Distributed Modeling Using HEC-HMS: A Continuum of Water Droplets

By

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ABSTRACT

Watershed subdivision is the process of breaking a large drainage basin into smaller sub-basins. The process of dividing a watershed into smaller pieces might be beneficial in areas of significant changes to land use or slope. However, too much or unnecessary subdivision leads to greater model complexity and less accuracy in the runoff hydrograph. Of interest to the Texas Department of Transportation (TxDOT) is the application of distributed modeling in HEC-HMS to assess the utility of distributed modeling and its effect on the outflow hydrograph.

Three watersheds in Texas were modeled using HEC-HMS 3.2. Datasets used in the analysis were created using Arc Hydro Tools and HEC-GeoHMS, both are ArcGIS extensions. Each watershed was modeled for a number of peak storm events using a distributed modeling approach, determined by reverse engineering the software used, and additional sub-basin delineation.

Based on analysis of runoff hydrographs for the subdivided watersheds computed using HEC-HMS, increased subdivision did not produce better results. In general, one sub-basin with the gridded parameter sets produced the “best” results, indicating that increased subdivision is unnecessary unless needed because of topographic or analysis-based reasons.

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CHAPTER 1

INTRODUCTION

An approach to problem solving nearly universal in engineering is the idea of taking a large and sometimes overwhelming problem and breaking it into its components. This method of analysis assumes that the components are more manageable to analyze and lead to the larger problem being solved more efficiently and possibly more accurately.

This same method of deconstructing a problem is sometimes used in the field of hydrology and hydraulics. In a hydrologic study a watershed is delineated and analyzed to determine the outflow hydrograph that will be produced under specific meteorological conditions. There are several logical reasons when additional subdivision within the watershed might be necessary. An example of where subdivision might be necessary would be if very different land use (rural versus urban) or extremely steep and then mild slopes occurred within a watershed. If these kinds of differences occur within the watershed, the analyst might rightfully decide to subdivide to separate the different regions of the watershed.

Although watershed subdivision is, at times, justifiable and even necessary, it is also greatly overused in an attempt to produce more "accurate" outflow results. Watershed subdivision increases the complexity of the modeling process. As more subdivisions are created, parameters affecting the runoff hydrograph produced from each sub-basin must be determined. As subdivision increases, the number of sub-basin parameters that must be estimated also increases. Generally, the analyst does not know enough about the finite areas produced through subdivision. This increases the uncertainty of the parameters estimated and used in the watershed analysis. With increased uncertainty, there is a tendency for the analyst to use average parameter values, effectively defeating the purpose of distributed-modeling. Although the analyst is well-intentioned to increase the "accuracy" of the produced results, there is the possibility that, because of the greater complexity caused by greater subdivision, the produced results are less accurate than what might be produced using a simplified approach.

This topic is of particular interest to the Texas Department of Transportation (TxDOT), as stated in TxDOT Project 0-5822, because of the lack of available guidance in when watershed subdivision should occur and to what extent. The objective of the research reported herein was to assess the utility of distributed models, specifically the gridded model approach available in HEC-HMS[1].

To achieve the objective, first, a review of pertinent literature was performed and reported in this document. Second, watershed analysis was performed for three Texas watersheds using six subdivision schemes based on approximately equal area. In addition to the equal area subdivisions, gridded parameter datasets were also created. Data file creation and watershed analysis were achieved using a geographic information system and the most current version of HEC-HMS available at the time of analysis. The process used in this research was determined by reverse engineering the software used in the analysis. The results of the analysis for all three watersheds studied are presented in this thesis.

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[1] Written communication with Dr. David Thompson on January 26, 2007.

CHAPTER 2

LITERATURE REVIEW

Numerous studies have been conducted to determine the affects of watershed subdivision on simulated output using various software programs. Some of the subdivision analyses are discussed herein.

2.1 SPUR Model

The U.S. Department of Agriculture's Agricultural Research Service (USDA-ARS) created and maintains the Simulation of Production and Utilization of Rangelands (SPUR) model (Sasowsky and Gardner, 1991). SPUR is a comprehensive hydrologic model, generally applied to semi-arid rangeland watersheds (Sasowsky and Gardner, 1991). Sasowsky and Gardner applied SPUR to a sub-watershed located in the Walnut Gulch watershed located in Arizona to assess the accuracy of model simulation based on watershed configuration. Sub-watershed delineations ranged from 3 to 66 sub-regions. A geographic information system (GIS) was used to analyze topographic features and calculate physical watershed characteristics (Sasowsky and Gardner, 1991). Simulation results led the analysts to conclude that greater model complexity increased model accuracy. A threshold was observed indicating that there was a point at which increased subdivision would not provide noticeable improvement in the model results (Sasowsky and Gardner, 1991). Modeled results, however, did show an inherent sensitivity between simulated runoff and basin curve number (Sasowsky and Gardner, 1991).

2.2 SWAT Model

Numerous researchers performed watershed subdivision analysis using the Soil and Water Assessment Tool (SWAT) (Mamillapalli et al., 1996; Bingner, et al., 1997; FitzHugh and Mackay, 2000; Hernandez, et al., 2002; Jha, et al., 2004). SWAT is a hydrologic basin scale model, available in the public domain, which is used to quantify the effect of land management changes in large watersheds.

Mamillapalli, et al. (1996) used SWAT to assess the effects of increased discretization on one Texas watershed. GRASS-GIS, a GIS interface used to create the input files for a SWAT model, was used to subdivide the watershed into sub-basins, ranging from 4 to 54 sub-basins (Mamillapalli, et al., 1996). The analysts observed a "general" increase in accuracy of the modeled results relative to increased subdivision, as measured using a coefficient of efficiency (Mamillapalli, et al., 1996). A threshold was observed in the simulation, which indicated that subdivision beyond such a point would not further increase model accuracy. Although briefly mentioned, the analysts did not address a specific subdivision configuration that was optimal for the watershed studied.

Bingner, et al. (1997) applied SWAT to model ten subdivision schemes for the Goodwin Creek watershed in Mississippi. The researchers determined that the degree of subdivision affected only the sediment yield from the watershed. The simulated runoff was not sensitive to the increased number of sub-areas, leading the analysts to suggest that for stream flow analysis, less subdivision is adequate for modeling watershed runoff response.

FitzHugh and Mackay (2000) analyzed the Pheasant Branch watershed in Wisconsin using eight subdivision schemes, ranging from 3 to 181 sub-watersheds. They concluded that stream flow increased by only 12 percent between the coarsest (3 sub-watersheds) and the finest (181 sub-watersheds) subdivisions. The minimal change in stream flow was attributed to the strong correlation between runoff and curve number, which was unchanged in the analysis. Although sediment generation was greatly affected by changes in subdivision, transported sediment simulated at the outlet was not. The researchers thus concluded that for the purpose of stream flow and sediment transport analysis, coarser watershed delineation is adequate (FitzHugh and Mackay, 2000).

Hernandez, et al. (2002) applied both SWAT and the Kinematic Runoff and Erosion (KINEROS) model to assess the quality assurance and control of the

Automated Geospatial Watershed Assessment (AGWA) tool. AGWA is a GIS-based interface used to derive spatially-distributed watershed parameters from a digital elevation model. Through analysis of model results, the AGWA tool was determined to be most applicable for relative comparisons of watershed response, not in the estimation of specific rainfall-runoff response for one particular storm event. The analysts did state that the numerous parameters required in the analysis can decrease model accuracy as subdivision increases because of an increased number of parameters that must be estimated (Hernandez, et al., 2002).

Jha, et al. (2004) modeled four Iowa watersheds to assess the effects of various subdivision schemes on stream flow and sediment yield, as well as to develop guidelines for threshold subdivision. The Better Assessment Science Integrating Point and Nonpoint Sources (BASINS) tool was used, through ArcView, to obtain topographic data from the digital elevation model, and to create the SWAT input files (Jha, et al., 2004). The simulated stream flow varied by only seven percent between the coarsest (three sub-basins) and the finest watershed delineations (53 sub-basins). Changes in land use or soils would have produced a greater impact in the stream flow because of their direct effect on infiltration/runoff potential (Jha, et al., 2004). Modeled sediment yield increased greatly, which is indicative of a direct relationship between the sediment transported and the amount of watershed subdivision (Jha, et al., 2004). Although the researchers intended to present guidelines for subdivision, this was not explicitly addressed. This is believed to be related to the inherent differences in watershed analysis, which would not be conducive to one set of specified guidelines.

2.3 HEC-HMS Model

U.S. Army Corps of Engineers' Hydrologic Engineering Center personnel created a widely used hydrologic modeling system (HEC-HMS). HEC-HMS evolved from the previous generation of rainfall-runoff modeling software, the DOS-based HEC-1. Norris and Haan (1993) used HEC-1 to demonstrate the effects of differing subdivision schemes on the hydrologic analysis of a watershed after undergoing some type of watershed modification, for example, reservoir installation or land cover and land use changes. The Little Washita watershed in Oklahoma was subdivided into one, two, five, ten, and fifteen sub-basins (Norris and Haan, 1993). The modeled peak discharges showed considerable variation relative to increased subdivision. Time to peak, however, remained relatively constant. The modelers determined that the outflow hydrographs were dependent on the number of sub-basins used in the analysis and that the number of sub-basins should be consistently used in all analysis of the watershed (Norris and Haan, 1993). It should be noted that Norris and Haan concluded that simulated runoff was not sensitive to the curve number (Norris and Haan, 1993).

McCormick (2003) investigated the effects of resolution and grid cell size on model output from the Upper Roanoke River watershed. HEC-GeoHMS (Hydrologic Engineering Center, 2003), an ArcGIS (ESRI, 2006) extension, was used to create HEC-HMS input files. McCormick used a fully-distributed hydrologic model and compared its output to a similar lumped model. In the distributed model results, there was a noticeable sensitivity to the curve number used (McCormick, 2003). The ModClark transform seemed relatively insensitive to grid cell resolution or size (McCormick, 2003). Application of the distributed model resulted in little improvement over use of the lumped model. Differences between the outflow hydrographs produced by the models were insignificant (McCormick, 2003).

Luong (2008) evaluated the impact of watershed subdivision based on the creation of sub-basins with approximately the same physical characteristics. Five Texas watersheds were subdivided using five subdivision schemes. Within each sub-basin, the hydrologic parameters were averaged to produce a lumped model approach (Luong, 2008). Luong's analysis assessed the impact of increased subdivision on the total runoff volume, peak discharge, and the time to peak discharge for each watershed. She observed that for the five Texas watersheds studied, increased subdivision did not significantly affect the simulated runoff hydrographs for the storm events which were modeled. The curve number, however, was found to be a sensitive parameter in terms of the modeled output. Luong (2008) concluded that, unless required for some reason, arbitrary subdivision is unnecessary.

2.4 Design Criteria

Hromadka (1986) presented guidelines for hydrologic analysis and design for San Bernardino County. In the San Bernardino County manual, Hromadka directly addressed the issue of simple and complex watershed subdivision. Subdivision is considered necessary only when the watershed consists of varying land uses and soil conditions, or storage effects that could offset the modeled results. When a complex watershed analysis is performed, a simple, one-basin, model is required for comparison (Hromadka, 1986). Because of increased unreliability in complex models, arbitrary subdivision is to be avoided (Hromadka, 1986).

2.5 Implications

Based on the results of work by other researchers, no definitive conclusion can be made as to the effect of increasing subdivision on hydrologic modeling. Some researchers believe simplicity in model creation is “better” than complexity. Other researchers presented models in which increased subdivision resulted in improved outflow hydrograph “accuracy.” With increasing technological capabilities it is possible to create fully-distributed datasets resulting in more concern about the possibility of sub-dividing a watershed beyond a justifiable level. It is pertinent to assess the utilities available and the results produced from such distributed modeling methods and software.

One primary objective of most of the aforementioned studies is to present an “optimal” sub-watershed or subdivision configuration criteria. Because of various differences in software used for analysis as well as the topographic differences inherent in different watershed analyses, one set of governing guidelines is difficult to determine and not addressed in the final conclusions of the previously described research.

The idea of a threshold also appeared in numerous reports. A threshold is a point of subdivision beyond which increased accuracy is negligible. A threshold is an important concept to understand and recognize, however, the threshold for subdivision will vary between different watersheds and perhaps different models. This complicates the idea of creating universal watershed subdivision guidelines.

A notable sensitivity in the models presented in the literature reviewed was attributable to use of the Natural Resources Conservation Service (NRCS) curve number for the runoff-generation process sub-model. The curve number is variable based on land cover and is inherently linked to the runoff produced from a watershed. It would be beneficial to quantify, if possible, the sensitivity of the curve number chosen relative to the runoff produced.

McCormick (2003) was the only researcher identified who analyzed fully-distributed models and the impact of gridded parameter sets on outflow results. Because of the lack of research in this particular area, the approach taken in this research was determined, in part, by trial and error.

CHAPTER 3

PROCEDURE

3.1 Study Location

Three watersheds were studied: South Mesquite Creek and Ash Creek in Dallas, Texas, and Little Pond Elm Creek northeast of Austin, Texas. The relative location of each watershed in Texas is shown on Figure 3.1. General information for the study watersheds are listed in Table 3.1.

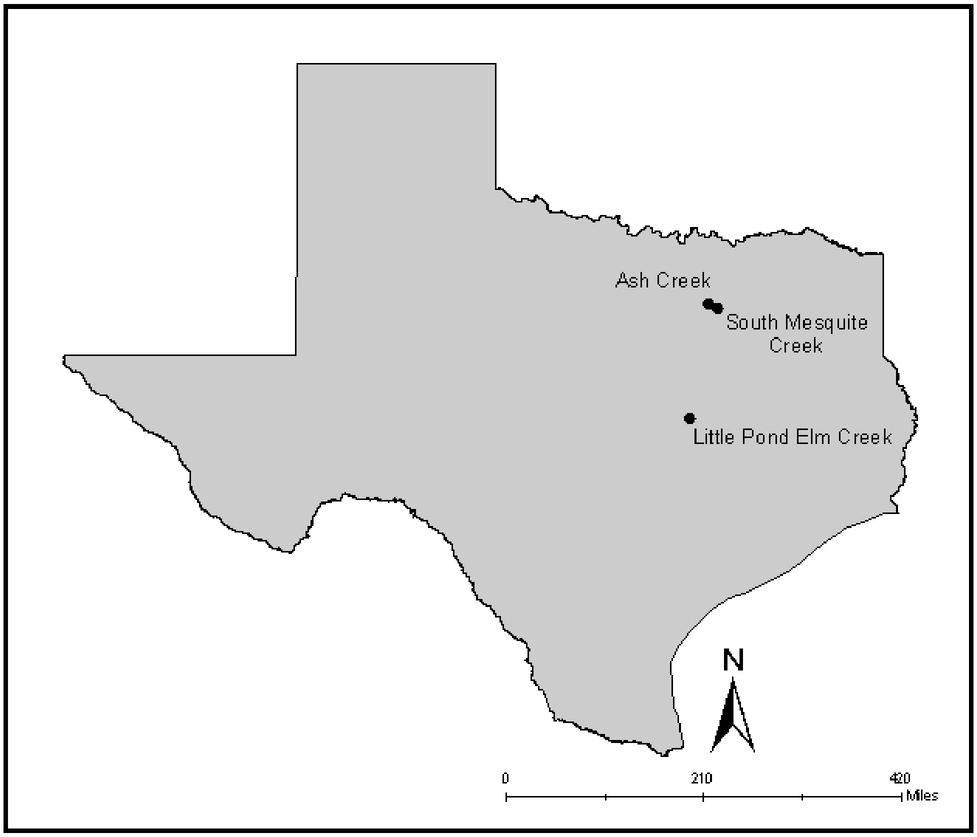

Figure 3.1. Study Locations.

Table 3.1. Study Locations General Information.

Watershed	Outlet Location			Area	Classification
	USGS Gage	Latitude	Longitude		
South Mesquite Creek	08061950	32°43'32"	96°34'12"	23 mi^2	Urban
Ash Creek	08057320	32°48'18"	96°43'04"	6.92 mi^2	Urban
Little Pond Elm Creek	08098300	31°01'35"	96°59'17"	22.2 mi^2	Rural

3.2 Application of Geographic Information System

The three watersheds were initially analyzed using a combination of tools in a Geographic Information System (GIS). The GIS was used to create the files that would later be used to model the watershed response to precipitation data.

The GIS datasets were downloaded from the internet and are freely available. The National Map Seamless Server (formerly the United States Geological Survey (USGS) Seamless Server) was used to obtain the 30-meter digital elevation model (DEM) and 2001 Land Cover data. Soil data were downloaded based on the county in which the watershed was located from the Natural Resource Conservation Service (NRCS) Soil Data Mart (formerly the Soil Survey Geographic (SSURGO) Database). High-resolution stream data, based on sub-basin location, were downloaded from the USGS National Hydrography Dataset.

A common map projection was applied to all GIS layers prior to beginning analysis. The projection used was the Universal Transverse Mercator (UTM) Zone 14 North. The datum was North American Datum (NAD) 83. Any additional layers created through the analysis maintained the same projection and datum.

3.3 Application of Arc Hydro Tools

Arc Hydro Tools (Maidment, 2002) is an extension to ArcGIS used to create and model geospatial and temporal data. The following procedure that will be discussed is briefly explained in Atkinson's *Introduction to HEC-GeoHMS* (2008). An explanation of some of the steps and their purpose is presented in *Arc Hydro: GIS for Water Resources* (Maidment, 2002).

The required procedure for application of Arc Hydro Tools is repetitive in nature. To maintain consistency in the analysis and also reduce potential errors, a base map was created for each study watershed after the completion of basic watershed processing. This base map was then used as the basis of all remaining analysis, including equal-area subdivision and HEC-HMS[2] file creation.

Multiple steps are required to produce the necessary layers in Arc Hydro Tools. The analysis steps are briefly discussed below.

1. DEM Reconditioning uses the stream locations, dictated by the stream dataset, to "burn" the stream centerlines into the DEM. This step is necessary to ensure that any "bumps" in the elevation model are removed which might act as a virtual pond or reservoir in the stream network. This step ensures that by removing the "bumps" there is consistency in the stream network that will be created. If potential bumps are not removed, the bumps may act as a dam (or other flow impedance), restricting water flow in the downstream direction.
2. Fill Sinks looks at every cell in the DEM and its surrounding cells to determine if the cell is a "dip" or a low point in the model. If all of the surrounding cells are at a higher elevation than the cell of interest, then the cell of interest is raised to the lowest elevation of the surrounding cells. Fill Sinks prevents unwanted water storage or water pooling to occur by removing low points from the DEM.
3. The Flow Direction grid is created by using the eight direction pour-point model to determine the direction of steepest descent. The eight direction pour-point model is shown in Figure 3.2.

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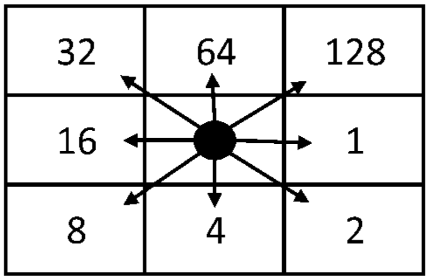
[2] HEC-HMS is the Hydrologic Modeling System created by the Army Corps of Engineers' Hydrologic Engineering Center.

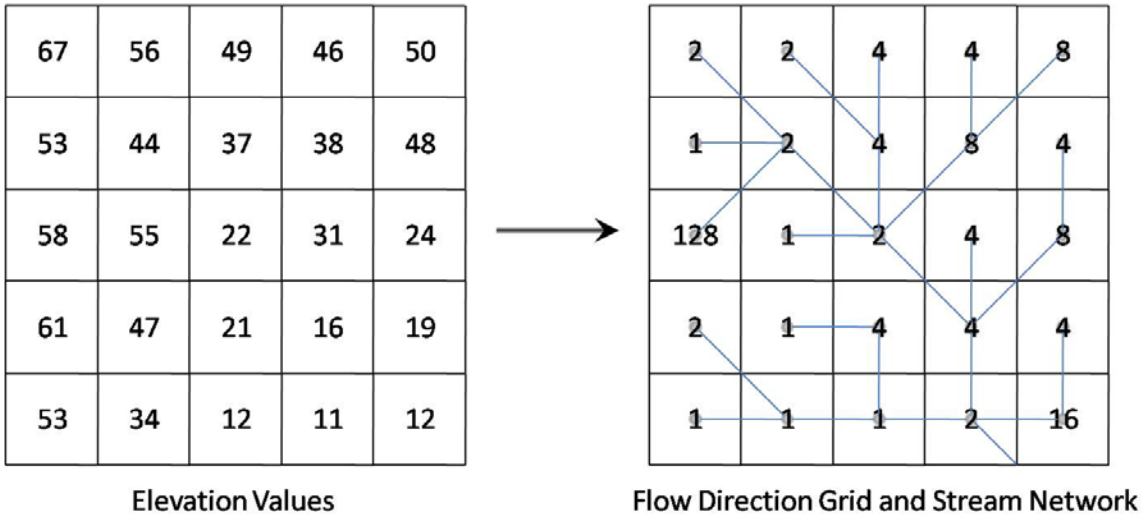


32	64	128
16		1
8	4	2

Figure 3.2. Eight Direction Pour-Point Model.

Elevation values are evaluated for each cell relative to its surrounding cells. The steepest descending elevation change is then color-coded according to the eight direction pour-point model value indicating the flow direction for each cell.

4. The Flow Direction grid is used to create the Flow Accumulation grid by calculating the number of cells flowing into each individual cell in the model. This process creates the initial stream network. The creation of the stream network from the DEM and Flow Direction grid is shown in Figure 3.3. The Flow Direction grid can be color-coded to show the accumulation of flow through the progression of the stream network.



67	56	49	46	50
53	44	37	38	48
58	55	22	31	24
61	47	21	16	19
53	34	12	11	12

Elevation Values

2	2	4	4	8
1	2	4	8	4
128	1	2	4	8
2	1	4	4	4
1	1	1	2	16

Flow Direction Grid and Stream Network

Figure 3.3. Stream Network Creation. (Maidment, 2002)

5. The Stream Definition grid is produced using a parameter called the stream threshold value. The stream threshold defines the starting point of a stream by indicating the minimum drainage area or number of cells that must flow to a single point for a stream to begin. The threshold value is chosen by the analyst and should be chosen based on understanding of the topography and extent of the study area. A typical threshold value of 5,000 cells is used with the National Elevation Dataset (NED; source of the DEM). Threshold values of less than 5,000 cells tend to cause an intricate stream network to be created, consisting of a number of small creeks that in reality would not exist. A greater threshold value will limit the stream network and could remove important stream segments from the stream network (Maidment, 2002). A threshold of 5,000 cells was used for South Mesquite Creek and Little Pond Elm Creek. The drainage area of the Ash Creek watershed was much less than the other two study watersheds. Therefore, a threshold value of 2,200 cells was used.
6. The Stream Segmentation grid was created from the Stream Definition grid by breaking the flow network into river segments at the stream confluences. Each stream segment was then assigned a unique value, and could be color-coded accordingly.
7. Catchment Grid Delineation was used to create unique basins based on the Flow Direction grid and the individual stream segments. The Flow Direction grid was used to define the drainage areas based on water flow because of steepest descent. Stream segments containing the same unique value were then used to "delineate" the catchments. The first connection between the physical characteristics of the terrain and the stream flow network is provided in this step (Maidment, 2002).
8. Catchment Polygon Processing was used to convert the Catchment Grid from a raster dataset to a vector dataset. This step results in individual polygons representing each catchment. The Drainage Line Processing

converts the raster streams to vector streams, represented as polylines. The raster to vector conversion process is required for later analysis steps.

9. During the conversion of the catchments from raster to vector form, some spurious or unnecessary polygons are produced. Adjoint Catchment Processing was used to "dissolve" the unnecessary polygons into their original catchment.
10. The Slope grid is created from the DEM, where the slope is calculated between each cell in the DEM. The Slope grid is used later in the GeoHMS analysis.

Application of the steps defined in the preceding text completes the pre-processing of the initial datasets, the DEM, and stream dataset. Through the pre-processing analysis, ArcGIS was used to create a terrain model for the area encompassing the study watershed. The direction of flow was determined by calculating the steepest descent between each cell, which was the first step in creating the channel network. The stream network was defined through a process of calculating the flow accumulation to each cell in the DEM, based on the steepest descent. The defined stream network and physical topography of the land, as provided in the DEM, is used to define the catchment polygons. Although the computer-generated catchments do not coincide with the specific area of interest for the study watershed(s), the catchment dataset is used to determine the break points for later watershed delineation.

3.4 Creation of the Curve Number Grid

The 2001 Land Cover dataset and the Soils dataset were used to create the curve number grid. The 2001 Land Cover was converted to vector data so it could be combined with the soils. The Soils dataset, when downloaded, has a soils database, which includes the hydrologic soil group classification, among other information. A table of curve numbers for different land cover types and hydrologic soil groups was created and used in the creation of the curve number grid. The curve numbers used in this analysis, listed in Table 3.2, were obtained from the United States Department of Agriculture's (USDA) Natural Resources Conservation Service (NRCS) Technical Release 55 (1986) and Viessman and Lewis (2003).

The manipulated Land Cover and Soils layers were used to create a curve number grid with a 100-meter cell size. The initial resolution of the Land Cover dataset was approximately 30 meters, which means that the raster grid was created with square grid cells 30 meters long on each side. The 30 meter DEM was re-sampled to increase the cell size to 100-meters. In reducing the cell density, computational time for the GIS procedures was reduced to a manageable amount of time.

Table 3.2. Curve Numbers.

Land Cover/Land Use	Hydrologic Soil Group			
	A	B	C	D
Open Water	100	100	100	100
Developed, Open Space	39	61	74	80
Developed, Low Intensity	57	72	81	86
Developed, Medium Intensity	77	85	90	92
Developed, High Intensity	98	98	98	98
Barren Land, Rock, Sand, Clay	63	77	85	88
Deciduous Forest	36	60	73	79
Evergreen Forest	36	60	73	79
Mixed Forest	36	60	73	79
Scrub/Shrub	35	56	70	77
Grasslands, Herbaceous	39	61	74	80
Pasture, Hay	49	69	79	84
Cultivated Crops	67	78	85	89
Woody Wetlands	100	100	100	100
Emergent Herbaceous Wetlands	100	100	100	100

3.5 Watershed Processing

Each watershed was delineated using Batch Watershed Delineation, an Arc Hydro Tools procedure. Watershed outlet locations were obtained from Asquith, et al. (2004) and are listed in Table 3.1. To include the outlet location in the analysis, the outlet was created as *X, Y* data and then re-projected to UTM Zone 14 North, NAD 83. From this outlet location, the watershed was delineated. Problems occurred in the delineation of Little Pond Elm Creek based on the actual watershed outlet location. To solve this problem the outlet for the Little Pond Elm Creek watershed was relocated by a fraction of an arc second (less than 30 meters or 0.02 miles). This required shift did not affect the delineated area compared with the area available in Asquith, et al. (2004). Reasons for this problem occurrence could be attributed to the location of the outlet relative to the terrain or to possible artifacts in the DEM that prevented delineation from occurring.

After the watershed was delineated, it was subdivided to create six subdivision schemes: one (no subdivision), two, three, five, seven, and nine sub-basins. The subdivision was based on approximately equal area of the sub-basins. Sub-basin areas for each watershed as well as figures showing the subdivision schematics are given in Appendix A. Sub-basin outlets were determined by the user and added as a batch point. To determine the initial location of the sub-basin outlets, the Flow Accumulation grid was queried based on the desired sub-basin area, using Equation 3.1,

$$\textit{Total Upstream Area} = \textit{Cell Value X (Cell Size}^{2}) \tag{3.1}$$

The calculated cell value, when queried, would then provide a starting location for sub-basin delineation. Batch Subwatershed Delineation, in Arc Hydro Tools, was used to delineate the sub-basins. Based on the delineated versus desired area, the sub-basin outlet location was manually adjusted and re-delineated. This trial and error process was repeated for all subdivision schemes to obtain sub-basin areas within 20 percent of the desired sub-basin equal area.

3.6 Application of HEC-GeoHMS

HEC-GeoHMS (Hydrologic Engineering Center, 2003) is an ArcGIS extension created by the Army Corps of Engineer's Hydrologic Engineering Center used to visualize, create, and analyze hydrologic models. The general HEC-

GeoHMS procedure was obtained from *Introduction to HEC-GeoHMS* (Atkinson, 2008). Atkinson's procedure was not followed precisely because of differences between the lumped procedure explained in his text and the distributed modeling performed in this analysis. The HEC-GeoHMS User's Manual was consulted; however, its utility was limited because the software appears to be more developed than presented in the document.

A GeoHMS project is started by the analyst defining area, point, and project names, project description, stream extraction method, and metadata. In this analysis, the stream threshold value used in Arc Hydro Tools Stream Definition was also used in GeoHMS. The datasets required for the GeoHMS analysis were those produced during steps outlined in previous sections of this report:

- Fill Sinks
- Flow Direction
- Flow Accumulation
- Stream Definition
- Stream Segmentation
- Catchment Grid Delineation
- Catchment Polygon Processing
- Drainage Line Processing
- Adjoint Catchment Processing
- Slope Grid
- Curve Number Grid

A project point is used to define the outlet of the watershed in GeoHMS. The GeoHMS project point was placed at the outlet location defined in the Batch Watershed Delineation (the watershed outlet). The GeoHMS project was then generated based on the datasets created during the GIS processing of the layers documented previously and the defined project point. The GeoHMS project created a new ArcGIS data frame in which the project was created. The required datasets were then copied to the newly-created data frame and the GeoHMS project was ready for additional analyses, as documented below.

1. The Basin Processing tools were used to change the predefined sub-basins and river segments that were created during GeoHMS project generation. Basin Merge combined all selected sub-basins together into one basin. This step was necessary to create the user-defined sub-basins of approximately equal area. The user-defined sub-basin outlets were imported into the GeoHMS project and delineated to re-subdivide the watershed. Numerous errors arose during Delineate Batch Points in the GeoHMS project, which resulted in failed sub-basin delineation. A trial and error process was used to make minor adjustments to the sub-basin outlet location to achieve the target sub-basin delineation. The minor adjustments to the outlet locations produced negligible differences in the sub-basin area from that created using Arc Hydro Tools.
2. The Basin Characteristics processes were used to extract the physical characteristics of the watershed/sub-basin area. Prior to continuing analysis, the z-unit was set to meters in both the DEM and FILL projection files. This step was required because elevation calculations require the unit of measurement used in the z-direction to be known. In this analysis, the z unit of measurement was the same as the x and y unit of measurement. The slope and curve number grids created in Arc Hydro Tools were copied into the GeoHMS project, because they are not part of the required layers but were necessary in this analysis. The specific basin characteristics were obtained for each sub-basin in the watershed. The river length and slope, basin slope, and longest flow path from the hydraulically most distance point on the watershed/sub-basin to the outlet were calculated. The basin centroid was computed as the midpoint of the longest flow path. The centroid elevation was then computed as well as the distance from the sub-basin centroid to the sub-basin outlet for each sub-area.

3. The Hydrologic Parameters procedure was used to define parameters used in the HEC-HMS model. The HMS processes, used to define the procedures used in the loss-model, transform-model, and the routing routine, were defined and stored in the respective sub-basin or river layer. The loss model, used to determine the hydrologic abstractions, was Gridded SCS (applying the gridded curve number created in a previous step). The transform model, which converts the runoff to the watershed hydrograph, was the Modified Clark (ModClark) method (Hydrologic Engineering Center, 2008). The ModClark method is a quasi-distributed unit hydrograph method implemented in HEC-HMS using distributed precipitation data (Hydrologic Engineering Center, 2008). The routing method, which moves or translates runoff from an upstream outlet to a downstream outlet, used was the lag method (Hydrologic Engineering Center, 2008). No base flow was included in this analysis. Names for sub-basins and river segments were generated by GeoHMS using the following convention, where the Hydro ID is assigned in ArcGIS and should remain unchanged through the analysis:
    - Sub-Basin: W + Hydro ID X 10
    - River Segment: R + Hydro ID X 10
4. The ModClark Processing procedure was used to create a Standard Hydrologic Grid (SHG) of polygon grid cells with a 100-meter cell size. The Sub-basin Parameters routine was used to calculate the curve number, using the curve number grid, for each cell in the ModClark grid. After completion of the Sub-basin Parameter calculation, corrections were made to null and zero value curve numbers, which were changed to a curve number of 79, the median value of the curve numbers (Table 3.2). Some values of the flow length generated using GeoHMS were incorrect (values of -9.999). These were changed to values similar to values assigned to adjacent cells. The reason for these two processing errors is unknown. However correction of the erroneous values allowed processing using GeoHMS to be completed.

5. The HMS procedure was used to prepare the data created in ArcGIS to be exported to HEC-HMS. The process included a conversion from map units (ArcGIS) to units appropriate for HEC-HMS. English units were used in the HEC-HMS models. A data check was performed to verify the uniqueness and validity of the data. This data check will identify any problems in the datasets created through the GeoHMS analysis, prior to creation of the HEC-HMS input files. One error was found in the Ash Creek nine sub-basin model that prevented creation of one of the sub-basins. However, the parameters for this basin were calculated. To correct this error, the missing basin was created in HEC-HMS. The HMS schematic was created, representing the movement of runoff through the watershed system. The schematic uses a node-link system in which the nodes represent the sub-basins and outlets (junctions) and the links represent the river segments. Coordinates indicating the position of each element and their respective elevations were calculated and assigned to each component of the system.
6. Datasets created using GeoHMS are used to export the data into HEC-HMS input files. A background map was created to display the watershed, sub-basins, and river segments used in the model. The grid cell parameter file, which is required for distributed modeling, was created. The grid cell parameter file indicates every cell in each sub-basin and gives the specific coordinates, cell travel length, and cell area. The basin file created included the node-link system and HMS processes that were specified during the analysis. To create the curve number file for HEC-HMS, a number of steps were required. First, the ModClark Intersect, after the Sub-basin Parameters routine was completed and corrected, was re-projected to Albers Equal Area Conic (USGS Version). Second, the re-projected ModClark Intersect was used to create an ASCII grid of all curve numbers in the grid. Third, the ASCII grid was converted to a Digital Storage System (DSS) file. HEC created the DSS file type to store data used in HEC products (HEC-HMS,

HEC-RAS, etc.). Through a trial and error process, the following parameters were required to be specified as shown below[3].

- A-SHG (Projection Type)
- B-SouthMesquite (Watershed Name)
- C-Curve Number
- Data Type-UNDEF
- Data Type-INST-VAL

HMS project setup was completed with creation of the HMS project file. All required project files were copied to the HEC-HMS project folder.

The completion of the GeoHMS application was difficult, problem prone, and time consuming. A well-documented application of GeoHMS was not found during the review of pertinent literature. Because of lack of documentation of its application, the GeoHMS procedure used herein was discovered, in part, by trial and error. In determining the correct procedure, numerous incorrect steps were taken which required much time and additional effort to correct. In the deliberate process of reverse-engineering the correct steps to take, many steps in the process were repeated to remove problematic issues that were faced.

3.7 Precipitation Data Creation

For application of a fully-distributed hydrologic model, gridded precipitation was also required. Although there is a method included in GeoHMS to create gridded precipitation datasets, this method was not used in this research because required precipitation information for the respective study areas was not available.

Creation of the gridded precipitation data was done by personnel located in the Austin, Texas Water Science Center. GageInterp, a program created by Army

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[3] Hydrologic Engineering Center presentation by Matt Fleming, received from Dr. David Thompson on July 3, 2008.

Corps of Engineers personnel, was used in conjunction with additional computer coding to create the gridded precipitation datasets (Hydrologic Engineering Center, 2006). The gridded precipitation dataset was created using the gage data available in Asquith, et al. (2004) for each of the three study areas. The precipitation datasets were created using a Standard Hydrologic Grid (SHG) with a 100-meter cell size. These datasets were created in concurrence with this analysis to ensure alignment between all grids used in the HEC-HMS modeling process.

3.8 Application of HEC-HMS

HEC-HMS is the Hydrologic Modeling System created by the Army Corps of Engineer's Hydrologic Engineering Center. HEC-HMS is used to predict rainfall-runoff watershed response. The distributed modeling approach in HEC-HMS requires four major components: Basin Model, Meteorologic Model, Control Specification, and Gridded Data.

The Basin Model is the physical representation of the watershed, the sub-basins, outlets, and river segments. This representation is produced using a node-link system shown in Figure 3.4. The nodes represent the sub-basins and the sub-basin outlets. The links represent the river segment (reaches) through which upstream runoff is routed to downstream outlets.

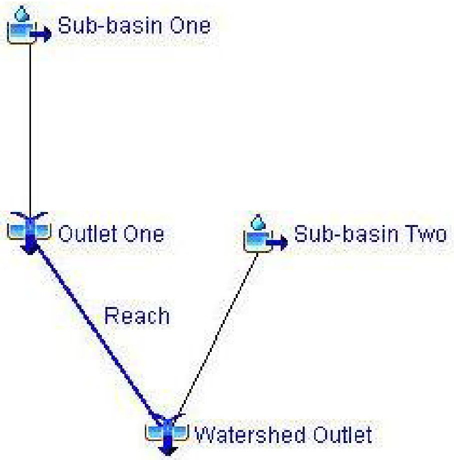

Figure 3.4. HEC-HMS Node-Link System.

Loss and Transform models are specified for each sub-basin. The Gridded SCS[4] Curve Number was used for the loss model, representing the hydrologic abstractions (Hydrologic Engineering Center, 2008). The initial abstraction (I_a) and maximum potential retention (S) needed to be determined. Standard curve numbers (NRCS tabulated values) for different land cover and hydrologic soil groups were derived assuming that $I_a = 0.2S$, that is the initial abstraction is one-fifth the maximum potential retention. Although these two values could be changed, the analyst decided that they should remain 0.2 and 1.0, respectively, to maintain consistency with curve numbers used in the analysis (Conservation Engineering Division, 1986).

The Modified Clark (ModClark) method was used as the transform model, representing the transformation of excess precipitation to surface runoff. The ModClark method is a quasi-distributed unit hydrograph method that uses gridded precipitation data. The ModClark method uses a scaled time index which relates the travel time for each grid cell in the watershed to the overall time of concentration.

The runoff generated from each grid cell is lagged by the time index and routed through a linear reservoir, where the grid cell runoff values are combined to create the final watershed hydrograph. (Hydrologic Engineering Center, 2008). Two parameters are required for the ModClark transform method: time of concentration and storage coefficient. The time of concentration (t_c) represents the travel time for the hydraulically most distant particle of water on the watershed/sub-basin to reach the watershed/sub-basin outlet. Time of concentration includes channelized and overland flow travel time. In this analysis, two methods were used to calculate the time of concentration: Kirpich Only and Kirpich plus 30 minutes (Roussel et al., 2005). The Kirpich method is used to calculate the travel time for runoff, mainly channel flow, on small watersheds. Kirpich plus 30 minutes represents both channelized and overland flow (the 30 minutes was used to account for overland

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[4] HEC continues to use SCS (Soil Conservation Service), however, the name has changed to NRCS (Natural Resources Conservation Service).

flow). For the analysis and results reported herein, the Kirpich Only method will be discussed. The storage coefficient (*R*) represents the lag attributable to natural storage effects (Kull and Feldman, 1998). It was estimated using Equation 3.2 related to Figure 3.5, where *Q* is the flow rate and *S* is the slope at the point of inflection on the falling limb of the hydrograph. The storage coefficient was estimated using the hydrographs for the five largest storm events based on peak discharge. The median *R*, listed in Table 7 of Appendix B, for each watershed was used for all subsequent HMS models.

$$R \cong \frac{-Q}{S} \qquad (3.2)$$

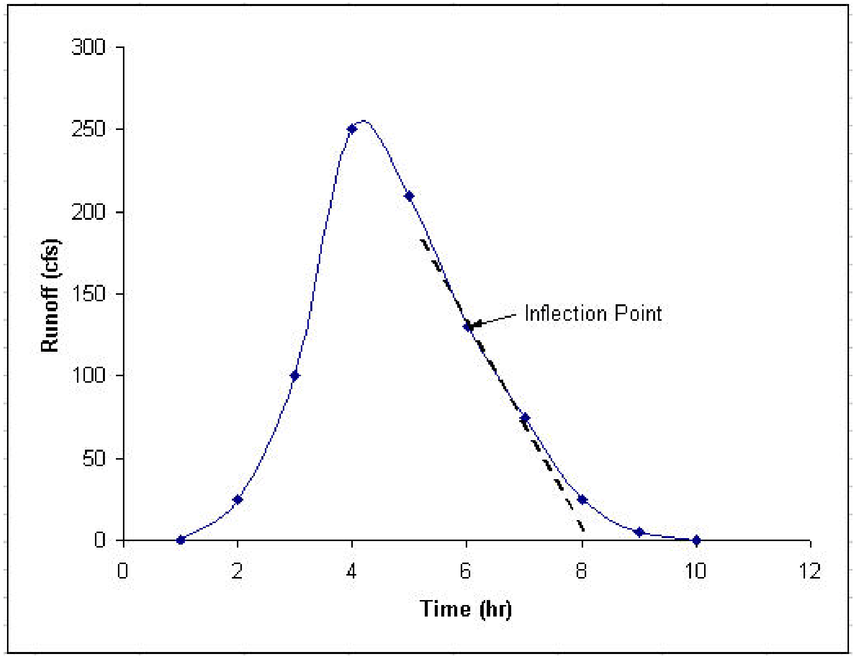


Figure 3.5. Graphical Determination of Storage Coefficient.

The routing procedure is used to move the discharge hydrograph from one sub-basin through the channel network to the outlet of the next sub-basin downstream (Hydrologic Engineering Center, 2008). For this application of HEC-HMS, the lag routing method was used. The lag routing method results when the outflow hydrograph is translated from the channel inlet to the outlet without attenuation. This minimal attenuation was a valid assumption because of the relatively short reach lengths and the inability of most streams to effect significant attenuation on the runoff hydrograph. The lag time was estimated using the Kirpich equation, Equation 3.3 (Roussel et al., 2005). In Equation 3.3, t_c is the time of concentration in minutes, L is the travel length in feet, and S is the slope in feet/feet. For the calculation of routing time, the travel length was the river reach length and the slope was estimated using the outlet elevations and river reach length. Because of the errors found in the slope calculations in HEC-GeoHMS, the outlet elevations were queried in the original DEM.

$$t_c = 0.0078L^{0.77}S^{-0.385} \tag{3.3}$$

The Meteorologic Model was used to define the meteorologic methods that will be used to calculate precipitation, evapo-transpiration, and snowmelt (Hydrologic Engineering Center, 2008). Each modeled storm was created in the Meteorologic Model and assigned a gridded precipitation method used to provide the precipitation datasets created by USGS to the HMS model. Evapo-transpiration and snowmelt were not included in this analysis because the requisite data were not available for the watersheds studied in this research.

Control Specifications were created to define the simulation run time. The beginning and ending times, plus the computational time interval, define the Control Specifications for a HEC-HMS model run (Hydrologic Engineering Center, 2008). A separate Control Specification was created for each storm event. The time interval used for the files containing the observed precipitation measurements was five minutes. This value was used as the basic computational time interval for HEC-HMS modeling.

The Grid Data Manager is a component of HEC-HMS which is required for distributed modeling. The Grid Data Manager is used to store all gridded datasets for the hydrologic model. Because HEC-HMS does not include the ability to internally create gridded parameter datasets; the gridded datasets must be created external to

HEC-HMS and stored in a Data Storage System (DSS) file (Hydrologic Engineering Center, 2008). Two gridded datasets were created for this analysis: the curve number grid and the precipitation grid. The curve number grid was created during pre-processing of the project datasets using Arc Hydro Tools and HEC-GeoHMS in ArcGIS. The gridded precipitation dataset was created by USGS personnel using tools developed specifically for this purpose. The gridded precipitation was created using a five-minute time interval. Because of the precipitation datasets being created by USGS personnel, it was not possible for the analyst to revise the datasets. When revisions to the data were required, these changes were made by USGS personnel and a new dataset was created. USGS personnel used software not available to the general public. Therefore, use of gridded precipitation datasets developed from point rain gage data remains an issue.

Six HEC-HMS models were created for each watershed relative to the subdivision scheme being modeled. The HEC-HMS models were uncalibrated. Procedures for calibrating model parameters exist in HEC-HMS (Hydrologic Engineering Center, 2008), but were not used for the research project reported herein. The objective of this research was to assess the utility of distributed modeling as available in HEC-HMS. To completely assess the distributed modeling approach, parameter values were used as they were obtained from ArcGIS. Calculations and modeled results were then able to assess the complete application of the distributed modeling process without having to quantify the effects of parameter modification as would occur in a calibrated model.

CHAPTER 4

RESULTS

The uncalibrated models were operated and analysis performed on the results to determine the effect of the fully-distributed parameter sets and additional watershed subdivision on the outflow hydrographs.

4.1 Runoff Hydrographs

For each watershed subdivision scheme, the outflow hydrograph at the watershed outlet and the observed hydrograph were plotted on the same set of coordinate axes for the suite of storm events used in this study. Some hydrographs are presented in the following discussion; the remaining hydrographs are presented in Appendix C.

4.1.1 South Mesquite Creek

Six storm events from the South Mesquite Creek watershed were modeled. The runoff hydrograph for the April 18, 1976 storm event is presented in Figure 4.1.

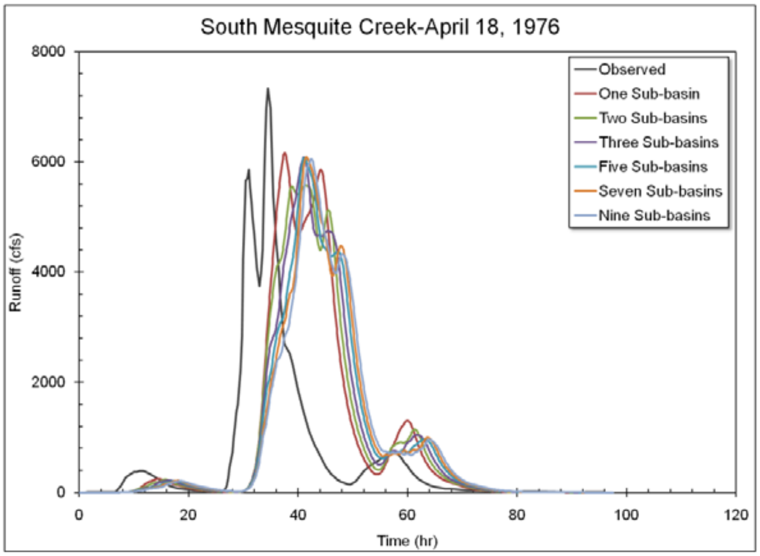


Figure 4.1. Runoff hydrographs from South Mesquite Creek watershed for the event of April 18, 1976.

The modeled peak discharge was significantly less than the observed peak discharge. Also, the modeled time to peak discharge was greater than the observed time to peak discharge. The noticeable differences in peak discharge and time to peak could be attributed to the uncalibrated nature of the models, discussed briefly at the end of the previous section, or to the additional conservatism inherent in the sub-basin routing method. The lag time used in the models was derived from slope and river lengths directly from ArcGIS. However, in a calibrated model the lag time could be adjusted, possibly minimizing the timing difference between observed and modeled hydrographs. Another noticeable occurrence in the modeled hydrograph is an increased number of peaks as the number of subdivisions increased. This wavy appearance, seen throughout the analysis, is most explainable by the introduction of sub-basins and the required routing of sub-basin outflows toward the watershed outlet.

The hydrograph for the storm event of October 17, 1971 is depicted in Figure 4.2.

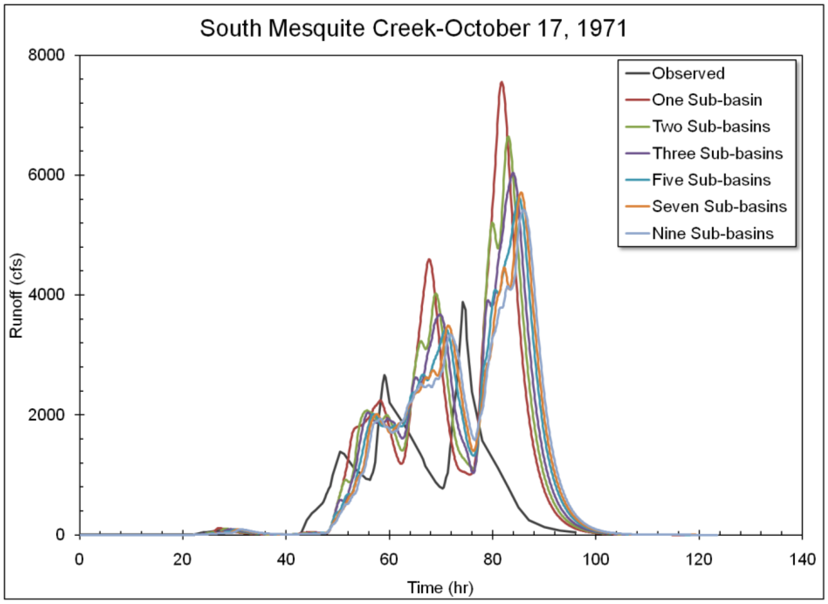


Figure 4.2. Runoff hydrographs from South Mesquite Creek watershed for the event of October 17, 1971.

For the storm event of October 17, 1971 the observed runoff is substantially less than the modeled runoff. This difference could be attributed to the precipitation datasets. The precipitation grids were interpolated based on the recorded precipitation at rain gages within the watershed. The specific location of the rain gages effects the areal distribution of the interpolated precipitation grid which would directly affect the runoff produced.

4.1.2 Ash Creek

The Ash Creek watershed was modeled for five storm events. The runoff hydrograph for June 3, 1973 is shown in Figure 4.3.

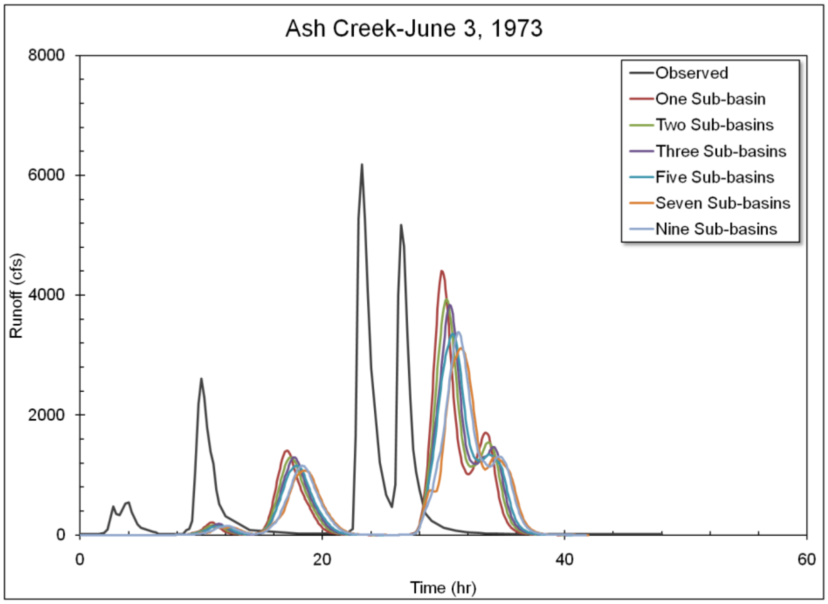


Figure 4.3. Runoff hydrographs from Ash Creek watershed for the event of June 3, 1973.

The hydrographs from the event of June 3, 1973, displayed in Figure 4.3, have a similar shape as the observed hydrograph. However, as seen in the South Mesquite Creek hydrographs, there is a noticeable decrease in modeled peak discharge and increase in modeled time to peak relative to the observed data. This, again, could be attributed to the uncalibrated models as well as the conservative travel time because of sub-basin routing routines.

4.1.3 Little Pond Elm Creek

The Little Pond Elm Creek watershed was modeled for six storm events. Figure 4.4 is the runoff hydrograph for May 16, 1965. The observed runoff hydrograph was estimated from incremental runoff values which caused the "blocky" hydrograph. The modeled hydrographs display the same pattern of lesser peak discharge and longer time to peak relative to the observed data as seen with South Mesquite Creek and Ash Creek.

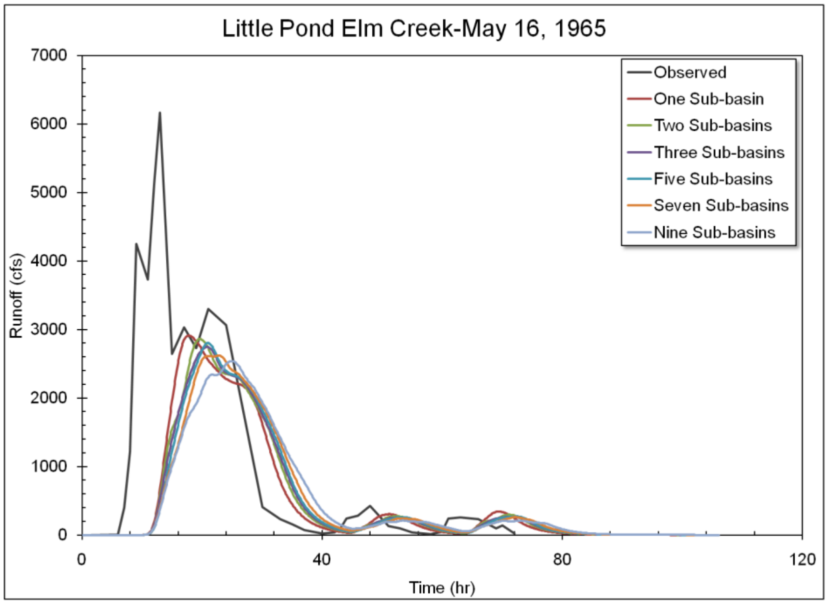


Figure 4.4. Runoff hydrographs from Little Pond Elm Creek watershed for the event of May 16, 1965.

4.2 Analysis Metrics

Three variables were used to assess the effects of parameter distribution and subdivision on the outflow results: time to peak, peak discharge, and runoff volume. Four metrics were used to compare the results. The relative error, Equation 4.1, was used to compute the error between modeled and observed data, where X_o is the modeled value and X is the observed value.

$$R_e = \frac{X_o - X}{X} \tag{4.1}$$

The arithmetic mean, Equation 4.2, was used to obtain the average relative error, where N is the number of parameters, in this case relative errors.

$$\overline{R_e} = \frac{1}{N} \sum R_e \tag{4.2}$$

The Relative Root Mean Square Error, Equation 4.3, was used to estimate the variation of the relative error as related to the arithmetic mean.

$$RRMSE = \sqrt{\frac{1}{N} \sum (R_e)^2} \tag{4.3}$$

A minimum count was then performed on each parameter of interest to determine the number of times (relative to the number of storms modeled) each subdivision scheme performed the “best” or produced the least error as compared to the other subdivision schemes.

4.2.1 Time to Peak

The time to peak represents the number of hours for the peak discharge to occur at the outlet. Using the metrics stated above, the results presented in Table 4.1 were developed.

Table 4.1. Time to peak assessed using metrics for South Mesquite Creek, Ash Creek, and Little Pond Elm Creek watersheds.

Watershed	Time to Peak	Number of Subdivisions						Best Performing Subdivision Scheme
		1	2	3	5	7	9	
South Mesquite Creek	Arith. Mean (%)	21%	25%	27%	30%	31%	33%	1
	RRMSE (%)	23%	27%	29%	32%	34%	35%	1
	Min. Count	5	1	0	0	0	0	1
Ash Creek	Arith. Mean (%)	51%	53%	56%	56%	60%	60%	1
	RRMSE (%)	64%	65%	69%	67%	74%	74%	1
	Min. Count	5	0	0	0	0	0	1
Little Pond Elm Creek	Arith. Mean (%)	64%	79%	83%	90%	92%	99%	1
	RRMSE (%)	70%	86%	89%	98%	98%	106%	1
	Min. Count	6	0	0	0	0	0	1

In general, the best performing subdivision scheme was one sub-basin. For South Mesquite Creek, the two sub-basin scheme demonstrated the least difference between observed and modeled time to peak as measured using relative root mean square error and minimum count. The lesser error relative to the observed time to peak for the one-basin model is based on the conservative travel time inherent in the multiple sub-basin models. As the number of sub-basins increased, the travel time across the watershed was adjusted to account for travel delays because of storage as well as the lag time to translate the upstream hydrograph to downstream outlets. The routing time required of multi-basin models created a conservative estimate of the travel time. How conservative the timing parameters were could be determined using a sensitivity analysis.

The graphs of time to peak errors relative to the arithmetic mean are shown in Figure 4.5. Excluding the five subdivision scheme for Ash Creek, all three watersheds exhibited a trend of increasing relative error with an increasing number of subdivisions. The lower relative error for Ash Creek's five sub-basin model could be because of two of the five sub-basins being routed directly to the watershed outlet.

The translation of the hydrograph directly to the watershed outlet instead of through another sub-basin outlet could have minimized the overall time to peak.

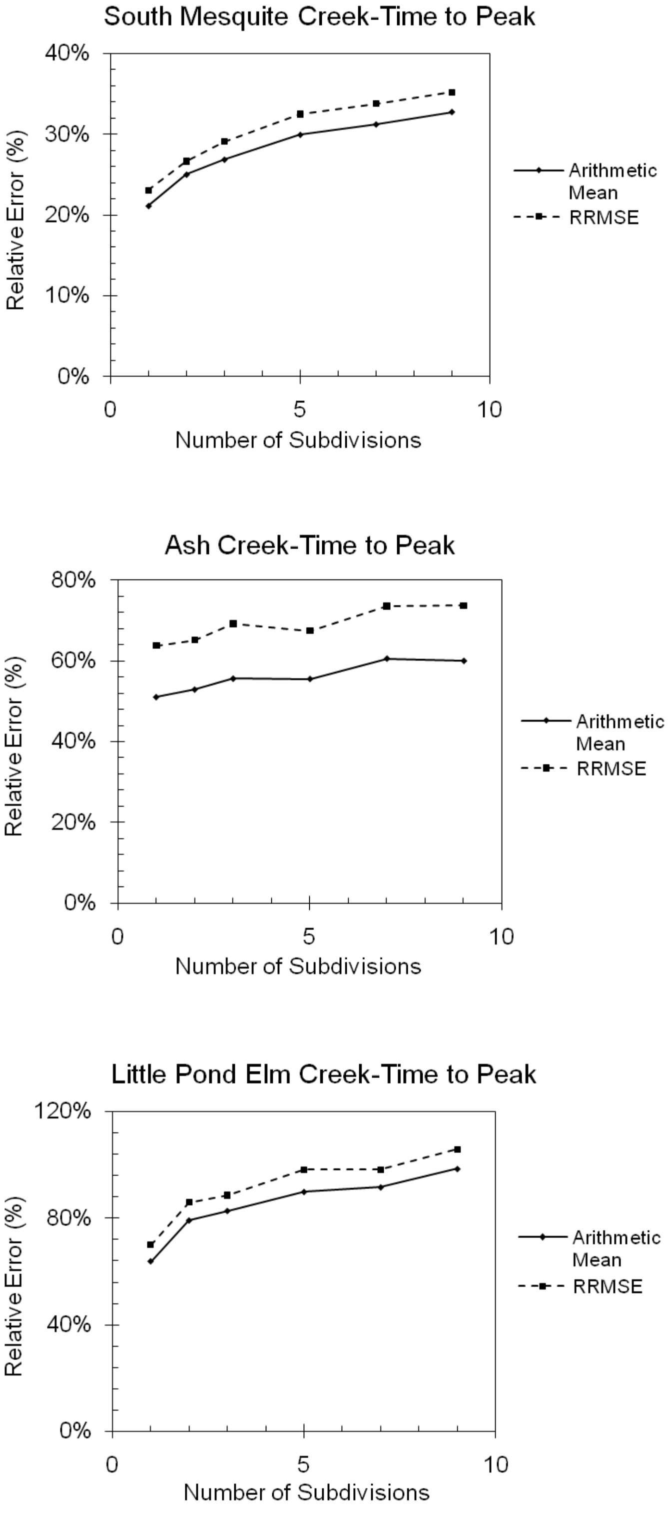


Figure 4.5. Time to Peak metrics for South Mesquite Creek, Ash Creek, and Little Pond Elm Creek watersheds.

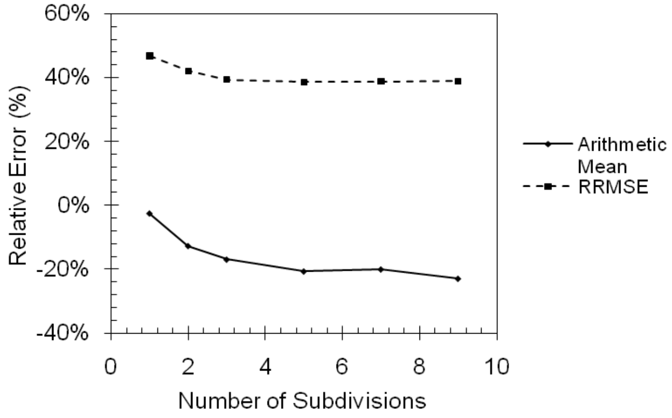
4.2.2 Peak Discharge

Values of the relative error for the modeled peak discharge compared to the observed peak discharge are listed in Table 4.2 and depicted in Figure 4.6. Ash Creek and Little Pond Elm Creek have increasingly greater error as related to the observed with an increase in subdivision. The South Mesquite Creek watershed, however, experienced a decrease in relative error up to three subdivisions. An increase in subdivision above three sub-basins caused negligible changes in the model accuracy. The peak discharge is directly related to the precipitation and curve number. The lesser error produced in the South Mesquite Creek watershed models lead the analyst to conclude that the actual watershed response, with specific regard to the meteorology and loss models, was more closely modeled than that of Ash Creek or Little Pond Elm Creek. Once again, in general the one sub-basin model produced the least relative error and would be considered the best performing subdivision scheme.

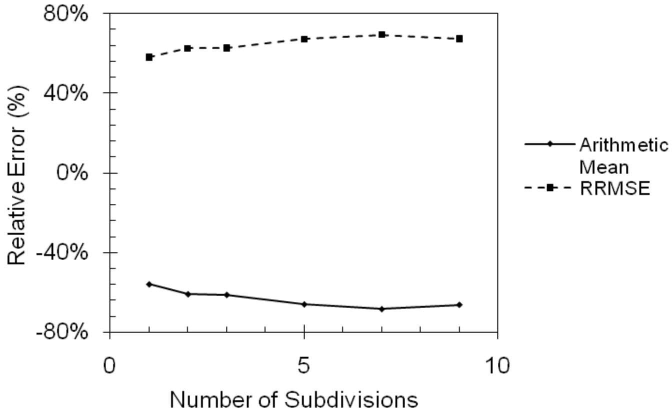
Table 4.2. Peak Discharge assessed using metrics for South Mesquite Creek, Ash Creek, and Little Pond Elm Creek watersheds.

Watershed	Peak Discharge	Number of Subdivisions						Best Performing Subdivision Scheme
		1	2	3	5	7	9	
South Mesquite Creek	Arith. Mean (%)	-3%	-13%	-17%	-21%	-20%	-23%	1
	RRMSE (%)	47%	42%	39%	39%	39%	39%	3
	Min. Count	4	1	0	0	0	1	1
Ash Creek	Arith. Mean (%)	-56%	-61%	-61%	-66%	-68%	-66%	1
	RRMSE (%)	58%	63%	63%	67%	69%	67%	1
	Min. Count	5	0	0	0	0	0	1
Little Pond Elm Creek	Arith. Mean (%)	-58%	-62%	-64%	-64%	-66%	-70%	1
	RRMSE (%)	59%	62%	65%	64%	66%	70%	1
	Min. Count	6	0	0	0	0	0	1

South Mesquite Creek-Peak Discharge



Ash Creek-Peak Discharge



Little Pond Elm Creek-Peak Discharge

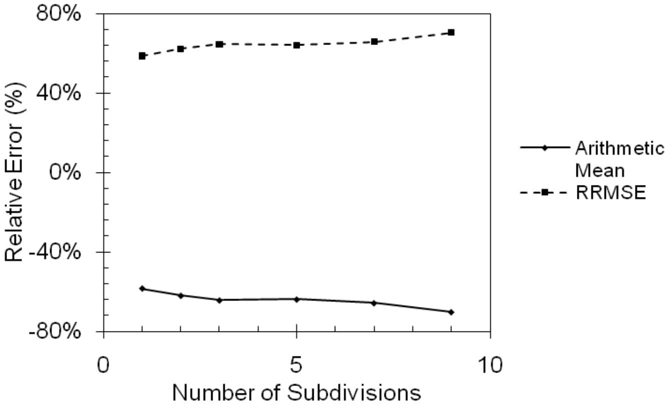


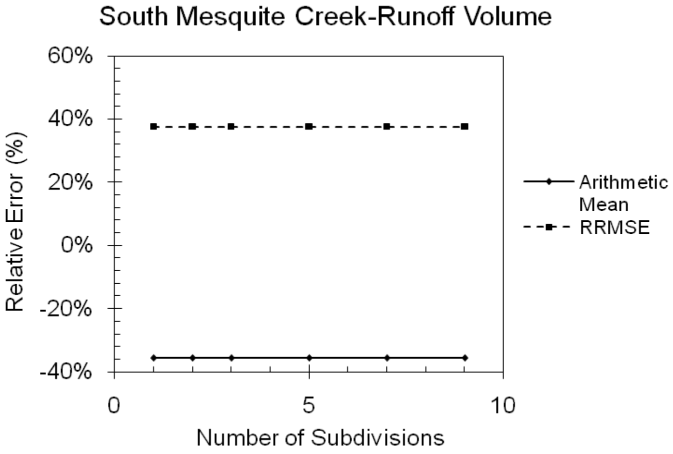
Figure 4.6. Peak Discharge metrics for South Mesquite Creek, Ash Creek, and Little Pond Elm Creek watersheds.

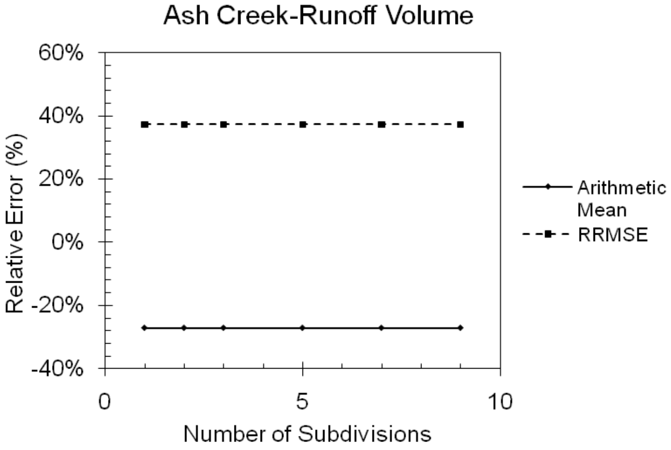
4.2.3 Runoff Volume

Values of the relative error between modeled and observed runoff volume over the watershed area are listed in Table 4.3 and depicted in Figure 4.7. As shown on both Table 4.3 and Figure 4.7, the arithmetic mean and relative error were unvarying. The unchanged error is because of the constant curve number grid used in the model. The runoff volume is directly related to the curve number, which determines the amount of precipitation that is abstracted and that which will turn into runoff. The curve number grid for each watershed was created in ArcGIS and remained constant for each watershed subdivision scheme. Because the curve numbers were not related to or changed based on the subdivision scheme, the runoff volume remained constant.

Table 4.3. Runoff Volume assessed using metrics for South Mesquite Creek, Ash Creek, and Little Pond Elm Creek watersheds.

Watershed	Runoff Volume	Number of Subdivisions						Best Performing Subdivision Scheme
		1	2	3	5	7	9	
South Mesquite Creek	Arith. Mean (%)	-36%	-36%	-36%	-36%	-36%	-36%	1
	RRMSE (%)	38%	38%	38%	38%	38%	38%	1
	Min. Count	-	-	-	-	-	-	-
Ash Creek	Arith. Mean (%)	-27%	-27%	-27%	-27%	-27%	-27%	1
	RRMSE (%)	37%	37%	37%	37%	37%	37%	1
	Min. Count	-	-	-	-	-	-	-
Little Pond Elm Creek	Arith. Mean (%)	-35%	-35%	-35%	-35%	-35%	-35%	1
	RRMSE (%)	41%	41%	41%	41%	41%	41%	1
	Min. Count	-	-	-	-	-	-	-





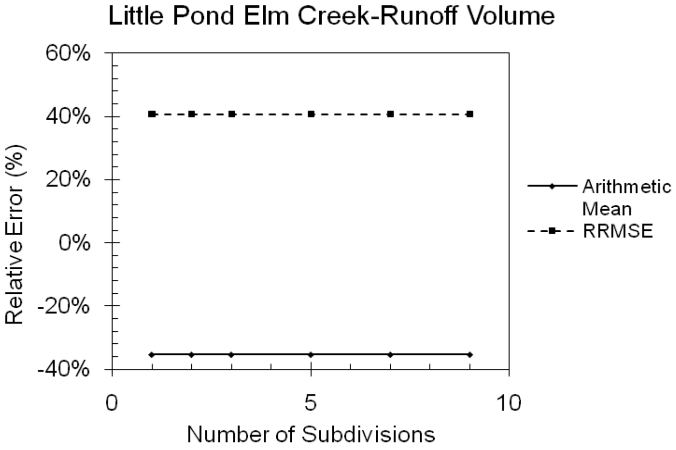


Figure 4.7. Runoff Volume metrics for South Mesquite Creek, Ash Creek, and Little Pond Elm Creek watersheds.

CHAPTER 5

SUMMARY AND CONCLUSIONS

The primary objective of this research was to assess the utility of distributed modeling using HEC-HMS. The secondary objective was to examine the impact of watershed subdivision, in a distributed-model context, on the runoff hydrograph from the watershed.

To achieve the objective a review of applicable literature was first performed in which few applications of the distributed modeling approach were reported. Three Texas watersheds were then studied using peak storm events and six subdivision schemes: one (no subdivision), two, three, five, seven, and nine sub-basins. Because of the lack of documentation, application of the distributed modeling approach using HEC-HMS was determined by reverse engineering the software used in the analysis. Numerous problems, which are discussed further in the proceeding text, were encountered. These problems easily derailed the modeling effort and required a large investment of time to correct.

Analysis of the three watersheds indicated that minimal to no improvement in the accuracy of the modeled runoff hydrographs occurred with increased subdivision. Any minor improvement in the model results were not seen to be significant enough to justify the increased effort to use a distributed model approach.

The application of distributed modeling using Arc Hydro Tools and HEC-GeoHMS for HEC-HMS file creation was not a simple process. The difficulties in applying the distributed-modeling approach implemented in HEC-HMS are attributable to many sources.

1. The distributed modeling approach using HEC-HMS is not well documented, making it difficult to overcome the problems that were encountered.

2. Application of the HEC-GeoHMS procedure is complex and problem-prone. This issue might be less of a problem if a useful User's Manual existed for HEC-GeoHMS. However, the available User's Manual from HEC is dated 2003 and details for application of the current version of HEC-GeoHMS are not available.
3. Because the gridded precipitation datasets were created outside of the analyst's control, coordination of numerous GIS map document properties were required to ensure proper alignment of all gridded datasets. If the precipitation data could be created under the same GIS umbrella with the other gridded datasets this problem might be alleviated. For the research reported herein, that was not possible, which led to initial problems in the creation of the gridded precipitation.
4. Corrections were required in the ModClark Intersect, the gridded layer of parameter values, after parameter calculations were performed. Although not major, these corrections could be problematic in a different analysis.
5. File type conversion to DSS files was difficult. The process to convert the files is seemingly simple. However, very specific projections and parameters must be specified for the file conversion to occur. This process is not clearly documented in the GeoHMS User's Manual.

The application of Arc Hydro Tools and HEC-GeoHMS using ArcGIS to create fully distributed hydrologic models was difficult and time consuming. The lack of documentation and lack of current software User's Manuals make the possible widespread use of distributed modeling to be unlikely in the near future.

Hydrographs from the subdivided watershed models were not substantially different when compared to hydrographs from less subdivided watershed models. This lack of improvement in the outlet hydrograph could be attributed to the parameter "assumptions" that must be made in a distributed model. The curve number was calculated for every 100-meter grid cell in the watershed. Realistically, the curve number for such a small area would not be known. The precipitation grid was created by interpolating a precipitation surface based on the precipitation at point locations within the watershed. The location of these points or even the number of known points used is unknown to the analyst. Because of uncertainties associated with the interpolated curve number and precipitation datasets, and whether these datasets adequately represent the watershed response, there is inherent uncertainty in the distributed parameters used in the models. This uncertainty can lead to compiling systematic error in the outlet hydrograph, reducing the accuracy of the modeled results.

Additional research could be invested into further assessment of the distributed modeling approach. A few suggestions for future research are:

1. The curve numbers used in the analysis could be varied. As stated in the literature review, previous researchers report that the curve number has a large impact on the watershed outlet results. This observation makes sense because of the nature of the curve number. Variation of the curve numbers used could then be used to assess the effect of the curve number on the outflow results. To vary the curve number, different curve number tables would be needed and then a different curve number grid could be created based on the different curve number tables.
2. The cell size used in the distributed modeling analysis could be varied. In this research, a grid cell size of 100-meters was used. Numerous grid cell sizes could be used, both large and small, to fully assess the effects of distributed-modeling as it is related to cell size. The impact of grid cell size could then be assessed by comparing the modeled hydrograph for each different cell size. Varying the cell size would require creation of multiple ModClark Intersects using the various grid cell sizes of interest. The parameters (curve number and precipitation) would then be calculated for each ModClark Intersect.
3. The variation of the subdivision scheme requirements, in this research equal area, could assess the effects of grouping similar terrain, which did not occur

in the subdivision schemes used herein. Implementing the distributed parameter set with additional subdivision could be performed based on changes in slope, land cover, or land use. Sub-dividing based on similar terrain could affect the timing parameters which would then affect the outflow hydrograph.

4. The use of GIS precipitation datasets would allow for the creation of gridded precipitation simultaneously with other required HMS files.
5. The GIS analysis could be attempted using the Albers Equal Area Conic (USGS Version) Projection. The curve number grid DSS file must be created from the Albers Equal Area Conic projection. This was discovered by trial and error. Once discovered the entire analysis was attempted in the Albers Equal Area Conic projection but numerous errors were encountered. If it was possible to perform the entire GIS analysis using this projection, later file creation problems could be avoided.
6. The HEC-HMS models could be calibrated. For distributed modeling, the timing parameters (time of concentration and lag time) would be the most influential parameters that could be used in the calibration process. Calibration of the timing parameters would affect the time to peak as well as the peak discharge. Calibrating the models to known runoff hydrographs could produce models that generally represent the watershed runoff response.

The primary objective of this research was to assess the utility of distributed modeling using HEC-HMS. The secondary objective was to examine the impact of watershed subdivision, in a distributed-model context, on the runoff hydrograph from the watershed. Application of the distributed-modeling approach using Arc Hydro Tools and HEC-GeoHMS is not applicable to most watershed applications. The complexity of the method is not offset by any increased accuracy in the outflow hydrograph. Distributed modeling currently appears to be an unnecessary investment of time and effort. Furthermore, there was little improvement in estimates of the hydrograph response that could be attributed to the increase in watershed subdivision. Improvement to estimates of the runoff hydrograph by watershed subdivision is not warranted for the watersheds studied as part of this research.

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APPENDIX A

WATERSHED SUBDIVISIONS

Table A1. South Mesquite Creek Sub-basin Areas.

South Mesquite Creek			
Total Area	**23.4**	**mi^2**	
2 Equal Sub-basins Area	11.7	mi^2	
			% Difference
Sub-basin 1	10.88	mi^2	-7%
Sub-basin 2	12.52	mi^2	7%
3 Equal Sub-basins Area	7.8	mi^2	
			% Difference
Sub-basin 1	6.56	mi^2	-16%
Sub-basin 2	8.89	mi^2	14%
Sub-basin 3	7.95	mi^2	2%
5 Equal Sub-basins Area	4.7	mi^2	
			% Difference
Sub-basin 1	4.93	mi^2	5%
Sub-basin 2	3.91	mi^2	-16%
Sub-basin 3	4.68	mi^2	0%
Sub-basin 4	4.85	mi^2	4%
Sub-basin 5	5.03	mi^2	7%
7 Equal Sub-basins Area	3.3	mi^2	
			% Difference
Sub-basin 1	3.23	mi^2	-3%
Sub-basin 2	3.83	mi^2	15%
Sub-basin 3	3.34	mi^2	0%
Sub-basin 4	3.17	mi^2	-5%
Sub-basin 5	3.95	mi^2	18%
Sub-basin 6	2.87	mi^2	-14%
Sub-basin 7	3.01	mi^2	-10%
9 Equal Sub-basins Area	2.6	mi^2	
			% Difference
Sub-basin 1	2.94	mi^2	13%
Sub-basin 2	2.40	mi^2	-8%
Sub-basin 3	2.31	mi^2	-11%
Sub-basin 4	2.83	mi^2	9%
Sub-basin 5	3.03	mi^2	17%
Sub-basin 6	2.51	mi^2	-3%
Sub-basin 7	2.33	mi^2	-10%
Sub-basin 8	2.75	mi^2	6%
Sub-basin 9	2.28	mi^2	-12%

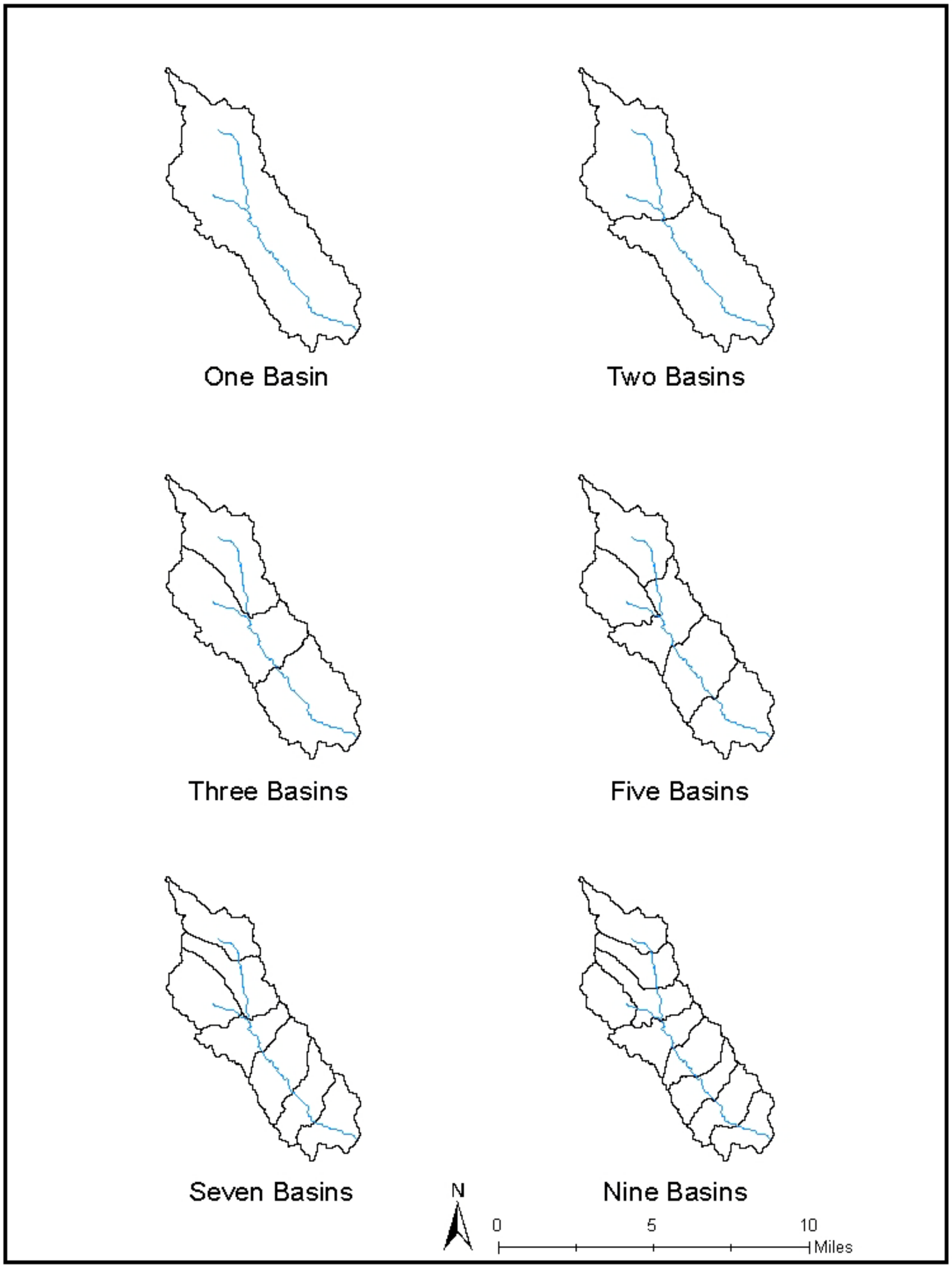

Figure A1. South Mesquite Creek Subdivision Schemes.

Table A2. Ash Creek Sub-basin Areas.

Ash Creek			
Total Area	**7.00**	**mi^2**	
2 Equal Sub-basins Area	3.50	mi^2	
			% Difference
Sub-basin 1	3.61	mi^2	3%
Sub-basin 2	3.38	mi^2	-3%
3 Equal Sub-basins Area	2.33	mi^2	
			% Difference
Sub-basin 1	2.64	mi^2	13%
Sub-basin 2	2.32	mi^2	-1%
Sub-basin 3	2.04	mi^2	-13%
5 Equal Sub-basins Area	1.40	mi^2	
			% Difference
Sub-basin 1	1.28	mi^2	-9%
Sub-basin 2	1.48	mi^2	6%
Sub-basin 3	1.53	mi^2	9%
Sub-basin 4	1.19	mi^2	-15%
Sub-basin 5	1.52	mi^2	9%
7 Equal Sub-basins Area	1.00	mi^2	
			% Difference
Sub-basin 1	0.85	mi^2	-15%
Sub-basin 2	1.03	mi^2	3%
Sub-basin 3	0.99	mi^2	-1%
Sub-basin 4	0.98	mi^2	-2%
Sub-basin 5	1.08	mi^2	8%
Sub-basin 6	0.99	mi^2	-1%
Sub-basin 7	1.08	mi^2	8%
9 Equal Sub-basins Area	0.78	mi^2	
			% Difference
Sub-basin 1	0.80	mi^2	3%
Sub-basin 2	0.70	mi^2	-10%
Sub-basin 3	0.92	mi^2	18%
Sub-basin 4	0.87	mi^2	12%
Sub-basin 5	0.73	mi^2	-6%
Sub-basin 6	0.79	mi^2	2%
Sub-basin 7	0.78	mi^2	0%
Sub-basin 8	0.65	mi^2	-16%
Sub-basin 9	0.75	mi^2	-4%

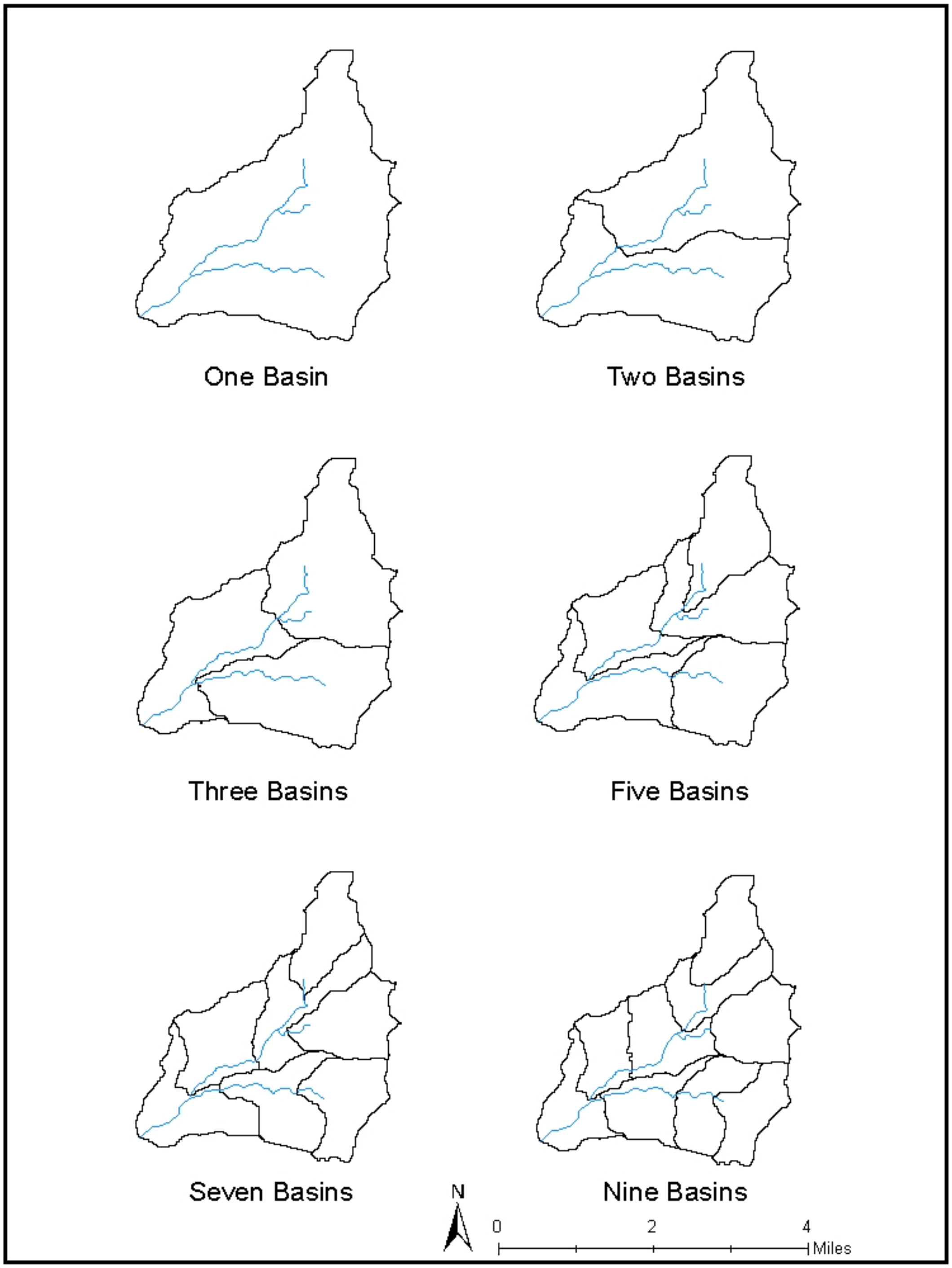

Figure A2. Ash Creek Subdivision Schemes.

Table A3. Little Pond Elm Creek Sub-basin Areas.

Little Pond Elm Creek			
Total Area	**22.80**	**mi^2**	
2 Equal Sub-basins Area	11.40	mi^2	
			% Difference
Sub-basin 1	11.59	mi^2	2%
Sub-basin 2	11.22	mi^2	-2%
3 Equal Sub-basins Area	7.60	mi^2	
			% Difference
Sub-basin 1	7.53	mi^2	-1%
Sub-basin 2	7.20	mi^2	-5%
Sub-basin 3	8.08	mi^2	6%
5 Equal Sub-basins Area	4.56	mi^2	
			% Difference
Sub-basin 1	4.70	mi^2	3%
Sub-basin 2	3.72	mi^2	-18%
Sub-basin 3	5.41	mi^2	19%
Sub-basin 4	4.02	mi^2	-12%
Sub-basin 5	4.96	mi^2	9%
7 Equal Sub-basins Area	3.26	mi^2	
			% Difference
Sub-basin 1	3.15	mi^2	-3%
Sub-basin 2	3.28	mi^2	1%
Sub-basin 3	3.26	mi^2	0%
Sub-basin 4	3.50	mi^2	7%
Sub-basin 5	3.27	mi^2	0%
Sub-basin 6	2.94	mi^2	-10%
Sub-basin 7	3.42	mi^2	5%
9 Equal Sub-basins Area	2.53	mi^2	
			% Difference
Sub-basin 1	2.52	mi^2	-1%
Sub-basin 2	2.57	mi^2	1%
Sub-basin 3	2.45	mi^2	-3%
Sub-basin 4	2.26	mi^2	-11%
Sub-basin 5	2.64	mi^2	4%
Sub-basin 6	2.80	mi^2	11%
Sub-basin 7	2.14	mi^2	-16%
Sub-basin 8	2.52	mi^2	-1%
Sub-basin 9	2.93	mi^2	16%

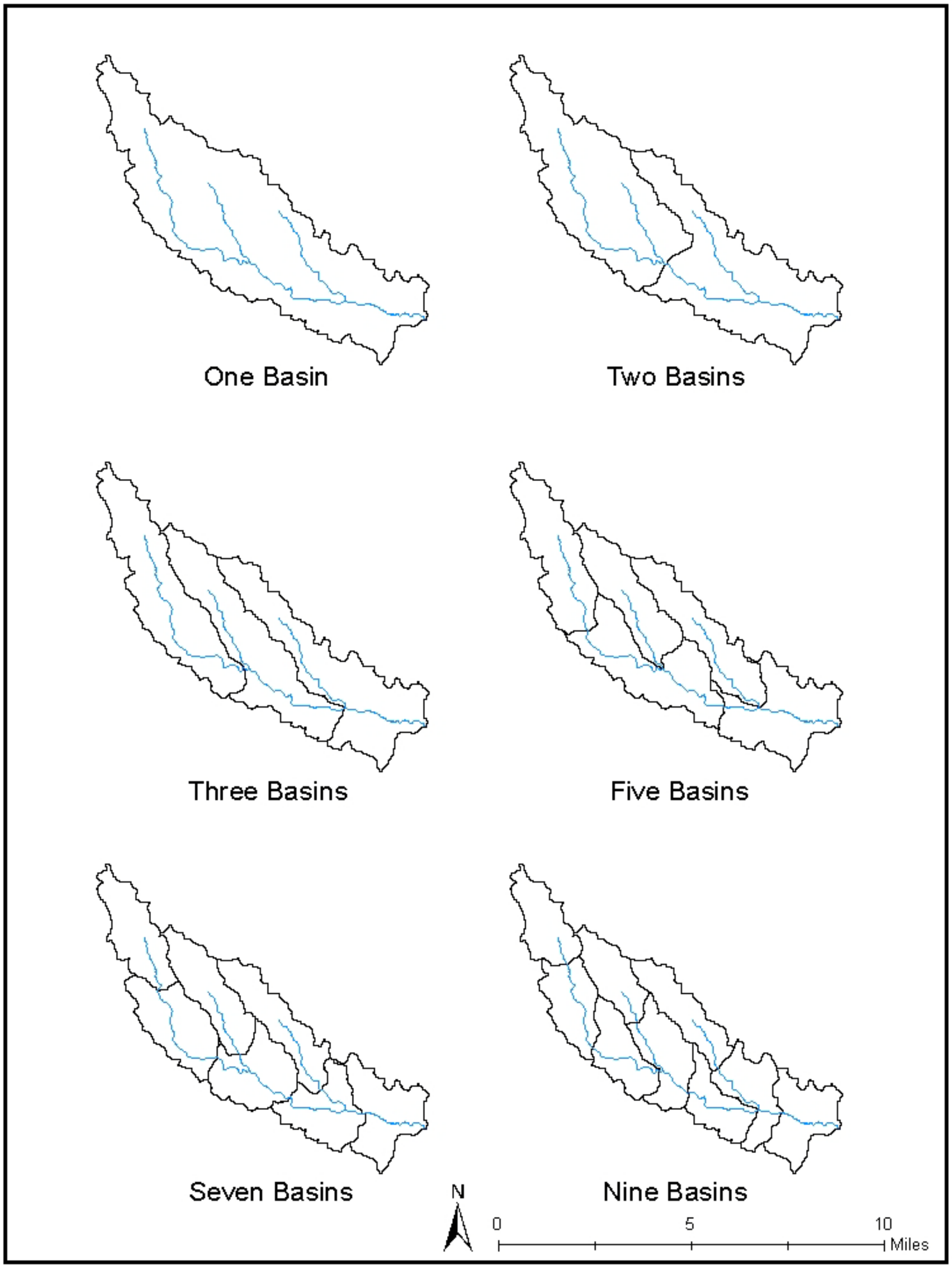

Figure A3. Little Pond Elm Creek Subdivision Schemes.

APPENDIX B

HYDROLOGIC MODEL PARAMETER CALCULATIONS

Table B1. South Mesquite Creek Watershed Transform Model Parameters.

		Channel Flow (Kirpich Method)						
Sub-basin	Area	Elevation_{US}	Elevation_{DS}	Slope	Travel Length	Travel Length	Time of Concentration t_c	Time of Concentration t_c
	mi^2	m	m	-	m	ft	min	hr
W110	23.40	181.85	118.83	0.003	20708	67941	382	6.4
W130	10.88	181.85	140.56	0.004	10689	35070	209	3.5
W120	12.52	169.69	118.83	0.004	12861	42196	239	4.0
W180	6.56	181.85	137.88	0.004	9793	32128	185	3.1
W280	8.89	174.18	130.99	0.004	9763	32032	185	3.1
W270	7.95	159.65	118.83	0.004	9131	29957	175	2.9
W130	4.93	181.85	145.67	0.005	7639	25062	149	2.5
W170	5.03	153.42	118.83	0.005	6605	21669	128	2.1
W230	3.91	174.18	139.04	0.006	5723	18776	108	1.8
W270	4.85	158.66	128.03	0.005	6514	21371	133	2.2
W280	4.68	167.56	134.21	0.005	6305	20686	123	2.1
W130	3.23	181.85	148.82	0.005	6489	21288	128	2.1
W380	3.83	174.18	140.14	0.006	5390	17685	102	1.7
W330	3.34	174.23	138.83	0.005	6871	22543	133	2.2
W370	3.17	169.69	133.96	0.008	4524	14842	82	1.4
W230	3.95	158.66	128.25	0.005	5817	19085	117	1.9
W280	2.87	159.75	124.76	0.006	5910	19388	112	1.9
W270	3.01	150.13	118.83	0.006	5654	18551	112	1.9
W130	2.94	181.85	149	0.005	6037	19806	118	2.0
W180	2.40	174.23	143.5	0.006	5031	16506	98	1.6
W230	2.31	170.75	143.24	0.006	4284	14056	85	1.4
W430	2.83	174.18	137.51	0.006	5913	19399	111	1.8
W480	3.03	169.69	134.21	0.008	4265	13993	77	1.3
W470	2.51	158.66	130.77	0.006	4572	15001	91	1.5
W330	2.33	159.75	128.02	0.007	4268	14003	80	1.3
W380	2.75	153.42	122.67	0.007	4491	14733	86	1.4
W370	2.28	155.59	118.83	0.008	4547	14919	82	1.4

Table B2. South Mesquite Creek Watershed Routing Parameters.

Routing Parameters							
Reach	From-To	Elevation$_{US}$	Elevation$_{DS}$	Slope	Travel Length	Travel Length	Travel Time
	-	m	m	-	m	ft	min
R50	1-Outlet	137.97	118.83	0.002	10048	32965	262
R30	1-2	137.88	130.99	0.002	4093	13427	138
R240	2-Outlet	130.99	118.83	0.002	6857	22498	201
R20	2-3	139.04	134.3	0.002	2273	7458	81
R30	1-2	146.03	139.04	0.003	2419	7935	75
R60	4-Outlet	128.04	118.83	0.002	4312	14146	131
R200	3-4	134.3	128.04	0.001	4313	14151	152
R350	1-3	148.82	137.6	0.003	3419	11218	93
R20	2-3	140.15	137.6	0.005	482	1582	17
R30	3-4	137.6	134.06	0.001	2463	8081	99
R240	4-5	134.06	128.22	0.002	3363	11035	117
R290	5-6	128.22	124.87	0.001	2292	7519	93
R60	6-Outlet	124.87	118.83	0.002	2711	8893	90
R200	1-2	149.42	143.72	0.003	2037	6682	66
R30	2-4	143.72	137.55	0.003	1955	6415	61
R20	3-4	143.24	137.55	0.004	1561	5120	49
R50	4-5	137.55	134.3	0.002	2083	6835	84
R290	5-6	134.3	130.95	0.001	2378	7800	97
R340	6-7	130.95	128.05	0.002	1895	6218	79
R400	7-8	128.05	121.97	0.003	2209	7248	71
R60	8-Outlet	121.97	118.83	0.001	2143	7030	88

Table B3. Ash Creek Watershed Transform Model Parameters.

		Channel Flow (Kirpich Method)						
Sub-basin	Area	Elevation$_{US}$	Elevation$_{DS}$	Slope	Travel Length	Travel Length	Time of Concentration, t_c	Time of Concentration, t_c
	mi^2	m	m	-	m	ft	min	hr
W110	7.00	184.09	130.71	0.006	9032	29631	156	2.6
W130	3.61	184.09	144.2	0.006	6605	21671	122	2.0
W120	3.38	174.38	130.71	0.006	6881	22577	123	2.1
W180	2.64	184.09	148.48	0.007	4920	16141	90	1.5
W130	2.32	174.38	131.2	0.008	5339	17516	92	1.5
W170	2.04	162.25	130.71	0.006	5063	16612	98	1.6
W130	1.28	184.09	149.41	0.007	4652	15264	86	1.4
W180	1.48	177.29	144.65	0.008	3886	12749	71	1.2
W280	1.19	162.25	130.79	0.009	3577	11737	66	1.1
W230	1.53	174.38	147.79	0.008	3423	11231	67	1.1
W270	1.53	163.02	130.71	0.006	5066	16621	97	1.6
W130	0.85	184.09	159.59	0.006	3826	12553	78	1.3
W280	0.99	177.33	139.23	0.010	3917	12852	68	1.1
W180	1.03	177.29	150.89	0.009	3020	9908	58	1.0
W330	0.98	174.38	151.91	0.008	2871	9421	58	1.0
W320	0.99	174.38	134.09	0.012	3374	11070	56	0.9
W380	1.08	162.25	131.72	0.009	3543	11625	66	1.1
W370	1.08	165.54	130.71	0.009	3841	12600	69	1.1
W440	0.80	184.09	161.54	0.007	3400	11156	70	1.2
W490	0.70	177.33	149.41	0.010	2897	9504	54	0.9
W540	0.92	177.29	152.77	0.010	2452	8043	47	0.8
W590	0.87	162.57	135.98	0.010	2650	8693	50	0.8
W640	0.73	174.38	152.44	0.009	2439	8001	48	0.8
W690	0.79	168.58	143.02	0.012	2202	7223	41	0.7
W740	0.78	163.02	134.15	0.009	3370	11058	63	1.1
W790	0.65	162.45	130.79	0.010	3026	9927	54	0.9
W780	0.75	161.56	130.71	0.010	3227	10588	59	1.0

Table B4. Ash Creek Watershed Routing Parameters.

Routing Parameters							
Reach	From-To	ElevationUS	ElevationDS	Slope	Travel Length	Travel Length	Travel Time
	-	m	m	-	m	ft	min
R40	1-Outlet	141.93	130.71	0.005	2466	8092	64
R40	1-Outlet	144.5	130.71	0.003	4152	13621	107
R50	2-Outlet	134.81	130.71	0.003	1583	5192	56
R20	1-2	149.16	144.19	0.007	764	2505	22
R300	2-4	144.19	134.69	0.004	2124	6968	57
R50	3-Outlet	147.79	130.71	0.005	3492	11458	81
R40	4-Outlet	134.69	130.71	0.003	1526	5006	54
R20	1-3	159.59	139.23	0.011	1881	6171	37
R30	2-3	150.89	139.23	0.010	1145	3758	26
R400	3-5	139.23	131.72	0.004	1810	5940	52
R250	4-6	152.61	134.09	0.010	1905	6250	39
R60	5-Outlet	131.72	130.71	0.001	1554	5099	94
R40	6-Outlet	134.09	130.71	0.002	2139	7018	86
R510	1-2	161.54	149.41	0.009	1280	4200	29
R460	2-4	149.41	136.15	0.008	1612	5290	36
R30	3-4	152.77	136.15	0.007	2306	7564	51
R810	4-8	136.15	134.69	0.001	1247	4091	63
R40	8-Outlet	134.69	130.71	0.003	1526	5006	54
R50	7-Outlet	134.1	130.71	0.002	1736	5695	67
R330	6-7	143.01	134.1	0.005	1825	5988	49
R710	5-6	152.99	143.01	0.011	922	3024	21

Table B5. Little Pond Elm Creek Watershed Transform Model Parameters.

		Channel Flow (Kirpich Method)						
Sub-basin	Area	Elevation$_{US}$	Elevation$_{DS}$	Slope	Travel Length	Travel Length	Time of Concentration, t_c	Time of Concentration, t_c
	mi^2	m	m	-	m	ft	min	hr
W110	22.81	180.51	119.3	0.003	22972	75367	435	7.3
W130	11.59	180.51	134.15	0.003	13848	45435	270	4.5
W120	11.22	156.05	119.3	0.003	13663	44827	291	4.8
W130	7.53	180.51	134.17	0.003	13395	43947	260	4.3
W180	7.20	166.94	126.86	0.003	13020	42717	266	4.4
W170	8.08	156.05	119.3	0.003	13663	44827	291	4.8
W130	4.70	180.51	143.29	0.004	8884	29146	176	2.9
W220	5.41	156.22	130.94	0.003	9440	30972	219	3.6
W230	3.72	166.94	134.14	0.004	7809	25619	159	2.7
W280	4.02	156.05	126.79	0.003	9629	31593	212	3.5
W270	4.96	135.78	119.3	0.002	6978	22895	182	3.0
W130	3.15	180.51	149.2	0.005	6699	21978	136	2.3
W180	3.28	166.94	137.09	0.004	6931	22740	144	2.4
W230	3.26	156.05	130.6	0.003	8079	26506	182	3.0
W280	3.50	168.05	137.49	0.005	6501	21329	132	2.2
W370	2.94	143.85	125.14	0.004	4774	15663	112	1.9
W380	3.27	150.59	131.12	0.004	5462	17921	129	2.1
W320	3.42	140.91	119.3	0.003	6226	20426	144	2.4
W130	2.52	180.51	152.51	0.005	5338	17514	109	1.8
W180	2.57	168.4	140.61	0.005	5766	18918	119	2.0
W230	2.26	166.94	140.16	0.005	5307	17411	110	1.8
W280	2.45	156.22	134.17	0.004	5591	18343	126	2.1
W330	2.80	155.71	133.85	0.004	5693	18679	129	2.2
W380	2.64	156.05	134.2	0.003	6772	22217	158	2.6
W430	2.14	144.4	126.86	0.003	5583	18316	137	2.3
W480	2.52	144.3	125.02	0.004	5406	17737	128	2.1
W470	2.93	138.13	119.3	0.004	5353	17564	127	2.1

Table B6. Little Pond Elm Creek Watershed Routing Parameters.

Routing Parameters

Reach	From-To	ElevationUS	ElevationDS	Slope	Travel Length	Travel Length	Travel Time
	-	m	m	-	m	ft	min
R50	1-Outlet	134.15	119.3	0.002	9164	30065	260
R30	1-2	134.17	126.86	0.001	5481	17983	188
R50	2-Outlet	126.86	119.3	0.002	4136	13570	134
R30	1-3	143.29	130.94	0.001	8435	27675	253
R50	3-Outlet	130.94	119.3	0.002	5687	18659	164
R20	2-3	134.15	130.94	0.001	3666	12029	163
R40	4-Outlet	126.79	119.3	0.002	4074	13366	133
R300	1-4	149.25	137.45	0.002	4826	15834	135
R30	4-5	137.45	131.12	0.001	4535	14879	160
R50	5-6	131.12	125.34	0.002	3761	12338	133
R20	2-5	137.09	131.12	0.002	3279	10758	113
R40	3-6	130.91	125.34	0.002	2439	8001	82
R60	6-Outlet	125.34	119.3	0.002	3186	10452	108
R200	1-2	152.51	140.45	0.003	4336	14227	119
R300	2-3	140.45	134.17	0.002	3795	12451	131
R30	3-6	134.17	133.85	0.0002	1724	5655	165
R20	4-6	140.16	133.85	0.002	4036	13243	140
R450	6-7	133.85	126.86	0.002	3677	12064	121
R50	7-8	126.86	125.1	0.001	1185	3889	56
R40	5-8	133.99	125.1	0.002	3975	13042	121
R60	8-Outlet	125.1	119.3	0.002	2951	9681	101

Table B7. Storage Coefficients.

South Mesquite Creek			Ash Creek			Little Pond Elm Creek		
Slope	Flow Rate, Q	Storage Coefficient, R	Slope	Flow Rate, Q	Storage Coefficient, R	Slope	Flow Rate, Q	Storage Coefficient, R
$ft^3/s/hr$	ft^3/s	hr	-3800	2500	0.66	-1614	4500	2.8
-1098	3000	2.7	-4120	2500	0.61	-442	1500	3.4
-1088	3000	2.8	-4600	2500	0.54	-1159	4000	3.5
-1314	4500	3.4	-5820	3500	0.60	-526	2250	4.3
-510	2500	4.9	-1714	1250	0.73	-495	2500	5.1
-800	2500	3.1	-	-	-	-	-	-

APPENDIX C

WATERSHED HYDROGRAPHS

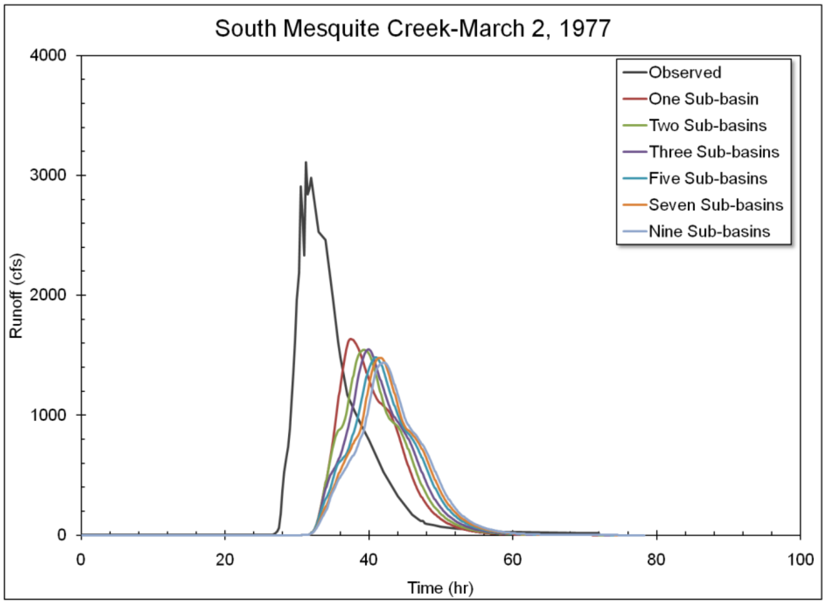

Figure C1. Runoff hydrographs from South Mesquite Creek watershed for the event of March 2, 1977.

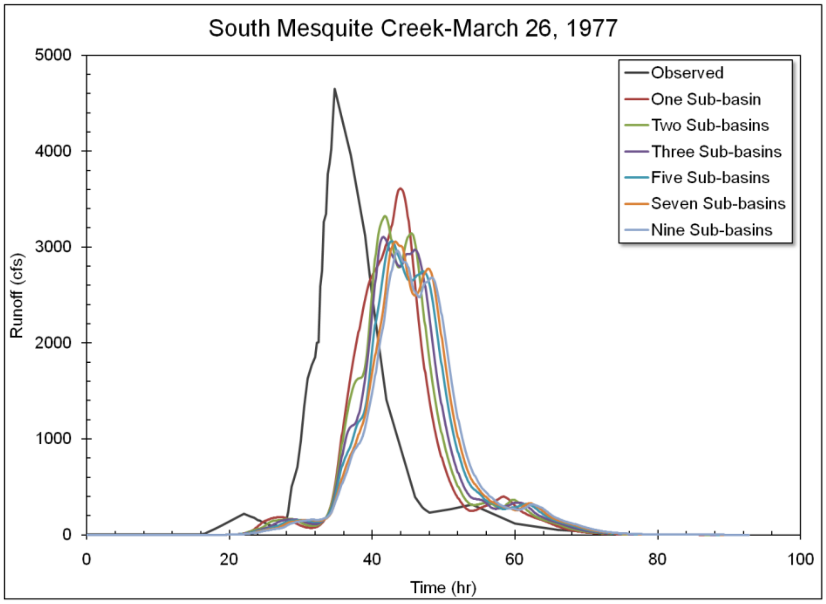

Figure C2. Runoff hydrographs from South Mesquite Creek watershed for the event of March 26, 1977.

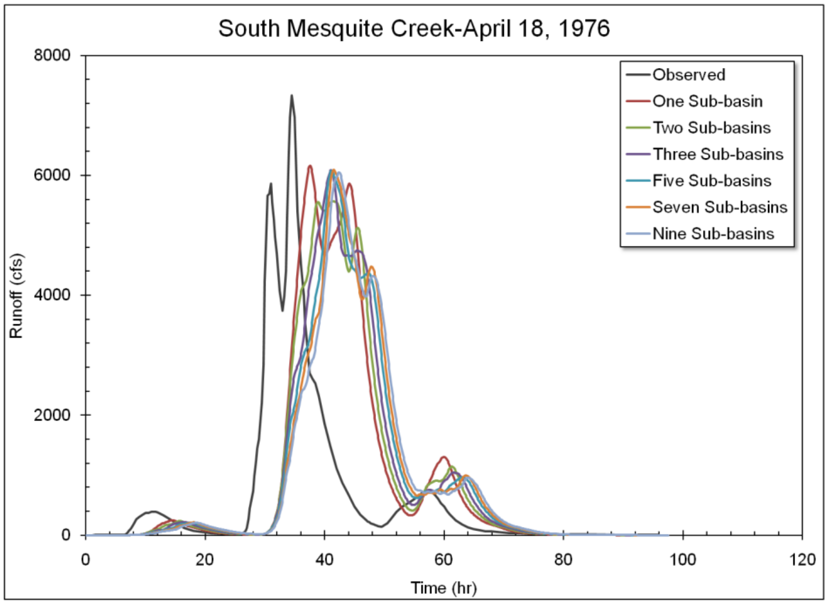

Figure C3. Runoff hydrographs from South Mesquite Creek watershed for the event of April 18, 1976.

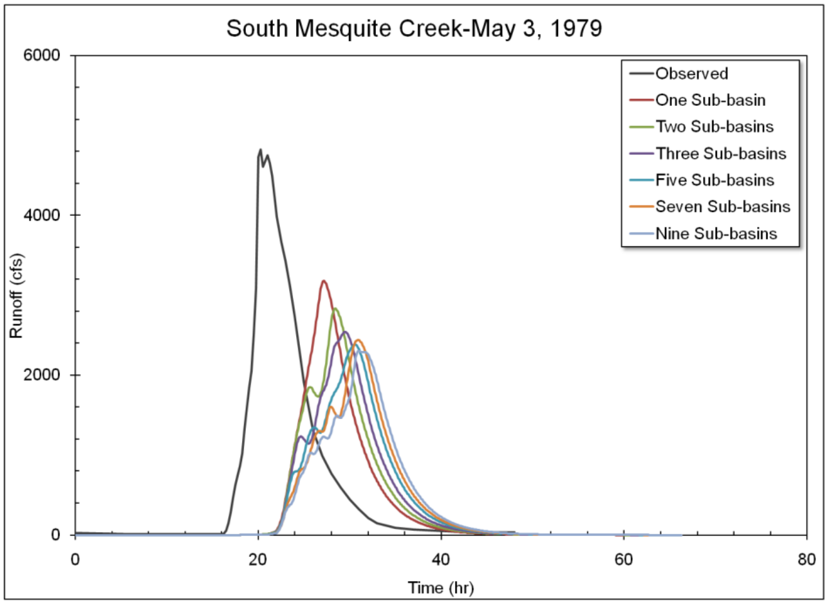

Figure C4. Runoff hydrographs from South Mesquite Creek watershed for the event of May 3, 1979.

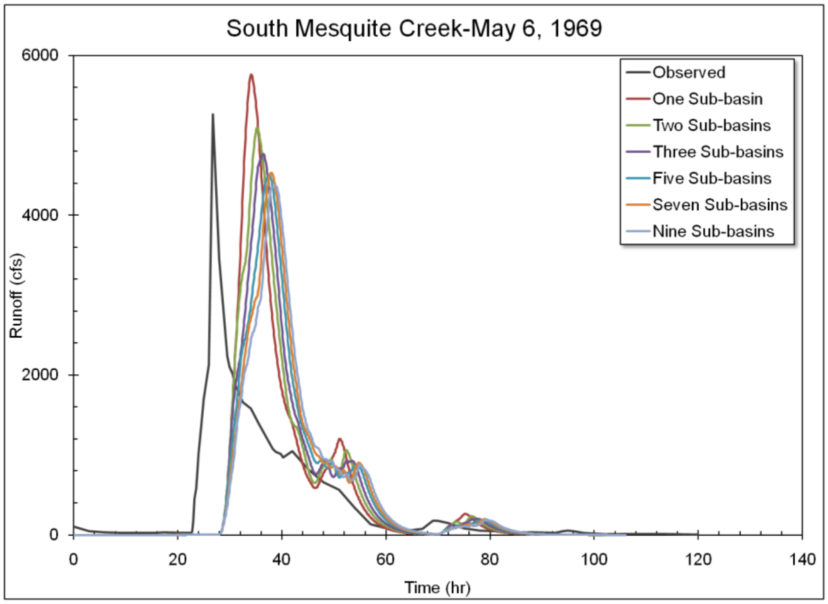

Figure C5. Runoff hydrographs from South Mesquite Creek watershed for the event of May 6, 1969.

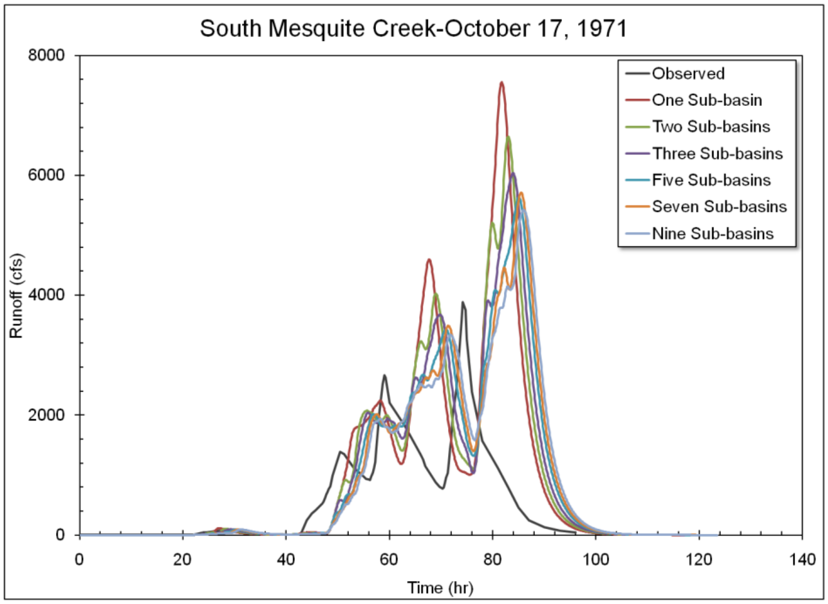

Figure C6. Runoff hydrographs from South Mesquite Creek watershed for the event of October 17, 1971.

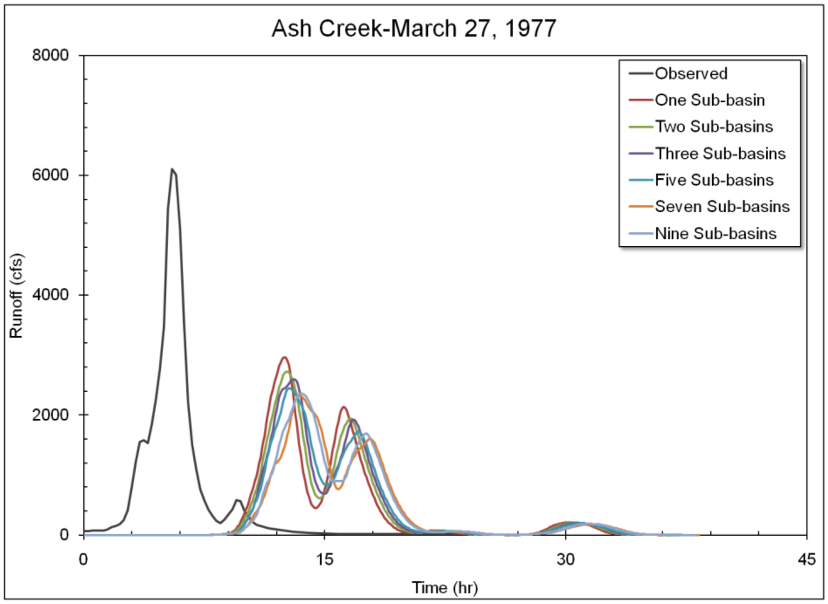

Figure C7. Runoff hydrographs from Ash Creek watershed for the event of March 27, 1977.

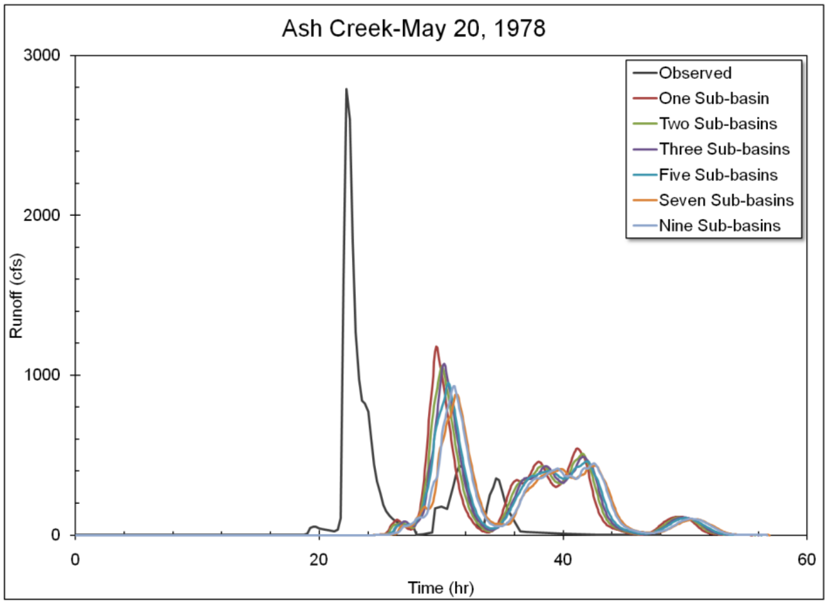

Figure C8. Runoff hydrographs from Ash Creek watershed for the event of May 20, 1978.

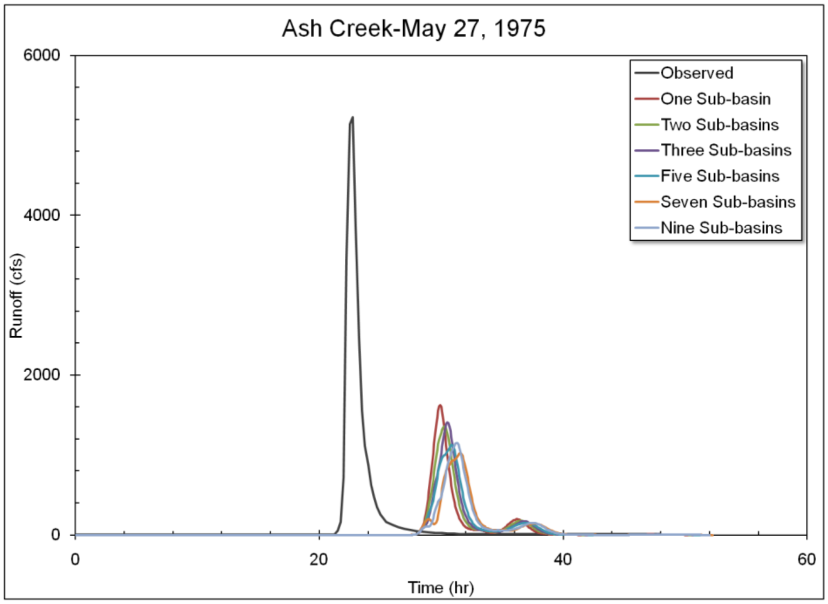

Figure C9. Runoff hydrographs from Ash Creek watershed for the event of May 27, 1975.

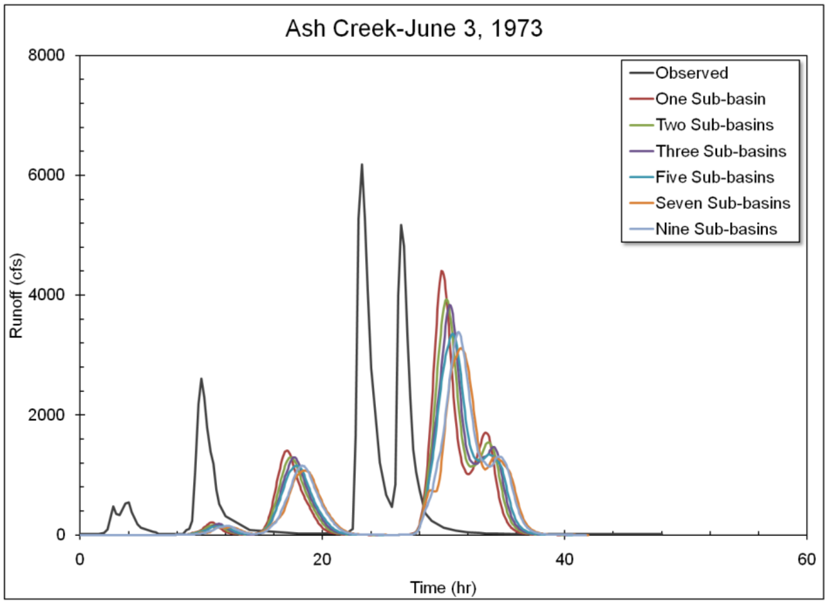

Figure C10. Runoff hydrographs from Ash Creek watershed for the event of June 3, 1973.

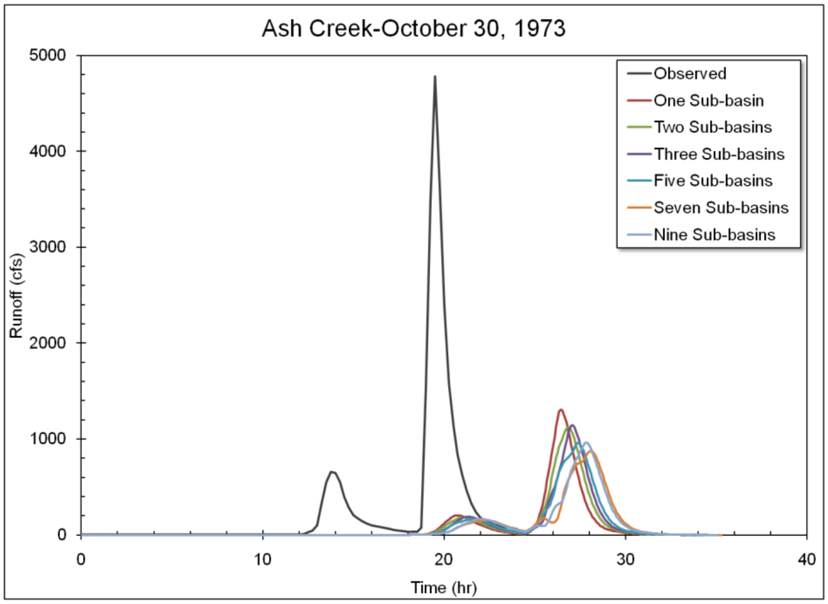

Figure C11. Runoff hydrographs from Ash Creek watershed for the event of October 30, 1973.

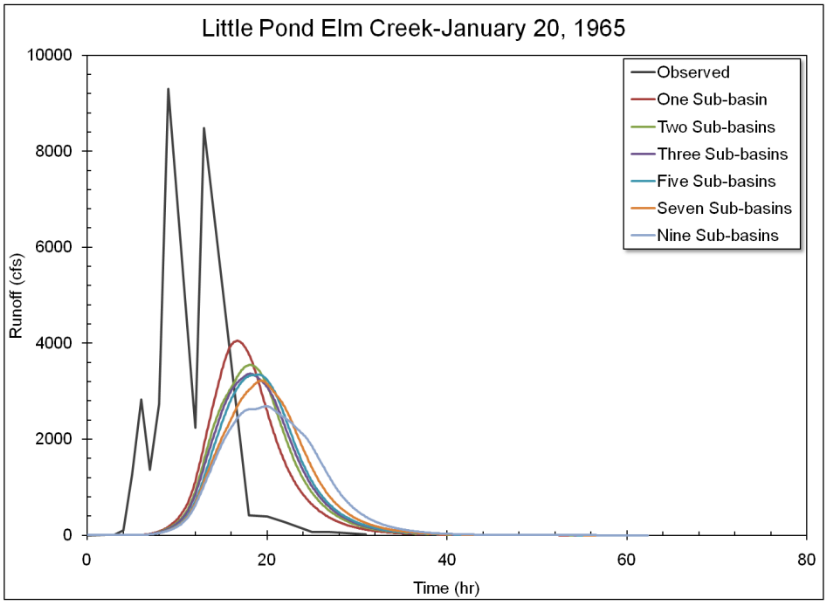

Figure C12. Runoff hydrographs from Little Pond Elm Creek watershed for the event of January 20, 1965.

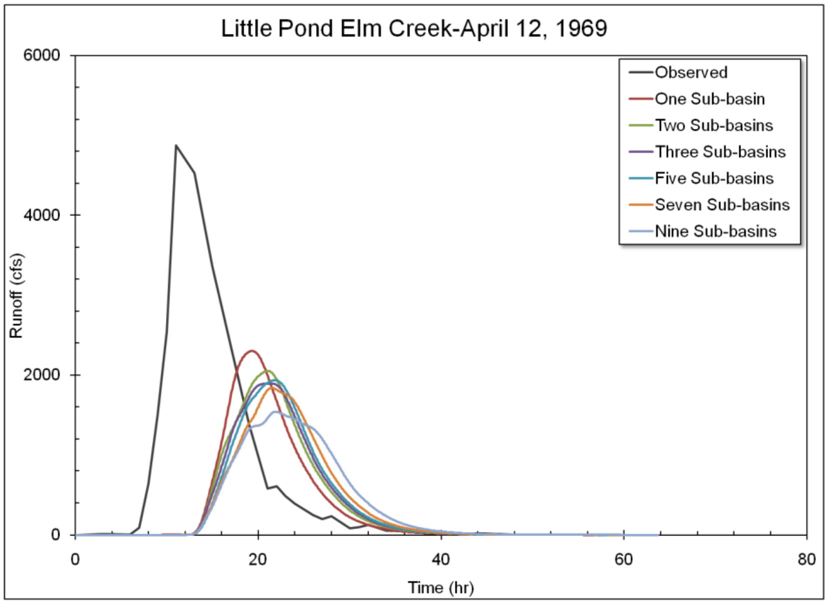

Figure C13. Runoff hydrographs from Little Pond Elm Creek watershed for the event of April 12, 1969.

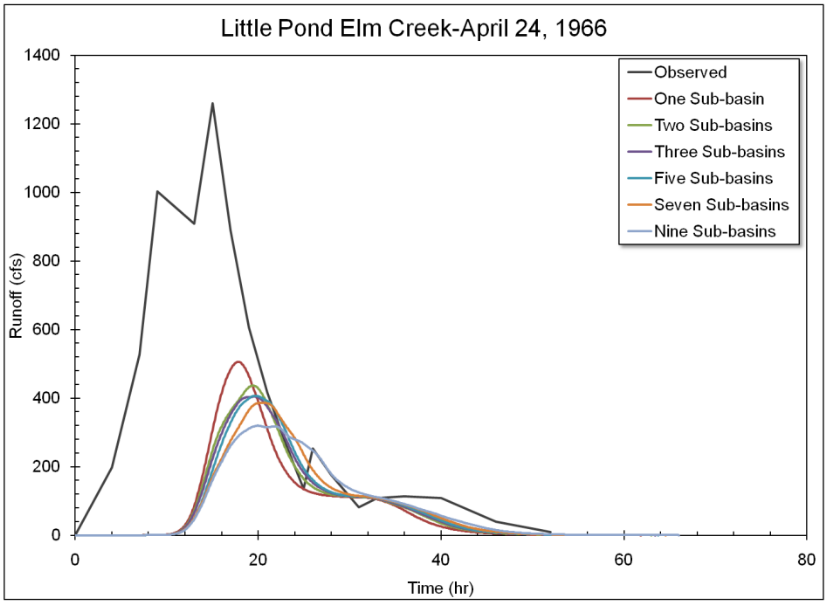

Figure C14. Runoff hydrographs from Little Pond Elm Creek watershed for the event of April 24, 1966.

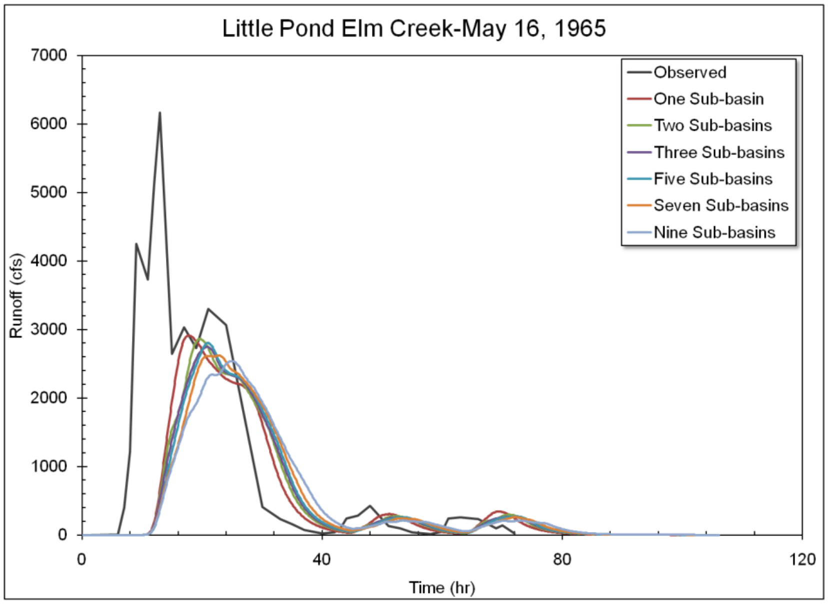

Figure C15. Runoff hydrographs from Little Pond Elm Creek watershed for the event of May 16, 1965.

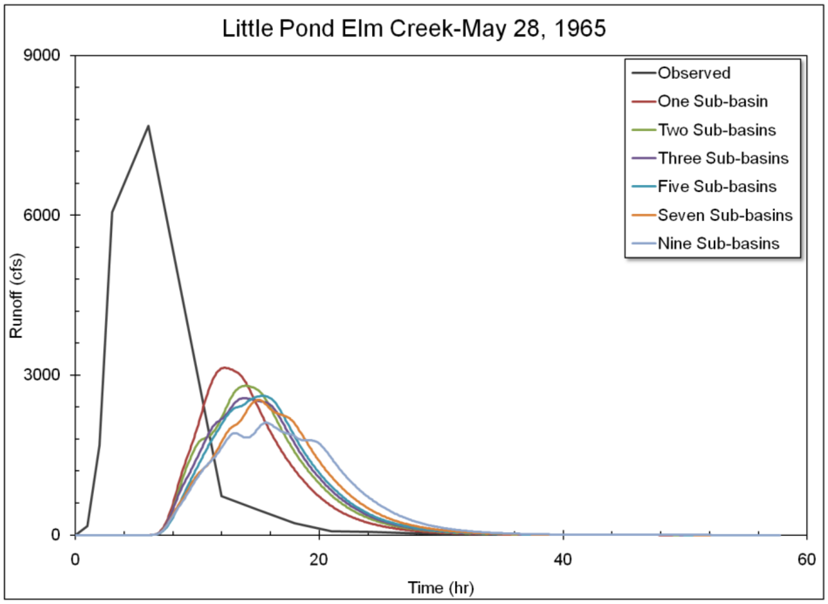

Figure C16. Runoff hydrographs from Little Pond Elm Creek watershed for the event of May 28, 1965.

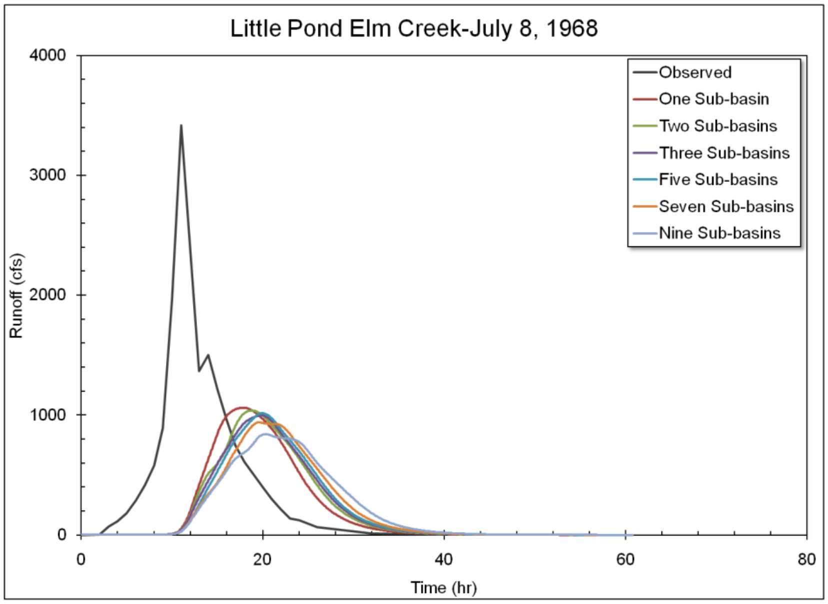

Figure C17. Runoff hydrographs from Little Pond Elm Creek watershed for the event of July 8, 1968.

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