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Assessment of flood mitigation strategies for reducing peak discharges in the Upper Cedar River watershed

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ASSESSMENT OF FLOOD MITIGATION STRATEGIES FOR REDUCING PEAK DISCHARGES IN THE UPPER CEDAR RIVER WATERSHED

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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Finally, thanks to my family for your continued love, support, and interest in my endeavors. I love you guys and look forward to spending time together in the future.

ABSTRACT

This thesis evaluates the effectiveness of several flood mitigation strategies for reducing peak discharges in the Upper Cedar River Watershed located in northeast Iowa. Triggered by record flooding in June 2008, the Iowa Watersheds Project was formed to evaluate and construct projects for flood reduction. The Upper Cedar was selected as a pilot watershed and a hydrologic assessment was performed to better understand its flood hydrology. Evaluation of different flood mitigation strategies was performed with HEC-HMS, a lumped parameter surface water model. The hydrologic model development is described and the model applications are analyzed.

The HMS model was used in several ways to better understand the flood hydrology of the Upper Cedar River Watershed. First, the runoff potential of the basin was assessed to identify the primary runoff generation mechanisms. Areas with agricultural land use and moderately to poorly draining soils had the highest runoff potential. Following, the model was used to evaluate the impact of several flood mitigation strategies – increased infiltration through land use changes, increased infiltration through soil improvements, and added storage in the watershed to hold runoff temporarily and reduce downstream flood peaks – for different flood frequency events (the 10-, 25-, 50-, and 100-year, 24-hour design rain storms) and the June 2008 flood. Although each scenario is hypothetical and simplified, they do provide benchmarks for the types of reductions physically possible and the effectiveness of strategies relative to one another. In order to reduce the impacts of flooding in the Upper Cedar, a combination of projects that enhance infiltration and/or store excess runoff will be necessary.

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CHAPTER 1: INTRODUCTION

1.1 Motivation and Overview of the Iowa Watersheds Project

As a result of persistent rain and saturated soil conditions during the summer of 2008, catastrophic flooding struck much of Iowa resulting in substantial economic, social, and environmental losses. Eighty-five of Iowa's 99 counties were declared federal disaster areas and, despite recovery efforts, impacts of the flood are still evident today. In response to this event, federal funding from the U.S. Department of Housing and Urban Development was provided to the state of Iowa to construct watershed flood mitigation projects. As a result, the Iowa Watersheds Project was formed in the spring of 2012 and is being facilitated by the Iowa Flood Center (IFC) located at IIHR-Hydroscience & Engineering at the University of Iowa. Four Iowa watersheds were selected to participate in the study, and the IFC works directly with the watershed management authority (WMA) – a group consisting of representatives from counties, cities, and soil and water conservation districts – formed within each respective watershed to solve flooding-related problems with a unified approach and outlook. The four watersheds selected to participate in the study include the Upper Cedar River, Turkey River, Middle Raccoon, and Soap/Chequest Creeks; they are shown in Figure 1.1.

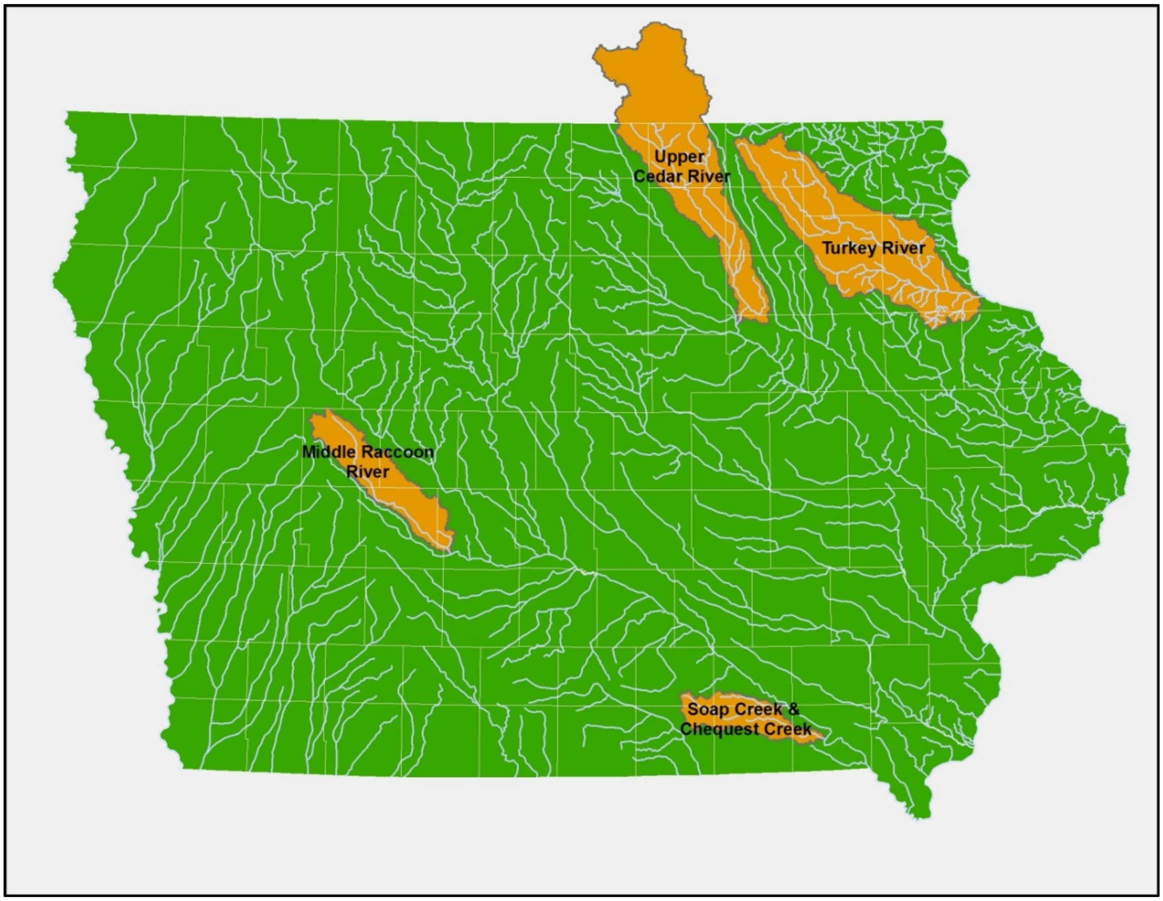

Figure 1.1. Iowa Watersheds Project study areas.

The purpose of the Iowa Watersheds Project is to plan, implement, and evaluate watershed projects to lessen the severity and frequency of flooding in Iowa. Specific project goals include maximizing soil water retention from heavy precipitation events, minimizing soil erosion and deposition during floods, managing runoff in upland regions under saturated conditions, and reducing both structural and non-structural flood damages ("Iowa Watersheds Project", 2014).

The project includes two phases and is estimated to be completed in 2017. Phase I began in June 2012 and involved performing a hydrologic assessment of each selected watershed to better understand the basin hydrology in order to locate areas where runoff reduction efforts and flood mitigation projects should be targeted; Phase I also focuses on evaluating the impact of different hypothetical watershed improvement projects. Phase I will be completed in the spring of 2014 following submission of the final draft of the hydrologic assessment report to the WMAs. This report summarizes the hydrologic conditions/trends in the watershed and evaluates the impact different flood mitigation strategies could have on reducing flood peak discharges. Phase II focuses on the construction of flood mitigation projects and began in the summer of 2013. The Upper Cedar River, Turkey River, and Soap/Chequest Creek Watersheds were selected for Phase II, each receiving $1.5 million in funding for project implementation. In order to better study and monitor the impact of potential projects – which may include a combination of farm ponds, wetlands, agricultural conservation practices, and urban stormwater best management practices – the three Phase I watersheds selected for Phase II were required to select a smaller subwatershed to focus project efforts. Project construction is planned to begin in the summer of 2014 and the Iowa Watersheds Project will conclude in the summer of 2017 following release of the Phase II report.

1.2 Outlook

The remainder of this Master's thesis focuses on the Phase I hydrologic assessment performed for the Upper Cedar River Watershed in northeast Iowa. The purpose of this research is to evaluate the impact of different hypothetical flood mitigation strategies on reducing flood peak discharges throughout the watershed using a lumped parameter hydrologic model. In order to achieve this goal, an accurate physical description of the watershed describing the relevant factors that impact the basin hydrology and runoff generation is needed. Following, a hydrologic model must be selected, built, and fine-tuned to reflect the hydrologic conditions present in the watershed in order to provide some level of predictive capability for how the watershed is expected to respond for different rainfall inputs. Once the simulated response of the hydrologic model reflects what was actually observed in the watershed with some degree of confidence, different hypothetical flood mitigation strategies may be simulated.

1.3 Chapter Summary

This introductory chapter provides an overview of the Iowa Watersheds Project and outlines the focus of this Master's thesis. As a result of the severe flooding that occurred in Iowa during the summer of 2008, the Iowa Watersheds Project was formed. This is a federally funded flood mitigation study whose purpose is to construct various watershed projects in Iowa to reduce the severity and frequency of flooding. Four Iowa watersheds geographically spread throughout the state were selected to participate in the study, and the IFC is working closely with each WMA to understand the pertinent flood-related issues specific to each watershed to help develop viable flood mitigation solutions.

This thesis focuses on the work performed as part of Phase I of the Iowa Watersheds Project, specifically for the Upper Cedar River Watershed in northeast Iowa. The goal of this research is to characterize the watershed's hydrologic response so the impact of different watershed improvement projects on reducing flood peak discharges may be evaluated using a hydrologic model. In order to do so, an inventory of the watershed physical characteristics pertinent to runoff generation is needed in order to develop a calibrated hydrologic model that can accurately simulate and predict the watershed response for various conditions. First, however, a sound understanding of watershed modeling for flood mitigation purposes in agricultural watersheds is necessary, which is the focus of the literature review in the next chapter.

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

Flooding has been and will continue to be a major concern in the U.S. Midwest. The hydrology of the Midwest is changing due to a variety of reasons. While the relative impact of each factor is less agreed upon, changes in land use and climate over time are two of the primary drivers altering the hydrology. Human induced land cover changes, including urbanization and agricultural expansion, have lessened the storage capacity of the landscape, resulting in more precipitation being converted to runoff. Prior to the 1830s, Iowa's landscape was dominated by native vegetation (tall-grass prairie). By 1980, 75% of Iowa's land had been converted to row crop agriculture. Additionally, increases in annual and seasonal precipitation totals have been observed in the Midwest since the 1970s (Takle, 2010). In part to both these factors, the Upper Cedar River Watershed has seen a statistically significant increase in mean daily discharge since the late 1970s (Villarini et al., 2011). Similar trends were observed for low flows as well, and while no statistically significant changes were detected for high flows, a generally increasing trend in peak annual discharge marked by observably higher annual variability is noticeable. Severe flooding has struck Iowa on multiple occasions over the past 20 years, marked by the well-known floods of 1993, 2008, and 2013. The 1993 flood resulted in statewide losses to crops, livestock, and personal property/income estimated at $1.45 billion (Gleason, 2008), while the 2008 flood resulted in statewide agricultural economic losses estimated to exceed $2 billion ("Iowa – Midwest Floods of 2008", 2010).

Because of the Midwest's changing hydrology and the substantial economic losses that can result from flooding, watershed modeling for flood mitigation has great value. Estimating the impact different watershed projects can have on reducing runoff and flood peak discharges can assist in selecting and locating flood mitigation projects that provide the greatest downstream benefits. This literature review provides some background information on the impact agriculture, specifically tile drainage, may have on a watershed's hydrology and provides several examples of watershed modeling studies performed that are similar in scope to the goals of Phase I of the Iowa Watersheds Project (the impacts of added storage on the landscape for reducing peak flows). A better knowledge of both these topics will assist in the calibration of the Upper Cedar River Watershed HEC-HMS model and help guide the flood mitigation analyses performed.

2.2 Impacts of Tile Drainage on the Flood Hydrology of Agricultural Watersheds

Because the Upper Cedar River Watershed is dominated by row crop agriculture (77% of the area) and moderately to poorly draining soils, tile drainage is hypothesized to be present throughout the watershed. Tile drainage is a practice often used by farmers to better agricultural productivity in areas with poorly draining soils by installing a system of underground pipes to enhance subsurface drainage. Knowledge of the impacts tile drainage can have on the flood hydrology of a watershed is needed to assist in model calibration and validation. Interestingly, while agricultural tile drainage has been in place since the mid-1800s, its hydrologic impacts at the watershed scale are not well understood. There are several findings that most historic studies tend to agree on, though exceptions can be found. According to Sloan (2013), generally accepted beliefs regarding the hydrologic impacts of tile drainage include:

1. The degree of impact tile drainage has on altering the hydrology of a basin depends on multiple variables including pre-drainage conditions, soil type, climate, land use, topography, and tile drain characteristics (depth and spacing).

2. Tile drainage tends to decrease peak flows in lower infiltrating soils (silts and clays) and increase peak flows in higher infiltrating soils (sands and gravels).
3. Tile drainage does not alter the total runoff volume but does increase the baseflow contribution.
4. The role precipitation plays on the impacts of tile drainage is not well understood. Some field studies showed the peak flow reduction increases with increasing rainfall intensity. The impact of storm size (frequency) is less understood.

In addition to possibly altering the peak flow magnitude and increasing the baseflow contribution in a watershed, tile drainage may also change other components of the flood hydrograph. Compared to the native prairie landscape that once covered much of Iowa, agricultural areas typically have less storage capacity and higher, flashier runoff potential. Subsurface drainage can increase the storage capacity temporarily in the upper layer of soil above the drains, allowing more infiltration (Blann et al., 2009). Because of this, tile drainage may reduce the flashiness of a basin, marked by an extension of the recession limb on the flood hydrograph. In some ways, the impacts of tile drainage resemble a water storage structure like a reservoir – water is stored temporarily in the subsurface and, depending on soil type, may drain back to a stream at a lower rate than if the rainfall was converted to surface runoff initially.

2.3 Lumped Parameter Watershed Modeling for Flood Mitigation

Watershed (hydrologic) modeling is performed using either a lumped parameter or physically-based model. Lumped parameter models "lump" or aggregate physical watershed characteristics, such as land use and soil type, into a single representative value to reflect the runoff potential of a certain area in an average sense. These types of models typically use mathematical expressions and empirical approximations to represent the different hydrologic processes in a watershed. On the other hand, physically-based models describe the hydrologic processes of a watershed at a localized scale and instantaneous rate (Ponce et al., 1996); the governing equations of physics and fluid flow (conservation of mass and momentum) are solved at a point and used to describe the hydrology over a given area. Both model types have advantages and disadvantages; for the Phase I modeling effort of the Upper Cedar River Watershed, a lumped parameter model (HEC-HMS) was selected because the rainfall-runoff response may be estimated using easily obtainable physical watershed characteristics, which lends itself nicely for a watershed as large as the Upper Cedar (nearly 1700 mi^2).

Various examples of lumped parameter watershed modeling for flood mitigation exist. Prior work in the Iowa Watersheds Project provides one example. HEC-HMS – a lumped parameter, surface water model – was used to evaluate the impact 132 distributed storage projects (primarily ponds) could have on reducing flood peak discharges in the Soap Creek Watershed in southeastern Iowa (253 mi^2). Twenty-four hour design rainstorms of different frequency (from the 2-year event to the 100-year event) were simulated over the entire watershed and peak discharges at select locations were compared with and without ponds present. Initial modeling results indicated the storage ponds could reduce peak discharges throughout the watershed by a substantial amount (approximately 15% for the 10-year rain event up to 40% for the 100-year rain event at the watershed outlet) (Wunsch, 2013).

In another study, the Soil and Water Assessment Tool (SWAT) was used to optimize the sizing and number of wetlands in upland areas to achieve greatest peak flow reductions in a tile-drained Indiana watershed (162 mi^2). SWAT is another lumped parameter model that operates on a daily time step, so while the simulation of flood events could not be performed at fine time scales, the overall flood reduction benefit provided by the wetlands was still able to be considered over a longer simulation period.

Results indicated that strategic placement of smaller wetlands with smaller drainage areas could achieve the same or higher peak flow reduction than fewer, larger wetlands (Babbar-Sebens et al., 2013).

Chennu et al. (2008) studied the impact a distributed network of dry ponds could have on reducing flood peak discharges in a French watershed (58 mi^2). Smaller ponds were placed upstream, larger ponds placed downstream, and the ponds were divided into regions adding the same total amount of storage to the watershed. The greatest flood reduction benefit relative to all other areas was observed in the most upstream region (a greater number of smaller ponds). The consistency in this finding among different studies is particularly relevant to the Iowa Watersheds Project since managing runoff in upland regions with smaller scale, distributed storage projects is a highly probable flood reduction strategy to be implemented.

2.4 Chapter Summary

In order to derive meaningful conclusions about flood mitigation practices in the Upper Cedar River Watershed, knowledge of hydrologic modeling in agricultural watersheds is necessary. Unique modeling challenges are likely to be encountered as a result of the U.S. Midwest being one of the most intensively managed areas in the world (Pimentel, 2012). The hydrologic impacts of subsurface drainage used to enhance agricultural productivity are not well understood; most evidence suggests tile drainage can alter the magnitude of peak flows depending on the underlying soil type and baseflow recession characteristics. Tile drainage practices may be extensive in the Upper Cedar River Watershed, so hydrologic model parameters will need to be adjusted to reflect these conditions. Finally, previous flood mitigation studies using lumped parameter hydrologic models can help define the appropriate capabilities/limits for the Upper Cedar HMS model and indicate the general trends in modeling results that should be expected.

CHAPTER 3: CONDITIONS IN THE UPPER CEDAR RIVER WATERSHED

3.1 Introduction

This chapter provides a description of the hydrologic trends and physical attributes of the Upper Cedar River Watershed. An overview of the basin's hydrologic trends and historic floods of record is provided, along with descriptions of current watershed conditions related to geology, soils, topography, land use, and hydrologic/meteorologic instrumentation. Some of these datasets are used to develop the hydrologic model discussed in Chapter 4.

3.2 Hydrology

The Upper Cedar River Watershed extends over southeast Minnesota and northeast Iowa (see Figure 3.1). The watershed as defined by its eight digit hydrologic unit code (HUC 8) 07080201 drains an area of 1685 mi^2. The watershed extends over 10 counties – four in Minnesota and six in Iowa. The Upper Cedar and its tributaries generally flow from north to south and the outlet of the watershed is at the confluence of the West Fork Cedar River with the Upper Cedar. The Upper Cedar River has one main tributary, the Little Cedar River, which drains 311 mi^2 along the eastern part of the watershed.

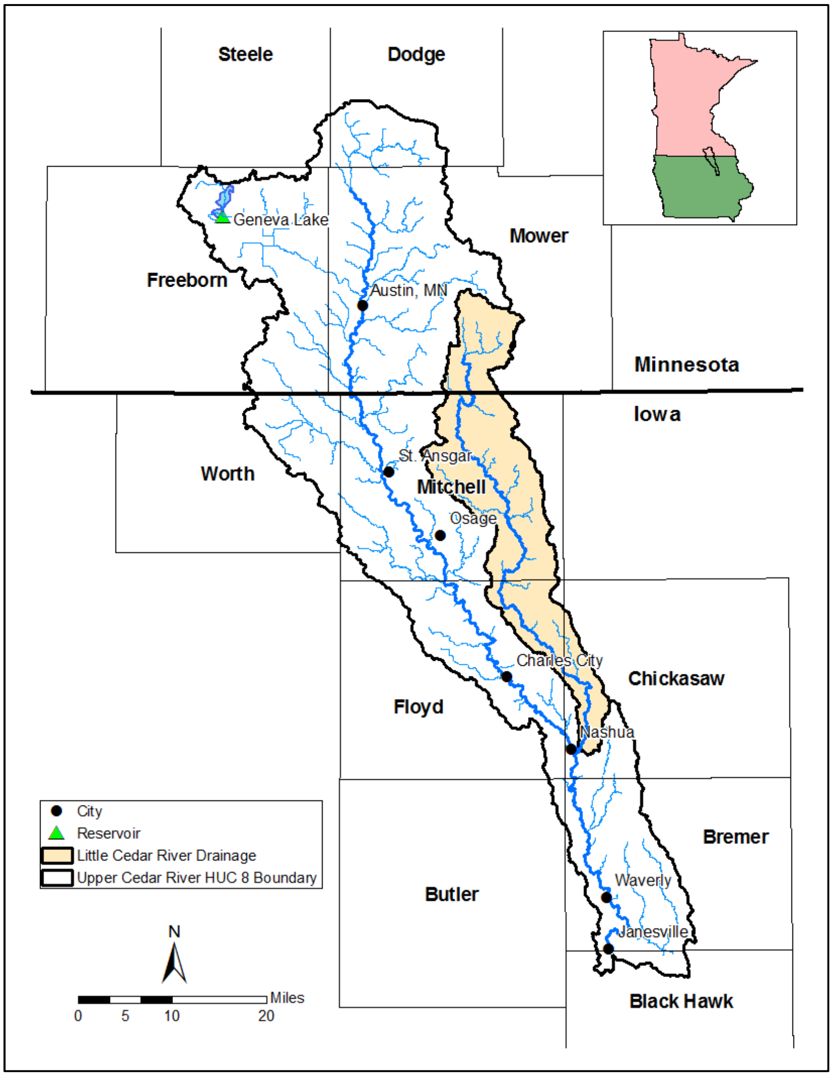

Figure 3.1. Overview of the Upper Cedar River Watershed (HUC 07080201). The watershed drains 1685 mi^2 and the Little Cedar River is the largest tributary.

3.2.1 Annual Water Cycle

Based on a 30-year period of record (1981-2010), the average annual precipitation for the Upper Cedar River Watershed is 35.1 inches ("30-yr Normal Precipitation", 2013). Most of this precipitation evaporates – either directly from open water bodies like lakes or streams, or by transpiration from vegetation. Based on a water budget analysis, approximately 68.5% (24 inches) of the total annual precipitation in the watershed evaporates (Bradley, 2014).

The remaining precipitation either infiltrates into the ground or becomes surface runoff. Both mechanisms replenish the water in streams and rivers, but the time period over which they do so varies immensely. During rain events and shortly following, some of the water quickly runs off the land surface as overland flow and fills streams and rivers. This occurs over time scales of hours and days. Based on U.S. Geological Survey (USGS) streamflow records for the Cedar River at Janesville, IA (USGS 05458500) during the same 30 year period (1981-2010), approximately 9.8% of annual precipitation (3.4 inches) becomes surface flow. The remaining 21.7% of precipitation (7.6 inches) in the watershed enters streams as baseflow. This water takes a much longer, slower path to reach the stream network compared to surface flow. The water must infiltrate into the ground, reach the groundwater table, and then slowly travel down gradient to the river network. This process occurs over days, months, and even years. The slow, steady contribution of baseflow maintains flow in perennial river systems during extended dry periods.

3.2.2 Monthly Water Cycle

Average monthly precipitation and streamflow for the Upper Cedar River Watershed is shown in Figure 3.2. Precipitation is lowest during winter months and increases in the spring to early summer months. Greatest precipitation is observed during May, June, and July. Streamflow is at a minimum during the winter and increases in the spring as well, but the months of greatest streamflow are observed earlier than the months of greatest precipitation. Greatest average monthly streamflow is observed in April in response to snow melt and spring runoff. During this time, temperatures are warm enough to melt snow that has collected over winter but the ground may still be frozen, inhibiting infiltration, and evaporation is small. As a result, spring streamflows are high. Despite substantial precipitation in the summer months, average monthly streamflow shows a decreasing trend due to greater losses of precipitation to evaporation and transpiration during the growing season.

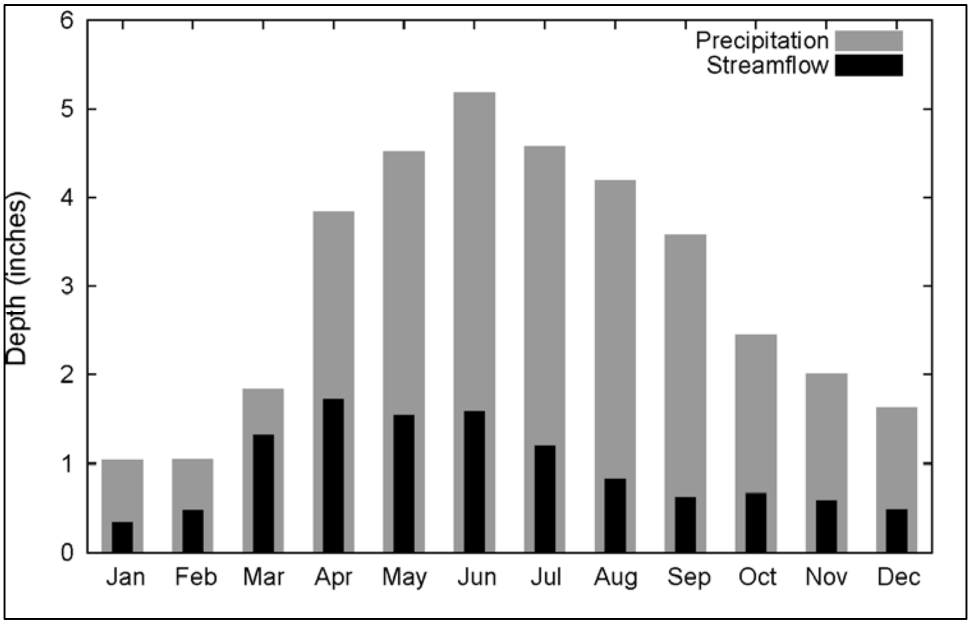

Figure 3.2. Monthly water cycle for the Upper Cedar River Watershed. The plot shows the average monthly precipitation and streamflow (in inches) over a 30 year period (1981-2010). Monthly streamflow estimates are based on records for the Cedar River at Janesville.

Source: Bradley, Allen. Water Budget Analysis for the Upper Cedar River Watershed (1981-2010). 1 Mar. 2014. Raw data. IIHR-Hydroscience & Engineering, University of Iowa, Iowa City.

3.2.3 Flood Climatology

In order to better understand the flood hydrology of the Upper Cedar River Watershed, knowledge of when floods typically occur is important. Figure 3.3 shows the annual maximum peak discharges and dates of occurrence for the Cedar River at Janesville over its 93-year period of record (intermittent periods between 1905 and 2014). Annual maximums often occur in March or April as a result of snow melt and spring runoff. However, the highest annual peak discharges have occurred in the summer months when the largest rain storms occur. These summer storms can produce great amounts of rainfall in a few hours or days. The horizontal line in Figure 3.3 defines the mean annual flood (the average of the annual maximums). The mean annual flood provides an approximate threshold for flooding. Comparing the annual maximums to the mean annual flood, in many years the annual peak discharge is not large enough to produce a flood. Floods are more frequent in March or April while floods of greatest magnitude (largest annual peak discharges) occur sporadically in the summer months.

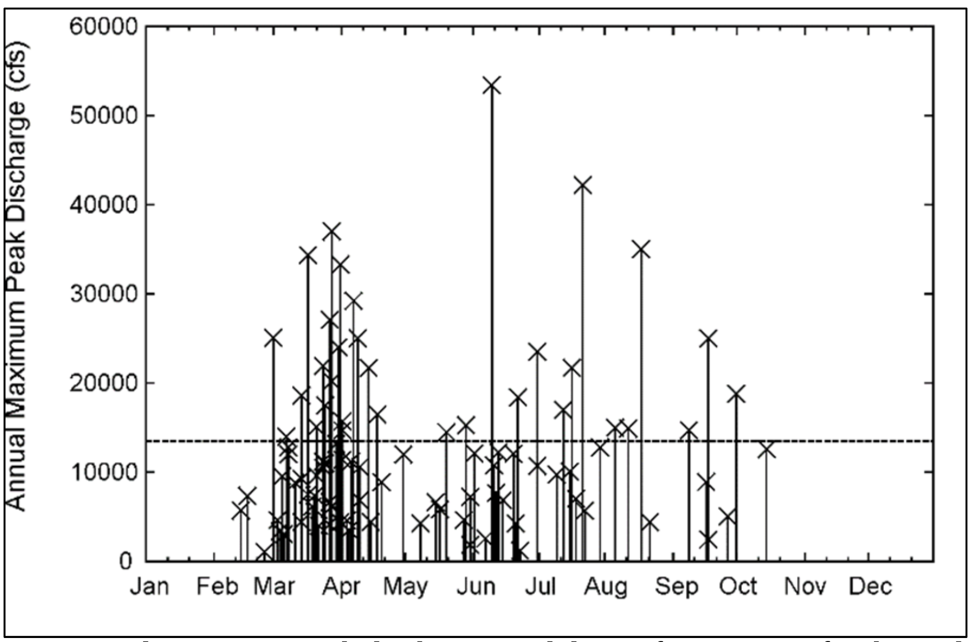

Figure 3.3. Annual maximum peak discharges and dates of occurrence for the Cedar River at Janesville (93-year period of record). The mean annual flood is shown by the horizontal line.

Source: Bradley, Allen. Water Budget Analysis for the Upper Cedar River Watershed (1981-2010). 1 Mar. 2014. Raw data. IIHR-Hydroscience & Engineering, University of Iowa, Iowa City.

Figure 3.4 shows the flood occurrence frequency by month for the Cedar River at Janesville. This plot shows the percentage of annual maximum discharges exceeding the mean annual flood for each month. Similar conclusions can be drawn from Figure 3.4 as from Figure 3.3. Flooding (defined as annual peak discharges exceeding the mean annual flood) is most frequent in March (around 35%) as a result of spring runoff. Flood occurrence frequency decreases over April, May, and June before a secondary, smaller peak is observed in July (around 12%) as the result of large rain events. Unlike Figure 3.3, Figure 3.4 does not provide any indication on the magnitude of the flood events, only the frequency of occurrence.

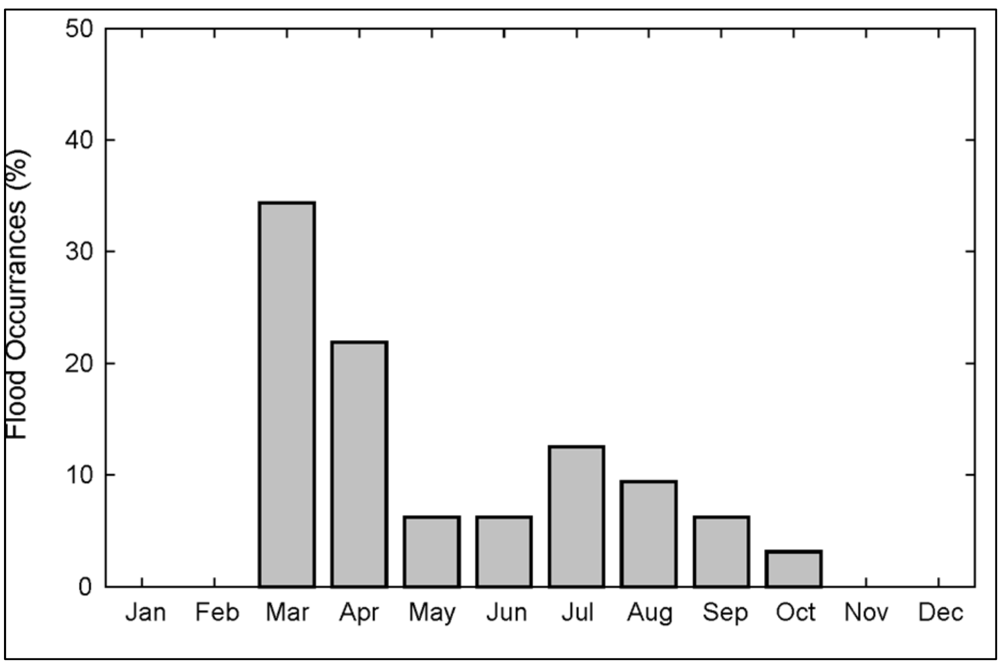

Figure 3.4. Flood occurrence frequency by month for the Upper Cedar River Watershed. The plot shows the percent of peak annual discharges exceeding the mean annual flood for the Cedar River at Janesville.

Source: Bradley, Allen. Water Budget Analysis for the Upper Cedar River Watershed (1981-2010). 1 Mar. 2014. Raw data. IIHR-Hydroscience & Engineering, University of Iowa, Iowa City.

3.2.4 Floods of Record

There have been several noteworthy floods in the watershed over the past 25 years, with the most well-known being the floods of 1993 and 2008. Flooding during the summer of 1993 struck much of the upper Midwest. On August 15, over 10 inches of rain was reported in southern Mower County, MN, resulting in severe flooding at Austin that produced a stage of 19.43 feet, the second highest to date. Downstream at Charles City, the stage reached 21.44 feet, the largest stage ever recorded at that time until a July 1999 flood (Welvaert, 2010). For reference, the flood stage established by the National Weather Service (NWS) at which minor flooding begins to occur at Charles City is 12 feet.

Flooding in July 1999 resulted from two severe thunderstorms that occurred within days of each other. A thunderstorm on July 18-19 produced 4-6 inches of rain over Chickasaw, Floyd, and Worth counties with unofficial estimates reaching 13 inches; 5.16 inches of rain were reported at Charles City. On July 20-21, 6-8 inches of rain were reported in the southern part of the watershed, with 6.65 inches reported at Charles City. A discharge of 31,200 cubic feet per second (cfs) and stage of 22.81 feet were measured at Charles City. Road closures, washouts, and residential flooding occurred (Welvaert, 2010).

The next major flood occurred during mid-September 2004. Heavy rain on September 14-15 over southeast Minnesota and northeast Iowa produced record flooding near Austin. Rainfall amounts of 11.5-13 inches were reported in some areas, and a record stage of 25 feet was recorded at Austin. Despite relatively little rainfall in the southern part of the basin, the Cedar River rose to a stage of 20.6 feet at Charles City, the seventh highest on record. Two casualties occurred at Austin (Welvaert, 2010).

The June 2008 flood produced some of the greatest discharges and stages on record throughout the basin. Basin average precipitation from June 3-12 totaled 8.3 inches, with nearly 50% of that falling on June 7-8. As a result of saturated soils and persistent precipitation, record discharges were recorded on the Cedar River at Austin (20,000 cfs, 23.26 feet), Charles City (34,600 cfs, 25.33 feet), Janesville (53,400 cfs, 19.45 feet, NWS flood stage defined as 13 feet), and on the Little Cedar River near Ionia (24,700 cfs, 21.32 feet, NWS flood stage defined as 10 feet). Damages were severe and recovery is still taking place (Welvaert, 2010).

3.3 Watershed Physical Description

This section provides a physical description of the Upper Cedar River Watershed. Various characteristics impacting the watershed hydrology and existing instrumentation in the watershed helpful for hydrologic model development are described.

3.3.1 Geology

The Upper Cedar River Watershed is located in two landform regions, the Iowan Surface and the Des Moines Lobe. The majority of the watershed is located in the Iowan Surface (80.2% of the area), which is composed primarily of glacial drift and loess (moderately to poorly draining soils), and has low relief. The northwest part of the watershed lies in the Des Moines Lobe (19.8% of the area), which is composed primarily of glacial till (similar drainage characteristics to the Iowan Surface), and has even less relief than the Iowan Surface (Hutchinson, 2013).

Areas of shallow carbonate bedrock and karst features including sinkholes, springs, and fractured bedrock are present throughout the watershed, particularly in Floyd and Mitchell counties. In much of Mitchell County and parts of the remaining Iowa counties, the depth to bedrock is less than 10 feet. The shallow carbonate bedrock and over 2100 sinkholes in the watershed can have a major impact on both the basin hydrology and water quality. The landform regions, sinkhole locations, and depth to bedrock in the Upper Cedar River Watershed are shown in Figure 3.5.

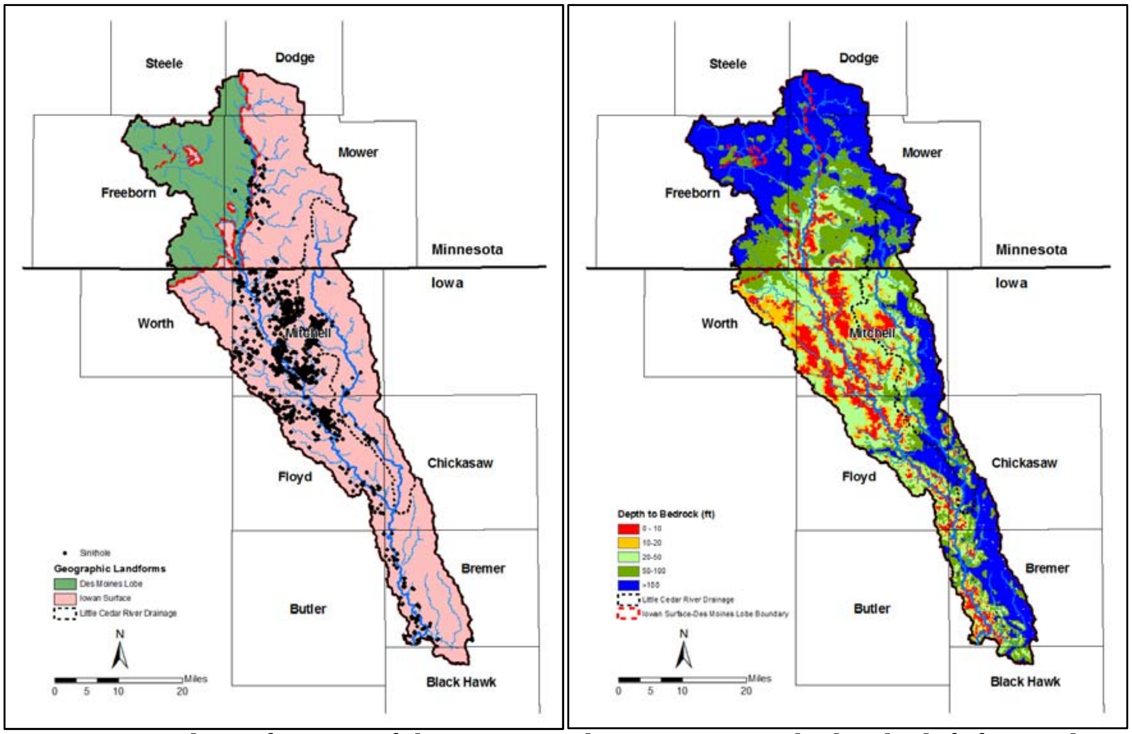

Figure 3.5. Geologic features of the Upper Cedar River Watershed. The left figure shows the two landform regions the watershed encompasses (Iowan Surface and Des Moines Lobe) and the 2132 sinkholes documented in the watershed. The right figure shows the depth to bedrock.

Source: Iowa Geological Survey, Iowa DNR, Minnesota DNR, and Minnesota Dept. of Geology and Geophysics. 2003-2013.

3.3.2 Soils

The basin is composed primarily of moderately to poorly 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 soils have the highest runoff potential (lowest infiltration capacity). A sand or gravel would classify as an A-type soil whereas a clay or silt would classify as a C or D-type soil. In addition, dual code soil classes A/D, B/D, and C/D can also be assigned. For dual soil code classes, even though the soil properties may be favorable for infiltration, a shallow groundwater table (within 24 inches of the surface) typically prevents much infiltration from occurring (Hoeft, 2007). For example, a B/D soil will have the runoff potential of a B-type soil if the shallow groundwater table were to be drained away or lowered, but the higher runoff potential of a D-type soil if it is not.

Figure 3.6 shows the distribution of HSGs in the Upper Cedar River Watershed. The parts of the watershed located in the Iowan Surface have primarily B, C, and C/D-type soils, resulting in areas that range from moderate to high runoff potential. The northwest part of the watershed in the Des Moines Lobe region contains a greater amount of B/D-type soils, reflecting the presence of a shallow groundwater table or depth to an impermeable layer. The large amount of dual code soils in the watershed (60%) gives strong reason to believe tile drainage practices exist throughout much of the watershed to better agricultural production in these poorly draining areas.

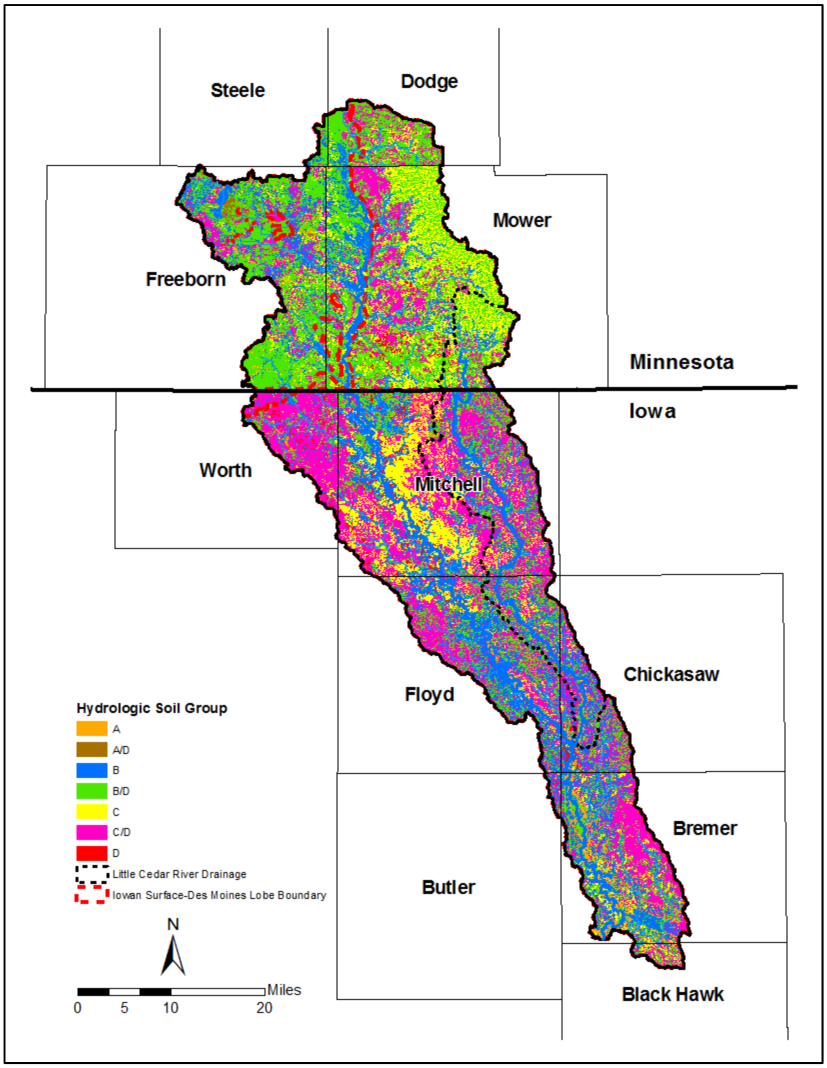

Figure 3.6. Distribution of Hydrologic Soil Groups in the Upper Cedar River Watershed. Hydrologic Soil Groups reflect the degree of runoff potential a particular soil has, with Type A representing the lowest runoff potential and Type D representing the highest runoff potential.

Source: *Soil Survey Geographic (SSURGO) Database*. Fort Worth: USDA-NRCS, 2012. <http://SoilDataMart.nrcs.usda.gov/>.

The HSG composition in the watershed (by percent area) is tabulated in Table 3.1.

Table 3.1. Hydrologic Soil Group composition in the Upper Cedar River Watershed.

Hydrologic Soil Group	Percent of Watershed Area
A	3.8
A/D	0.3
B	19.5
B/D	30.2
C	15.2
C/D	29.5
D	1.4

3.3.3 Topography

The Upper Cedar River Watershed has low relief and mildly rolling terrain. Elevations range from 1438 feet above mean sea level (MSL) in the uppermost part of the watershed to 814 feet above MSL at the outlet (624 feet of relief). The terrain steepens moving from north to south out of the Des Moines Lobe region and into the Iowan Surface. Typical land slopes are between 0.6-5.6% (25th and 75th percentiles), with the steepest areas most common along the eastern half of the watershed or near the river channel network; land slopes lessen moving from south to northwest. Figure 3.7 shows the elevations and land slopes for the Upper Cedar River Watershed.

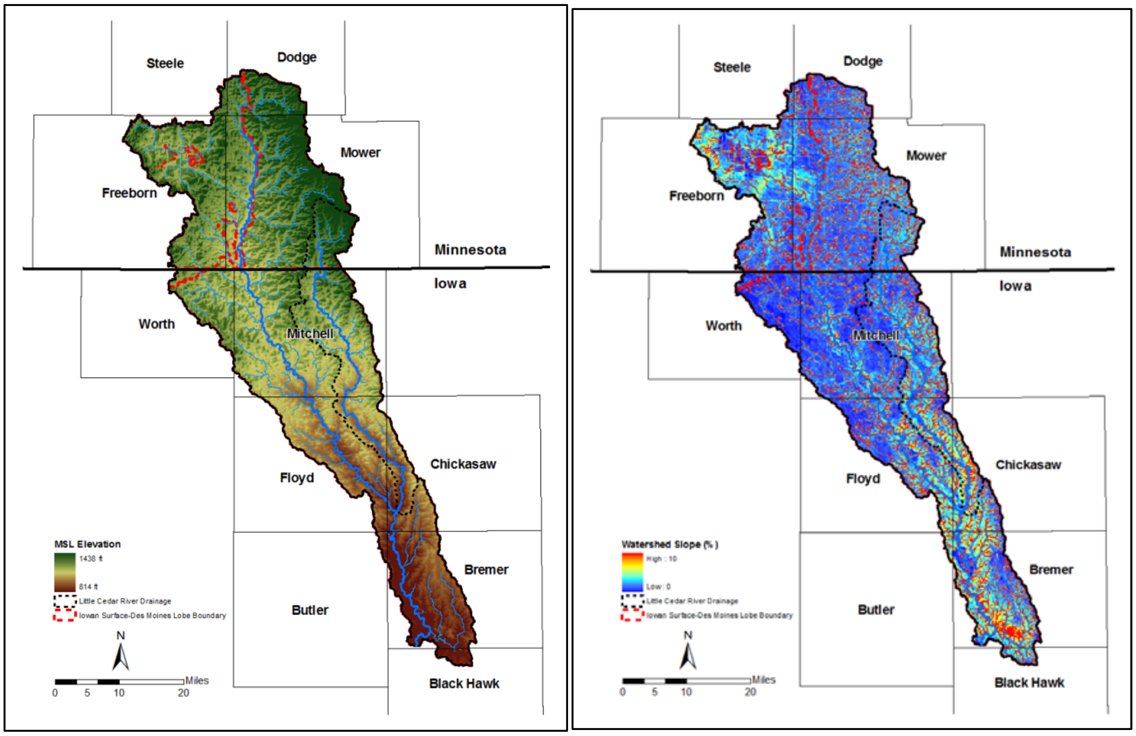

Figure 3.7. Topography of the Upper Cedar River Watershed. The left figure shows watershed elevations (in feet) and the right figure shows land slopes ranging from 0-10%.

Source: *National Elevation Dataset*. 1/3 arc-second. Sioux Falls, SD: USGS, 2009. <http://nationalmap.gov>.

3.3.4 Land Use

Land use in the Upper Cedar River Watershed is dominated by agriculture (corn and soybeans) at approximately 77.2% of the acreage, followed by grass/hay/pasture (9.2%), developed areas (8.3%), forest (2.7%), and open water/wetlands (2.5%), per the 2006 National Land Cover Dataset (NLCD). Approximately 99% of the watershed is privately owned (*Rapid Watershed Assessment, 2012*). Land use/cover in the watershed is shown in Figure 3.8.

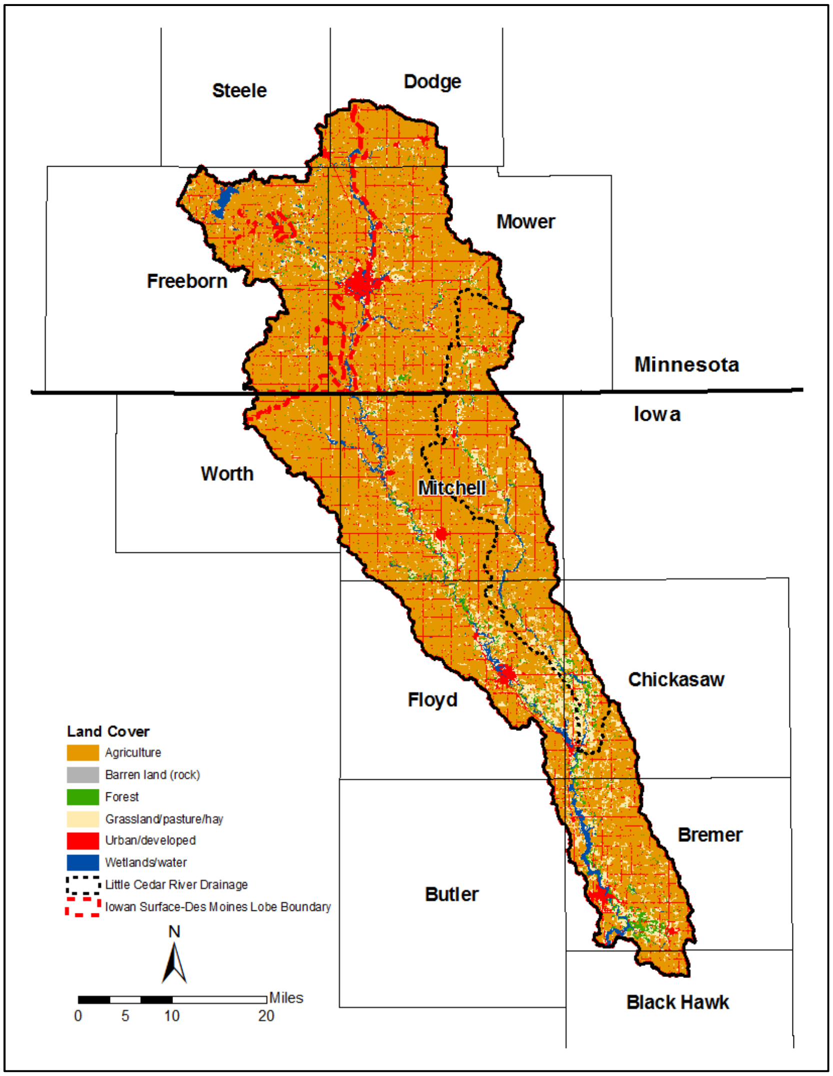

Figure 3.8. Land use composition in the Upper Cedar River Watershed. Agricultural land use is shown in orange.

Source: *National Land Cover Dataset (2006)*. Sioux Falls, SD: USGS, 2011. <http://nationalmap.gov>.

3.3.5 Hydrologic and Meteorologic Instrumentation

The Upper Cedar River Watershed has instrumentation installed to collect and record stream stage, discharge, and precipitation measurements. There are six USGS stage/discharge gages and five Iowa Flood Center (IFC) stream stage sensors located within the watershed. There are nine National Oceanic and Atmospheric Administration

(NOAA) hourly or daily precipitation gages within or near the watershed. Table 3.2 and Figure 3.9 below detail the periods of record and locations of the hydrologic/meteorologic instrumentation.

Table 3.2. Periods of record for hydrologic and meteorologic instrumentation in or near the Upper Cedar River Watershed.

Gage Type	Location	Period of Record
Stage/Discharge Gages (11)		
USGS Stage/Discharge	Cedar River near Austin, MN (05457000)	1909- present
USGS Stage/Discharge	Cedar River at Osage, IA (05457505)	2010- present
USGS Stage/Discharge	Cedar River at Charles City, IA (05457700)	1965- present
USGS Stage/Discharge	Cedar River at Waverly, IA (05458300)	2000- present
USGS Stage/Discharge	Cedar River at Janesville, IA (05458500)	1905-present
USGS Stage/Discharge	Little Cedar River near Ionia, IA (05458000)	1955- present
IFC Stream Sensor (stage)	Cedar River near Ortranto, IA (SRS0157)	3/29/2013 - present
IFC Stream Sensor (stage)	Cedar River near St. Ansgar, IA (SRS0161)	3/29/2013 - present
IFC Stream Sensor (stage)	Cedar River near Plainfield, IA (SRS0143)	7/24/2013 - present
IFC Stream Sensor (stage)	Little Cedar River near Orchard, IA (SRS0151)	3/29/2013 - present
IFC Stream Sensor (stage)	Little Cedar River near Nashua, IA (SRS0162)	3/29/2013 - present
Precipitation Gages (9)		
NOAA Hourly Precipitation	Dodge Center, MN (COOP: 212166)	1948-present
NOAA Daily Precipitation	Austin WWTP, MN (GHCND:USC00210355)	1937-present
NOAA Daily Precipitation	Albert Lea 3 SE, MN (GHCND:USC00210075)	1893-present
NOAA Daily Precipitation	Northwood, IA (GHCND:USC00136103)	1896-present
NOAA Hourly Precipitation	St. Ansgar, IA (COOP: 137326)	1948-present
NOAA Daily Precipitation	Osage, IA (GHCND:USC00136305)	1893-present
NOAA Daily Precipitation	Charles City, IA (GHCND:USC00131402)	1893- present
NOAA Daily Precipitation	Tripoli, IA (GHCND:USC00138339)	1946- present
NOAA Hourly Precipitation	Shell Rock 2W, IA (COOP: 137602)	1948- present

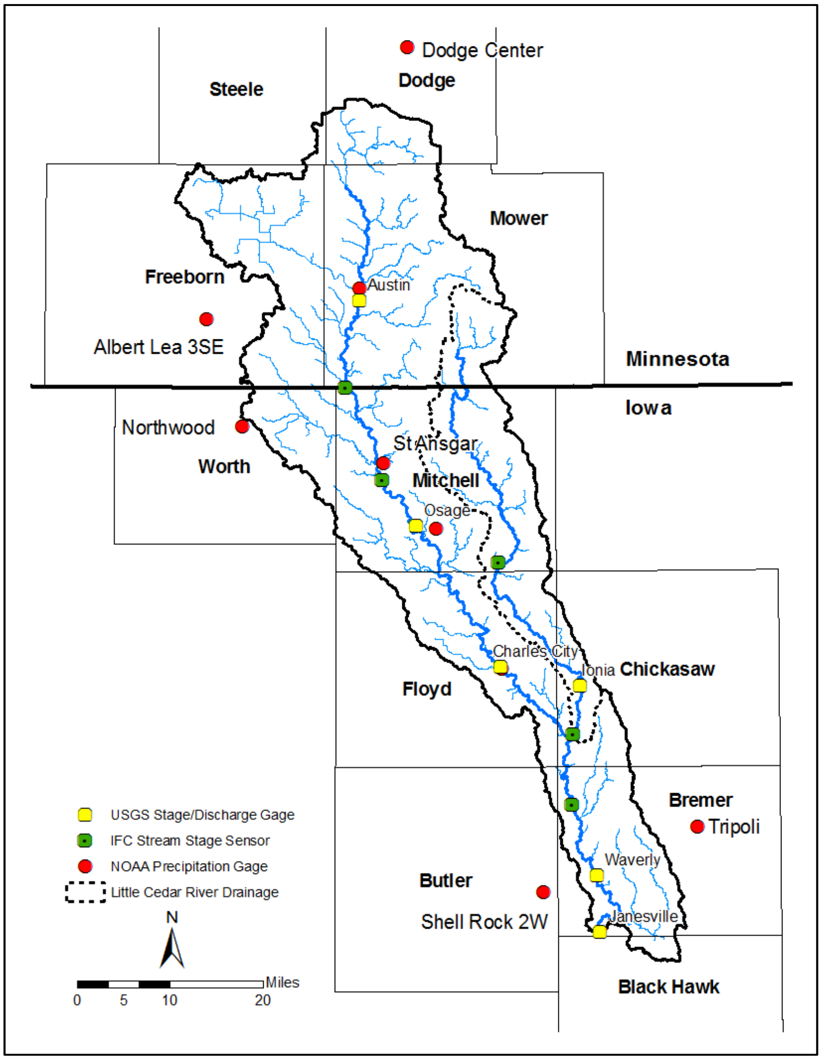

Figure 3.9. Hydrologic and meteorologic instrumentation in the Upper Cedar River Watershed. Stage/discharge gages (11) are shown in yellow/green and precipitation gages (9) are shown in red.

3.4 Chapter Summary

Chapter 3 details some of the hydrologic trends and physical characteristics of the Upper Cedar River Watershed. The Upper Cedar River Watershed (HUC 07080201) drains 1685 mi^2 in southeast Minnesota (580 mi^2: 34%) and northeast Iowa (1105 mi^2: 66%). Average annual precipitation in the watershed is about 35 inches; nearly 69% of this evaporates, while 10% enters streams as surface flow and 22% enters streams as baseflow. Greatest amounts of precipitation are observed from May to July, while streamflow is typically greatest in March, April, and May as a result of snowmelt and spring runoff. Flooding is most frequent during the early spring (March and April), but the largest floods occur in the summer months as a result of heavy rainstorms.

The watershed is characterized by low relief, moderately to poorly draining soils, and is agriculturally-based. Eighty percent of the watershed is located in the Iowan Surface landform region, and the watershed has some karst characteristics that include over 2100 sinkholes and areas with shallow carbonate bedrock. Runoff potential is moderate to high, as over 90% of the basin is defined by some form of Hydrologic Soil Group B or C. Like much of Iowa, row crop agriculture is the dominant land use, accounting for 77% of the acreage. Finally, the basin is well-gaged with hydrologic and meteorologic instrumentation, which is helpful for hydrologic model development discussed in future chapters.

CHAPTER 4: UPPER CEDAR RIVER HYDROLOGIC MODEL DEVELOPMENT

4.1 Introduction

This chapter summarizes the development of the hydrologic model used for the Upper Cedar River Watershed. The modeling was performed using the U.S. Army Corp of Engineers' (USACE) Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS), Version 3.5. The general procedure involved using ESRI Arc GIS to perform terrain analysis and break the watershed into smaller pieces based on user-defined thresholds; the datasets from Chapter 3 were used to develop the model parameters needed in HEC-HMS.

4.2 Hydrologic Model Selection: HEC-HMS

HEC-HMS was selected for the Phase I hydrologic modeling of the Upper Cedar River Watershed for several reasons. Reasons include its diverse range of applicability, ability to model at fine time scales, small computational cost, and simplicity. HMS is applicable in a wide range of geographic areas and for watersheds ranging in size from very small (a few acres) to very large (the size of the Upper Cedar River Watershed or larger). HMS also has the ability to model at fine time scales (a time step as small as one minute). This is important since characterizing the watershed response to large rain events lasting hours or days is one of the primary goals of Phase I. HMS also has a relatively small computational cost compared to some other hydrologic models; rather than taking hours or days to run a simulation, especially at the scale of the Upper Cedar, simulations in HMS can typically be run in minutes. Finally, HMS is widely used in both academics and industry, so its strengths and weaknesses are well known. It makes use of relatively simple methods derived from measurable watershed characteristics and widely available datasets to quantify how water moves throughout a watershed for different rainfall inputs and initial conditions.

4.2.1 HEC-HMS Assumptions and Limitations

HMS is a mathematical, lumped parameter, uncoupled, surface water model. Each of these items will be briefly discussed. The fact that HMS is a mathematical model implies the different hydrologic processes are represented by mathematical expressions that 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 instead of varying over time. HMS is an uncoupled model, meaning the different hydrologic processes are solved independent 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 when saturated conditions are expected since overland flow is expected to dominate the partitioning of rainfall.

4.3 Model Construction

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, defines 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 it rains over the watershed.

Inputs required for the basin and meteorologic models of the Upper Cedar River Watershed were developed in Arc GIS 9.3. The watershed was broken into smaller units, called subbasins in HMS, and the stream network was delineated. Once the skeleton of the model was constructed, HMS model inputs and parameters needed for precipitation, runoff estimation, and river routing were created from the datasets of Chapter 3 using

HEC-GeoHMS – an Arc GIS extension specifically for HMS model development. Finally, these attributes were exported to the appropriate format so most of the model setup was complete upon opening in HMS.

4.3.1 River Network Definition and Subbasin Delineation

The area upstream of Janesville, IA (1663 mi^2) was modeled in HMS for the Upper Cedar River Watershed. Janesville was selected as the most downstream point in the model because it is very near the official Upper Cedar HUC 8 watershed outlet and corresponds to the location of the furthest downstream USGS stage/discharge gage in the basin. The watershed was divided into 320 smaller units, called subbasins in HMS, with an average area of about 5.2 mi^2 but as large as 19.5 mi^2. The subbasin delineation of the Upper Cedar River Watershed implemented into HMS is shown in Figure 4.1.

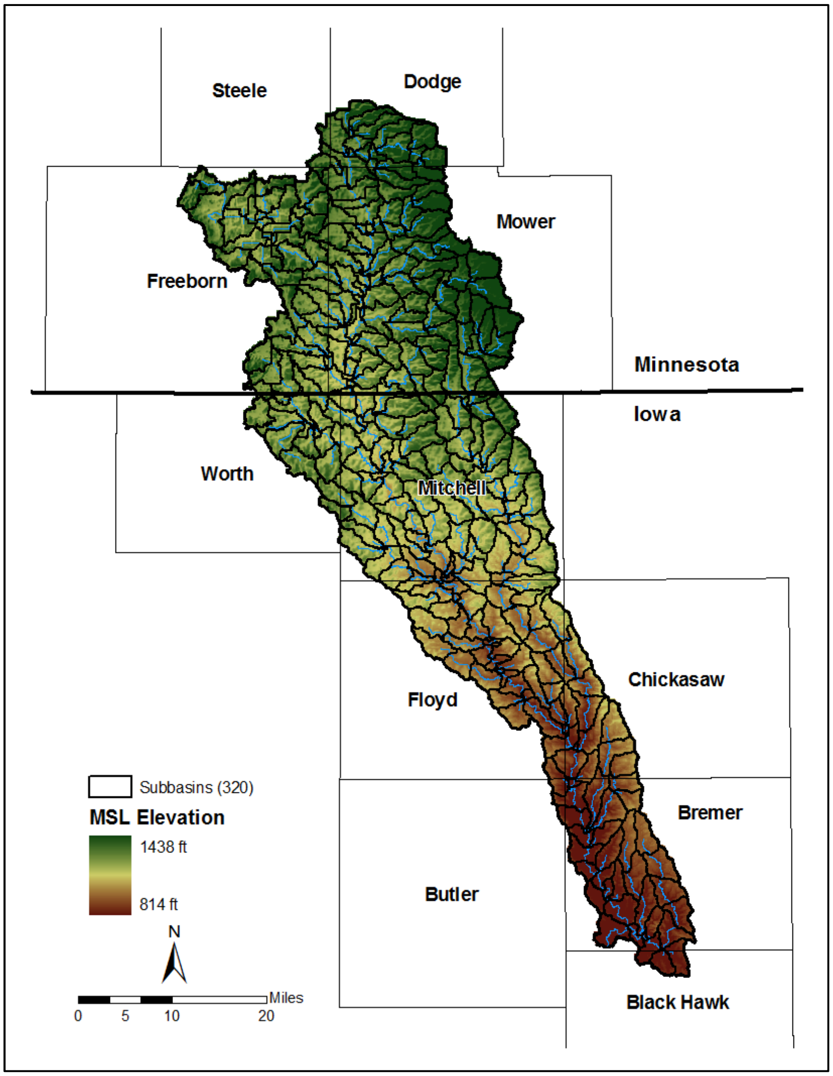

Figure 4.1. Subbasin delineation of the Upper Cedar River Watershed. The watershed was divided into 320 subbasins in Arc GIS to better refine model parameters.

A combination of Arc GIS and Arc-Hydro tools were used to define the stream network and delineate subbasins for the watershed based on user-defined thresholds. Delineating the stream network and subbasins required developing a digital elevation model (DEM) of the watershed. Elevation data was obtained from the USGS National Elevation Dataset (NED). Four blocks (n43w093, n43w094, n44w093, and n44w094) of 1/3 arc-second (approximately 10-meter) resolution digital elevation models covering the extent of the watershed were downloaded from the USGS National Map Viewer

Platform, clipped to the watershed extents using ArcGIS, and then joined into a seamless DEM.

Terrain preprocessing was performed using Arc-Hydro tools to refine the DEM, define the stream network, and delineate subbasins. Preprocessing steps were performed sequentially and included DEM reconditioning, fill sinks, flow direction, flow accumulation, stream definition, stream segmentation, catchment grid delineation, catchment and drainage line processing, adjoint catchment processing, and drainage point processing (Merwade, 2012). A brief description of these functions is provided in Table 4.1.

Table 4.1. Summary of terrain preprocessing functions performed in Arc GIS to delineate the stream network and subbasins for the Upper Cedar River Watershed HMS model.

Terrain Preprocessing Function	**Description**
DEM reconditioning	Modifies original DEM to be consistent with river network
Fill sinks	Modifies DEM in depressional areas
Flow direction	Computes direction water will flow from each DEM grid cell based on direction of steepest descent from that cell
Flow accumulation	Computes accumulated number of cells upstream of a given cell
Stream definition	Creates initial stream delineation grid based on user-defined upstream area threshold
Stream segmentation	Creates a grid of stream segments that have a unique identification
Catchment grid delineation	Creates a grid that defines which catchment each cell belongs to
Catchment polygon processing	Converts the catchment grid to a feature class (shapefile)
Drainage line processing	Converts the stream segment grid to a feature class (shapefile)
Adjoint catchment processing	Generates a feature class containing the aggregated upstream catchments
Drainage point processing	Generates the outlet (drainage) point for each catchment

The stream network was defined to begin when the upstream drainage area was 10 km^2 (3.86 mi^2), and subbasins were delineated such that a subbasin was defined upstream of all stream confluences. This resulted in the creation of 251 subbasins ranging in size from 0.04 acres to 37.6 mi^2, each possessing one river reach. GIS-defined subbasins were further manually split in some cases to create an outlet point at each USGS stage/discharge gage location as well as the outlet of one incorporated structure – Geneva Lake, MN. Extremely small subbasins (a few acres) were merged with adjacent ones, while large subbasins (typically greater than 10 mi^2) were further delineated to better account for soil and land use heterogeneities.

Once the stream network and subbasins were delineated, basic watershed properties were derived for each subbasin or stream reach to better describe the drainage patterns of the Upper Cedar River Watershed using HEC-Geo HMS. The area, average basin slope, and longest flowpath were obtained for each subbasin; reach length and slope were obtained for each stream segment within a subbasin. Because HMS is a lumped parameter model, area-weighted averaging is performed within the boundary of each subbasin to assign each subbasin a single value for the parameter being developed.

4.3.2 Incorporated Structures

One reservoir, Geneva Lake (see Figure 3.1), was incorporated into the model to account for attenuation and delay of the flood hydrograph due to surface storage. Geneva Lake is a shallow lake in Freeborn County, Minnesota, 10 miles north of Albert Lea. The lake drains approximately 21.5 mi^2 and has a surface area of 1955 acres, storage of 6,021 acre-feet, and average depth of three feet at the normal pool level (Lipetzky, 2005). The stage-storage-discharge rating curve used in the HMS model was obtained from Ducks Unlimited, Inc. of Bismarck, North Dakota and is available in Appendix B. No other existing water storage structures were incorporated into the HMS model.

4.4 Development of Model Inputs and Parameters

This section provides an overview of data inputs used and assumptions made to develop the HMS model. This includes the methods used for estimating rainfall, partitioning rainfall into runoff, subbasin hydrograph development, baseflow, and river routing in HMS.

4.4.1 Rainfall

4.4.1.1 Historical Rain Storms

Stage IV radar rainfall estimates were used as the precipitation input for simulation of historical (actual) rain events known to have occurred within the watershed. The Stage IV dataset 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 U.S. and combining them into a nationwide gridded hourly precipitation estimate data set. Each grid cell is approximately 4 km x 4 km (2.5 miles x 2.5 miles). Stage IV radar rainfall estimates are available from January 1, 2002 to the present.

Additional analysis was required in Arc GIS in order to use radar rainfall estimates in HMS. HEC-GeoHMS tools were used to intersect the subbasins with the appropriate grid system to allow use of the Stage IV radar rainfall estimates. Stage IV radar rainfall estimates are reported using the Hydrologic Rainfall Analysis Project (HRAP) grid, so a grid cell parameter file containing the coordinates and area of each HRAP grid cell within each subbasin was generated. HMS uses this file to assign a single area-weighted radar rainfall value to each subbasin during each time step of a simulation.

Figure 4.2 shows an example of the Stage IV radar rainfall product. The cumulative rainfall estimated for each grid cell during a one hour period is shown. For this one hour time period, a single rainfall amount would be assigned to each subbasin in

HMS based on the grid cell parameter file. This figure helps demonstrate the gridded nature of the radar rainfall estimates as well as the distributed nature of rainfall in time and space.

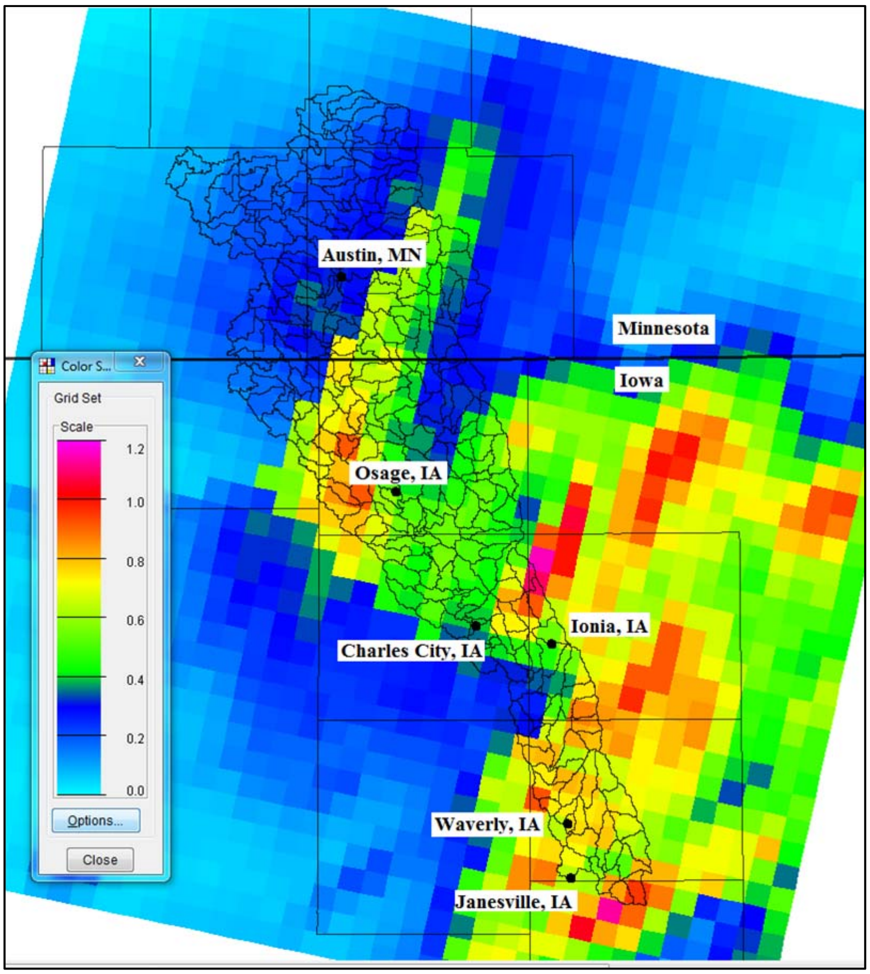

Figure 4.2. Example of the Stage IV radar rainfall product used as the precipitation input for historical storms in the Upper Cedar River Watershed HMS model. The Stage IV product provides hourly rainfall estimates for each 4 km x 4 km grid cell. The scale shown refers to the depth of rainfall (in inches) estimated for a one hour period.

Use of radar rainfall estimates provides increased resolution of the spatial and temporal distribution of precipitation over the watershed, and Stage IV estimates provide a level of manual quality control performed by the NWS that incorporates available rain gage measurements into the rainfall estimates. HMS simulations of historical storms using Stage IV data were the basis for model calibration and validation.

4.4.1.2 Hypothetical Rain Storms

Design rain storms were used for hypothetical or comparative analyses such as potential runoff generation, increased infiltration capacity through land use changes or soil improvements, and increased distributed storage within the watershed. These hypothetical design storms apply a uniform depth of rainfall across the entire watershed with the same timing everywhere. Soil Conservation Service (SCS or NRCS) Type II distribution, 24-hour storms were used for all hypothetical storms. The temporal distribution of rainfall for the SCS Type II, 24-hour storm is shown in Figure 4.3. Nearly 60% of the rainfall is applied in hours 11-13, with over 40% being applied in hour 12 of the 24-hour design storm.

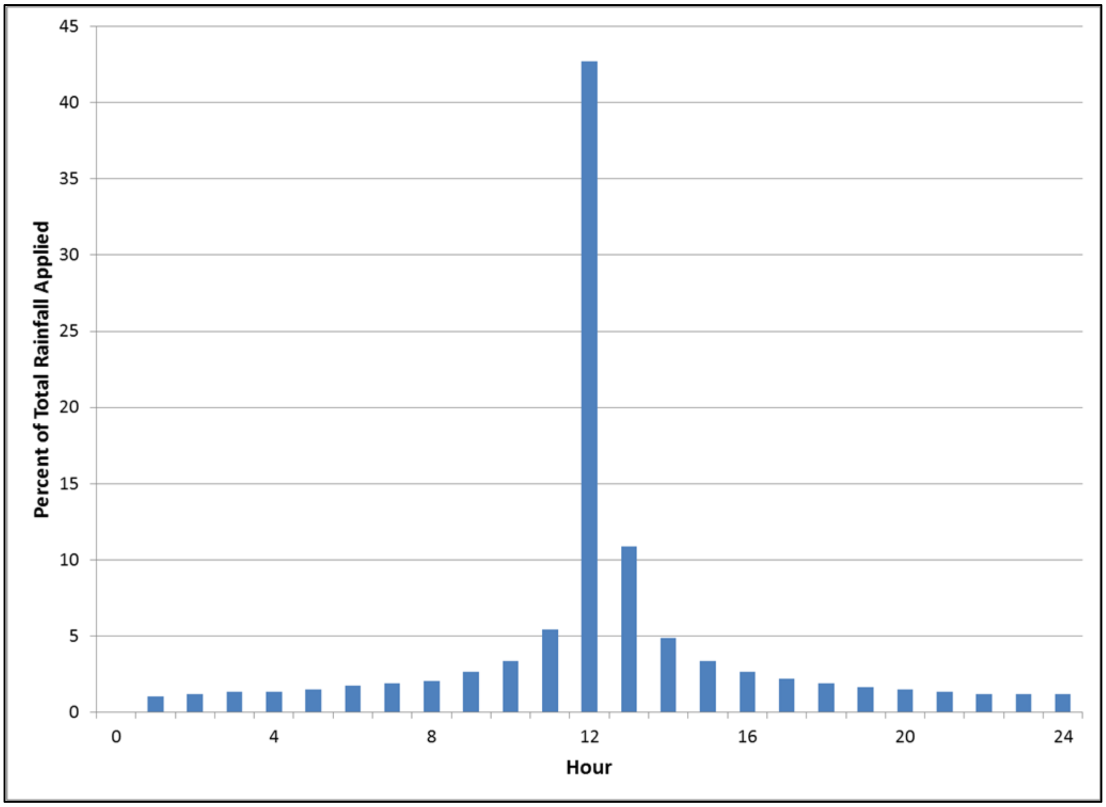

Figure 4.3. Temporal distribution of the SCS Type II, 24-hour hypothetical storm for a given rainfall amount. Hypothetical storms were used in HMS for comparative analyses of different watershed improvements.

Point precipitation values (rainfall depths) for the 10-, 25-, 50-, and 100-year average return period, 24-hour storms were derived using the online version of NOAA Atlas 14 – Point Precipitation Frequency Estimates (Perica et al., 2013). Point precipitation frequency estimates were collected at the centroid of the Upper Cedar River Watershed. The values were similar to the point precipitation frequency estimates at St. Ansgar and Osage (within 0.1 inches), both of which are centrally located in the watershed, so this served as a check that the basin centroid precipitation estimates were reasonable.

Studies have been performed on the spatial distribution characteristics of heavy rainstorms in the U.S. Midwest. Point precipitation frequency estimates are generally only applicable for drainage areas up to 10 mi^2 before the assumption of spatial uniformity is no longer valid. For drainage areas between 10 and 400 mi^2, relations have been established between point precipitation estimates and a reduced, areal mean precipitation approximation. Areal reduction factors based on storm duration and drainage area can be found in the *Rainfall Frequency Atlas of the Midwest* (Huff and Angel, 1992). This reference does not recommend adjusting point estimates for watersheds beyond 400 mi^2, as the dependence between the point and areal values breaks down for watersheds larger than this. Areal reduction factors for a 24-hour storm derived by Huff and Angel are summarized in Table 4.2.

Table 4.2. Areal reduction factors for point precipitation estimates for drainage areas of 10-400 mi^2.

Drainage Area (mi^2)	10	25	50	100	200	400
24-Hour Storm Areal Reduction Factor	0.99	0.97	0.95	0.94	0.93	0.91

Source: Huff, Floyd A., and James R. Angel. *Rainfall Frequency Atlas of the Midwest*. Illinois State Water Survey, Champaign, Bulletin 71, 1992.

For the comparative analyses that were performed in this modeling effort, an extrapolation was performed to get an areal reduction factor beyond 400 mi^2. It is agreed that this depth of rainfall would not fall uniformly across a watershed this large; however, to have reasonable rainfall depth estimates for the average recurrence interval 24-hour storms, extrapolation of the areal reduction factors in Table 4.2 was made to an area the size of the modeled portion of the watershed (1663 mi^2 at Janesville). The extrapolation is shown in Figure 4.4. Using the power law relationship estimated from the Least Squares method, an areal reduction factor of 0.88 was applied to the point precipitation frequency estimates for the Upper Cedar River Watershed.

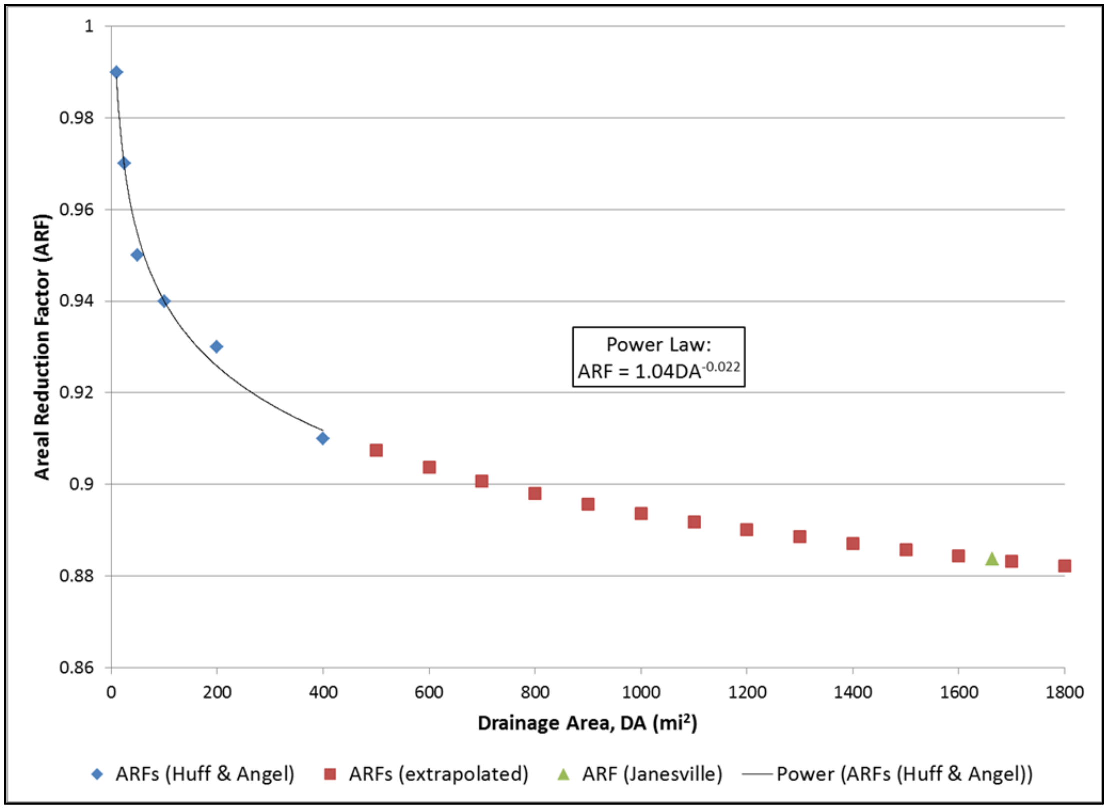

Figure 4.4. Estimation of an areal reduction factor for the Upper Cedar River Watershed. Extrapolation of areal reduction factors beyond 400 mi^2 was performed to estimate a value for the modeled portion of the Upper Cedar River Watershed. An areal reduction factor of 0.88 was selected.

Table 4.3 summarizes the point precipitation frequency estimates estimated at the basin centroid for the 10-, 25-, 50-, and 100-year, 24-hour design storms, along with the areal reduced values used for the hypothetical analyses in HMS.

Table 4.3. Summary of NOAA point precipitation frequency estimates and areal reduced values for the 10-, 25-, 50-, and 100-year, 24-hour design storms.

24-Hour Design Storm Return Period (years)	NOAA Point Precipitation (inches)	Areal Reduced Precipitation (inches)
10	4.58	4.05
25	5.71	5.05
50	6.67	5.89
100	7.71	6.81

4.4.2 Runoff Estimation

The NRCS Curve Number (CN) methodology was used to determine the rainfall-runoff partitioning in the Upper Cedar River Watershed HMS modeling. The CN method uses measurable watershed characteristics to estimate infiltration and runoff depths. The CN serves as an index for runoff potential and typical 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. Precipitation excess (or runoff depth) over a given area is estimated using the following well-known relationship:

$$Q_e = \frac{(P - I_a)^2}{(P - I_a) + S}$$

Q_e = runoff depth (inches)
P = total precipitation (inches)
S = potential maximum soil retention (inches)
I_a = initial abstraction (inches)

The initial abstraction (I_a) refers to the initial amount of rainfall that must fall before any runoff begins and accounts for losses due to plant interception, soil wetting, and depressional storage on the surface (Ponce, 1996). The remaining precipitation is partitioned between runoff and infiltration. The potential maximum retention (S) reflects the soil's ability to infiltrate and store water and is inversely related to the CN:

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

Essentially, the CN parameter maps the soil storage on to a 0-100 scale, where higher CN values represent a smaller soil storage capacity.

CNs are an estimated parameter based on the intersection of a specific land use and the underlying soil type. General guidelines for CNs based on land use and soil type are available in technical references from the NRCS. The CNs assigned to each land use and soil type combination in the Upper Cedar River HMS model are shown in Table 4.4 below.

Table 4.4. Curve Number assignment in the Upper Cedar River Watershed based on land use and soil type.

		Hydrologic Soil Group			
2006 NLCD Code	*Description*	*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/scub	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

NOTE: Curve Number combinations derived from *Urban Hydrology for Small Watersheds* (TR-55), Table 2-2, June 1986.

In cases where a specific land use or soil type was undefined for an area, Hydrologic Soil Group B (Iowa DOT, 2013) or the row crop agriculture land use class was assigned. Additionally, soils that had been assigned a dual soil code (A/D, B/D, and

C/D) were reassigned to the 100% drained condition (A, B, or C, respectively) to account for agricultural tile drainage practices assumed present throughout the watershed.

A CN grid was generated for the Upper Cedar River Watershed in ArcGIS by intersecting the digital Soil Survey Geographic Database (SSURGO) soils dataset (Figure 3.6) with the 2006 NLCD (Figure 3.8). After the CN grid was created, HEC-GeoHMS tools were used to perform area-weighted averaging within each subbasin to assign a single CN to each subbasin. The CN grid and corresponding subbasin composite CNs developed for the Upper Cedar Watershed are shown in Figure 4.5. Over 50% of the subbasin CNs are between 78 and 81 (25th-75th percentiles), reflecting the large amount of row crop agriculture and B and C-type soils in the basin.

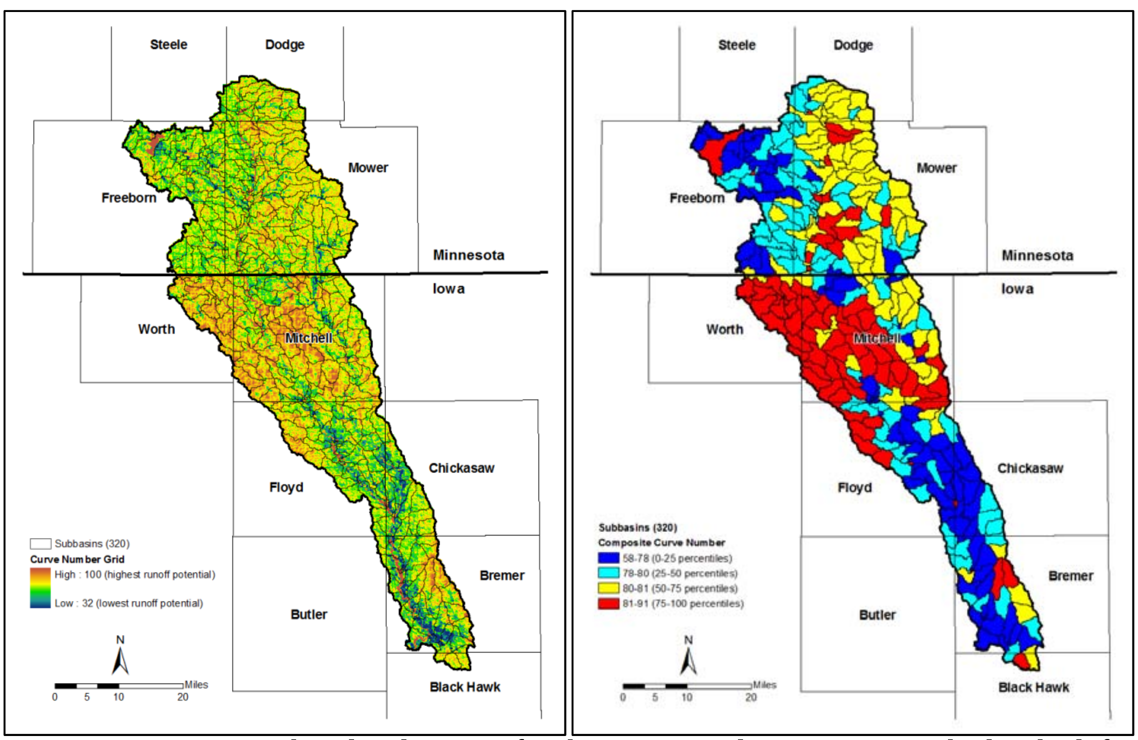

Figure 4.5. Curve Number development for the Upper Cedar River Watershed. The left figure shows the Curve Number grid and the right figure shows the single Curve Number calculated for each subbasin through area-weighted averaging.

4.4.3 Runoff Hydrographs

Excess precipitation was converted to a direct runoff hydrograph for each subbasin using the Clark Unit Hydrograph method. The Clark Unit Hydrograph model is a well-known synthetic unit hydrograph method developed in the 1940s that accounts for translation (delay) of the excess precipitation as it travels to the subbasin outlet and attenuation (reduction) of the peak discharge at the subbasin outlet due to surface storage. Travel time effects are accounted for by lagging the excess precipitation by its estimated travel time to the subbasin outlet. This translation hydrograph is then routed through a linear reservoir to reflect temporary stream channel storage effects. An average unit hydrograph reflecting the discharge at the subbasin outlet is derived, and convolution is performed to create the direct runoff hydrograph at the outlet of each subbasin (Kull and Feldman, 1998).

The Clark Unit Hydrograph method was selected as a way to represent the hydrologic impacts of tile drainage. As described in the Literature Review (Chapter 2), a delayed hydrologic response is hypothesized for tiled areas since the rain may infiltrate into the ground several feet before intersecting a tile and draining to a stream. For the same area without tiling, much of the rain may never infiltrate into the ground as a result of the poorly drained soil characteristics and instead taken a much faster path to a stream as surface runoff. The lag due to travel time effects incorporated in the Clark method is one way to represent this. Likewise, tiled areas may provide additional subsurface storage for temporary infiltration as a result of the groundwater table being lowered to the elevation of the tile. Routing the translation hydrograph through a linear reservoir is one way to represent the aggregated impacts of both surface and subsurface storage effects.

Two different versions of the Clark Unit Hydrograph method were used in HMS depending on the type of event being simulated. The ModClark method was used for simulating historical rain events while the traditional Clark method was used for hypothetical storm simulations. The difference between the two methods lies in the way the translation hydrograph is derived. With the traditional Clark method, a pre-developed time-area histogram reflecting a generalized watershed shape (parabola) defines the portion of subbasin area contributing runoff to the subbasin outlet as a function of time. The ModClark method allows radar rainfall to be used and uses a grid-based travel time model. Instead of lagging the precipitation excess to the subbasin outlet by a pre-defined time-area relationship, the excess precipitation from each HRAP grid cell within a subbasin is lagged to the subbasin outlet by a scaled amount of the estimated subbasin time of concentration (T_c). Assuming a constant travel speed, the travel time of excess precipitation from each grid cell to the subbasin outlet is:

$$[Travel\ Time]_{cell} = T_c \left[\frac{[Travel\ Length]_{cell}}{Maximum\ of\ Cell\ Travel\ Lengths\ in\ Subbasin}\right]$$

Distances of each grid cell to the subbasin outlet are specified in the same grid cell parameter file required to run radar rainfall. Once the translation hydrograph is developed, both Clark methods route the hydrograph through a linear reservoir to account for temporary storage effects (Kull and Feldman, 1998).

Besides the way the translation hydrograph is derived, the Clark and ModClark methods require two additional inputs – time of concentration (T_c) and a time storage coefficient (R). 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 and accounts for travel time effects. This was estimated as 5/3 times the lag time, where lag time is the time difference between the center of mass of the excess precipitation and the peak discharge of the runoff hydrograph. This is a reasonable approximation according to NRCS methodology (Woodward, 2010). Inputs required to determine the lag time for each subbasin include the subbasin slope in percent (Y), length of the longest flowpath in the subbasin in feet (L), and maximum potential retention (S) in the subbasin in inches, which is inversely related to the subbasin CN. An NRCS approximation was used to estimate time of concentration (in hours):

$$T_c = \frac{5}{3}\left[\frac{L^{0.8}(S+1)^{0.7}}{1900\sqrt{Y}}\right]$$

While time of concentration is a measure of lag due to travel time effects as water moves through the watershed, the time storage coefficient (R) is a measure of lag due to natural storage effects in the subbasin. Based on the literature, one way of estimating R is from the combined parameter $R/(T_c+R)$, with a ratio of 0.65 providing an initial starting point. A final value of the time storage coefficient for each subbasin is determined through calibration (Kull and Feldman, 1998).

4.4.4 Baseflow

Baseflow for each subbasin was approximated by the first order exponential decay relationship

$$Q_t = Q_o k^t$$

where Q_t is the baseflow at time t, Q_o is the initial baseflow at the beginning of a simulation, and k is an exponential decay constant describing the rate of decay of baseflow with time. Q_o, k, and a threshold indicating when baseflow should be reactivated were required for each subbasin in HMS.

Records from the six USGS stage/discharge gages were used to set initial baseflow conditions (Q_o) prior to each historical storm event simulation. One option for defining Q_o is as a discharge to area ratio (cfs/mi^2); therefore, the discharge estimated at each USGS stage/discharge gage at the starting time of a historical storm event simulation was normalized by the gage's drainage area. This value was then assigned as the initial baseflow condition to all subbasins upstream of that gage. Three initial baseflow conditions were calculated for each historical storm event simulation and applied to the appropriate upstream subbasins – one for the Cedar River and its tributaries upstream of the Austin, MN gage; one for the Little Cedar River and its tributaries upstream of the Ionia gage; and one for all other subbasins using the average cfs/mi^2 ratio calculated from the Osage, Charles City, Waverly, and Janesville gages.

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 for each subbasin. Typical recession constants for interflow and groundwater components range from 0.8-0.95 (Feldman, 2000). A value in this range was initially selected for all subbasins and adjusted through calibration. Finally, baseflow was assigned to reactivate on the receding limb of the hydrograph when the current discharge was 10% of the peak subbasin discharge. This parameter was also adjusted through calibration.

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

4.4.5 Flood Wave Routing

Conveyance of runoff through the river network, or flood wave routing, was executed using the Muskingum routing method. Derived from Conservation of Mass, the Muskingum method assumes a linear but non-level water surface so both increases and decreases in storage can be calculated as a flood wave passes through the reach.

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 initially by dividing the reach length by a reasonable travel velocity (1-5 feet per second, in general), but is generally refined through calibration.

Flow routing through Geneva Lake reservoir and hypothetical ponds was executed using level pool routing. A level water surface is assumed. A storage-outflow discharge relationship is required along with an initial storage or discharge 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 or ponds modeled were assumed to be filled to the normal pool level at the beginning of each simulation.

4.4.6 Additional Notes

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 considered since short duration, event-based storms were the primary focus.

4.5 Chapter Summary

Chapter 4 details the hydrologic model development of the Upper Cedar River Watershed. HEC-HMS, a lumped parameter surface water model, is being used to simulate both historical (use of radar rainfall estimates) and hypothetical (use of SCS Type II, 24-hour design storms) storms in the watershed. Using Arc GIS and HEC-GeoHMS tools, the watershed was broken into smaller components and the stream network and subbasins were delineated based on user-defined thresholds. In total, 320 subbasins with an average area of 5.2 mi^2 were created in HMS. Terrain analysis was performed to derive physical watershed characteristics including areas, slopes, and river reach lengths. The HEC-GeoHMS extension in Arc GIS provides a convenient way to create, organize, and output the necessary datasets needed to build a project in HMS.

Well established methods are being used to describe the hydrology of the Upper Cedar River Watershed in the HMS model. The NRCS Curve Number methodology is being used for rainfall-runoff portioning, the Clark Unit Hydrograph converts excess precipitation to discharge and attempts to account for tile drainage effects, and the Muskingum routing method conveys flows through river reaches. Initial parameters for each of these components were derived from Arc GIS or from the literature; final adjustments to parameters were made through calibration (Chapter 5).

CHAPTER 5: MODEL CALIBRATION

5.1 Introduction

Model calibration is a process of taking an initial set of parameters developed for the hydrologic model through Arc GIS and other means and making adjustments to them so that simulated results produced by the model match as close as possible to an observed time series, typically stream discharge at a gaging station. Doing so provides some level of confidence that the model will reasonably predict the watershed response for different rainfall inputs and initial conditions. Once a satisfactory level of calibration is achieved, hypothetical scenarios – different flood mitigation strategies for reducing flood peak discharges – can be considered knowing the simulated results will be reasonable. It should be noted that adjustments to parameters should not be made to great extremes just to manipulate the end results to match the observed time series. If this is necessary, the model does not reasonably reflect the physical processes occurring in the watershed and alternative methods should be considered.

This chapter details the calibration efforts made for the Upper Cedar River Watershed. This includes the major calibration measures taken, a summary of the final calibrated model parameters, and results for the six historical storms simulated.

5.2 Calibration Measures

The Upper Cedar River Watershed HMS model was calibrated to six storm events that occurred between September 2004 and June 2013. Storms were selected based on their magnitude, time of year they took place, and based on the availability of Stage IV radar rainfall estimates and USGS discharge estimates. Large, high runoff storms occurring between May and September were selected so the impacts of snow, rain on frozen grounds, and freeze-thaw effects that can exist during late fall to early spring were minimized. Global adjustments were made to the model parameters described in Chapter 4 to best match the simulated hydrograph to the observed discharge time series at each

USGS stage/discharge gage location. Model performance was evaluated based on how well the simulated hydrograph peak discharge, time of peak discharge, total runoff volume, and general shape resembled the observed hydrograph at a particular USGS gage location.

Two major calibration measures were taken to improve the HMS model performance. Antecedent moisture conditions (AMC) prior to a historical storm simulation were accounted for in a different way than typically done by NRCS CN methodology, and Clark Unit Hydrograph parameters were adjusted to account for the effects of tile drainage on the basin hydrology.

5.2.1 Antecedent Moisture Conditions

Rainfall-runoff partitioning for an area is dependent on the antecedent soil moisture conditions (how wet the soil is) at the time rain falls on the land surface. 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 runoff volumes.

Existing NRCS methodology accounts for antecedent soil moisture conditions by classifying CNs into one of three classes: AMC I (dry), AMC II (average or normal), or AMC III (wet), which are statistically defined as the 10^{th}, 50^{th}, and 90^{th} percentiles of runoff depth, respectively (Hjelmfelt, 1982). Equations exist for adjusting CNs from the base AMC II condition to either the AMC I or III condition based on the seasonal five-day antecedent rainfall total prior to the event being simulated. The five-day antecedent rainfall totals defining the three AMC CN classes for the growing season developed by the NRCS are shown in Table 5.1.

Table 5.1. Five-day antecedent rainfall totals defining the AMC I, II, and III Curve Number classes for the growing season developed by the NRCS.

AMC Class	Runoff Depth Percentile	Growing Season 5-Day Antecedent Rainfall Total (inches)	Curve Number Adjustment
I (dry)	10^{th}	< 1.4	$CN(I) = \frac{4.2CN(II)}{10 - 0.058CN(II)}$
II (normal)	50^{th}	1.4-2.1	---
III (wet)	90^{th}	> 2.1	$CN(III) = \frac{23CN(II)}{10 + 0.13CN(II)}$

While this method provides a simple way to adjust CNs to reflect antecedent moisture conditions based on the five-day antecedent rainfall total, it is over simplified. The five-day antecedent rainfall totals listed in Table 5.1 define the AMC classes *everywhere*, regardless of geographic location or climate. Additionally, the five-day antecedent rainfall total applies equal weight to each of the five days preceding a storm to reflect the soil moisture conditions. Hence, rain that fell five days before or one day before the event being simulated is treated the same in determining the appropriate AMC CN class. In reality, the soil moisture conditions may be significantly different depending on how close in time the rain fell prior to the event being simulated. Finally, existing NRCS methodology provides only three discrete classes for CNs based on AMC. Substantial changes in CN can occur for only small differences in the five-day antecedent rainfall total (e.g. 2.09 inches (AMC II) versus 2.11 inches (AMC III)), which could result in drastic overestimations or underestimations of runoff volume.

Using records from the nine NOAA hourly/daily precipitation stations listed in Table 3.2 and shown in Figure 3.9, basin average daily precipitation was computed for the Upper Cedar River Watershed over a 65-year period (1948-2013) using Thiessen

Polygons. Since historical storms that occurred primarily during the growing season (May-September) were simulated in HMS, only precipitation that fell in this time period was considered; precipitation between October 1 and April 30 of each year was not considered. Using the basin average daily precipitation calculated for the 65-year period, the five-day rainfall total cumulative distribution function (CDF) was developed and compared to existing NRCS AMC definitions for the growing season (Table 5.1). The basin average, five-day rainfall total CDF developed for the Upper Cedar River Watershed is shown in Figure 5.1.

Evident from Figure 5.1, using existing NRCS definitions for defining AMC classes (Table 5.1) would place the Upper Cedar River Watershed in the AMC I (dry) class most of the time, as 86% of the five-day, basin average rainfall totals are less than 1.4 inches. ‘Normal’ conditions for the watershed, defined by five-day antecedent rainfall totals between 1.4 and 2.1 inches (AMC II), would only occur around 10% of the time, and ‘wet’ conditions (AMC III) would rarely occur (5% of the time). In other words, the existing NRCS five-day rainfall totals defining the three AMC classes are not well-suited for the Upper Cedar River Watershed. Applying the statistical definitions of the three AMC classes to the Upper Cedar River Watershed, the AMC I class would be defined by five-day rainfall totals between 0-0.01 inches (0-10th percentiles), the AMC II class by five-day rainfall totals between 0.01-1.63 (10th-90th percentiles), and the AMC III class by five-day rainfall totals greater than 1.63 inches (90th-100th percentiles). The five-day rainfall total in the Upper Cedar River Watershed over the 65-year period was zero for 6% of the records.

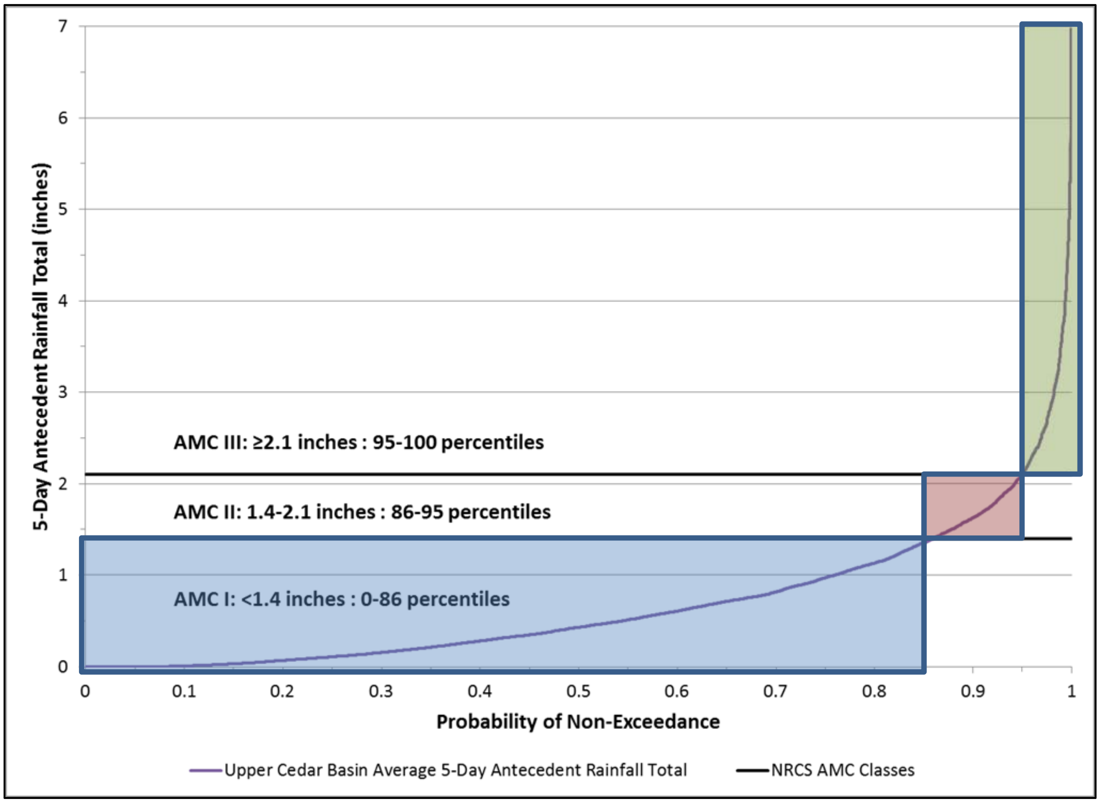

Figure 5.1. Five-day rainfall total cumulative distribution function (CDF) for the Upper Cedar River Watershed. The CDF was computed using the daily basin average precipitation computed between May and September of each year over a 65 year period (1948-2013).

To better account for AMC at the beginning of a simulation in the HMS model, a soil moisture proxy known as the antecedent precipitation index (API) was used instead of the five-day antecedent rainfall total. The API may be calculated over a longer time period and uses a temporal decay constant (k) that accounts for soil moisture losses and allows more or less weight to be applied to precipitation that fell closer in time to the event of interest. The API on day t is calculated as

$$API_k(t) = kAPI_k(t-1) + P(t)$$

where P is the gaged, basin average precipitation. As with the five-day antecedent rainfall total, a greater API reflects a wetter initial condition and greater runoff potential.

The goal of this analysis was to relate CN to API so appropriate CN adjustments could be made in the HMS model to reflect soil moisture conditions at the beginning of a simulation. Rather than have only three discrete AMC classes for CN classification, a continuous function was developed so a unique CN change could be applied for all AMC. The basin average AMC I, II, and III CNs were calculated using the subbasin CNs derived in Chapter 4 (Figure 4.5) so the percent change from the AMC II CN could be computed for the AMC I and III classes. Then, linear interpolation was performed between the percent changes for the basin average AMC I, II, and III CNs and their statistical definitions (10^{th}, 50^{th}, and 90^{th} percentiles, respectively). In this way, a global CN adjustment (applied to all 320 subbasin CNs) could be determined for any API percentile.

In order to apply a CN adjustment based on some API percentile, an analysis was performed to determine the optimal value of k to use to compute the API. The decay constant (k) is reported in the literature to vary between 0.8 and 0.98 (Beck et al., 2009). Various values of k in this range were assumed and the CDF for each API alternative was computed using the basin average daily precipitation records. For each k considered, each of the six calibration storms were simulated using the appropriate CN adjustment predicted from the percentile corresponding to the API for the day before the historical event simulation. The optimal k was selected as the one that yielded the most similar results to the observed hydrographs at the USGS stage/discharge gage locations considering all six calibration storms. This resulted in a value of 0.8 being selected for k. For comparison, considering a five-day period and assuming equal precipitation fell on each day, the five-day antecedent rainfall total is computed assuming equal weight (20%) for each day, while the API (k = 0.8) is computed by applying a weight of nearly 30% to the precipitation that fell one day before the event and a weight of 12% to precipitation that fell five days before the event.

The AMC methodology derived for the Upper Cedar River Watershed is shown in Figure 5.2. For each historical storm event, the API for the day before the start of the simulation was calculated, its percentile referenced from the CDF, and the percentile was used to determine the percent adjustment in CN to apply to all subbasin CNs in the HMS model. A separate analysis was also performed in which the optimal subbasin CN adjustment for each calibration storm was determined through trial and error (independent of API); these results are shown by the crosses in Figure 5.2. These results were used to adjust the initial CN-API curve (coarser dashed line) to better the simulated results. Because the original curve tended to lie above the best fit calibration events (crosses), the 50th percentile point was shifted down by the average percent difference between the CN adjustments predicted by the original curve and the CN adjustments determined through trial and error. This resulted in a 4.78% reduction of the base AMC II CN. New basin average AMC I, II, and III CNs were calculated and the percent change of the AMC I and AMC III CNs from the AMC II CN defined the endpoints of the curve. The final curve used to adjust CNs to reflect the AMC prior to a historical storm is shown by the solid black line in Figure 5.2. The finer dashed line shows how CNs would be defined into one of three classes if the NRCS methodology (Table 5.1) were being used (same curve as Figure 5.1).

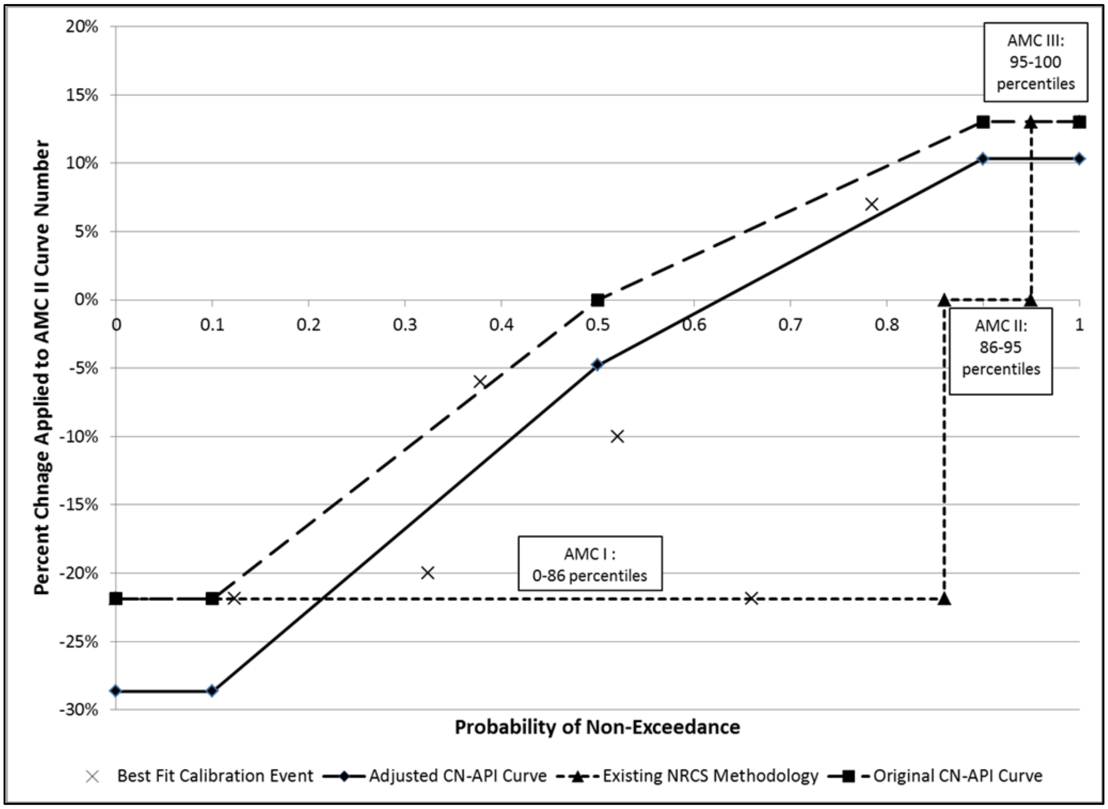

Figure 5.2. Accounting for antecedent moisture conditions in the Upper Cedar River Watershed HMS model through use of the antecedent precipitation index (API). Precipitation gage records were used to calculate the API prior to each historical storm event and a corresponding percent change in Curve Number was applied to each subbasin Curve Number to account for the initial soil moisture conditions.

5.2.2 Adjustment of Clark Basin Storage Coefficient to Account for Tile Drainage Impacts

The other primary calibration measure taken was adjustment to one of the Clark Unit Hydrograph parameters to reflect the presence of tile drainage in the watershed. Under normal conditions, a delayed hydrologic response downstream of tiled areas (installed in poorly draining soils) is hypothesized since a greater amount of rain is temporarily stored in the subsurface before intersecting a tile and draining to a stream. To account for this, the time storage coefficient (R), a measure of lag due to natural storage effects, in the Clark Unit Hydrograph method was increased in each subbasin. By doing so, the peak subbasin discharge was reduced and the receding limb of the hydrograph extended. The time storage coefficient for each subbasin was initially determined based on a ratio of 0.65 for the combined parameter $R/(T_c+R)$ reported in the literature, corresponding to an R-value 1.86 times greater than the time of concentration (Kull and Feldman, 1998). To reflect tile drainage impacts, the ratio for $R/(T_c+R)$ was increased to 0.87, corresponding to an R-value 6.69 times greater than the time of concentration. Hence, the time storage coefficient for each subbasin was increased by a factor of 3.6 through calibration to reflect the additional temporary storage effects and attenuation that may result from tile drainage.

5.3 Summary of Calibrated HMS Model Parameters

Table 5.2 summarizes the final set of HMS model parameters determined through calibration for the Upper Cedar River Watershed.

Table 5.2. Summary of calibrated HMS model parameters for the Upper Cedar River Watershed.

Parameter	Initial Value	Calibrated Value	Comments
Runoff Estimation			
Curve Number (CN)	Three discrete classes based on initial moisture state; subbasin CNs determined from Table 4.4	AMC II CNs reduced by 4.78%, AMC I and III CNs recalculated based on new AMC II CN	Development of continuous function to adjust CN values based on initial moisture state
Initial Abstraction (I_a)	20% of potential maximum retention (S)	20% of potential maximum retention (S)	
Subbasin Hydrograph Development			
Time of Concentration (T_c)	5/3 of subbasin lag time	5/3 of subbasin lag time	Lag time calculated using original, uncalibrated CNs
Basin Storage Coefficient (R)	Based on $R/(T_c+R)$ ratio = 0.65	Based on $R/(T_c+R)$ ratio = 0.87	Increased to account for tile drainage effects
River Routing			
Muskingum Flood Wave Travel Time (K)	Based on a velocity of 1 m/s (3.28 ft/s)	Based on a velocity of 0.9 m/s (2.95 ft/s)	
Muskingum Attenuation Factor (X)	0.2	0.2	Little impact on model output
Baseflow			
Decay Constant (k)	0.8	0.8	Reasonable value for interflow
Baseflow Reset on Receding Limb	10% of subbasin peak discharge	10% of subbasin peak discharge	

5.4 Calibration Storm Results

The Upper Cedar River Watershed HMS model was calibrated to six storm events that occurred between September 2004 and June 2013. High runoff storms occurring between May and September were selected for several reasons. While high streamflows are most frequent in March/April, the largest magnitude floods have typically occurred later in the summer months (section 3.2). Additionally, projects to be constructed in the watershed, particularly runoff reduction practices, are likely to provide greater flood reduction benefits during the growing season as opposed to earlier in the year when soil infiltration is more inhibited due to frozen conditions. Thus, the HMS model was calibrated to high runoff storms occurring between May and September to reflect the time period when built projects are likely to have greatest impact.

Model performance was evaluated based on how well the simulated hydrograph peak discharge, time of peak discharge, total runoff volume, and general shape resembled the observed hydrograph at a particular USGS discharge gage location. Calibration results (simulated hydrographs) for the six historical storms are presented and possible reasons for difference between the simulated and observed responses are discussed.

5.4.1 September 2004

The September 14-22, 2004 storm was characterized by a basin average Stage IV radar rainfall total of 4.1 inches and an observed runoff coefficient of 0.37 and peak discharge of 25,000 cfs at Janesville. Drier than normal conditions were present before the storm (12th percentile of API), so the uncalibrated subbasin CNs were reduced by 27.3% (determined from solid black line of Figure 5.2). The model did a reasonable job simulating this event but the simulated hydrographs are typically early and underestimate runoff volume; at Janesville, the peak flow is underestimated by only 6%, but the timing of the peak flow is approximately a day early and the runoff volume is underestimated by 28%.

Underestimation of runoff volume may be due to the radar rainfall estimates being approximately 8% less than the rain gage estimates (4.44 inches), 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. Simulated and observed hydrographs at the USGS stage/discharge gages operational during this time period are shown in Figure 5.3.

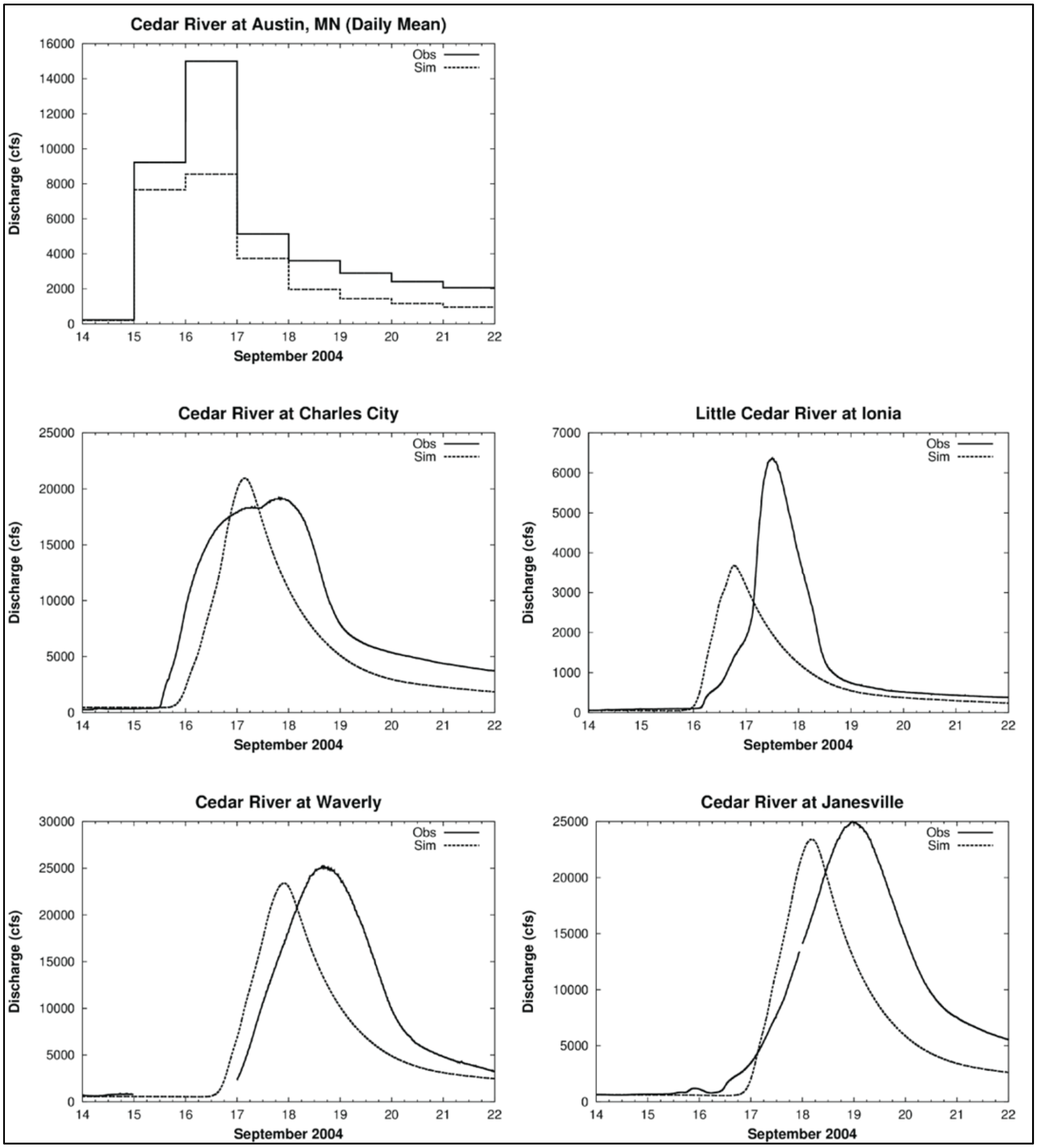

Figure 5.3. Hydrograph comparisons for the September 2004 calibration storm.

5.4.2 August 2007

The August 18-27, 2007 storm was characterized by a basin average Stage IV radar rainfall total of 7.49 inches and an observed runoff coefficient of 0.14 and peak discharge of 8,260 cfs at Janesville. Slightly wetter than normal conditions were present before the storm (66^{th} percentile of API), so the uncalibrated subbasin CNs were increased by 1.3%. Despite wetter antecedent conditions, very little rain was actually converted to runoff; as a result, simulated runoff volumes and peak flows are significantly overestimated in the model (overestimation of runoff volume and peak flow at Janesville by 298% and 439%, respectively). The simulated runoff coefficient at Janesville was 0.57.

Substantial overestimations in runoff volume and peak flows may be due to the radar rainfall estimates being approximately 10% greater than the rain gage estimates and lack of accounting for evapotranspiration losses which may be significant in mid-August near the peak of the growing season. As a result of this second point, this storm may have been driven by a greater subsurface flow mechanism, which is not considered in HMS. Despite these anomalies, one would still expect a storm yielding 7-8 inches of rain to produce more than one inch of runoff given the wet initial conditions. Simulated and observed hydrographs at the USGS stage/discharge gages operational during this time period are shown in Figure 5.4.

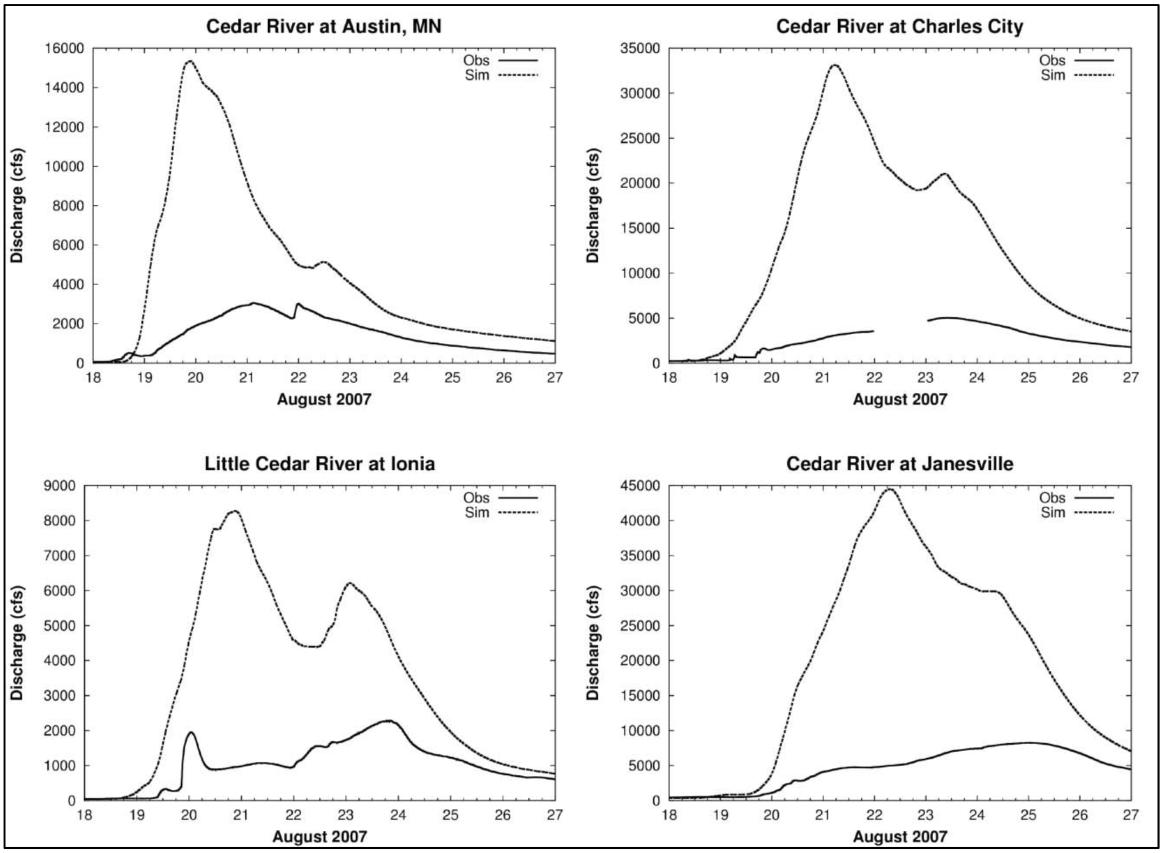

Figure 5.4. Hydrograph comparisons for the August 2007 calibration storm.

5.4.3 July 2010

The July 22-28, 2010 storm was characterized by a basin average Stage IV radar rainfall total of 3.07 inches and an observed runoff coefficient of 0.17 and peak discharge of 6,500 cfs at Janesville. Drier than normal conditions were present before the storm (32^{nd} percentile of API), so the uncalibrated subbasin CNs were reduced by 15.3%. Simulated runoff volumes and peak flows were still generally overestimated at most USGS gage locations. The simulated runoff coefficient at Janesville was 0.23.

Overestimation of the simulated runoff volumes and peak flows may be due to several reasons. Ignoring evapotranspiration losses may be a poor assumption in mid-July near the peak of the growing season. Also, despite substantial precipitation, this event produced little runoff, evident from the observed runoff coefficient at Janesville of

0.17 (the observed peak flow of 6,500 cfs at Janesville corresponds to less than the two-year return period streamflow). Therefore, the hydrologic response that is observed may be driven by a greater subsurface flow component. Simulated and observed hydrographs at the USGS stage/discharge gages operational during this time period are shown in Figure 5.5.

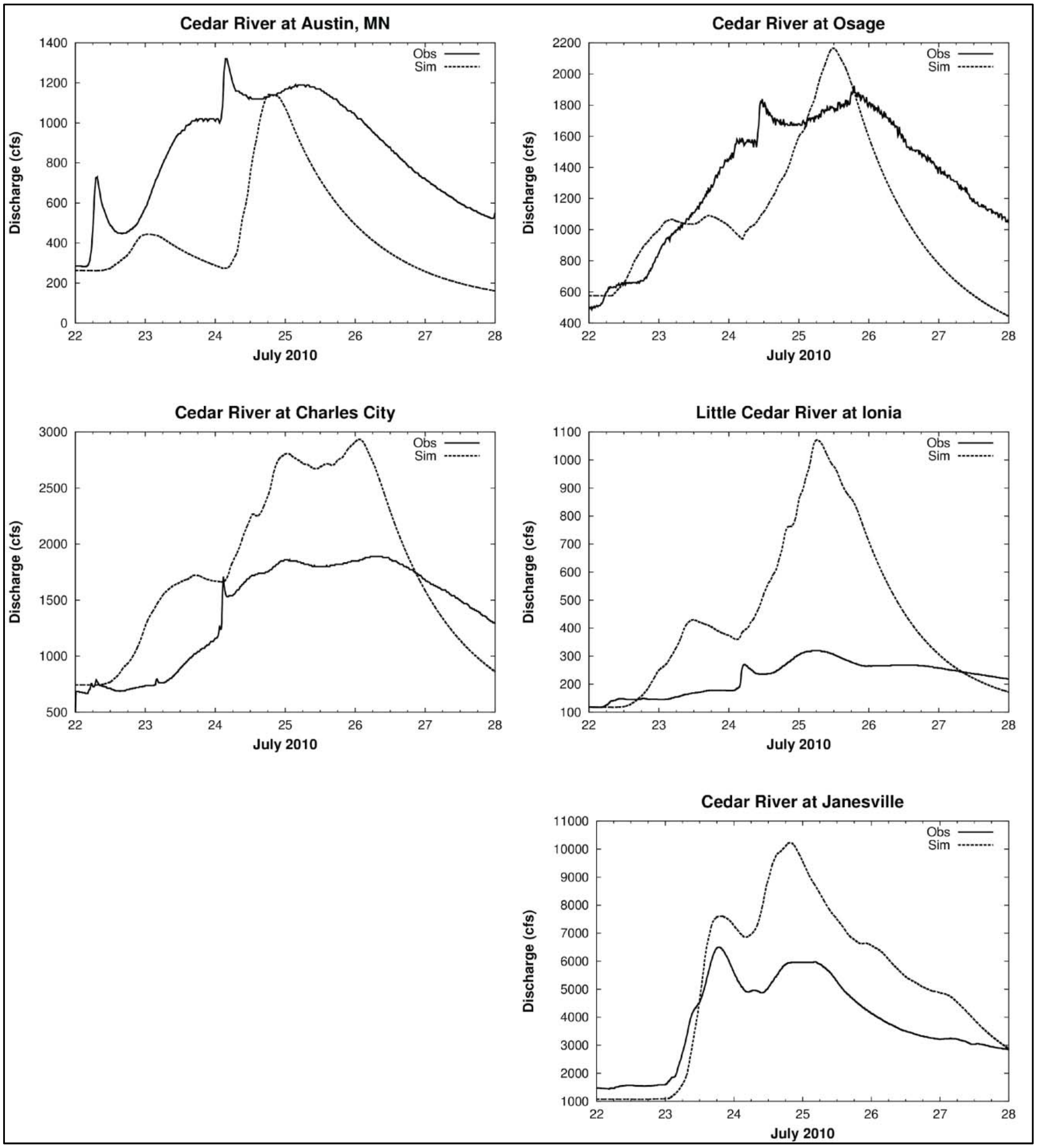

Figure 5.5. Hydrograph comparisons for the July 2010 calibration storm.

5.4.4 May 1-8, 2013

The May 1-8, 2013 storm was characterized by a basin average Stage IV radar rainfall total of 2.76 inches and an observed runoff coefficient of 0.30 and peak discharge of 11,000 cfs at Janesville. Near normal soil moisture conditions were present before the storm (52nd percentile of API), so the uncalibrated subbasin CNs were reduced by 4% according to Figure 5.2. The simulated runoff volume and peak flow at Janesville are overestimated by 13% and 25%, respectively, but overall are reasonable. The simulated runoff coefficient at Janesville was 0.34.

The greatest discrepancy between the simulated and observed hydrographs is the timing of the peak flow. The simulated response at all USGS gage locations occurs much earlier than was observed. The simulated peak flows are 1-3 days early. With model parameters already adjusted to create a more delayed response due to tile drainage, it is difficult to explain why the simulated response is so much earlier than the observations. Simulated and observed hydrographs at the USGS stage/discharge gages operational during this time period are shown in Figure 5.6.

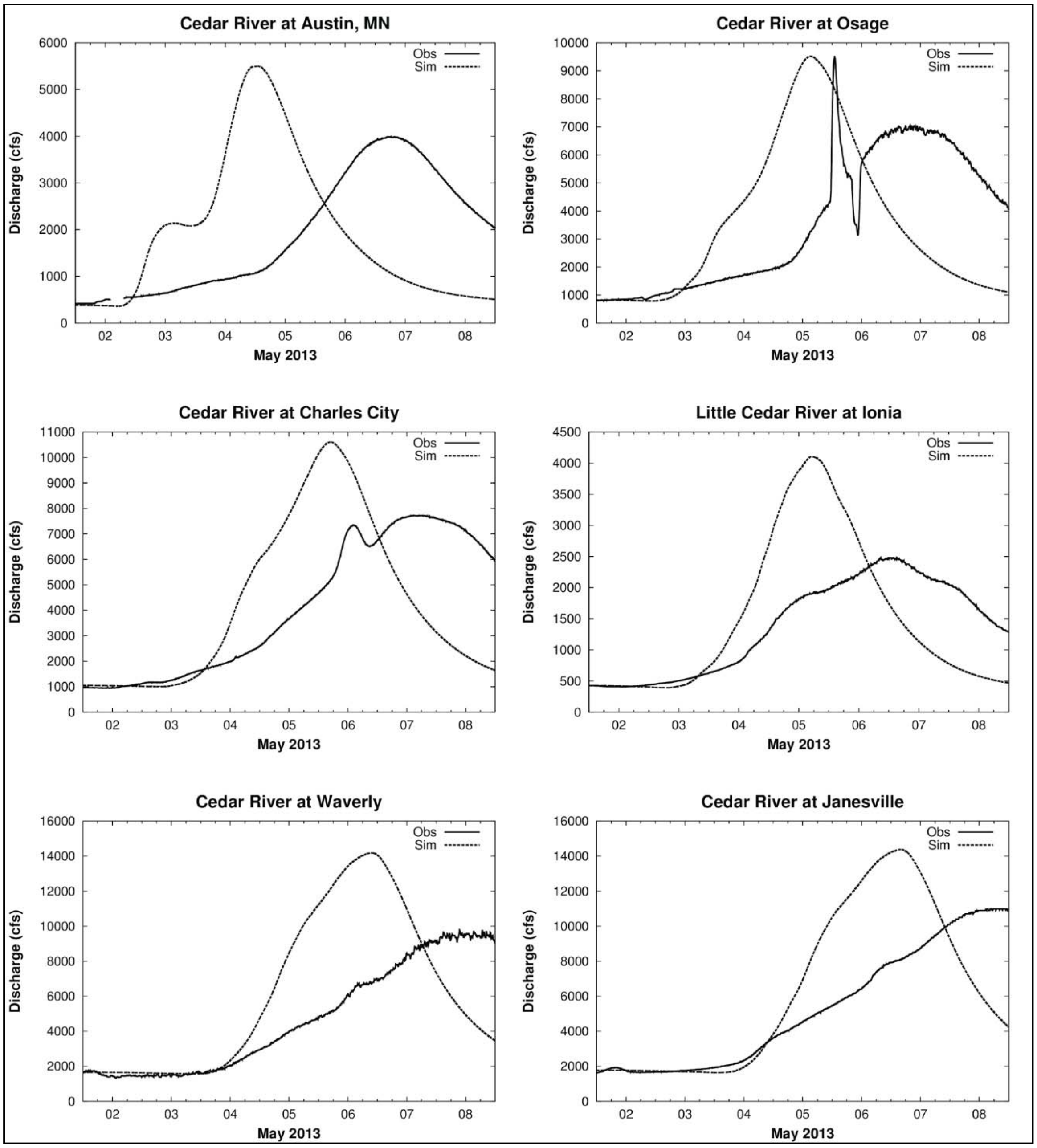

Figure 5.6. Hydrograph comparisons for the May 1-8, 2013 calibration storm.

5.4.5 May 16-25, 2013

The May 16-25, 2013 storm was characterized by a basin average Stage IV radar rainfall total of 4.76 inches and an observed runoff coefficient of 0.42 and peak discharge of 27,600 cfs at Janesville. Slightly drier than normal conditions were present before the storm (38th percentile of API), so the uncalibrated subbasin CNs were reduced by 12.1%. The HMS model did an acceptable job simulating this storm; the simulated peak flow and runoff volume at Janesville are underestimated by 6% and 8%, respectively, and the time of the peak flow is within a couple hours of when it was actually observed.

Reasonable agreement between the simulated and observed responses is expected since this was a larger storm event (the peak flow of 27,600 cfs at Janesville corresponds to approximately the 18-year return period streamflow) that partitioned a greater amount of rain into surface runoff. Simulated and observed hydrographs at the USGS stage/discharge gages operational during this time period are shown in Figure 5.7.

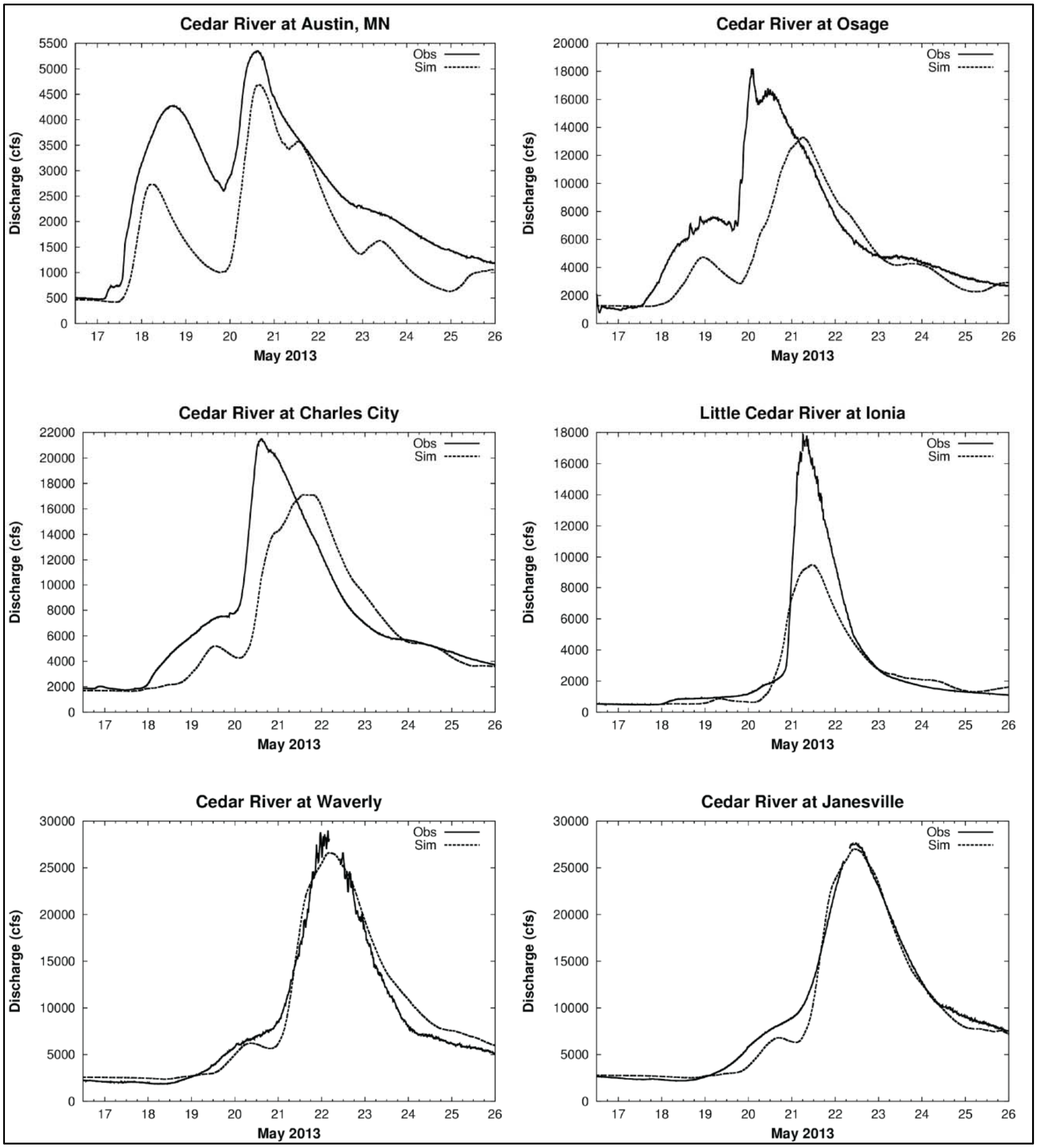

Figure 5.7. Hydrograph comparisons for the May 16-25, 2013 calibration storm.

5.4.6 June 2013

The June 10-19, 2013 storm was characterized by a basin average Stage IV radar rainfall total of 2.38 inches and an observed runoff coefficient of 0.55 and peak discharge of 13,800 cfs at Janesville. Wetter than normal conditions were present before the storm

(79^{th} percentile of API), so the uncalibrated subbasin CNs were increased by 6%. The HMS model did a reasonable job simulating this storm; the simulated peak flow at Janesville is overestimated by 3% while the runoff volumes are nearly identical.

Good agreement between the simulated and observed response is explained by similar reasons as for the May 16-25, 2013 storm – although this was a smaller storm event, over 50% of the rain was converted to surface runoff which plays to the strength of the HMS model. Simulated and observed hydrographs at the USGS stage/discharge gages operational during this time period are shown in Figure 5.8.

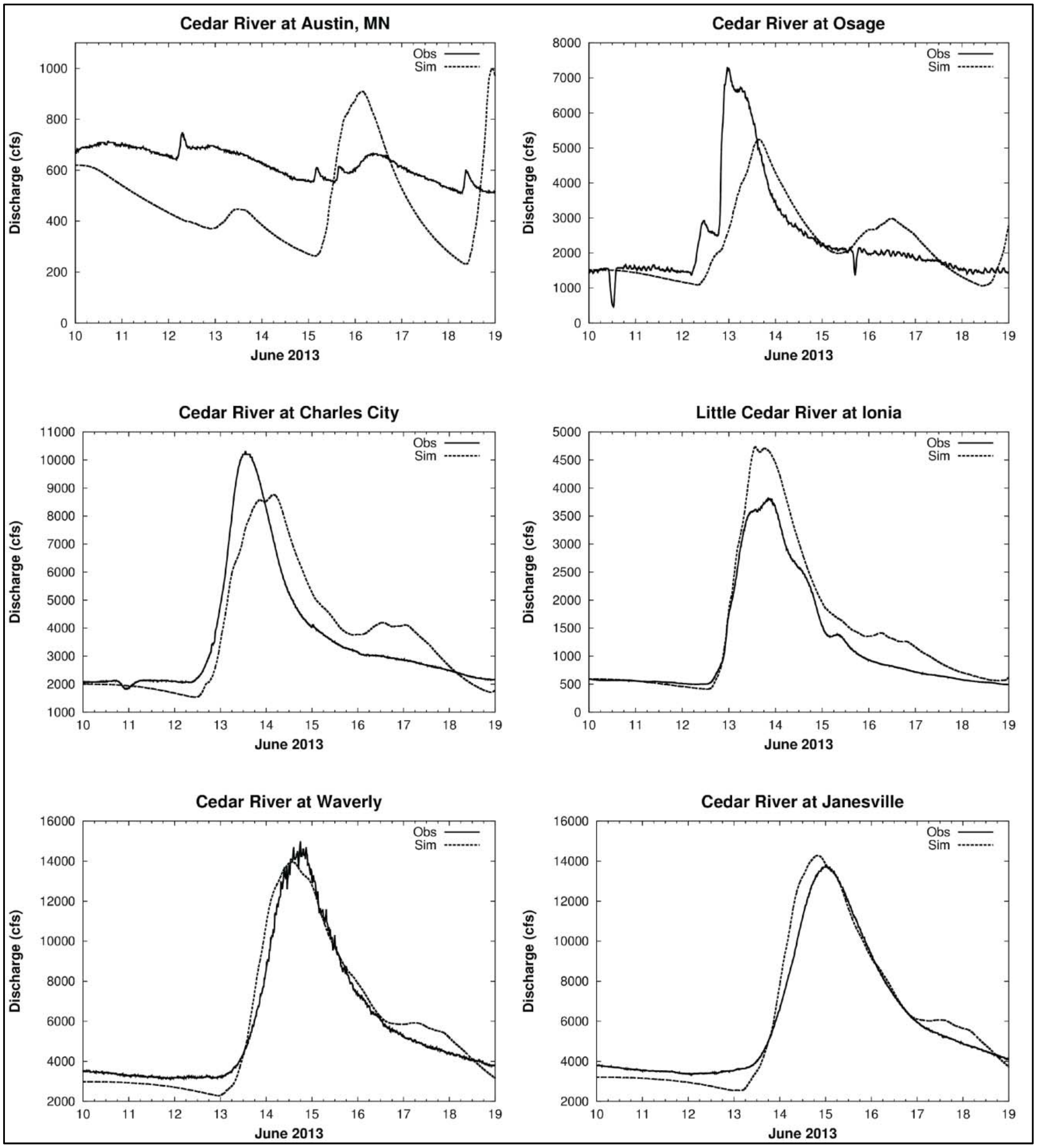

Figure 5.8. Hydrograph comparisons for the June 2013 calibration storm.

5.5 Summary of HMS Model Performance for Calibration Storms

The Upper Cedar River HMS model was evaluated by comparing the simulated peak discharge, time to peak discharge, total runoff volume, and general hydrograph shape to observations at the USGS stage/discharge gages in the basin. Apparent from section 5.4, model performance varied for the six calibration storms.

Figure 5.9 compares the simulated and observed peak discharges at the operational USGS stage/discharge gage locations for all six calibration storms. Comparing simulated and observed peak discharges indicates whether or not the magnitude of the flood simulated is reasonable. Overall, the HMS model did a reasonable job simulating the correct peak discharge magnitudes throughout the basin for the calibration storms. In general, the model tends to overestimate peak discharges of smaller flood events (e.g. August 2007) and does a better job of matching the peak discharges observed for larger flood events (September 2004 and May 2013 events). This is expected since greater peak discharges are characteristic of larger flood events, and larger flood events generally have a greater overland flow component.

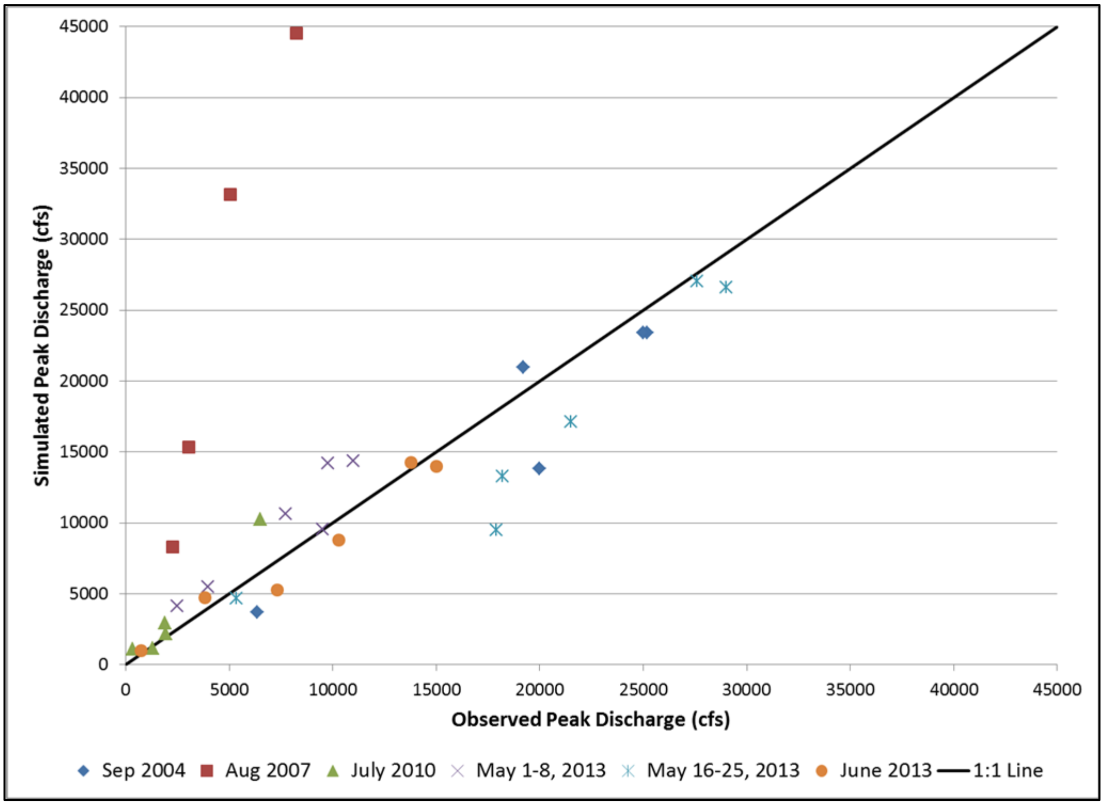

Figure 5.9. Comparison of simulated and observed peak discharges at operational USGS stage/discharge gage locations in the Upper Cedar River Watershed for the six calibration storms.

Figure 5.10 evaluates the ability of the HMS model to simulate the correct time to peak discharge at each of the USGS stage/discharge gage locations for the six calibration storms. For each calibration storm, the anomaly in the time to peak discharge at each operational USGS stage/discharge gage is plotted. Each anomaly (in hours) was calculated by taking the difference in the time to peak discharge between observed and simulated; a positive anomaly indicates the simulated time to peak discharge was later than observed, while a negative anomaly indicates the simulated time to peak discharge was earlier than observed. It should be noted that the simulated time to peak discharge at Austin, MN for the June 2013 event was nearly seven days late, but all other positive anomalies were less than 30 hours, so the upper limit was set at 30 hours to allow for greater viewing detail of timing anomalies for all other storm events.

The Upper Cedar HMS model tends to predict peak discharges earlier than observed for the September 2004 (15-20 hours), August 2007 (30-70 hours), and May 1-8, 2013 (10-50 hours) storm events. On the other hand, the model does a better job matching the time to peak discharge for the July 2010, May 16-25, 2013, and June 2013 storm events; the greatest anomalies for these storms are positive indicating the simulated times to peak discharge are later than was observed.

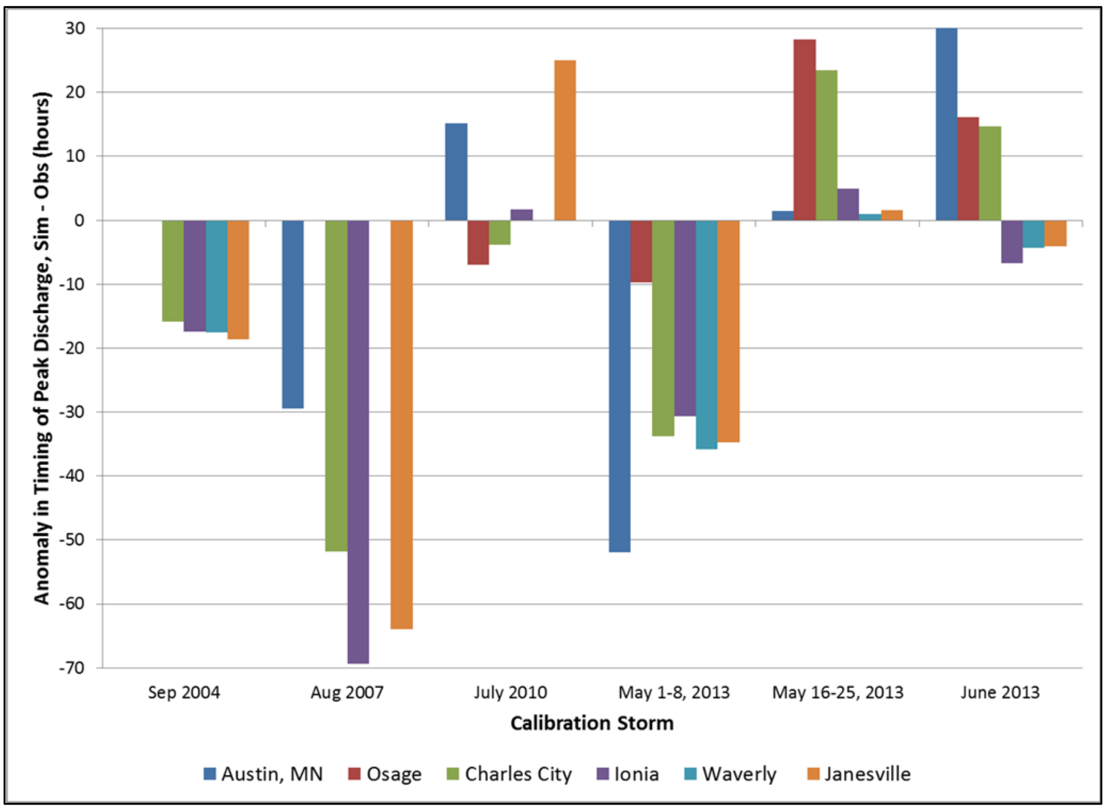

Figure 5.10. Anomalies in time to peak discharge between simulated and observed hydrographs at the operational USGS stage/discharge gage locations in the Upper Cedar River Watershed for the six calibration storms. A positive anomaly indicates the simulated time to peak was later than observed (and vice versa).

Figure 5.11 compares the simulated and observed runoff depths (runoff volume normalized by upstream drainage area) at the operational USGS stage/discharge gage locations for the six calibration storms. The simulated and observed runoff volumes at a particular location were only plotted if the observed discharge time series was complete. Except for the August 2007 and May 16-25, 2013 storms, the HMS model does a reasonable job correctly simulating runoff volumes throughout the basin. Simulated runoff volumes are best for the June 2013 storm and worst for the August 2007 storm. This general trend is observed for estimating peak discharge magnitude and time to peak as well. It is important to consider all three quantities – peak discharge magnitude, time to peak, and total runoff volume – when evaluating model performance. For example, while the simulated peak discharges for the September 2004 storm are reasonable, simulated times to peak are consistently early by over 15 hours and simulated runoff volumes are generally underestimated. Finally, while the HMS model is expected to perform better for higher runoff events where surface flow dominates the partition of rainfall, Figure 5.11 also reveals the HMS model can reasonably simulate runoff volumes for smaller events as well under certain conditions. Simulated runoff volumes are best for the May 1-8, 2013 and June 2013 storms, both of which had wetter than normal AMC. This reveals that in addition to larger magnitude storms, the HMS model will also perform better when a wetter initial condition is expected.

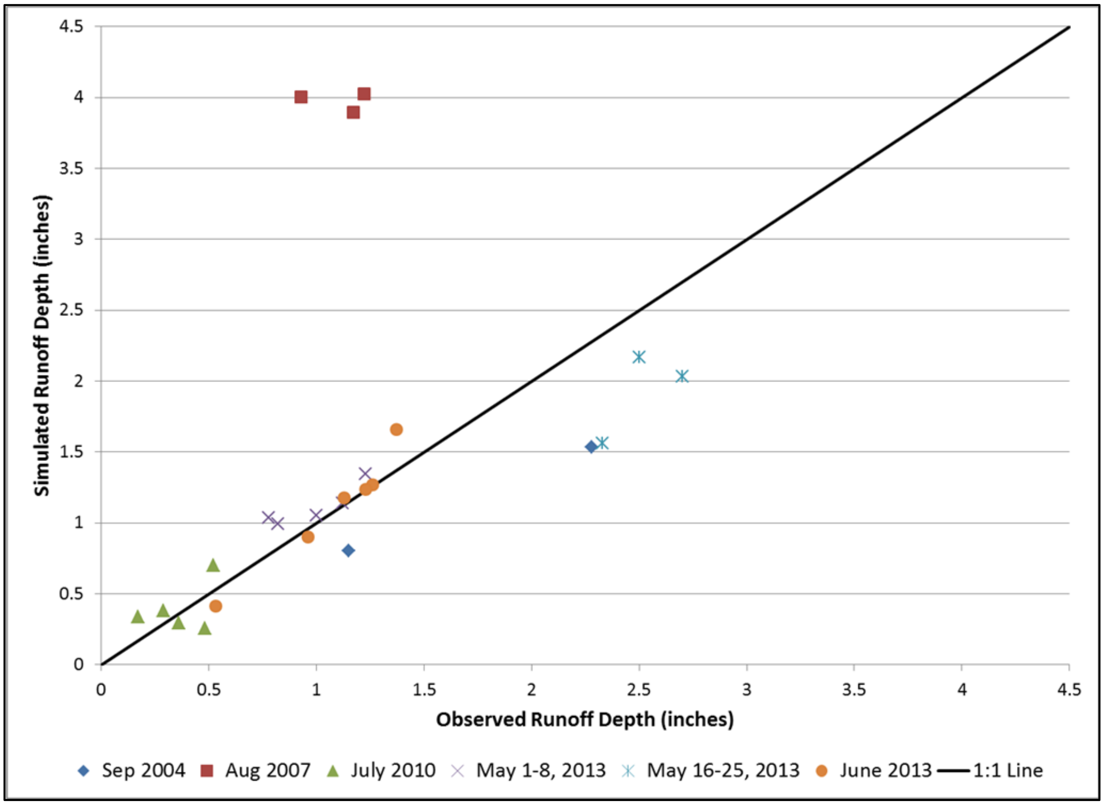

Figure 5.11. Comparison of simulated and observed runoff depths at operational USGS stage/discharge gage locations in the Upper Cedar River Watershed for the six calibration storms.

5.6 Chapter Summary

This chapter describes the calibration of the Upper Cedar River Watershed HMS model. Calibration refers to the process of taking an initial set of model parameters and adjusting them so the simulated results produced by the model reflect what was actually observed. The HMS model was calibrated to six historical storms occurring between September 2004 and June 2013 by comparing simulated hydrographs to observations at the six USGS stage/discharge gages in the watershed. To improve model performance, existing NRCS definitions for antecedent moisture conditions were modified to develop a continuous relationship that predicts the Curve Number adjustment that should be applied at the beginning of a simulation using the antecedent precipitation index. This is an improvement over the traditional NRCS methodology that provides only three discrete classes for CN classification. The time storage coefficient (R) of the Clark Unit Hydrograph method was also modified to account for tile drainage hypothesized to delay the hydrologic response downstream of tiled areas.

HMS model performance varied for the six calibration storms considered. Model performance was evaluated by comparing simulated and observed peak discharges, times to peak, and runoff volumes at the operational USGS stage/discharge gages throughout the basin. As expected, the model performed better for larger runoff events or when wetter than normal initial conditions were expected. In each case, either a greater amount of rain (high runoff events) or proportion of rain (near saturated initial conditions) is partitioned into surface runoff. In cases where the model did not perform well (e.g. August 2007), possible reasons for error include the size of the storm event considered (smaller storms are likely to have a greater subsurface flow component), not accounting for evapotranspiration during the growing season, and the presence of karst geologic features in the watershed (sinkholes, shallow carbonate bedrock) that were not directly accounted for in the HMS model. Following calibration, the HMS model of the Upper Cedar River Watershed was validated to several historical storms, which is described in the next chapter.

CHAPTER 6: MODEL VALIDATION

6.1 Introduction

For model validation, the intent is to use the model parameters developed during calibration to simulate other events and evaluate how well the model is 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. Four historical storms were considered for model validation. Results for these storms are presented and discussed.

6.2 Validation Storm Results

6.2.1 May 2004

The May 20-28, 2004 validation storm was characterized by a basin average Stage IV radar rainfall total of 6.17 inches and an observed runoff coefficient of 0.29 and peak discharge of 22,600 cfs at Janesville. Wetter than normal conditions were present before the storm (70th percentile of API), so uncalibrated subbasin CNs were increased by 2.6% according to Figure 5.2. Despite wetter than normal conditions, only a small fraction of rain was converted to runoff. As a result, simulated runoff volumes and peak flows are significantly overestimated in the model (overestimation of runoff volume and peak flow at Janesville by 55% and 118%, respectively). The simulated runoff coefficient at Janesville was 0.63, more than double the observed runoff coefficient.

Overestimations in runoff volume and the magnitude of peak flows may be partially attributed to the radar rainfall estimates being approximately 8% greater than the rain gage estimates. However, overestimation of runoff volume is primarily due to an inaccurate prediction of initial soil moisture conditions using the API. Simulated and observed hydrographs at the USGS stage/discharge gages operational during this time period are shown in Figure 6.1.

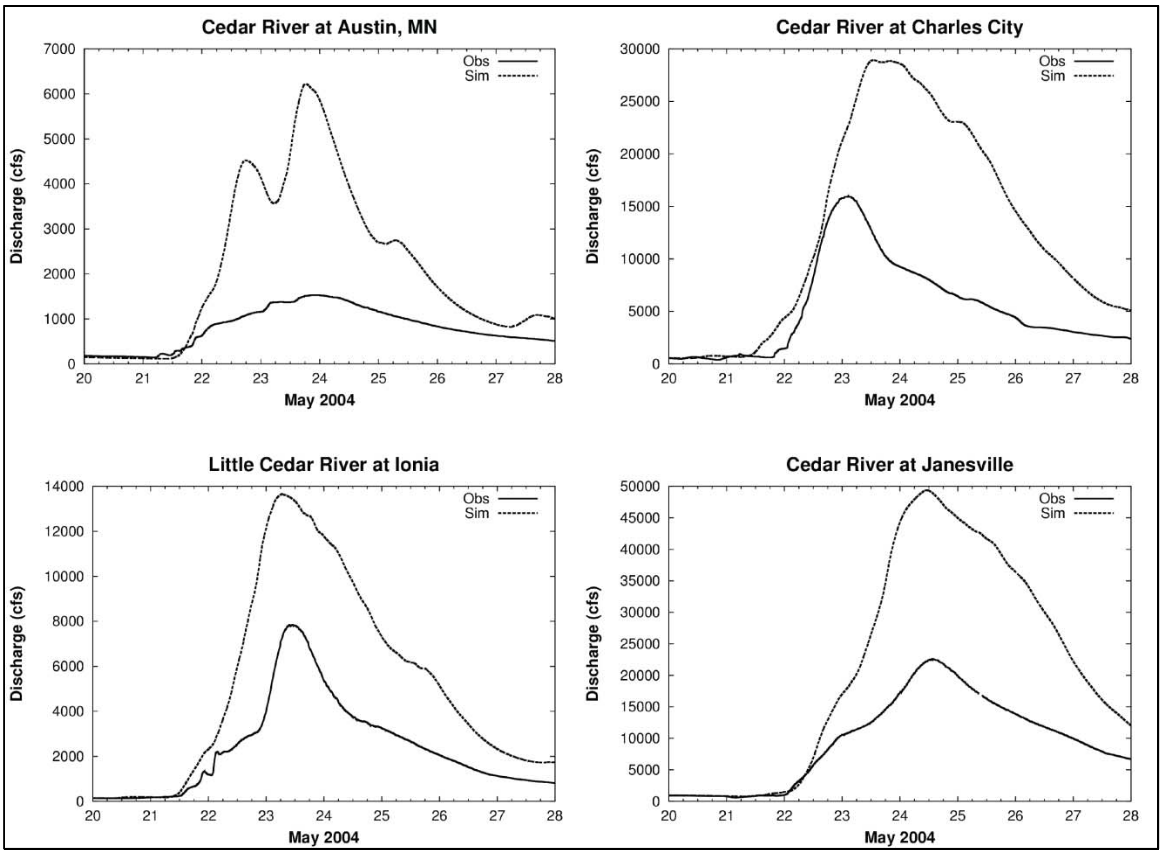

Figure 6.1. Hydrograph comparisons for the May 2004 validation storm.

6.2.2 September 2010

The September 22 – October 1, 2010 validation storm was characterized by a basin average Stage IV radar rainfall total of 3.14 inches and an observed runoff coefficient of 0.44 and peak discharge of 15,300 cfs at Janesville. Near normal soil moisture conditions were present before the storm (51st percentile of API), so uncalibrated subbasin CNs were decreased by 4.7%. Although the model did not perform as poorly for this event as the May 2004 storm, simulated runoff volumes and peak flows are still overestimated (runoff volume and peak flow at Janesville overestimated by 11% and 69%, respectively). The simulated runoff coefficient at Janesville was 0.49. Simulated and observed hydrographs at the USGS stage/discharge gages operational during this time period are shown in Figure 6.2.

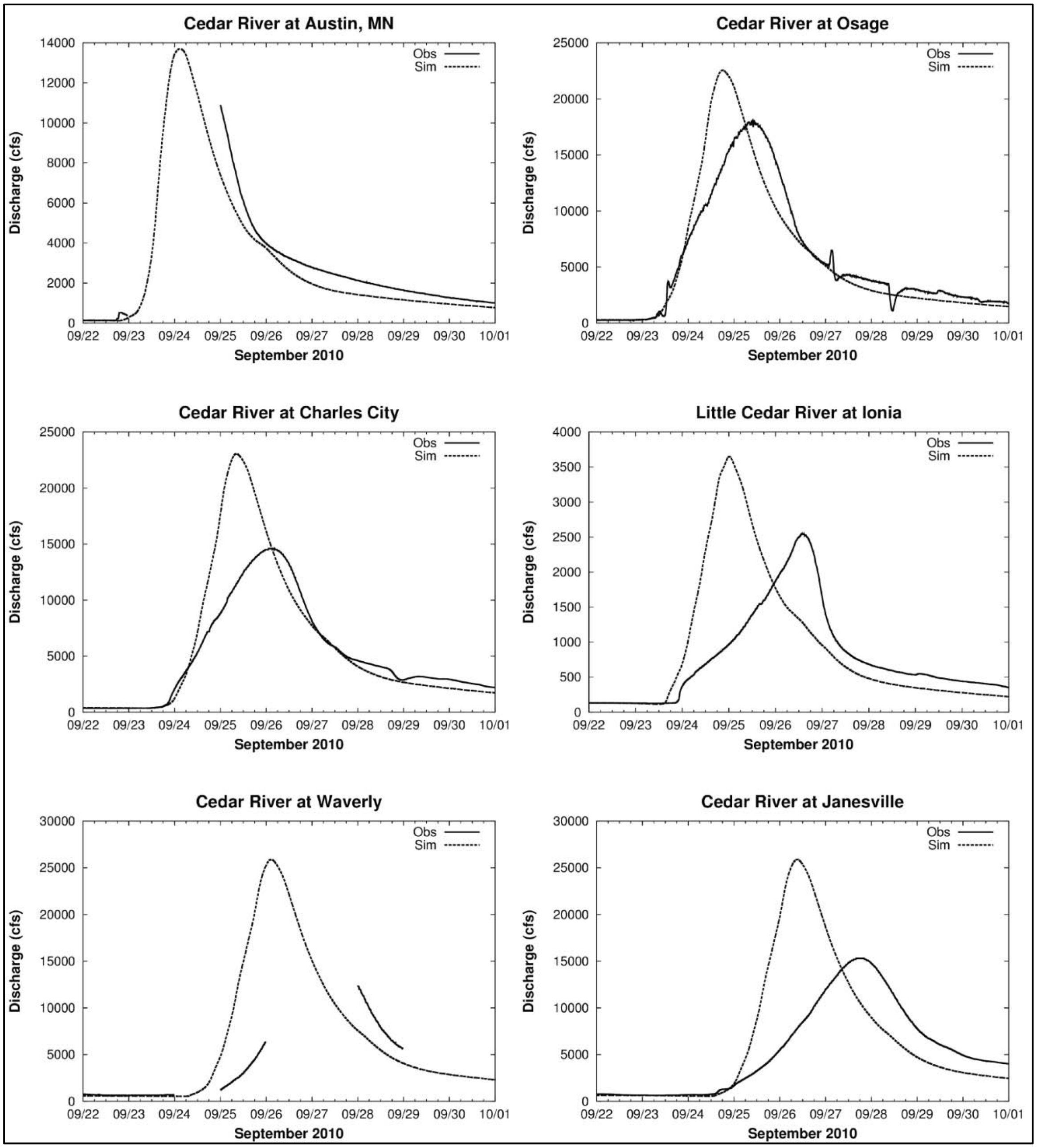

Figure 6.2. Hydrograph comparisons for the September 2010 validation storm.

6.2.3 July 2011

The July 14-22, 2011 validation storm was characterized by a basin average Stage IV radar rainfall total of 2.76 inches and an observed runoff coefficient of 0.23 and peak discharge of 7,790 cfs at Janesville. Normal soil moisture conditions were present before the storm (50th percentile of API), so uncalibrated subbasin CNs were reduced by 4.7%. Once again, the modeled response was substantially overestimated (overestimation of runoff volume and peak flow at Janesville by 52% and 126%, respectively). The simulated runoff coefficient at Janesville was 0.35.

Runoff volume overestimation by the HMS model is likely due to similar reasons discussed previously. Evapotranspiration losses during the growing season may be considerable; the small amount of runoff generated (the peak flow of 7790 cfs at Janesville corresponds to less than the two-year return period streamflow) suggests this event may have been influenced by a greater subsurface flow component. Interestingly, the API is greater a couple days before and after the start of the simulation (7/14/2011), which would lead to an even greater overestimation of runoff volume. Simulated and observed hydrographs at the USGS stage/discharge gages operational during this time period are shown in Figure 6.3.

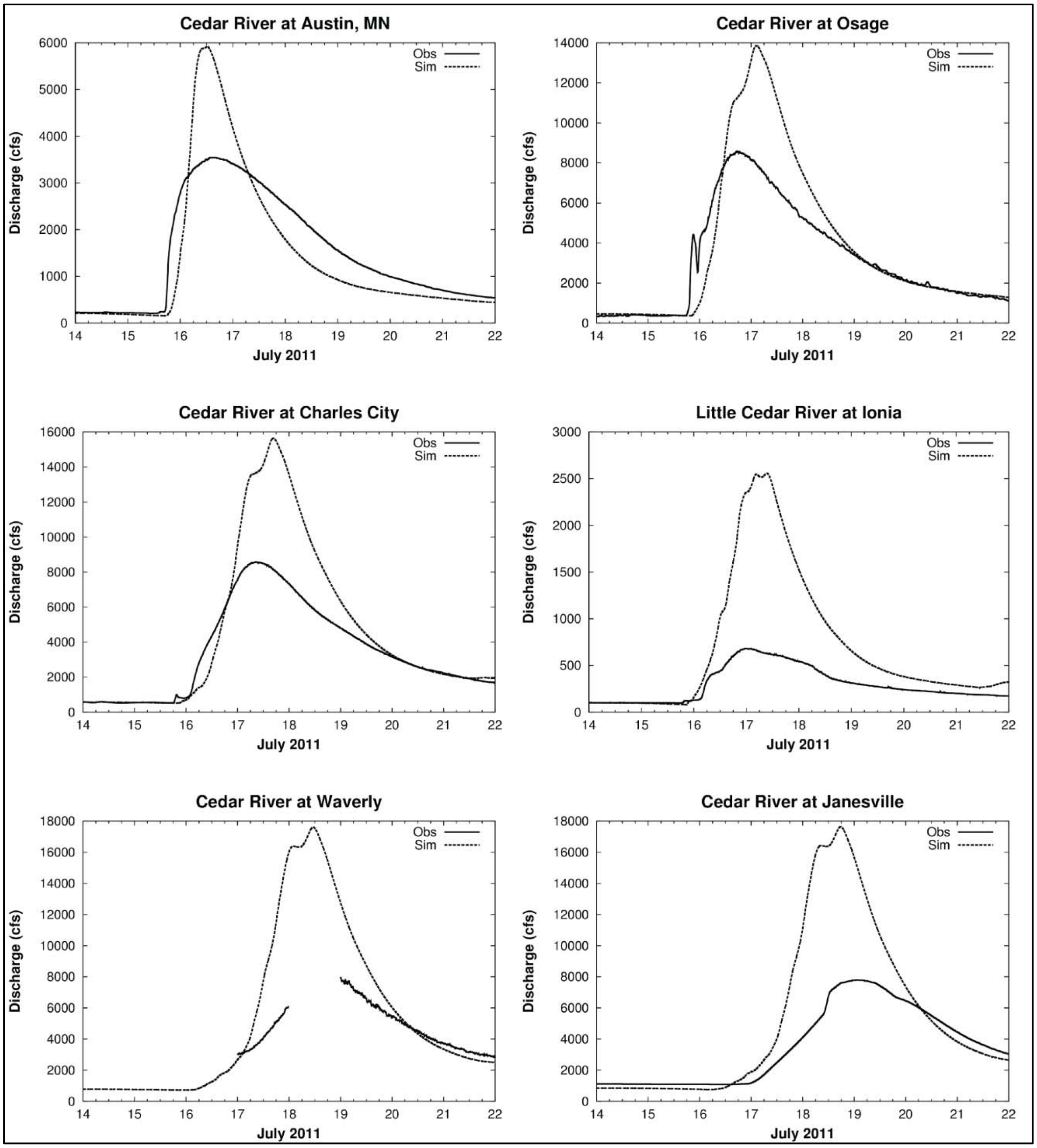

Figure 6.3. Hydrograph comparisons for the July 2011 validation storm.

6.2.4 June 2008

Best validation results were observed for the June 7-17, 2008 flood event that produced a record discharge of 53,400 cfs at Janesville. The ground was nearly saturated before the flood (89th percentile of API), so uncalibrated subbasin CNs were increased by

9.9%. Although many of the USGS discharge records during the flood are unavailable, reasonable agreement is observed between the HMS simulation and the available USGS discharge estimates/measurements available. The simulated peak discharge at Janesville is overestimated by only 6% and the timing is within five hours of the measured peak.

Overestimation of the simulated response is partially explained by the radar rainfall estimates being greater (8%) than the rain gage estimates. Overall, good model performance was achieved for the June 2008 flood because of the saturated soil conditions prior to the start of the storm. Most of the ensuing rain was converted to surface runoff because of the landscape's diminished infiltration capacity. Simulated and observed hydrographs at the USGS stage/discharge gages operational during this time period are shown in Figure 6.4.

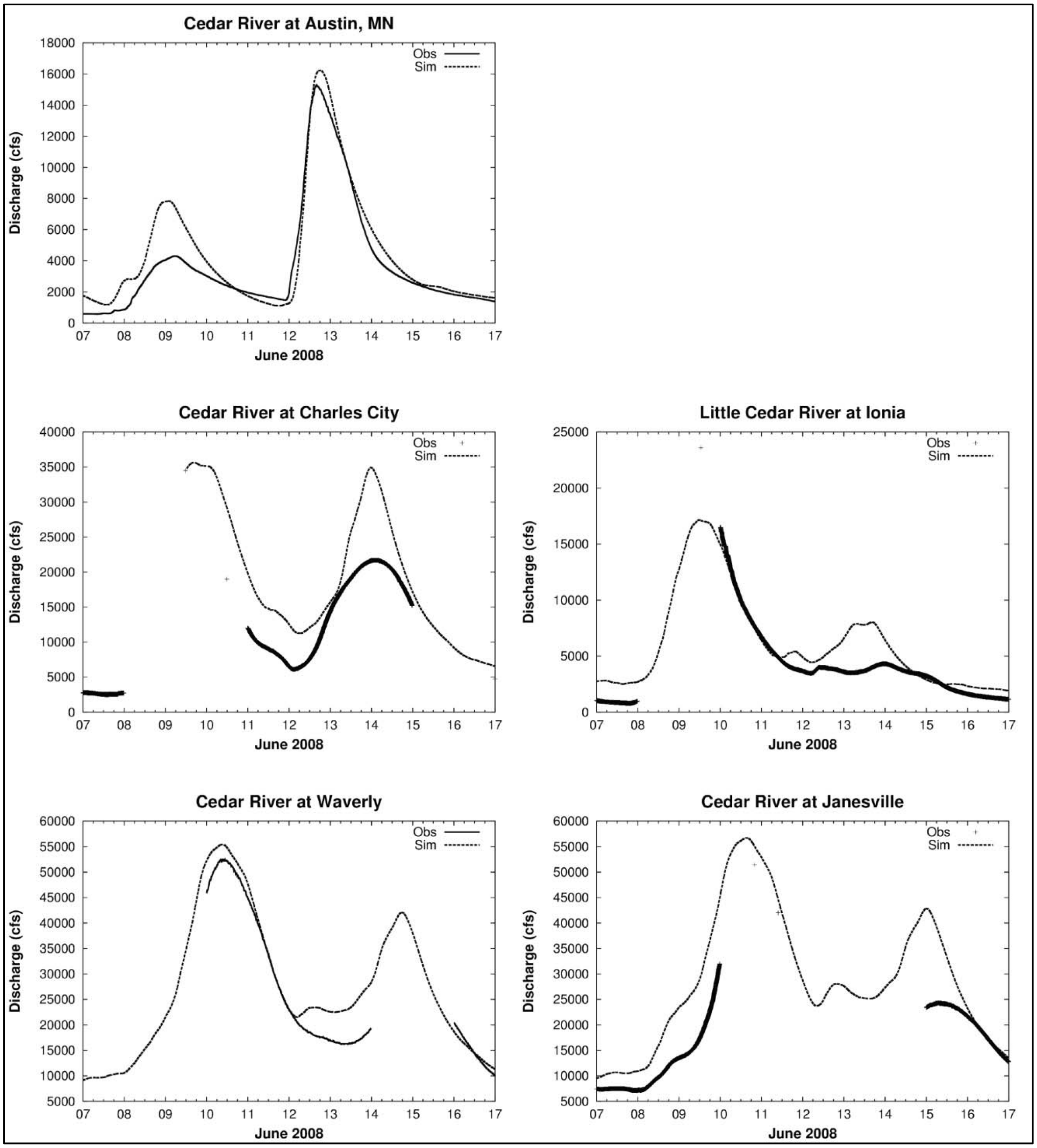

Figure 6.4. Hydrograph comparisons for the June 2008 validation storm.

6.3 Summary of HMS Model Performance for Validation Storms

As with calibration, HMS model performance for the four validation storms was evaluated by comparing how well simulated peak discharges, times to peak, and runoff volumes matched observations at operational USGS stage/discharge gages in the basin. Figure 6.5 compares the simulated and observed peak discharges at the operational USGS stage/discharge gage locations for all four validation storms. The HMS simulated peak discharges consistently overestimate the observed peak discharges except for the June 2008 flood. Once again, the model tends to perform better, at least in terms of peak discharge prediction, for larger events.

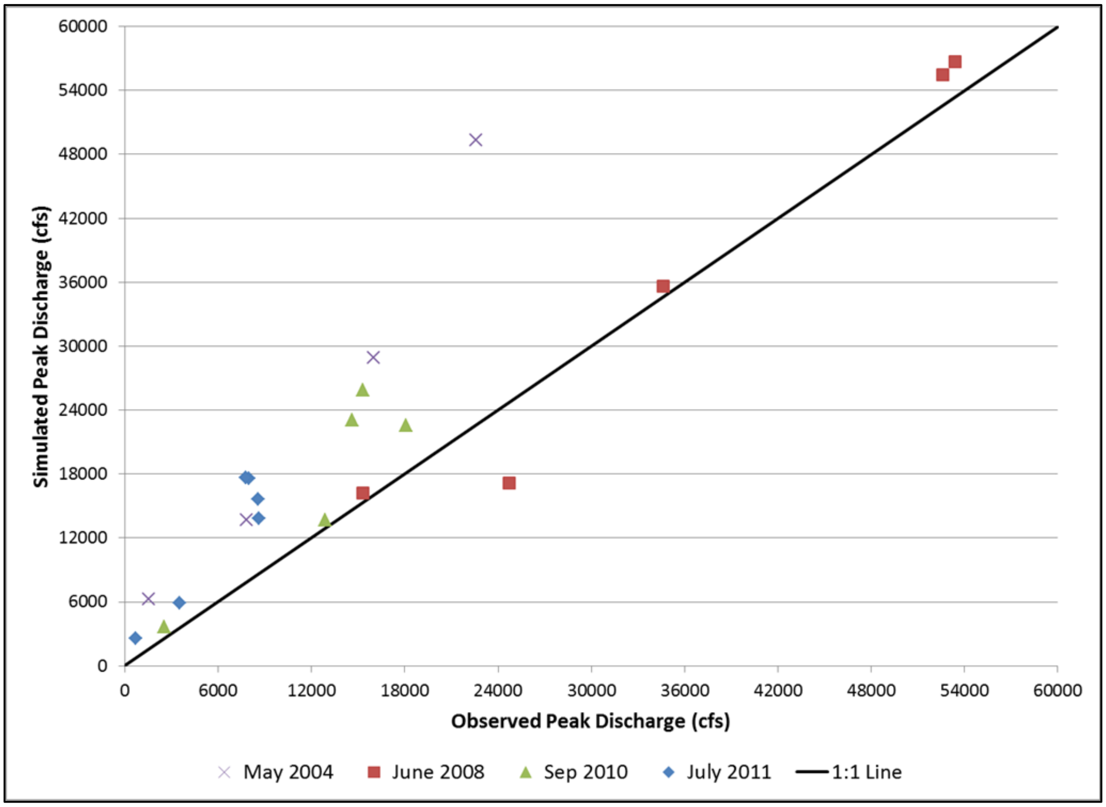

Figure 6.5. Comparison of simulated and observed peak discharges at operational USGS stage/discharge gage locations in the Upper Cedar River Watershed for the four validation storms.

Figure 6.6 plots the anomalies in the time to peak discharge between the simulated and observed hydrographs at each operational USGS stage/discharge gage for the four validation storms. The predicted times to peak for the May 2004 and July 2011 storms are reasonable, but as shown in Figure 6.5, peak discharges are substantially overestimated. Evident from all events except the September 2010 storm, the times to peak tend to improve moving downstream. Because further downstream locations have a greater upstream area, a greater amount of basin averaging takes place in which areas predicting the peak discharge too early are balanced by areas where the time to peak is predicted too late. As a result, times to peak may be inaccurate at upstream locations but reasonable agreement between simulated and observed times to peak can still be achieved at downstream locations near the basin outlet.

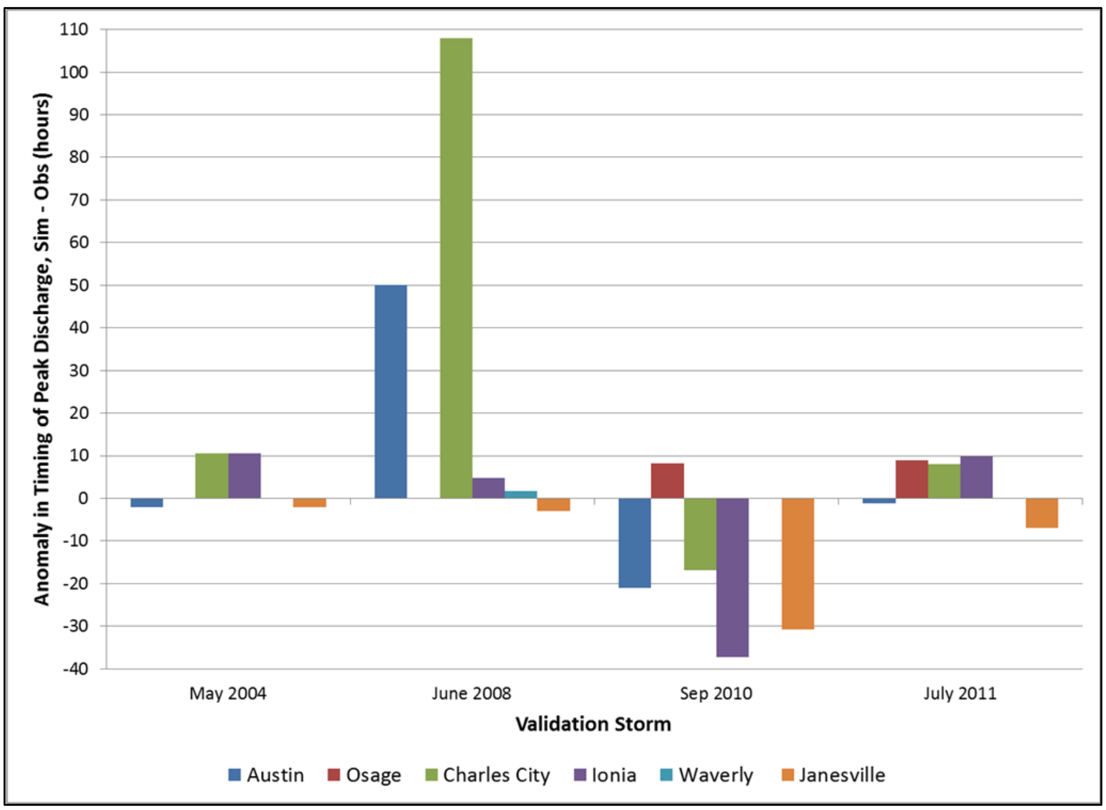

Figure 6.6. Anomalies in time to peak discharge between simulated and observed hydrographs at the operational USGS stage/discharge gage locations in the Upper Cedar River Watershed for the four validation storms. A positive anomaly indicates the simulated time to peak was later than observed (and vice versa).

Figure 6.7 compares the simulated and observed runoff depths at the operational USGS stage/discharge gage locations for the four validation storms. The simulated and observed runoff volumes at a particular location were only plotted if the observed discharge time series was complete. In a similar manner as for peak discharge prediction, simulated runoff volumes consistently overestimate observations. Predicted runoff volumes were worst for the May 2004 event. Despite substantial rainfall and wetter than normal initial conditions, much less runoff was generated than expected. Runoff prediction for the June 2008 event appears reasonable, but only one complete discharge time series was available for comparison.

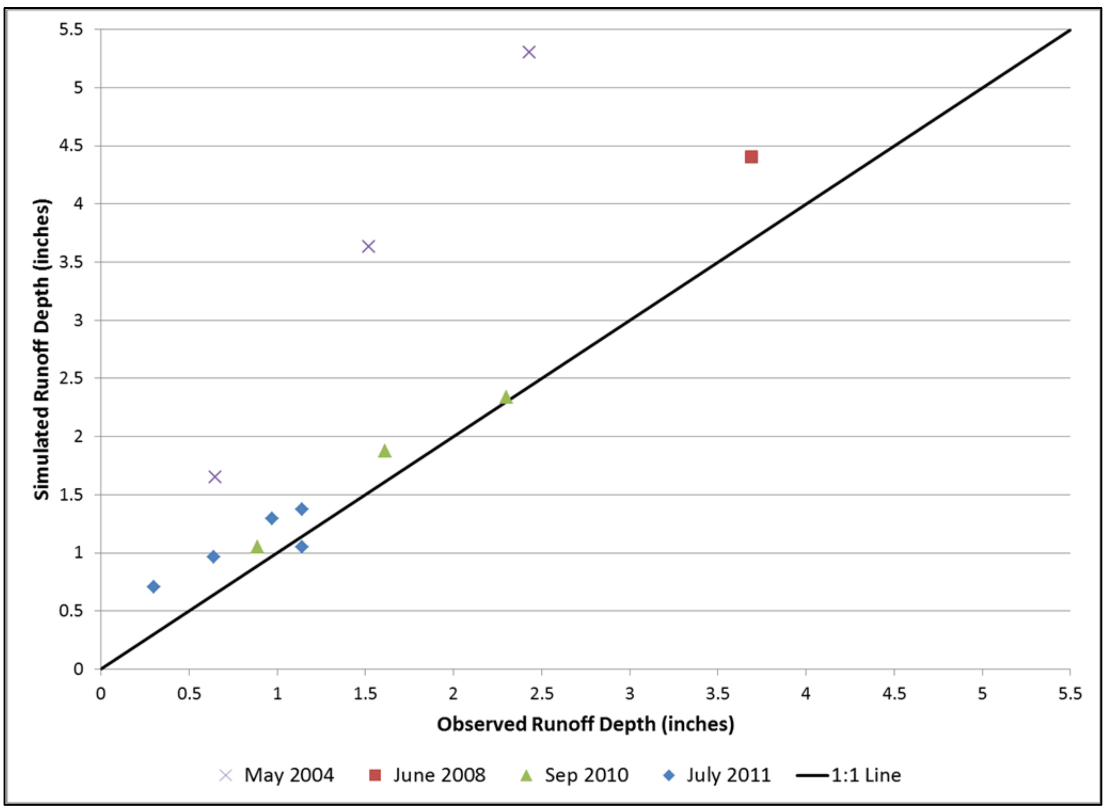

Figure 6.7. Comparison of simulated and observed runoff depths at operational USGS stage/discharge gage locations in the Upper Cedar River Watershed for the four validation storms.

6.4 Chapter Summary

Using the model parameters determined from calibration, the Upper Cedar River Watershed HMS model was validated to four historical storm events. Validation is performed without adjusting parameters beforehand to assess the predictive capability of the model.

As with calibration, the HMS model validation results are not perfect. The HMS simulated results consistently overestimate the USGS discharge observations both in magnitude of the peak flow and total runoff volume. While this likely reflects the model parameters responsible for runoff, namely the subbasin CNs, are overestimated, other possible reasons for error are similar to those discussed for calibration – having to select smaller storm events that yielded less runoff, not accounting for evapotranspiration losses during the growing season, and differences in the radar rainfall and rain gage estimates. However, the HMS model did acceptable simulating the June 2008 flood that produced a record discharge of 53,400 cfs at Janesville, reiterating the concept that the model does a better job simulating surface flow-dominated 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 flooding threat.

The Upper Cedar HMS model has several strengths, weaknesses, and assumptions that should be reiterated. First, the Upper Cedar HMS model is a surface water-only model, so subsurface and groundwater flow components were not accounted for explicitly. Baseflow was represented by a first order exponential decay relationship, which represented the aggregated effects of all subsurface flow contributions (interflow and groundwater flow). While the karst subsurface is expected to increase the baseflow contribution in the basin, no significant changes to baseflow parameters needed to be made to reflect this condition for the historical storms selected for calibration and validation. Additionally, the HMS model is only applicable for estimating the watershed response to storm events occurring between May and September. While flooding is common at other times of the year as well, particularly in March and April, this time period was not considered during calibration for several reasons. Reasons include project goals (constructed projects are likely to perform better during the late spring to summer months), flood seasonality (the largest floods have occurred sporadically in the summer months), and model limitations (snowmelt was not considered in the model). Finally, the HMS model performs best when surface runoff is expected to dominate the partitioning of rainfall. This typically occurs for larger storm events when a greater overall amount of precipitation is converted to runoff or for near saturated initial conditions. This observation is supported by the fact the model performed well for the June 2013 and June 2008 events. For the June 2013 event, less than 2.5 inches of rain fell across the basin on average, but wet initial conditions resulted in more than 50% of the rain being converted to runoff. The model performed well for the June 2008 event because a large amount of rain fell across the basin (over eight inches on average) and because near saturated initial conditions existed.

Keeping in mind the limitations, strengths, and weaknesses of the HMS model, hypothetical watershed improvement analyses may now be performed comparing differences between a hypothetical flood mitigation scenario and a baseline scenario reflecting existing watershed conditions determined from calibration and validation.

CHAPTER 7: HIGH RUNOFF POTENTIAL AREAS

7.1 Introduction

The HEC-HMS model of the Upper Cedar River watershed was used to identify areas in the watershed with high runoff potential. Identifying areas of the watershed with higher runoff potential is the first step in selecting project sites for flood mitigation. High runoff areas offer the greatest opportunity for retaining more water from large rainstorms on the landscape and reducing downstream flood peaks.

7.2 Method

In the HMS model of the Upper Cedar River Watershed, the runoff potential for each subbasin is defined by the NRCS 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 was simulated with the HMS model for the same hypothetical rainstorm. A rainstorm with a total accumulation of 5.05 inches in 24 hours was selected for this analysis (25-year average return period design storm). The calibrated subbasin CNs were used and normal soil moisture conditions were assumed (AMC II).

7.3 Results

Figure 7.1 shows the runoff coefficient as a percentage (from 0% for no runoff to 100% 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 subbasin areas — namely, hydrologic units defined by the USGS. The smallest hydrologic units, known as HUC 12 watersheds, are also shown in Figure 7.1. Area-weighted average runoff coefficients were determined for each of the 47 HUC 12 watersheds in the Upper Cedar basin. Areas in the Upper Cedar with the highest runoff potential are primarily located in Mitchell, Worth, and parts of Floyd counties. For the 25-year average return period design storm, runoff coefficients around 50% are common. Agricultural land use dominates these counties (and the entire watershed in general). However, these areas have moderately to poorly drained soils, which are characteristic of the Iowan Surface and Des Moines Lobe geographic landforms. 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 three counties in Iowa with the highest runoff areas have very flat terrain; the average subbasin slopes are at or below the basin average (5.3%). Flat terrain would make the siting of flood mitigation ponds more challenging. Of course, 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 places with repetitive loss of crops or roads/road structures is also valuable in selecting locations.

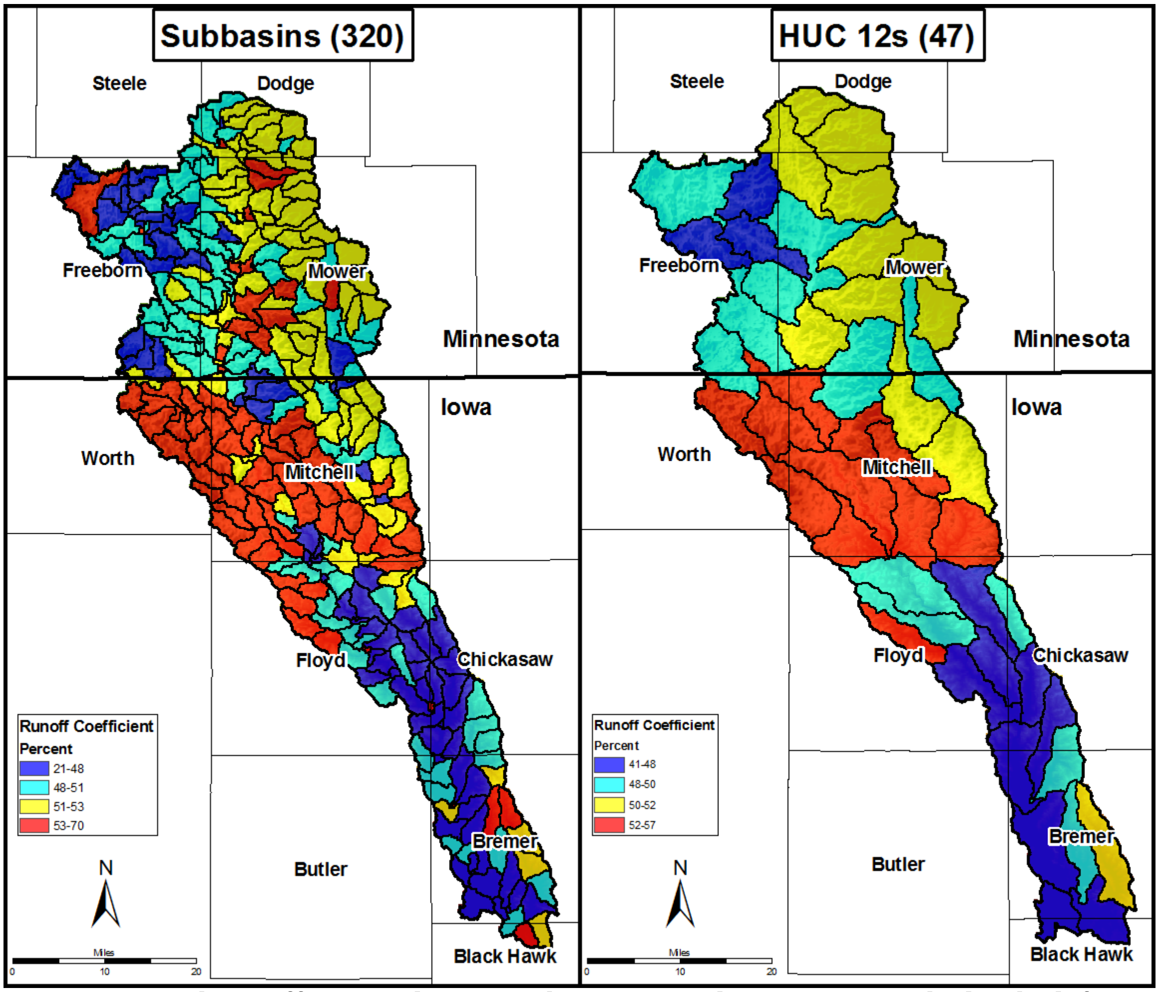

Figure 7.1. High runoff potential areas in the Upper Cedar River Watershed. The left figure shows the runoff coefficient for each subbasin for the 25 year – 24 hour storm (5.05 inches of rain) and the right figure shows the aggregated runoff coefficient calculated for each HUC 12 watershed. Higher runoff coefficients are shown in red.

While runoff coefficient values will vary depending on the storm magnitude (higher runoff coefficients are expected for larger storms assuming the same antecedent moisture conditions), it is important to recognize the relative ranking of high runoff areas will not change when a uniform depth of rainfall is applied everywhere. This is because the runoff potential for each subbasin is defined strictly by the CN. Therefore, Figure 7.1 can also be viewed as a map showing the relative ranking of CN in the watershed. This is confirmed in Figure 7.2, which shows the calibrated AMC II CNs for the subbasins and the aggregated values for the HUC 12 watersheds. As expected, Figures 7.1 and 7.2 look identical since the color coding for each is based on the same percentile rankings.

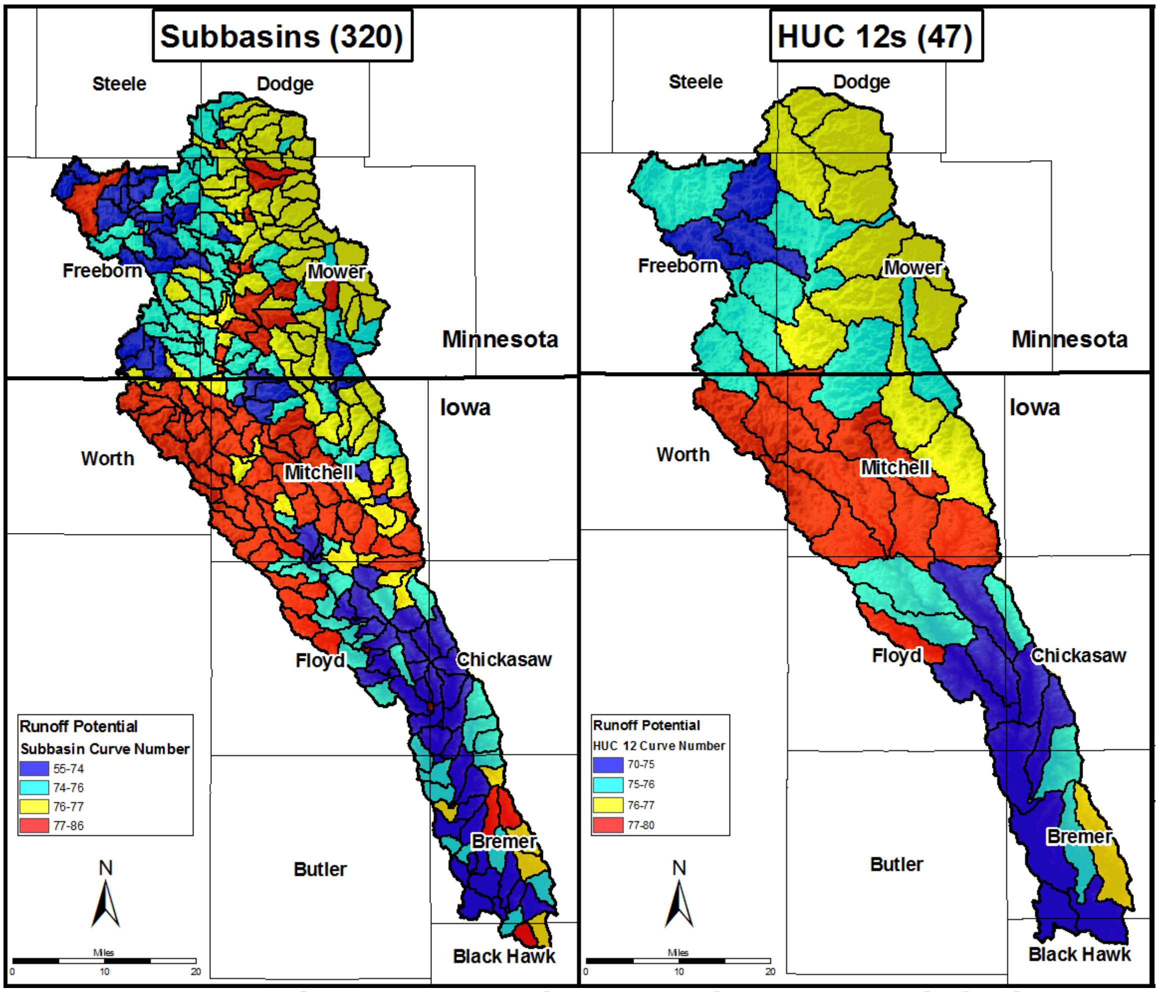

Figure 7.2. Curve Number assignment in the Upper Cedar River Watershed. The AMC II CNs determined from calibration are shown for the subbasins and HUC 12 watersheds.

Showing areas of high runoff potential in both ways is useful. The runoff coefficients define the runoff potential in an absolute sense – how much rainfall is converted to runoff for a particular storm. The CNs define the runoff potential in a relative sense – the relative ranking of runoff potential among different areas – but do not explicitly reveal the runoff magnitude or severity. To summarize, while Figure 7.1

does indicate the highest runoff areas are expected in Mitchell, Worth, and parts of Floyd counties, Figure 7.2 reveals these areas do not pose a substantially higher runoff risk than other areas in the watershed. CNs between 74 and 77 are common throughout much of the watershed, reflecting the relatively equal distribution of land use and soils throughout the watershed dominated by row crop agriculture and B and C-Type soils.

7.4 Chapter Summary

Chapter 7 details the first application of the calibrated HMS model of the Upper Cedar River Watershed to identify areas of high runoff potential. Identifying areas of high runoff potential is the first step in selecting flood mitigation project sites as these areas offer the greatest opportunity for retaining more water on the landscape from large rain events to reduce downstream flood peaks.

To determine areas of high runoff potential in the watershed, a hypothetical rain storm applying 5.05 inches of rain in 24 hours (25-year average return period design rain storm) was run in HMS and the runoff coefficient was computed for each subbasin. For this particular storm, runoff coefficients around 50% were common and the highest runoff areas were in the central part of the watershed (Mitchell, Worth, and Floyd counties). However, the runoff potential of these areas was not substantially greater than other parts of the watershed, reflecting the agricultural land use and moderately to poorly drained soils that dominate the watershed.

While the results of this analysis provide initial recommendations on where to focus flood mitigation efforts, other factors must be considered in selecting potential project locations. Other considerations include knowledge of the site criteria required for certain types of flood mitigation projects, landowner willingness, and awareness of existing conservation practices that may already be in place in potential project areas.

CHAPTER 8: HYPOTHETICAL INCREASED INFILTRATION WITHIN THE WATERSHED – LAND USE CHANGES

8.1 Introduction

Reducing runoff from areas with high runoff potential may be accomplished by increasing how much rainfall 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. Even if the infiltrated water reaches a stream, it arrives much later (long after the storm ends). Also, its late arrival keeps rivers running during long periods without rain.

In this chapter, runoff reduction resulting from increased infiltration is examined through land use changes. Two hypothetical scenarios are considered. The first is the conversion of row crop agriculture back to native tall-grass prairie; the second is the improvement of existing agricultural land by planting cover crops. The first hypothetical example – conversion of row crop agriculture back to native-tall grass prairie – is meant to represent a natural landscape condition to provide an indication of what the flood hydrology was possibly like in the watershed historically. This example is not a project proposal but does provide a valuable benchmark on the limits of flood reduction that are physically possible with runoff reduction. The second hypothetical example – improved agricultural conditions due to planting cover crops – is meant to illustrate the impacts planting cover crops during the dormant season (after the harvest of row crops) could have on reducing flood peaks during the growing season (after the planting of row crops).

8.2 Method

An analysis was performed to quantify the impact of human-induced land use changes on the flood hydrology of the Upper Cedar River Watershed. In the first example, all current agricultural land use is converted to native tall-grass prairie with its much higher infiltration characteristics. Some evidence suggests 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 soils instead of rapidly traveling to a nearby stream as surface flow. Once in the soils, much of the water was actually taken up by the root systems of the prairie grasses. Obviously, returning to this pre-settlement condition is unlikely to occur. Still, this scenario is an important benchmark to compare with any watershed improvement project considered.

In the second example, all existing agricultural land use is improved to a lower runoff condition to represent planting cover crops during the dormant season. This example does not represent the replacement of row crops with cover crops, but rather the slightly improved soil infiltration characteristics expected for existing agricultural land (constituting row crops primarily) during the growing season that might result from planting cover crops in the fall following harvest of row crops.

Cover crops have been around for some time but are becoming more widely used as a farming conservation practice. Cover crops are typically planted following the harvest of either corn or soybeans and “cover” the ground through winter until the next growing season begins. The cover crop can be killed off in the spring by rolling it or grazing it with livestock; following, row crops can be planted directly into the remaining cover crop residue. Cover crops provide a variety of benefits including improved soil quality and fertility, increased organic matter, increased infiltration and percolation, reduced soil compaction, and reduced erosion and soil loss. They also retain soil moisture and enhance biodiversity (Mutch, 2010). One source suggests that for every one percent increase in soil organic matter that can result over time from planting cover crops, the soil retains an additional 17,000-25,000 gallons of water per acre per year (Archuleta, 2014). Examples of cover crops include clovers, annual and cereal ryegrasses, winter wheat, and oilseed radish (Mutch, 2010). Once again, the purpose of this hypothetical example is to illustrate the impact planting cover crops during the dormant season could have on improving soil quality and infiltration during the growing season when row crops are planted.

To simulate both land use change scenarios with the HMS model, the model parameters affecting runoff potential across the landscape were adjusted to reflect either the tall-grass prairie or cover crop condition. Specifically, existing agricultural land use, which accounts for 77% of the watershed area, was redefined as either tall-grass prairie or an improved agricultural condition representing the impacts of cover crops. Figure 8.1 shows the agricultural areas (orange) in the watershed compared to all other land uses (red).

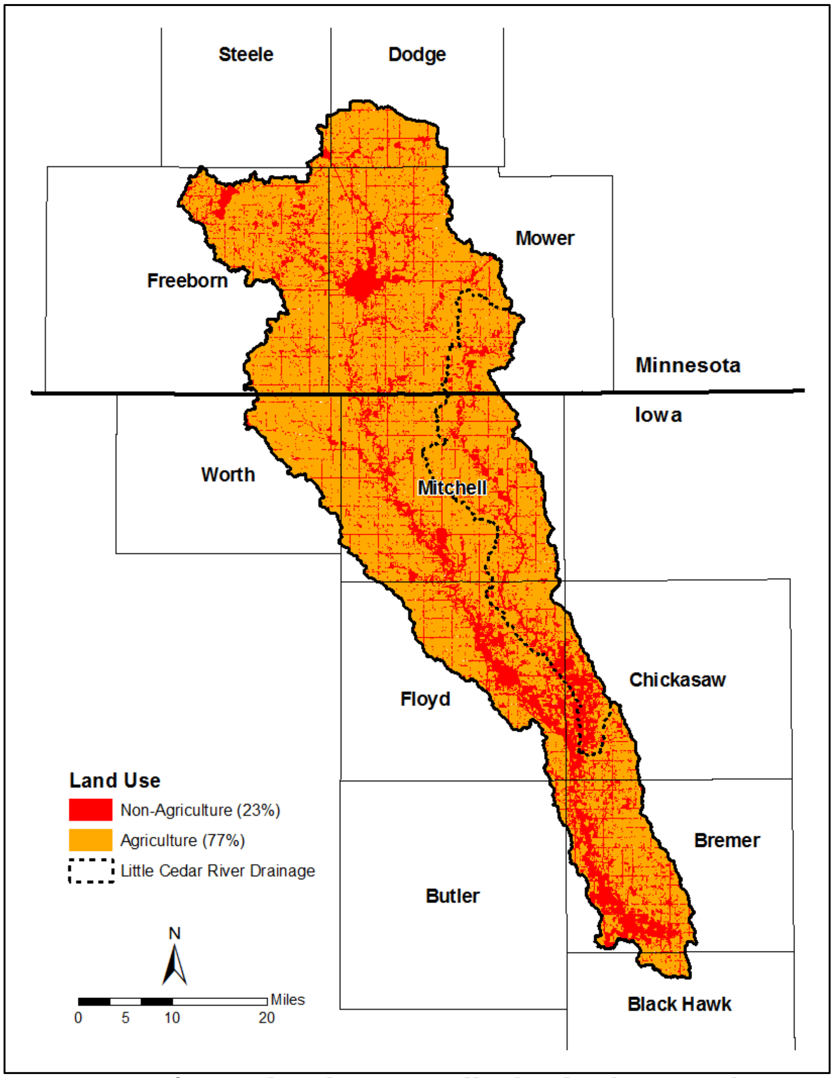

Figure 8.1. Comparison of agricultural areas to all other land uses in the Upper Cedar River Watershed. All agricultural areas (orange) were converted to prairie for this hypothetical scenario to assess the impacts of increased infiltration.

New NRCS CNs reflecting the lower runoff potential of prairie or improved soil conditions due to cover crops were assigned to each subbasin. The CNs used to define both land use changes are provided in Table 8.1. A new CN grid was generated in Arc GIS reflecting each land use change and area-weighted averaging was performed to assign new subbasin CNs. On average, subbasin CNs were reduced by 16% to reflect the native tall-grass prairie condition and 3.6% to reflect the improved row crop condition resulting from cover crops being planted during the dormant season.

Table 8.1. Curve Numbers used to define the tall-grass prairie and cover crop land use conditions.

	Hydrologic Soil Group			
Land Use	A	B	C	D
Row Crops	67	78	85	89
Tall-Grass Prairie	30	58	71	78
Row Crops After Planting Cover Crops	64	74	81	85

NOTE: Curve Number combinations derived from the *Urban Hydrology for Small Watersheds* (TR-55), Table 2-2, June 1986.

It is important to note that other parameters estimated from CNs, such as the water flow travel time through the subbasin, were not adjusted. Thus, this scenario (and all future scenarios considered where CNs were adjusted) only considers the reduction in runoff volume resulting from the hypothetical scenario and not the additional attenuation and delay in the timing of the peak discharge that would be expected due to a higher surface roughness.

Following new assignment of subbasin CNs, the model was run for a set of design storms. Normal antecedent moisture conditions (AMC II) were assumed. Comparisons were made between the current, baseline simulation and the hypothetical land use change scenario 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.

8.3 Results

8.3.1 Conversion of Row Crop Agriculture to Tall-Grass Prairie

As expected, converting 77% of the watershed from row crop agriculture to native tall-grass prairie has 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 infiltrates 0.8 inches more into the ground than the current agricultural landscape. The additional infiltration increases to 1.0 inch for a 25-year storm (5.05 inches of rain in 24 hours), 1.2 inches for a 50-year storm (5.89 inches of rain in 24 hours), and 1.3 inches for a 100-year storm (6.81 inches of rain in 24 hours). As a result of increased infiltration across the landscape, the river response is dampened.

Figure 8.2 shows several locations in the watershed that were selected as points of reference (index points) for comparing a particular watershed improvement scenario to current conditions. The six USGS stage/discharge gages and the outlet of Beaver Creek in Chickasaw County were selected as index locations. The Beaver Creek outlet was selected because Phase II of the Iowa Watersheds Project will construct projects in this HUC 12 watershed. Subbasins W2760 in northern Mower County, Minnesota and W3900 in eastern Worth County were also used for comparing watershed improvement scenarios to current conditions at the smaller, subbasin scale.

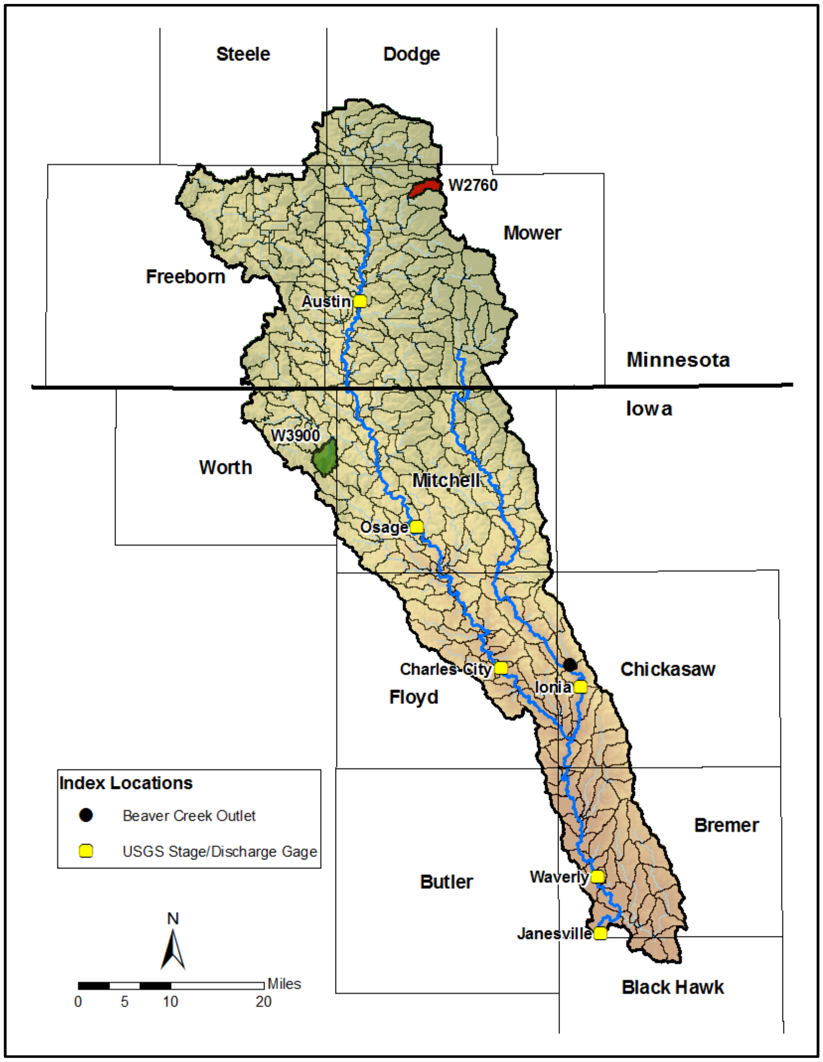

Figure 8.2. Index locations selected for comparing watershed improvement scenarios to current conditions. The six USGS stage/discharge gages and the outlet of Beaver Creek served as points of reference to compare scenario results to existing conditions. Subbasins W2760 and W3900 were also used to demonstrate the impact of a particular scenario at the subbasin scale.

Figure 8.3 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 (5.89 inches of rain in 24 hours). For all four locations shown – from an upstream subbasin area (panel a) to the outlet of the Upper Cedar River at Janesville – the river discharges and peak discharge rates are significantly less for a tall-grass prairie landscape. The smallest drainage area shown, Subbasin W3900

(7.6 square miles), along the eastern border of Worth County, currently has a large percentage of agricultural area (91%). About 1.2 additional inches of rainfall would infiltrate if this area were tall-grass prairie, resulting in a 33% reduction in its flood peak discharge. At downstream locations, the peak discharge reduction remains fairly uniform (30 to 40%), reflecting the relatively even distribution of agriculture throughout the watershed.

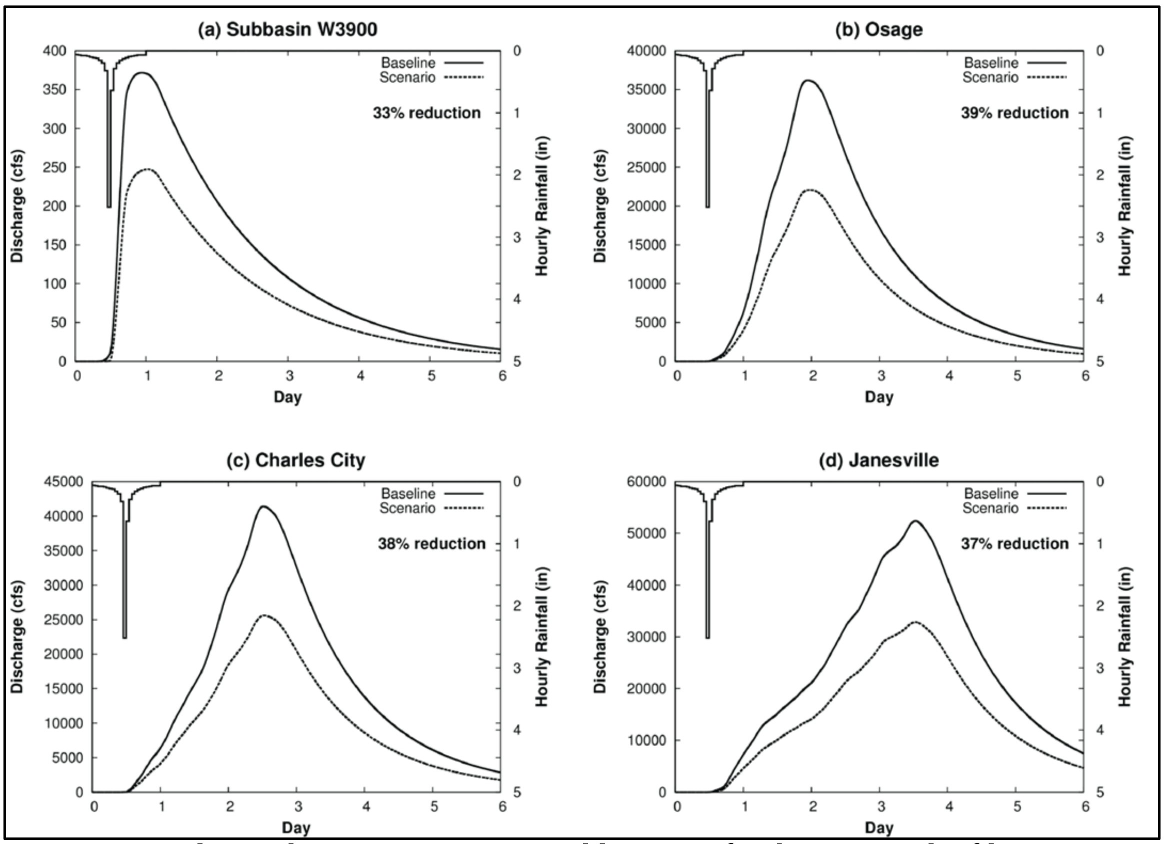

Figure 8.3. Hydrograph comparison at several locations for the increased infiltration scenario resulting from hypothetical land use changes (conversion of row crop agriculture to native prairie). Results shown are for the 50 year – 24 hour storm (5.89 inches of rain in 24 hours).

Figure 8.4 shows the percent reduction in peak discharge for each subbasin and the seven index locations resulting from a tall-grass prairie landscape for the 50-year, 24-hour design storm (5.89 inches of rain in 24 hours). Peak discharge reductions of 32-41% are common at the subbasin scale. Greatest subbasin peak discharge reductions are observed in the Minnesota portion of the watershed in parts of Freeborn, Steele, and Dodge counties. This results from these areas having a large percentage of agricultural area combined with poorly draining soils characteristic of the Des Moines Lobe region. As expected based on Figure 8.3, peak discharge reductions remain fairly uniform at the seven index locations throughout the watershed (36-39%), reflecting the relatively even distribution of agriculture throughout the watershed.

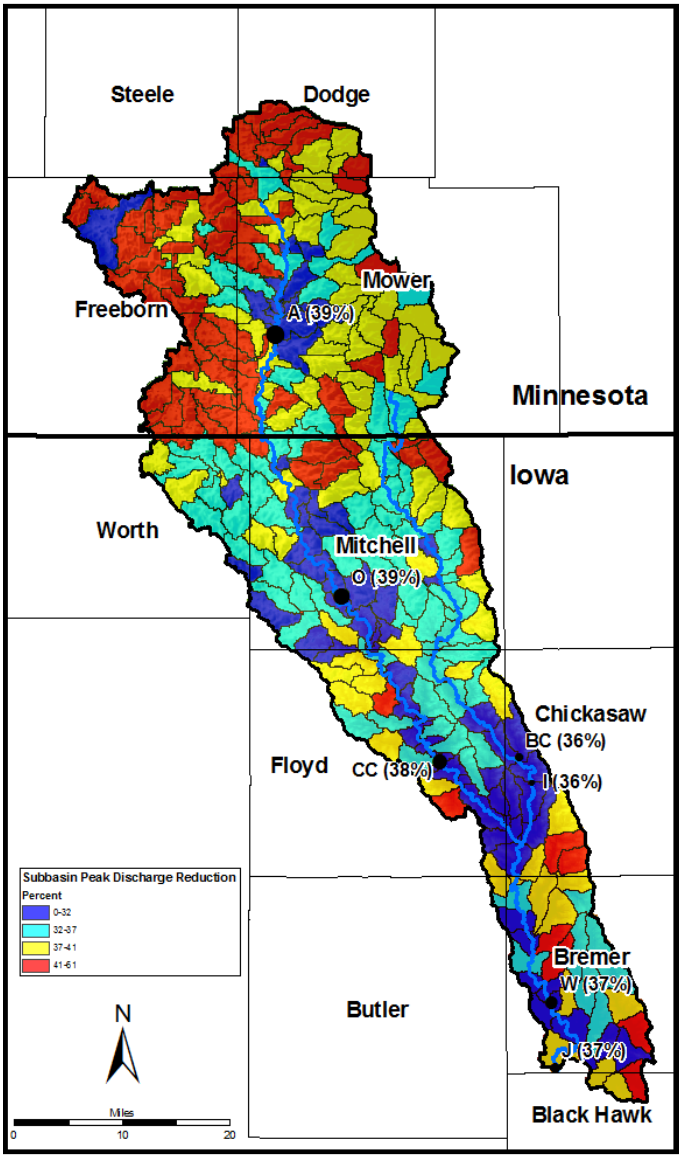

Figure 8.4. Subbasin peak discharge reductions resulting from the tall-grass prairie landscape for the 50-year, 24-hour design storm (5.89 inches of rain in 24 hours).

Figure 8.5 shows the percent reductions in peak discharge resulting from this hypothetical tall-grass prairie scenario at the seven index locations (the six USGS stream gages and the Beaver Creek Outlet) for the four design storms. The restoration of native tall-grass prairie typically results in peak discharge reductions of 30-50%. The peak reduction is largest for the smallest design storm (10-year return period), and decreases with larger rainfall amounts (up to the 100-year return period). In other words, the runoff reduction benefits of increased infiltration are greater for smaller rainfall events; still, for this tall-grass prairie scenario, there is a significant peak reduction benefit for large floods. Note also that the percent reduction in peak discharge is fairly uniform at all locations. Again, this outcome reflects the relatively equal distribution of agricultural land throughout the watershed.

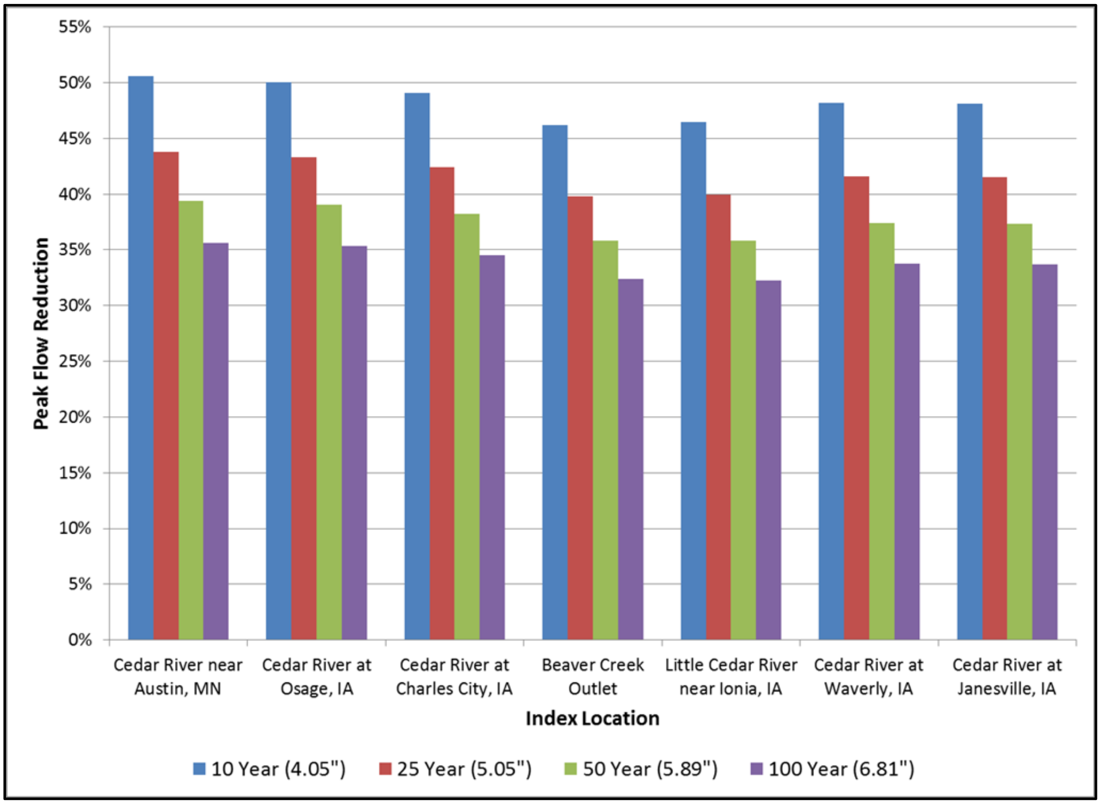

Figure 8.5. Percent reductions in peak discharge for the increased infiltration scenario due to land use changes (conversion of row crop agriculture to native prairie). Peak flow reductions at seven index locations progressing from upstream (left) to downstream (right) are shown for four different 24 hour design storms (4.05-6.81 inches of rain in 24 hours).

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 shown in Figure 8.5, the corresponding reduction in flood stage is between 2 and 7 feet. This reduction was estimated at the USGS stage/discharge gage locations, where the relationship between river stage and discharge – also known as a rating curve – has been measured.

Although a 2-7 foot reduction in flood stage would substantially reduce the flood inundation area, flooding still occurs in the native tall-grass prairie simulation. For instance, based on the flood stage level reported by the NWS at Janesville, water levels above flood stage are expected for both the current agricultural and the tall-grass prairie landscapes for a large rainstorm – those the size of the 25- (5.05 inches), 50- (5.89 inches), or 100-year (6.81 inches) return period design storm levels. For a smaller 10-year design storm, flooding would still occur for the agricultural landscape, but not for the tall-grass prairie landscape. Hence, conversion from the existing agriculture to tall-grass prairie landscape does not eliminate flooding, but would reduce its severity and frequency.

8.3.2 Improved Agricultural Conditions Due to Cover Crops

The second land use change example – where planting cover crops during the dormant season improves agricultural conditions during the growing season – results in less reduction of runoff and peak discharges than the tall-grass prairie simulation, which was expected. On average for the basin, 0.2-0.3 inches of additional infiltration occur for the four design storms.

Figure 8.6 shows the percent reduction in peak discharge for each subbasin and the seven index locations resulting from improved agricultural conditions due to cover crops for the 50-year, 24-hour design storm (5.89 inches of rain in 24 hours). Peak discharge reductions of 7-10% are common at the subbasin scale. As was observed for the tall-grass prairie simulation, greatest subbasin peak discharge reductions are observed in the Minnesota portion of the watershed due to these areas having a large percentage of agricultural area combined with slightly better soil drainage characteristics (dominated by B-type soils) than the Iowa portion of the watershed (dominated by C-type soils). Peak discharge reductions of around 9% are observed at all seven index locations throughout the watershed, reflecting the relatively even distribution of agriculture throughout the watershed.

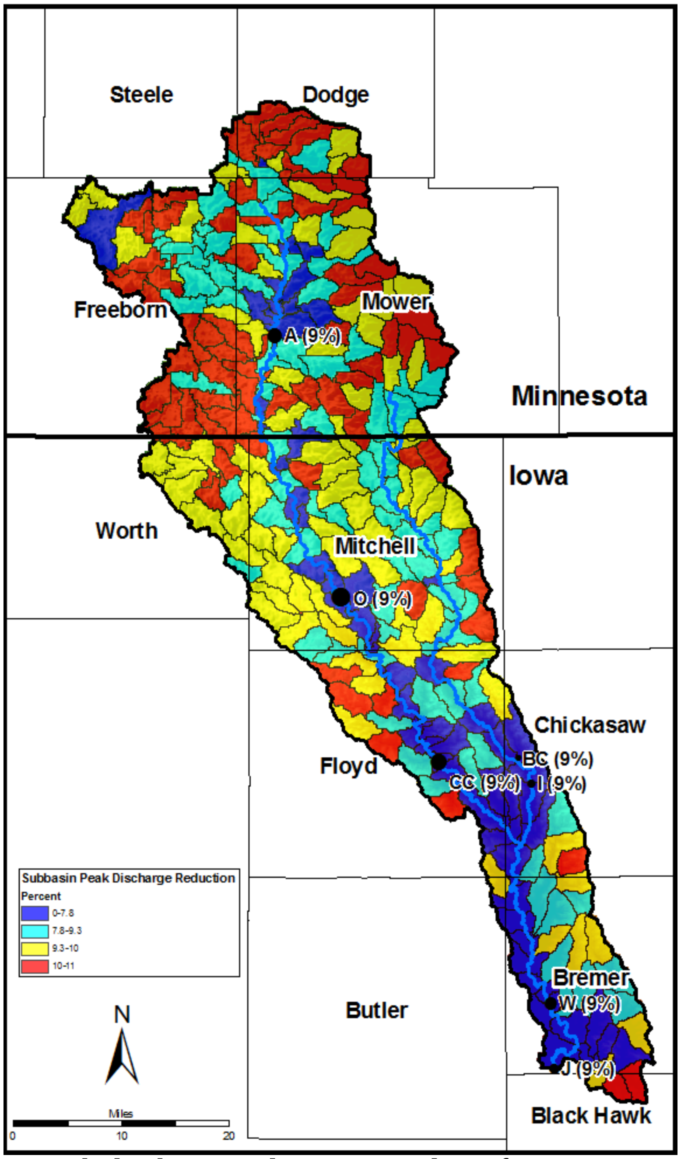

Figure 8.6. Subbasin peak discharge reductions resulting from improved agricultural conditions due to cover crops for the 50-year, 24-hour design storm (5.89 inches of rain in 24 hours).

Figure 8.7 shows the percent reductions in peak discharge resulting from the cover crop scenario at the seven index locations for the four design storms. The improved agricultural condition due to cover crops typically results in peak discharge reductions of 8-12%; flood stages are reduced by up to 1.5 feet. As for the tall-grass prairie scenario, the peak reduction is largest for the smallest design storm (10-year return period), and decreases with larger rainfall amounts (up to the 100-year return period). Finally, the percent reduction in peak discharge is fairly uniform at all locations, reflecting the relatively equal distribution of agricultural land throughout the watershed.

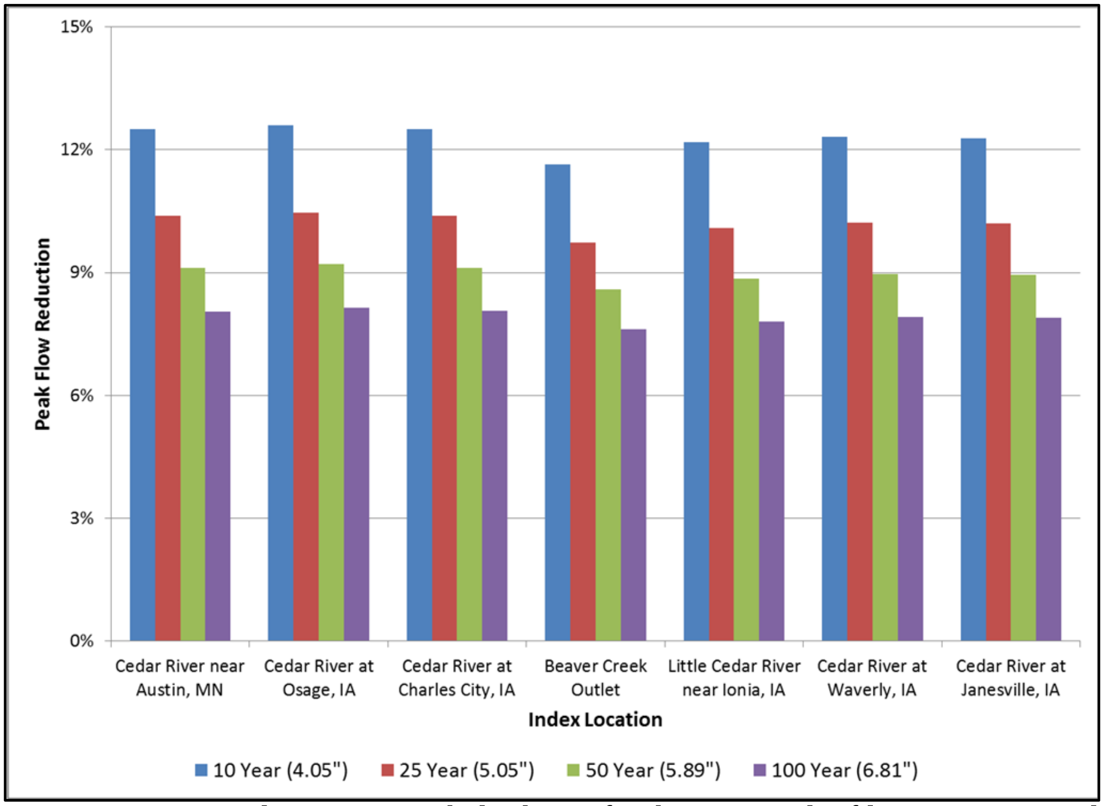

Figure 8.7. Percent reductions in peak discharge for the increased infiltration scenario due to land use changes (improved row crop agriculture conditions due to cover crops). Peak flow reductions at seven index locations progressing from upstream (left) to downstream (right) are shown for four different 24 hour design storms (4.05-6.81 inches of rain in 24 hours).

8.4 Chapter Summary

In this chapter, runoff reduction through land use changes is examined. Two hypothetical scenarios were considered to evaluate changes to the dominant land use in the watershed – row crop agriculture (77% of the area).

In the first scenario, all areas of row crop agriculture in the watershed are converted back to native-tall grass prairie that once covered much of Iowa. Tall-grass prairie has much better infiltration characteristics than the current agricultural landscape, so substantial reductions in runoff volumes and peak discharges were expected and observed. This hypothetical example provided a benchmark on the limits of flood reduction that are physically possible for any other watershed improvement project considered. For the 10-, 25-, 50-, and 100-year, 24-hour design storms considered (4.05-6.81 inches of rain applied in 24 hours), the simulated tall-grass prairie infiltrated 0.8-1.3 inches more into the ground than the current agricultural landscape. The restoration of native tall-grass prairie resulted in peak discharge reductions of 30-50% at various reference points throughout the watershed; 2-7 foot reductions in flood stage were estimated at the USGS stage/discharge gage locations. While these reductions in stage would substantially reduce the flood inundation area, it is important to recognize that the conversion to the tall-grass prairie landscape would not eliminate flooding completely; rather, the severity and frequency of flooding would be reduced.

In the second hypothetical scenario, the impact of cover crops was evaluated. The lower runoff condition resulting from planting cover crops during the dormant season across all agricultural areas in the watershed infiltrated 0.2-0.3 inches more into the ground than the current landscape. Peak flow reductions of 6-12% (flood stage reductions of up to one foot) were estimated at the seven index locations.

For both scenarios, runoff reduction benefits of increased infiltration were greatest for smaller rain events (10-year return period) and decreased with larger rainfall amounts (up to the 100-year return period). For a given rain storm, peak flow reductions were fairly uniform at the reference locations considered, reflecting the relatively equal distribution of agricultural land throughout the watershed.

CHAPTER 9: HYPOTHETICAL INCREASED INFILTRATION WITHIN THE WATERSHED – IMPROVING SOIL QUALITY

9.1 Introduction

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 Upper Cedar River watershed were improved, it could potentially reduce flood damages. Once again, this is a hypothetical example intended to provide a benchmark on the limits of flood reduction that are physically possible with runoff reduction.

To simulate improved soil quality with the HMS model, it is hypothesized 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.

9.2 Method

In the HMS model of the Upper Cedar River Watershed, the effects of improved soil health through conservation and best management practices are represented by changes in the NRCS hydrologic soil groups. Two cases were examined. The most dominant soil type in the Upper Cedar River Watershed is Type B (or B/D), which makes up 49.7% of the area. In the first case, improved soil quality is assumed to improve these soils to Type A. Type C (or C/D) soils make up 44.8% of the area, almost the same as for Type B soils. In the second case, improved soil quality is assumed to improve these soils to Type B. Figure 9.1 shows the spatial extents of the Type B (or B/D) and Type C (or C/D) soils in the watershed.

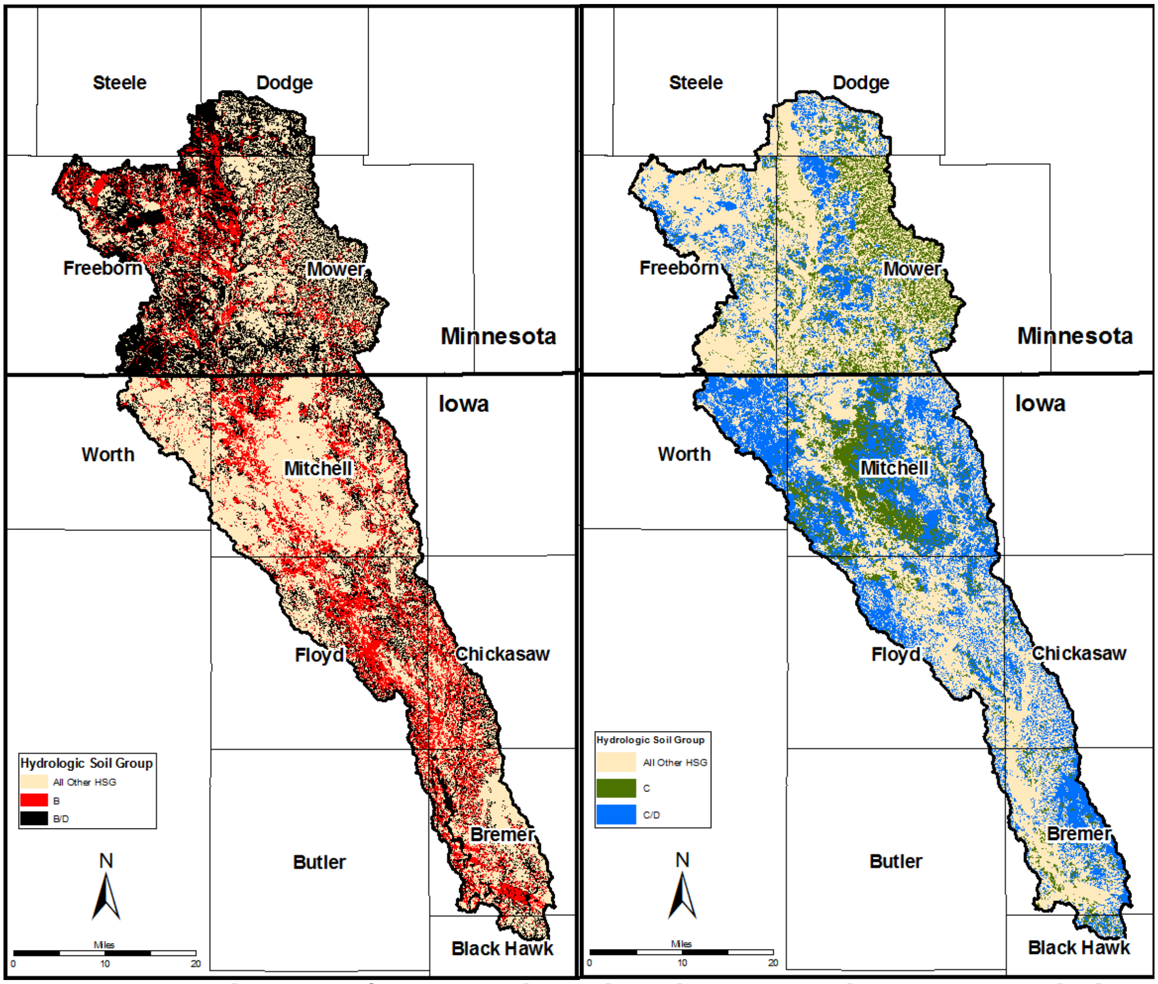

Figure 9.1. Spatial extents of Type B and C soils in the Upper Cedar River Watershed. The left figure shows Type B (red) or B/D (black) soils and the right figure shows Type C (green) or C/D (blue) soils. All other soil types in each figure are indicated by a tan color.

For both cases, new NRCS CNs reflecting the lower runoff potential with improved soil quality were assigned to each subbasin. A new CN grid was generated in Arc GIS reflecting these soil changes and area-weighted averaging was performed to assign new subbasin CNs. Table 9.1 lists the CNs assigned to different land use and soil combinations in Arc GIS (same as Table 4.4 but excluding the CNs defined for Type D soils). As an example, if an area of pasture/hay overlaid a Type C soil, its CN would be 79 under current conditions. If conservation practices improved the soil quality in this area to Type B, however, its CN would reduce to 69; similarly, if conservation measures improved the soil quality from Type B to A, the CN would reduce from 69 to 49.

Table 9.1. Curve Number assignments for different land use and soil (Type A, B, and C) combinations.

	Hydrologic Soil Group		
Land Use Description	*A*	*B*	*C*
Open Water	100	100	100
Woody wetlands	100	100	100
Emergent herbaceous wetlands	100	100	100
Developed, open space	49	69	79
Developed, low intensity	57	72	81
Developed, medium intensity	81	88	91
Developed, high intensity	89	92	94
Bare rock/sand/clay	98	98	98
Deciduous forest	32	58	72
Evergreen forest	32	58	72
Mixed forest	32	58	72
Shrub/scub	32	58	72
Grassland/herbaceous	49	69	79
Pasture/hay	49	69	79
Row crops	67	78	85

NOTE: Areas originally defined by Type B or C soils were assigned new CNs to reflect improved soil quality for the hypothetical scenario.

Following new assignment of subbasin CNs, the model was run for a set of design storms. Normal antecedent moisture conditions (AMC II) were assumed. Comparisons were made between current and improved soil quality simulations for the 10-, 25-, 50-, and 100-year return period, 24-hour SCS design storms. The same reference locations throughout the watershed as used for the land use scenarios in Chapter 8 were used for comparing a particular watershed improvement scenario to current conditions; the locations are shown again in Figure 9.2.

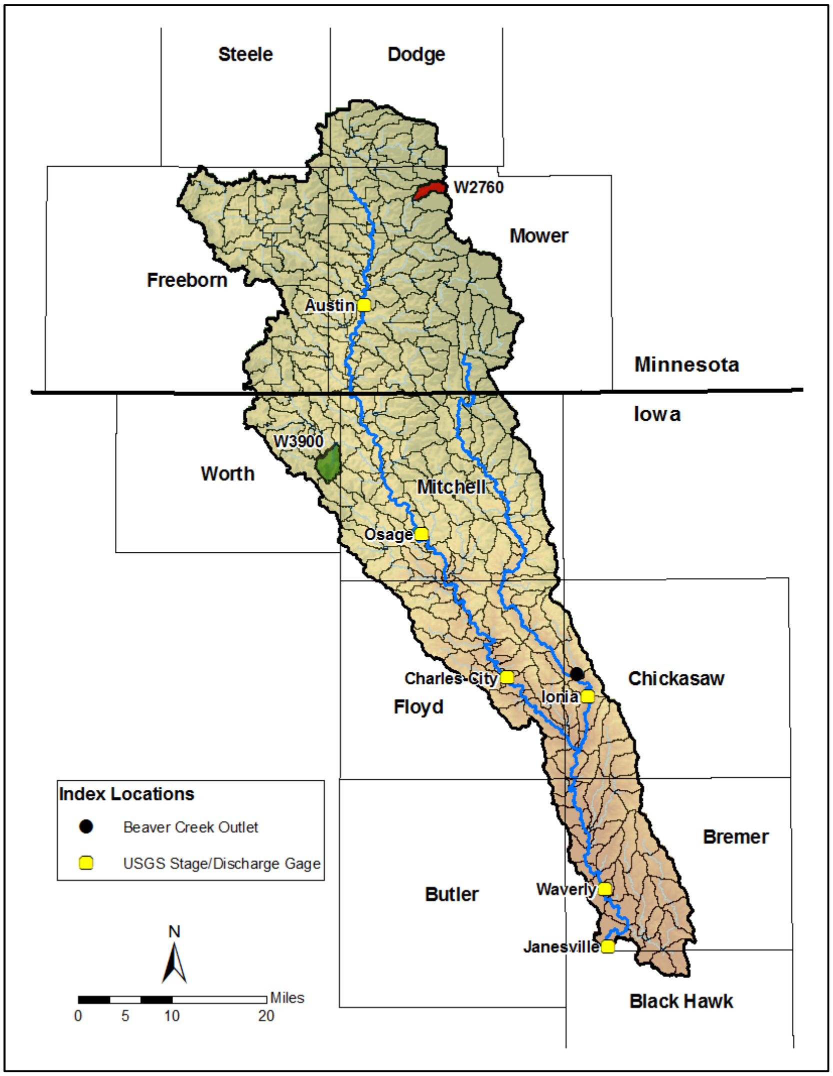

Figure 9.2. Reference locations for comparing watershed improvement scenarios to current conditions.

9.3 Results

9.3.1 Soil Improvement: Type B to A

The first soil improvement case – where all Type B soils improve to Type A – results in approximately 0.4 inches more infiltration than current soil conditions for the 10-year return period design storm (4.05 inches of rain in 24 hours). Additional infiltration increases to about 0.5 inches for the 25-year storm (5.05 inches of rain in 24 hours), and levels off at about 0.6 inches for the 50-year (5.89 inches of rain in 24 hours) and 100-year (6.81 inches of rain in 24 hours) storms. For this soil improvement case, subbasin CNs were reduced by 8% on average from the current condition.

Figure 9.3 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 (5.89 inches of rain in 24 hours). The smallest drainage area shown, Subbasin W3900 located along the eastern border of Worth County (panel a), was earlier identified as a high runoff potential area. It contains only a small percentage of Type B soils (about 10%), so in the first case, soil improvement (Type B to A) has minimal effects; the peak discharge reduction is only 3%, as only about 0.1 additional inches of rainfall infiltrates. However, for the larger downstream drainage areas (panels b-d), where Type B soils are more common, the peak discharge reduction is more significant; the sites all show a fairly uniform peak reduction of 17-19%.

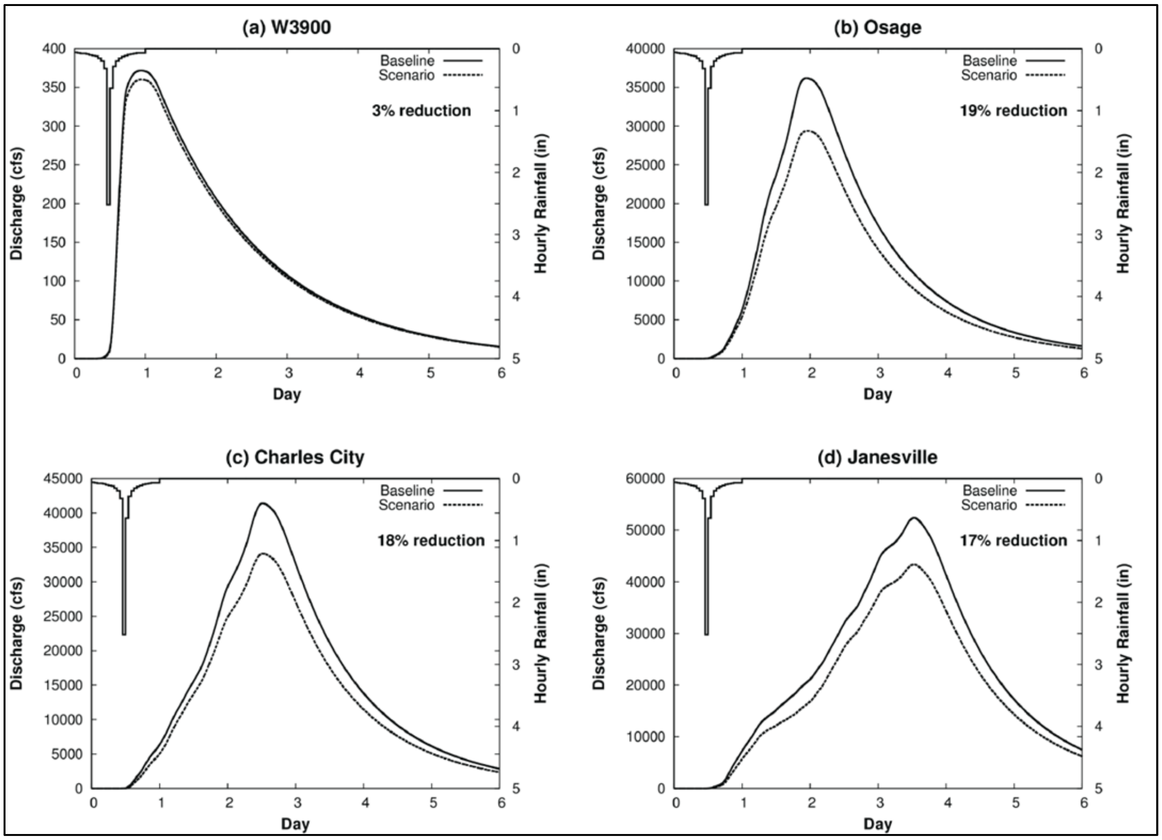

Figure 9.3. Hydrograph comparison at several locations for the increased infiltration scenario due to soil improvements (Type B to A). Improved soil quality was represented by converting all Hydrologic Soil Group B and B/D to A. Results shown are for the 50 year – 24 hour storm (5.89 inches of rain in 24 hours).

Figure 9.4 shows the percent reduction in peak discharge for each subbasin and the seven index locations as a result of the first soil improvement case for the 50-year, 24-hour storm (5.89 inches of rain in 24 hours). Peak discharge reductions of 14-24% are common at the subbasin scale. The greatest subbasin peak discharge reductions are observed in the northwest part of the watershed in Minnesota as well as in the lower third of the watershed in parts of Floyd, Chickasaw, and Bremer counties. The Minnesota portion of the watershed with the greatest reductions is in the Des Moines Lobe geographic landform region, which is dominated by Type B or B/D soils. The lower third of the watershed where the greatest peak flow reductions are observed is in a portion of the Iowan Surface where a higher concentration of Type B soils are found.

The middle part of the watershed is dominated by Type C soils, so this first soil improvement case yields the smallest peak discharge reductions in this area. Peak discharge reductions are fairly uniform at the seven index locations (16-21%).

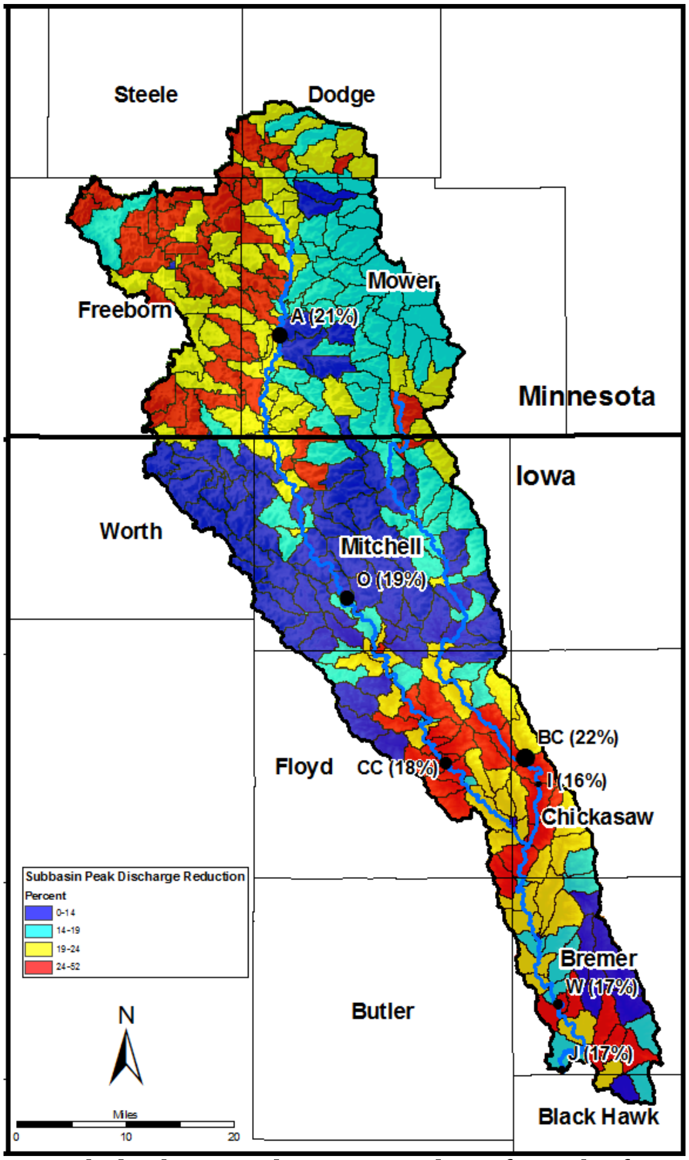

Figure 9.4. Subbasin peak discharge reductions resulting from the first soil improvement case (Type B to A) for the 50-year, 24-hour design storm (5.89 inches of rain in 24 hours).

Figure 9.5 shows the percent reductions in peak flow resulting from the first soil improvement case at the seven index locations for the four design storms. Peak flows are typically reduced by 15-25%. As a result, flood stages are reduced by 1-3 feet. As with the two land change use scenarios, the peak reduction is largest for the smallest design storm (10-year return period), and decreases with larger rainfall amounts (up to the 100-year return period). This outcome reflects the landscape's diminished capacity to infiltrate additional water as rain rates increase.

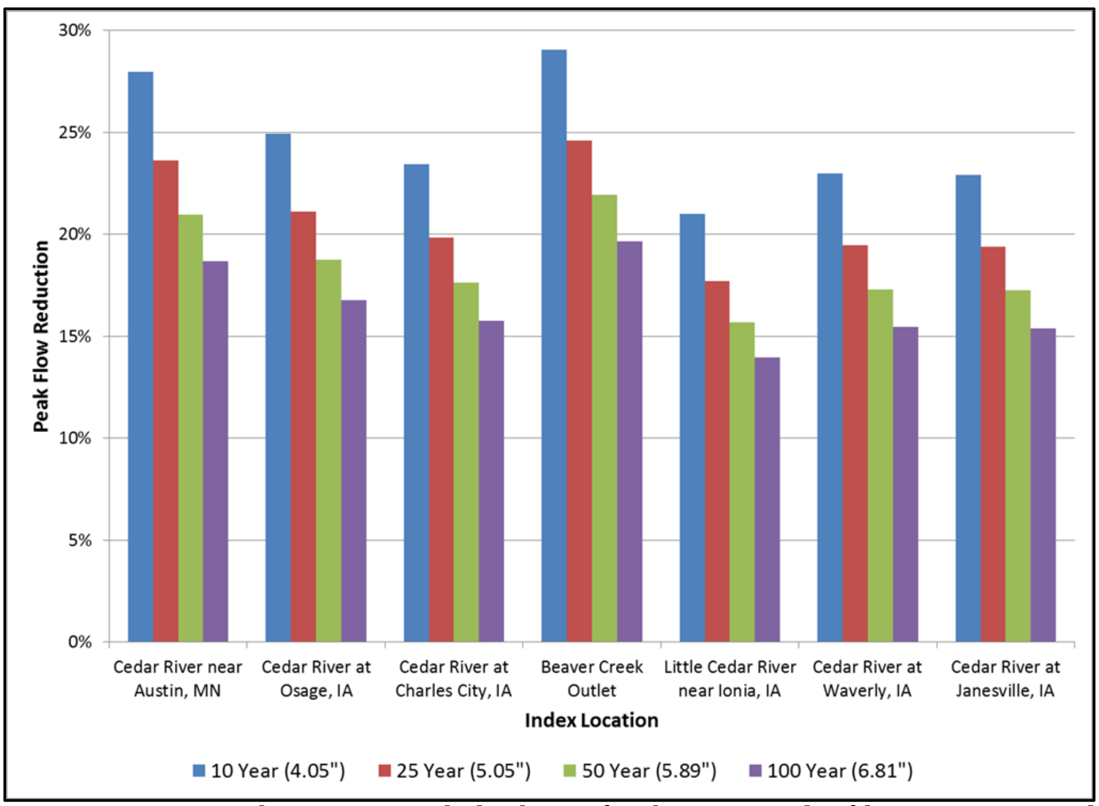

Figure 9.5. Percent reductions in peak discharge for the increased infiltration scenario due to soil improvements (Type B to A). Peak flow reductions at seven index locations progressing from upstream (left) to downstream (right) are shown for four different 24 hour design storms (4.05-6.81 inches of rain in 24 hours).

9.3.2 Soil Improvement: Type C to B

The second soil improvement case – where all Type C soils improve to Type B – results in less reduction of runoff and peak discharges than the first case. Subbasin CNs were only reduced by 4% on average for this second soil improvement scenario compared to 8% for the first case. On average for the basin, only 0.2-0.3 inches of additional infiltration occur for the four design storms.

Figure 9.6 shows the percent reduction in peak discharge for each subbasin and the seven index locations as a result of the second soil improvement case for the 50-year, 24-hour storm (5.89 inches of rain in 24 hours). Reductions are roughly half of those for the first soil improvement case. Peak discharge reductions of 6-12% are common at the subbasin scale. The greatest subbasin peak discharge reductions are observed in the Iowa portion of the watershed (parts of Worth, Mitchell, Floyd, and Bremer counties) where Type C and C/D soils constitute a large percentage of the area. Once again, peak discharge reductions are fairly uniform at the seven index locations, ranging from 7-11%.

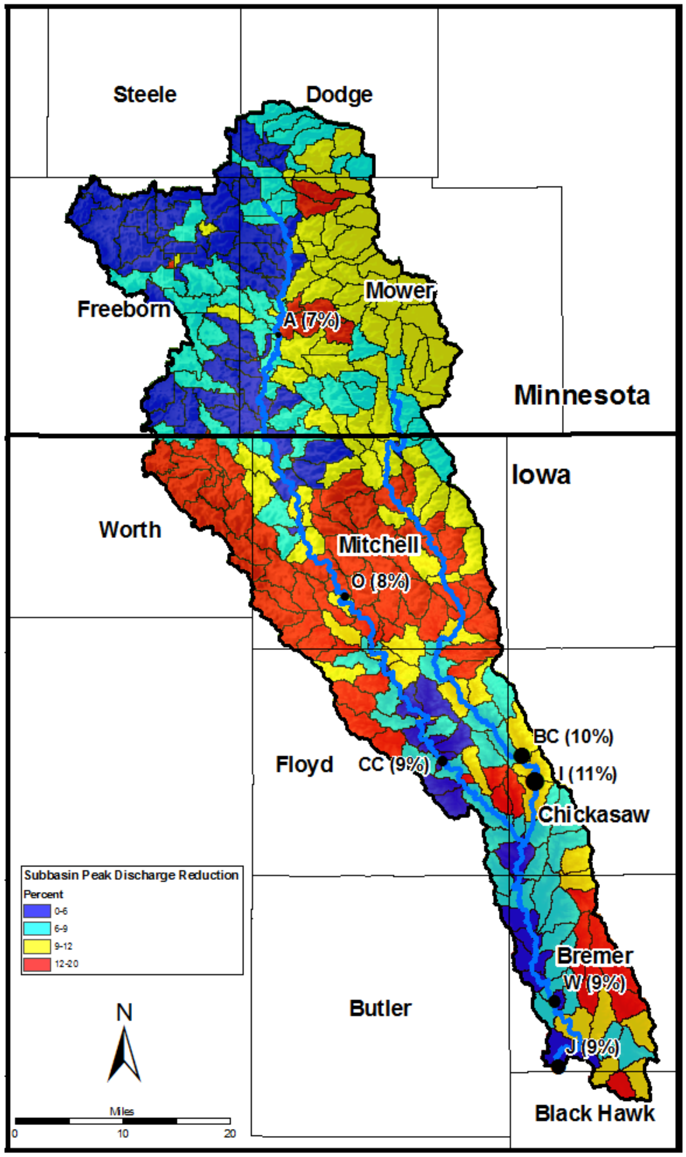

Figure 9.6. Subbasin peak discharge reductions resulting from the second soil improvement case (Type C to B) for the 50-year, 24-hour design storm (5.89 inches of rain in 24 hours).

Figure 9.7 shows the percent reductions in peak flow resulting from this hypothetical scenario at the seven index locations for the four design storms. Peak flows are typically reduced by 6-15%; flood stages are reduced by up to one foot. Because a greater amount of Type C soil is found in Iowa, particularly in Worth and Mitchell counties, the percent reductions in peak flow show an increasing trend moving downstream from Austin to Charles City. Moving downstream from Charles City, fewer

Type C soils are present. As a result, peak reductions at Waverly and Janesville decrease and are almost identical. Finally, as with the prior simulations, the peak reduction is largest for the smallest design storm (10-year return period), and decreases with larger rainfall amounts (up to the 100-year return period), reflecting the landscape's diminished capacity to infiltrate additional water as rain rates increase.

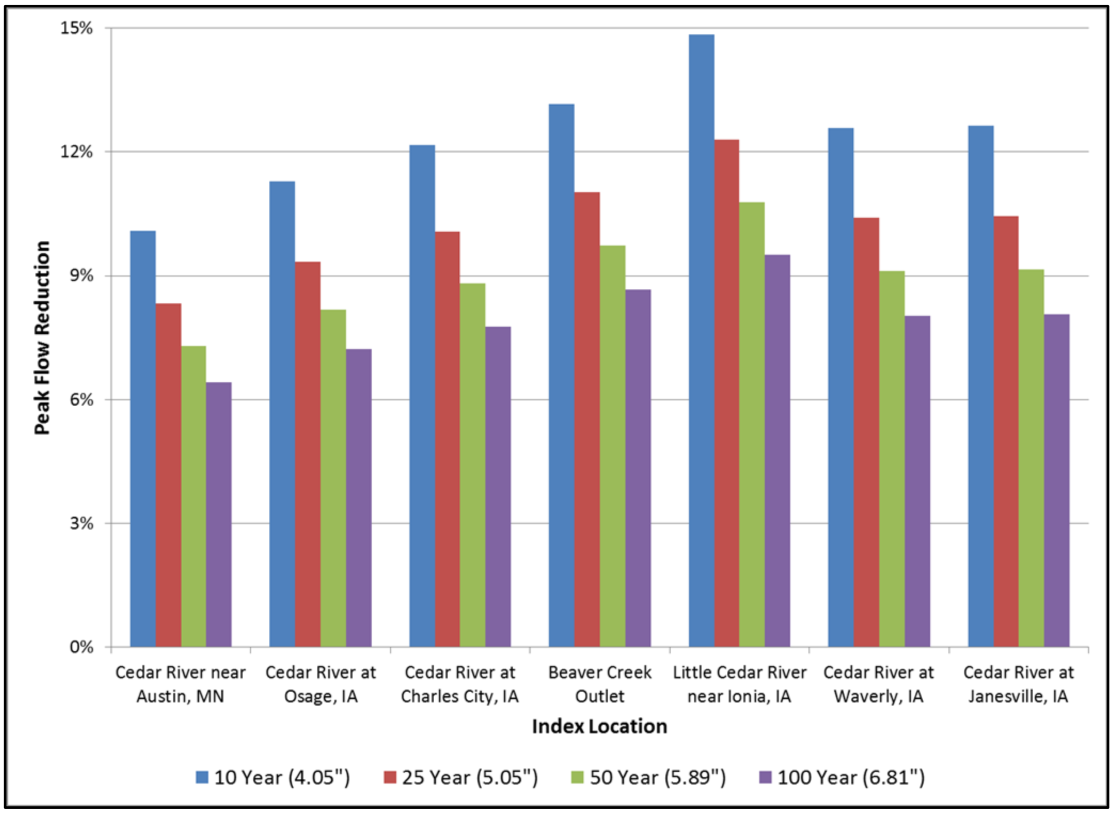

Figure 9.7. Percent reductions in peak discharge for the increased infiltration scenario due to soil improvements (Type C to B). Peak flow reductions at seven index locations progressing from upstream (left) to downstream (right) are shown for four different 24 hour design storms (4.05-6.81 inches of rain in 24 hours).

9.4 Chapter Summary

In this chapter, runoff reduction through soil quality improvements is examined. The effects of improved soil health through conservation and best management practices (such as increased organic content and the introduction of cover crops) are represented by changes in the NRCS hydrologic soil group. Two cases were examined. In the first case, improved soil health was assumed to improve all Type B (or B/D) soils (49.7% of the area) to Type A soils. This had a noticeable impact on the flood hydrology in the basin, but not to the same extent as the native prairie land use change scenario. For the 10-, 25-, 50-, and 100-year, 24-hour design storms considered (4.05-6.81 inches of rain applied in 24 hours), the first soil improvement case infiltrated 0.4-0.6 inches more into the ground than existing conditions. Greatest peak flow reductions were observed in the lower and upper thirds of the watershed where Type B soils are more heavily concentrated. Peak discharge reductions of 15-25% were observed at the seven reference locations throughout the watershed, and flood stage reductions of 1-3 feet were estimated.

In the second case, improved soil quality was assumed to improve all Type C (or C/D) soils (44.8% of the area) to Type B soils. Runoff and peak flow reductions were approximately half of those seen with the first soil improvement case. On average, only 0.2-0.3 inches of additional infiltration occurred for the four design storms. Greatest reductions in peak discharge were observed in the middle third of the watershed (Mitchell and Worth counties) where a large amount of Type C soils are present. Peak flows were typically reduced by 6-15% at the seven index locations (up to a one foot reduction in flood stage). For both soil improvement scenarios, runoff reduction benefits are greatest for smaller rain events (10-year return period) and decrease with larger rainfall amounts, reflecting the landscape’s diminished infiltration capacity as rain rates increase.

CHAPTER 10: MITIGATING THE EFFECTS OF HIGH RUNOFF WITH FLOOD STORAGE

10.1 Introduction

Another way to mitigate the effects of high runoff is with flood storage. The most common type of flood storage is a pond. In agricultural areas, ponds usually hold some water all the time. However, ponds also have the ability to store extra water during high runoff periods. This so-called flood storage can be used to reduce flood peak discharges.

Unlike 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 (see Figure 1.1). They adopted a plan that included identifying the locations for 154 distributed storage structures (mainly ponds) that could be built within the watershed. As of 2014, 132 of these structures have been constructed.

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 areas throughout the Upper Cedar 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 instead. Therefore, this example is not a proposed plan for siting a system of storage ponds; it was not 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 from water storage structures.

10.2 Method

To examine the impact additional flood storage would have on the flood hydrology of the Upper Cedar River Watershed, a system of storage ponds was placed in the headwater regions of the basin and comparisons were made between this scenario and the baseline, no ponds scenario. In order to evaluate the impact of distributed storage projects on reducing flood peaks, a prototype pond design specifying the pond geometry (size) and outflow characteristics needed to be developed.

10.2.1 Prototype Storage Pond Design

Many ponds in Iowa have been constructed to provide flood storage. A pond schematic is illustrated in Figure 10.1. The pond is created by constructing an earthen embankment across a stream. A typical pond holds some water all the time (called 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. The water level at the principal spillway elevation is also called the normal pool level. 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.

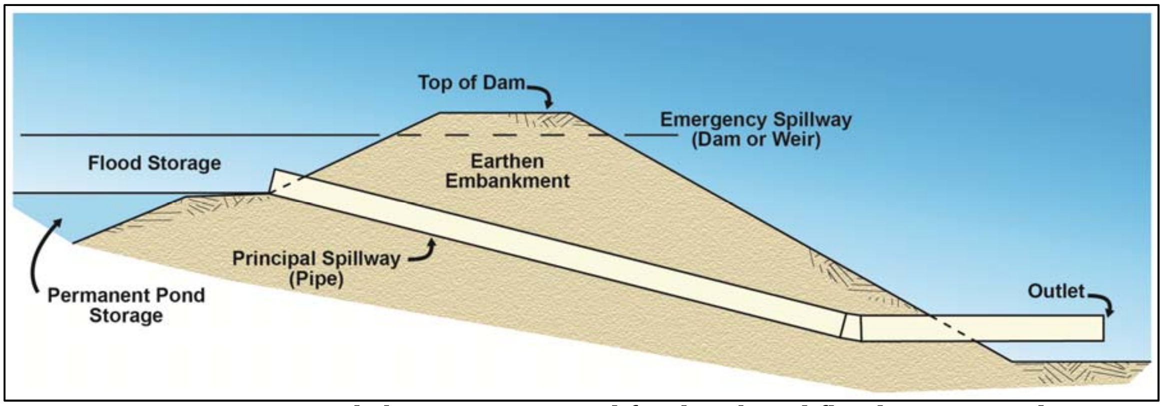

Figure 10.1. Prototype pond characteristics used for distributed flood storage analysis.

10.2.1.1 Prototype Pond Outlet and Emergency Spillway

Using information from ponds constructed in Soap Creek and NRCS technical references on pond design, a prototype pond outlet and emergency spillway were defined for the simulation experiments. A 12-inch pipe outlet was assumed for the principal spillway. The elevation of the principal spillway above the pond bottom was selected to achieve a normal pool area between two and five acres. A 20-foot wide overflow opening was assumed for the emergency spillway. The top of the dam was set two feet above the emergency spillway.

The elevation difference between the principal and emergency spillways was varied; simulations were done with elevation differences of 3, 5, and 7 feet. As the elevation difference increases, the available flood storage increases exponentially. Therefore, simulations for ponds with a 7 foot elevation difference have much more flood storage than those with a 3 foot difference.

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 reaches the emergency spillway, outflow discharge from the pond also includes contributions from the emergency spillway. Discharge from the emergency spillway was determined using NRCS Technical

References assuming “C-Type” retardance on the spillway, which was determined to be a reasonable design assumption (based on discussions with regional NRCS engineers).

10.2.1.2 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 is also unwise, unless good sites for pond structures are selected in the first place (for each and every pond). As a compromise, the relationship between stage and storage at a few potential pond sites in the Upper Cedar River watershed were analyzed, and the results were averaged to define a prototype pond shape.

The first step was to select some potential pond sites in the Upper Cedar River watershed for topographic analysis. Figure 10.2 shows the 320 subbasins in the HMS model. Of these, 121 are headwater basins. These are the subbasins where the stream network begins based on the stream threshold specified in the model development (Chapter 4). 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, seven of the 121 headwater basins were selected as exploratory sites. These seven subbasins are scattered throughout the watershed and encompass both geographic landform regions (Des Moines Lobe and Iowan Surface).

In each of the seven subbasins, locations for a pond embankment were selected. In most cases, one location was selected per subbasin, but multiple locations were selected for a couple subbasins[1]. Each site was chosen where it was felt there was 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 Arc GIS analysis using the 10-meter DEM of the local terrain. An elevation is specified, and the volume between the horizontal plane projected at this elevation and the surface of the DEM is calculated. The calculation is then 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.

[1] 2-3 sites were selected in two exploratory subbasins to compare the variability in the stage-storage relationships at the smaller subbasin scale to the larger watershed scale. As expected, the stage-storage relationships derived within a single subbasin were quite similar.

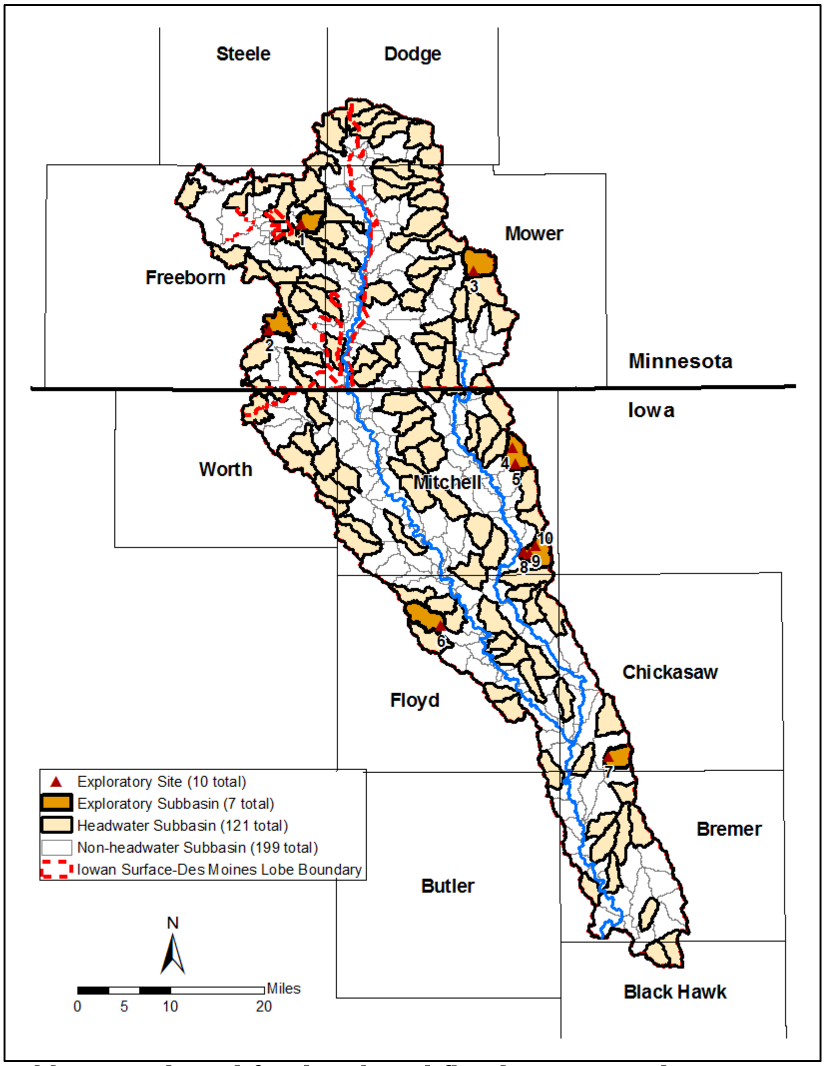

Figure 10.2. Subbasins selected for distributed flood storage analysis. Prototype ponds were placed in 121 headwater subbasins (tan) and seven of these subbasins (orange) were used as exploratory sites to develop a representative stage-storage relationship for the prototype pond.

The last step was to compare the different stage-storage relationships developed for the exploratory locations. Figure 10.3 shows the stage-storage relationships developed at the 10 exploratory sites. As expected, stage-storage relationships can 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. Overall, the local terrain was found to be more important than the geographic landform region of the site. Therefore, a single stage-storage relationship – the best fit to relationships from all 10 sites – was selected for use in all the simulations. The final prototype pond stage-storage relationship was derived after having set the principal spillway elevation.

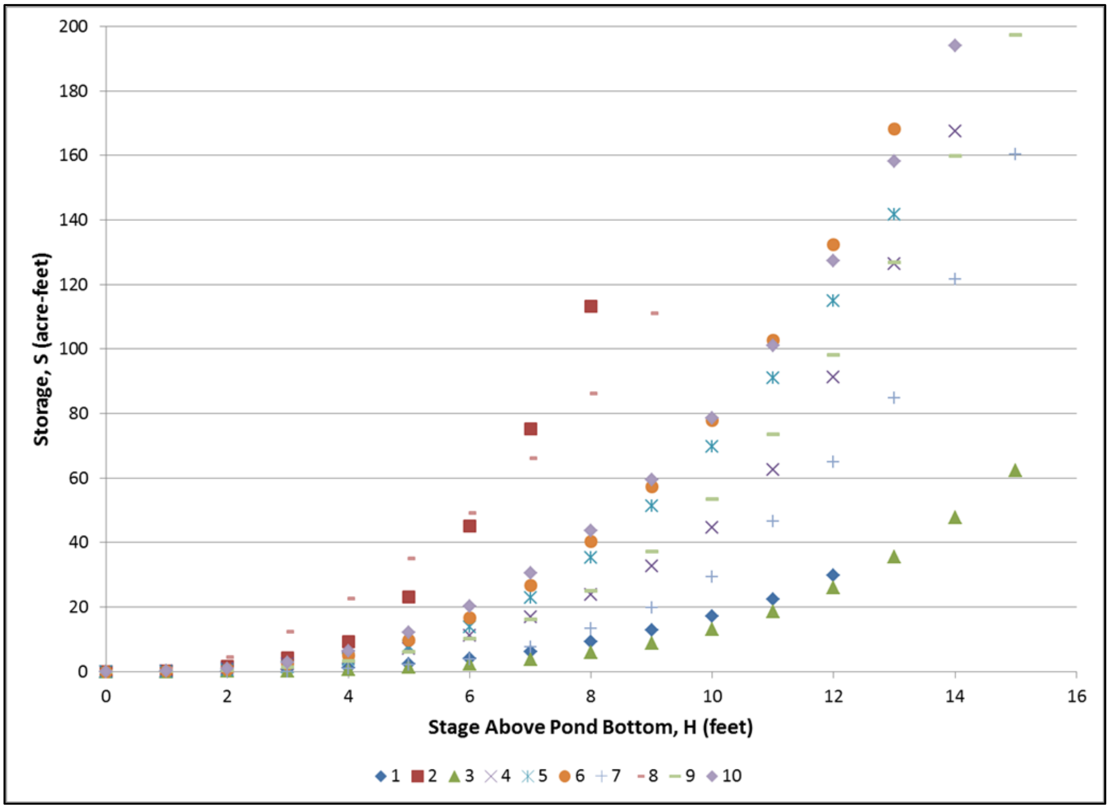

Figure 10.3. Stage-storage relationships developed for the 10 exploratory pond sites. Storage volumes were computed in Arc GIS.

In addition to computing storage volumes at each stage level, surface areas were also computed. The stage-area relationships for the 10 exploratory pond locations were used to determine the elevation of the principal spillway above the pond bottom. As mentioned previously, the principal spillway was set at an elevation above the pond bottom that achieved a 2-5 acre surface area. This guideline was determined from review of the Soap Creek project data. Figure 10.4 shows the stage- area relationships developed at the 10 exploratory sites for elevations 1-5 feet above the pond bottom. Using the average surface area of all 10 sites at each elevation to define the stage-area curve for the prototype pond, the principal spillway was set three feet above the pond bottom. The (normal) pool area at this elevation is 2.32 acres and the permanent storage is 2.76 acre-feet (average of the storages shown in Figure 10.3 at a stage of three feet).

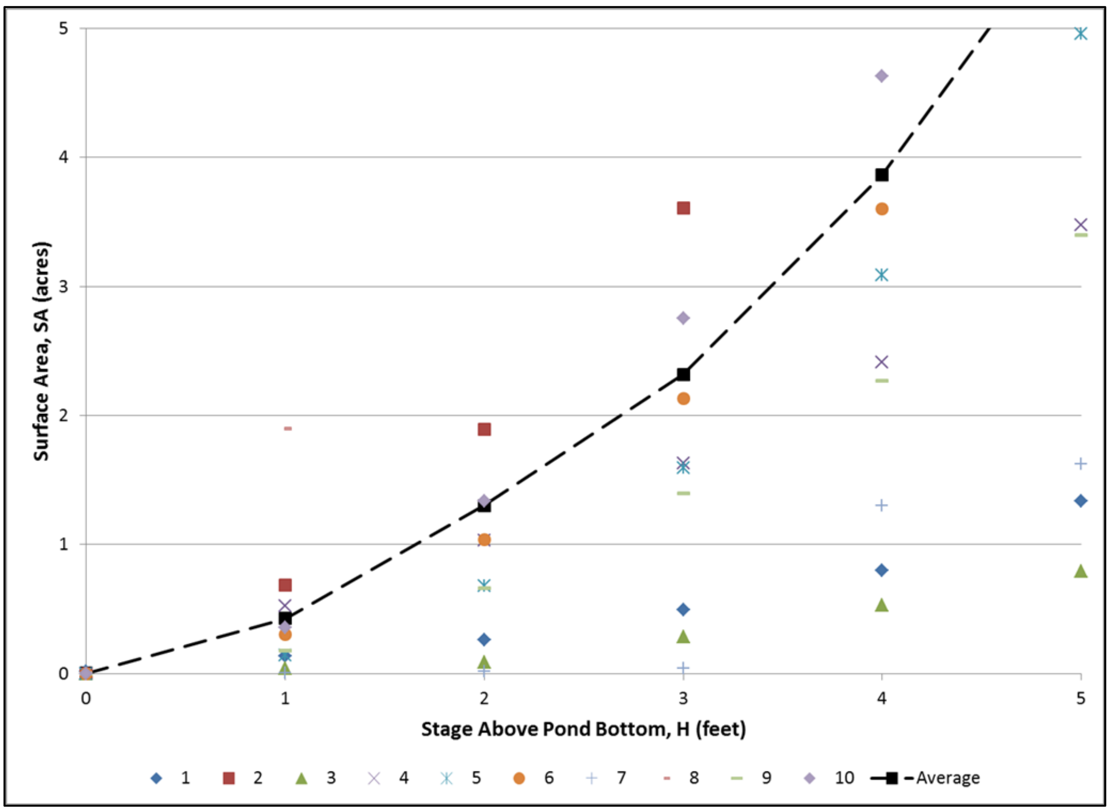

Figure 10.4. Stage-area relationships developed for the 10 exploratory pond sites. Surface areas were computed in GIS.

Having set the principal spillway elevation, the final stage-storage relationship was developed for the prototype pond. Figure 10.5 shows the *net* (flood) storage as a function of stage above the principal spillway for the 10 exploratory sites. The net storage was calculated by reducing the storages shown in Figure 10.3 by the storage at the principal spillway elevation (3 feet above the pond bottom) for each site. The power law curve in Figure 10.5, representing the best fit to all 10 relationships, was used to define the prototype pond stage-storage relationship.

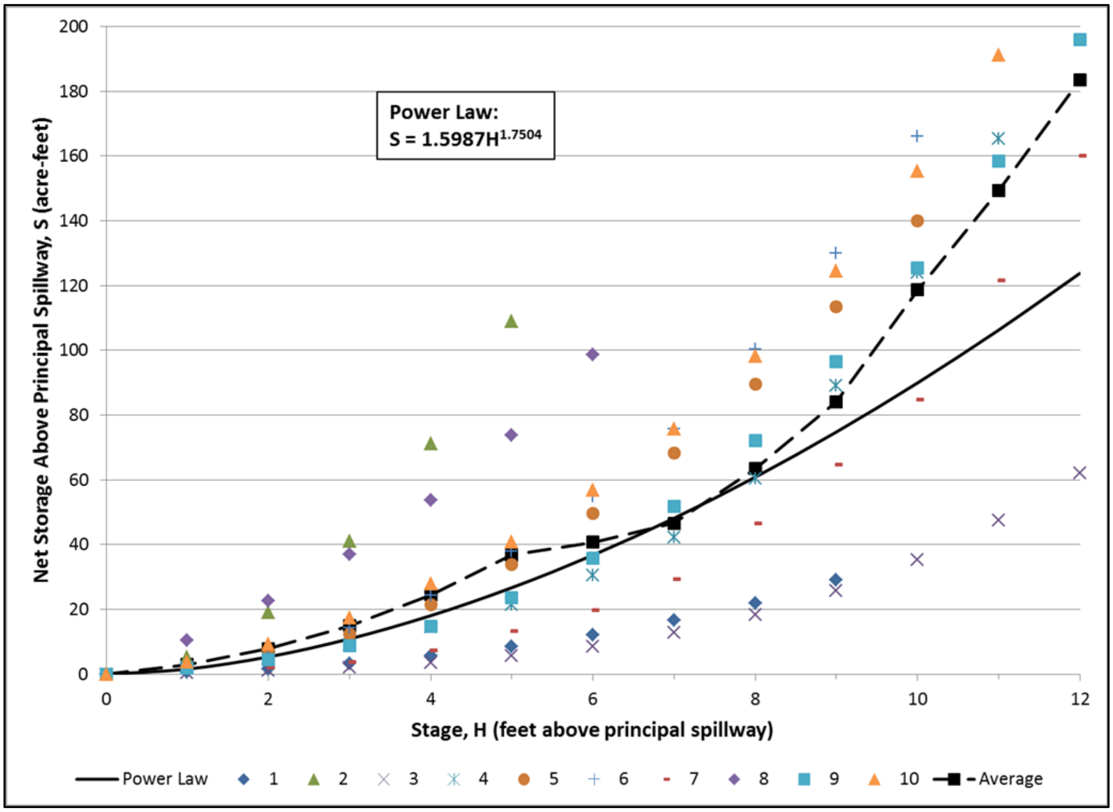

Figure 10.5. Stage-storage relationships developed for the 10 exploratory pond sites reflecting the elevation and volume above the principal spillway (set 3 feet above the pond bottom).

10.2.1.3 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 for pond simulations.

In all, three different prototype ponds are used. For the small pond, the emergency spillway elevation is set to 3 feet above the primary spillway; this results in a flood storage capacity of 10.9 acre-feet per prototype pond. For the medium-sized pond, the emergency spillway elevation is set to 5 feet above the primary spillway; this results in a flood storage capacity of 26.8 acre-feet per prototype pond. For the large pond, the emergency spillway elevation is set to 7 feet above the primary spillway; this results in a flood storage capacity of 48.2 acre-feet per prototype pond. The stage-storage-discharge relationships for the three prototype ponds are found in Appendix B.

10.2.2 Siting of Hypothetical Ponds in the Upper Cedar River Watershed

To examine the hypothetical impact that flood storage would have on the flood hydrology of the Upper Cedar River watershed, prototype ponds were placed throughout the headwater subbasins (see again Figure 10.2). In the Soap Creek Watershed, where flood storage is already used extensively, the average pond density is one built pond for every 1.9 mi^2 of drainage area. Therefore, for the flood storage simulations for the Upper Cedar River watershed, ponds were placed in headwater subbasins at a density of one pond for every 2 mi^2 of drainage area.

The 121 headwater subbasins range in size from 3.9-9.3 mi^2. Hence, all the subbasins contain more than one pond. For example, if a headwater subbasin drainage area was 6 mi^2, it would have three ponds. Furthermore, not all the area within a subbasin will drain to a pond; some water would flow into the stream below the ponds and not be temporarily stored. To handle these conditions in the HMS model, it was assumed that half the subbasin area drains through a pond, and half does not. The area draining through a pond and the area bypassing the pond maintained the same Clark parameters (time of concentration and time storage coefficient), which assumes the water travel time to the subbasin outlet has not changed even though the subbasin area has been halved. 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 is most efficiently accomplished in the model by creating a single aggregate pond in each headwater subbasin. That is, if there were three ponds in a subbasin, it has the same aggregate effect of a single pond that has three times the storage and three times the outflow of a single prototype pond. So from an HMS modeling standpoint, the half of the subbasin that drains through a pond can more simply be routed through a single aggregated pond. In this way, the effects of the pond storage can be estimated without having to specify the exact physical locations of any pond.

For the 121 headwater subbasins, a total of 372 prototype ponds were simulated. All the headwater subbasins contained between 2-5 ponds. Figure 10.6 shows the 121 headwater subbasins, and the number of prototype ponds assigned to each. In HMS, the 372 prototype ponds were represented by 121 aggregated ponds, one for each of the 121 headwater subbasins. Overall, the ponds control flows from a total area of 375 mi^2; in other words, 23% of the watershed area drains through the simulated prototype ponds.

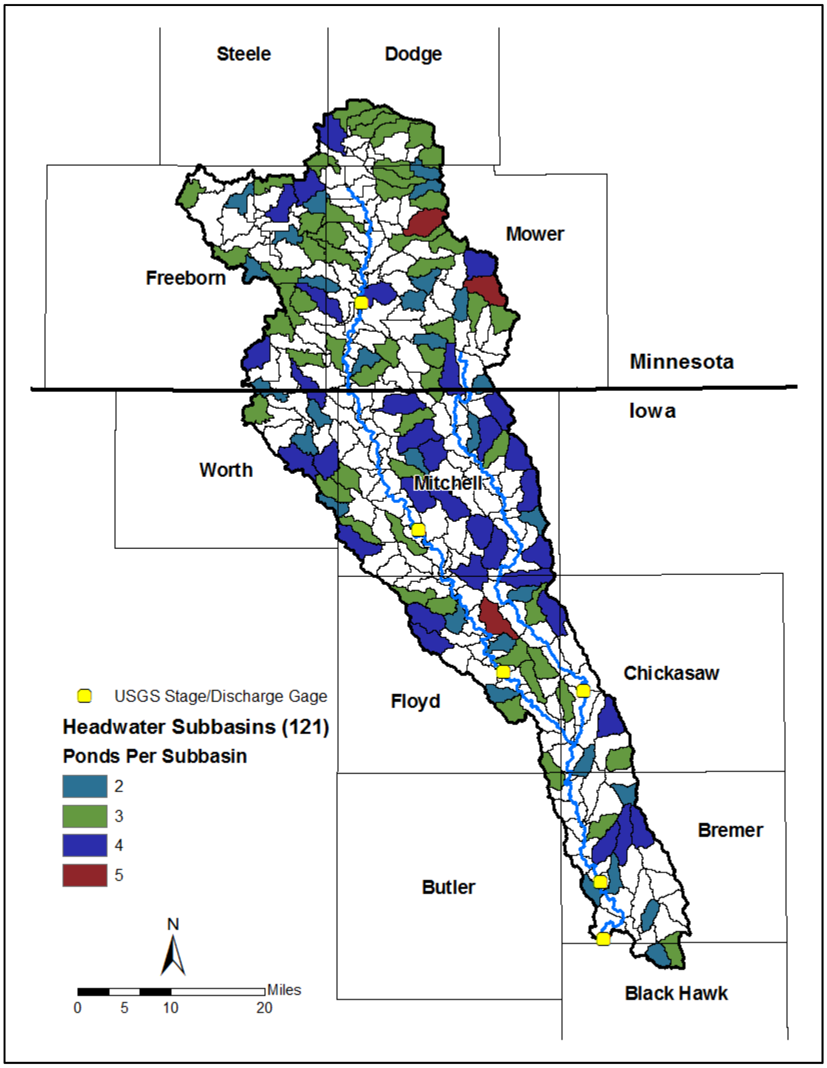

Figure 10.6. Headwater subbasins selected for distributed flood storage analysis and number of prototype ponds assigned to each subbasin. Each headwater subbasin contained between 2-5 ponds, resulting in 372 prototype ponds that were aggregated into 121 ponds, one for each headwater subbasin.

The pond characteristics upstream of the seven index locations are characterized in Table 10.1. Overall, the percentage of the upstream area controlled by ponds is relatively consistent; it ranges from 21.4% for the Little Cedar River at Ionia, to a maximum of 25.8% for the Cedar River at Austin.

Table 10.1. Summary of pond characteristics upstream of the seven index locations.

Location	Drainage Area (mi^2)	Number of Aggregated Ponds Upstream	Number of Prototype Ponds Upstream	Drainage Area Controlled by Ponds (mi^2)	Percent Controlled
Cedar River at Austin	393	34	102	101.4	25.8
Cedar River at Osage	833	66	199	199.3	23.9
Cedar River at Charles City	1069	83	256	257.3	24.1
Beaver Creek Outlet	17	1	4	4.3	24.9
Little Cedar River at Ionia	294	19	62	63.0	21.4
Cedar River at Waverly	1550	114	353	355.5	22.9
Cedar River at Janesville	1663	121	372	375.4	22.6

Table 10.2 summarizes the flood storage available upstream of the seven index locations. For small ponds, the total flood storage is 4,069 acre-feet; this amount of water placed over the upstream drainage area controlled by ponds would have a water depth of 0.2 inches. Hence, the ponds can temporarily store roughly 0.2 inches of runoff from a storm event. For medium-sized ponds, the total flood storage is 9,949 acre-feet; this is equivalent to roughly 0.6 inches of runoff. For large ponds, the total storage is 17,930 acre-feet; this is equivalent to roughly 0.9 inches. As shown in Table 10.2, these average storage depths are relatively consistent at the seven locations.

Table 10.2. Summary of the flood storage available upstream of the seven index locations for the small, medium, and large wet pond scenarios.

Pond Size Flood Storage/Pond		Small 10.9 acre-feet		Medium 26.8 acre-feet		Large 48.2 acre-feet	
Location	Percent of Total Flood Storage in Watershed	Total Flood Storage Upstream (acre-feet)	Total Flood Storage Upstream, Uniform Depth (inches)	Total Flood Storage Upstream (acre-feet)	Total Flood Storage Upstream, Uniform Depth (inches)	Total Flood Storage Upstream (acre-feet)	Total Flood Storage Upstream, Uniform Depth (inches)
Cedar River at Austin	27	1116	0.21	2728	0.50	4916	0.91
Cedar River at Osage	53	2177	0.20	5322	0.50	9591	0.90
Cedar River at Charles City	69	2800	0.20	6847	0.50	12339	0.90
Beaver Creek Outlet	1	44	0.19	107	0.46	193	0.83
Little Cedar River at Ionia	17	678	0.20	1658	0.49	2988	0.89
Cedar River at Waverly	95	3861	0.20	9441	0.50	17014	0.90
Cedar River at Janesville	100	4069	0.20	9949	0.50	17930	0.90

Additionally, ponds can be classified as “wet” ponds or “dry” ponds. The ponds discussed previously are classified as a “wet” pond because no outlet exists at the bottom of the pond, so the pond holds some water most of the time (permanent storage). In addition to the principal and emergency spillways, a “dry” pond as defined here has another much smaller outlet placed at the bottom of the pond so it has no permanent storage and drains to empty between rain events. A 2-inch diameter pipe was assumed for this smaller outlet and outflow discharge was calculated assuming submerged orifice flow using a discharge coefficient of 0.8 (Sturm, 2010). The dry pond alternative should typically result in slightly greater peak flow reductions downstream than the wet pond alternative since each dry pond provides slightly more flood storage (the permanent storage of the wet pond) while throttling down the outflow discharge from the pond through the 2-inch pipe until the water reaches the primary spillway elevation (three feet above the pond bottom). For each pond size scenario, both the wet and dry pond alternatives were simulated to evaluate how much additional flood reduction was gained from the dry pond alternative.

Table 10.3 summarizes the flood storage available upstream of the seven index locations for the dry pond alternative for each pond size. Each prototype dry pond provides 2.76 acre-feet of additional flood storage (the permanent storage of the wet prototype pond); this represents a 25% increase in the flood storage provided per prototype pond for the small pond scenario, a 10% increase for the medium-sized pond scenario, and a 6% increase for the large pond scenario. For each pond scenario, dry ponds provide an additional 1,028 acre-feet of flood storage to the watershed; this means dry ponds could store 0.05 inches more of runoff than wet ponds. For small ponds, the total flood storage provided by the dry pond alternative increases from 4,069 acre-feet (0.20 inches) to 5,097 acre-feet (0.25 inches). For medium-sized ponds, the total flood storage provided by the dry pond alternative increases from 9,949 acre-feet (0.50 inches) to 10,977 acre-feet (0.55 inches). For large ponds, the total flood storage provided by the dry pond alternative increases from 17,930 acre-feet (0.90 inches) to 18,958 acre-feet (0.95 inches).

Table 10.3. Summary of the flood storage available upstream of the seven index locations for the small, medium, and large dry pond scenarios.

Pond Size Flood Storage/Pond		Small 13.7 acre-feet		Medium 29.5 acre-feet		Large 51.0 acre-feet	
Location	Percent of Total Flood Storage in Watershed	Total Flood Storage Upstream (acre-feet)	Total Flood Storage Upstream, Uniform Depth (inches)	Total Flood Storage Upstream (acre-feet)	Total Flood Storage Upstream, Uniform Depth (inches)	Total Flood Storage Upstream (acre-feet)	Total Flood Storage Upstream, Uniform Depth (inches)
Cedar River at Austin	27	1398	0.26	3010	0.56	5198	0.96
Cedar River at Osage	53	2727	0.26	5872	0.55	10141	0.95
Cedar River at Charles City	69	3508	0.26	7554	0.55	13046	0.95
Beaver Creek Outlet	1	55	0.24	118	0.51	204	0.88
Little Cedar River at Ionia	17	849	0.25	1830	0.54	3160	0.94
Cedar River at Waverly	95	4837	0.26	10417	0.55	17989	0.95
Cedar River at Janesville	100	5097	0.25	10977	0.55	18958	0.95

10.3 Results

The HMS model was run with ponds to simulate the effects of flood storage on peak discharges. Separate model runs were made assuming small, medium-sized, and large ponds were in place. For each pond size, the wet and dry pond alternatives were evaluated. For clarity, “wet” and “dry” descriptors will be used to specify which pond alternative is being considered. Comparisons were then made to 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.

The same reference locations throughout the watershed were used from before and are shown in Figure 10.7.

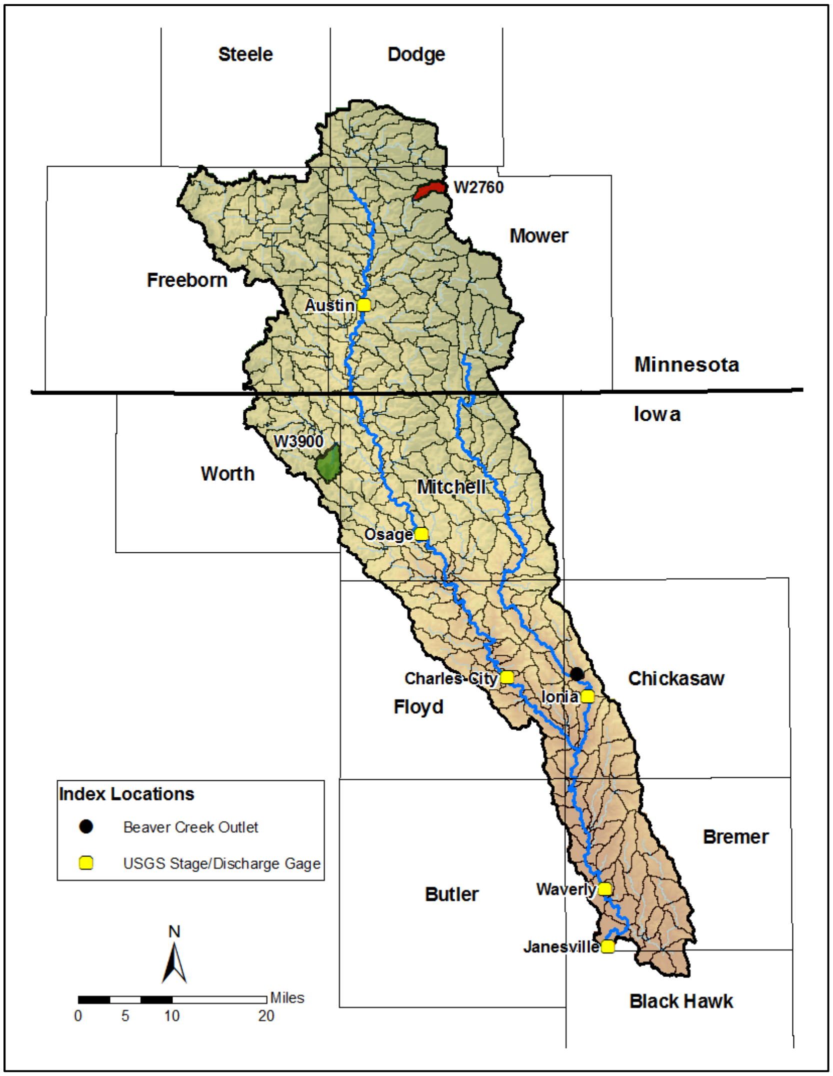

Figure 10.7. Reference locations for comparing watershed improvement scenarios to current conditions.

10.3.1 Small Pond Scenario

10.3.1.1 Wet Ponds

Figure 10.8 shows the peak discharge reductions at the seven index locations for the small pond scenario with wet ponds (3 foot emergency spillway elevation). In this scenario, each prototype pond provides 10.9 acre-feet of flood storage, resulting in a total of 4,069 acre-feet of flood storage for the entire watershed. For the small ponds, the percent reduction is greatest for the 10-year return period flood, and decreases for larger floods; the small ponds fill rapidly for large floods, at which point little attenuation in the flood peak is achieved. As noted above, the peak reduction effect varies with drainage area. It is typically larger for small drainage areas, where the location is closer to the headwater ponds, and decreases in the downstream direction. The one exception is the Little Cedar River at Ionia, which has the lowest proportion of upstream drainage area controlled by ponds, and a very low peak reduction effect. For smaller upstream locations, the peak reduction range is much larger; at Austin it varies from about 9% (10-year event) to 4% (100-year event), whereas at the downstream-most location of Janesville, it is near 3% for nearly all the simulated flood events.

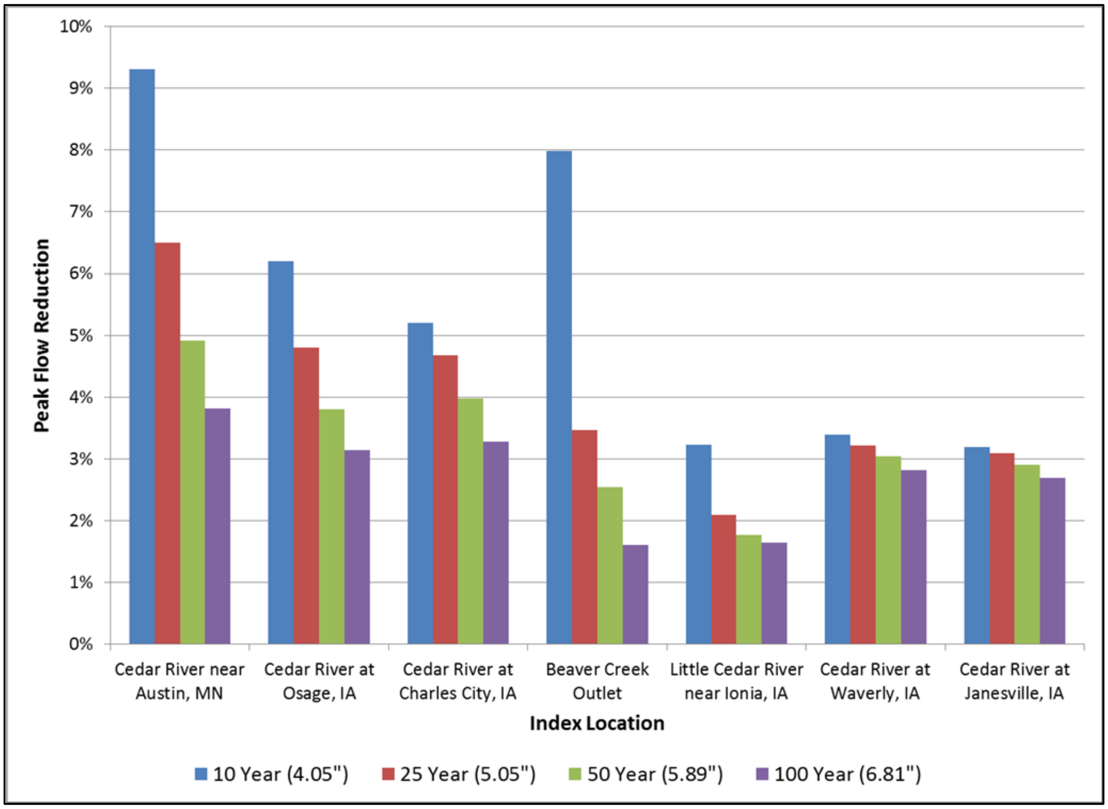

Figure 10.8. Peak discharge reductions for the small pond scenario with wet ponds (3 foot emergency spillway elevation). Percent reductions in peak flow are shown at seven index locations moving from upstream (left) to downstream (right) for four different 24 hour design storms (4.05-6.81 inches of rain in 24 hours).

10.3.1.2 Dry Ponds

Each prototype dry pond provides an additional 2.76 acre-feet of flood storage, a 25% increase from the wet pond alternative. This adds 1,028 acre-feet of flood storage to the watershed that increases the total amount to 5,097 acre-feet. Figure 10.9 compares the performance of the wet and dry pond alternatives for reducing flood peaks for the small pond scenario. For each index location, the anomaly in the peak flow reduction between the wet and dry pond alternative is plotted. Each anomaly was calculated by subtracting the percent reduction in peak flow for the wet pond alternative (those shown in Figure 10.8) from the percent reduction in peak flow for the dry pond alternative. A positive anomaly indicates a greater peak flow reduction was achieved with dry ponds.

For instance, the anomaly for the 10-year, 24-hour storm at Austin is approximately 1.3%, meaning the peak flow reduction at Austin was about 10.6% with dry ponds and 9.3% with wet ponds.

Although small, the dry ponds do perform slightly better than the wet ponds at reducing peak discharges at the seven index locations. The dry pond alternative provides about a 1% increase in the peak flow reduction at the seven index locations for the 10-year event, which decreases to less than a 0.5% increase in the peak flow reduction for the 100-year event. Flood storage provided by each prototype dry pond is 25% greater (2.76 acre-feet) than the wet pond alternative, while total discharge at the emergency spillway elevation only increases by about 3% (0.3 cfs) due to the smaller 2-inch pipe outlet; in other words, the additional flood storage benefit outweighs the increase in outflow discharge.

For a given rain storm, the anomaly in the peak discharge reduction decreases moving downstream. Moving downstream, generally a lesser percent of the upstream area is drained by ponds and the added flood storage provided by the dry ponds has minimal effect on increasing the peak discharge reduction. The anomaly at a given location also decreases with increasing storm size. Once the flood storage in the pond is utilized, water rapidly passes over the emergency spillway and the wet and dry ponds perform essentially the same.

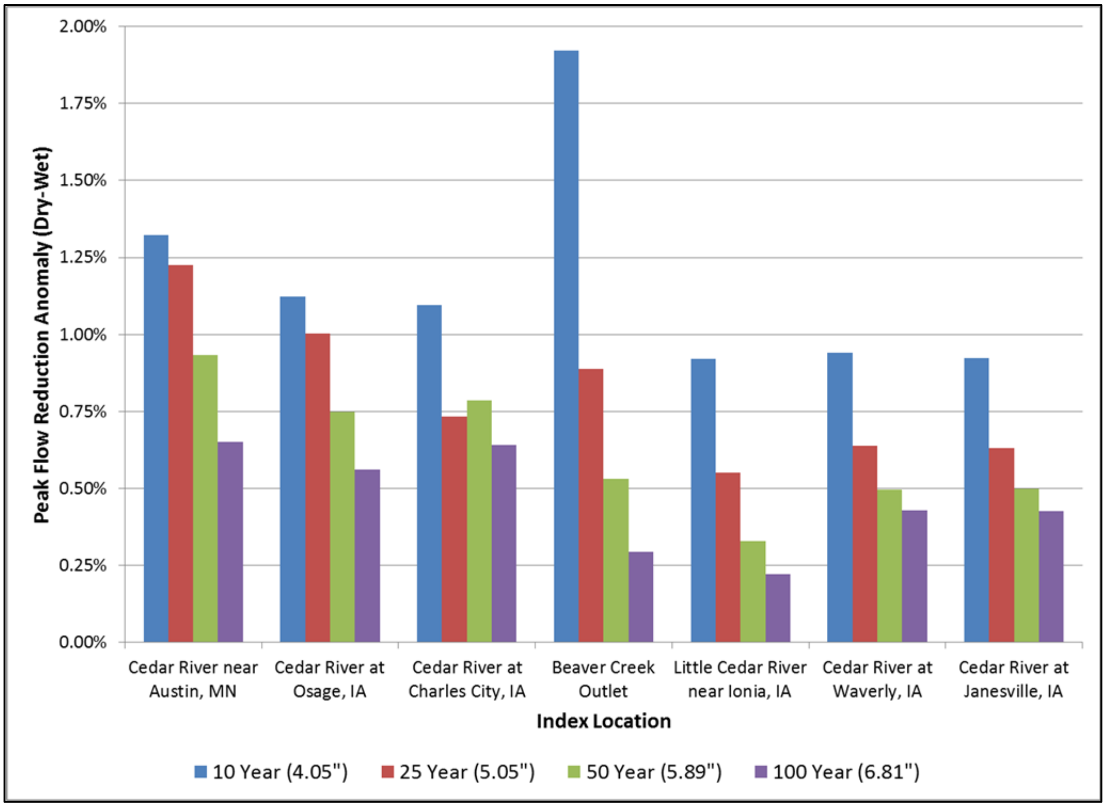

Figure 10.9. Peak discharge reduction anomalies between wet and dry pond alternatives for the small pond scenario. Anomalies were calculated as the difference between the percent reduction in peak flow for the dry and wet pond alternatives; positive anomalies indicate a greater reduction was achieved with dry ponds and negative anomalies indicate a greater reduction was achieved with wet ponds.

10.3.2 Medium Pond Scenario

10.3.2.1 Wet Ponds

Figure 10.10 compares the simulated flood hydrographs for the current no pond condition (Baseline) to those with medium-sized wet ponds (Scenario) for the 50-year return period, 24-hour design storm (5.89 inches of rain in 24 hours). The smallest drainage area shown, Subbasin W2760 located in northern Mower County, Minnesota (panel a), has a drainage area of 4.2 mi^2. Two prototype ponds were placed upstream. As a result, the peak discharge is reduced by 10%. The operation of the ponds is most evident at this location. Initially water exits the subbasin without significant delay. Then the rise in the discharge is halted, as water is stored in the ponds. After water begins flowing over the emergency spillway, discharge increases rapidly again. Still there is sufficient flood storage available to reduce the peak discharge from 317 cfs (with no ponds) to 286 cfs (with ponds).

At Austin, where 102 prototype ponds were placed upstream, the peak flow reduction is the maximum observed at the seven sites (13%). Austin also has the maximum upstream area controlled by ponds (25.8%). Even though the area controlled is very similar downstream, the peak flow reduction is not; the peak flow reduction gradually decreases to its minimum (5%) at the downstream-most site at Janesville. In other words, the flood reduction effect is largest at locations closer to the headwater ponds for a 50-year return period design event.

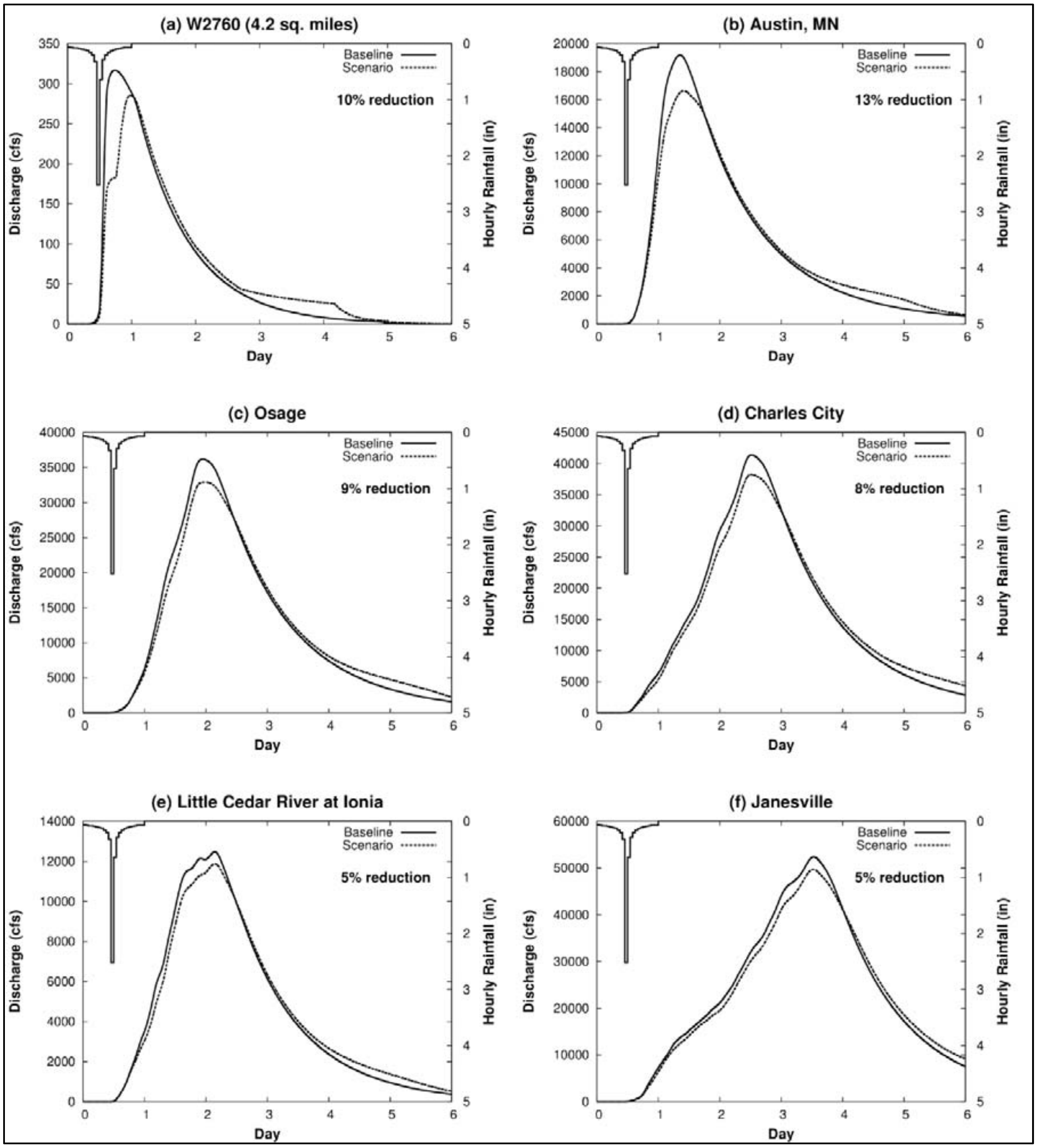

Figure 10.10. Comparison of hydrographs at several locations with and without medium-sized wet ponds for the 50 year – 24 hour storm (5.89 inches of rain in 24 hours). For the hydrographs shown, peak flow reductions range from 5-13%.

Figure 10.11 shows the peak discharge reductions for the medium-sized pond scenario with wet ponds (5 foot emergency spillway elevation). In this scenario, each prototype pond provides 26.8 acre-feet of flood storage, resulting in a total of 9,949 acre-feet of flood storage for the entire watershed. As one would expect with more storage, the peak reduction effect is significantly larger for the medium-sized ponds. Still, while the storage volume increases by about 2.4 times, the increases in peak reduction are less than that. Comparing the effects at a location for different flood events, the percent reduction is larger for the smaller flood events, and decreases for larger floods. However, because the flood storage in the watershed is not fully exhausted for the 10-year design flood, the peak reductions at Austin and the Beaver Creek outlet increase slightly for the 25-year design flood. For even larger flood events, all the flood storage is utilized, and the peak reduction decreases. Also as seen before, the peak reduction tends to be greater nearer to the headwater ponds (smaller drainage areas), and decreases for larger drainage areas downstream (with Ionia again being the exception).

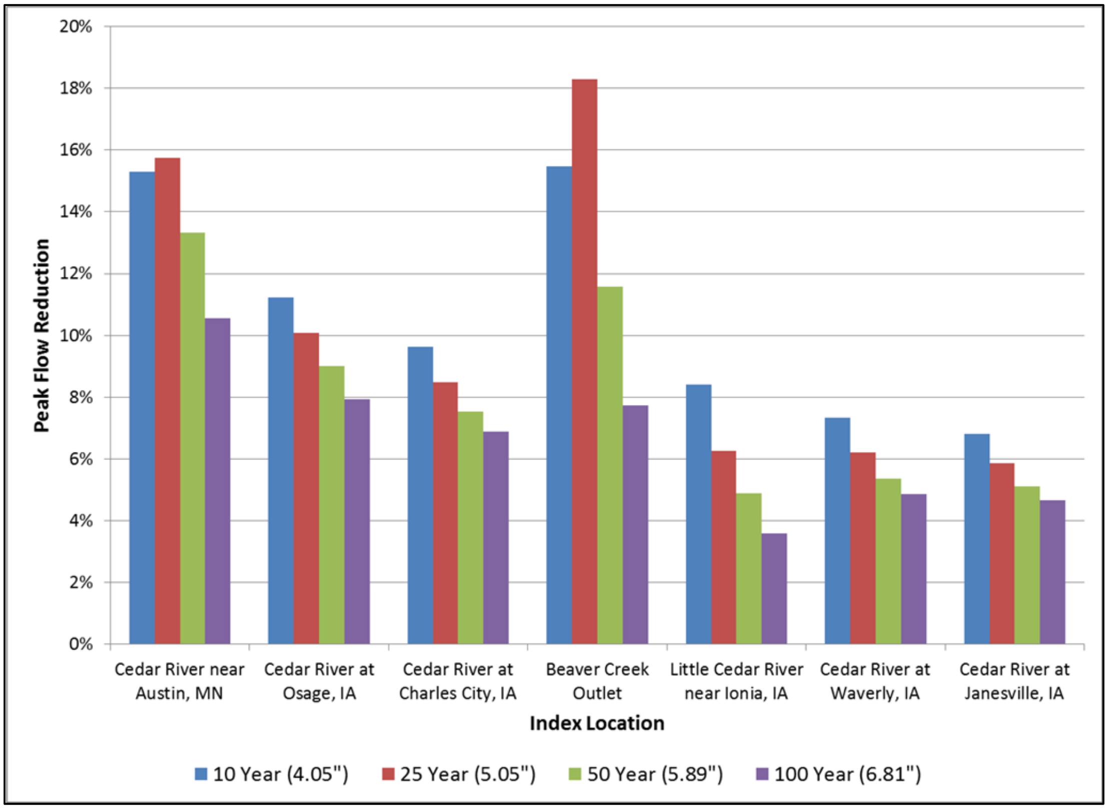

Figure 10.11. Peak discharge reductions for the medium-sized pond scenario with wet ponds (5 foot emergency spillway elevation). Percent reductions in peak flow are shown at seven index locations moving from upstream (left) to downstream (right) for four different 24 hour design storms (4.05-6.81 inches of rain in 24 hours).

10.3.2.2 Dry Ponds

Dry ponds for the medium-sized pond scenario increase the total flood storage in the watershed by 10% from 9,949 acre-feet to 10,977 acre-feet. Figure 10.12 compares the performance of wet and dry ponds for the medium-sized pond scenario. As with the small pond scenario, each anomaly was calculated by subtracting the percent reduction in peak flow for the wet pond alternative (those shown in Figure 10.11) from the percent reduction in peak flow for the dry pond alternative. Positive anomalies indicate a greater peak flow reduction was achieved with dry ponds and negative anomalies indicate a greater peak flow reduction was achieved with wet ponds. For the medium pond scenario, positive anomalies are typically smaller than for the small pond scenario at the seven index locations, indicating the dry ponds provide less additional flood reduction benefit for the medium-sized ponds. For the medium-sized pond scenario, the 2.76 acre-feet of additional flood storage per prototype dry pond increases the flood storage per pond by 10%, while total outflow discharge at the emergency spillway elevation increases by about 3% (0.4 cfs); because the percent of flood storage added by the dry ponds has decreased while the increase in discharge through the two pipe outlets has remained about the same, less additional flood reduction benefit is provided by dry ponds for the medium-sized pond scenario compared to the small pond scenario. For all index locations except the Beaver Creek outlet, the dry pond alternative provides less than a 1% increase in the peak flow reduction for all four design storms compared to wet ponds. At the outlet of Beaver Creek, wet ponds perform slightly better than dry ponds for the smaller 10- and 25-year events; however, the additional peak discharge reduction provided by the wet ponds is quite small (less than one cfs difference). For the larger 50- and 100-year events, the additional storage provided by the dry ponds results in slightly greater peak flow reductions than with wet ponds at the Beaver Creek outlet. Overall, however, the wet and dry ponds perform similarly for the medium-sized pond scenario with dry ponds generally providing slightly greater peak discharge reductions as expected.

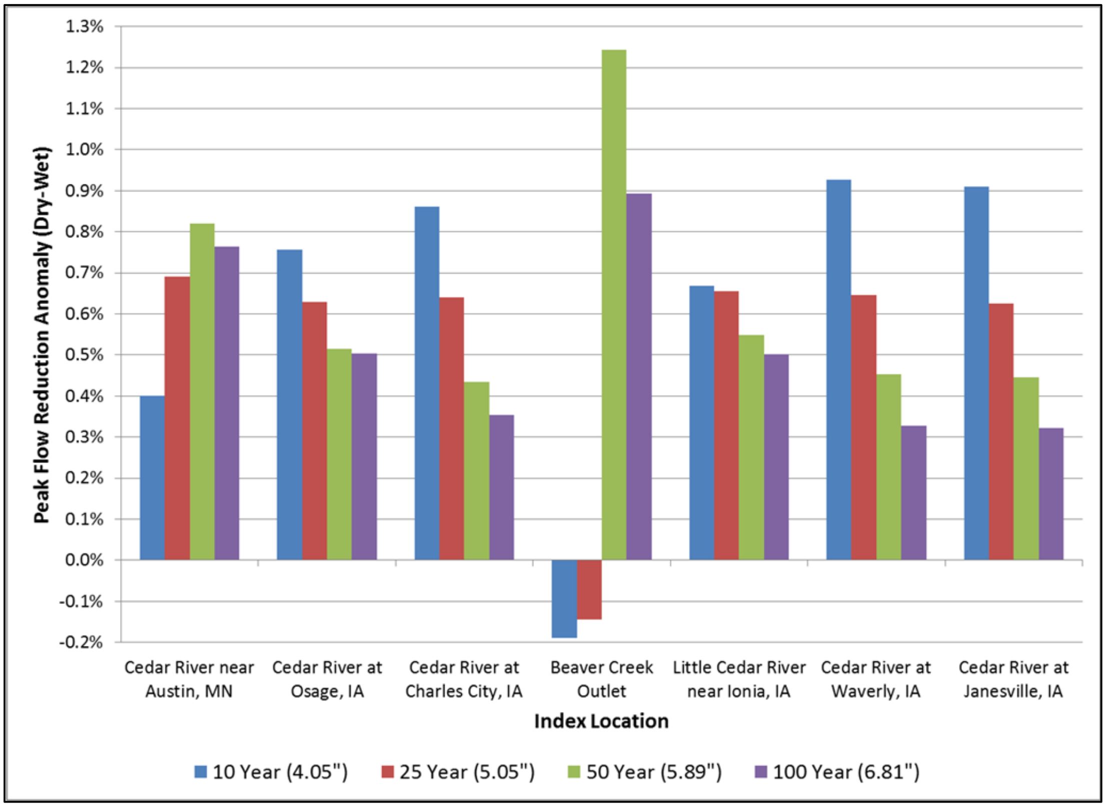

Figure 10.12. Peak discharge reduction anomalies between wet and dry pond alternatives for the medium-sized pond scenario. Anomalies were calculated as the difference between the percent reduction in peak flow for the dry and wet pond alternatives; positive anomalies indicate a greater reduction was achieved with dry ponds and negative anomalies indicate a greater reduction was achieved with wet ponds.

10.3.3 Large Pond Scenario

10.3.3.1 Wet Ponds

Figure 10.13 shows the peak discharge reductions for the large pond scenario with wet ponds (7 foot emergency spillway elevation). In this scenario, each pond provides 48.2 acre-feet of flood storage, resulting in a total of 17,930 acre-feet of flood storage for the entire watershed. With this additional flood storage, the peak reduction is again increased. Although the storage volume is about 1.8 times larger than with medium-sized ponds, the increase in peak reduction is much less than that. In a similar manner to medium-sized ponds, the flood storage is not fully exhausted for the smaller design floods. Hence, the peak reduction effect is at its maximum for the 25-year flood at most locations; for Austin the maximum is for the 50-year design flood, and for the Beaver Creek outlet it is actually greatest for the 100-year flood. As always, the peak reduction tends to be greater nearer to the headwater ponds (smaller drainage areas), and decreases for larger drainage areas downstream (with Ionia again being the exception).

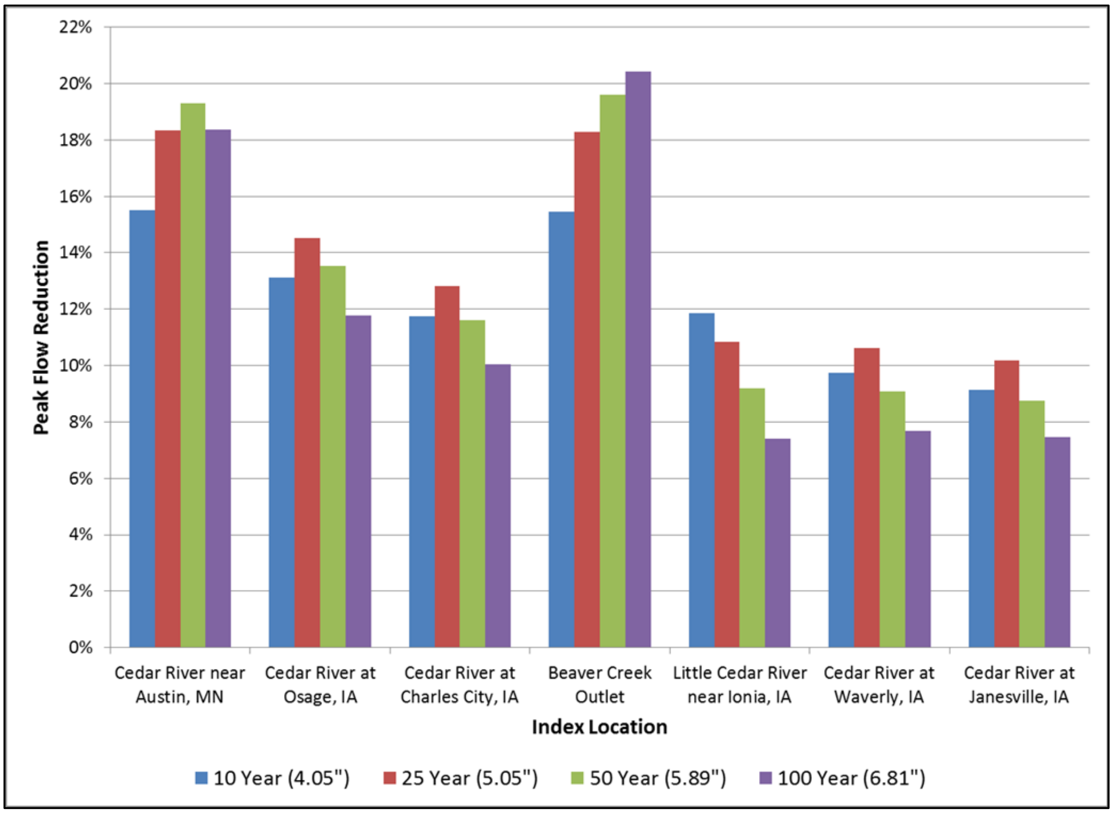

Figure 10.13. Peak discharge reductions for the large pond scenario with wet ponds (7 foot emergency spillway elevation). Percent reductions in peak flow are shown at seven index locations moving from upstream (left) to downstream (right) for four different 24 hour design storms (4.05-6.81 inches of rain in 24 hours).

10.3.3.2 Dry Ponds

Dry ponds for the large pond scenario increase the total flood storage in the watershed by nearly 6% from 17,930 acre-feet to 18,958 acre-feet. Figure 10.14 compares the performance of wet and dry ponds for the large pond scenario. For the large pond scenario, the wet and dry pond alternatives perform almost identically; the increase in the peak discharge reduction provided by dry ponds at any index location for the four design storms is less than 0.5%. This is due to the prototype dry pond providing only a 6% increase in flood storage (2.76 acre-feet), while total outflow discharge at the emergency spillway elevation increases by about 3% (0.4 cfs). The added flood storage benefit of the dry ponds for the large pond scenario is even less than for the medium-sized pond scenario, so peak flow reductions throughout the basin are also less. As with the medium-sized pond scenario, wet ponds provide a slightly greater peak flow reduction at the Beaver Creek outlet for all but the 100-year event, but the additional reduction is very small. Although the dry ponds release less water than the wet ponds upstream of the Beaver Creek outlet, differences in timing result in a slightly greater discharge being simulated at Beaver Creek for the dry pond alternative. Again, the anomalies are small, and it would be difficult to discern any noticeable difference in practice between wet and dry ponds for the large pond scenario.

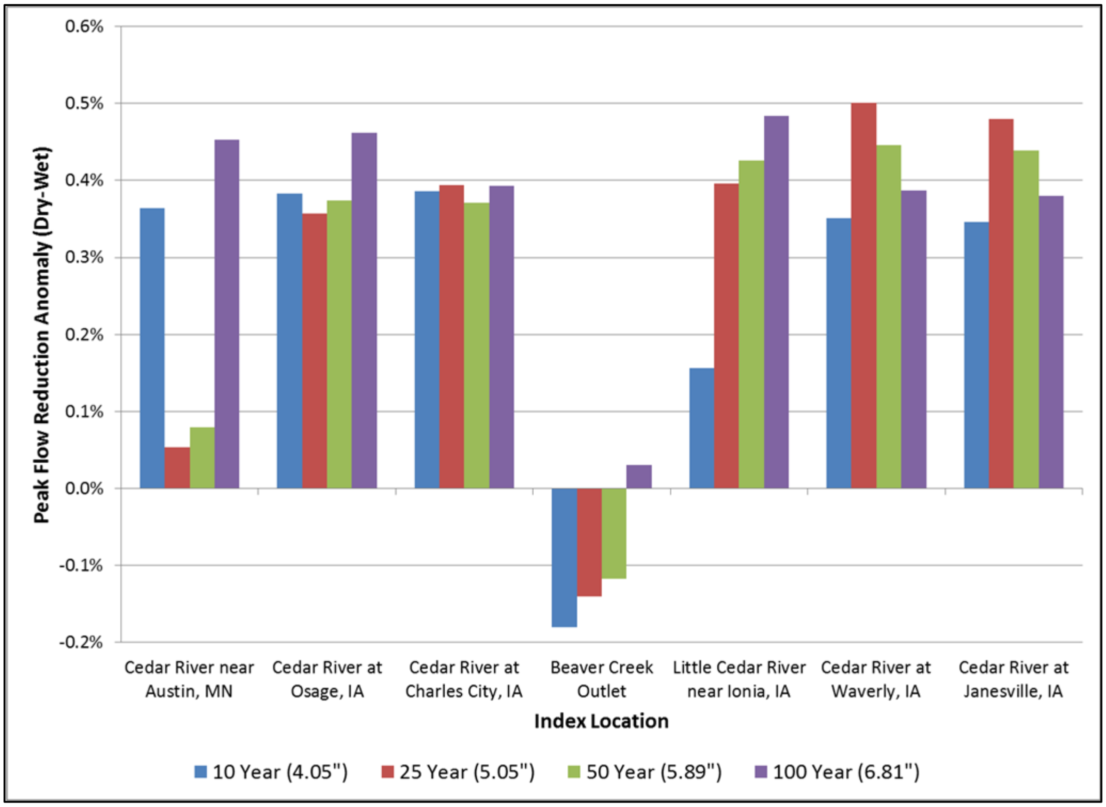

Figure 10.14. Peak discharge reduction anomalies between wet and dry pond alternatives for the large pond scenario. Anomalies were calculated as the difference between the percent reduction in peak flow for the dry and wet pond alternatives; positive anomalies indicate a greater reduction was achieved with dry ponds and negative anomalies indicate a greater reduction was achieved with wet ponds.

10.3.4 Summary of Pond Performance Characteristics

To illustrate how effectively the wet and dry pond alternatives utilize their storage in the simulated flood events, the pond performance characteristics are summarized in Tables A-1 through A-3 of Appendix A for the 10-, 50-, and 100-year return period, 24-hour design storms. In addition to showing pond performance – either the percent of flood storage utilized for each aggregated pond or the water depth flowing over the emergency spillway for larger events – Figures A-5 through A-10 of Appendix A also show the peak discharge reductions with wet ponds or dry ponds for each pond scenario at the seven index locations for the 10-, 50-, and 100-year, 24-hour design storms.

For the 10-year return period design flood (Table A-1 and Figures A-5 and A-6), the water level reaches the emergency spillway elevation for all 121 of the wet and dry ponds for the small pond scenario (3 foot emergency spillway elevation). For the medium-sized pond scenario (5 foot emergency spillway elevation), the water level reaches the emergency spillway for 92 (76%) of the wet ponds and 83 (69%) of the dry ponds. For the large pond scenario (7 foot emergency spillway), the water level reaches the emergency spillway for 15 (12%) of the wet ponds and only 9 (7%) of the dry ponds. As a result, less of the flood storage is utilized using dry ponds compared to wet ponds for the small pond scenario. Depending on the pond size, wet ponds exhaust 70-100% of the available flood storage while dry ponds exhaust 68-100% of the available flood storage for the 10-year event.

Pond performance characteristics and peak flow reductions at the seven index locations for the 25-year event are summarized in Table 10.4 and Figures 10.15-10.16 below. The 25-year event illustrates the effectiveness of each pond size for temporarily storing runoff. The water level reaches the emergency spillway elevation for all 121 small and medium-sized ponds (wet and dry); for the large pond scenario, 106 (88%) wet ponds and 99 (82%) dry ponds activate the emergency spillway. The wet and dry pond performance characteristics are more similar for the medium-sized pond scenario compared to the small pond scenario for the 25-year event.

Table 10.4. Pond performance characteristics for the 25-year, 24-hour design storm (5.05 inches of rain in 24 hours).

	Wet			*Dry*		
Pond Scenario	Ponds Activating Emergency Spillway (121 total)	Ponds with Peak Water Level One Foot Above Emergency Spillway	Percent of Total Flood Storage Utilized	Ponds Activating Emergency Spillway (121 total)	Ponds with Peak Water Level One Foot Above Emergency Spillway	Percent of Total Flood Storage Utilized
Small	121	26	100.0%	121	25	100.0%
Medium	121	9	100.0%	121	7	100.0%
Large	106	3	99.1%	99	2	98.2%

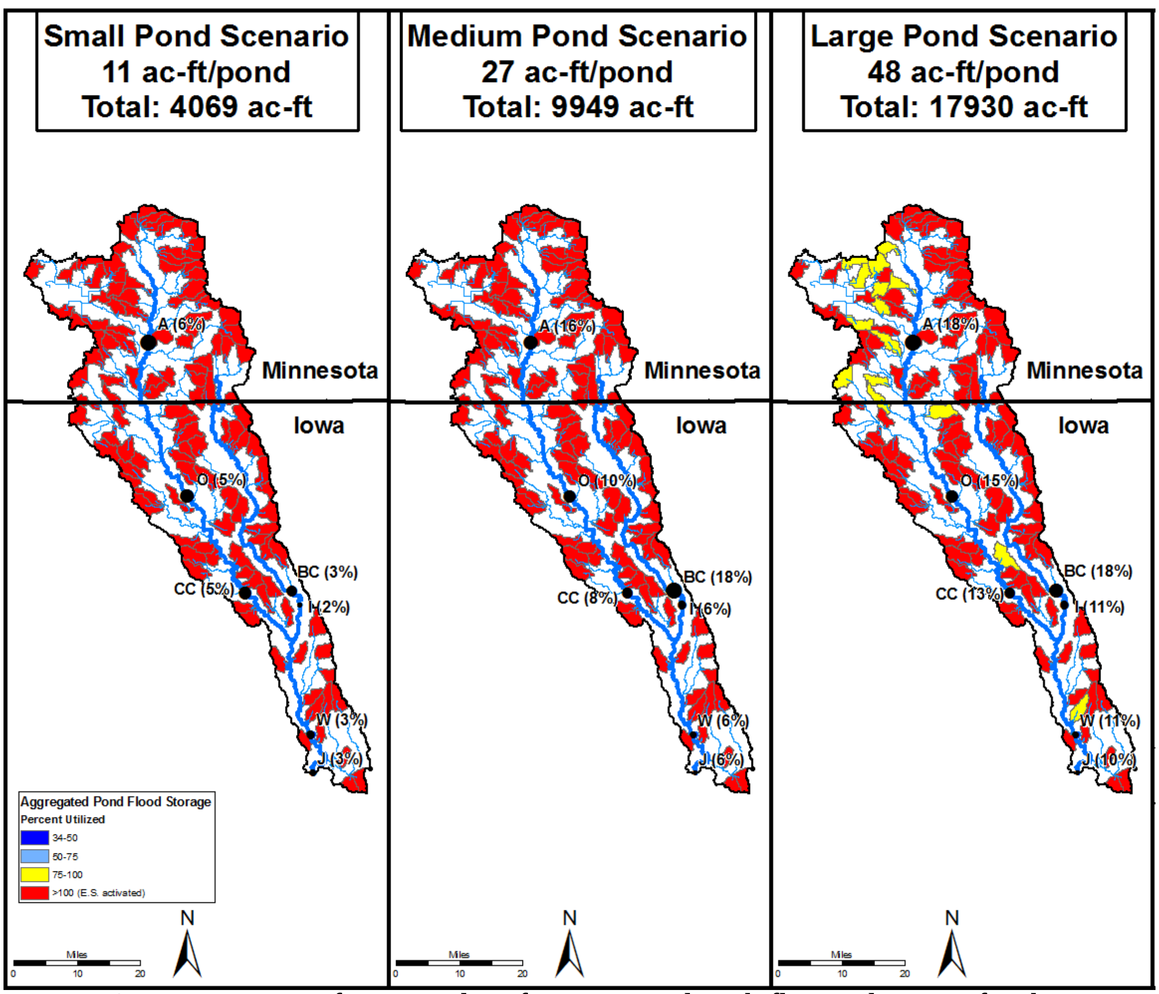

Figure 10.15. Summary of wet pond performance and peak flow reductions for the 25-year, 24-hour design storm (5.05 inches of rain in 24 hours).

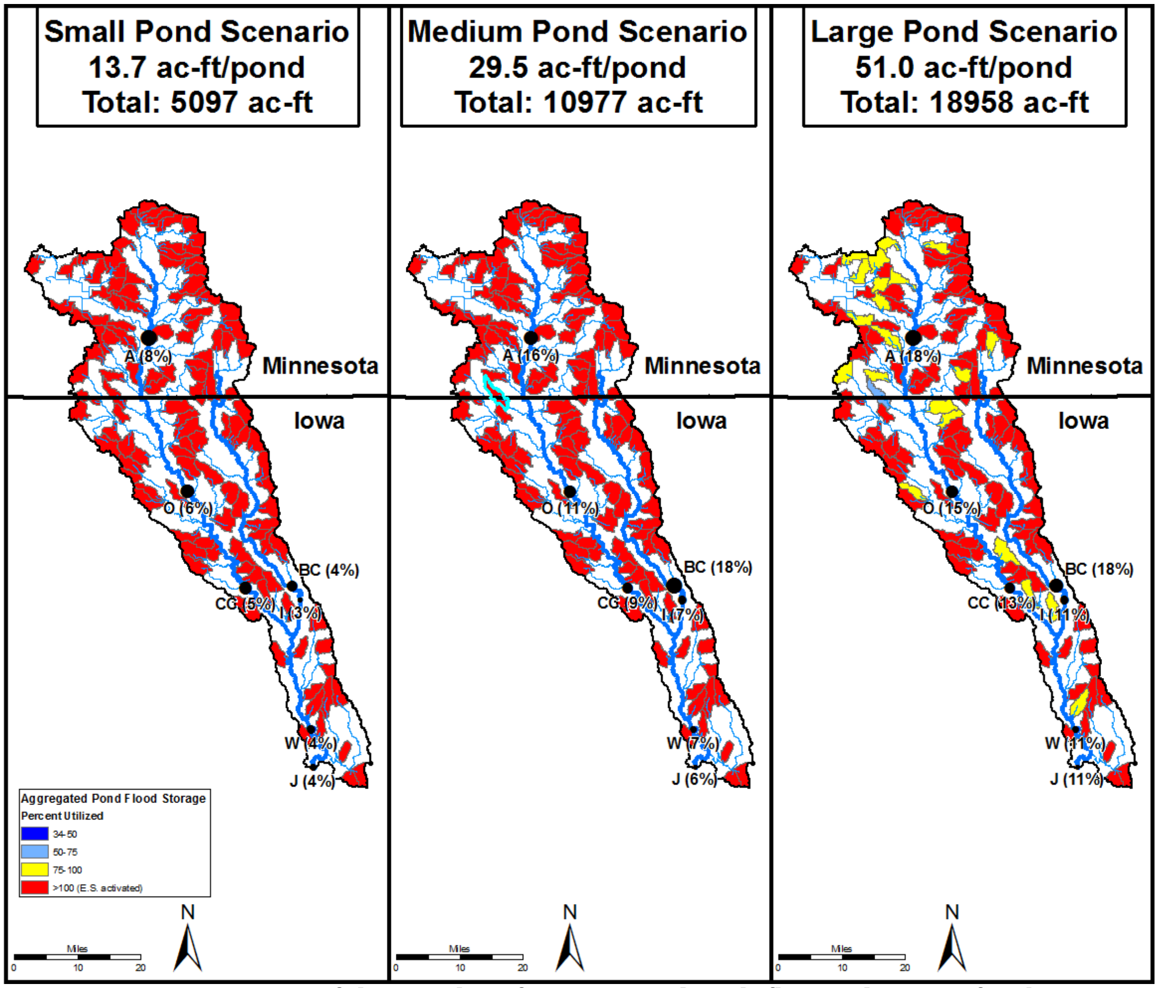

Figure 10.16. Summary of dry pond performance and peak flow reductions for the 25-year, 24-hour design storm (5.05 inches of rain in 24 hours).

By the 50-year design flood (Table A-2 and Figures A-7 and A-8), the water level reaches the emergency spillway for all the wet and dry ponds for the large pond scenario. For the 100-year design flood (Table A-3 and Figures A-9 and A-10), no overtopping of the dam occurs in any scenario, but the water level does reach at least one foot above the emergency spillway in a greater number of wet ponds than dry ponds. The number of wet ponds with a water level at least one foot above the emergency spillway ranges from 66 for the large pond scenario (57 for the dry pond alternative) to 117 for the small pond scenario (117 for dry pond alternative as well).

10.4 Chapter Summary

This chapter evaluates the impacts flood storage could have on mitigating the effects of high runoff by reducing flood peak discharges in the Upper Cedar River Watershed. Unlike the previous analyses considered that reduced the amount of runoff for a given rain storm, ponds store floodwater temporarily and release it at a lower rate. As a result, flood peak discharges are reduced downstream of the storage ponds.

A prototype pond design was developed to describe the storage capacity from likely watershed topography and included hydraulic design features common to previously constructed NRCS projects. To be clear, the typical pond design is not site specific, and while it incorporates several NRCS design recommendations, none of the ponds meet standard NRCS design criteria. In order for this to be achieved, a site specific design would be needed for each pond location, which was not the purpose of this exercise.

For the hypothetical distributed flood storage scenario, 121 aggregated ponds providing flood storage were placed in the headwater regions of the Upper Cedar River Watershed. The ponds drain approximately 23% of the watershed. Wet and dry ponds were considered for three different pond sizes (small, medium, and large); the wet ponds provide 4,069-17,930 acre-feet of total flood storage, while dry ponds provide an additional 1,028 acre-feet of flood storage for each pond size. For the upstream areas draining to a pond, this is equivalent to an added storage depth of 0.2 inches (small ponds) to 0.9 inches (large ponds) for the wet ponds and 0.25 inches (small ponds) to 0.95 inches (large ponds) for the dry ponds. For the watershed as a whole, the wet ponds provide 0.05-0.2 inches of added storage depth and the dry ponds provide 0.06-0.21 inches of added storage depth. To put this in perspective, Coralville Reservoir north of Iowa City drains 3,084 mi^2 and provides 421,000 acre-feet of flood storage, equivalent to an added storage depth of 2.56 inches over the upstream area (*Coralville Lake*, 2012). Clearly, Coralville Lake provides a much greater amount of flood storage on a per area basis than the system of distributed storage ponds developed for the Upper Cedar. The added storage depths of the distributed storage ponds are also much less than the storage depth added by improved infiltration in the previous scenarios, so the peak flow reductions resulting from this distributed storage scenario tend to be less.

The wet and dry pond alternatives provided similar downstream peak flow reduction benefits. In general, dry ponds performed slightly better than wet ponds at reducing peak discharges downstream since they provide slightly more flood storage and throttle down the discharge leaving the pond until the water level reaches the primary spillway (due to the smaller pipe located on the pond bottom). In general, downstream peak flow reductions were within 1-2% (or less) of each other for the wet and dry pond alternatives, and the two pond alternatives behaved more similarly when larger pond sizes and flood events were considered. For wet ponds, peak flow reductions at the seven index locations for all four design storms ranged from 2-9% for the small pond scenario, 4-18% for the medium-sized pond scenario, and 7-20% for the large pond scenario.

Peak flow reductions tended to decrease moving downstream, verifying that the greatest reductions are achieved directly downstream of the ponds and at locations where a greater percent of the upstream area is drained by ponds. Peak flow reductions also decreased with increasing storm size. Once all the flood storage is utilized, water passes over the emergency spillway at close to the uncontrolled rate and minimal attenuation or delay in the flood peak is observed. While this analysis demonstrated the ability of storage ponds to reduce flood peak discharges downstream, it is also important to recognize that using distributed storage practices as a prominent flood mitigation strategy in the Upper Cedar may be difficult to achieve due to the flat topography of the watershed that makes siting ponds more difficult.

CHAPTER 11: MITIGATING THE EFFECTS OF HIGH RUNOFF WITH INCREASED INFILTRATION AND FLOOD STORAGE

11.1 Introduction

In this chapter, a combination of flood mitigation strategies are considered to evaluate their effect on reducing flood peak discharges. The prior scenarios evaluated the flood reduction benefit of increased infiltration (Chapters 8-9) or added storage (Chapter 10) independent of one another. Both strategies reduce peak discharges in different ways; enhanced infiltration, achieved through land use changes or improved soil conditions, is a runoff reduction strategy while distributed storage does not alter the total amount of runoff but stores the floodwater temporarily in ponds and releases it at a lower rate. Projects to be constructed in the Upper Cedar are likely to rely on both flood reduction strategies, so gaining a sense for the potential flood reduction benefit of both together is important.

In this chapter, flood reduction resulting from both storage ponds and improved agricultural conditions due to cover crops is examined. Both strategies are applied in a more limited capacity than considered previously to represent a more realistic and feasible implementation scheme. Once again, this is a hypothetical and simplified example but does provide an indication of the types of peak flow reductions that may be achievable for a specific combination of flood mitigation practices.

11.2 Method

Using information from the prior scenarios, the HMS model was used to simulate the effect both storage ponds and cover crops could have on reducing flood peak discharges.

11.2.1 Storage Pond Implementation Scheme

The large, wet pond design described in Chapter 10 was selected for this analysis so the maximum peak reductions achievable from both flood storage and a given cover crop implementation scheme could be determined. This prototype pond design has the principal spillway set three feet above the pond bottom and the emergency spillway set seven feet above the primary spillway. Each prototype large, wet pond provides 48.2 acre-feet of flood storage.

The hypothetical, large wet ponds were distributed in headwater areas with the greatest amount of topographic relief to represent a more realistic implementation scheme. Of the 121 aggregated ponds considered in Chapter 10, half were kept in the steepest headwater subbasins. Figure 11.1 shows the 60 headwater subbasins selected for pond placement for the blended scenario. Not surprising, most of the aggregated ponds are located in the Iowan Surface geographic landform region, which has more relief than the Des Moines Lobe region. The 60 aggregated ponds drain 185 mi^2 (11% of the watershed), about half the drainage area of all 121 ponds.

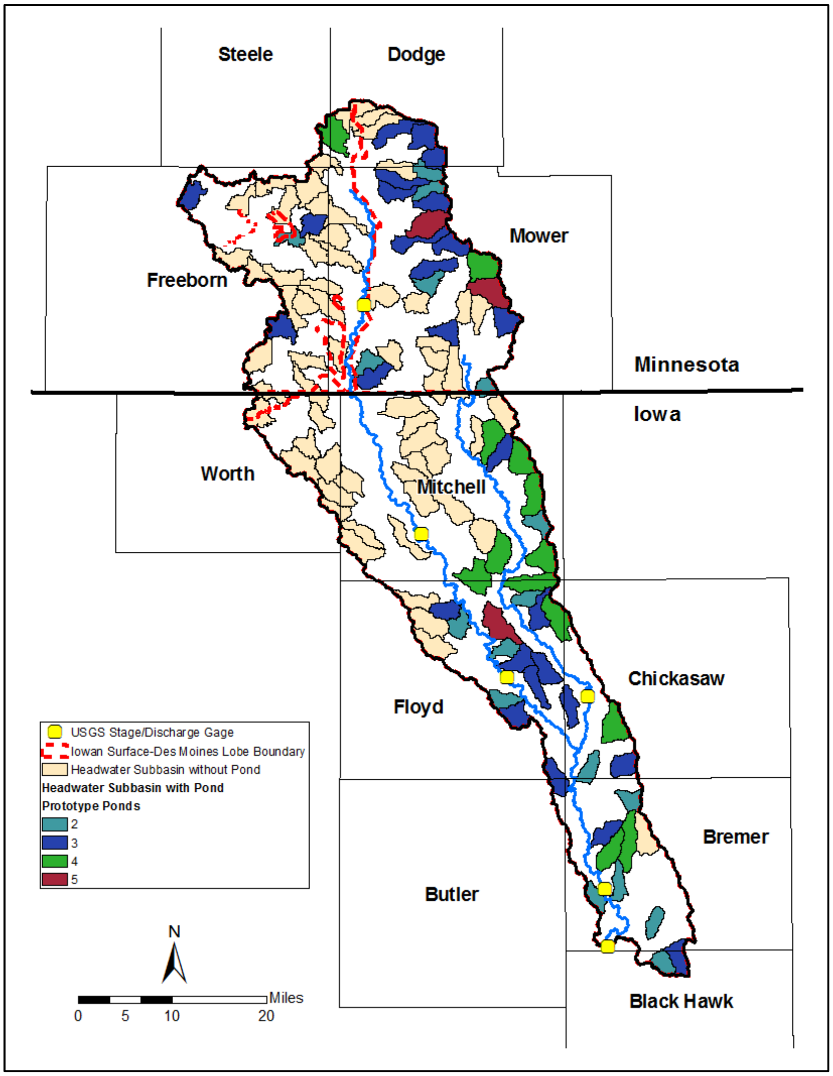

Figure 11.1. Headwater subbasins selected for pond placement for the blended hypothetical scenario. Sixty ponds were placed in the steepest headwater subbasins, which tend to be located in the Iowan Surface region.

The pond characteristics upstream of the seven index locations are characterized in Table 11.1. The percentage of the upstream area controlled by ponds is more variable than for all 121 ponds; it ranges from 8.4% at Charles City to a maximum of 24.9% at the Beaver Creek outlet. The percentage of the upstream area controlled by ponds generally decreases moving downstream, as expected. However, a slight increase in the percent controlled is seen at Waverly and Janesville because the southern part of the watershed is generally steeper than the northern part, so more ponds were retained in this region.

Table 11.1. Summary of pond characteristics upstream of the seven index locations for the blended scenario.

Location	Drainage Area (mi^2)	Number of Aggregated Ponds Upstream	Number of Prototype Ponds Upstream	Drainage Area Controlled by Ponds (mi^2)	Percent Controlled
Cedar River at Austin	393	16	48	48.4	12.3
Cedar River at Osage	833	23	70	71.4	8.6
Cedar River at Charles City	1069	29	89	90.3	8.4
Beaver Creek Outlet	17	1	4	4.3	24.9
Little Cedar River at Ionia	294	13	43	43.8	14.9
Cedar River at Waverly	1550	54	167	169.2	10.9
Cedar River at Janesville	1663	60	182	185.4	11.1

Table 11.2 summarizes the flood storage available upstream of the seven index locations. The 60 aggregated ponds add 8,772 acre-feet of storage to the watershed, which is about 49% of the total storage added by all 121 ponds. The 8,772 acre-feet of storage provided by the 60 ponds represents a uniform depth of about 0.9 inches over the upstream area drained by the ponds. As shown in Table 11.2, these average storage depths are relatively consistent at the seven locations.

Table 11.2. Summary of the flood storage available upstream of the seven index locations for the blended scenario utilizing large, wet ponds.

Location	Percent of Total Flood Storage in Watershed	Total Flood Storage Upstream (acre-feet)	Total Flood Storage Upstream, Uniform Depth (in)
Cedar River at Austin	26	2313	0.90
Cedar River at Osage	38	3374	0.89
Cedar River at Charles City	49	4290	0.89
Beaver Creek Outlet	2	193	0.83
Little Cedar River at Ionia	24	2073	0.89
Cedar River at Waverly	92	8049	0.89
Cedar River at Janesville	100	8772	0.89

11.2.2 Cover Crop Implementation Scheme

In addition to a select number of storage ponds placed in the steepest areas of the watershed, agricultural conditions were improved to reflect planting cover crops, though to a lesser extent than considered in Chapter 8. Using cover crops as a farming conservation practice was assumed to take place in 50% of agricultural areas, on average, rather than all (100%) agricultural areas in the watershed assumed in Chapter 8.

To reflect improved agricultural conditions due to planting cover crops, changes were made to the NRCS CN for each subbasin. To represent cover crops being implemented on 50% of the agricultural land in the watershed in HMS, each subbasin was assigned a CN corresponding to the average of the CNs from the baseline simulation (existing agricultural landscape) and the cover crop simulation of Chapter 8 (all agricultural area improved as a result of cover crops). Subbasin CNs were reduced by 1.8% on average from the baseline condition to reflect 50% of the agricultural land implementing cover crops compared to a 3.6% average CN reduction to reflect 100% of the agricultural land implementing cover crops.

The same reference locations throughout the watershed were used from before to compare the baseline simulation to the blended scenario and are shown in Figure 11.2.

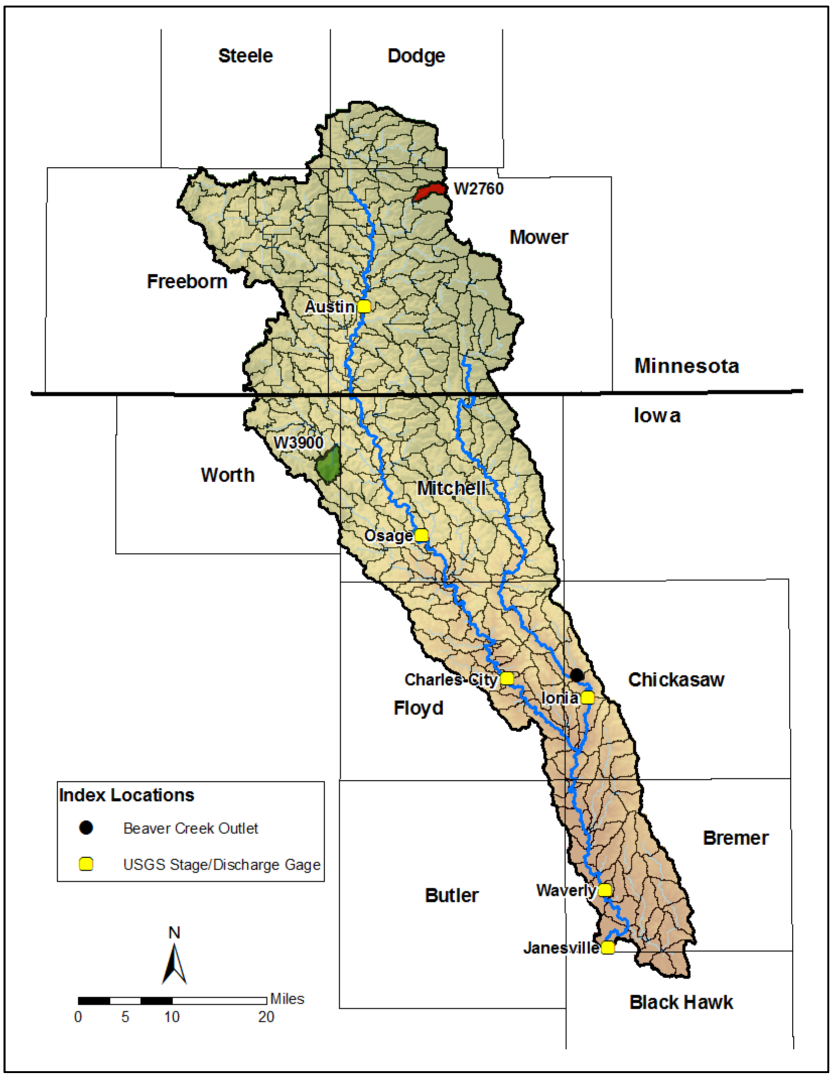

Figure 11.2. Reference locations for comparing watershed improvement scenarios to current conditions.

11.3 Results

The blended scenario – where half the agricultural area in the watershed is improved as a result of cover crops and half (60) the number of large, wet storage ponds are implemented – results in less flood reduction than either more extensive flood mitigation strategy (Chapters 8 and 10) . As expected, improving 50% of agricultural land, on average, through cover crop planting improves infiltration by approximately half as much as when all agricultural land is improved by planting cover crops; on average, 0.1-0.2 inches of additional infiltration occur for the four design storms for the blended scenario compared to 0.2-0.3 inches when all existing agricultural areas are improved by planting cover crops.

Figure 11.3 compares the simulated flood hydrographs for the current agricultural landscape (Baseline) to those for the blended scenario (Scenario) for the 50-year return period, 24-hour design storm (5.89 inches of rain in 24 hours). The smallest drainage area shown, Subbasin W2760 located in northern Mower County, Minnesota (panel A), is 4.2 mi^2, has two prototype ponds placed upstream, and is nearly 90% agriculture by area. About 0.2 additional inches of rainfall would infiltrate if 50% of the agricultural land in this subbasin were improved by planting cover crops during the dormant season. The flood storage provided by the large pond combined with the enhanced infiltration due to cover crops reduces the peak discharge leaving this subbasin by 30%. This is greater than the peak flow reduction achieved through either 100% cover crops (10% reduction) or distributed storage with large wet ponds (24% reduction). Moving downstream, the peak flow reduction drastically decreases. At Janesville, the peak flow reduction is 7% which is less than was observed for the hypothetical scenarios involving 121 large, wet ponds or improvement of all agricultural areas due to cover crops; the peak flow reduction at Janesville for both these scenarios was around 9%.

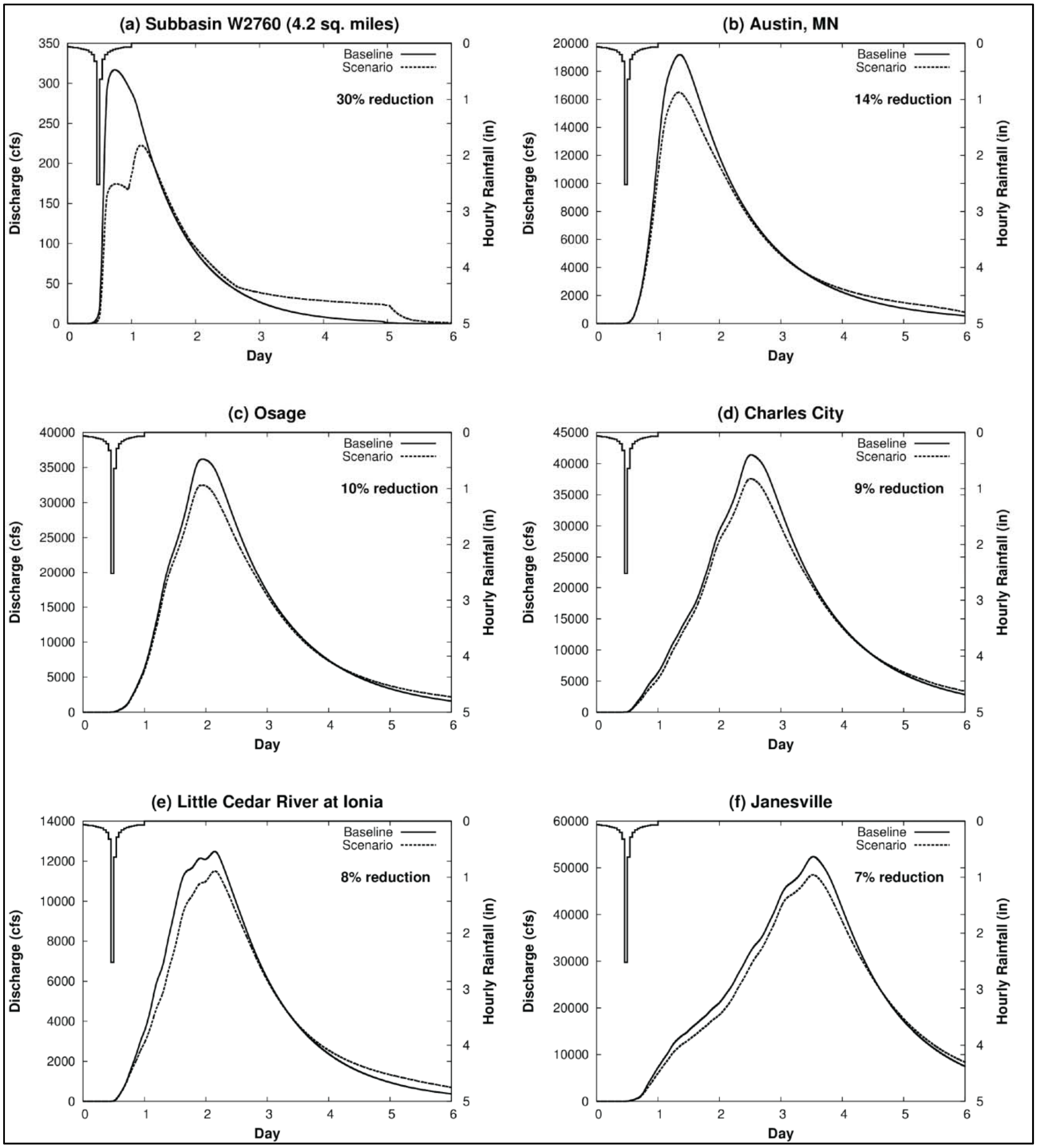

Figure 11.3. Comparison of hydrographs at several locations for the blended scenario for the 50 year – 24 hour storm (5.89 inches of rain in 24 hours). For the hydrographs shown, peak flow reductions range from 7-30%.

Figure 11.4 shows the percent reductions in peak discharge resulting from the blended scenario at the seven index locations for the four design storms. Peak flow reductions at the seven index locations range from 6-24%; excluding the Beaver Creek outlet, peak flow reductions are less than 15%. Flood stages were reduced by 0.5-1.5 feet. The peak flow reductions at the index locations are similar but generally less than either idealized hypothetical scenario; for the large pond scenario (121 ponds), peak flow reductions range from 7-20% and for the 100% cover crop scenario, reductions were between 8 and 12%.

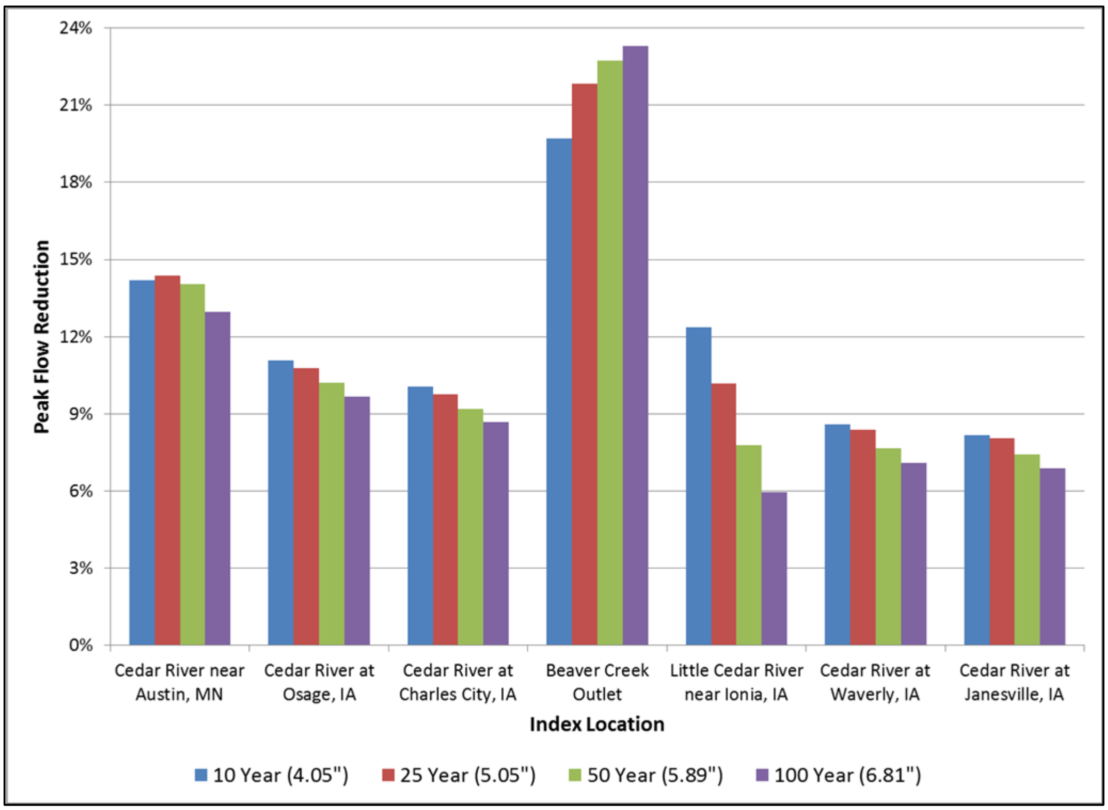

Figure 11.4. Peak discharge reductions for the blended scenario. Percent reductions in peak flow are shown at seven index locations moving from upstream (left) to downstream (right) for four different 24 hour design storms (4.05-6.81 inches of rain in 24 hours).

The large, wet pond performance characteristics for the four design storms are summarized in Table 11.3. Figures A-11 and A-12 of Appendix A also show the pond performance along with the peak discharge reductions estimated at the seven index locations for the four design storms. The proportion of ponds activating their emergency spillways, having a water depth of at least one foot over the emergency spillway, and the percent of total flood storage utilized are similar to the 121 large, wet ponds described in Chapter 10. For the 10-year return period design flood, only four of the 60 ponds (7%) activate their emergency spillway and 68% of the flood storage is utilized (compared to 70% for the 121 large, wet ponds). For the 25-year design flood, the water level reaches the emergency spillway for 54 (90%) of the ponds and 99% of the flood storage is utilized (compared to 99% for the 121 large, wet ponds). By the 50-year design flood, the water level reaches the emergency spillway for all ponds and all (100%) of the flood storage is utilized. For the 100-year design flood, no overtopping of the dam occurs for any pond, but the water level does reach at least one foot above the emergency spillway for 33 (55%) of the ponds.

Table 11.3. Summary of large, wet pond performance characteristics for the blended scenario for the 10-, 25-, 50-, and 100-year, 24-hour design storms (4.05-6.81 inches of rain in 24 hours).

Design Storm	Ponds Activating Emergency Spillway (60 total)	Ponds with Peak Water Level One Foot Above Emergency Spillway	Total Flood Storage Utilized (acre-feet)	Percent of Total Flood Storage Utilized
10-Year, 24-Hour	4	0	5932.2	68
25-Year, 24-Hour	54	1	8688.2	99
50-Year, 24-Hour	60	10	8772.0	100
100-Year, 24-Hour	60	33	8772.0	100

11.4 Chapter Summary

The blended scenario implements a couple flood mitigation strategies – increased infiltration resulting from cover crops and added watershed storage through ponds – to evaluate their combined impact on reducing flood peak discharges. The scenario uses half the number of large, wet ponds considered in the earlier flood storage analysis (60 – located in the steepest headwater subbasins) and assumes 50% of the existing agricultural landscape has been improved by planting cover crops. Improvement of half the agricultural land results in about half as much added infiltration (0.1-0.2 inches) as when all agricultural areas are improved through cover crop planting (0.2-0.3 inches of additional infiltration). Peak discharge reductions of 7-14% were observed at the index locations, excluding the Beaver Creek outlet. These are similar but generally a few percent less than either more extensive flood mitigation strategy considered separately in Chapters 8 and 10. Flood stages were reduced by 0.5-1.5 feet. At the Beaver Creek outlet, slightly greater peak flow reductions (20-23%) are observed than for either more extensive flood mitigation strategy considered separately because the same amount of flood storage is available and infiltration is improved. As with the other scenarios, this scenario is not a project proposal but does provide an indication of the flood reduction benefits that could potentially be gained from a combination of flood mitigation projects.

CHAPTER 12: EVALUATION OF FLOOD MITIGATION STRATEGIES FOR THE JUNE 2008 VALIDATION STORM

12.1 Introduction

In addition to evaluating flood mitigation strategies for reducing peak discharges throughout the watershed using SCS design rain storm events, each flood mitigation scenario was also run for the June 5-17, 2008 validation storm. Assessing the effectiveness of the different flood mitigation strategies for this particular historical storm was done for a couple reasons. First, the June 2008 flood is the largest flood on record in the watershed and people are still very aware of its consequences. Second, the HMS model arguably predicted the watershed response for this event best out of all calibration and validation storms considered, so there is greater confidence the amount of flood reduction provided by the different strategies is reasonable of what might be expected for an event of this magnitude.

In this chapter, flood reduction resulting from the previously considered flood mitigation strategies is examined for the June 2008 flood. Flood mitigation strategies considered include enhanced infiltration from land use changes and improving soil quality (Chapters 8-9), distributed flood storage (Chapter 10), and the blended scenario involving the combination of enhanced infiltration from improved agricultural conditions due to cover crops and distributed flood storage (Chapter 11).

12.2 Method

The HMS model was used to simulate the impact each flood mitigation strategy considered in Chapters 8-11 could have on reducing peak discharges throughout the watershed for the June 2008 validation storm. As described in Chapter 6, flooding in June 2008 produced some of the largest discharges on record throughout the Upper Cedar River Watershed. As a result of heavy rainfall and near saturated soil conditions, record discharges were measured at all operational USGS stage/discharge gages, ranging from

20,000 cfs at Austin to 53,400 cfs downstream at Janesville. The cumulative basin average rainfall for June 5-17, 2008 was over eight inches, with the central part of the watershed (Mitchell and Floyd counties) receiving the most rain (9-11 inches on average). Figure 12.1 shows the cumulative rainfall estimated for the watershed for June 5-17, 2008 using the Stage IV radar rainfall estimates. The left figure shows the gridded cumulative rainfall (scale in inches) and the right figure shows the cumulative rainfall estimated for each subbasin from the gridded estimates through area-weighted averaging.

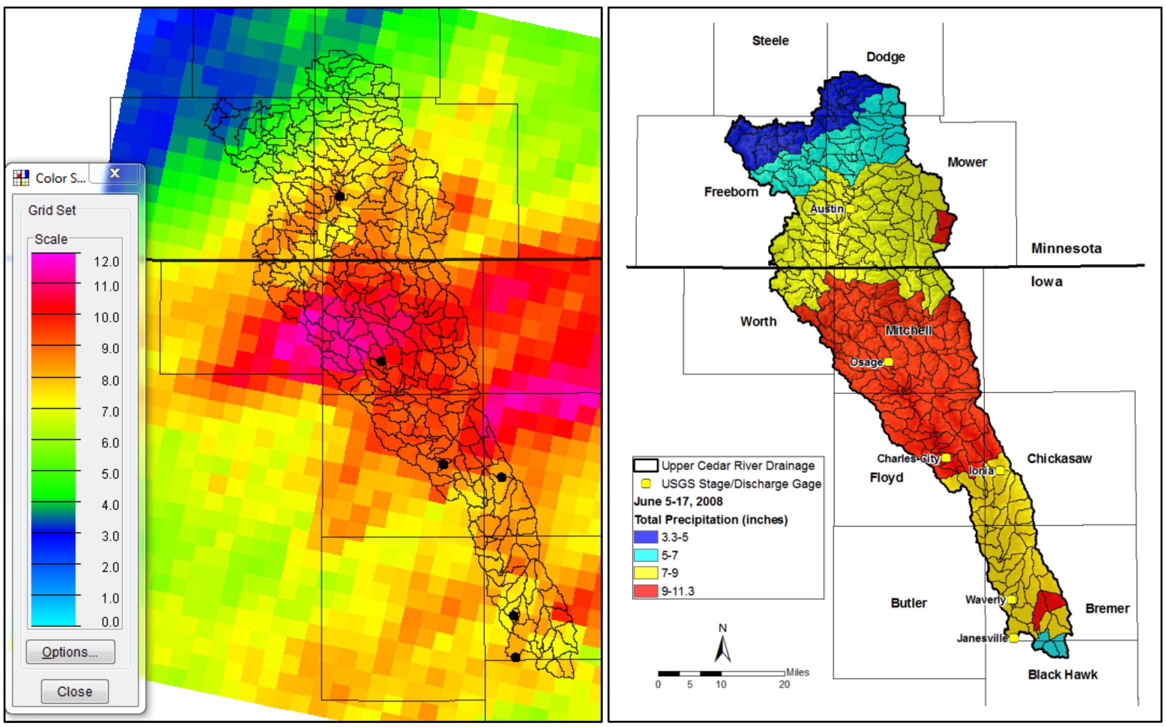

Figure 12.1. Cumulative rainfall estimated for the June 5-17, 2008 validation storm from Stage IV radar rainfall estimates. The Stage IV gridded estimates are shown on the left (in inches) and the cumulative rainfall estimated for each subbasin from the gridded estimates is shown on the right (in inches).

Peak discharges were compared between each hypothetical flood mitigation scenario and the June 2008 validation simulation. Model parameters of each hypothetical scenario were adjusted in the same way as for the validation simulation to accurately reflect the conditions of the June 2008 flood. Each hypothetical scenario used the same baseflow parameters and method to account for AMC as used for the validation simulation. Wetter than normal conditions existed prior to the start of the simulation on June 5th (89th percentile of API), so subbasin CNs of each hypothetical scenario were increased accordingly to represent a higher runoff initial condition. The same reference locations throughout the watershed were used from before to compare each flood mitigation scenario to the June 2008 validation simulation and are shown in Figure 12.2.

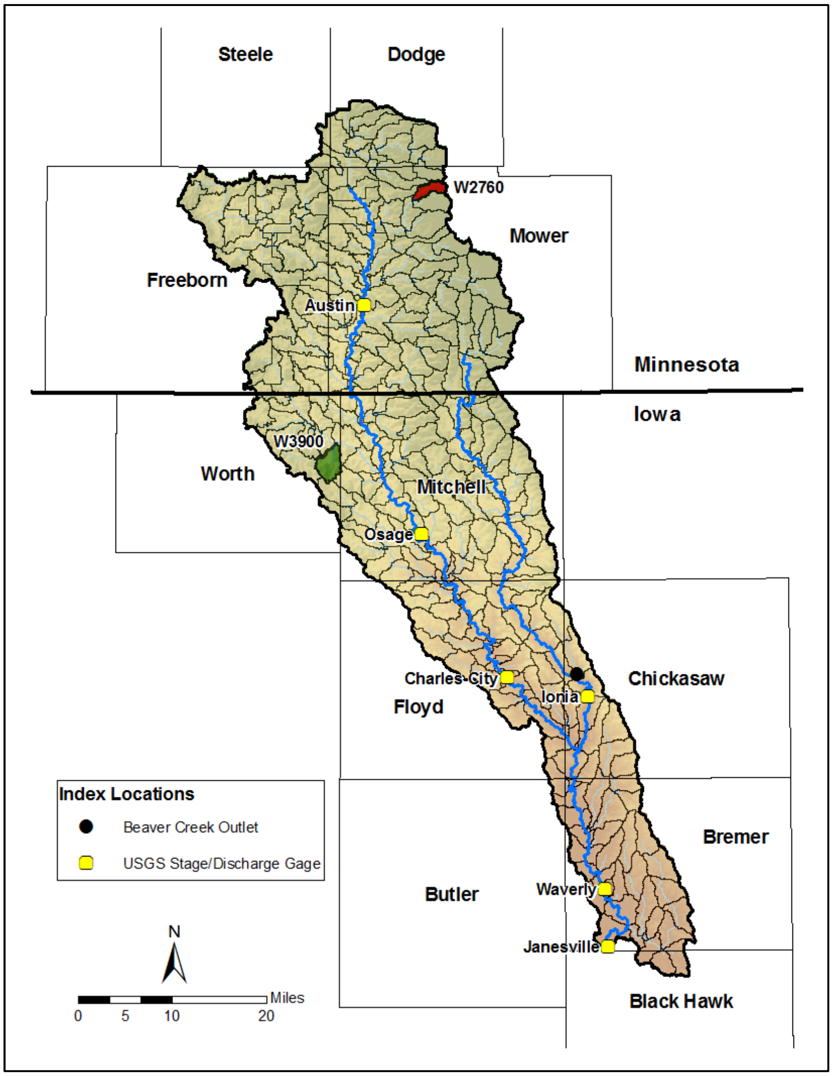

Figure 12.2. Reference locations for comparing watershed improvement scenarios to the June 2008 validation simulation.

12.3 Results

The HMS model was run with each hypothetical flood mitigation strategy – enhanced infiltration due to land use changes, enhanced infiltration due to soil improvements, and flood storage – to see the impact each respective practice could have on reducing flood peak discharges for the June 2008 validation storm.

12.3.1 Effects of Enhanced Infiltration

Figure 12.3 shows the peak discharge reduction for each subbasin and the seven index locations resulting from a tall-grass prairie landscape (left), improved agricultural conditions due to planting cover crops (middle), and the blended scenario (right) which involved a combination of enhanced infiltration (improved agricultural conditions in 50% of the watershed on average by planting cover crops) and flood storage (60 large, wet ponds placed in the steepest headwater subbasins) practices. As expected, the tall-grass prairie landscape had the greatest impact on reducing peak discharges throughout the watershed. The tall-grass prairie landscape increased infiltration by 1.7 inches (109% increase) compared to the baseline validation simulation, and peak discharge reductions of 15-24% are common at the subbasin scale (25th-75th percentiles). Despite spatially variable rainfall for this historical event, peak discharge reductions at the seven index locations are fairly even, ranging from 19% at the Beaver Creek outlet to 25% at Janesville. This reflects the relatively even distribution of agriculture throughout the watershed as well as the near saturated initial condition that caused the majority of rainfall to be converted to runoff regardless of land cover type.

Improved agricultural conditions from planting cover crops throughout the basin resulted in much less runoff reduction compared to the tall-grass prairie landscape. On average throughout the basin, infiltration was increased by about 0.4 inches (23% increase) and peak discharge reductions of 2-5% are common at the subbasin scale (25th-75th percentiles). Peak discharge reductions at the seven index locations range from 3-5%. For the blended scenario where 50% of the agricultural land in the watershed is improved by planting cover crops, about half as much additional infiltration occurs as for the 100% cover crop scenario; infiltration is increased by about 0.2 inches (15% increase). Nearly all (about 98%) the flood storage in the watershed for the blended scenario is exhausted by this event, but no overtopping of the dam occurs for any pond. The water is flowing over the emergency spillway of each large wet pond except for a couple in the northernmost part of the watershed where less rain occurred. Peak discharge reductions for the blended scenario generally decrease moving downstream, ranging from 11% at Austin to 4% at Janesville. For all three land use scenarios, peak discharge reductions are lowest in the middle part of the watershed, reflecting the diminished infiltration capacity of the landscape where the largest amount of rain fell.

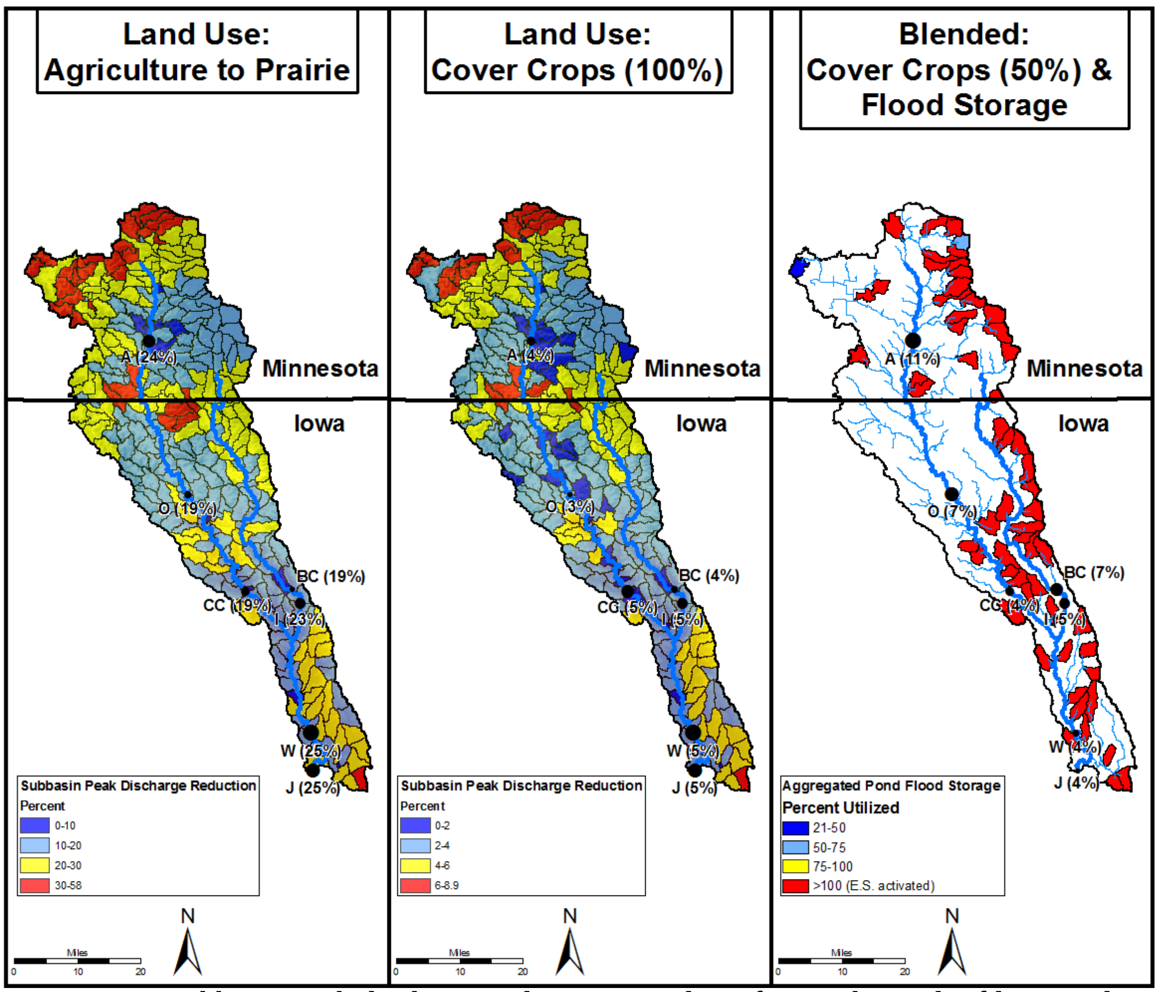

Figure 12.3. Subbasin peak discharge reductions resulting from enhanced infiltration due to land use changes for the June 2008 validation storm. Peak discharge reductions are shown for the tall-grass prairie landscape (left), improved agricultural conditions from planting cover crops (middle), and the blended scenario (right).

Similarly, Figure 12.4 shows the peak discharge reduction for each subbasin and the seven index locations resulting from enhanced infiltration due to soil improvements (B to A and C to B). Improving soil quality represented by converting all B-type soils to A-type soils increases infiltration by about 0.8 inches (49% increase) and peak discharge reductions of 5-13% are common at the subbasin scale (25^{th}-75^{th} percentiles). Greatest peak discharge reductions are observed in the northern and southern thirds of the watershed where the greatest amount of Type B soil exists. Peak discharge reductions at the seven index locations throughout the watershed remain fairly even, ranging from 9-12%.

The C to B soil improvement scenario increases infiltration in the watershed by about half as much as the B to A scenario; infiltration is increased by about 0.4 inches (26% increase) and peak discharge reductions of 2-5% are common at the subbasin scale (25^{th}-75^{th} percentiles). Greatest peak discharge reductions are observed in the middle third of the watershed where the greatest amount of Type C soils exist. Peak discharge reductions at the seven index locations range from 3-6%, about half the reductions seen for the B to A scenario.

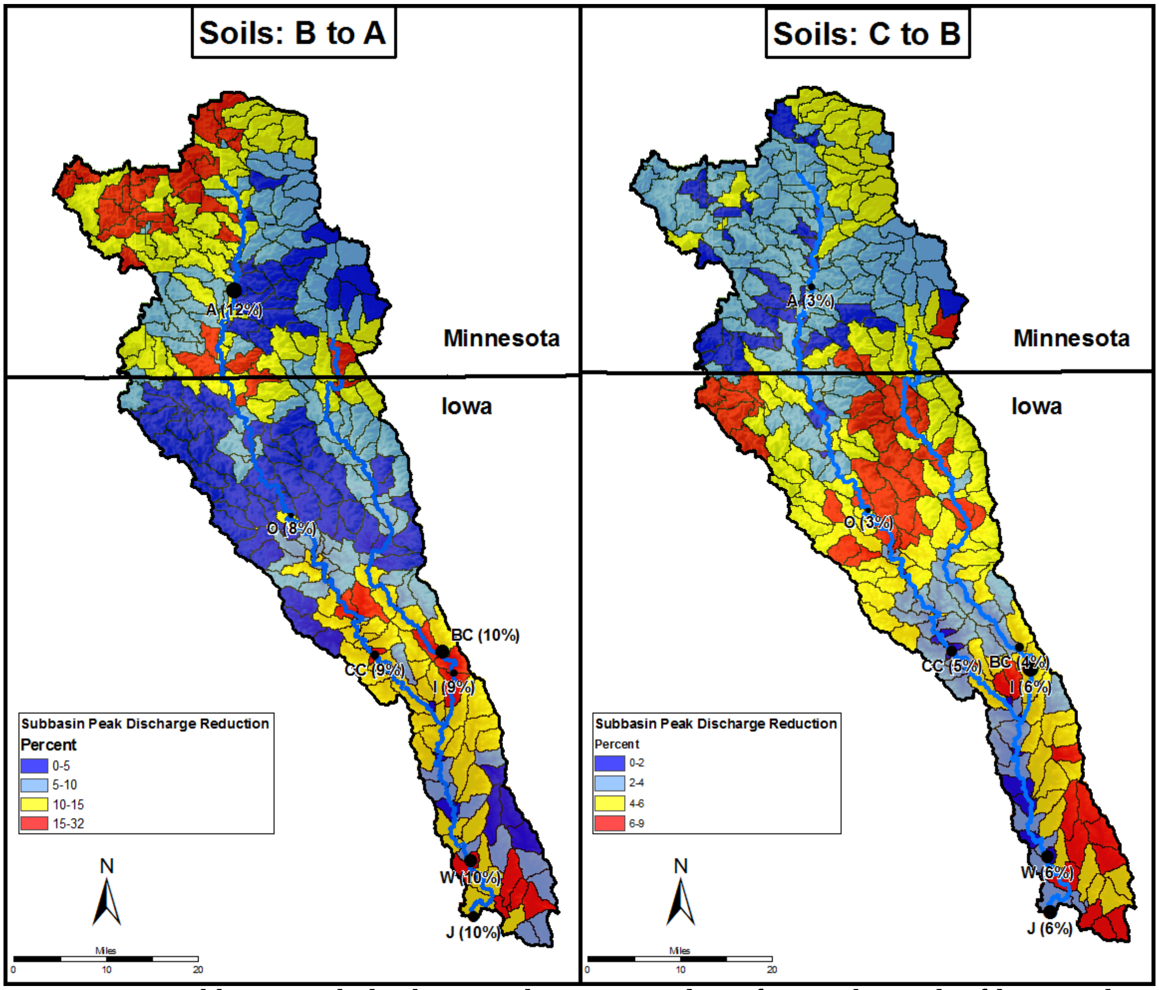

Figure 12.4. Subbasin peak discharge reductions resulting from enhanced infiltration due to soil improvements for the June 2008 validation storm. Peak discharge reductions are shown for the B to A (left) and C to B (right) soil improvement scenarios.

Figure 12.5 compares the performance of each enhanced infiltration practice for reducing peak discharges at the seven index locations for the June 2008 validation storm. The restoration of native tall-grass prairie results in the greatest peak discharge reductions of about 18-25%; flood stages are reduced by up to 2.5 feet. The greatest reduction resulting from any other scenario is less than 12% and the maximum flood stage reduction is about one foot. The cover crop (100%) and C to B soil improvement scenarios generally provided the least amount of flood reduction. In general, peak flow reductions are fairly even throughout the watershed for each flood mitigation scenario, and the lowest reductions typically occur in the middle part of the watershed (Osage, Charles City, and the Beaver Creek outlet) where rainfall was most intense. Peak discharge reductions are fairly even at the seven index locations for each runoff reduction scenario despite spatially variable rainfall. This is largely because the ground was nearly saturated prior to the start of the storm simulation. As a result, most of the rain was converted to runoff regardless of land cover type.

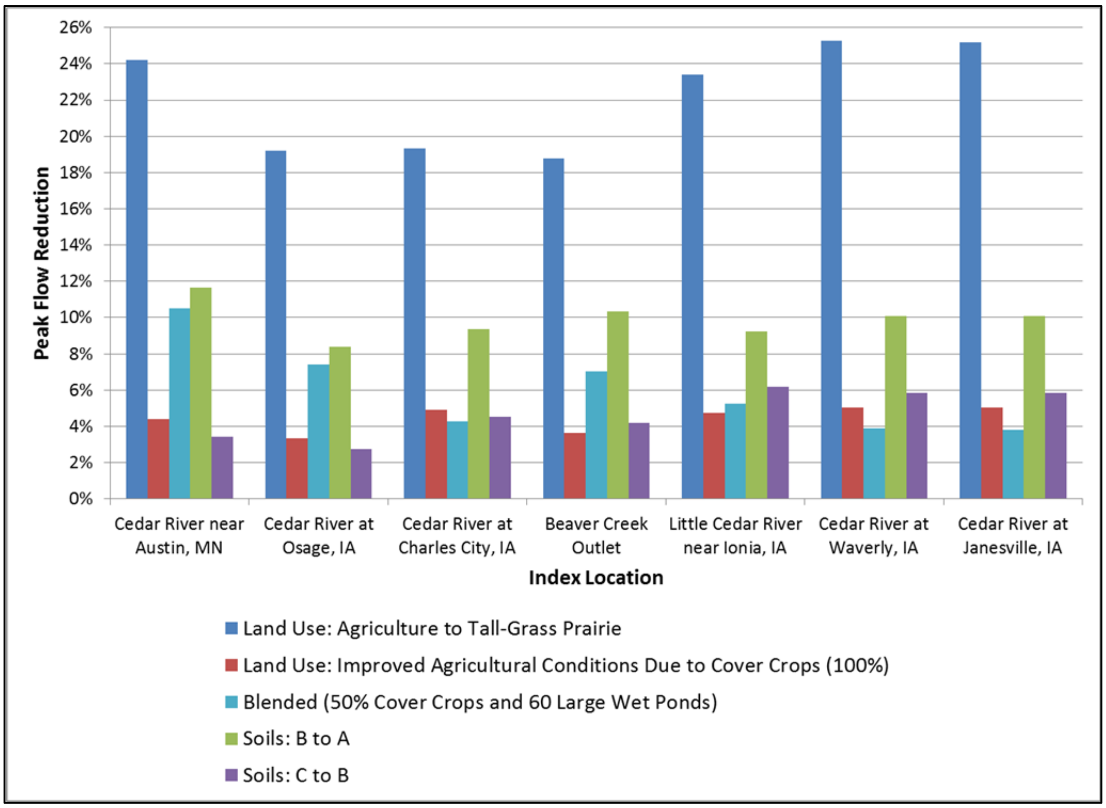

Figure 12.5. Peak discharge reductions at the seven index locations resulting from the enhanced infiltration scenarios for the June 2008 validation storm.

12.3.2 Effects of Distributed Flood Storage

The effects of distributed flood storage for reducing peak discharges for the June 2008 validation storm were also analyzed. Simulations were run for the three pond sizes (elevation differences of 3-, 5- and 7- feet between the primary and emergency spillways) described in Chapter 10, and both wet and dry pond alternatives were considered.

Pond performance characteristics and peak discharge reductions at the seven index locations for the June 2008 validation storm are summarized in Table 12.1 and Figure 12.6 below. For all pond scenarios, nearly 100% of the flood storage in the watershed is exhausted. The wet and dry pond alternatives for each pond size perform similarly at reducing downstream peak discharges. The water level reaches the emergency spillway for all 121 wet and dry ponds for the small pond scenario, 116 (96%) of the wet and dry ponds for the medium-sized pond scenario, and 110 (91%) of the wet and dry ponds for the large pond scenario. The water level reached the emergency spillway for 58 of 60 (97%) large wet ponds for the blended scenario, and nearly 98% of the total flood storage is exhausted. Because the wet and dry ponds perform essentially the same for the June 2008 flood, only wet pond performance for each pond size is shown in Figure 12.6. The flood storage of each pond is completely utilized except for a few of the medium and large ponds in the northern part of the watershed (where a lesser amount of rain fell). No overtopping of the dam occurs in any scenario, but the water level reaches at least one foot above the emergency spillway for 82 (68%) of the small ponds, 68 (56%) of the medium-sized ponds, and 51 (42%) of the large ponds.

Table 12.1. Summary of pond performance characteristics for the June 2008 validation storm.

	Wet			*Dry*		
Pond Scenario	Ponds Activating Emergency Spillway (121 total)	Ponds with Peak Water Level One Foot Above Emergency Spillway	Percent of Total Flood Storage Utilized	Ponds Activating Emergency Spillway (121 total)	Ponds with Peak Water Level One Foot Above Emergency Spillway	Percent of Total Flood Storage Utilized
Small	121	82	100.0%	121	82	100.0%
Medium	116	68	99.0%	116	68	99.0%
Large	110	51	96.0%	110	50	96.0%
Blended (Large, Wet Ponds)	58 (60 total)	24	97.7%			

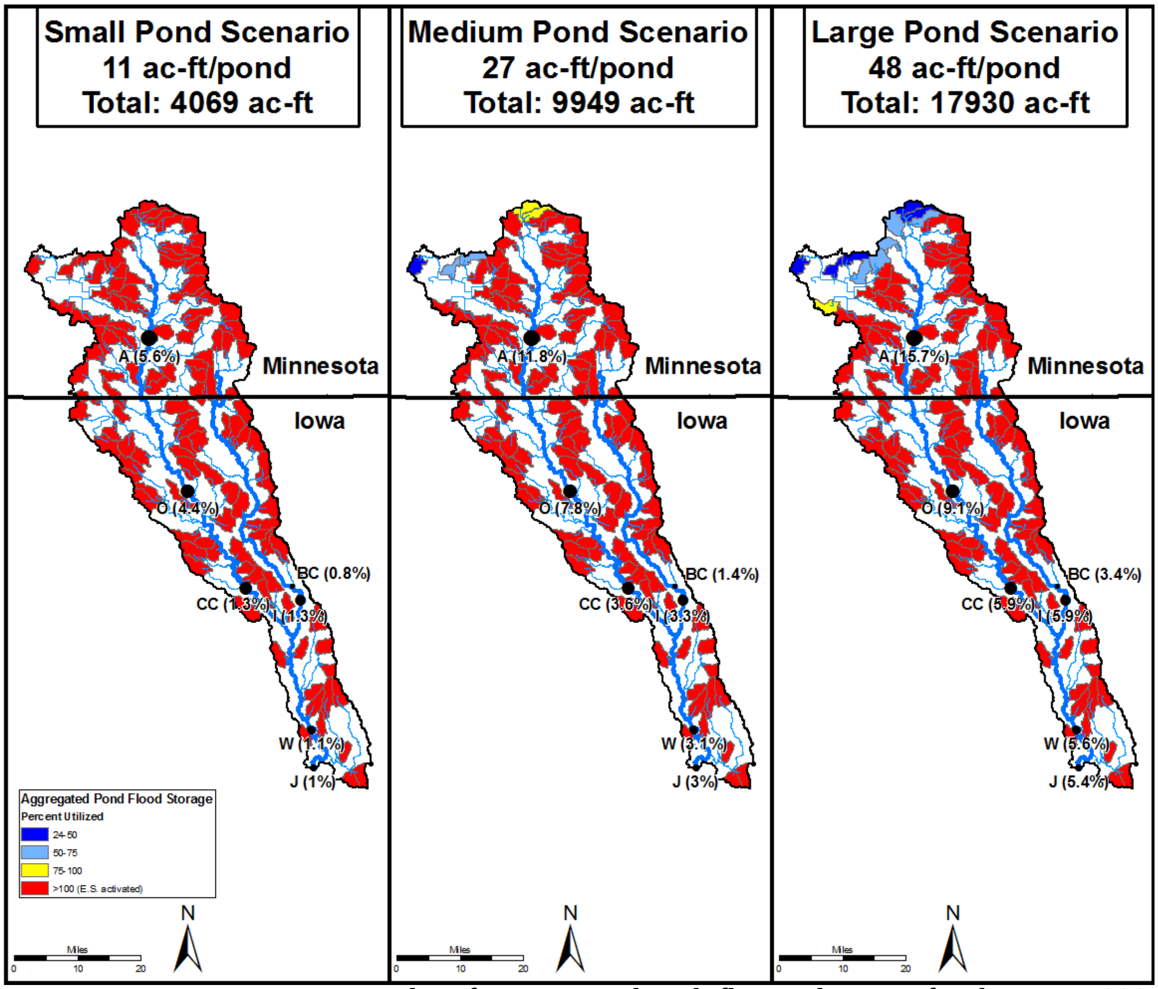

Figure 12.6. Summary wet pond performance and peak flow reductions for the June 2008 validation storm.

Figure 12.7 compares the peak discharge reductions estimated at the seven index locations for the small, medium, and large wet pond scenarios for the June 2008 validation storm. As expected, the peak discharge reduction at each location increases with increasing pond size since a greater amount of storage is available to temporarily retain floodwaters. Also, the reductions tend to decrease moving from upstream to downstream. This is partly due to the same reason as described for the design rain events – the percent of upstream area drained by ponds generally decreases moving downstream. However, the spatial variability of rainfall for this actual event also impacts the peak flow reductions observed throughout the watershed. Because rain was heaviest in the central part of the basin, peak discharge reductions are generally smallest at Charles City and the Beaver Creek outlet since the flood storage is exhausted more quickly in this region. For the small pond scenario, peak discharge reductions range from 0.8% at the Beaver Creek outlet to almost 6% at Austin; flood stages are reduced by up to half a foot. For the medium-sized pond scenario, peak discharge reductions range from 1.4% at the Beaver Creek outlet to almost 12% at Austin; flood stages are reduced by up to one foot. For the large pond scenario, peak discharge reductions range from 3.4% at the Beaver Creek outlet to almost 16% at Austin; flood stages are reduced by up to 1.5 feet.

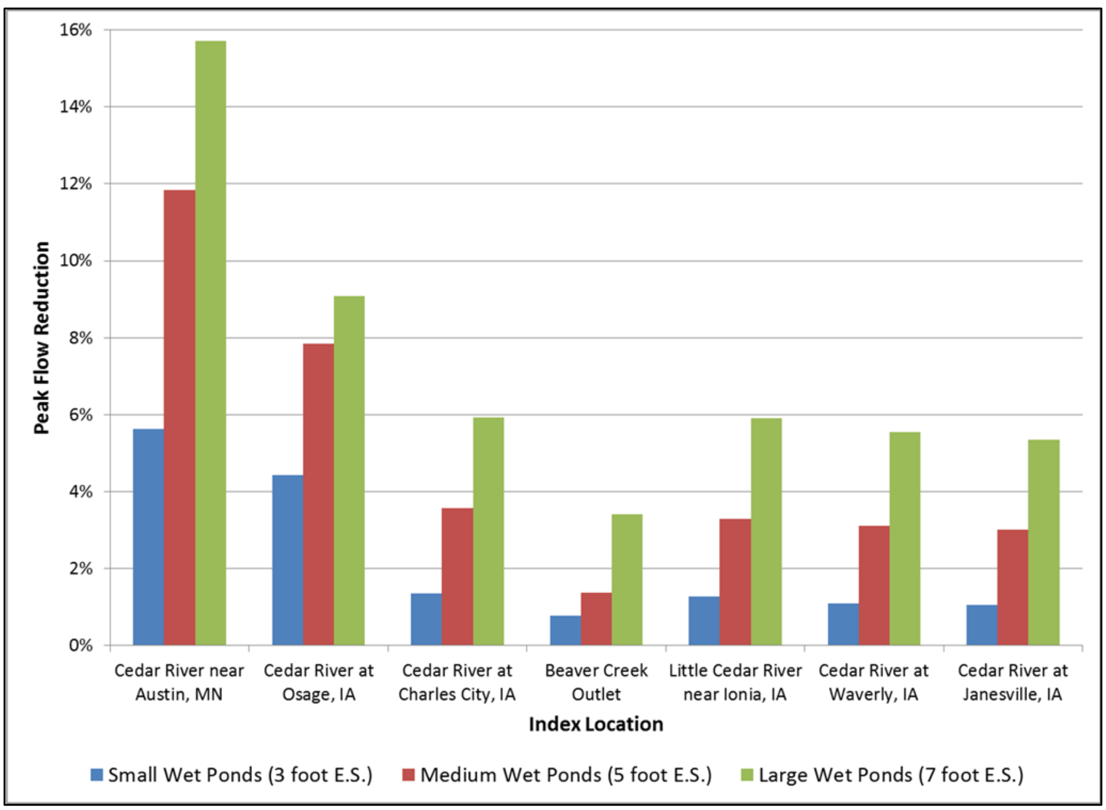

Figure 12.7. Peak discharge reductions at the seven index locations resulting from distributed flood storage (small, medium and large pond scenarios) for the June 2008 validation storm.

12.4 Chapter Summary

The final analysis performed evaluates the impact the different flood mitigation strategies discussed in Chapters 8-11 could have on reducing peak discharges for the June 2008 flood. The impacts of runoff reduction practices – enhanced infiltration due to land use changes and improved soil conditions – and distributed flood storage were both considered. Restoring the agricultural landscape back to native tall-grass prairie had the greatest flood reduction impact. The native tall-grass prairie landscape could reduce peak discharges throughout the watershed by up to 25% and flood stages by a couple feet. While this is significant and would lessen the severity and frequency of the June 2008 flood by today's standards, flooding would still have been substantial even with the tall-grass prairie landscape. Because the ground was nearly saturated at the start of the event, the majority of rain was converted to runoff regardless of land cover or soil type, so peak discharge reductions for each runoff reduction scenario are fairly even throughout the basin. Improved agricultural conditions throughout the basin from planting cover crops during the dormant season had a relatively small effect on reducing peak discharges (less than 5% at any index location).

Distributed flood storage throughout the watershed showed the ability to reduce peak discharges for the June 2008 event by small but noticeable amounts depending on pond size and location. As expected, the largest pond scenario provided the greatest peak flow reductions throughout the watershed. Reductions were greatest at Austin (over 15% for the large pond scenario), but decreased rapidly moving downstream to less than 6% downstream of Charles City. The small and medium-sized pond scenarios provided noticeable peak flow reductions in the upper part of the watershed, but reductions were minimal in the lower two-thirds of the watershed. The blended scenario, which utilized both cover crops as a runoff reduction strategy and flood storage, showed the ability to reduce peak discharges by 4-11%; flood stages were reduced by up to one foot. This reemphasizes the value of combining flood mitigation strategies to reduce flood damages.

CHAPTER 13: SUMMARY AND CONCLUSIONS

The overall goal of this Master's thesis is to evaluate the impact different flood mitigation strategies could have on reducing runoff and peak discharges from flood events in the Upper Cedar River Watershed in northeast Iowa. To do so, a hydrologic model of the watershed was developed using HEC-HMS – a lumped parameter, surface water model – to predict the watershed response for different rainfall events. The model was developed in Arc GIS using relevant topographic, land use, and soil datasets needed to describe the runoff characteristics of the watershed as required by the NRCS CN methodology. The HMS model was calibrated and validated to several historical, large rainstorms that occurred in the watershed over the last decade. Appropriate changes were made to the runoff and timing parameters of the model to reflect the intensively managed agricultural landscape and account for antecedent moisture conditions. Following, the model was used to assess the runoff potential in the basin and the effects of different flood mitigation strategies for significant design flood events – the 10-, 25-, 50-, and 100-year storms that apply 4.05-6.81 inches of rainfall over the entire watershed over a 24-hour period. Three flood reduction strategies were considered independently or in combination – enhancing infiltration through land use changes, enhancing infiltration through soil quality improvements, and storing runoff in ponds placed throughout the watershed to reduce downstream peak flows. The various flood mitigation strategies were also evaluated for the June 2008 flood to provide a benchmark on the amount of flood reduction possible for the largest flood on record in the watershed.

13.1 Assessing the Runoff Potential in the Upper Cedar River Watershed

To better understand the flood hydrology of the Upper Cedar River Watershed, the HMS model was used to identify areas of high runoff potential in the basin. This analysis provides a first step in prioritizing areas to focus flood mitigation efforts. The runoff coefficient was used to assess the runoff potential throughout the watershed. Locations with agricultural land use and moderately to poorly draining soils have the highest runoff potential. Highest runoff areas were observed in the central part of the watershed in parts of Worth, Mitchell, and Floyd counties. However, the runoff potential in these areas was not substantially different from other areas throughout the watershed, reflecting the relatively equal distribution of row crop agriculture and moderately to poorly draining soils in the watershed.

13.2 Increased Infiltration in the Watershed: Land Use Changes

To demonstrate the effectiveness of one flood reduction strategy, infiltration in the watershed is increased through hypothetical land use changes. Two scenarios were considered in which all existing agricultural areas were changed to a landscape with lower runoff characteristics. In the first case, all row crop agriculture in the watershed is converted to native-tall grass prairie. This scenario had the greatest impact on reducing runoff and peak discharges of all flood mitigation strategies considered. The infiltration characteristics of the tall-grass prairie landscape are much better than the existing agricultural landscape; as a result, substantial peak discharge reductions (30-50%) and flood stage reductions (2-7 feet) were estimated at various index locations throughout the watershed. While the native tall-grass prairie landscape would substantially reduce the extent of flood inundated areas, flooding would not be eliminated completely; rather, the severity and frequency of flooding would be reduced. This hypothetical example provides a benchmark on the limits of flood reduction that are physically possible with other watershed improvement projects and suggests that flood mitigation projects focusing on enhancing infiltration through land use changes could be an effective flood reduction strategy.

In the second land use scenario, existing agricultural areas are improved to reflect planting cover crops during the dormant season. Flood reduction benefits are less than for the native prairie scenario, as expected, but are still noticeable. On average for the basin, infiltration increases by 0.2-0.3 inches for the four design storms considered. Peak flows are reduced by 6-15% at the seven index locations, representing up to one foot reductions in flood stage at the USGS stage/discharge gage locations. While runoff and peak flow reductions are substantially less for the cover crop scenario than for the native prairie scenario, cover crops provide a realistic approach for runoff reduction and could be implemented at large scales.

13.3 Increased Infiltration in the Watershed: Improving Soil Quality

In addition to land use changes, enhancing infiltration through soil quality improvements could also provide some amount of flood reduction. The HMS model was used to simulate the effects of improved soil health through conservation and best management practices. These improvements were represented through changes to NRCS hydrologic soil groups. Runoff reduction benefits were similar to or better than the cover crop land use scenario. The best soil improvement scenario resulted in peak flow reductions of 15-25% and flood stage reductions of 1-3 feet at the index locations. Continued efforts to improve soil quality, such as increasing soil organic material and planting cover crops, could aid in reducing flood damages across the watershed.

13.4 Increased Storage on the Landscape

Distributed flood storage was also evaluated as a flood mitigation strategy. Rather than mitigating flood impacts through runoff reduction, storage ponds reduce downstream flood peaks by storing floodwater temporarily and releasing it at a lower rate. For the hypothetical pond scenarios, a typical pond design was developed reflecting the topography of the watershed; 121 ponds were placed in the headwater regions of the watershed. The ponds drain approximately 23% of the watershed. Wet and dry pond alternatives were considered for three different pond sizes. Wet ponds added between 4,069 acre-feet and 17,930 acre-feet of storage to the watershed. For the upstream areas draining to a pond, this is equivalent to an added storage depth of 0.2 inches (small ponds) to 0.9 inches (large ponds); for the watershed as a whole, the wet ponds provide 0.05-0.2 inches of added storage depth. The dry ponds add an additional 1,028 acre-feet of storage, equivalent to an added storage depth of 0.05 inches over the upstream areas draining to a pond and 0.01 inches for the watershed as a whole.

Peak flow reductions were similar for both the wet and dry pond alternatives. Considering wet ponds, peak flow reductions ranged from 2-9% for the small pond scenario (4,069 acre-feet of added storage), 4-18% for the medium pond scenario (9,949 acre-feet of added storage), and 7-20% for the large pond scenario (17,930 acre-feet); flood stage reductions of up to 1.5 feet were estimated at the USGS stage/discharge gages. The ponds were effective at reducing peak discharges immediately downstream of their headwater locations. Moving further downstream, however, the ponds are less effective at reducing flood peaks. Distributed flood storage represents a realistic flood mitigation strategy but will be more difficult to implement in the Upper Cedar than some other Phase I watersheds because of the flat topography.

13.5 Combined Effect of Increased Infiltration and Flood Storage

The impact of both flood storage and enhanced infiltration practices for flood reduction was also considered. For this scenario, storage ponds located in the steepest headwater regions in the watershed and cover crops implemented across half the basin on average helped to reduce flood damages. The 60 storage ponds provide 8,772 acre-feet of storage to the watershed, approximately half that provided by all 121 large, wet ponds.

As expected, the added infiltration due to improved soil conditions from planting cover crops on 50% of the agricultural land is approximately half as much (0.1-0.2 inches) as when cover crops are planted on the entire agricultural landscape (0.2-0.3 inches); the combination of cover crops and flood storage reduce peak discharges by 7-14% at six of the seven index locations with up to 24% reductions observed at the Beaver Creek outlet. While not as effective as either more extensive flood mitigation strategy considered independently, this is one example of the impact a combination of projects could have on reducing flood peak discharges.

13.6 Evaluation of Flood Mitigation Strategies for the June 2008 Flood

The flood mitigation strategies described in sections 13.1-13.5 were also evaluated for the June 2008 flood. The June 2008 event is the largest flood on record in the watershed and its impacts are still evident six years later. Restoring the agricultural landscape back to native tall-grass prairie provided the greatest amount of flood reduction of any hypothetical scenario considered. Peak discharges at the seven index locations were reduced by 18-25%; flood stages were reduced by a couple feet. While the restored tall-grass prairie landscape would significantly reduce the flood severity and frequency of the June 2008 flood, substantial flooding would still have occurred because of the saturated initial conditions and persistent rainfall. The other hypothetical runoff reduction scenarios were much less effective at reducing peak discharges, in large part due to the ground being saturated prior to the start of the event which caused most rain to be converted to runoff regardless of land cover type.

Distributed flood storage demonstrated the ability to reduce peak discharges by a small but noticeable amount for this high flow event. Nearly all the flood storage in the watershed was exhausted for each pond scenario. While relatively small peak flow reductions were predicted at the seven index locations, storage ponds are still effective at reducing peak discharges directly downstream of their locations. Peak flow reductions resulting from the blended scenario (combination of cover crops and flood storage) were also small but do provide a benchmark for how a specific combination of flood mitigation practices might perform for a large flood event.

13.7 Final Remarks

As part of the Iowa Watersheds Project, several flood mitigation strategies were modeled in the Upper Cedar River Watershed using HEC-HMS to evaluate their effectiveness at reducing flood peak discharges. The flood mitigation strategies considered focus on either reducing runoff from the landscape through enhanced infiltration (land use changes and soil improvements) or by storing the runoff and controlling its release rate (distributed flood storage). Although hypothetical and simplified, the scenarios provide guidelines for the flood reduction benefits that are physically possible in the Upper Cedar. Hopefully these findings can assist modeling efforts and project selection in Phase II of the Iowa Watersheds Project as well as provide a reference/resource to the rest of the watershed for future flood mitigation work.

APPENDIX A – ADDITIONAL RESULTS FOR HYPOTHETICAL SCENARIOS

Appendix A contains additional figures and tables for the hypothetical scenario analyses performed in Chapters 8-11.

Chapter 8: Hypothetical Increased Infiltration within the Watershed – Land Use Changes

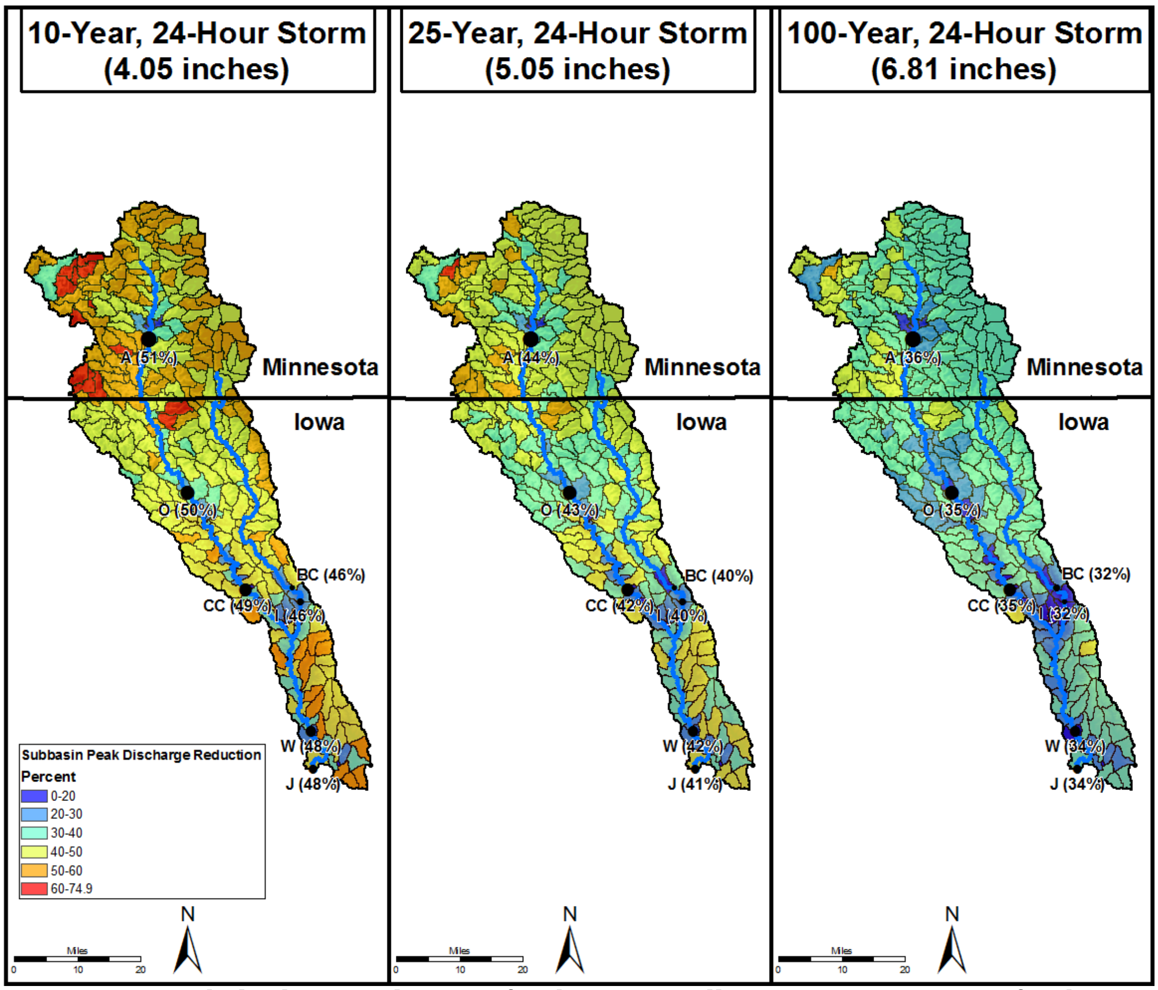

Figure A-1. Peak discharge reductions for the native-tall grass prairie scenario for the 10-, 25-, and 100-year, 24-hour design storms.

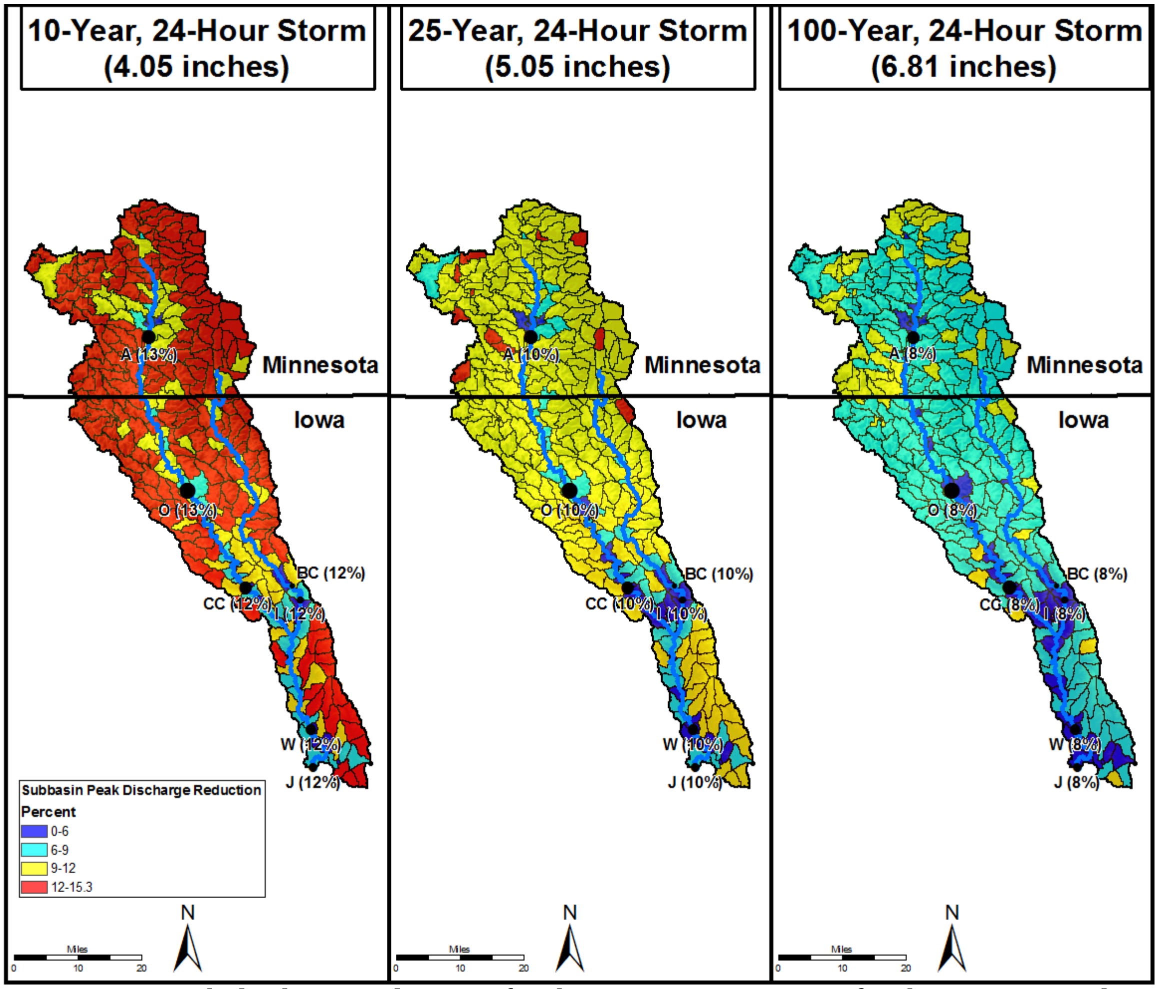

Figure A-2. Peak discharge reductions for the cover crop scenario for the 10-, 25-, and 100-year, 24-hour design storms.

Chapter 9: Hypothetical Increased Infiltration within the Watershed – Improving Soil Quality

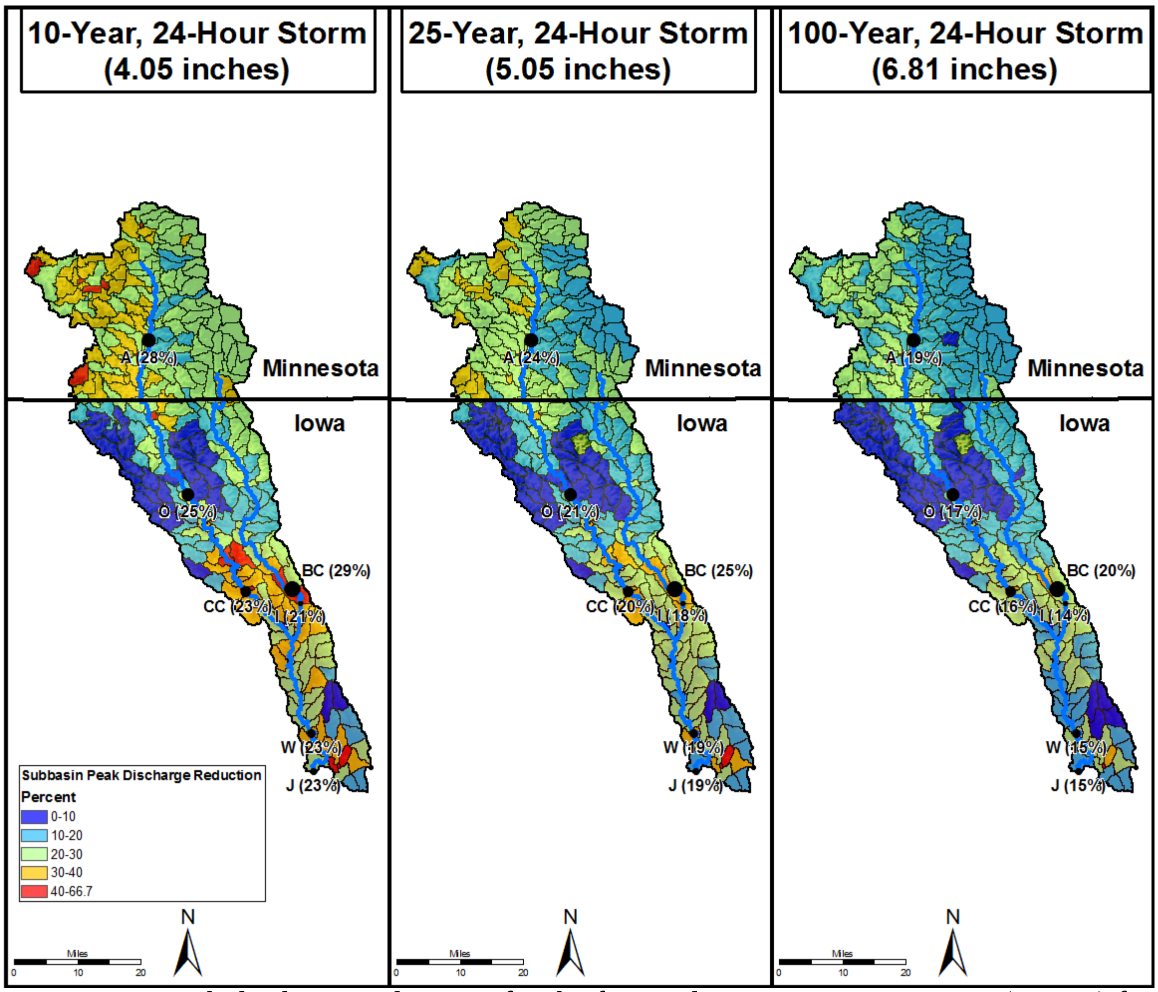

Figure A-3. Peak discharge reductions for the first soil improvement scenario (B to A) for the 10-, 25-, and 100-year, 24-hour design storms.

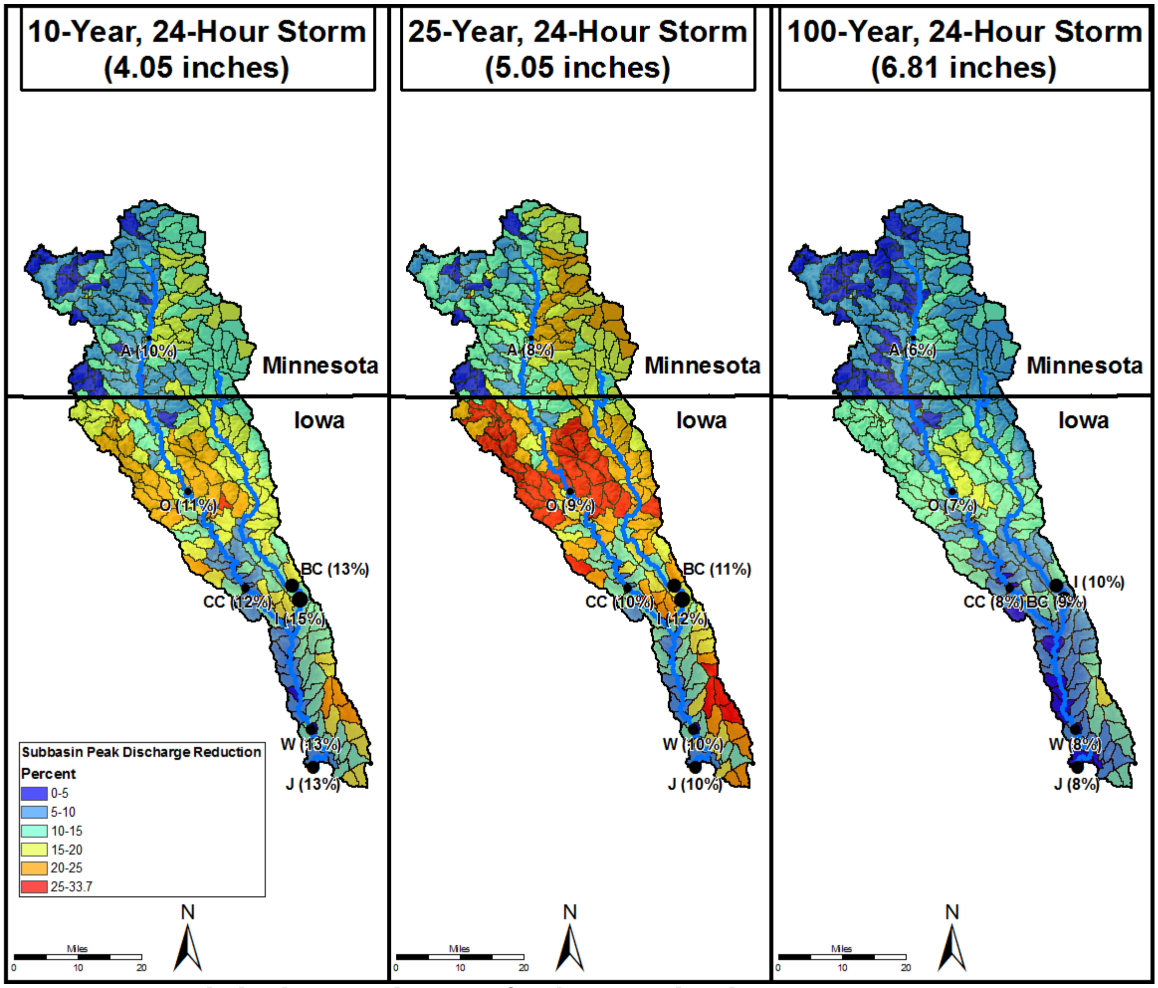

Figure A-4. Peak discharge reductions for the second soil improvement scenario (C to B) for the 10-, 25-, and 100-year, 24-hour design storms.

Chapter 10: Mitigating the Effects of High Runoff With Flood Storage

Table A-1. Pond performance characteristics for the 10-year, 24-hour design storm (4.05 inches of rain in 24 hours).

	Wet			*Dry*		
Pond Scenario	Ponds Activating Emergency Spillway (121 total)	Ponds with Peak Water Level One Foot Above Emergency Spillway	Percent of Total Flood Storage Utilized	Ponds Activating Emergency Spillway (121 total)	Ponds with Peak Water Level One Foot Above Emergency Spillway	Percent of Total Flood Storage Utilized
Small	121	2	100.0%	121	1	100.0%
Medium	92	0	96.6%	83	0	94.8%
Large	15	0	70.2%	9	0	67.5%

Table A-2. Pond performance characteristics for the 50-year, 24-hour design storm (5.89 inches of rain in 24 hours).

	Wet			*Dry*		
Pond Scenario	Ponds Activating Emergency Spillway (121 total)	Ponds with Peak Water Level One Foot Above Emergency Spillway	Percent of Total Flood Storage Utilized	Ponds Activating Emergency Spillway (121 total)	Ponds with Peak Water Level One Foot Above Emergency Spillway	Percent of Total Flood Storage Utilized
Small	121	73	100.0%	121	70	100.0%
Medium	121	50	100.0%	121	47	100.0%
Large	121	21	100.0%	121	19	100.0%

Table A-3. Pond performance characteristics for the 100-year, 24-hour design storm (6.81 inches of rain in 24 hours).

	Wet			*Dry*		
Pond Scenario	Ponds Activating Emergency Spillway (121 total)	Ponds with Peak Water Level One Foot Above Emergency Spillway	Percent of Total Flood Storage Utilized	Ponds Activating Emergency Spillway (121 total)	Ponds with Peak Water Level One Foot Above Emergency Spillway	Percent of Total Flood Storage Utilized
Small	121	117	100.0%	121	117	100.0%
Medium	121	104	100.0%	121	98	100.0%
Large	121	66	100.0%	121	57	100.0%

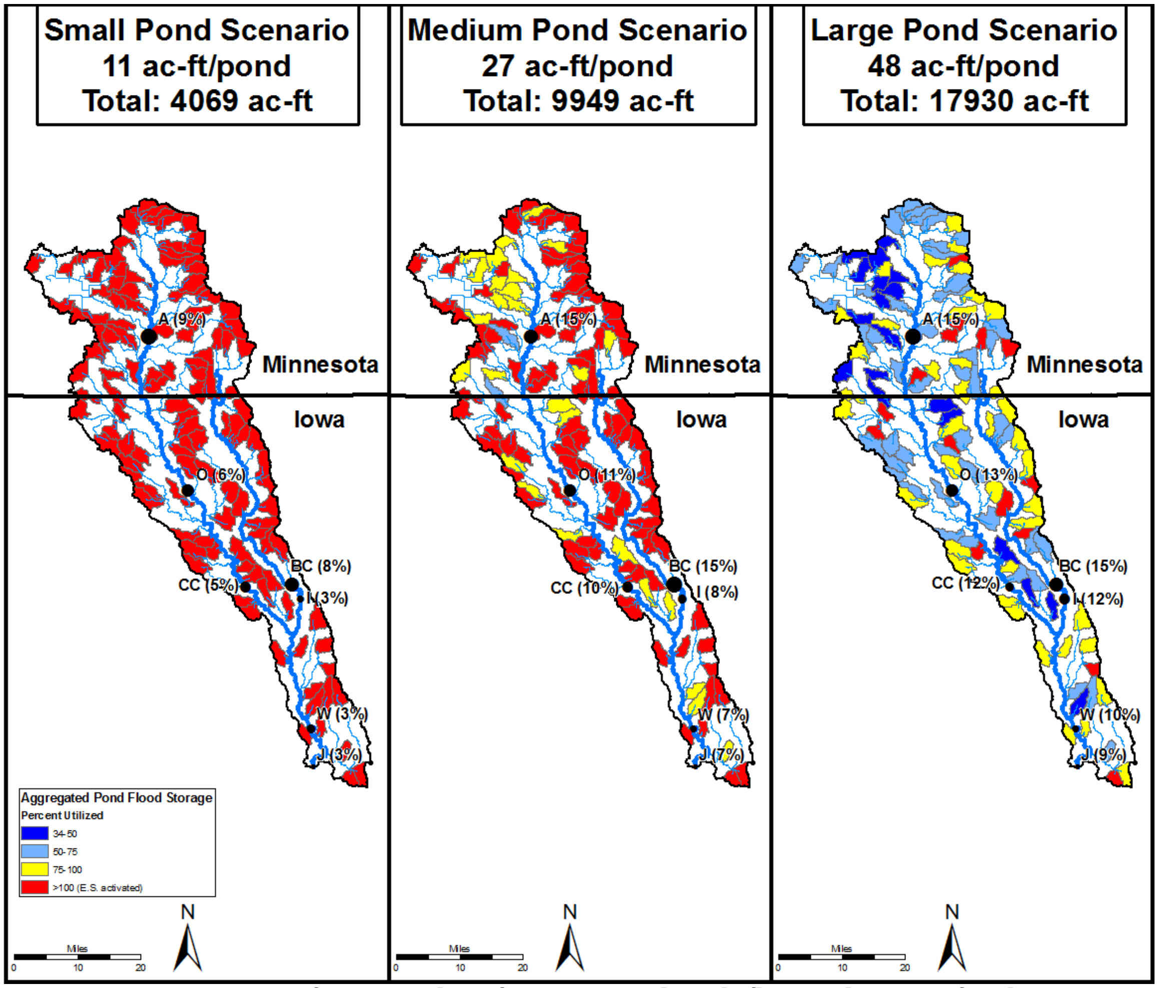

Figure A-5. Summary of wet pond performance and peak flow reductions for the 10-year, 24-hour design storm (4.05 inches of rain in 24 hours).

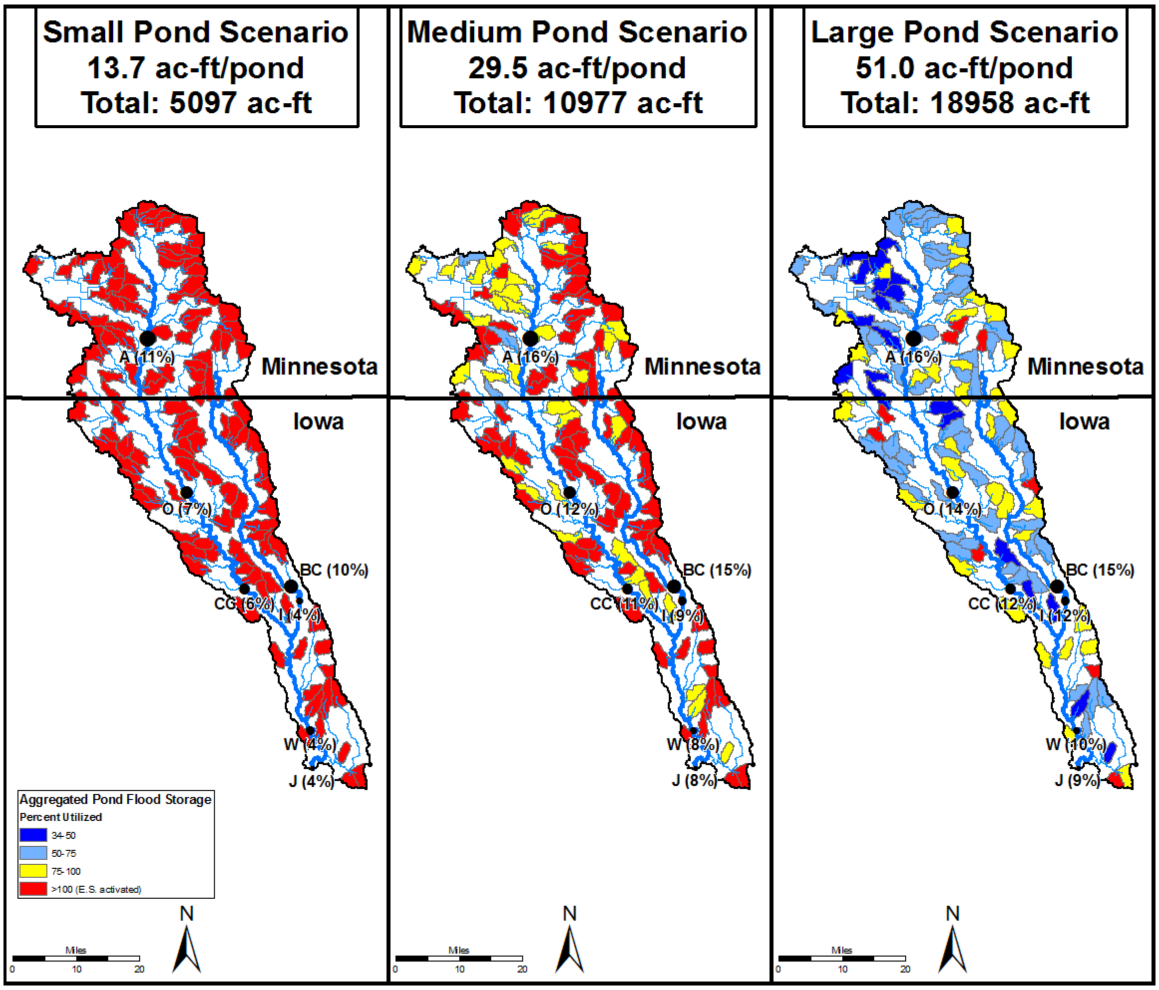

Figure A-6. Summary of dry pond performance and peak flow reductions for the 10-year, 24 hour design storm (4.05 inches of rain in 24 hours).

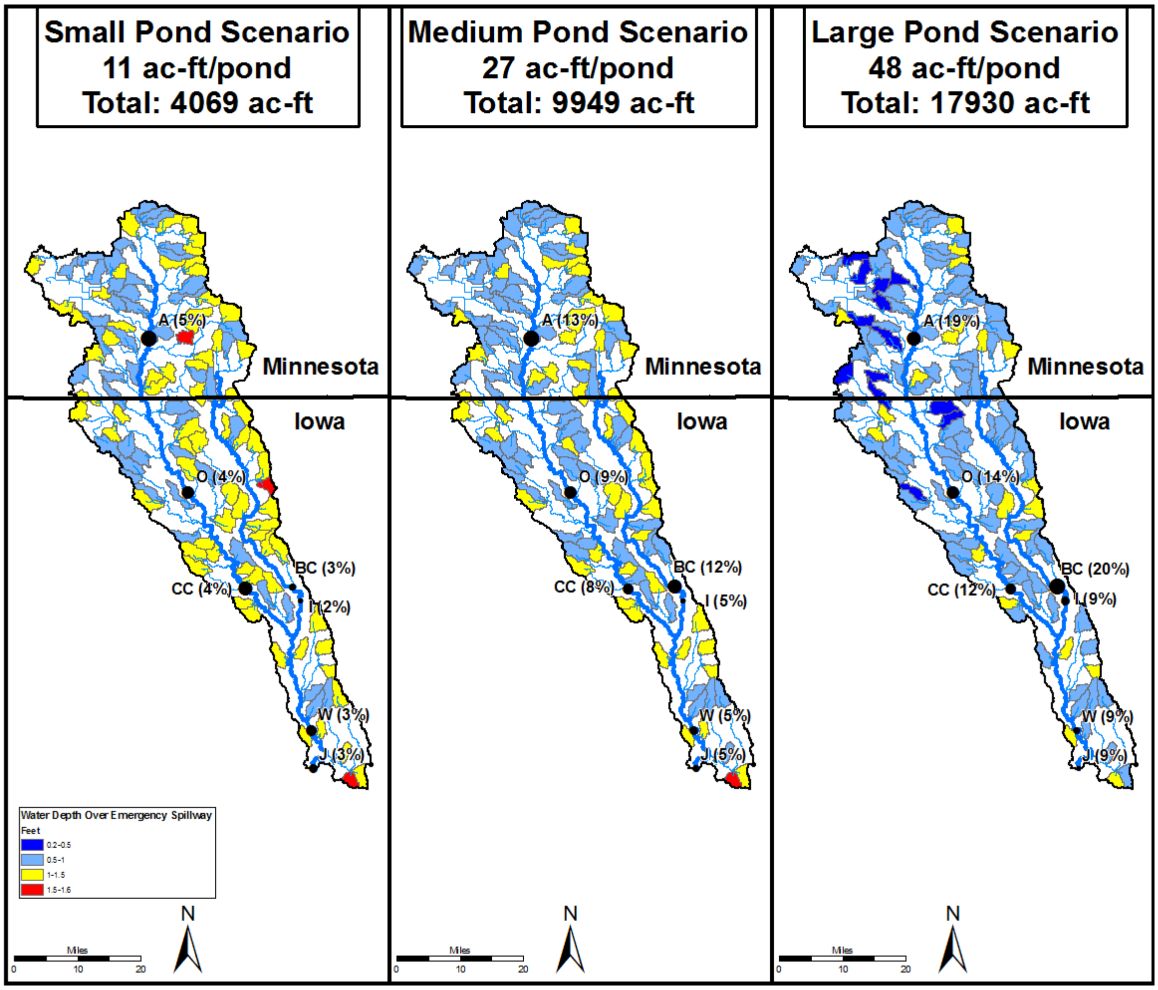

Figure A-7. Summary of wet pond performance and peak flow reductions for the 50-year, 24-hour design storm (5.89 inches of rain in 24 hours).

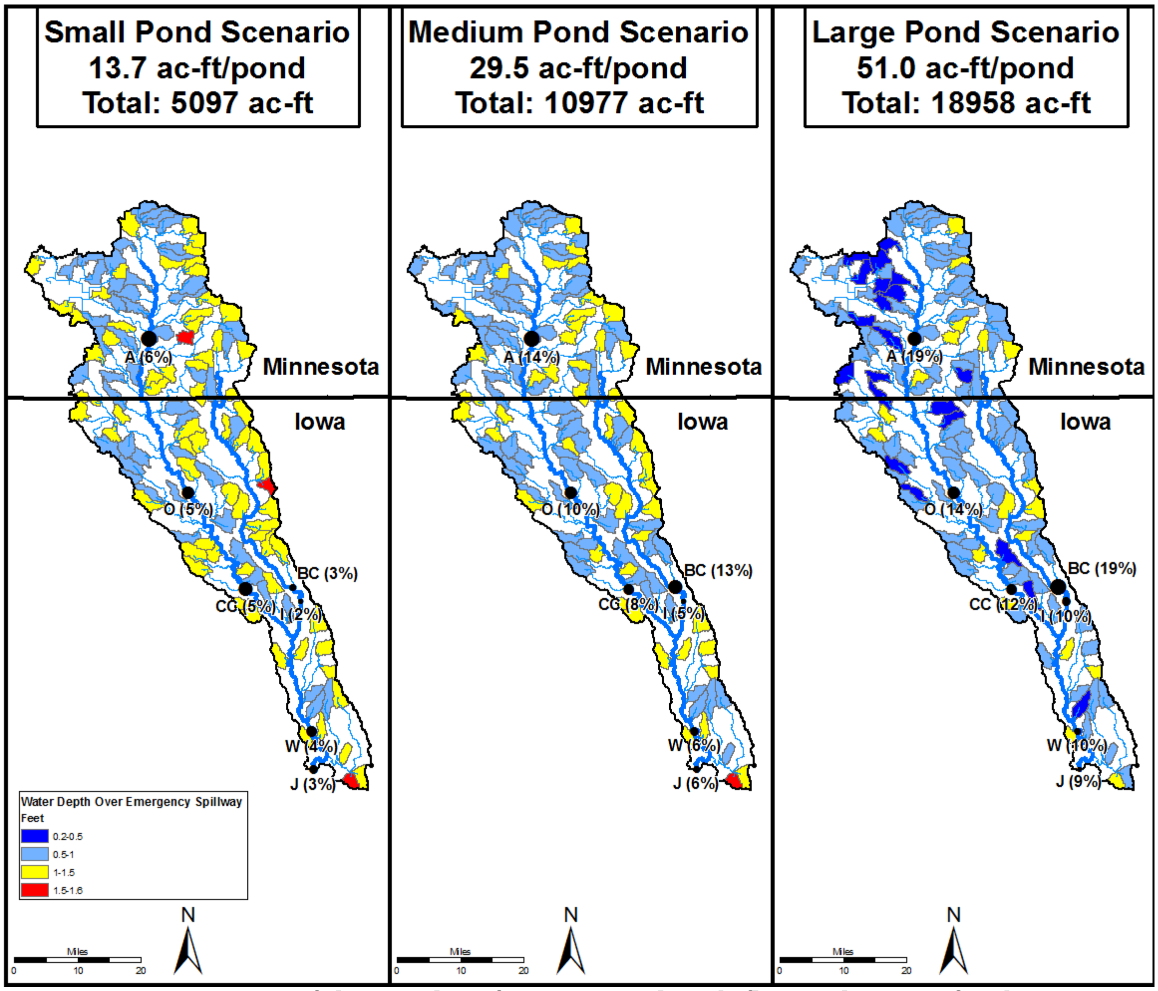

Figure A-8. Summary of dry pond performance and peak flow reductions for the 50-year, 24 hour design storm (5.89 inches of rain in 24 hours).

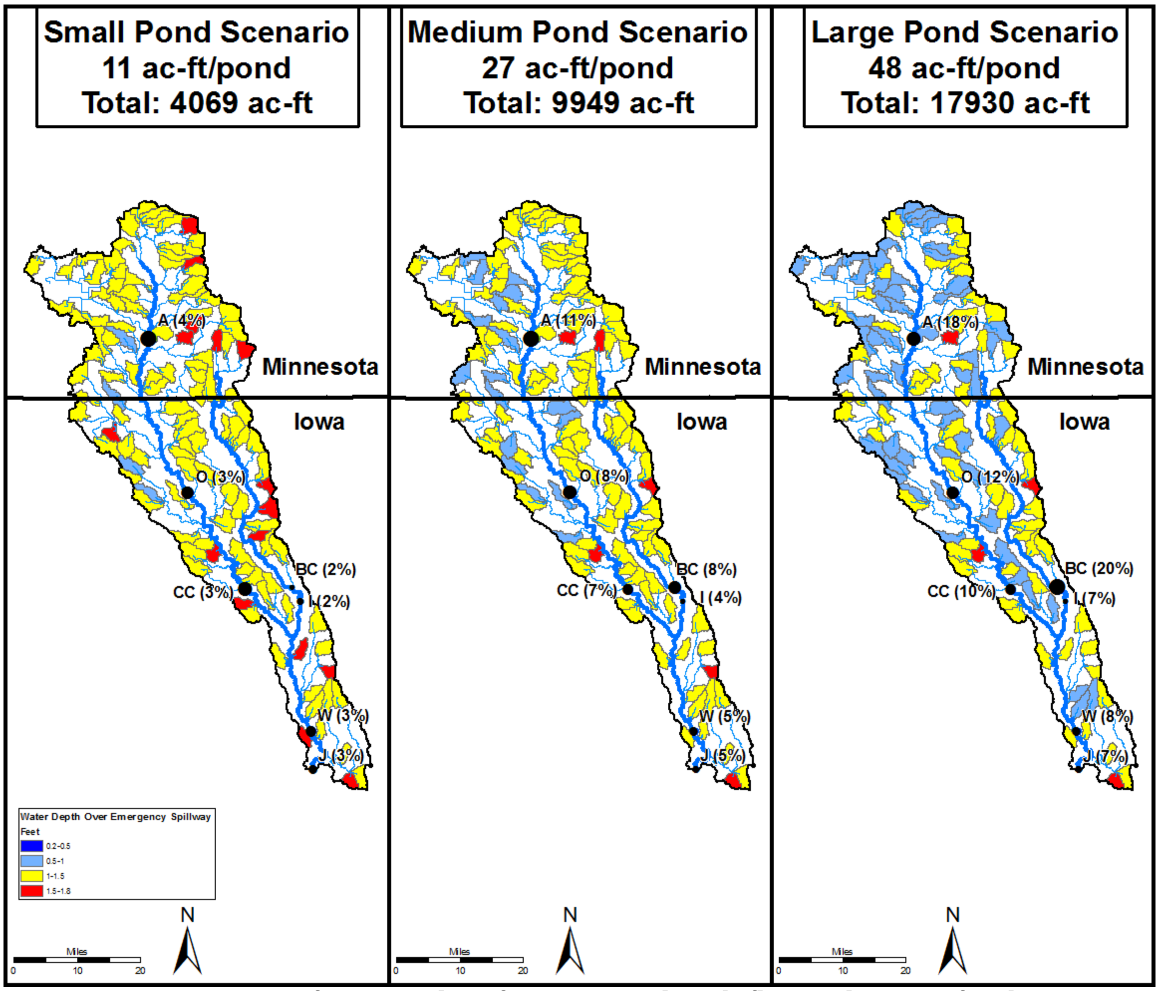

Figure A-9. Summary of wet pond performance and peak flow reductions for the 100-year, 24-hour design storm (6.81 inches of rain in 24 hours).

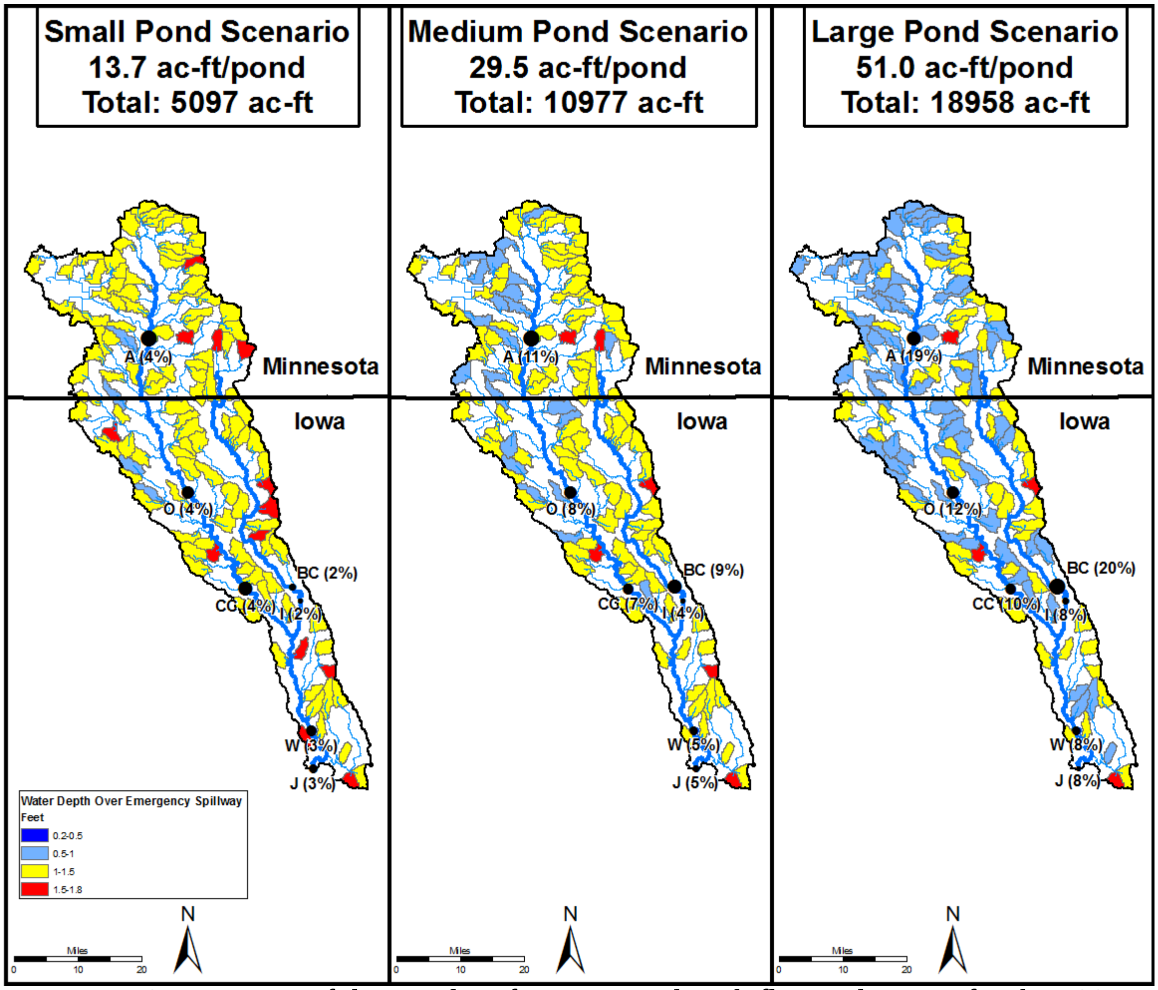

Figure A-10. Summary of dry pond performance and peak flow reductions for the 100-year, 24-hour design storm (6.81 inches of rain in 24 hours).

Chapter 11: Mitigating the Effects of High Runoff with

Increased Infiltration and Flood Storage

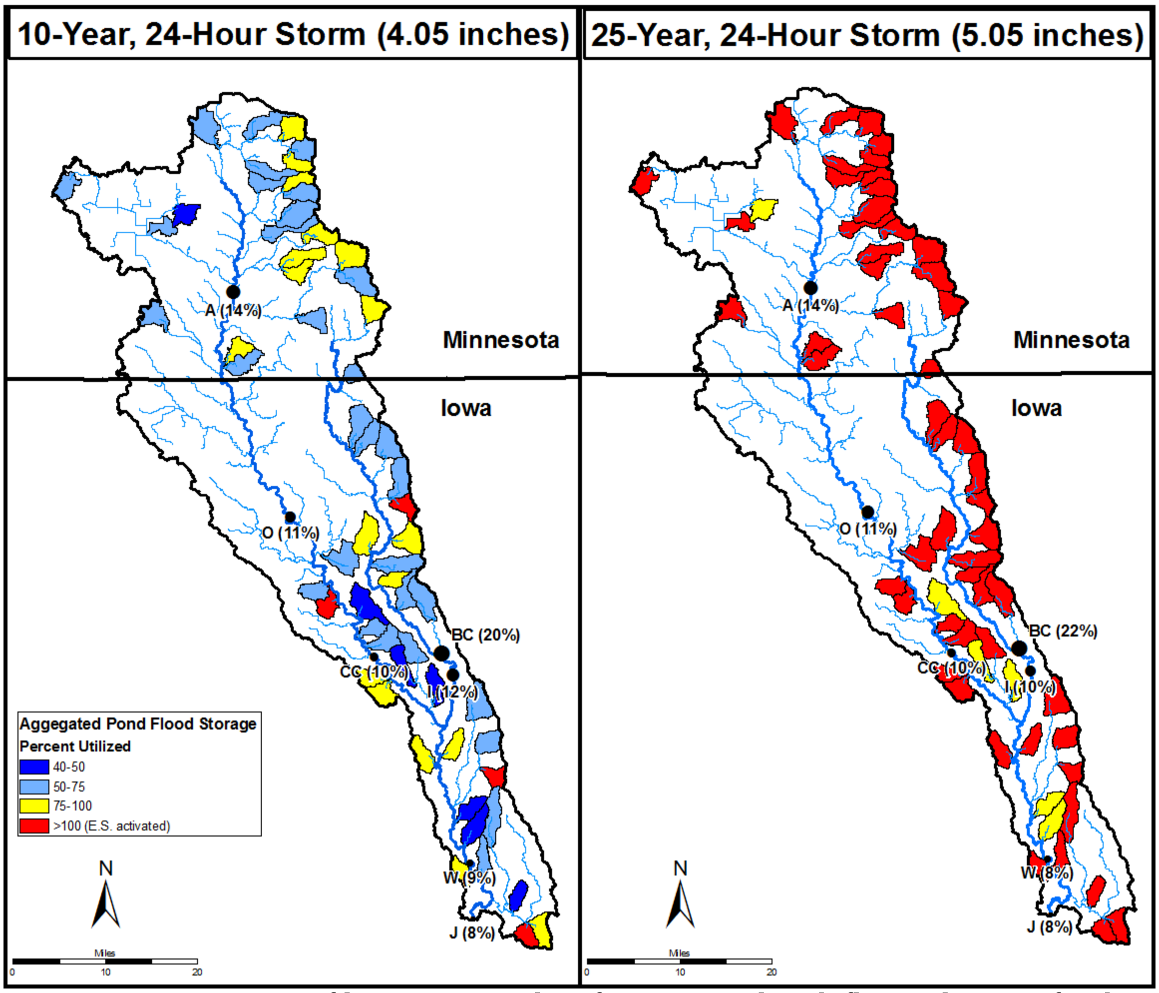

Figure A-11. Summary of large, wet pond performance and peak flow reductions for the blended scenario for the 10- and 25-year, 24-hour design storms (4.05 and 5.05 inches of rain in 24 hours).

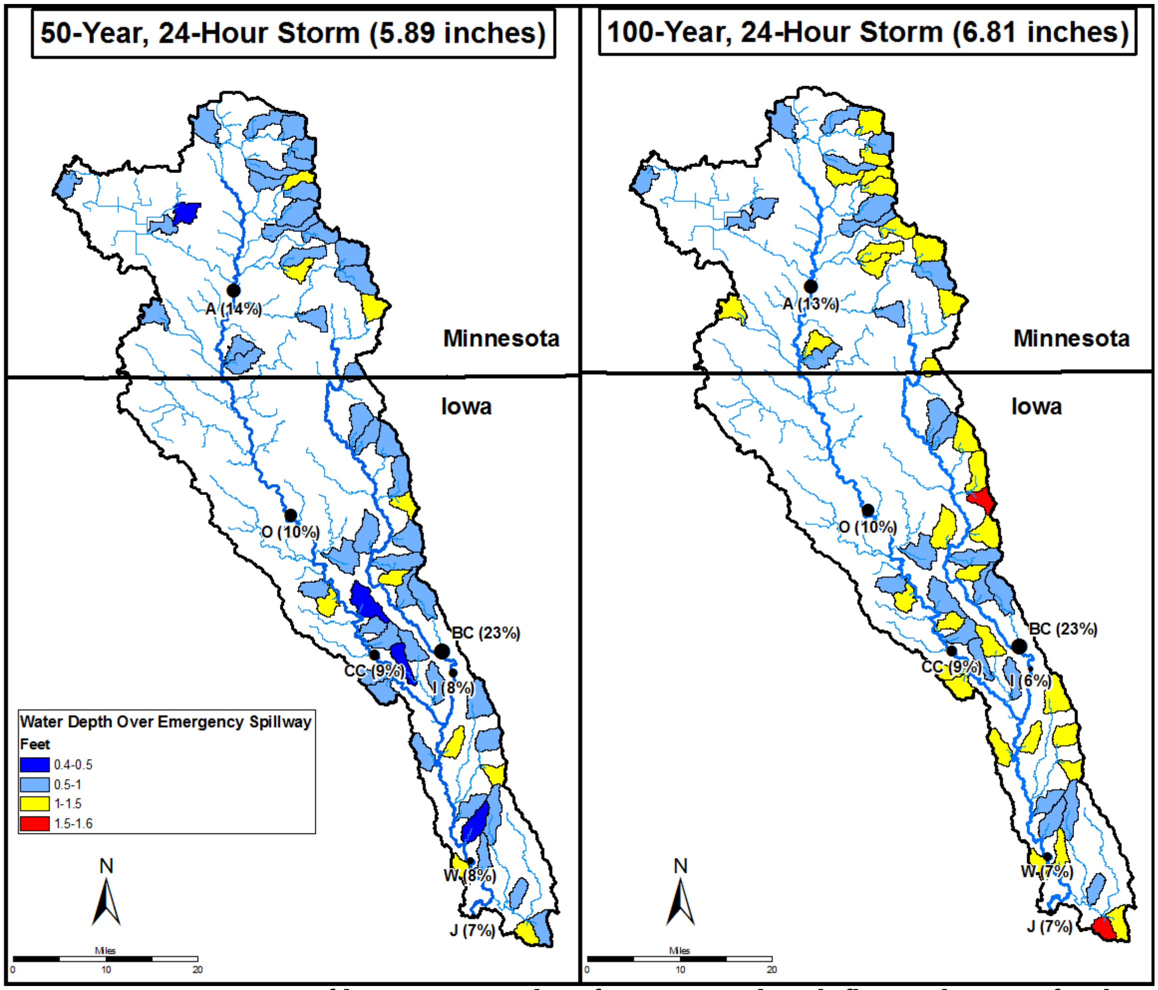

Figure A-12. Summary of large, wet pond performance and peak flow reductions for the blended scenario for the 50- and 100-year, 24-hour design storms (5.89 and 6.81 inches of rain in 24 hours).

APPENDIX B – INCORPORATED STRUCTURES

This appendix includes all stage-storage-discharge tables for existing and hypothetical ponds or reservoirs in the Upper Cedar River Watershed.

The stage-storage-discharge table for Geneva Lake is listed below.

Table B-1. Geneva Lake stage-storage-discharge table.

Elevation (ft)	Stage (ft)	Storage (ac-ft)	Discharge (cfs)	Notes
1210.5	0	0	0	Normal pool level
1210.7	0.2	394.3	3.61	
1210.9	0.4	788.6	10.32	
1211	0.5	985.75	14.48	
1211.1	0.6	1187.85	21.22	
1211.3	0.8	1592.05	38.49	
1211.5	1	1996.25	61.1	
1211.7	1.2	2400.45	88.93	
1211.9	1.4	2804.65	122.18	
1212	1.5	3006.75	139.94	
1212.1	1.6	3215.45	159.32	
1212.3	1.8	3632.85	199.37	
1212.5	2	4050.25	241.2	
1212.6	2.1	4258.95	263.94	
1212.7	2.2	4467.65	286.38	
1213	2.5	5093.75	300	Top of embankment
1213.5	3	6170.25	322	
1214	3.5	7246.75	344	Discharge is extrapolated
1220	9.5	21544.75	608	Discharge is extrapolated

The stage-storage-discharge tables used for the hypothetical pond scenarios are listed below. Each stage-storage-discharge relationship is for a single prototype pond.

Table B-2. Small pond scenario (wet pond): stage-storage-discharge table.

Elevation Above Principal Spillway (ft)	Storage (ac-ft)	Principal Spillway (cfs)	Emergency Spillway (cfs)	Total Outflow (cfs)
0	0	0	0	0
1	1.6	2.2	0	2.2
2	5.4	11.1	0	11.1
3-Emergency Spillway	10.9	11.5	0	11.5
3.5	14.3	11.7	14.0	25.7
4	18.1	11.9	40.0	51.9
4.5	22.2	12.1	80.0	92.1
5-Top of Dam	26.7	12.3	140.0	152.3
5.5	31.6	12.5	448.1	460.6
6	36.8	12.6	609.1	621.7
6.5	42.3	12.8	870.6	883.4
7	48.2	13.0	1099.7	1112.7
7.5	54.4	13.2	1211.9	1225.1
8	60.9	13.4	1363.8	1377.2
8.5	67.7	14.5	1507.0	1521.5
9	74.8	15.6	1643.1	1658.8

Table B-3. Small pond scenario (dry pond): stage-storage-discharge table.

Elevation Above Pond Bottom (ft)	Storage (ac-ft)	2-inch Pipe (cfs)	Principal Spillway (cfs)	Emergency Spillway (cfs)	Total Outflow (cfs)
0	0	0	0	0	0
1	0.2	0.13	0	0	0.1
2	1.0	0.19	0	0	0.2
3-Principal Spillway	2.8	0.24	0	0	0.2
4	4.4	0.27	2.2	0	2.5
5	8.1	0.31	11.1	0	11.4
6-Emergency Spillway	13.7	0.34	11.5	0	11.8
6.5	17.1	0.35	11.7	14.0	26.1
7	20.9	0.37	11.9	40.0	52.3
7.5	25.0	0.38	12.1	80.0	92.5
8-Top of Dam	29.5	0.39	12.3	140.0	152.7
8.5	34.4	0.40	12.5	448.1	461.0
9	39.6	0.42	12.6	609.1	622.1
9.5	45.1	0.43	12.8	870.6	883.8
10	51.0	0.44	13.0	1099.7	1113.1
10.5	57.1	0.45	13.2	1211.9	1225.6
11	63.7	0.46	13.4	1363.8	1377.7
11.5	70.5	0.47	14.5	1507.0	1522.0
12	77.6	0.48	15.6	1643.1	1659.2

Table B-4. Medium pond scenario (wet pond): stage-storage-discharge table.

Elevation Above Principal Spillway (ft)	Storage (ac-ft)	Principal Spillway (cfs)	Emergency Spillway (cfs)	Total Outflow (cfs)
0	0	0	0	0
1	1.6	2.2	0	2.2
2	5.4	11.1	0	11.1
3	10.9	11.5	0	11.5
4	18.1	11.9	0	11.9
5-Emergency Spillway	26.7	12.3	0	12.3
5.5	31.6	12.5	14.0	26.5
6	36.8	12.6	40.0	52.6
6.5	42.3	12.8	80.0	92.8
7-Top of Dam	48.2	13.0	140.0	153.0
7.5	54.4	13.2	448.1	461.3
8	60.9	13.4	609.1	622.5
9	74.8	15.6	1099.7	1115.3

Table B-5. Medium pond scenario (dry pond): stage-storage-discharge table.

Elevation Above Pond Bottom (ft)	Storage (ac-ft)	2-inch Pipe (cfs)	Principal Spillway (cfs)	Emergency Spillway (cfs)	Total Outflow (cfs)
0	0	0	0	0	0
1	0.2	0.13	0	0	0.1
2	1.0	0.19	0	0	0.2
3-Principal Spillway	2.8	0.24	0	0	0.2
4	4.4	0.27	2.2	0	2.5
5	8.1	0.31	11.1	0	11.4
5.5	10.7	0.32	11.3	0	11.6
6	13.7	0.34	11.5	0	11.8
6.5	17.1	0.35	11.7	0	12.1
7	20.9	0.37	11.9	0	12.3
7.5	25.0	0.38	12.1	0	12.5
8-Emergency Spillway	29.5	0.39	12.3	0	12.7
8.5	34.4	0.40	12.5	14	26.9
9	39.6	0.42	12.6	40	53.0
9.5	45.1	0.43	12.8	80	93.2
10-Top of Dam	51.0	0.44	13.0	140	153.4
10.5	57.1	0.45	13.2	448.1	461.8
11	63.7	0.46	13.4	609.1	623.0
11.5	70.5	0.47	14.5	870.6	885.6
12	77.6	0.48	15.6	1099.7	1115.8

Table B-6. Large pond scenario (wet pond): stage-storage-discharge table.

Elevation Above Principal Spillway (ft)	Storage (ac-ft)	Principal Spillway (cfs)	Emergency Spillway (cfs)	Total Outflow (cfs)
0	0	0	0	0
1	1.6	2.2	0	2.2
2	5.4	11.1	0	11.1
3	10.9	11.5	0	11.5
3.5	14.3	11.7	0	11.7
4	18.1	11.9	0	11.9
4.5	22.2	12.1	0	12.1
5	26.7	12.3	0	12.3
5.5	31.6	12.5	0	12.5
6	36.8	12.6	0	12.6
6.5	42.3	12.8	0	12.8
7-Emergency Spillway	48.2	13.0	0	13.0
7.5	54.4	13.2	14	27.2
8	60.9	13.4	40	53.4
8.5	67.7	14.5	80	94.5
9-Top of Dam	74.8	15.6	140	155.6
9.5	82.3	15.8	448.1	463.9
10	90.0	16.0	609.1	625.1
10.5	98.0	16.2	870.6	886.8
11	106.3	16.4	1099.7	1116.1

Table B-7. Large pond scenario (dry pond): stage-storage-discharge table.

Elevation Above Pond Bottom (ft)	Storage (ac-ft)	2-inch Pipe (cfs)	Principal Spillway (cfs)	Emergency Spillway (cfs)	Total Outflow (cfs)
0	0	0	0	0	0
1	0.2	0.13	0	0	0.1
2	1.0	0.19	0	0	0.2
3-Principal Spillway	2.8	0.24	0	0	0.2
4	4.4	0.27	2.2	0	2.5
5	8.1	0.31	11.1	0	11.4
6	13.7	0.34	11.5	0	11.8
6.5	17.1	0.35	11.7	0	12.1
7	20.9	0.37	11.9	0	12.3
7.5	25.0	0.38	12.1	0	12.5
8	29.5	0.39	12.3	0	12.7
8.5	34.4	0.40	12.5	0	12.9
9	39.6	0.42	12.6	0	13.0
9.5	45.1	0.43	12.8	0	13.2
10-Emergency Spillway	51.0	0.44	13.0	0.0	13.4
10.5	57.1	0.45	13.2	14.0	27.7
11	63.7	0.46	13.4	40.0	53.9
11.5	70.5	0.47	14.5	80.0	95.0
12-Top of Dam	77.6	0.48	15.6	140.0	156.1
12.5	85.0	0.49	15.8	448.1	464.4
13	92.7	0.50	16.0	609.1	625.6
13.5	100.8	0.51	16.2	870.6	887.3
14	109.1	0.52	16.4	1099.7	1116.6

REFERENCES

Archuleta, Ray. "Cedar River Watershed Coalition Meeting - Cover Crops and Other Conservation Practices." Personal interview. 1 Apr. 2014.

Babbar-Sebens, Meghna, Robert Barr, and Enore Tedesco. "Spatial Identification and Optimization of Upland Wetlands in Agricultural Watersheds." *Ecological Engineering* 52 (2013): 130-42. *Web of Knowledge*. Web. 8 Nov. 2013. >.

Beck, Hylke E., Richard A. De Jeu, Jaap Schellekens, Albert I. Van Dijk, and L. A. Bruijnzeel. "Improving Curve Number Based Storm Runoff Estimates Using Soil Moisture Proxies." *IEEE Journal of Selected Topics in Applied Earth Observations and Remote Sensing* 2.4 (2009): 250-59.

Blann, Kristen L., Anderson, James L., Sands, Gary R. and Vondracek, Bruce (2009) 'Effects of Agricultural Drainage on Aquatic Ecosystems: A Review', Critical Reviews in Environmental Science and Technology, 39: 11, 909-1001.

Bradley, Allen. Water Budget Analysis for the Upper Cedar River Watershed (1981-2010). 1 Mar. 2014. Raw data. IIHR-Hydroscience & Engineering, University of Iowa, Iowa City.

Chennu, S., J. Gresillon, J. Faure, E. Leblois, C. Poulard, and D. Dartus. "Flood Mitigation Strategies at Watershed Scale Through Dispersed Structural Measures." *Institute for Catastrophic Loss Reduction: 4th International Symposium on Flood Defence, Toronto, 6-8 May 2008*.

Chow, V.T., Maidment, D., and Mays, L. (1988). *Applied Hydrology*. McGraw-Hill, Inc.

Coralville Lake. Rep. Rock Island, IL: U.S. Army Corps of Engineers, 2012. Print.

Feldman, Arlen, ed. *Hydrologic Modeling System HEC-HMS: Technical Reference Manual*. Davis, CA: U.S. Army Corps of Engineers, 2000.

Gleason, Karin; "2008 Midwestern U.S. Floods"; July 2008; NOAA's National Climatic Data Center, Asheville, NC .

Hjelmfelt AT Jr, Kramer KA, Burwell RE. 1982. Curve numbers as random variables. In *Proceeding, International Symposium on Rainfall-Runoff Modeling*, Singh VP (ed.). Water Resources Publication: Littleton, CO; 365–373.

Hoeft, Claudia. "Ch.7: Hydrologic Soil Groups." *Part 630 Hydrology: National Engineering Handbook*. Washington D.C.: USDA NRS, 2007.

Huff, Floyd A., and James R. Angel. *Rainfall Frequency Atlas of the Midwest*. Illinois State Water Survey, Champaign, Bulletin 71, 1992.

Hutchinson, K.J., and Christiansen, D.E., 2013, Use of the Soil and Water Assessment Tool (SWAT) for simulating hydrology and water quality in the Cedar River Basin, Iowa, 2000–10: U.S. Geological Survey Scientific Investigations Report 2013–5002, 36 p.

Iowa DOT. "Chapter 10 - Roadside Development and Erosion Control." *Iowa DOT Design Mannual*. N.p.: n.p., 2013. 4-5. Print.

Iowa Geological Survey, Iowa DNR, Minnesota DNR, and Minnesota Dept. of Geology and Geophysics. 2003-2013.

"Iowa – Midwest Floods 2008: News & Statistics." *MCEER: Earthquake Engineering to Extreme Events*. MCEER, 2010. Web. 15 Mar. 2014. <https://mceer.buffalo.edu/infoservice/disasters/iowa-flood-news-statistics.asp>.

"Iowa Watersheds Project." *Iowa Flood Center*. Iowa Flood Center, n.d. Web. 01 Mar. 2014. <http://iowafloodcenter.org/projects/watershed-projects/>.

Kull, Daniel W., and Arlen D. Feldman. "Evolution of Clark's Unit Graph Method to Spatially Distributed Runoff." *Journal of Hydrologic Engineering* 3.1 (1998): 9-19. Web.

Lipetzky, Douglas J. *Design Report for Geneva Lake Structure Reconstruction*. Bismarck, ND: Ducks Unlimited, 2005.

Merwade, Venkatesh. "Tutorials." *Website of Venkatesh Merwade*. Venkatesh Merwade, 4 Apr. 2012. Web. 27 Apr. 2013.

Mutch, Dale R. "Cover Crop Overview." *Michigan Cover Crops*. Michigan State University, 2010. Web. 25 Mar. 2014. <http://www.covercrops.msu.edu/general/general.html>.

National Elevation Dataset. 1/3 arc-second. Sioux Falls, SD: USGS, 2009. <http://nationalmap.gov>.

National Land Cover Dataset (2006). Sioux Falls, SD: USGS, 2011. <http://nationalmap.gov>.

Perica, Sanja, Deborah Martin, Sandra Pavlovic, Ishani Roy, Michael St. Laurent, Carl Trypaluk, Dale Unruh, Michael Yekta, and Geoffrey Bonnin. "NOAA Atlas 14 Point Precipitation Frequency Estimates: IA." *Hydrometeorological Design Studies Center: Precipitation Frequency Data Server*. NOAA, 2013. Web. Nov. 2013.

Pimentel, D. 2012. Biofuels Causing Malnutrition in the World. In Global Economic and Environmental. Pimentel, David (Editor). Aspects of Biofuels. Taylor & Francis, Boca Raton, FL. Pp. 1-13.

Ponce, Victor M., and Richard H. Hawkins. "Runoff Curve Number: Has It Reached Maturity?" *Journal of Hydrologic Engineering* 1.1 (1996): 11-19.

Rapid Watershed Assessment: Upper Cedar. Des Moines, IA: NRCS, 2012.

Scharffenberg, William, and Matthew Fleming. *Hydrologic Modeling System HEC-HMS: User's Manual*. Davis, CA: U.S. Army Corps of Engineers, 2010.

Sloan, Brandon P. *Hydrologic Impacts of Tile Drainage in Iowa*. Thesis. University of Iowa, 2013.

Soil Survey Geographic (SSURGO) Database. Fort Worth: USDA-NRCS, 2012. <http://SoilDataMart.nrcs.usda.gov/>.

Sturm, Terry W. *Open Channel Hydraulics*. 2nd ed. New York: McGraw-Hill, 2010.

Takle, E.S. (2010). “Was Climate Change Involved?” Chapter in A Watershed Year: Anatomy of the Iowa Floods of 2008, C. Mutel (editor), University of Iowa Press, Iowa City, IA. pp. 111 – 116.

"30-yr Normal Precipitation: Annual (1981-2010)." PRISM Climate Group. Oregon State University, 2013. Web. Feb. 2014. <http://www.prism.oregonstate.edu/normals/>.

Villarini, Gabriele, James A. Smith, Mary Lynn Baeck, and Witold F. Krajewski, 2011. Examining Flood Frequency Distributions in the Midwest U.S. Journal of the American Water Resources Association (JAWRA) 47(3): 447-463. DOI: 10.1111/j.1752-1688.2011.00540.x.

Welvaert, Mike. "Major Historical Floods and Flash Floods in the La Crosse (ARX) Hydrologic Service Area." Historical Floods for Southeast MN, Northeast IA, and Western WI. National Weather Service, Jan. 2010. Web. 13 Nov. 2013. <http://www.crh.noaa.gov/arx/?n=historicalfloods>.

Woodward, Donald E. "Ch. 15: Time of Concentration." *National Engineering Handbook: Part 630 Hydrology*. N.p.: NRCS, 2010. 2-3.

Wunsch, Matthew J. *Distributed Storage Modeling in Soap Creek for Flood Control and Agricultural Practices*. Thesis. University of Iowa, 2013.