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The effects of increased infiltration and distributed storage on reducing peak discharges in an agricultural Iowa watershed: the Middle Raccoon River

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THE EFFECTS OF INCREASED INFILTRATION AND DISTRIBUTED STORAGE ON REDUCING PEAK DISCHARGES IN AN AGRICULTURAL IOWA WATERSHED: THE MIDDLE RACCOON RIVER

by

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CERTIFICATE OF APPROVAL

MASTER'S THESIS

This is to certify that the Master's thesis of

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has been approved by the Examining Committee
for the thesis requirement for the Master of Science
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Dedicated to my Father, Michael D. Klingner, and Grandfather, William H. Klingner, whose contributions to the fields of Water Resources and Civil Engineering have inspired me to follow in their footsteps.

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ABSTRACT

The devastating Floods throughout Iowa in 2008 caused homes to be lost, people to be displaced, and total economic losses exceeding $10 billion dollars (Baldwin, 2008). This left State Officials pondering how to limit the damages of large magnitude floods in the future. From the legislative sessions following this tragedy came the Iowa Flood Center (IFC) and funding through the Department of Housing and Urban Development (HUD), among others, to begin the Iowa Watersheds Project (IWP). Through House File 2459, the project was tasked with the planning, implementation and evaluation of watershed projects to lessen the severity and frequency of flooding in Iowa. One test watershed studied was the Middle Raccoon River watershed in West Central Iowa.

To study the impacts of basin-wide flood mitigation strategies on the Middle Raccoon River watershed, the hydrologic modeling software HEC-HMS was used in conjunction with the geographic analysis software, ArcGIS. A model was developed and calibrated to best represent the observed hydrologic response at USGS stream gages located at Bayard, IA and Panora, IA. Once completed, a series of flood mitigation techniques were applied to the watershed model, and run with the 10-, 25-, 50-, and 100-year SCS design storms. These techniques include increasing infiltration by modifying land use, and applying a distributed storage system (ponds). Both practices are shown to have the ability to reduce peak discharge, from 4 percent to 56 percent, depending on the location in the watershed, the severity of the design storm, and the extent of the flood mitigation technique.

Although research describing the effects of distributed storage and increased infiltration currently exist, this study details the process in which these effects can be modeled in a heavily agricultural Iowa watershed using a simplified lumped parameter model, HEC-HMS. With recent major flooding events in Iowa, and increased interest

in alternative flood mitigation techniques, the methods and tools in this report will be valuable in predicting the effectiveness of flood projects prior to project construction.

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CHAPTER 1: INTRODUCTION

Project Description

Floods along the major rivers and tributaries in Iowa, and throughout the Midwest, are not new to those living among its floodplains and river banks. Notable floods in the summers of 1993 and 2008 and the spring of 2013 brought economic, social, and environmental hardships to individuals and communities in watersheds throughout the region. The Great Flood of 1993 alone caused 12 to 16 billion dollars of damage along the Upper Mississippi River corridor (Galloway, 1994), with similarly high amounts caused by the Flood of 2008—$10 billion in Iowa alone (Baldwin, 2008). In response to these events, Iowa has seen an increase in flood research funds and the formation of new Watershed Management Authorities (WMA) – quasi-governmental bodies consisting of representation from municipalities, counties, and soil and water conservation districts. This report will focus on one such authority—the South Middle Raccoon River WMA. The WMA has sought to mitigate the effects of large flooding events using a unified watershed wide approach. The purpose of this paper is to analyze the effects of watershed improvements and flood control structures on peak discharge in the heavily agricultural, Middle Raccoon River watershed. While the hydrologic effects of traditional flood control structures such as levees, floodwalls, large reservoir storage, diversion channels, and floodplain improvements have been extensively studied and measured, the modeling and effects of distributed storage on a peak discharge in a heavily agricultural, Midwestern watershed have not.

This analysis of the Middle Raccoon River watershed is part of the greater Iowa Watersheds Project. The project originated from 2008 disaster funding of which the state received $84.1 million from the Federal Department of Housing and Urban Development (HUD) to benefit the 85 presidentially declared disaster counties in Iowa (Figure 1.1).

Specifically, the purpose was to "plan, implement, and evaluate watershed projects to lessen the severity and frequency of flooding in Iowa", (IIHR, 2012).

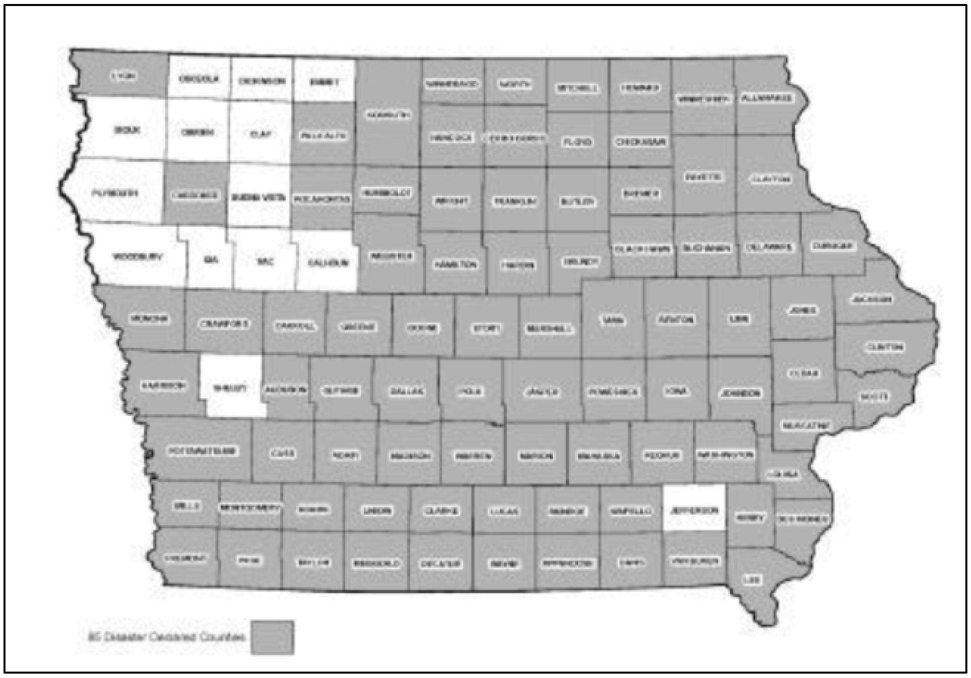

Figure 1.1: Federally declared disaster areas after the 2008 flood (Ralston, 2010)

To achieve this overarching project goal, four watersheds across Iowa were studied to understand the hydrology – movement of water – within the local watersheds and the effects of certain flood mitigation techniques, particularly distributed storage. Along with the Middle Raccoon River, three other watersheds were studied – the Turkey River, the Upper Cedar River, and the Soap/Chaquest Creeks (Figure 1.2).

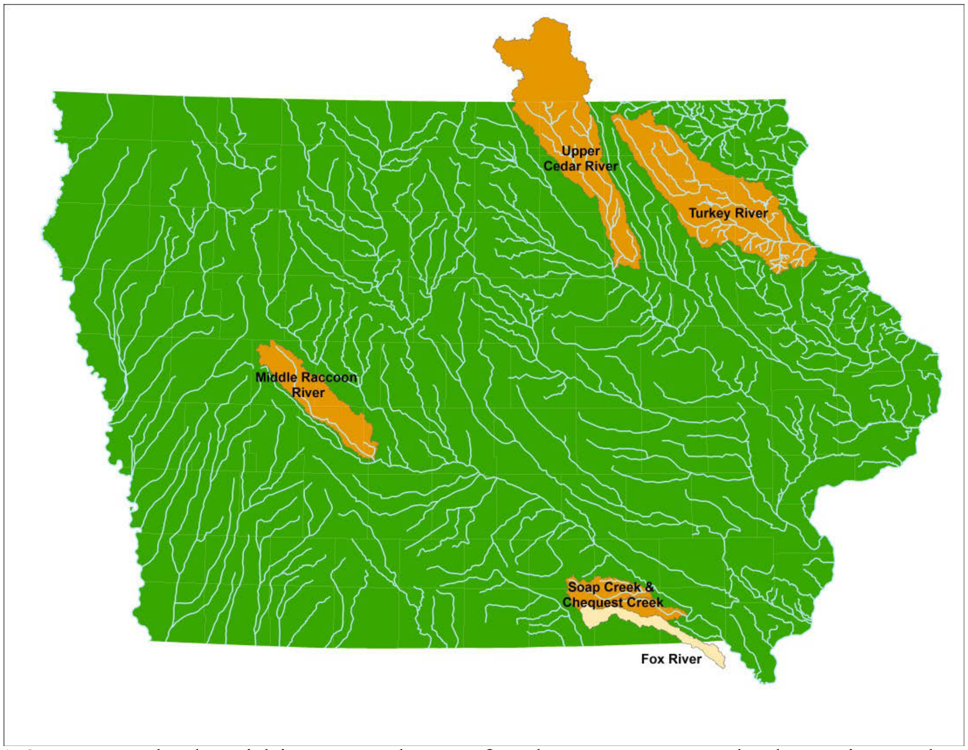

Figure 1.2: Watersheds within Iowa chosen for the Iowa Watersheds Project, Phase I funding. These watersheds include the Upper Cedar River, the Turkey River, Soap and Chaquest Creeks, and the Middle Raccoon River.

$8.8 million of the $84.1 million was allocated to these watersheds through the Iowa Department of Economic Development and supplemental Community Development Block Grant funding in order to fulfill the aforementioned goal.

The Iowa Watersheds Project is broken into two phases. Phase one focuses on the watershed selection, community engagement, hydrologic development using the United States Army Corps of Engineers (USACE) HEC-HMS program, and the identification of flood control projects and their respective locations. Phase two involves a more detailed hydrologic analysis on the scale which projects will be implemented, Hydrologic Unit Code (HUC) 12, using Hydro GeoSphere. It will also include the actual project construction and implementation. Project types may include active and passive distributed storage, floodplain restoration, buffer strips, advanced tile drainage, urban/rural infiltration practices, and floodplain easement acquisition. The focus of this research will be on Phase One of the project, particularly passive storage and rural infiltration practices. This is accomplished by developing a hydrologic model using HEC-GeoHMS in the Middle Raccoon River Watershed.

Phase One of the Iowa Watersheds Project began in June 2012 and concluded with the delivery of the Middle Raccoon Hydrologic Assessment Report in the spring of 2014. Phase Two began in fall of 2013, and is scheduled for completion in the summer of 2017 with the delivery of the Phase Two Report.

Selection of the Middle Raccoon River Watershed

The selection of watershed locations for the Iowa Watersheds project began with a request for information (RFI) sent to the 85 declared disaster counties in Iowa. The RFI requested information on watersheds, no larger than HUC 8 scale, which were interested in reducing the effect of flooding within the watershed bounds. The group that was to become the Middle South Raccoon River Watershed Management Authority replied to the RFI, and was ultimately chosen for Phase 1 funding.

As in many watersheds across Iowa, the Middle Raccoon River watershed has witnessed changing hydrologic conditions since its permanent settlement in the late 19th Century. During this period, land cover over the Iowa and the Middle Raccoon River watershed changed from native tall-grass prairies and forests to higher runoff-producing row crops. Along with changing land cover, the region has seen increases in annual and seasonal precipitation totals, along with changing frequencies, intensities, and seasonality of rainfall events (Takle, 2010). These watershed alterations may be two of the driving factors contributing to statistically significant shifts in mean daily discharge at the Raccoon River at Van Meter (Villarini et al., 2011). Figure 1.3 shows mean daily discharges increasing around 1968.

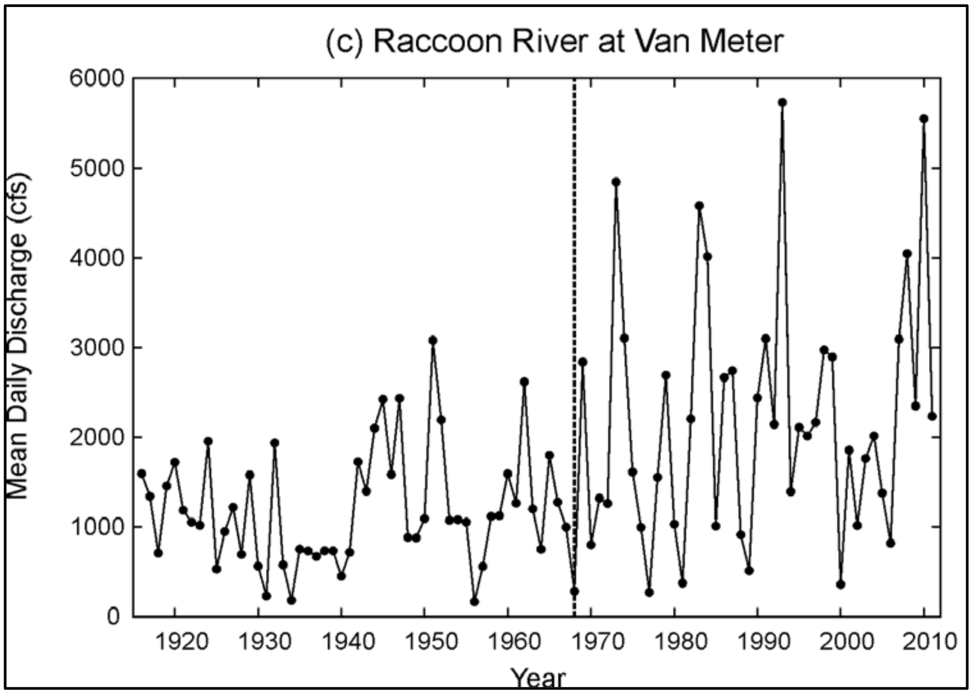

Figure 1.3: Increasing mean daily discharge on the Raccoon River at Van Meter, IA. Significant statistical increase occurs around 1968. The Raccoon River is located directly downstream of the Middle Raccoon River.

These known hydrologic conditions, along with voluntary participation from the South Middle Raccoon River Watershed Management Authority, were two of the reasons this watershed participated in the Iowa Watersheds Project.

Thesis Outline

This thesis will contain a brief literature review detailing some of the published research on the topics related to the modeling of flood mitigation in the Middle Raccoon River watershed. These include the determination of locations of high potential runoff, the effects of tile drainage on flood hydrography, and the use of distributed storage in reducing flood severity, frequency, and damages. These topics are discussed within the context of their potential uses in a lumped parameter model such as HEC-HMS. I will then discuss the general information of the Middle Raccoon River watershed, including its current land uses, soil types, and topography. The Middle Raccoon River watershed back ground information section will also include pertinent information on the data and instrumentation used in this research. In the following sequence, I will discuss the application of the Middle Raccoon River watershed data, and how it was used to develop the HEC-HMS Hydrologic model. The model calibration and validation are important aspects of Hydrologic Development chapter. Next, I will analyze the hypothetical increased infiltration and distributed storage systems applied to the calibrated hydrologic model, and quantify each scenario's peak discharge reduction capabilities. Lastly, the most important outcomes and lessons learned from the modeling of the Middle Raccoon River watershed will be explained in the Summary and Conclusions Section.

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CHAPTER 2: LITERATURE REVIEW

Introduction

The purpose of the Middle Raccoon River watershed literature review is to gain an understanding of similar studies that have been conducted regarding the topics of this report. These include methods for determining where flood mitigation projects could have their greatest impact, the effect of antecedent moisture and tile drainage on flood hydrography for the purpose of validation and calibration, and the impact of distributed storage project on reducing flood severity, frequency and damages, using lumped parameter models. Of course, this research is unique in its location, approach, and results; yet, lessons learned and methods used in other similar studies can be critical resources.

Siting Distributed Storage and other Flood Mitigation Projects

The siting of distributed storage ponds in other similar flood mitigation projects has a significant impact on the overall success of the project, and therefore, needed to be addressed in this study. One such study analyzed the effects of flood control on peak discharge potential within a $110 mi^2$ watershed in Iran. The study concluded that land use changes close to the catchment outlet will have the least impact on flood peaks, and subbasins, with times of concentration equal to approximately 50 percent of the overall watershed time of concentration, are the most efficient (Raughani et. al, 2005). The location of distributed storage reservoirs, using mathematical algorithms to maximize flood water storage, was analyzed, based upon potential highest reservoir depth and highest storage potential in the Clear Creek Watershed in East Central Iowa. The report also looked at the economic and legal aspects of applying a large scale distributed storage system (Baxter, 2011). Flood project sites were again studied to develop a method to determine the locations of potential wetlands in tile-drained, Midwestern landscapes

(Babar-Sebens et. al., 2012). The study used GIS-data, Storm Water Assessment Tool (SWAT) modeling, and optimization techniques to apply wetlands across the landscape. Using this method, the wetland projects were able to capture runoff from 29 percent of the watershed area. The use of distributed storage for flood control, and the need to site these projects are both critical themes in this report. While it is clear that these aspects have been studied in a variety of scenarios, the Middle Raccoon Watershed has its own unique circumstances, which required the use of additional parameters and methods for the siting flood control projects, resulting in potential flood reductions unique to this watershed.

Calibration/Validation Techniques: Antecedent Moisture and Tile Drainage

Special calibration and validation efforts had to be made to reflect the hydrologic response of the Middle Raccoon River watershed. Most notable were the efforts in tile drain hydrology and antecedent moisture conditions. Tile drains are used extensively throughout the Midwest in an attempt to improve growing conditions. By 1987, more than 17 percent of U.S. cropland and up to 30 percent of cropland in the Upper Midwest had been altered by artificial surface or subsurface drainage (Pavelis, 1987). Tile drains work by removing excess water from the soil and creating an environment that allows greater plant uptake of nutrients. Drainage significantly increases crop growth and productivity (Zucker & Brown, 1998), Figure 2.1. However, the effect of tile drainage reaches beyond plant productivity; the removal of excess water in the soil also has a profound effect on the hydrologic response of the watersheds in which they are installed. The tile drains provide an underground preferential flow path, by which water that falls on the flat Midwestern landscape can infiltrate quickly and move through the tile drains much quicker than its natural conditions would allow. Although tile drainage is thought to predominately affect the baseflow portion of the stream hydrograph, it also may affect storm flow (Schilling and Libra, 2003). Sloan reviewed upward of twenty field studies around the world, relating peak stream flows with tile drainage and soil type, and concluded that in highly clay soils, tile drains attenuate the flow to a greater extent than surface pathways, which lead to an observed decrease in peak flows. In sandy soils, the permeability of soils is already high, therefore, the installation of tiles increases this already high subsurface flow, resulting in higher peak stream flow (Sloan, 2013). Restated, this means that where land already has been converted to agricultural production, the addition of subsurface drainage may reduce runoff and peak outflow rates (Blann et at., 2009). The decrease in peak flow, along with the fact that subsurface drainage can create increased temporary storage capacity in the upper layer of soil, allowing water to infiltrate and spread through the soil over a longer period (Blann et al., 2009), and the recession curves (after a storm event) from a tile-drained watershed which appear to be more linear than less-tiled watersheds (Schilling & Helmers, 2008), are all used in the justification of the calibrated parameters in this report.

Antecedent moisture, in the most general sense, relates the magnitude of the five day precipitation to three moisture states: dry (AMC I), average (AMC II), and wet (AMC III), (NRCS, 1972). The value of these three moisture conditions is show in Table 2.1. Statically, these three conditions relate to the 90, 50, and 10 percent cumulative probability of exceedances of runoff depth for a given rainfall (Hjelmfelt et al., 1982). However, this creates sudden unrealistic “jumps” in the curve number, resulting in widely varying percentages of runoff (Ram Kumar Ahu et al., 2010). The definition and underlying issues with antecedent moisture conditions are addressed and accounted for in the calibration of hydrologic parameters.

Lumped Parameter Modeling for Analyzing the Impact of Distributed Storage on Peak Discharge

Lumped parameter models are models in which the physical characteristics of the watershed, such as land use and soil type, are "lumped" together to form a single representative value for a given land area. The use of distributed storage as a means of flood control is an emerging method of flood mitigation that replaces the single, large reservoir projects that were popular during the mid-20th century. Existing distributed storage systems in Iowa have been previously studied using the program HEC-HMS, where Wunsch, 2013 showed potential flood reductions of 70 percent or more in the Soap Creek Watershed in Southeastern Iowa (253 mi^2). A similar study using HEC-HMS was performed on the smaller scaled Valley Creek watershed (24 mi^2) in suburban Philadelphia where more than 100 storm water detention basins showed a maximum potential peak flow reduction of 9 percent (Emerson, et. al, 2005). Other software packages, such as the Soil and Water Assessment Tool (SWAT) are also lumped parameter models that have studied the effects of distributed storage. SWAT is a tool that is used for long term continuous simulations, while HEC-HMS is more appropriate for short event based simulation. SWAT was used in conjunction with ArcGIS to determine wetland locations and quantify the effects of wetland distribution in the Eagle Creek Watershed near Indianapolis, IN. It concluded that the locations of these wetlands have a significant impact on peak flow reduction, with maximum peak flow reductions of nearly 20 percent (Babbar-Sebens, 2012). The SWAT program has been used to determine nutrient processes in the Raccoon watershed, of which the Middle Raccoon makes up approximately one third the area (Jha et. al., 2007). While this study did not use distributed storage to determine peak flow reduction, it did model and calibrate some of the hydrologic components in an area which directly incorporates the study area of this project. Therefore, parameters in the SWAT model could be utilized for the purposes of this study as quality control.

Chapter Summary

The purpose of this section is to review the research that has been developed and published for topics relating to the use of distributed storage as flood control on the Middle Raccoon River watershed. The references in this section provide guides and tools on how similar research has been performed in the past, and allow one to build upon the knowledge gained from their review.

CHAPTER 3: BACKGROUND INFORMATION & WATERSHED DESCRIPTION

Introduction

This chapter provides an overview of the current Middle Raccoon River watershed conditions including hydrology, geology, topography, land use, hydrologic/meteorologic instrumentation, as well as a summary of previous floods of record. A thorough investigation of the most current, applicable, and accurate data available for the Middle Raccoon River watershed was required in order to develop the appropriate Geospatial data layers in ArcGIS. From the data gathered, the initial parameters and watershed characteristics were developed for hydrologic model development.

Hydrology

The Middle Raccoon River watershed is comprised of 590 square miles (mi^2) in West Central Iowa. The Watershed encompasses approximately half the area of the South Raccoon River eight-digit Hydrologic Unit Code (HUC8) 07100007. It is made of Four HUC10's, and 15 HUC12's. The majority of the watershed boundary falls within four counties—Carroll, Greene, Guthrie, and Dallas. The main stem of the Middle Raccoon is located in the southern portion of the watershed and is fed by three primary tributaries from the north—Storm Creek, Willow Creek, and Mosquito Creek. The Middle Raccoon River drains to the South Raccoon River near Redfield, IA, and then flows east to meet the Des Moines River in Des Moines, IA.

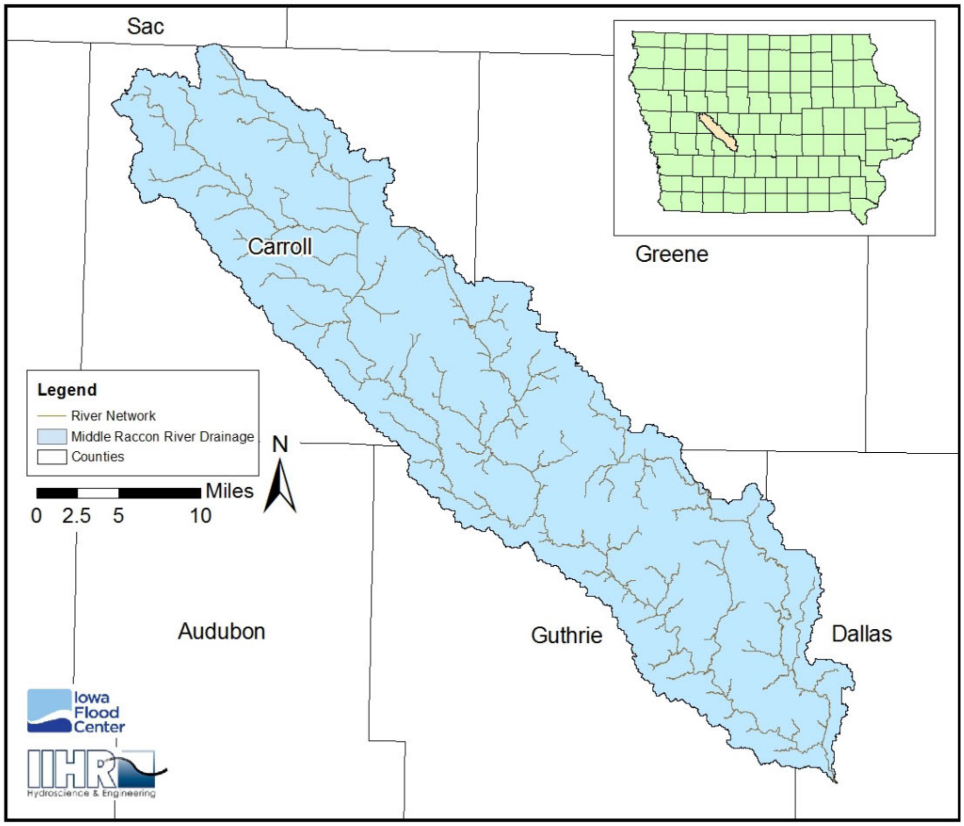

Figure 3.1: The drainage area for the Middle Raccoon River watershed, part of the Southern Raccoon River HUC 0710007. The watershed drains 590 mi², in West Central Iowa.

Geology and Soils

The Middle Raccoon River watershed is split by two identified landform regions—the Des Moines Lobe and the Southern Iowa Drift Plain, each of which has a unique influence on the rainfall-runoff characterization of the watershed. The Southern Drift Plain Region of Southern Iowa covers 33 percent of the watershed and is characterized by numerous rills, creeks, and rivers which branch out across the landscape, shaping glacial deposits into steeply rolling hills and valleys. In contrast, the Des Moines Lobe Region of Central Iowa covers 67 percent of the watershed and is characterized by a poorly drained landscape of pebbly deposits, with broadly curved bands of ridges and knobby hills set among irregular ponds and wetlands, punctuating the otherwise subtle terrain (Iowa Geological & Water Survey, Iowa Department of Natural Resources, 2013).

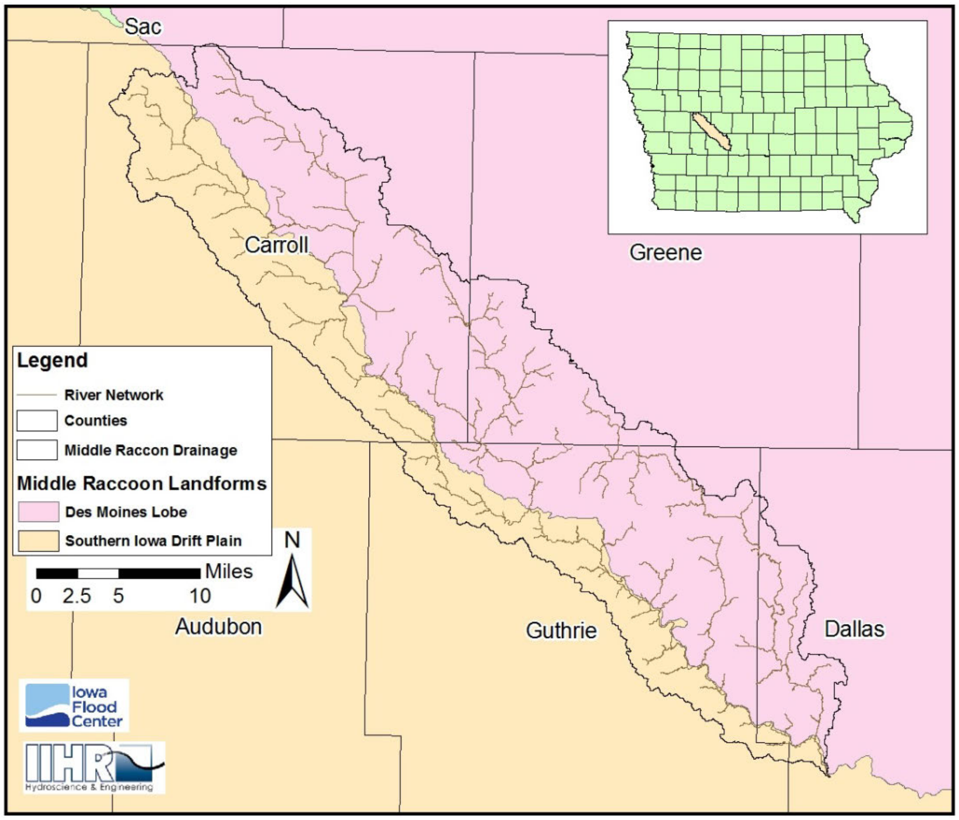

Figure 3.2: Defined landform regions of the Middle Raccoon River watershed. The Southern Iowa Drift Plain in the South is characterized by heavy relief. The flatter Des Moines Lobe is to the North.

The basin is composed primarily of moderately drained soils. Soils are classified into four Hydrologic Soil Groups (HSG) by the Natural Resources Conservation Service (NRCS) based on the soil's runoff potential. The four HSGs are A, B, C, and D, where A-type soils have the lowest runoff potential (highest infiltration capacity) and D-type have the highest runoff potential (lowest infiltration capacity). For reference, sand or gravel would classify as an A-type soil, whereas clay or silt would classify as a C or D-type soil. In addition, there are dual code soil classifications—A/D, B/D, and C/D—

which are assigned to certain wet soils. In the case of these soil groups, the soil properties may be favorable to allow infiltration, but a shallow groundwater table (within 24 inches of the surface) typically prevents much infiltration from occurring (Hoeft, 2007). For example, a B/D soil would have the runoff potential of a B-type soil if the shallow water table was to be drained away or lowered, but the higher runoff potential of a D-type soil if it was not. Complete descriptions of the Hydrologic Soil Groups can be found in USDA-NRCS National Engineering Handbook, Part 630 – Hydrology, Chapter 7.

Based on Figure 3.3, showing the HSG distribution in the Middle Raccoon River Watershed, the watershed consists primarily of B (66 percent) and B/D (27 percent) type soils, resulting in the majority of the area classified moderately well-draining. The portion of the watershed deemed B/D reflects a shallow groundwater table. A shallow groundwater table can result in increased runoff potential and greater reason to believe tile drainage practices are present to improve agricultural production. The Iowa Department of Natural Resources has also published Geospatial data, mapping locations where soils may require subsurface drainage to maximize agricultural productivity (Figure 3.4). Tile drain practices have further been confirmed in discussions with watershed stakeholders. The soils data from the USDA-NRCS Web Soil Survey (WSS) is available by county. The Counties of Sac, Carroll, Greene, Guthrie, and Dallas were downloaded, and then merged using ArcGIS tools.

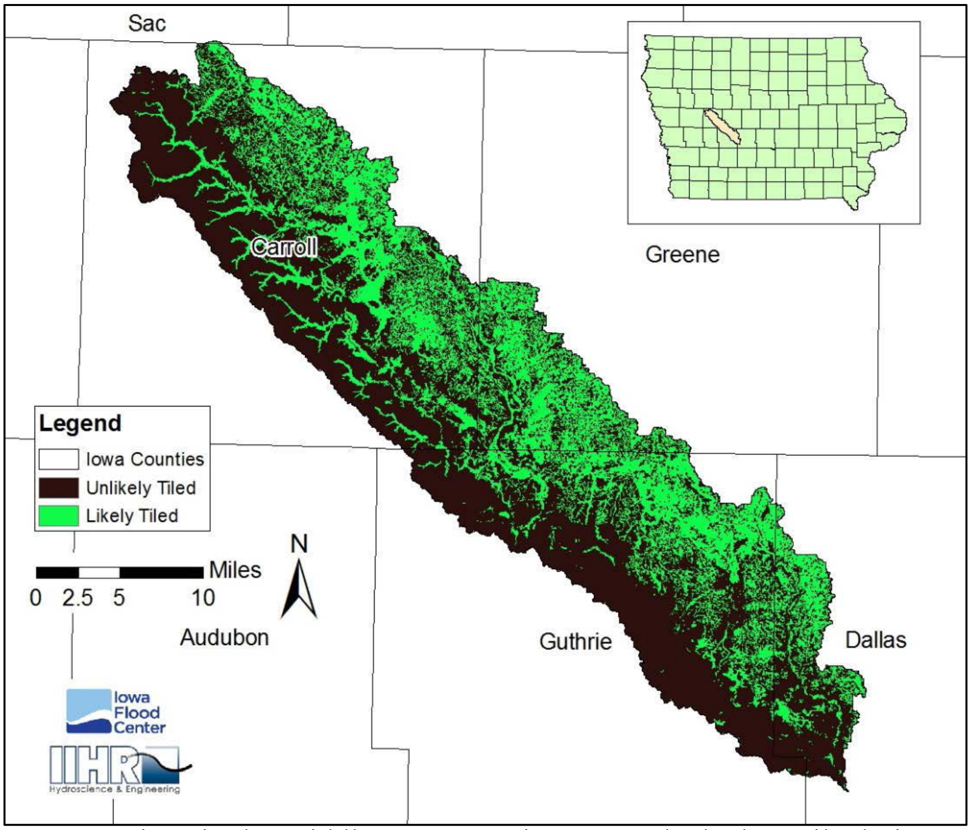

Figure 3.3: Locations in the Middle Raccoon River watershed where tile drainage is needed for maximum agricultural production, 85 percent.

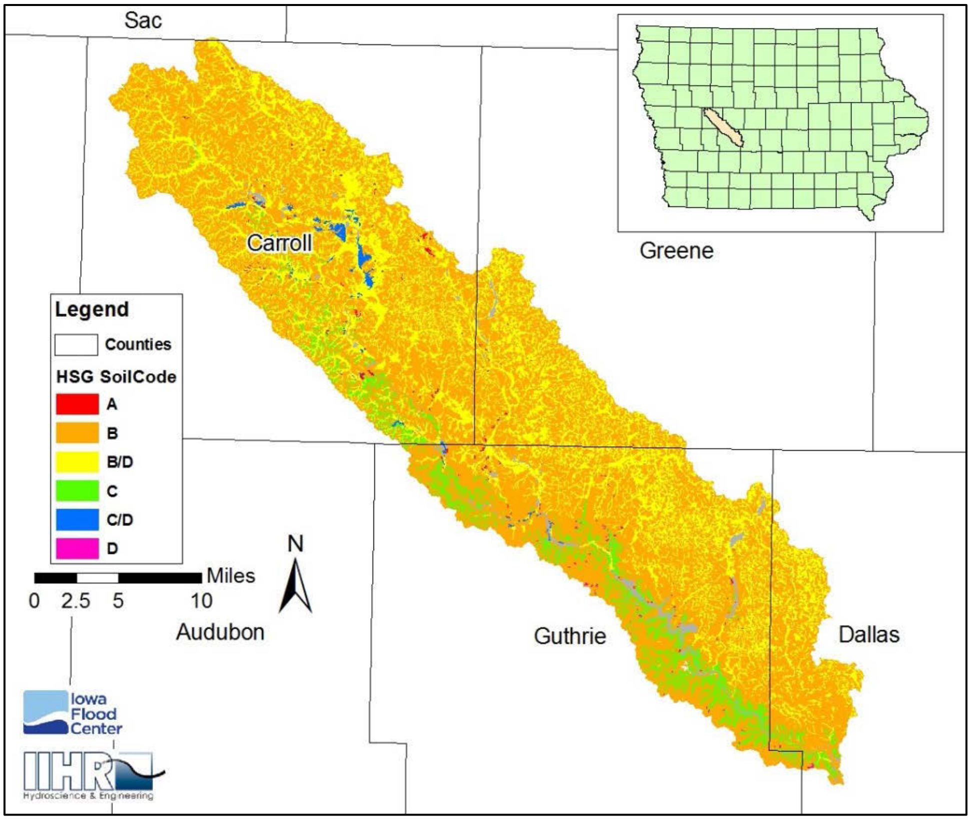

Figure 3.4: Soil Distribution of the Middle Raccoon River watershed. Hydrologic Soil Groups (HSG) reflect the degree of runoff potential for a particular soil, with Type A (Red) representing the lowest runoff potential and Type D (Purple) representing the highest runoff potential. The dominant soil type in the basin is HSG B (66 percent).

Table 3.1: Hydrologic Soil Group distribution (by percent area) in the Middle Raccoon River watershed.

Hydrologic Soil Group	Portion of Watershed (%)
A	0.4
A/D	0.0
B	66.4
B/D	27.1
C	5.3
C/D	0.5
D	0.2

Topography

The topography of the Middle Raccoon River watershed is relatively flat, particularly in the Des Moines Love region, and consists primarily of rolling hills and farm land. Elevations range from 1475 feet above sea level in the uppermost part of the watershed to 900 feet at its outlet (525 feet of relief). The terrain tends to be slightly steeper near the river channel and on the southern side of the Middle Raccoon River main stem, where the Southern Iowa Drift Plain Region is dominant. About 65 percent of the watershed has a slope of less than 5 percent and approximately 95 percent of the basin has a slope of less than 30 percent.

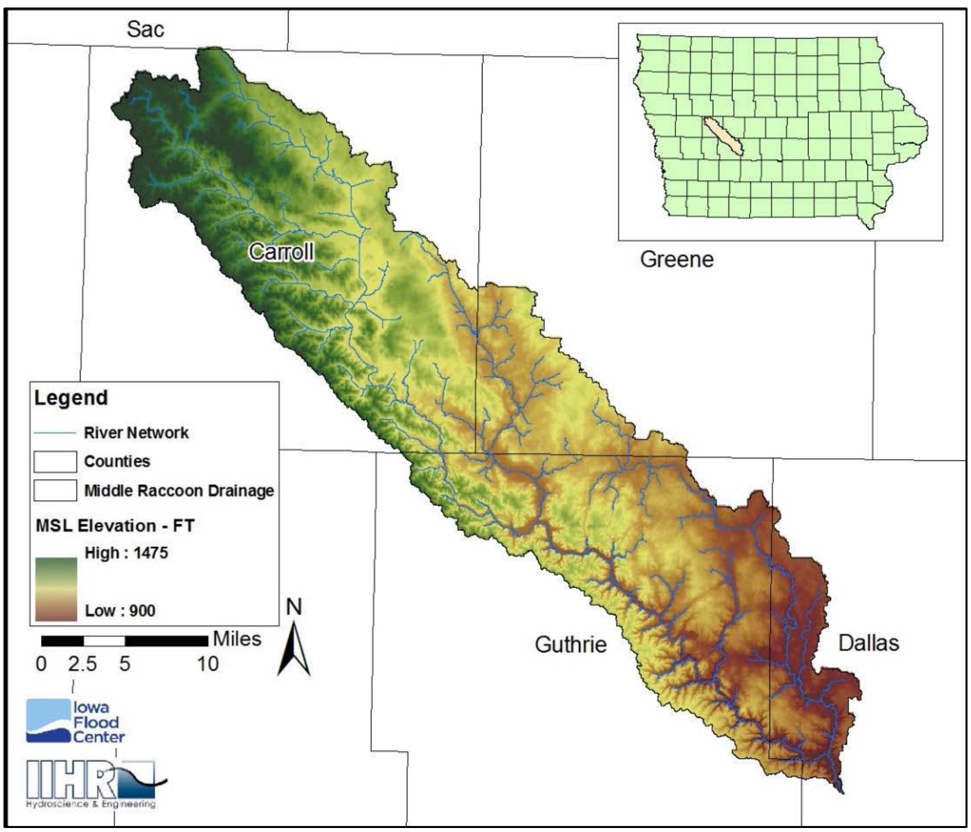

Figure 3.5: Topography of the Middle Raccoon River watershed. The Middle Raccoon is a relatively flat basin with elevations ranging from 900 feet to 1475 feet.

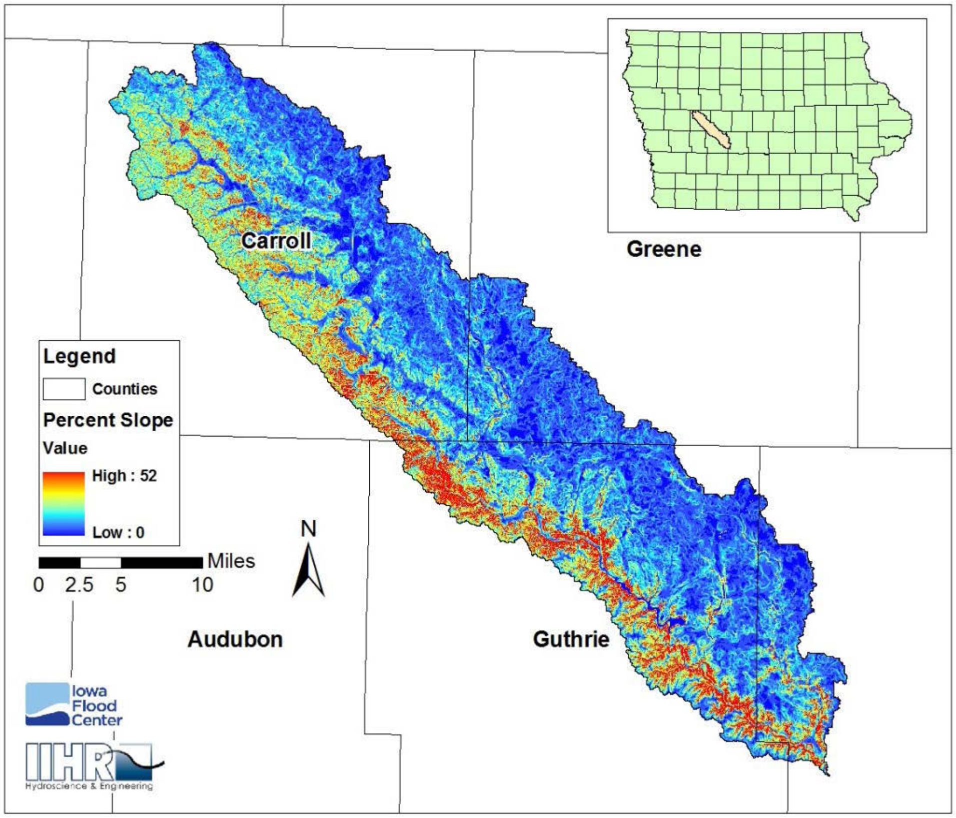

Figure 3.6: Slope of the Middle Raccoon River watershed. Slopes range from 0 to 52 percent.

Land Use

The Middle Raccoon River Watershed is predominantly agricultural, dominated by cultivated crops (corn/soybeans) at approximately 77 percent of the acreage, followed by pasture (9 percent), developed/commercial (7 percent), and forest (4 percent), per the 2006 National Land Cover Data (NLCD) Set. There are also several small towns located in the watershed—Carroll, Panora, Coon Rapids, Redfield, Lidderdale, and Bayard, among others.

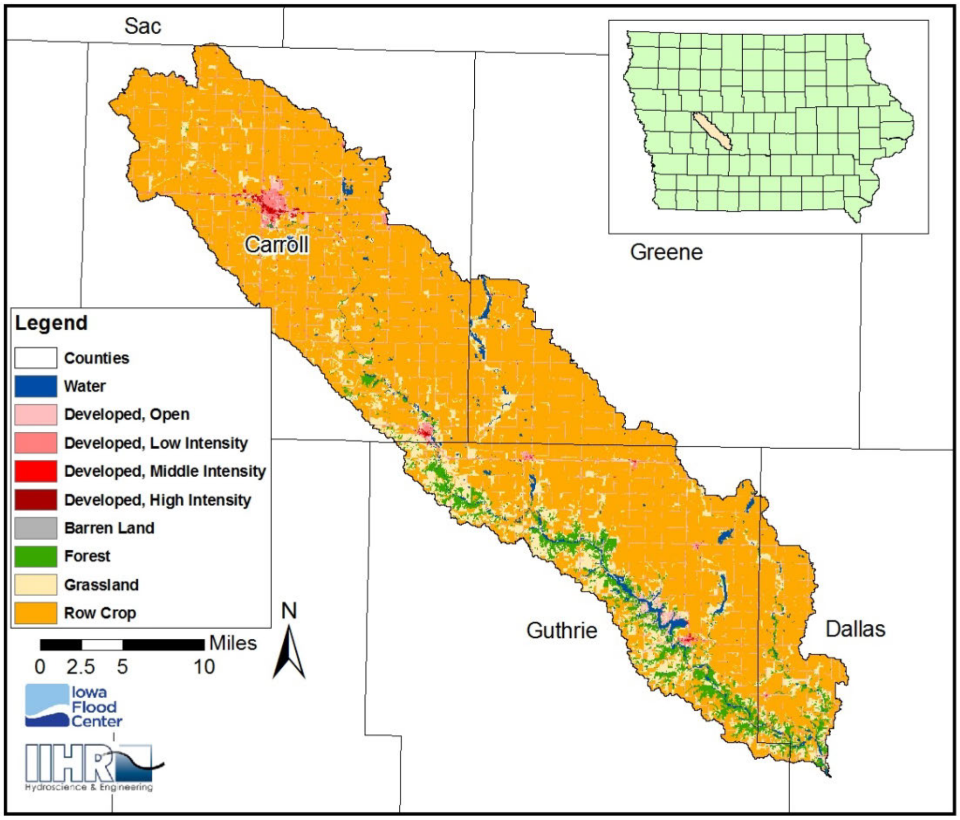

Figure 3.7: Land use compostion in the Middle Raccoon River watershed. Agricutural land use is dominant, and shown in orange.

Instrumentation and Data Records

The Middle Raccoon River watershed has instrumentation installed to collect and record stream stage, discharge, and precipitation measurements. There are two United States Geological Survey (USGS) owned stage and discharge gauges, one USGS owned stage-only gauge and three Iowa Flood Center (IFC) stream stage sensors located within the watershed. While the USGS gauges are owned by the USGS, they are maintained and operated by the Lake Panorama Association. There are four National Oceanic and Atmospheric Administration (NOAA) precipitation gauges within, or near, the watershed used for this study. Only rain gauges, with a period of record longer than 25 years, were considered. Table 3.2-3.3 and Figure 3.7 below detail the period of record and location of the hydrologic and meteorologic instrumentation.

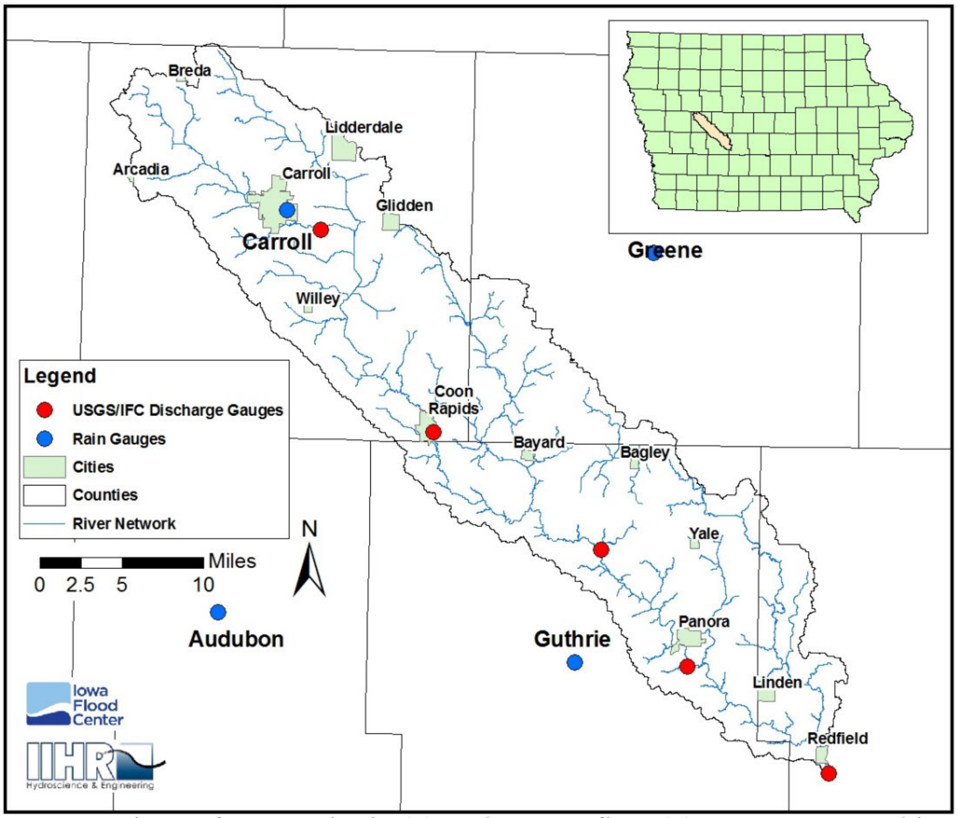

Figure 3.8: Locations of meterorlogic (4) and stream flow (5) gauges are used in the model development, calibration, and validation of the Middle Raccoon River watershed model. Meterorlogic gauges are shown in blue, stream flow in red.

Table 3.2: Periods of record for the hydrologic instumentation in the Middle Raccoon River watershed.

Gauge Type Stage/Discharge Gauges	Location	Period of Record
USGS 05483450 (stage,discharge)	Middle Raccoon near Bayard, IA	March 1979- present
USGS 0583470 (stage)	Middle Raccoon Lake Panorama	May 1979- present
USGS 05483600 (stage,discharge)	Middle Raccoon near Panora, IA	June 1958- present
IFC MDDLRCCN03 (stage)	Middle Raccoon near Carroll, IA	October 2013- present
IFC MDDLRCCN02 (stage)	Middle Raccoon near Coon Rapids, IA	October 2013-present
IFC MDDLRCCN01 (stage)	Middle Raccoon near Redfield, IA	October 2013- present

Table 3.3: Periods of record for meteorlogic instrumentation in, or near, the Middle Raccoon River watershed.

Gauge Type Meterorlogic Gauges	Location	Period of Record
GHCND: USC00130385	Audubon, IA	January 1883 - Present
GHCND: USC00131233	Carroll, IA	January 1883 - Present
GHCND: USC00136566	Jefferson, IA	January 1883 - Present
GHCND: USC00133509	Guthrie Center, IA	January 1895 - Present

Floods of Record

There have been several noteworthy floods in the watershed over the past 25 years, with perhaps the most well-known being the floods of 1993. The memorable flood during the summer of 1993 struck much of the upper Midwest, and resulted in a stage of 29.02, shattering the prior high water mark by over 4 feet. Rainfall data from the storm of July 8-9, 1993 show that nearly 11 inches of rain fell on the upper reaches of the Middle Raccoon River and the surrounding watersheds. (Prestegaard et. al, 1994).

In total, four floods, greater than 10,000 cubic feet per second, have been recorded at the USGS Middle Raccoon gauging station at Bayard, Iowa since 1973. These four flood peaks on June 03, 1973 – 14,600 cfs; June 30, 1986 – 12,300 cfs; July 09, 1993 – 27,500 cfs; and June 15, 2013 – 13,200 cfs are the four largest discharges observed during the continuous operation of this gauge. Flood details can be seen in Table 3.4.

The National Weather Service has not determined flood stages for the USGS gauges in the watershed. However, it has determined an action stage of 13 ft. This action stage was exceeded in all flood events tabulated above. The emergency spillway at Lake Panorama (El. 1048) has only been activated once since its construction, in the Flood of 1993.

Table 3.4: Floods of record on the Middle Raccoon River at Bayard, IA.

Date	Gauge Height/Stage (ft)	Peak Streamflow (cfs)
July 03, 1973	21.63	14,600
June 30, 1986	24.70	12,300
July 09, 1993	29.02	27,500
June 15, 2013	24.94	13,200

Chapter Summary

The purpose of this chapter is to describe the physical characteristics of the Middle Raccoon River watershed that were pertinent in the developments and analysis of the hydrologic model. Land use, soil type, topography, and available gauge data are all valuable inputs, or potential analysis tools, which will be discussed throughout the remainder of this report.

CHAPTER 4: HYDROLOGIC MODEL DEVELOPMENT

This chapter summarizes the development of the hydrologic model used in the Phase I Hydrologic Assessment for the Middle Raccoon River watershed. The modeling was performed using the United States Army Corps of Engineers (USACE) Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS), Version 3.5.

HEC-HMS is designed to simulate rainfall-runoff processes of a watershed. It is applicable in a wide range of geographic areas and for watersheds ranging in size from small (a few acres) to very large (1,000 acres or more).

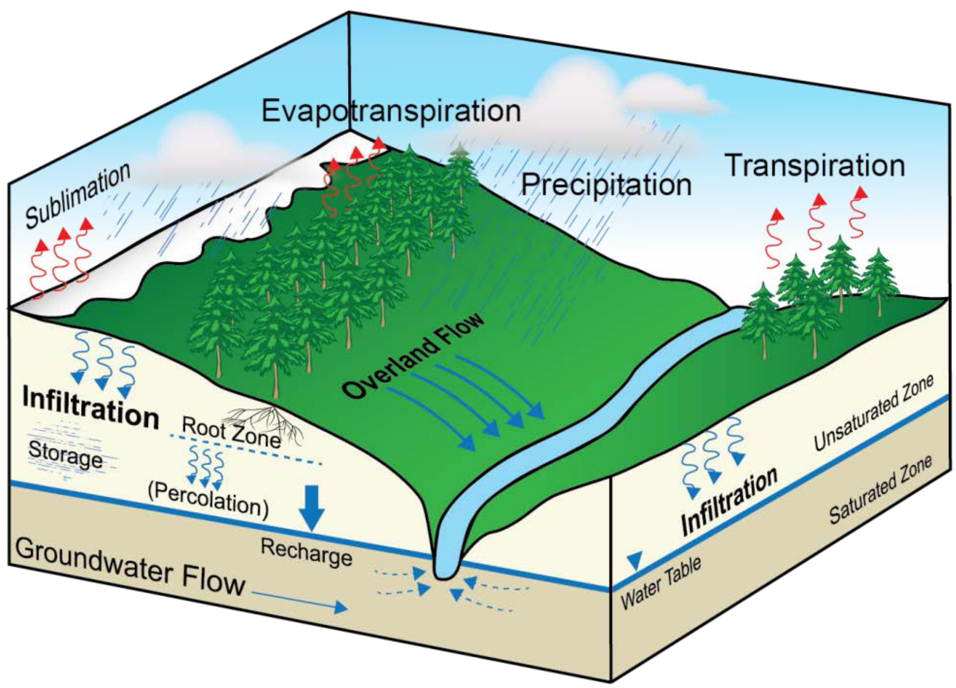

Figure 4.1: Hydrologic processes that occur in a watershed. HEC-HMS only considers precipitation, infiltration, and overland flow.

HMS is a mathematical, lumped parameter, uncoupled, surface water model. Each of these characteristics will be briefly discussed. The fact that HMS is a mathematical model implies that the different hydrologic processes are represented by mathematical expressions that were often empirically developed to best describe observations or controlled experiments. HMS is also a lumped parameter model, meaning physical characteristics of the watershed, such as land use and soil type, are "lumped" together into a single representative value for a given land area. Once these averaged values are established within HMS, the value remains constant throughout the simulation, rather than varying over time. HMS is an uncoupled model, meaning the different hydrologic processes are solved independently of one another, rather than jointly. In reality, surface and subsurface processes are dependent on one another, and their governing equations should be solved simultaneously (Scharffenberg and Fleming, 2010). Finally, HMS is a surface water model, meaning it works best for simulating (large) storm events or wet antecedent conditions where overland flow is expected to dominate the partitioning of rainfall. This thesis chose to use HEC-HMS over other modeling software packages due to it wide spread use throughout the hydrologic community, and its ability to model events on an event based, one minute time scale. Many other hydrologic models are developed for longer term simulations, and therefore have time steps in the magnitude of one day.

The two major components of the HMS hydrologic model are the basin model and the meteorologic model. The basin model defines the hydrologic connectivity of the watershed, how rainfall is converted to runoff, and how water is routed from one location to another. The meteorologic model stores the precipitation data that defines when, where, and how much rain occurs over the watershed. The model outputs simulated hydrographs at certain locations, which can then be compared to measured data, if available.

Model Development

The Middle Raccoon River watershed, as modeled and detailed herein, is approximately 590 square miles (mi^2). The watershed was divided into 349 smaller units, referred to as subbasins in HMS, with an average area of about 1.7 mi^2, but as large as 8.2 mi^2.

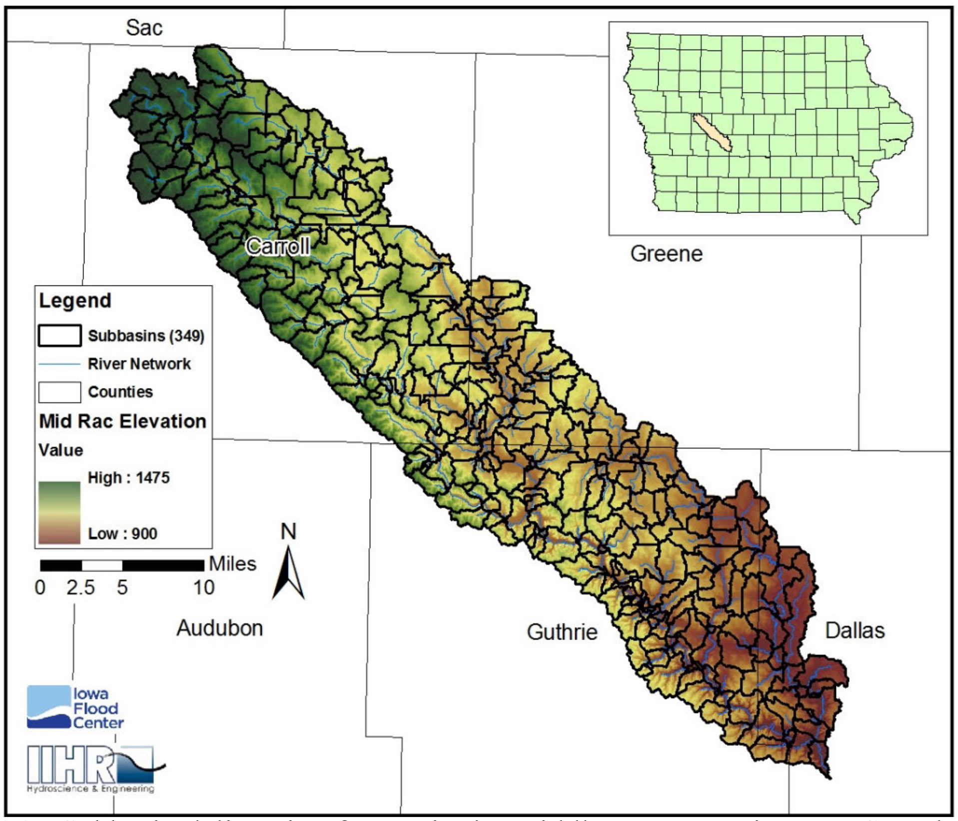

Figure 4.2: Subbasin delineation for use in the Middle Raccoon River HMS Hydrologic Model. Subbasins are smaller watershed units for which unique parameters can be assigned, such as soil type and land use. The Middle Raccoon Model has 349 subbasins with an average size of 1.7 mi^2.

ESRI/ArcGIS and Arc Hydro tools were used for terrain preprocessing, creating flow direction and flow accumulation grids, defining the stream network, and subbasin delineation. The stream network was defined to begin when the upstream drainage area was 4 square kilometers (1.16 mi^2), and subbasins were delineated such that a subbasin was defined upstream of all stream confluences. GIS-defined subbasins were further manually split to create an outlet point at each USGS gauge location, as well as the discharge point of two incorporated structures (Bays Branch Lake and Lake Panorama; refer to Chapter 5). In HMS, the averaging previously described for lumped parameter models is performed within the boundary of each subbasin and then each subbasin is assigned a single value for the parameter being developed.

Incorporated Structures

Two Reservoirs, Lake Panorama and Bays Branch Lake, were incorporated in the HMS model. Lake Panorama is located in Guthrie County, Northwest of Panora, IA. It drains approximately 440 mi^2, has a surface area of 1270 acres, and a normal storage of 19,700 acre-feet (Shive-Hattery, 1977). The Dam at Lake Panorama is controlled by a 100 foot long, 9.8 foot high, Bascule Gate (i.e. Hinge Crested Gate) and is operated by the Dam Supervisor. Since Lake Panorama is not intended for flood storage, the gate is designed to allow inflow to equal outflow, whenever possible. In order to mimic this gate operation in the model, the computed hydrograph at Bayard (the gauge directly upstream of Lake Panorama) was translated, via specified discharge, to the Bascule Gate location. Therefore, timing, peak flows, and total volumes from the Bayard Gauge location and the Bascule Gate location are identical. Since the supervisor uses the Bayard Gauge as one of the deciding factors in gate operation, this is a reasonable assumption and was confirmed by comparing the observed hydrograph at Bayard (upstream of Lake Panorama) to the observed hydrograph at Panora (downstream of Lake Panorama)—Figure 4.3.

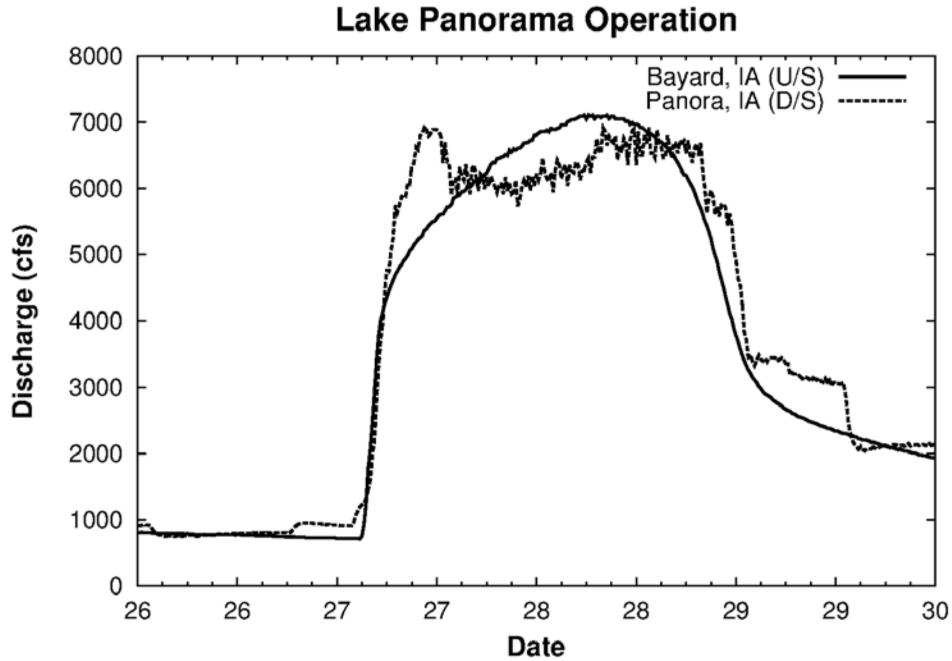

Figure 4.3: Observed hydrographs at both upstream of Lake Panorama (at Bayard,IA) and downstream of Lake Panorama (at Panora, IA) for the June, 2010 flood event. The figure shows how the upstream gauges are used to determine the discharge released from Lake Panorama.

Bays Branch is also located in Guthrie County, but Northeast of Panora, IA. It drains approximately 15 mi^2, has a surface area of 272 acres, and a normal storage of 1,088 acre-feet (Hall, 2006). The Bays Branch Dam was modeled using the storage, elevation, and discharge relationships that were obtained from the Iowa Department of Natural Resources—Bays Branch Dam Safety Inspection Report—and is available in Appendix B. No existing farm ponds or other possible water storage structures were included in the HMS baseline model.

Development of Model Inputs and Parameters

A brief overview of the data input used and the assumptions that have been made to develop the HMS model are provided in the following paragraphs. Later chapters of this report provide more detailed information on the hydrologic model development.

Rainfall

Stage IV radar rainfall estimates (NCEP/EMC 4KM Gridded Data {GRIB} Stage IV Data) were used as the precipitation input for simulation of actual rainfall events known to have occurred within the watershed. The Stage IV data set is produced by the National Center for Environmental Prediction (NCEP) by taking Stage III radar rainfall estimates, produced by the 12 National Weather Service (NWS) River Forecast Centers across the Continental United States, and combining them into a nationwide 4km x 4km (2.5mile x 2.5mile) gridded hourly precipitation estimate data set. Stage IV radar rainfall estimates are available from January 1, 2002 – present.

Figure 4.4, shows an example of Stage IV radar rainfall estimates of cumulative rainfall during the event of June 13-15, 2013 in the Middle Raccoon River watershed. This figure helps demonstrate the gridded nature of the radar rainfall estimate data, as well as the distributed nature of rainfall in time and space.

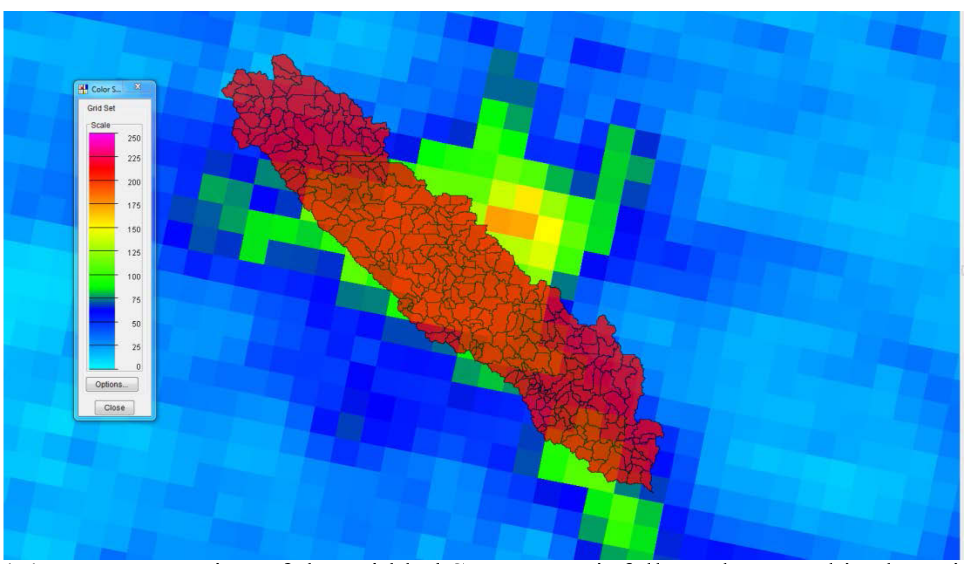

Figure 4.4: Demonstration of the gridded Stage IV rainfall product used in the Middle Raccoon River Watershed HMS model, June 13-15, 2013. On the map, yellow represents approximately 6 inches of rainfall.

Use of radar rainfall estimates provide increased accuracy of the spatial and time distribution of precipitation over the watershed, and Stage IV estimates provide a level of manual quality control (QC) performed by the NWS that incorporates available rain gauge measurements into the rainfall estimates. Actual storms using Stage IV data were the basis for model calibration and validation.

Hypothetical storms were developed for comparative analyses such as potential runoff generation, increased infiltration capacity through land use changes or soil improvements, and application of distributed storage within the watershed. These hypothetical storms apply a uniform depth of rainfall across the entire watershed with the same timing at each location. Soil Conservation Service (SCS) Type-II distribution, 24-hour storms were used for all hypothetical storms (Figure 4.5). Point Precipitation values (rainfall depths) for the 10, 25, 50, and 100-year average recurrence interval, 24 hour storms, were derived using the online version of National Oceanic and Atmospheric Administration (NOAA) Atlas 14 – Point Precipitation Frequency Estimates (Perica et at., 2013). Point estimates were obtained for several locations throughout the watershed; these estimates remained fairly consistent. Therefore, point estimates at Carroll, IA were used since this was also the location of the GHCND rainfall gauge used for estimating antecedent moisture conditions.

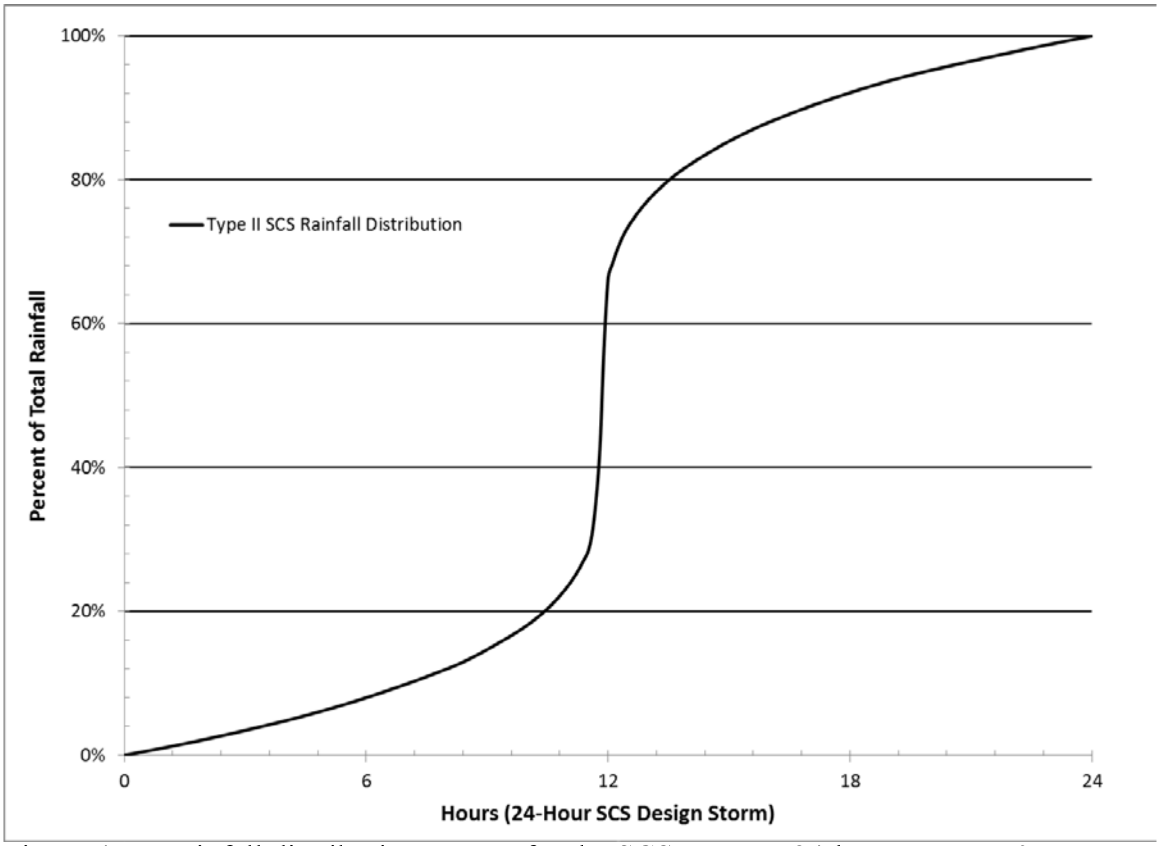

Figure 4.5: Rainfall distribution pattern for the SCS Type II, 24-hour storm. 50-percent of rain falls near the 12^{th} hour.

Studies have been performed on the spatial distribution characteristics of heavy rainstorms in the Midwestern United States (Huff and Angel, 1992). Point precipitation frequency estimates are generally only applicable for drainage areas up to 10 square miles, before the assumption of being uniformly distributed is no longer valid. Thus, for drainage areas between 10 and 400 square miles, relations have been established between point precipitation estimates and an areal mean precipitation approximation. Areal reduction factors, based on storm duration and drainage area, can be found in *Rainfall Frequency Atlas of the Midwest* (Huff and Angel, 1992). NOAA does not recommend adjusting point estimates to account for watershed size beyond 400 mi^2, as the dependence between the point and the areal values break down for watersheds larger than this.

For the comparative analyses that were performed in this modeling effort, an extrapolation was performed to get an areal reduction factor beyond 400 mi². It is understood that this depth of rainfall would not fall uniformly across a watershed this large; however, to have reasonable rainfall depth estimates, with a general relationship to the average recurrence interval 24-hour storms, the point rainfall estimates were reduced by a factor of 0.90 (the areal reduction factor for the 590 mi² drainage area at Redfield).

Table 4.1: Rainfall depths used in this modeling analysis for the 10-year through 100-year SCS design storms.

24 Hour Hypothetical Design Storm (years)	NOAA Point Precipitation (inches)	Areal Reduced Precipitation (inches)
10	4.48	4.03
25	5.64	5.08
50	6.67	6.00
100	7.82	7.04

Topography

Elevation data was obtained from the National Elevation Dataset (NED). The Digital Elevation Model (DEM) data was downloaded for the 5 counties in the watershed (DEM3MI05, DEM3MI14, DEM3MI25, DEM3MI37, DEM3MI39) at 3-meter resolution, covering the extent of the Middle Raccoon River watershed. They were clipped to the watershed extents using ESRI ArcGIS, then merged into a single seamless DEM. NED data was distributed in geographic coordinates in units of decimal degrees, in conformance with the North American Datum of 1983 (NAD 83). All elevation values are in meters and are referenced to the North American Vertical Datum of 1988 (NAVD 88).

Runoff Volume

Soil Conservation Service (SCS) Curve Number methodology was used to determine the rainfall-runoff partitioning for the Middle Raccoon River Watershed HMS modeling. Curve Number (CN) serves as a runoff index and values range from 30-100. As the CN becomes larger, there is less infiltration of water into the ground and a higher percentage of runoff occurs. CN values are an estimated parameter, based primarily on the intersection of a specific land use and the underlying soil type, not a measured parameter. General guidelines for curve numbers, based on land use and soil type, are available in technical references from the NRCS. The CN's assigned to each land use and soil type combination for the Middle Raccoon River HMS model are shown below.

Table 4.2: Curve Numbers assigned to each land use and soil type combination.

NLCD 2006	Description	Hydrologic Soil Group			
		A	B	C	D
11	Open Water	100	100	100	100
90	Woody wetlands	100	100	100	100
95	Emergent herbaceous wetlands	100	100	100	100
21	Developed, open space	49	69	79	84
22	Developed, low intensity	57	72	81	86
23	Developed, medium intensity	81	88	91	93
24	Developed, high intensity	89	92	94	95
31	Bare rock/sand/clay	98	98	98	98
41	Deciduous forest	32	58	72	79
42	Evergreen forest	32	58	72	79
43	Mixed forest	32	58	72	79
52	Shrub/scrub	32	58	72	79
71	Grassland/herbaceous	49	69	79	84
81	Pasture/hay	49	69	79	84
82	Row crops	67	78	85	89

Soils that had been assigned a dual soil code (A/D, B/D, and C/D) were reassigned to the undrained condition since tile drainage conditions were represented using the Clark Unit Hydrograph transform method. (See the *Runoff* section of this chapter.)

A CN grid was generated for the Middle Raccoon River watershed using ESRI ArcGIS with the HEC-GeoHMS extension tools. These tools intersect the 2006 National Land Cover Data Set with digital soils data (SSURGO) available from the USDA-NRCS Web Soil Survey (WSS). Upon completion of the CN Grid, HEC-GeoHMS tools were used to perform area-weighted averaging within each subbasin to assign a composite CN to each subbasin. Curve number grid figures are seen in Appendix A.

The NRCS curve number methodology for rainfall-runoff partitioning accounts for precipitation losses due to initial abstraction, which is the initial amount of rainfall that must fall before any runoff begins (losses due to plant interception, soil wetting, and storage in surface depressions), and the amount of precipitation that is estimated to infiltrate into the ground during the simulation. The remaining precipitation is considered excess precipitation and is converted to runoff. Evaporation and transpiration (evapotranspiration) were neglected in the modeling, as the focus is to simulate short duration, large rain events when evapotranspiration is thought to be a minimal component of the water balance. CN regeneration, in which the initial abstraction is reset after some time period, was not used since only short duration/ event-based storms were considered.

Antecedent Moisture Conditions

Rainfall-runoff partitioning for an area is also dependent on the antecedent soil moisture conditions (how wet the soil is) at the time rain falls on the land surface. In essence, the wetter the soil is, the less water is able to infiltrate into the ground and more rain is converted to runoff. Therefore, a methodology was needed to adjust subbasin CNs to reflect the initial soil moisture conditions at the beginning of a storm simulation in order to better predict direct runoff volumes.

To account for antecedent moisture conditions, a soil moisture proxy, known as the 5-day Antecedent Moisture Condition, (5-Day AMC) was used. Traditionally, 5-Day AMC is defined by the five day cumulative rainfall prior to the period of study; then based on the total amount of rainfall in those five days, it is broken into three levels—AMC I (dry), AMC II (normal/average), and AMC III (wet) (Table 3.3). AMC level two is calculated in ArcGIS, using the soil type and landuse grids. They are shifted upwards during wet soil conditions, resulting in higher runoff generation; the opposite effect occurs during dry soil conditions. AMC levels one and three are converted to a change in Curve Number, using the equations seen in Table 3.3. These three values statistically correspond to the 10 percent, 50 percent, and 90 percent cumulative non-exceedance probabilities of runoff depth, respectively (Hjelmfelt, 1982).

Table 4.3: Traditional definition for antecedent moisture conditions. (Chow et al., 1988)

5-Day AMC Group	5-Day Cumulative Rainfall (inches)	Equation used to calculate CN, based on AMC
I	Less than 1.4	$CN(I) = \frac{4.2\ CN(II)}{10 - 0.058\ CN(II)}$
II	1.4 to 2.1	Computed using current soils and landuse data
III	More than 2.1	$CN(III) = \frac{23\ CN(II)}{10 + 0.13CN(II)}$

Rainfalls in the Middle Raccoon River watershed at the Carroll, IA rainfall gauge were analyzed to determine if rainfall volumes in the region fit the traditional definition of AMC I, II, and III. After classifying a 113 year record (1900-2013) into a series of 5-day antecedent moisture values at the NOAA GHCND Carroll, IA rainfall gauge, it was determined that AMC definitions for the Middle Raccoon would need to be modified to fit the hydrology seen in the watershed. In order for the moisture conditions to reflect the hydrology seen in the Middle Raccoon, new AMC I, II, and III values were calculated so that they reflected the 10 percent, 50 percent, and 90 percent cumulative nonexceedance probabilities of 5-day rainfall in the watershed. Between the new AMC conditions, I, II, & III moisture conditions were assumed to act linearly in order to better account for AMC states between the three points. In this way, a continuous relationship describing the change in Curve Number that should be applied, based on 5-Day AMC, was developed, as opposed to traditional NRCS methodology, which allows only three discrete possibilities for Curve Number manipulation (the AMC I, II, and III Curve

Numbers). Once it was determined what curve numbers were required for the optimal peak discharge correlation in the calibrated storms (independent of 5-Day AMC), the AMC II condition was shifted upwards 2.9-percent, and AMC I and III values were recalculated as per the equations in Chow et al., 1988. This physically represents a slightly higher volume of runoff for the same moisture condition in a typically defined watershed. Figure 3.5 shows the existing NRCS definition for antecedent moisture conditions, along with the changes described here, and carried out for model calibration and validation.

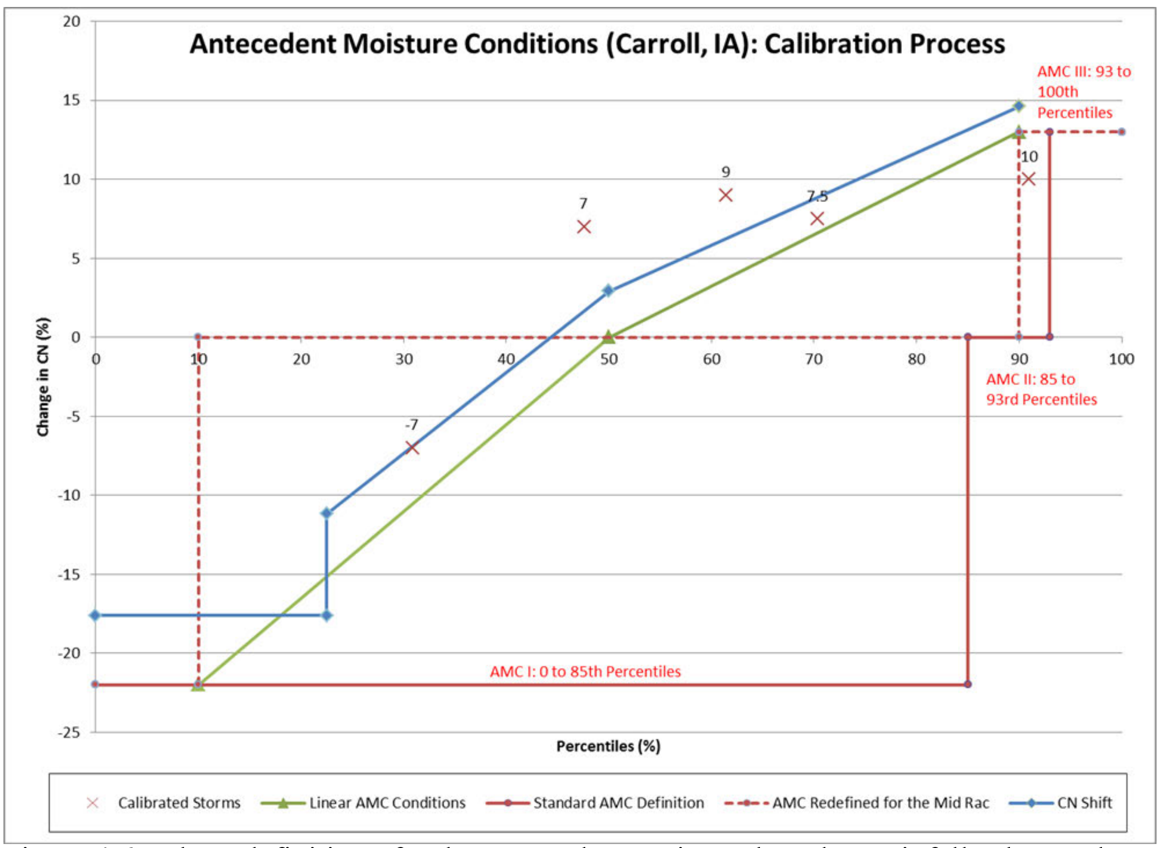

Figure 4.6: The redefinition of 5-day antecedent moisture based on rainfalls observed at Carroll, IA. The y-axis represents the change in Curve Number associated with AMC I, II, and III. The x-axis represents the 5-day rainfall probability of nonexceedance. Final AMC/CN values used in the HEC-HMS simulations were calculated using the blue line.

Runoff Hydrographs

The Clark and ModClark Unit Hydrograph methods were used to convert excess precipitation to a direct runoff hydrograph for each subbasin. The ModClark method requires the same grid used for radar rainfall, so this method was used for simulating historical storms used for calibration and validation, while the traditional Clark method was used for hypothetical design storm analysis. Both methods account for translation (delay) and attenuation (reduction) of the peak subbasin hydrograph discharge, due to the time it takes the excess precipitation to travel to the subbasin outlet and natural storage effects. The primary difference between the two methods is that the traditional Clark Unit Hydrograph method uses a pre-developed time-area histogram while the ModClark method uses a grid-based travel time model to account for translation (lag) of the subbasin hydrograph. Both methods route the translation unit hydrograph through a linear reservoir to account for temporary storage effects.

Both unit hydrograph methods require two inputs—time of concentration and a time storage coefficient. The time of concentration is the time required for water to travel from the hydraulically most remote point in the subbasin to the subbasin outlet. This was estimated as 5/3rd's lag time, where lag time is the time difference between the center of mass of the excess precipitation and the peak of the runoff hydrograph. A scaling coefficient of 5/3 is a reasonable approximation, according to SCS methodology (Woodward, 2010). Inputs required to determine the basin lag time for each subbasin include the subbasin slope, the length of the longest flowpath in the subbasin, and maximum potential retention (the maximum depth of water the soil can retain) in the subbasin, which is determined from the subbasin Curve Number. ESRI ArcGIS tools were used for terrain analysis to identify subbasin slopes and the longest flowpaths. While time of concentration is a measure of lag due to travel time effects, as water moves through the watershed, the time storage coefficient is a measure of lag due to natural storage effects in the subbasin (Kull and Feldman, 1998). Based on the literature, it can be estimated as a multiple of the time of concentration. Figures 4.7-4.8 illustrate the SCS methodologies for runoff volume estimation, as well as the Clark and ModClark methodologies for conversion of the excess precipitation into a runoff hydrograph.

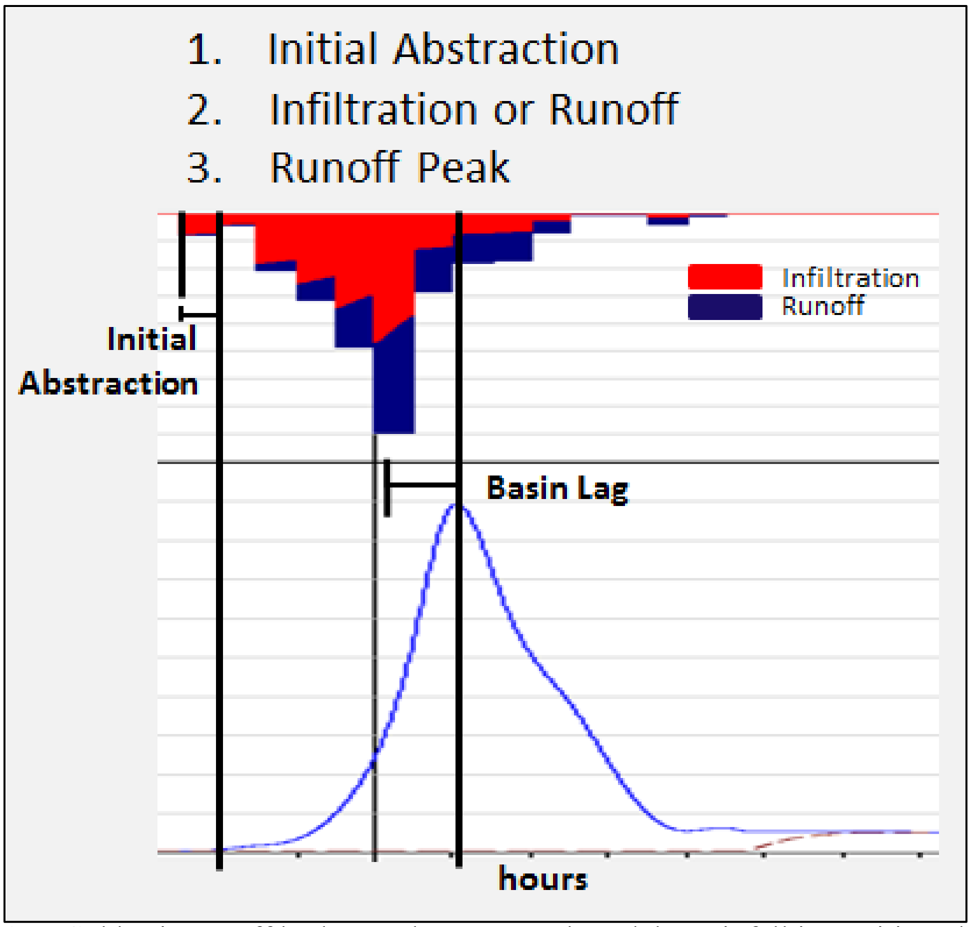

Figure 4.7: Subbasin runoff hydrograph conceptual model. Rainfall is partitioned into runoff depth (Curve Number Method). The runoff is then converted to discharge using the Clark Unit Hydrograph Method.

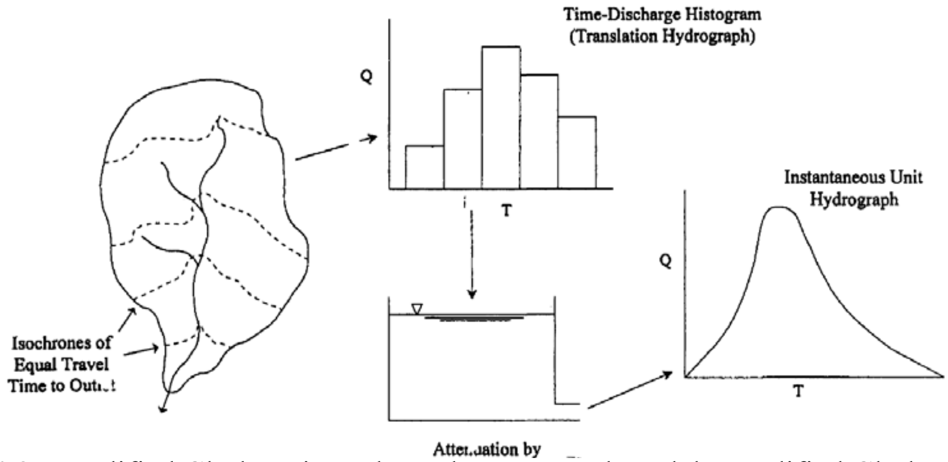

Figure 4.8: Modified Clark Unit Hydrograh conceptual model. Modified Clark uses grid cells rather than subbasins to generate runoff. (Kull and Feldman, 1998)

Modeling Tile Drainage

Since HEC-HMS is a precipitation-runoff-routing model only (HEC-HMS, 2000), it does not model subsurface processes. Under many circumstances, large scale event based models do not have a large subsurface component since the available storage in the soil is used up quickly and streamflow is almost entirely surface flow driven. However, after examination of the river response in the Middle Raccoon River, and after conversations with stakeholders in the watershed, it was determined that subsurface tile drains could play a significant role in streamflow, even during high precipitation, low frequency events.

Tile drains affect the hydrology by removing excess water and lowering the groundwater table from frequently inundated fields, through a network of underground pipes, usually made of clay or corrugated plastics (Sloan, 2013). This effect is demonstrated in Figure 4.9. In Iowa, it has been estimated that 25 to 35-percent of all cropland is artificially drained (Schilling, 2008). In the case of the Middle Raccoon River watershed, conversations with stakeholders have given the impression that nearly all land used in agricultural productions has been tile drained, or up to 77-percent of the watershed. This argument is strengthened upon analysis of Figure 3.3, where up to 85-percent of the watershed has been determined to need artificial drainage in order to maximize crop production.

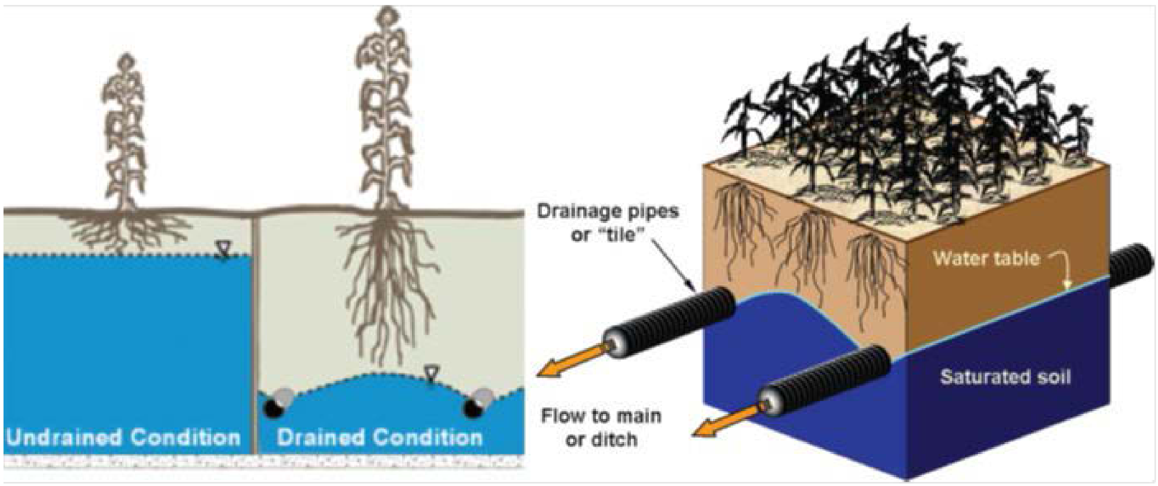

Figure 4.9: The use of tile drains in an agricultural setting acts to lower the watertable to below a crops root line. (Minnesota Extension Services)

For the purposes of the Middle Raccoon River hydrologic model, we focused on three potential effects of tile drainage on a flood hydrograph:

1.) Recession curves (after a storm event) from a tile drained watershed appear to be more linear than less tiled watersheds (Schilling, 2008)
2.) Where land already has been converted to agricultural production, the addition of subsurface drainage may reduce runoff and peak outflow rates (Blann et al., 2009)
3.) Relative to cropland drained by surface drainage alone, subsurface drainage can create increased temporary storage capacity in the upper layer of soil, allowing water to infiltrate and spread through the soil over a longer period (Blann et al., 2009)

In order to mimic these effects in the HEC-HMS model, the basin storage coefficient was manipulated so that the recession limb was more linear, the peak discharge was less, and the recession limb was drawn out. The final calibrated parameter increased the basin storage coefficient ratio from the recommended 0.65 (Kull and Feldman, 1998) to 0.92. (See equation 4.1 for the basin storage coefficient ratio.) The basin storage coefficient is a measure of a watershed's natural ability to store, hold and release water, for example, surface depressions or wetlands. By increasing this value, we allow each subwatershed to retain water longer and release it at slower rate. While this constant was not developed to account for subsurface drainage, it allows us to recreate its effect on the watershed hydrograph. Figure 4.10 shows how increasing the basin storage coefficient affects a single subbasin (W3990) in the Middle Raccoon River watershed.

$$Basin\ Storage\ Coefficent\ Ratio\colon\ 0.65 = \frac{R}{t_c + R}$$

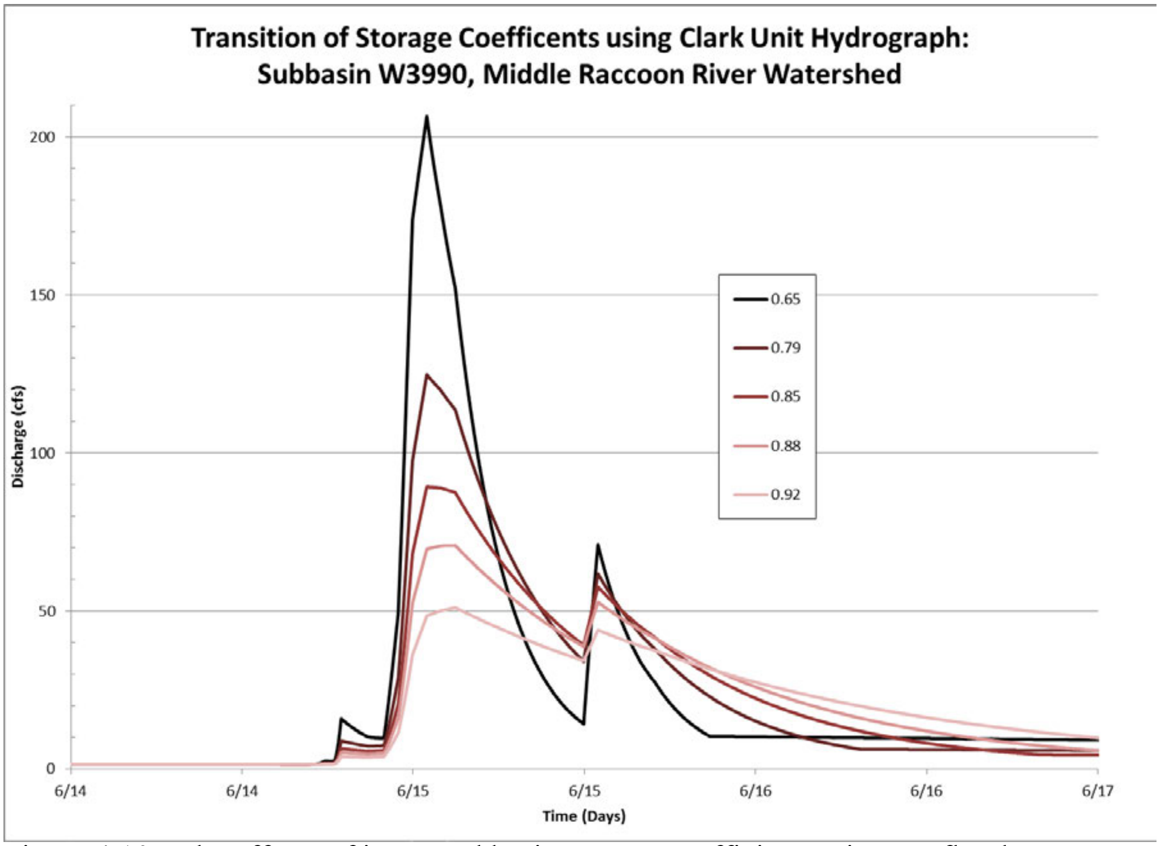

Figure 4.10: The effects of increased basin storage coefficient ratio on a flood hydrograph.

ArcGIS to HEC-HMS

Upon completion of GIS processing to prepare the basin topography data, establish the stream network, delineate the subbasins, and develop and assign the necessary parameters to describe the rainfall-runoff partitioning for each subbasin, HEC-GeoHMS tools were used to intersect the subbasins with the appropriate grid system (HRAP) to allow use of the Stage IV radar rainfall estimates. Lastly, from ArcGIS, HEC-GeoHMS tools were used to create a new HMS project and export all of the data developed in ArcGIS to the appropriate format, such that the model setup was mostly complete upon opening HMS for the first time. Once in the HEC-HMS user's interface, quality checks were performed to ensure that the connectivity of the subbasins and stream network of the watershed were imported correctly.

Baseflow

Baseflow was approximated by a first order exponential decay relationship for all historical storms. The USGS stage/discharge gauges for the Middle Raccoon River near Bayard, IA were used to develop discharge-drainage area (cubic feet per second/per square mile) relationships to set initial conditions for streamflow prior to each actual storm event simulation. These unique initial conditions were applied to the appropriate corresponding subbasins within the HMS interface for each actual storm event simulation. A baseflow recession constant, describing the rate of decay of baseflow per day, and a threshold indicating when baseflow should be reactivated, were also specified.

No baseflow was modeled for the hypothetical (design) storms, as theses analyses are more concerned with the effects of the amount of direct runoff produced. The contribution of baseflow during these design storm analyses is assumed to be relatively small compared to the amount of direct runoff produced.

Flood Wave Routing

Conveyance of runoff through the river network, or flood wave routing, was executed using the Muskingum routing method. Two inputs are required to use the Muskingum routing model in HMS – the flood wave travel time in a reach (K) and a weighting factor that describes storage within the reach as the flood wave passes through (X). The allowable range for the X parameter is 0-0.5, with values of 0.1-0.3 generally being applicable to natural streams. A value of 0.2 is frequently used in engineering practice and was used in this modeling analysis. Great accuracy in determining X may not be necessary because the results are relatively insensitive to the value of this parameter (Chow et. al, 1988). The flood wave travel time, K, is much more important and can be estimated by dividing the reach length by a reasonable travel velocity (1-5 feet per second, in general) as a starting point, but it is generally best obtained by adjustment in the model calibration process using measured discharge records, if available.

Flow routing through the Lake Panorama reservoir was executed using level pool routing. A level water surface is assumed and the methodology is derived from Conservation of Mass, similar to the Muskingum model. A specified discharge relationship was used, along with an initial condition; specified discharges were gathered from the flows seen at the Bayard, IA USGS stream gauge location. Therefore, the discharges at the Bayard, IA USGS Gauge location and at the Lake Panorama Bascule Gate are identical in both timing and flow. This represents how the Dam Supervisor uses the Bayard, IA USGS stream gauge to determine flows approaching the dam, so he can release water accordingly.

Flow routing through Bays Branch Lake reservoir was also executed using level pool routing. A storage-outflow discharge relationship was used, along with an initial condition, from which HMS computes the outflow from the reservoir at each time step, based on the known inflow and change in storage. All reservoirs and ponds incorporated into the model were assumed filled to the normal pool level at the beginning of each simulation.

Calibration Process

Model calibration is a process of taking an initial set of parameters, developed for the hydrologic model through GIS and other means, and making adjustments to them so that simulated results, produced by the model, match as closely as possible to an observed time series, typically stream discharge at a gauging station. However, adjustments to parameters should not be made to great extremes simply to manipulate the end results to match the observed time series. If this is necessary, the model does not reasonably represent the watershed, and it is upon the modeler to change methods used within the model or find what parameter(s) might be needed to better represent the watershed's hydrologic response.

The Middle Raccoon River watershed was calibrated to five storms events which occurred in April 2007, June 2008, June 2010, May 2013, and June 2013. Storms were selected based on their magnitude, time of year they took place, and on the availability of Stage IV radar rainfall estimates and USGS discharge estimates. Stage IV rainfall data is not available before 2002; therefore, historic storms, such as the flood of 1993, could not be modeled. Large, high runoff storms, occurring between May and September, were selected so that the impacts of precipitation on frozen grounds, and late fall to early spring freeze-thaw effects were minimized. Global adjustments were made to the runoff (CN) and timing (river routing and unit hydrograph) parameters to best match the simulated response to the observations at each of the USGS discharge gauge locations.

Calibration Events

The April 2007 storm was characterized by a basin wide average rainfall depth of approximately 3.25 inches, an antecedent moisture condition in the 31^{st} percentile, and a peak discharge of 5,890 cfs at Bayard, IA. Dry conditions were present before the storm; rainfall the five previous days amounted to 0.1 inches at Carroll, IA. Curve Numbers in the HMS model were reduced to reflect these dry conditions (average CN -7.6 percent) and the model did a reasonable job simulating this particular storm as the simulated peak flow is only 7-percent underestimated, the timing of the peak flow is approximately two hours late, and the runoff volume is underestimated by 18 percent. Underestimation of runoff volume may be due to the inaccuracies in radar rainfall estimates, but the very dry conditions before the storm would suggest a greater initial abstraction would need to be overcome to produce runoff, and a lesser amount of rainfall would be converted to runoff.

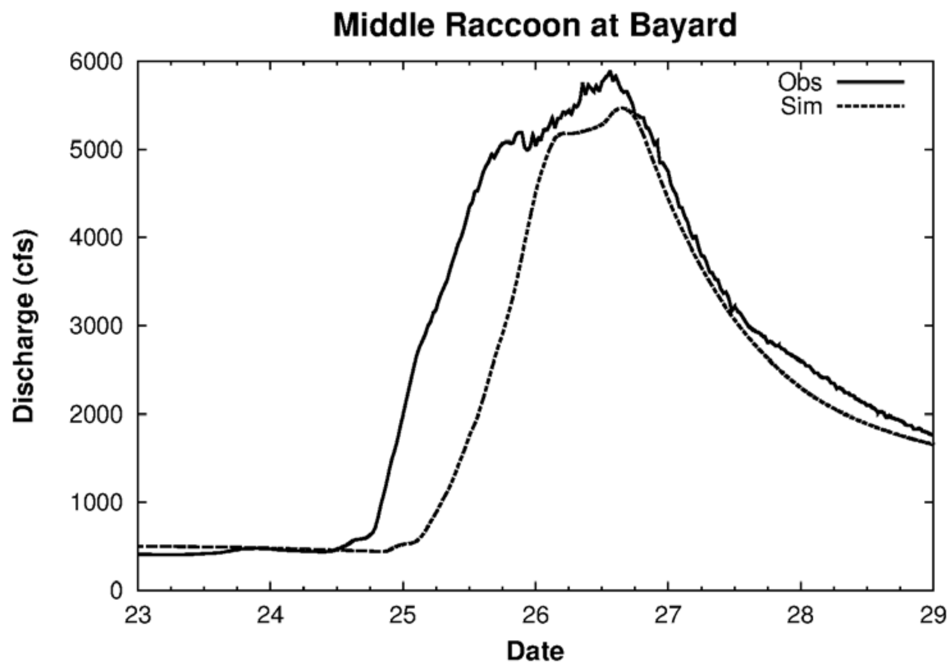

Figure 4.11: Observed and simulated hydrographs at Bayard, IA USGS stream gauge location. Run for the April, 2007 rainfall event with post calibration parameters.

The June, 2008 storm was characterized by a rainfall depth of approximately 3.1 inches, an antecedent moisture condition in the 91st percentile, and a peak discharge of 7,190 cfs at Bayard, IA. Very wet conditions were present before the storm; rainfall the five previous days amounted to 1.9 inches at Carroll, IA. CNs in the HMS model were increased to reflect these wet conditions (average CN +15.9 percent). The simulated peak flow was 22 percent overestimated, the timing of the peak flow was approximately ten minutes early, and the runoff volume was overestimated by 1 percent. The difference in peak flows may be due to the very wet antecedent moisture condition, which accounted for the greatest increase in curve number seen in any of the calibrated events. The model tends to be more accurate as antecedent moisture conditions move closer to the average.

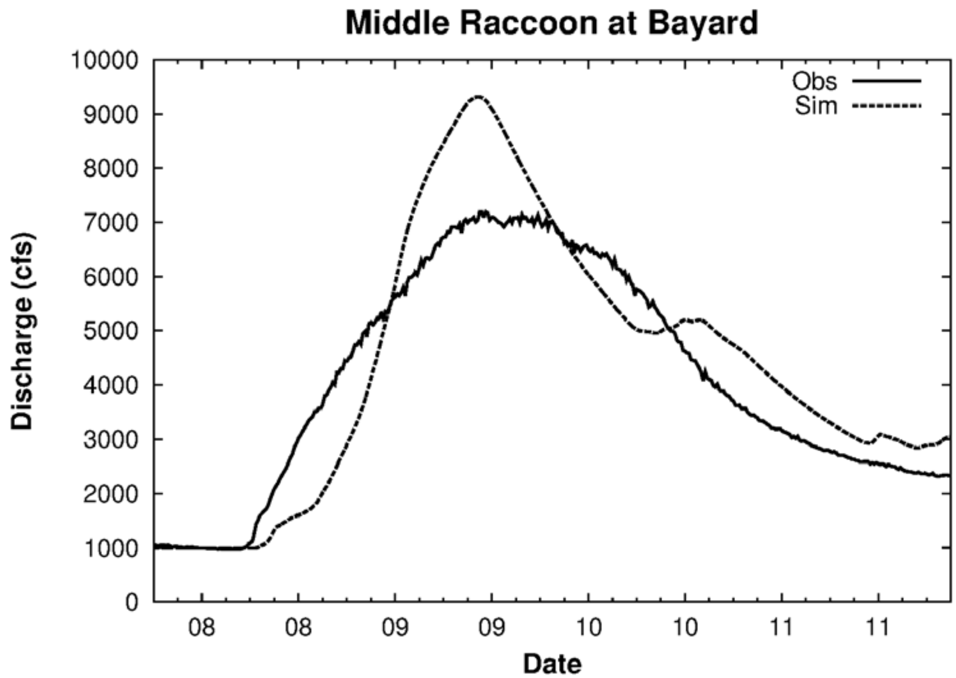

Figure 4.12: Observed and simulated hydrographs at Bayard, IA USGS stream gauge location. Run for the June, 2008 rainfall event with post calibration parameters.

The June, 2010 storm was characterized by a rainfall depth of approximately 2.7 inches, an antecedent moisture condition in the 61st percentile, and a peak discharge of 7,100 cfs at Bayard, IA. Wet conditions were present before the storm; rainfall the five previous days amounted to 0.6 inches at Carroll, IA. CNs in the HMS model were increased to reflect these wet conditions (average CN +6.6 percent). The simulated peak flow was 16 percent underestimated, the timing of the peak flow was approximately three hours late, and the runoff volume was underestimated by 23 percent. Differences in the hydrographs could be due to the abnormally "flashy" response observed in this storm.

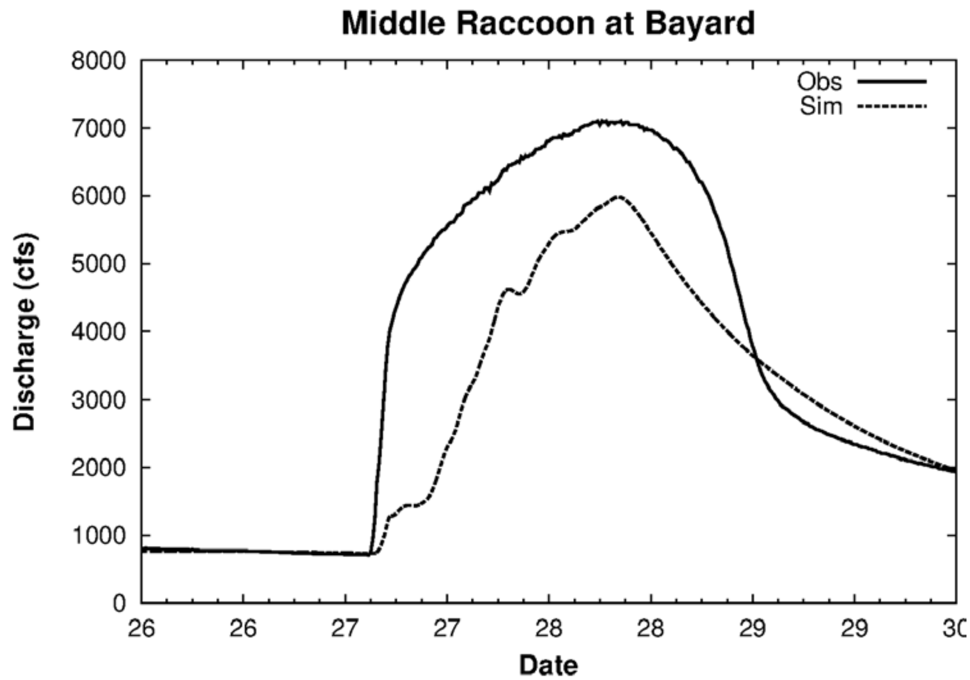

Figure 4.13: Observed and simulated hydrographs at Bayard, IA USGS stream gauge location. Run for the June, 2010 rainfall event with post calibration parameters.

The May 2013 storm was characterized by a rainfall depth of approximately 2.8 inches, an antecedent moisture condition in the 70th percentile, and a peak discharge of 8,030 cfs at Bayard, IA. Wet conditions were present before the storm; rainfall the five previous days amounted to 0.8 inches at Carroll, IA. CNs in the HMS model were increased to reflect these wet conditions (average CN +9.5 percent). The simulated peak flow was 6 percent overestimated, the timing of the peak flow was approximately four hours early, and the runoff volume was overestimated by 23 percent. The calibrated parameters seemed to do a reasonable job in reflecting the hydrologic response to this storm.

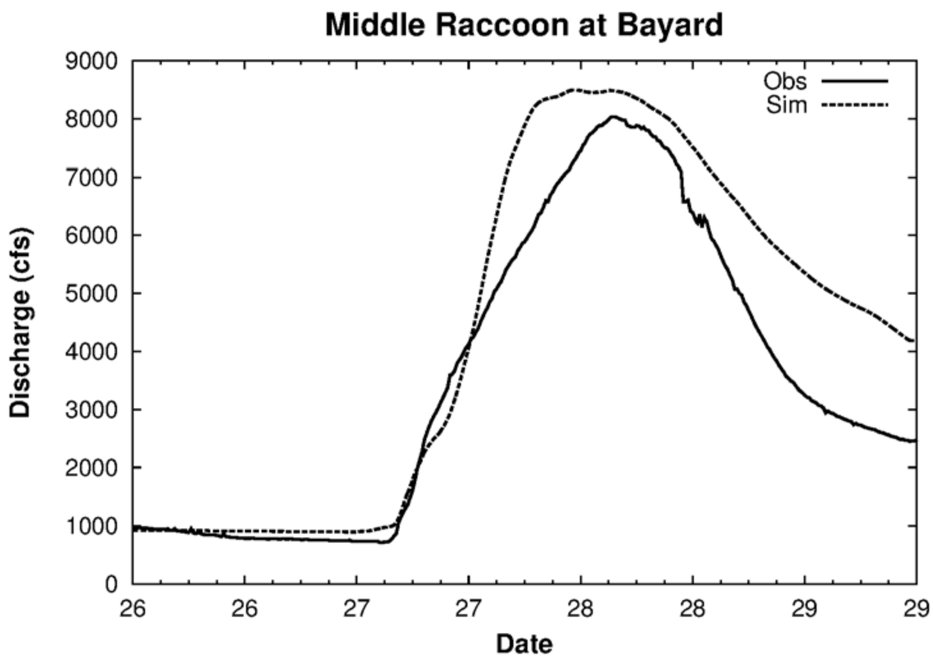

Figure 4.14: Observed and simulated hydrographs at Bayard, IA USGS stream gauge location. Run for the May, 2013 rainfall event with post calibration parameters.

The June 2013 storm was characterized by a basin wide average rainfall depth of approximately 2.5 inches, an antecedent moisture condition in the 48^{th} percentile, and a peak discharge of 13,200 cfs at Bayard, IA. Average conditions were present before the storm; rainfall the five previous days amounted to 0.3 inches at Carroll, IA. CNs in the HMS model were slightly increased to reflect these conditions (average CN +1.6 percent). The simulated peak flow was 17 percent underestimated, the timing of the peak flow was within five minutes of the observed flow, the runoff volume was overestimated by 8 percent. The calibrated parameters seemed to do a reasonable job in reflecting the hydrologic response to this storm, but may be underestimated due to its close proximity to the May, 2013 storm. The large event in May, two weeks earlier, would indicate very wet conditions. However, since the model only knows the 5-day moisture conditions prior to the event, the AMC may be underestimated.

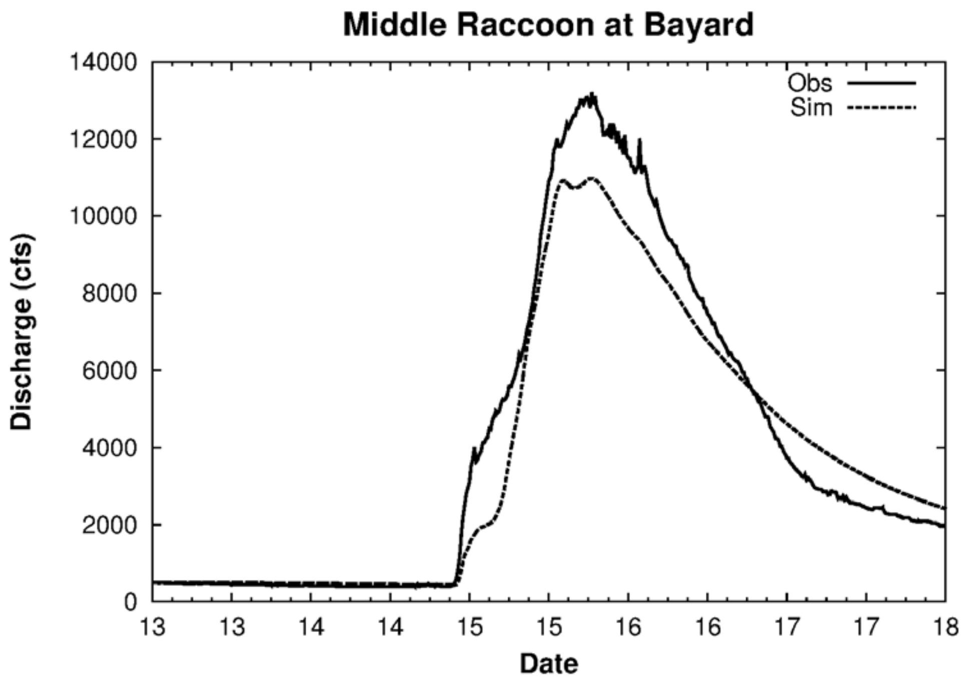

Figure 4.15: Observed and simulated hydrographs at Bayard, IA USGS stream gauge location. Run for the June, 2013 rainfall event with post calibration parameters.

The model was not calibrated to fit one storm perfectly. Instead, parameters were altered in an attempt to reflect a variety of historic rainfall events that varied in intensity, seasonality, and antecedent moisture conditions. The efforts of this multi-storm approach to calibration and validation can be seen in Figure 4.14. While none of the peak flows calibrated matched the peak flow observed at the Bayard, IA USGS gauge location exactly, they each did a reasonable job of estimating flows within a realistic range for the magnitude of rainfall events simulated.

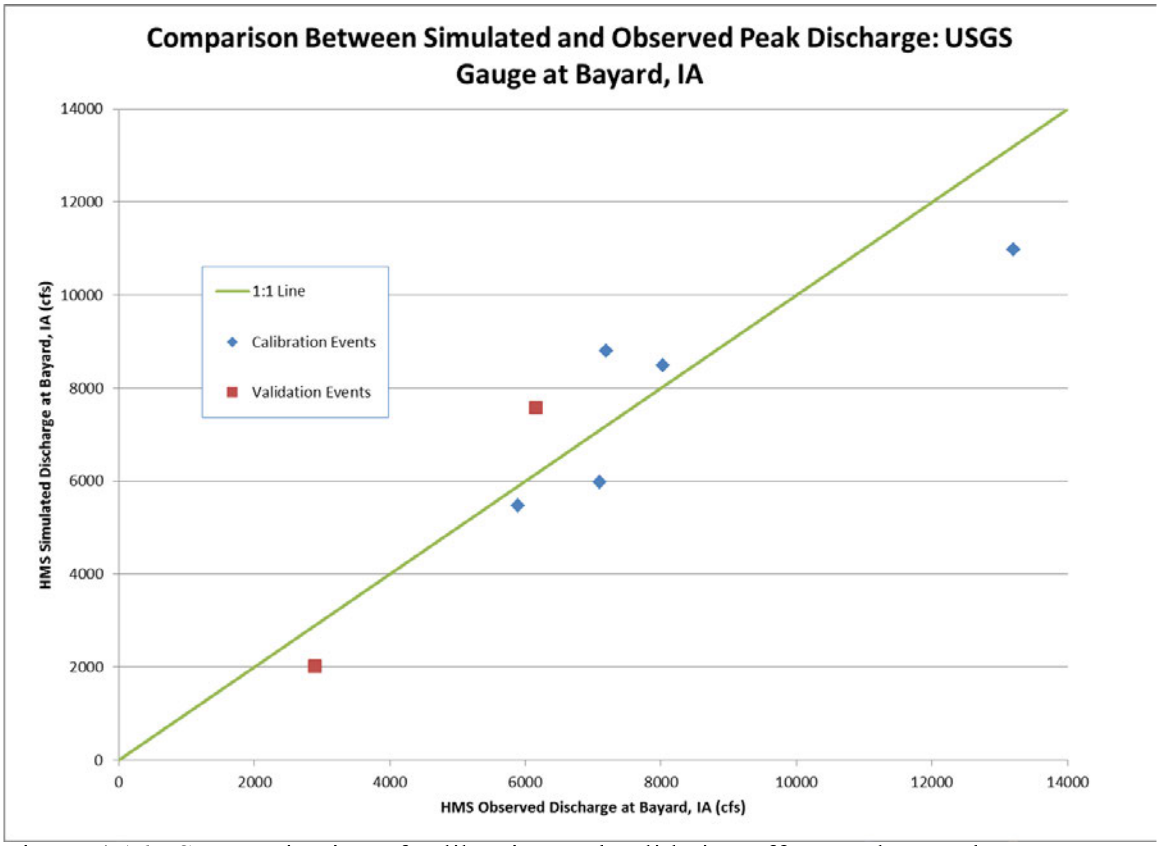

Figure 4.16: Summarization of calibration and validation efforts. The graph compares the observed discharges (x-axis) and simulated discharges (y-axis). A perfect correlation would be located directly on the 1:1 (green line).

<u>Validation Process</u>

For model validation, the intent was to use the model parameters, developed during calibration, to simulate other events and evaluate how well the model was able to replicate observed stream flows. With several of the largest storms already having been selected for calibration or having occurred before the availability of Stage IV radar rainfall estimates (January 2002), the next best available storms were selected. Two storms were considered for model validation—July, 2008 and August, 2010.

As with calibration, the HMS model validation results are not perfect. Differences may be due to the sizes of the storms considered. Relative to the calibration events, these were smaller storms in terms of total runoff produced. These smaller storms tend to have a greater subsurface flow component than larger storms since the ground is likely to have a greater capacity to infiltrate water, depending on antecedent moisture conditions. Because HMS is a surface water model, it struggles to simulate these types of conditions where surface flow is not the dominant partitioning of rainfall. Secondly, the storms occurred in, or near, the peak of the growing season, when precipitation losses, due to evapotranspiration and plant root uptake, are at a maximum. This is reflected in the observations, as most of the storms produced a small amount of runoff, despite a substantial amount of rain, even with some storms having wetter than normal antecedent conditions. Lack of accounting for evapotranspiration losses in the HMS model may also contribute to runoff discrepancies.

Despite these differences, the HMS model did an acceptable simulation of the July, 2008 flood (the larger of the two validation storms) that produced a discharge of 6,150 cfs at Bayard, IA, providing reassurance that the existing HMS model can reasonably simulate large runoff events where overland flow is expected to dominate the partitioning of rainfall. For the August, 2010 event, the model did an acceptable job simulating the peak flow of 2,890 cfs at Bayard, IA. However, timing of the peak was delayed nearly two days.

Validation Events

In validation, the model overestimated the wet antecedent moisture (79th percentile) July, 2008 event and underestimated the very dry (0-22nd percentile, i.e. 0 inches) August, 2010 event. Both of these events had lower observed peak discharges when compared with the calibrated events. Although a reasonable simulated response is sought for all storm sizes, greater precedence is placed on more accurately modeling large events since they typically pose a greater threat in terms of flooding.

The July 2008 validation storm was characterized by a rainfall of approximately 1.8 inches and a peak discharge of 6,150 cfs at the Bayard, IA USGS gauge location. Wetter than normal conditions were present before the storm—1.1 inches of rainfall in the 5 days prior, or the 79th percentile. The wet initial conditions increased the curve number and allowed more rainfall to be converted to runoff. This increase in curve number increased peak discharge to a level that better represented discharges. Despite these conditions, the simulation still overestimated peak discharge by 23 percent and volumes by 31 percent. Validation for this storm showed that the use of calibrated parameters better reflected the observed conditions when compared with uncalibrated parameters.

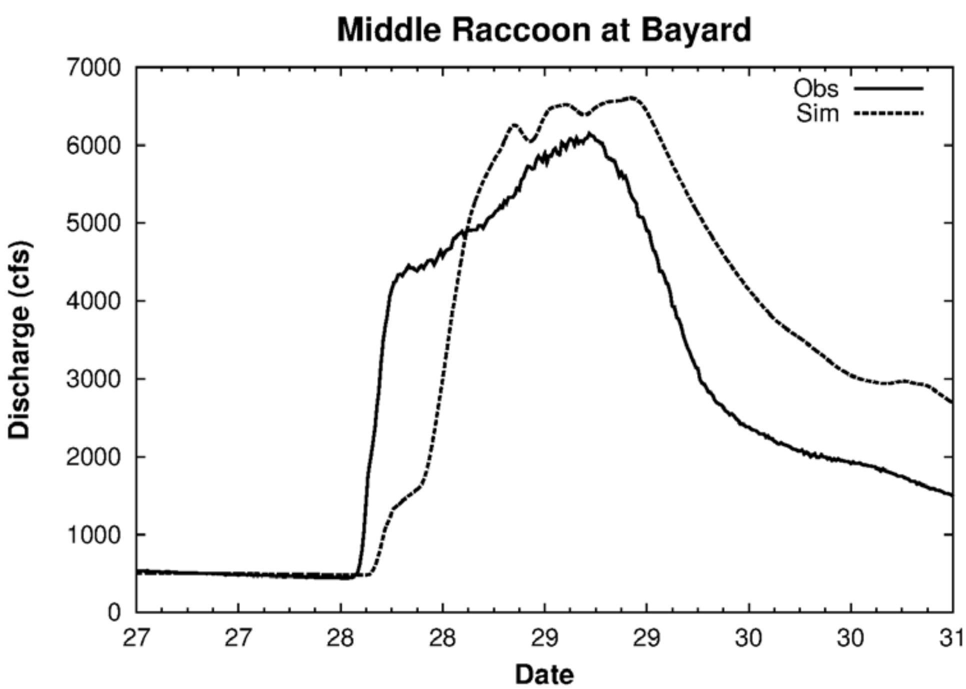

Figure 4.17: Observed and simulated hydrographs at Bayard, IA USGS stream gauge location. Validation using the July, 2008 rainfall event with post calibration parameters.

The August, 2010 validation storm was characterized by a peak discharge of 2,890 cfs at the Bayard, IA USGS gauge location, by far the smallest event simulated. Much dryer than normal conditions were present before the storm—0 inches of rainfall in the 5 days prior, or the 0-22nd percentile. The very dry initial conditions decreased the curve number and allowed less rainfall to be converted to runoff.   In this event, peak discharges were underestimated and the timing of the peak flow was late by approximately two day. The simulation still underestimated peak discharge by 30 percent and volumes by 11 percent. Difference in the observed and simulated hydrographs may be due to the smaller nature of this event; the model tends to more accurately predict large, high surface flow events. Despite these conditions, the model performed better using the calibrated parameters when compared with the uncalibrated parameters.

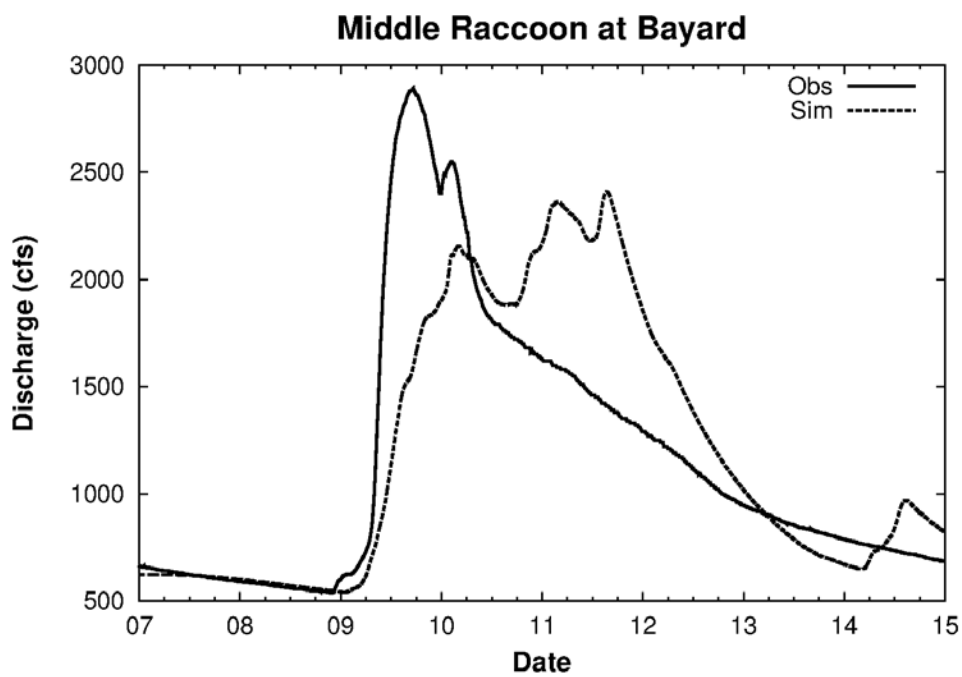

Figure 4.18: Observed and simulated hydrographs at Bayard, IA USGS stream gauge location. Validation using the August, 2010 rainfall event with post calibration parameters.

Chapter Summary

This chapter focused on the procedures and challenges faced when calibrating the Middle Raccoon River HEC-HMS model. It detailed the methods and parameters used for rainfall, runoff generation, antecedent moisture conditions, baseflow, and river routing. It also discussed the results of these parameters during the calibration and validation process. Special notes from this chapter include the redefinition of antecedent moisture, where a historical rainfall record at Carroll, IA was used to determine the rainfalls for AMC conditions I, II, and III, as well as the results from calibration and validation. The six total events modeled showed that the calibrated parameters did well predicting the magnitude of the storm event, even though no storm was ever calibrated perfectly. Pre and post calibrated parameters can be seen in Table 4.4.

Table 4.4: Pre and post calibration parameters used in HEC-HMS modeling.

Parameter	Initial Value	Calibrated Value
Curve Number	Calculated for Each Subbasin using soil and land use characteristics	2.94% Increase in curve number, values vary based upon initial moisture state
Antecedent Moisture	Three initial states I, II, III	Varies based on Figure 4.6
Time of Concentration	Calculated For Each Subbasin using basin lag/0.6	No Change
Storage Coefficient Ratio	0.65	0.92
Initial Abstraction	0.2 * Storage	No Change
Baseflow Recession Constant	0.9	No Change
Ratio to Peak	0.1	0.2
Muskingum Velocity	1 m/s	0.95 m/s

CHAPTER 5: LOCATIONS OF HIGH POTENTIAL RUNOFF

The HEC-HMS model of the Middle Raccoon River watershed was used to identify areas in the watershed with high runoff potential and to run simulations to help in understanding the potential impact of alternative flood mitigation strategies in the watershed.

In the HMS model of the Midgtodle Raccoon River watershed, the runoff potential for each subbasin is defined by the NRCS Curve Number (CN). The CN assigned to a subbasin depends on its land use and the underlying soils. The fraction of rainfall that is converted to runoff — also known as the runoff coefficient — is a convenient way to illustrate runoff potential. Areas with higher runoff coefficients have higher runoff potential. To evaluate the runoff coefficient, the runoff from each subbasin area is simulated with the HMS model for the same rainstorm; we chose a rainstorm with a total accumulation of 5.08 inches in 24 hours (i.e the 25-year, 24-hour SCS design storm). Only one storm was simulated for this analysis, since only total runoff will vary with increased rainfall, and not the high runoff locations. For this analysis, the model was run using the current watershed conditions; therefore, this simulation detects the current location of high runoff areas. Current watershed parameters were developed in ArcGIS and then imported into HEC-HMS for hydrologic simulations. Changes in land use, soil type, or land conservation practices all have an impact on runoff, so high runoff potential areas change over time.

Figure 5.1 shows the runoff coefficient as a percentage (from 0 percent for no runoff to 100 percent when all rainfall is converted to runoff). Since the subbasin areas shown were defined for numerical modeling purposes, the results were aggregated to more commonly used drainage areas — namely, hydrologic units defined by the U.S. Geological Survey (USGS). The smallest hydrologic units, known as HUC 12 watersheds, are shown in Figure 5.2. Area-weighted average runoff coefficients were determined for each of the 15 HUC 12 watersheds in the Middle Raccoon River basin. Areas in Iowa with the highest runoff potential are primarily located in the Des Moines Lobe portions of Carroll and Greene counties. Runoff coefficients exceed 50 percent in many areas. Although agricultural land use dominates the entire watershed, it does even more so in these two counties, which drives up the average Curve Number. From a hydrologic perspective, flood mitigation projects that can reduce runoff from these high runoff areas would be a priority.

Still, high runoff potential is but one factor in selecting locations for potential projects. Alone, it has limitations. For example, the two counties in Iowa with the highest runoff areas have very flat terrains; the average subbasin slopes are at, or below, the basin average. Flat terrain would make the siting of flood mitigation ponds more challenging. Indeed, there are many factors to consider in site selection. Landowner willingness to participate is essential. Also, existing conservation practices may be in place, or areas, such as timber, that should not be disturbed. Stakeholder knowledge of locations with repetitive loss of crops or road structures is also valuable in selecting locations.

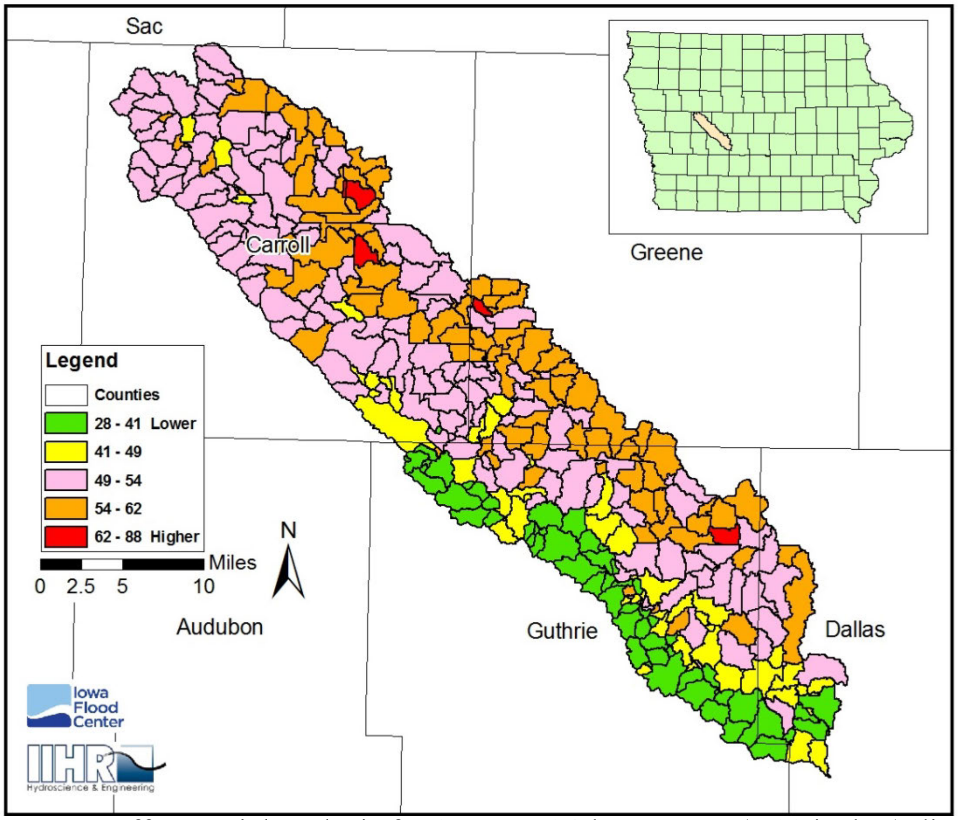

Figure 5.1: Runoff potential analysis for 25-year, 24-hour storm (5.08 inches) displayed by subbasin boundary. Higher potential runoff areas are labeled in orange and red.

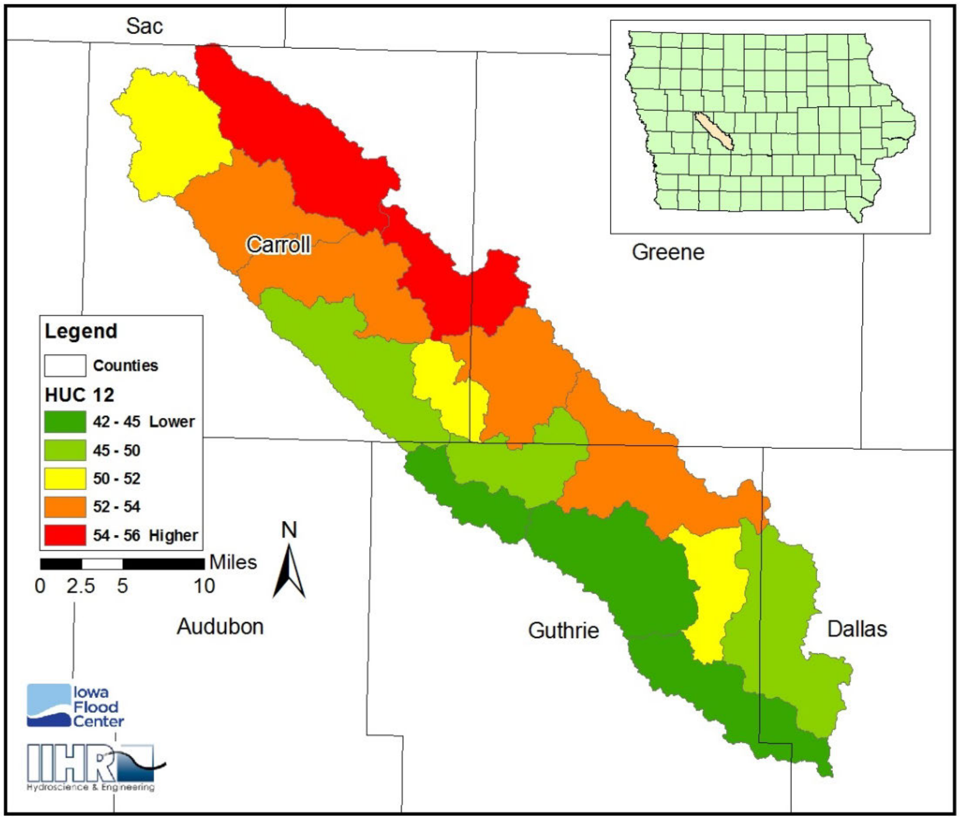

Figure 5.2: High runoff potential analysis for the 25-year, 24-hour storm (5.08 inches) aggregated to the HUC 12 boundaries. High potential runoffs are labeled in orange and red.

Summary of High Runoff Areas

For the analysis of high runoff potential areas, the model incorporated the current land use and soil type conditions in the watershed. These two combined parameters produce a composite curve number, which essentially estimates a region's runoff potential. Two separate region types were examined—the subbasin scale (approximately 1.7 mi²) and the HUC 12 scale (approximately 40 mi²). High curve numbers result in more rainfall converted to runoff. The highest curve numbers, and therefore, the highest runoff potential areas, are found in the northeast portion of the watershed, primarily in Carroll and Greene Counties. Flood mitigation projects built in these regions could have the greatest impact on peak discharge. However, economic factors, project feasibility, landowner participation, specific site conditions, and other factors all need to be considered before designing and siting a potential project.

CHAPTER 6: MITIGATING THE EFFECTS OF HIGH RUNOFF WITH INCREASED INFILTRATION

Reducing runoff from areas with high runoff potential may be accomplished by increasing the amount of rainfall that infiltrates into the ground. Changes that result in higher infiltration reduce the volume of water that drains off the landscape during, and immediately after, the storm. The extra water that soaks into the ground may later evaporate or it may slowly travel through the soil, either seeping deeper into the groundwater storage or traveling beneath the surface to a stream. Increasing infiltration has several benefits—if the infiltrated water reaches a stream, it arrives much later (long after the storm ends), and its late arrival keeps rivers running during long periods without rain. This late arrival also helps minimize the flashy response of the river.

In this section, we examine three alternatives for reducing runoff. The first is the conversion of row crop agriculture to forest. The second is the conversion of row crop agriculture back to native tall-grass prairie. The third is improving soil quality. All three are hypothetical examples; they are meant to illustrate the potential effects on flood reduction. In addition, the examples are not project proposals; they would be neither recommended nor practically feasible. Still, the hypothetical examples do provide valuable benchmarks on the limits of flood reduction that are physically possible with runoff reduction.

Land Use Change: Agriculture to Forest

An analysis was performed to quantify the impact of land use changes on the flood hydrology of the Middle Raccoon River watershed. In this first example, all current agricultural land use was converted to forest. This scenario was run first because forest land use has the highest infiltration capacity that the landscape could reasonably support. Obviously, moving to this condition is unlikely to occur, but this scenario is an important benchmark to compare with any watershed improvement project considered.

To simulate the conversion to forest with the HMS model, the model parameters affecting runoff potential across the landscape (Curve Number) were adjusted to reflect the forest condition. Specifically, existing agricultural land use, which accounts for 77 percent of the watershed area, was redefined as forest. New SCS Curve Numbers, reflecting the lower runoff potential of forest, were assigned to each subbasin. The basin average curve number was lowered from 82.2 to 65.7. It is important to note that other parameters estimated from Curve Numbers — such as the water flow travel time through the subbasin — were not adjusted. Thus, this scenario only considered the reduction in runoff volume resulting from the enhanced infiltration capacity of the forest; the attenuation and delay in the timing of the peak discharge that would also be expected, due to a much higher surface roughness and travel time, was not considered. Following the assignment of new subbasin Curve Numbers, the model was run for a set of design storms. Comparisons were made between current and forest simulations for the 10-, 25-, 50-, and 100-year return period 24-hour SCS design storms. Using design storms of different severity illustrates how flooding characteristics change during more intense rainstorms.

As expected, converting 77 percent of the watershed from row crop agriculture to forest had a significant effect on the flood hydrology. For the 10-year return period design storm (4.03 inches of rain in 24 hours), the simulated forest infiltrated 0.9 inches more into the ground than the current agricultural landscape. The additional infiltration increased to 1.1 inch for a 25-year storm, 1.3 inches for a 50-year storm, and 1.4 inches for a 100-year storm. As a result of increased infiltration across the landscape, the river response was dampened.

Figure 6.1 shows several locations in the watershed that were selected as points of reference in comparing flood flows for watershed improvement scenarios to current conditions. The two USGS stream gauges and the three IFC stage gauges in the watershed were selected as reference (index) points.

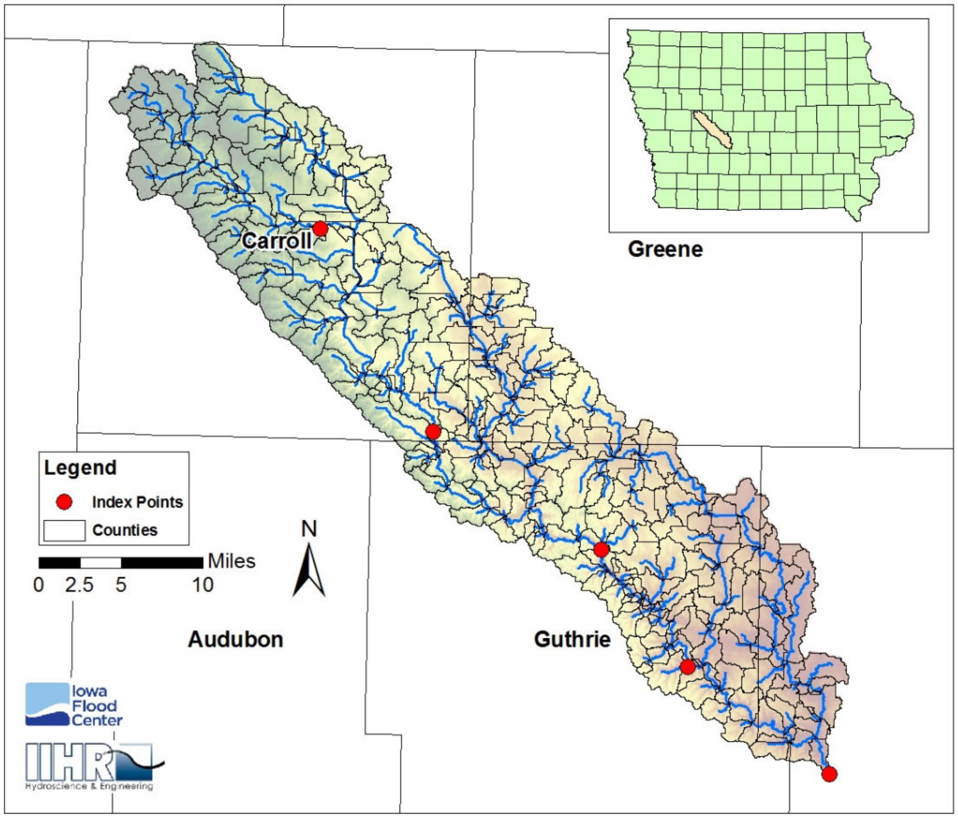

Figure 6.1: Index locations used for comparing watershed improvement scenarios to current conditions. The two USGS discharge gauges and the three IFC stage gauges served as points of reference to compare scenario results to exisitng conditions.

Figure 6.2 compares the simulated flood hydrographs for the current agricultural landscape (Baseline) to those for the forest landscape scenario for the 50-year return period, 24-hour design storm (6.00 inches of rain in 24 hours). For four locations shown — from an upstream subbasin area (Carroll, IA) to the outlet of the Middle Raccoon River at Redfield, IA — the river discharges and peak discharge rates are significantly less than that of a forest landscape. At Carroll, the smallest drainage area shown (73.8 mi^2), about 1.6 additional inches of rainfall would infiltrate, if this area were forest, resulting in a 44 percent reduction in its flood peak discharge. At downstream locations, the peak discharge reduction remains nearly uniform (between 40 and 42 percent), reflecting the relatively even distribution of agriculture throughout the watershed. Figure

6.3 summarizes the peak discharge for current conditions, the peak discharge for the hypothetical forest scenario, and the peak reduction effect, at all five index locations for the 50-year, 24-hour design storm event.

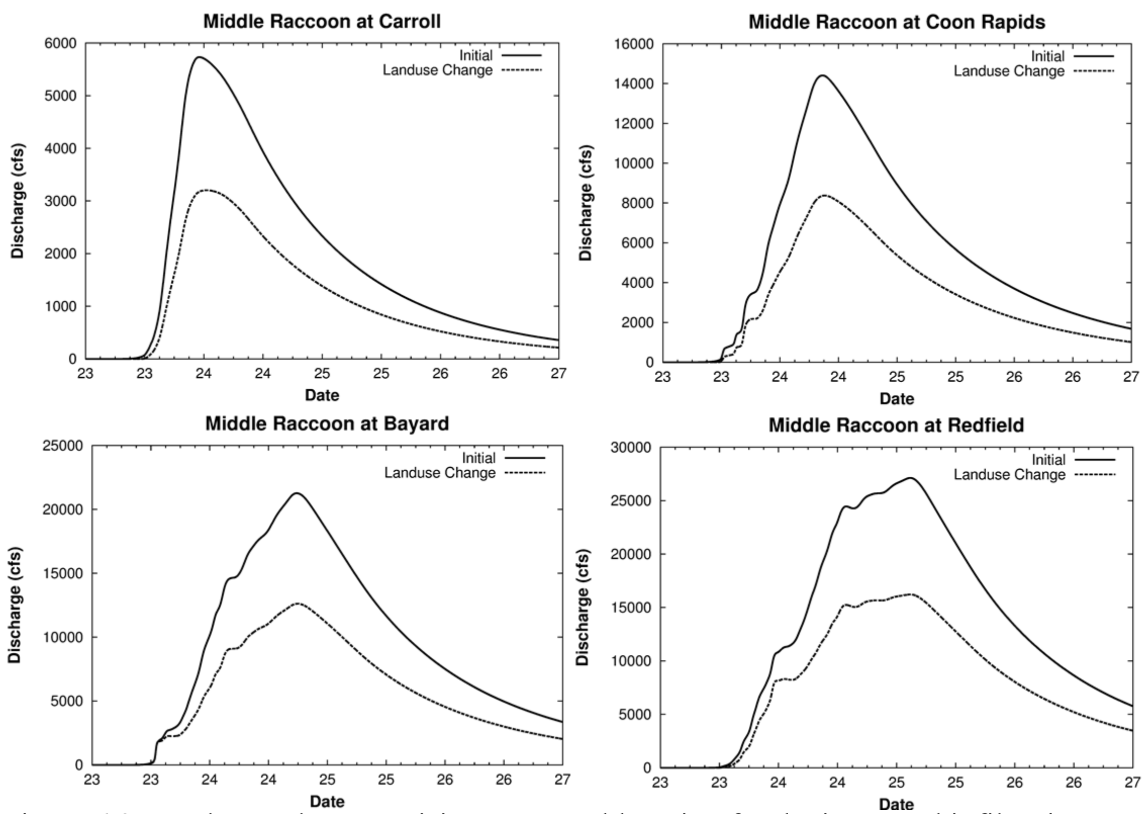

Figure 6.2: Hydrograph comparision at several location for the increased infiltration scenario resulting from hypothetical land use changes (conversion of row crop agriculture to forest). Results shown are for the 50-year, 24-hour storm (6.00 inches).

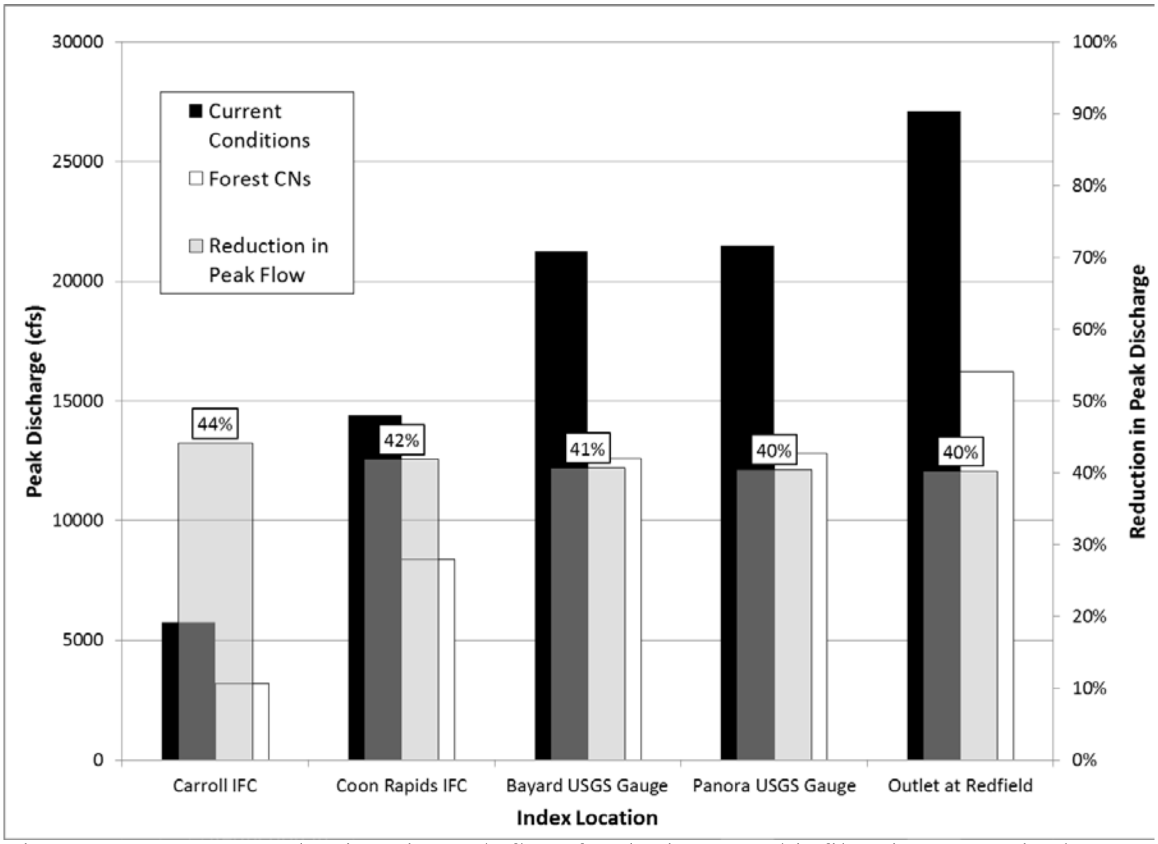

Figure 6.3: Percent reductions in peak flow for the increased infiltration scenario due to land use changes (conversion of row crop agriculture to forest). Peak flow reductions at five index locations progressing from upstream (left) to downstream (right) are shown for the 50-year, 24-hour design storms (6 inches).

Table 6.1 summarizes the percent reductions in peak discharge resulting from this hypothetical forest scenario at the five index locations for all the design storm events. The conversion of agriculture to forest resulted in peak discharge reductions of 36 to 56 percent. The peak reduction was largest for the smallest design storm (10-year return period), and decreased with larger rainfall amounts (up to the 100-year return period). In other words, the runoff reduction benefits of increased infiltration were greater for smaller rainfall events; still, for this forest scenario, there was a significant peak reduction benefit for large floods as well. Note also that the percent reduction in peak discharge was fairly uniform at all locations. Again, this outcome reflects the relatively equal distribution of agricultural land throughout the watershed.

Table 6.1: Percent reductions in peak discharge for agriculture to forest scenario.

Index Location	Percent Peak Discharge Reduction Based on Storm Return Period (%)			
	10-YR (4.03 in)	25-YR (5.08 in)	50-YR (6.00 in)	100-YR (7.04 in)
Carroll IFC Gauge	56.1	49.0	44.1	39.7
Coon Rapids IFC Gauge	53.9	46.7	41.9	37.5
Bayard USGS Gauge	52.6	45.5	40.7	36.4
Panora USGS Gauge	52.4	45.2	40.4	36.2
Redfield IFC Gauge (Outlet)	52.1	44.9	40.2	35.9

Reducing peak flood discharge also reduces the peak water height (or stage) in a river during the flood. During a flood, the river stage is higher than the channel itself, so water flows out of the channel and inundates the surrounding floodplain. Hence, even small reductions in flood stage can significantly reduce the inundation area. For the peak discharge reductions in the agriculture to forest scenario, the corresponding reduction in flood stage was between 2.6 and 4.6 feet. This reduction was estimated for the USGS stream-gauge locations, where the relationship between river stage and discharge — also known as a rating curve — has been measured. The rating curves developed by USGS can be seen in Appendix A.

Although a 2.6 to 4.6 foot reduction in flood stage would substantially reduce the flood inundation area, flooding still occurs in the forest simulation. For instance, based on the flood stage level reported by the National Weather Service at Bayard, water levels above action stage (13 feet) are expected for both the current agricultural and the forest landscapes for all rain events. Hence, conversion from agricultural to forest landscape does not eliminate flooding, but would reduce its severity and frequency.

Land Use Change: Agriculture to Native Tall-Grass Prairie

Much has been documented about the historical water cycle of the native tall-grass prairie of the Midwest. Some evidence suggests that the tall-grass prairie could handle up to six inches of rain without having significant runoff. The deep, loosely packed organic soils, and the deep root systems of the prairie plants, allowed a high volume of the rainfall to infiltrate into the ground. The water was retained by the soil instead of rapidly traveling to a nearby stream as surface flow. Once in the soil, much of the water was actually taken up by the root systems of the prairie grasses (Bradley, 2014).

Similar to the previous scenario, an analysis was performed to quantify the impact of human-induced land use changes on the flood hydrology of the Middle Raccoon River watershed. In this example, all current agricultural land use was converted to native tall-grass prairie with its much higher infiltration capacity. Obviously, returning to this pre-settlement condition is unlikely to occur. Still, this scenario is an important benchmark to compare current conditions to the most favorable hydrologic conditions historically seen in this area. Prior to 1830, Iowa's landscape was dominated by tall-grass prairies and broad –leaved flowering plants (Petersen, 2010).

To simulate the conversion to native tall-grass prairie with the HMS model, the model parameters affecting runoff potential across the landscape were adjusted to reflect the tall-grass prairie condition. Specifically, existing agricultural land use, which accounts for 77 percent of the watershed area, was redefined as tall-grass prairie. New SCS Curve Numbers, reflecting the lower runoff potential of prairie, were assigned to each subbasin. The basin average curve number was lowered from 82.2 to 69.6. Again, other parameters estimated from Curve Numbers — such as the water flow travel time through the subbasin — were not adjusted. Thus, this scenario only considered the reduction in runoff volume resulting from the enhanced infiltration capacity of the native prairie; the attenuation and delay in the timing of the peak discharge that would be expected as well, due to a much higher surface roughness and travel time, was not considered. Following assignment of new subbasin Curve Numbers, the model was run for a set of design storms. Comparisons were made between current and tall-grass prairie simulations for the 10-, 25-, 50-, and 100-year return period 24-hour SCS design storms.

As expected, converting 77 percent of the watershed from row crop agriculture to native tall-grass prairie had a significant effect on the flood hydrology. For the 10-year return period design storm (4.05 inches of rain in 24 hours), the simulated tall-grass prairie infiltrated 0.7 inches more into the ground than the current agricultural landscape. The additional infiltration increased to 0.9 inch for a 25-year storm, 1.0 inches for a 50-year storm, and 1.1 inches for a 100-year storm. As a result of increased infiltration across the landscape, the river response was dampened.

Figure 6.4 compares the simulated flood hydrographs for the current agricultural landscape (Baseline) to those for a native tall-grass prairie landscape scenario for the 50-year return period, 24-hour design storm (6.00 inches of rain in 24 hours). For all four locations shown — from an upstream subbasin area (Carroll, IA) to the outlet of the Middle Raccoon River at Redfield — the river discharges and peak discharge rates were significantly less for a tall-grass prairie landscape. At Carroll, the smallest drainage area shown (73.8 square miles), about 1.2 additional inches of rainfall would infiltrate if this area were tall-grass prairie, resulting in a 32 percent reduction in its flood peak discharge. At downstream locations, the peak discharge reduction remained fairly uniform (30 to 32 percent), reflecting the relatively even distribution of agriculture throughout the watershed. Figure 6.5 summarizes the peak discharge for current conditions, the peak discharge for the hypothetical tall-grass prairie scenario, and the peak reduction effect, at all five index locations for the 50-year, 24-hour design storm event.

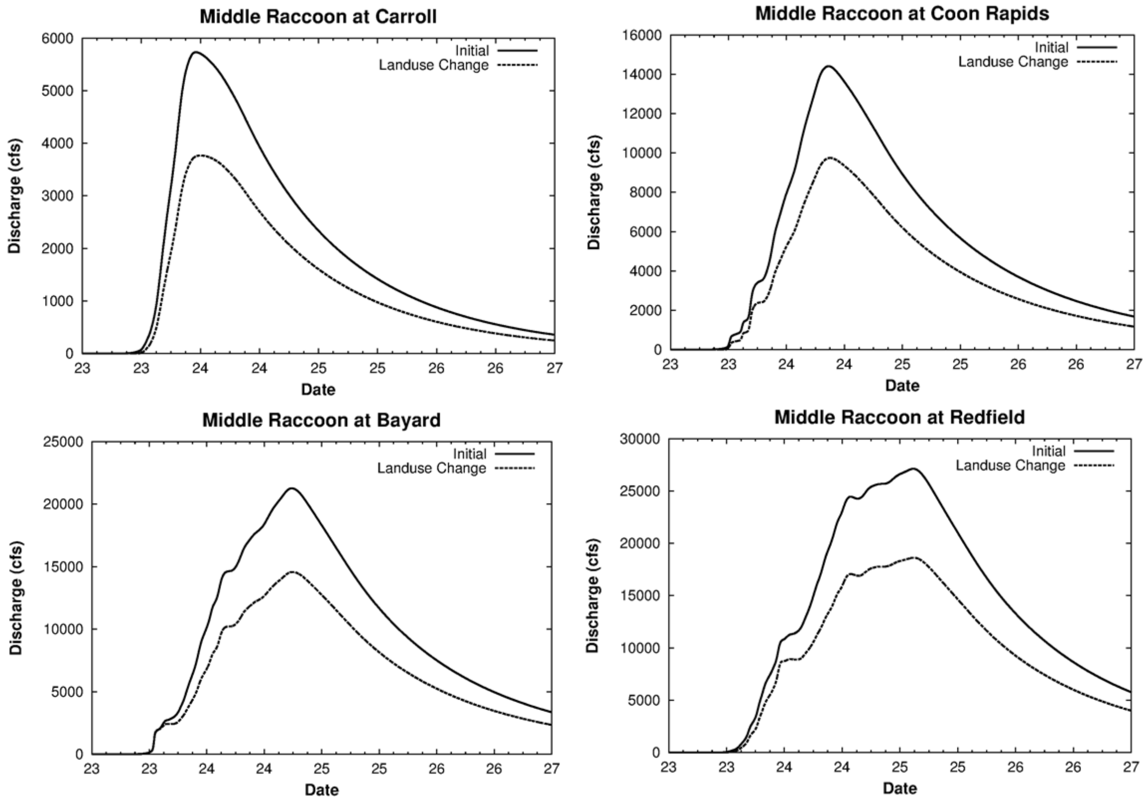

Figure 6.4: Hydrograph comparision at several locations for the increased infiltration scenario resulting from hypothetical land use changes (conversion of row crop agriculture to native tall-grass prairie). Results shown are for the 50-year, 24-hour storm (6.00 inches).

Table 6.2 summarizes the percent reductions in peak discharge resulting from this hypothetical native tall-grass prairie scenario at the five index locations for all the design storm events. The restoration of native tall-grass prairie typically resulted in peak discharge reductions of 28 to 45 percent. As in the forest scenario, the peak reduction was largest for the smallest design storm (10-year return period), and decreased with larger rainfall amounts (up to the 100-year return period). Again, note also that the percent reduction in peak discharge was fairly uniform at all locations.

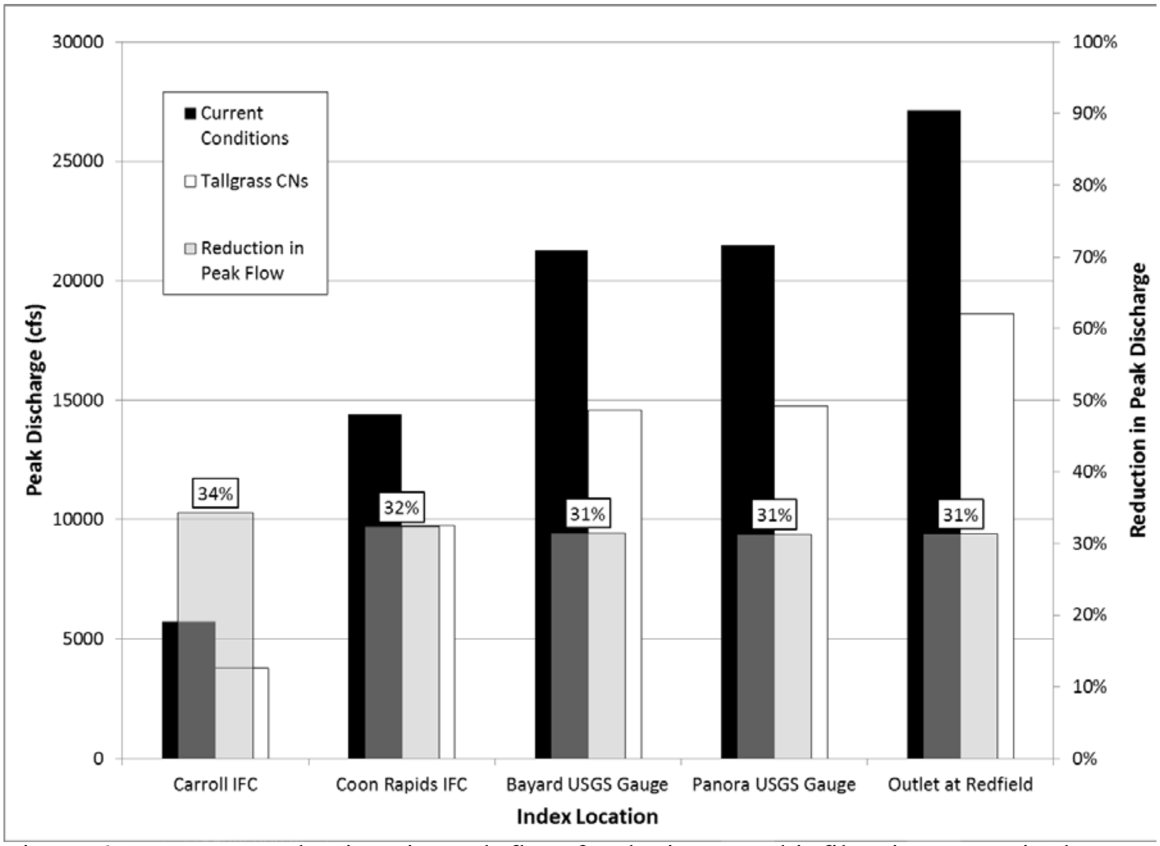

Figure 6.5: Percent reductions in peak flow for the increased infiltration scenario due to land use changes (conversion of row crop agriculture to native tall-grass prairie). Peak flow reductions at five index locations progressing from upstream (left) to downstream (right) are shown for the 50-year, 24-hour design storms (6 inches).

Table 6.2: Percent reductions in peak discharge for the agriculture to native prairie tall-grass scenario.

Index Location	Percent Peak Discharge Reduction Based on Storm Return Period (%)			
	10-YR (4.03 in)	25-YR (5.08 in)	50-YR (6.00 in)	100-YR (7.04 in)
Carroll IFC Gauge	44.8	38.5	34.3	30.5
Coon Rapids IFC Gauge	42.8	36.5	32.3	28.7
Bayard USGS Gauge	41.7	35.5	31.5	27.9
Panora USGS Gauge	41.5	35.3	31.3	27.7
Redfield IFC Gauge (Outlet)	41.6	35.4	31.3	27.8

Reducing peak flood discharge also reduces the peak water height (or stage) in a river during the flood. For the peak discharge reductions in the agriculture to tall-grass prairie scenario, the corresponding reduction in flood stage is between 1.9 and 3.5 feet. This reduction was estimated for the USGS stream-gauge locations, where the rating curves have been developed. Although a 1.9 to 3.5 foot reduction in flood stage would substantially reduce the flood inundation area, flooding still occurred in the native tall-grass prairie simulation. For instance, based on the flood stage level reported by the National Weather Service at Bayard, IA, water levels above action stage (13 feet) are expected for both the current agricultural and the tall-grass prairie landscapes for all rain events. Hence, conversion from agricultural to tall-grass prairie does not eliminate flooding, but would reduce its severity and frequency.

Increased Infiltration Through Soil Quality Improvements

Another way to reduce runoff is to improve soil quality. Better soil quality effectively lowers the runoff potential of the soil. If soil quality throughout the Middle Raccoon River watershed was improved, it could potentially reduce flood damages.

To simulate improved soil quality with the HMS model, we hypothesize that improvements translate to changes in the NRCS hydrologic soil group. As discussed previously, NRCS rates the runoff potential of soils with four hydrologic soil groups (A through D). Type A soils have the lowest runoff potential; type D soils have the highest runoff potential. The NRCS relies primarily on three quantities to assign a hydrologic soil group—saturated hydraulic conductivity (the rate water flows through the soil under saturated conditions), depth to an impermeable layer, and depth to the ground water table (Hoeft, 2007). Soils with a greater saturated hydraulic conductivity, or greater depth to an impermeable layer or ground water table, are assigned to a hydrologic soil group of lower runoff potential. To increase infiltration into the soil, one or more of these three quantities must be targeted. Obviously, the removal of all poorly draining soils throughout the watershed and replacement with higher infiltrating soils (like sands and gravels) is unrealistic. However, certain conservation and best management practices, such as increasing the organic material content in the soil and the introduction of cover crops, could aid in improving soil health to some degree.

In the HMS model of the Middle Raccoon River watershed, the effects of improved soil health through conservation and best management practices are represented by changes in the NRCS hydrologic soil group. The most dominant soil type in the Middle Raccoon River watershed is Type B, which makes up 66 percent of the area. In this scenario, improved soil quality was assumed to improve these soils to Type A. New SCS Curve Numbers, reflecting the lower runoff potential with improved soil quality, were assigned to each subbasin. The basin average curve number was lowered from 82.2 to 71.6. The model was then run for a set of design storms. Comparisons were made between the current and improved soil quality simulation for the 10-, 25-, 50-, and 100-year return period 24-hour SCS design storms.

The soil improvement case — where all Type B soils improve to Type A — resulted in approximately 0.7 inches more infiltration than current soil conditions for the 10-year return period design storm. Additional infiltration increased to about 0.9 inches for the 25-year storm, 1.0 inches for the 50-year and 1.1 inches for the 100-year storms. Figure 6.6 compares the simulated flood hydrographs for the current soil condition (baseline) to those for the first soil improvement case scenario for the 50-year return period, 24-hour design storm (6.00 inches of rain in 24 hours). Type B soils are relatively evenly distributed throughout the watershed, so the percent reduction in peak flow did not vary greatly from the headwaters (27 percent) to the basin outlet at Redfield (26 percent) for the 50-year, 24-hour event. Figure 6.7 summarizes the peak discharge for current conditions, the peak discharge for the hypothetical soil quality improvement scenario, and the peak reduction effect, at all five index locations for the 50-year, 24-hour design storm event.

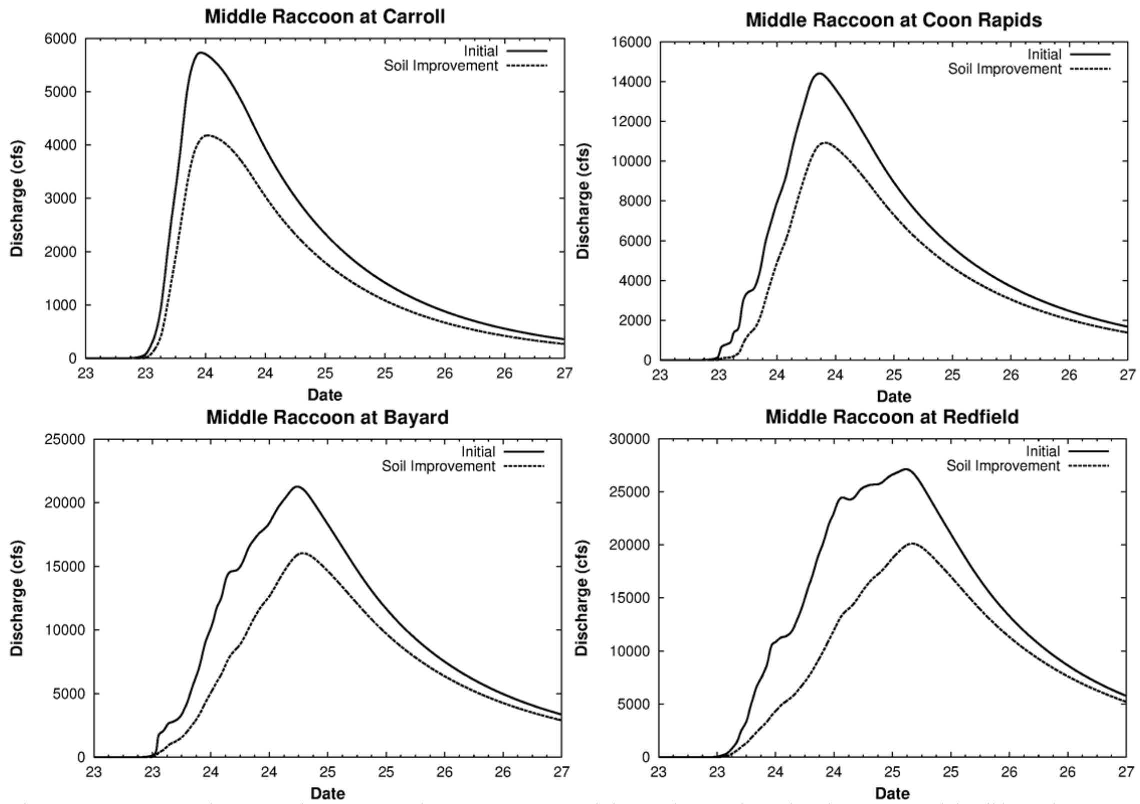

Figure 6.6: Hydrograph comparison at several locations for the increased infiltration scenario due to soil improvements. Improved soil quality was represented by converting all Hydrologic Soil Group B to A. Results are shown for the 50-year, 24-hour storm (6.00 inches)

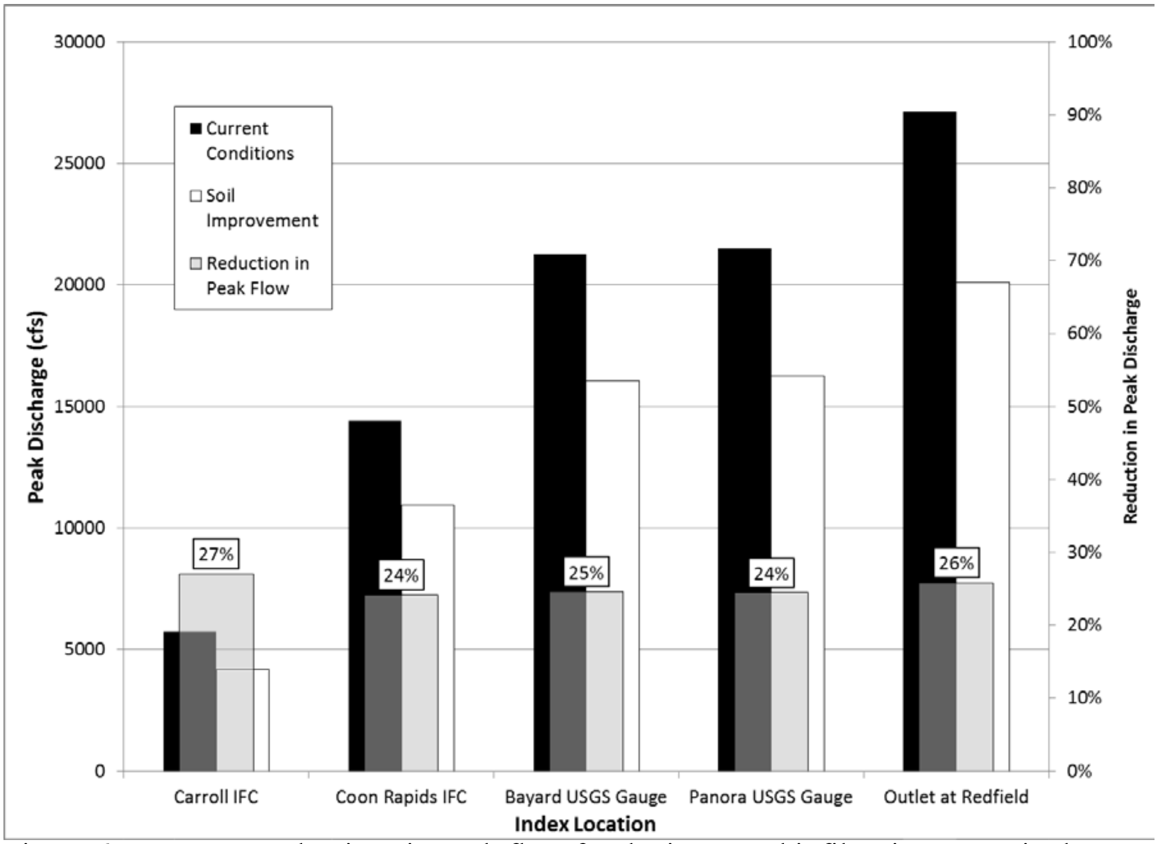

Figure 6.7: Percent reductions in peak flow for the increased infiltration scenario due to soil improvements (conversion soil HSG B to A). Peak flow reductions at five index locations, progressing from upstream (left) to downstream (right) are shown for the 50-yesr, 24-hour design sorms (6 inches).

Table 6.3 summarizes the percent reductions in peak discharge resulting from this hypothetical soil quality improvement scenario at the five index locations for all the design storm events. Improving soil quality typically resulted in peak discharge reductions of 21 to 36 percent. As a result, flood stages were reduced by 1.6 to 2.6 feet. As in the two other enhanced infiltration scenarios, the peak reduction was largest for the smallest design storm (10-year return period), and decreased with larger rainfall amounts (up to the 100-year return period). This outcome reflected the landscape's diminished capacity to infiltrate additional water as rain rates increase. Also as seen before, the percent reduction in peak discharge was fairly uniform at all locations.

Table 6.3: Percent reductions in peak discharge for the improved soil conditions scenario.

Index Location	Percent Peak Discharge Reduction Based on Storm Return Period (%)			
	10-YR (4.03 in)	25-YR (5.08 in)	50-YR (6.00 in)	100-YR (7.04 in)
Carroll IFC Gauge	35.8	30.5	27.0	24.0
Coon Rapids IFC Gauge	32.4	27.5	24.2	21.4
Bayard USGS Gauge	33.0	27.9	24.6	21.7
Panora USGS Gauge	32.9	27.8	24.4	21.5
Redfield IFC Gauge (Outlet)	34.7	29.3	25.8	22.8

Use of Conservation Practices: Agriculture Use to Agricultural plus Cover Crop

While it is evident that the change of land use from the current agricultural state to that of forest or native prairie tall-grass land use has a large impact on the reduction of peak flow, on the watershed-wide scale, it is neither economically feasible nor desirable. However, like changing land use, there are other methods with which to increase infiltration, without having to take the land entirely out of agricultural production. One common practice is the use of cover crops. Cover crops, such as oats and rye, are typically grown between periods of cash crops in order to fill a void in times where soil nutrients may otherwise be lost (Dabney, 1998). They affect the hydrology of a watershed by increasing hydraulic roughness, canopy and surface detention storage, and water infiltration rate (Dabney, 1998). For the purposes of this scenario, we focused on a cover crop's ability to increase the water infiltration rate. This was represented in the hydrologic model by a decrease in curve number. In this scenario, basin-wide curve numbers were reduced from 82.2 to 79.0.

Similar to the land use change scenarios, an analysis was performed to quantify the impact of applying uniform cover crops on the flood hydrology of the Middle Raccoon River watershed. In this example, all current agricultural land use was assumed to use cover crops. It would be a rare or improbable case in which all the agriculturally productive land was planted with cover crops, yet this scenario quantified the maximum reductions in peak flood discharge that could be expected for watersheds using cover crop conservation practices.

To simulate the application of cover crops with the HMS model, the model parameters affecting runoff potential across the landscape were adjusted to reflect hydrology of a watershed using cover crops. Specifically, existing agricultural land use, which accounts for 77 percent of the watershed area, was redefined as agriculture with cover crops. New SCS Curve Numbers, reflecting the lower runoff potential, were assigned to each subbasin. It is important to note that, as in the prior scenarios, other parameters estimated from Curve Numbers — such as the water flow travel time through the subbasin — were not adjusted. Thus, this scenario only considered the reduction in runoff volume resulting from the enhanced infiltration capacity of the cover crops; the attenuation and delay in the timing of the peak discharge that would be expected, due to the increased canopy and surface detention, increased hydraulic roughness, and increase in evaporation and transpiration, were not considered. Following assignment of new subbasin Curve Numbers, the model was run for a set of design storms. Comparisons were made between current and cover crop simulations for the 10-, 25-, 50-, and 100-year return period 24-hour SCS design storms. Using design storms of different severity illustrates how flooding characteristics changed during more intense rainstorms.

As expected, assuming 77 percent of the watershed was using the cover crop conservation practice, the impact on peak discharge was large. For the 10-year return period design storm (4.05 inches of rain in 24 hours), the simulated cover crops infiltrated 0.3 inches more into the ground than the current agricultural landscape. The additional infiltration generally increased to 0.3 inch for a 25-year storm, 0.4 inches for a 50-year storm, and 0.4 inches for a 100-year storm. As a result of increased infiltration across the landscape, the river response was slightly dampened.

Figure 6.8 compares the simulated flood hydrographs for the current agricultural landscape (Baseline) to those for a cover cropped landscape scenario for the 50-year return period, 24-hour design storm (6.00 inches of rain in 24 hours). For all four locations shown — from an upstream subbasin area (Carroll, IA) to the outlet of the Middle Raccoon River at Redfield, IA — the river discharges and peak discharge rates were less for a landscape using cover crops. At Carroll, the smallest drainage area shown (73.8 square miles), about 0.3 additional inches of rainfall would infiltrate if this area was using cover crops, resulting in a 9 percent reduction in its peak flood discharge. Downstream locations remained fairly uniform (7 to 9 percent), reflecting the relatively even distribution of agriculture throughout the watershed. Figure 6.9 summarizes the peak discharge for current conditions, the peak discharge for the cover crop scenario, and the peak reduction effect, at all five index locations for the 50-year, 24-hour design storm event.

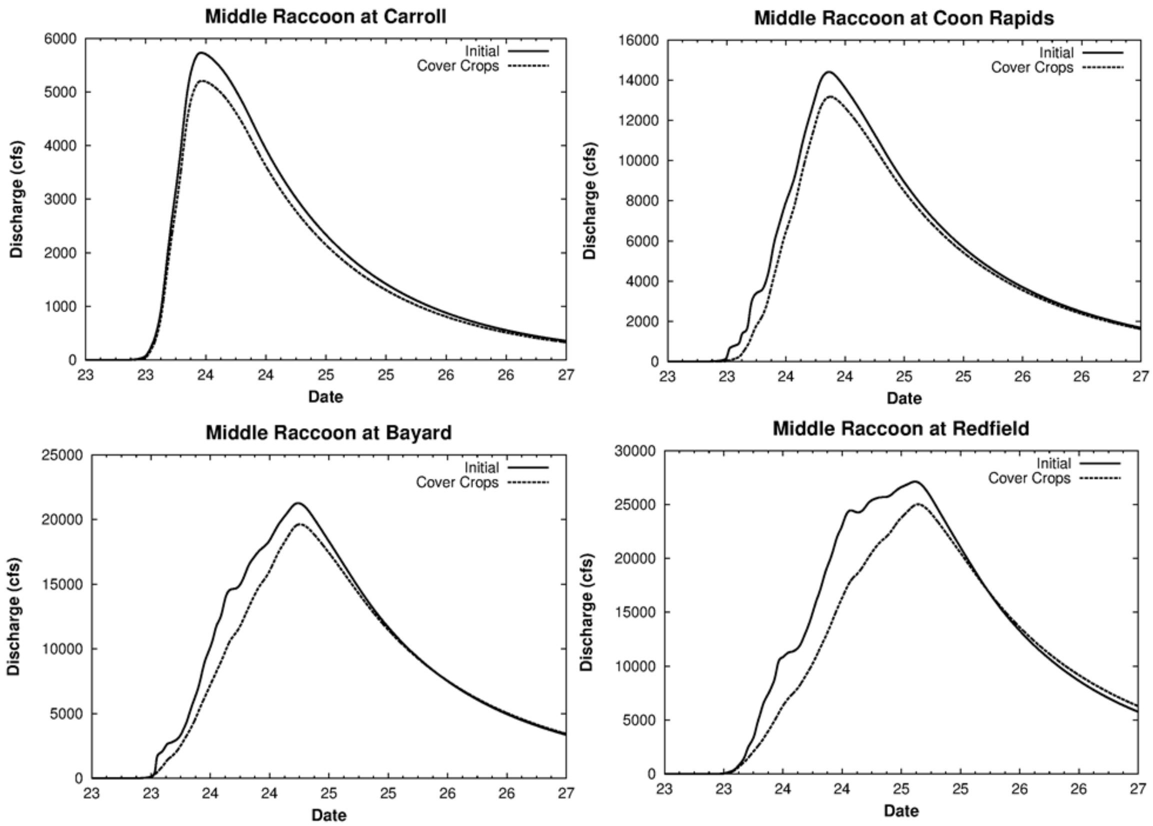

Figure 6.8: Hydrograph comparision at several locations for the increased infiltration scenario resulting from hypothetical conservation practice (conversion of row crop agriculture to agriculture using cover crops). Results shown are for the 50-year, 24-hour storm (6.00 inches).

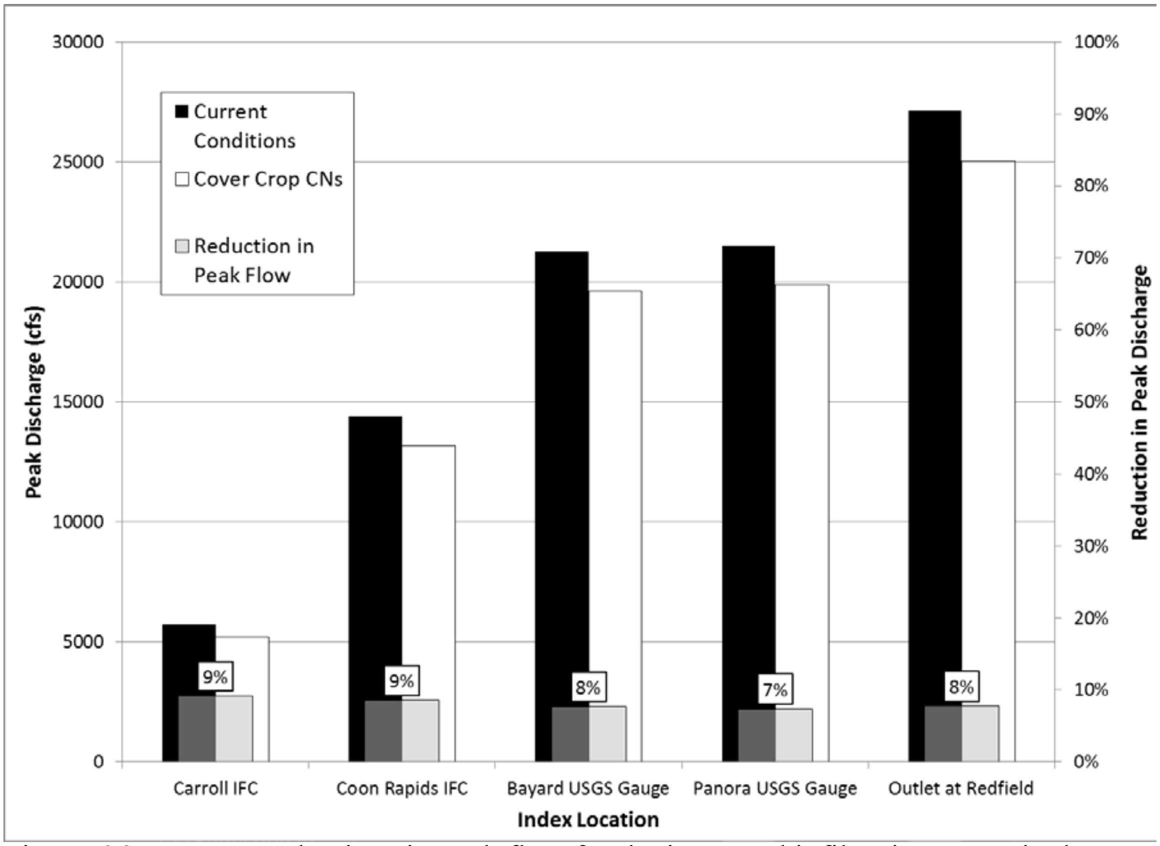

Figure 6.9: Percent reductions in peak flow for the increased infiltration scenario due to land use changes (conversion of row crop agriculture to agriculture using cover crops). Peak flow reductions at five index locations progressing from upstream (left) to downstream (right) are shown for the 50-year, 24-hour design storms (6 inches).

Table 6.4 summarizes the percent reductions in peak discharge resulting from this hypothetical cover crop scenario at the five index locations for all the design storm events. The cover crops typically resulted in peak discharge reductions of 6 to 13 percent. As in the other increased infiltration scenarios, the peak reduction was largest for the smallest design storm (10-year return period), and decreased with larger rainfall amounts (up to the 100-year return period). Again, note also that the percent reduction in peak discharge was fairly uniform at all locations.

Table 6.4: Percent reductions in peak discharge for the use of cover crops scenario.

Index Location	Percent Peak Discharge Reduction Based on Storm Return Period (%)			
	10-YR (4.03 in)	25-YR (5.08 in)	50-YR (6.00 in)	100-YR (7.04 in)
Carroll IFC Gauge	12.8	10.5	9.4	7.9
Coon Rapids IFC Gauge	12.2	9.9	8.5	7.4
Bayard USGS Gauge	11.2	9.0	7.7	6.5
Panora USGS Gauge	10.9	8.7	7.4	6.2
Redfield IFC Gauge (Outlet)	11.5	9.2	7.8	6.5

Reducing peak flood discharge also reduced the peak water height (or stage) in a river during the flood. For the peak discharge reductions in the agriculture to agriculture with cover crops scenario, the corresponding reduction in flood stage was between 0.4 and 0.8 feet. This reduction was estimated for the USGS stream-gauge locations, where the rating curves have been developed. Although a 0.4 to 0.8 foot reduction in flood stage would slightly reduce the flood inundation area, flooding would still occur. Again, based on the flood stage level reported by the National Weather Service at Bayard, water levels above action stage (13 feet) are expected for both the current agricultural and the agriculture with cover crops for all rain events. Hence, the addition of cover crops does not eliminate flooding, but would reduce its severity and frequency.

Summary of Flood Mitigation Using Increased Infiltration

The four scenarios in this chapter each represent a method which could help the Middle Raccoon watershed increase its infiltration capacity. These four scenarios used changes in land use, soil improvements, or conservation practices to decrease runoff and increase infiltration. In the hydrologic model, this was represented by a decrease in curve number. The most drastic changes to the watershed, such as converting all the current agricultural land to forest or native prairie tall-grass, had the most significant changes in peak discharge (up to 56 percent). The less drastic, but more feasible, scenarios such as application of cover crops as a conservation practice, also produced a reduction in peak discharge and stage (up to 13 percent), but not to the extent acquired by changing the entire land use. Table 6.5 below summarizes the average reduction in curve number for each of the scenarios analyzed in this chapter, along with their increases in infiltration, decreases in discharge, and decreases in flood stage. All scenarios analyzed in this chapter effectively reduced discharge in some capacity; therefore, a watershed wide flood mitigation plan could include any, or all, of the above strategies. A more localized and detailed analysis should be done on any specific project before being implemented.

Table 6.5: Summary of the effects of increased infiltration on peak discharge, 50-year, 24-hour event.

Scenario	Decrease in CN (%)	Increase in Infiltration (in)	Decrease in Discharge (%) – Bayard, IA	Decrease in Stage (ft) – Bayard, IA
Ag to Forest	18.9	1.3	40.2	2.9
Ag to Prairie	16.9	1.0	31.3	2.1
Soil Improvements	13.9	1.0	25.8	1.6
Cover Crops	3.5	0.4	7.8	0.5

CHAPTER 7: MITIGATING THE EFFECTS OF HIGH RUNOFF WITH DISTRIBUTED FLOOD STORAGE

Another way to mitigate the effects of high runoff is with distributed flood storage. Ponds provide the most common type of flood storage. In agricultural areas, ponds usually hold some amount of water at all times. However, ponds also have the capacity to store additional water during high runoff periods. This so-called flood storage can be used to reduce flood peak discharges.

Unlike the increased infiltration approaches for reducing runoff, storage ponds do not change the volume of water that runs off the landscape. Instead, storage ponds hold floodwater temporarily, and release it at a lower rate. Therefore, the peak flood discharge downstream of the storage pond is lowered. The effectiveness of any one storage pond depends on its size (storage volume) and how quickly water is released. By adjusting the size and the pond outlets, storage ponds can be engineered to efficiently utilize their available storage for large floods.

A system of ponds located throughout a watershed could be an effective strategy for reducing flood peaks at many stream locations. As an example, in the 1980s, landowners in southern Iowa came together to form the Soap Creek Watershed Board. Their motivation was to reduce flood damage and soil loss within the Soap Creek watershed. They adopted a plan that included the identification of locations for 154 distributed storage structures (mainly ponds) which could be built within the watershed. As of 2014, 132 of these structures have been built. (Wunsch, 2013)

In this section, the HMS model is used to simulate the effect of pond storage on flood peaks. For this hypothetical example, many ponds are distributed in tributary regions throughout the Middle Raccoon River watershed; because an actual storage pond design requires detailed site-specific information, a prototype pond design that mimics the hydrologic impacts of flood storage was used. Therefore, this example is not a proposed plan for siting a system of storage ponds, as it has not been determined whether suitable sites are available in the simulated locations. Still, this hypothetical example does provide a quantitative benchmark on the effectiveness of distributed flood storage and the flood reduction benefits that are physically possible.

Prototype Storage: Pond Design

Many ponds in Iowa have been constructed to provide flood storage. A pond schematic is illustrated in Figure 7.1. The pond is created by constructing an earthen embankment across the stream. A typical pond holds some water at all times (referred to as permanent pond storage). However, if the water level rises high enough, an outlet passes water safely through the embankment. This outlet is called the principal spillway. As the water level rises during a flood, more water is stored temporarily in the pond. Eventually, the water level reaches the emergency spillway. The emergency spillway is constructed as a means to release water rapidly so the flow does not damage or overtop the earthen embankment. Storage between the permanent pool and emergency spillway is referred to as the total flood storage.

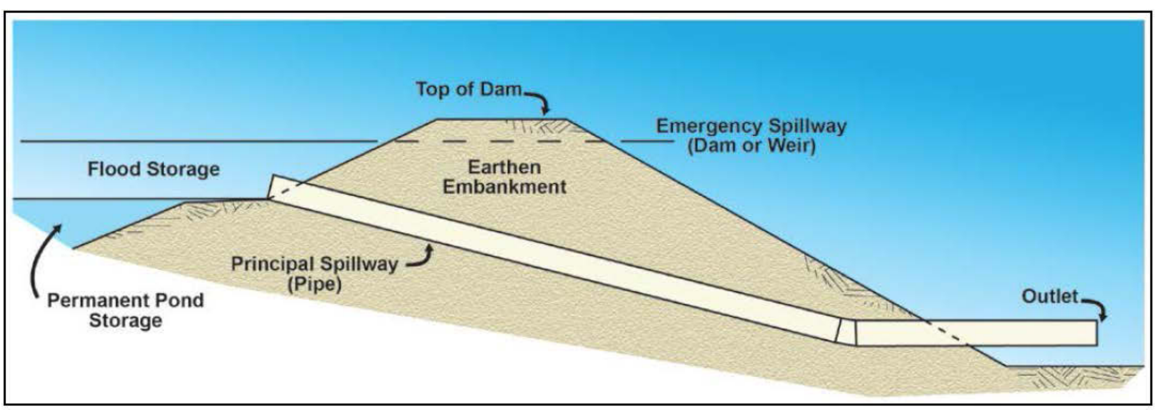

Figure 7.1: Prototype pond used for distributed flood storage analysis.

In addition to the typical pond design above, a second "dry" pond design was considered. A dry pond does not hold water under normal circumstances and, therefore, has no permanent pond storage. In this design, an additional 2-inch diameter outlet is set at the bottom of the pond so that, under normal conditions, inflow will roughly equal outflow. This allows for additional flood storage during times of high runoff, but also means that the pond will not serve additional purposes such as irrigation or watering animals. All of the other design characteristics in the dry pond scenario remained the same.

Prototype Pond Outlet and Emergency Spillway

Using information from ponds constructed in Soap Creek, as well as NRCS Technical References on pond design, a prototype pond outlet and emergency spillway were defined for the simulation experiments. In all cases, a 12-inch pipe outlet was assumed for the principal spillway, a 20 foot wide overflow opening was assumed for the emergency spillway, and the top of the dam was set two feet above the emergency spillway. In the case of dry ponds, an additional 2-inch pipe was considered at the pond bottom.

The elevation difference between the principal and emergency spillways varied; for the typical pond design, simulations were done with elevation differences of 3, 5, 7, and 10 feet. As the elevation difference increased, the available flood storage increased exponentially. Therefore, simulations for ponds with a 10 foot elevation difference have much more flood storage than those with a 3 foot difference. The elevations of the spillway in the pond designs were dependent upon the landform region where the pond was located. Due to its steeper topography, the Southern Iowa Drift Plain region needed higher emergency spillways to match the storage values seen in the flatter Des Moines Lobe region. Emergency spillways in the Southern Iowa Drift Plain were designed at 7 and 10 feet, while emergency spillways in the Des Moines Lobe were designed at 3 and 5 feet.

For the dry pond design, an additional 2 inch pipe was simulated in the Des Moines Lobe ponds; this pipe remained fixed at the bottom of the pond. Simulations were done with the emergency spillway set at 8 and 10 feet above the bottom of the pond. Therefore, the total pond volume did not change between the typical and dry pond designs. For example, in both designs, the total volume of the 3 foot typical pond and the 8 foot dry pond remained fixed at 62.8 acre-feet; only the amount of flood storage varied—26.8 acre-feet versus 34.2 acre-feet for the typical and dry ponds, respectively. In this way, the effects of two ponds were compared with roughly equivalent construction and operation costs, but serving different functions.

The amount of water released downstream by the pond depends on the water depth. The discharge from the principal spillway was determined using pipe flow hydraulic calculations. Once the water depth reached the emergency spillway, releases also included contributions from the emergency spillway. Discharge of the emergency spillway was determined using NRCS Technical References, assuming “C-Type” retardance, which was determined to be a reasonable design assumption (based on discussions with regional NRCS engineers). Discharge downstream began immediately in both ponds, since the typical pond is considered full (at the elevation of the principal spillway) prior to the rainfall event. However, it should be noted that more water would be released through the principal spillway, at the same relative elevation, in the dry pond design compared to the typical design. Since, the dry pond has an additional 2 inch pipe set five feet lower, discharge from the pipe began earlier in this design.

Prototype Pond Shape

Although pond design specifications and built ponds in Iowa provide a reasonable prototype for a pond outlet, the amount of water stored behind an earth embankment requires local knowledge of the topography behind the embankment. For hundreds of unique pond locations, the effort to compute a precise relationship between pond stage (water level) and water storage for each would be enormous. The effort would also be unwise, unless suitable sites for pond structures were selected in the first place (for each and every pond). As a compromise, the relationship between stage and storage at eight potential pond sites in the Middle Raccoon River watershed was analyzed, and the results were averaged to define a prototype pond shape.

The first step was to select several potential pond sites in the Middle Raccoon River watershed for topographic analysis. Figure 7.2 shows the subbasins in the HMS model. Of these, 160 were headwater basins. Headwater basins make good locations for flood storage ponds; they have relatively small drainage areas, and typical pond outlets (like the prototype above) can effectively reduce flood discharge at this scale. Hence, eight of the 160 headwater basins were selected as exploratory sites. These eight were scattered throughout the watershed and encompassed both geographic landform regions (four in the Des Moines Lobe and four in the Southern Iowa Drift Plain).

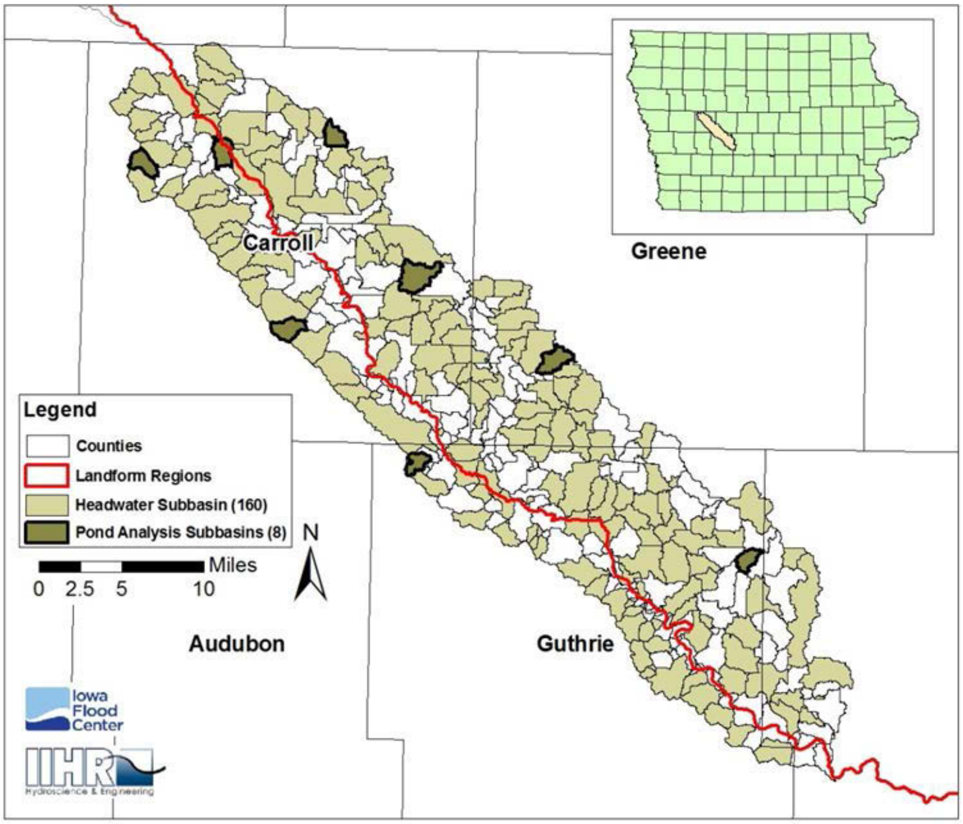

Figure 7.2: Subbasin locations selected for distributed flood storage analysis. Hypothetical ponds were placed in 160 headwater subbasin (beige) and eight of these subbasins (darkened) were used as exploratory sites to develop relevant pond characteristics needed for the HMS model.

In each of the eight subbasins, a location for a pond embankment was selected. Each site was chosen based on sufficient topographic relief to support the construction of a pond. Then, for a given water level, the volume of water that would be impounded behind the dam was computed. This calculation was done by ArcGIS 3D analyst, using the area and volume statistics tool and the 3 m^2 digital elevation model (DEM) of the local terrain. Once the pond location was defined, the tool could calculate volumes and areas for a given water surface elevation; the calculation was repeated for many different water levels. The final result — the storage volume in the pond for different water levels — is known as a stage-storage relationship. Figure 7.3 shows how the stage storage relationship was developed for one location analyzed (W4380). As can be observed, the site was selected due to the available space, suitable land use, and the natural "pinch point" in the topography, which would result in a shorter, more economically feasible dam. The three shades of blue represent the pond surface area at the principal spillway, the emergency spillway, and the Top of Dam (TOD).

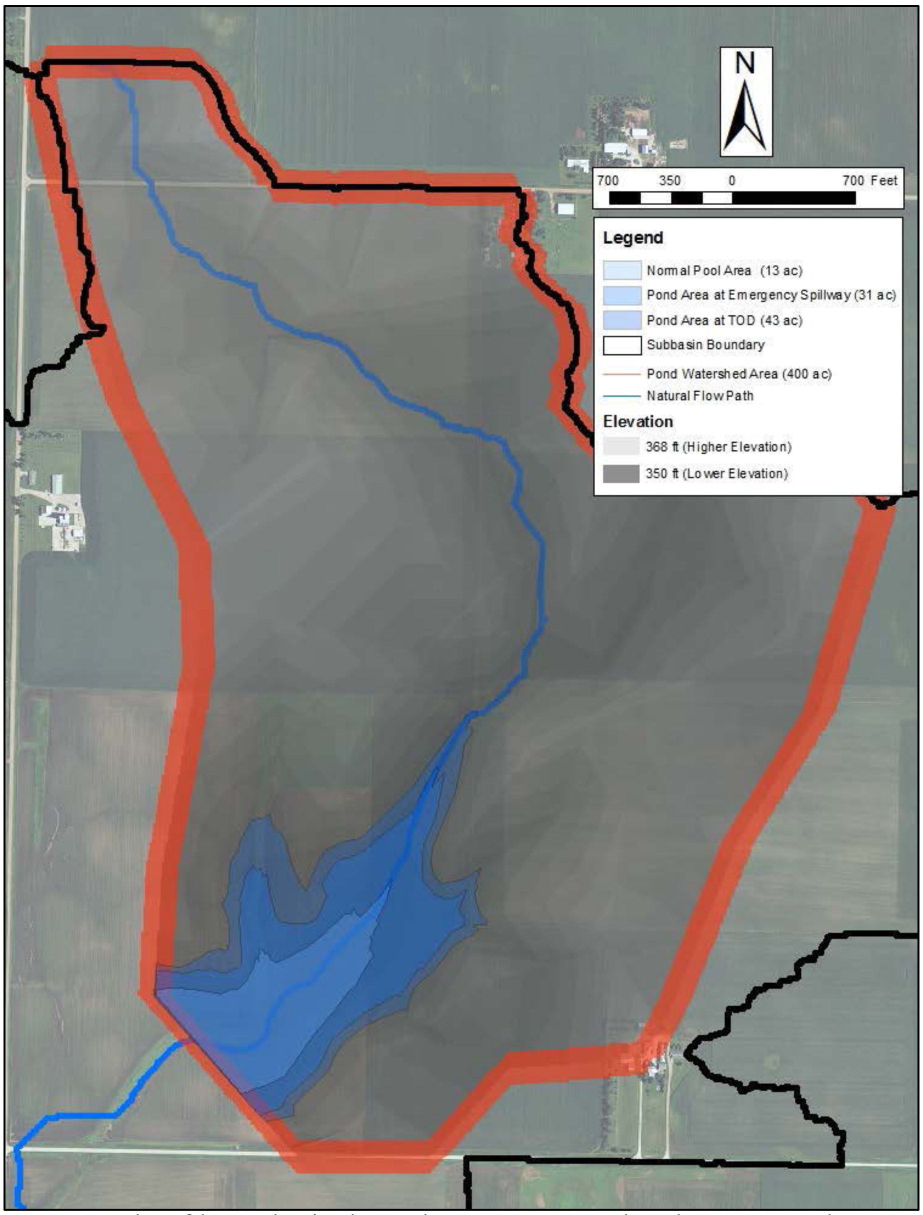

Figure 7.3: Example of hypothetical pond stage-storage development and topographic analysis at subbasin W4380.

The last step was to compare the different stage-storage relationships developed for the eight pond locations. The stage-storage relationships for similar projects constructed in the Soap Creek watershed were also examined. As expected, stage-storage relationships could be very different at different sites. Indeed, one would anticipate that pond storages for flat topography would be quite different from those for steep topography. As a result, different stage-storage relationships were discovered in the Des Moines Lobe (with its flatter terrain) compared to those in the Southern Iowa Drift Plain (with its steeper terrain). Therefore, two different stage-storage relationships were developed—one for the ponds in the flatter Des Moines Lobe, and another for the ponds in the steeper Southern Iowa Drift Plain. The stage-storage tables for all of the ponds used in distributed storage scenarios can be seen in Appendix B.

Prototype Pond Hydraulics

The pond shape defines the stage-volume relationship as water levels change in the pond. In contrast, the pond outlet defines the stage-discharge relationship for the pond. This information is combined to define the prototype storage-discharge hydraulic relationship needed in HEC-HMS for pond simulations.

In all, 6 different prototype pond storage discharge tables were used. First, for the typical pond designs, four sizes were considered. For the small pond scenario, the emergency spillway elevation was set to 3 feet above the primary spillway in the Des Moines Lobe and 7 feet above the primary spillway in the Southern Iowa Drift Plain; this resulted in a flood storage capacity of 23.8 acre-feet in the Southern Iowa Drift Plain and 26.8 acre-feet in the Des Moines Lobe. For the large pond scenario, the emergency spillway elevation was set to 5 feet above the primary spillway in the Des Moines Lobe and 10 feet above the primary spillway in the Southern Iowa Drift Plain; this resulted in a total flood storage capacity of 38.6 acre-feet in the Southern Iowa Drift Plain and 54.5 acre-feet in the Des Moines Lobe.

For the dry pond design, two sizes were considered. The design of these ponds was identical to the typical pond designs mentioned above, except an additional 2 inch outlet was set at the bottom of the pond. In these ponds, water was not stored under normal circumstances. This resulted in a larger total storage available for flood waters. For these scenarios, dry ponds were only assumed in the Des Moines Lobe landform region. This was due to the fact that the Des Moines Region is much flatter, and a dry pond in this location could reasonably be farmed during non-flood conditions. A dry pond in the Southern Iowa Drift Plain would have much steeper banks and would, therefore, not be conducive to farming practices, and could be seen as undesirable by land owners. For the small dry pond, the emergency spillway was set to 8 feet above the pond bottom in the Des Moines Lobe and 7 feet above the principal spillway in the Southern Iowa Drift Plain; this resulted in a total storage capacity 23.8 acre-feet in the Southern Iowa Drift Plain and 34.2 acre-feet om the Des Moines Lobe. For the large dry pond scenario, the emergency spillway elevation was set to 10 feet above the pond bottom in the Des Moines Lobe and 10 feet above the principal spillway in the Southern Iowa Drift Plain; this resulted in a flood storage capacity of 38.6 acre-feet in the Southern Iowa Drift Plain and 62.8 acre-feet in the Des Moines Lobe. The stage-storage-discharge relationships for all of the typical prototype pond scenarios are found in Appendix B.

Siting of Hypothetical Ponds

To examine the hypothetical impact that flood storage would have on the flood hydrology of the Middle Raccoon River watershed, prototype ponds were placed throughout the headwater subbasins (see again Figure 7.2). In the Soap Creek watershed, where flood storage is already used extensively, the average pond density was 1 built pond for every 1.9 square miles of drainage area. Therefore, for the flood storage simulations for the Middle Raccoon River watershed, it was decided to place pond structures in headwater subbasins at a density of 1 pond for every 2 square miles of drainage area.

The 160 headwater subbasins ranged in size from 0.1 to 8.2 square miles. Hence, all the subbasins contained between one and four ponds. For example, if a subbasin drainage area was 4.2 square miles, it would have two ponds (number of ponds was rounded to the nearest whole number). Furthermore, not all the area within a subbasin drained to a pond; some water would flow into the stream below the ponds and would not be temporarily stored. To handle these conditions in the HMS model, it was first assumed that half the subbasin areas drain through a pond, and half do not. Next, for areas that drain through a pond, it was assumed that the water passes through only one pond (and not from one to the next and so on). This step was most efficiently accomplished in the model by creating a single aggregate pond. That is, if there were 3 ponds in a subbasin, it had the same aggregate effect of a single pond that had three times the storage and three times the outflow. So from an HMS modeling standpoint, the half of the subbasin that drained through a pond could more simply be routed through a single aggregated pond. In this way, the effects of the pond storage could be estimated, without having to specify the exact physical locations of any pond.

For the 160 headwater subbasins, a total of 198 prototype ponds were simulated. All the subbasins contained between 1 and 4 ponds. Figure 7.4 shows the 160 headwater subbasins, and the number of ponds assigned to each. In HMS, the 198 prototype ponds were represented by 160 aggregated ponds, one for each of the 160 subbasins. Overall, the ponds controlled flows from a total area of 175 square miles (or 30 percent of the watershed); in other words, 30 percent of the watershed area drained through the simulated prototype ponds.

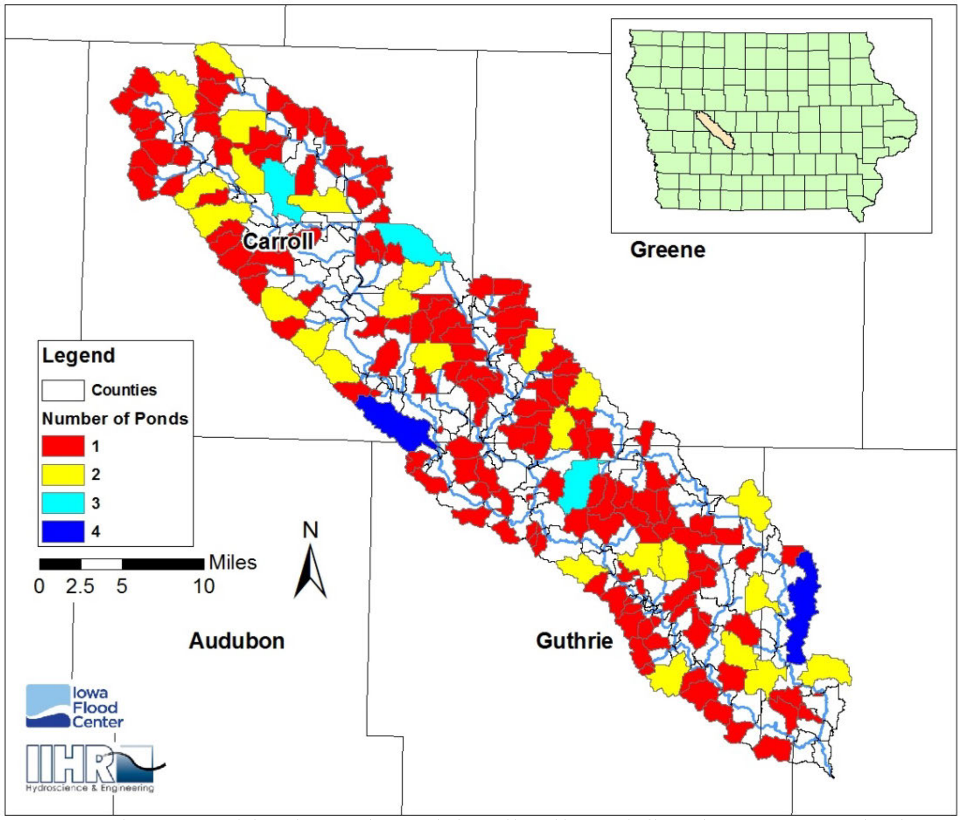

Figure 7.4: Headwater subbasins selected for distributed flood storage analysis and the number of prototype ponds assigned to each subbasin.

For the two USGS stream-gauges and the three IFC stream-gauges, the pond characteristics upstream of the locations are characterized in Table 7.1. Overall, the percentage of the upstream area controlled by ponds was relatively consistent; it ranged from approximately 30 percent for the Middle Raccoon River at Coon Rapids, Bayard, and Redfield, to a maximum of 35 percent for the Middle Raccoon River at Carroll. For the typical ponds, the small ponds had a total flood storage of 4,709 acre-feet; this amount of water placed over the upstream drainage area would have a water depth of 0.5 inches. Hence, the ponds could temporarily store roughly 0.5 inches of runoff from upstream of the ponds before filling completely. For large ponds, the total storage was 9,693 acre-feet; this is equivalent to roughly 1.0 inch.

For the dry ponds, the small ponds had a total storage of 6,051 acre-feet; this amount of runoff placed over the upstream drainage area would have a water depth of 0.6

inches. For large ponds, the total flood storage was 10,765 acre-feet; this is equivalent to roughly 1.2 inches. These average storage depths were relatively consistent for the upstream areas of the five locations.

Table 7.1: Summary of pond characteristics for the distributed flood storage analysis at five index locations.

Location	Drainage Area (mi^2)	Number of Headwater Subbasins Upstream	Number of Ponds Upstream	Drainage Area Controlled by Ponds (mi^2)	Percent Controlled
Middle Raccoon at Carroll	74	21	28	26	35%
Middle Raccoon at Coon Rapids	217	58	72	66	30%
Middle Raccoon at Bayard	382	110	132	116	30%
Middle Raccoon at Panora	426	129	155	131	31%
Middle Raccoon at Redfield	590	160	189	175	30%

Distributed Storage Simulations

The HMS model was run with ponds to simulate the effects of flood storage on peak discharges. Separate model runs were created for the typical pond design and the dry pond design; each pond design was broken into two separate scenarios—small and large ponds. For the small ponds scenario, in the case of the typical pond, each simulation started with all pond water levels at the principal spillway elevation; this assumed that the permanent storage was full as the storm began. For the dry pond, each simulation started with completely empty ponds (inflow equal to outflow). Comparisons were then made for the simulated flows without ponds in place (the existing baseline condition). Flood hydrographs were compared for the 10-, 25-, 50-, and 100-year return period 24-hour SCS design storms.

Typical Pond Results

Figure 7.5 compares the simulated flood hydrographs for the current no pond condition (Baseline) to those with small prototype ponds for the 50-year return period 24-hour design storm (6.00 inches of rain in 24 hours). The smallest drainage area shown, at Carroll, IA, has a drainage area of 73.8 square miles. Twenty-eight prototype ponds were placed upstream. As a result, the peak discharge was reduced by 7 percent. The water runoff from this storm quickly filled the available storage and engaged the emergency spillway, so there was limited benefit from ponds of this size. There was only sufficient flood storage available to reduce the peak discharge from 5,733 cfs (with no ponds) to 5,338 cfs (with small ponds).

Even though the area controlled was very similar throughout the basin, the peak flow reduction was not. At Carroll, where the ponds upstream mostly lie in the Southern Iowa Drift Plain, the peak flow reduction was minimal. The smaller prototype ponds in the Southern Iowa Drift Plain filled faster than the larger ponds in the Des Moines Lobe. Even though a larger percentage of the watershed at Carroll drained through ponds, it had a smaller percentage of available storage. At Coon Rapids, the next index location downstream, the peak reduction was at a maximum (11 percent). At this location, a larger percentage of ponds upstream lie in the Des Moines Lobe. Even though the area controlled by ponds was very similar downstream, and the mix of ponds from the Southern Iowa Drift Plain and Des Moines Lobe were similar, the peak reduction gradually decreased downstream to the basin outlet at Redfield (6 percent). Generally speaking, the small typical pond design was not sufficiently sized to handle rainfalls of this magnitude.

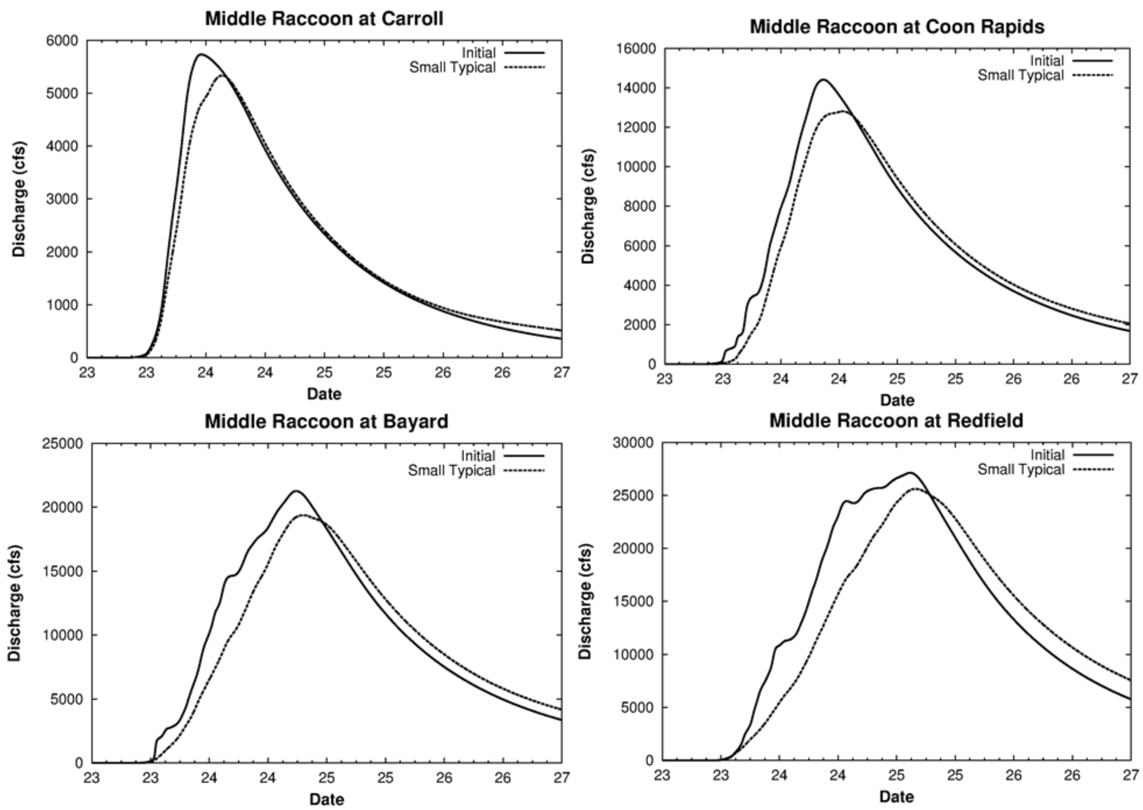

Figure 7.5: Comparisons of hydrographs with and without small ponds for the 50-year, 24-hour storm (6.00 inches). For the hydrographs shown, peak flow reduction ranges from 6-11%

Figure 7.6 shows the peak discharge reductions at the two USGS discharge gauge locations and the three IFC discharge gauge locations for the small pond scenario (3 foot and 7 foot emergency spillway elevations) for the 50-year, 24-hour event (6.00 inches).

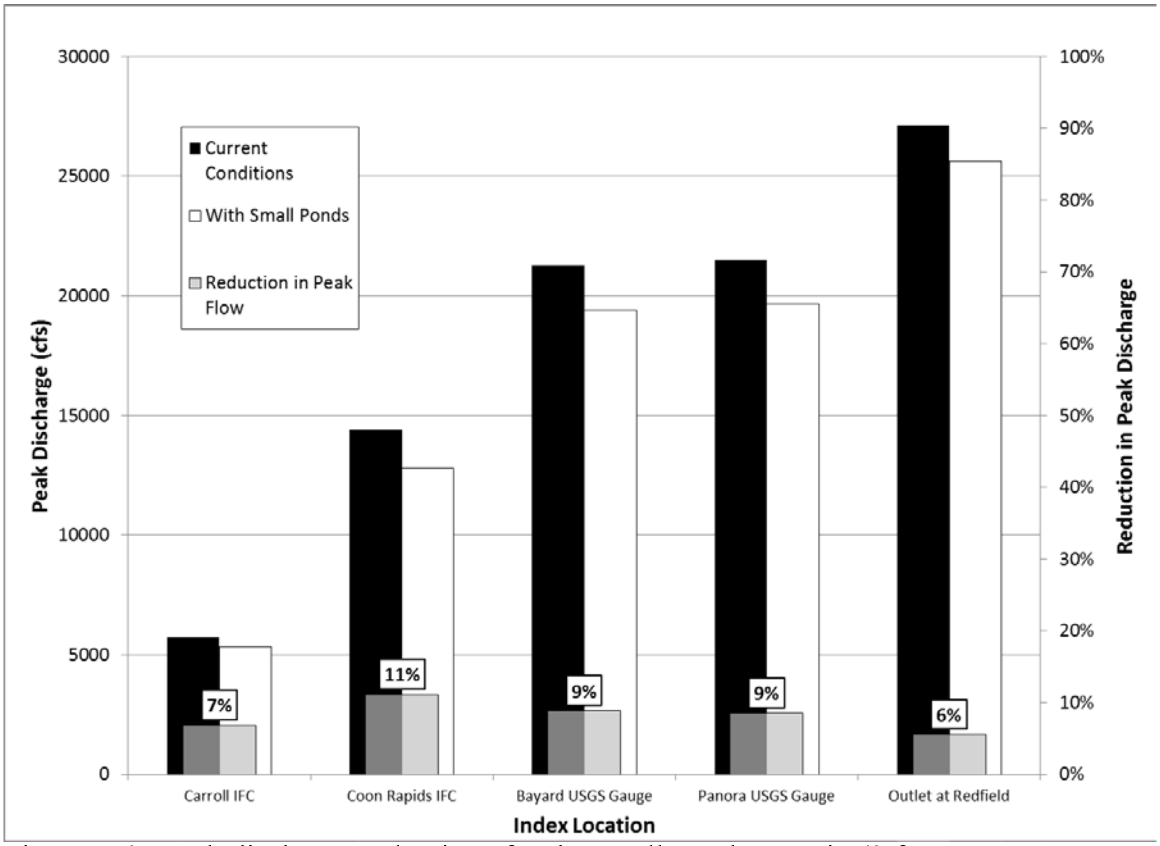

Figure 7.6: Peak discharge reductions for the small pond scenario (3 foot emergency spillway). Results are shown at five index points moving from upstream (left) to downstream (right) for the 50-year, 24-hour design storm (6 inches)

Table 7.2 summarizes the percent reductions in peak discharge for the small typical pond scenario at the five index locations for all the design storm events. In this scenario, each pond in the Southern Iowa Drift Plain provided 23.8 acre-feet of flood storage and each pond in the Des Moines Lobe provided 26.8 acre-feet of flood storage, resulting in a total of 4,709 acre-feet of flood storage for the entire watershed. For the small ponds, the percent reduction was greatest for the 10-year return period flood, and decreased for larger floods; the small ponds fill rapidly for large floods, at which point little attenuation in flood peak was achieved. As noted above, the peak reduction effect varied with drainage area. It was typically larger for small drainage areas, where the location was closer to the headwater ponds, and decreased in the downstream direction.

The one exception was the IFC gauge location at Carroll, for the 25- to 100- year events, where its upstream area was primarily in the Southern Iowa Drift Plains (where ponds are smaller and less flood storage is available). Otherwise, the peak reduction range was larger at smaller upstream locations; at Coon Rapids it varied from about 16 percent (10-year event) to 8 percent (100-year event), whereas at the downstream-most location of Redfield, it varied from 10 percent (10-year event) all the way to 4 percent (100-year event).

Table 7.2: Percent reduction in peak discharge using the typical small pond design (3 or 7 foot emergency spillway elevations).

Location	Percent Peak Discharge Reduction			
	10–YR (4.03 inches)	25-YR (5.08 inches)	50-YR (6.00 inches)	100-YR (7.04 inches)
Carroll IFC Gauge	18.0	10.3	6.9	4.6
Coon Rapids IFC Gauge	15.8	14.0	11.2	8.4
Bayard USGS Gauge	13.0	10.7	8.9	7.1
Panora USGS Gauge	12.7	10.3	8.6	6.9
Redfield IFC Gauge (Outlet)	9.8	7.0	5.6	4.3

Figure 7.7 compares the simulated flood hydrographs for the current no pond condition (Baseline) to those with large prototype ponds for the 50-year return period 24-hour design storm (6.00 inches of rain in 24 hours). At Carroll twenty-eight prototype ponds were placed upstream. As a result, the peak discharge was reduced by 16 percent. The operation of the ponds is most evident at this size pond and at this location. Initially, water discharged from the subbasin without significant delay. Then, the rise in the discharge was halted, as water was stored in the ponds. After water began to flow over

the emergency spillway, discharge increased rapidly again. The additional flood storage in this scenario reduced the peak discharge from 5,733 cfs (with no ponds) to 4,841 cfs (with large ponds).

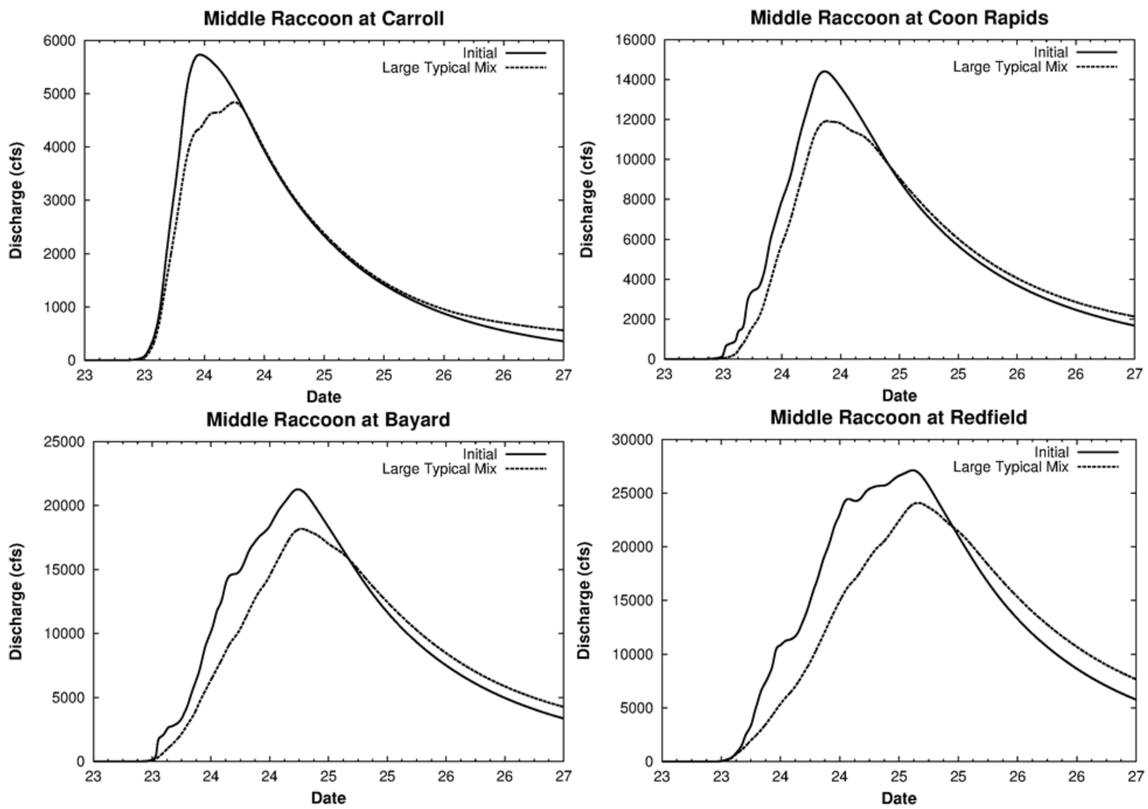

Figure 7.7: Comparisons of hydrographs with and without large ponds (7 foot emergency spillway) for the 50-year, 24-hour storm (6.00 inches). For the hydrographs shown, peak flow reduction ranges from 11-17 percent.

Figure 7.8 shows the peak discharge reductions at the two USGS discharge gauge locations and the three IFC discharge gauge locations for the large pond scenario (5 foot and 10 foot emergency spillway elevations) for the 50-year, 24-hour event (6.00 inches).

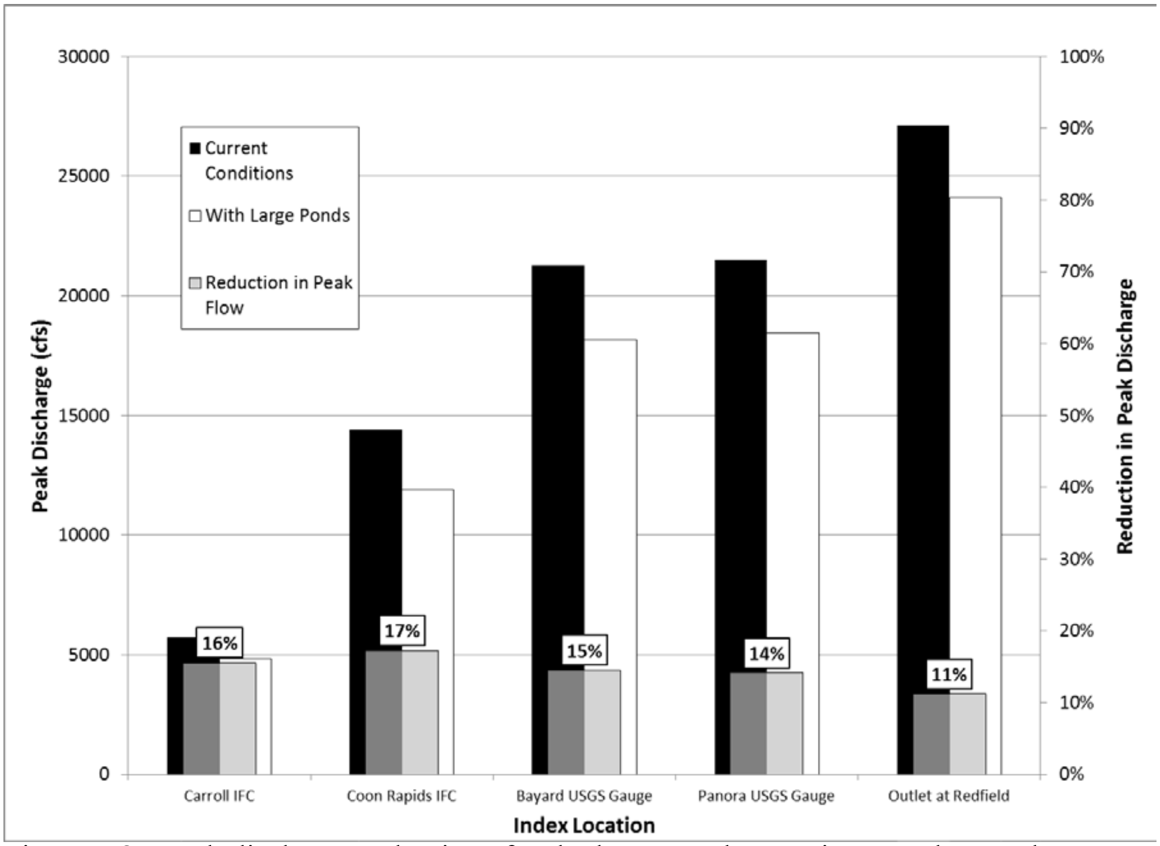

Figure 7.8: Peak discharge reductions for the large pond scenario. Results are shown at five index points moving from upstream (left) to downstream (right) for the 50-year, 24-hour design storm (6 inches).

Table 7.3 summarizes the percent reductions in peak discharge for the large pond scenario at the five index locations for all the design storm events. In this scenario, each pond provided 38.6 acre-feet of flood storage for the Southern Iowa Drift Plain and 54.5 acre-feet of total storage in the Des Moines Lobe Region, resulting in a total of 9,693 acre-feet of total storage for the entire watershed. With this additional flood storage, (approximately 2.1 times the small pond flood storage), the peak reduction increased. Percent reduction in peak flow remained relatively constant for the 10- through 50-year design storm events at the watershed outlet, yet it was a maximum at the 25-year event (14 percent). This was due to pond utilization of the potential flood storage to its maximum potential during this event (i.e. most ponds were relatively full but not engaging the emergency spillway). As expected, the peak reduction tended to be greater nearer to the headwater ponds (smaller drainage areas), and decreased for larger drainage areas downstream (with Carroll again being the exception). It should be noted that Carroll, IA had a maximum reduction for the 10-year and 25-year, 24-hour events; this was due to the ability of the smaller Southern Iowa Drift Plain ponds to store the 10-year and 25-year events when the spillway was raised to 10 feet. For the small pond designs, most emergency spillways were engaged much earlier.

Table 7.3: Percent reductions in peak discharge using the large typical pond design (5 foot and 10 foot emergency spillway elevations).

Location	Percent Peak Discharge Reduction			
	10–YR (4.03 inches)	25-YR (5.08 inches)	50-YR (6.00 inches)	100-YR (7.04 inches)
Carroll IFC Gauge	22.2	21.3	15.6	10.5
Coon Rapids IFC Gauge	19.7	17.9	17.3	15.9
Bayard USGS Gauge	17.5	16.1	14.5	12.8
Panora USGS Gauge	17.1	15.7	14.1	12.4
Redfield IFC Gauge (Outlet)	15.2	13.6	11.2	9.0

The maps in Appendix A show the percent reduction in peak flow with ponds, as compared to those without ponds, at the five index locations for the scenarios with small and large ponds. The maps also show each headwater basin and how well the ponds in the basins are utilized. The ponds in the Southern Iowa Drift Plain, south of the Middle Raccoon River main stem utilized their entire flood storage capacity at much smaller storm events; therefore, higher emergency spillways were used in this region.

To illustrate how effectively the ponds utilized their storage in the simulated flood events, the resulting peak discharge and potential stage reductions are shown in Table 7.4. Results are shown for the 10-, 25-, 50-, and 100-year return period 24-hour SCS design storms. For the 10-year return period design flood, the water level reached the emergency spillway elevation for 127 of the 160 (79 percent) of the small ponds (3 foot and 7 emergency spillway elevations). In contrast, the water level reached the emergency spillway for only 50 (31 percent) of the large ponds (5 foot and 10 foot emergency spillway elevations). As a result, nearly all of the flood storage was utilized in a 10-year flood for small ponds, with decreasing utilization for the large ponds. For the 25-year design flood, the water level reached the emergency spillway elevation for 145 of 160 small ponds (91 percent), and 115 of 160 large ponds (72 percent). By the 50-year and 100-year design floods, the water level reached the emergency spillway for the vast majority of all ponds, regardless of size.

Table 7.4: Reductions in stage at the USGS gauge locations due to the reduction in peak discharge for all typical pond scenarios.

Pond Size	Reduction in Stage due to Reduction in Peak Discharge (ft)							
	Bayard, IA USGS Gauge				Panora, IA USGS Gauge			
	10-YR	25-YR	50-YR	100-YR	10-YR	25-YR	50-YR	100-YR
Small	0.7	0.6	0.5	0.4	0.9	0.9	0.9	0.7
Large	1.0	0.9	0.9	0.8	1.2	1.4	1.6	1.4

Dry Pond Results

The same distributed storage analysis was again run for the dry pond design. The dry ponds initially had no stored water since a 2-inch pipe was set at the lowest elevation in the pond. Therefore, dry ponds had a greater amount of total storage which could allow for greater potential peak flow reductions.

Figure 7.9 compares the simulated flood hydrographs for the current no pond condition (Baseline) to those with small dry prototype ponds for the 50-year return period 24-hour design storm (6.00 inches of rain in 24 hours). The smallest drainage area shown, at Carroll, IA, has a drainage area of 73.8 square miles. Twenty-eight prototype ponds were placed upstream. As a result, the peak discharge was reduced by 7 percent. The limited amount of flood storage available in these ponds reduced the peak discharge from 5,733 cfs (with no ponds) to 5,339 cfs (with ponds). In general, the additional storage in the dry ponds amounted to an additional 1 percent reduction in discharge.

Even though the area controlled was very similar throughout the basin, the peak flow reduction was not. At Carroll, where the ponds upstream mostly lie in the Southern Iowa Drift Plain, the peak flow reduction was minimal. The smaller prototype dry ponds in the Southern Iowa Drift Plain filled faster (and therefore engaged the emergency spillway much earlier) than the larger ponds in the Des Moines Lobe. Even though a larger percentage of the watershed at Carroll drained through ponds, it had a smaller percentage of available storage. At Coon Rapids, the next index location downstream, the peak reduction was at a maximum (11 percent). At this location, a larger percentage of ponds upstream lie in the Des Moines Lobe. Even though the area controlled by ponds was very similar downstream, and the mix of ponds from the Southern Iowa Drift Plain and Des Moines Lobe were similar, the peak reduction gradually decreased downstream to the basin outlet at Redfield (6 percent).

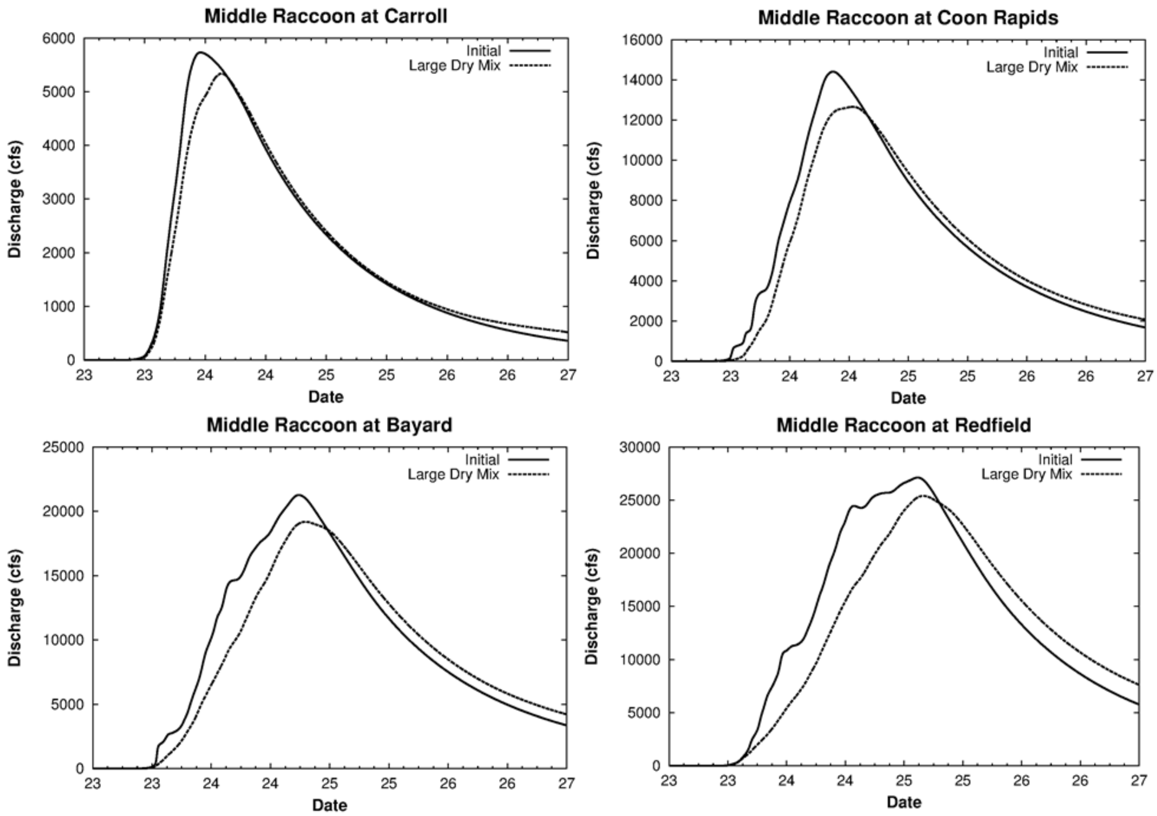

Figure 7.9: Comparisons of hydrographs with and without small dry ponds (8 foot emergency spillway) for the 50-year, 24-hour storm (6.00 inches). For the hydrographs shown, peak flow reduction ranges from 2-9 percent.

Figure 7.10 shows the peak discharge reductions at the two USGS discharge gauge locations and the three IFC discharge gauge locations for the small dry pond scenario (7 foot and 8 foot emergency spillway elevations) for the 50-year, 24-hour event (6.00 inches).

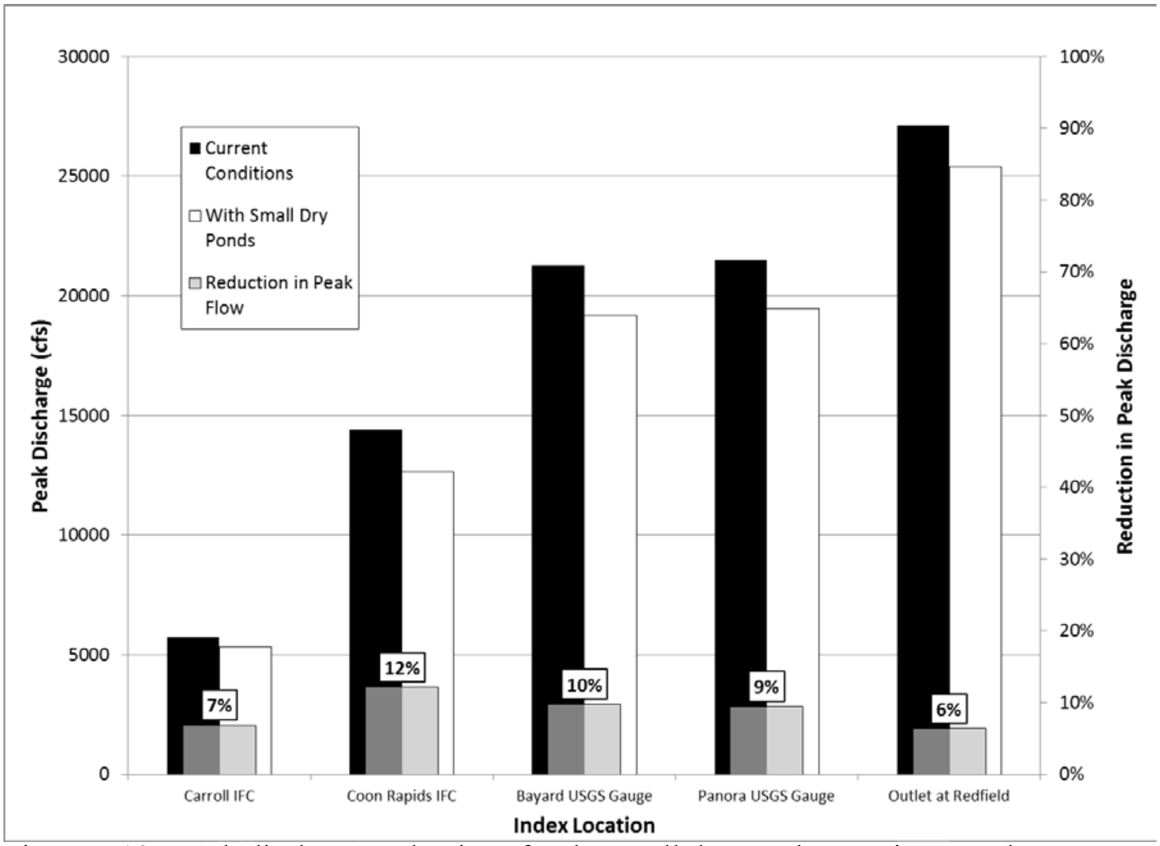

Figure 7.10: Peak discharge reductions for the small dry pond scenario. Results are shown at five index points moving from upstream (left) to downstream (right) for the 50-year, 24-hour design storm (6 inches).

Table 7.5 summarizes the percent reductions in peak discharge for the small dry pond scenario at the five index locations for all the design storm events. In this scenario, each pond in the Southern Iowa Drift Plain provided 23.8 acre-feet of flood storage and each pond in the Des Moines Lobe provided 34.2 acre-feet of flood storage, resulting in a total of 6,051 acre-feet of flood storage for the entire watershed. For the small dry ponds, the percent reduction was greatest for the 10-year return period flood, and decreased for larger floods; the small dry ponds filled rapidly for large floods, at which point little attenuation in flood peak was achieved.

As noted above, the peak reduction effect varied with drainage area. It was typically larger for small drainage areas, where the location was closer to the headwater ponds, and decreased in the downstream direction. The one exception was the IFC gauge location at Carroll where its upstream area was primarily in the Southern Iowa Drift Plains (where ponds are smaller and less flood storage is available). At Coon Rapids it varied from about 11 percent (10-year event) to 9 percent (50-year event), whereas at the downstream-most location of Redfield, it varied from 8 percent (10-year event) all the way to 5 percent (50-year event). The increased storage in the small dry pond scenario increased peak flow reductions by an average of approximately 1 percent.

Table 7.5: Percent reduction in peak discharge using the small dry pond design (7 foot and 8 foot emergency spillway elevations).

Location	Percent Peak Discharge Reduction			
	10–YR (4.03 inches)	25-YR (5.08 inches)	50-YR (6.00 inches)	100-YR (7.04 inches)
Carroll IFC Gauge	18.6	10.1	6.9	5.0
Coon Rapids IFC Gauge	16.8	14.8	12.1	9.5
Bayard USGS Gauge	14.2	11.7	9.8	8.0
Panora USGS Gauge	13.8	11.4	9.5	7.7
Redfield IFC Gauge (Outlet)	11.1	8.1	6.4	5.0

Figure 7.11 compares the simulated flood hydrographs for the current no pond condition (Baseline) to those with large dry prototype ponds for the 50-year return period 24-hour design storm (6.00 inches of rain in 24 hours). At Carroll, IA, peak discharge was reduced by 16 percent. The increased amount of storage going from small dry ponds to large dry ponds decreased peak discharge by an additional 9 percent. These ponds

reduced the peak discharge from 5,733 cfs (with no ponds) to 4,843 cfs (with large dry ponds).

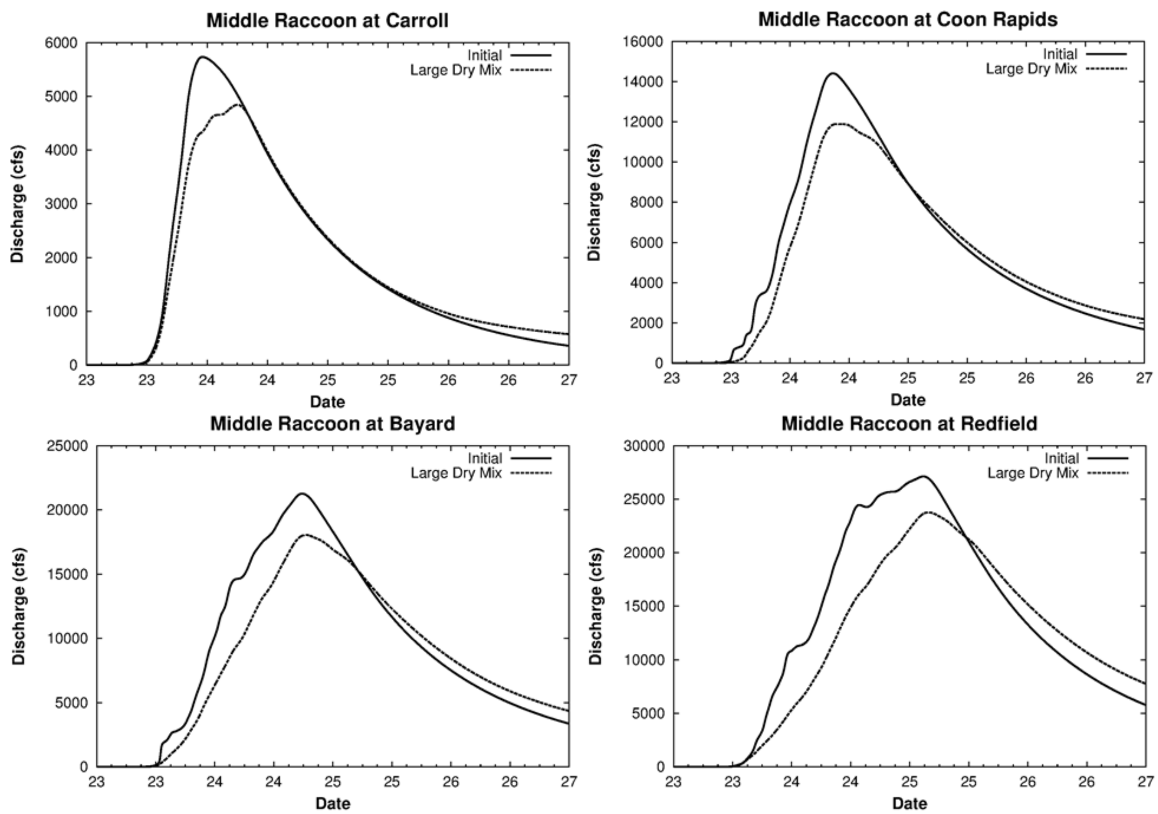

Figure 7.11: Comparisons of hydrographs with and without the large dry ponds (10 foot and 12 foot emergency spillways) for the 50-year, 24-hour storm (6.00 inches). For the hydrographs shown, peak flow reduction ranges from 12-17%

In the case of the large dry pond scenario, flow reductions seemed to be relatively uniform at all of the index locations throughout the Middle Raccoon River main stem. This was especially true for the 10-year and 25-year event. In these scenarios, the ponds upstream of Carroll, IA had sufficient capacity to retain the majority of runoff; this was not true for the other pond designs where discharge reductions at Carroll were minimal. For the 50-year event, percent reductions were more similar to those seen in the smaller pond scenarios with a reduction range from 16 percent at Carroll, IA to 12 percent at the watershed outlet (Redfield, IA).

Figure 7.12 shows the peak discharge reductions at the two USGS discharge gauge locations and the three IFC discharge gauge locations for the large dry pond scenario (10 and 12 foot emergency spillway elevations) for the 50-year, 24-hour event (6.00 inches).

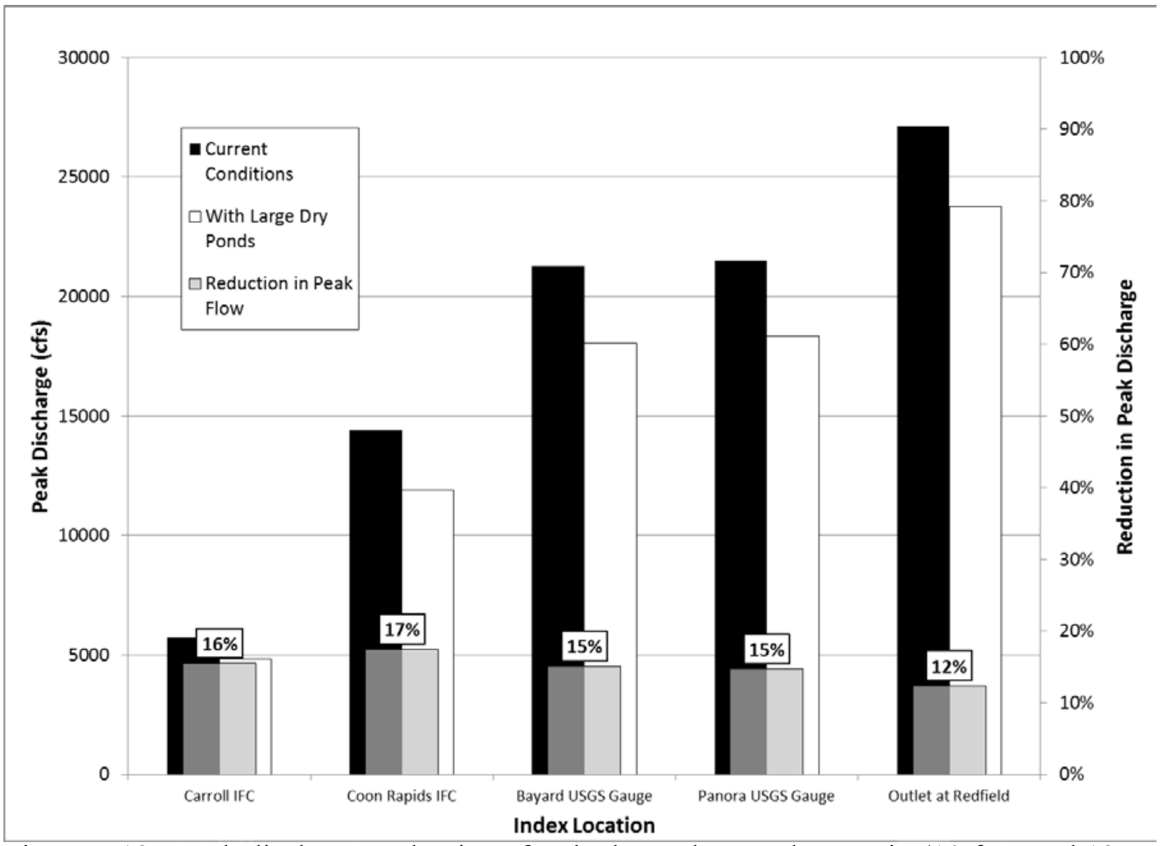

Figure 7.12: Peak discharge reductions for the large dry pond scenario (10 foot and 12 foot emergency spillways). Results are shown at five index points moving from upstream (left) to downstream (right) for the 50-year, 24-hour design storm (6 inches).

Table 7.6 summarizes the percent reductions in peak discharge for the large pond scenario at the five index locations for all the design storm events. In this scenario, each pond provided 38.6 acre-feet of flood storage for the Southern Iowa Drift Plain and 62.8 acre-feet of flood storage in the Des Moines Lobe Region, resulting in a total of 10,765 acre-feet of flood storage for the entire watershed. With this additional flood storage (approximately 1.8 times the small dry pond flood storage), the peak reduction was again increased. Percent reduction in peak flow remained relatively constant for the 10- through 100-year design storm events at the watershed outlet, yet it was at a maximum at the 10-year event (15 percent). This was due to the ponds' utilization of the potential flood storage to its maximum potential (i.e. most ponds were relatively full but not engaging the emergency spillway). As expected, the peak reduction tended to be greater nearer to the headwater ponds (smaller drainage areas), and decreased for larger drainage areas downstream. In this case, Carroll no longer seemed to be the exception, at least in the smaller events. Table 7.6 shows that for the 10-year event, peak discharge reduction was at a maximum (22.6 percent) at Carroll, IA. The topography upstream of Carroll dictated that more elevation was needed between the pond bottom and emergency spillway in order to reduce the flows to the same capacity as the locations downstream.

Table 7. 6: Percent reduction in peak discharge using the large dry pond design (10 foot emergency spillway elevation).

Location	Percent Peak Discharge Reduction			
	10–YR (4.03 inches)	25-YR (5.08 inches)	50-YR (6.00 inches)	100-YR (7.04 inches)
Carroll IFC Gauge	22.6	21.2	15.5	10.6
Coon Rapids IFC Gauge	20.5	18.2	17.5	16.0
Bayard USGS Gauge	17.7	16.2	15.1	13.7
Panora USGS Gauge	17.4	15.8	14.7	13.3
Redfield IFC Gauge (Outlet)	15.0	14.0	12.4	10.2

Maps in Appendix A show the percent reduction in peak flow with dry ponds, as compared to that without ponds, at the five index locations for the scenarios with small, and large dry ponds.

To illustrate how effectively the dry ponds utilize their storage in the simulated flood events, the resulting peak discharge and potential stage reductions are shown in Table 7.7. Results are shown for the 10-, 25-, 50-, and 100-year return period 24-hour SCS design storms.

Table 7. 7: Reductions in stage at the USGS gauge locations due to the reduction in peak discharge for all dry pond scenarios.

Pond Size	Reduction in Stage due to Reduction in Peak Discharge (ft)							
	Bayard, IA USGS Gauge				Panora, IA USGS Gauge			
	10-YR	25-YR	50-YR	100-YR	10-YR	25-YR	50-YR	100-YR
Small	0.8	0.7	0.6	0.5	1.0	1.0	1.0	0.8
Large	1.0	1.0	0.9	0.9	1.2	1.4	1.6	1.5

For the 10-year return period design flood, the water level reached the emergency spillway elevation for 104 of the 160 (65 percent) of the small dry ponds (7 and 8 foot emergency spillway elevations). In contrast, the water level reached the emergency spillway for only 32 (20 percent) of the large dry ponds (10 foot emergency spillway elevations). As a result, the dry ponds did a slightly better job than the typical ponds in utilizing flood storage. The majority of ponds did not activate the emergency spillway for the large dry ponds. For the 25-year design flood, the water level reached the emergency spillway elevation for 144 of 160 small dry ponds (90 percent), and 93 of 160 large dry ponds (58 percent). In the 25-year design flood, a large increase was seen in the activation of the emergency spillways for the small dry ponds (65 percent to 90 percent). The small dry ponds were, therefore, most impactful for the 10-year event. For the 50-year design flood, the water level reached the emergency spillway for 145 of 160 small dry ponds (91 percent) and 130 of 160 large dry ponds (81 percent). Lastly, for the 100-year design flood, the water level reached the emergency spillway for 148 of 160 small dry ponds (92 percent) and 144 of 160 large dry ponds (90 percent). Based on when the majority of the emergency spillways were activated, in the most general sense, these results would indicate that the small dry ponds are best designed for approximately a 10-year event, and the large dry ponds are best designed for between a 10-year and 25-year event.

Summary of Flood Mitigation Techniques Using Flood Storage

In this chapter, we analyzed the effect of distributed storage on peak discharge at five index points in the Middle Raccoon River watershed. Two pond designs were implemented and simulated—the typical pond design and the dry pond design. The typical design was assumed to have a permanent pool elevation of five feet. This allows the pond to be used for activities such as watering animals or irrigation. The dry pond design was assumed to have no permanent pool, meaning an additional outlet was placed at the pond bottom. This allows for more flood storage, but doesn't allow for other uses. Both designs were shown to have an impact on peak discharge reduction. On the scale of the entire watershed (590 mi^2), the dry ponds generally outperformed the typical ponds on the scale of one to two percent, regardless of the size of the storm.

Both pond designs reduced peak flooding when compared with the current conditions simulation. The reductions ranged from 4 percent (typical small pond design, 100-year at Carroll, IA) to 23 percent (large dry pond design, 10-year at Carroll, IA). The maximum percent peak discharge reduction at the watershed outlet (Redfield, IA) was 15.0 percent, for the 10-year event using both the large dry pond design and large typical pond design. Max peak discharge reduction for each design pond scenario was dependent upon which storm event most efficiently utilized the ponds' flood storage. Reductions were greatest at the 10-year event for both small ponds, and the 10-year event for the large typical ponds, the 10-year event for the small dry ponds, and the 25-year event for the large dry ponds. Reduction in peak flow resulted in stage reductions; this ranged from 0.4 feet for the small typical pond design during the 100-year event to 1.6 feet for the large dry pond design for the 50-year event. An additional benefit from the use of distributed storage was the potential delay in peak flows. For the pond designs, peak flow delay ranged from a few minutes (10-year, 24-hour event at Carroll, IA) to approximately 7 hours (50-year, 24-hour event at Carroll, IA). This delay could allow for advanced evacuation warnings and might allow people to remove belongings from the floodplain, thereby limiting a flood's damage potential. Distributed storage in the Middle Raccoon River watershed does have a positive effect on peak discharge reduction and could be successful as part of a flood mitigation strategy.

CHAPTER 8: MITIGATING THE EFFECTS OF HIGH RUNOFF WITH INCREASED INFILTRATION AND DISTRIBUTED FLOOD STORAGE

Chapters 6 and 7 described how increased infiltration and distributed storage independently affect peak discharge. Both were shown to decrease peak discharge, but they used entirely different mechanisms to do so. Increased infiltration reduced runoff volume by removing runoff from the system and allowing more water to infiltrate the subsurface. Distributed storage did not reduce runoff, but rather stored excess runoff in ponds, which could then be released in a slower manner.

In this chapter, we used the HMS model to simulate the effect on peak discharge using a combination of both methods. For this hypothetical example, we attempted to create a feasible watershed flood mitigation strategy. This was accomplished by applying the cover crop scenario from Chapter 6 to the small and large, typical pond design from Chapter 7. This example is not a proposed flood mitigation plan; it is only meant to provide quantitative knowledge on how a realistic combination of flood control measures could affect peak discharge.

Prototype Storage: Pond Design

For this scenario, we used the small and large typical pond design seen in Chapter 7. This design assumes a principal spillway elevation of 5 feet above the pond bottom. Therefore, during normal circumstances, the pond holds water up to this permanent pool elevation. The elevation of the emergency spillway was fixed at 7 or 10 feet above the principal in the Southern Iowa Drift Plain and 3 or 5 feet in the Des Moines Lobe. The volume between this principal and emergency spillway was the flood storage which was available during high runoff events. Figure 7.1 in the previous chapter shows the typical pond design used in the scenario.

The typical pond design was chosen for this scenario due to its high impact on peak discharge reduction (up to 22 percent) and its ability to hold water under normal flow conditions. This is desirable for land owners in the watershed, as it can be used for farming or recreational purposes. The dry pond scenario, while it has greater flow reduction potential, does not provide the additional uses of the typical pond design. Methods and procedures for applying the ponds in HMS are identical to those in Chapter 7; pond design and siting are also identical.

Increased Infiltration: Cover Crops

Similar to the method in Chapter 6, every subbasin was assumed to have 100 percent of its agricultural land converted to agricultural land with the cover crop conservation practice. The increase in infiltration associated with cover crops was modeled in HMS by a basin wide decrease in the Curve Number. Methods and procedures for applying cover crops in HMS are identical to those in Chapter 6.

Blended Pond Simulations

The HMS model was run with the ponds and cover crops to simulate the effects of the combination of flood mitigation methods on peak discharges. Two models were created for this scenario. The first blended the small typical pond design with the lowered cover crop practices curve numbers; the second blended the large typical pond design with the lowered cover crop practices curve numbers. Only the typical pond designs were used due to the assumption that they would be desirable for landowners, and therefore, would be a more realistic scenario. For the locations of the ponds, each simulation started with all pond water levels at the principal spillway elevation; this assumed that the permanent storage was completely utilized as the storm began. Comparisons were then made for the simulated flows without the blended flood mitigation practices in place (the existing baseline condition). Flood hydrographs were compared for the 10-, 25-, 50-, and 100-year return period, 24-hour SCS design storms.

Blended Practices Results

Figure 8.1 compares the simulated flood hydrographs for the current no pond condition (Baseline) to those with the application of small blended flood mitigation practices for the 50-year return period, 24-hour design storm (6.00 inches of rain in 24 hours). The simulations used the small typical pond design specified in Chapter 7, where the emergency spillway is set at 7 feet above the principal spillway in the Southern Iowa Drift Plain and 3 feet above the principal spillway in the Des Moines Lobe. The smallest drainage area, shown at Carroll, IA, has a drainage area of 73.8 mi^2. There, the peak discharge was reduced by 17 percent. Ponds upstream of Carroll, IA were smaller due to the Southern Iowa Drift Plain topography; therefore, reductions there were not as significant as the downstream index locations. The results showed fairly uniform reductions of flows, ranging from 14 percent (at Redfield, IA) to over 21 percent (at Coon Rapids, IA). Figure 8.2 summarizes the peak discharge for current conditions, the peak discharge for the small blended practices scenario, and the percent peak reduction, at all five index locations for the 50-year, 24-hour design storm event.

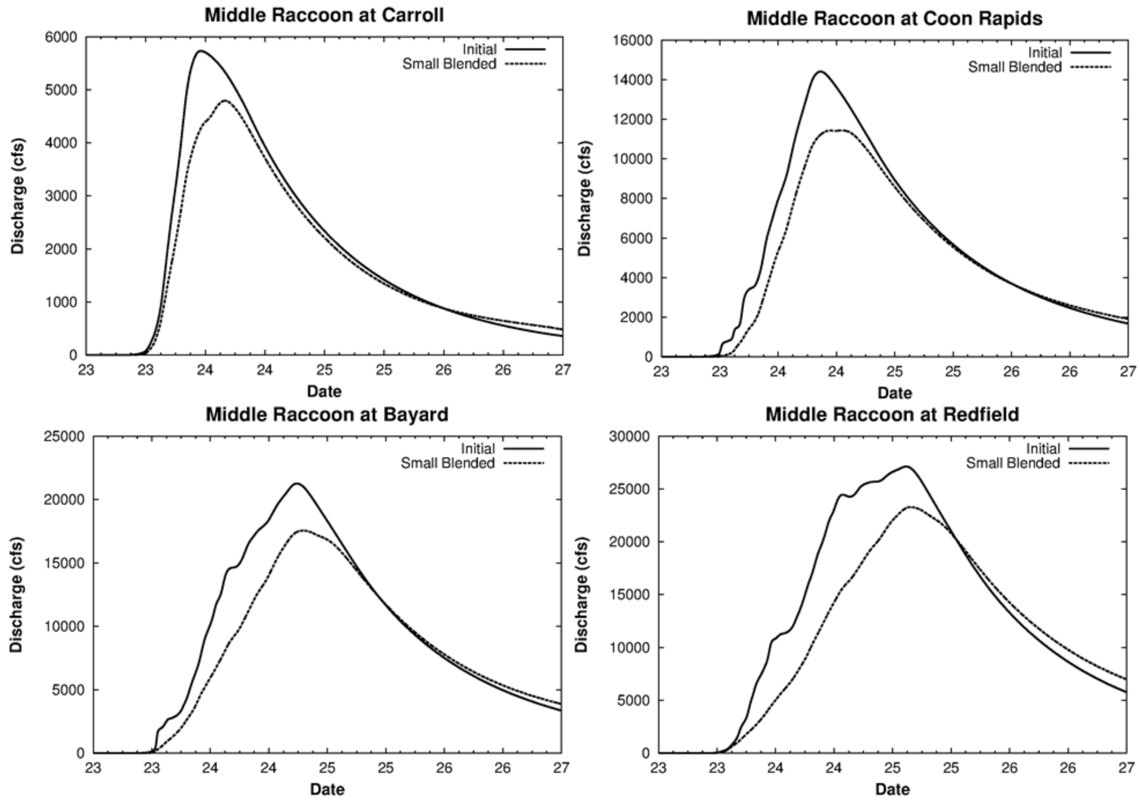

Figure 8.1: Comparisons of hydrographs with and without the small blended scenario for the 50-year, 24-hour storm (6.00 inches). For the hydrographs shown, peak flow reduction ranges from 14-21 percent.

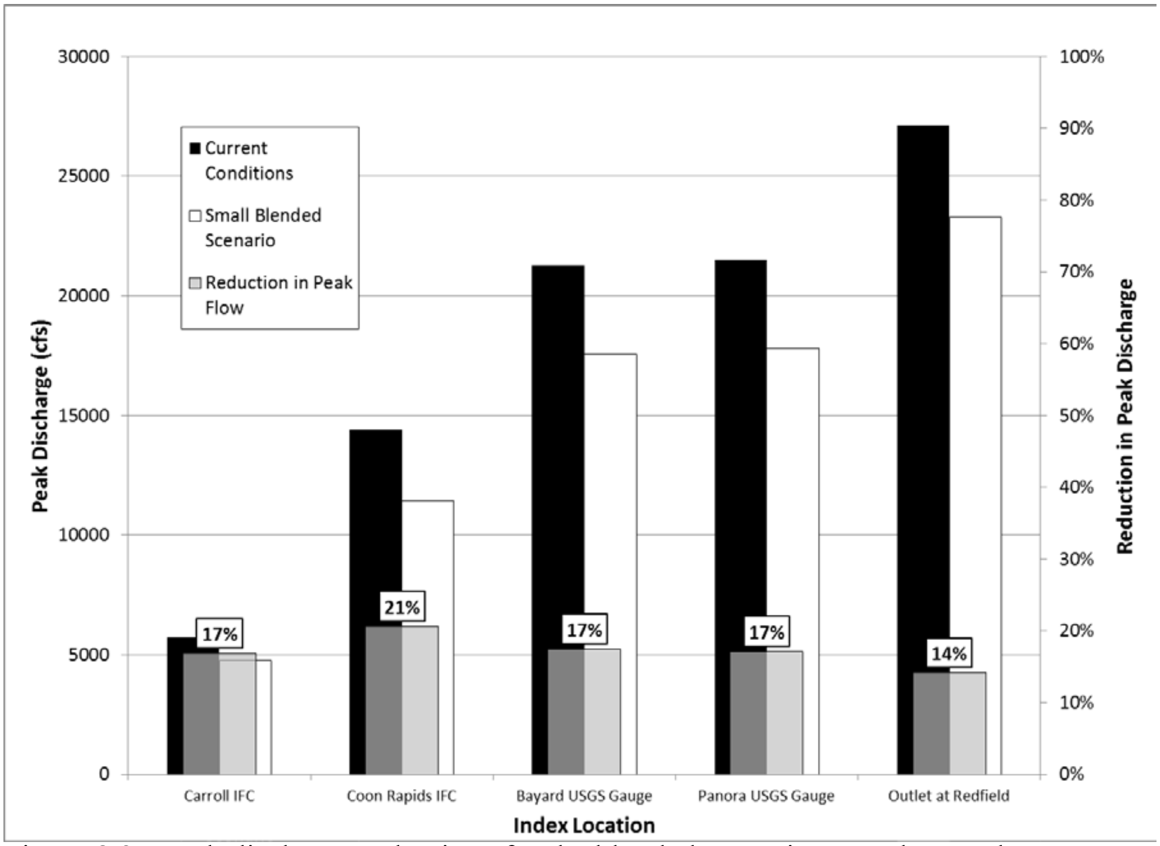

Figure 8.2: Peak discharge reductions for the blended scenario. Results are shown at five index points moving from upstream (left) to downstream (right) for the 50-year, 24-hour design storm (6 inches)

Table 8.1 summarizes the percent reduction in peak discharge resulting from this hypothetical small blended scenario at the five index location for all the design storm events. The small blended scenario resulted in peak discharge reduction between 12 and 29 percent. For this scenario, the maximum flow reductions were found during the 10-year rainfall event. At this event, the ponds utilized their storage most efficiently. At the larger scaled drainage areas, the effects of the cover crop seem to dominate. Therefore, based on the results in Chapter 6, we expect increased infiltration to have its greatest impact during small design storms where the percentage of rainfall infiltrated is the greatest. The reduction in peak flow was relatively uniform at all locations for each event.

Table 8.1: Percent reduction in peak discharge using the small blended scenario.

Index Location	Percent Peak Discharge Reduction Based on Storm Return Period (%)			
	10-YR (4.03 in)	25-YR (5.08 in)	50-YR (6.00 in)	100-YR (7.04 in)
Carroll IFC Gauge	29.3	21.9	16.9	12.7
Coon Rapids IFC Gauge	26.5	23.7	20.6	16.7
Bayard USGS Gauge	24.2	20.2	14.5	14.8
Panora USGS Gauge	23.8	19.8	17.1	14.4
Redfield IFC Gauge (Outlet)	21.5	16.9	14.2	11.8

Reducing peak flood discharge also reduced the peak water height (or stage) in a river during the flood. For the peak discharge reductions in the small blended flood mitigation practices, the corresponding reduction in flood stage was between 1.0 and 1.9 feet. This reduction was estimated for the USGS stream-gauge locations, where the rating curves have been developed. Although a 1.0 to 1.9 foot reduction in flood stage would slightly reduce the flood inundation area, flooding still occurs. Again, based on the flood stage level reported by the National Weather Service at Bayard, IA, water levels above action stage (13 feet) are expected for both the current conditions and small blended scenario. Hence, the addition of cover crops does not eliminate flooding, but would reduce its severity and frequency.

Figure 8.3 compares the simulated flood hydrographs for the current no pond condition (Baseline) to those with the application of large blended flood mitigation practices for the 50-year return period, 24-hour design storm (6.00 inches of rain in 24 hours). The simulations used the small typical pond design specified in Chapter 7, where the emergency spillway is set at 10 feet above the principal spillway in the Southern Iowa

Drift Plain and 5 feet above the principal spillway in the Des Moines Lobe. The smallest drainage area shown at Carroll, IA has a drainage area of 73.8 mi². There, the peak discharge was reduced by 26 percent. In this scenario, the ponds upstream of Carroll, IA were much larger than in the small blended practices scenario; therefore, reductions there were at a maximum, while in the small blended scenario, they were not. The results showed fairly uniform reductions of flows, ranging from 20 percent (at Redfield, IA) to 26 percent (at Carroll, IA). Figure 8.4 summarizes the peak discharge for current conditions, the peak discharge for the large blended practices scenario, and the percent peak reduction, at all five index locations for the 50-year, 24-hour design storm event.

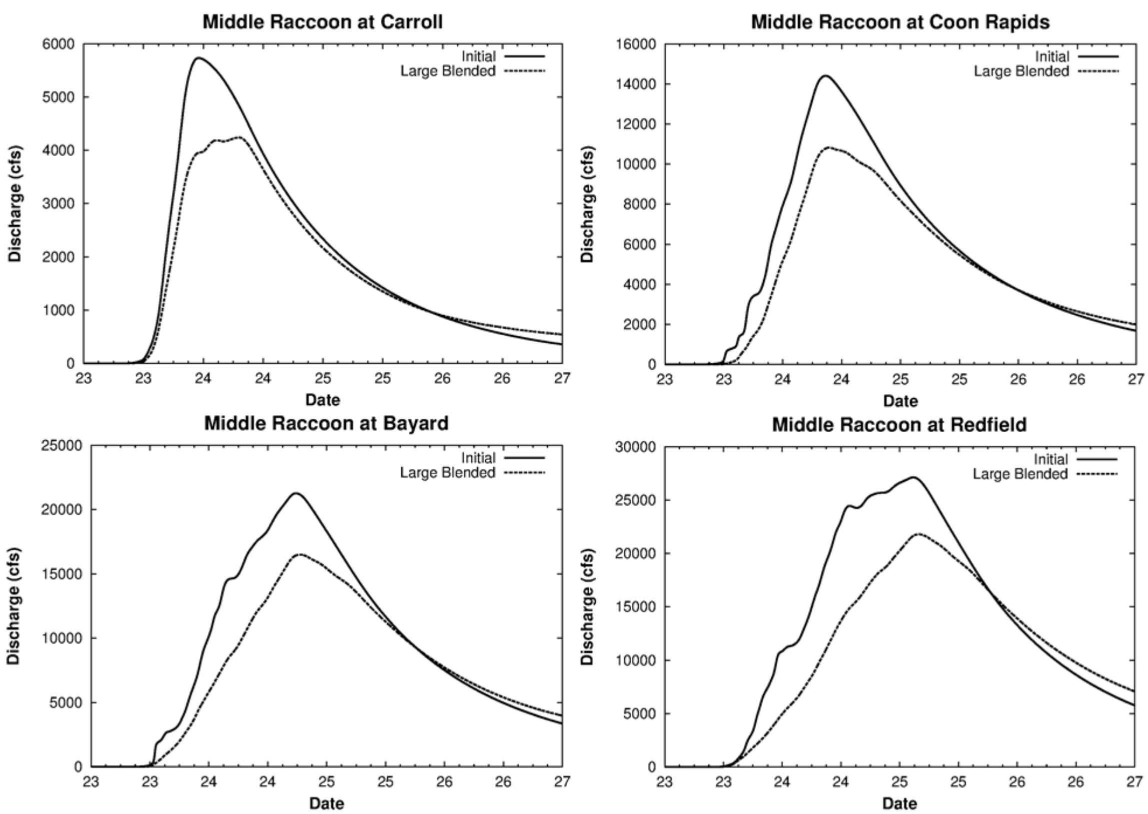

Figure 8.3: Comparisons of hydrographs with and without the blended scenario for the 50-year, 24-hour storm (6.00 inches). For the hydrographs shown, peak flow reduction ranges from 20-26 percent.

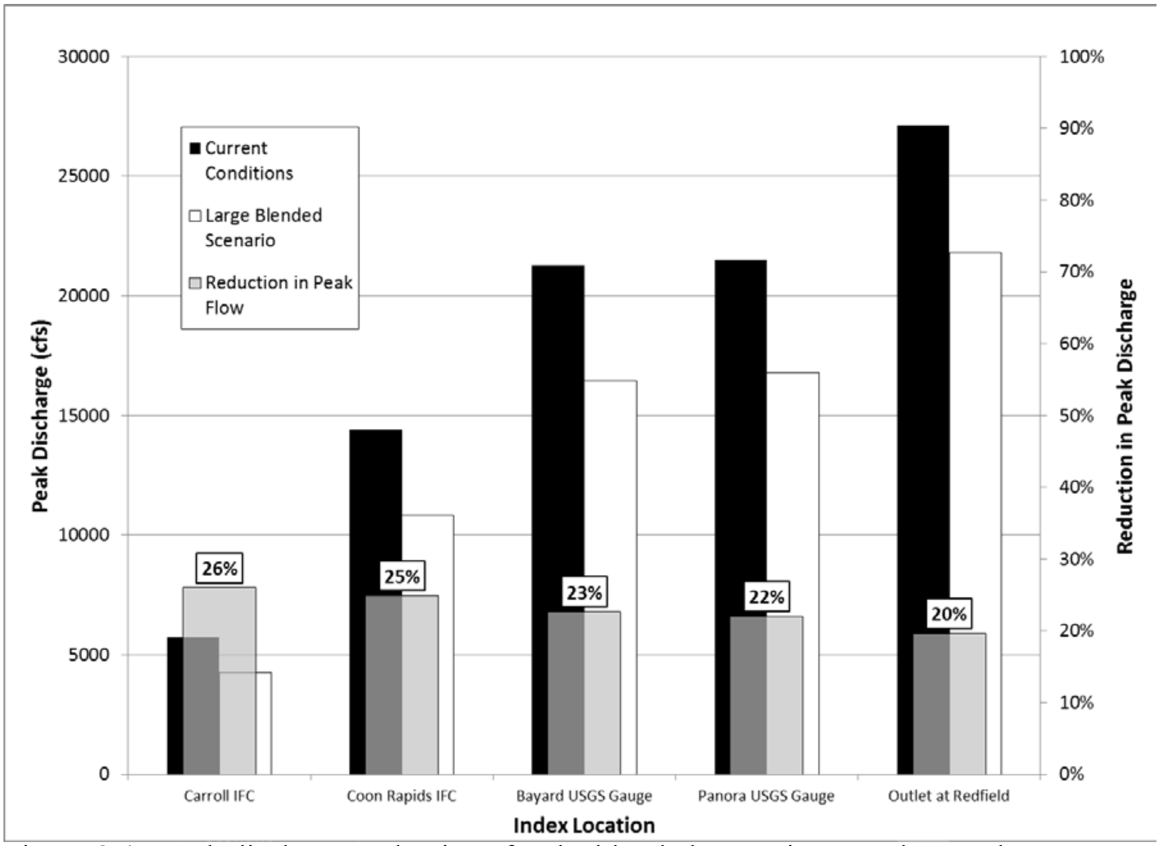

Figure 8.4: Peak discharge reductions for the blended scenario. Results are shown at five index points moving from upstream (left) to downstream (right) for the 50-year, 24-hour design storm (6 inches)

Table 8.2 summarizes the percent reduction in peak discharge resulting from this hypothetical large blended scenario at the five index location for all the design storm events. The large blended scenario resulted in peak discharge reduction between 16 and 31 percent. For this scenario, the maximum flow reductions were found during the 10-year rainfall event. During this event, the ponds utilize their storage most efficiently. At the larger scaled drainage areas, the effects of the cover crop seem to dominate. Therefore, based on the results in Chapter 6, we expect increased infiltration to have its greatest impact during small design storms where the percentage of rainfall infiltrated is the greatest. The reduction in peak flow was relatively uniform at all locations for each event.

Table 8.2: Percent reduction in peak discharge using the large blended scenario.

Index Location	Percent Peak Discharge Reduction Based on Storm Return Period (%)			
	10-YR (4.03 in)	25-YR (5.08 in)	50-YR (6.00 in)	100-YR (7.04 in)
Carroll IFC Gauge	31.0	30.6	26.0	19.1
Coon Rapids IFC Gauge	29.0	26.7	24.9	23.1
Bayard USGS Gauge	26.7	25.1	22.6	19.9
Panora USGS Gauge	26.3	24.7	22.0	19.5
Redfield IFC Gauge (Outlet)	24.5	23.1	19.6	16.4

Reducing peak flood discharge also reduces the peak water height (or stage) in a river during the flood. For the peak discharge reductions in the large blended flood mitigation practices, the corresponding reduction in flood stage was between 1.3 and 2.5 feet. This reduction was estimated for the USGS stream-gauge locations, where the rating curves have been developed. Although a 1.5 to 2.5 foot reduction in flood stage would slightly reduce the flood inundation area, flooding still occurs. Again, based on the flood stage level reported by the National Weather Service at Bayard, IA, water levels above action stage (13 feet) are expected for both the current conditions and the small blended scenario. Hence, the addition of cover crops and large ponds does not eliminate flooding, but would reduce its severity and frequency.

Summary of Flood Mitigation Using Increased Infiltration and Distributed Storage

In this chapter, two scenarios were analyzed to represent a feasible mix of increased infiltration and distributed storage as flood control measures. The scenarios were created using the headwater subbasins defined in Chapter 7 and applying either the large or small typical ponds. In addition to ponds, cover crops were applied, as defined in Chapter 6. In HEC-HMS, the ponds are represented by a storage-discharge table that is reflective of the topographic conditions and pond hydraulics. The cover crops are represented with a reduced Curve Number which allow for more infiltration and less runoff. Since two practices were applied in these scenarios, peak flow reductions were shown to increase when compared to using a single practice. A maximum reduction of 31 percent was observed at Coon Rapids, IA during the 10-year SCS storm event and at Carroll, IA during the 25-year SCS storm event. Reductions were relatively uniform for all locations throughout the watershed, and relatively uniform for every storm event analyzed. From the analysis completed in Chapters 6 and 7, we know that increased infiltration is most effective at reducing peak discharge at small scale storm events, while distributed storage can be most effective at larger events (if the correct pond sizes are selected). Therefore, the blended scenario with cover crops and large typical ponds maximizes benefit at both the small and large design storms. This may be why we see the relatively uniform reduction in discharge.

The reductions seen in this scenario may best represent reductions that could be expected should the watershed adopt a basin wide flood mitigation strategy. Table 8.3 compares the reductions between the typical ponds only, the cover crops only, and the blended scenarios at the watershed outlet (Redfield, IA).

Table 8.3: Comparison of peak discharge reductions between distributed storage, cover crops, and blended scenarios.

Scenario	Percent Peak Discharge Reduction Based on Storm Return Period, at Redfield, IA (%)			
	10-YR (4.03 in)	25-YR (5.08 in)	50-YR (6.00 in)	100-YR (7.04 in)
Small Typical Pond Design Only	9.8	7.0	5.6	4.3
Large Typical Pond Design Only	15.2	13.6	11.2	9.0
Cover Crop Application Only	11.5	9.2	7.5	6.8
Small Blended Scenario	21.5	16.9	14.2	11.8
Large Blended Scenario	24.5	23.1	19.6	16.4

CHAPTER 9: APPLICATION OF FLOOD MITIGATION PRACTICES ON HISTORIC RAINFALL EVENTS

The application of design storms, which apply uniform rainfall over the entire watershed, provided great value in predicting the effects of flood mitigation practices for the entire basin. Design storms are also easily applied for comparative analysis. However, it is unlikely that the Middle Raccoon River watershed would ever receive a uniform depth of rainfall along the entire 590 mi^2 region. For this reason, we examined the effects of the large typical pond application, discussed in Chapter 7, and the large blended cover crops and typical pond application, discussed in Chapter 8, on two historic rainfall events—the storms of June 2008 and June 2013. These two storms were chosen based upon their use in the calibration phase of modeling effort, for the large nature of the events, and because they occurred within the last decade. The recent time frame means the effects of these flooding events can still be easily remembered. Applying flood mitigation practices to real storm events, not only allows watershed stakeholders to better visualize the possible flood reduction benefit, but also provides insight as to how the practices would perform under a most probable non-uniform rainfall.

June 2008

The storm of June 8-12, 2008 was characterized by heavy rainfalls falling primarily in the northwest and southeast corners of the watershed. In these locations, rainfall totals reached approximately 4 inches. Lighter rainfalls fell in the central portion of the basin, averaging approximately 1 to 2 inches. Figure 9.1 shows the spatial variation of rainfall for this event, where green represents rainfalls of between 3 and 5 inches and blue represents rainfalls of between 1 and 3 inches. The combination of heavy rains and wet antecedent moisture conditions resulted in a modeled peak discharge of 8,806 cfs at the USGS gauge located in Bayard, IA.

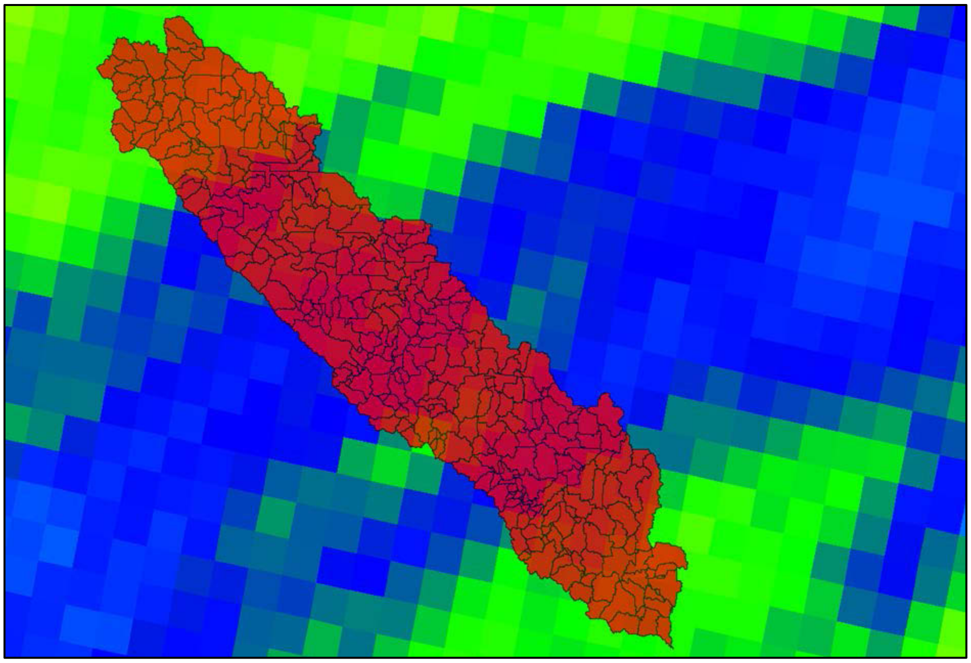

Figure 9. 1: Spatial distribution of rainfall for the June 5-9, 2008 rainfall event

Figure 9.2 compares the simulated flood hydrographs for the current no pond condition (Baseline) to those with the application of the large typical pond flood mitigation practices for the June 2008 rainfall event. The simulations used the large typical pond design specified in Chapter 7, where the emergency spillway is set at 10 feet above the principal spillway in the Southern Iowa Drift Plain and 5 feet above the principal spillway in the Des Moines Lobe. The smallest drainage area shown at Carroll, IA has a drainage area of 73.8 mi². There, the peak discharge was reduced by 21 percent. Ponds upstream of Carroll, IA were more fully utilized than the ponds located downstream due to the heavier rainfalls experienced in the northwest corner of the watershed. For this reason, reductions at the Carroll, IA location were at a maximum for the watershed during this event. Downstream, the results showed steadily decreasing reductions of flows, ranging from 14 percent (at Redfield, IA) to 19 percent (at Coon Rapids, IA). Figure 9.3 summarizes the peak discharge for current conditions, the peak discharge for the large typical pond scenario, and the percent peak reduction, at all five index locations for the June 2008 rainfall event.

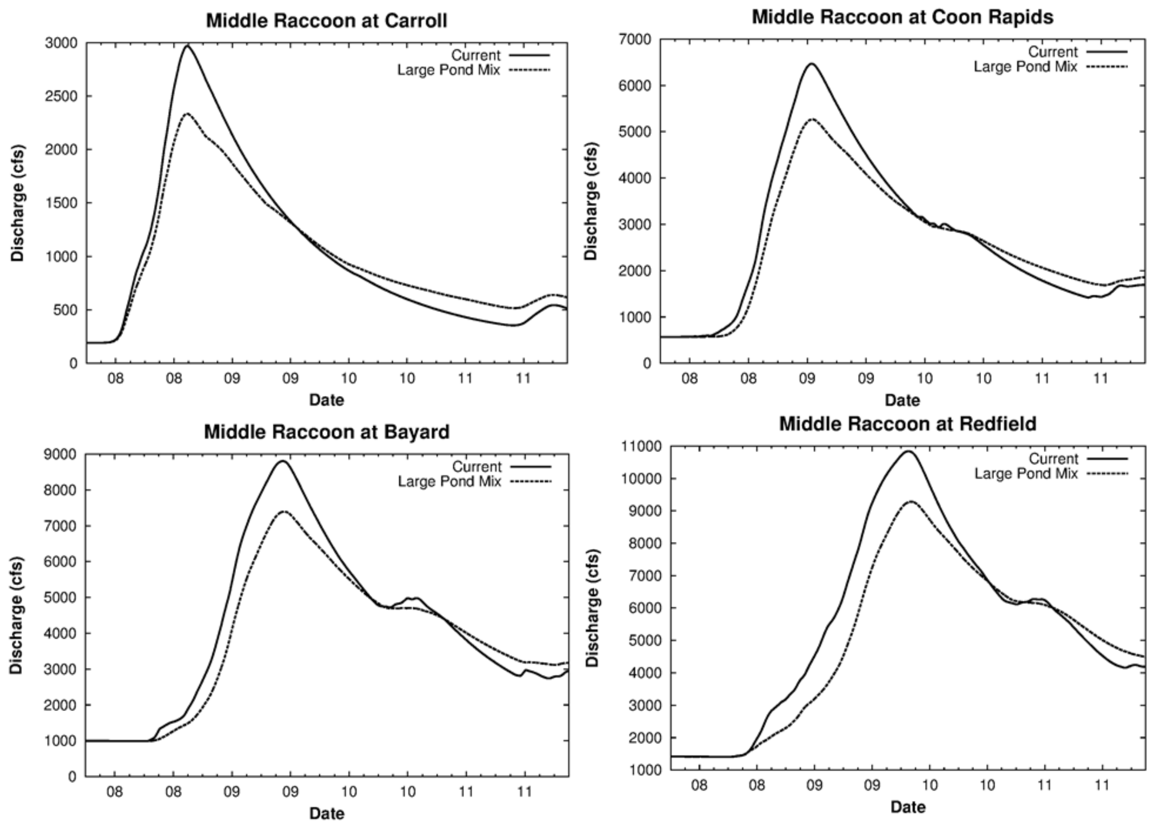

Figure 9. 2: Comparisons of hydrographs with and without the large typical pod scenario for the June 2008 rainfall event. For the hydrographs shown, peak flow reduction ranges from 14-21 percent.

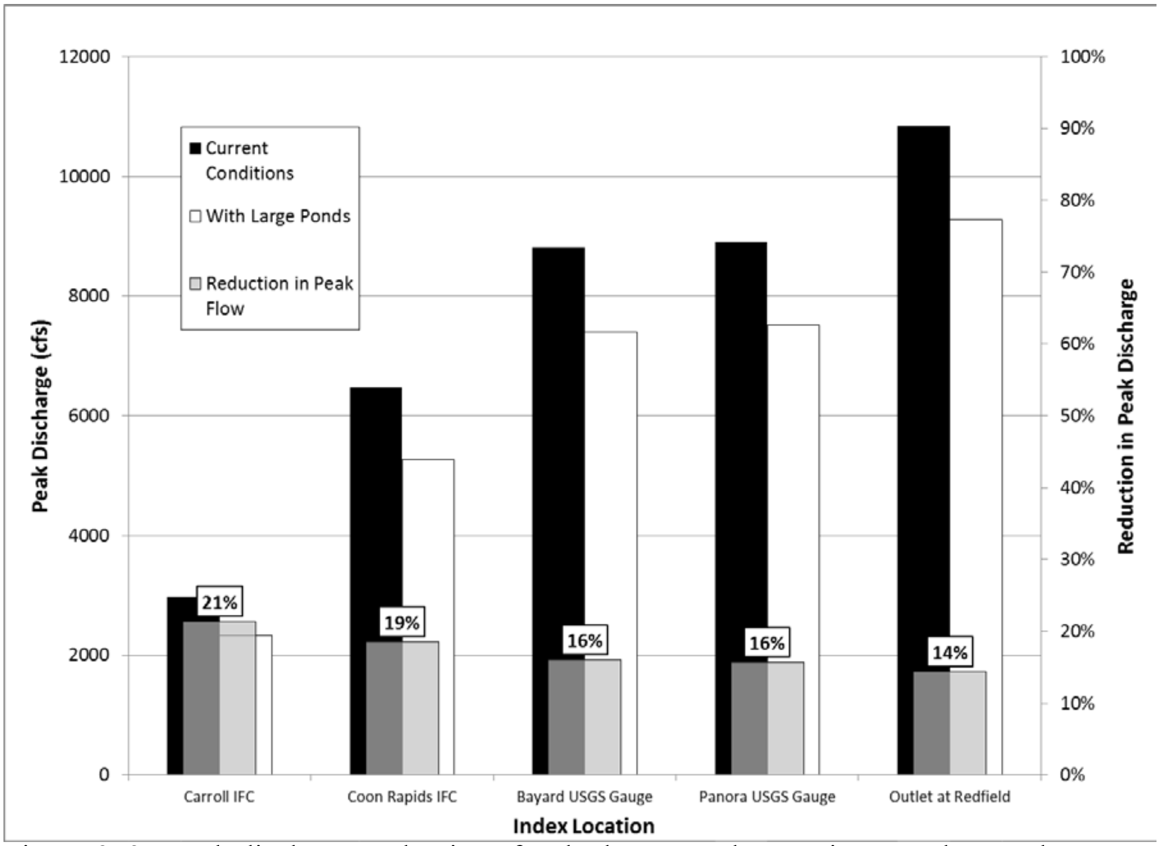

Figure 9. 3: Peak discharge reductions for the large pond scenario. Results are shown at five index points moving from upstream (left) to downstream (right) for the June 2008 rainfall event

Reducing peak flood discharge also reduces the peak water height (or stage) in a river during the flood. For the peak discharge reductions in the large pond flood mitigation practice, the corresponding reduction in flood stage was 0.8 feet at the USGS gauge location at Bayard, IA and 0.9 feet for the USGS gauge at Panora, IA. This reduction was estimated for the USGS stream-gauge locations, where the rating curves have been developed. Although a 0.8 and 0.9 feet reductions in flood stage would slightly reduce the flood inundation area, flooding still occurs. Based on the flood stage level reported by the National Weather Service at Bayard, IA, water levels above action stage (13 feet) are expected for both the current conditions and the large pond scenario. The addition of the large pond flood mitigation practice would not have eliminated the flood of 2008 in the Middle Raccoon River watershed. However, it would be reasonable to assume that a reduction in flood stage of approximately 1 foot may be capable of protecting some homes and properties which were inundated during this event.

Figure 9.4 compares the simulated flood hydrographs for the current no pond condition (Baseline) to those with the application of the large typical pond and cover crops flood mitigation practices for the June 2008 rainfall event. The simulations used the large typical pond design and cover crops specified in Chapter 8, where the emergency spillway is set at 10 feet above the principal spillway in the Southern Iowa Drift Plain and 5 feet above the principal spillway in the Des Moines Lobe. Cover crops were applied to every agricultural acre. The smallest drainage area shown at Carroll, IA has a drainage area of 73.8 mi^2. There, the peak discharge was reduced by 31 percent. Ponds upstream of Carroll, IA were more fully utilized than the ponds located downstream due to the heavier rainfalls experienced in the northwest corner of the watershed. For this reason, reductions at the Carroll, IA location were at a maximum for the watershed during this event. Downstream, the results showed steadily decreasing reductions of flows, ranging from 26 percent (at Redfield, IA) to 29 percent (at Coon Rapids, IA). Compared to the June 2008 storm using large ponds alone, the addition of cover crops created more uniform reductions in flow from the upstream most to downstream most index location. In general, the addition of cover crops reduced peak flows by an additional 10 percent when compared with large typical ponds alone. Figure 9.5 summarizes the peak discharge for current conditions, the peak discharge for the large typical pond with cover crops scenario, and the percent peak reduction, at all five index locations for the June 2008 rainfall event.

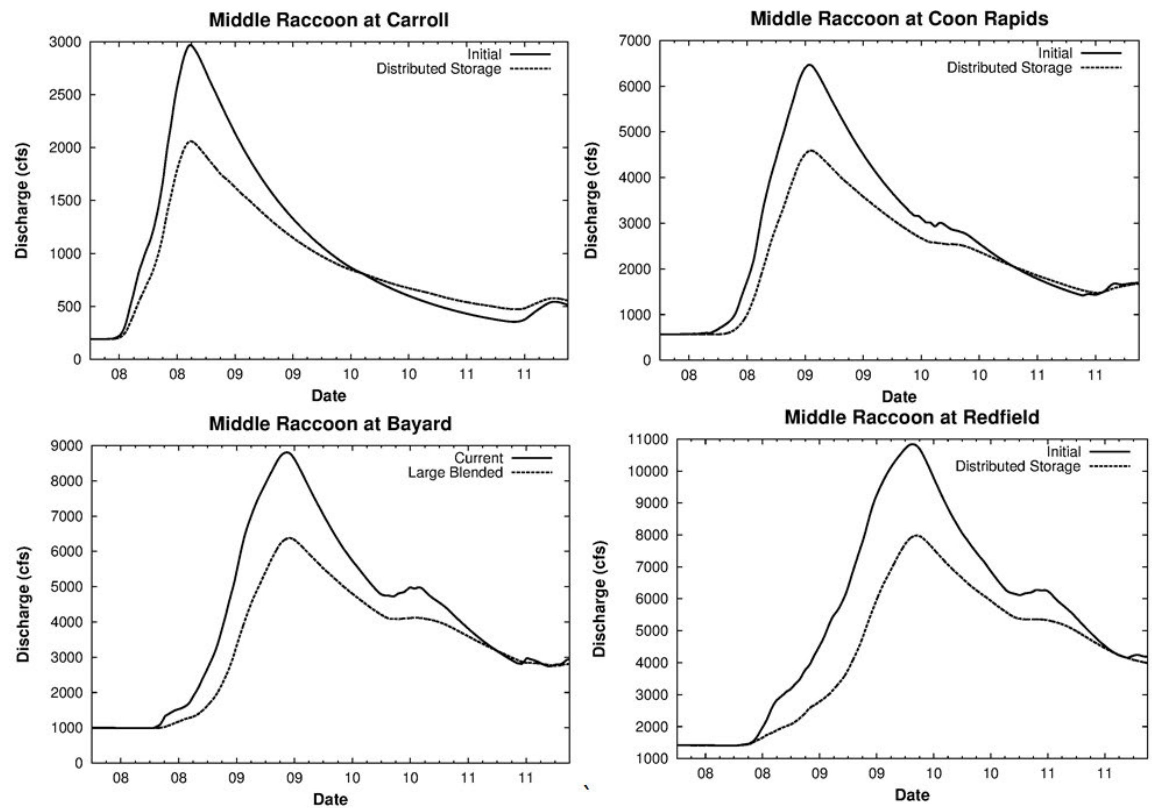

Figure 9. 4: Comparisons of hydrographs with and without the large blended scenario for the June 2008 rainfall event. For the hydrographs shown, peak flow reduction ranges from 26-31 percent.

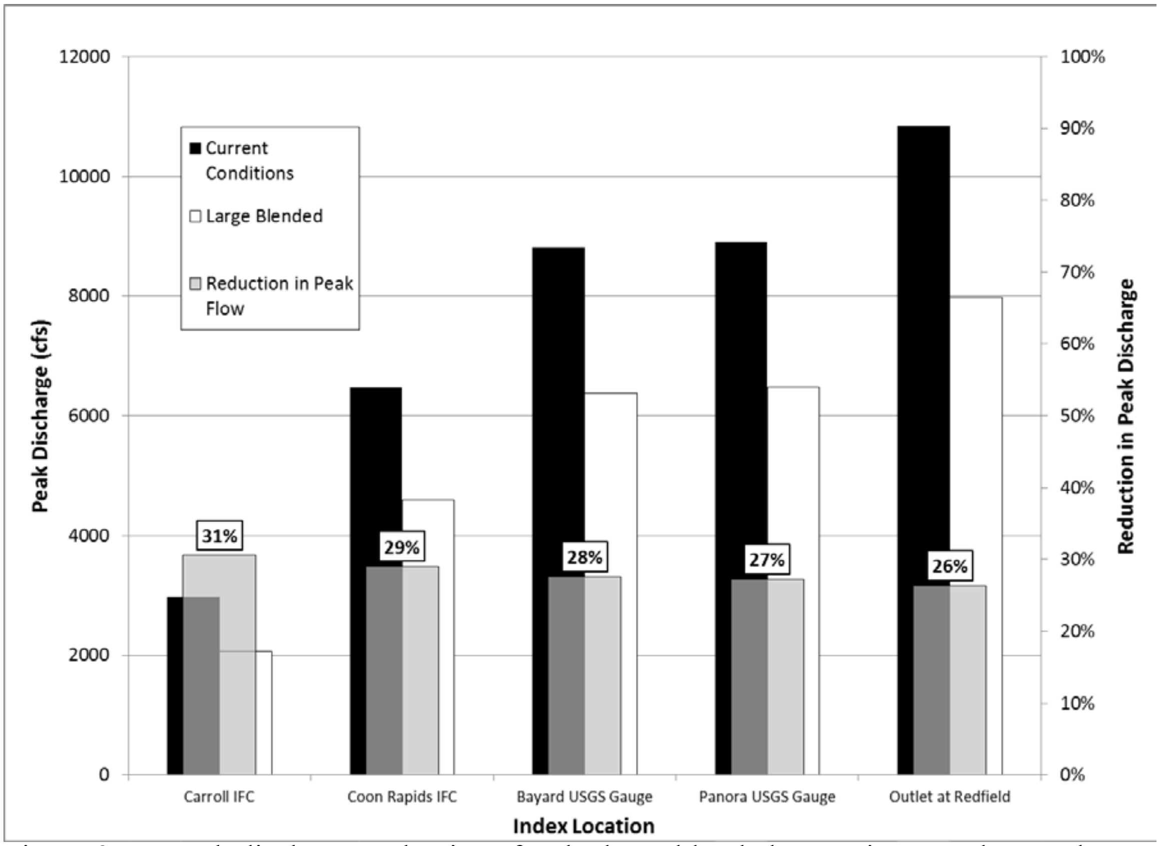

Figure 9. 5: Peak discharge reductions for the large blended scenario. Results are shown at five index points moving from upstream (left) to downstream (right) for the June 2008 rainfall event

For the peak discharge reductions in the large blended flood mitigation practice, the corresponding reduction in flood stage was 1.5 feet at the USGS gauge location at Bayard, IA and 1.7 feet for the USGS gauge at Panora, IA. This reduction was estimated for the USGS stream-gauge locations, where the rating curves have been developed. Although a 1.5 and 1.7 feet reductions in flood stage would slightly reduce the flood inundation area, flooding still occurs. Again, based on the flood stage level reported by the National Weather Service at Bayard, IA, water levels above action stage (13 feet) are expected for both the current conditions and the large pond scenario. The addition of the large blended flood mitigation practice would not have eliminated the flood of 2008 in the Middle Raccoon River watershed. However, it would be reasonable to assume that a reduction in flood stage of approximately 1.5 feet may be capable of protecting some homes and properties which were inundated during this event.

June 2013

The storm of June 13-17, 2013 was characterized by heavy rainfalls falling primarily in the center of the watershed. In this location, rainfall totals reached nearly 6 inches. Lighter rainfalls fell in the northern and southern portion of the basin, averaging approximately 1 to 4 inches in these locations. Figure 9.6 shows the spatial variation of rainfall for this event, where yellow represent rainfalls of greater than 6 inches, green represents rainfalls of between 3 and 5 inches, and blue represents rainfalls of between 1 and 3 inches. The heavy rains resulted in a modeled peak discharge of 10,970 cfs at the USGS gauge located in Bayard, IA.

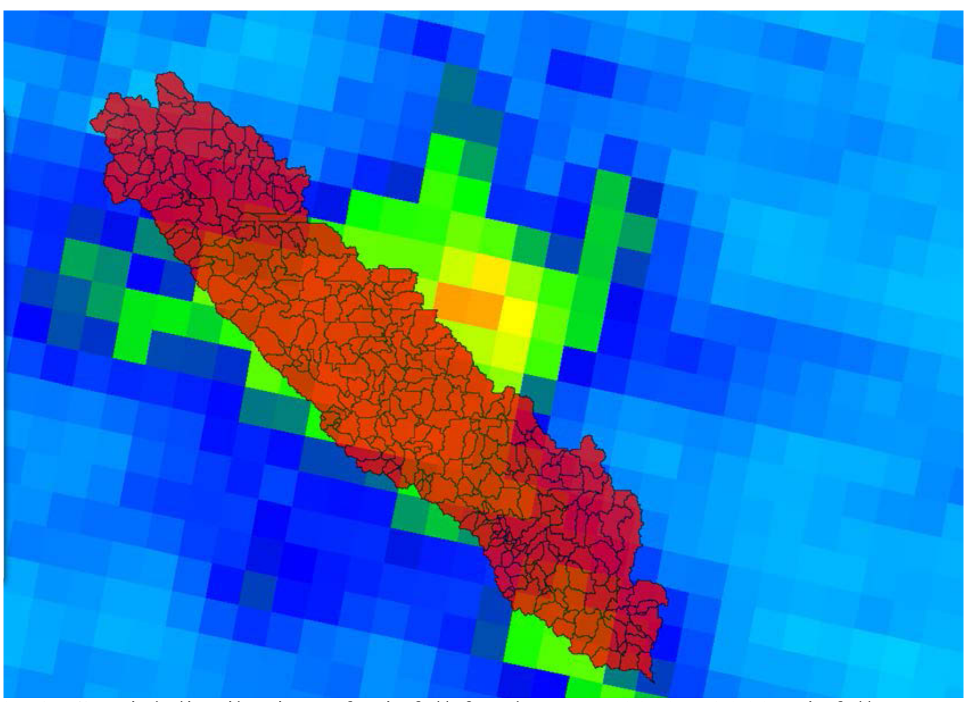

Figure 9. 6: Spatial distribution of rainfall for the June 13-17, 2013 rainfall event

Figure 9.7 compares the simulated flood hydrographs for the current no pond condition (Baseline) to those with the application of the large typical pond flood mitigation practices for the June 2013 rainfall event. The simulations used the large typical pond design specified in Chapter 7, where the emergency spillway is set at 10 feet above the principal spillway in the Southern Iowa Drift Plain and 5 feet above the principal spillway in the Des Moines Lobe. The smallest drainage area shown at Carroll, IA has a drainage area of 73.8 mi^2. There, the peak discharge was reduced by 13 percent. Ponds upstream of Carroll, IA were not fully utilized due to very little rainfall in the area draining to Carroll. For this reason, reductions at the Carroll, IA location were at a minimum for the watershed. Downstream, the results showed a sharp increase in peak flow reduction where rainfalls were heaviest, ranging from 13 percent (at Carroll, IA) to 27 percent (at Redfield, IA). Figure 9.8 summarizes the peak discharge for current conditions, the peak discharge for the large typical pond scenario, and the percent peak reduction, at all five index locations for the June 2013 rainfall event.

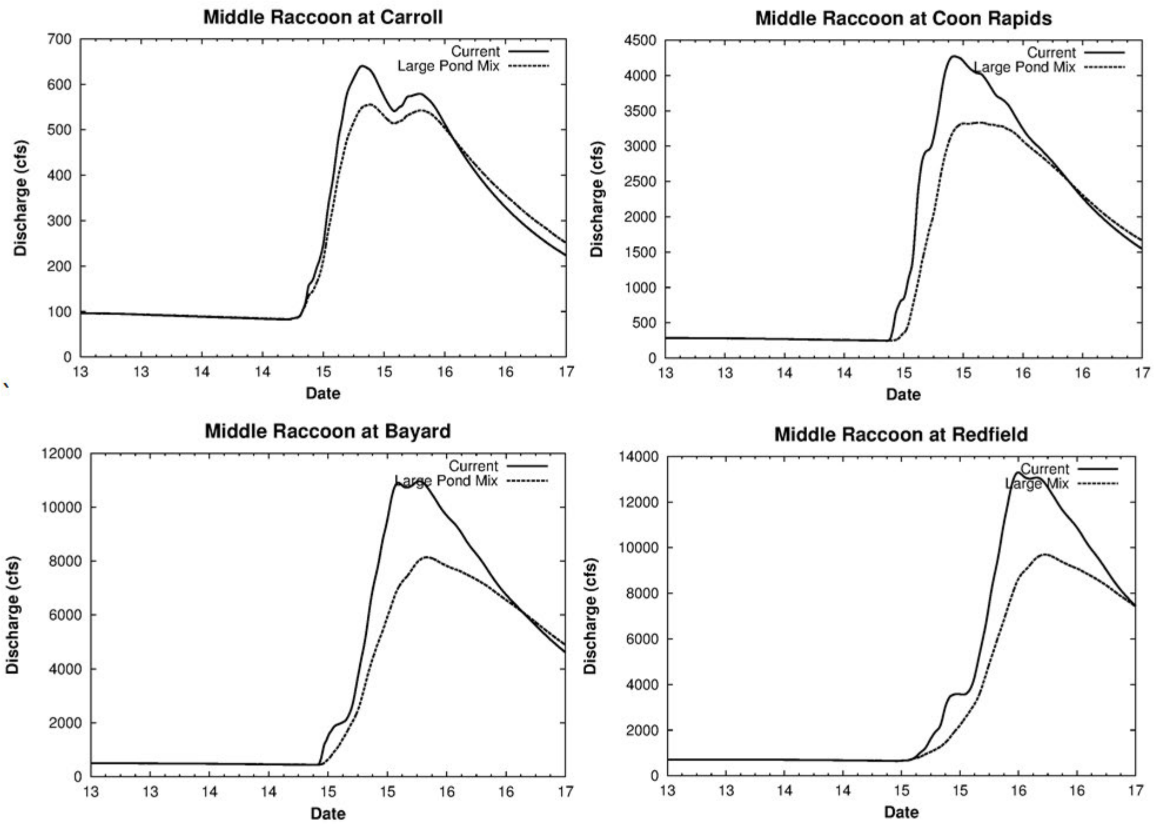

Figure 9. 7: Comparisons of hydrographs with and without the large typical pod scenario for the June 2013 rainfall event. For the hydrographs shown, peak flow reduction ranges from 14-21 percent.

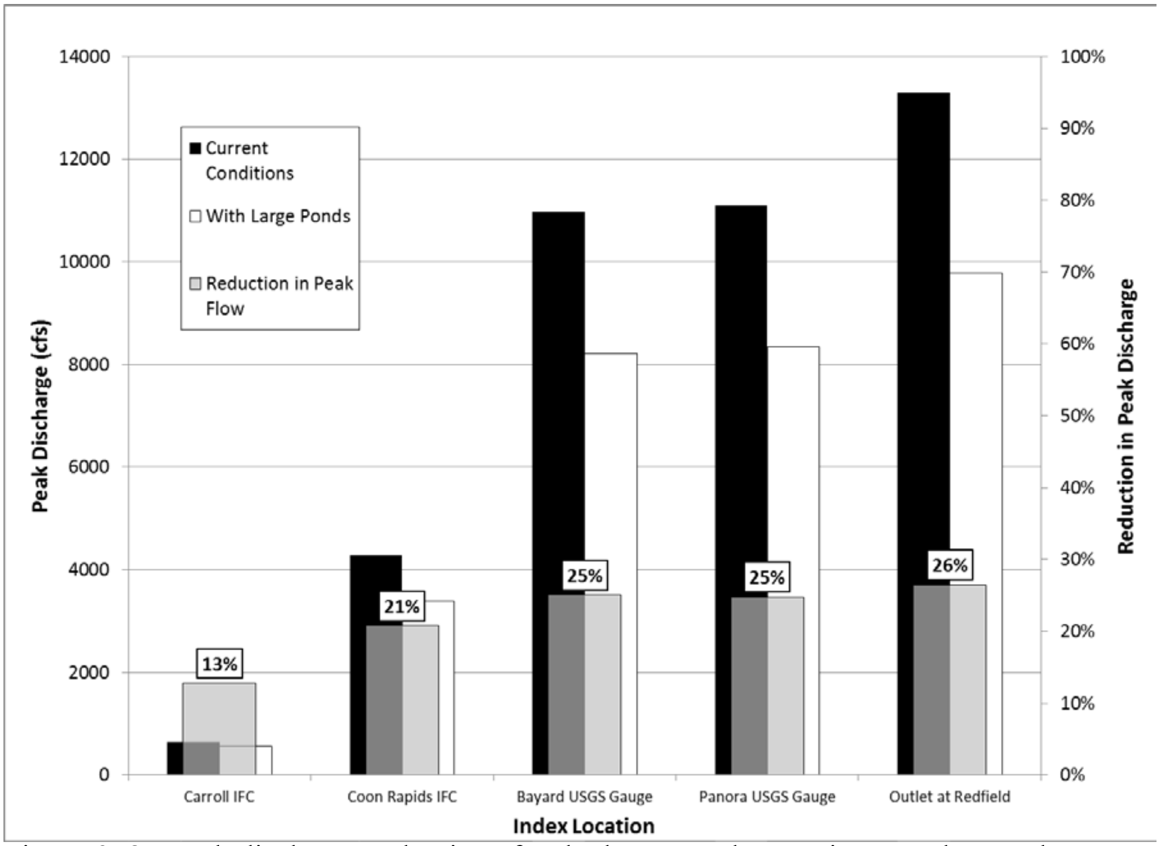

Figure 9. 8: Peak discharge reductions for the large pond scenario. Results are shown at five index points moving from upstream (left) to downstream (right) for the June 2013 rainfall event

Reducing peak flood discharge also reduces the peak water height (or stage) in a river during the flood. For the peak discharge reductions in the large pond flood mitigation practice, the corresponding reduction in flood stage was 1.5 feet at the USGS gauge location at Bayard, IA and 1.7 feet for the USGS gauge at Panora, IA. This reduction was estimated for the USGS stream-gauge locations, where the rating curves have been developed. Although a 1.5 and 1.7 feet reductions in flood stage would slightly reduce the flood inundation area, flooding still occurs. Again, based on the flood stage level reported by the National Weather Service at Bayard, IA, water levels above action stage (13 feet) are expected for both the current conditions and the large pond scenario. The addition of the large pond flood mitigation practice would not have eliminated the flood of 2013 in the Middle Raccoon River watershed. However, it would be reasonable to assume that a reduction in flood stage of approximately 1.5 foot may be capable of protecting some homes and properties which were inundated during this event.

Figure 9.9 compares the simulated flood hydrographs for the current no pond condition (Baseline) to those with the application of the large typical pond and cover crops flood mitigation practices for the June 2013 rainfall event. The simulations used the large typical pond design and cover crops specified in Chapter 8, where the emergency spillway is set at 10 feet above the principal spillway in the Southern Iowa Drift Plain and 5 feet above the principal spillway in the Des Moines Lobe. Cover crops were applied to every agricultural acre. The smallest drainage area shown at Carroll, IA has a drainage area of 73.8 mi^2. There the peak discharge was reduced by 30 percent. Ponds upstream of Carroll, IA were not fully utilized; however, a large increase in peak flow reduction can be seen when comparing this practice to the large typical pond practice for this event. This was due to a large increase in the percentage of rainfall being absorbed into the soil using the cover crop practices for the small amount of rainfall which fell in the Carroll, IA drainage area. However, peak flow reduction was still at a minimum at Carroll, IA during this event. Downstream, the results varied, based upon the spatial location of rainfall, ranging from 30 percent (at Carroll, IA) to 34 percent (at Redfield, IA). Compared to the June 2013 storm, using large ponds along with the addition of cover crops, more uniform reductions in flow were created from the upstream most to downstream most index location. In general, the addition of cover crops reduced peak flows by an additional 8 percent when compared with large typical ponds alone. Figure 9.10 summarizes the peak discharge for current conditions, the peak discharge for the large typical pond with cover crops scenario, and the percent peak reduction, at all five index locations for the June 2013 rainfall event.

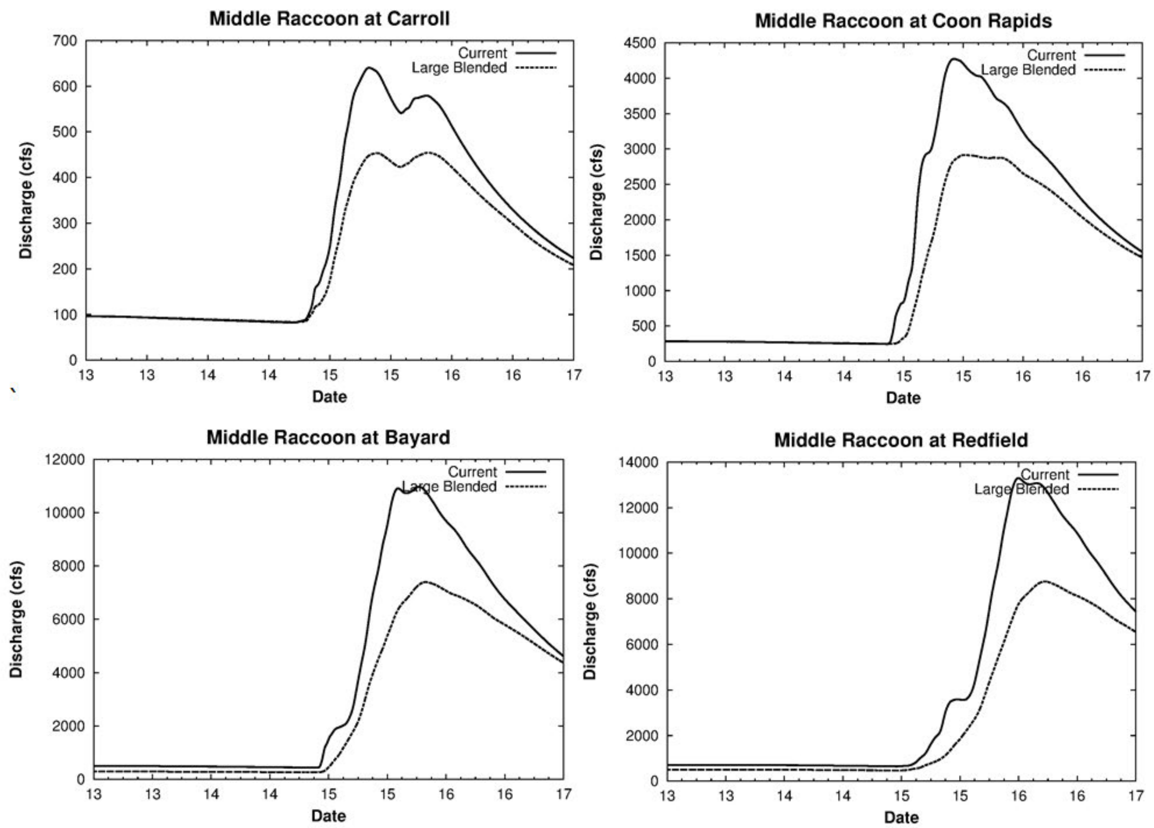

Figure 9. 9: Comparisons of hydrographs with and without the large blended scenario for the June 2013 rainfall event. For the hydrographs shown, peak flow reduction ranges from 30-34 percent.

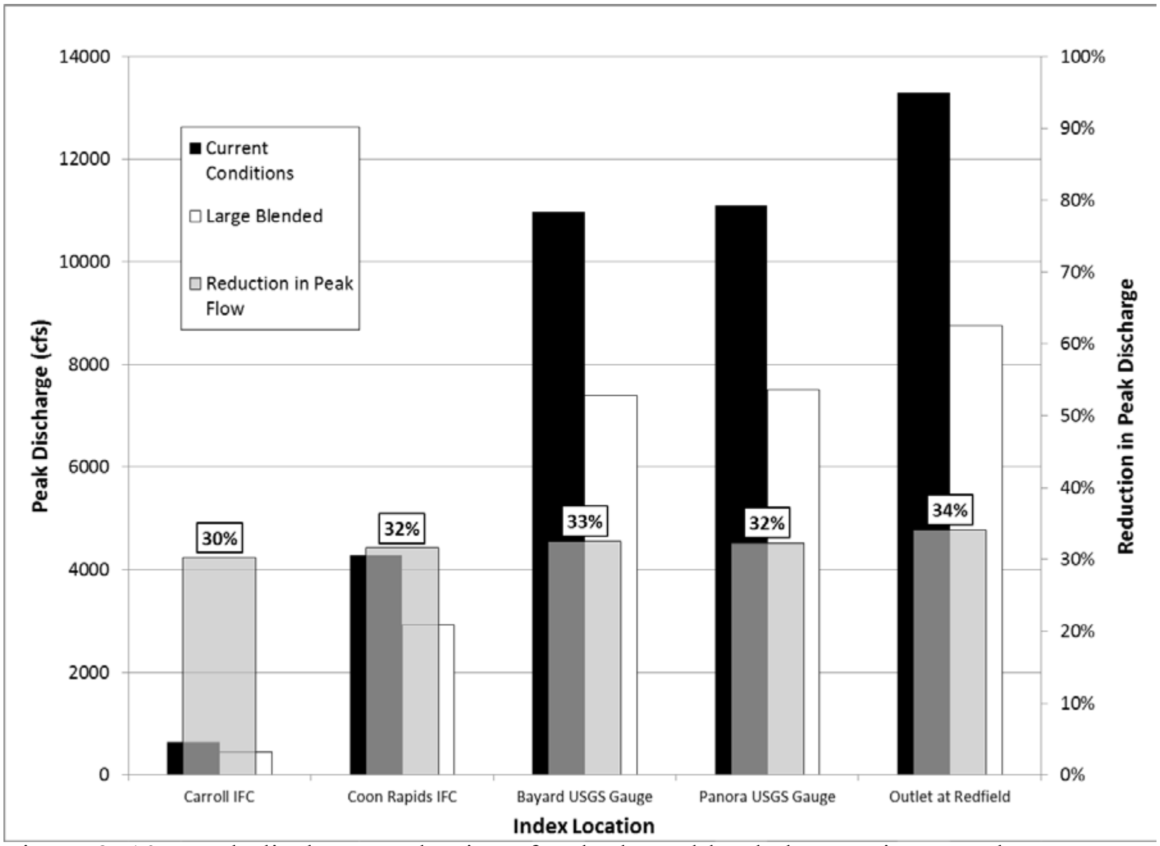

Figure 9. 10: Peak discharge reductions for the large blended scenario. Results are shown at five index points moving from upstream (left) to downstream (right) for the June 2013 rainfall event

For the peak discharge reductions in the large blended flood mitigation practice, the corresponding reduction in flood stage was 1.9 feet at the USGS gauge location at Bayard, IA and 2.3 feet for the USGS gauge at Panora, IA. This reduction was estimated for the USGS stream-gauge locations, where the rating curves have been developed. Although a 1.9 and 2.3 feet reductions in flood stage would slightly reduce the flood inundation area, flooding still occurs. Again, based on the flood stage level reported by the National Weather Service at Bayard, IA, water levels above action stage (13 feet) are expected for both the current conditions and the large pond scenario. The addition of the large blended flood mitigation practice would not have eliminated the flood of 2013 in the Middle Raccoon River watershed. However, it would be reasonable to assume that a reduction in flood stage of approximately 2 feet may be capable of protecting some homes and properties which were inundated during this event.

Summary of Flood Mitigation Using Increased Infiltration and Distributed Storage

In this chapter, two historic rainfall events were analyzed to represent the impact of flood mitigation practices on the watershed during times of past flooding. The scenarios were created using the large typical pond design defined in Chapter 7 and the addition of cover crops were applied in some simulations as defined in Chapter 8. In HEC-HMS, the ponds are represented by a storage-discharge table that is reflective of the topographic conditions and pond hydraulics. The cover crops are represented with a reduced Curve Number which allow for more infiltration and less runoff. A maximum peak flow reduction of 31 percent was found during the June 2008 event at Carroll, IA. A maximum peak flow reduction of 34 percent was found during the June 2013 event at Redfield, IA. Reductions were more varied for the simulations run with the ponds only, and more uniform when using the combination of both ponds and cover crops. From the analyses completed in Chapters 6 and 7, we know that increased infiltration is most effective at reducing peak discharge at small scale storm events, while distributed storage can be most effective at larger events (if the correct pond sizes are selected). The 2008 rainfall event was roughly equivalent to a 10-year rainfall in the northern portion of the watershed; therefore, most ponds were utilized, but the emergency spillway was not engaged. The June 2013 rainfall event was roughly equivalent to a 25-year rainfall in the center of the watershed; therefore, a number of ponds in the center of the basin were fully utilized and the emergency spillway was engaged. Pond utilization figures can be seen in Appendix B. Had flood mitigation practices been in place during these events, the simulations show that they could have had a significant impact on flooding throughout the basin.

CHAPTER 10: SUMMARY AND CONCLUSIONS

To better understand the flood hydrology of the Middle Raccoon River watershed, and to evaluate potential flood mitigation strategies, the HEC-HMS model of the watershed was used in several ways. We first assessed the runoff potential throughout the basin, using the HEC-HMS model's representation of runoff generation. Locations with agricultural land use, and moderate to poorly drained soils, have the highest runoff potential; mitigating the effects of high runoff from these areas is a priority for flood mitigation planning. Note that other land uses – particularly urban development in towns and cities – may have even higher runoff. But because their size is small compared to that of the HEC-HMS modeled subbasins (the basic element for runoff simulation), individual communities are not identified by this technique (only individual subbasins, which may include a small portion of urban land, are identified). Still, typical strategies employed to manage urban storm water are needed in these communities (e.g., storm water detention and low-impact development practices).

To quantify the potential effects of flood mitigation strategies, the HEC-HMS model was used to simulate river flows throughout the Middle Raccoon River watershed. Five strategies were considered — enhancing local infiltration though changes in land-use (from agriculture to forest or native tall-grass prairies), enhancing local infiltration though improvements in soil quality, enhancing local infiltration using conservation practices (cover crops), storing floodwaters temporarily in ponds throughout the watershed, and a combination of cover crops and ponds all to reduce downstream discharges. The effects of these strategies were simulated for significant design flood events – those resulting from a 10-, 25-, 50-, and 100-year return period, 24-hour design storm rainfall. These events correspond to rainfall amounts of 4.03, 5.08, 6.00, and 7.04 inches in 24 hours over the entire Middle Raccoon River watershed. Scenarios were also run for the historic rainfall events of June 2008 and June 2013. The results for these strategies were compared to simulations of flows for the existing watershed condition. Although each simulated scenario was hypothetical and simplified, the results provide valuable insights on the relative performance of each strategy for flood mitigation planning.

Increased Infiltration: Land Use Change

From the simulated results, enhancing local infiltration through changes in land use was found to have the most significant impact on runoff. The model predicts that the changes from an agricultural to a forest or native tall-grass prairie landscape would increase infiltration during large storms between 0.7 inches (tall-grass prairie, 10-year event) to 1.4 inches (forest, 100-year event). The increased infiltration results in peak flow reductions between 28 percent (tall-grass, 100-year event) and 56 percent (forest, 10-year event). This means the conversion of the native landscape to agricultural land uses, which has been occurring since the mid-19th century, has resulted in a significant reduction in the infiltration capacity, and more runoff. Obviously, converting the entire landscape back to forest or tall-grass prairie (as was simulated) is not a practical or an economically desirable strategy. Still, from a hydrologic point of view, targeted projects that enhance infiltration by land-use change could be an effective part of the watershed's flood mitigation efforts.

Increased Infiltration: Improving Soil Quality

Even without changes to land use, the storage capacity of the soils could be better utilized by improving soil quality to enhance infiltration. The hypothetical improved soil quality scenario suggests that it is a slightly less effective strategy than land use change. The improved soil quality scenarios predict an increased infiltration during large storms between 0.7 inches (for the 10-year design event) to 1.1 inches (100-year design event). The increased infiltration results in peak flow reductions between 21 percent and 36 percent. In locations where the land use must remain the same, such increases in infiltration (and the resulting downstream reduction in flood flows) are very significant. For the Middle Raccoon River watershed, where agricultural land use will continue to dominate for the foreseeable future, efforts to improve soil quality can also be an effective part of a watershed-wide flood mitigation strategy.

Increased Infiltration: Application of Cover Crops

Hydrology is altered when cover crops are applied to heavily agricultural landscapes during a crop's dormant season. Cover crops act to increase infiltration by the prevention of surface sealing, increased available water storage capacity, and increased soil macroporosity, (Dabney, 1998). The hypothetical cover crop application results in the least drastic reductions in peak discharge reductions and stage reductions. However, less drastic results were expected in this scenario since they did not require large scale changes to the watershed's primary agricultural function. This scenario may, therefore, be the most feasible of the four increased infiltration scenarios. The application of cover crops scenario predicts an increased infiltration during large storms between 0.2 inches (for the 10-year event) to 0.4 inches (for the 100-year event). The increases in infiltration result in peak discharge reductions between 6 percent (for 100-year design event) and 13 percent (for the 10-year design event). Since the application of cover crops between cash crop seasons has become more popular in recent years, and the removal of agricultural land to tall-grass prairie or forest is not realistic on a large scale, this cover crop scenario can provide input as to the upper bounds of expected peak flow reduction, should every tillable acre apply this conservation practice. The results of this scenario show that, while not as effective as other increased infiltration techniques, the application of cover crops can still be part of a basin-wide flood mitigation strategy.

Increased Storage on the Landscape: Typical Ponds

In some ways, using ponds to temporarily store floodwaters is an attempt to replace the loss of water that was stored in soils during a pre-agricultural landscape. In the hypothetical scenarios involving pond storage, between 4,709 acre-feet and 9,693 acre-feet of flood storage was added to the Middle Raccoon River watershed. For the watershed, the added storage depth ranges from 0.5 inches (using small ponds) to 1.0 inches of rainfall for drainage areas upstream of the ponds (using large ponds). Compared to the extra water that was removed by infiltration in the previous scenario simulations, the amount of storage replaced by ponds is much smaller. As a result, the overall flood peak reduction with storage ponds is less than predicted for the other scenarios. However, compared to the infiltration scenarios, flood storage is more realistically achievable. In this scenario peak discharges were reduced between 4 percent (small pond) and 22 percent (large ponds).

As a flood mitigation strategy, ponds are very effective in reducing flood peaks immediately downstream of their headwater sites. Further downstream, floodwaters originating from locations throughout the watershed arrive at vastly different times; some areas are controlled by ponds, others are not. As a result, as one moves further downstream in the watershed, the flood peak reduction of storage dampens and reductions are less.

Increased Storage on the Landscape: Dry Ponds

Another way to consider the temporary storage of water floodwaters is to design ponds that do not maintain a permanent pool. This means that during times of normal flow, the ponds in this scenario would not hold water. By adding and an outlet (2 inch pipe) at the pond bottoms, the water stored in the permanent pool in the typical pond design scenario, becomes flood storage in the dry pond design scenario. Over the entire Middle Raccoon River watershed, this adds approximately 1,000 acre-feet of flood storage. In this scenario, a willing land owner would be trading other functions of the pond, such as watering animals and irrigation, for flood storage. This may not be as appealing, and may require extra incentive. However, the dry pond scenarios show that, under most conditions, the extra storage provides an increased benefit in peak flow reductions. Peak discharge in these scenarios was reduced between 5 percent (small dry pond) to 23 percent (large dry pond).

Blended Practices: Increased Infiltration and Distributed Storage

One last scenario was run to get an approximation for peak flow reduction, should the watershed adopt a flood mitigation plan that incorporated both distributed storage and increased infiltration. In HEC-HMS, the cover crop increased infiltration scenario (where every agricultural acre was converted to agricultural plus the application of cover crops) was combined with the typical pond scenarios. The typical pond design was used, due to the increased expectation of landowner willingness. The two flood mitigation strategies applied in this scenario were assumed to be a likely part of any flood mitigation strategy. Therefore, it can give an approximation for realistic peak flow reductions that the watershed could expect from adopting such a plan. Peak flow reductions for this scenario ranged from 12 percent (Small Blended storm at Redfield, IA) to 31 percent (Large Blended at Carroll, IA). Flow reductions remained relatively consistent for the entire range of design storms considered. This is likely due to the fact that increased infiltration has its greatest impact at smaller design storm events, and distributed storage can have its greatest effect at larger design storm events. The mixing of the practices results in flow reductions that never vary more than 8 percent from the upstream most to downstream most locations, and under most design storms variations were far lower. Compared to cover crops only and typical ponds only, the blended practices have a much larger impact on peak discharges, up to 11 percent.

Historic Rainfall Events

In addition to analyzing the effects of flood mitigation practices on peak discharge using design storms, we also took into consideration the effects of these practices on historic rainfall events. In particular, two events were simulated June 2008, and June 2013 both using the Large Typical Pond scenario and the Large Blended Practices scenario. By analyzing actual past rainfall events, we were able to get a sense of how well these practices would have performed in non-uniform, past rainfall events. It also gave us an idea as to how much peak flow reduction we can expect in future large flooding events. As expected, the reduction in peak discharge for these events was highly dependent on the spatial distribution of rainfall. For June 2008, the majority of rain fell on the northwest portion of the watershed, so this is where peak flow reductions were at a maximum. For June 2013, the majority of rainfall fell on the center of the watershed, so similarly this is where peak flow reductions were at a maximum. For both events the large blended scenarios outperformed the large typical pond scenarios by an average peak flow reduction of 8 percent. Reductions in peak flows for June 2008 ranged from 14 percent (Large typical ponds at Redfield, IA) to 31 percent (Large blended at Carroll, IA). Reductions in peak flows for June 2013 ranged from 13 percent (Large typical ponds at Carroll, IA) to 34 percent (Large blended at Redfield, IA)

Scenario Reductions in Flood Stage

While reductions in peak discharge describe the hydrologic impact of the simulated scenarios, the most important factor is how that peak discharge reduction translates to a decrease in river stage. Table 10.1 summarizes the effects of all the flood mitigation scenarios analyzed in terms of their stage reduction at the USGS gauges in Bayard, IA and Panora, IA for the design storm simulations. Table 10.2 summarizes the effects of the flood mitigation scenarios when used in the June 2008 and June 2013 historic rainfall events.

Table 10. 1: Stage reductions (ft) for all hypothetical flood mitigation simulations using design storm analysis

Scenario	Reduction in Stage due to Reduction in Peak Discharge (ft)							
	Bayard, IA USGS Gauge				Panora, IA USGS Gauge			
	10-YR	25-YR	50-YR	100-YR	10-YR	25-YR	50-YR	100-YR
Ag to Forest	3.5	3.1	2.9	2.6	4.1	4.3	4.6	4.6
Ag to Tallgrass	2.6	2.3	2.1	1.9	3.1	3.3	3.5	3.4
Soil Improvement	2.0	1.7	1.6	1.4	2.4	2.6	2.7	2.6
Addition of Cover Crops	0.6	0.5	0.5	0.4	0.8	0.8	0.8	0.6
Ponds - Small	0.7	0.6	0.5	0.4	0.9	0.9	0.9	0.7
Ponds - Large	1.0	0.9	0.9	0.8	1.2	1.4	1.6	1.4
Dry Ponds - Small	0.8	0.7	0.6	0.5	1.0	1.0	1.0	0.8
Dry Ponds - Large	1.0	1.0	0.9	0.9	1.2	1.4	1.6	1.5
Small Ponds and Cover Crops	1.4	1.2	1.1	1.0	1.4	1.8	1.9	1.6
Large Ponds and Cover Crops	1.5	1.5	1.4	1.3	1.9	2.3	2.5	2.3

Table 10. 2: Stage reductions (ft) for all hypothetical flood mitigation simulations using historic storm analysis

Scenario	Reduction in Stage due to Reduction in Peak Discharge (ft)			
	Bayard, IA USGS Gauge		Panora, IA USGS Gauge	
	June 2008	June 2013	June 2008	June 2013
Ponds - Large	0.8	1.5	0.9	1.7
Large Ponds and Cover Crops	1.5	1.9	1.7	2.3

As a final note, it is important to recognize that the modeling scenarios evaluate the hydrologic effectiveness of the flood mitigation strategies and not their effectiveness in other ways. For instance, while certain strategies are more effective from a hydrologic point of view, they may not be more effective economically. As part of the flood mitigation planning process, factors such as the cost and benefits of alternatives and landowner willingness to participate need to be considered along with the hydrology.

This thesis lays out a methodology for analyzing peak flow reductions in an agricultural Iowa watershed, using a widely used lumped parameter hydrologic model, HEC-HMS. The techniques used in the report could potentially be applied in other similar watersheds across the Midwest to determine the possible locations of flood control projects and the impact these projects could have on peak discharge. In the case of the Middle Raccoon River watershed, watershed projects that increase infiltration such as land use changes, soil improvements, and conservation practices can decrease peak discharge between 6 and 56 percent. Watershed projects that apply distributed storage can reduce peak discharge between 4 and 23 percent for design storms and between 13 to 27 percent for historic storms. Watershed projects that apply both distributed storage and cover crops can reduce peak discharge between 12 and 31 percent for design storms and between 26 and 34 percent for historic storms. The projects applied in this report have been simplified for the application on a large scale basin. Any project built in the watershed for the purpose of peak discharge reduction should be completed with a more detailed, project specific study.

APPENDIX A: ADDITIONAL MAPS AND FIGURES

Rating Curves

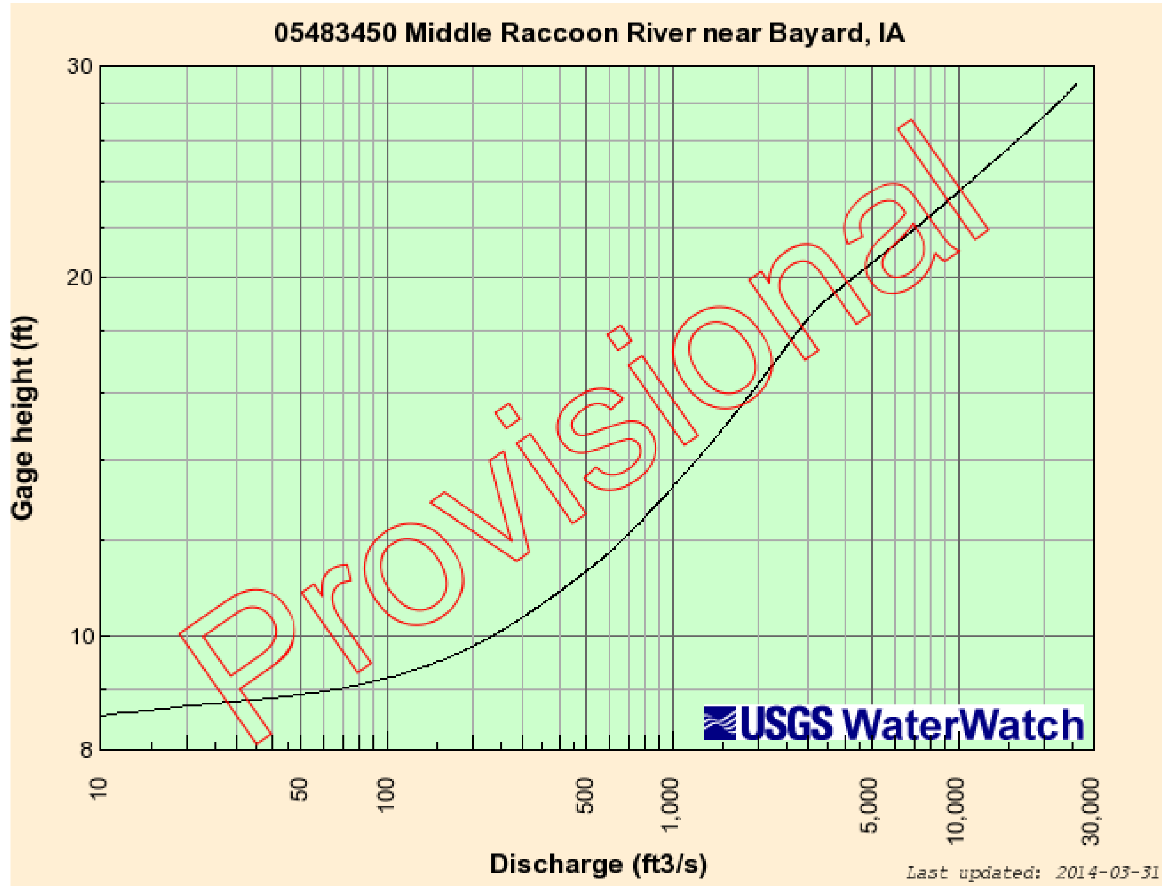

Figure A.1: Rating curve used to determine flood stages at the Bayard, IA USGS stream gauge location (USGS, 2014).

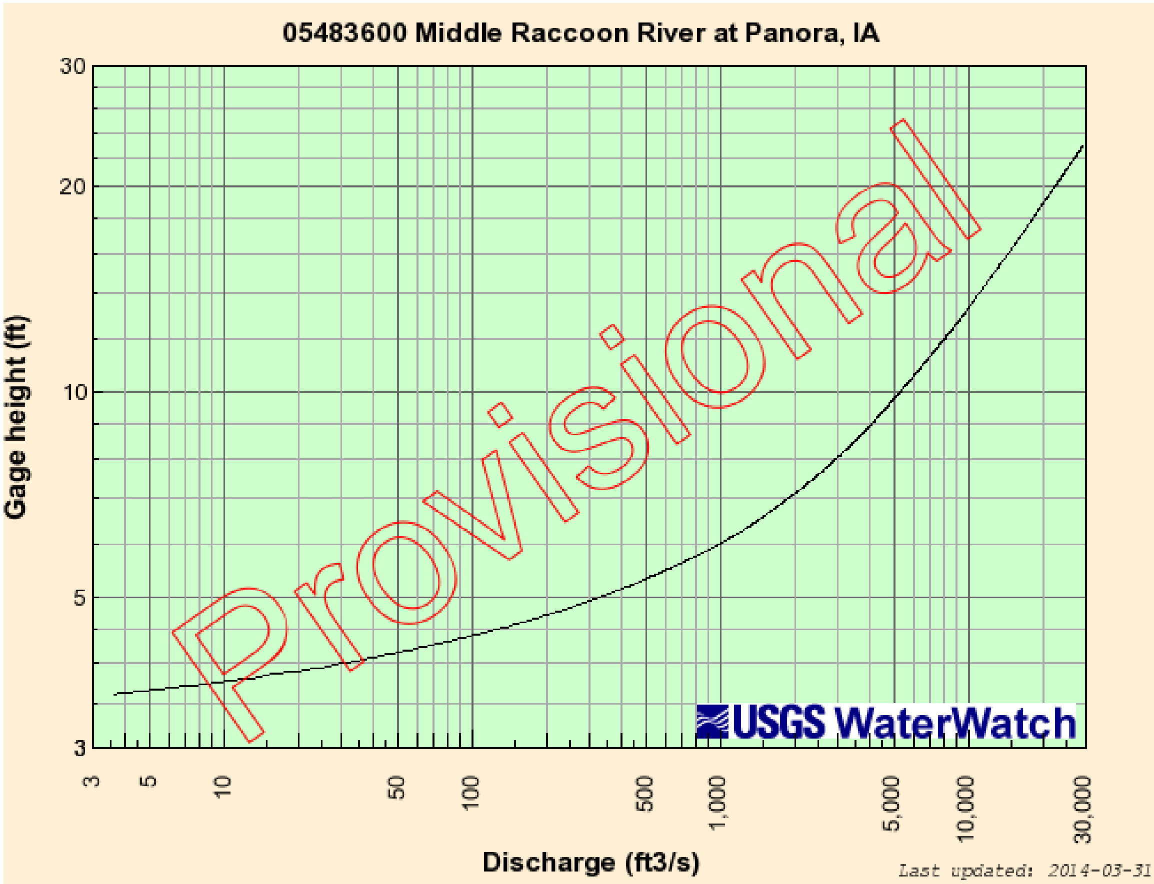

Figure A.2: Rating curve used to determine flood stages at the Panora, IA USGS stream gauge location (USGS, 2014).

Curve Number Grids

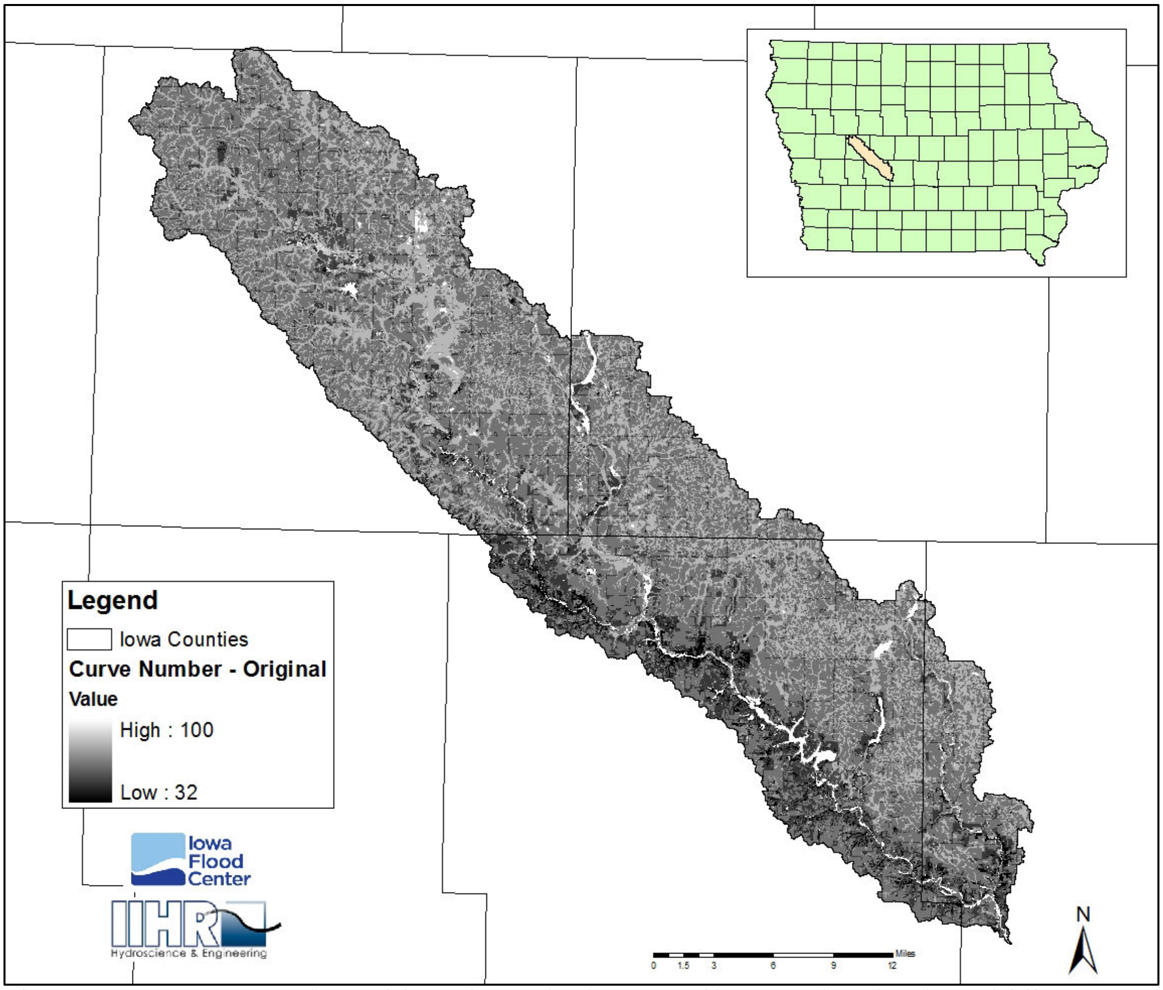

Figure A.3: Curve number grid used to calculate runoff generation for the Middle Raccoon River watershed, current conditions scenario. Light colors indicate high curve number, dark colors indicate low curve numbers. The basin-wide average curve number is 82.2.

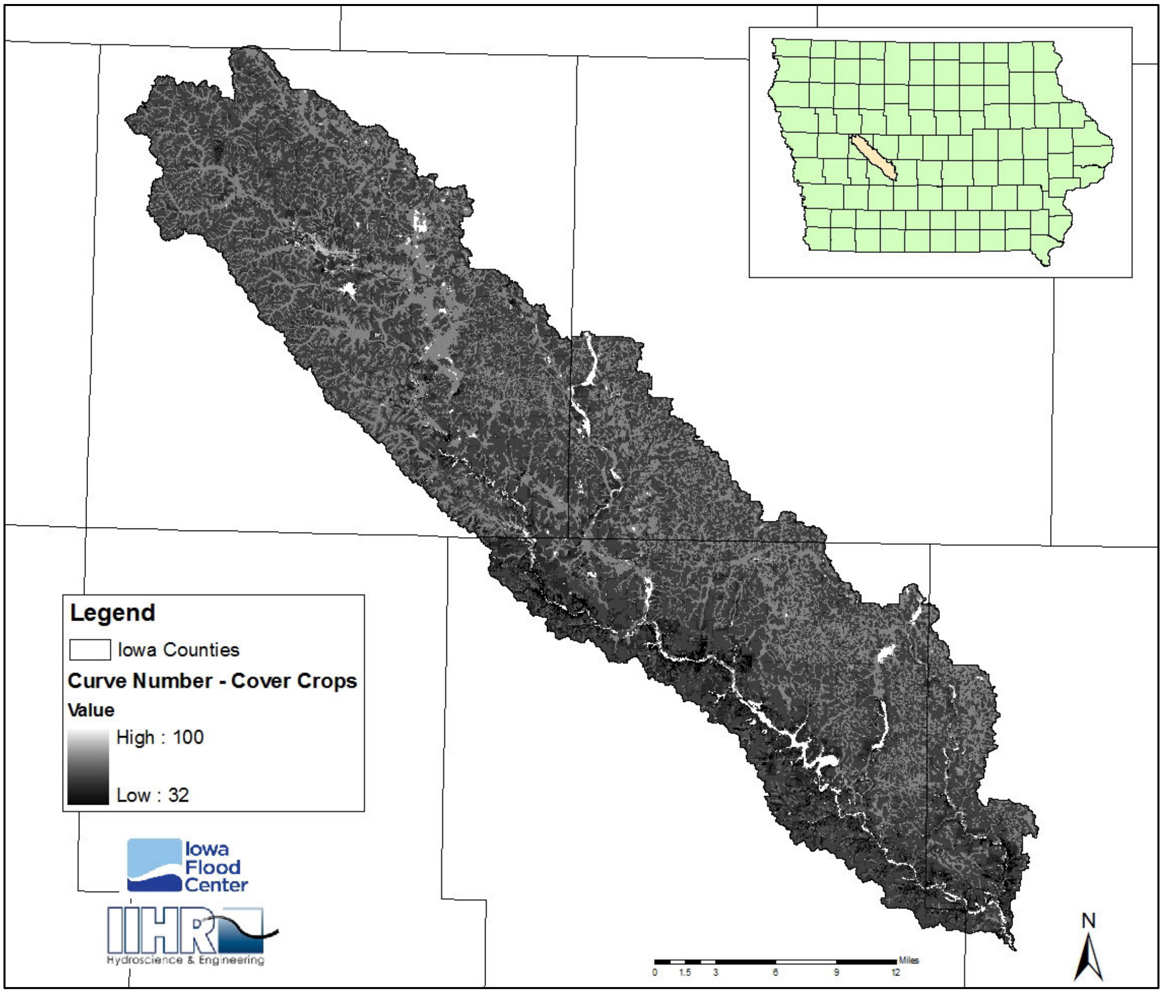

Figure A.4: Curve number grid used to calculate runoff generation for the Middle Raccoon River watershed, cover crop scenario. Light colors indicate high curve number, dark colors indicate low curve numbers. The basin-wide average curve number is 79.0.

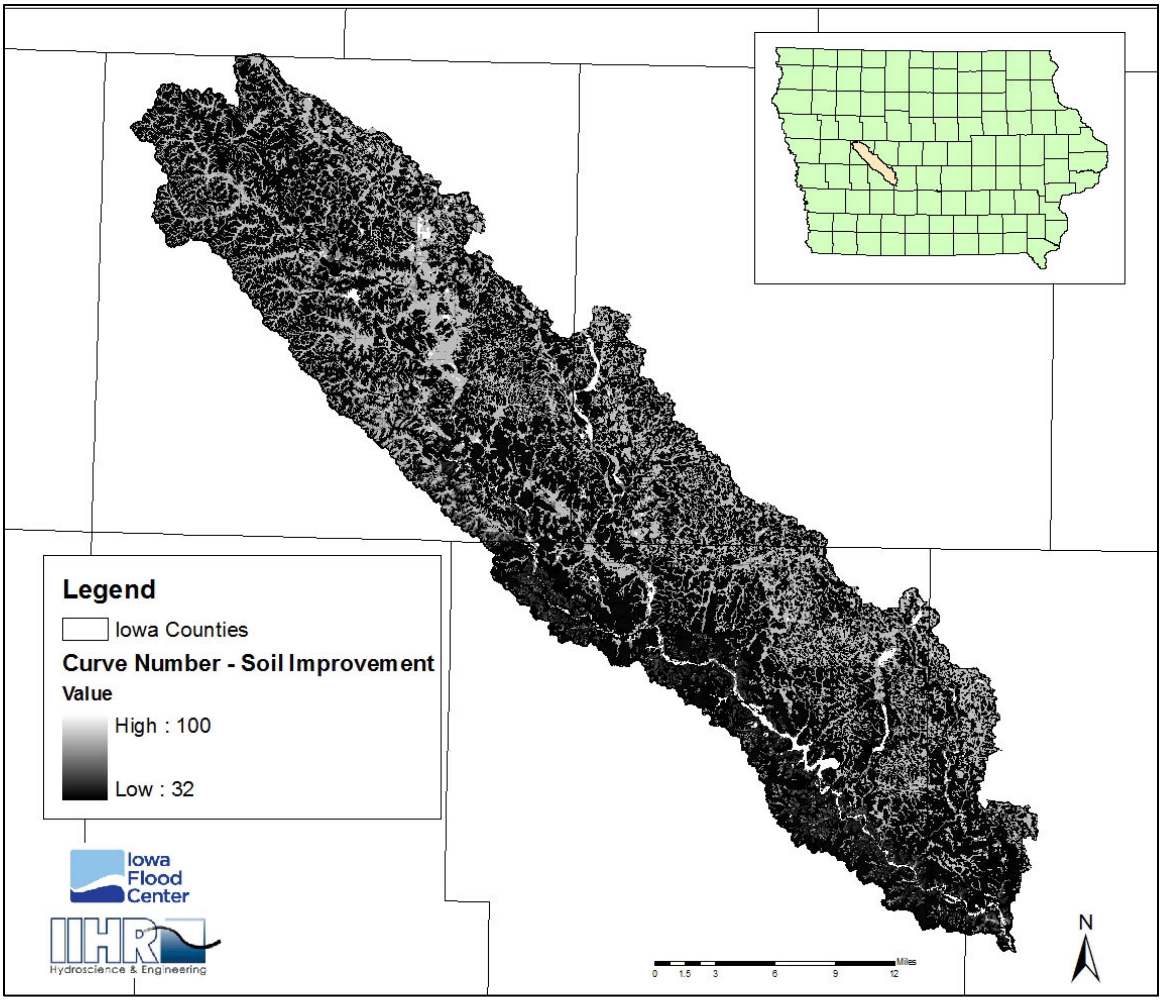

Figure A.5: Curve number grid used to calculate runoff generation for the Middle Raccoon River watershed, improved soil conditions scenario. Light colors indicate high curve number, dark colors indicate low curve numbers. The basin-wide average curve number is 71.6.

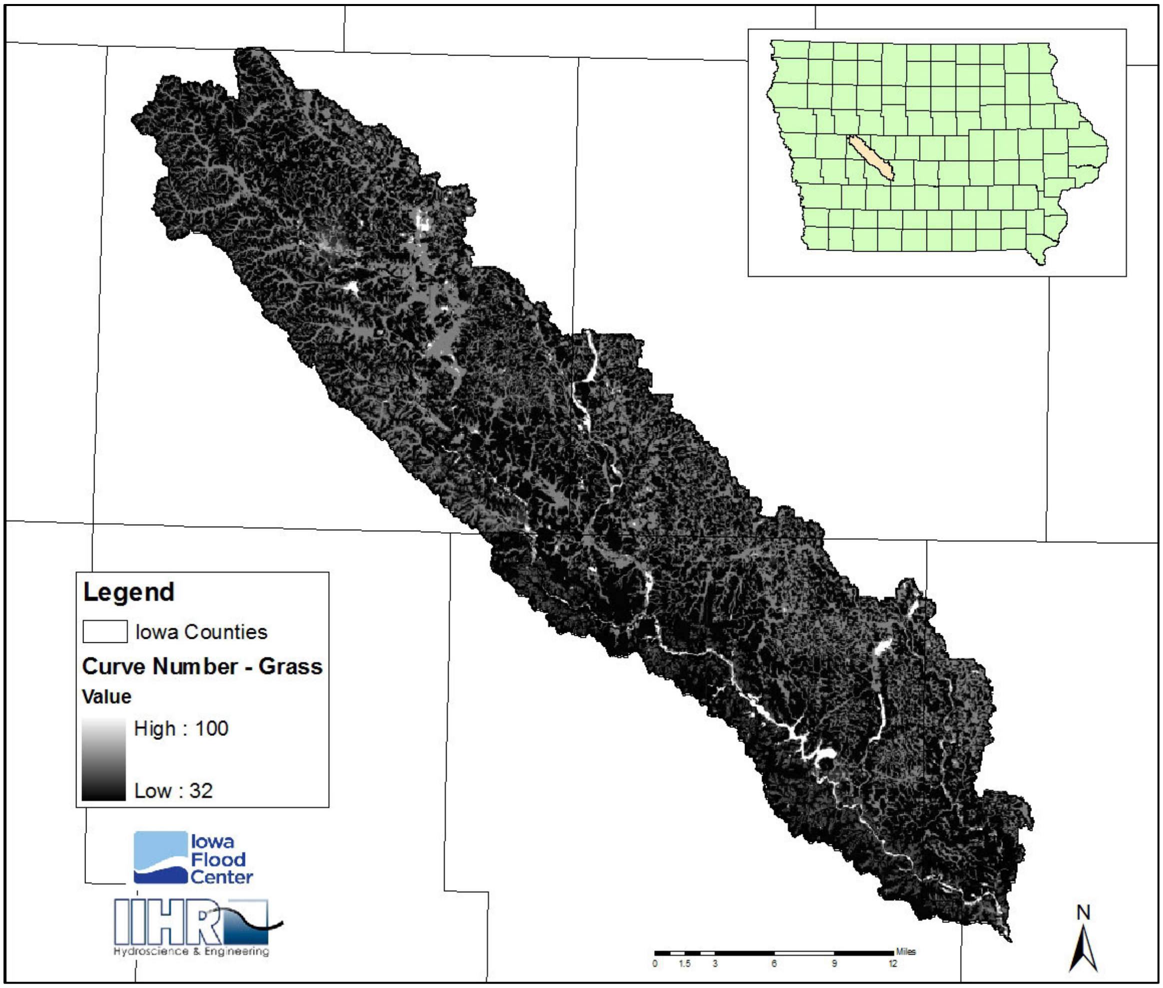

Figure A.6: Curve number grid used to calculate runoff generation for the Middle Raccoon River watershed, agriculture to native prairie tall-grass scenario. Light colors indicate high curve number, dark colors indicate low curve numbers. The basin-wide average curve number is 69.6.

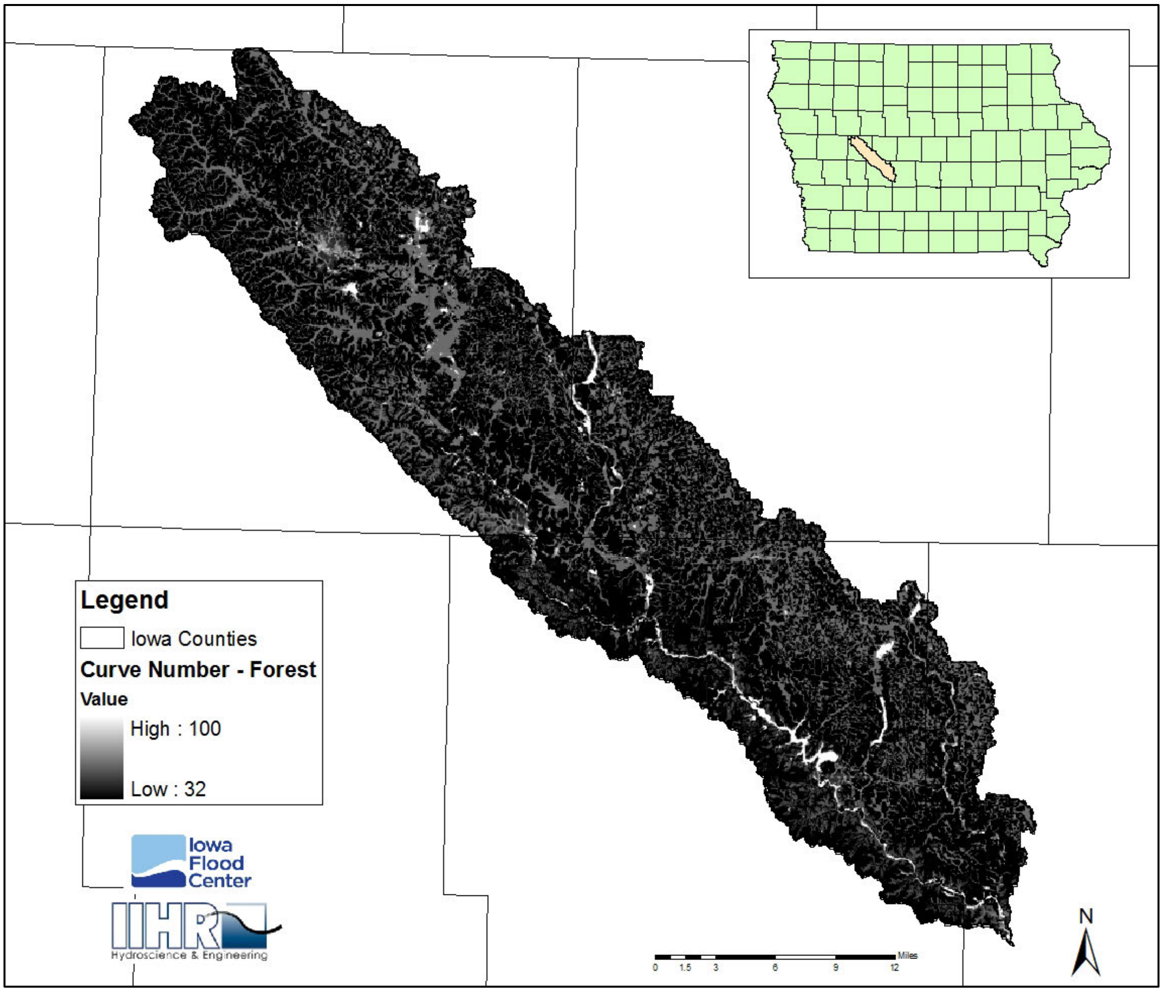

Figure A.7: Curve number grid used to calculate runoff generation for the Middle Raccoon River watershed, agriculture to forest scenario. Light colors indicate high curve number, dark colors indicate low curve numbers. The basin-wide average curve number is 65.7.

Distributed Storage: Pond Utilization Figures

Typical Pond Design: Small Pond

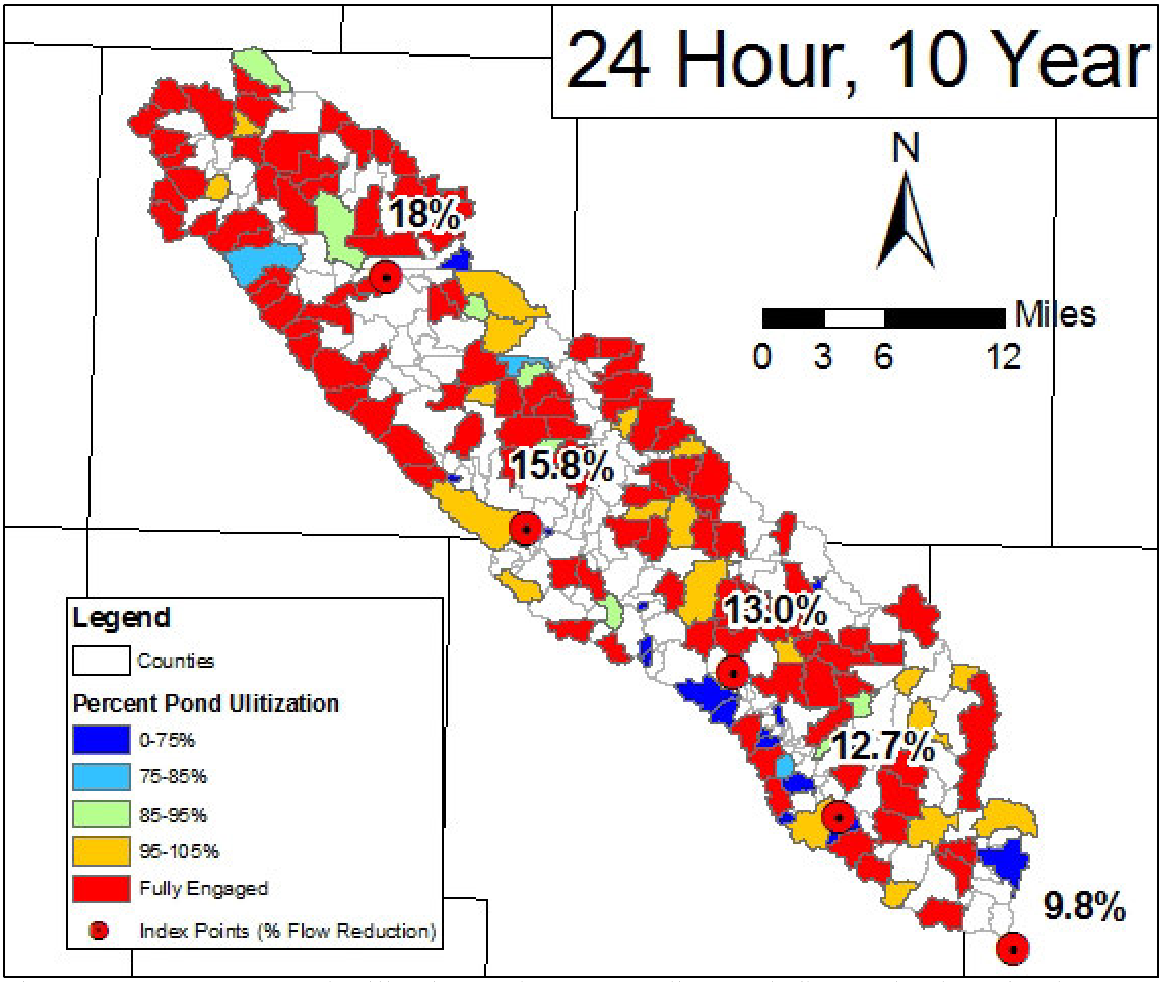

Figure A.8: Percent pond utilization and corresponding peak flow reductions for the small typical pond design. Run using the 10-year, 24-hour design storm (4.03 inches)

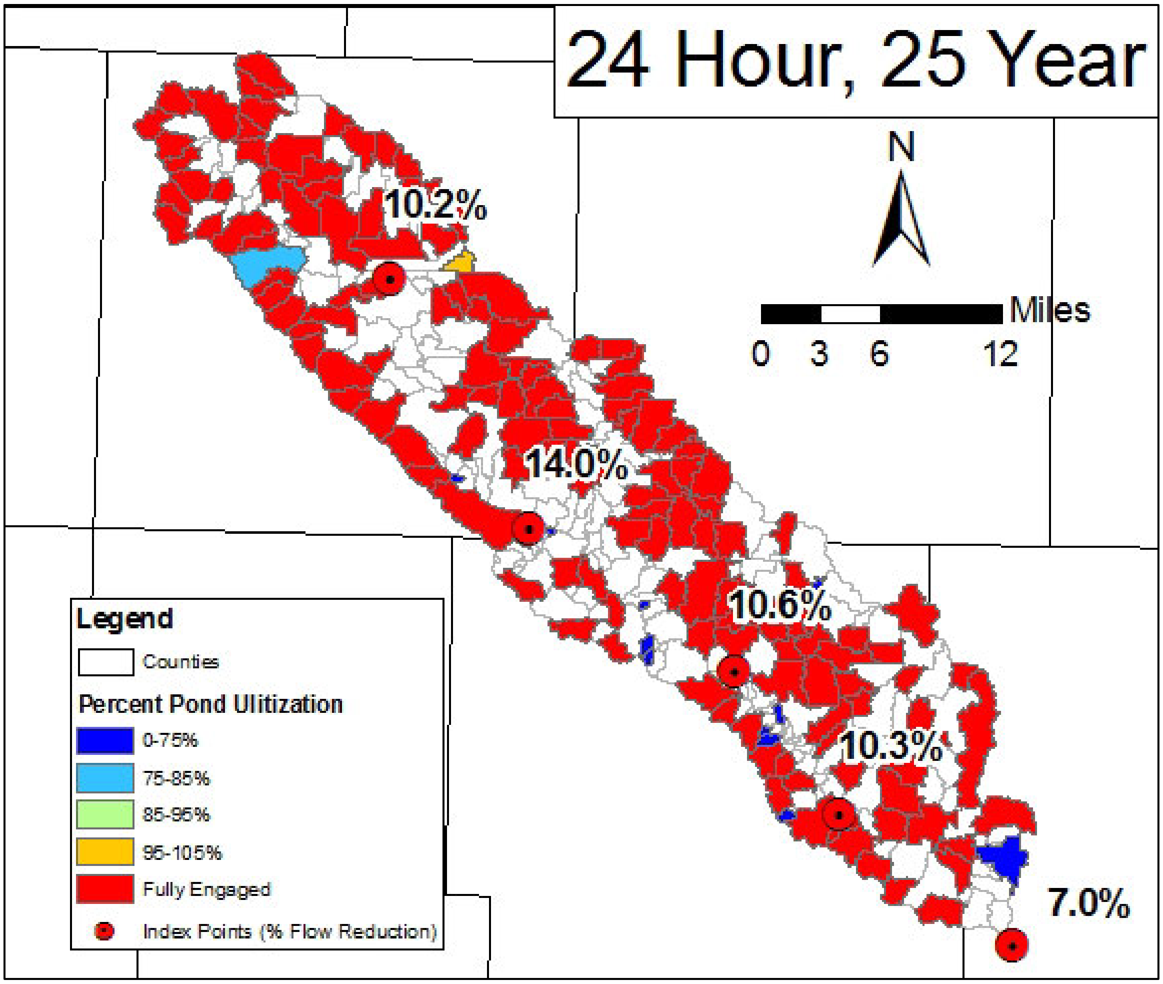

Figure A.9: Percent pond utilization and corresponding peak flow reductions for the small typical pond design. Run using the 25-year, 24-hour design storm (5.08 inches)

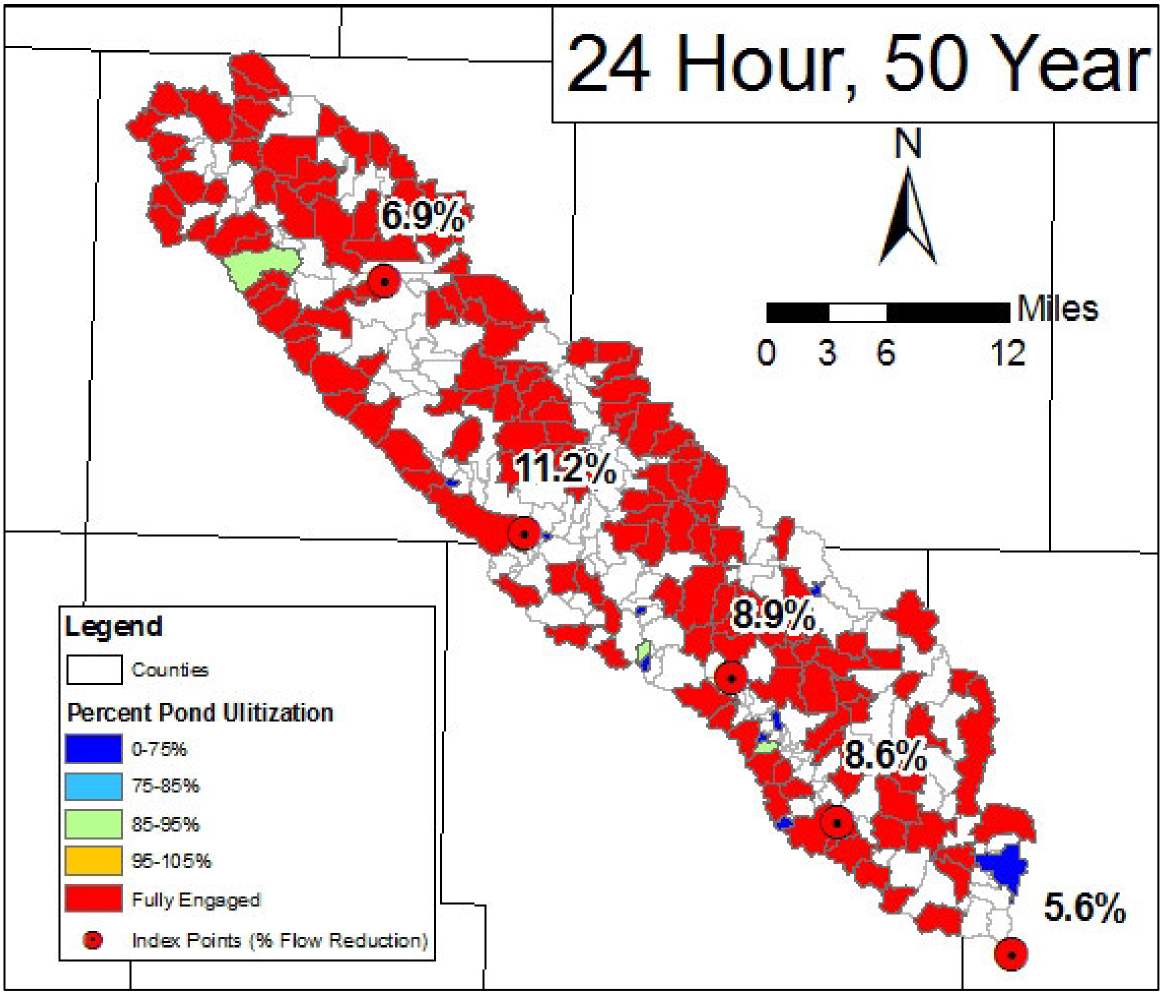

Figure A.10: Percent pond utilization and corresponding peak flow reductions for the small typical pond design. Run using the 50-year, 24-hour design storm (6.00 inches)

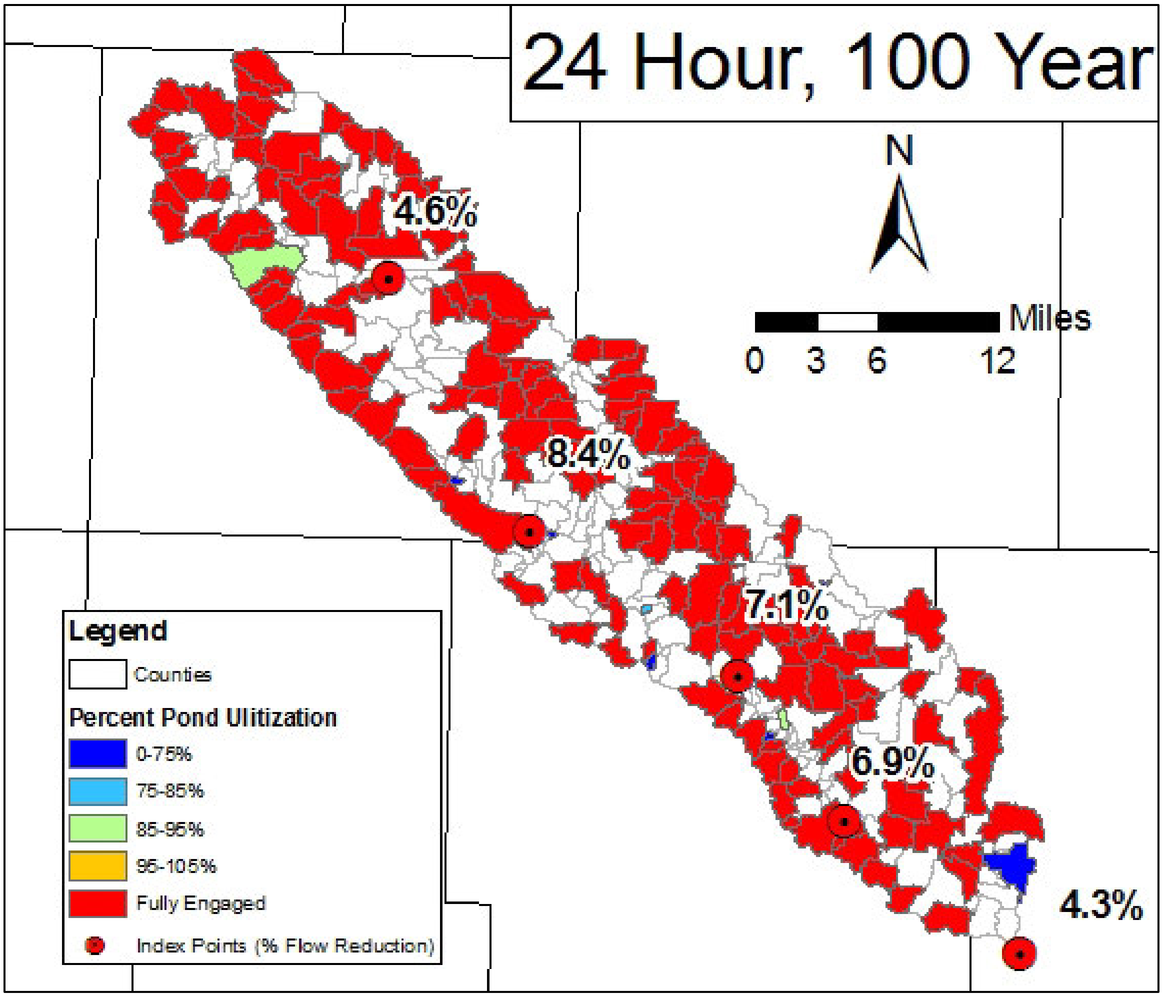

Figure A.11: Percent pond utilization and corresponding peak flow reductions for the small typical pond design. Run using the 100-year, 24-hour design storm (7.04 inches)

Typical Pond Design: Large Pond

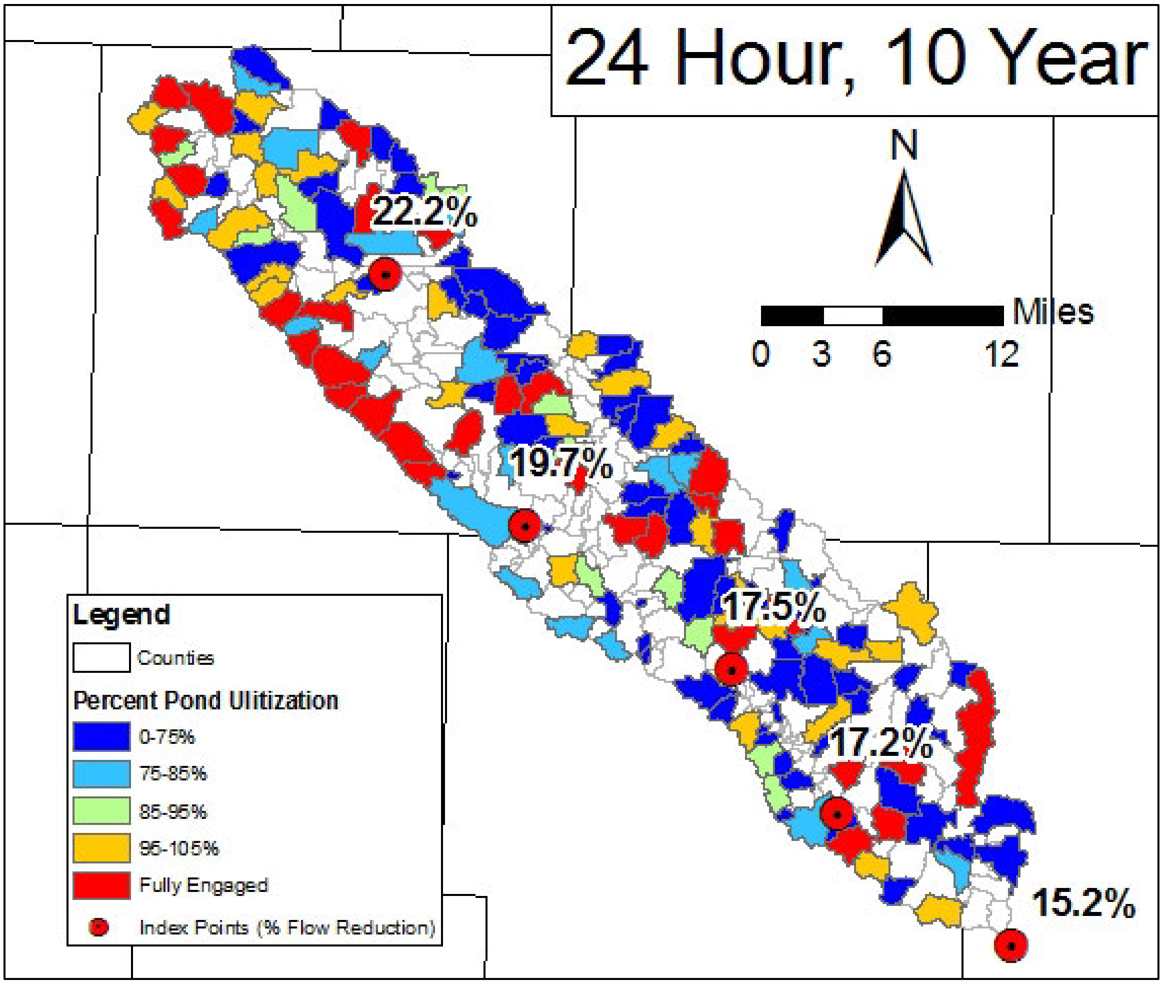

Figure A.12: Percent pond utilization and corresponding peak flow reductions for the large typical pond design. Run using the 10-year, 24-hour design storm (4.03 inches)

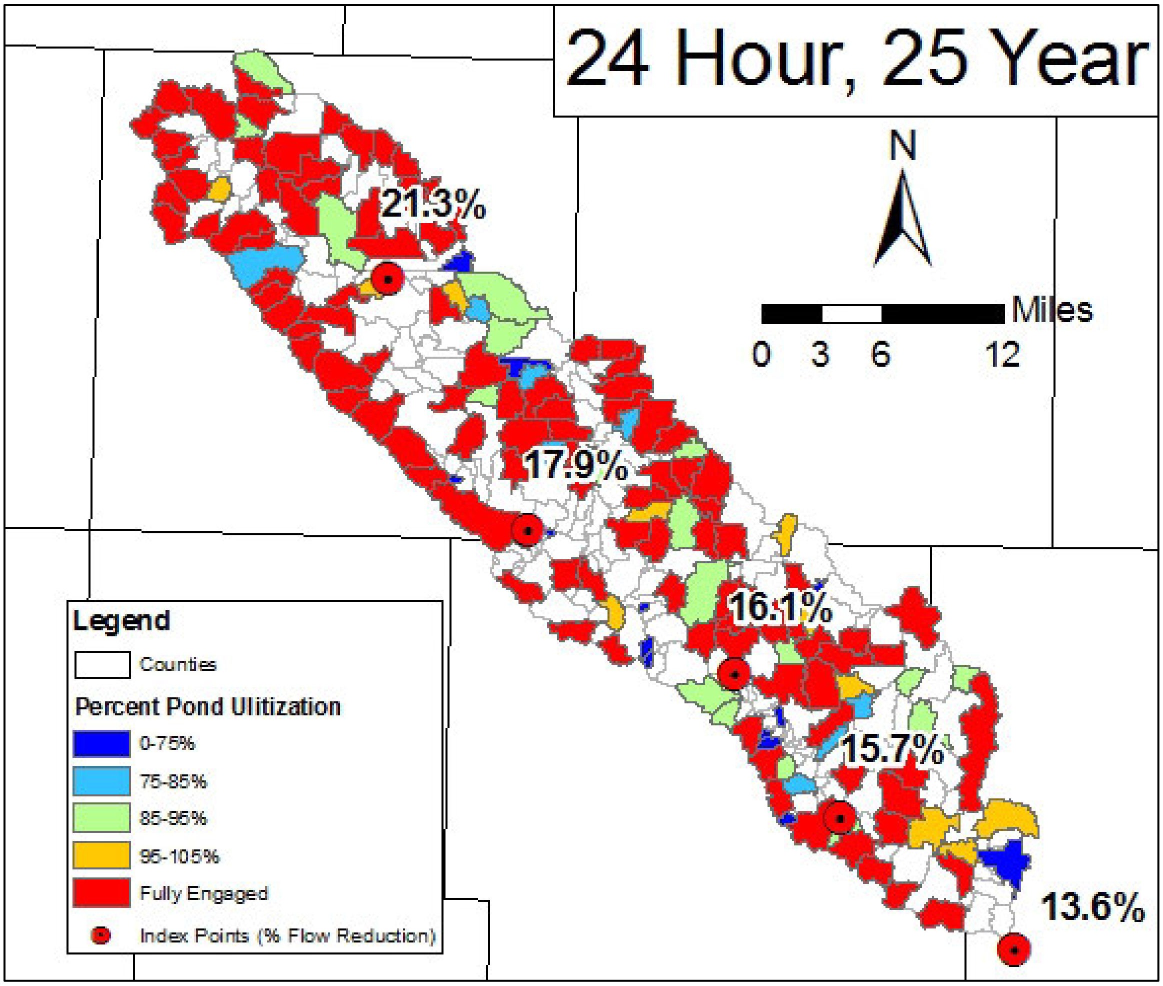

Figure A.13: Percent pond utilization and corresponding peak flow reductions for the large typical pond design. Run using the 25-year, 24-hour design storm (5.08 inches)

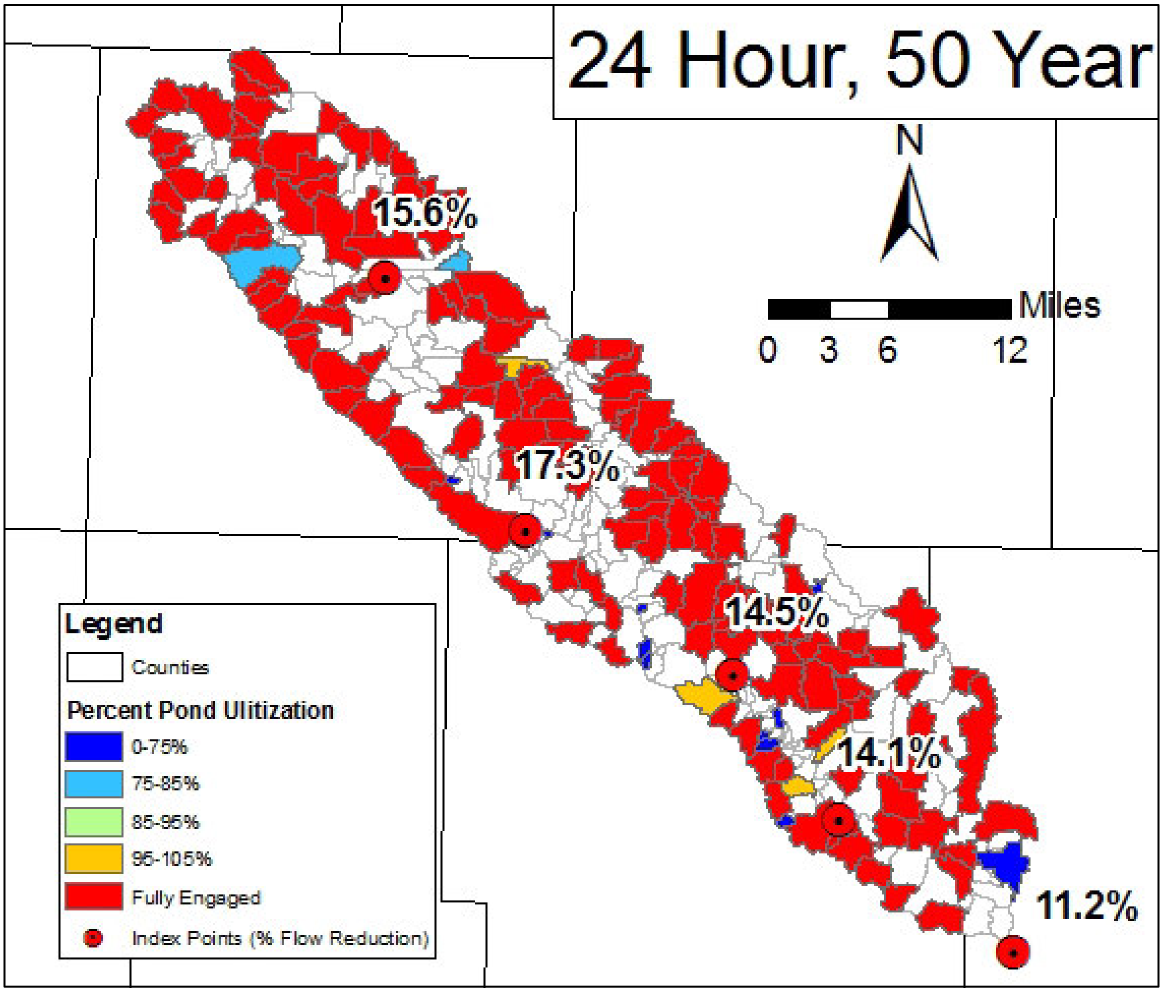

Figure A.14: Percent pond utilization and corresponding peak flow reductions for the large typical pond design. Run using the 50-year, 24-hour design storm (6.00 inches)

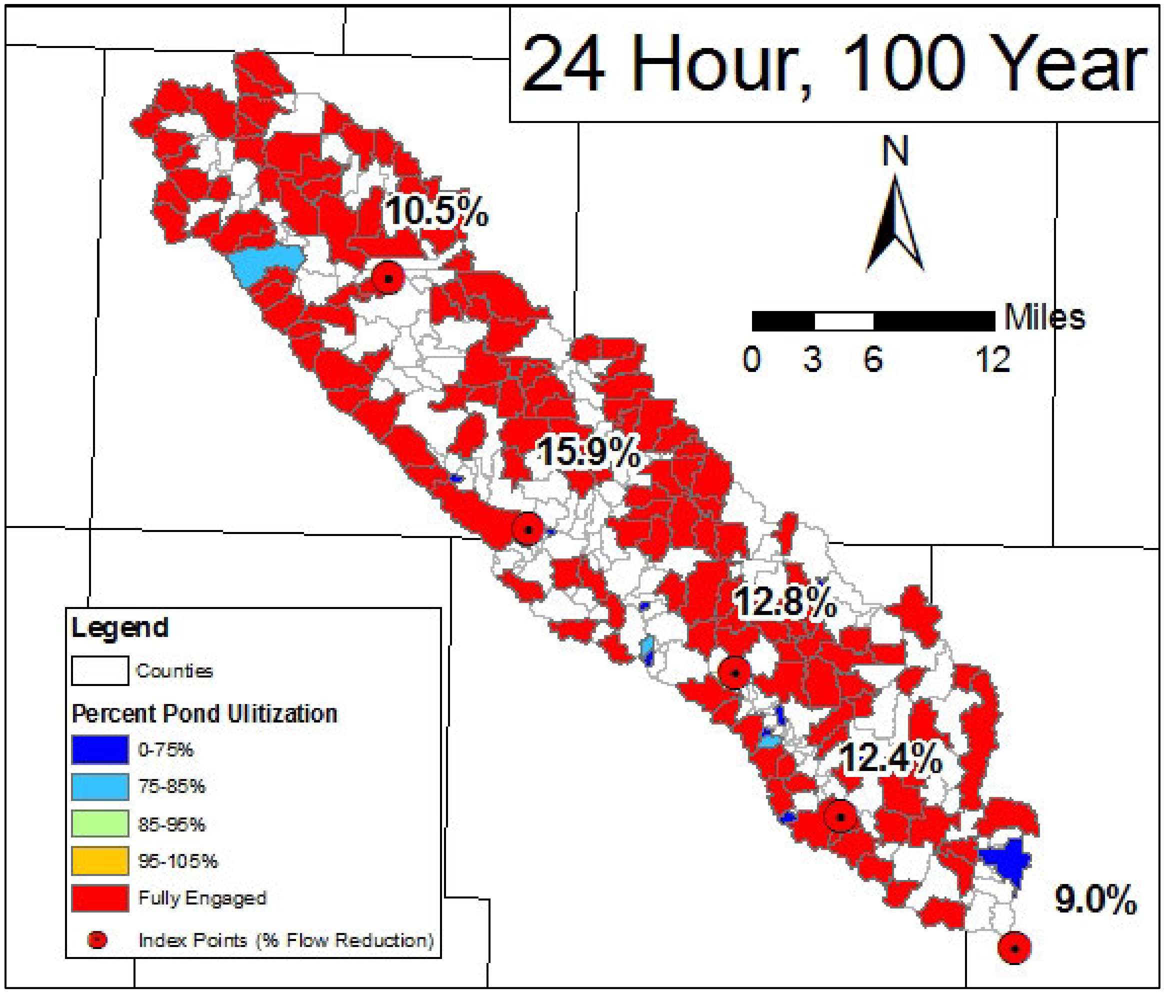

Figure A.15: Percent pond utilization and corresponding peak flow reductions for the large typical pond design. Run using the 100-year, 24-hour design storm (7.04 inches)

Dry Pond Design: Small Pond

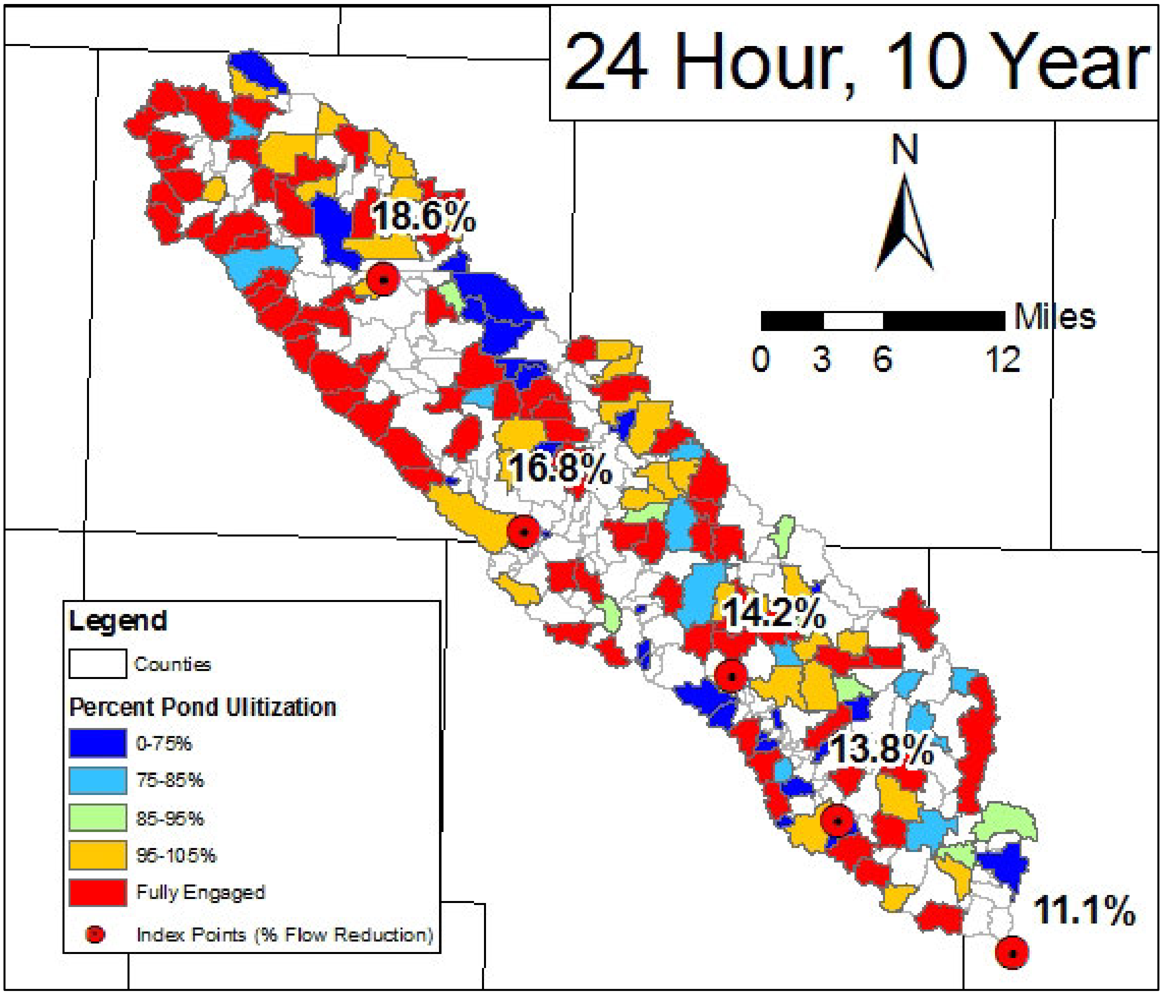

Figure A.16: Percent pond utilization and corresponding peak flow reductions for the small dry pond design. Run using the 10-year, 24-hour design storm (4.03 inches)

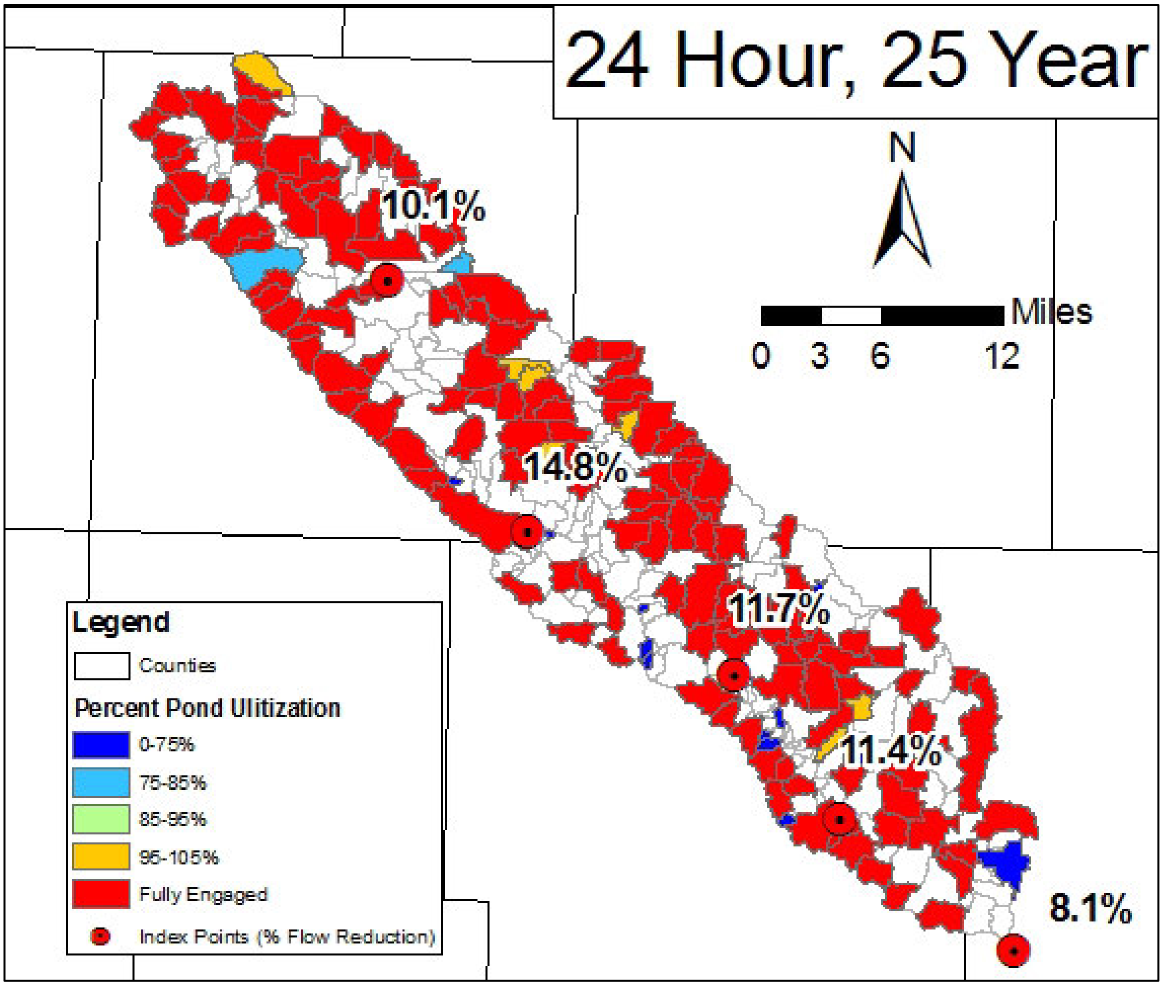

Figure A.17: Percent pond utilization and corresponding peak flow reductions for the small dry pond design. Run using the 25-year, 24-hour design storm (5.08 inches)

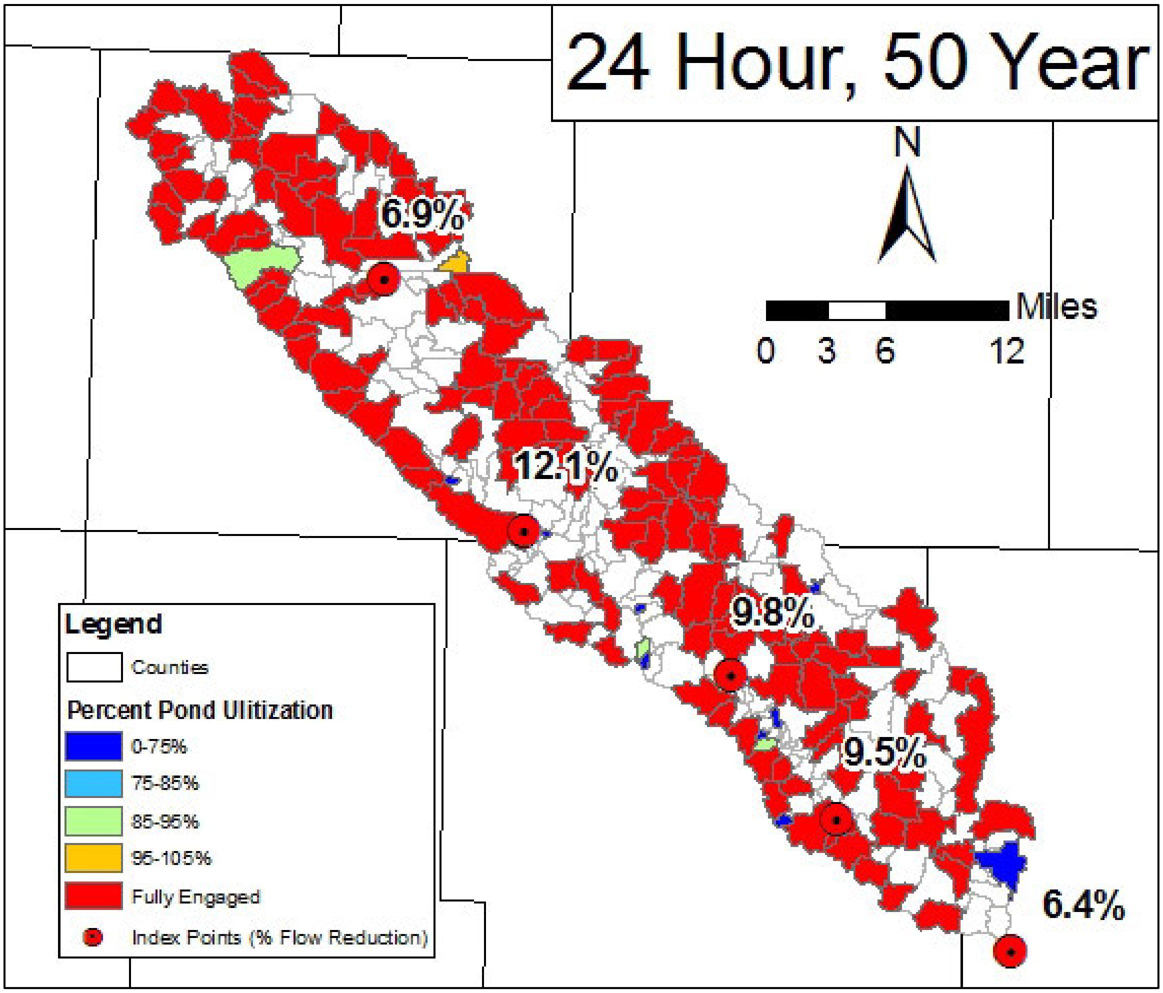

Figure A.18: Percent pond utilization and corresponding peak flow reductions for the small dry pond design. Run using the 50-year, 24-hour design storm (6.00 inches)

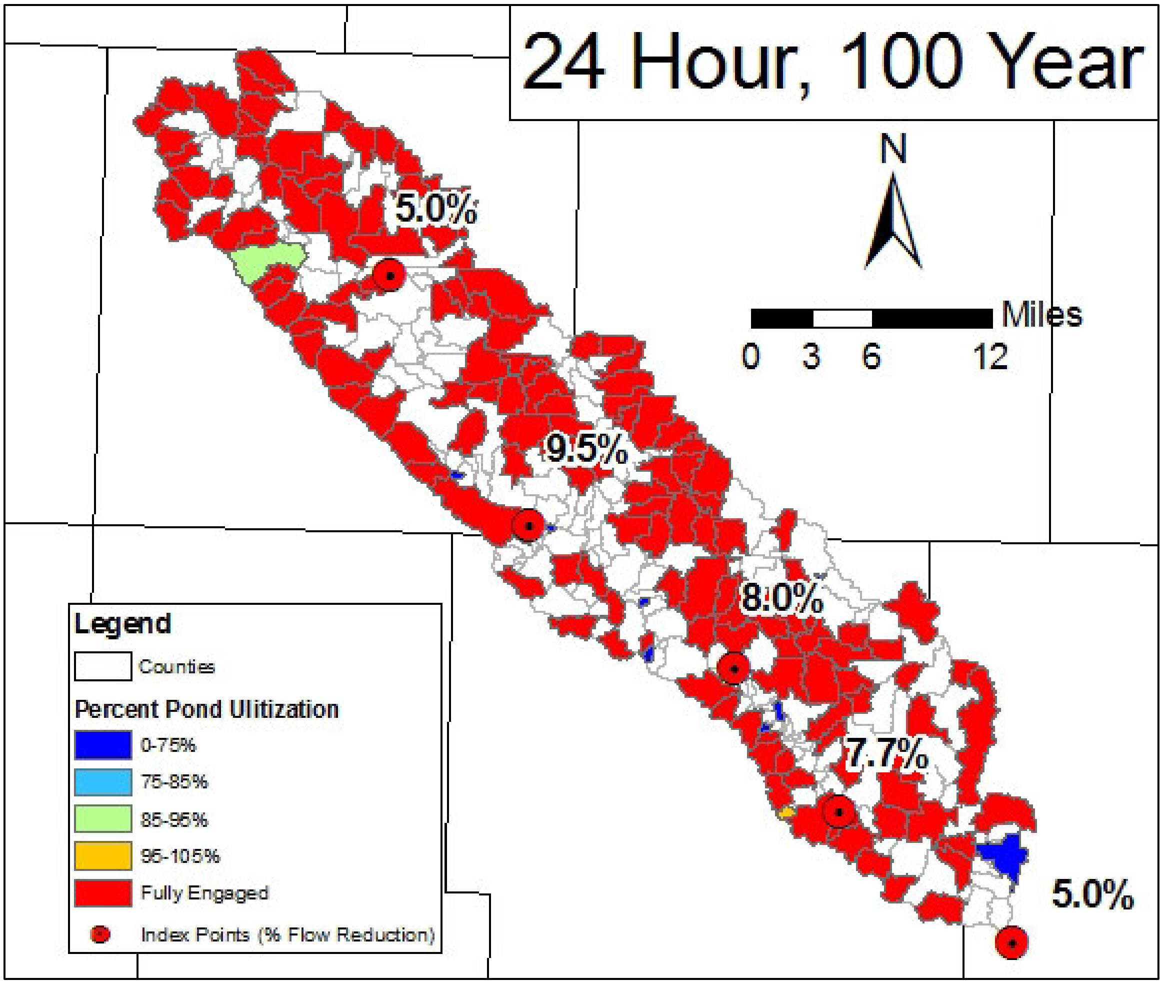

Figure A. 19: Percent pond utilization and corresponding peak flow reductions for the small dry pond design. Run using the 100-year, 24-hour design storm (7.04 inches)

Dry Pond Design: Large Pond

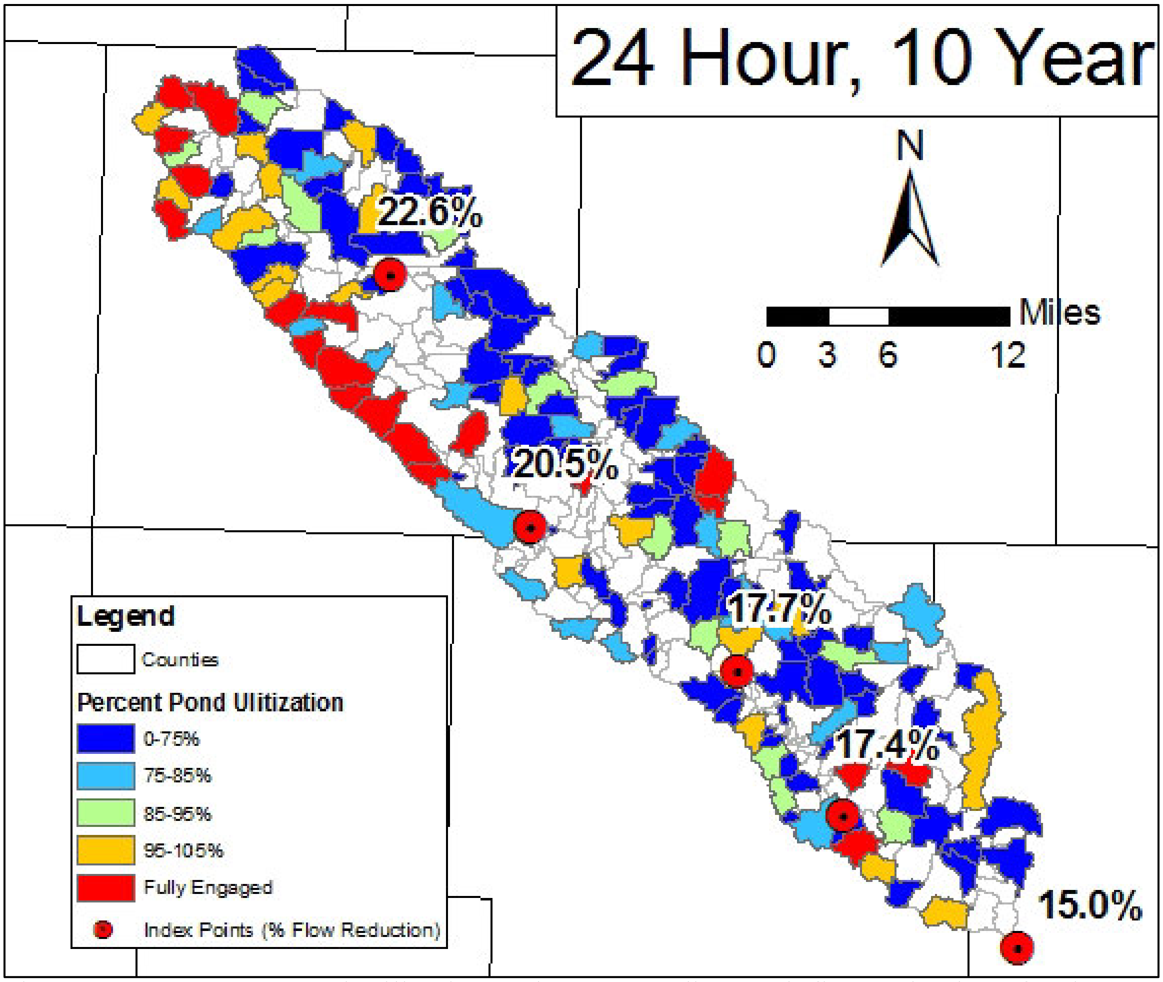

Figure A.20: Percent pond utilization and corresponding peak flow reductions for the large dry pond design. Run using the 10-year, 24-hour design storm (4.03 inches)

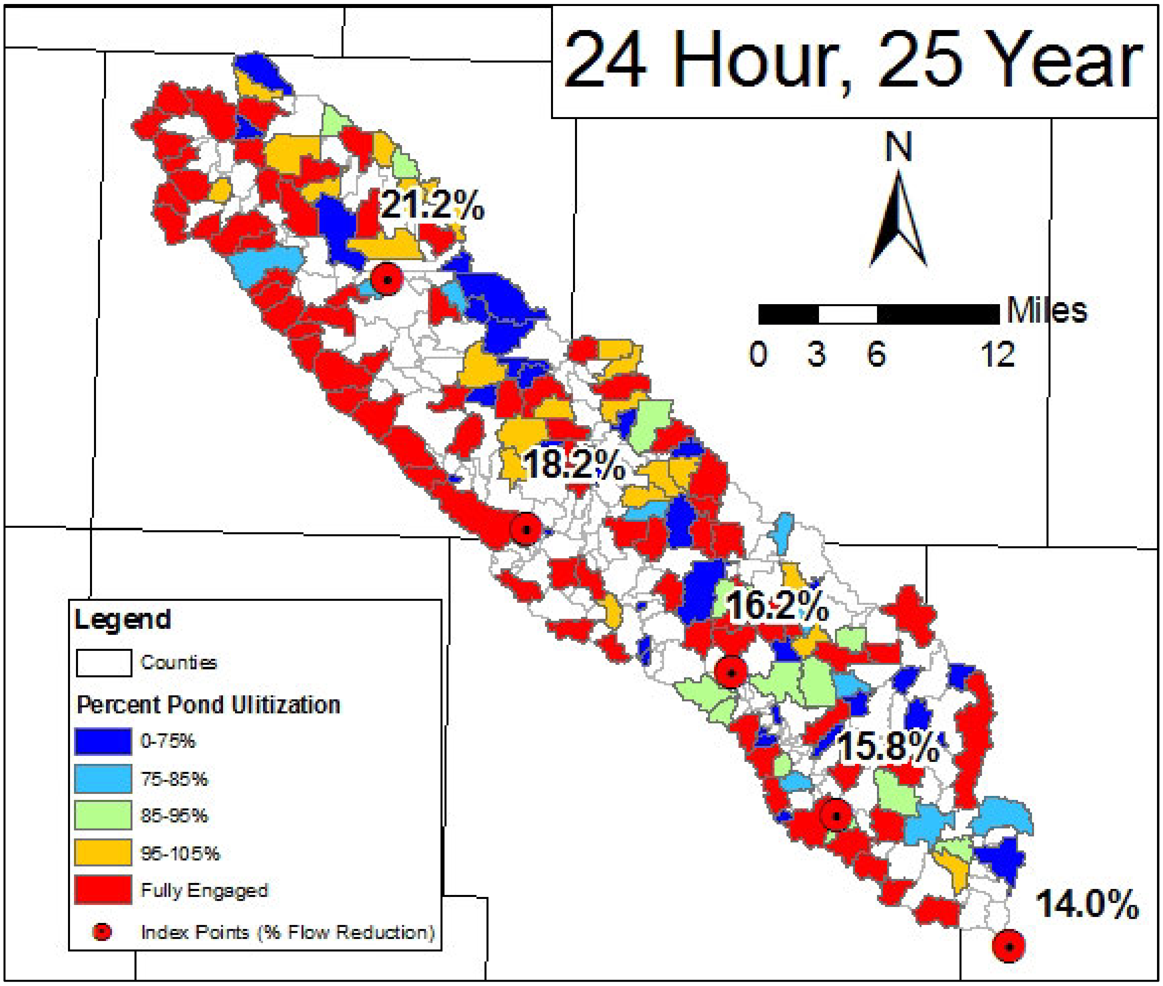

Figure A.21: Percent pond utilization and corresponding peak flow reductions for the large dry pond design. Run using the 25-year, 24-hour design storm (5.08 inches)

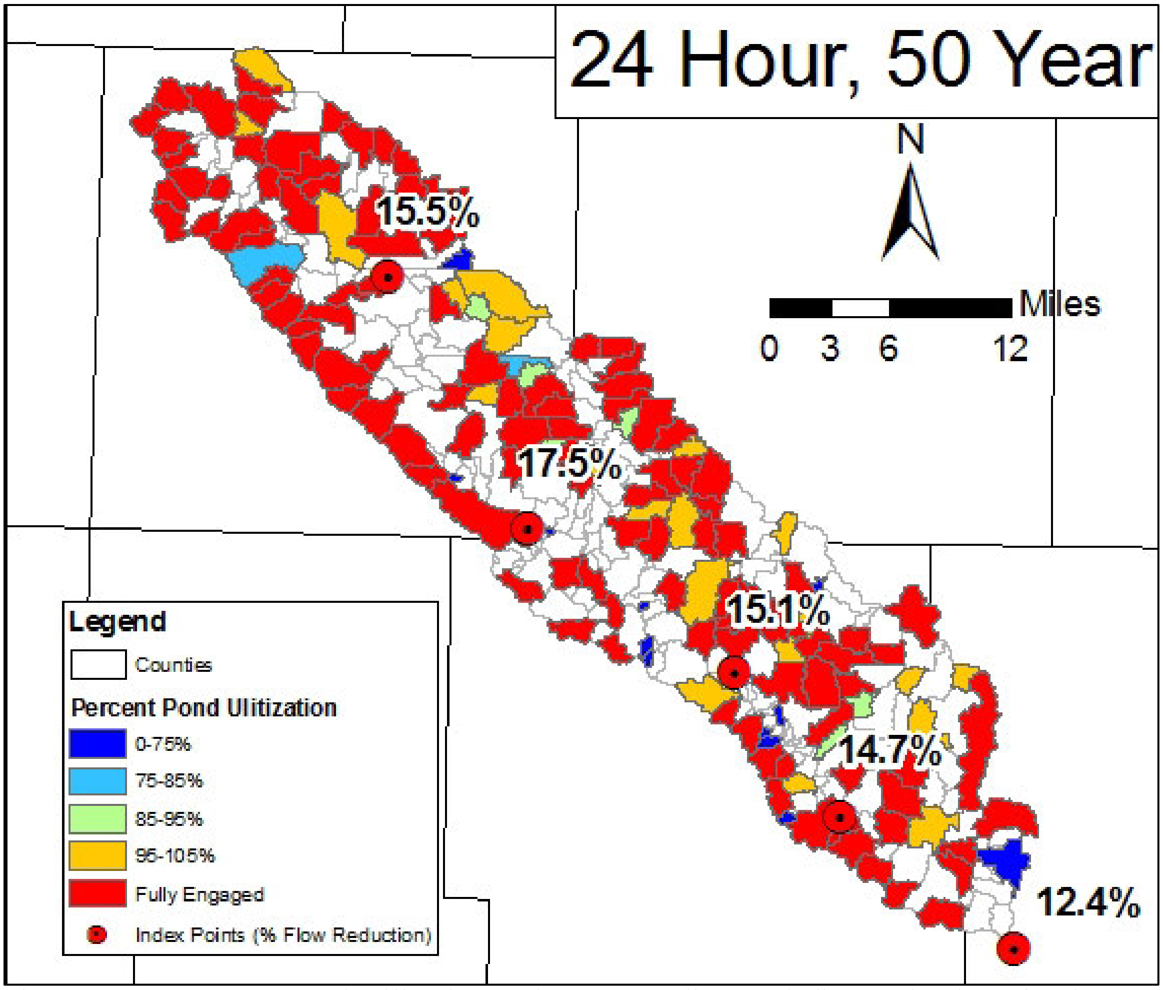

Figure A.22: Percent pond utilization and corresponding peak flow reductions for the large dry pond design. Run using the 50-year, 24-hour design storm (6.00 inches)

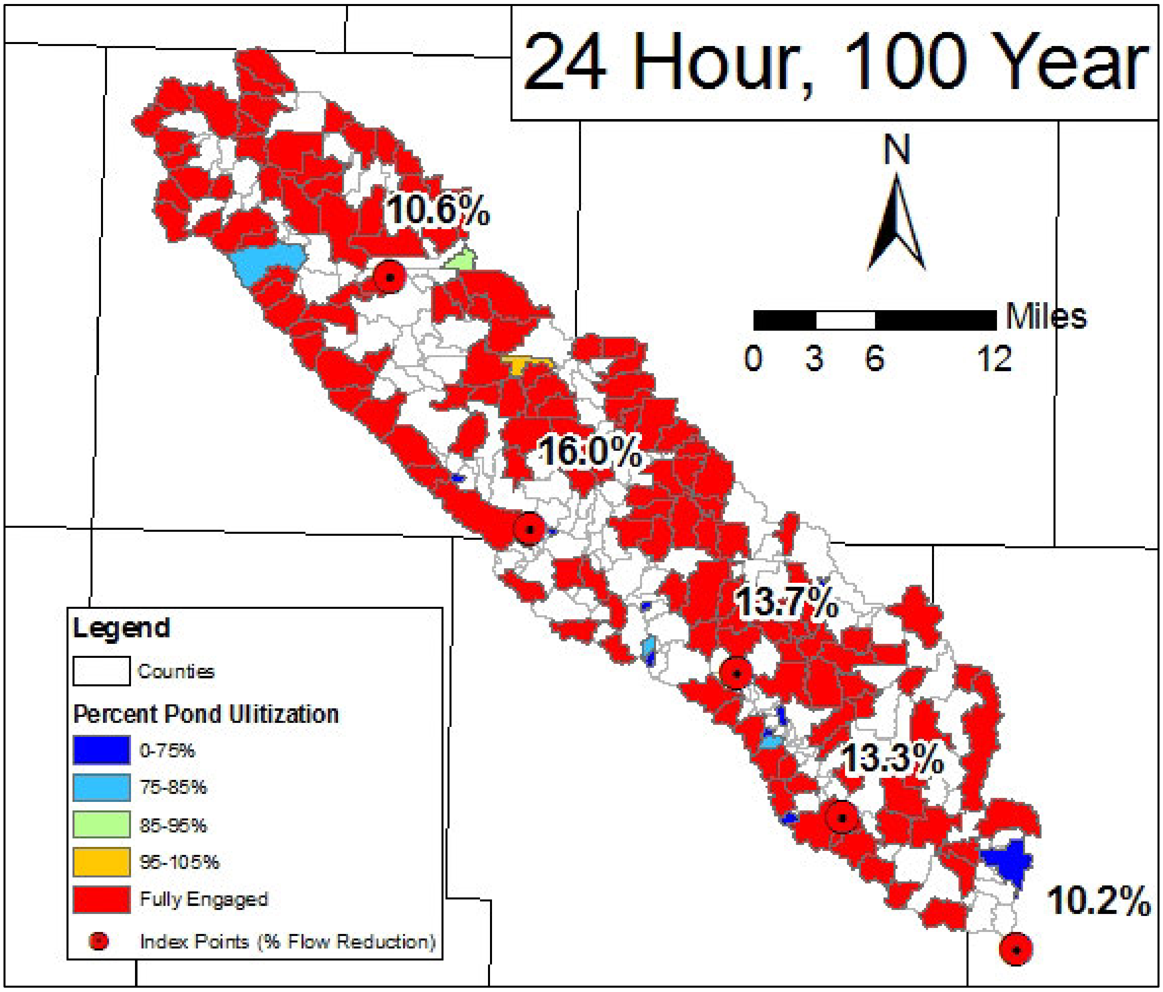

Figure A.23: Percent pond utilization and corresponding peak flow reductions for the large dry pond design. Run using the 100-year, 24-hour design storm (7.04 inches)

Cover Crop and Small Pond Design: Blended Scenario

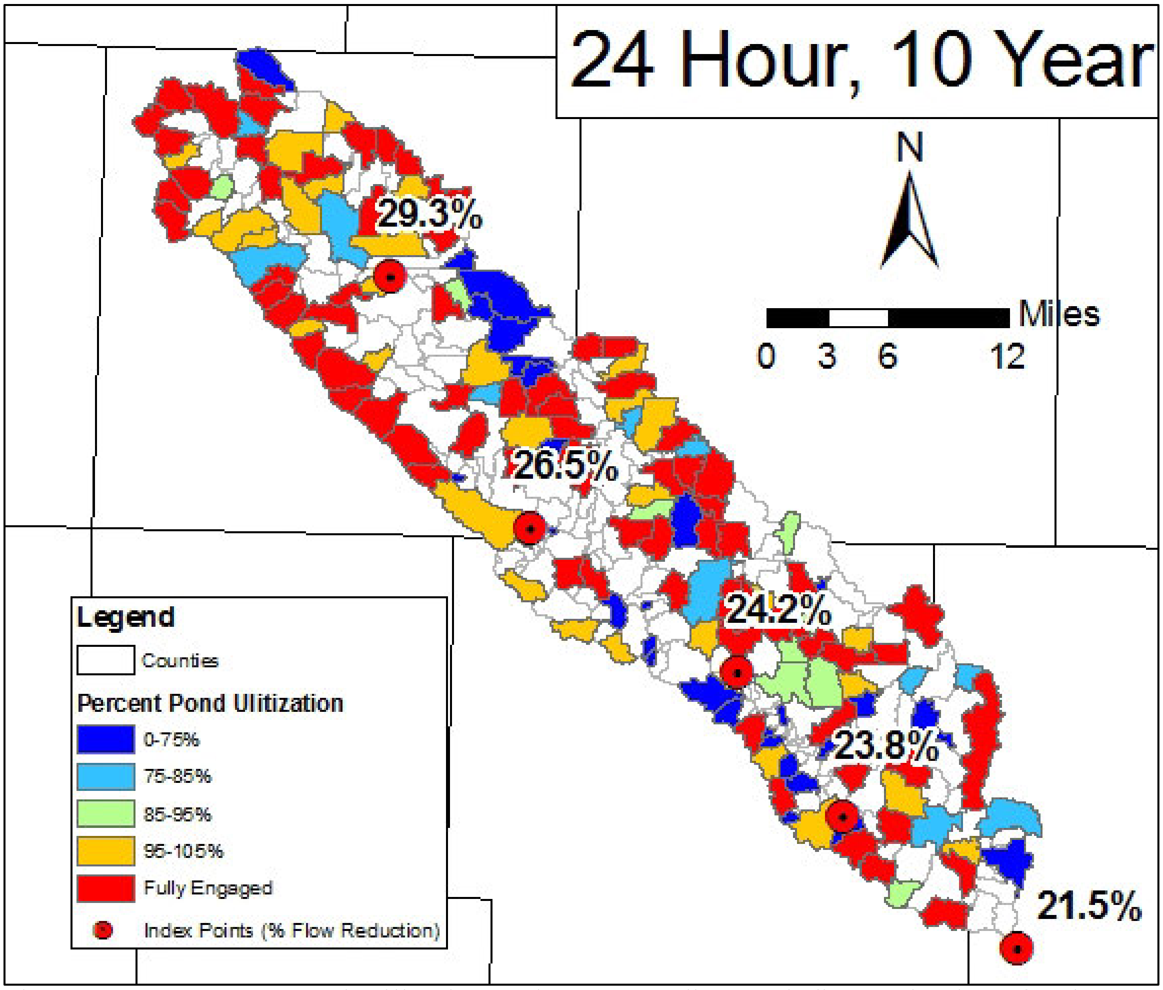

Figure A.24: Percent pond utilization and corresponding peak flow reductions for the small blended scenario. Run using the 10-year, 24-hour design storm (4.03 inches)

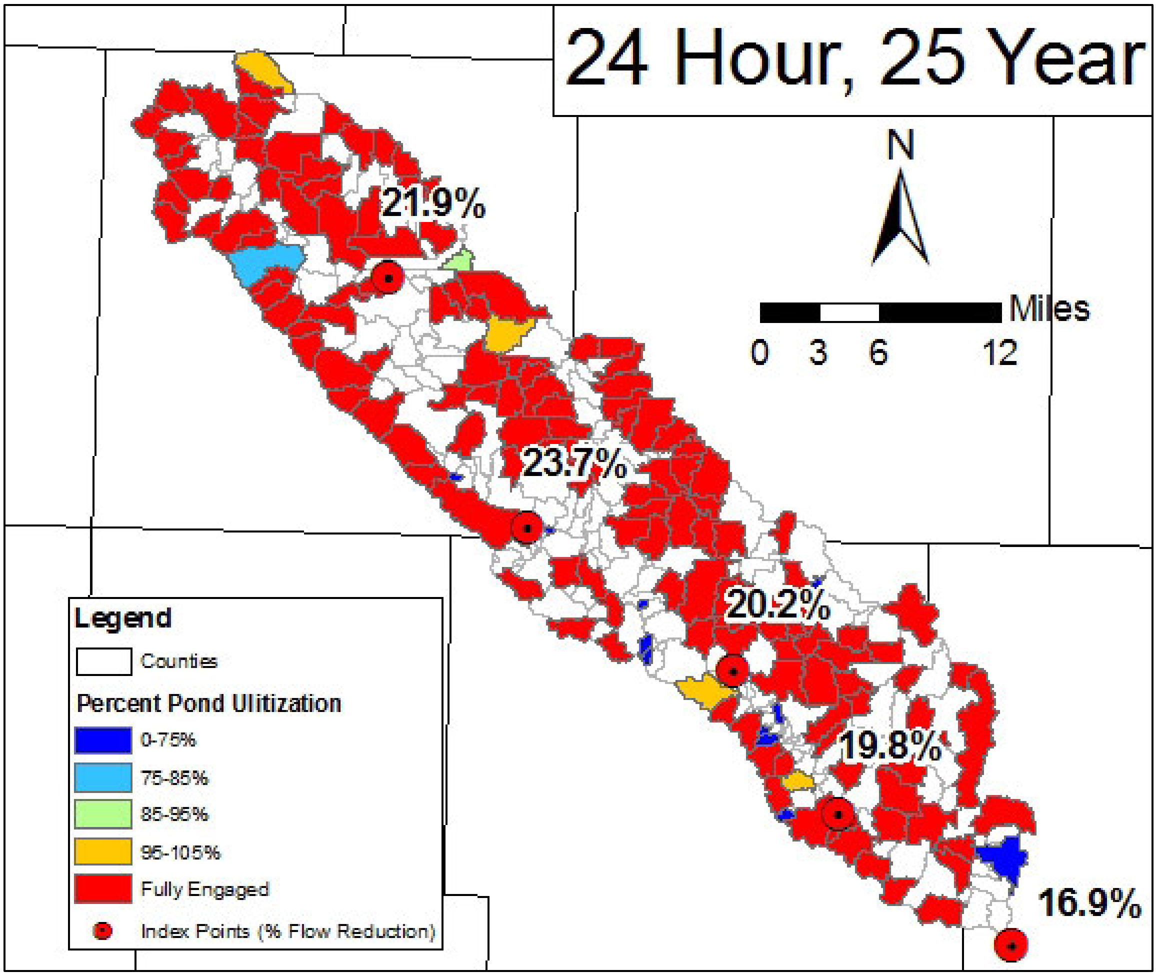

Figure A.25: Percent pond utilization and corresponding peak flow reductions for the small blended scenario. Run using the 25-year, 24-hour design storm (5.08 inches)

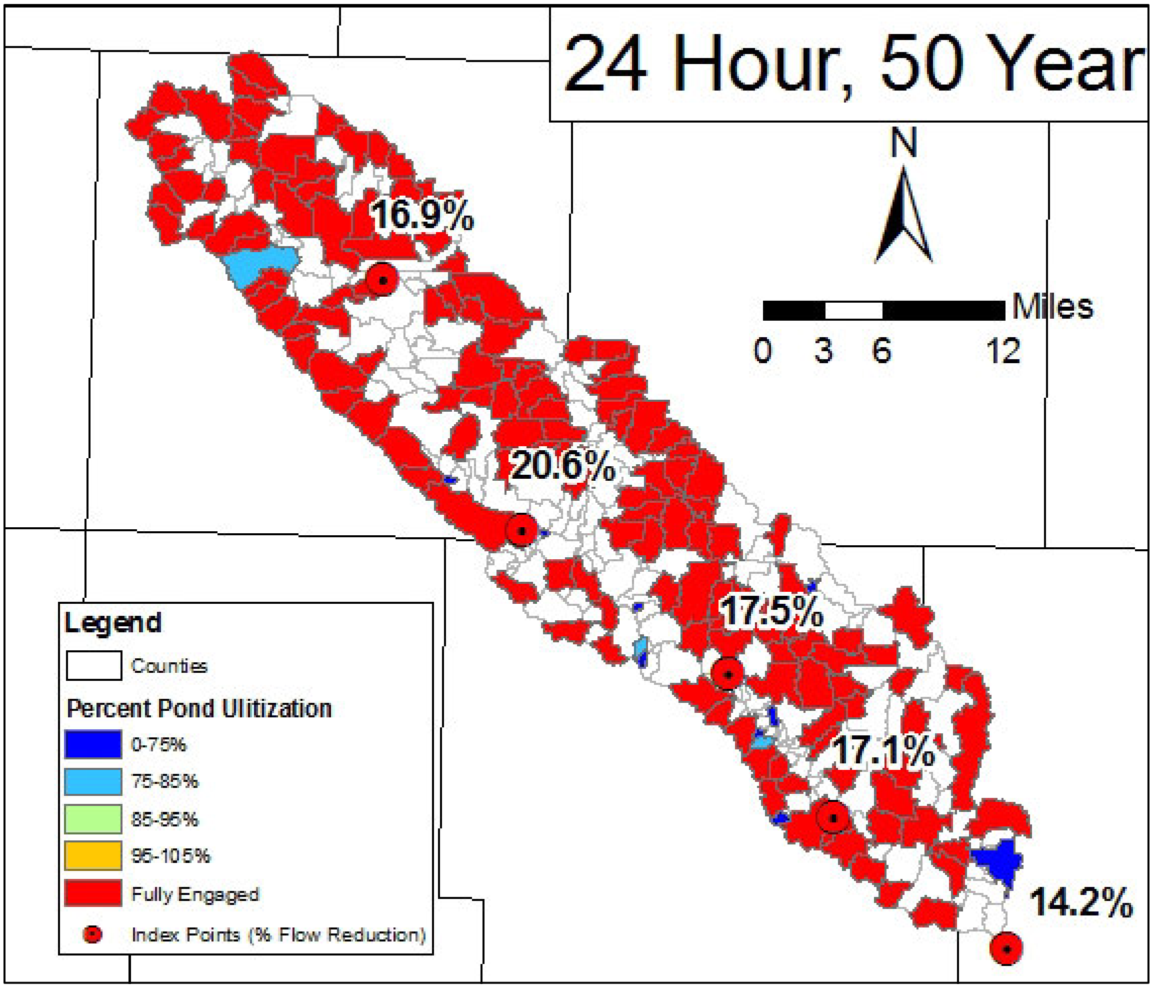

Figure A.26: Percent pond utilization and corresponding peak flow reductions for the small blended scenario. Run using the 50-year, 24-hour design storm (6.00 inches)

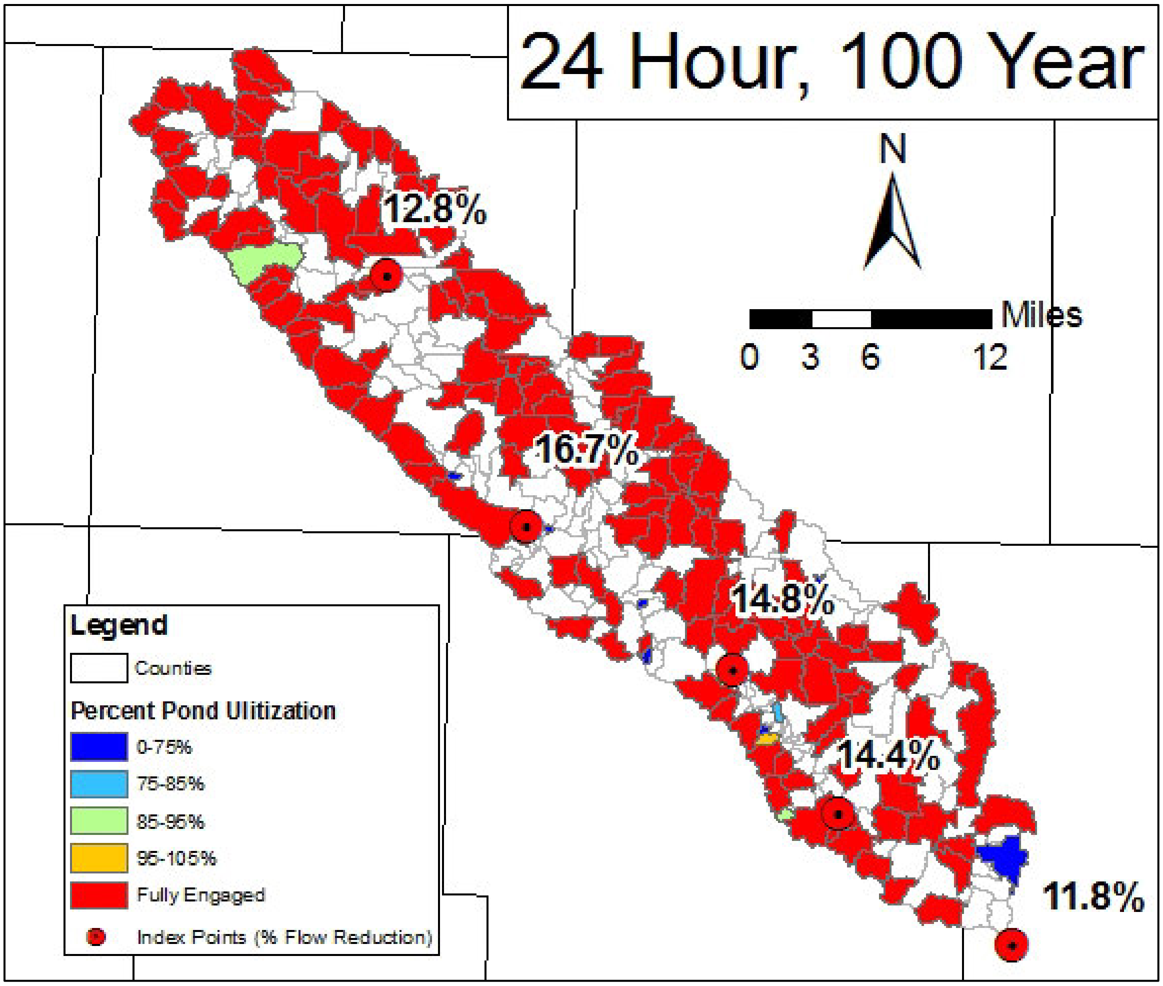

Figure A. 27: Percent pond utilization and corresponding peak flow reductions for the blended scenario. Run using the 100-year, 24-hour design storm (7.04 inches)

Cover Crop and Large Pond Design: Blended Scenario

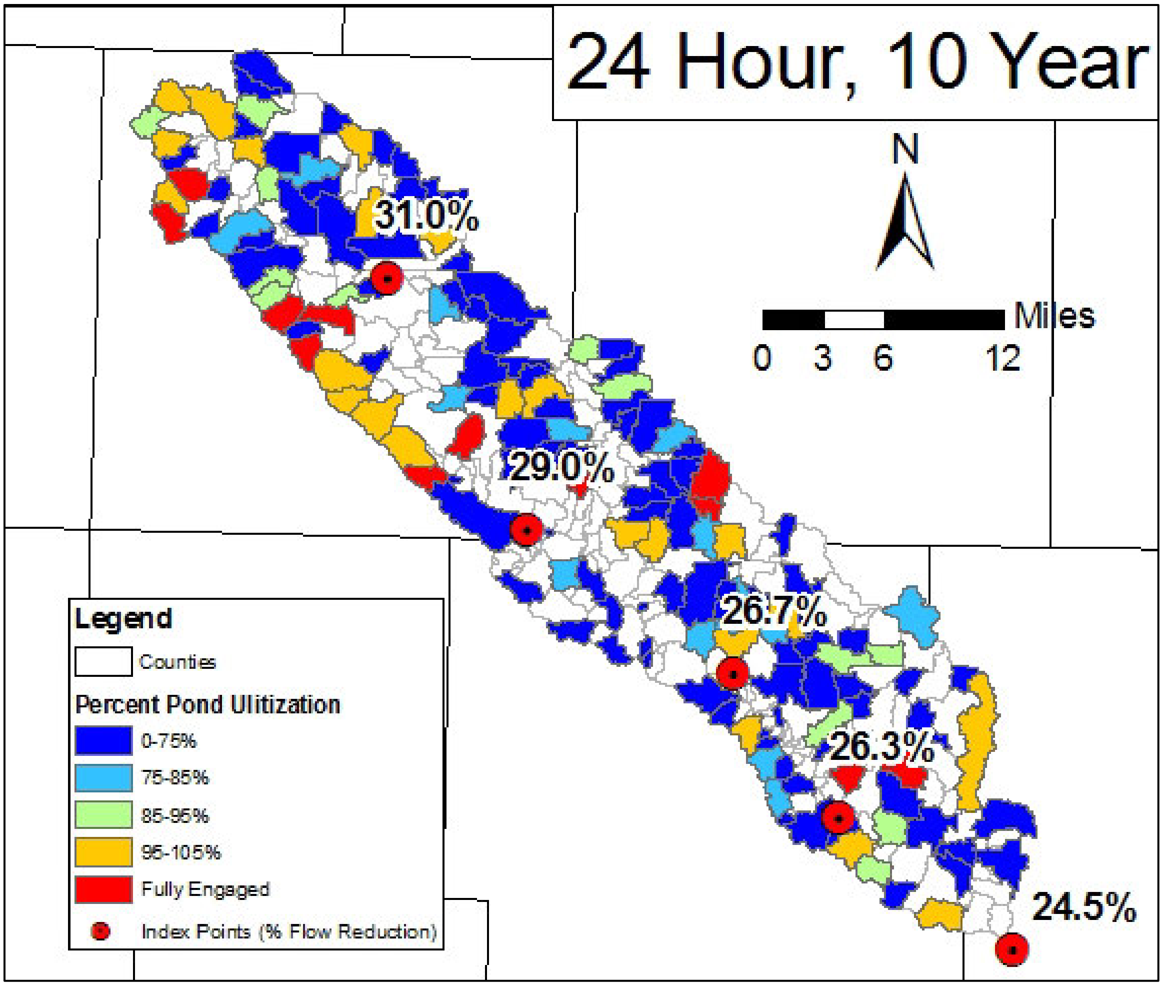

Figure A.28: Percent pond utilization and corresponding peak flow reductions for the large blended scenario. Run using the 10-year, 24-hour design storm (4.03 inches)

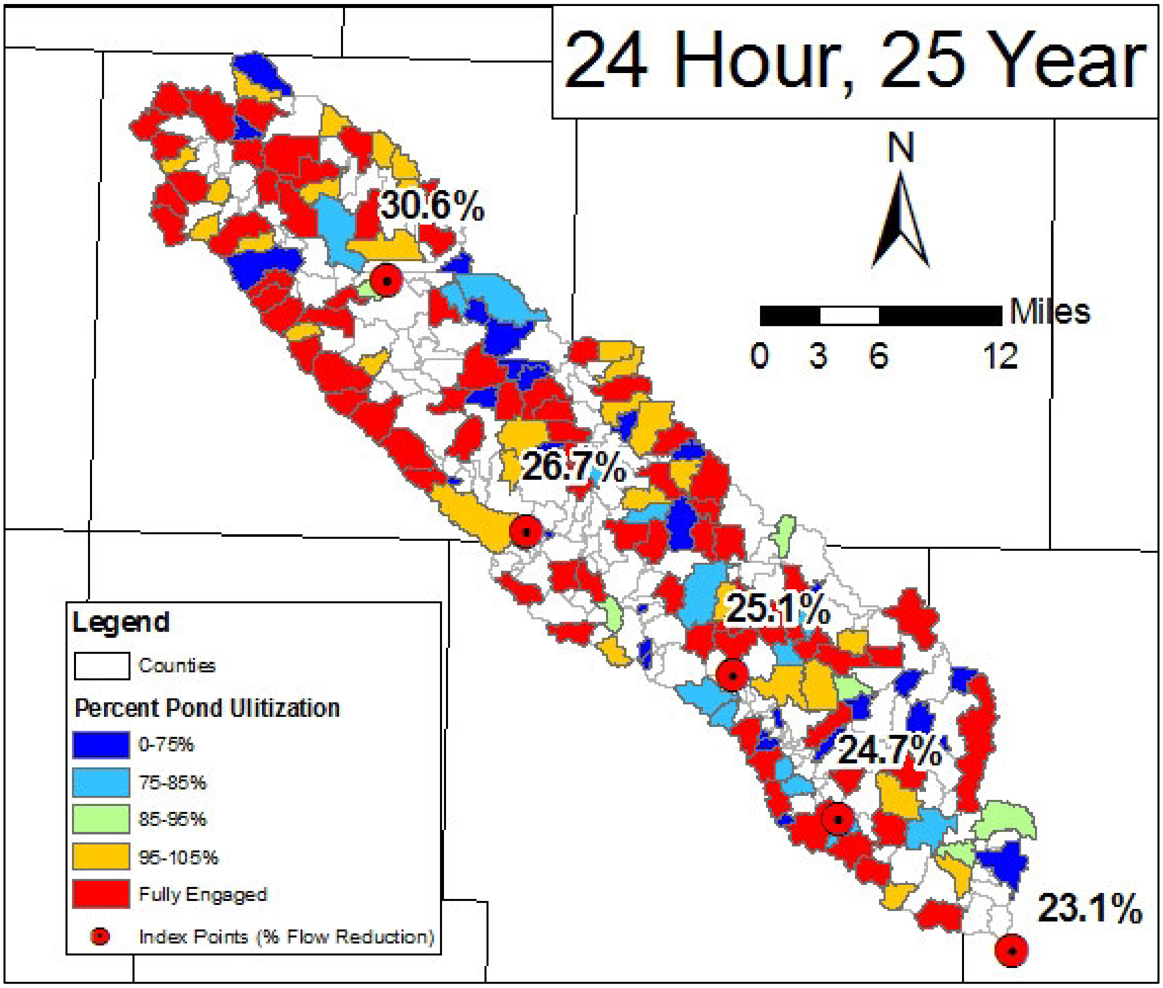

Figure A.29: Percent pond utilization and corresponding peak flow reductions for the large blended scenario. Run using the 25-year, 24-hour design storm (5.08 inches)

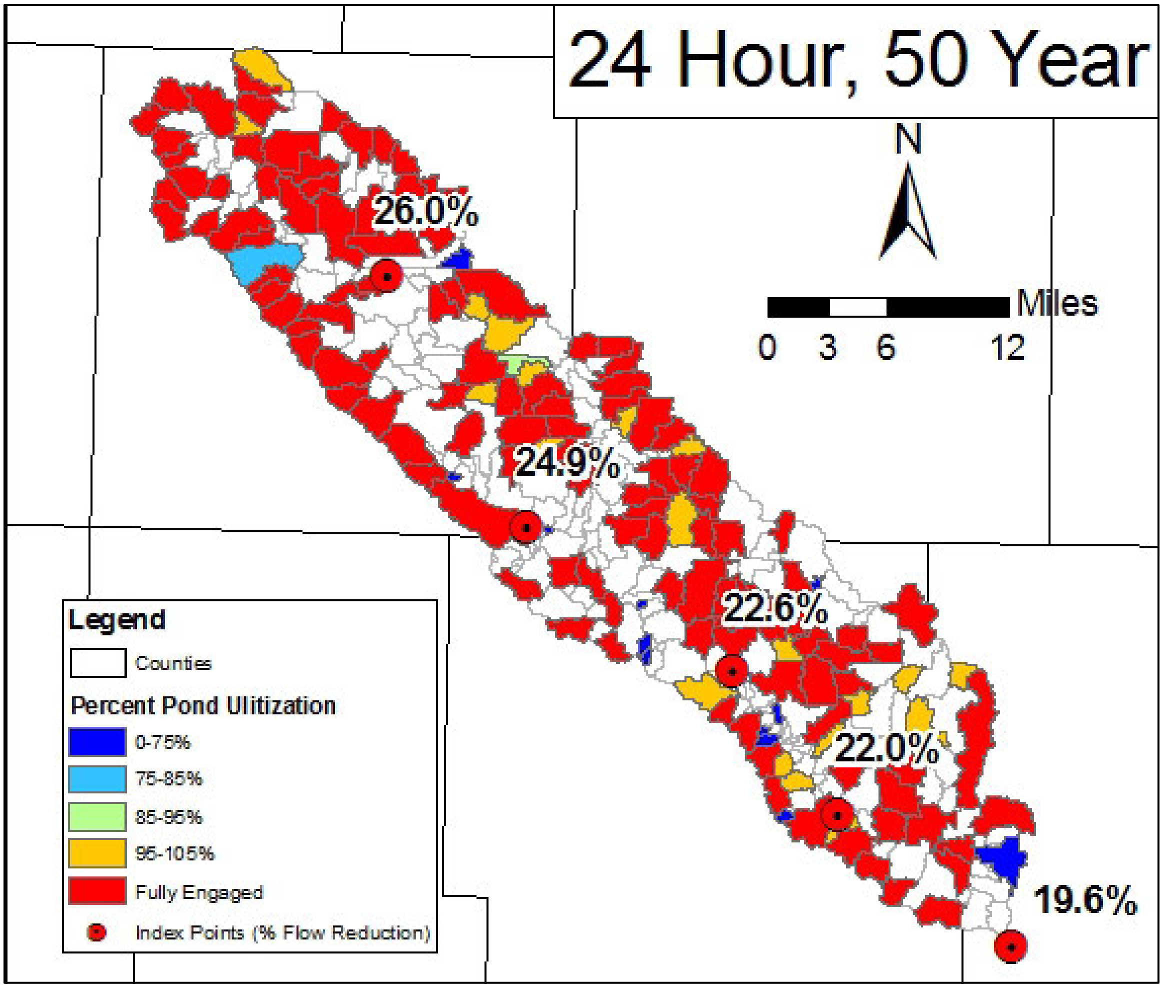

Figure A.30: Percent pond utilization and corresponding peak flow reductions for the large blended scenario. Run using the 50-year, 24-hour design storm (6.00 inches)

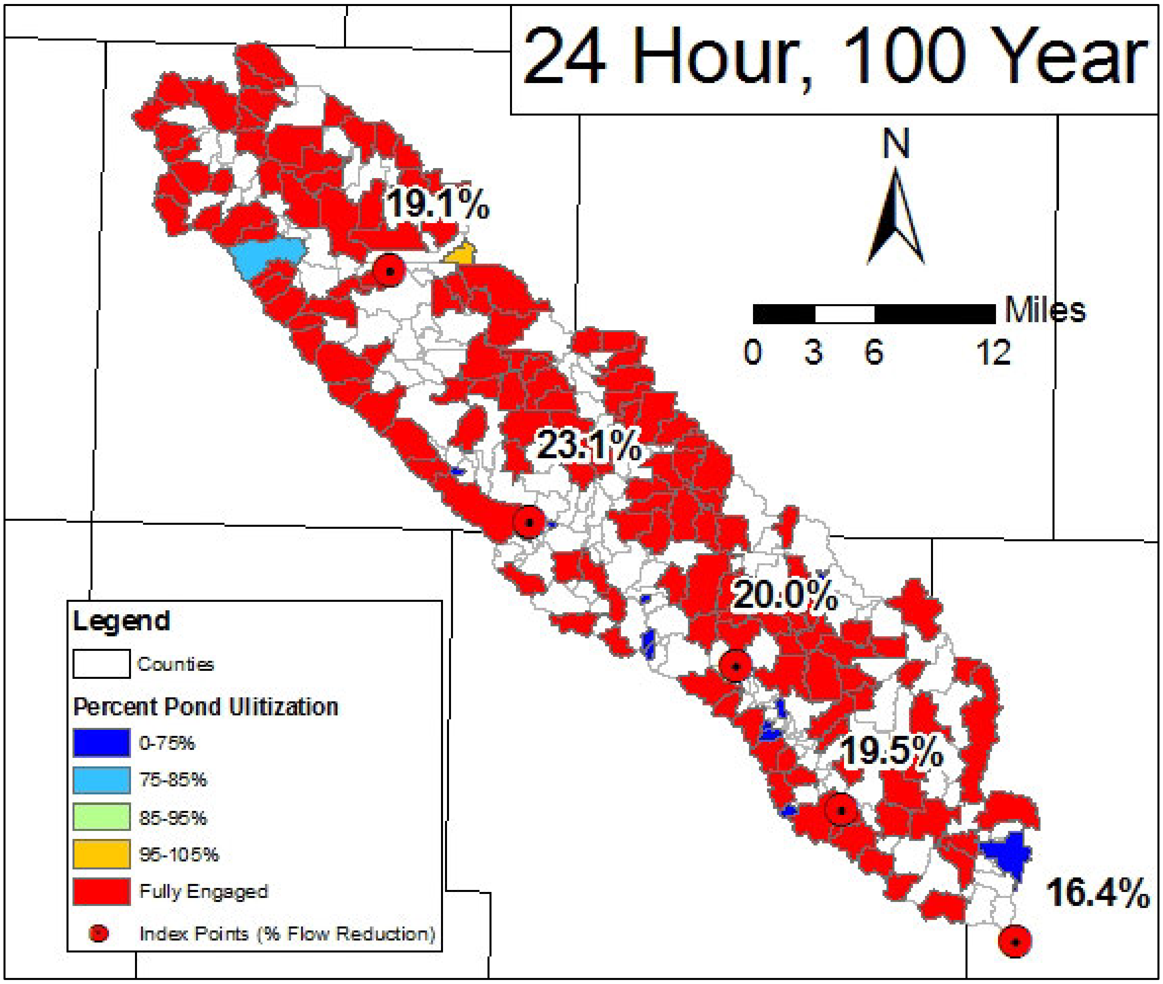

Figure A. 31: Percent pond utilization and corresponding peak flow reductions for the blended scenario. Run using the 100-year, 24-hour design storm (7.04 inches)

Historic Rainfall Event: June 2008

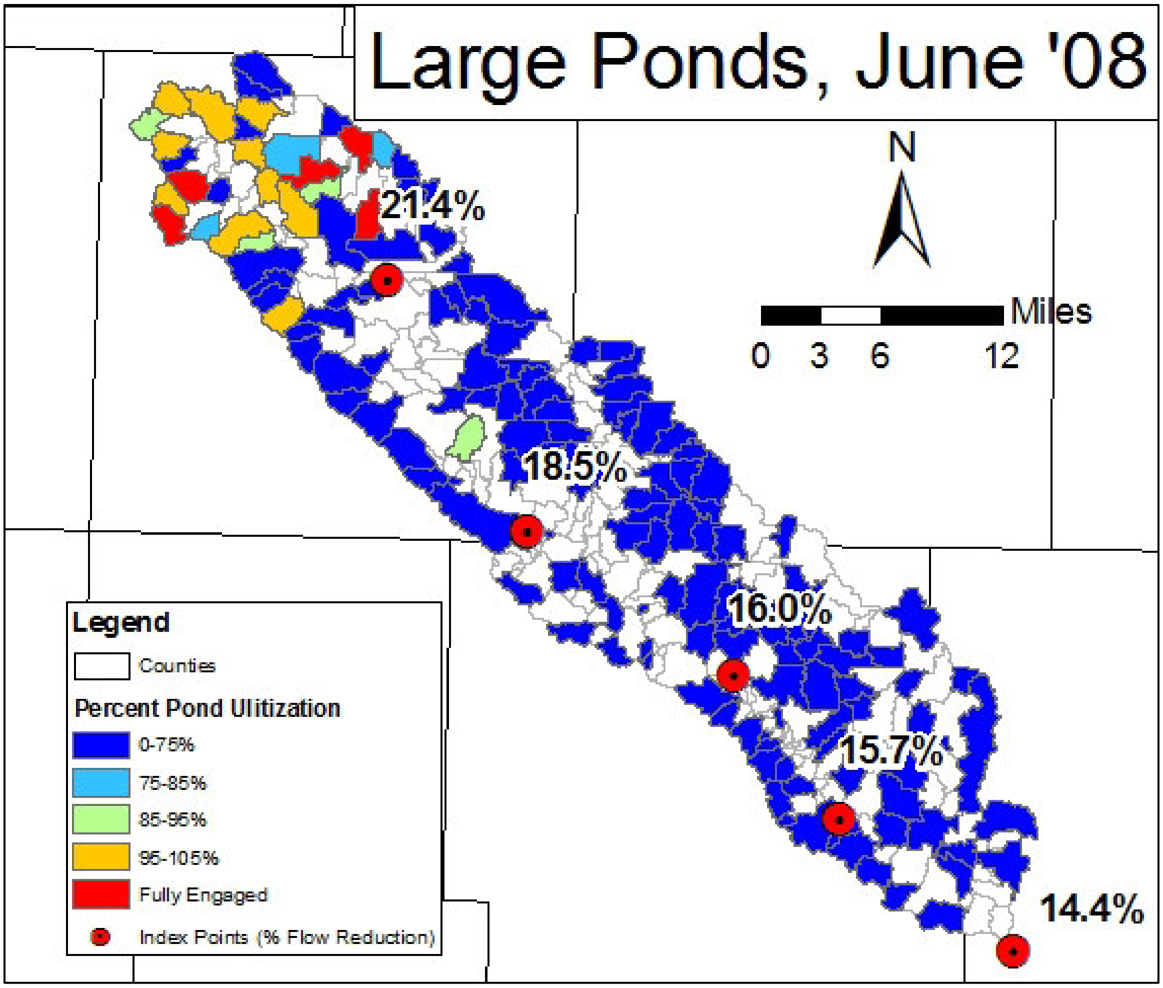

Figure A.32: Percent pond utilization and corresponding peak flow reductions for the large pond scenario. Run using the June 2008 rainfall event.

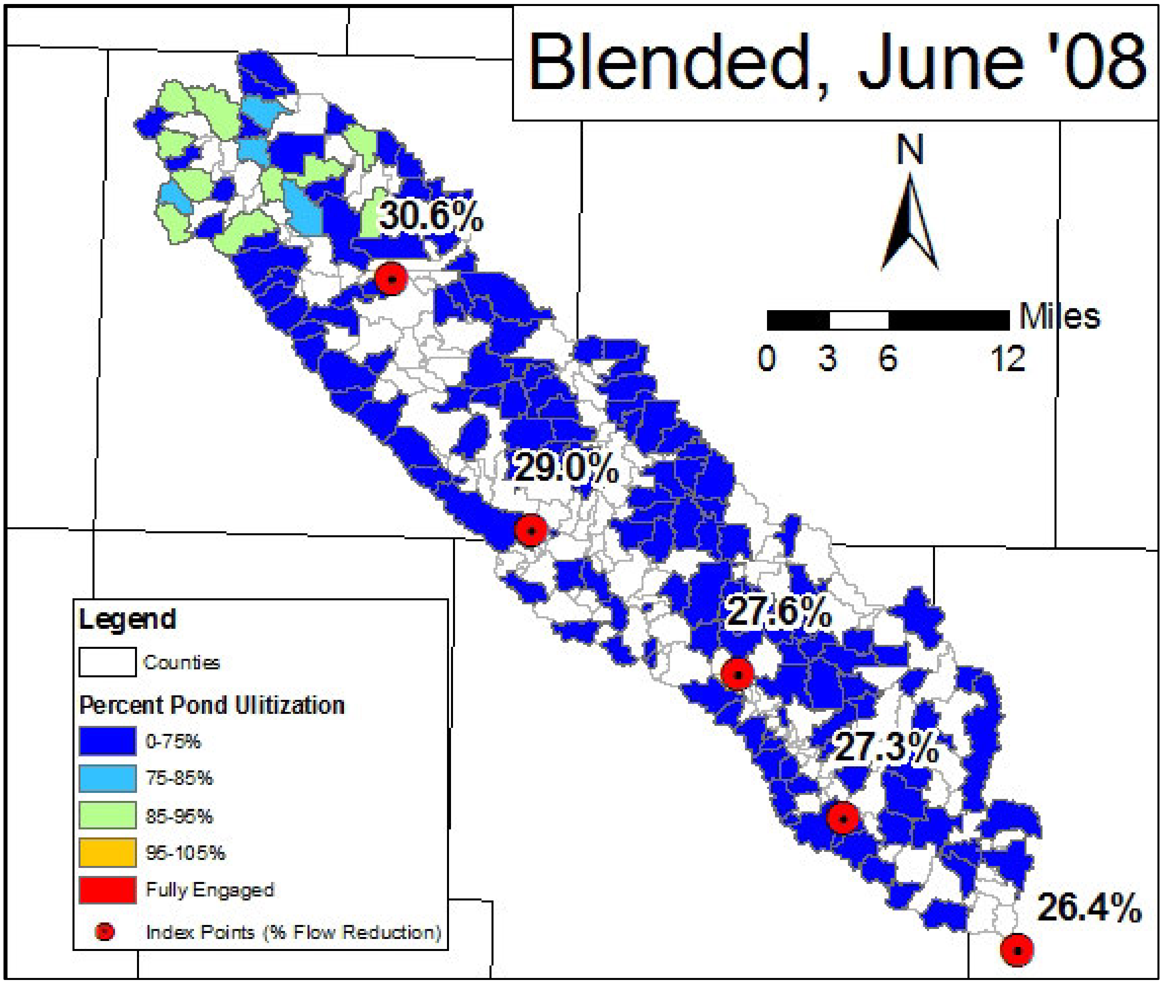

Figure A.33: Percent pond utilization and corresponding peak flow reductions for the large blended scenario. Run using the June 2008 rainfall event.

Historic Rainfall Event: June 2013

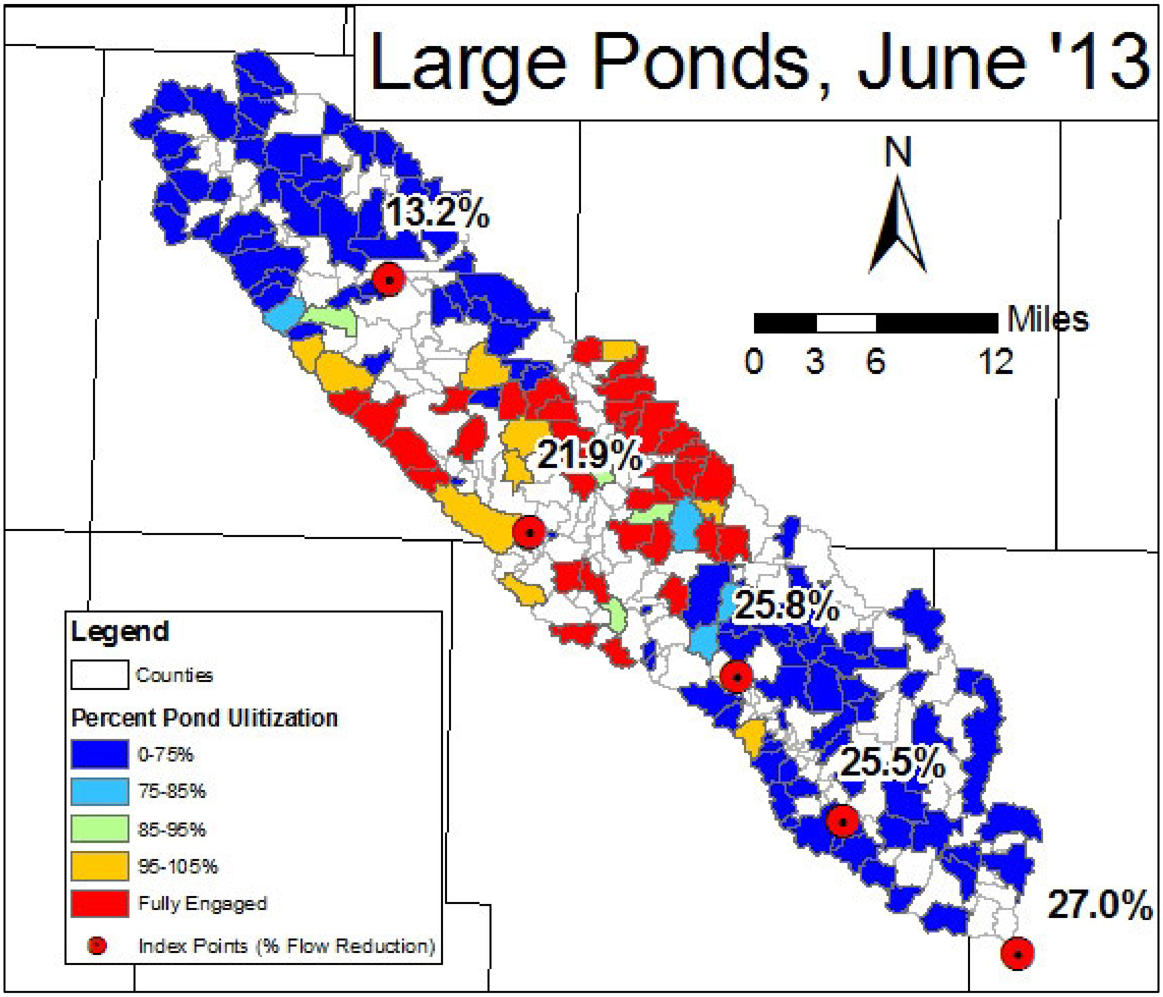

Figure A.34: Percent pond utilization and corresponding peak flow reductions for the large pond scenario. Run using the June 2013 rainfall event.

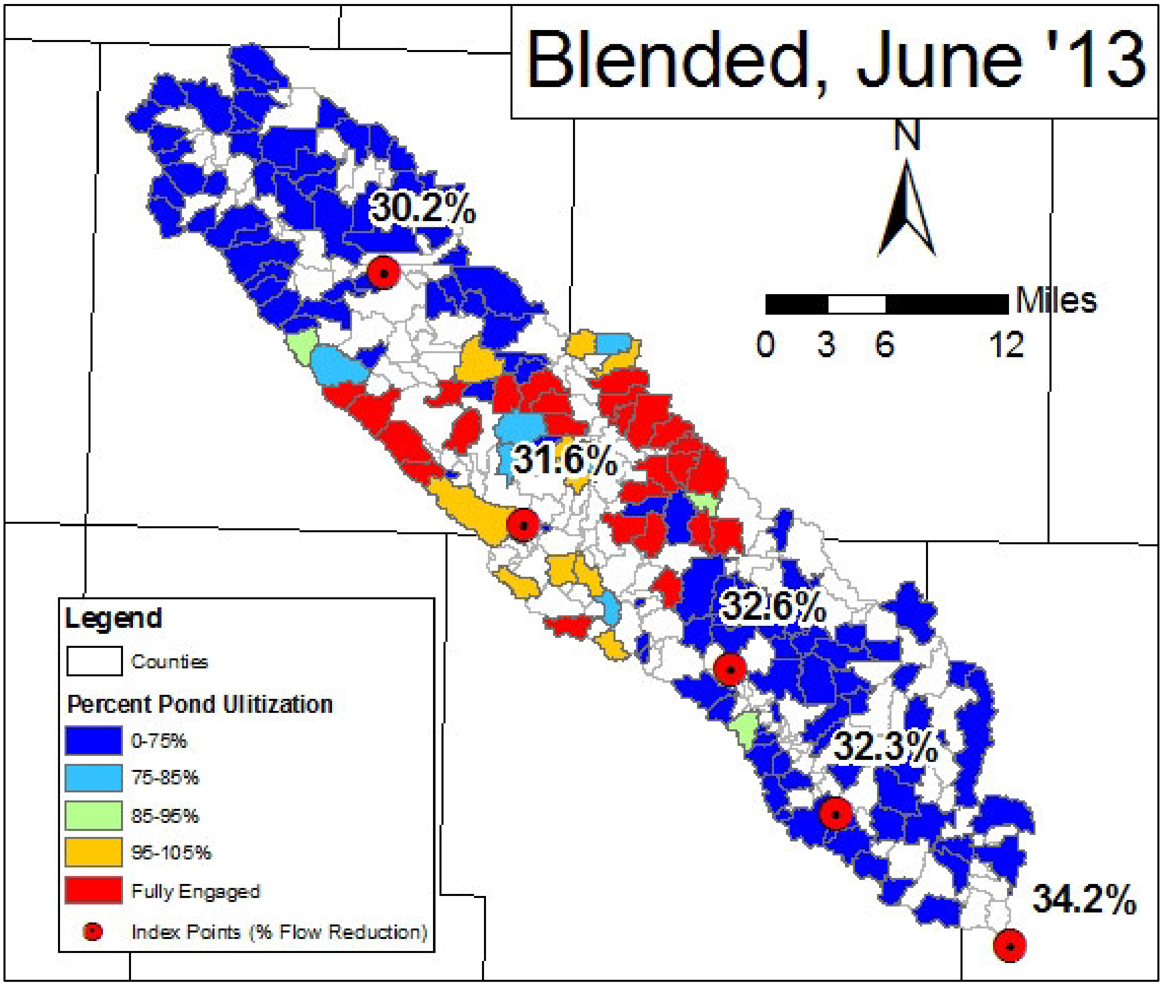

Figure A.35: Percent pond utilization and corresponding peak flow reductions for the large blended scenario. Run using the June 2013 rainfall event.

APPENDIX B – SUPPLEMENTAL TABLES

Storage-Elevation-Discharge Tables: Existing Structures

Lake Panorama – Discharges translated from discharges at Bayard, IA USGS Stream Gauge location to water released at the Lake Panorama Bascule Gate, via specified discharge. Therefore hydrographs at Bayard and hydrographs being released from Lake Panorama are identical.

Table B.1: Bays Branch elevation-storage-discharge relationship.

Elevation (ft)	Storage (ac-ft)	Discharge (cfs)
1040	1088	0
1041	1338	125
1042	1588	375
1043	1838	750
1044	2088	1125
1045	2343	1525
1046	2608	1950
1047	3128	2700

Storage-Discharge Tables: Typical Pond Design

Table B.2: Typical small pond stage-discharge relationship in the Des Moines Lobe Region

Elevation Above Primary Spillway (ft)	Storage (ac-ft)	Outflow Pipe (cfs)	Outflow Emergency Spillway (cfs)	Total Outflow (cfs)
0	0	0	0	0
1	5.9	2.2	0	2.2
2	15.3	11.1	0	11.1
3	26.8	11.5	0	11.5
3.5	33.2	11.7	14	25.7
4	40.0	11.9	40	51.9
4.5	47.1	12.1	80	92.1
5	54.5	12.3	140	152.3
5.5	62.2	12.45	448.1	460.55
6	70.2	12.6	609.1	621.7
6.5	78.4	12.8	1099.7	1112.5
7	86.9	13	1370.6	1383.6
7.5	95.7	13.2	1787.9	1801.1
8	104.6	13.4	2107.6	2121.0
8.5	113.8	13.6	2492.4	2506.0
9	123.2	13.8	2833.8	2847.6
10	142.5	14.1	3567.3	3581.4

Table B.3: Typical small pond stage-discharge relationship in the Southern Iowa Drift Plain region

Elevation Above Primary Spillway (ft)	Storage (ac-ft)	Outflow Pipe (cfs)	Outflow Emergency Spillway (cfs)	Total Outflow (cfs)
0	0	0	0	0
1	1.7	2.2	0	2.2
2	4.3	11.1	0	11.1
3	7.5	11.5	0	11.5
4	11.1	11.9	0	11.9
5	15.0	12.3	0	12.3
5.5	17.1	12.45	0	12.45
6	19.3	12.6	0	12.6
6.5	21.5	12.8	0	12.8
7	23.8	13	0	13
7.5	26.1	13.2	14	27.2
8	28.5	13.4	40	53.4
8.5	31.0	13.6	80	93.5766
9	33.5	13.8	140	153.7654
9.5	36.0	14.0	448.1	462.0542
10	38.6	14.1	609.1	623.243
10.5	41.3	15.6	1099.7	1115.34

Table B.4: Typical large pond stage-discharge relationship in the Des Moines Lobe Region

Elevation Above Primary Spillway (ft)	Storage (ac-ft)	Outflow Pipe (cfs)	Outflow Emergency Spillway (cfs)	Total Outflow (cfs)
0	0	0	0	0
1	5.9	2.2	0	2.2
2	15.3	11.1	0	11.1
3	26.8	11.5	0	11.5
4	40.0	11.9	0	11.9
5	54.5	12.3	0	12.3
5.5	62.2	12.45	14	26.45
6	70.2	12.6	40	52.6
6.5	78.4	12.8	80	92.8
7	86.9	13	140	153
7.5	95.7	13.2	448.1	461.3
8	104.6	13.4	609.1	622.5
9	123.2	15.64	1099.7	1115.34

Table B.5: Typical large pond stage-discharge relationship in the Southern Iowa Drift Plain region

Elevation Above Primary Spillway (ft)	Storage (ac-ft)	Outflow Pipe (cfs)	Outflow Emergency Spillway (cfs)	Total Outflow (cfs)
0	0	0	0	0.00
1	1.7	2.2	0	2.20
2	4.3	11.1	0	11.10
3	7.5	11.5	0	11.50
4	11.1	11.9	0	11.90
5	15.0	12.3	0	12.30
5.5	17.1	12.45	0	12.45
6	19.3	12.6	0	12.60
6.5	21.5	12.8	0	12.80
7	23.8	13	0	13.00
7.5	26.1	13.2	0	13.20
8	28.5	13.4	0	13.40
8.5	31.0	13.6	0	13.58
9	33.5	13.8	0	13.77
9.5	36.0	14.0	0	13.95
10	38.6	14.1	0	14.14
10.5	41.3	15.6	14	29.64
11	44.0	16.9	40	56.89
11.5	46.7	17.4	80	97.42
12	49.5	18.0	140	157.95
13	55.2	19.0	609	628.02

Storage-Discharge Tables: Dry Pond Design

Table B.6: Small dry pond stage-discharge relationship in the Des Moines Lobe Region.

Elevation Above Primary Spillway (ft)	Storage (ac-ft)	2" Pipe Outflow	Primary Discharge (cfs)	Outflow Emergency Spillway (cfs)	Total Outflow (cfs)
0	0.00	0.00	0	0	0.00
1	0.12	0.94	0	0	0.94
2	0.78	1.32	0	0	1.32
3	2.35	1.62	0	0	1.62
4	5.16	1.87	0	0	1.87
5	9.48	2.09	0.00	0	2.09
6	15.59	2.29	2.20	0	4.49
7	23.73	2.48	11.10	0	13.58
8	34.16	2.65	11.50	0	14.15
9	47.11	2.81	11.90	40	54.71
10	62.80	2.96	12.30	140	155.26
11	81.44	3.11	12.60	609	624.81

Table B.7: Large dry pond stage-discharge relationship in the Des Moines Lobe Region.

Elevation Above Primary Spillway (ft)	Storage (ac-ft)	2” Pipe Outflow	Primary Discharge (cfs)	Outflow Emergency Spillway (cfs)	Total Outflow (cfs)
0	0.00	0.00	0.00	0	0.00
1	0.12	0.94	0.00	0	0.94
2	0.78	1.32	0.00	0	1.32
3	2.35	1.62	0.00	0	1.62
4	5.16	1.87	0.00	0	1.87
5	9.48	2.09	0.00	0	2.09
6	15.59	2.29	2.20	0	4.49
7	23.73	2.48	11.10	0	13.58
8	34.16	2.65	11.50	0	14.15
9	47.11	2.81	11.90	0	14.71
10	62.80	2.96	12.30	0	15.26
11	81.44	3.11	12.60	40	55.71
12	103.26	3.24	13.00	140	156.24
13	128.46	3.38	13.40	609	625.88

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