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DISTRIBUTED STORAGE MODELING IN SOAP CREEK FOR FLOOD CONTROL  
AND AGRICULTURAL PRACTICES

by  
Matthew John Wunsch

A thesis submitted in partial fulfillment  
of the requirements for the Master of  
Science degree in Civil and Environmental Engineering  
in the Graduate College of  
The University of Iowa

May 2013

Thesis Supervisors: Professor Larry J. Weber  
Adjunct Associate Professor Nathan C. Young

Graduate College  
The University of Iowa  
Iowa City, Iowa

CERTIFICATE OF APPROVAL

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MASTER'S THESIS

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This is to certify that the Master's thesis of

Matthew John Wunsch

has been approved by the Examining Committee  
for the thesis requirement for the Master of Science  
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## ACKNOWLEDGMENTS

First and foremost the mentorship, advice, and openness put forth by my thesis advisors Drs. Larry Weber and Nathan Young were invaluable not only for the work on this project but for continued guidance in the future. I thank the members of the floodplain mapping and watershed project groups in providing counsel and assistance when needed. Without their help this project would not have come to fruition. To the staff and students of IIHR - Hydroscience & Engineering and the Iowa Flood Center, the friends I have made and support I have been given has been invaluable and will be unforgettable. Last, I would like to thank my family for supporting my decisions past, present, and future. They are always a source of positive thinking when I needed them and without them I would not be where I am today.

## ABSTRACT

In 1988, the counties of Appanoose, Davis, Monroe, and Wapello created the Soap Creek Watershed Board. This group put in place a plan to fund and construct 154 farm ponds in an effort to provide water for agriculture practices as well as provide flood protection for the residents inside the Soap Creek watershed. Through collaborative efforts and funding from federal, state, and local sources, to date 132 ponds have been constructed.

Currently there is no stream monitoring in place in the watershed to observe stream conditions. This leads to no stored data on the benefits of the projects in the basin and the reduced flood impacts. With funding from the Iowa Watershed Projects (IWP) through the IIHR - Hydroscience & Engineering lab, a lumped parameter surface water model was created to show the benefits of the constructed projects. Using detailed LiDAR data, a Hydrologic Engineering Center-Hydrologic Modeling System (HEC-HMS) model was created. This model used ArcHydro and HEC-GeoHMS, tools in ArcGIS. Detailed LiDAR, SURGGO soil data, and land cover data was used to create the model parameters. Several design and historical storms were modeled to quantify the benefits in peak flow reductions and in amounts of water stored behind the projects.

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## CHAPTER 1: INTRODUCTION

Ever since populations have been utilizing land in floodplains, there have been efforts made to mitigate or eliminate flooding. In an effort to protect homes, businesses, or agriculture there have been many attempts at reducing the effects of flooding. Throughout time many technologies have been created to combat flooding along rivers, these techniques include floodwalls, levees, large reservoirs, diversions, or floodplain improvement. Many of these techniques were implemented to protect small areas, usually populated town or cities. There are few examples of basin wide practices to reduce the effects of flooding.

For many populated cities along major rivers, the preferred method of protection is the installation of floodwalls. Floodwalls only provide protection to the area in which they are built. An area that is known for their floodwall protection system is located along the Red River of the North in Minnesota, North and South Dakota. The Red River lies in an area that is very flat so when flooding occurs the water is able to spread out over a vast area. Figure 1.1 shows the expansive flooding along the Red River in 2011. Many of the projects were built to hold back the river to elevations that exceed the 100 year flood (Fargo's Long-Term Flood Protection Plan, 2013). These floodwalls have been built to protect the cities of Fargo/Morehead and Grand Forks. Another option being investigated in the Red River Basin is the so called "Waffle". This study is looking at the feasibility of using the natural depressions inside the basin as storage reservoirs. These depressions are usually bounded by county roads, ditches, or wetlands (Kurz, et al., 2008). Once the depressions are identified, the project wants to build up the bounding features and add a controlled release mechanism.

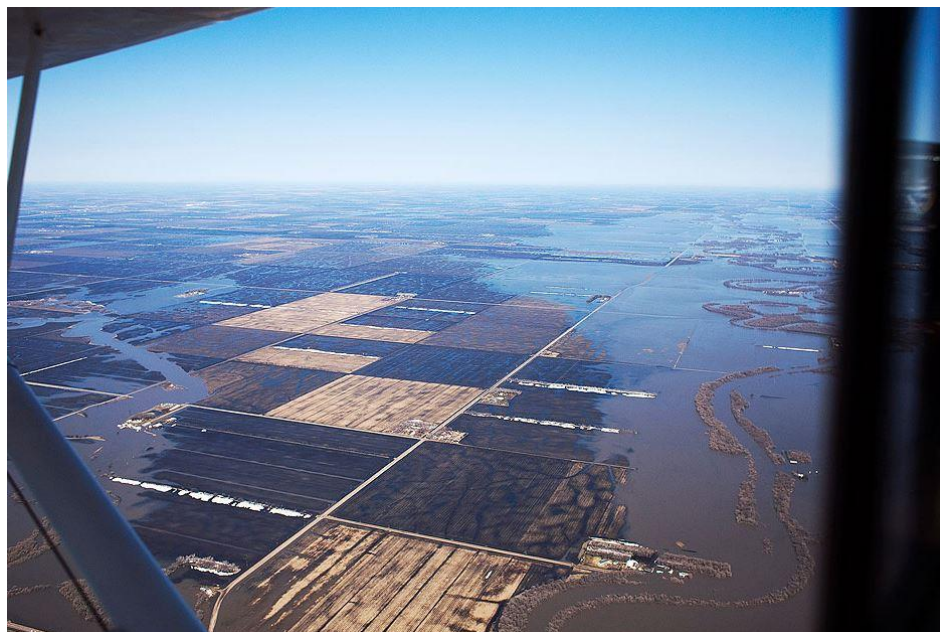


Figure 1.1: Flooding along the Red River of the North in 2011 (Miller, 2011)

The Missouri River provides an example of a distributed flood protection system along the main stem of a river. When Congress passed the Flood Control Act of 1944, they provided the funding for the most comprehensive river basin development plan taken at the time (USACE, 1947). This plan called for the creation and maintenance of six large reservoirs as well as many miles of flood control levees and floodwalls, see Figure 1.1 for reservoir locations. The six reservoirs control the basin upstream of Yankton, South Dakota, and can store up to 73.4 million acre-feet of water (USACE, 2002). Figure 1.2 shows the Gavins Point dam and reservoir located in South Dakota during a flood in 2011. All of the reservoirs are placed on the main stem. The reservoirs are used to provide flood control in the lower basin, irrigation for the upper basin, and recreation. These reservoirs only provide protection to the areas below the projects and do not protect any of the upstream headwaters or tributaries. For the area below the reservoirs, which starts in Sioux City, Iowa and downstream to the confluence of the

Missouri and Mississippi Rivers, protection is provided by levees. Similar to floodwalls levees only protect the areas adjacent to where they were constructed.

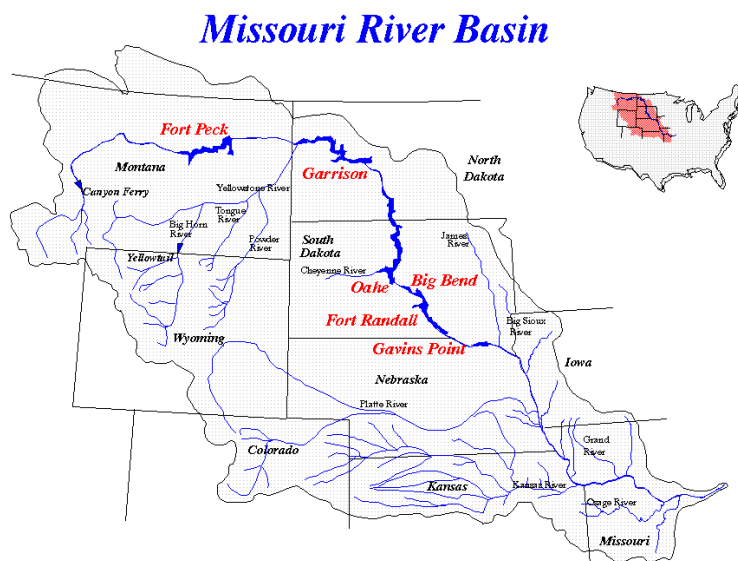


Figure 1.2: Missouri River Basin, reservoir locations in red (USACE, 2002)



Figure 1.3: Missouri River, Gavins Point Dam and Reservoir (Tretbar, 2011)

These examples show some of the methods used to control flooding along the main stem of a river. The purpose of this study is to provide basin wide reduction of floods using best control practices for the study area. In order to determine where structures should be built and what type of structure should be implemented, an accurate model needs to be used. There are various modeling softwares available to users. One of the most widely used modeling software's available is the Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS).

The HEC-HMS model is a lump parameter model. This means that instead of using a detailed mesh of calculation nodes, an area is delineated and all of the parameters are then area averaged to give one calculation per element. Many watershed studies have been run using this software. There are many examples of using this lumped parameter modeling for storm event modeling. A coupled HEC-HMS/HEC-RAS model was used to show the effects of a storm event in San Antonio using NEXRAD rainfall (Knebl, Yang, Hutchison, & Maidment, 2005). All of the HMS modeling was run using a gridded setup based on a 4 km grid. The basin modeled was approximately 4000 square miles but was only delineated into 12 subbasins. Another HMS model was used to forecast the flow into one basin that is controlled by a reservoir in central California. (Anderson, Chen, Kavvas, & Feldman, 2002) This study was using a rainfall forecast model as input to get a forecasted amount of water that will flow into the reservoir. Once the model is calibrated the results can be used to forecast releases through the reservoir to control flooding.

These modeling examples show how HMS can be used for different scales and using different rainfall methods. For the study of Soap Creek the tools available in HMS will be used to model the flows through the basin. Flows will be modeled for 24 hour design storms, and historical storms using NEXRAD rainfall. The results can point out which areas would be best suited for further investigation and potential construction sites for control measures.



## CHAPTER 2: BACKGROUND INFORMATION

After the Floods of 2008, which devastated much of Central and Eastern Iowa, funding was provided to the state through the office of Housing and Urban Development (HUD). This funding was then distributed to various agencies in the state. A portion of this funding was allocated to IIHR - Hydrosience & Engineering at the University of Iowa (UI). The funding was used for the Iowa Watershed Projects (IWP). As part of IWP, the Soap Creek Watershed Board applied to take part in this project.

### 2.1: Iowa Watershed Projects

Following the floods and tornadoes that affected the state in 2008, the state received \$84.1 million in funding from the federal HUD department. \$8.8 million was allocated to IIHR to evaluate, plan, and implement watershed projects (IIHR, 2012). The specific goals of the projects were detailed in House File 2459 as follows. Maximize soil water holding capacity from precipitation, minimize severe scour erosion and sand deposition during floods, manage water runoff in uplands under saturated soil moisture conditions, and reduce and mitigate structural and nonstructural flood damage. IIHR is partnering with the Iowa Department of Natural Resources (IDNR), the Iowa Department of Agriculture and Land Stewardship (IDALS), the Iowa Economic Development Authority (IEDA), and the USDA Natural Resources Conservation Service (NRCS) for the efforts of the Iowa Watershed Projects.

A Request for Information (RFI) went out to the 85 counties that were declared Federal Disaster Areas during the floods of 2008; see Figure 2.1 for the map of disaster areas. This RFI asked for watersheds no larger than a HUC-8 who expressed interest in reducing the effects of flooding within the watershed to partner in a multi-year project. The project is broken into two phases, Phase 1 entails assessing the watershed and Phase 2 is the construction and monitoring of flood mitigation projects.

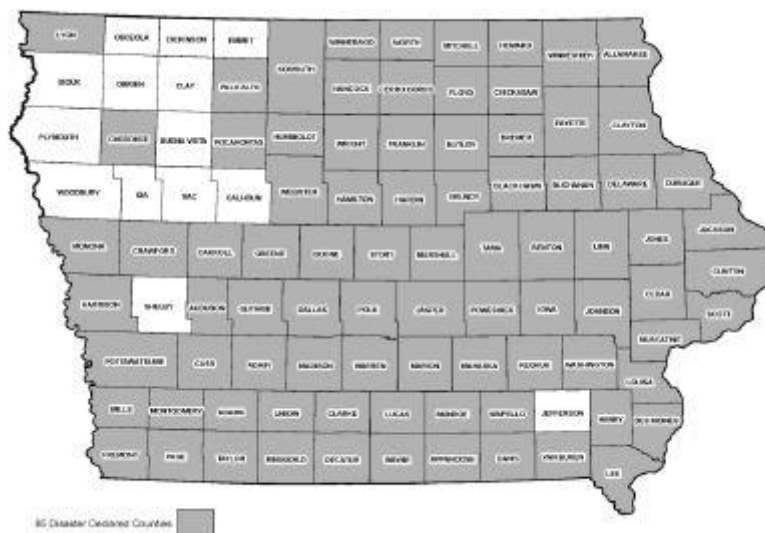


Figure 2.1: Federal Disaster declared counties after the 2008 floods (Ralston, 2010)

Four watersheds were selected for Phase 1 study, the Upper Cedar River, Turkey River, Soap and Chequest Creek, and Middle Raccoon. The locations of the selected watersheds are shown in Figure 2.2. Each selected watershed will then have a detailed analysis done. This assessment includes data collection, hydrologic modeling, and assessment of areas with the best chances to reduce flooding and flood damages downstream (IIHR, 2012). Phase 1 also includes the installation of IFC Stage Sensors in some of the basins to assist in model calibration and for stream monitoring. Three of the watersheds were then chosen for Phase 2. Within each selected watershed one HUC-12 will be chosen for project construction.



Figure 2.2: Location of the four IWP watersheds, state geologic characteristics in background (IWP, 2012)

## 2.2: Soap Creek Watershed Plan

In 1988 the counties of Appanoose, Davis, Monroe and Wapello gathered to discuss the flooding and agriculture issues facing them. Together all four County Board of Supervisors, and County Soil and Water Conservation Boards (CSWB) drafted the Soap Creek Watershed Plan. This comprehensive plan detailed the desire to construct up to 154 retention ponds for use of flood mitigation.

### 2.2.1: Watershed Plan

The Soap Creek is a 162,000 acre watershed located in the Lower Des Moines HUC-8. Of the 162,000 acres 96% is privately owned, with the remaining 4% owned by local governments (Soap Creek Watershed Board, 1988). The watershed has approximately 10,680 acres of cropland, 1,670 acres of pasture, and 970 acres of forest in the floodplain. All projects that are built must have a greater benefit compared to cost. The National Resources Conservation Service (NRCS), formally the Soil Conservation Service (SCS) funded construction cost of new projects. With funding from IWP

constructions cost are split 75% by the NRCS and 25% by the land owner. Operation and maintenance will be the responsibility of the land owners with assistance by the local CSWB.

### 2.2.2: Problem and Opportunity Identification

Annual average damages in Soap Creek were calculated to be \$1.4 million in 1988. These values were based on damages obtained in the 14,150 acre floodplain of the main stem and the major tributaries along the Soap Creek. Damages include loss of crops, pastures, roads, and bridges. Erosion in the basin has also led to pipelines being exposed and possible damage (Soap Creek Watershed Board, 1988). The high priority concerns identified by the watershed plan were floodwater, floodplain scour, and sedimentation reduction. Transportation damages and effects to the ecosystem were considered as well. These concerns must be considered in the analysis of alternatives. Other concerns that were identified are not required to be considered in the analysis of alternatives.

### 2.2.3: Alternatives

There are two alternatives identified in the watershed plan. These options are the no action plan, and the National Economic Development (NED) objective. The first option implies that there will be no action taken in the watershed, and to leave all the present conditions as is. The second option was to construct the 154 projects, project locations are in Figure 2.3. The total cost of the projects was estimated at \$6.52 million in 1988. Based on studies of the watershed the NED plan will reduce flooding in the 14,150 acres of the floodplain, reduce sediment discharge into the Des Moines river by approximately 30,500 tons per year, and create approximately \$1 million in annual benefits to the watershed. The NED Plan was selected as it maximizes benefits and land owners would prefer more control structures to reduce flooding.

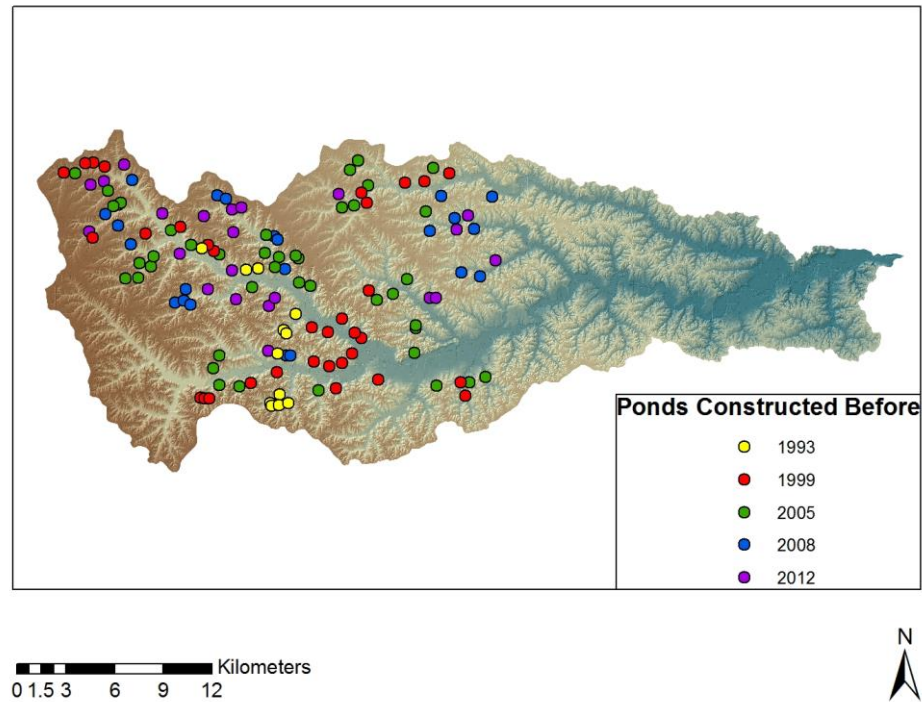


Figure 2.3: Locations of the 132 ponds that have been built and progression of construction

#### 2.2.4: Plan Implementation

The NED plan called for the construction of 154 dams to reduce the effects of flooding. All of the dams were required to be Class (a) structures. Class (a) structures require that the only damage from failure is to farm buildings, agricultural land, or township and county roads. (Soap Creek Watershed Board, 1988) All of the dams must be built from the earth available on site. Many of the identified project locations were found to be in the upper watershed. Most of the projects will be designed to store the runoff from the 10, 25, and 50 year storm between the principal and emergency spillways. NRCS guidelines will be followed in the design and construction of the projects. To date 132 of the original 154 projects have been completed.

### 2.3 NRCS Pond Guidelines

The NRCS provides guidelines on the planning, design, and construction of ponds. A typical embankment pond is made by building a dam across a stream or watercourse. The natural depression must be deep enough to store at least 5 feet of water behind it. The dam must be less than 35 feet high and if there is dam failure then the resulting damages must not include loss of life, damage to commercial or industrial buildings, main highways, railroads, or interrupt use of public utilities. The recommended minimum pond depth for Iowa is suggested to be between 7 and 8 feet. (NRCS, 2013) The principal spillway is normally chosen to pass the runoff from a 24 hour storm with return periods of 1 to 10 years. Auxiliary spillways should have the ability to contain the 10 to 50 year storm depending on the amount of drainage area being controlled. SCS methodology is used in the design of all ponds. The SCS methods are described in Sections 3.2.1 and 3.2.2.

## CHAPTER 3: HEC-HMS MODELING

HEC-HMS is a modeling program developed and distributed by the United States Army Corps of Engineers (USACE) to simulate rainfall-runoff processes in dendritic watersheds. The program relies on separating the hydrologic cycle into separate pieces and constructing watershed boundaries. Each component of the water cycle is then able to be represented by a separate mathematical model. (Scharffenber and Fleming, 2010)

### 3.1 Components

The hydrologic cycle is separated into two parts; a basin model and a meteorological model. Once the basin is defined and a rainfall delivery process is chosen, a control model defines the model run time. These components will be described in the sections that follow. The user interface for HMS showing the components of the Soap Creek project is shown in Figure 3.1.

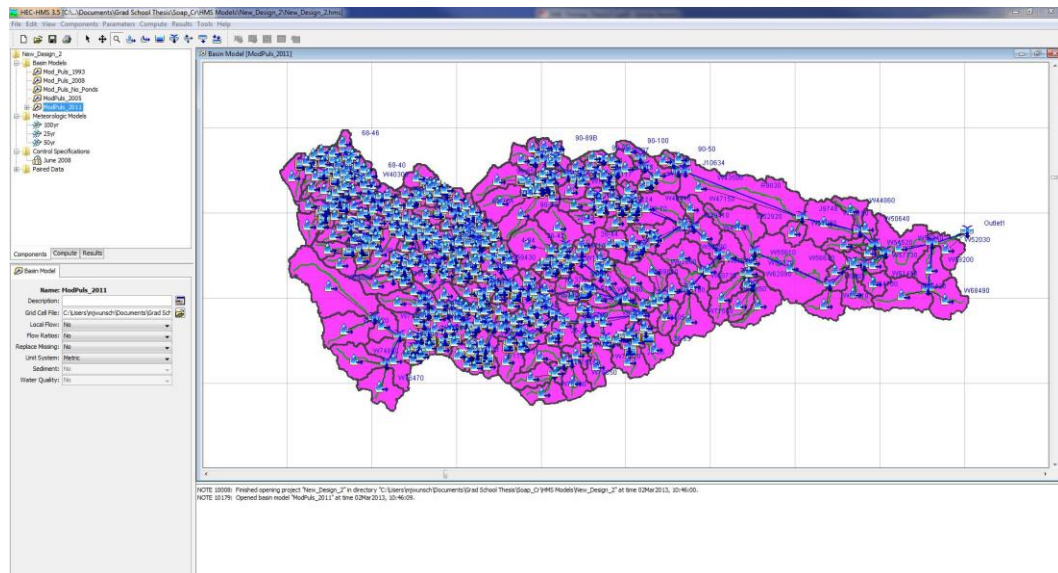


Figure 3.1: Graphical user interface of HEC-HMS

### 3.1.1 Basin Model

The basin model provides the physical characteristics of the land surface inside of the model area. The basin model contains the components that describe how rainfall moves along the surface and how it is transformed into stream flow. The components included in the basin model are infiltration, surface runoff, baseflow, and river routing. Elements that make up the basin are subbasins, reaches, reservoirs, junctions, diversions, sources, and sinks. These elements are then assembled together to create a dendritic basin. Within each element there are a series of parameters that describe how water moves through each element. These parameters are described in Section 3.2.

### 3.1.2 Meteorological Model

The meteorological model is designed as a way to create the boundary conditions that act on a watershed. (Scharffenber and Fleming, 2010) This model describes how water will be generated for each subbasin element inside of a basin model. The meteorological model is also responsible for precipitation, evapotranspiration, and snowmelt processes. Precipitation may be distributed over the basin using precipitation gages, frequency storms, or gridded precipitation. Evapotranspiration can be controlled using monthly averages or the Priestley Taylor method. Snowmelt is controlled by temperature indexing.

### 3.1.3 Control Model

The third component of the HEC-HMS model is the control model. This component sets the duration for a model run as well as what time step will be used during the modeling run. The time step must be set so that each element can be calculated at least once. This is most important with working with lag times, or velocities in streams.



### 3.2 Parameters

The following section describes the various parameters that are needed to populate the different models inside a project. In depth descriptions of the loss, transform, routing, and baseflow methods that were created within the basin model will be discussed. Creating the inputs of the meteorological model will also be discussed.

#### 3.2.1 Loss Parameters

Determining the distribution of rainfall into infiltration and runoff is the first step in creating the final element output hydrograph. The loss is calculated at the end of each time interval by finding the difference in the volumes of each adjacent time interval. (Scharffenber and Fleming, 2010) There are 11 loss methods available inside of HEC-HMS, the SCS Curve Number method was chosen for implantation in this study. Using the SCS Curve Number, the rainfall is able to be partitioned into the two components of infiltration and runoff. The curve number is calculated in GIS using land cover and soil data; this will be detailed in Section 4.2. Once the curve number is calculated for each basin this number represents the percentage of rainfall that will be converted into direct runoff minus any initial abstractions. The initial abstraction is the amount of rainfall that must fall before any excess runoff can occur. The default value of initial abstraction is calculated from the SCS method as  $I_a = 0.2S$ .  $S$  is defined as a value of maximum potential retention which is calculated from the curve number.

$$S = \frac{1000}{CN} - 10$$

CN=Curve Number

The last input for calculating loss is the percent of subbasin impervious. Since any areas with the potential to have impervious conditions was taken into account while creating the curve numbers this variable was left at zero for all subbasins across all model runs.

### 3.2.2 Transform Parameters

Converting the runoff that was created from the loss component into a physical hydrograph is done through the transform method in HEC-HMS. Inside of HMS there are seven transform methods to choose from, and the SCS Unit Hydrograph was selected for modeling. When using the SCS Unit Hydrograph, the runoff that was produced from the loss calculation is converted into an outflow hydrograph. The general hydrograph takes the time lag from each subbasin and scales the outflow for use in the unit hydrograph. (Scharffenber and Fleming, 2010) Using the standard graph type, the lag time is defined as the time between the centroid of precipitation and the peak flow of the hydrograph. This time lag is calculated in GIS using HEC-GeoHMS and uses the curve number method from the 2010 NRCS Handbook. The equation is as follows.

$$Lag = \frac{(L^{0.8} * (S + 1)^{0.7})}{(1900 * Y^{0.5})}$$

L=Longest Flow Path (feet)

Y=Basin Slope (%)

S=Maximum Possible Retention

The SCS Unit Hydrograph then transforms the lag time into a ratio to unit hydrograph peak discharge described below.

$$U_p = \frac{\Delta t}{2} + Lag$$

$\Delta t$ =model time step

### 3.2.3 Baseflow Parameters

Once the loss method splits the rainfall into infiltration and runoff, the water that infiltrates into the soil is not represented in the model. The baseflow methods try to simulate the response of the infiltrated water back into the stream network. There are

five methods of baseflow open to use in HMS. Prior to the IWP there were no monitoring gages or equipment in Soap Creek. Now there are four IFC stream sensors in place. Two are reporting continuously and from these, model validation is possible. In order to match the hydrograph shape with the gage data baseflow was taken into account. This was modeled in HMS using the recession method. The recession method uses a single exponential equation that relates what baseflow will start at and at what rate it will decay. Baseflow also requires a value that will indicate when baseflow resumes the full flow in an element; this was done using the ratio to peak method. A ratio to peak of 1% was chosen for modeling in this project. The baseflow equation is shown below.

$$Q_t = Q_0 k^t$$

$Q_t$  = baseflow  
 $Q_0$  = initial discharge  
 $k$  = recession constant  
 $t$  = time

#### 3.2.4 Routing Parameters

Once the all of the basin parameters converting the rainfall into runoff have been calculated, the hydrographs need to be moved down stream through the stream network. This is done through routing methods in HEC-HMS. There are six routing methods built into HMS. Since there are no gages or historical data available to calibrate to, the best routing method available for use is the modified puls. Modified puls uses a storage-discharge relationship that defines how water will be attenuated through the reach using conservation of mass (Scharffenberg and Fleming, 2010).

$$\left( \frac{S_t}{\Delta t} + \frac{O_t}{2} \right) = \left( \frac{I_{t-1} + I_t}{2} \right) + \left( \frac{S_{t-1}}{\Delta t} - \frac{O_{t-1}}{2} \right)$$

$S_{t,t-1}$  = Storage at time t and t-1  
 $I_{t,t-1}$  = Inflow discharge at times t and t-1  
 $O_{t,t-1}$  = Outflow discharge at times t and t-1  
 $\Delta t$  = simulation time step

### 3.2.5 Meteorological Parameters

Once the basin model is set up and all of the characteristics of how the water is moved over the surface are filled, the description of how water is applied to that surface needs to be described. There are three options on how much water is generated and when it is applied. These options are precipitation, evapotranspiration, and snowmelt. Since all of the modeling in this investigation is event based, only precipitation is considered in the meteorological model. There are seven methods of applying precipitation to the subbasins, three of which will be discussed in this section.

To run the 24 hour design storms, the SCS Storm method was chosen. This method uses the 24 hour design storm that was created by the NRCS. The same hyetograph is applied to each subbasin and there are no reductions on the storm total depth of precipitation. For this method the user needs to define the type of hyetograph distribution which is based on location, See Figure 3.2. The state of Iowa is located in the Type 2 distribution. Using the 1992 Rainfall Frequency Atlas of the Midwest from the University of Illinois to get the storm depths, the hyetographs are then calculated.

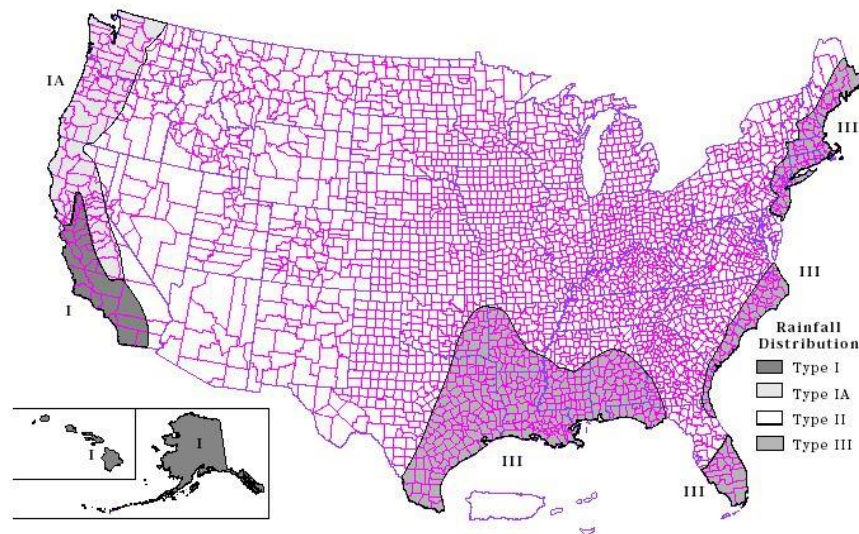


Figure 3.2: SCS design storm type distribution (NRCS, 1986)

The second precipitation method applied was gridded precipitation from NEXRAD radar products. In order to use radar data, a grid was set up in GIS that segmented the subbasins into a 4.5km grid that is used by the National Weather Service. The rainfall is then averaged from the grid cells into the subbasins to be used as the precipitation. The data is hourly, and for time steps that are under an hour the precipitation is divided evenly so that at the end of the hour the total hourly precipitation had been met. The grid is created using ASCII files and converting them into a HEC-DSS table. The table is then linked to the meteorological model and the data can be shifted in time to adjust to different reporting formats (UTC, Central, and Daylight Savings). The last input is how to handle missing data, there are two options. The model can either set missing data to zero or to abort the simulation. For the simulation runs used in this study the “set to zero” option was chosen so that the control model could be set to run longer than a specified storm to show how the watershed drains.

The last precipitation method used was the specified hyetograph. This method allows the user to create a specific plot of rainfall to be used for each subbasin or applied over the entire watershed uniformly. The data must be entered as a precipitation gage in order to use this method. Once all of the hyetographs are input the user must assign each subbasin to a gage. If the simulation control period is set longer than the hyetograph extent then the user must specify how to handle the missing data. For the purposes of this exercise the missing data are set to assume zero.

### 3.3 Chapter Summary

HEC-HMS is a surface water runoff modeling program that uses basin averaged parameters that characterize the land surface and stream network. HMS compiles how water moves through the watershed using mathematical models. The meteorological model describes how that water is applied to the subbasins, through gages, design storms,

and radar rainfall. Once the precipitation is on the land surface the basin model distributes the water through the subsurface infiltration and surface runoff. The surface water is then moved from the land into the stream network. Eventually the water is routed through the system to the outlet. As the water moves through the system hydrographs are able to be produced at any element in the system.

## CHAPTER 4: MODEL BUILDING

HEC-HMS modeling is dependent on being able to supply all of the values needed to fill all the parameters inside of each component. Doing these calculations by hand would present a large time commitment. Using the tools and extensions supplied in ArcGIS, I was able to manipulate data sets to create the information needed to input into HEC-HMS. Combining LiDAR, land cover, and soil data sets, ArcGIS is able to create the import files that are then used in HEC-HMS. Steps to create the required data are described in detail in this chapter.

### 4.1 Arc Hydro

Using published information by the Iowa Department of Natural Resources (IDNR), United States Geological Survey (USGS), and United States Department of Agriculture (USDA), data sets are input to produce required GIS layers. LiDAR derived rasters, NHD streamlines, NLCD land use products, and NRCS Soil products are discussed in the following section.

#### 4.1.1: Digital Elevation Model

The digital elevation model (DEM) that was used in Soap Creek was created using Light Detecting and Ranging (LiDAR) technology. This tool is able to measure the distance from the emitting object to another surface by measuring the travel time of laser pulses (Sanborn, 2013). The resulting product is the one meter DEM from IDNR. Both the one meter and three meter DEMs are used for the modeling of Soap Creek. The three meter DEM is created by averaging the one meter DEM over the 3 meter domain. Figure 4.1 shows the resolutions of the two DEMs that were used.

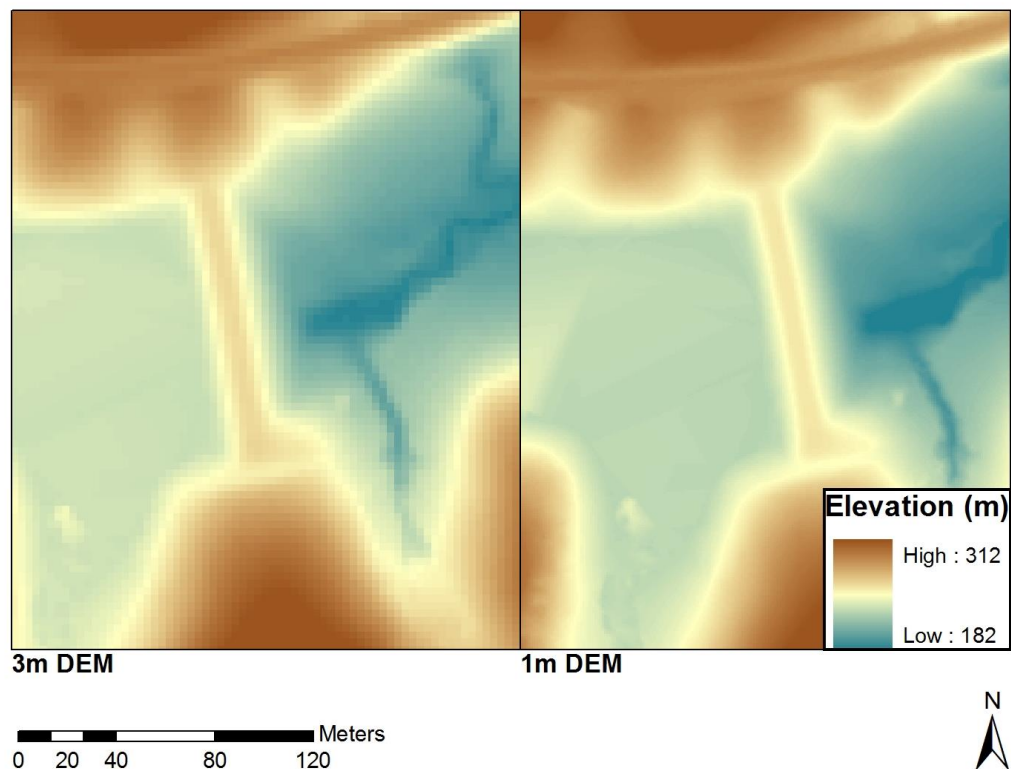


Figure 4.1: 3 meter and 1 meter DEM resolution comparison

#### 4.1.2 Stream Centerlines

In order to use the ArcHydro tools the raw DEM needs to be run through several terrain pre-processing tools that ensure that water will flow into the stream network and that all of the water flows to the outlet. The first step is to create the stream network through the DEM Reconditioning tool. Since the three meter DEM is being used, the National Hydrography Dataset (NDH) centerlines that are from 2007 may not represent the present conditions or the high resolution DEM. To account for this dataset, new stream centerlines were created by researchers at UI using the high resolution one meter DEM and are shown in Figure 4.2.



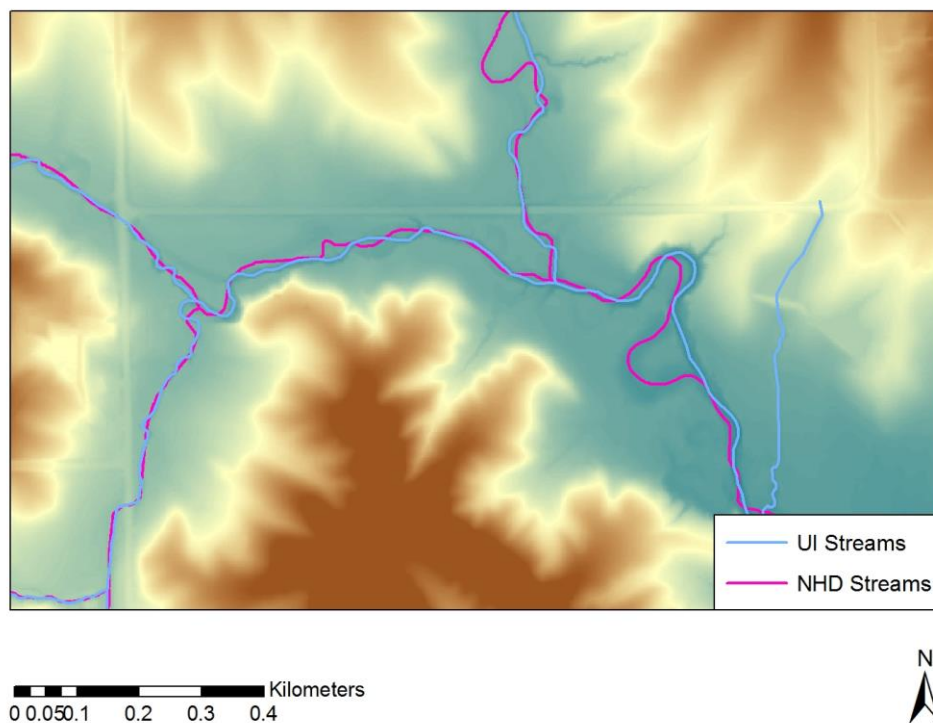


Figure 4.2: Comparison of NHD stream centerlines to centerlines created by UI

#### 4.1.3: DEM Reconditioning

When the new stream centerlines are created they might not line up with the lowest points in the DEM, so as a result water will not flow into the exact location of the stream. To correct this problem the DEM reconditioning tool inside of ArcHydro is used to lower the elevation of the areas that fall along the stream centerline. To make sure that the stream is the lowest point in the DEM the user sets the raw DEM and stream centerlines as inputs to the reconditioning pop-up window. The user must also set the number of cells that will be used as a buffer to create a smooth transition to the new low point, and set how much of a reduction in the DEM will be used. Once all of the inputs are set and the tool is run a new DEM raster is created with the default name of AgreeDEM, see Figure 4.3.

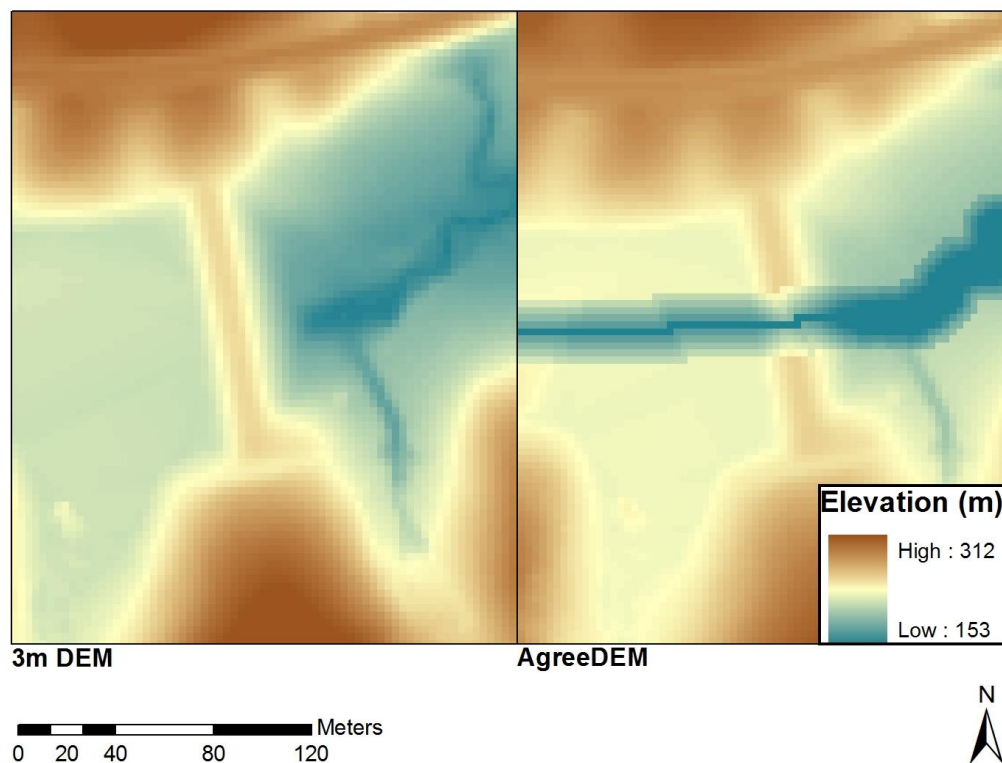


Figure 4.3: Raw DEM versus AgreeDEM

#### 4.1.4 Fill Sinks

After creating the reconditioned DEM there still might be areas where water will flow into local ponds instead of moving across the land into streams. Using the Fill Sinks tool, the DEM will be altered to ensure that water will flow into the stream network. The tool will take these “sinks” in the DEM and level them out with the highest elevation of the pond. In order to run all of the processes in HEC-geoHMS all sinks must be filled. The resulting output is another raster file with the default name of Fil, shown in Figure 4.4.

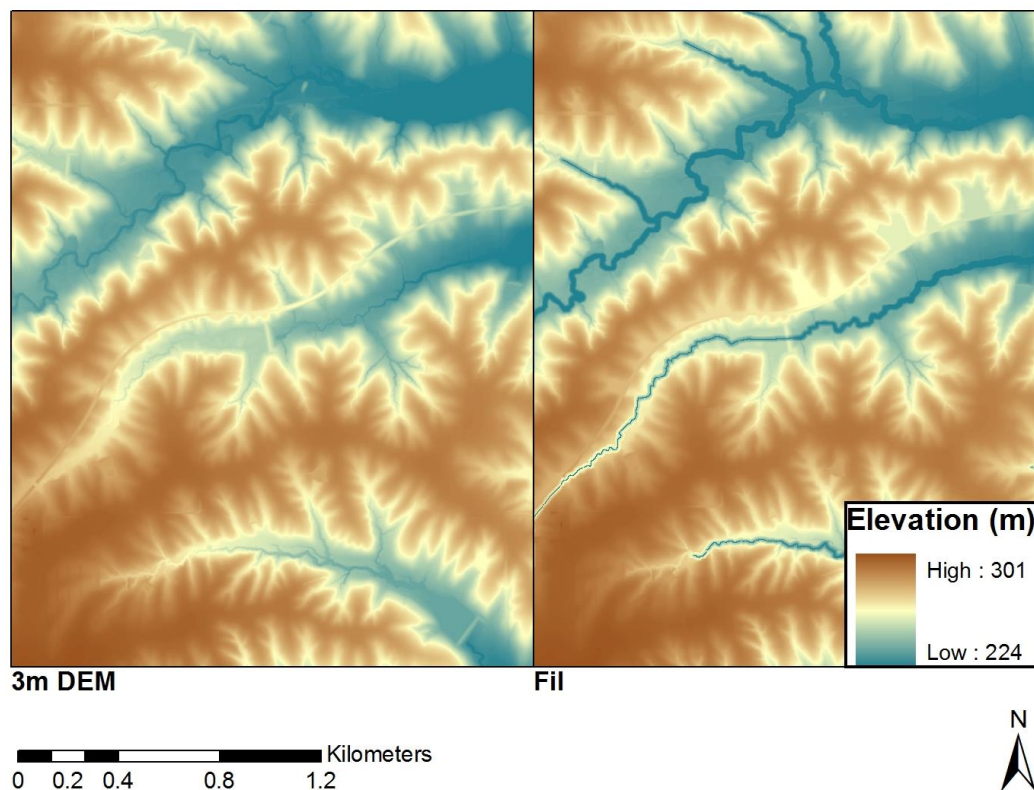


Figure 4.4: DEM showing the fill sinks operation

#### 4.1.5 Flow Direction

Now that there is a final DEM that ensures water to flow from the land to the stream network it is time to define how the water gets to the stream network. The flow direction tool will define which direction water will flow from cell to cell using an eight point compass. For each cell the tool will find which neighboring cell has the greatest elevation change that is adjacent to the main cell (Fleming & Doan, 2010). Once it finds the steepest elevation it assigns the direction that water will travel away from that cell. Once this process has run the output is a new grid that is called Fdr, See Figure 4.5.

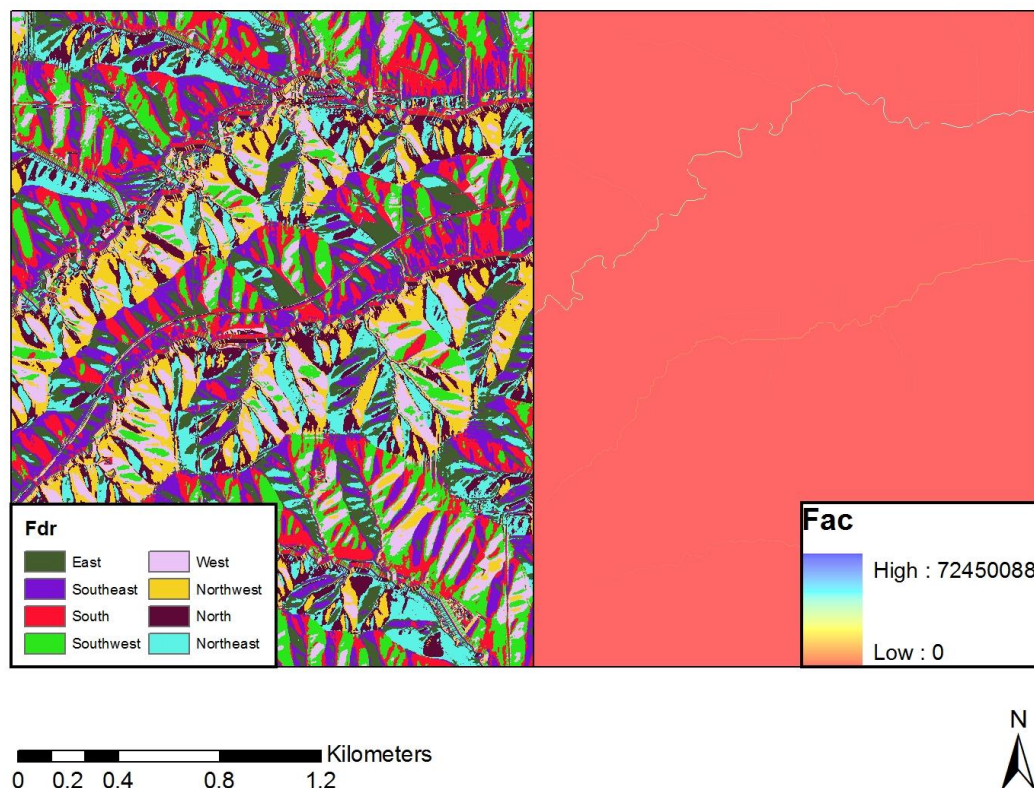


Figure 4.5: Flow direction and flow accumulation grids

#### 4.1.6 Flow Accumulation Grid

Using the flow direction grid, ArcHydro can create a new layer that sums the total number of cells that drain into a given cell. The Fdr grid is used to follow how water moves through the watershed at a given point. Since the Fdr grid is based on the reconditioned DEM as the program moves from the headwaters to the outlet, the places with the most cells that contribute to a point will form a rough stream network. The new layer that is created has the default name of Fac. A portion of the fac grid is shown in Figure 4.5.

#### 4.1.7 Stream Definition

After generating the Fac grid the next step in the generation of the layers needed for HMS is the stream definition layer. This tool will take the Fac grid and a user defined

threshold to create the stream network. The number of cells was set to be equal to the same threshold that was used in the Iowa Floodplain Mapping Project (FPM). This threshold is 24 acres or 10,792 cells based on using the three meter DEM (Thomas, 2011). This creates a new shapefile with the stream network, shown in Figure 4.6.

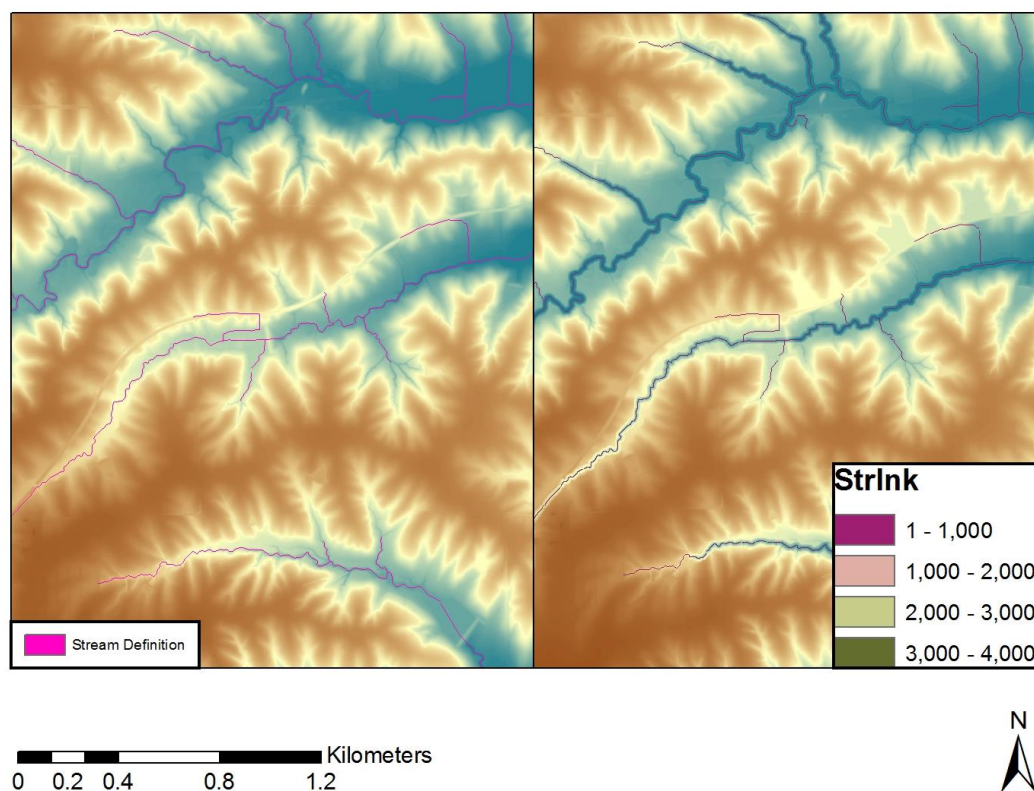


Figure 4.6: Stream definition using a 24 acre threshold

#### 4.1.8 Stream Segmentation

Once the stream network is generated the network is split into segments. These segments are sections of the stream that connect two junctions, a junction and an outlet, or a junction and the drainage divide. (Fleming & Doan, 2010) These segments form the reaches that are used in HMS. In order to run this tool the stream definition layer and the

flow direction grid are input, and will output a new shapefile called StrLnk. This layer is shown in Figure 4.6.

#### 4.1.9 Catchment Grid and Polygon

The catchment grid is created by delineating a subbasin for each stream segment. The inputs needed to run this process are the stream segmentation and the flow direction layers. The output is a new layer called Cat. After creating the catchment grid, ArcHydro can create a polygon layer of the subbasins. This step is called Catchment Polygon Processing. This process takes the Cat layer that already exists and creates a vector layer of all of the subbasins. The catchment grid is input and after the tool runs the catchment polygon is output. There were 3,857 subbasins created in Soap Creek using the 24 acre threshold. The default name is Catchment and can be seen Figure 4.7.

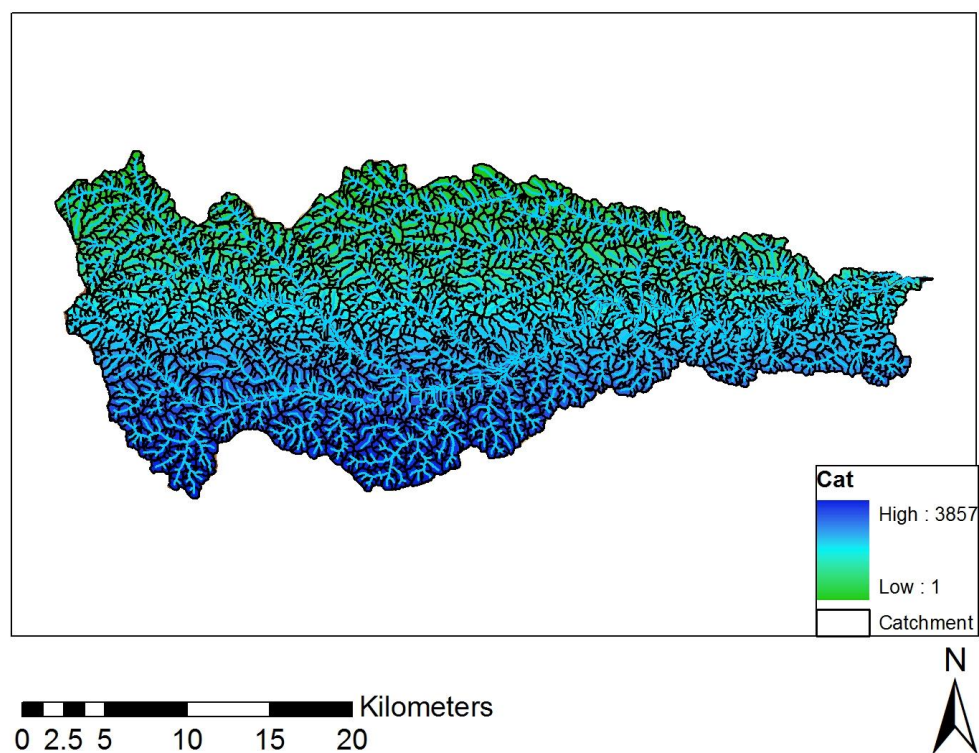


Figure 4.7: Soap Creek subbasin delineation at the 24 acre threshold results in 3,857 subbasins.

#### 4.1.10 Drainage Line Processing

Once the subbasin layer has been turned into a vector the same step needs to be taken with the stream network. This step is done with the drainage line processing tool. The inputs for this step are the stream link grid and the flow direction grid. The output layer that is created has the default name of DrainageLine. The output layer is displayed in Figure 4.8.

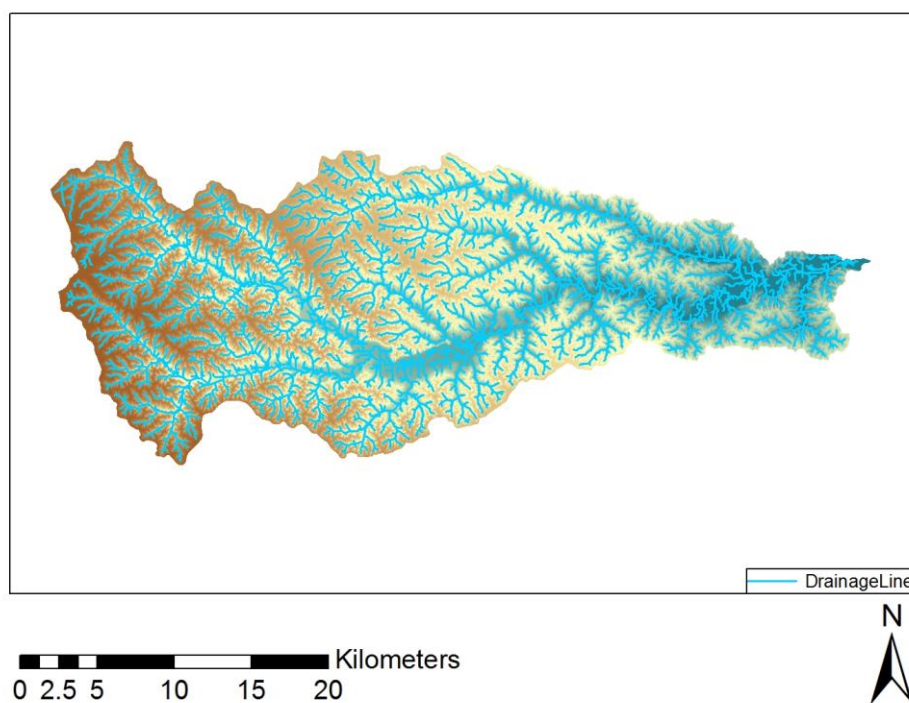


Figure 4.8: Stream network in vector form using the 24 acre threshold

#### 4.1.11 Watershed Aggregation

This step takes the Catchment and DrainageLine layers and aggregates the upstream subbasins at every stream confluence. This step is taken to improve performance when interactively delineating subbasins in HEC-geoHMS as well as to enhance data extraction. This outputs a new layer called AdjointCatchment. (Fleming & Doan, 2010) Adjoint catchment is show in Figure 4.9.

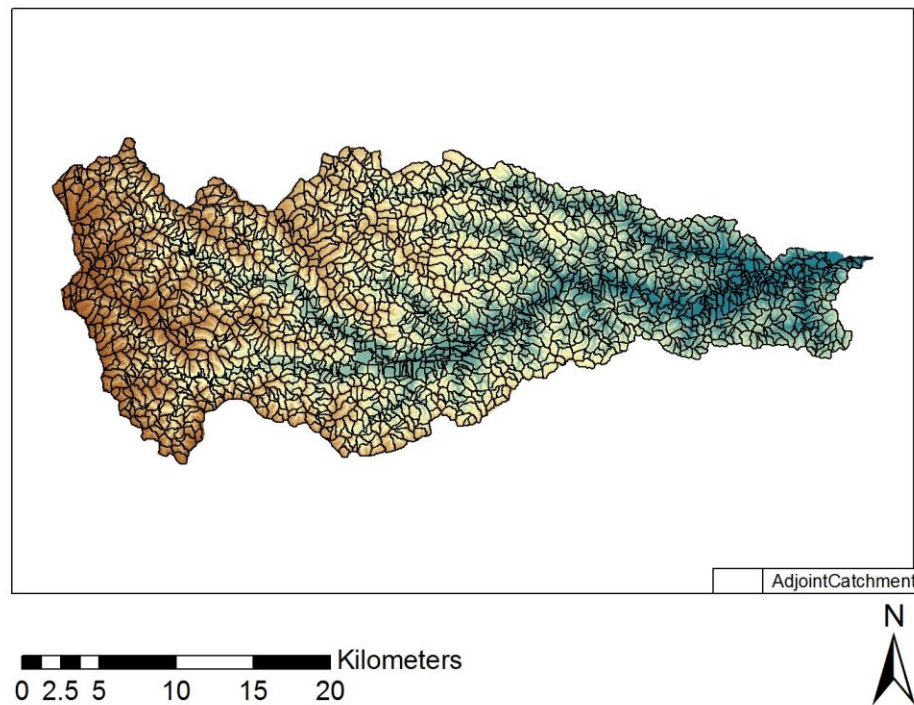


Figure 4.9: Adjoint catchment delineation output map

#### 4.1.12 Watershed Slope

The slope tool will take the raw DEM and create a new raster that contains the slopes over the entire watershed. The slope is reported in units of percent. This calculation is done by extracting the vertical units from the defined projection of the raw DEM and then applying the correct factor of the elevations to create a slope, see Figure 4.10 (ESRI, 2011).



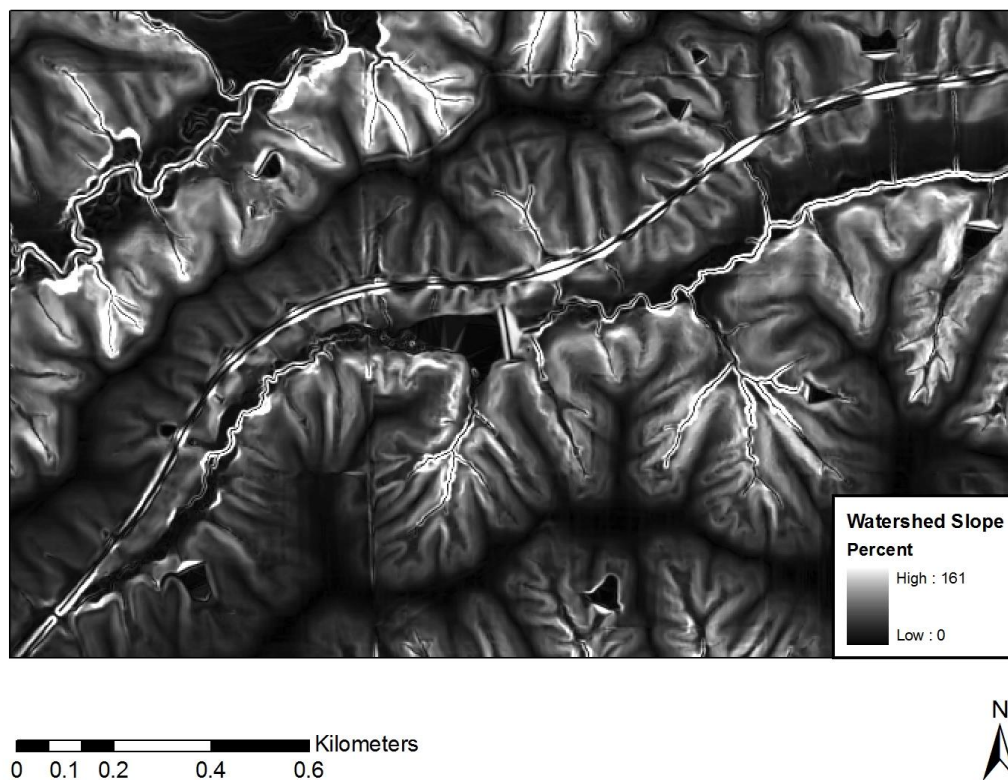


Figure 4.10: Watershed slope shown as a percentage

#### 4.2 Curve Number Grid

ArcHydro creates the layers that make up the hydrology of the model, but there are no physical characteristics for the land surface. Using the SCS methods as a way to describe the properties of the land surface provide a method to detail how water is retained and moved off the surface. The following sections describe how the curve number grid was generated using tools in GIS.

##### 4.2.1 SSURGO Data

Gathering the soil data that will be used for the creation of the curve number grid is done through the NRCS Soil Mart. Once at the online source the user must choose the state and county that will be requested. Users can only request data one county at a time. The project area for this investigation includes parts of four counties. Users are given the

option to download the data in three ways; spatial / tabular / spatial and tabular. Once at the checkout the user must select the data format and a coordinate system. Options chosen for this study were to download both the tabular and spatial data, and to format it as an arcview shapefile in the UTM Zone 15 coordinate system. Once the data is delivered the user needs to organize both the spatial and tabular data.

Merging the spatial data is done using ArcCatalog. Inside the folder where the downloaded data was sent, a geodatabase is created and two of the soil shapefiles are imported for each county (Merwade, 2012). The two shapefiles that are needed are the map unit boundary polygons, and the soil survey area boundary polygons. These two files go into a feature class, for this application the feature class was named Soap\_Soil. Another feature class that needs to be created in the geodatabase is the boundary of the watershed. Using the clip tool in ArcCatalog the four counties can be clipped to match the watershed in order to save some calculation time by trimming the excess data.

Once the spatial data have been merged the user needs to merge the tabular data. This process is done using the software and tools in Microsoft Access. Once Access is opened the user needs to navigate to one of the counties and import the data. This process needs to be repeated for each county that is needed for the project. Once the import is finished the new Access tables can then be imported into ArcCatalog. A portion of the merged soil data is shown in Figure 4.11. The tables that need to be imported are labeled as chfrags, chorizon, component, and mapunit. (Merwade, 2012)

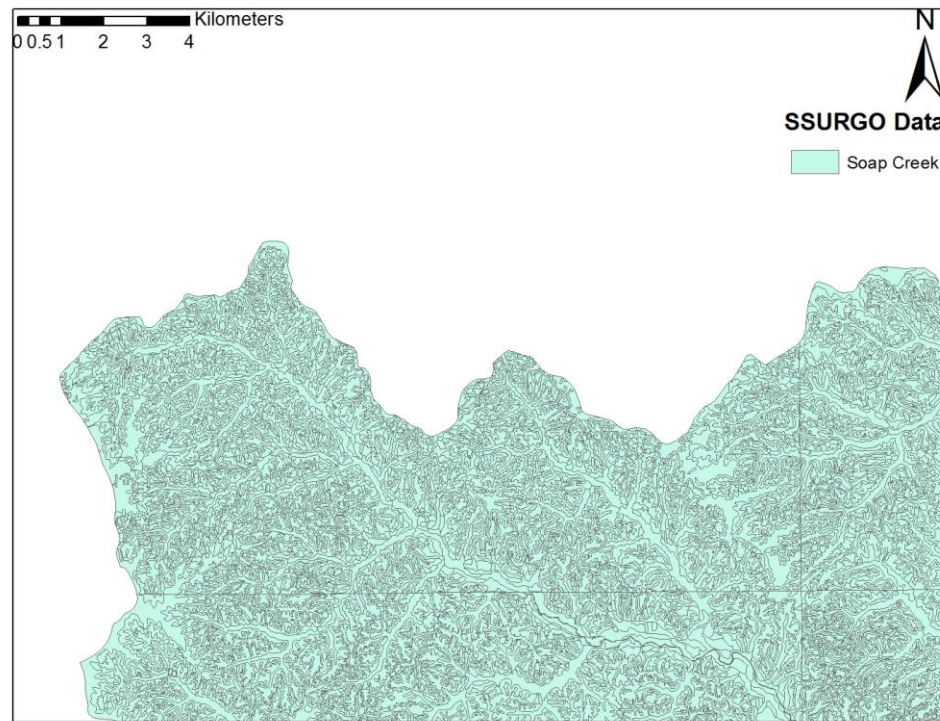


Figure 4.11: Merged SSURGO data

#### 4.2.2 Land Cover Data

After obtaining the soil data the user still needs to know what type of land cover is in place throughout the watershed to know how water will move either into the soil or over the land surface. The latest land cover data can be obtained from the USGS Map Viewer. Once the user has the spatial extent in view they can download the land cover data as an ArcGrid. The 2006 land cover was chosen as the desired dataset. Similar to the soil data once the land cover data is received the dataset can be trimmed using the watershed outline and renamed soap\_lu.

#### 4.2.3 Reclassifying Land Use Data

After clipping the land cover data there are many classifications given inside the data. To easily calculate the curve number the dataset will be manipulated into four categories; water, medium residential, forest, and agricultural. (Merwade, 2012) Using

the USGS land cover institute as a guide in grouping the land cover codes into the four groups, and the reclassify tool in the Arc Toolbox for spatial analyst, the original land cover data is converted into the four new classes data set. The new data is output in a new layer called lu\_reclass. One final step in converting the data into a suitable format to match the soil data is done by using the conversion tool, Raster to Polygon in Arc Toolbox. The inputs for this process are the lu\_reclass raster and changing the field option to value. The reclassified land use map is displayed in Figure 4.12. After running the tool there is a new layer named landuse\_poly.

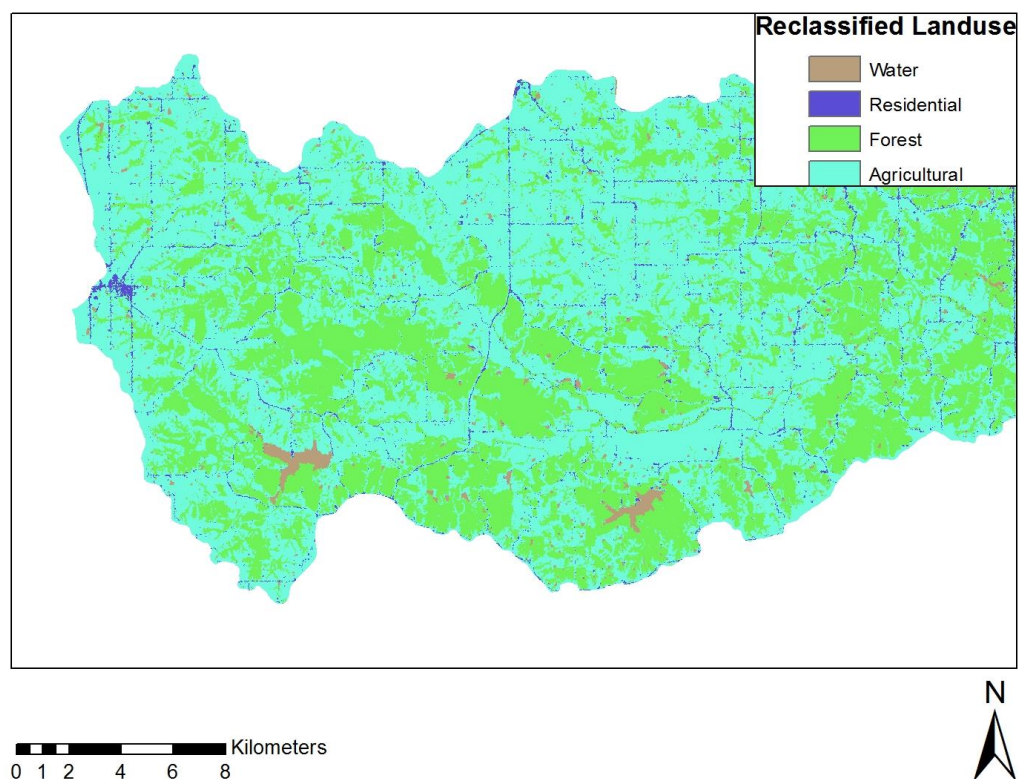


Figure 4.12: Final reclassified land use map

#### 4.2.4 Creating Soil Code Table

To be able to extract curve number values there needs to be a soil group for each polygon in the soap\_soil\_clip feature class. In the attribute table for this layer the user must add the field SoilCode with the type of field set to text. The data needed to fill this new field is inside the component table from the SSURGO data geodatabase from Section 4.2.1. In order to fill this field, a join between soap\_soil\_clip and the component table from Section 4.2.1 needs to be created. Using the join function in the soap\_soil\_clip attribute table, select mukey as the field in the layer you want to join, and select mukey as the field in the table to base to join on. This will result in all of the fields from the component table being shown in the soap\_soil\_clip attribute table. Using the field calculator equate the SoilCode to the component.hydgrp values. (Merwade, 2012) This will fill the SoilCode field with the appropriate soil type of A/B/C/D, See Figure 4.13 for a soil type distribution example. The soil types are classified on how much runoff potential there is. Type A has the lowest potential while Type D is the highest (NRCS, 1986). Checking the soil table there might be values that are labeled as <null>. Using the data round these values they can be filled in to match the surrounding values. Once the SoilCode field is filled, add four columns to the table that represent the percentages of each soil type for each polygon. For dual soil codes, a 50/50 split was assigned to each soil type. Each field has to be filled in order to continue so each entry will have a 0, 50, or 100.

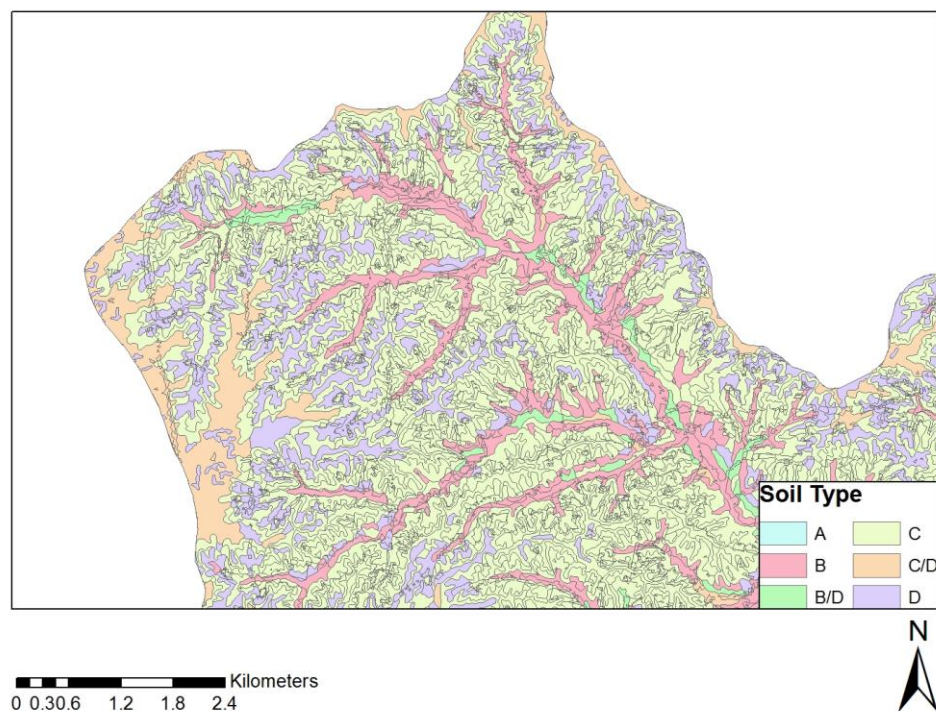


Figure 4.13: Section of Soap Creek soil type

Once the percentages are filled in, the soil and land cover data are ready to be merged. Using the union tool in the ArcToolbox under analysis tools, input the soap\_soil\_clip and the landuse\_poly features. Name the output as soap\_soil\_lu and run the tool. This new layer will have the features from both inputs, but there may be data around the edges of the watershed that have missing data that will have a value of -1 in the attribute table. These polygons represent areas that only have values from one of the input layers (Merwade, 2012). They can be found in the column named FID\_soap\_soil\_clip. Select all of the values with -1 and delete them. Once this step is completed the surface data is ready to be used to create the curve number grid.

#### 4.2.5 Curve Number Generation

Using ArcCatalog create a table inside the soil database created in Section 4.2.1. Name the table CNLookUp and create fields for the land use value, description, and the

A/B/C/D soil codes. Using the SCS TR55 report from 1986 the table can be filled with the corresponding values for each soil code, See Table 4.1. Using the HEC-GeoHMS Project View toolbar select the Create Parameter Grids tool. Once selected, enter the corresponding data for the Hydro DEM, Soil Landuse Polygon, and Curve Number Lookup, and name the output as CNGrid. Once the tool finishes a new layer is created with the final curve number grid for the basin. The Soap Creek curve number grid is show in Figure 4.14.

Table 4.1: Curve number look up table for assigning curve numbers based on soil types.

Landuse Value	Description	A	B	C	D
1	Water	100	100	100	100
2	Residential	57	72	81	86
3	Forest	30	58	71	78
4	Agricultural	67	77	83	87

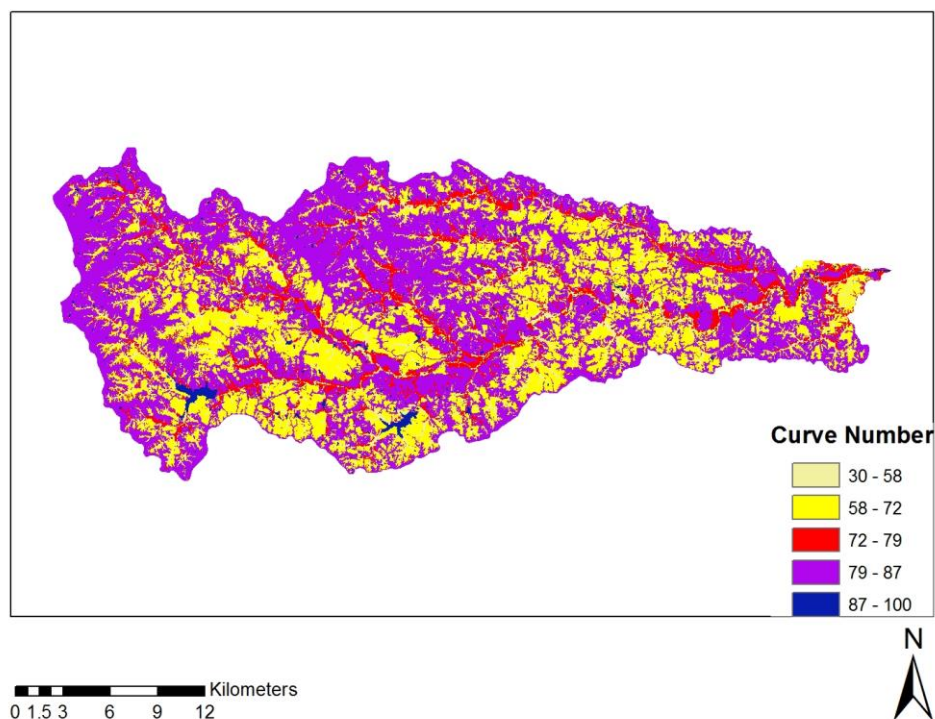


Figure 4.14: Curve number grid showing distribution of curve numbers over the entire Soap Creek watershed

### 4.3 HEC-GeoHMS

Now that all of the necessary basin layers and curve number grid have been created, the characteristics of the basin need to be filled in. Using the tools in the HEC-GeoHMS Project View and HEC-GeoHMS Main View toolbars, GIS is able to calculate and populate the necessary fields to create the export files needed to create the HEC-HMS model. The processes required to create all of the data for the export files will be discussed in the following sections.

#### 4.3.1 Project Setup

In order to use Project View, there first needs to be a project in which to use the tools. Selecting start a new project from the Project View setup will open a new window in which the user can accept the defaults of the ProjectArea and Project Points layers. Project area will show the upstream area of the outlet point and the project point will show the location of the outlet. Once these layers are created the program will ask to define the project.

Here the user needs to name to project, give a description, choose an extraction method, and to set a location to where all new data will be stored. For this example the extraction method that was selected is original stream definition, this option uses the same stream segments and subbasins that were delineated using the terrain preprocessing tools described in Sections 4.1.8 and 4.1.9.

After completing this step the program will ask to select a project point. This is done by zooming in to the area of the outlet using the Fac and StrLnk grids. Select a point near the desired outlet and be sure to click on a cell that is highlighted by both the Fac and StrLnk layers. This will open a new window that will ask the user to check the layers necessary to generate the project. They layers included are the following; Raw DEM, Fil DEM, Fdr, Fac, Str, StrLnk, Catchment, and AdjointCatchment. Once the necessary layers are selected to the proper locations select OK to continue. This will clip



and generate a new data frame in GIS with all of the layers starting with the outlet and working upstream of that location to the map boundary.

#### 4.3.2 Basin Processing

After generating the project the user is able to modify the watershed to their project needs. The first step is to finalize the subbasins. Users can manipulate the number of subbasins by using the merge and subdivide basins tools. When merging subbasins the tool will only allow basins that share a common confluence or are immediately up or down stream of each other to be merged (Fleming and Doan, 2010). Once subbasins are selected to be merged a warning window will pop up asking to confirm the new area before the program will continue with the process. When splitting a subbasin the user can only operate along the stream network. The options for splitting are to split at a confluence or to split at any point along a reach. After running the basin split and basin merge tools the Soap Creek watershed was delineated into 302 subbasins. See Figure 4.16 for the final subbasin and river network.

Once the user had finalized the number and layout of their subbasins they have the option to merge or remove stream segments. The river merge tool allows for joining segments to be modeled as one reach in HMS. River segments were removed to lessen the number of elements that would be created so that each subbasin would only have one stream running through it. After removing the extra stream segments there were 565 reaches remaining. This allows for more than one segment in a subbasin. Another option available to users in the basin processing menu is the ability to see the river profile. This can be viewed by segment or for a longer section of a stream by simply highlighting one or more segments of the stream network that are connected to one another and selecting the river profile tool. The final distribution of subbasins, and river network is show in Figure 4.15.

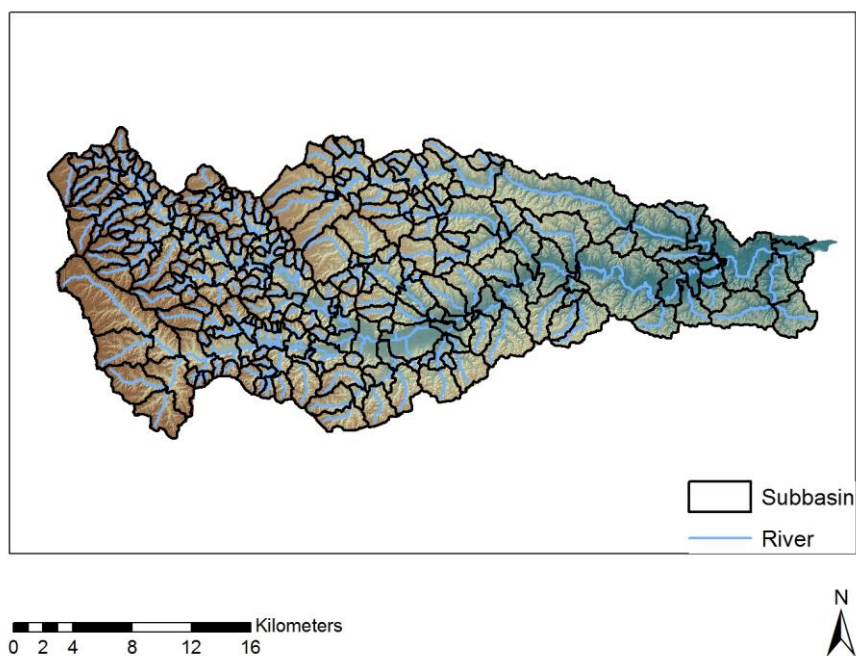


Figure 4.15: Soap Creek's final subbasin delineation and stream network from HEC-GeoHMS

#### 4.3.3 Basin Characteristics

GeoHMS has the ability to calculate many of the characteristics for the streams and subbasins using the topographic information provided by the layers created during the terrain pre-processing steps. These characteristics are then stored in the attribute tables for the river and subbasin layers.

The river length and slope are calculated for each segment that remains in the river layer after any alterations may have been made using the merge tools described in the previous section. The length is based on the number of cells each segment intersects and is assigned the same units as the DEM. (Fleming & Doan, 2010) The river slope is calculated by the elevation at the upstream and downstream ends of each segment and then divided by the length of the segment to compute the slope. If no vertical units are included in the DEM spatial reference the program will then assume the units for both the horizontal and vertical are the same.

Basin slope is calculated by inputting the slope field that was generated using the slope function in ArcHydro and the subbasin layer. The slope is generated by averaging the slope values over each subbasin. The slope values that are calculated will be used in the calculation of the curve number lag that was discussed in Section 3.2.2.

Calculating the longest flow path will create a new layer and fill in a number of physical characteristics. For each flow path the length, up and down stream elevation and the slope between the points will be calculated. The longest flow path is chosen as the flow path that has the greatest length from a subbasin boundary to the subbasin outlet. (Fleming & Doan, 2010) See Figure 4.16 for an example of the output shapefile.

The basin centroid tool creates a new layer that will contain the centroid of each subbasin. There are four ways to calculate this location, using center of gravity, longest flow path, fifty percent area, and user specified. For this application the center of gravity method was chosen. After this process is complete the centroid elevation tool is selected, and this tool uses the DEM to assign the elevation of each point inside the centroid layer. The basin centroids are shown in Figure 4.16

The last process to be run in the basin characteristics menu is the centroidal longest flow path. This tool calculates the longest flow path by projecting the centroid onto the longest flow path that has already been created. (Fleming & Doan, 2010) Once the location of where the centroid intersects the longest flow path is found a new length is calculated from that point to the subbasin outlet and the information is stored in a new layer called CentroidalLongestFlowpath. The processed centroidal longest flowpaths are shown in Figure 4.16.

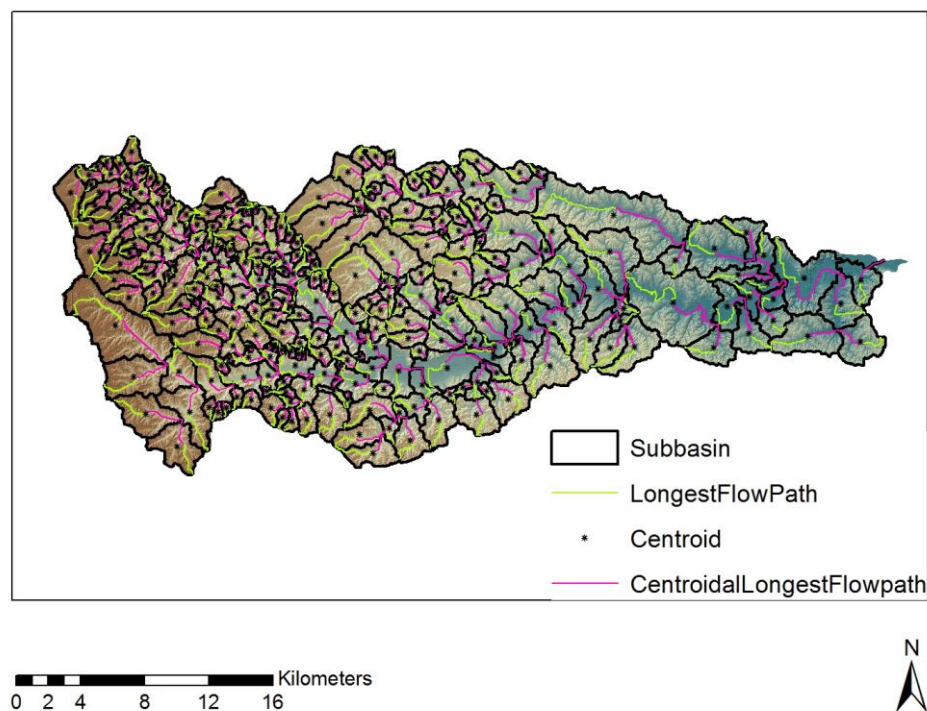


Figure 4.16: Basin characteristics of Soap Creek

#### 4.3.4 Hydrologic Parameters

GeoHMS has the ability to calculate many of the hydrologic parameters used in HMS by way of the data in the soil and land use tables. The first step is to decide what processes are going to be used in the HMS model for the loss, transport, and routing methods. Using a dialog window that provides a dropdown list for each method available the user is able to select the methods that will be used. For the model of Soap Creek the SCS Loss and SCS Unit Hydrograph Transform methods were selected and the routing method was left empty.

Following the selection of the HMS processes, GeoHMS can assign names to all of the reach and subbasin elements. The auto naming tool assigns each subbasin a name starting with a W for watershed and any reach a name starting with a R. These names carry over into HMS as a way to identify and sort the elements in the user interface.

In order to get the subbasin averaged curve number, GeoHMS needs to run a script that can run zonal statistics. Using the subbasin parameters from raster tool, GeoHMS can calculate the SCS loss parameters of initial abstraction, curve number and percent impervious. (Fleming & Doan, 2010) After selecting the tool a window will ask for inputs to a variety of optional grids. For the purposes of this study only the curve number grid was selected. Once the script runs the subbasin averaged curve number will be calculated and shown in the subbasin attribute table.

The last parameter that needs to be calculated is the curve number lag time for the transform method in HMS. In order to run this script the basin curve number, basin slope and the longest flow path fields all need to have been calculated previously. Using the NRCS National Engineering Handbook from 2010 the lag time can be calculated from the equation described in Section 3.2.2.

#### 4.3.5 Grid Cell Processing

Another important tool inside the hydrologic parameter menu is the grid cell processing setup. This tool sets up a new layer that serves as a key for the use of gridded rainfall. There are two grid definitions to choose from, Standard Hydrologic Grid (SHG) or the Hydrologic Rainfall Analysis Project (HRAP). The HRAP grid system is used by the National Weather Service (NWS) as their method of reporting radar rainfall and was selected for use in this project. HRAP consist of cells that are 4.7625 km. The Y axis is set to align with the central meridian which is located at 105<sup>0</sup> W. The coordinates are set that as the location moves east and north the coordinates will increase. The North Pole is the upper right hand corner of the extent and is located at (401, 1601) (Fleming & Doan, 2010). Once the grid cell processing tool runs the output is a series of layers. This output layer is shown in Figure 4.17. The GridCellIntersect layer contains the data necessary for the application of radar rainfall. The attribute table shows what HRAP cells are contained within each subbasin. This map layer will provide the information for the

application of radar rainfall while running the historical storms that will be discussed in Section 5.3.

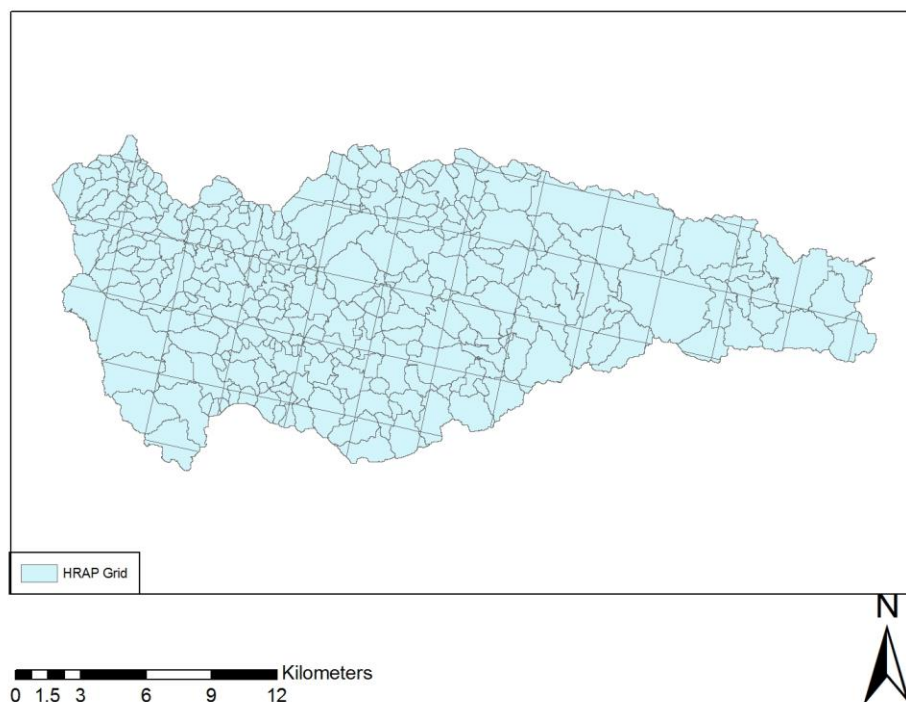


Figure 4.17: Soap Creek subbasins intersected with the NWS HRAP grid

#### 4.3.6 Building HMS

Once all of the basin and hydrologic characteristics have been calculated and assigned to the subbasin and river shapefiles, the export files need to be created. The options inside of the HMS menu in GeoHMS will assist in creating these files that will be exported to HEC-HMS. The first step is to convert the ArcMap to HMS units. After selecting the layers to match the requested data the program will ask to select the units that HMS will report in. This study was done using data that was collected in metric units so the SI reporting method was selected.

Once the units have been converted, the next step is to run a script that checks for errors in the HMS layers. This script checks for unique names, river containment, centroid containment, river connectivity, and VIP relevance. Unique names checks that

there are no repeating names for the river segments and subbasins. River and centroid containment checks for any stream segments or centroids that are not contained by the watershed. River connectivity checks for any places where the river has any discontinuities. These checks are in place to keep track of the relationship between elements that may have been broken by use of subdivide and merge tools (Fleming & Doan, 2010). Once the check runs, the user can open a file that contains the errors. Any errors that come up in the file can be fixed using GeoHMS or in HMS.

When any errors have been fixed the user can then create a schematic of the HMS layout in GIS. This schematic shows the connectivity of the elements that will be exported into HMS. There are two layers that are created when the basin schematic tool is run, the HMS Link and Node layers. HMS Link shows the connectivity and HMS Node show the location of the subbasin and junctions elements. See Figure 4.18 for the HMS element locations and layout. Once the process is complete the user can toggle the legend between HMS and Regular.

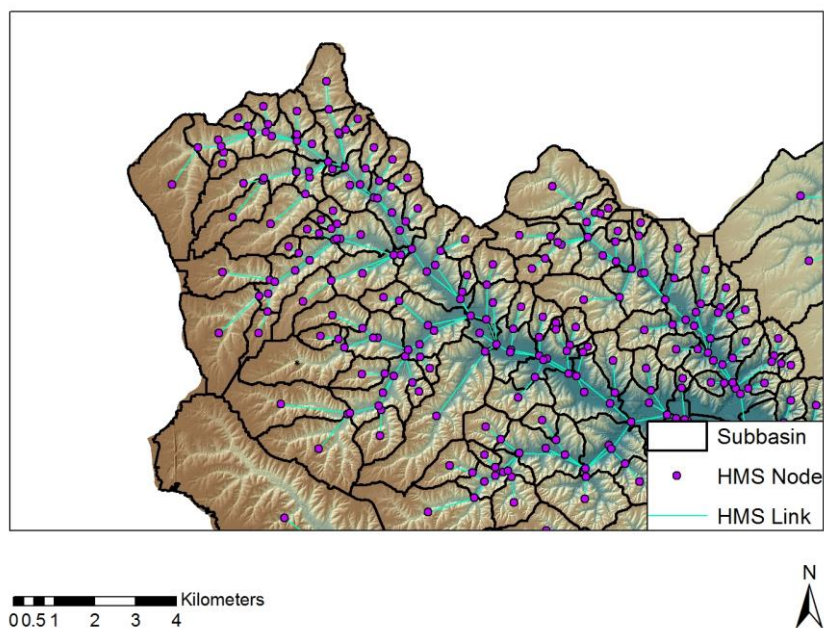


Figure 4.18: HMS element locations

The HMS schematic that was created in the previous step does not have geographic coordinates. The Add Coordinates script adds coordinates to the attribute tables of HMS Link and Node. Adding the coordinates allows for the export file to be in ASCII format and keep the geospatial information. The data now needs to be prepared for export. Not all of the parameters required to run a HMS model can be created in GeoHMS. Once the Prepare Data for Model Export tool is run, all of the parameters that were calculated in the subbasin and river layers are moved into a basin file that can be read into HMS.

#### 4.3.7 Generating Export Files

The first files that can be created are ones that are optional. The background shape files of the basin and river are created by using the Background Shape File Tool in the HMS menu. This creates a file in the HEC-HMS library with a .map extension. Another option is to import the Arc shapefiles in HMS.

The next file to export is the basin model file. This tool can be found in the HMS menu under Basin Model File. The basin model contains all of the hydrologic elements, their connectivity and geographic information. The data is output in an ASCII file in the same library as the map, but has an extension of .basin.

Now the grid cell file is ready for export. This file is exported as an ASCII file that contains the data showing which grid cells are contained in each subbasin. This file has an extension of .mod.

Meteorological models can be created in GeoHMS and exported which will add a new file that ends with a .met extension. There are only three models that can be set up; specified hyetograph, gage weights, and inverse distance. The specified hyetograph method creates a gage that has the same name as the corresponding subbasin. This meteorological model method was created and exported.



The last step in GeoHMS is to create the HEC-HMS Project. This tool will move all of the created export files to the location that the user selects to store HMS data. This tool will also create the .HMS file that serves as the main file that contains all of the HEC-HMS project information. Once all of the export files are brought into HMS any remaining parameters that were not calculated in GeoHMS must be filled before running a model.

#### 4.4 Reach Building

Selecting the routing method of modified puls requires a storage-discharge table for each reach. For cross section analysis the 1 meter DEM was used in order to get the maximum detail and accuracy possible in all calculations. Using tools in GIS and converting the geometry data into values of volume and discharge are discussed in the following sections.

##### 4.4.1 Cross Sections

For each of the 565 reaches that were created in the GeoHMS process, a storage-discharge relationship was needed to provide the routing in HMS. The geometry of each reach was extracted using the 3D Analyst in GIS. See Figure 4.19 for an example of a cross section. We drew one cross section per reach to provide simplicity in the calculations as it assumes one geometry for the entire reach length. The cross sections were chosen in areas that best represented where the midpoint of a stream was in order to get an average channel geometry. Cross sections were drawn from left bank to right bank looking in the downstream direction in an effort to maintain uniformity throughout the data collection process. Once the cross sections were drawn, the data was then saved in the format of Microsoft Excel spreadsheets. An example cross section and location is seen in Figure 4.19.

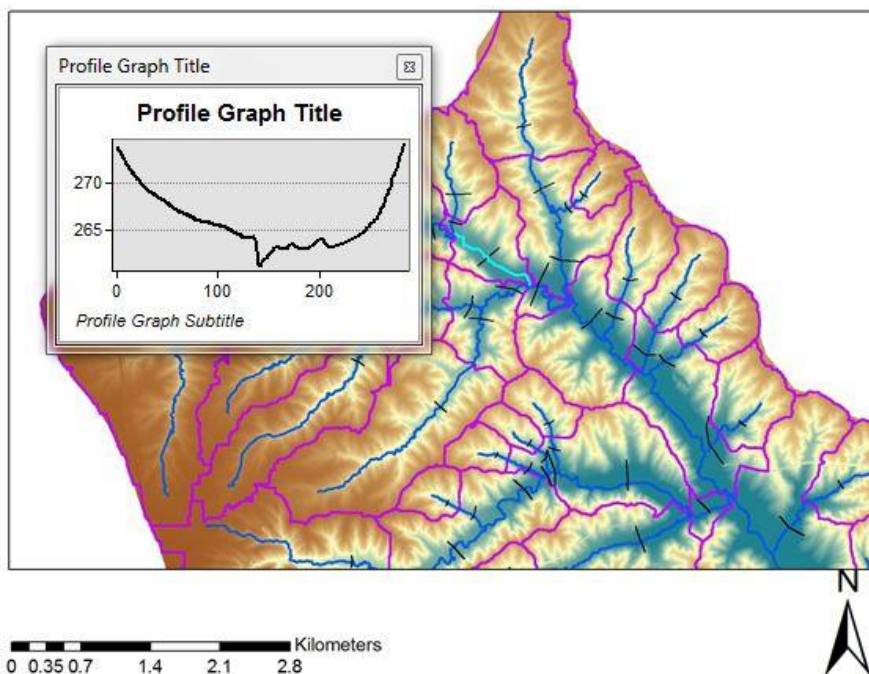


Figure 4.19: Cross Section of a single reach in GIS, reach R96890 is highlighted

#### 4.4.2 Storage-Discharge Tables

We used Manning's Equation as the basis to get the stream velocities, the first calculations were to get the area and wetted perimeter. Calculating the area between a given water elevation and the channel cross section required the use of an If statement in excel. The function called on a water surface elevation to be input and if the cross section elevation was lower than the water surface to calculate the area between the water surface and the channel bed. If the water surface elevation was higher than the channel elevation then a zero was returned in the area column. The input if statement was written as follows:  $=\text{if}(\text{average}(y_1:y_2) < \text{water surface elevation}, (\text{water surface elevation} - \text{average}(y_1:y_2)) * (x_2 - x_1), 0)$ . The next calculations were to find the differential changes in both x and y at each new data point. The calculation for wetted perimeter required another if statement similar to the first as the value needed to be calculated between the water surface and the channel bed. Once again all calculations for channel elevations that were above the water surface elevation, a zero was returned. The statement used to

calculate the wetted perimeter is as follows:  $=\text{if}(y_2 \leq \text{water surface elevation}, (\text{sqrt}(dx^2 + dy^2)), 0)$ . The area and wetted perimeter for each water surface elevation was summed over the entire cross section to get a single value for each input elevation. These calculations were repeated to form the basis of the storage-discharge tables. Table 4.2 shows an example of a completed storage-discharge relationship.

Table 4.2: Storage-Discharge table for reach R96890

Elevation (m)	Depth (m)	Volume (1000 m <sup>3</sup> )	Discharge (m <sup>3</sup> /s)
261.1174	0.000	0.000	0.000
261.25	0.133	0.144	0.024
261.5	0.383	0.851	0.280
262	0.883	3.814	2.038
262.5	1.383	9.631	7.319
263	1.883	18.427	13.530
263.5	2.383	48.921	34.684
263.75	2.633	70.915	58.635
264	2.883	95.826	88.644
264.25	3.133	124.876	125.150
264.5	3.383	157.723	173.465

The other variables for Manning's Equation are Manning's N and slope. Slope was calculated in GeoHMS for each reach so this value was brought into excel. Manning's N was assumed based on drainage area similar to FPM practices (Thomas, 2011). Drainage areas less than 10 mi<sup>2</sup> were assigned an N value equal to 0.045 and drainage areas greater than 10 mi<sup>2</sup> were assigned 0.035. Drainage areas were calculated using the same methodology as FPM. The area for the last drainage point in a given reach was used as the reference point to assign the Manning's N. Using these variable inputs as well as the area and wetted perimeter values that were calculated in the previous step the stream velocities were calculated using Manning's Equation.

$$V = \frac{1}{n} \left( \frac{A}{P} \right)^{\frac{2}{3}} S^{\frac{1}{2}}$$

V=Velocity (m/s)

A=Area (m<sup>2</sup>)

P=Wetted Perimeter

S=Slope

$$Q=AV$$

Stream volumes were calculated by multiplying the stream length, which was calculated in GIS by GeoHMS, by the volume of the stream at a given elevation. SI units were used throughout resulting in volumes of cubic meters. For use in HMS the metric unit of volume is 1000's of m<sup>3</sup> so each volume was then divided by 1000 to achieve the desired unit.

Discharge was calculated by multiplying the area by the velocity at a given elevation. Using SI units this results in a discharge given in cubic meters per second (cms). This is the standard unit in HMS so no manipulation was needed to this variable. Once these calculations were completed the final storage-discharge calculations were ready for use in HMS.

#### 4.5 Pond Building

The HMS program uses the element of the reservoir to model ponds. When modeling ponds there needs to be a storage-discharge relationship similar to what was used in the routing method through the streams. Once knowing the location of all 132 structures that have been built in the basin, finding the areas of each to use in calculating the volume and storage of the outlet could be done in GIS. This doesn't provide enough information as I still would need to know many of the design specifications such as pipe diameter, pipe material, pipe elevation, and spillway elevation among others to calculate the discharges. With the knowledge of the ponds being built to NRCS specifications a

request for data was sent to the local NRCS offices, See Figure 4.20 for an example of point locations. This led to a data gathering trip in Des Moines, as the state NRCS office had the design data on all of the constructed ponds. Included in the design reports were stage-storage-discharge relationships for each pond. This data was transferred from physical paper into an excel datasheet to be used in HMS. The tables were reported in English units of feet for elevation, acre-feet for volume, and cubic feet per second (cfs) for discharge, See Table 4.3 of the input table of Project 68-47. These tables were directly input to HMS in English units as the program can perform the conversions when the model is run.

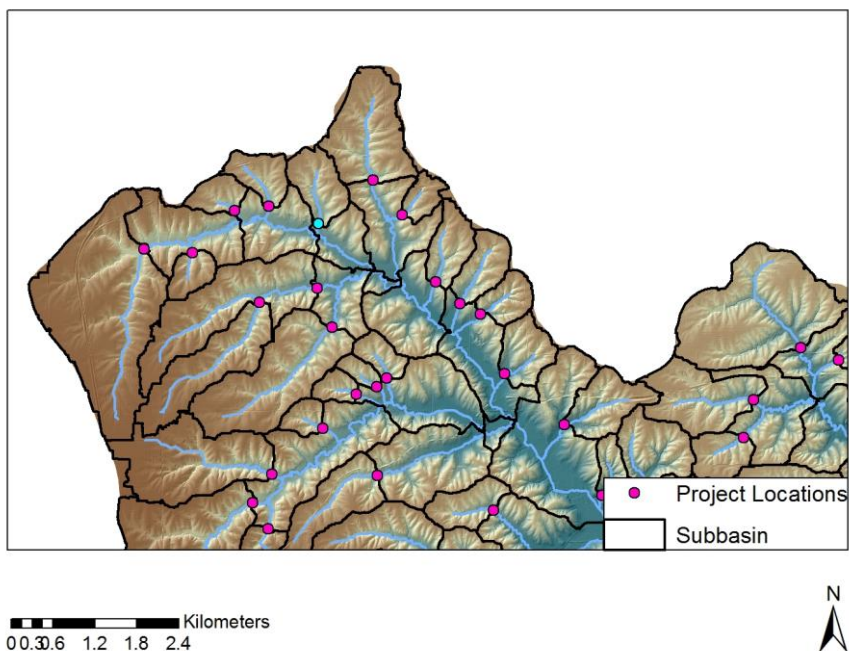


Figure 4.20: Location of Soap Creek Projects, project 68-47 is highlighted

Table 4.3: Storage-Discharge table for Project 68-47

Elevation (ft)	Storage (ac-ft)	Outflow (cfs)
888	0	0
889	3.62	1.78
890	9.67	8.12
891	16.42	8.38
892	23.87	8.63
893	31.91	8.87
894	40.42	9.11
894.5	44.92	19.67
895	49.41	52.25
895.5	54.15	114.86
896	58.88	194.36
896.5	63.93	297.57
897	68.98	431.34
897.5	74.43	591.93
898	79.88	807.35
899	91.58	1232.21
900	104.08	1793.39

#### 4.6 Chapter Summary

ArcGIS provides many powerful tools that can be used to create the many layers and parameters that are needed to make a HMS model. ArcHydro is used to set up the geometry and base layers that will be used to populate the basin characteristics. HEC-GeoHMS provides tools to alter how the subbasins and stream network layout. GeoHMS also calculates many of the parameters that are associated with the basin model inside HMS. The program is able to calculate many of the loss, and transform parameters as well as set up some of the baseflow methods and meteorological models. The creation of the HMS model layout in terms of element number and location are addressed in GeoHMS. The files that create the basin, meteorological and project files are created and exported as part of GeoHMS.

Once the files are imported into a HEC-HMS project the missing parameters need to be calculated. For the Soap Creek model this primarily consists of adding the storage-

discharge tables for the reach lengths and ponds. Reaches were created using single cross sections and Manning's Equation. Pond data was obtained through the previous design work done with the NRCS.

## CHAPTER 5: RESULTS

A series of 24 hour SCS design storms ranging from the 2yr to 100yr return period were modeled as well as three historical storms using NEXRAD rainfall data. The historical storms were selected from knowledge of recent heavy rain events across the state. June 2008 and 2011 and April 2013 were selected for historical analysis. A March 2013 storm was selected to serve as a comparison to new stage data from IFC stream sensors in Soap Creek, and a sensitivity run was calculated by adjusting the curve number to the various antecedent soil moisture conditions. Each modeled scenario was run using the following project conditions; No Ponds, Ponds built before 1993, Ponds built before 2005, ponds built before 2008, and all Ponds built before 2013. Figure 5.1 shows the locations of points where data is collected and presented for each model. Table 5.1 shows the area upstream of each hydrograph point and the area that is controlled by ponds in each region.

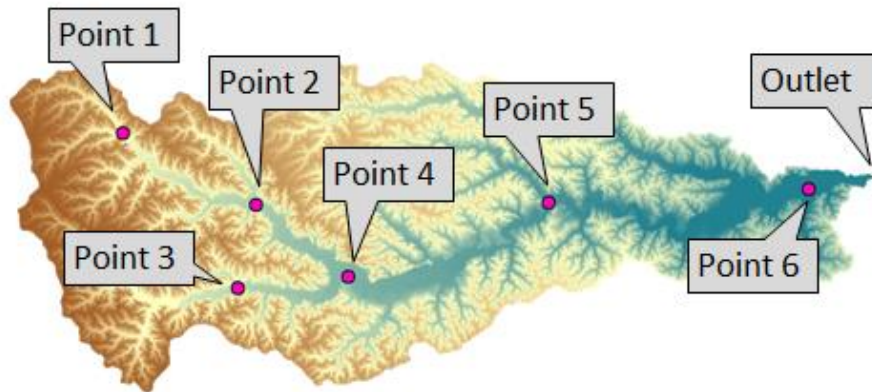


Figure 5.1: Hydrograph and calculation locations



Table 5.1: Table showing the area upstream of each hydrograph point, area controlled by ponds for each point and the percentage of the area that is controlled

	Total Area Upstream (acres)	Area Controlled By Ponds (acres)	Percentage Controlled
Point 1	8,594	4,499	52%
Point 2	29,376	13,085	45%
Point 3	19,176	1,210	6%
Point 4	61,739	20,885	34%
Point 5	106,942	30,641	29%
Point 6	159,857	37,936	24%

### 5.1 Validation

Four IFC stage sensors were deployed in Soap Creek in November 2012. These sensors record the stage of the stream and store the data to be used by IFC researchers. In the time since the sensors have been deployed there has been little rainfall and no flooding. Finding a stream response to the spring melting conditions supplied data that could be used to visually inspect if the model is performing as the stream would. During the period of 8-11 March 2013 the state received a steady rain when combined with mild temperatures melted most of the snowpack that was present in the basin. With the ground still frozen from the winter and snowpack this provided a sample of how the watershed responds to a increased flow, and would provide a validation of model performance. The validation and calibration that is being performed is to visually check the timing and shape of the modeled hydrograph compared to the time series of stage from the IFC gages. There are currently no rating curves for the IFC gages in Soap Creek and as a result a direct calculation from stage to discharge was not used for these simulations. I was able to compare stage from the IFC gages to output discharges from the HMS results using the storage-discharge tables at the location of the sensors. Once I had the peak from HMS, I was able to go backwards through the location storage-discharge table to get an approximate stage value.

Figure 5.2 displays the hyetograph that was generated from the hourly rain gage in Ottumwa, Iowa from 9 March until 11 March 2013. A storm total precipitation of 3.9 inches was recorded over the 2 day period with a majority of rainfall rates no greater than 3 mm per hour. Figure 5.3 indicates the locations of two IFC gages that were reporting during the March event. The gages were located in the Little Soap Creek and on the main stem of the Soap Creek.

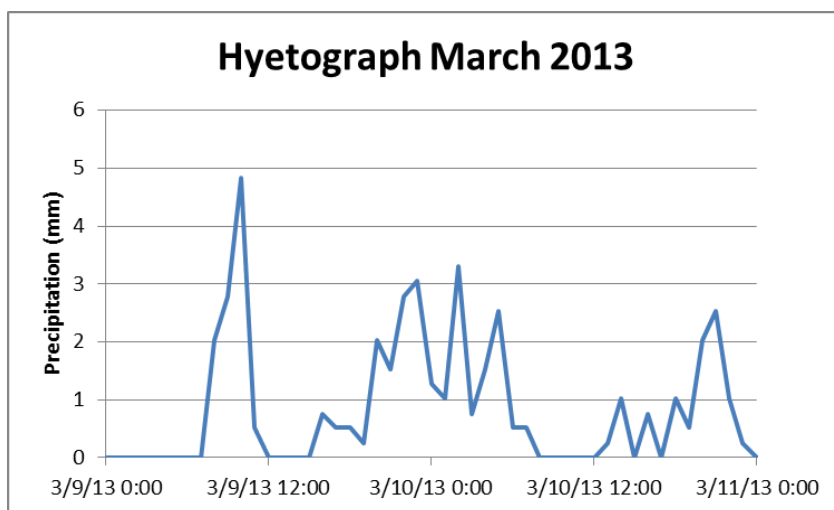
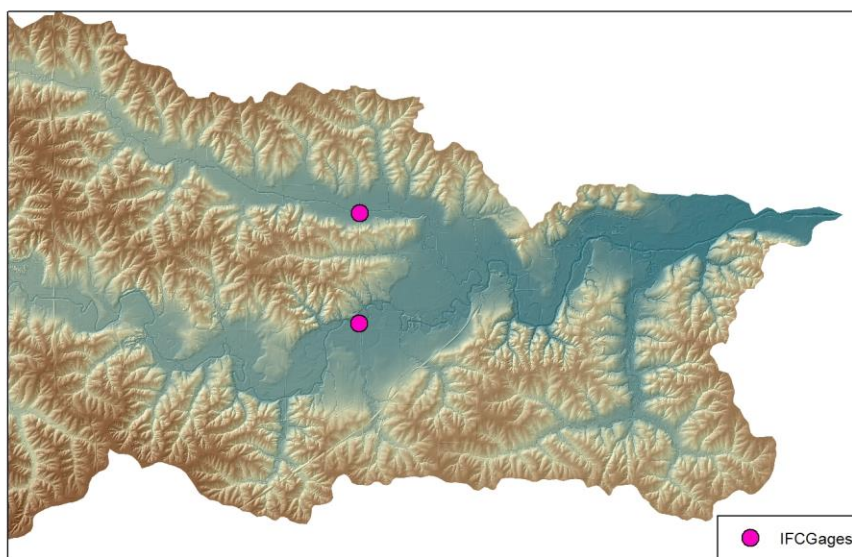


Figure 5.2: Ottumwa hyetograph from 9-11 March 2013



0 0.4 0.8 1.6 2.4 3.2 Miles



Figure 5.3: IFC gage locations in the lower portion of Soap Creek

The first simulation was run using all of the parameters that were originally calculated in GIS, this included initial abstractions of 20% the maximum possible retention, curve numbers based on normal antecedent soil moisture conditions (Type II), GIS calculated lag times, and recession baseflow with a recession constant of 0.9 and a ratio to peak of 1%. These results are shown in Figures 5.4 and 5.5 with the red line. The results show that the timing of the peak is off by a day and that the shape of the hydrograph is reversed with the second peak being higher than the first.

Seeing the initial results led to the need to calibrate the model to better match the timing and shape of the hydrograph. The biggest obstacle to overcome first was the timing issue. In order to move the peak forward in time, the time it takes to get water into the system needs to be reduced. This was first modeled by reducing the initial abstraction to 10% of the maximum possible retention (S). This allows for direct runoff to occur quicker as the surface is modeled to only accept half the original abstractions. All other parameters were maintained from the first simulation. The resulting hydrographs can be seen in Figures 5.4 and 5.5 with a green line. This result got the timing much closer and switched the peaks to be in the right order. The concern I had from this simulation going forward, and modeling the design storms as well as the historical storms is that they are simulating the summer months where initial abstractions are likely to be 20% of S. This led to another simulation being run to try and account for all of the simulations that will be run.

The last simulation that was modeled used the antecedent soil moisture (AMC) conditions that assumed soil saturation. This is the Type III case. Using AMC III the basin curve numbers are adjusted up to account for higher runoff and the initial abstractions are recalculated based on the new curve numbers. The formula below describes how the curve numbers are altered from AMC II (normal conditions) to AMC III (saturated)

$$CN3 = \frac{23 * CN2}{10 + (0.13 * CN2)}$$

When calculating the new initial abstractions the 20% of S was used to better maintain properties between simulations. The lag times and baseflow were conserved across all simulations. The AMC III results are show in Figures 5.4 and 5.5 with the orange line. These results best matched the timing of the peaks from the IFC gages and are closest in maintaining the shape present in the IFC data. The general flashy nature of the modeled results were left. In order to smooth out the results, the lag times would have needed to be increased which would have led to the peaks being pushed further away and having the same problem as in which was originally presented. Going forward all design storms will be modeled based on AMC II conditions and the historical storms will be based on the previous 5 days precipitation (Silveira, Charbonnier, & Genta, 2000).

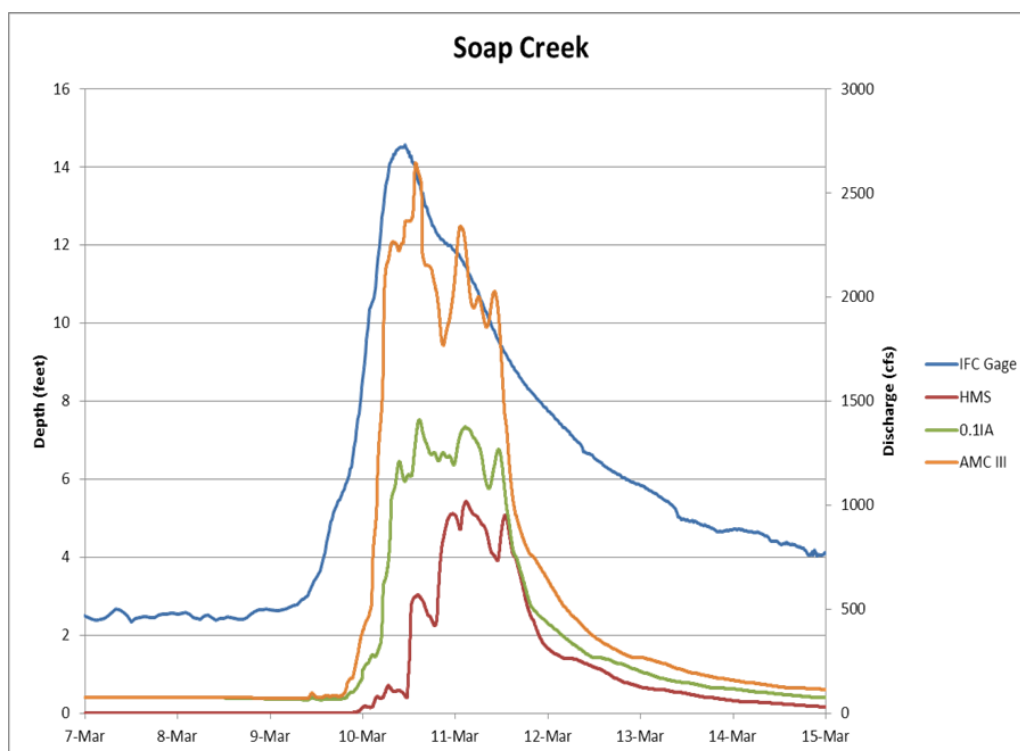


Figure 5.4: Soap Creek IFC gage and HMS modeled results from March 2013

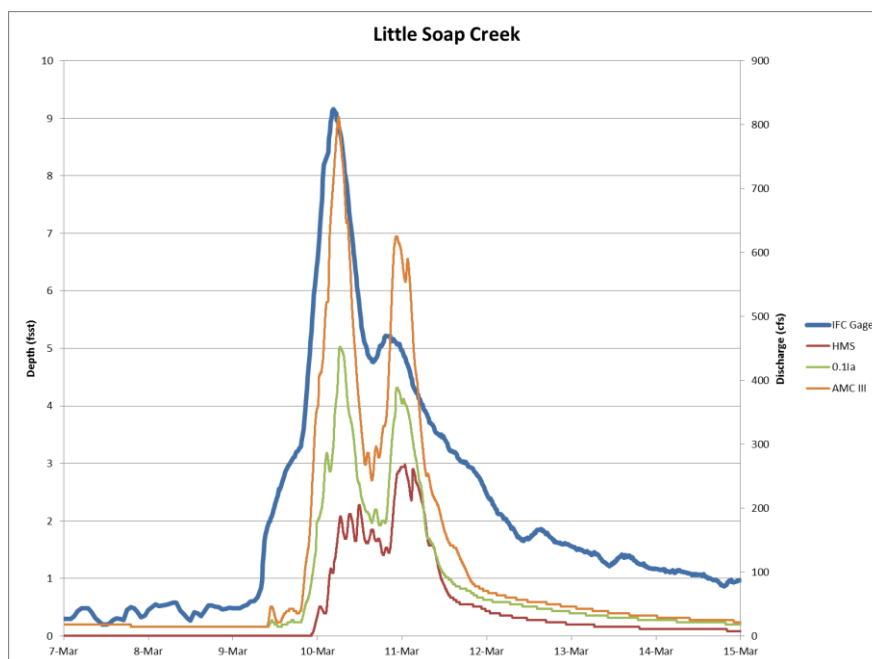


Figure 5.5: Little Soap Creek IFC gage and HMS modeled results from March 2013

Using site measured cross sections obtained from IFC Researchers, I was able to compare peak values from HMS to the cross section data by working backwards through the storage-discharge tables that were created using the cross sections. The Little Soap Creek had a peak discharge of 813 cfs which corresponds to a depth of 4.46 feet according to the storage-discharge table. This is only half of the observed depth of 9.16 feet. The result was accepted before we had the cross sections and future calibration will be run to better match the IFC gage data. Discharge in the Soap Creek was modeled to be 2,645 cfs which correlates to 10.16 feet and the IFC gage was showing a peak of 14.58 feet. Once again the depths are off by 4.0-4.5 feet. Since both gage locations are off by a similar value that would suggest that a single correction to be made to both locations. This will be considered during future modeling and calibration.

## 5.2 Design Storms

As stated in Section 2.2.4 all of the Soap Creek projects were designed to hold the 50yr storm below the elevation of the emergency spillway. Using the design storm

results the outlet conditions can be compared to the USGS regional regression equations to confirm if the results are acceptable. Since the Soap Creek watershed is 250 mi<sup>2</sup> the 2001 Eash equations are used to determine the outlet discharge based on drainage area. Table 5.2 shows the discharges from the regression equations for the 2, 5, 10, 25, 50, and 100 year return periods. Each storm uses the SCS Type II storm distribution of rainfall. This is shown in Figure 5.6. All design storms were modeled with the original parameters in GIS with the baseflow added in HMS. Baseflow was calculated using the recession method with the recession constant set at 0.9 and the ratio to peak at 1%.

Table 5.2: Outlet discharge estimates for Soap Creek based on the 2001 regression equations, Soap Creek is located entirely in Region 3.

Eash (2001)	Discharge (cfs)	Region 1	Region 2	Region 3
	Q2	1264.635	3588.880	5516.478
	Q5	2300.107	6942.383	9658.452
	Q10	3063.888	9487.974	12746.594
	Q25	4083.415	12785.260	17011.233
	Q50	4863.323	15215.383	20331.322
	Q100	5668.145	17799.902	23690.734

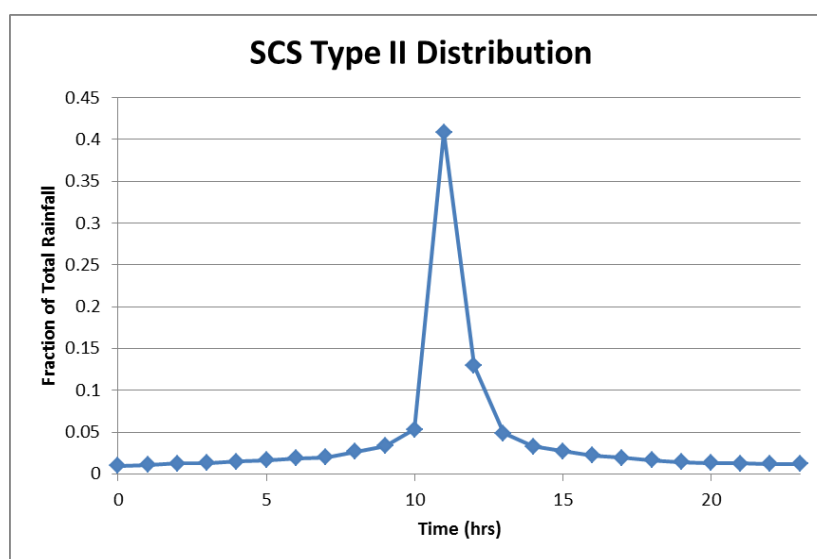


Figure 5.6: Distribution of rainfall using the SCS Type II storm

### 5.2.1 Two Year Storm

The two year storm was modeled as a way to generate a baseline or bankfull conditions within the watershed. Using a rainfall atlas (Huff & Angel, 1992), the two year, 24 hour storm produces a storm total rainfall of 3 inches that is uniformly distributed over the basin. The no pond condition results in a discharge at the outlet of 5,703 cfs. As time passes and more ponds are built, the peak discharge decreases to 4,697 cfs in 2005 and to 4,563 cfs in 2012. This represents a 20 percent reduction in the peak flow. Figure 5.7 shows a detailed progression of the flow and the reduction through the watershed from the No Pond condition to the current construction of 132 ponds. The first thing to notice in Figure 5.7 is the hydrograph in the top left Point 1, this point shows a very large reduction in the peak which is seen through all of the model simulations run. As the water moves down stream the area that is controlled compared to the total area decreases, this corresponds with a decrease in the peak reductions also falling as the water reaches the outlet. At point 2 and point 4 there are large, quick drops that are observed in the recession limbs of the hydrographs. These drops are attributed to large changes in the storage-discharge tables over small changes in elevation around the height of the floodplain. Figure 5.8 shows the peak storage values of the ponds in terms of depth. Across most of the basin the ponds are storing up to 1.00 inch of the rain at their peaks. These small numbers are due to most of the water being able to pass through the pipe of the pond without filling much of the storage available in the ponds.

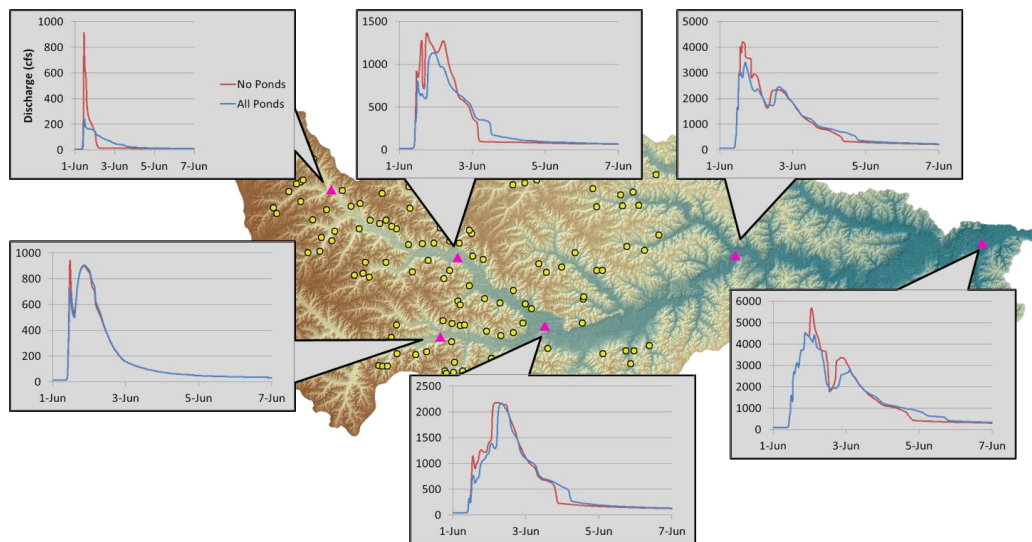


Figure 5.7: Two year storm results showing hydrographs at six locations and reductions in flow

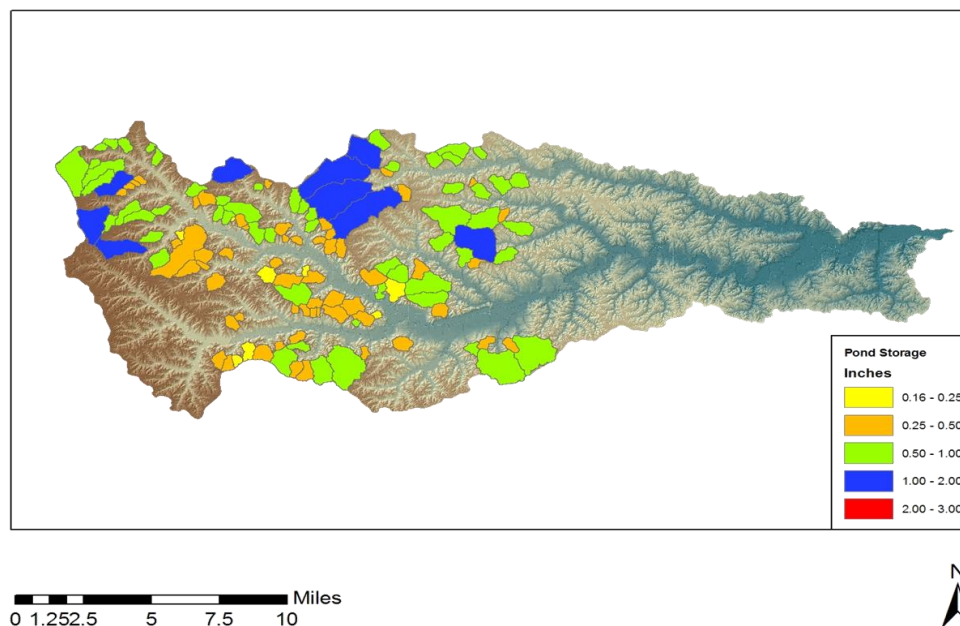


Figure 5.8: Peak storage in each pond for the 2 year storm, compared to storm total rainfall of 3 inches

### 5.2.2 Five Year Storm

The five year storm was chosen as a way to show the reductions in flow for mild flooding in the basin. The rainfall atlas shows 3.5 inches of rain fall over the basin in 24



hours for the five year storm. This flow results in some over land flooding in the head water areas but down stream near the outlet the flows are still contained within the banks. This results in some of the flows near the outlet showing very little reduction in terms of the peak but the hydrographs still show benefits in duration of high flows. When there are no ponds in the basin the discharge is modeled to be 8,175 cfs. As ponds have been built the flow is reduced to 8,048 cfs in 2005 to 7,649 cfs as of 2012. This results in a 6.44 percent reduction in the whole watershed. Figure 5.9 shows the hydrographs on the basin and how the watershed responds to the ponds in the five year storm. Between Points 5 and 6 in the basin the reaches are being filled to limit of their banks. This results in the most efficient flow possible and the resulting flows down stream show the peaks are not reduced, but the time of high flow is shorter and the lower flows are prolonged. This still provides benefit to the areas between Points 5 and 6. Figure 5.10 shows the peak storage in the ponds in inches compared to the total rainfall. Many of the ponds are holding between 0.75 and 2.00 inches of the 3.5 inches of rain.

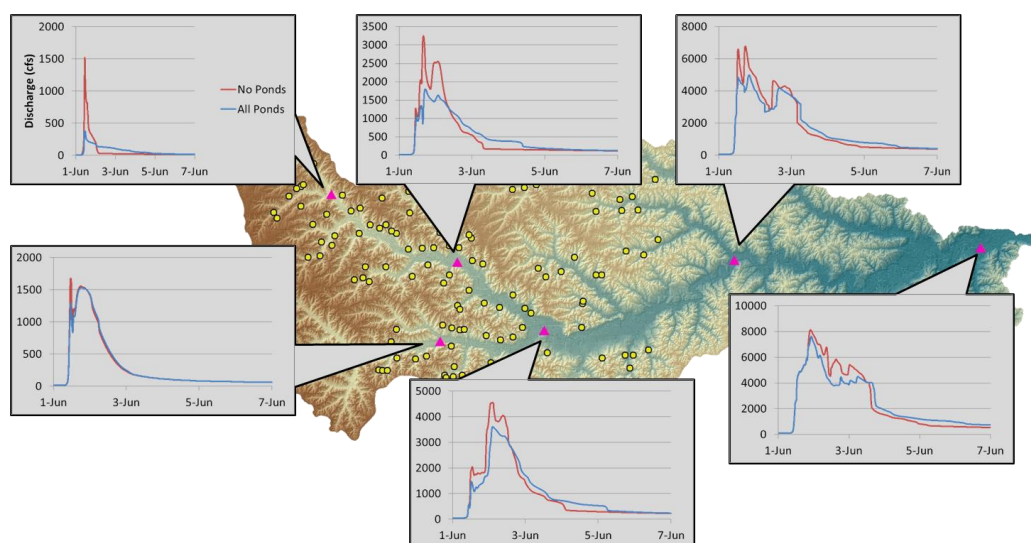


Figure 5.9: Five year storm hydrographs showing the no pond condition (red) and the modeled flows will all of the ponds implemented (blue)

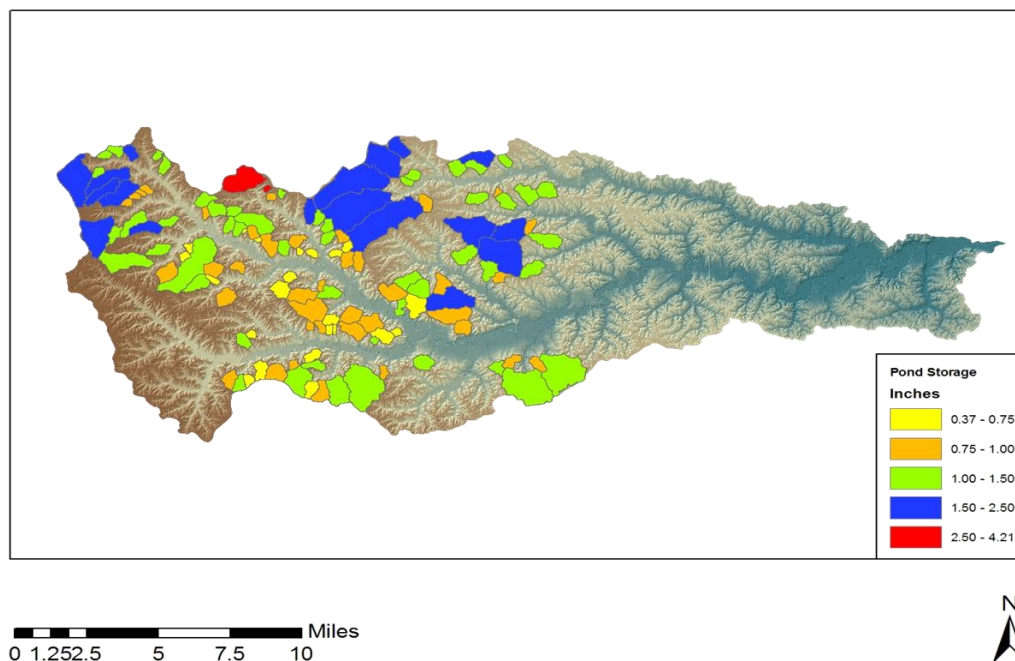


Figure 5.10: Peak storage for all ponds during the 5 year storm, compared to the total storm rainfall of 3.5 inches.

### 5.2.3 10 Year Storm

The 10 year storm signifies the amount of water for which some ponds will reach their emergency spillways. Using Huff & Angel, the 10 year storm will produce a total accumulation of 4.75 inches of rain. The no pond condition model at the outlet shows a discharge at the outlet of 11,975 cfs. In 2005 the peak has been cut down to 10,086 cfs. By the time all of the projects have been implemented in 2012 the peak flow is still 10,086 cfs, but many of the secondary peaks have been reduced and the flood has been extended. This result shows that there still is benefit to adding more ponds over time. Figure 5.11 show the progression of the flood through the watershed and Figure 5.12 shows how much water was being stored at the time of the peak. As noted before the 10 year storm is one in which some of the ponds may start reaching their emergency spillways. Figure 5.13 shows how much of the pond is being utilized during the peak

conditions. Six of the 132 ponds are shown as reaching their emergency spillways during the 10 year storm.

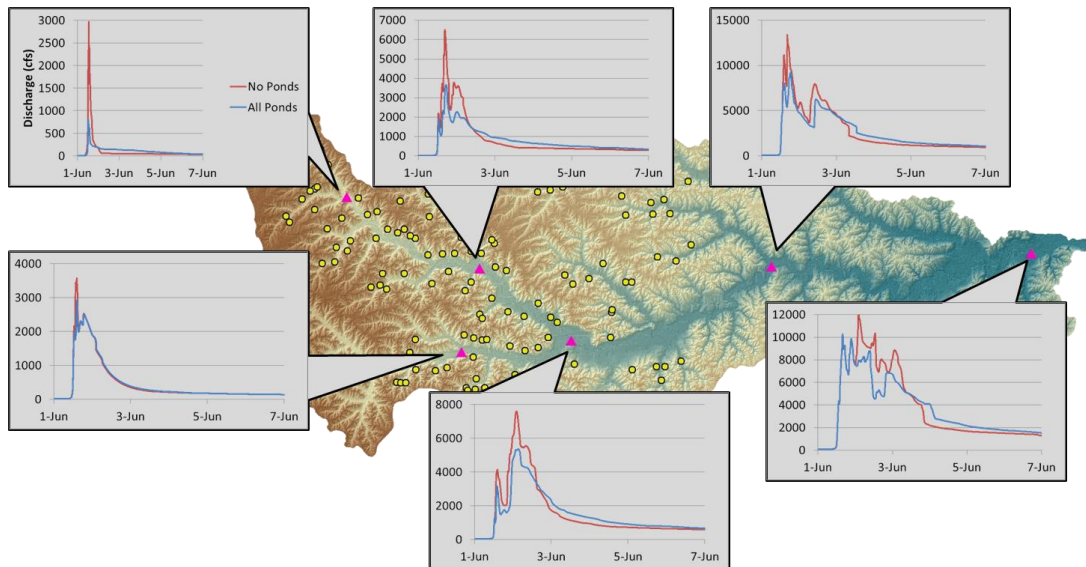


Figure 5.11: Basin wide hydrographs showing the benefits of the 132 projects in Soap Creek when modeling the 10 year storm

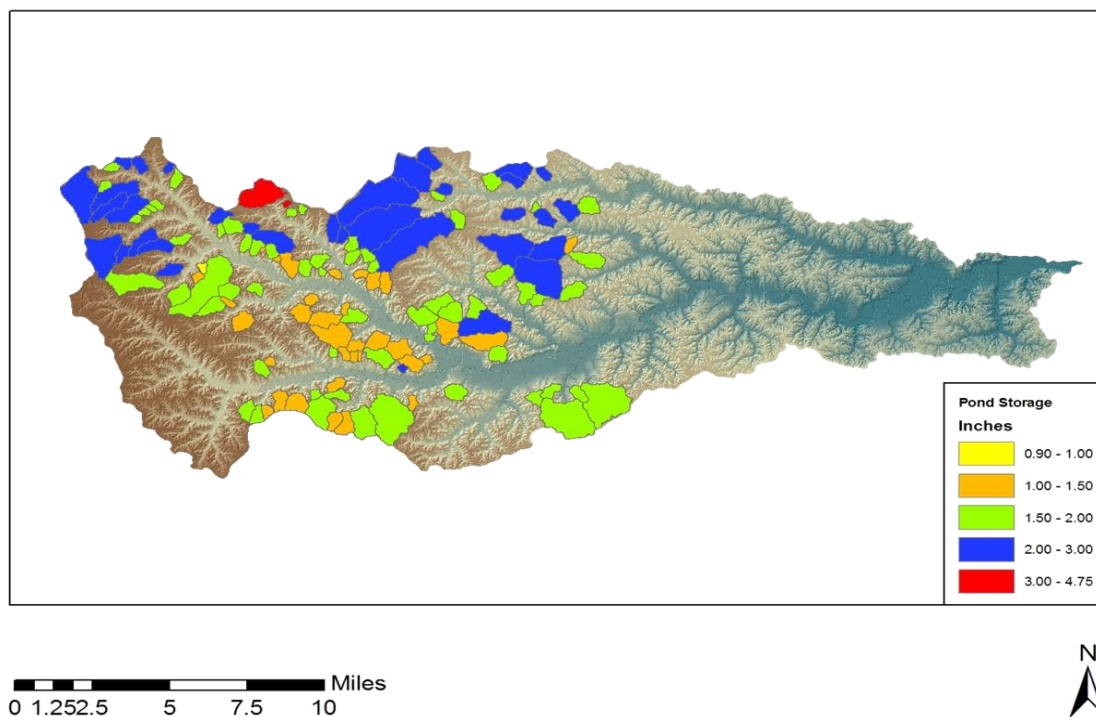


Figure 5.12: Peak storage in every project for the 10 year storm, total precipitation was 4.75 inches

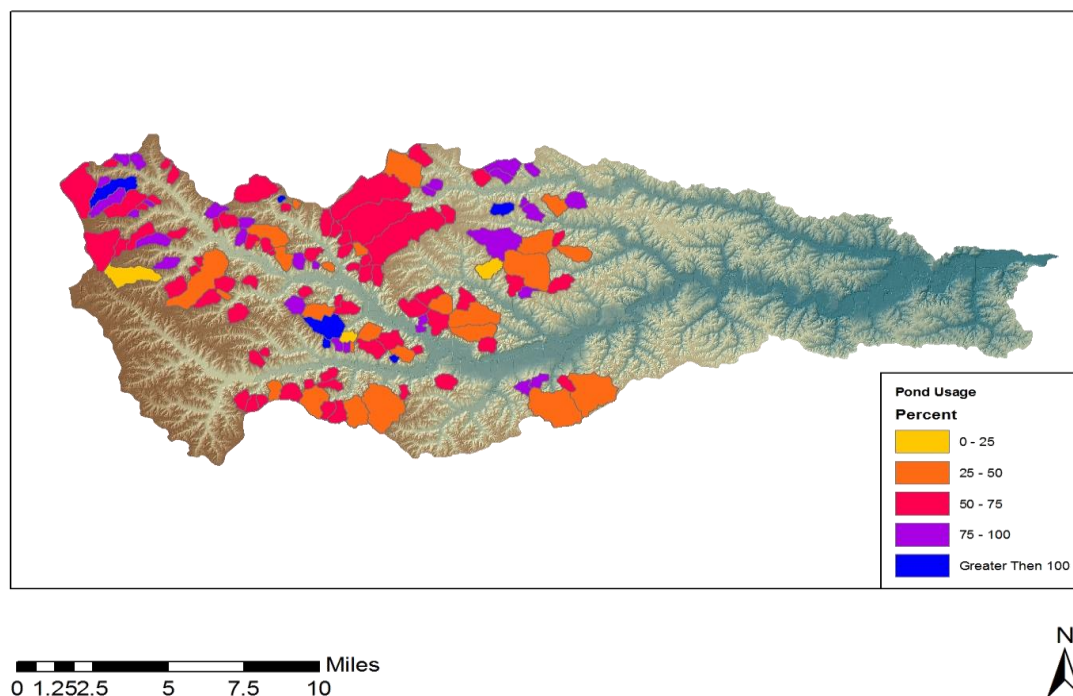


Figure 5.13: Peak storage compared to maximum storage from the 10 year storm, 6 projects have reached their storage capacity designated by the blue shading

#### 5.2.4 25 Year Storm

The 25 year storm was modeled to show the benefits during a moderate flooding event. The total depth of rainfall that was applied over the basin was 5.5 inches. Discharge at the outlet when there are no projects in place was modeled to be 27,598 cfs. By the time 12 of the ponds were built in 1993 the flow is already seeing a reduction to 26,461 cfs at the outlet. In 2005 when 88 of the 132 ponds have been built the flow was reduced to 19,875 cfs, and when all 132 projects were completed the flow was reduced to 16,722 cfs. This represents a 39% reduction in the peak. The total storage held in the system was equal to 8,094 acre-feet compared to a total available of 12,419 acre-feet. Figure 5.14 shows hydrograph results throughout the basin with all of the plots showing decreases in peak flow. Figure 5.15 is showing that many of the ponds are still storing much of the total storm precipitation during the peak with many ponds storing 2.00 to

3.50 inches of the rain. Figure 5.16 shows that there are now 28 ponds that have reached their maximum flood storage. This plot also shows that many of the ponds that are storing at maximum levels are the smaller ponds. As the storms increase in intensity the larger ponds will begin to reach capacity as well.

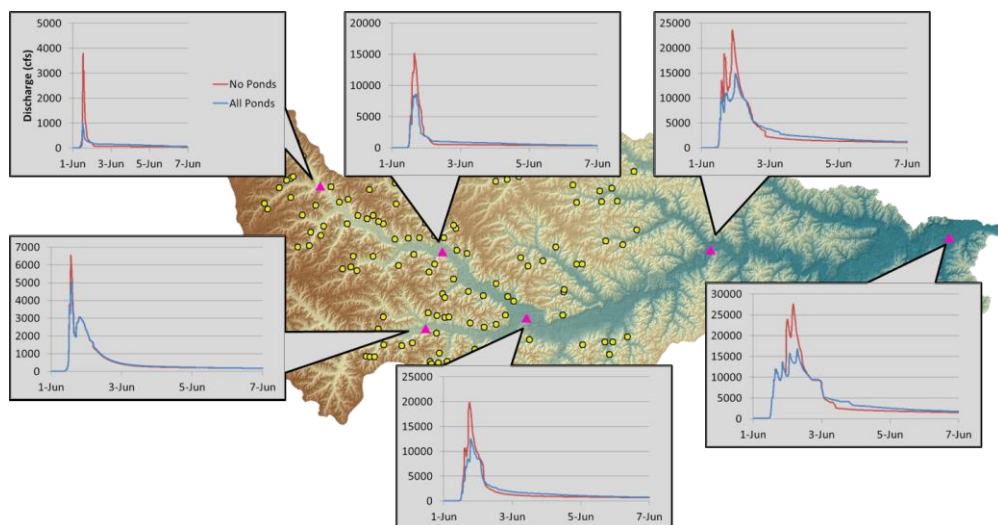


Figure 5.14: Progression of hydrographs through the watershed for the 25 year storm

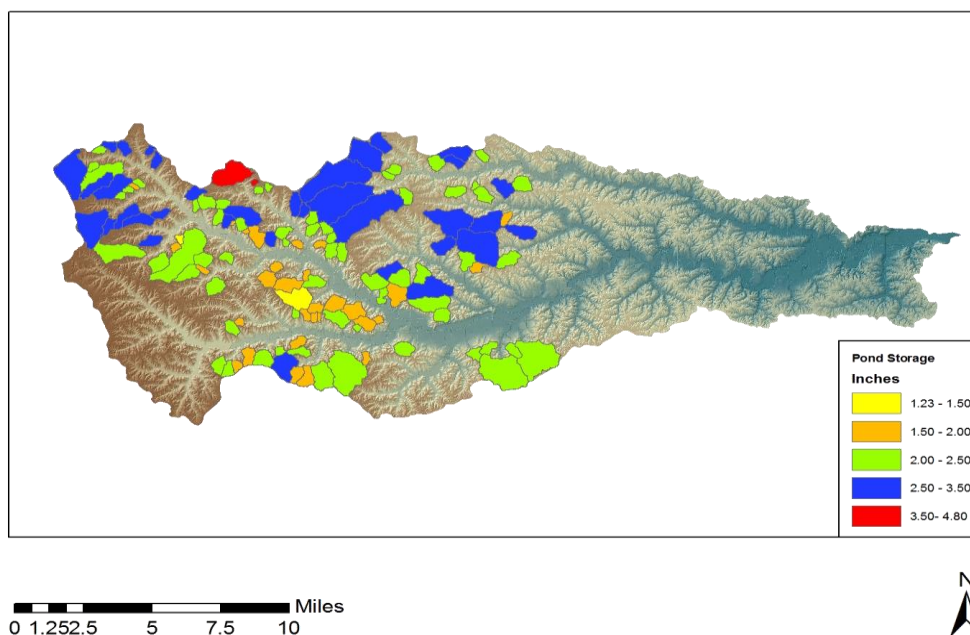


Figure 5.15: Peak storage as a function of depth for the 25 year storm, total precipitation was 5.5 inches

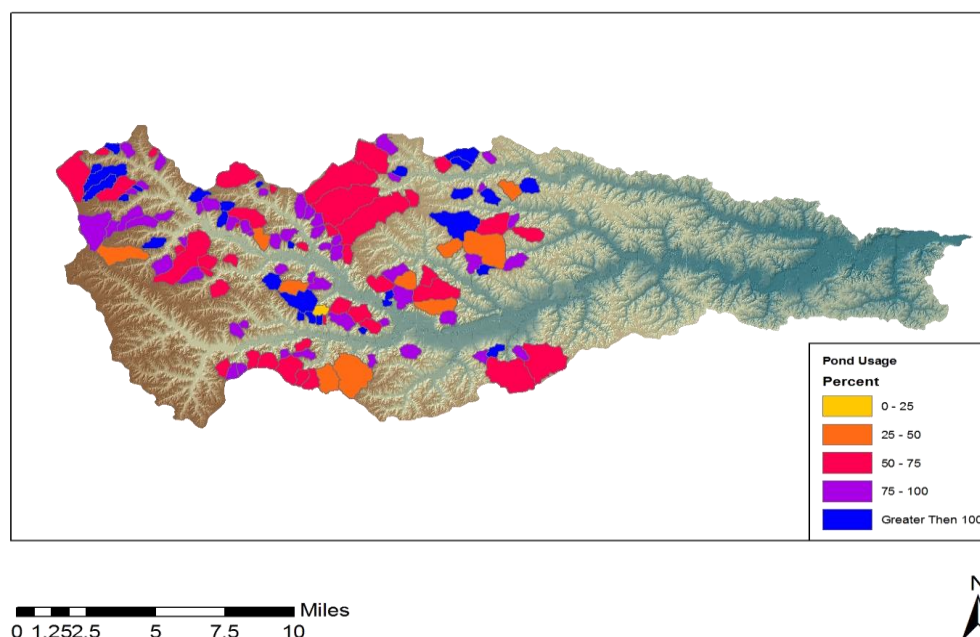


Figure 5.16 Peak storage compared to maximum storage below the emergency spillway for the 25 year storm, 28 ponds have reached their maximum storage values

### 5.2.5 50 Year Storm

All of the projects in Soap Creek are designed for the 50 year storm. To model this event 6.5 inches of precipitation was applied to the basin. The precipitation resulted in a discharge of 37,776 cfs at the outlet when there are no ponds in the basin. Comparing to the results obtained from the ponds that were in place by the end of 2005 discharge is down to 26,694 cfs, and finally by the time all of the ponds were built the flow has been reduced to 23,135 cfs in 2012. This represents a reduction in the peak flow of nearly 39%. Figure 5.17 shows the hydrograph comparisons between the no pond and all pond conditions. The 50 year results follow the trend that was set by the 25 year storm in that the hydrographs maintain their shape but the values have increased from the high rainfall amount. Figure 5.18 shows that many of the ponds are holding at least 2.5 inches of the total accumulation during the peak which is just below 40% of the total

precipitation. Figure 5.19 shows that now 36 of the ponds have reached their designed capacity during the 50 year storm. Another 64 are within 25% of being full as well so that brings the total to 100 ponds that are full or near capacity. The total system storage during peak conditions at each project resulted in 10,284 acre-ft being held back. This represents 83% of the total available flood storage in the system. This result confirms that the ponds have met the design criteria of being able to handle the 50 year storm as was described in the watershed plan from Section 2.2.1.

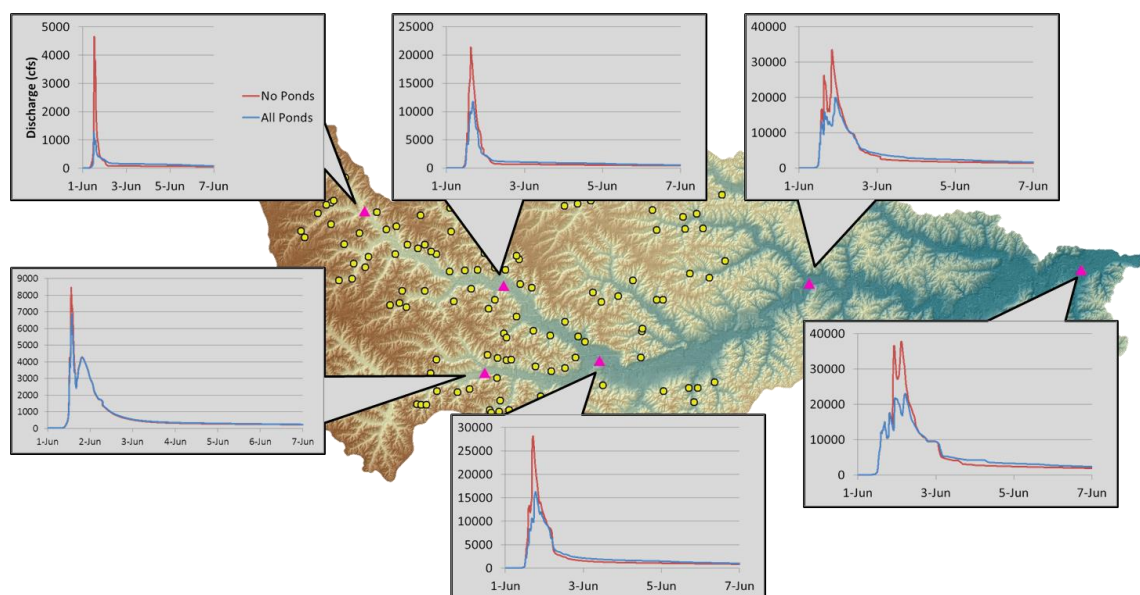


Figure 5.17: Hydrographs of the 50 year storm modeled with no ponds and all of the ponds

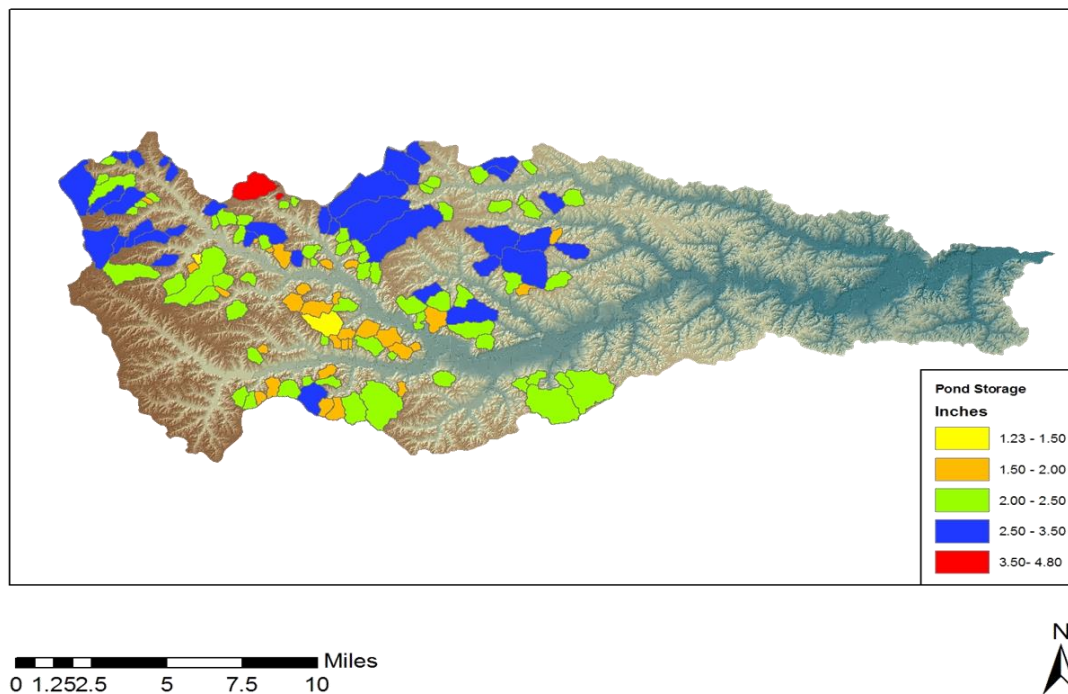


Figure 5.18: Peak storage of the 50 year event with all of the projects constructed, many of the ponds are holding at least 40% of the total rainfall accumulation

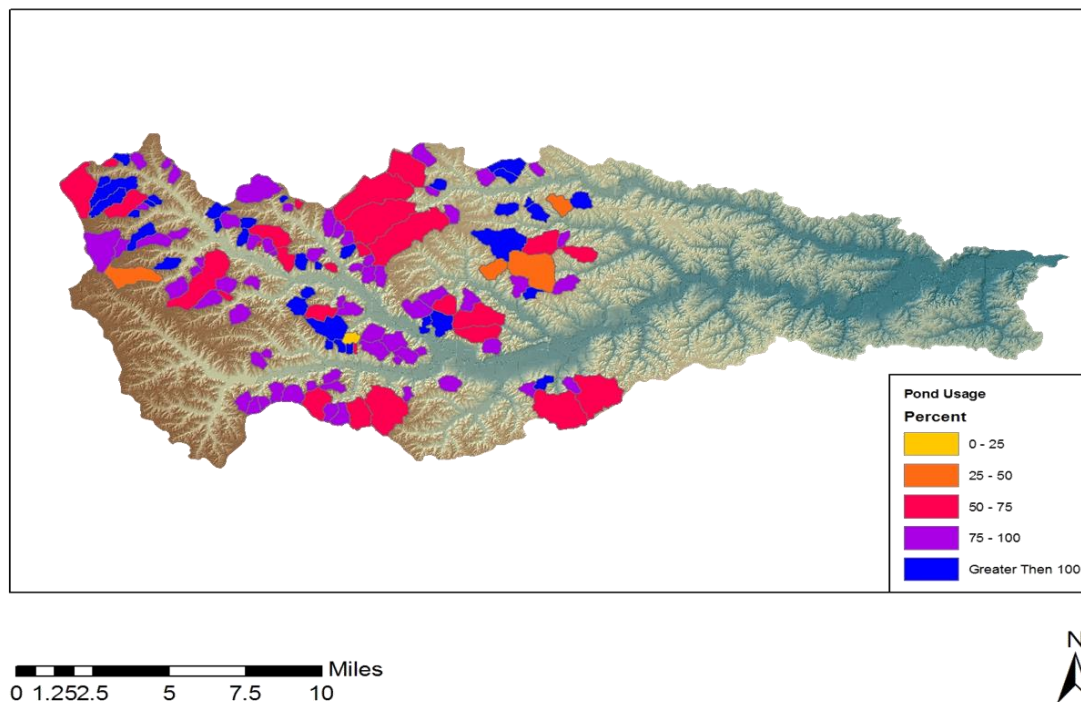


Figure 5.19: The 50 year storm results in 36 ponds being filled to capacity and another 64 between 75-100% maximum flood storage



### 5.2.6 100 Year Storm

The 100 year storm was the last design storm modeled as was done to show the effects that the system has on a major flood. According to the rainfall atlas the 100 year storm will result in 7.5 inches of rainfall over the entire basin. The no pond condition outlet discharge was modeled to be 52,305 cfs, this results in flows that are above the 30% error in the USGS regression equations. While the USGS equations provide a guide the 100 year 24 hour storm may not result in the 100 year return period discharge. This suggest that this very intense rain could result in the discharges being higher than the estimated USGS values. After the first 12 projects are built by 1993 there is a small reduction down to 49,515 cfs, but by 2005 the peak had been reduced to 35,993 cfs. The reduction is greatest after all of the projects have been implemented in 2012, the final modeled values of discharge was 30,226 cfs. This results in a peak reduction of 42%. Figure 5.20 shows the hydrographs through the basin and how the system of ponds alters the flow. Once again the hydrograph shape remains very similar to the 25 and 50 year, but the peaks have increased as a result of the heavier rain. Figure 5.21 shows the peak storage in terms of water depth. Many of the ponds are holding at least 3.0 inches of rain out of 7.5 at their peak. Figure 5.22 shows the number of ponds that have reached capacity has increased from 85 to 101, and that the number of ponds that are near or at capacity is up to 123. The total valume of water stored at the peak is 11,682 acre-feet. This number shows that the system is nearing the total capacity and that the ponds wouldn't be able to control much more precipitation.

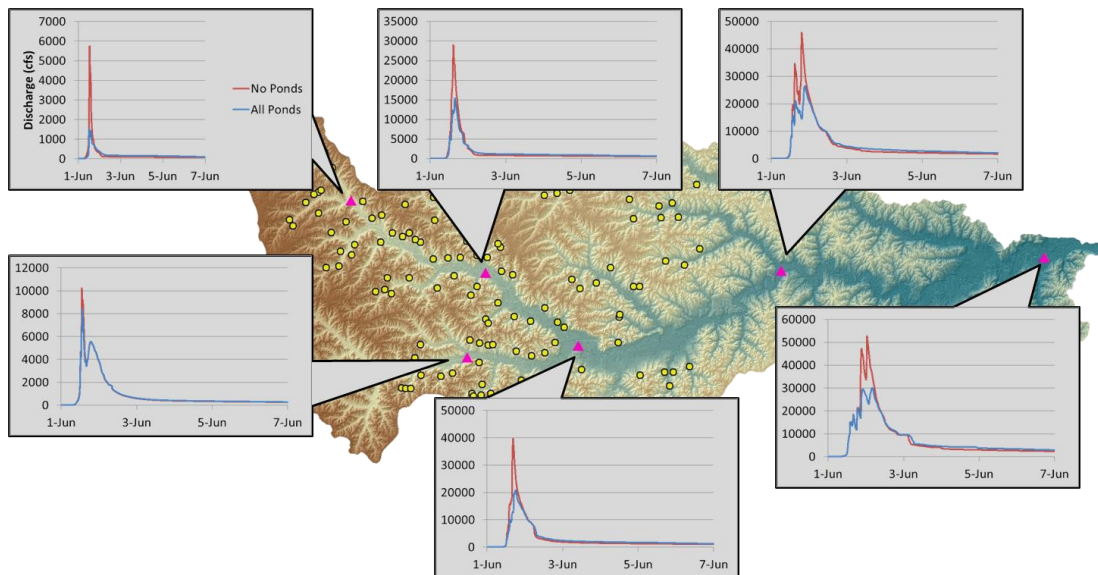


Figure 5.20: 100 year storm hydrographs along the main stem of Soap Creek

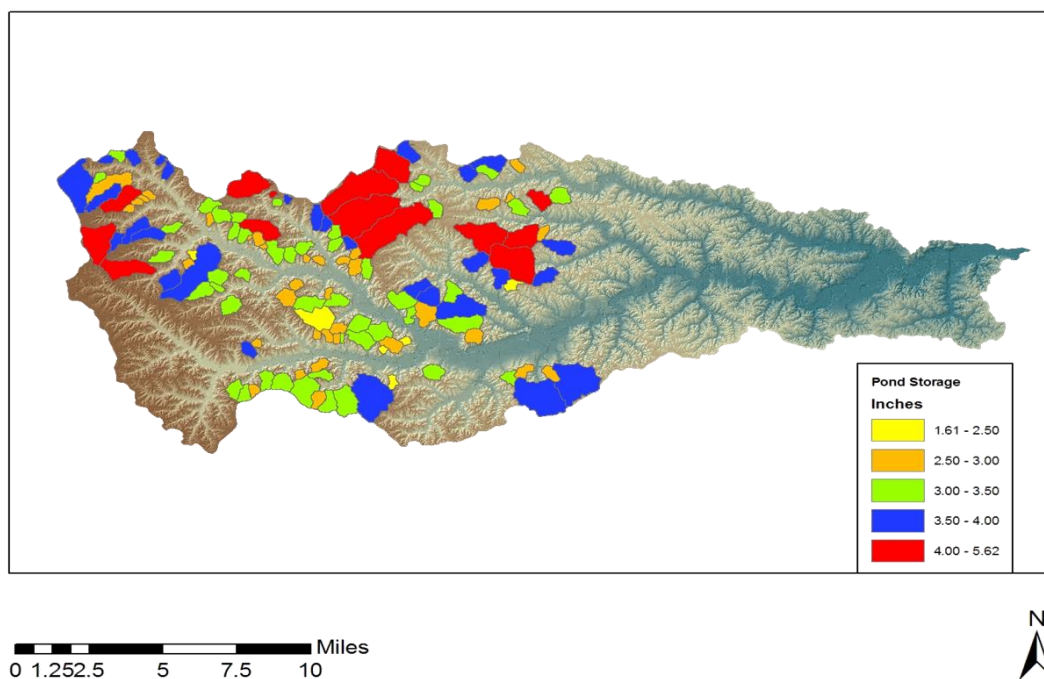


Figure 5.21: Peak pond storage from the 100 year storm, 43 of the ponds are storing at least 3.5 inches of rain

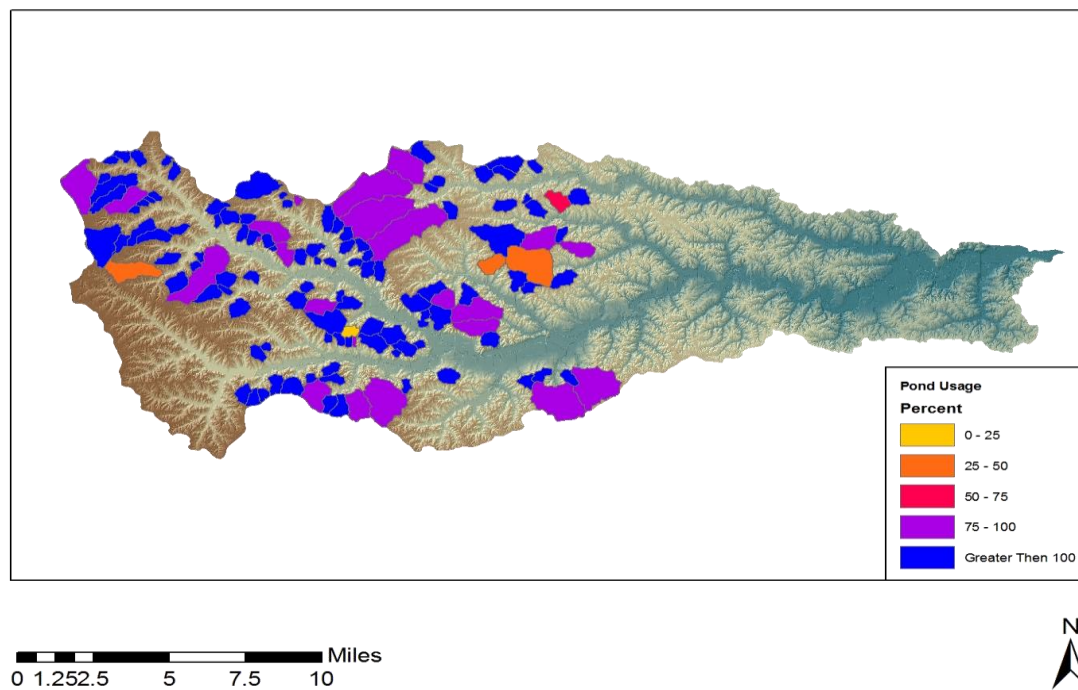


Figure 5.22: 100 year storm peak storage compared to the maximum available in each pond, 103 ponds have reached their maximum flood storage

### 5.2.7 Extreme Storm

After observing the peak flow reductions across the watershed always increasing regardless of total precipitation a last design storm was run to confirm there is a maximum storm event that will result in the storage of the system to be over topped and as a result a significant amount of water to be released over the spillway of the ponds. This should result in a decrease of the overall system effectiveness and a lower value of peak reduction. To model such an event a 12 inch rainfall was applied to the system using the same design storm parameters. The resulting effects from such a heavy rain are shown and discussed below. Figure 5.23 shows the response of the watershed to the heavy rainfall.

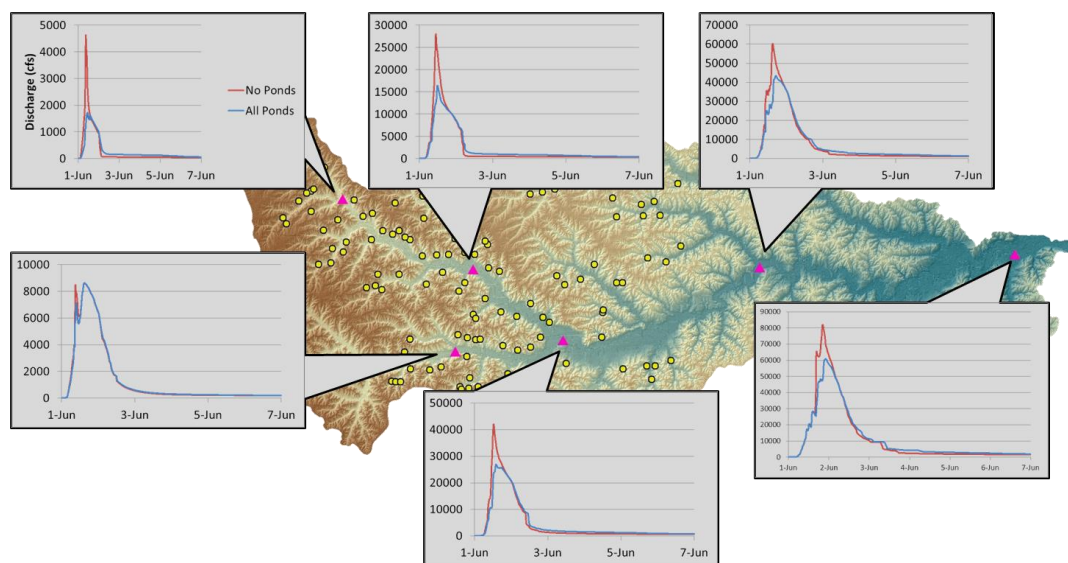


Figure 5.23: Hydrograph plots of watershed response to 12'' extreme storm event

Figure 5.24 show how much of the total precipitation each pond was able to hold during the peak retention. This figure is showing that many of the ponds are holding at least 3.5 inches of the total precipitation during peak storage conditions. This represents a minimum of 30% of the total rainfall. Figure 5.25 shows how much flood storage each pond was holding at the peak of the simulation. This figure uses a different color scheme as most of the ponds have reached their respective maximum flood storages. This is reflected in Figure 5.25 as the greens and blues represent all of the ponds that have surpassed their maximum flood storage. The extreme event has surpassed the total storage available to the system in terms of total storage in the ponds, but there are 15 ponds with small amounts of flood storage available to them. Overall there is 13,750 acre-feet of water being stored out of the total flood storage of 12,419 acre-feet.

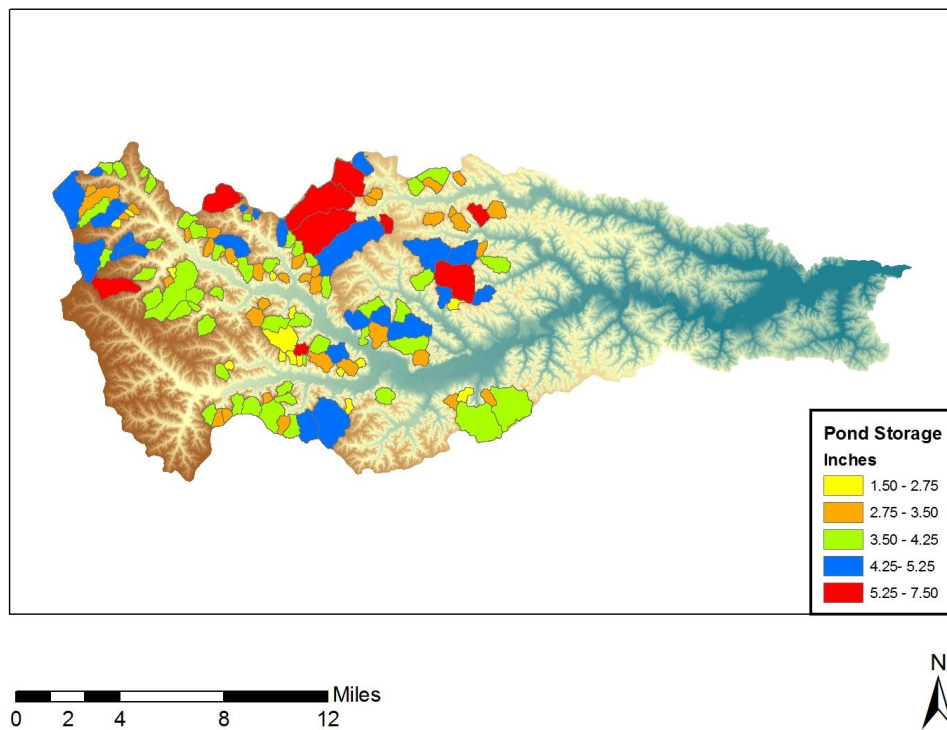


Figure 5.24: Peak pond storage in each pond for the 12” extreme event design storm.

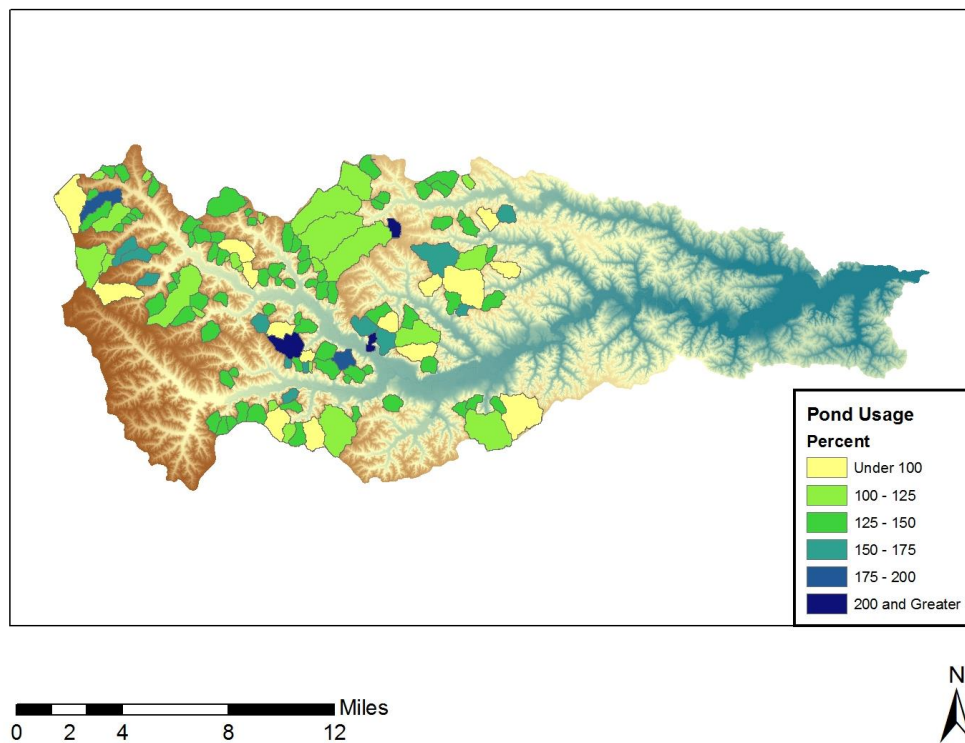


Figure 5.25: Pond usage in percent of available flood storage for the extreme event

### 5.2.8 Design Storm Summary

Throughout modeling the design storms a few trends emerged. The highest reductions in peak flow are in the upper half of the watershed (Points 1, 2, and 4) where most of the projects were located. As was shown in the previous sections there is a low in the reductions from the five year storm that that results from the streams not overflowing their banks in the lower portion of the watershed. This results in the streams moving water at their peak efficiency. When the 10 year storm is modeled this portion of the watershed floods and once again the peak is reduced to a greater extent. As more ponds become full the flow out of the ponds become unregulated and behave as if the pond was not there. Figure 5.26 shows the percent reduction at each hydrograph point for all six design storms that were modeled, comparing the no pond and all pond conditions. Table 5.3 shows the comparison between the 2001 USGS regression equations and the final modeled flows in HMS.

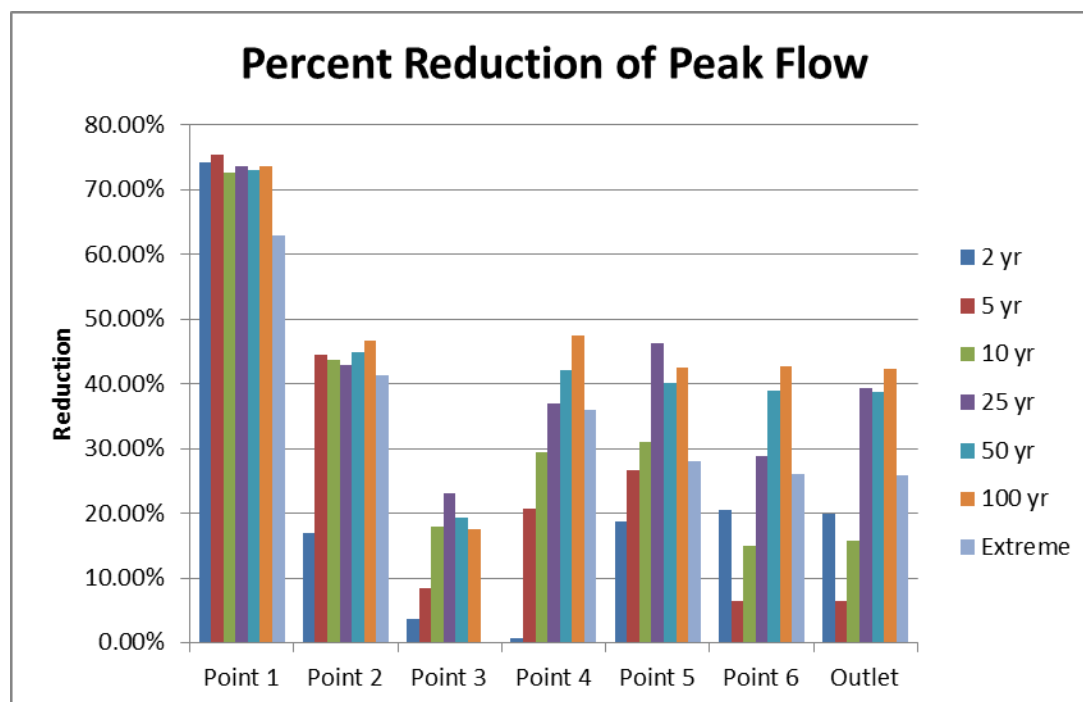


Figure 5.26: Percent reduction at each hydrograph location of all six design storms

Table 5.3: Results of the outlet peak flows compared to the 2001 USGS regression equations

Eash (2001)	Discharge (cfs)	Region 1	Region 2	Region 3	HMS Results
	Q2	1264.635	3588.880	5516.478	5703.320
	Q5	2300.107	6942.383	9658.452	8175.350
	Q10	3063.888	9487.974	12746.594	11975.210
	Q25	4083.415	12785.260	17011.233	27598.440
	Q50	4863.323	15215.383	20331.322	37776.130
	Q100	5668.145	17799.902	23690.734	52304.600

### 5.3 Historical Storms

Once the design storms were modeled work began on modeling historical events. Three storms were chosen for historical significance these events included 3-13 June 2008, 9-20 June 2011, and 14-21 April 2013. These events correspond to the time periods of the 2008 Iowa floods and two local high water events. Storm details and results will be discussed in the following sections. The three historical events were modeled using NEXRAD rainfall data. SCS loss parameters were chosen based on Table 5.4 which describes which AMC type to select based on the previous five days total precipitation (Silveira, Charbonnier, & Genta, 2000). The original lag times and baseflow were left unaltered for model these events.

Table 5.4: Antecedent soil moisture condition selection based on the previous five days of precipitation

Selecting AMC Type		
5 Day Total Precip (mm)		
	Dormant Season	Growing Season
Type I	Less than 13	Less than 36
Type II	13 to 28	36 to 53
Type III	More than 28	More than 53

### 5.3.1 June 2008 Storm

The flood of 2008 is known as the costliest natural disaster to affect the state. Flooding was prevalent throughout much of the state resulting in 85 of the 99 counties to be declared Federal Disaster Areas (Baldwin, 2008). Ottumwa set the fourth highest crest recorded at the USGS gage in 2008 and in Keosauqua the 2008 event was the second largest crest (USGS, 2013). The Soap Creek drains into the Des Moines River between these points. Comparing the output flow to these points will provide a window of acceptable values of discharge from Soap Creek. Table 5.4 shows the previous rainfall from Ottumwa and based on the Silveira study this results in AMC II being used for modeling this event.

Table 5.5: Previous five days of rainfall in Ottumwa, IA from 30 May to 3 June for assigning antecedent soil moisture conditions

2008	
Date	Precip (mm)
30-May	10.41
31-May	0.00
1-Jun	0.00
2-Jun	1.02
3-Jun	36.58
Total	48.01
Type II	

The model was run using rainfall from June 1<sup>st</sup> through June 4<sup>th</sup> and then allowing the basin to drain with no added precipitation. The precipitation was applied using hourly NEXRAD data and the total precipitation is shown in Figure 5.27. The rainfall is distributed in such a way that the most intense rain was located along the southern most area of the basin where the number of projects is smallest. This results in the hydrographs at the outlet to show no reduction in the peak. This simulation resulted in a peak outflow at the outlet in the no pond condition of 2,790 cfs. While the peak at



the outlet is reduced, it is only lowered by 14 cfs due to the location of heavy rain in the southern and lower portions of the watershed as well as a lack of projects in those areas. Table 5.6 shows the reduction at each hydrograph location. Figure 5.28 shows that along the stream there is benefit from the projects, but the results are not seen in the peak flow.

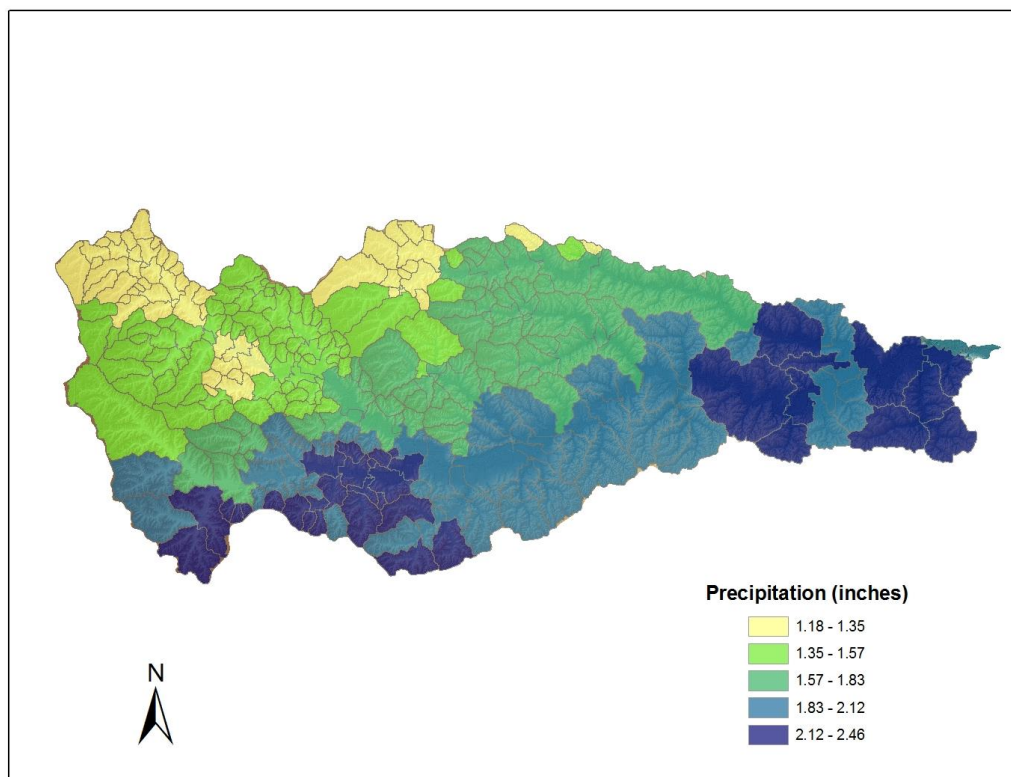


Figure 5.27: Total precipitation distribution from 1-4 June 2008

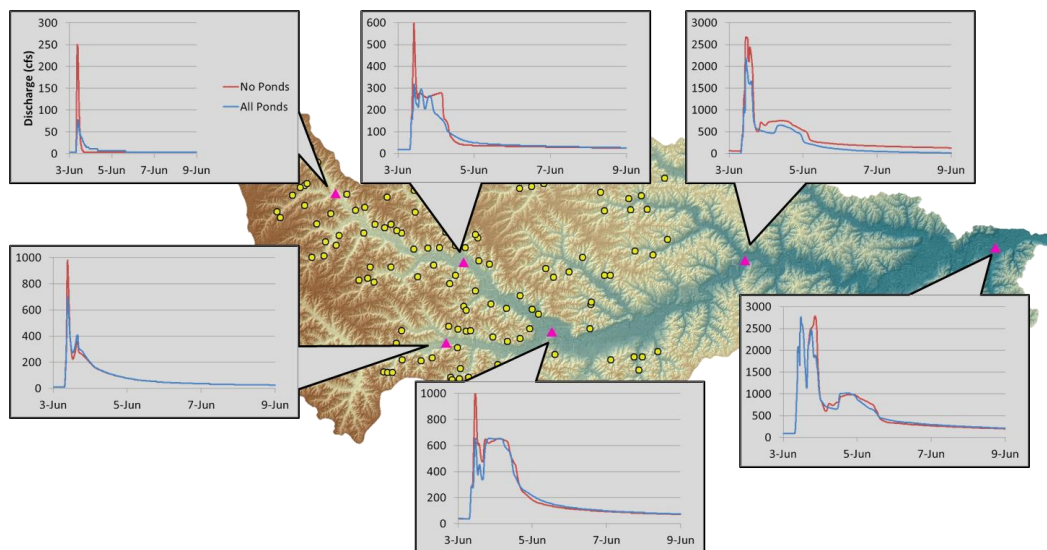


Figure 5.28: Hydrographs from 1-11 June 2008

Table 5.6: Peak reduction and percentage of reduction in flow from 1-11 June 2008

	1	2	3	4	5	6	Outlet
No Ponds	250.73	603.88	981.75	1020.6	2687.45	2789.86	2789.86
All Ponds	77.69	317.83	699.23	656.85	2196.57	2768.67	2775.74
Reduction	173.04	286.05	282.52	363.75	490.88	21.19	14.12
Percentage	69.01%	47.37%	28.78%	35.64%	18.27%	0.76%	0.51%

The peak in each hydrograph occurred during the day of June 3<sup>rd</sup>, to check if the outlet discharge is acceptable, the value is compared to the discharge at Ottumwa and Keosauqua from the same day. Average flow for June 3<sup>rd</sup> at Ottumwa was 20,600 cfs and flow at Keosauqua was 25,700 cfs (USGS, 2013). This equates to a 5,100 cfs difference and the HMS output was 2,543 so this would suggest that the output from the model is acceptable. Since the rain was distributed over three days the peak storage values are relatively small with water stored at the peak being around 1.0 inch out of a storm total of up to 2.46 inches. Peak storages are shown in Figure 5.29. Figure 5.30 shows that with this longer duration event the peak usage in many of the ponds was no higher than 50%.

Out of a total available storage of 12,419 acre-feet only 2,182 acre-feet was used during this event.

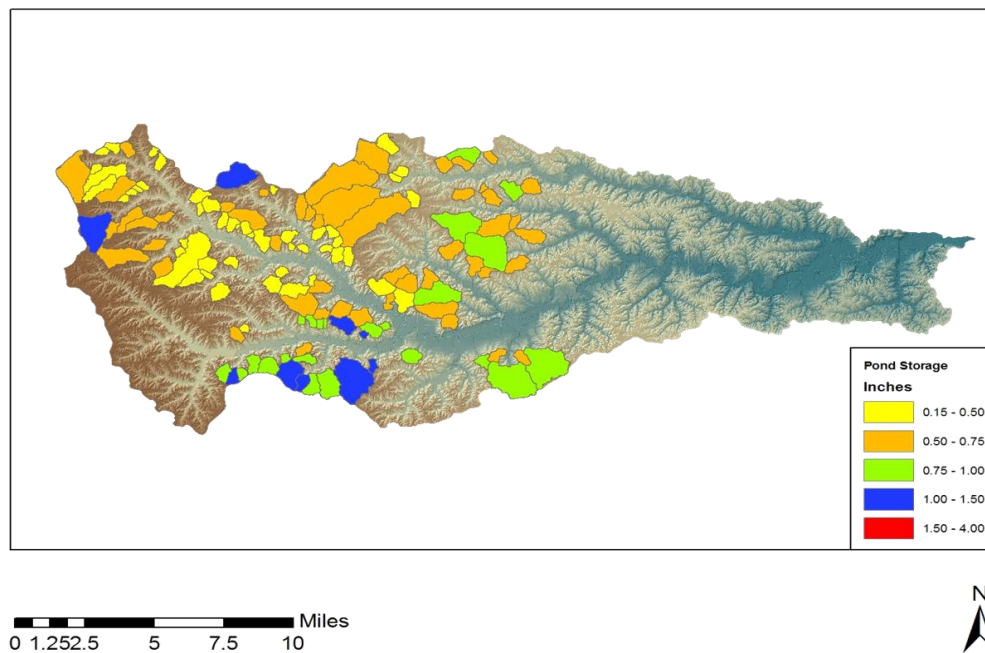


Figure 5.29: Peak pond storage during the 1-11 June 2008 event

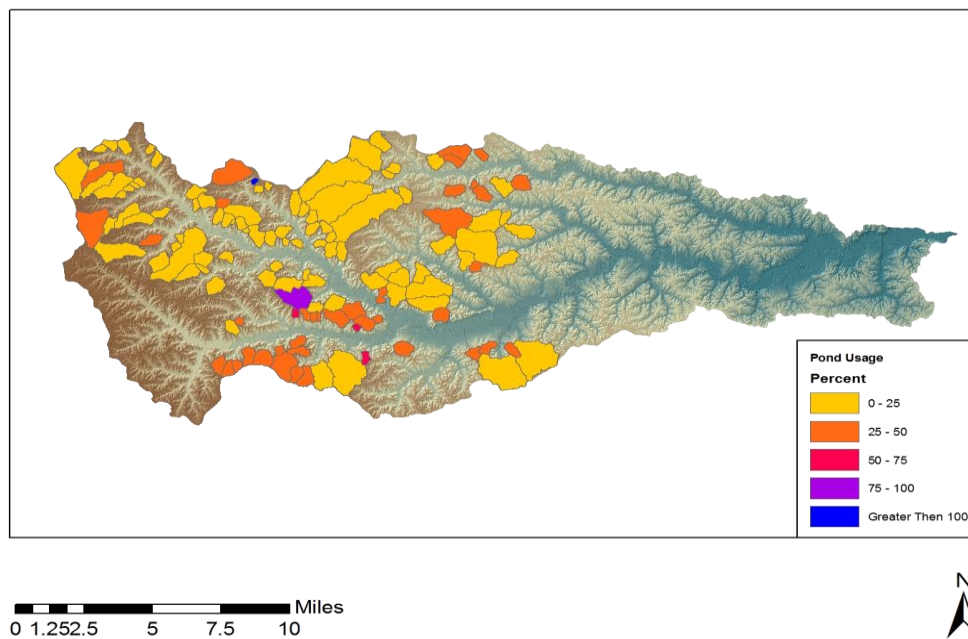


Figure 5.30: Percent of total pond storage used during peak conditions for the 1-11 June 2008 event.

### 5.3.2 June 2011 Storm

The rainfall from June 2011 was a much more localized event compared to the widespread heavy rain that was distributed over the state in 2008. In a two week period from June 9<sup>th</sup> to June 20<sup>th</sup> the rain gage in Ottumwa recorded over 10 inches of rain while 4-8 inches fell in Albia, Bloomfield, and Centerville (Iowa State University, 2012). The model only used rainfall from June 13-14, in this period 1-3 inches fell over the area. The total accumulated HRAP rainfall is shown in Figure 5.31.

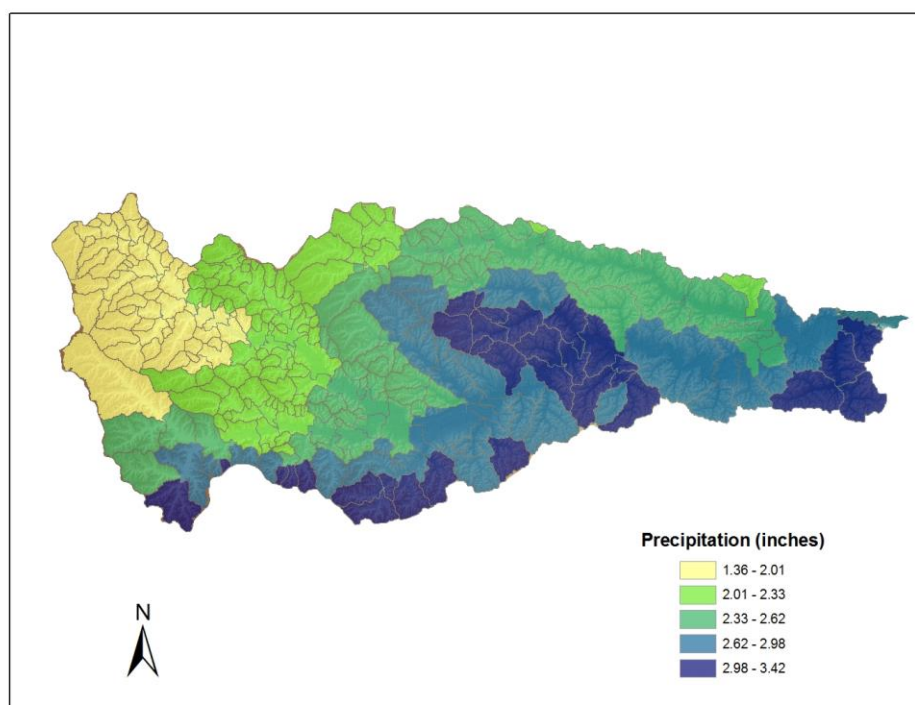


Figure 5.31: Total rainfall accumulation map for 13-14 June 2011

Once again the heaviest rainfall is concentrated over the southern half of the basin away from most of the projects. Using this distribution of rainfall the peak outlet discharge for the no pond condition was 10,266 cfs. If the same distribution of rain was applied to the pond conditions that were in place in 2005 the peak would have been

reduced to 9,747 cfs, and finally when all of the ponds are built and in place the peak is reduced to 9,726 cfs. This only represents a total basin reduction of 5% in peak flow. Similar to what was shown in the 2008 event since the heaviest rain was in the unprotected areas the benefits that are shown elsewhere are washed out by the higher flows in the unprotected areas. This is shown in Figure 5.32 and Table 5.7 with the hydrograph breakdown. The peak outflow was observed on June 14<sup>th</sup>. The daily average discharge at Ottumwa for the 14<sup>th</sup> was 42,800 cfs and 56,100 cfs in Keosauqua. This is a 13,300 cfs difference. The result from HMS of 10,266 cfs is below the difference between the gages and with the other contributing areas between Ottumwa and Keosauqua the modeled result is acceptable.

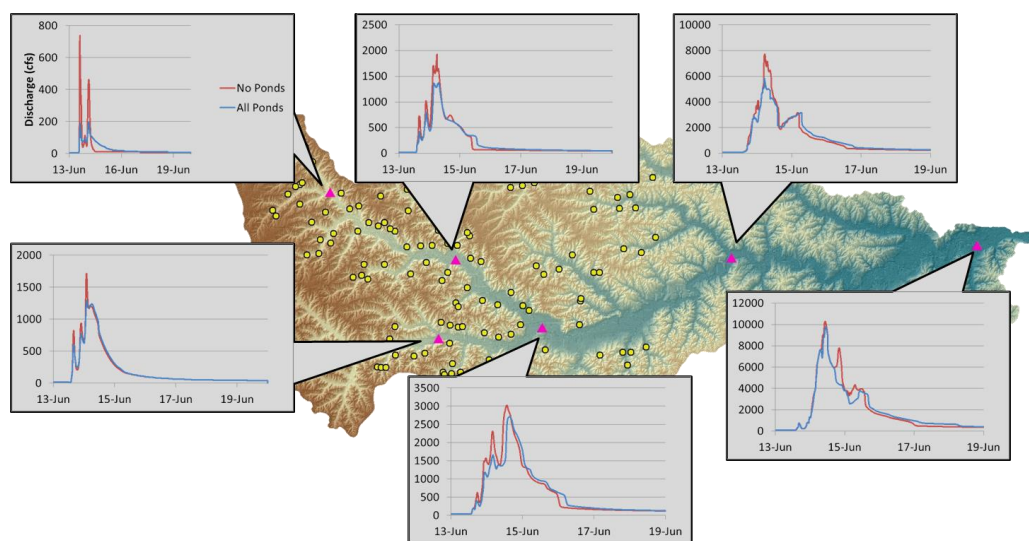


Figure 5.32: Hydrograph progression based on the June 2011 event

Table 5.7: Peak flow reduction and percentage reduction for Soap Creek during the June 2011 event

	1	2	3	4	5	6	Outlet
No Ponds	738.08	1931.71	1712.76	3026.47	7730.39	10290.7	10265.98
All Ponds	194.23	1370.21	1296.05	2708.64	5834	9750.4	9725.67
Reduction	543.85	561.5	416.71	317.83	1896.39	540.3	540.31
Percentage	73.68%	29.07%	24.33%	10.50%	24.53%	5.25%	5.26%

The ponds were able to store 3,127 acre-feet of water at their respected peaks resulting in only 25% of the total storage available being used. A breakdown of how each pond performed visually is shown in Figures 5.33 and 5.34. Most of the ponds were storing at least a half a inch of rain at their peaks while most of the ponds were all below 50% of maximum flood storage.

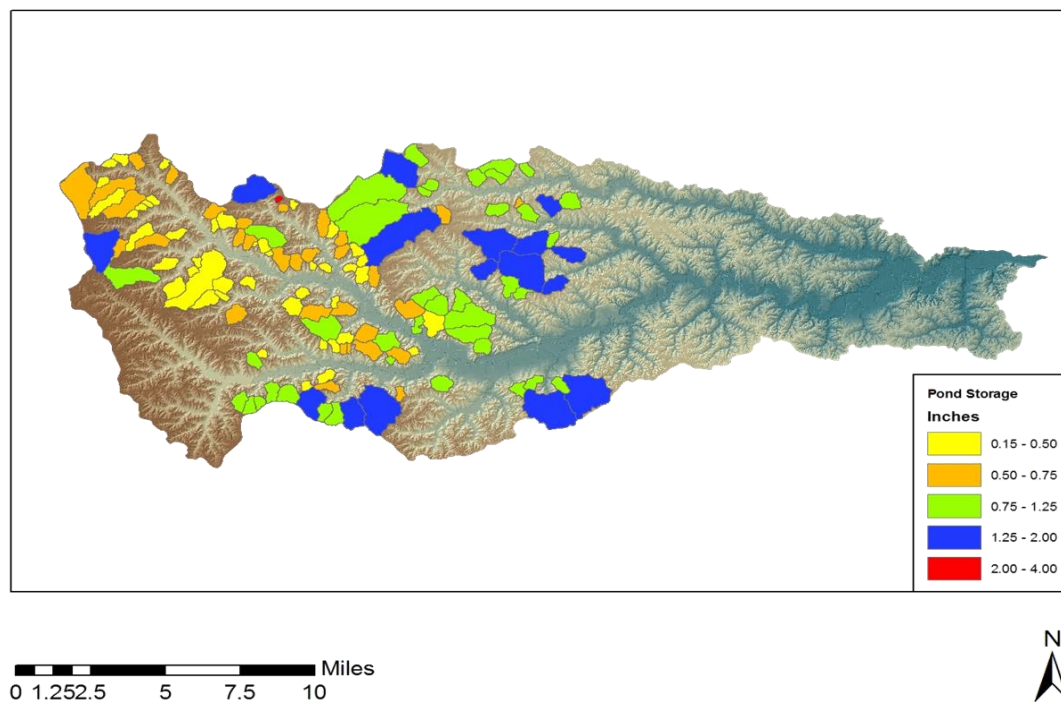


Figure 5.33: Peak pond storage displayed in inches from the June 2011 storm

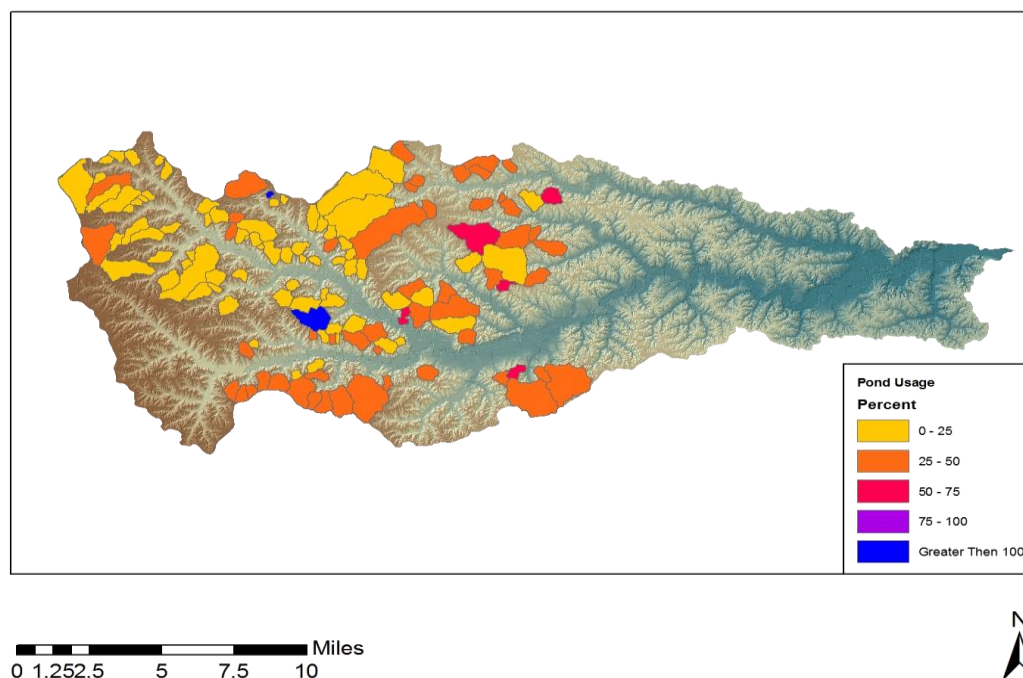


Figure 5.34: Pond storage from the 2011 storm reported in percent of pond usage

### 5.3.3 April 2013 Event

In the mid-section of April 2013 a line of very intense rainfall formed over the Midwest, with highest amounts recorded in Iowa and Illinois. In just over 24 hours many parts of the state received over 6 inches of rain. This led to wide scale flash flooding as well as moderate to major river flooding along many streams and major rivers. Once again NEXRAD radar rainfall was applied to better handle the spatial and temporal differences associated with rainfall events. The total storm duration rainfall is shown in Figure 5.35. Using the previous five day rainfall data shown in Table 5.8, the Type I AMC parameters were used to model this event.

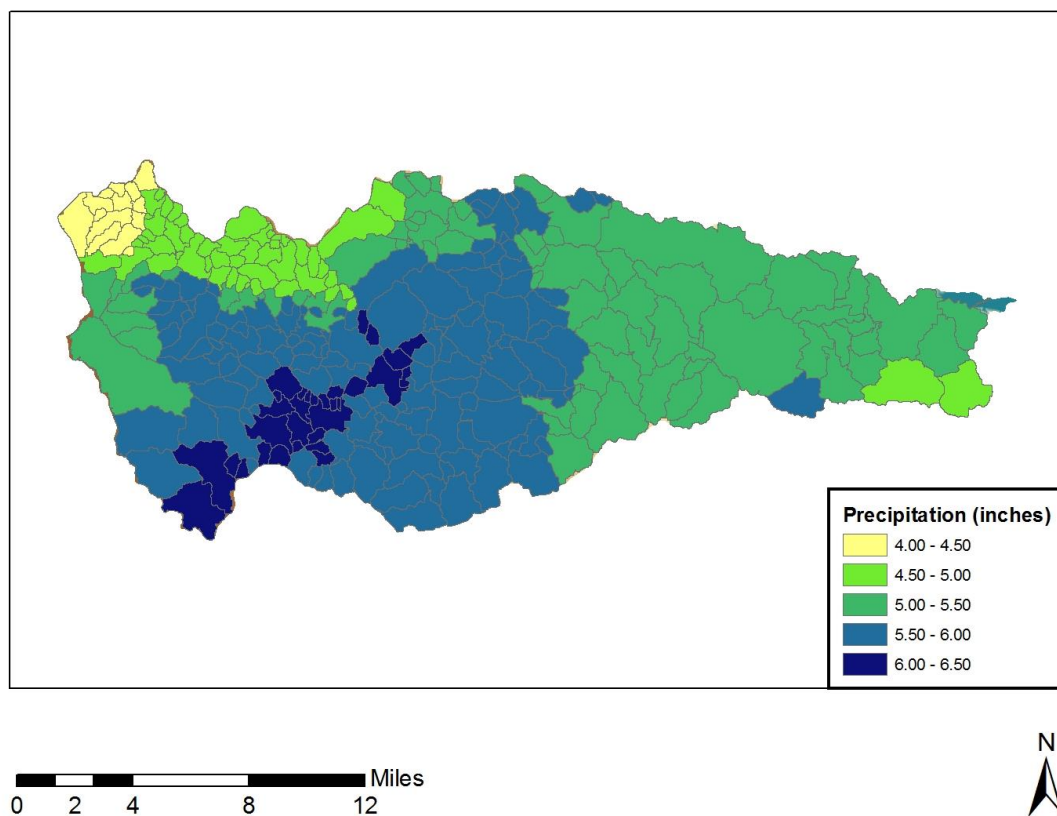


Figure 5.35: Total precipitation distribution from 14-19 April 2013

Table 5.8: Previous five day rainfall at Ottumwa, IA to determine AMC type

2013	
Date	Precip (mm)
12-Apr	0.00
13-Apr	0.00
14-Apr	10.16
15-Apr	0.76
16-Apr	0.00
Total	10.92
Type I	



Before looking at basin wide results the all pond model was run to determine how the model performed against the two IFC gages. Figure 5.36 and 5.37 show how the AMC I conditions performed versus the IFC gage. Both locations showed significant differences between the peak observed stage and the modeled values. These results are shown in the red line in both figures. Since these values were deemed too low and second approach was taken. Looking back at Table 5.4 we can see that the no growing season threshold for determining Type I and Type II ACM values are at 13 mm. The measured rainfall from Ottumwa, IA for the previous 5 days was reported at 10.92 mm. This is only a difference of 1.08 mm. As a result I then modeled the AMC II conditions and compared these results to the IFC gages. Once again looking at Figures 5.36 and 5.37, this time using the green lines we can see that both peaks are now much closer and the timing has not been greatly affected. Using these results the models were run using the AMC II conditions.

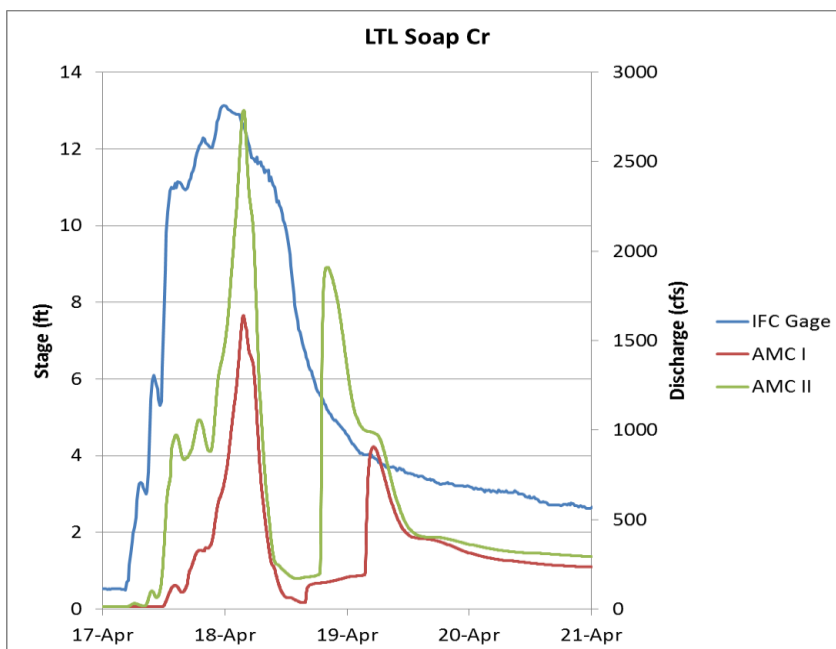


Figure 5.36: Hydrograph comparing IFC gage at Little Soap Creek to HMS results using AMC I and AMC II conditions

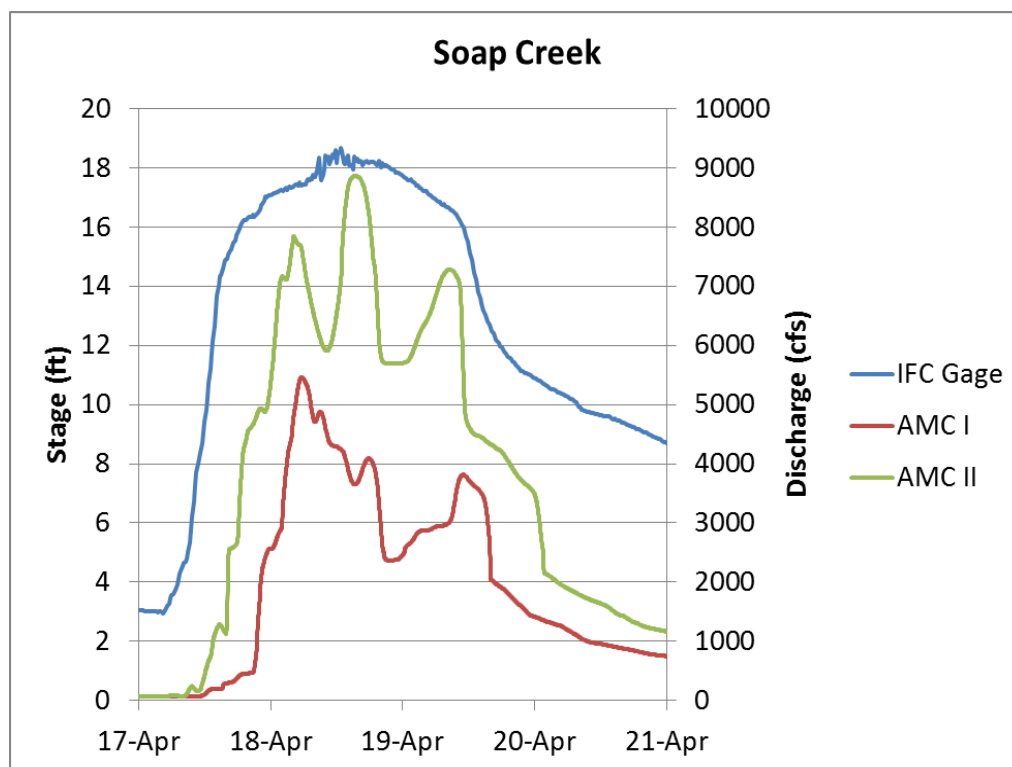


Figure 5.37: Hydrograph comparisons of IFC gage on Soap Creek to the HMS results of AMC I and AMC II soil conditions.

Once the loss conditions were selected as AMC II, I then ran the simulations for the no pond and all pond conditions. When looking at the hydrograph results shown in Figure 5.38 we see that the peak reductions are smaller than any seen in the design storms but basin wide reductions are large than the other two historical storms that were simulated. Using the total rainfall amounts we can see that results should follow those that we saw in the 25 year design storm. While the two storms do follow the same trends when looking at straight reductions the values are lower since the rain is spread out over a slightly longer time and not all concentrated in one pulse as in the design storms. The reductions are large over the entire watershed as a result of having a more uniform rain as compared to the other historical events. Figure 5.39 shows how much of the total rainfall each pond is holding during peak conditions. Many of the ponds are holding at least 2 inches of rain which corresponds to a minimum of 20-30% of total rain. This falls in line

with what was observed in the other events. Figure 5.40 show how much flood storage was utilized during this event. Since the rainfall was spread out over time and not all in one pulse we can see that the majority of the ponds are between 50% and 100% of total flood capacity. This results is 60% of the total flood storage being used, this also corresponds to the 25 year design storm.

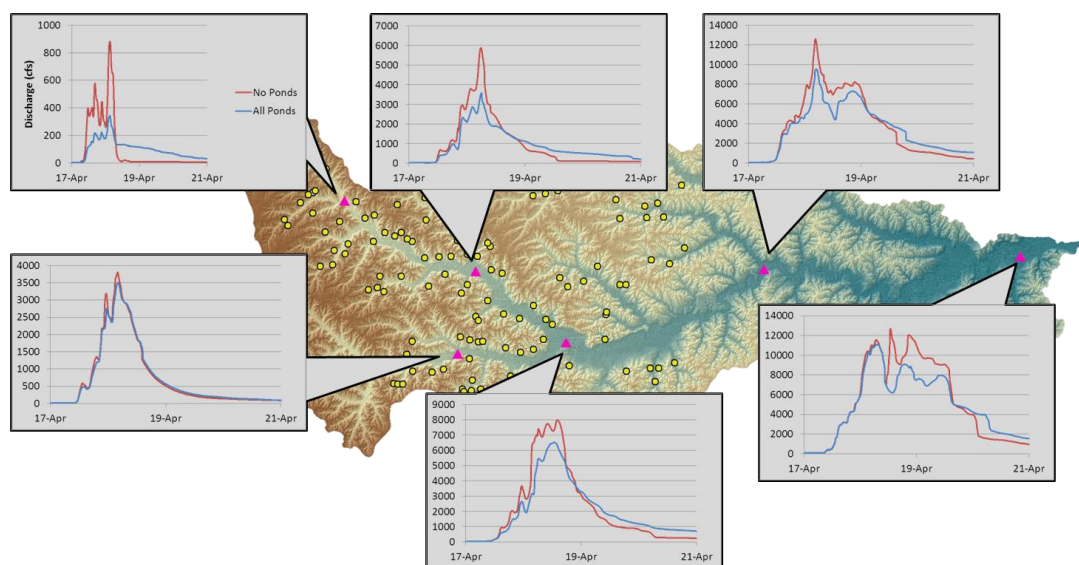


Figure 5.38: Hydrograph distribution showing results from the no pond and all pond conditions from the April 2013 event

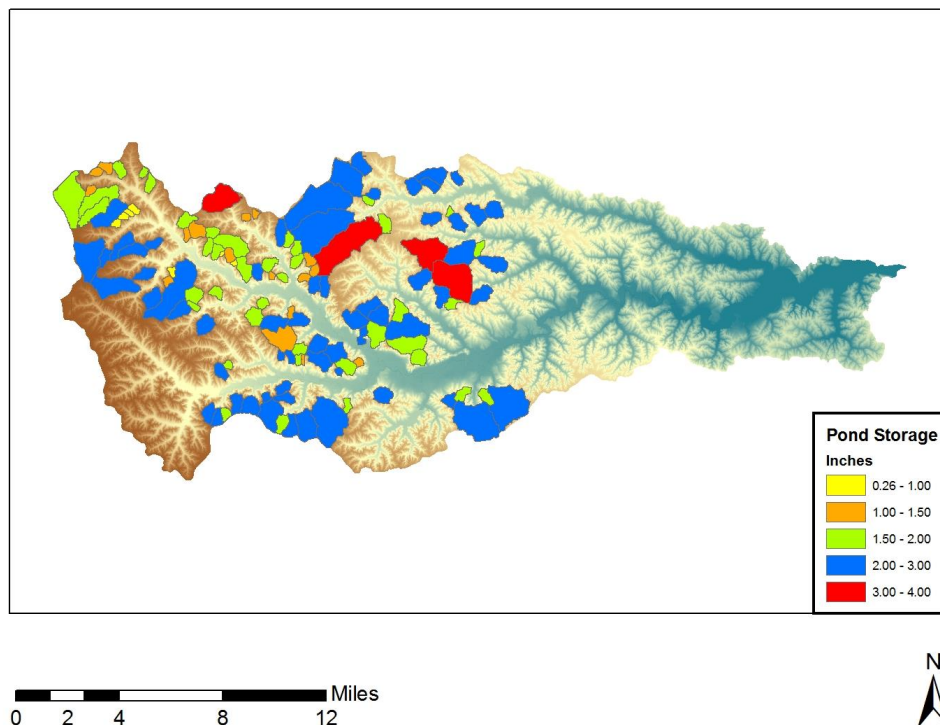


Figure 5.39: Peak pond storage for the April 2013 storm event, converted to a depth in inches

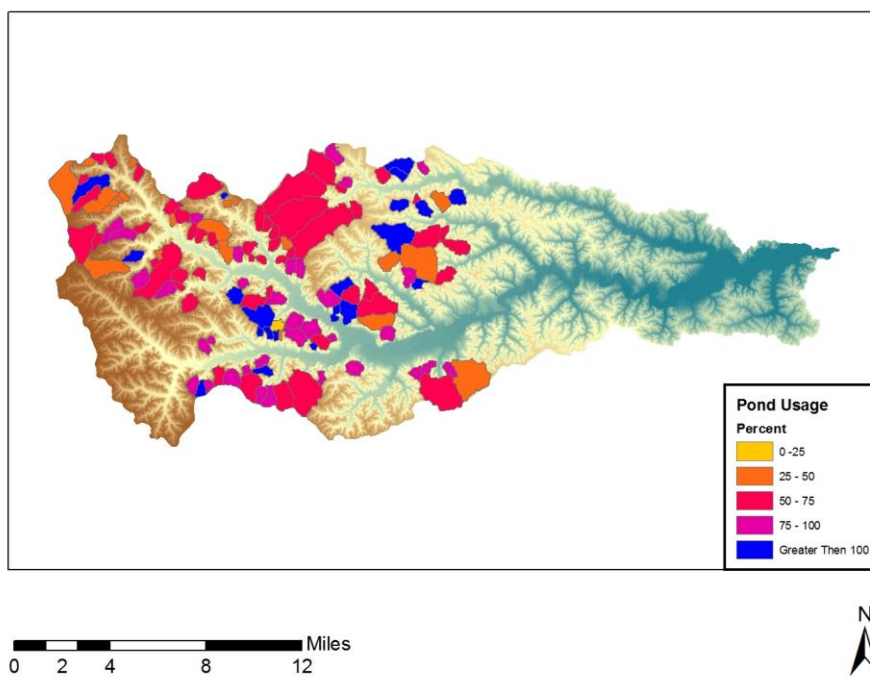


Figure 5.40: Pond usage from the April 2013 event, 60% of total flood storage was utilized

### 5.3.4 Historical Storm Summary

Three historical events were modeled in HMS as a way to convey results to the public and to provide prospective to where some events fall in the intensity of the design storms. All events were modeled using hourly NEXRAD data that gives the model a way to better handle the natural variability of rainfall distribution. These results showed that there is a benefit from the ponds that are in place, but also show that when a storm produces rain in the area of no projects that the effects downstream are not altered and may provide an example of where to build future projects.

### 5.4 Sensitivity Analysis

A simple sensitivity analysis was performed on the watershed to provide results on how the basin performs to changes in the basin characteristics. Only the 100 year design storm with the no pond condition was modeled in this analysis. The curve number was changed between the three soil conditions. Dry (CN1) and saturated (CN3) soil conditions were calculated from the normal conditions (CN2) used in previous simulations. To convert from one soil type to another the following equations are used. Figure 5.41 shows how the curve number changes for the different soil types.

$$CN1 = \frac{4.2 * CN2}{10 - (0.058 * CN2)}$$

$$CN3 = \frac{23 * CN2}{10 + (0.13 * CN2)}$$

Once the new curve numbers were calculated the remaining SCS methods were changed in response to the new curve number. These calculations included changing the initial abstractions and the basin lag times. Baseflow was omitted. When using the dryer CN1 values this lowers the curve number and increases the initial abstractions as well as increases the lag time. This allows for more water to be stored on the land surface,

requiring more rain before runoff can occur and increasing the time from water to runoff the land surface. The opposite happens when the soil is saturated.

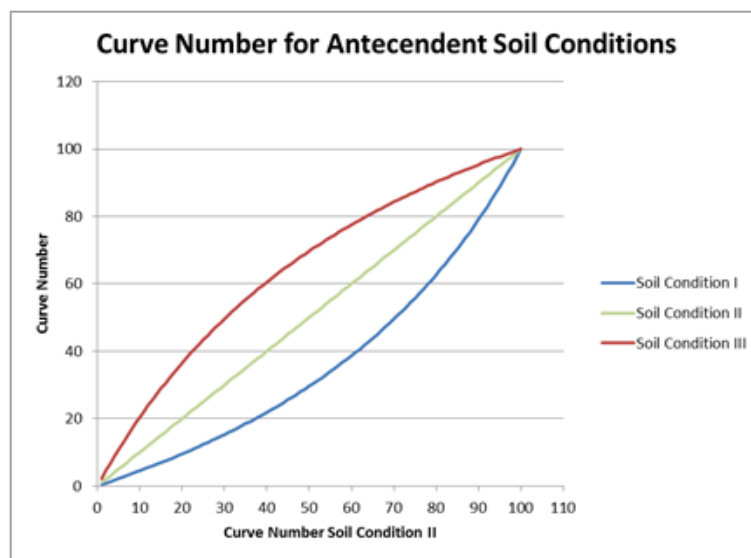


Figure 5.41: Curve number based on soil moisture conditions

Once all of the parameters have been changed the model was run for each soil moisture condition. Soil types II and III are based on the corresponding soil moisture conditions from CN1 and CN2. As expected all of the hydrographs maintain the same shape but have different peaks. The saturated soil condition peaks occur faster than the others while the dry conditions have the latest peaks and smallest peaks. The hydrograph break down is in Figure 5.42 and calculations of differences between soil conditions are shown in Table 5.9.

## Sensitivity

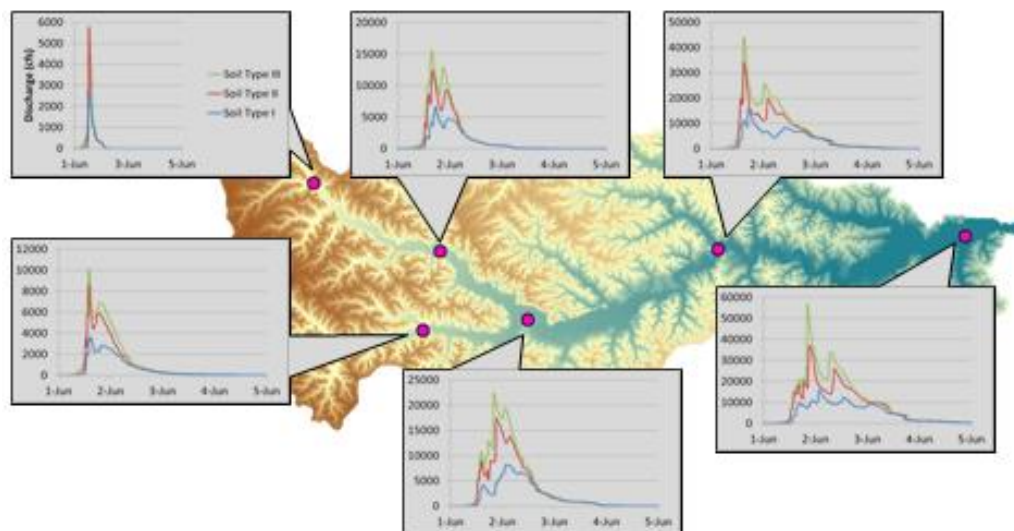


Figure 5.42: Sensitivity analysis of curve number based on the 100 year design storm

Table 5.9: Calculation breakdown of differences between soil conditions

Point	1	2	3	4	5	6	Outlet
CN1	2775.74	6635.63	3669.2	8214.2	15616.16	15838.64	15768.01
CN2	5752.77	12423.71	8574.41	17473.71	34350.61	37105.16	36815.57
Difference	2977.03	5788.08	4905.21	9259.51	18734.45	21266.52	21047.56
% Change	51.75%	46.59%	57.21%	52.99%	54.54%	57.31%	57.17%
Point	1	2	3	4	5	6	Outlet
CN2	5752.77	12423.71	8574.41	17473.71	34350.61	37105.16	36815.57
CN3	5862.24	15686.79	10163.57	22424.83	44072.75	57004.99	56637.72
Difference	109.47	3263.08	1589.16	4951.12	9722.14	19899.83	19822.15
% Change	1.87%	20.80%	15.64%	22.08%	22.06%	34.91%	35.00%

### 5.5 Chapter Summary

This chapter explored the numerous simulations that were run and the results that were produced. The first simulations used IFC gage data at two points as a way to validate that the model was working properly. Once the model was calibrated to match the gage data the design storms were modeled. Using six 24 hour design storms the model was able to produce results that showed how the system responds to quick intense rain. Starting with the 2 year storm the results were used to get a baseline or bankfull conditions. Continuing to the 10 year storm where the first ponds reached their capacity. While these results didn't meet the Soap Creek Watershed Plan design of holding all water up to the 50 year storm below the emergency spillway they can still contribute to the overall effectiveness of the system. Once the 50 year storm was run, 36 of the projects were shown to be at capacity which is on target per the guidelines provided in the watershed plan and with NRCS guidelines. The hydrographs showed that the most benefits are in the head waters where most of the projects are located.

After the design storms were modeled attention was focused on simulating historical events using NEXRAD rainfall data. Three events were modeled for their historical significance, the 2008 floods that devastated the entire state and two localized heavy rainfall events in 2011 and 2013. These results are to be used to provide perspective to local residents who remember high water marks but did not know what those levels correspond to and what those levels might have been if they didn't have the protection of the 132 ponds in place. Lastly a simple sensitivity analysis was done to compare how different soil conditions affect the basin. It was shown that when there are dryer soil conditions the effect on the basin is greater than when the soil is saturated.



## CHAPTER 6: CONCLUSIONS AND FUTURE WORK

From the numerous simulations that were run some general trends can be pulled out of the data and some conclusions can be made. Out of these preliminary results many refinements can also be made to make a better, more robust model. These conclusions and suggestions for future work will be described in the following chapter.

### 6.1 Conclusions

When examining the data that was gathered from the simulations a few trends and results begin to emerge. The biggest trend that shown in the results is the almost uniform flow reduction in the Point 1 hydrographs from all of the design storms. With the exception of the extreme event all of the simulations show a 70% or greater reduction of peak flow. This suggest that the pond location, design, and modeled results in the area upstream of Point 1 were done so well, that for a very wide range of precipitation there is always water being stored in the pond, but when heavier rain is distributed the ponds are able to store the increased volume while also passing enough water through to maintain this high reduction of flow.

Another trend that was observed during the modeling of the design storms was the basin wide trend of peak reduction was always increasing for the 2-100 year return periods. This led to the decision to model the extreme event to ensure that there is a point in which the effectiveness of the system is no longer able to keep up with the increased precipitation. This is shown in the results from the extreme event in which all of the hydrographs are showing the peak reductions begin to decrease from their levels that were seen during the design storm modeling process. This result shows that somewhere between the 7.5" and 12" rain there is a point where all of the available flood storage is consumed and any extra water is being allowed to flow over the spillways of the projects will be uncontrolled. This leads to a decrease in the peak reduction.

As mentioned in Section 5.5 the modeled results from the design storms suggest that the ponds were behaving as described in the NRCS guidelines and according to the design criteria of the Soap Creek Watershed Plan. These were shown in the 10yr flow of many of the ponds being 50% or less of flood capacity with a total system usage of 51%. When looking at the 50yr storm results, we saw that many of the ponds were between 75-100% full and 87% of the total system storage in use. This result didn't directly match with the design specifications of the ponds being able to hold the water up to the 50yr storm. By the time we modeled the 100yr storm and saw the results more than 100 of the 132 ponds had reached their individual maximum flood storage, with another 15 nearing full utilization. As a result 93% of all available flood control was being used. This also is nearing the peak effectiveness of the system.

The historical storms also provided some trends that differed from the design storm results. The first example of these differing trends can be seen in the lower values of peak reduction. This result shows how an uneven distribution in rain can affect the perceived basin wide benefit. As shown in Figures 5.28 and 5.32, the peak reductions near the outlet are much smaller compared to the design storm results, while these numbers are discouraging to see the hydrographs also show many of the secondary peaks have been eliminated so there are benefits, they just are not seen in the peak reduction. Second, these historical events show where new projects could be considered in the future. While many of the southern areas feed directly into the main stem where many of the older ponds were built in higher order stream locations, ponds built in these southern areas could provide a benefit for events where heavier rainfall is accumulated in the lowest portions of the basin.

Another trend that is shown from the historical events is one of timing for rainfall. Using real world precipitation instead of a design storm that applies a majority of the rain in a three hour window, the total storm precipitation is applied over a longer period. This allows the ponds more time to pass water through their pipes and allow for more storage

to be available over time. This showed how the issues associated with this type of modeling are presented in both space and time.

## 6.2 Future Work

This study provides the ground work for future study of the Soap Creek and the other IWP watersheds. My work was in creating an approximate model of the area to provide the Soap Creek Watershed Board and the public a model that can be refined to give higher accuracy results. At the same time my work provides the base results that show the model works as intended and to begin providing qualitative values for the benefits created from implementing the Soap Creek Watershed Plan.

Future work can be done in refining the model by using compound channels in the storage-discharge relationships. Currently all streams were modeled using a single Manning's N, this results in the floodplains conveying water more efficiently than the real world would allow. Using a compound n value across the channel at elevations out of channel would provide added detail in the model. Another refinement in the channel would be to add more cross sections within each reach to get a better understanding in the geometry of the channel as well as the floodplains. Right now my model assumes a constant geometry within each reach segment. Using more cross sections will provide more detail in generating the volumes in the storage-discharge table that are required in the Modified Puls routing method that was selected for modeling the Soap Creek.

A third item for future model implementation would be gathering reliable data from the remaining two IFC gages that were not used in the validation/calibration process described in Section 5.1. These additional gages would provide upstream calibration conditions to better validate the model to ensure that all locations are providing results that best mimic reality. When only using the two downstream locations for validating/calibrating there is a vast majority of the watershed one would need to look at

when trying to diagnose a potential error in the model. When using locations spread throughout the watershed the errors become easier to locate and as a result quicker to fix.

### 6.3 Chapter Summary

This chapter examined some the trends that were discovered during data examination of all of the modeled simulations. Trends that were observed included validation of pond performance comparing against the NRCS specifications and Soap Creek design criteria. Another trend that was noticed was the increasing system reduction of peak reduction within the hydrograph results. These results were then confirmed by modeling an extreme event to show that there is a point at which the system is overwhelmed and peak reductions begin to fall. Historical events provided information of what happens when the rainfall is not uniformly distributed, as well as some insight on where potential future projects could be located to combat rainfall in the lower portion of the watershed.

Many refinements can be made to produce a more accurate model. Many of these adjustments could be made in the relationships that make of the reach storage-discharge tables. Changes include using composite Manning's N values for channel and floodplain designation, and using multiple cross sections to better represent channel geometry. These will result in more accurate representation of channel discharges and volumes. Using all four IFC gages in the Soap Creek will also provide more opportunities to validate and calibrate the model to achieve optimal performance.

## CHAPTER 7: SUMMARY

Following in the work done in watershed around the globe, flood control has been an extensively studied topic. Most of the work that has been done to date has been on the main stem of major rivers. Using the Missouri River as an example of distributed flood control gave the current study a baseline. While these practices were done on the main stem the USACE was able to better control and regulate the lower basin for navigation. This study was designed to show the effects of distributed storage in headwaters of smaller basins and the benefits basin wide.

After the 2008 floods the state of Iowa received a federal grant from the Federal Department of Housing and Urban Development (HUD) to fund flood recovery and future flood protection in the state. A portion of this fund was given to IIHR - Hydroscience & Engineering for the Iowa Watershed Projects (IWP). Through this funding four watersheds were selected to have a detailed hydrologic assessment completed. Soap Creek was chosen as one of the four watersheds. The results of this study will be used as a source of information for future work in the IWP. Soap Creek presented a unique attribute in that compared to the other selected watersheds they already had a plan in place and had spent the last 20 plus years constructing 132 ponds in the headwaters in an effort to control flooding and provide water for various agricultural practices.

Modeling of the watershed was done using the HEC-HMS software developed by the USACE. Another unique attribute of Soap Creek was that there was no stream monitoring in place. This created the challenge of having to base all of the model setup and parameters on physical characteristics. This required that the models be set up using SCS methods for the loss and transform methods. The modified puls method was selected for the routing as there were no stream gages to calibrate to. Rainfall was

applied using uniform 24 hour SCS design storms, NEXRAD radar, and uniform hourly rain gage data.

Model set up was done using tools in ArcGIS. ArcHydro was used to handle all of the terrain preprocessing. This included refining the DEM to ensure water flows into the stream network to creating the subbasins and generating the stream network. All physical characteristics were calculated in HEC-GeoHMS, and extension in ArcGIS. Once all of the basins and streams were edited to best fit the needed layout all of the files were exported to HEC-HMS.

Once a simulation was run data was pulled from seven locations along the stream to show progression of water through the basin. Each scenario was modeled with no ponds and with a progression of pond construction until the model was run with all 132. The 2, 5, 10, 25, 50, and 100 year 24 hour SCS storm were modeled to show how the basin handles an increasing amount of precipitation. The 25 year storm was show to provide the greatest basin wide benefit. Three historical storms were modeled to show how the basin moves water through the system when the precipitation is not uniformly distributed. Two of these storms used NEXRAD data the third used hourly gage data. The third historical storm looked at how the System handles a winter snowmelt event. A sensitivity analysis was performed to show how soil conditions can affect the results.

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