE 2006).

The lag method is selected for the river routing in this research. Lag routing is the simplest of the routing models within HEC-HMS. The volume and shape of the outflow hydrograph is the same as the inflow hydrograph, but is simply offset in time (USACE 2000). For this research, the lag time is computed using the Kirpich Equation for channel travel time (Equation 2.6).

Rainfall-Runoff database for Texas Watersheds

The rainfall-runoff database used for this research was provided by the United States Geologic Survey. It contains over 1600 storms for over 90 different watersheds located in central Texas. The database is divided into five “modules” based on the geographic locations of the watersheds. The five modules are: Austin, Ft. Worth, Dallas, San Antonio, and small rural watersheds. Asquith and others (2004) give a detailed analysis of the development of this database.

CHAPTER III

METHODS AND PROCEDURES

Introduction

In Chapter 3, the methods and procedures used to address the objectives stated in Chapter 1 are described. These objectives are, in summary, to:

1. Use ArcGIS and the extension tools Arc Hydro and HEC-GeoHMS to delineate watersheds and sub-watersheds and to extract modeling parameters for each,
2. To evaluate the enhanced or diminished prediction value on watershed modeling as a function of subdivision, and
3. To determine if there is a certain percentage of a watershed that needs to be significantly different from the rest in order to justify subdividing.

Watershed Locations

Five watersheds were selected for this research project: Walnut Creek, Ash Creek, South Mesquite Creek, Calaveras Creek, and Pond-Elm Creek. Watershed drainage areas ranged between 7.1 and 46.1 square miles. The watersheds were selected based on certain attributes unique to each one. The locations of the watersheds are shown in Figure 3.1.

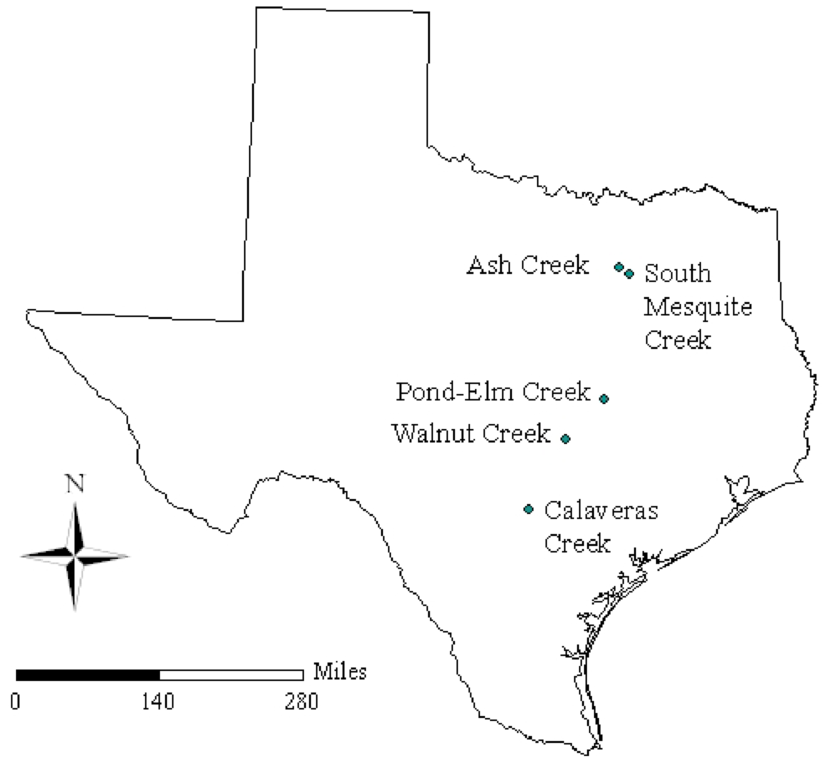

Figure 3.1. Texas watershed locations.

Walnut Creek is located near Austin and is considered an urban watershed. It is mostly developed, but does have areas that are undeveloped. Ash Creek and South Mesquite Creek are both located in Dallas and are urban watersheds. Ash Creek contains two distinct sections within the watershed. The northern 1/3 of the watershed is relatively flat and does not have distinct channel segments, while the southern 2/3 contains steeper sections and has distinct channel properties. South Mesquite Creek has a main channel running almost the entire length of the watershed with relatively short side branches.

South Mesquite Creek is also a common watershed between completed and concurrent research projects.

Calaveras Creek is located in a rural part of Texas near San Antonio and is mostly undeveloped. It has a distinct main channel section with multiple branching side channels. Pond-Elm Creek is also an undeveloped watershed located in a rural section of Texas. This watershed contains a long slender channel along the western side of the watershed and has relatively flat areas at the upstream end. These five watersheds are summarized in Table 3.1 below. Maps showing each watershed are presented in Appendix A.

Table 3.1. Summary of study watersheds.

Watershed	Module	Urban / Rural	Area (mi^2)
Walnut Creek	Austin	Urban	26.5
Ash Creek	Dallas	Urban	7.7
South Mesquite Creek	Dallas	Urban	23.3
Calaveras Creek	Small Rural	Rural	7.1
Pond-Elm Creek	Small Rural	Rural	46.1

Watershed Delineation

The steps used to accomplish the first objective stated in Chapter 1, which was to use the geographic information system ArcGIS to delineate watersheds and to extract the modeling parameters, are described in this section. The beginning of this section contains a brief description of the digital datasets used in ArcGIS along with a description of the data's source and intended use.

National Elevation Dataset (NED)

The digital elevation dataset used in this project was obtained from the USGS. USGS personnel developed the National Elevation Dataset (NED) by merging the highest-resolution, best quality elevation data available across the United States into a seamless raster format (USGS 1999b). ArcGIS and, specifically, Arc Hydro use the DEM as the base layer for all of the watershed delineation steps. The digital elevation data can be downloaded free of charge from the USGS website at: http://www.seamless.usgs.gov/.

National Hydrography Dataset (HND)

The National Hydrography Dataset (NHD) provides hydrographic data throughout the United States. This dataset consists of millions of features, including water bodies, water features such as streams and rivers, and also point features such as springs and wells. The NHD was created from a cooperative effort between the U.S. Environmental Protection Agency (USEPA) and the USGS (USGS 1999a).

The river flowlines within the NHD dataset were used in the DEM reconditioning step in the watershed delineation process. In the DEM reconditioning step, the location of the river within the NHD dataset is "burned" onto the DEM by lowering the elevation value in the raster cells. This ensures that the stream locations delineated from the DEM in subsequent steps match NHD river locations. The NHD dataset can be downloaded free of charge from the USGS website at: http://nhdgeo.usgs.gov.viewer.htm.

Land Use and Land Cover

Land use and land cover (LULC) data were described using the 2001 National Land Cover Database (NLCD). The NLCD was produced and funded through the Multi-Resolution Land Characteristics Consortium (MRLC). The Consortium consisted of 10 federal agencies, each of which required land cover data for their needs (Homer and others 2004). Land uses are classified by the scheme shown in Table 3.2.

The NLCD is the first of two layers needed to create the curve number grid within ArcGIS. As mentioned in Chapter 2, curve numbers are calculated from the Hydrologic Soil Groups and LULC classifications. In its downloaded form, the LULC classifications do not directly correlate with the land cover descriptions used in the NRCS curve number method. This discord requires judgment on the part of the modeler to determine an appropriate curve number from the digital data. This process is described in more detail later in this chapter. The NLCD is available to download free of charge from the USGS website: http://www.seamless.usgs.gov/.

Table 3.2. NLCD land use classifications adapted from Homer (2004).

Category	Classification	Description
Water	11	Open Water
	12	Perennial Ice/Snow
Developed	21	Developed, Open Space
	22	Developed, Low Intensity
	23	Developed, Medium Intensity
	24	Developed, High Intensity
Barren	31	Barren Land
	32	Unconsolidated Shore
Forested Upland	41	Deciduous Forest
	42	Evergreen Forest
	43	Mixed Forest
Shrubland	51	Dwarf scrub
	52	Shrub/Scrub
Herbaceous Upland	71	Grassland/Herbaceous
	72	Sedge/Herbaceous
	73	Lichens
	74	Moss
Planted / Cultivated	81	Pasture/Hay
	82	Cultivated Crops
Woody Wetlands	90	Woody Wetlands
	91	Palustrine Forested Wetland
	92	Palustrine Shrub/Shrub Wetland
	93	Estuarine Forested Wetland
	94	Estuarine Shrub/Shrub Wetland
	95	Emergent Herbaceous Wetlands
Emergent Herbaceous	96	Palustrine Emergent Wetland (Persistent)
Wetlands	97	Estuarine Emengent Wetland
	98	Palusrine Aquatic Bed
	99	Estuarine Aquatic Bed

Soil Survey Geographic data

The Soil Survey Geographic (SSURGO) database used in this research was collected, stored, maintained, and distributed by the NRCS. The SSURGO database contains physical and chemical soil properties for approximately 18,000 soil series across the United States. The SSURGO database was designed primarily for farm and ranch, landowner, township, or county natural resource planning and management (NRCS 1995).

The SSURGO soil layer is the second of two layers needed to create the curve number grid within ArcGIS. As mentioned in Chapter 2, curve numbers are calculated from the Hydrologic Soil Groups and LULC database. The SSURGO database contains all information necessary for use in the NRCS curve number method. The SSURGO dataset is available to download free of charge from the NRCS website: http://www.soildatamart.nrcs.usda.gov/.

Watershed and Stream Channel Delineation

The steps taken to delineate the watersheds and stream channels in ArcGIS are described in this section. ESRI (2007) provides detailed instructions on processing GIS data using Arc Hydro. Although the procedures described in the following sections apply to all study watersheds, only South Mesquite Creek will be used to illustrate the process. A flowchart describing Arc Hydro terrain processing steps from the GIS datasets to watershed delineation is shown in Figure 3.2.

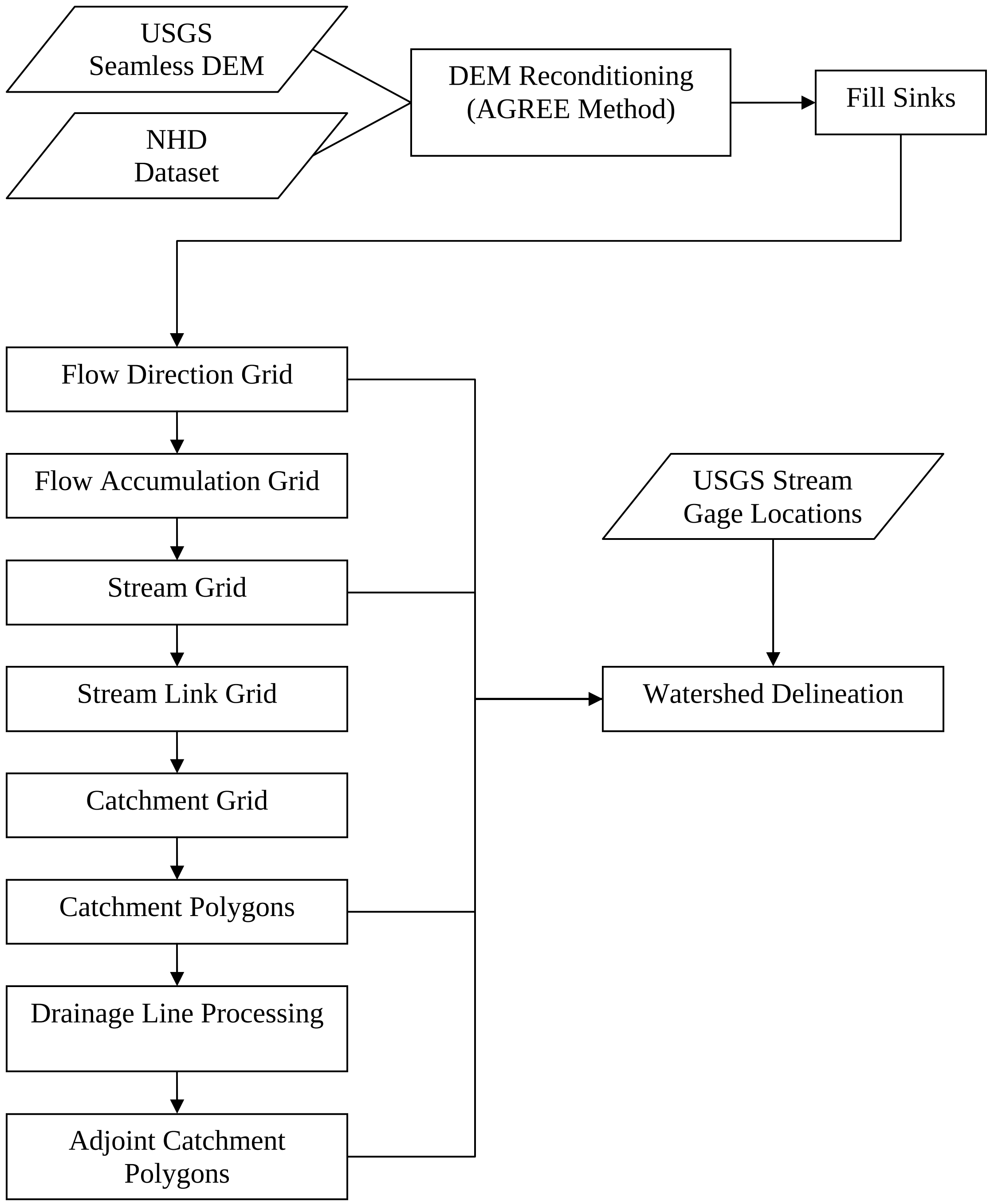

Figure 3.2. Terrain processing flowchart for Arc Hydro (adapted from ESRI 2007).

The DEM underwent two modifications before the watershed and stream channels were delineated. The first modification is called DEM reconditioning. Under DEM reconditioning (also called the AGREE method), the location of the rivers within the NHD were "burned" onto the DEM by the lowering and raising grid cell elevations along the line feature (USACE 2003). AGREE reconditioning was used to avoid potential problems that occur when filling sinks along the channel (Hellweger 1997).

The second modification creates a "hydrologically correct," depressionless DEM by increasing the elevation of any low spots, or sinks, within the DEM where water would be trapped (ESRI 2007). The elimination of these sinks is necessary for the application of the eight-point pour method developed by O'Callaghan and Mark (1984). The depressionless DEM for South Mesquite Creek is shown in Figure 3.3.

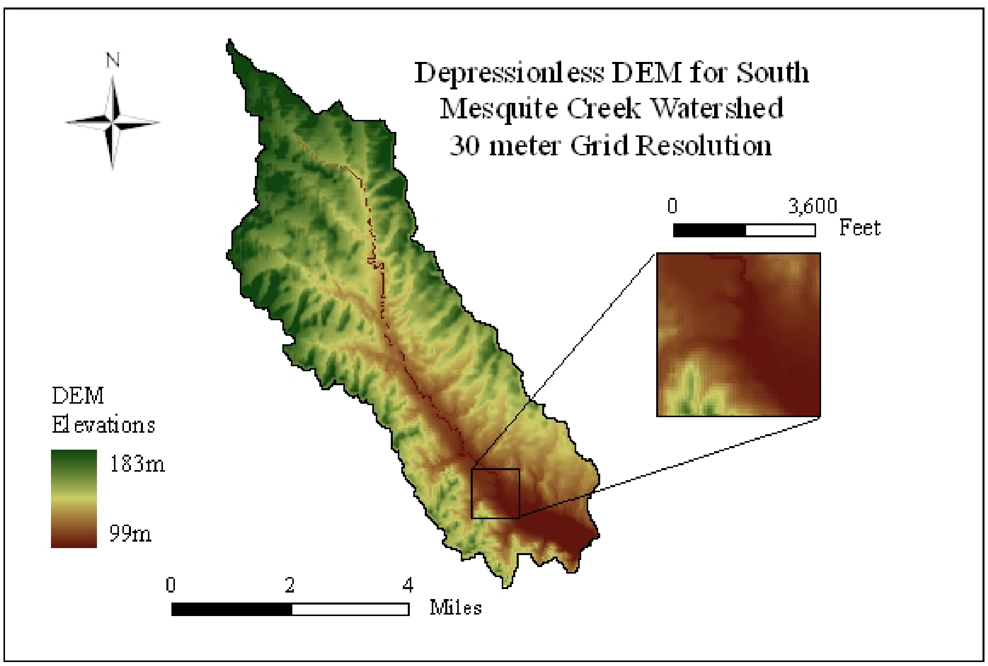

Figure 3.3. Depressionless DEM for South Mesquite Creek.

Flow direction for each of the cells was calculated from the depressionless DEM using the eight-point pour method. Flow direction is defined as the direction of steepest decent for a certain cell in relation to the eight cells surrounding it. In Figure 3.4, the eight possible flow directions are shown.

1	=	east
2	=	southeast
4	=	south
8	=	southwest
16	=	west
32	=	northwest
64	=	north
128	=	northeast

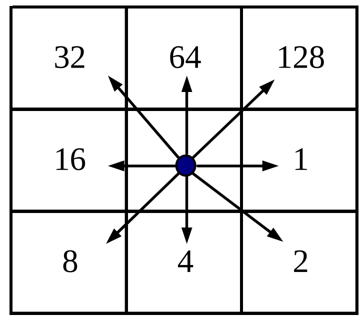

Figure 3.4. Eight-point pour flow directions (adapted from Maidment 2002).

The flow directions layer computed for the South Mesquite watershed using the eight point pour method is shown in Figure 3.5.

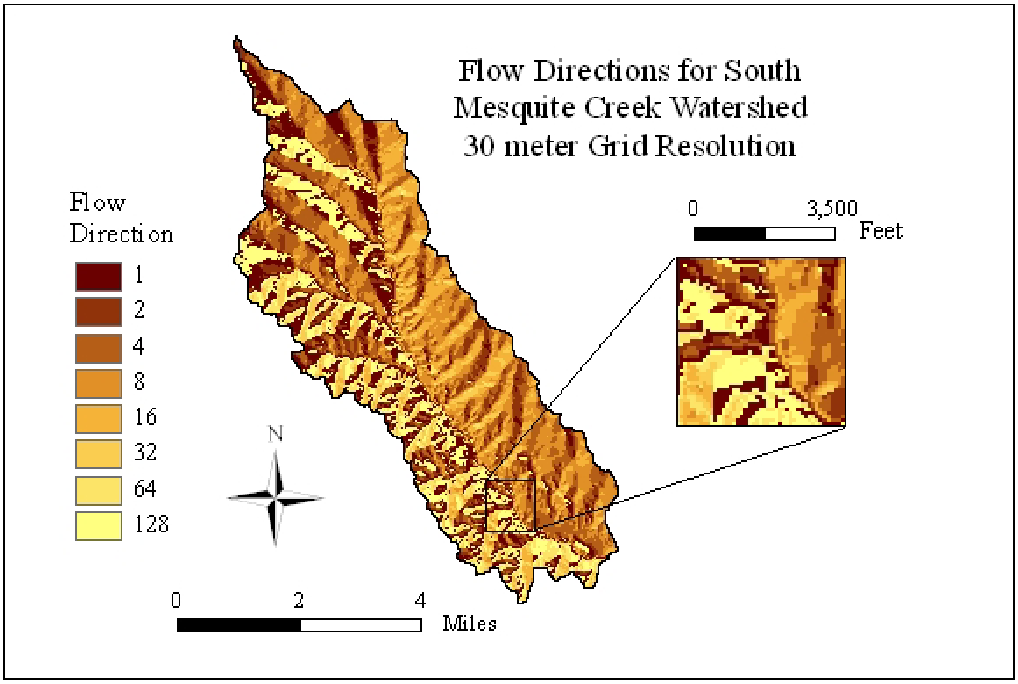

Figure 3.5. Flow directions for South Mesquite Creek.

The flow accumulation grid is calculated from the flow direction grid. The flow accumulation grid is created by assigning a value to a cell which equals the total number of upstream cells that drain into it. The total upstream drainage area can be determined by multiplying the flow accumulation cell value by the cell area, in this case 30m x 30m ($900m^2$). The flow accumulation grid calculated for South Mesquite Creek is shown in Figure 3.6.

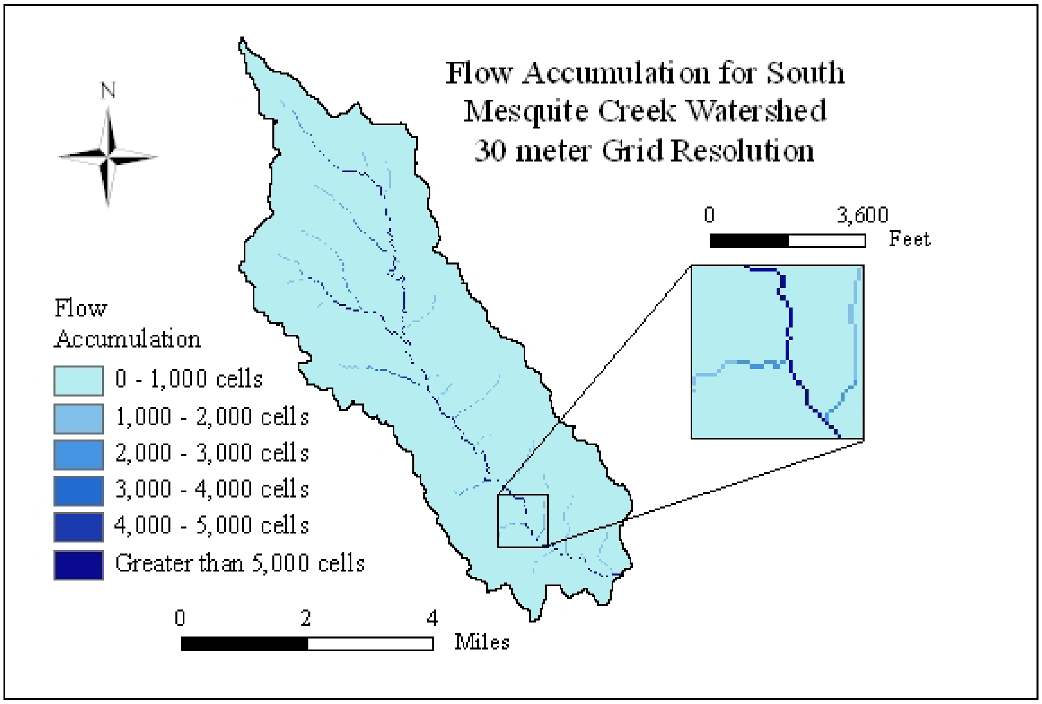

Figure 3.6. Flow accumulation for South Mesquite Creek.

After the flow accumulation grid was generated, streams were defined through the use of a threshold drainage area or flow accumulation value. The NHD uses a minimum value of 5,000 cells for stream definition. This means that all cells with a flow accumulation value greater than 5,000 are classified as stream cells and are assigned a value of 1, while all other cells are assigned a value of NODATA (Maidment 2002). For this research, the 5,000 cell count was used to define a stream. Selection of the threshold value was not a critical step in this process because the sub-watersheds defined for this research were based on physical features of the watershed and not stream definition.

The next step was to divide the stream network into stream segments called links. Links are defined as the river segments between stream confluences (Maidment 2002).

Each stream link has a unique numerical value assigned to it. The flow accumulation grid and the stream link grid were used together in order to define the catchment of cells whose drainage flows through each stream link. The result of the delineation is a catchment grid whose values match the stream link grid (Maidment, 2002). Stream definition using the 5,000 cell threshold and the corresponding catchments are shown in Figure 3.7.

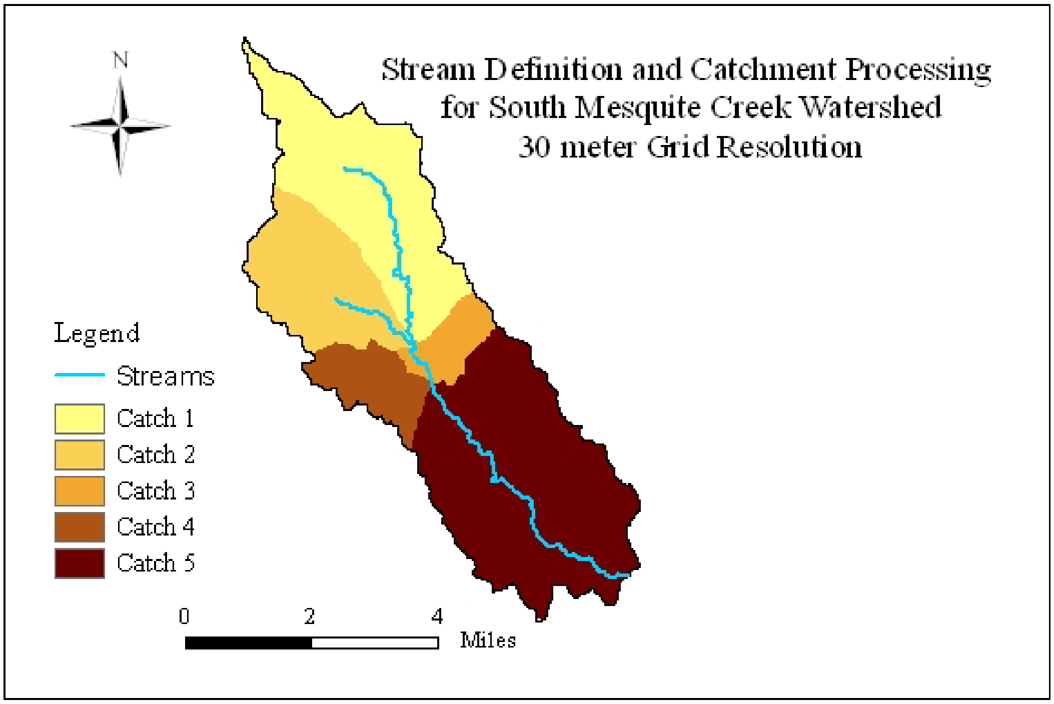

Figure 3.7. Stream and catchment processing for South Mesquite Creek.

The catchment and stream grids were converted to vector format (polygons and lines). Polygons and lines within ArcGIS contain attribute tables which allow the user to extract and store information. The results from terrain analysis, such as watershed areas and channel lengths, were stored in the corresponding attribute tables.

The methods described above are a preprocessing step for watershed delineation using Arc Hydro (Maidment 2002). Using this process, catchments are defined using the stream link grid as guidance. User specified points may also be used to delineate watersheds and sub-watershed. The outlet locations of the five watersheds in this research coincide with USGS streamgage locations. The streamgage locations were aggregated into a point file and ArcGIS was used, with the point file, to delineate the entire watershed. A list of the watersheds and the corresponding USGS stream gage locations are shown in Table 3.3.

Table 3.3. USGS streamgage information (Asquith and others 2004).

Watershed	USGS Station No.	Latitude	Longitude
Walnut Creek	08158200	30°22’30”	97°39’37”
Ash Creek	08057320	32°48’18”	96°43’04”
South Mesquite Creek	08061950	32°43’32”	96°34’12”
Calaveras Creek	08182400	29°22’49"	98°17’33"
Pond-Elm Creek	08108200	30°55’52"	97°01’13"

Sub-watershed Delineation

In the method described above, the watershed outlet locations corresponded to the USGS gages. Watersheds and sub-watersheds can also be defined using arbitrary points. There are a number of different techniques a modeler can use to subdivide a watershed into sub-watersheds. Some of these methods include:

1. An iso-characteristic approach, where each subdivision has about the same physical characteristics (area, length, etc.),
2. An iso-temporal approach, where each sub-watershed has about the same response time (time of concentration),
3. A gage defined approach, where the watershed is divided at stream gage locations,
4. A stream-order/bifurcation approach, where watersheds are subdivided based on stream branches within the drainage network, and
5. A heuristic approach, where watershed subdivision locations are based on characteristics that are unique to each watershed.

For this research, the heuristic approach was chosen, which means that a subdivision scheme applied to one watershed may not apply to another watershed. The criteria used for the location of the watershed subdivisions are described by the following sequence:

1. Subdivide where there is a distinct change in land use or land cover,

2. Where there is a noticeable change in channel or watershed slope, and
3. In areas where there are stream branches within the drainage network.

For example, if a mostly-urban watershed had a section which was undeveloped according to NLCD land cover data then this undeveloped area would be divided out from the total watershed first. Next, areas which could no longer be subdivided according to differences in land use or land cover characteristics were subdivided based on changes in the watershed slope. As the number of subdivisions increased and the sub-watershed areas decreased, land use and slopes become relatively consistent across each sub-watershed. When this happened, stream branches within the drainage network determined subdivision locations.

Each of the five study watersheds was subdivided into 3, 5, 7, 10, 15, and 30 sub-watersheds using the method described above. Maps showing the location of the subdivisions for each watershed are presented in Appendix A. The sub-watershed configurations for South Mesquite Creek are shown in Figure 3.8.

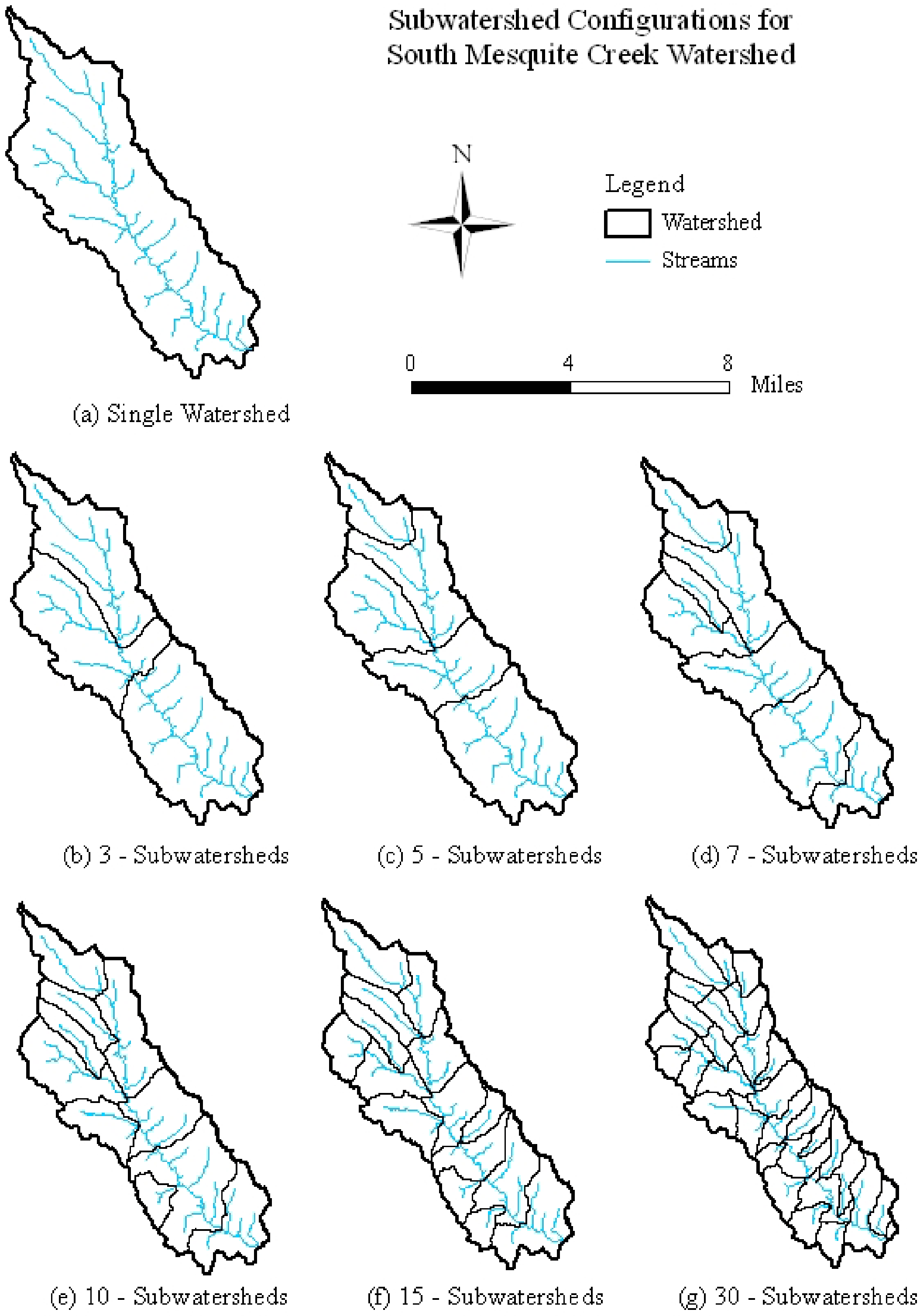

Figure 3.8. Sub-watershed configuration for South Mesquite Creek.

Creation of the NRCS curve number grid

The NRCS curve number method used in HEC-HMS, as described in Chapter 2, requires a curve number assigned to each watershed and sub-watershed. HEC-GeoHMS created a curve number grid using the SSURGO soil dataset and the LULC dataset. A curve number grid is a layer of 30m x 30m cells. Each cell has a curve number value assigned to it. HEC-GeoHMS has tools that can be used to average cell values over a specified area to estimate an overall watershed and sub-watershed curve number. A summary of the steps taken to create the curve number grid is shown in Figure 3.9. Additional details are presented by Merwade (2008a).

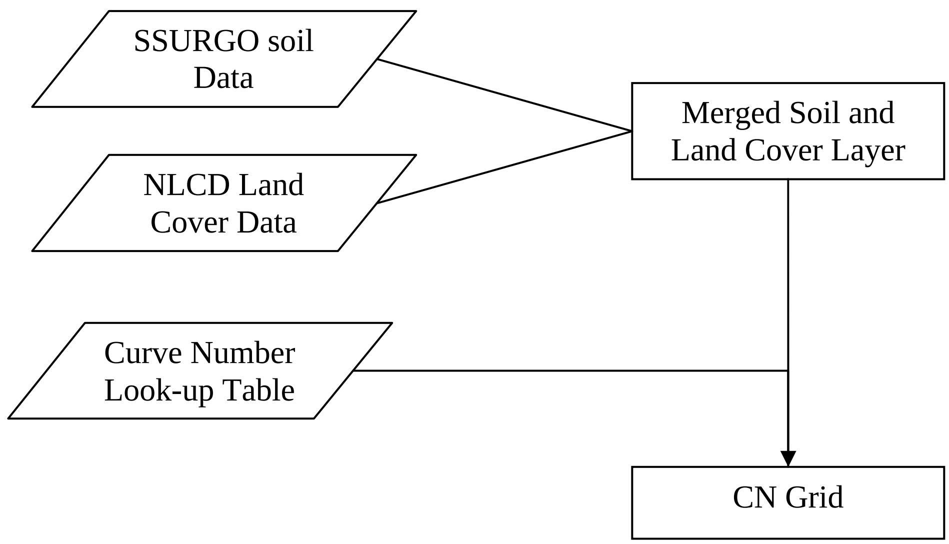

Figure 3.9. Flowchart for CN grid processing (adapted from Merwade 2008a).

The spatial location and descriptions of each soil type within the watershed are contained in the SSURGO layer. The hydrologic soil group (A, B, C, and D), which is needed for NRCS curve number method, is also contained in the SSURGO layer. Additional details on organizing the SSURGO database are presented by Merwade (2008b). When the SSURGO soil data layer and the NLCD land cover layer are merged, a layer is created that has land cover classifications and hydrologic soil group descriptions at every location within the watershed.

Because the NLCD land cover classifications do not directly correlate with the cover descriptions within the NRCS curve number method, it was necessary to apply judgment and make assumptions concerning the most appropriate curve number to assign to each soil and land cover combination. This was accomplished by creating a curve number look-up table in spreadsheet format. The curve number values chosen for each combination of hydrologic soil group and LULC used for this research are shown in Table 3.4. HEC-GeoHMS uses the look-up table in correlation with the merged SSURGO and LULC layer to create a new gridded curve number layer. The gridded curve number layer for South Mesquite Creek is shown in Figure 3.10.

Table 3.4. NRCS curve number values for SSURGO and NLCD attributes.

Class.	NLCD Land Description	A	B	C	D	NRCS (1986) TR-55 Cover Descriptions
11	Open Water	100	100	100	100	
21	Developed, Open Space	39	61	74	80	Open Space - Good Condition
22	Developed, Low Intensity	57	72	81	86	Residential - 1/3 acre lots
23	Developed, Medium Intensity	77	85	90	92	Residential - 1/8 acre lots
24	Developed, High Intensity	98	98	98	98	Impervious areas - Paved lots, roofs, drives, etc.
31	Barren Land, Rock, Sand, Clay	63	77	85	88	Desert Shrub - Poor Condition
41	Deciduous Forest	36	60	73	79	Woods - Fair Condition
42	Evergreen Forest	36	60	73	79	Woods - Fair Condition
43	Mixed Forest	36	60	73	79	Woods - Fair Condition
52	Scrub/Shrub	35	56	70	77	Brush - Fair Condition
71	Grasslands, Herbaceous	39	61	74	80	Pasture/ Grass/ Range - Good Condition
81	Pasture, Hay	49	69	79	84	Pasture/ Grass/ Range - Fair Condition
82	Cultivated Crops	67	78	85	89	Row Crops / Straight Row - Good Condition
90	Woody Wetlands	100	100	100	100	
95	Emergent Herbaceous Wetlands	100	100	100	100	

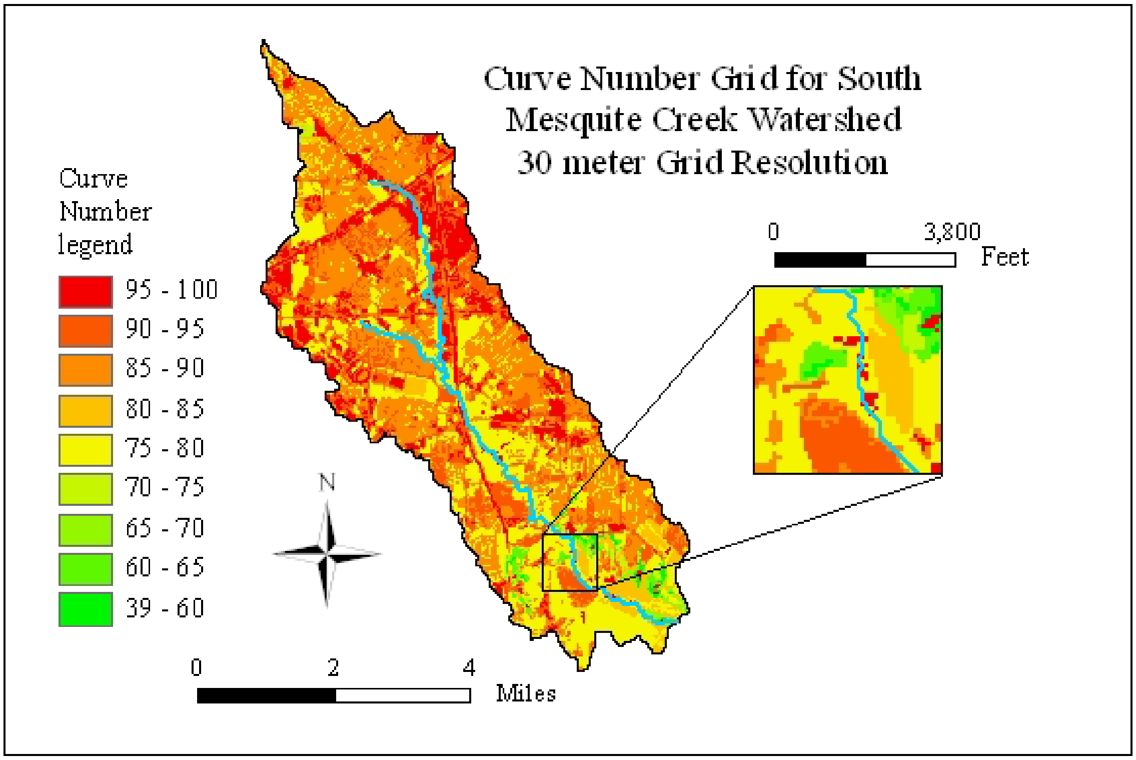

Figure 3.10. Curve Number grid for South Mesquite creek

Watershed Parameters

Once the sub-watersheds were delineated and curve number grids created, the physical properties of each sub-watershed were determined. Watershed and sub-watershed areas can be calculated by adding the number of cells within the basin then multiplying by the cell area. Arc Hydro has tools that determine both the longest flow path of each sub-watershed and the channel routing flow path. Once these flow paths were determined, length and slope parameters were calculated by comparing the flow path to the DEM. These parameters were then used with the Kerby and Kirpich equations to estimate the time of concentration. As mentioned in Chapter 2, the basin lag time (t_{lag}) is 60 percent of the time of concentration.

Hydrologic Modeling System (HEC-HMS)

The second objective stated in Chapter 1 was to evaluate the enhanced or diminished prediction value on watershed modeling as a function of subdivision. Once the watershed and sub-watershed parameters were determined in ArcGIS, they were input into HEC-HMS for analysis. HEC-HMS is designed to simulate the precipitation-runoff processes of watershed systems. A HEC-HMS model is made up of three components: the basin model, meteorologic model, and control specifications. Descriptions of these components are summarized below.

Basin Model

A basin model in HEC-HMS describes the physical representation of watersheds and river channels. The physical landscape is modeled by a series of hydrologic elements connected in a dendritic, or link- node, network. Some of the hydrologic elements include subbasins, reaches (river or stream segments), and junctions. The computation process proceeds from an upstream to downstream direction (USACE 2006). In addition to the basin model containing the physical description of the watershed, it also contains the methods for computing losses, transforming excess precipitation, and routing the channel flow. The methods chosen for each of these models are described in more detail in Chapter 2.

For this research, base flow is ignored. According to Asquith and others (2004), the majority of the USGS streamgages represented in the study are located on small ephemeral streams and base flow represents a small component of the total flow. The majority of the beginning and ending flows from the observed runoff hydrographs begin at or near zero.

Meteorologic Model

The meteorologic model describes the spatial and temporal variation of precipitation inputs to the basin model. The precipitation data may be observed rainfall from historic events or a frequency based hypothetical rainfall. For this research, specified hyetographs from historic storms were used. The hyetographs were obtained from databases provided in Asquith and others (2004). An example hyetograph data file for the Walnut Creek watershed is shown in Figure 3.11.

```
# HYETOGRAPH FILE
# Filename=rain_sta08158200_1976_0418.dat
# site=08158200 Walnut Creek at Dessau Road, Austin, Texas.
# latitude=30()22'30"
# longitude=97()39'37"
# drainagearea (mi2)=26.2
# DATE_TIME=date and time in MM/DD/YYY@HH:MM
# PRECIP1=1WLN raw recorded data
# PRECIP2=2WLN raw recorded data
# ACCUM_WTD_PRECIP=accumulated weighted precipitation in
inches
DATE_TIME HOURS_PASSED PRECIP1 PRECIP2 ACCUM_WTD_PRECIP
04/18/1976@00:30:00 0.0000 0.0000 0.0000 0.0000
04/18/1976@00:35:00 0.0833 0.0100 0.0000 0.0100
04/18/1976@00:40:00 0.1667 0.1200 0.0000 0.0600
04/18/1976@00:45:00 0.2500 0.1500 0.0300 0.0900
04/18/1976@00:50:00 0.3333 0.1900 0.0400 0.1200
04/18/1976@00:55:00 0.4167 0.4200 0.1300 0.2800
04/18/1976@01:00:00 0.5000 0.4900 0.2700 0.3800
04/18/1976@01:05:00 0.5833 0.5900 0.3300 0.4600
04/18/1976@01:10:00 0.6667 0.6300 0.4100 0.5200
04/18/1976@01:15:00 0.7500 0.6600 0.4400 0.5500
```

Figure 3.11. Example hyetograph data file (Asquith and others 2004).

The ACCUM_WTD_PRECIP (accumulated weighted precipitation) column in Figure 3.11 was used for the rainfall hyetographs in this study. According to Asquith and others (2004), the accumulated weighted precipitation values are the best available estimate of rainfall for the entire watershed and were derived from the PRECIP1 and PRECIP2 values. When entering the rainfall data, HEC-HMS requires the data to have a regular time interval between entries. Because many of the hyetograph files used for this study contained irregular time steps, the data was converted to 15-minute time intervals using a spreadsheet application. There were five to nine different storms for each watershed selected from the database for HEC-HMS analysis. The storms chosen occurred between the years 1965 and 1986.

Control Specifications

The control specifications of an HMS model describe the start and end date of the simulation, as well as the time interval for the model results. Actual storm dates and times were used for the control specifications. The time interval can range anywhere from 1 minute to 24 hours. To ensure that the peak of the hydrograph is captured, an appropriate time step was computed. According to USACE (2000), when using the SCS (NRCS) unit hydrograph method, the time interval must be less than 29 percent of the lag time. Because the minimum lag time in this study was 22 minutes, an approximate time step of 5 minutes was used. For simplicity, the 5 minute time step was used for all storms for the five study watersheds.

The resulting HEC-HMS runoff hydrographs were compared to observed runoff hydrographs for analysis. The hydrographs were compared using storm volume, peak discharge, and time to peak discharge. The objective was to evaluate what effect, if any, subdividing a watershed had on runoff results when compared to the observed data. The models were intentionally left uncalibrated with respect to observed runoff behaviors. In this sense the watershed models represent the judgment of the hydrologic analyst, as would be applied at ungaged sites.

Changing Watershed Parameters

The third objective stated in the Chapter 1 was to determine if there is a certain percentage of watershed area that needs to be significantly different from the rest in order to justify subdividing. The following three watershed parameters were analyzed: curve numbers, subbasin lag time, and channel routing lag time.

The HEC-HMS models created in the previous section were used as the base models in this section. First, the watershed loss method (the NRCS curve number) was analyzed. The procedure involved making approximately 1/5, 1/3, and then 1/2 of the watershed completely impervious. This would be equivalent to assigning a curve number value of 100. According to USACE (2006), loss calculations are not carried on the impervious area. All of the precipitation that falls on the impervious portion of the watershed becomes excess precipitation. While it was not possible to divide the watersheds into the exact proportions mentioned above (except in the case of the lumped model where the percent impervious could be directly entered) attempts were made to get as close to these proportions as possible.

Next, the basin's transformation lag time was arbitrarily decreased by 20 percent for the same areas that were analyzed in the loss method. In order to evaluate the effect of transformation lag time only, all other parameters were converted back to original values. Finally, the routing lag time was analyzed. The lag time was also arbitrarily decreased by 20 percent. The results of these experiments will be discussed in Chapter 4.

CHAPTER IV

RESULTS

Introduction

In this chapter, the results of the methods and procedures conducted in Chapter 3 will be discussed. This chapter is divided into three sections. In the first section, the watershed and sub-watershed parameters extracted from ArcGIS will be shown. The results from the HEC-HMS subdivision analysis will be shown in the second section and in the last section, the results of the watershed parameter changes will be shown.

Watershed Parameters

The first objective of this research was to apply Arc Hydro and HEC-GeoHMS to delineate the five study watersheds and to extract modeling parameters for each one. Once this was completed, the five watersheds were subdivided into 3, 5, 7, 10, 15, and 30 sub-watersheds. ArcGIS was used to develop modeling parameters for the sub-watersheds. The results of this analysis are shown within Tables 4.1 to 4.5.

Table 4.1. Basin and routing parameters for Walnut Creek.

Model Config.	SubBasin ID	Area (mi^2)	CN	t_{lag} (min)	Subbasin DownStream Routing Connection	Routing lag time (min)	Routing Downstream Connection
Lumped	A11	26.45	84	158	Outlet	-	-
3 - Unit Simulated	A31	7.40	83	79	R31	201	Outlet
	A32	4.20	84	81	R32	101	Outlet
	A33	14.85	84	140	Outlet	-	-
5 - Unit Simulated	A51	7.40	83	79	R51	36	R52
	A52	5.35	86	77	R52	121	R54
	A53	4.20	84	81	R53	13	R54
	A54	4.53	84	68	R54	95	Outlet
	A55	4.98	82	82	Outlet	-	-
7 - Unit Simulated	A71	1.29	86	43	R71	87	R72
	A72	6.11	83	70	R72	36	R73
	A73	5.35	86	77	R73	27	R74
	A74	1.13	87	54	R74	108	R76
	A75	3.39	83	52	R76	95	Outlet
	A76	4.20	84	81	R75	13	R76
	A77	4.98	82	82	Outlet	-	-
10 - Unit Simulated	A101	1.29	86	43	R101	87	R102
	A102	3.09	82	78	R102	36	R104
	A103	3.01	83	70	R102		
	A104	1.37	87	56	R103	29	R104
	A105	3.98	85	77	R104	27	R105
	A106	4.20	84	81	R106	13	R107
	A107	1.13	87	54	R105	117	R107
	A108	3.39	83	52	R107	95	Outlet
	A109	1.84	83	52	R108	110	Outlet
	A1010	3.13	82	48	Outlet	-	-
15 - Unit Simulated	A151	1.29	86	43	R151	87	R153
	A152	2.15	82	78	R153	36	R156
	A153	0.94	82	48	R152	42	R153
	A154	3.01	83	70	R153		
	A155	1.37	87	56	R154	30	R156
	A156	1.91	84	64	R155	36	R156
	A157	2.07	86	45	R156	27	R157
	A158	1.13	87	54	R157	117	R1511
	A159	2.67	83	52	R1511	95	Outlet
	A1510	0.72	81	43	R158	36	R1511
	A1511	2.70	84	62	R159	55	R1510
	A1512	1.51	82	61	R1510	13	R1511
	A1513	1.84	83	52	R1512	110	Outlet
	A1514	2.52	82	48	Outlet	-	-
	A1515	0.59	79	39	R1513	4	Outlet

Table 4.1.(cont.) Basin and routing parameters for Walnut Creek.

Model Config.	SubBasin ID	Area (mi^2)	CN	t_{lag} (min)	Subbasin DownStream Routing Connection	Routing lag time (min)	Routing Downstream Connection
30 - Unit Simulated	A301	0.71	86	58	R301	87	R306
	A302	0.59	85	50	R301		
	A303	1.40	82	56	R306	36	R3011
	A304	0.76	82	53	R302	71	R306
	A305	0.94	82	48	R303	42	R306
	A306	0.54	81	35	R304	81	R306
	A307	2.14	84	62	R306		
	A308	0.33	86	25	R305	21	R306
	A309	0.57	89	31	R307	57	R308
	A3010	0.79	86	53	R308	29	R3011
	A3011	1.91	84	58	R309	36	R3011
	A3012	0.50	85	38	R309		
	A3013	0.90	88	33	R3010	31	R3011
	A3014	0.67	84	31	R3011	27	R3012
	A3015	1.13	87	52	R3012	117	R3018
	A3016	1.78	83	52	R3018	95	Outlet
	A3017	0.41	84	25	R3016	107	R3018
	A3018	0.48	84	29	R3018		
	A3019	0.72	81	43	R3017	36	R3018
	A3020	1.25	84	40	R3013	65	R3014
	A3021	1.45	85	46	R3014	55	R3015
	A3022	0.79	84	38	R3014		
	A3023	0.71	80	44	R3015	13	R3018
	A3024	0.58	82	41	R3019	37	R3020
	A3025	0.73	85	42	R3019		
	A3026	0.53	81	33	R3020	110	Outlet
	A3027	0.56	83	39	R3021	80	Outlet
	A3028	0.23	84	22	R3022	36	Outlet
	A3029	1.76	82	48	Outlet	-	-
	A3030	0.59	79	45	R3023	4	Outlet

Table 4.2. Basin and routing parameters for Ash Creek.

Model Config.	SubBasin ID	Area (mi^2)	CN	t_{lag} (min)	Subbasin DownStream Routing Connection	Routing lag time (min)	Routing Downstream Connection
Lumped	A11	7.65	82	119	Outlet	-	-
3 - Unit Simulated	A31	1.10	88	80	R31	98	Outlet
	A32	2.35	84	58	R32	57	Outlet
	A33	4.20	78	86	Outlet	-	-
5 - Unit Simulated	A51	1.25	87	80	R51	98	Outlet
	A52	0.91	83	57	R51		
	A53	1.12	83	49	R52	103	Outlet
	A54	2.35	84	58	R53	57	Outlet
	A55	2.03	74	75	Outlet	-	-
7 - Unit Simulated	A71	1.25	87	80	R71	38	R73
	A72	0.91	83	57	R71		
	A73	1.12	83	49	R72	39	R73
	A74	1.30	86	36	R74	51	R75
	A75	1.05	83	50	R75	57	Outlet
	A76	0.70	76	29	R73	82	Outlet
	A77	1.33	72	69	Outlet	-	-
10 - Unit Simulated	A101	0.71	90	76	R101	24	R103
	A102	0.54	83	59	R103	38	R105
	A103	0.51	82	53	R102	14	R103
	A104	0.39	83	45	R103		
	A105	1.12	83	49	R104	39	R105
	A106	1.30	86	48	R106	51	R107
	A107	1.05	83	50	R107	57	Outlet
	A108	0.70	76	44	R105	41	R107
	A109	0.66	75	51	R107		
	A1010	0.67	71	47	Outlet	-	-
15 - Unit Simulated	A151	0.71	90	76	R151	24	R153
	A152	0.54	83	59	R153	38	R156
	A153	0.51	82	53	R152	14	R153
	A154	0.39	84	45	R153		
	A155	0.57	83	39	R154	26	R155
	A156	0.54	83	48	R155	39	R156
	A157	0.86	85	42	R157	20	R158
	A158	0.44	88	38	R158	24	R159
	A159	0.53	86	37	R159	36	R1511
	A1510	0.51	79	43	R1511	57	Outlet
	A1511	0.35	76	48	R156	40	R1511
	A1512	0.36	76	43	R156		
	A1513	0.26	74	45	R1510	27	R1511
	A1514	0.40	75	42	R1511		
	A1515	0.67	71	47	Outlet	-	-

Table 4.2.(cont.) Basin and routing parameters for Ash Creek.

Model Config.	SubBasin ID	Area (mi^2)	CN	t_{lag} (min)	Subbasin DownStream Routing Connection	Routing lag time (min)	Routing Downstream Connection
30 - Unit Simulated	A301	0.42	91	52	R301	61	R302
	A302	0.29	90	56	R302	24	R305
	A303	0.30	84	40	R303	43	R305
	A304	0.24	82	47	R305	38	R3011
	A305	0.51	82	53	R304	14	R305
	A306	0.29	85	40	R304		
	A307	0.10	77	32	R305		
	A308	0.28	83	35	R307	11	R308
	A309	0.29	84	23	R308	26	R3010
	A3010	0.29	85	37	R309	27	R3010
	A3011	0.26	81	42	R3010	39	R3011
	A3012	0.40	85	35	R3012	18	R3013
	A3013	0.13	85	33	R3013	20	R3015
	A3014	0.33	84	36	R3013		
	A3015	0.21	93	25	R3014	7	R3015
	A3016	0.23	84	38	R3015	10	R3017
	A3017	0.21	87	30	R3016	8	R3017
	A3018	0.11	86	29	R3017	18	R3018
	A3019	0.21	84	26	R3018	19	R3019
	A3020	0.16	79	30	R3019	28	R3022
	A3021	0.24	81	36	R3019		
	A3022	0.12	76	27	R3022	57	Outlet
	A3023	0.36	76	43	R3011	40	R3022
	A3024	0.17	78	40	R306	19	R3011
	A3025	0.18	75	38	R3011		
	A3026	0.26	74	45	R3020	27	R3022
	A3027	0.21	79	37	R3021	21	R3022
	A3028	0.18	69	39	R3022		
	A3029	0.23	69	34	R3023	54	Outlet
	A3030	0.44	71	43	Outlet	-	-

Table 4.3. Basin and routing parameters for South Mesquite Creek.

Model Config.	SubBasin ID	Area (mi^2)	CN	t_{lag} (min)	Subbasin DownStream Routing Connection	Routing lag time (min)	Routing Downstream Connection
Lumped	A11	23.33	86	223	Outlet	-	-
3 - Unit Simulated	A31	6.59	88	115	R31	65	R32
	A32	6.89	87	93	R32	252	Outlet
	A33	9.84	84	132	Outlet	-	-
5 - Unit Simulated	A51	2.17	86	57	R51	146	R52
	A52	4.42	90	94	R52	96	R53
	A53	4.01	88	87	R52		
	A54	4.41	87	53	R53	231	Outlet
	A55	8.32	83	98	Outlet	-	-
7 - Unit Simulated	A71	2.17	86	57	R71	146	R73
	A72	4.42	90	94	R73	96	R74
	A73	1.13	87	66	R72	66	R73
	A74	2.89	88	87	R73		
	A75	4.41	87	53	R74	175	R75
	A76	5.48	84	75	R75	92	Outlet
	A77	2.83	82	56	Outlet	-	-
10 - Unit Simulated	A101	2.17	86	57	R101	146	R104
	A102	3.15	91	78	R104	96	R106
	A103	1.28	87	60	R102	105	R104
	A104	1.13	87	66	R103	66	R104
	A105	2.89	88	87	R104		
	A106	2.66	87	67	R106	175	R108
	A107	1.74	87	41	R105	48	R106
	A108	4.66	84	75	R108	92	Outlet
	A109	0.83	85	24	R107	106	R108
	A1010	2.83	82	56	Outlet	-	-
15 - Unit Simulated	A151	2.17	86	57	R151	71	R152
	A152	1.48	92	49	R152	105	R155
	A153	1.67	90	53	R155	96	R157
	A154	1.28	87	60	R152		
	A155	1.13	87	66	R153	66	R155
	A156	1.28	90	45	R154	91	R155
	A157	1.61	87	53	R155		
	A158	1.74	87	41	R156	48	R157
	A159	0.77	89	49	R157	101	R158
	A1510	1.90	86	67	R157		
	A1511	1.62	86	56	R158	106	R1510
	A1512	0.83	85	24	R158		
	A1513	0.66	82	47	R159	52	R1510
	A1514	2.37	84	76	R1510	92	Outlet
	A1515	2.83	82	56	Outlet	-	-

Table 4.3.(cont.) Basin and routing parameters for South Mesquite Creek.

Model Config.	SubBasin ID	Area (mi^2)	CN	t_{lag} (min)	Subbasin DownStream Routing Connection	Routing lag time (min)	Routing Downstream Connection
30 - Unit Simulated	A301	1.74	86	56	R301	4	R302
	A302	0.43	86	34	R302	71	R305
	A303	0.60	90	31	R304	69	R305
	A304	0.88	83	43	R305	105	R3011
	A305	0.53	86	42	R303	41	R305
	A306	0.74	88	30	R305		
	A307	0.54	89	45	R3010	29	R3011
	A308	1.12	90	53	R3011	64	R3013
	A309	0.77	89	45	R308	35	R309
	A3010	0.51	90	34	R308		
	A3011	0.64	87	49	R306	42	R307
	A3012	0.49	88	32	R307	66	R3011
	A3013	0.71	88	23	R309	70	R3011
	A3014	0.90	87	40	R3011		
	A3015	0.95	88	32	R3012	24	R3013
	A3016	0.79	87	43	R3013	48	R3014
	A3017	1.14	86	54	R3013		
	A3018	0.76	87	50	R3014	102	R3017
	A3019	0.77	89	49	R3014		
	A3020	0.32	84	38	R3015	94	R3017
	A3021	0.36	87	47	R3016	58	R3017
	A3022	0.94	85	44	R3017	106	R3021
	A3023	0.83	85	24	R3017		
	A3024	0.65	86	56	R3019	86	R3021
	A3025	0.69	84	53	R3020	24	R3021
	A3026	1.02	82	40	R3021	92	Outlet
	A3027	0.66	82	47	R3018	52	R3021
	A3028	0.59	83	34	R3021		
	A3029	0.61	80	45	R3022	10	Outlet
	A3030	1.64	82	56	Outlet	-	-

Table 4.4. Basin and routing parameters for Calaveras Creek.

Model Config.	SubBasin ID	Area (mi^2)	CN	t_{lag} (min)	Subbasin DownStream Routing Connection	Routing lag time (min)	Routing Downstream Connection
Lumped	A11	7.11	77	95	Outlet	-	-
3 - Unit Simulated	A31	1.69	77	66	R31	83	Outlet
	A32	1.53	77	58	R31		
	A33	3.88	77	76	Outlet	-	-
5 - Unit Simulated	A51	1.69	77	66	R51	45	R52
	A52	1.53	77	58	R51		
	A53	1.90	77	64	R52	52	Outlet
	A54	0.77	78	80	R53	44	Outlet
	A55	1.21	77	70	Outlet	-	-
7 - Unit Simulated	A71	1.69	77	66	R72	45	R74
	A72	0.83	78	58	R72		
	A73	0.69	76	63	R71	10	R72
	A74	1.42	77	59	R74	52	Outlet
	A75	0.48	76	62	R73	32	R74
	A76	0.77	78	80	R75	44	Outlet
	A77	1.21	77	70	Outlet	-	-
10 - Unit Simulated	A101	0.83	77	45	R101	55	R103
	A102	0.87	76	46	R103	45	R105
	A103	0.83	78	58	R103		
	A104	0.69	76	63	R102	10	R103
	A105	0.48	76	62	R104	32	R105
	A106	0.70	77	59	R105	52	Outlet
	A107	0.72	78	56	R105		
	A108	0.77	78	80	R106	44	R105
	A109	0.77	76	72	R107	10	Outlet
	A1010	0.44	78	43	Outlet	-	-
15 - Unit Simulated	A151	0.24	82	41	R151	18	R152
	A152	0.33	75	44	R152	55	R155
	A153	0.26	75	28	R152		
	A154	0.87	76	46	R155	45	R158
	A155	0.44	77	46	R153	24	R155
	A156	0.39	78	40	R155		
	A157	0.69	76	63	R154	10	R155
	A158	0.48	76	62	R156	32	R158
	A159	0.70	77	59	R158	52	Outlet
	A1510	0.38	79	45	R157	24	R158
	A1511	0.35	76	45	R158		
	A1512	0.65	78	80	R1510	44	Outlet
	A1513	0.12	78	35	R159	22	R1510
	A1514	0.77	76	72	R1511	10	Outlet
	A1515	0.44	78	43	Outlet	-	-

Table 4.4.(cont.) Basin and routing parameters for Calaveras Creek.

Model Config.	SubBasin ID	Area (mi^2)	CN	t_{lag} (min)	Subbasin DownStream Routing Connection	Routing lag time (min)	Routing Downstream Connection
30 - Unit Simulated	A301	0.24	82	41	R301	18	R302
	A302	0.33	75	44	R302	35	R304
	A303	0.26	75	28	R302		
	A304	0.15	80	26	R303	30	R304
	A305	0.47	76	41	R304	32	R3010
	A306	0.25	74	42	R3010	15	R3011
	A307	0.29	78	45	R305	24	R3010
	A308	0.15	77	42	R305		
	A309	0.10	82	25	R306	23	R3010
	A3010	0.29	77	46	R3010		
	A3011	0.20	75	43	R307	16	R308
	A3012	0.20	77	36	R308	34	R309
	A3013	0.29	76	44	R309	10	R3010
	A3014	0.44	76	49	R3011	32	R3016
	A3015	0.20	75	48	R3012	31	R3013
	A3016	0.29	77	45	R3013	32	R3016
	A3017	0.25	78	32	R3016	61	R3021
	A3018	0.18	76	45	R3014	22	R3015
	A3019	0.20	72	29	R3014		
	A3020	0.18	76	42	R3015	9	R3016
	A3021	0.17	76	33	R3016		
	A3022	0.15	78	54	R3017	23	R3018
	A3023	0.23	76	35	R3018	40	R3020
	A3024	0.27	79	47	R3020	40	R3021
	A3025	0.12	78	35	R3019	22	R3020
	A3026	0.21	78	37	R3021	17	Outlet
	A3027	0.22	77	54	R3022	23	R3023
	A3028	0.25	76	41	R3023	22	R3024
	A3029	0.31	75	41	R3024	10	Outlet
	A3030	0.23	78	38	Outlet	-	-

Table 4.5. Basin and routing parameters for Pond-Elm Creek.

Model Config.	SubBasin ID	Area (mi^2)	CN	t_{lag} (min)	Subbasin DownStream Routing Connection	Routing lag time (min)	Routing Downstream Connection
Lumped	A11	46.07	86	373	Outlet	-	-
3 - Unit	A31	19.20	86	288	R31	248	Outlet
Simulated	A32	11.88	86	151	R32	74	Outlet
	A33	15.00	86	224	Outlet	-	-
5 - Unit	A51	8.68	86	115	R51	399	R52
Simulated	A52	10.51	86	200	R52	248	Outlet
	A53	5.42	87	131	R52		
	A54	11.88	86	151	R53	74	Outlet
	A55	9.58	86	182	Outlet	-	-
7 - Unit	A71	8.68	86	115	R71	399	R72
Simulated	A72	10.51	86	200	R72	169	R73
	A73	5.42	87	131	R72		
	A74	6.51	86	147	R73	121	Outlet
	A75	4.95	87	104	R74	131	Outlet
	A76	6.92	85	130	R75	74	Outlet
	A77	3.07	85	118	Outlet	-	-
10 - Unit	A101	3.29	86	60	R101	157	R102
Simulated	A102	5.39	86	90	R102	186	R103
	A103	5.00	87	96	R103	283	R104
	A104	5.52	86	147	R104	169	R106
	A105	5.42	87	131	R104		
	A106	2.70	88	108	R105	152	Outlet
	A107	3.82	85	89	R106	121	Outlet
	A108	4.95	87	104	R107	131	R108
	A109	6.92	84	130	R108	74	Outlet
	A1010	3.07	85	118	Outlet	-	-
15 - Unit	A151	3.29	86	60	R151	157	R152
Simulated	A152	5.39	86	90	R152	186	R153
	A153	5.00	87	96	R153	283	R155
	A154	5.52	86	147	R155	169	R157
	A155	3.01	88	101	R154	81	R155
	A156	2.41	87	100	R155		
	A157	2.70	88	108	R156	152	R157
	A158	3.82	85	89	R157	121	Outlet
	A159	2.30	88	72	R158	80	R159
	A1510	2.66	85	77	R159	131	R1511
	A1511	1.32	86	83	R157		
	A1512	2.71	83	83	R159		
	A1513	1.17	87	65	R1510	17	R1511
	A1514	3.05	85	93	R1511	74	Outlet
	A1515	1.75	84	61	Outlet	-	-

Table 4.5.(cont.) Basin and routing parameters for Pond-Elm Creek.

Model Config.	SubBasin ID	Area (mi^2)	CN	t_{lag} (min)	Subbasin DownStream Routing Connection	Routing lag time (min)	Routing Downstream Connection
30 - Unit Simulated	A301	1.01	86	43	R301	33	R302
	A302	0.66	85	53	R302	199	R303
	A303	1.62	87	60	R302		
	A304	1.94	87	70	R303	58	R304
	A305	0.87	86	43	R303		
	A306	0.96	83	45	R304	99	R305
	A307	1.61	86	50	R304		
	A308	1.72	86	49	R305	125	R306
	A309	2.34	87	69	R306	83	R307
	A3010	0.93	87	74	R306		
	A3011	1.77	87	63	R307	167	R308
	A3012	2.40	87	86	R308	114	R3011
	A3013	1.35	85	55	R3011	93	R3013
	A3014	1.57	88	65	R309	79	R3010
	A3015	1.44	88	79	R3010	81	R3011
	A3016	1.02	87	68	R3010		
	A3017	1.39	86	77	R3011		
	A3018	2.70	88	108	R3012	74	R3013
	A3019	2.04	85	59	R3013	104	R3014
	A3020	1.77	84	60	R3014	121	Outlet
	A3021	1.32	86	83	R3014		
	A3022	2.30	88	72	R3015	80	R3018
	A3023	1.57	84	70	R3018	61	R3019
	A3024	1.09	86	63	R3016	41	R3018
	A3025	1.64	85	45	R3017	96	R3018
	A3026	1.07	81	64	R3018		
	A3027	1.85	85	63	R3019	92	R3020
	A3028	1.20	86	51	R3021	74	Outlet
	A3029	1.17	87	65	R3020	17	R3021
	A3030	1.75	84	61	Outlet	-	-

Watershed Subdivision Analysis

The second objective of this research was to evaluate the enhanced or diminished prediction value on watershed modeling as a function of subdivision. Appendix B contains the charts of the computed and observed runoff hydrographs for each storm and subdivision scheme. Three metrics were used to evaluate differences between computed and observed runoff hydrographs. These metrics used the concept of relative error (Equation 4.1). For each storm event and subdivision scheme, the relative error was computed for runoff volume, peak flow, and time to peak. In a “perfect” hydrologic model, this equation would equal zero.

$$RE = \left(\frac{\left| X_C - X_O \right|}{X_O} \right), \tag{4.1}$$

where: RE = Relative Error (%),

X_C = Computed values (Runoff Volume, Peak Flow, or Time to Peak), and

X_O = Observed Values (Runoff Volume, Peak Flow, or Time to Peak).

The first metric used to compare the relative errors was the arithmetic mean. The arithmetic mean is simply the sum of the relative errors divided by the number of storms modeled for that particular watershed,

$$ArithmeticMean = \frac{1}{N} \sum_{i=1}^{N} \left(\frac{\left| X_C - X_O \right|}{X_O} \right)_i, \tag{4.2}$$

where: N = number of storms modeled in a watershed.

The second metric used to compare the relative errors was the root-mean-square-error (RMSE). RMSE is used to study the amount of dispersion within the data. RMSE, as shown in equation 4.2, is defined as the square root of the arithmetic means of the values squared. Since the relative errors are squared, larger values have more influence than smaller values (Davis 1937).

$$RMSE = \sqrt{\frac{1}{N}\sum_{i=1}^{N}\left(\frac{X_C - X_O}{X_O}\right)_i^2} \tag{4.3}$$

The third metric used to compare the relative errors is called MIN_COUNT, and is similar to the acceptance approach used in Cleveland and others (2006). This metric is simply a count of the number of storms for which a particular subdivision scheme performed better than the others.

Runoff Volume

The effect of watershed subdivision on runoff volume is evaluated within this section. Runoff volume is determined in HEC-HMS by the choice of the loss model. For this study the NRCS curve number method was used. The results of the arithmetic mean and RMSE runoff volume calculations for each watershed and subdivision scheme are shown in Figures 4.1 through 4.5. Values near zero imply greater model accuracy.

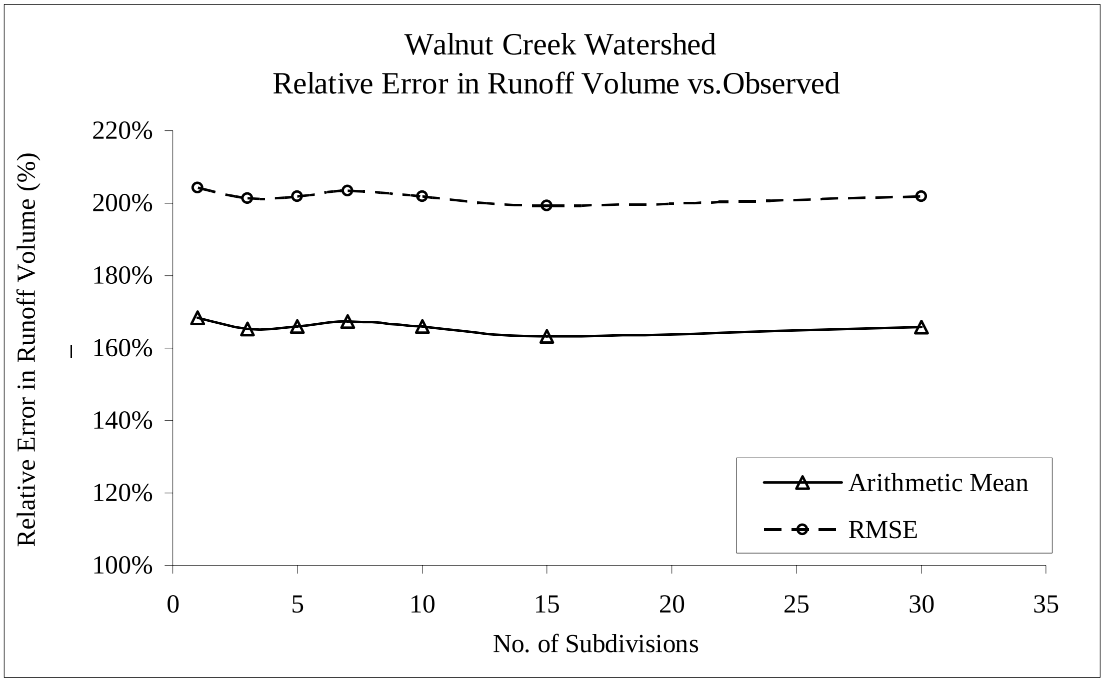

Figure 4.1. Relative error in runoff volume – Walnut Creek.

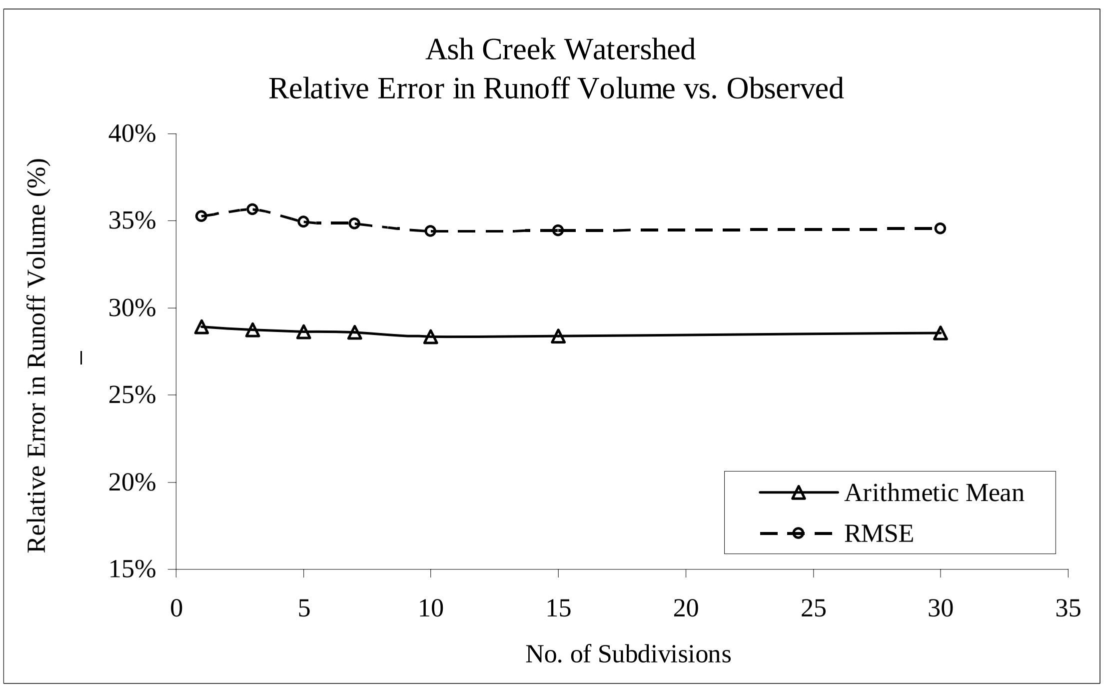

Figure 4.2. Relative error in runoff volume – Ash Creek.

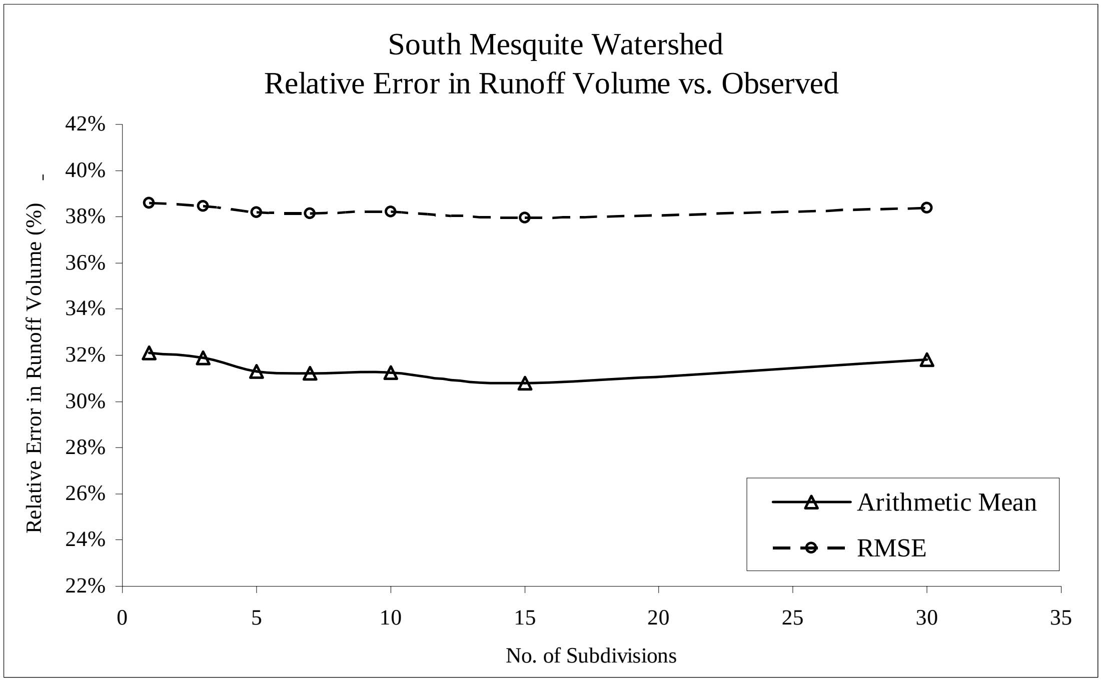

Figure 4.3. Relative error in runoff volume – South Mesquite Creek.

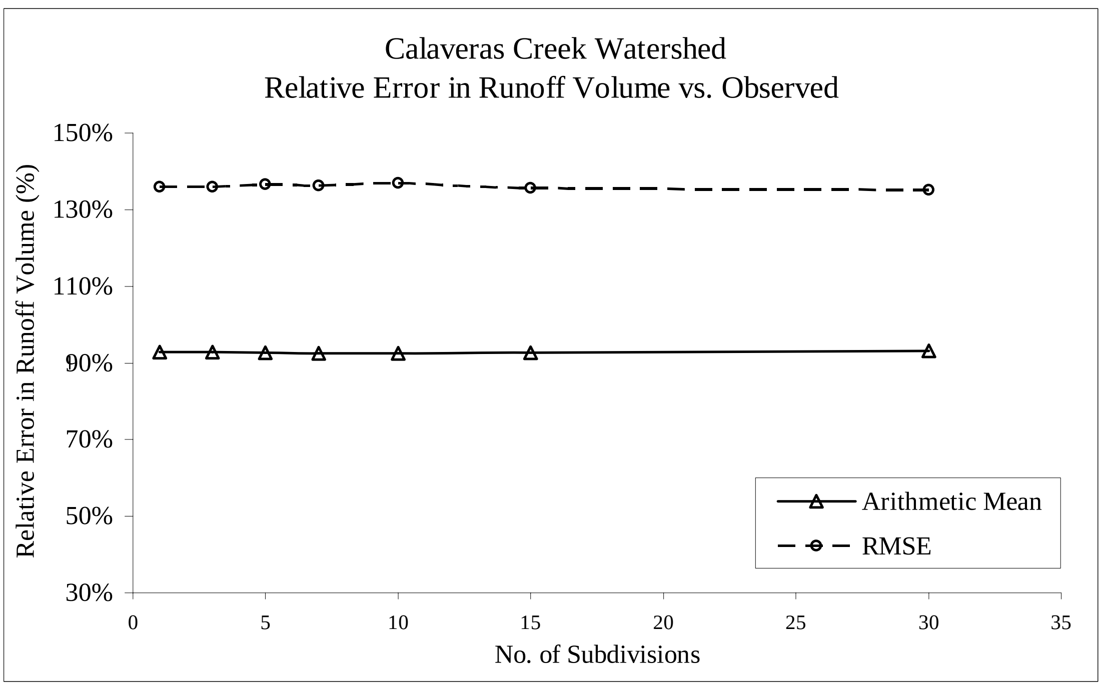

Figure 4.4. Relative error in runoff volume – Calaveras Creek.

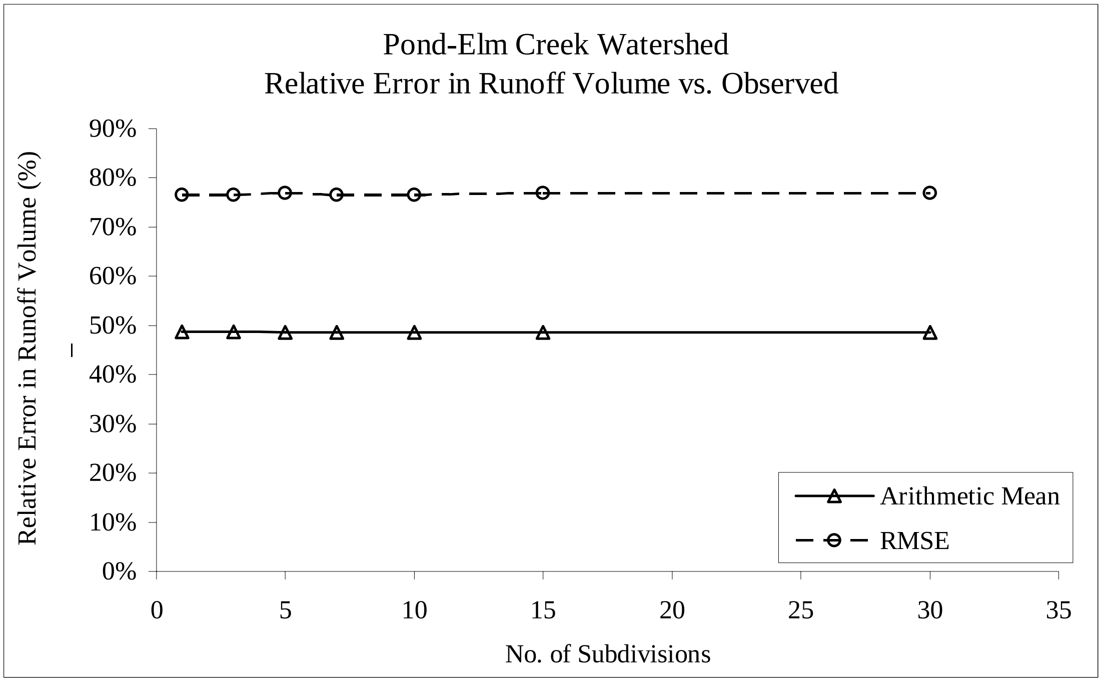

Figure 4.5. Relative error in runoff volume – Pond-Elm Creek.

It is shown in Figures 4.1 to 4.5 that there is little change in runoff volume as the amount of subdivisions increase. One reason behind this is that the curve numbers over the entire watershed remain constant no matter how many times a watershed is subdivided. Because the same ArcGIS produced curve number grid was used for watershed and sub-watershed curve number calculation, the results remain fairly constant. Small variations occur due to the averaging of the curve number grid over the smaller sub-watersheds.

Peak Flow

The effect of watershed subdivision on peak flow is evaluated in this section. The results of the arithmetic mean and RMSE on peak flow calculations for each watershed and subdivision scheme are shown in Figures 4.6 through 4.10.

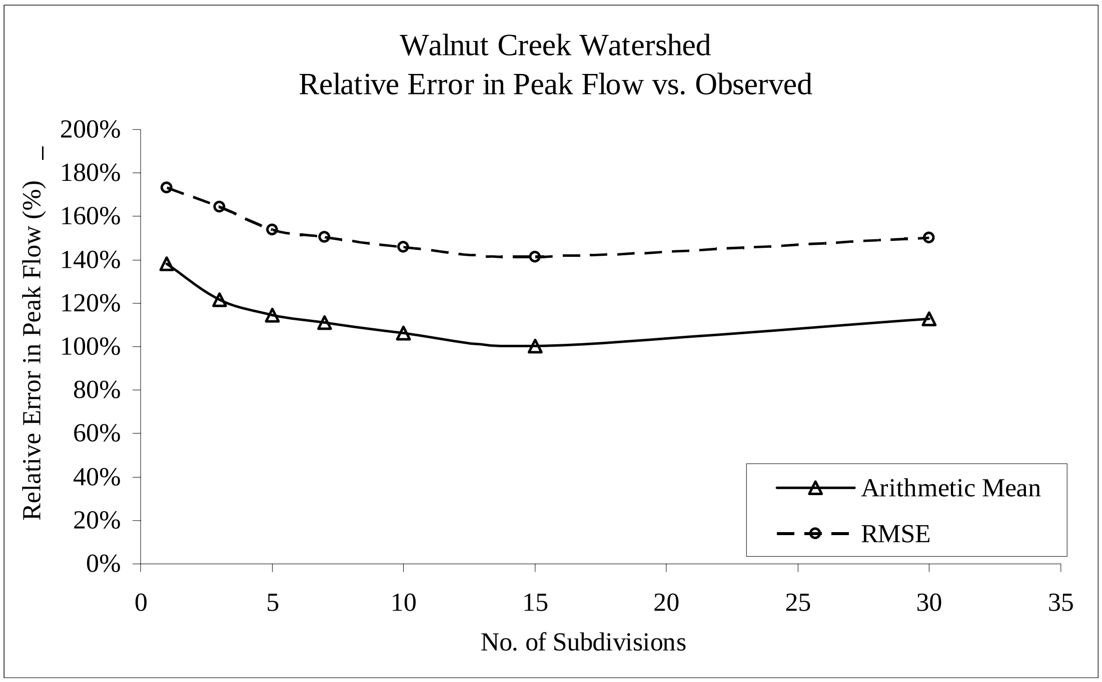

Figure 4.6. Relative error in peak flow – Walnut Creek.

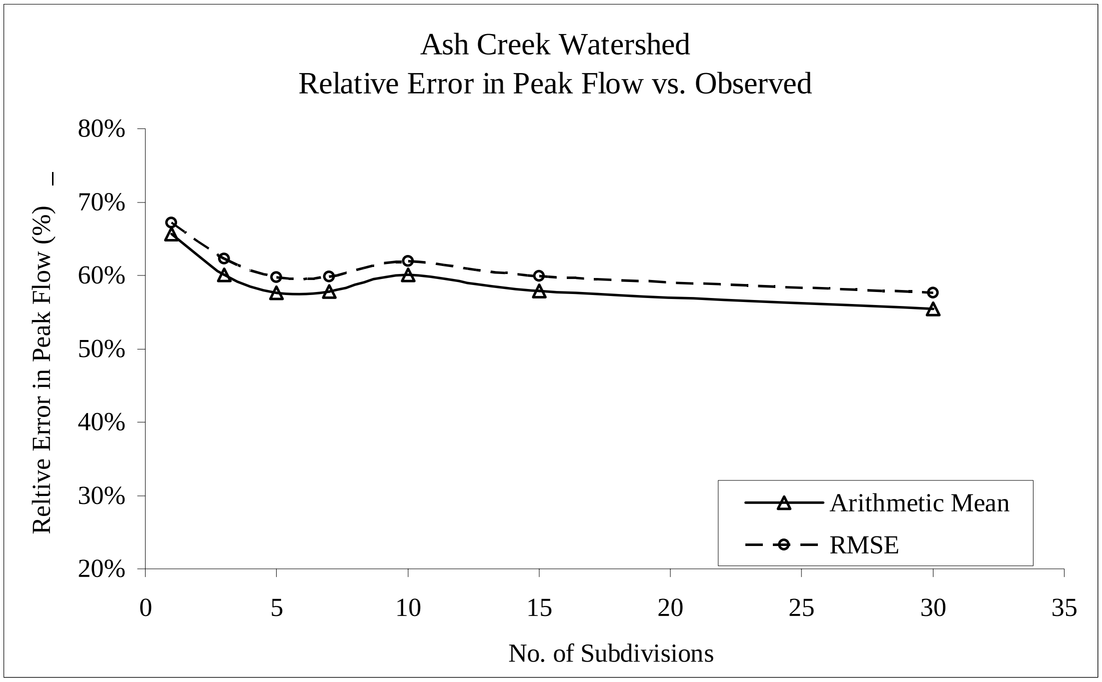

Figure 4.7. Relative error in peak flow – Ash Creek.

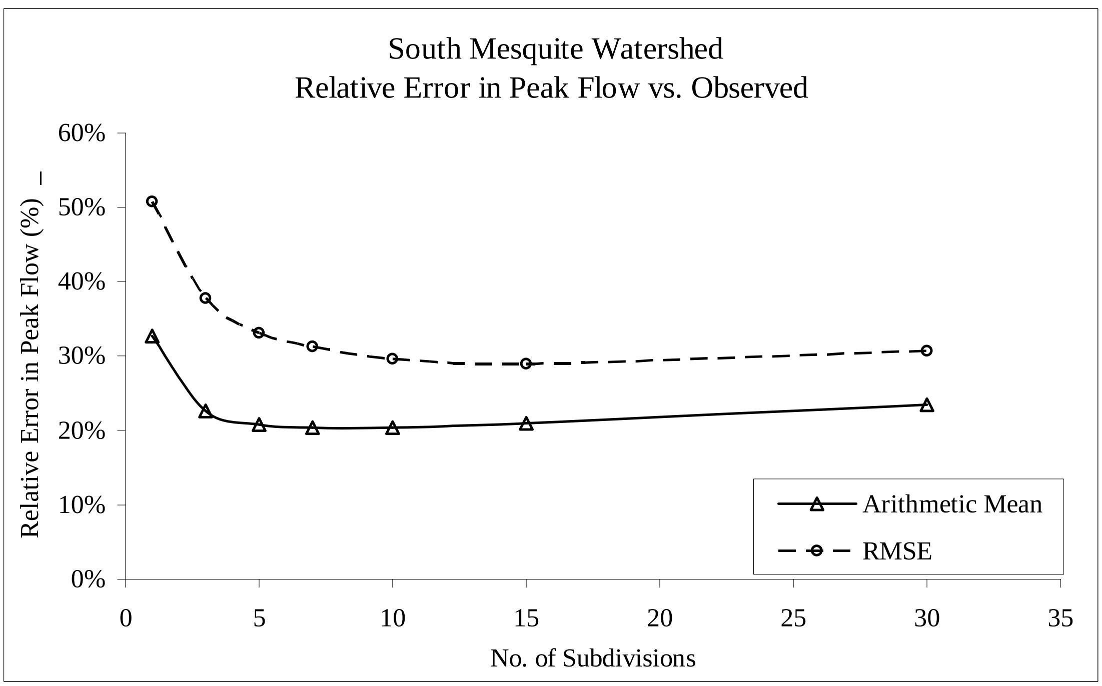

Figure 4.8. Relative error in peak flow – South Mesquite Creek.

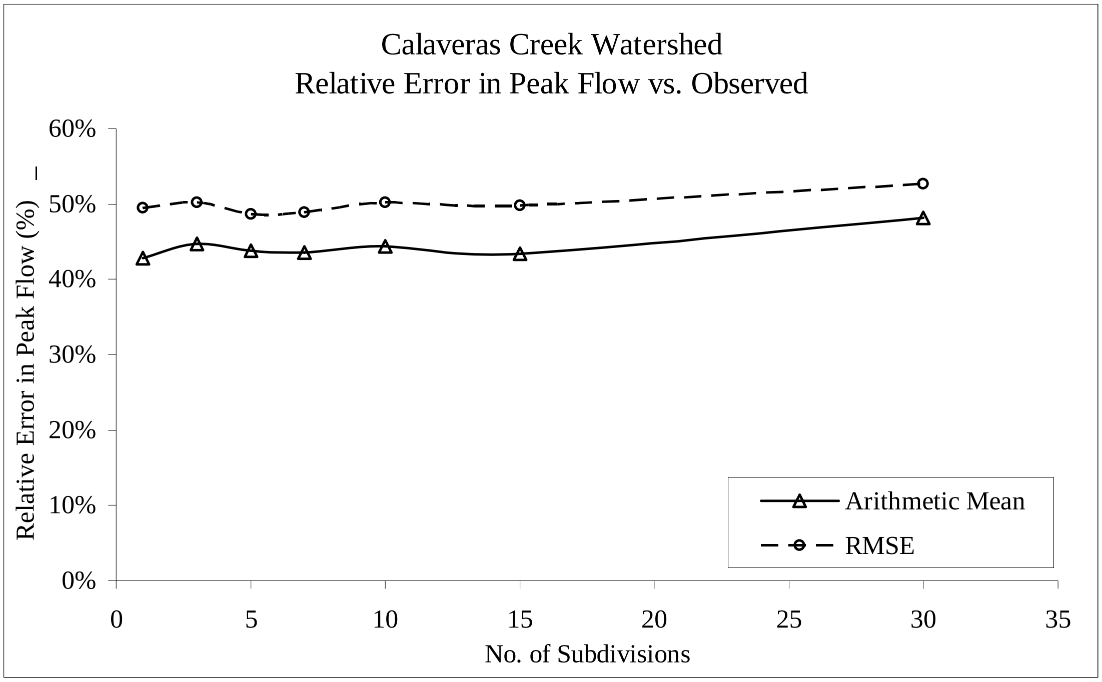

Figure 4.9. Relative error in peak flow – Calaveras Creek.

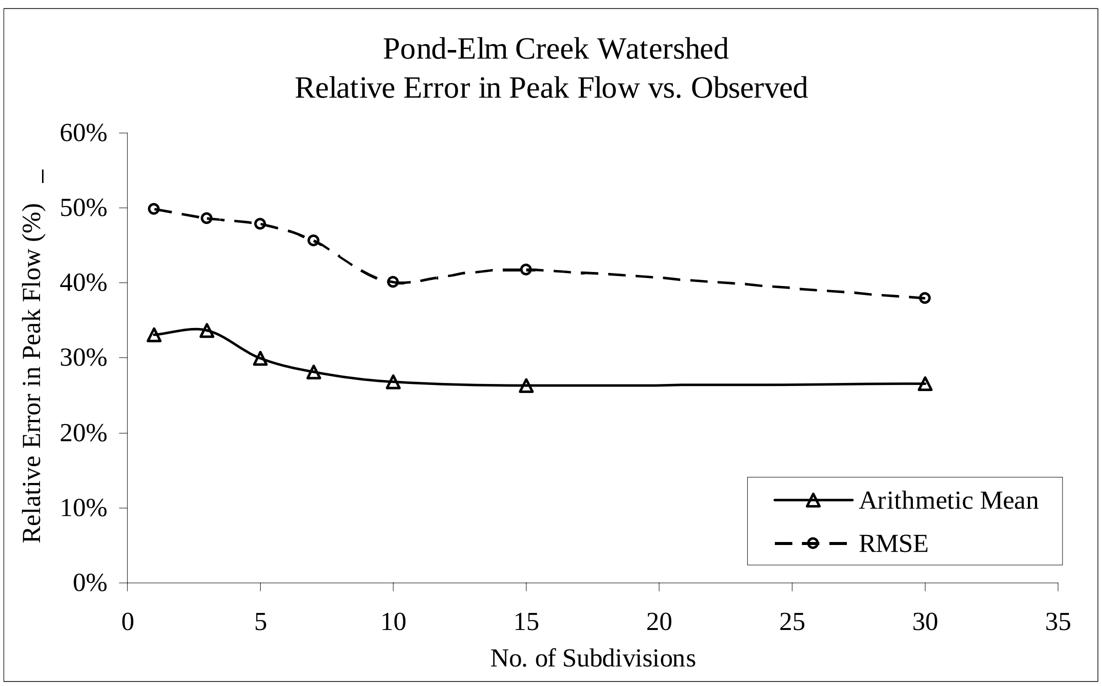

Figure 4.10. Relative error in peak flow – Pond-Elm Creek.

In Figures 4.6 to 4.10 above, it is shown that, with the exception of Calaveras creek, subdividing a watershed tends to increases the accuracy of peak flow when compared to the lumped model. It is also shown in these figures that the rate of accuracy increase levels off after 5 to 10 subdivisions.

Time to Peak

The effect of watershed subdivision on time to peak is evaluated in this section. The results of the arithmetic mean and RMSE on time to peak calculations for each watershed and subdivision scheme are shown in Figures 4.11 through 4.15. Values near zero imply greater model accuracy.

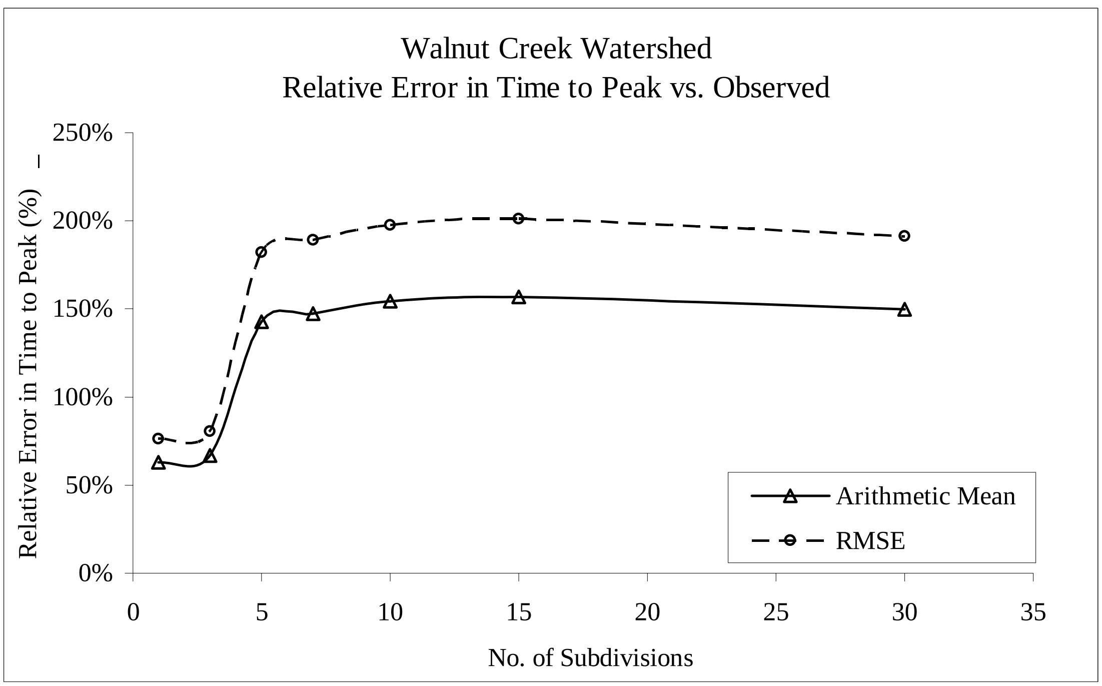

Figure 4.11. Relative error in time to peak – Walnut Creek.

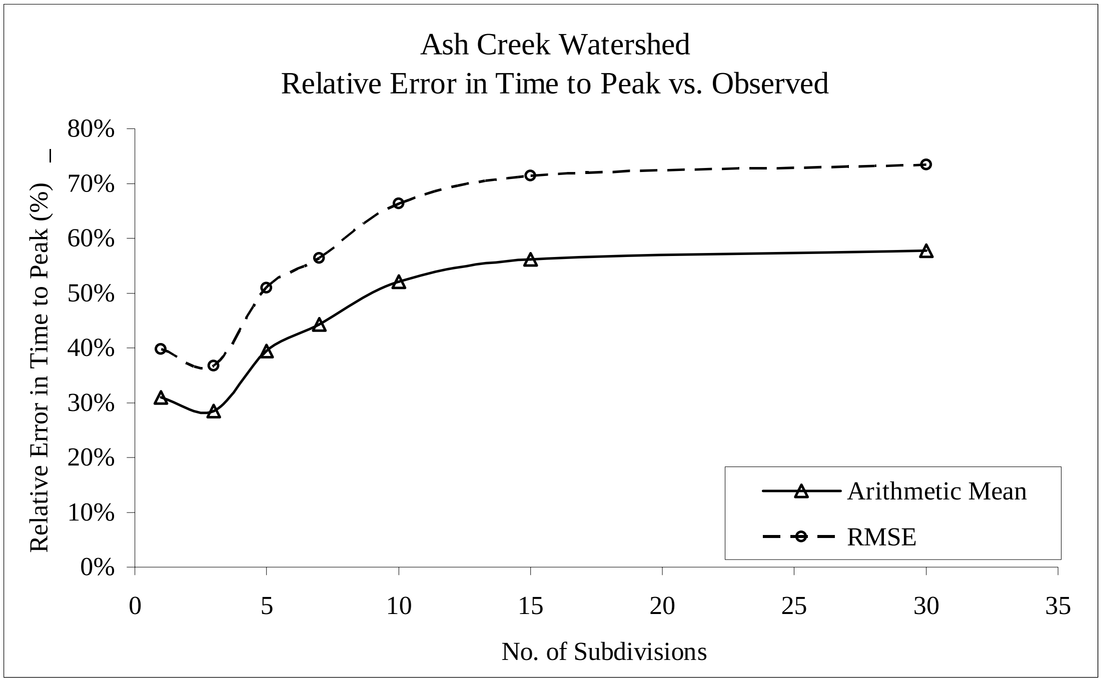

Figure 4.12. Relative error in time to peak – Ash Creek.

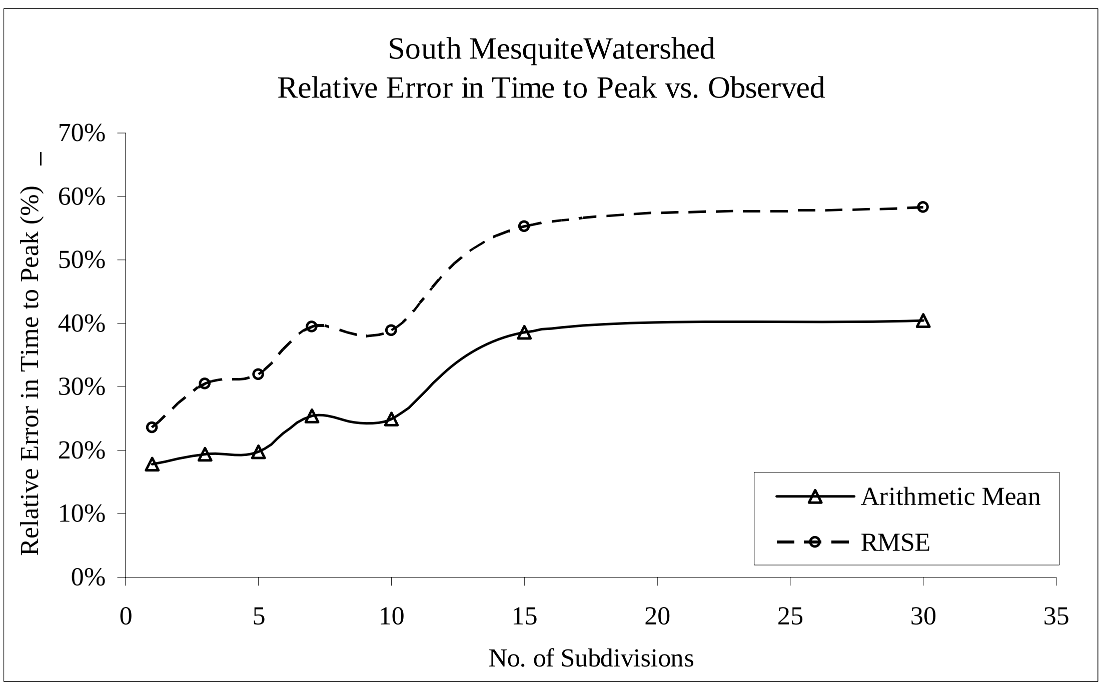

Figure 4.13. Relative error in time to peak – South Mesquite Creek.

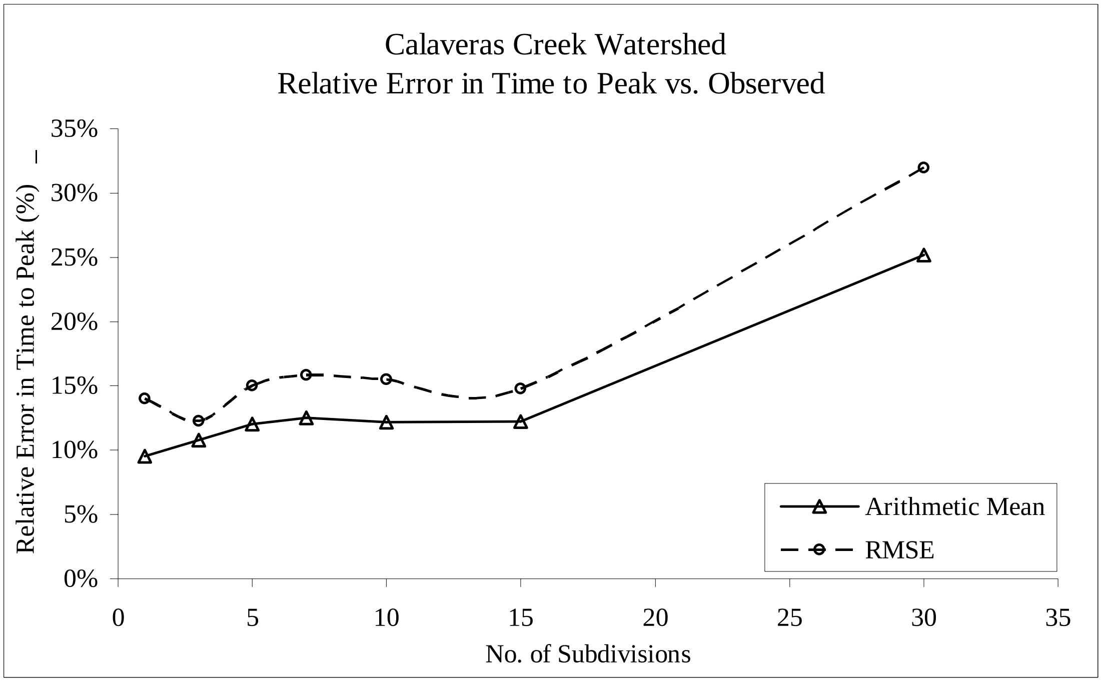

Figure 4.14. Relative error in time to peak – Calaveras Creek.

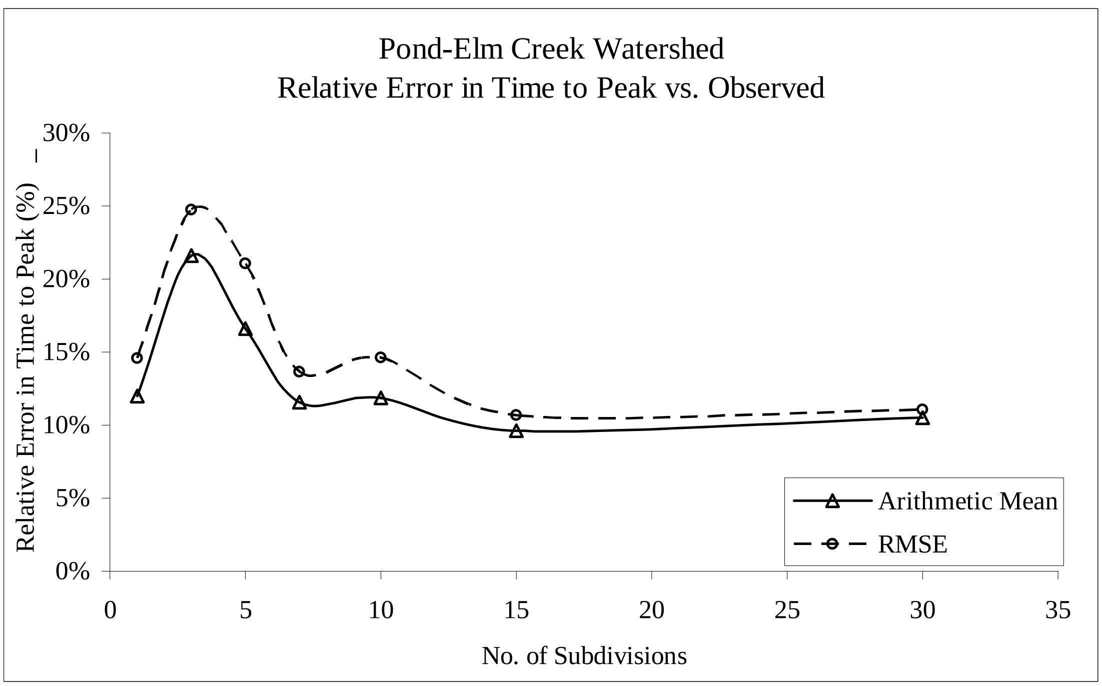

Figure 4.15. Relative error in time to peak – Pond-Elm Creek.

In Figures 4.11 to 4.15 above, it is shown that, with the exception of Pond-Elm creek, time to peak estimates tend to become less accurate as the number of subdivisions increase.

Summary

A summary of the metrics, including MIN_COUNT, are shown in Tables 4.6 to 4.8. In the last column of each table, the best performing subdivision scheme for each of the three metrics used is shown.

Table 4.6. Summary of runoff volume analysis.

Watershed	Runoff Volume	Number of Subdivisions							Best Performing Subdivision Scheme
		1	3	5	7	10	15	30	
Walnut	AM (%)	168%	165%	166%	167%	166%	163%	166%	15
	RRMSE (%)	204%	201%	202%	203%	202%	199%	202%	15
	MIN_COUNT	0	1	0	0	0	5	0	15
Ash	AM (%)	29%	29%	29%	29%	28%	28%	29%	10
	RRMSE (%)	35%	36%	35%	35%	34%	34%	35%	10
	MIN_COUNT	1	2	0	0	2	0	0	3, 10
South Mesquite	AM (%)	32%	32%	31%	31%	31%	31%	32%	15
	RRMSE (%)	39%	38%	38%	38%	38%	38%	38%	15
	MIN_COUNT	2	0	0	0	0	6	1	15
Calaveras	AM (%)	93%	93%	93%	92%	93%	93%	93%	7
	RRMSE (%)	136%	136%	137%	136%	137%	136%	135%	30
	MIN_COUNT	2	0	1	0	2	0	1	1, 10
Pond-Elm	AM (%)	49%	49%	49%	49%	49%	49%	49%	5
	RRMSE (%)	77%	77%	77%	77%	77%	77%	77%	7
	MIN_COUNT	2	0	4	0	0	0	0	5

Table 4.7. Summary of peak flow Analysis.

Watershed	Peak Flow	Number of Subdivisions							Best Performing Subdivision Scheme
		1	3	5	7	10	15	30	
Walnut	AM (%)	138%	122%	115%	111%	106%	100%	113%	15
	RRMSE (%)	173%	164%	154%	150%	146%	141%	150%	15
	MIN_COUNT	0	0	0	0	0	6	0	15
Ash	AM (%)	66%	60%	58%	58%	60%	58%	55%	30
	RRMSE (%)	67%	62%	60%	60%	62%	60%	58%	30
	MIN_COUNT	0	1	0	0	0	0	4	30
South Mesquite	AM (%)	33%	23%	21%	20%	20%	21%	23%	10
	RRMSE (%)	51%	38%	33%	31%	30%	29%	31%	15
	MIN_COUNT	2	2	1	1	0	0	3	30
Calaveras	AM (%)	43%	45%	44%	44%	44%	43%	48%	1
	RRMSE (%)	49%	50%	49%	49%	50%	50%	53%	5
	MIN_COUNT	1	0	4	0	0	0	1	5
Pond-Elm	AM (%)	33%	34%	30%	28%	27%	26%	27%	15
	RRMSE (%)	50%	49%	48%	46%	40%	42%	38%	30
	MIN_COUNT	1	0	1	1	0	1	2	30

Table 4.8. Summary of time to peak analysis.

Watershed	Time to Peak	Number of Subdivisions							Best Performing Subdivision Scheme
		1	3	5	7	10	15	30	
Walnut	AM (%)	63%	67%	143%	147%	154%	157%	150%	1
	RRMSE (%)	76%	81%	182%	189%	197%	201%	191%	1
	MIN_COUNT	4	0	0	1	1	0	0	1
Ash	AM (%)	31%	28%	39%	44%	52%	56%	58%	3
	RRMSE (%)	40%	37%	51%	56%	66%	71%	73%	3
	MIN_COUNT	1	4	0	0	0	0	0	3
South Mesquite	AM (%)	18%	19%	20%	25%	25%	39%	40%	1
	RRMSE (%)	24%	30%	32%	40%	39%	55%	58%	1
	MIN_COUNT	4	1	3	0	1	0	0	1
Calaveras	AM (%)	10%	11%	12%	13%	12%	12%	25%	1
	RRMSE (%)	14%	12%	15%	16%	16%	15%	32%	3
	MIN_COUNT	4	0	1	1	0	0	0	1
Pond-Elm	AM (%)	12%	22%	17%	12%	12%	10%	10%	15
	RRMSE (%)	15%	25%	21%	14%	15%	11%	11%	15
	MIN_COUNT	2	0	1	1	2	0	0	1, 10

As shown in tables 4.6 to 4.8, no single subdivision scheme consistently outperformed other schemes in all situations. General trends that can be observed from the above results include:

1. Storm runoff volume is not greatly affected by watershed subdivision when the NRCS curve number loss method is used, as applied in this research. Because the runoff volume is determined by the loss model, it is important to define the curve number values of a watershed accurately,

2. Overall, the accuracy of the peak flow calculations increased as the number of subdivisions increased, but the incremental improvement in accuracy is negligible beyond 5 to 10 subdivisions. As a guidance, subdividing the watershed beyond 10 subwatersheds as done in this research is not useful, and

3. When the Kerby-Kirpich method was used to estimate timing parameters, the lumped watershed model predicted the time to peak flow better than the other subdivision schemes.

Watershed Parameter Analysis

The third objective of this research was to determine if there is a certain percentage of a watershed that needs to be significantly different from the rest of the watershed in order to justify subdividing. For this study, “significantly different” is defined as the percentage of the watershed area that, when certain parameters are changed, moves the watershed behavior outside the 1/3 $\log_{10}$ region from the unchanged model (Cleveland 2008). In order to determine if the hydrograph results (volume, peak flow, and time to peak) moved outside the 1/3 $\log_{10}$ region from the base model, they were converted into the $\log_{10}$ format. In the following charts, the vertical lines above and below the base model represent 1/3 of a $\log_{10}$ cycle. Any area-parameter combination which falls outside of the 1/3 $\log_{10}$ cycle area would provide guidance for watershed subdivision.

First, the parameter dealing with precipitation loss (the NRCS curve number) was analyzed. The procedure involved assigning approximately 1/5, 1/3, and then 1/2 of the watershed a curve number of 100. This would be equivalent to making the assigned watersheds completely impervious. In Figures 4.17 to 4.31 below, these are represented by 1/5 CN, 1/3 CN, and 1/2 CN respectively.

Next, the basin's transformation lag time was arbitrarily decreased by 20 percent for the same 1/5, 1/3, and 1/2 areas that were analyzed for the loss method. In Figures 4.17 to 4.31 below, these are represented by 1/5 lag, 1/3 lag, and 1/2 lag. Similarly, the routing lag time was decreased by 20 percent for approximately 1/5, 1/3, and 1/2 of the watershed area. These changes are represented by 1/5 Rout, 1/3 Rout, and 1/2 Rout in Figures 4.17 to 4.31.

Decreasing the watershed and sub-watershed response time was one way to measure the affect of slope on the runoff hydrograph. For example, a channel that has a length of 1000 feet and a slope of 2 percent would have a travel time of approximately 7.18 minutes using the Kirpich equation (Equation 2.6). If this travel time was decreased by 20 percent to 5.74 minutes, and the length of 1000 feet remained unchanged, then the slope needed would be 3.6 percent. This change in slope from 2 to 3.6 percent is almost an 80 percent increase. Figure 4.16 was created by decreasing the travel time by varying percentages and determining the corresponding percent increase in slope.

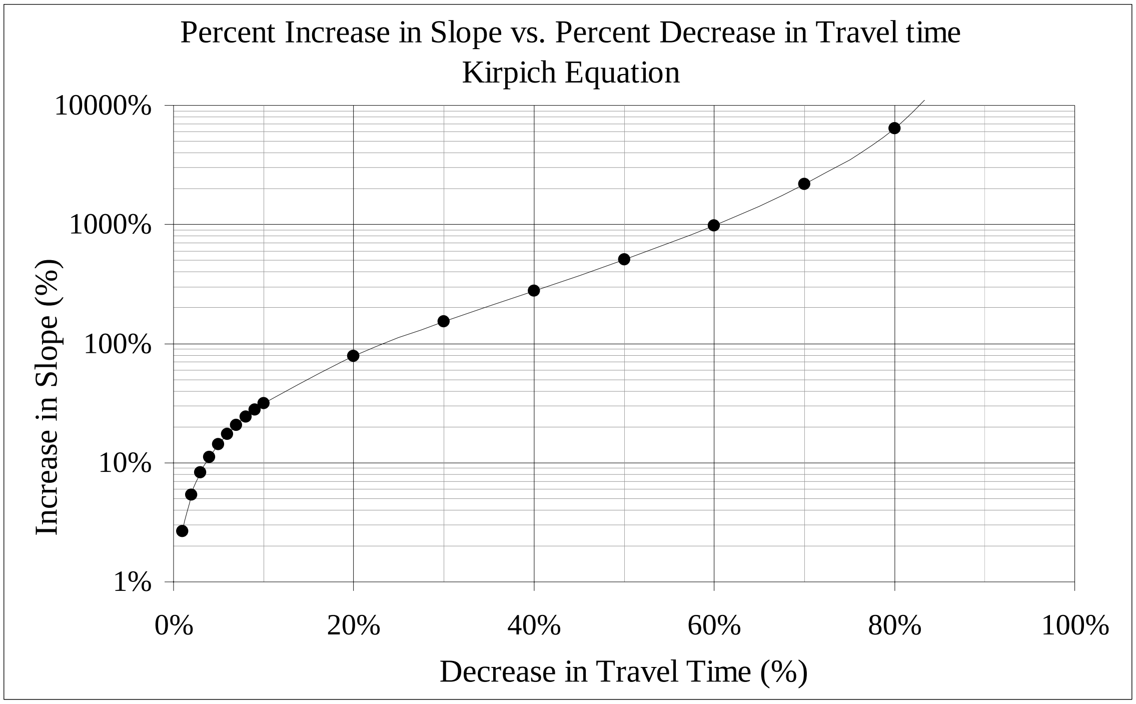

Figure 4.16. Increase in Slope vs. Decrease in Travel Time (%) – Kirpich Equation

Runoff Volume

In Figures 4.17 to 4.21 below, the results of changing the curve number, watershed lag time, and channel routing time on runoff volume for each subdivision scheme are shown.

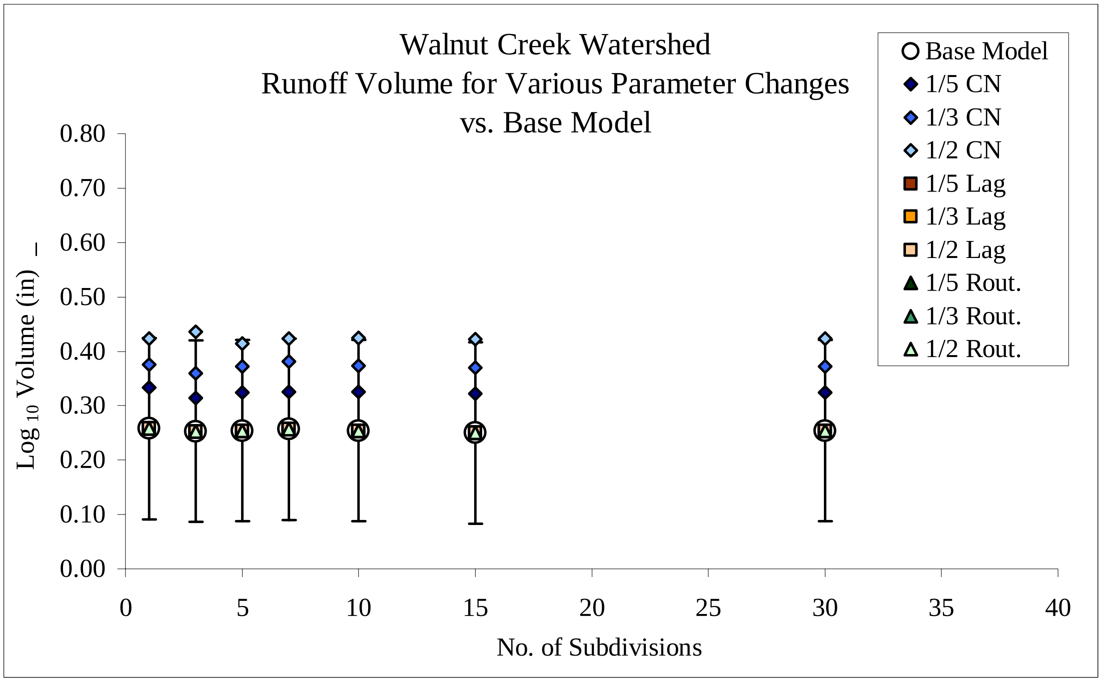

Figure 4.17. Effect of parameter changes on runoff volume – Walnut Creek.

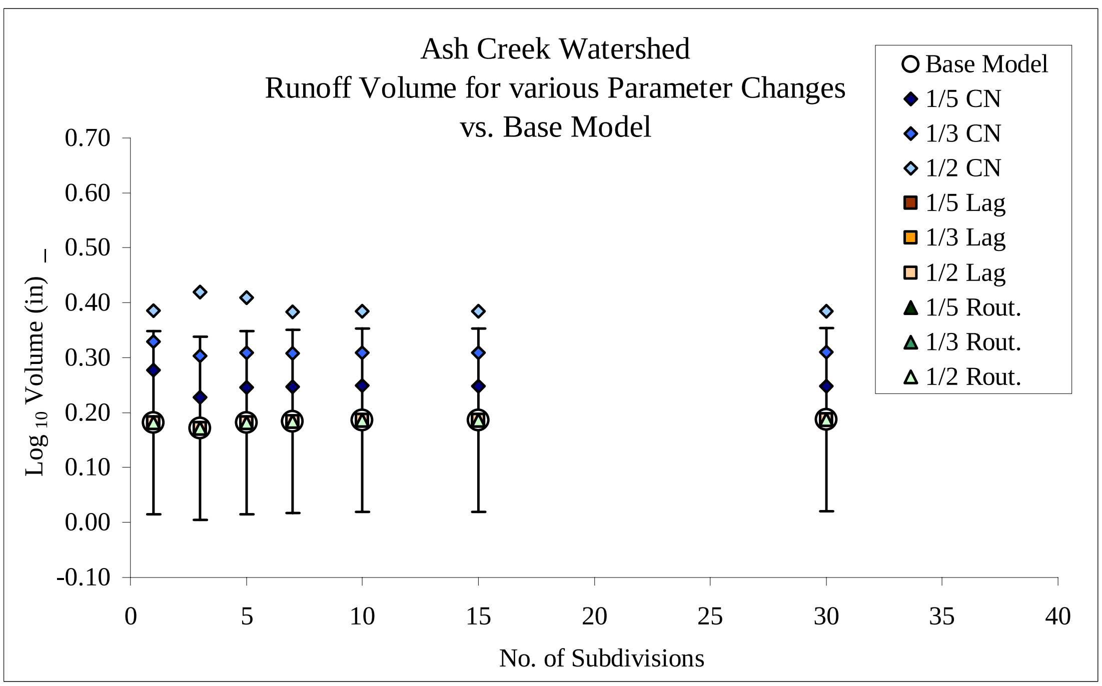

Figure 4.18. Effect of parameter changes on runoff volume – Ash Creek.

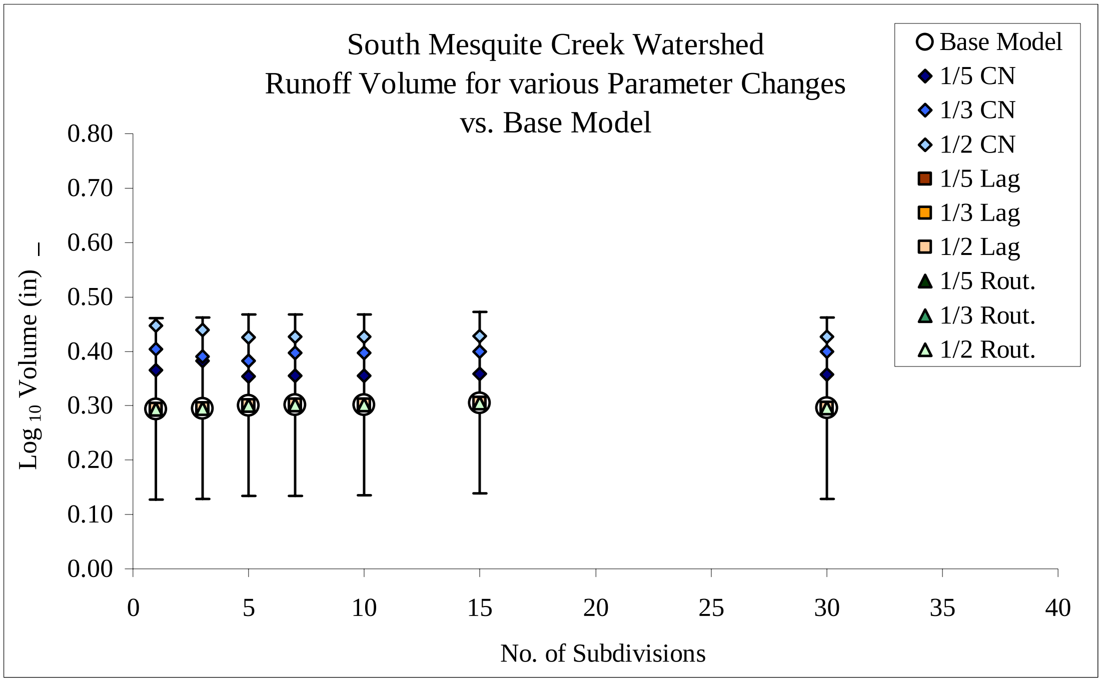

Figure 4.19. Effect of parameter changes on runoff volume – South Mesquite Creek.

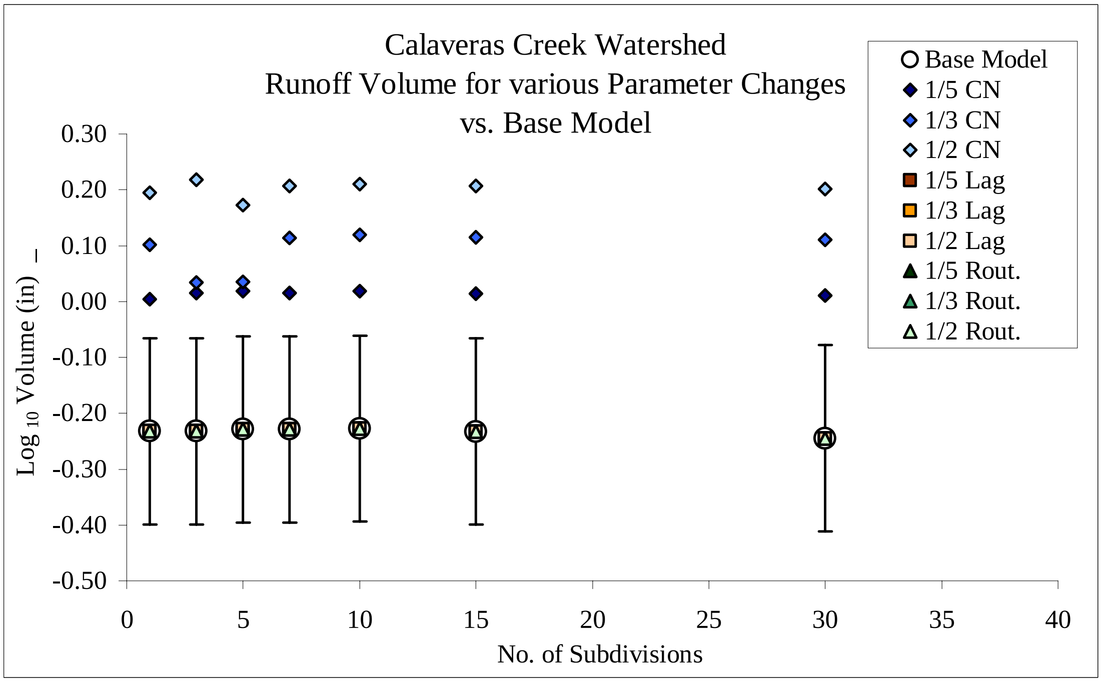

Figure 4.20. Effect of parameter changes on runoff volume – Calaveras Creek.

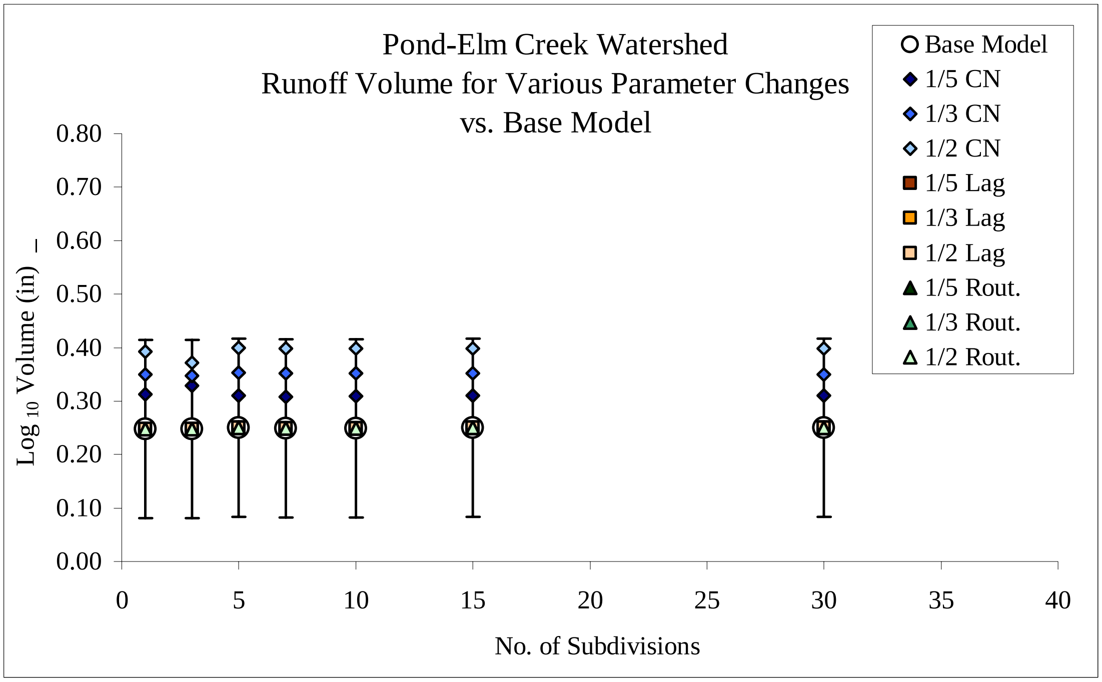

Figure 4.21. Effect of parameter changes on runoff volume – Pond-Elm Creek.

In the figures above, it is shown that the changes in subbasin lag time and channel routing time have relatively little affect on runoff volume. According to USACE (2000), HEC-HMS computes runoff volume by computing the volume of water that is intercepted, infiltrated, stored, evaporated, or transpired and subtracts it from the precipitation. Within HEC-HMS, these components are described using curve numbers.

In regards to watershed subdivision guidance, a few general conclusions can be deduced from the charts above. The first and most obvious conclusion is that timing parameters, and thus changes in watershed slope, do not affect the volume of runoff. A second conclusion is that the larger the difference is between curve number values in different areas, the greater the affect on runoff volume. For example, in examining the charts for South Mesquite and Pond-Elm watersheds, making even 1/2 of the watershed totally impervious did not move the volume outside of the 1/3 $\log_{10}$ cycle. Before any changes were made to the curve numbers, both of these watersheds had an average CN value of 86. For the Calaveras watershed, all three of the curve number area-parameter combinations moved outside the 1/3 $\log_{10}$ cycle. The Calaveras watershed had an average CN value of 77.

Peak Flow

In Figures 4.22 to 4.26 below, the results of changing the curve number, watershed lag time, and channel routing time on peak flow for each subdivision scheme are shown.

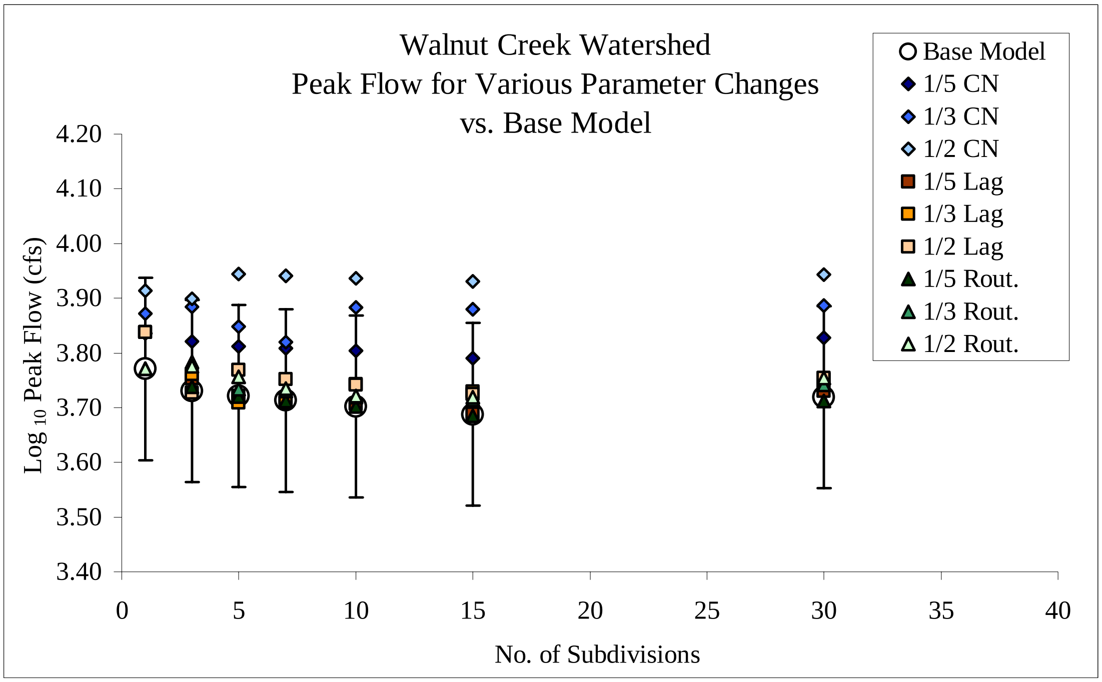

Figure 4.22. Effect of parameter changes on peak flow – Walnut Creek.

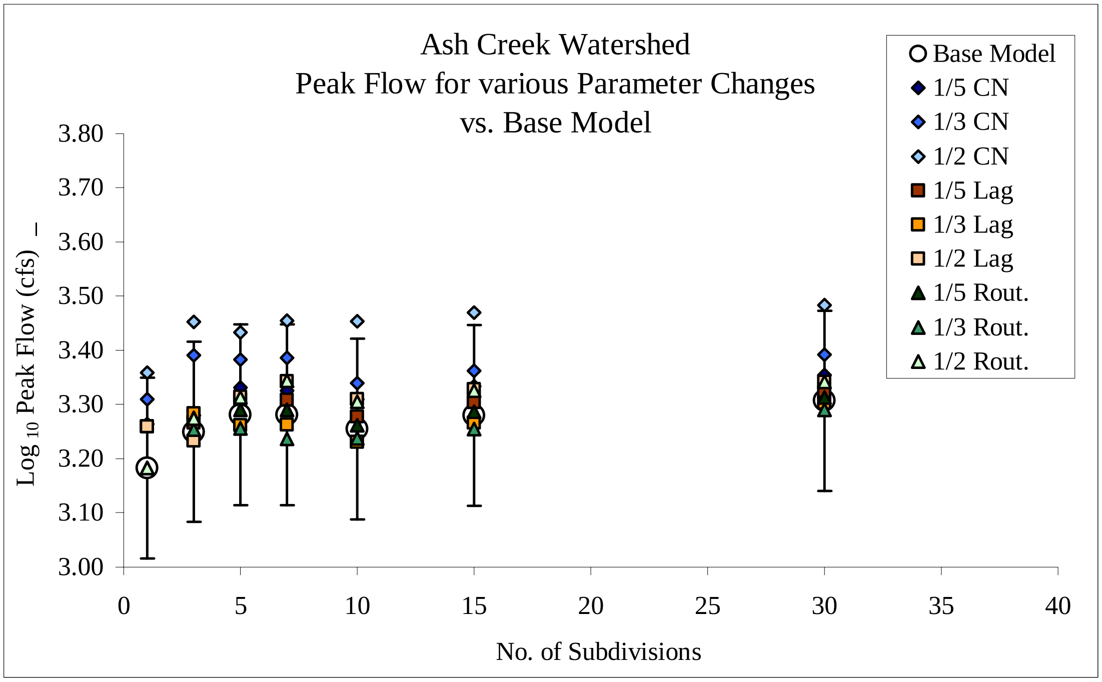

Figure 4.23. Effect of parameter changes on peak flow –Ash Creek.

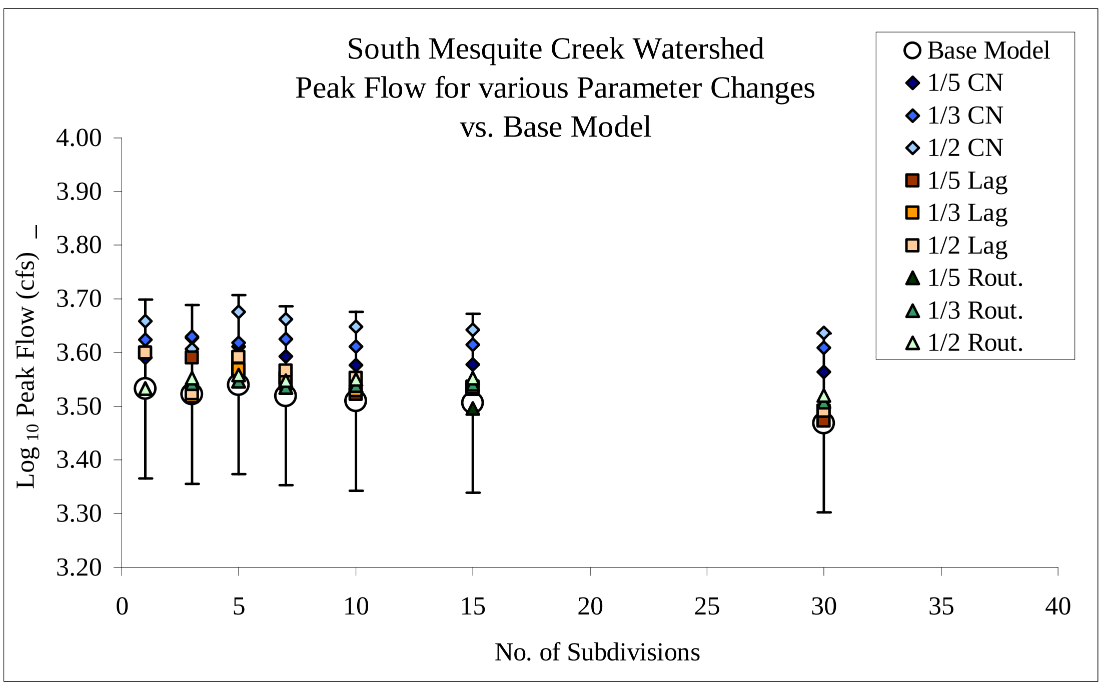

Figure 4.24. Effect of parameter changes on peak flow –South Mesquite Creek.

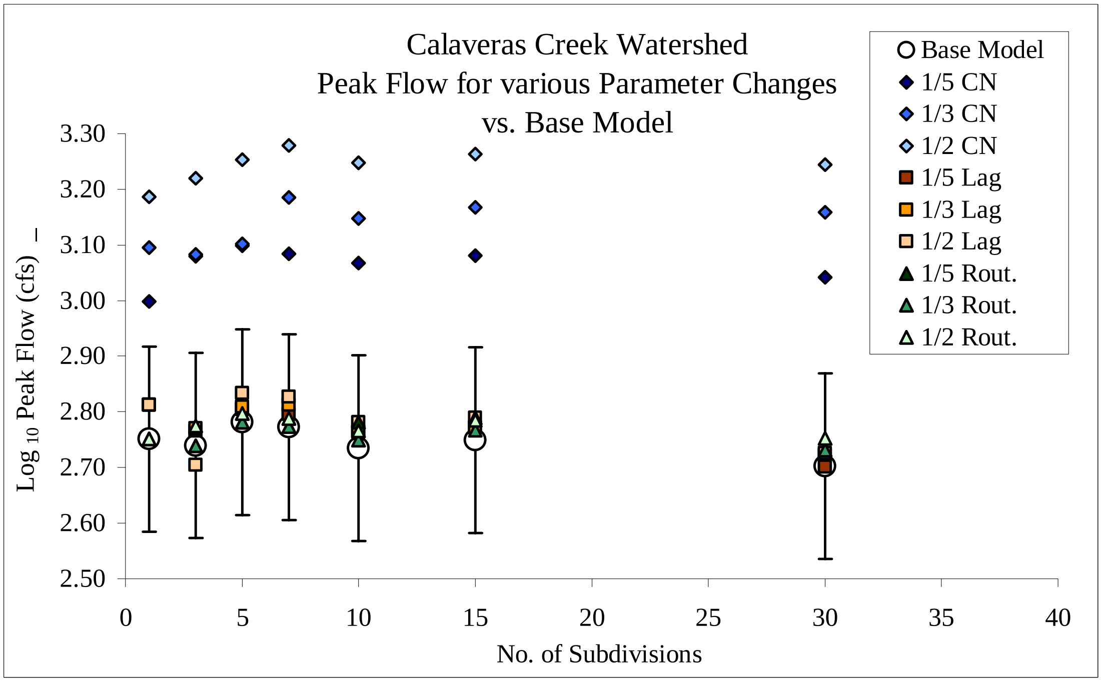

Figure 4.25. Effect of parameter changes on peak flow –Calaveras Creek.

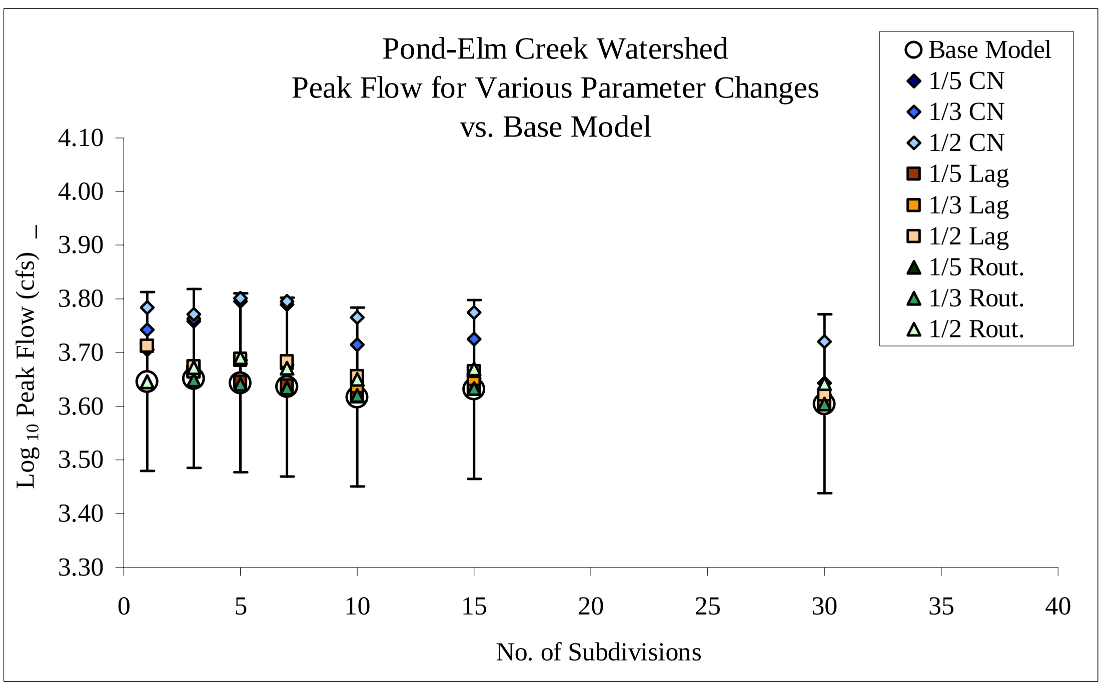

Figure 4.26. Effect of parameter changes on peak flow –Pond-Elm Creek.

In regards to watershed subdivision guidance, a few general conclusions can be deduced from the charts above. Of the three parameters changed in the models, changes to the curve number have the largest impact on peak flow. The same comment made in the runoff volume section on the degree of difference between the CN values can be applied to peak flow as well. A second conclusion that can be made is that the timing parameters, while having more of an impact than for the runoff volume, still do not move the results outside the 1/3 $\log_{10}$ cycle. Therefore, areas within a watershed that have varying curve number values would have a higher priority as locations to subdivide than areas with varying slopes.

Time to Peak

In Figures 4.27 to 4.31 below, the results of changing the curve number, watershed lag time, and channel routing time on time to peak for each subdivision scheme are shown.

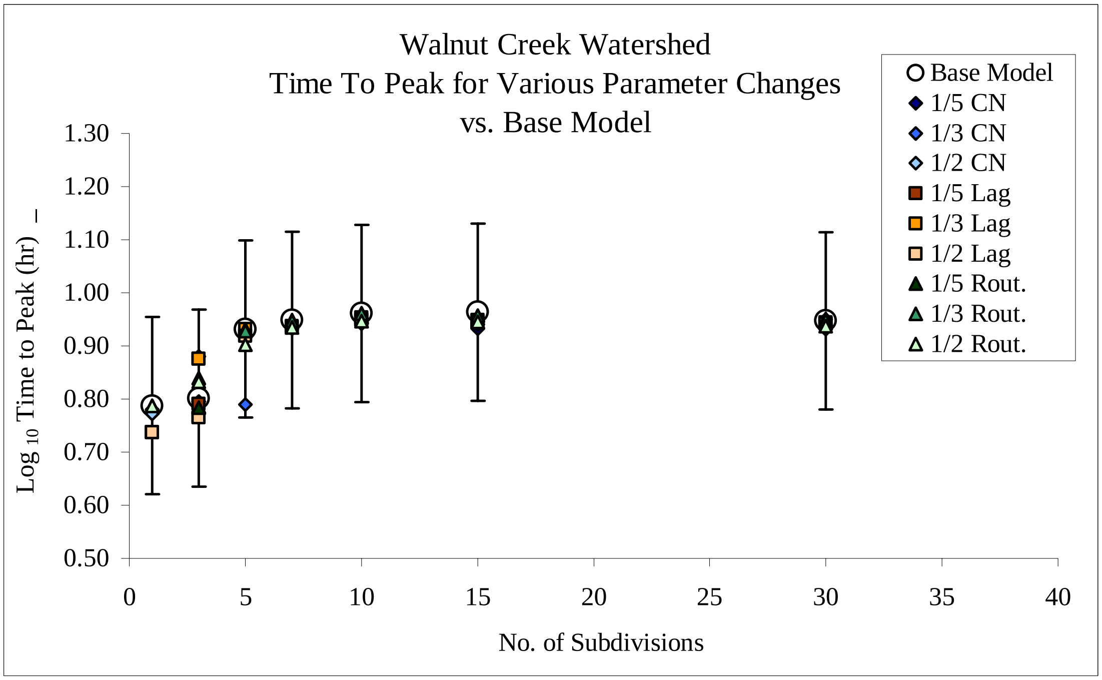

Figure 4.27. Effect of parameter changes on time to peak –Walnut Creek.

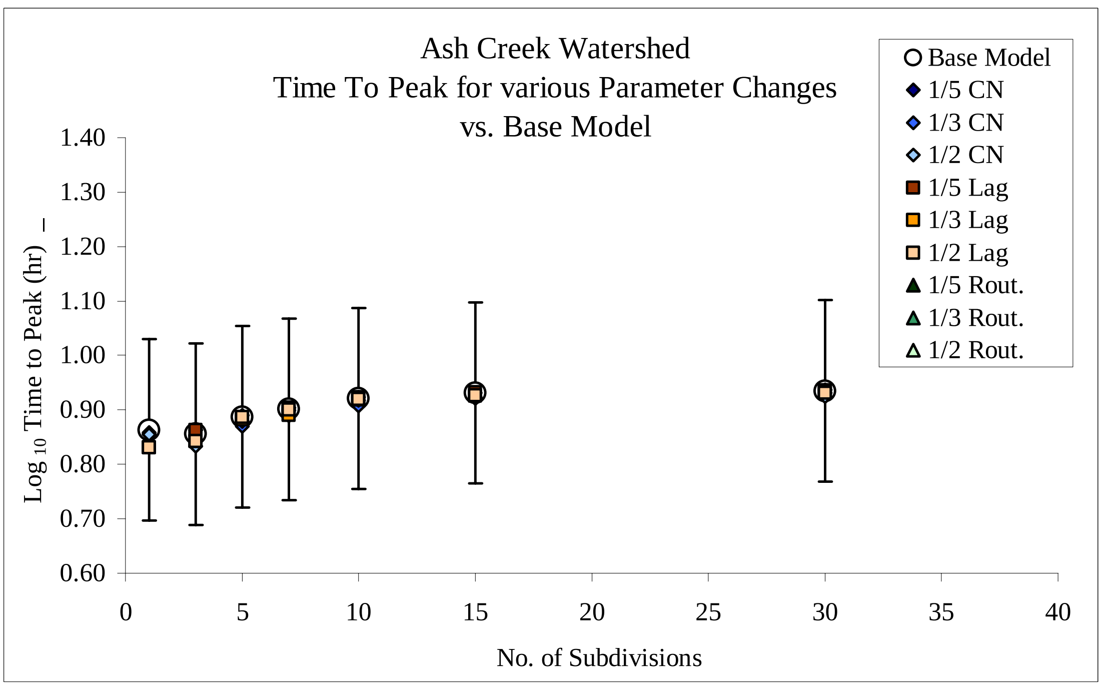

Figure 4.28. Effect of parameter changes on time to peak –Ash Creek.

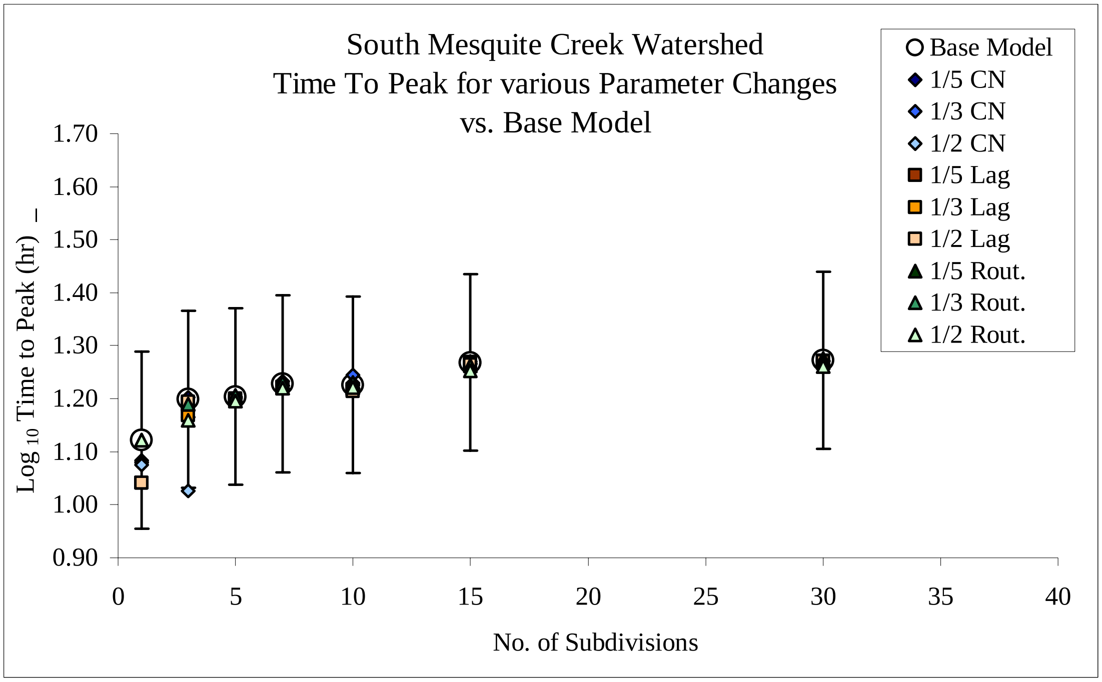

Figure 4.29. Effect of parameter changes on time to peak –South Mesquite Creek.

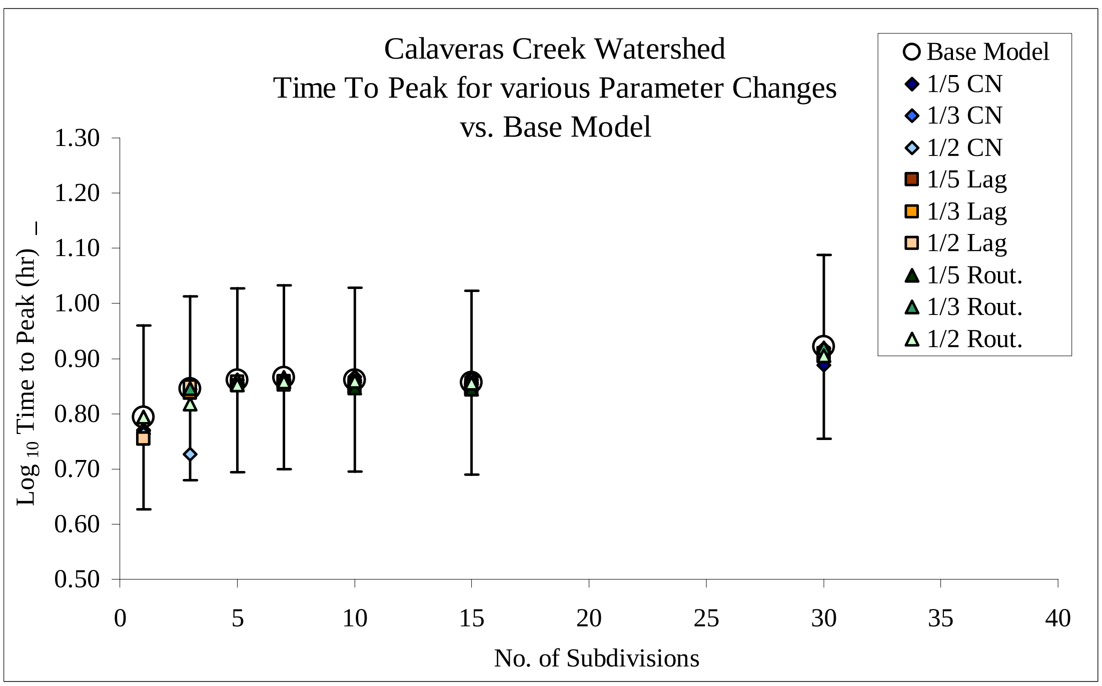

Figure 4.30. Effect of parameter changes on time to peak –Calaveras Creek.

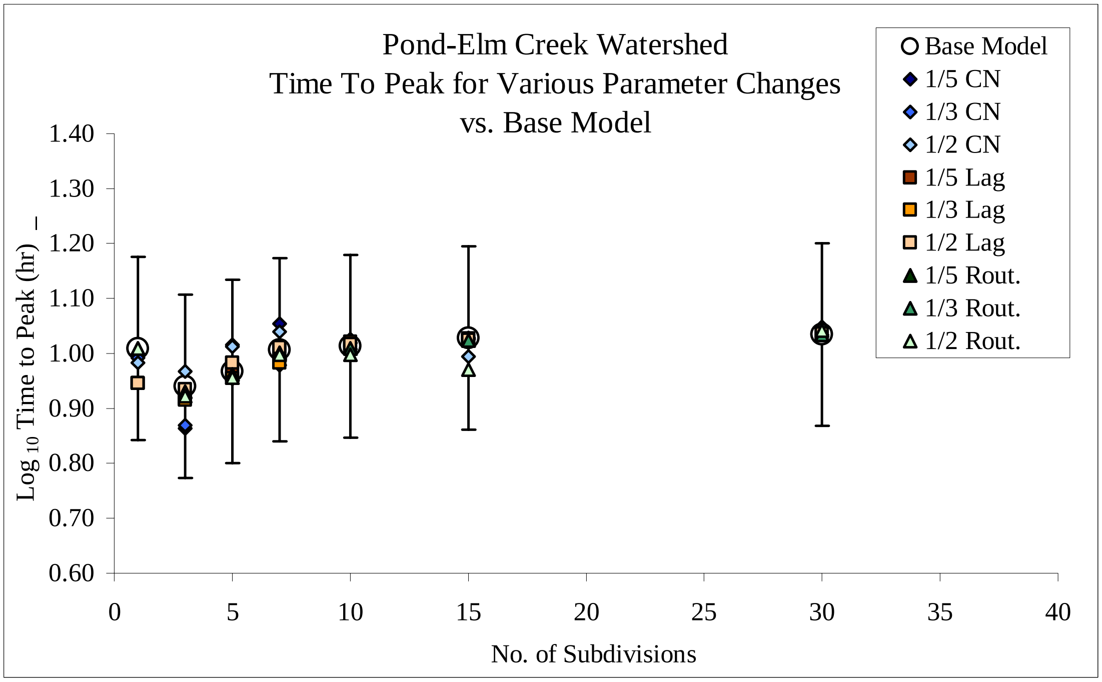

Figure 4.31. Effect of parameter changes on time to peak –Pond-Elm Creek.

In the charts above, it is shown that the various area-parameter changes, except in one instance, do not move the time to peak results outside of the 1/3 $\log_{10}$ cycle. It is important to note that when the watershed parameters were changed, in most instances they were changed at the upstream end of the watersheds. Changing the parameters at the upstream end of the watershed forced the storm runoff, especially in the higher numbered subdivision schemes, to travel through the channel routing system before reaching the outlet. This time in the routing system appears to have a dissipating effect on the results. The area-parameter points for the 30 sub-watershed scheme are generally closer to the base model and less dispersed than the other subdivision schemes.

In the smaller numbered subdivision schemes, especially 3 and 5 sub-watersheds, there are area-parameter points that are noticeably different from the rest. This may be attributed to the fact that the fewer sub-watersheds a model has the fewer basin components the runoff has to go through in order to reach the watershed outlet. In other words, the closer to the outlet the parameter changes occurred, the greater the impact to the timing results. In some cases the sub-watershed was connected directly to the outlet. For the most part these points are still inside the 1/3 $\log_{10}$ cycle, but they are close enough to this boundary that further study is needed in order to provide definitive subdivision guidance.

Summary

The third objective of this research was to determine if there is a certain percentage of a watershed that needs to be significantly different from the rest of the watershed in order to justify subdividing. In Figures 4.17 to 4.31 above, it is shown that there is not a definitive answer to this objective. In the situation at Calaveras creek, where the difference between the original curve number value and the changed curve number value was 23, 1/5 of the watershed was easily large enough to move the results outside of the 1/3 $\log_{10}$ cycle for runoff volume and peak flow. In the situation from South Mesquite creek, where the difference between curve number values was only 14, 1/2 of the watershed area was not large enough to move the results outside of the 1/3 $\log_{10}$ cycle. When the heuristic subdivision method is used, it is difficult to develop a set of guidelines that would encompass all situations. There are some general conclusions that can be derived from the figures:

1. Decreasing the basin lag and channel routing timing parameters by 20 percent, a surrogate for a slope increase of approximately 80 percent, does not significantly affect any of the three properties of the runoff hydrograph,

2. Of the three watershed parameters analyzed during this analysis, the curve number has the greatest affect on the runoff hydrograph. Because of this, changes in land use, land cover, or soil properties should be given priority over changes in slope when deciding where to subdivide a watershed.

3. The larger the difference is between curve number values across the watershed, the greater the impact subdividing has on the runoff hydrograph. Conversely, the more uniform the curve number values are across a watershed, the less of an impact subdividing has on the runoff hydrograph. For example, there was a dramatic difference between the curve number changing from a value of 77 to 100 (Calaveras Creek) than from changing from 86 to 100 (South Mesquite).

CHAPTER V

CONCLUSIONS

One of the main goals of TxDOT Project 0-5822, "Subdivision of Watersheds for Modeling," is to develop a set of defensible guidelines for watershed subdivision in order to improve the design process within TxDOT. In this thesis, five Texas watersheds with drainage areas between 7.1 and 46.1 square miles were analyzed. The computer program ArcGIS was used to develop watershed hydrologic parameters. The modeling software HEC-HMS was used to model the five watersheds. Runoff hydrographs for each watershed were computed using a heuristic subdivision approach for the lumped, 3, 5, 7, 10, 15, and 30 sub-watershed schemes.

Three components of the runoff hydrograph (runoff volume, peak flow, and time to peak) were analyzed and compared to determine the impact of watershed subdivision on modeling results. The models were intentionally left uncalibrated with respect to observed runoff behaviors. In this sense the watershed models represent the judgment of the hydrologic analyst, as would be applied at ungaged sites. It was found that no single subdivision scheme (3, 5, 7, etc.) consistently performed better than any other for every situation. There were some general trends observed in the results:

1. Storm runoff volume is not greatly affected by watershed subdivision when the NRCS curve number loss method is used (as applied in this research). Because the runoff volume is determined by the loss model, it is important to define the curve number values of a watershed accurately,

2. Overall, the accuracy of the peak flow calculations increased as the number of subdivisions increased, but the incremental improvement in accuracy is negligible beyond 5 to 10 subdivisions. As a guidance, subdividing the watershed beyond 10 subwatersheds (as done in this research) is not useful, and
3. When the Kerby-Kirpich method was used to estimate timing parameters, the lumped watershed model predicted the time to peak flow better than the other subdivision schemes.

Therefore, the amount of watershed subdivision would seem to depend on the objectives of the modeler. If the objective is to calculate the volume of storm runoff, than the lumped model produces results with the same accuracy as the different subdivision schemes when using NRCS curve numbers. If the goal is to determine the time to peak discharge, the lumped model produced more accurate results when the Kerby-Kirpich equations were used. If the objective is to determine peak flow, which is most often the case, then subdividing the watershed seems to improve the accuracy of peak flow calculations. The accuracy improvement tended to stabilize between 5 and 10 subdivisions.

One of the objectives of this research was to determine if there is a certain percentage of a watershed that needs to be significantly different from the rest of the watershed in order to justify subdividing. Although a definitive answer could not be determined for all situations, there were some general conclusions derived from the results:

1. Decreasing the basin lag and channel routing timing parameters by 20 percent, a surrogate for a slope increase of approximately 80 percent, does not significantly affect any of the three properties of the runoff hydrograph,

2. Of the three watershed parameters analyzed during this analysis, the curve number has the greatest affect on the runoff hydrograph. Because of this, changes in land use, land cover, or soil properties should be given priority over changes in slope when deciding where to subdivide a watershed.

3. The larger the difference is between curve number values across the watershed, the greater the impact subdividing has on the runoff hydrograph. Conversely, the more uniform the curve number values are across a watershed, the less of an impact subdividing has on the runoff hydrograph. For example, there was a dramatic difference between the curve number changing from a value of 77 to 100 (Calaveras Creek) than from changing from 86 to 100 (South Mesquite).

Future study is needed to find out if the results in this study would change if a further decrease in subbasin lag and channel routing times are made. Decreasing the travel times by 40 to 50 percent would be equivalent to increasing the slope about 300 to 500 percent. Future study would also be needed to determine if changing the watershed parameters near the outlet, as opposed to the upstream end, would change the results.

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APPENDIX A

WATERSHED SUBDIVISION SCHEMES

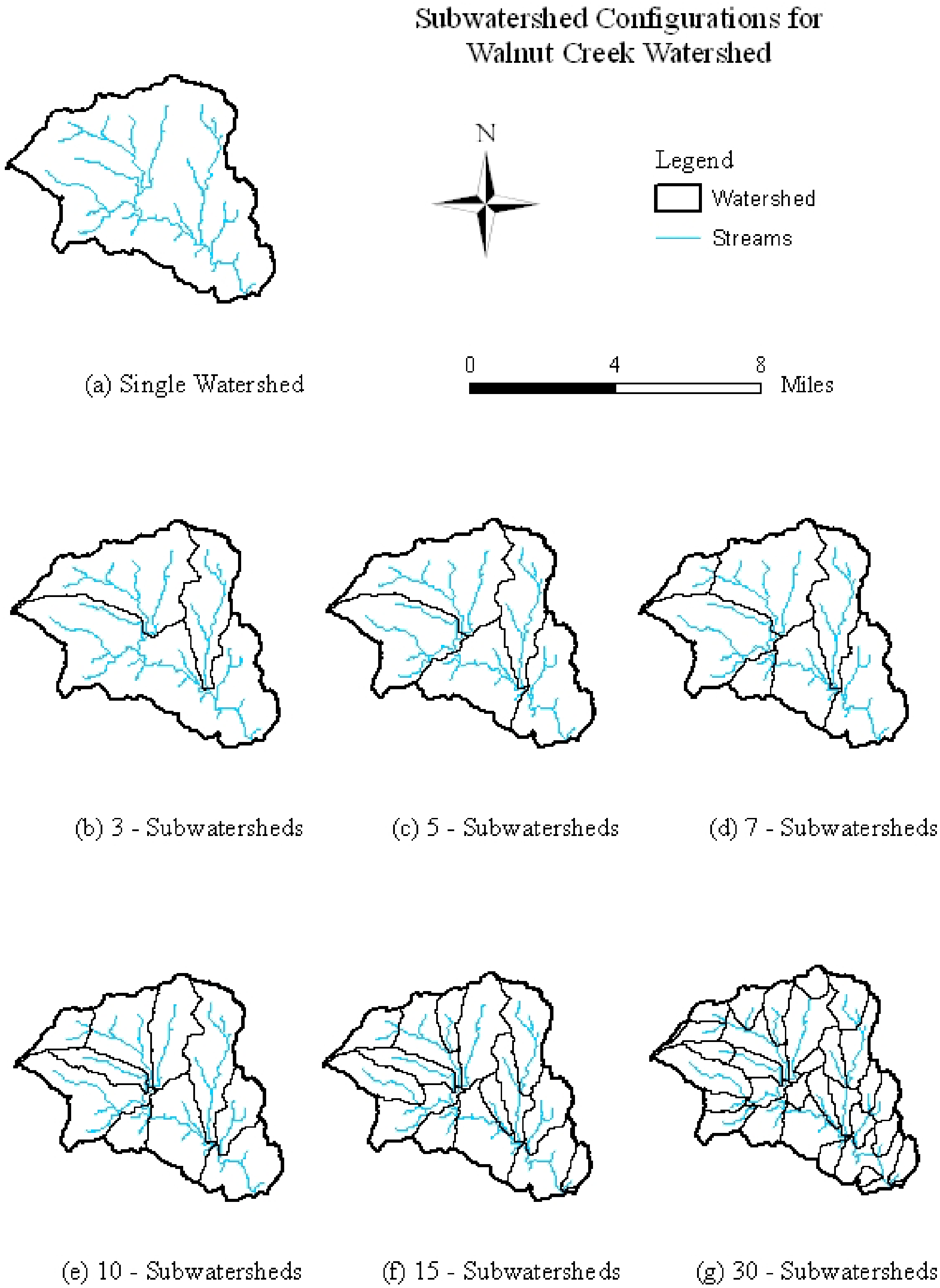

Figure A.1. Sub-watershed configuration for Walnut Creek

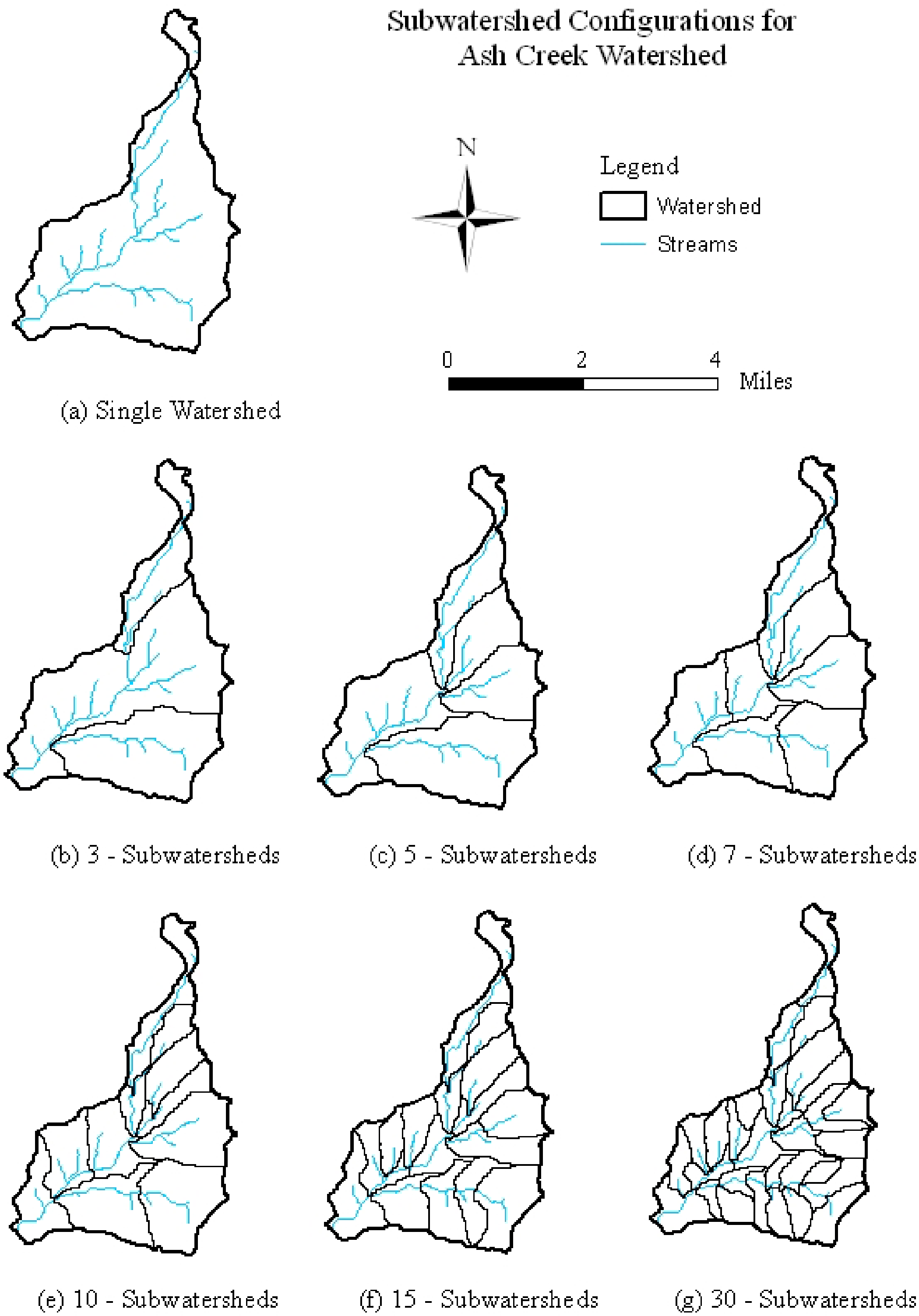

Figure A.2. Sub-watershed configuration for Ash Creek

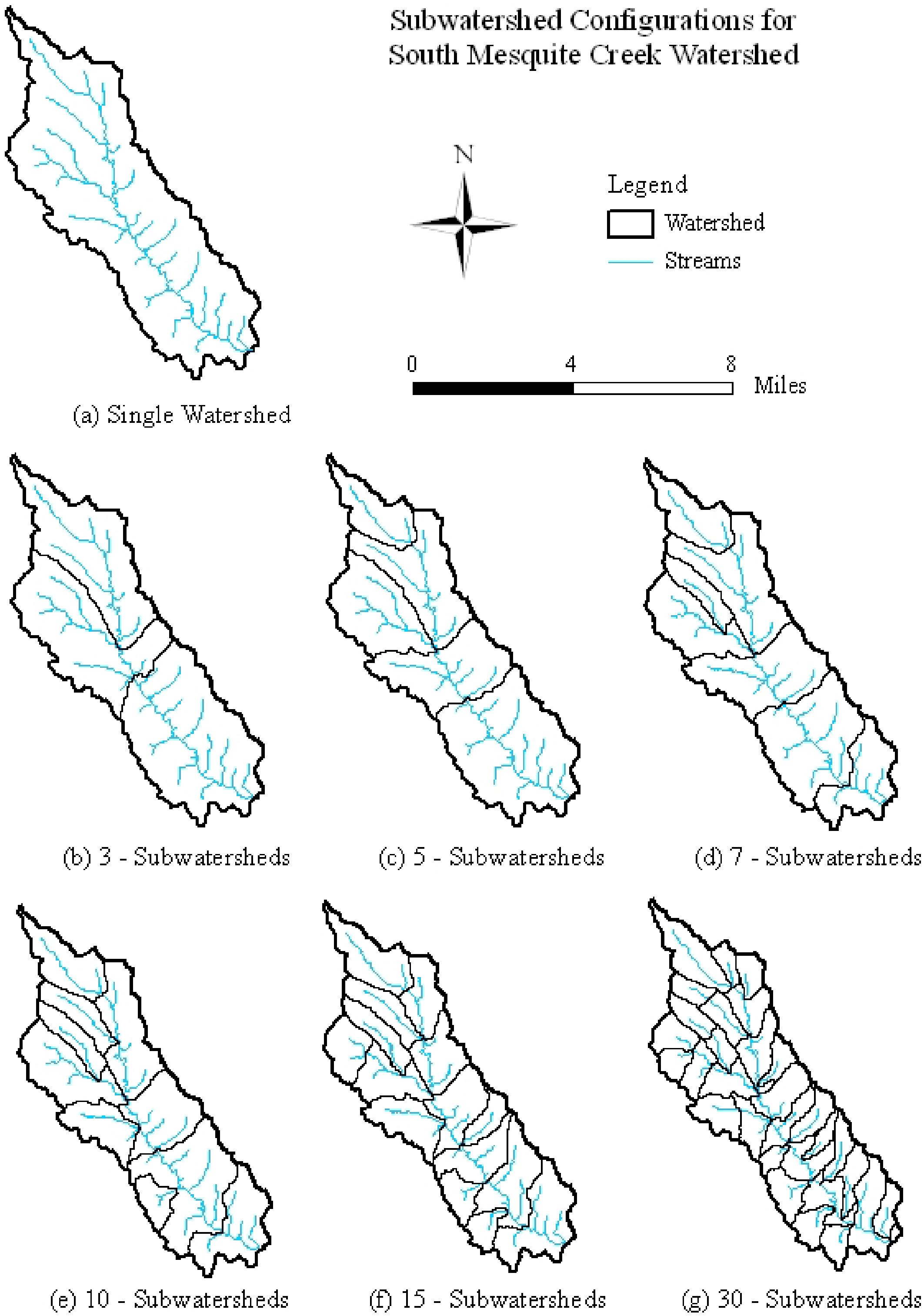

Figure A.3. Sub-watershed configuration for South Mesquite Creek

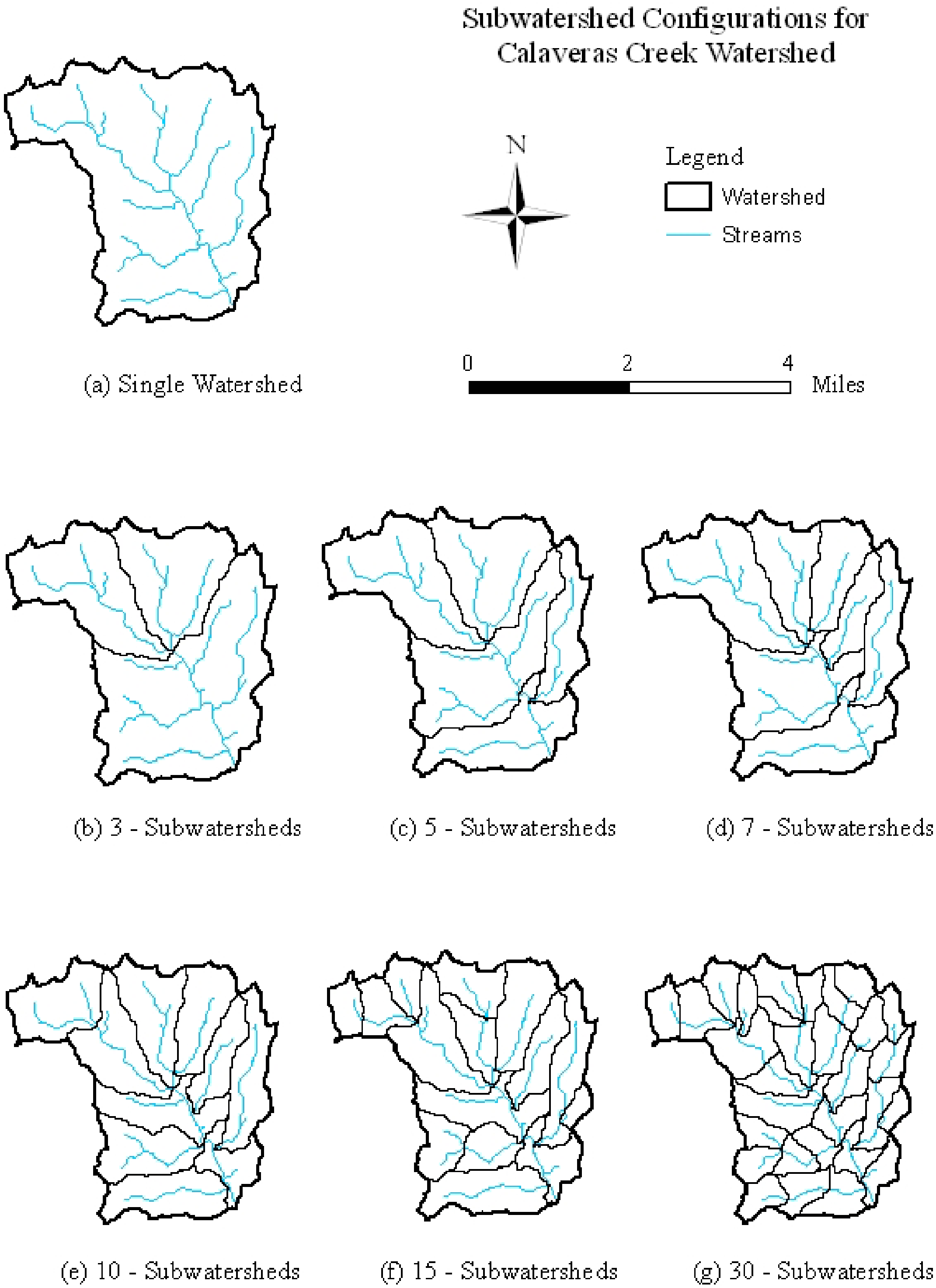

Figure A.4. Sub-watershed configuration for Calaveras Creek

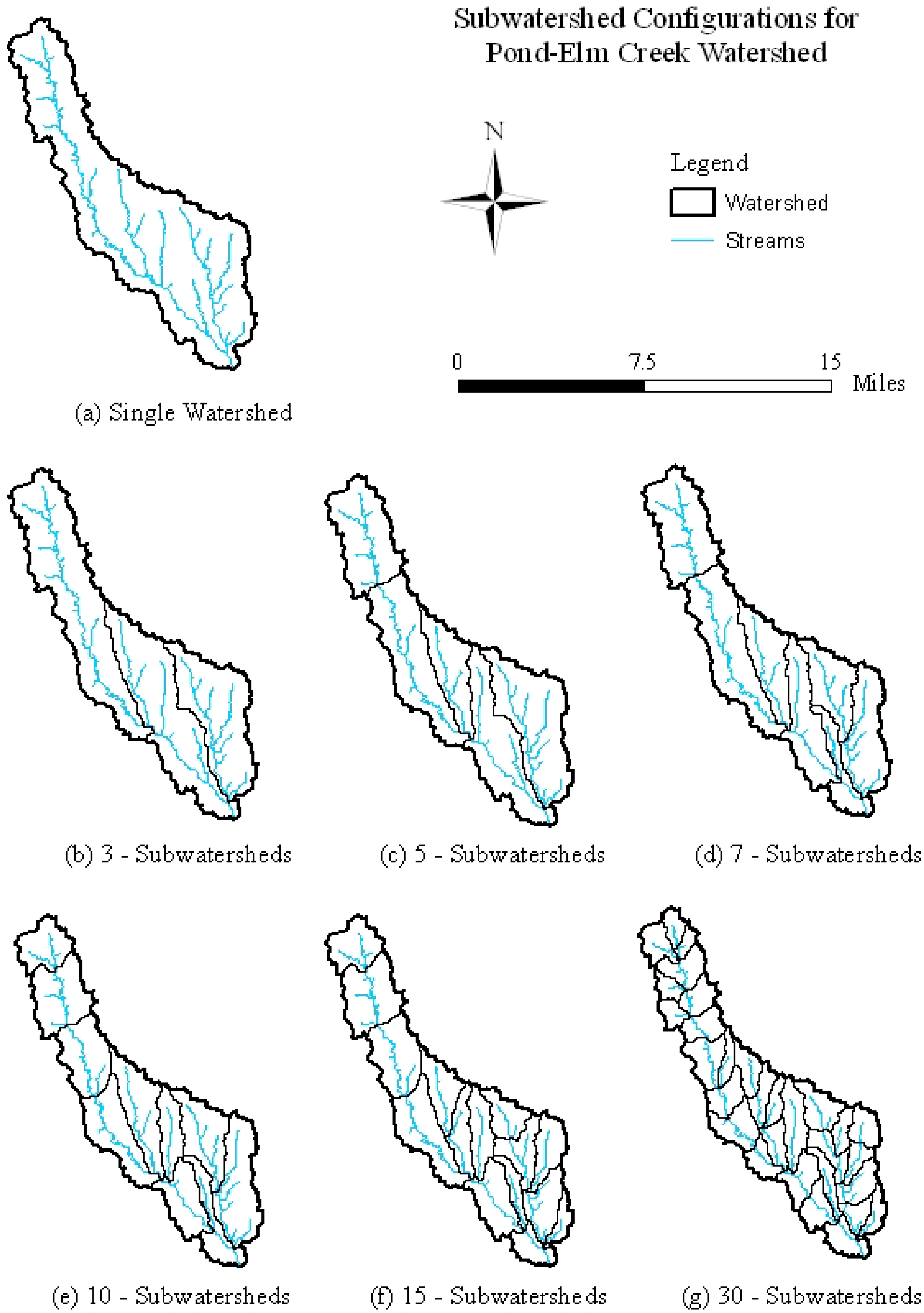

Figure A.5. Sub-watershed configuration for Pond-Elm Creek

APPENDIX B

RUNOFF HYDROGRAPHS FOR SUBDIVISION SCHEMES

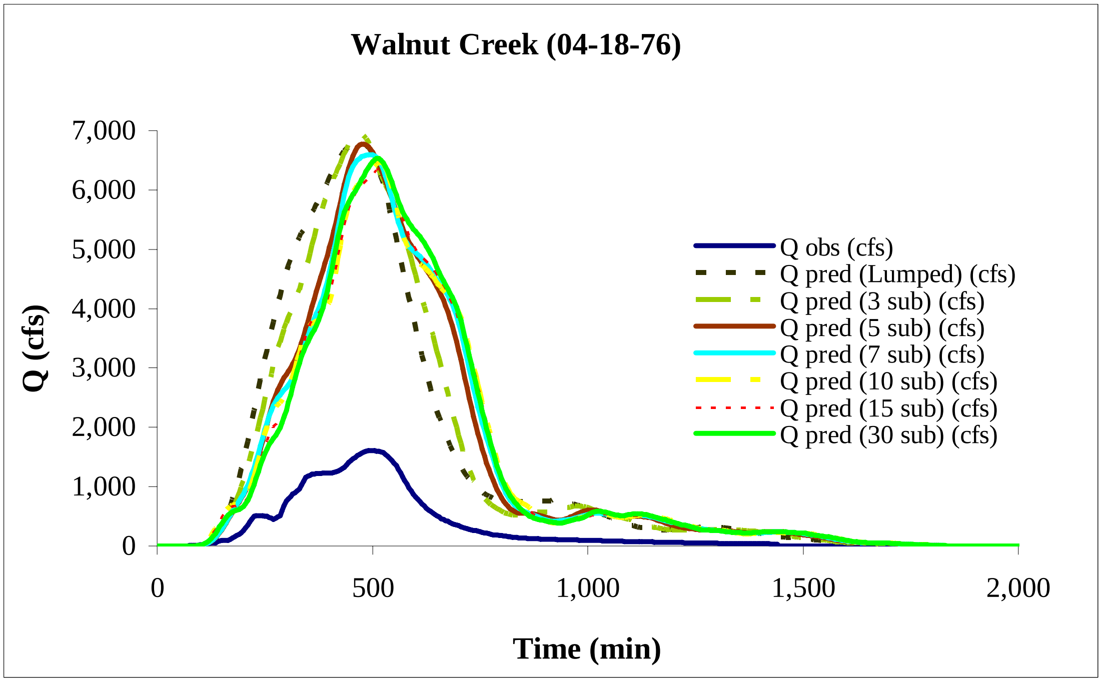

Figure B.1. Simulated and Observed Hydrographs – Walnut (04-18-76)

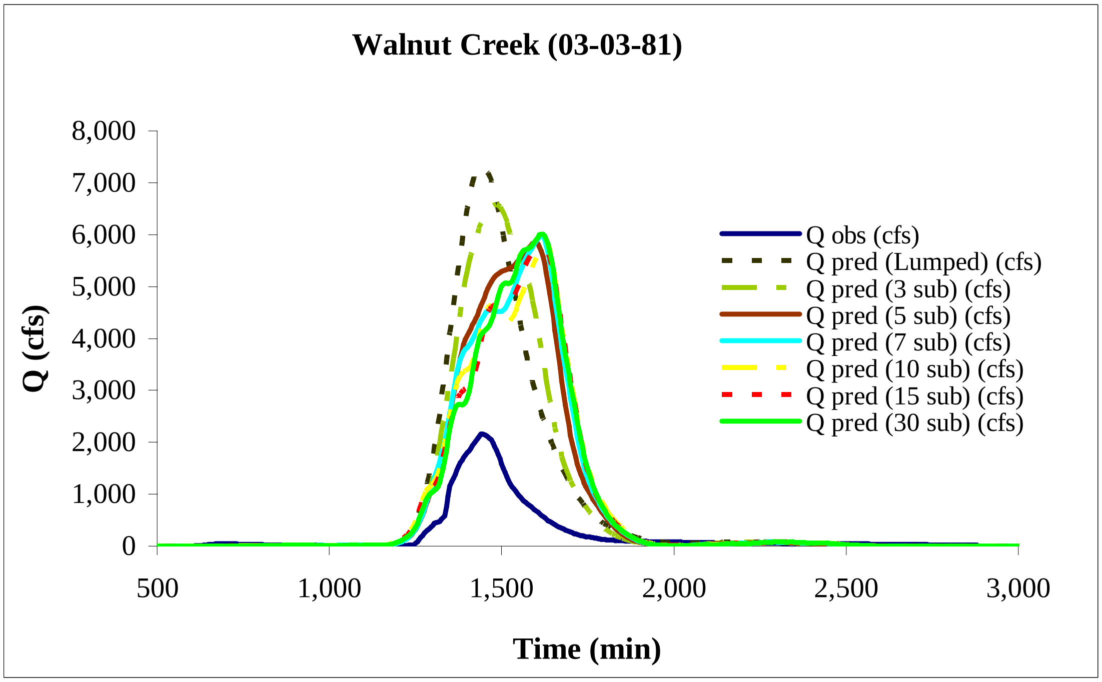

Figure B.2. Simulated and Observed Hydrographs – Walnut (03-03-81)

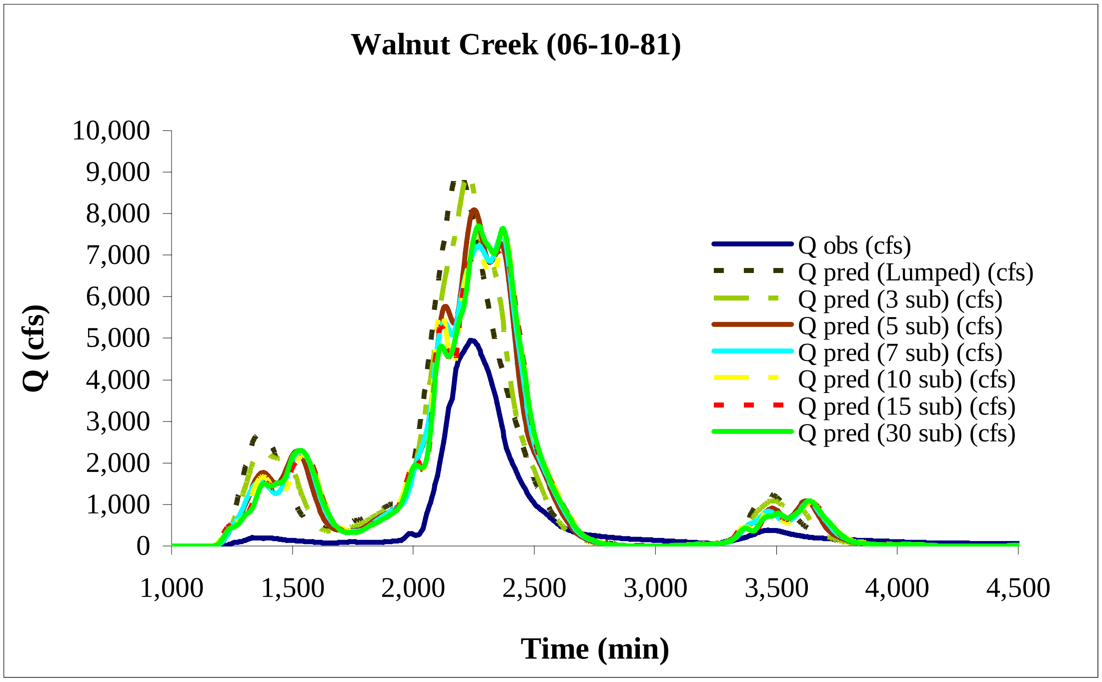

Figure B.3. Simulated and Observed Hydrographs – Walnut (06-10-81)

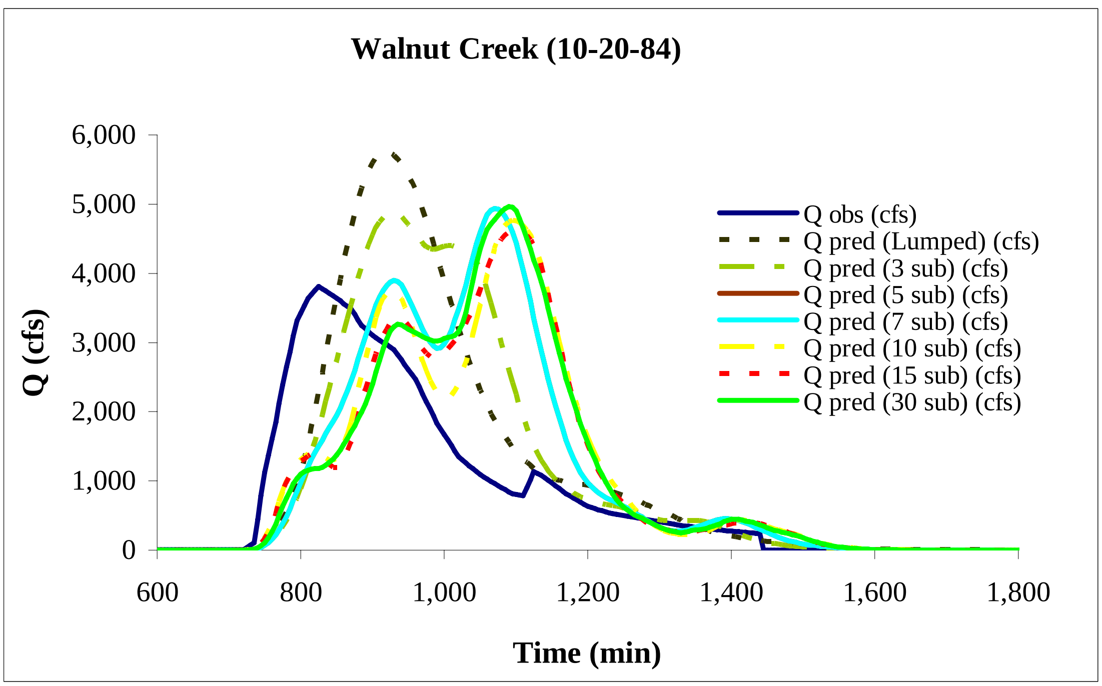

Figure B.4. Simulated and Observed Hydrographs – Walnut (10-20-84)

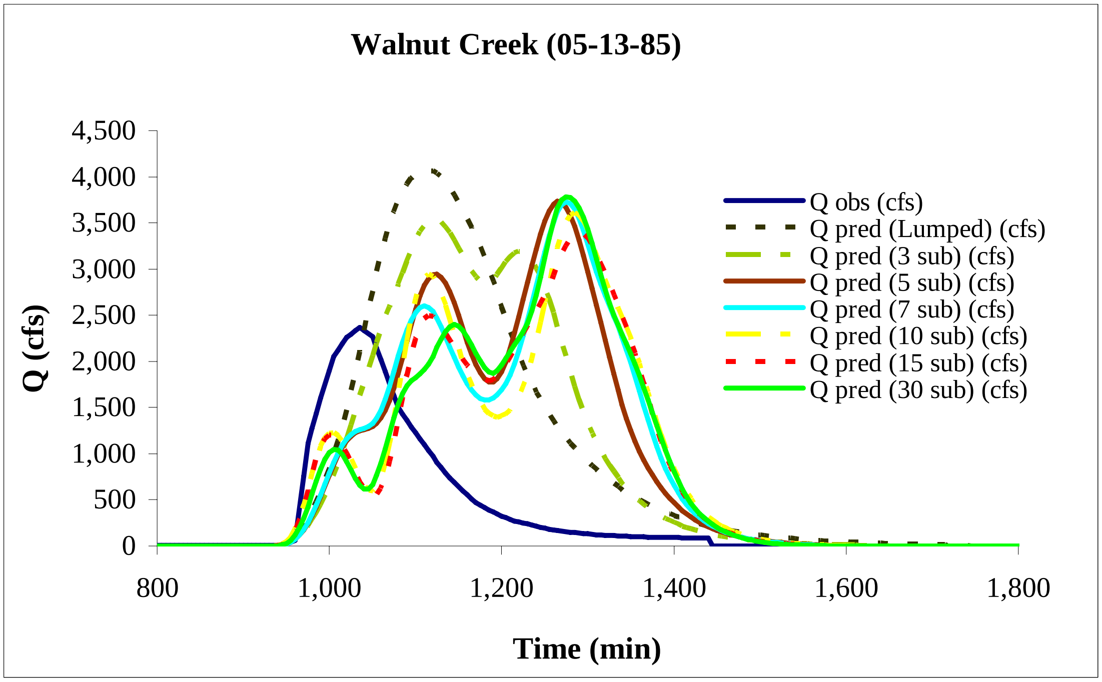

Figure B.5. Simulated and Observed Hydrographs – Walnut (05-13-85)

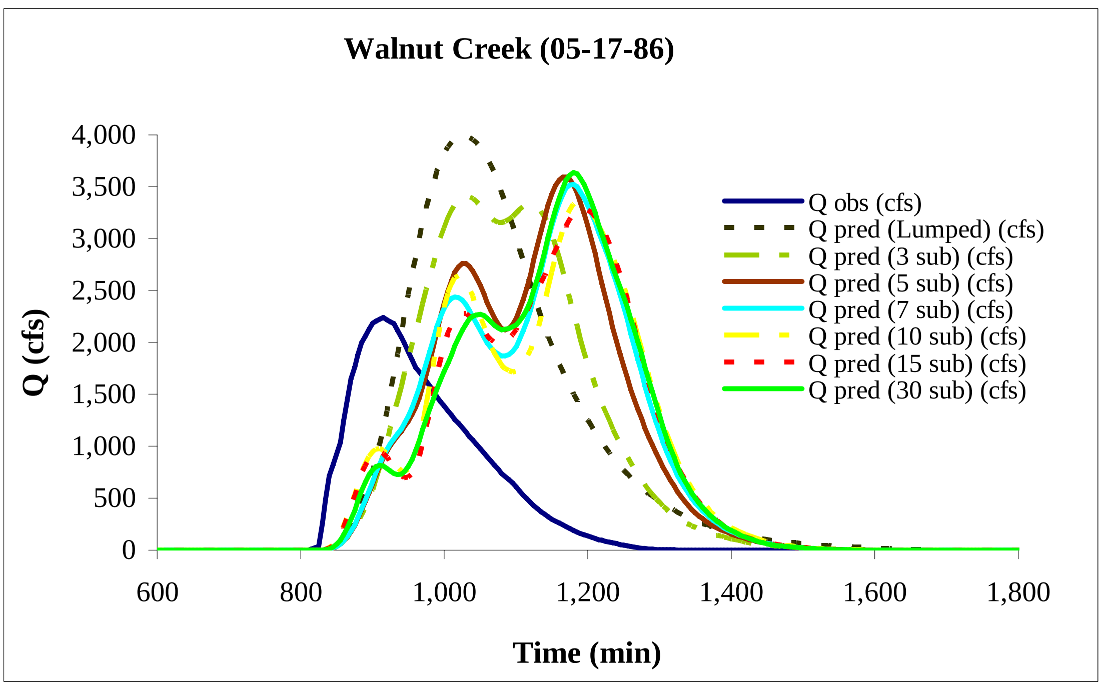

Figure B.6. Simulated and Observed Hydrographs – Walnut (05-17-86)

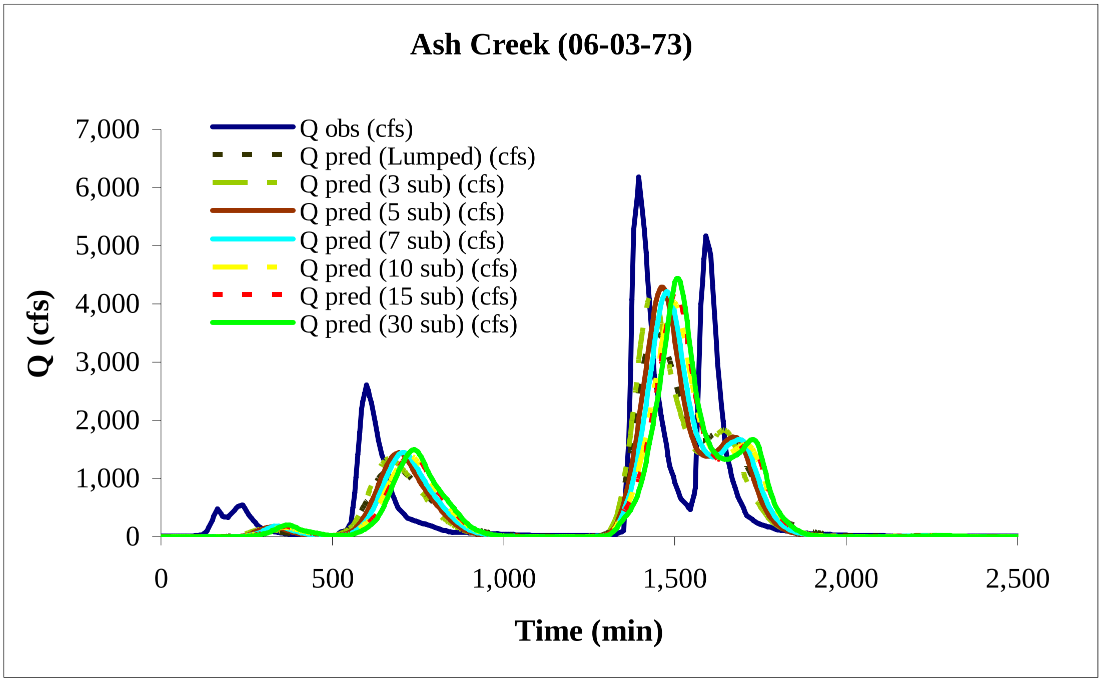

Figure B.7. Simulated and Observed Hydrographs – Ash (06-03-73)

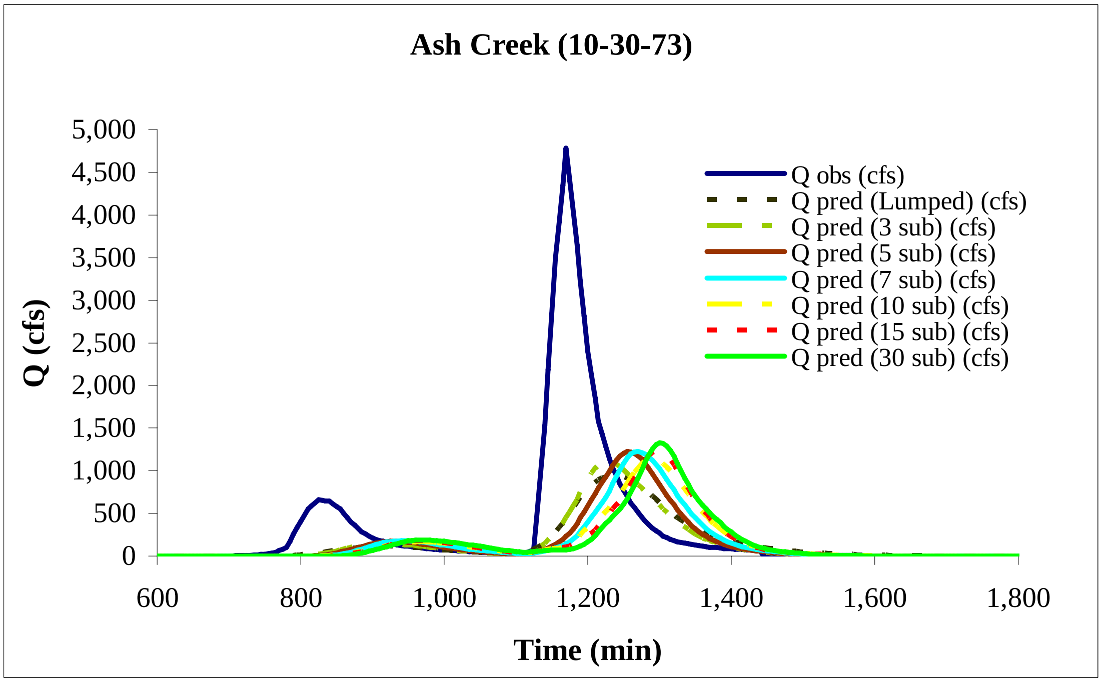

Figure B.8. Simulated and Observed Hydrographs – Ash (10-30-73)

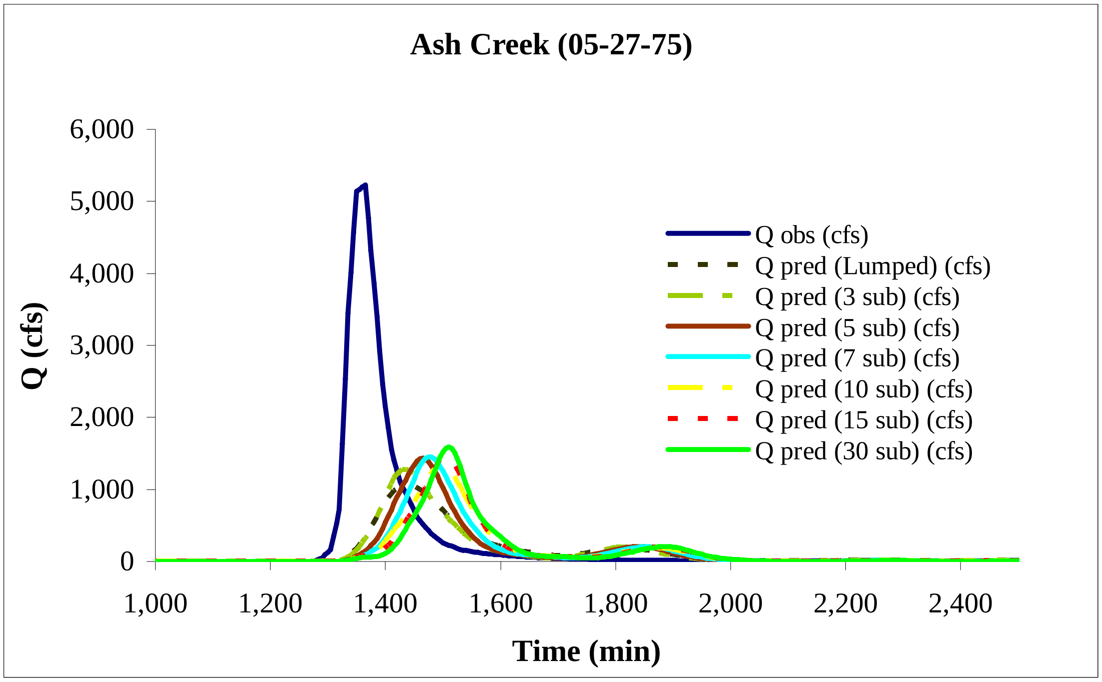

Figure B.9. Simulated and Observed Hydrographs – Ash (05-27-75)

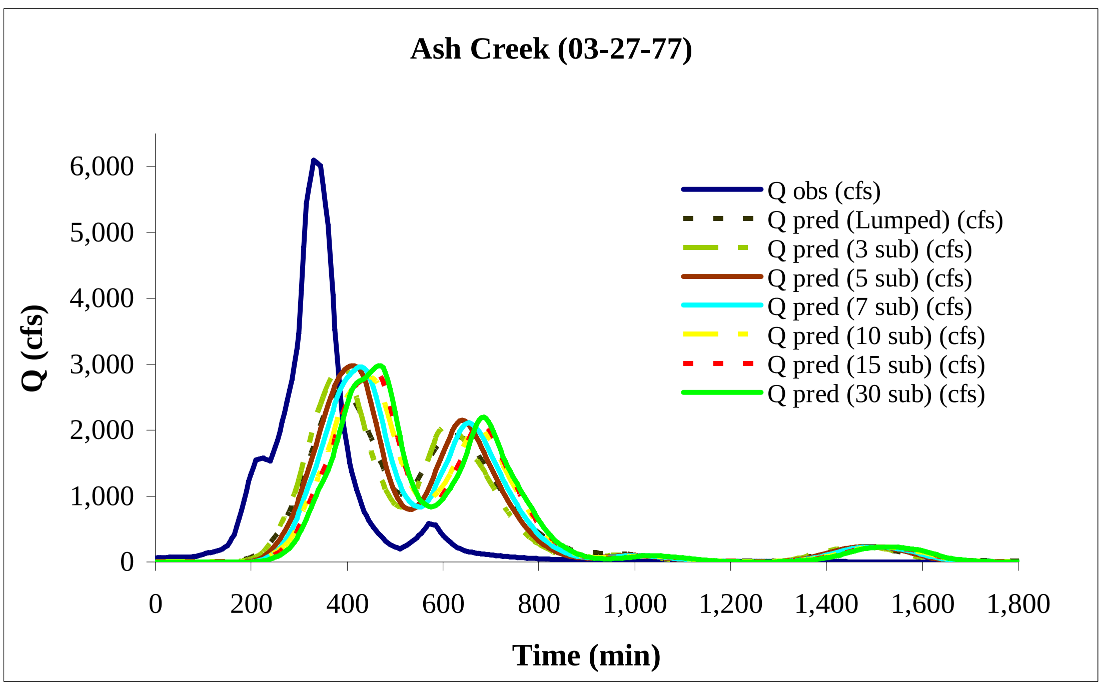

Figure B.10. Simulated and Observed Hydrographs – Ash (03-27-77)

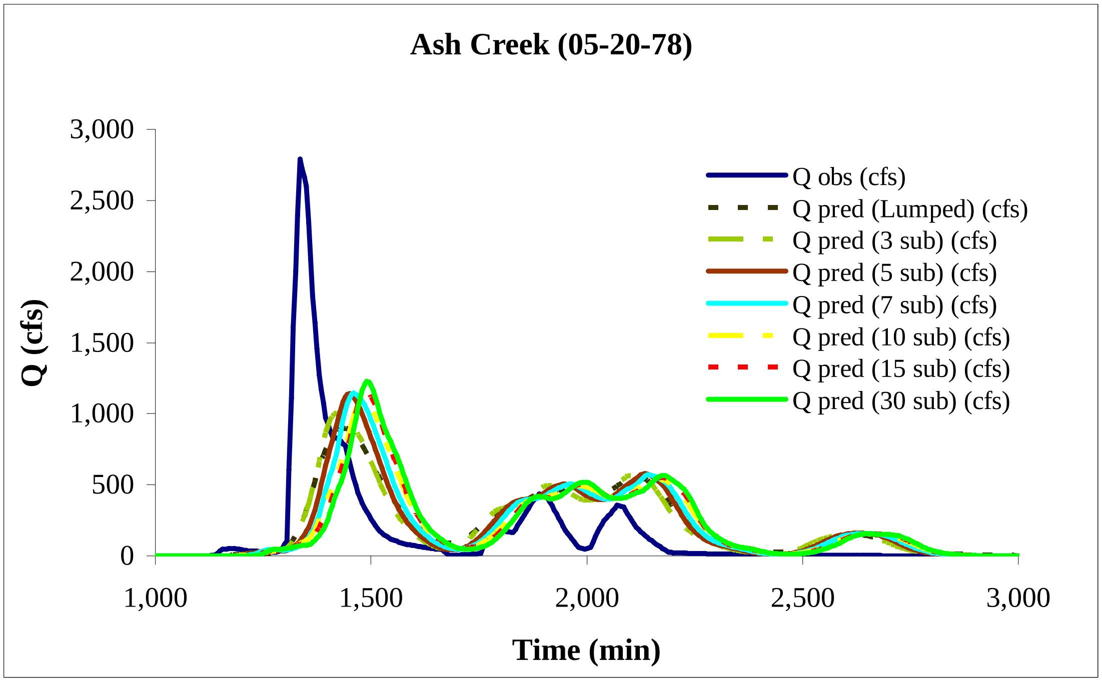

Figure B.11. Simulated and Observed Hydrographs – Ash (05-20-78)

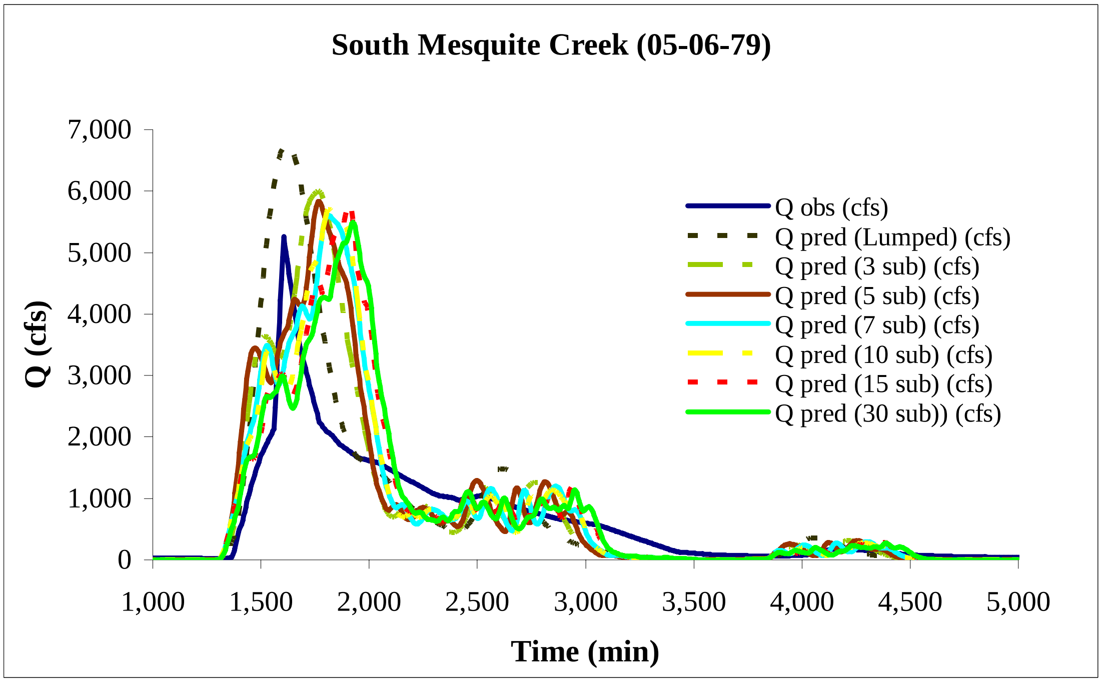

Figure B.12. Simulated and Observed Hydrographs – South Mesquite (05-06-79)

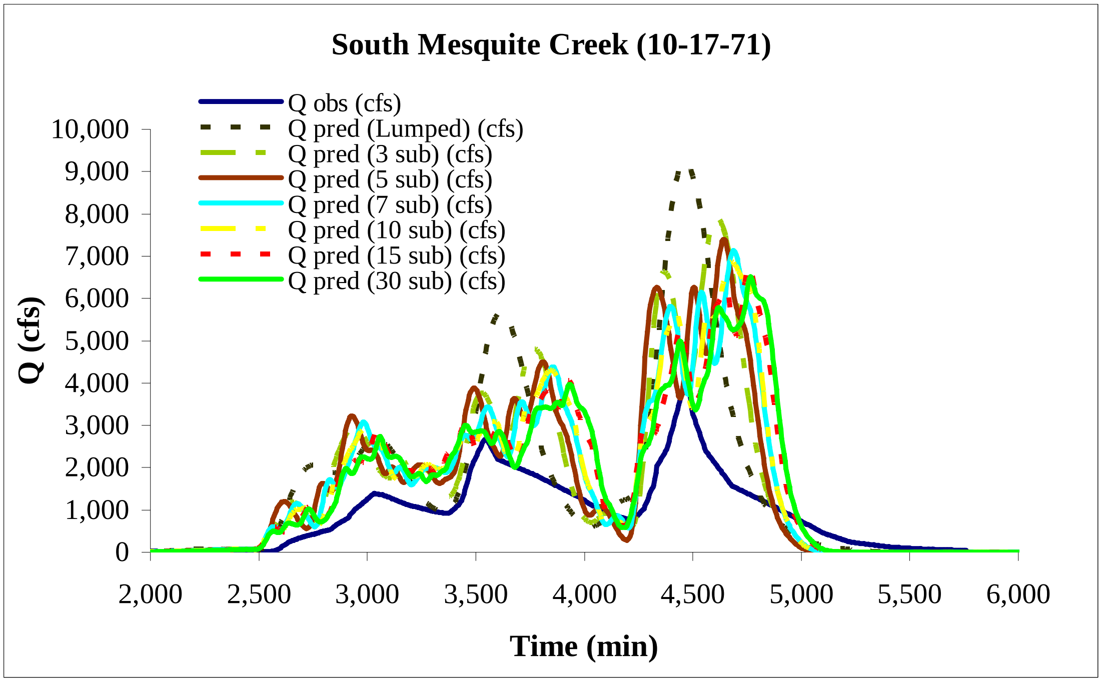

Figure B.13. Simulated and Observed Hydrographs – South Mesquite (10-17-71)

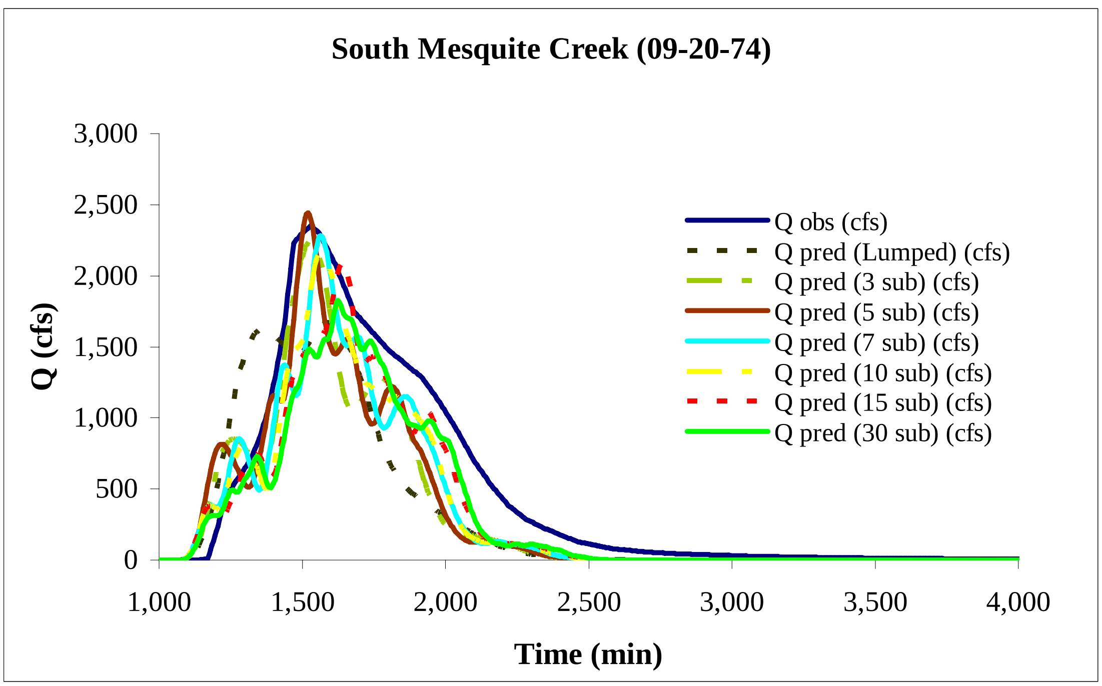

Figure B.14. Simulated and Observed Hydrographs – South Mesquite (09-20-74)

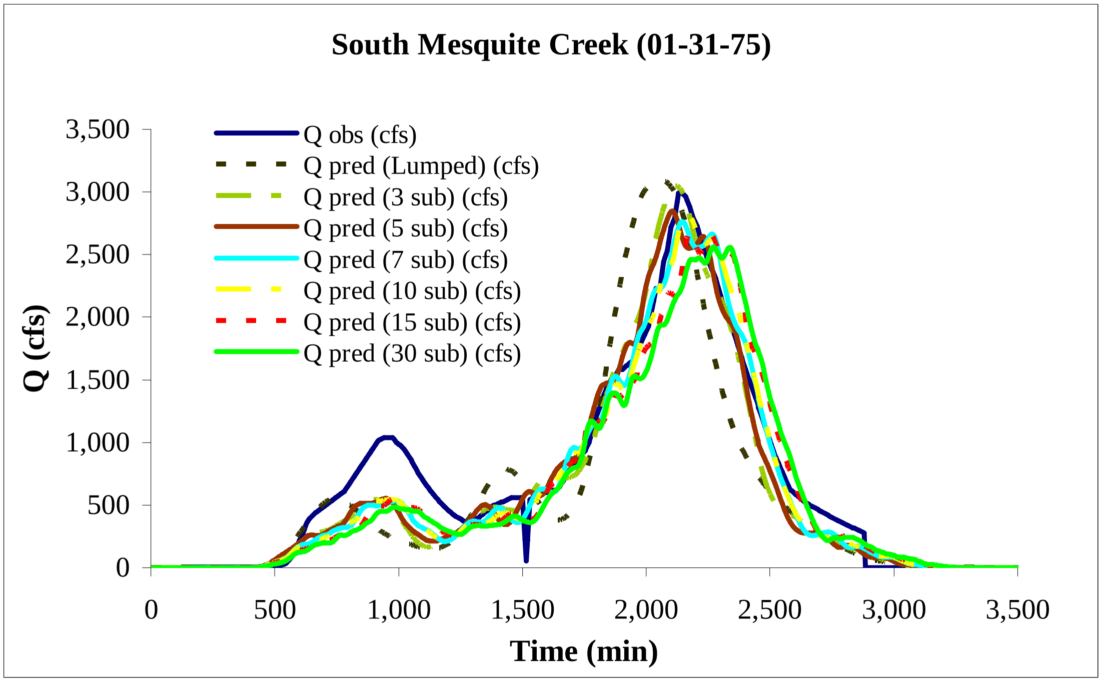

Figure B.15. Simulated and Observed Hydrographs – South Mesquite (01-31-75)

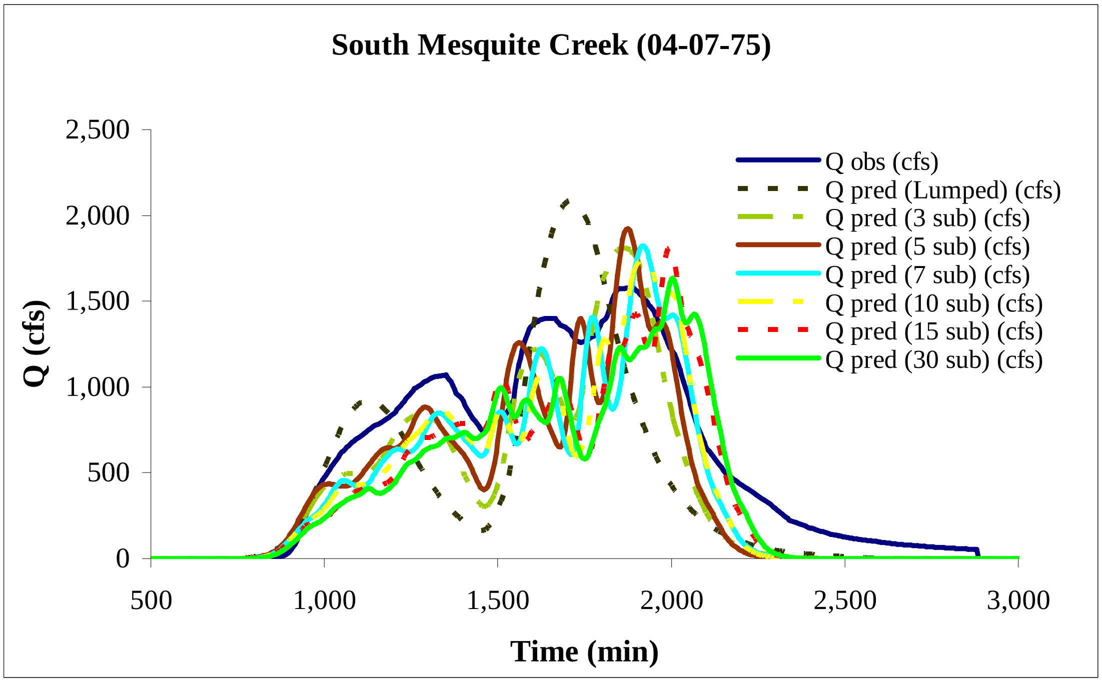

Figure B.16. Simulated and Observed Hydrographs – South Mesquite (04-07-75)

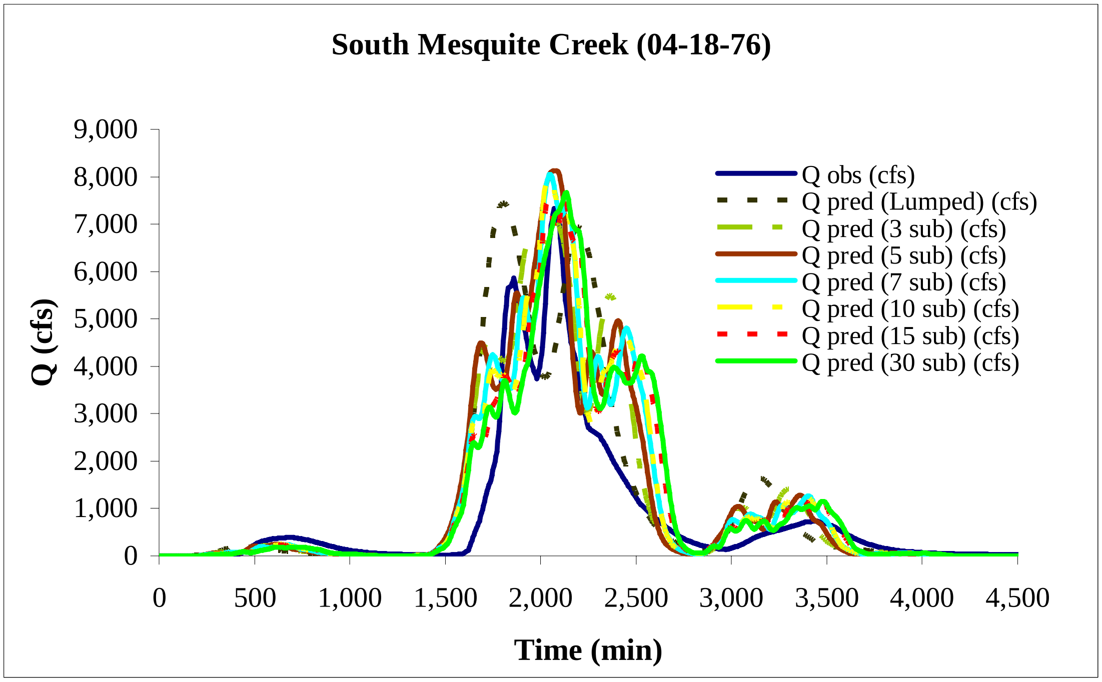

Figure B.17. Simulated and Observed Hydrographs – South Mesquite (04-18-76)

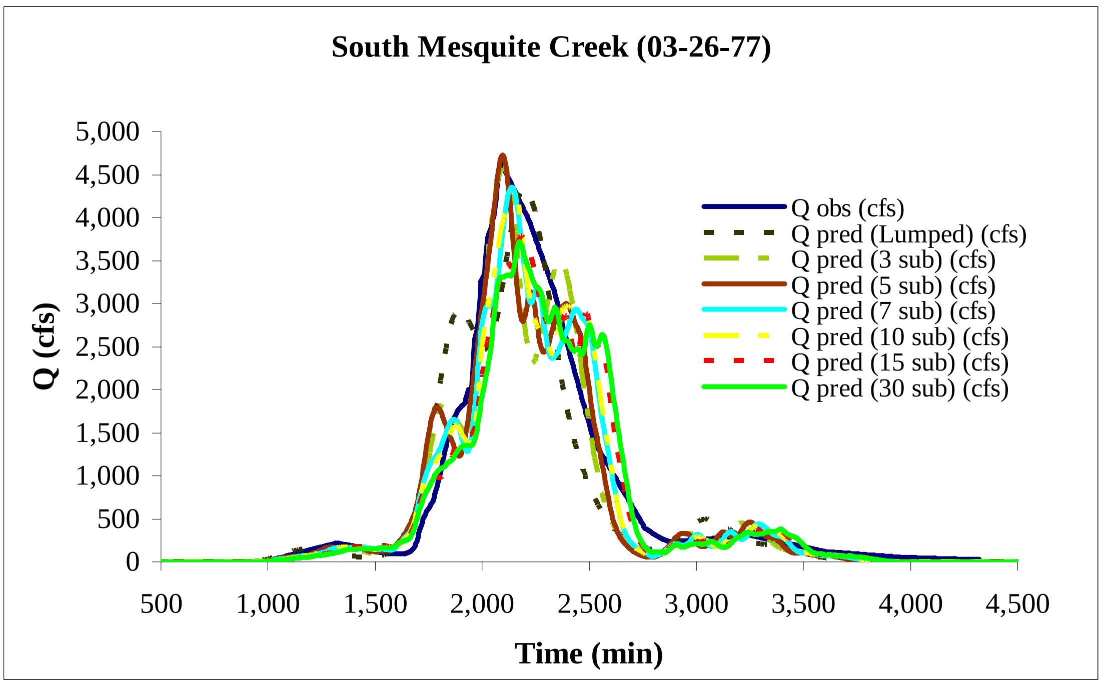

Figure B.18. Simulated and Observed Hydrographs – South Mesquite (03-26-77)

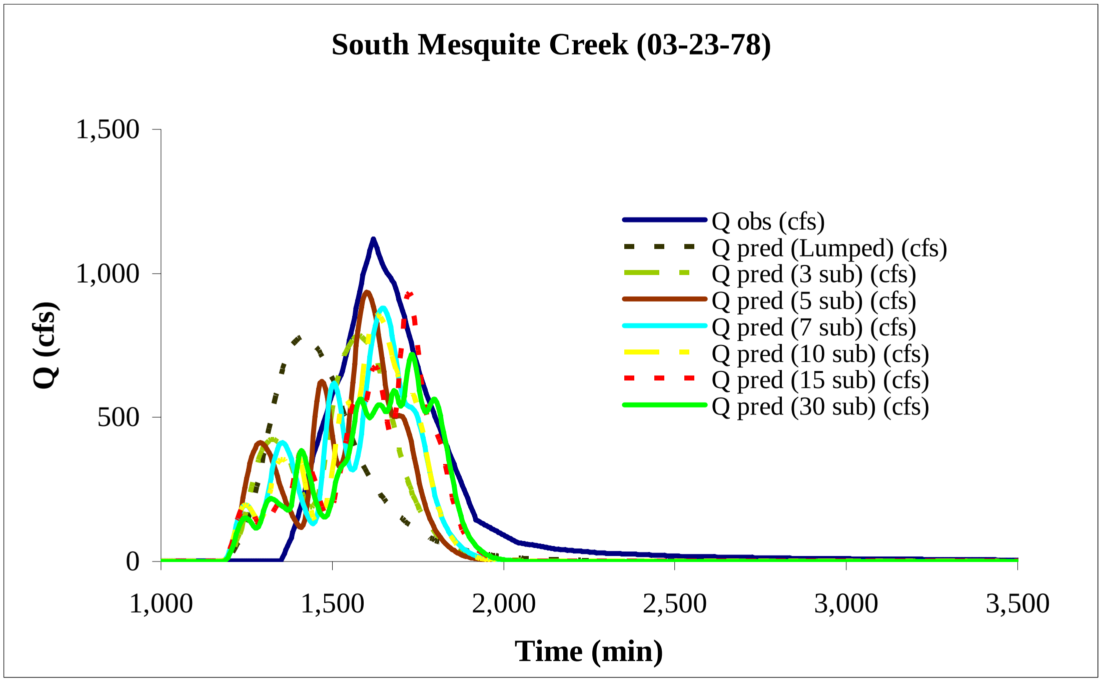

Figure B.19. Simulated and Observed Hydrographs – South Mesquite (03-23-78)

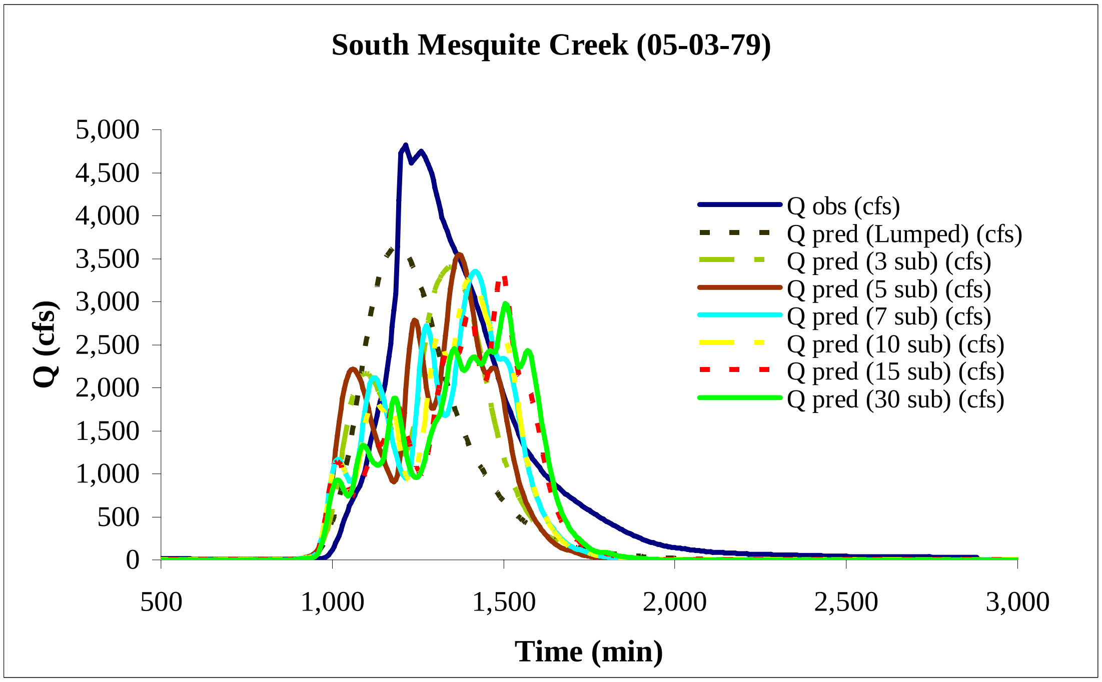

Figure B.20. Simulated and Observed Hydrographs – South Mesquite (05-03-79)

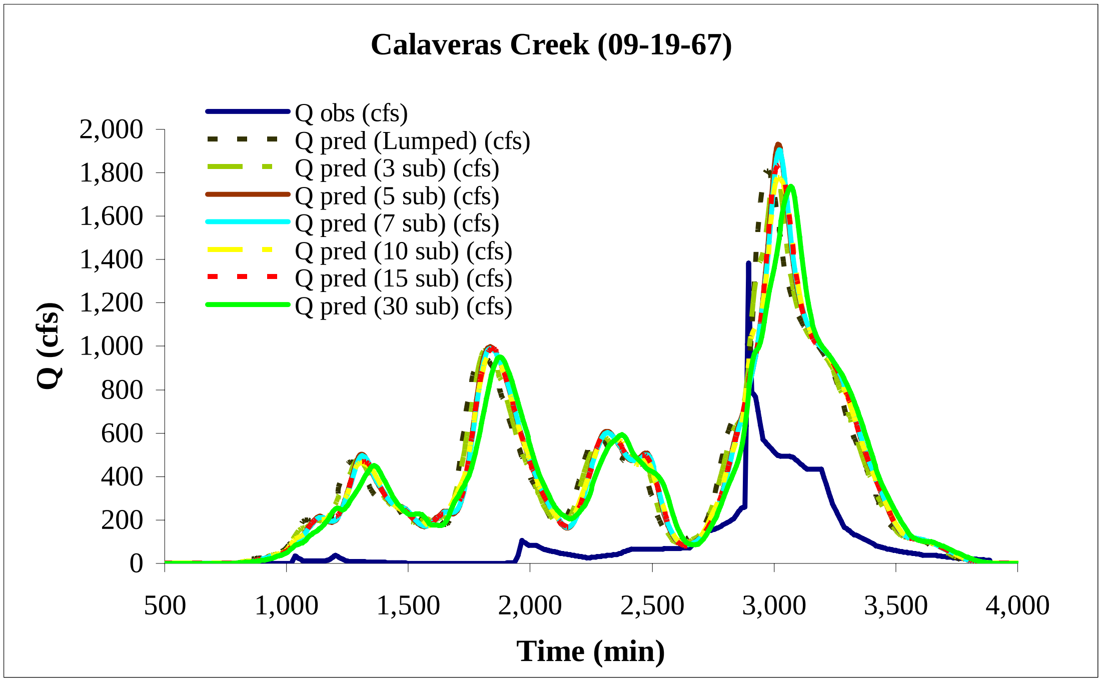

Figure B.21. Simulated and Observed Hydrographs – Calaveras (09-19-67)

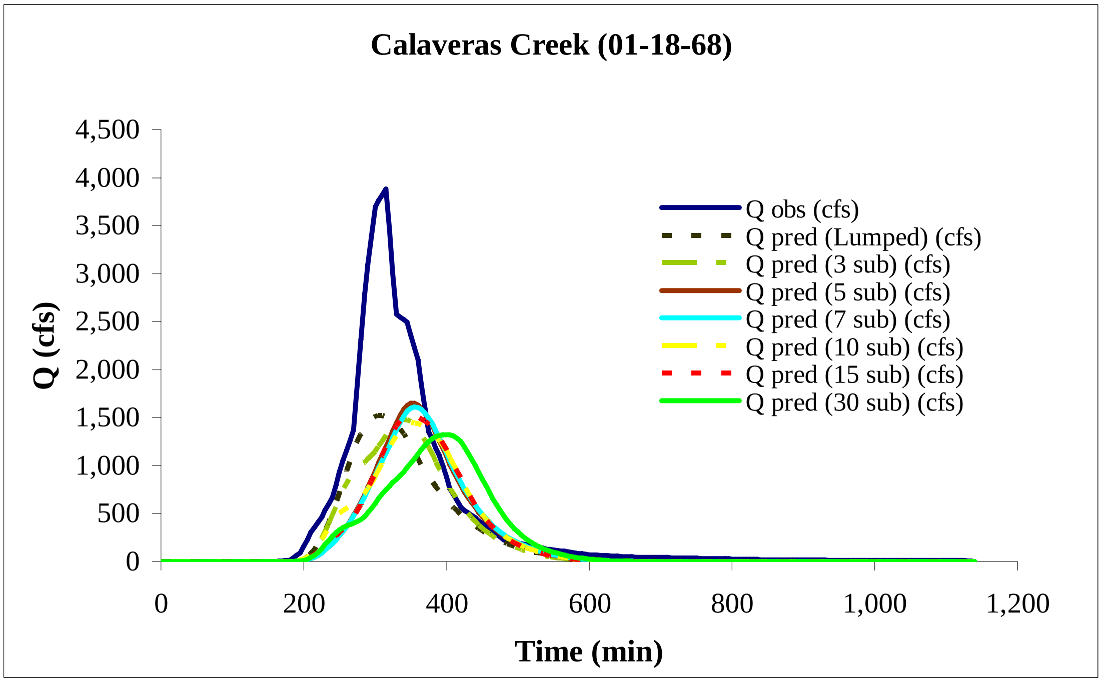

Figure B.22. Simulated and Observed Hydrographs – Calaveras (01-18-68)

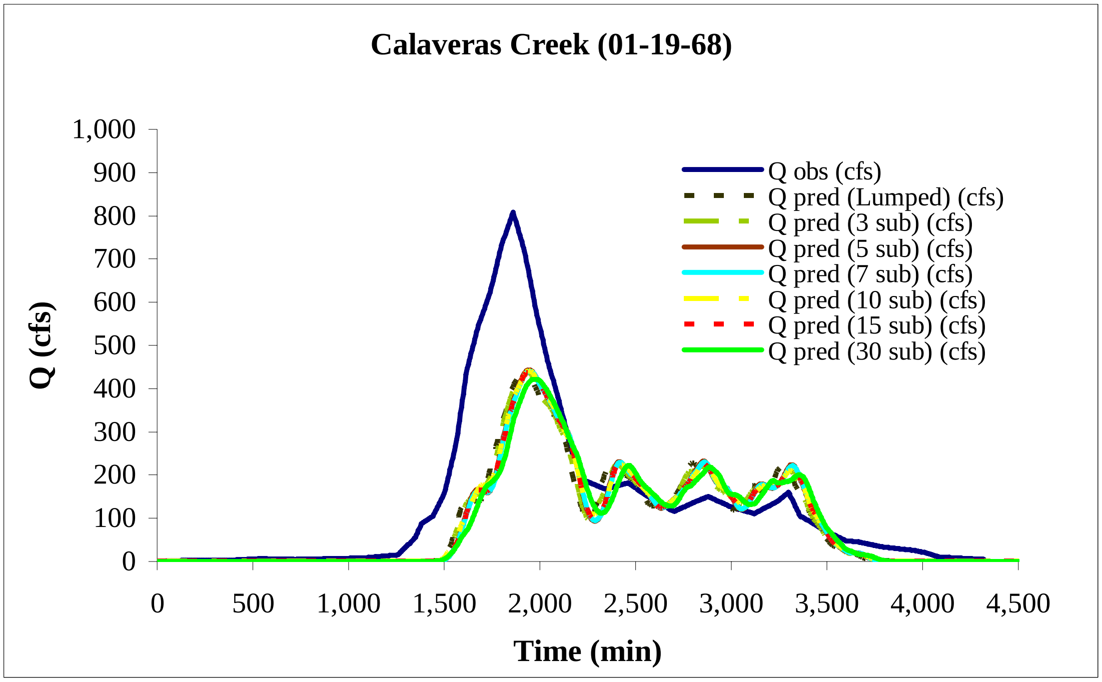

Figure B.23. Simulated and Observed Hydrographs – Calaveras (01-19-68)

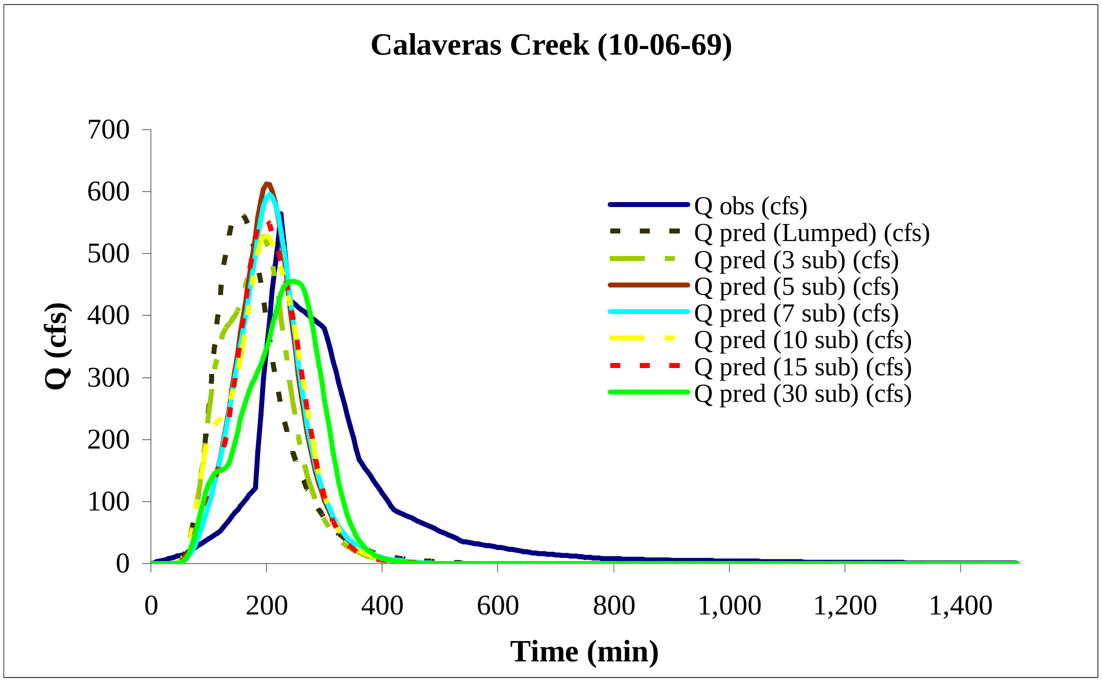

Figure B.24. Simulated and Observed Hydrographs – Calaveras (10-06-69)

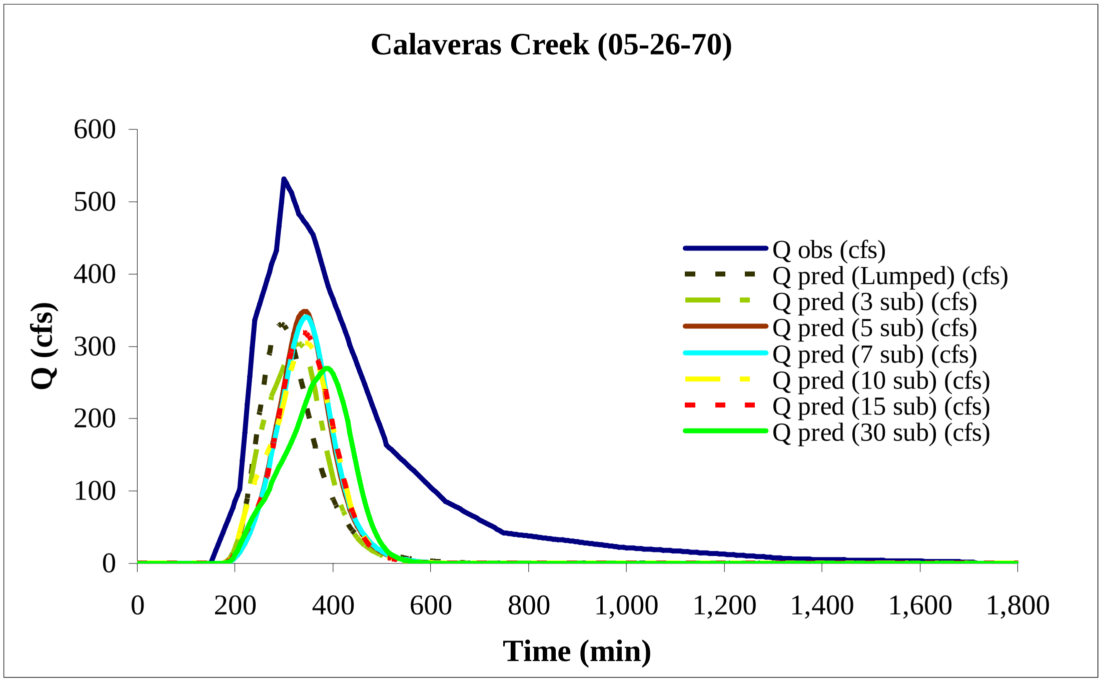

Figure B.25. Simulated and Observed Hydrographs – Calaveras (05-26-70)

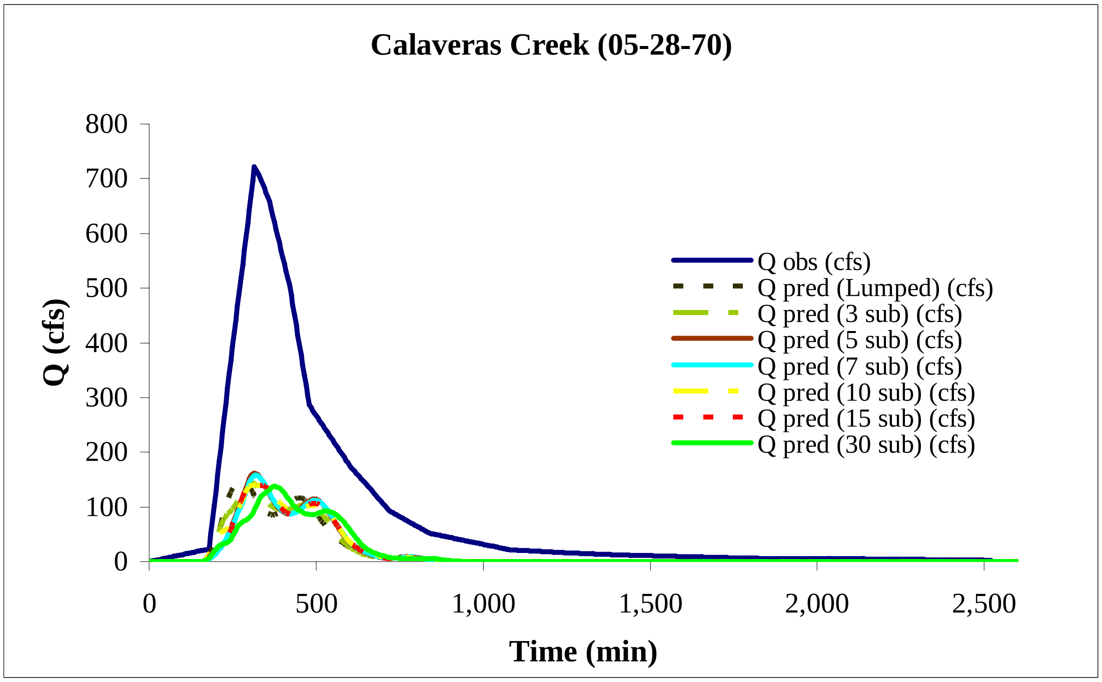

Figure B.26. Simulated and Observed Hydrographs – Calaveras (05-28-70)

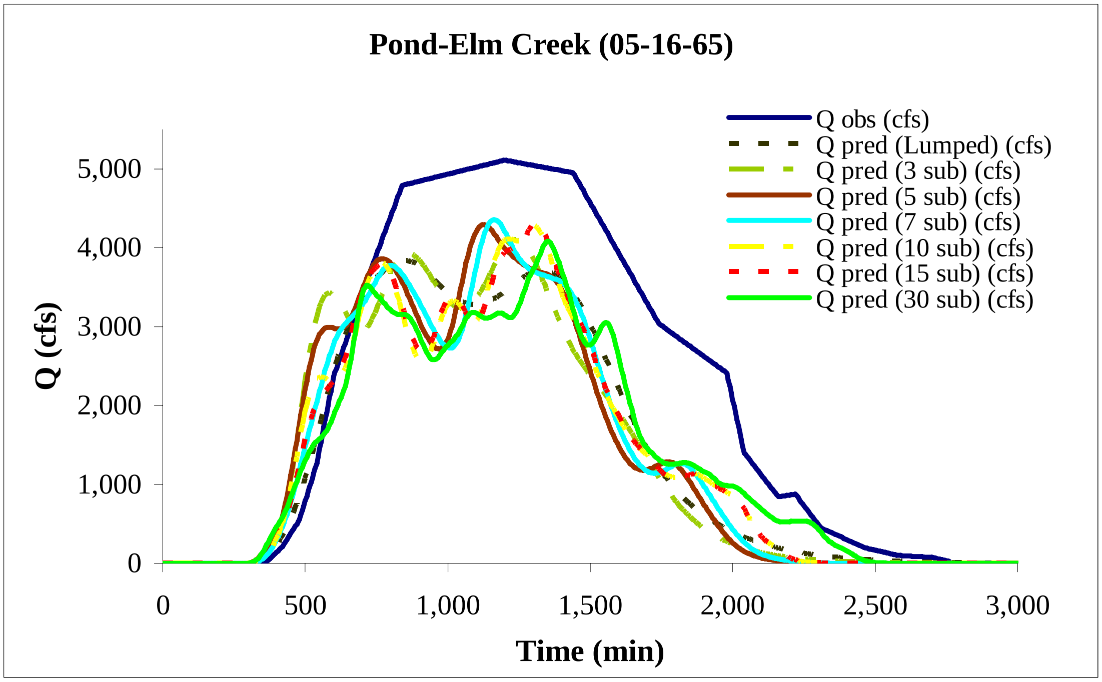

Figure B.27. Simulated and Observed Hydrographs – Pond-Elm (05-16-65)

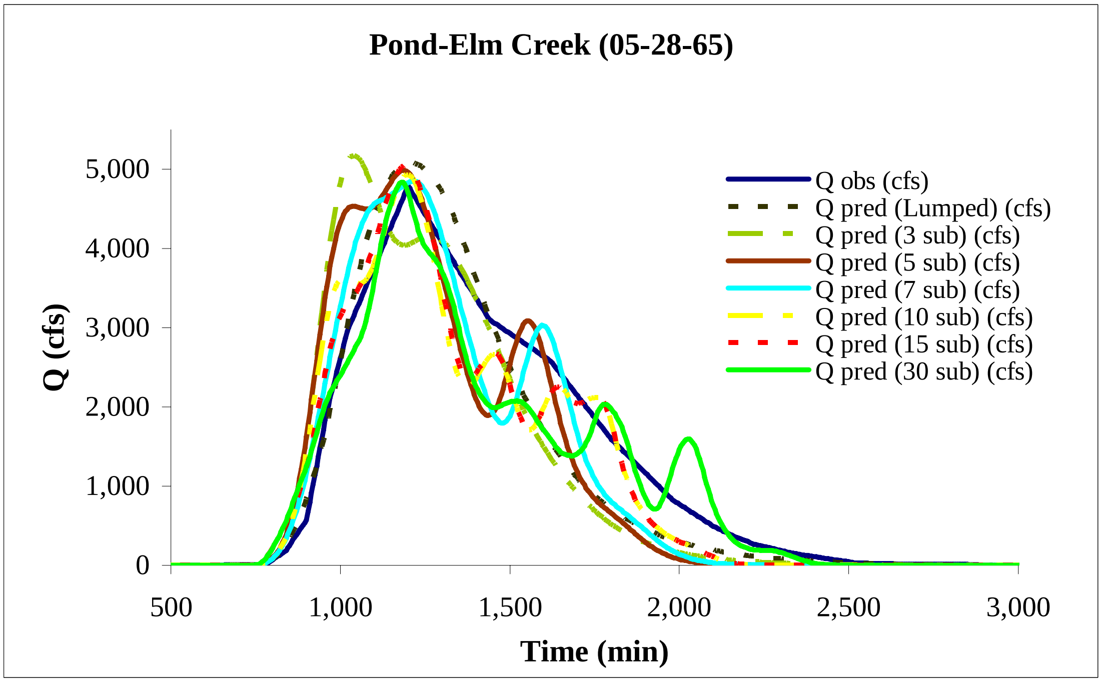

Figure B.28. Simulated and Observed Hydrographs – Pond-Elm (05-28-65)

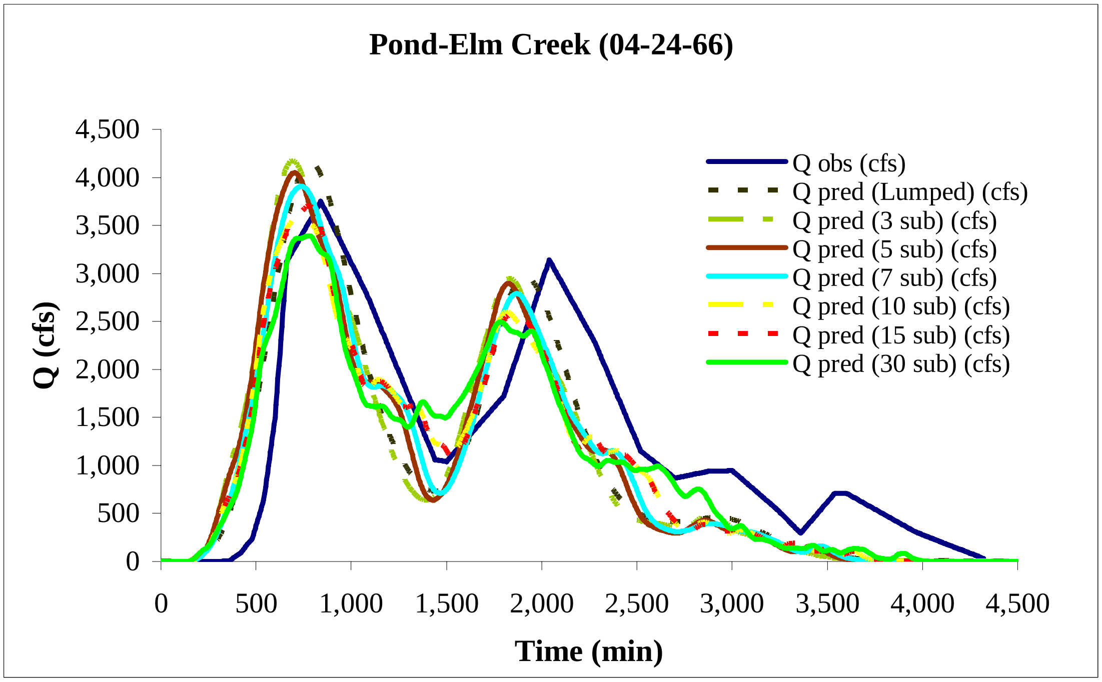

Figure B.29. Simulated and Observed Hydrographs – Pond-Elm (04-24-66)

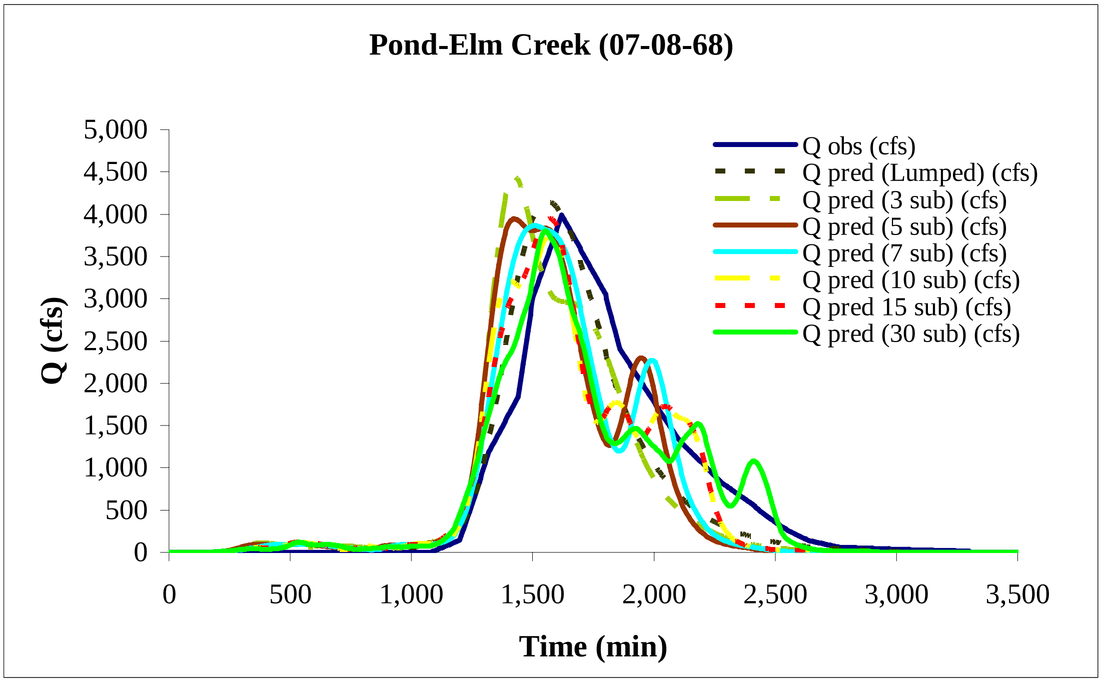

Figure B.30. Simulated and Observed Hydrographs – Pond-Elm (07-08-68)

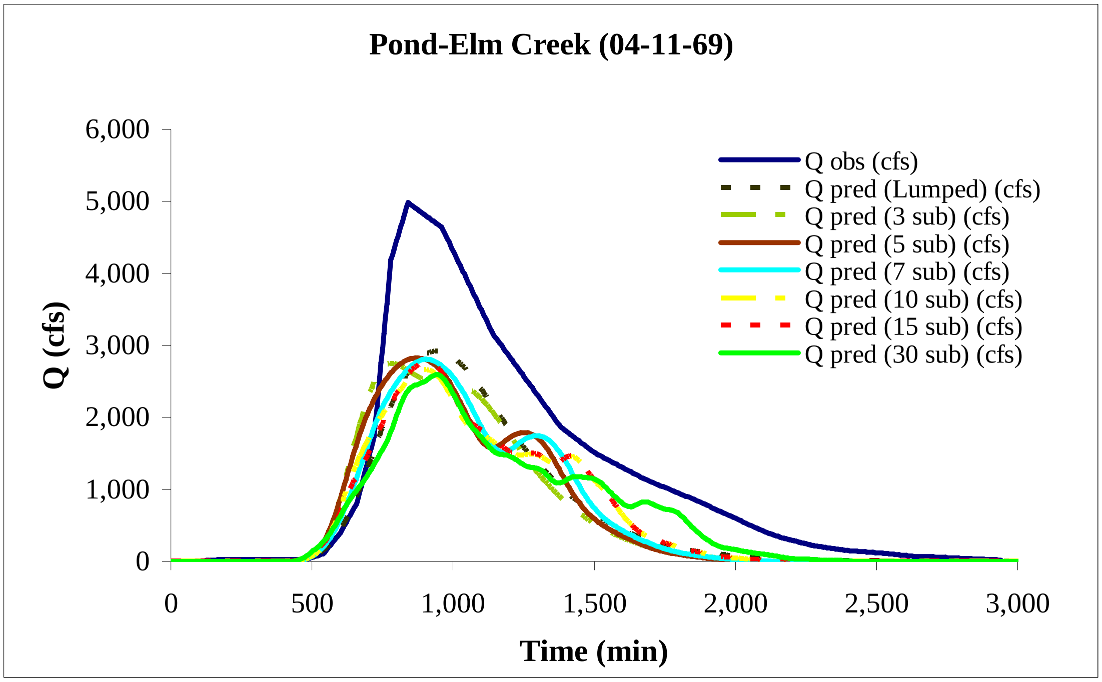

Figure B.31. Simulated and Observed Hydrographs – Pond-Elm (04-11-69)

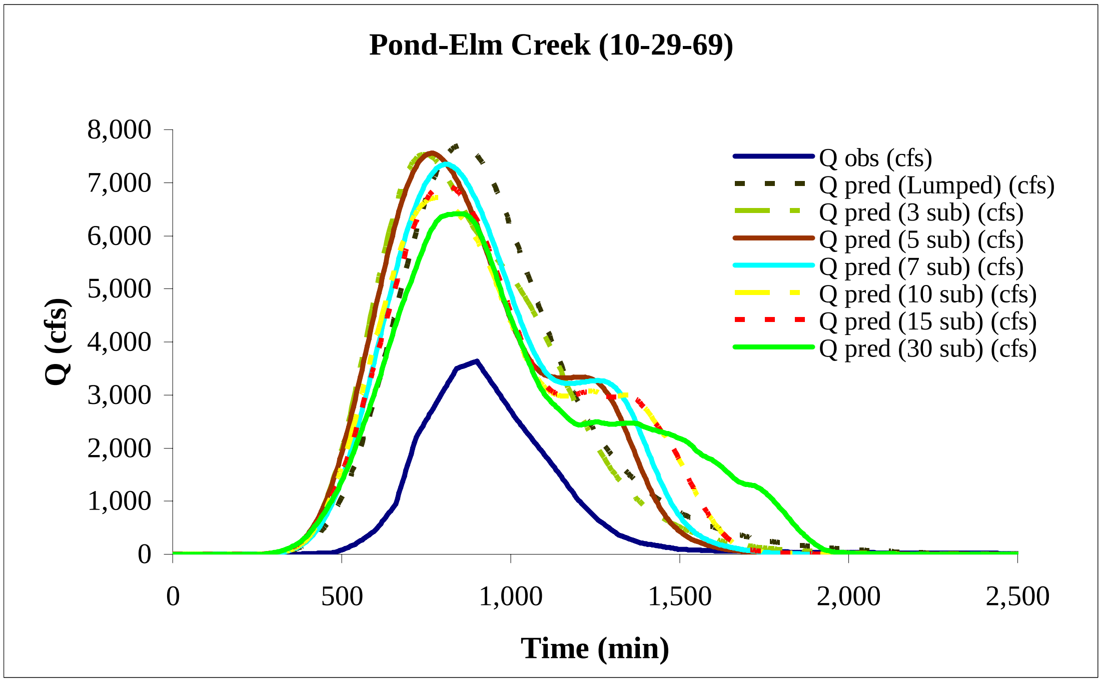

Figure B.32. Simulated and Observed Hydrographs – Pond-Elm (10-29-69)

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