Hydrologic Assessment of the Upper Cedar River Watershed

July 2014

Iowa Flood Center

IIHR Hydroscience & Engineering

Hydrologic Assessment of the Upper Cedar River Watershed

July 2014

IIHR Technical Report No. 489

Prepared by:
Iowa Flood Center | IIHR—Hydroscience & Engineering
The University of Iowa
C. Maxwell Stanley Hydraulics Laboratory
Iowa City, Iowa 52242

Acknowledgments

The Iowa Flood Center and IIHR—Hydroscience & Engineering would like to thank the following individuals and agencies for providing relevant data and engaging in discussions that contributed to this assessment:

Ducks Unlimited, Inc. of Bismarck, ND
Floyd County Soil and Water Conservation District
Iowa Department of Natural Resources
Mitchell County Soil and Water Conservation District
Natural Resources Conservation Service
Upper Cedar River Watershed Management Authority

Table of Contents

List of Figures

List of Tables

Introduction

Heavy rains and subsequent flooding during the summer of 2008 brought economic, social, and environmental impacts to many individuals and communities in watersheds across the state of Iowa. In the response and recovery aftermath, a handful of Watershed Management Authorities –bodies consisting of representatives from municipalities, counties, and soil and water conservations districts – were formed locally to tackle local challenges with a unified watershed approach.

This assessment is part of the Iowa Watersheds Project, a project being undertaken in four watersheds across Iowa by the Iowa Flood Center located at IIHR—Hydroscience & Engineering on the University of Iowa campus and the respective watershed management authorities in each watershed.

The assessment begins by outlining trends and hydrologic conditions across Iowa, characterizes the conditions within the Upper Cedar River watershed and compares local conditions to those in three other Iowa watersheds – the Middle Raccoon River, the Turkey River, and Soap/Chequest Creeks.

A hydrologic model of the Upper Cedar River Watershed, using HEC-HMS, was used to identify areas in the watershed with high runoff potential and run simulations to help understand the potential impact of alternative flood mitigation strategies in the watershed. Focus for the scenario development was placed on understanding the impacts of (1) increasing infiltration in the watershed through land use change and application of cover crops and (2) implementing a system of distributed storage projects (ponds) across the landscape.

The assessment is meant to provide local leaders, landowners and watershed residents in the Upper Cedar River Watershed an understanding of the hydrology within the watershed and the potential impact of various hypothetical flood mitigation strategies. The hydrologic assessment provides watershed residents and community leaders an additional source of information and should be used in tandem with additional reports and watershed plans working to enhance the social, economic, and environmental sustainability and resiliency of the Upper Cedar River Watershed.

1. Iowa's Flood Hydrology

This chapter illustrates facts about Iowa's water cycle and flood hydrology across the state. Historical records for precipitation and streamflow are examined to describe how much precipitation falls, how that water moves through the landscape, when storms typically produce river flooding, and how Iowa's hydrology has changed over the past decades and century. As the context for this discussion, we examine the water cycle of the Upper Cedar River Watershed, as well as that for the three other Iowa watersheds participating in the Iowa Watersheds Project (see Figure 1.1).
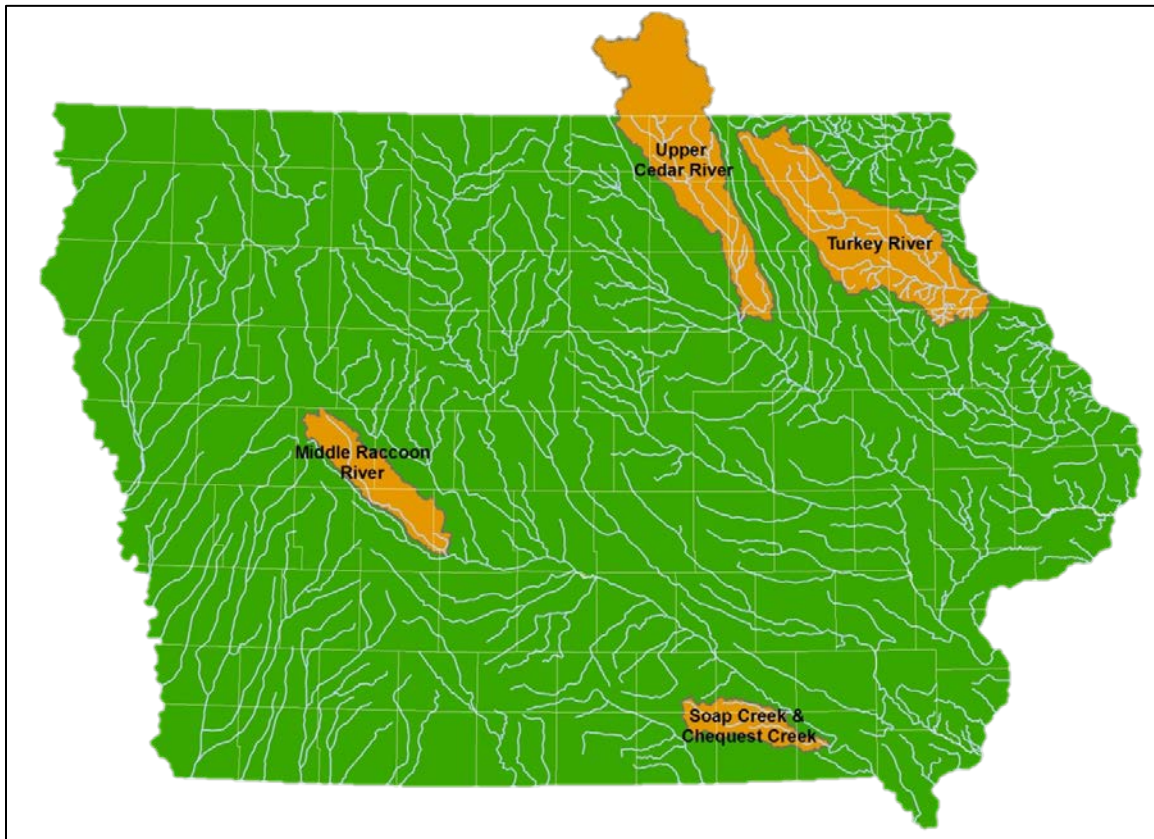
Figure 1.1. Iowa Watersheds Project study areas.

The Upper Cedar begins in Minnesota, and drains 1,661 square miles (mi²) — mostly from the Iowan Surface landform (USGS 05458500 Cedar River at Janesville). The Turkey River (USGS 05412500 Turkey River at Garber) drains 1,545 mi², and includes portions of the Iowan Surface and karst topography of the Paleozoic Plateau. The Middle Raccoon River drains 375 mi² (USGS 05483450 Middle Raccoon River near Bayard), and is located in the west-central part of the state. The upper part of the Middle Raccoon basin is located in flat terrain of the Des Moines Lobe, while the lower part is located within the Southern Iowa Drift Plain. Soap and Chequest Creeks in the southern part of the state are located in the Southern Iowa Drift Plain.

a. Hydrology in Iowa and the Iowa Watersheds Project Study Areas

i. Statewide Precipitation

Iowa's climate is marked by a smooth transition of annual precipitation from the southeast to the northwest (see Figure 1.2). The average annual precipitation reaches 40 inches in the southeast corner, and drops to 26 inches in the northwest corner. Of the four Iowa Watersheds Project study areas, Soap/Chequest along the southern border has the largest annual precipitation (38.8 inches), followed by the Turkey River (36.3 inches) and the Upper Cedar River (35.1 inches) in the northeast portion of the state, and then the Middle Raccoon (35.0 inches) in the western half of the state.
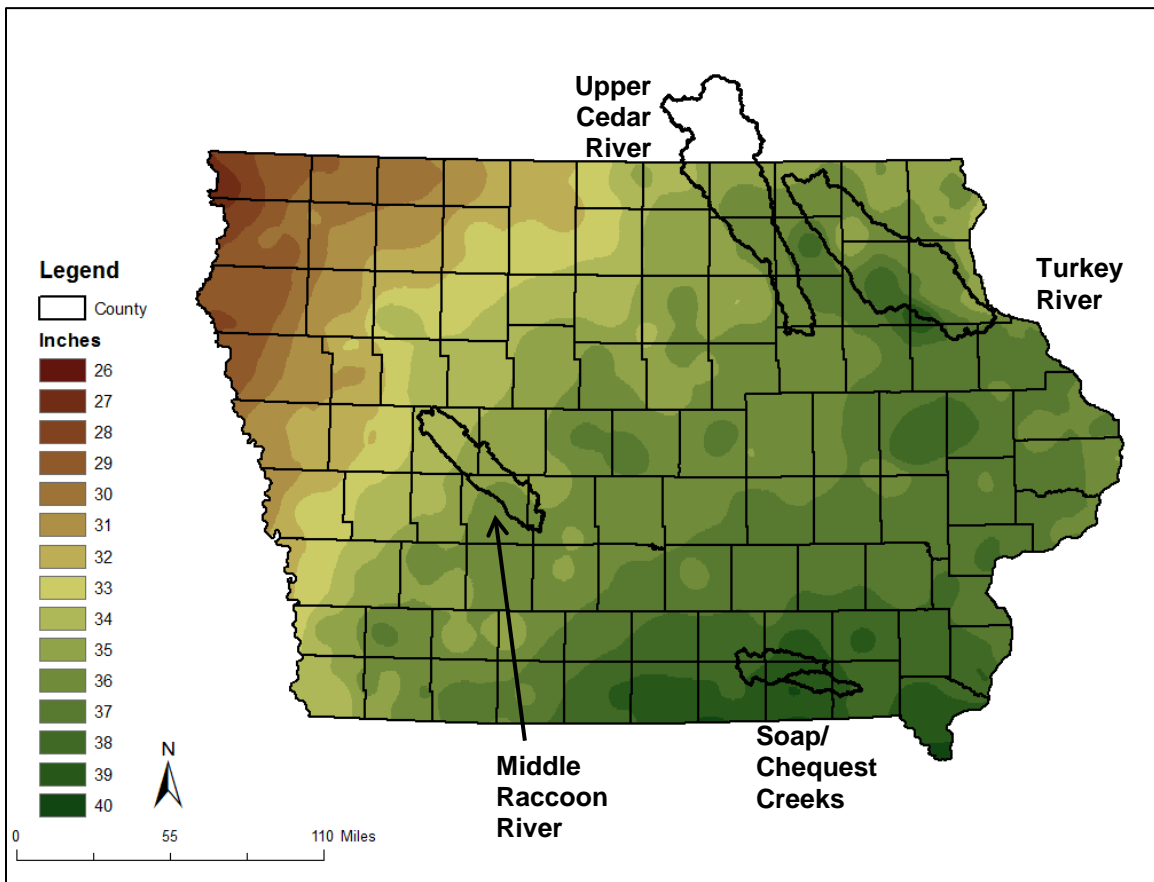
Figure 1.2. Average annual precipitation for Iowa. Precipitation estimates are based on the 30-year annual average (1981-2010) for precipitation gauge sites. Interpolation between gauge sites to an 800 m grid was done by the PRISM (parameter-elevation relationships on independent slopes model) method. (Data source: http://www.prism.oregonstate.edu/).

ii. The Water Cycle in Iowa

Of the precipitation that falls across the state, most of it evaporates into the atmosphere — either directly from lakes and streams, or by transpiration from crops and vegetation. What does not evaporate drains into streams and rivers (see Table 1.1).

Table 1.1. Iowa water cycle for four watersheds. The table shows the breakdown of the average annual precipitation (100% of the water in each watershed).

	Precipitation (%)	*Evaporation (%)*	*Surface Flow (%)*	*Baseflow (%)*
Middle Raccoon	100	73.5	8.9	17.5
Upper Cedar	100	68.5	9.8	21.7
Turkey	100	69.4	9.0	21.6
Fox[1]	100	69.2	19.2	11.6

Evaporation
In Iowa, the majority of water leaves by evaporation; for the four Iowa watershed study areas, evaporation accounts for about 68% of precipitation in the Upper Cedar, and 69% in the Fox and Turkey Rivers. As one moves westward in the state, a larger fraction evaporates; for the Middle Raccoon, evaporation accounts for almost 74% of the precipitation.

Surface Flow
The precipitation that drains into streams and rivers can take two different paths. During rainy periods, some water quickly drains across the land surface, and causes streams and rivers to rise in the hours and days following the storm. This portion of the flow is often called "surface flow", even though some of the water may soak into the ground and discharge later (e.g., a tile drainage system).

Baseflow
The rest of the water that drains into streams and rivers takes a longer, slower path; first it infiltrates into the ground, percolates down to the groundwater, and then slowly moves towards a stream. The groundwater eventually reaches the stream, maintaining flows in a river even during extended dry periods. This portion of the flow is often called "baseflow".

A watershed's geology helps determine the partitioning of precipitation runoff into surface flow and baseflow. The Turkey River has the largest ratio of baseflow to surface flow (2.4): about 22% of precipitation leaves as baseflow, and 9% leaves as surface flow. Most likely, the karst limestone geology in portions of the watershed (with its enhanced surface drainage) contributes to a higher baseflow ratio. The ratio of baseflow to surface flow is slightly lower in the Upper Cedar (2.2), with its 22% baseflow and 10% surface flow, and the Middle Raccoon (2.0), with its 17% baseflow and 9% surface flow. For the Fox River, the partitioning is reversed; more water

---

[1] Both Soap and Chequest Creek watersheds are ungauged, so historical records of streamflow are unavailable. However, the adjoining Fox River watershed, located directly south of Soap and Chequest Creek, has a long streamflow record (USGS 05495000 Fox River at Wayland, drainage area of 400 mi²); we will use the flow records at the adjoining Fox River as an indicator of the hydrology in this portion of the state.

leaves as surface flow (19%) than as baseflow (12%), so its baseflow ratio is less than one (0.6). This region consists of loess ridges and glacial till side slopes; steep slopes move water quickly to the valley, and those locations with flatter slopes typically contain high clay contents (42 to 48% in the subsoil) that limit infiltration in the ground. Figure 1.3 illustrates the water cycle components for the four Iowa watersheds, and clearly illustrates that the Fox is a more surface flow dominated river.
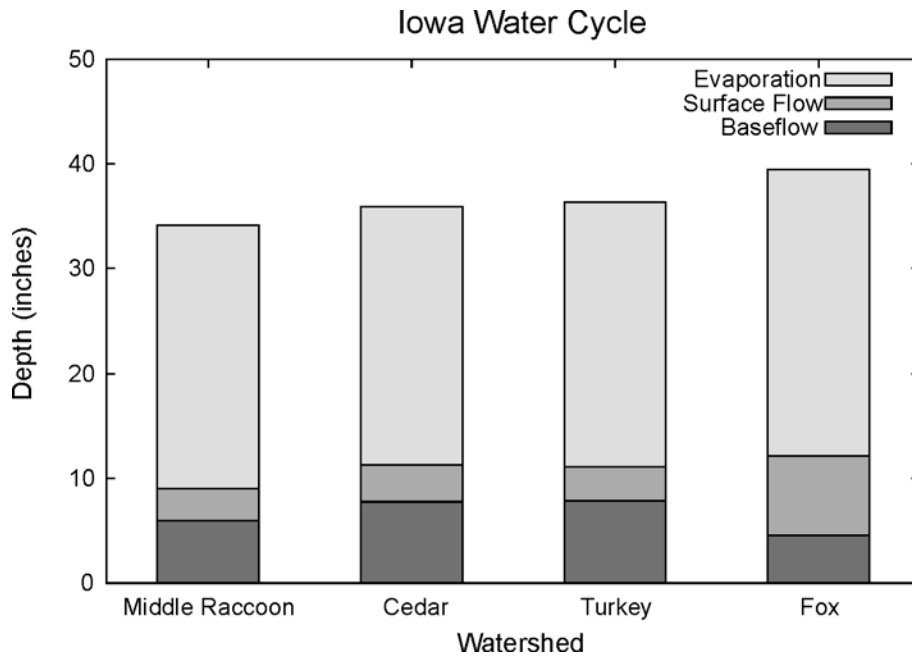
Figure 1.3. Iowa water cycle for four watersheds. The chart shows the partitioning of the average annual precipitation depth (in inches) into evaporation, surface flow, and baseflow components.[2]

---

[2] The average annual precipitation estimates are based on the 30-year averages for the state (see Figure 1.2). Flow records were obtained for USGS stream-gages for the same 30-year period (1981-2010); a continuous baseflow separation filter was used to estimate the surface flow and baseflow components. Evaporation was estimated by water budget analysis.

iii. Monthly Water Cycle

Across the state, Iowa watersheds exhibit a similar cycle of average monthly precipitation and streamflow (see Figure 1.4). Precipitation is at its lowest in winter months; still, the precipitation is often in the form of snow, and can accumulate within the watershed until it melts (especially in the northernmost watersheds). Spring is marked by an increase in precipitation, the melting of any accumulated winter snow, and low evaporation before the growing season begins; these factors combine to produce high springtime streamflows.

Northern watersheds tend to see their peak average monthly streamflow in early spring (March or April), as snow accumulation and melt is more pronounced; southern watersheds tend to see their peak in late spring or summer (May and June). As crops and vegetation evaporate more and more water as we enter the summer months, moisture in the soil is depleted and the average monthly streamflow decreases (even though average monthly rainfall amounts are relatively high).
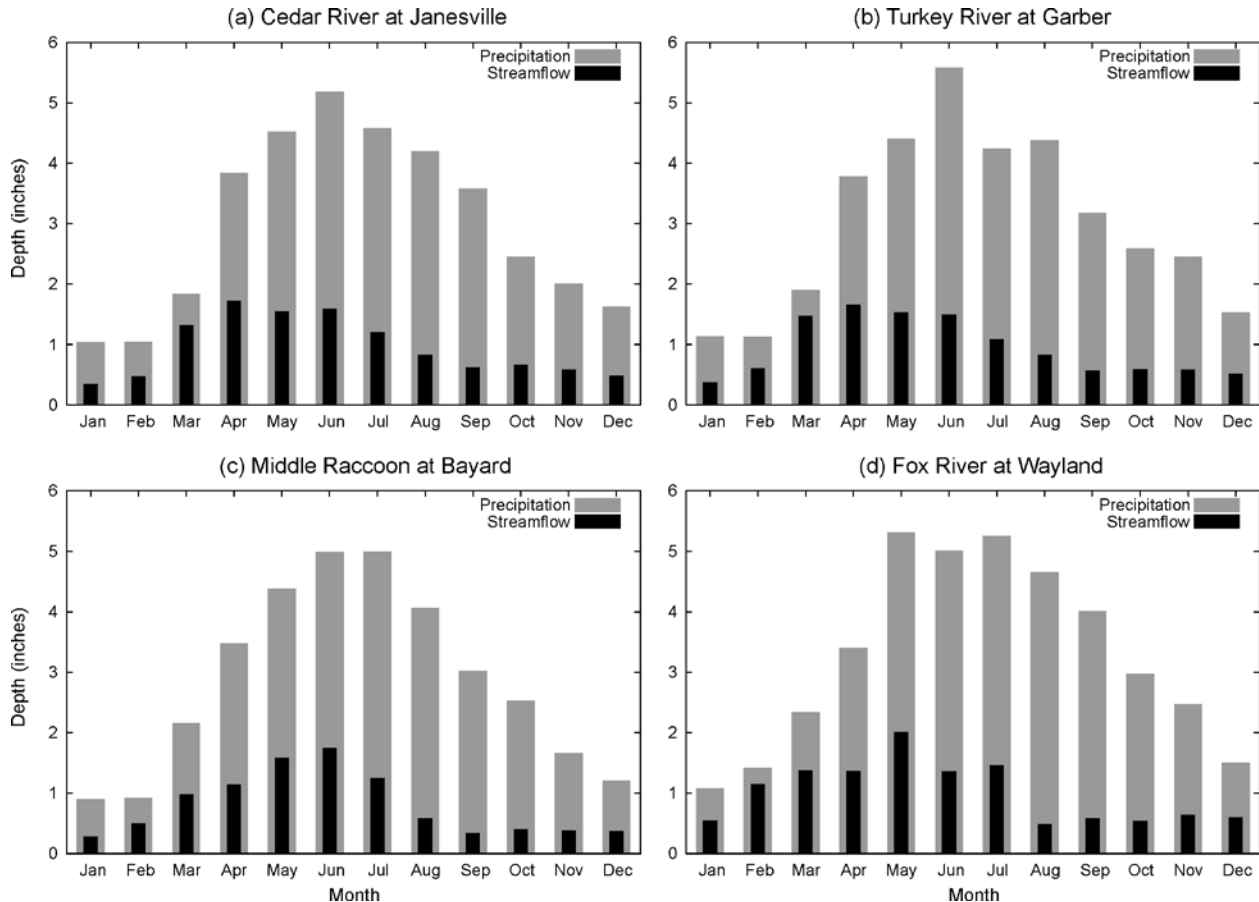
Figure 1.4. Monthly water cycle for four Iowa watersheds. The plots show the average monthly precipitation (in inches) and the average monthly streamflow (in inches). The average monthly estimates for precipitation and streamflow are based on the same 30-year period (1981-2010).

iv. Flood Climatology

The largest flows observed in Iowa's rivers follow a slightly different seasonal pattern. Figure 1.5 shows the annual maximum peak discharges (or the largest stream flow observed each year) and the calendar day of occurrence.
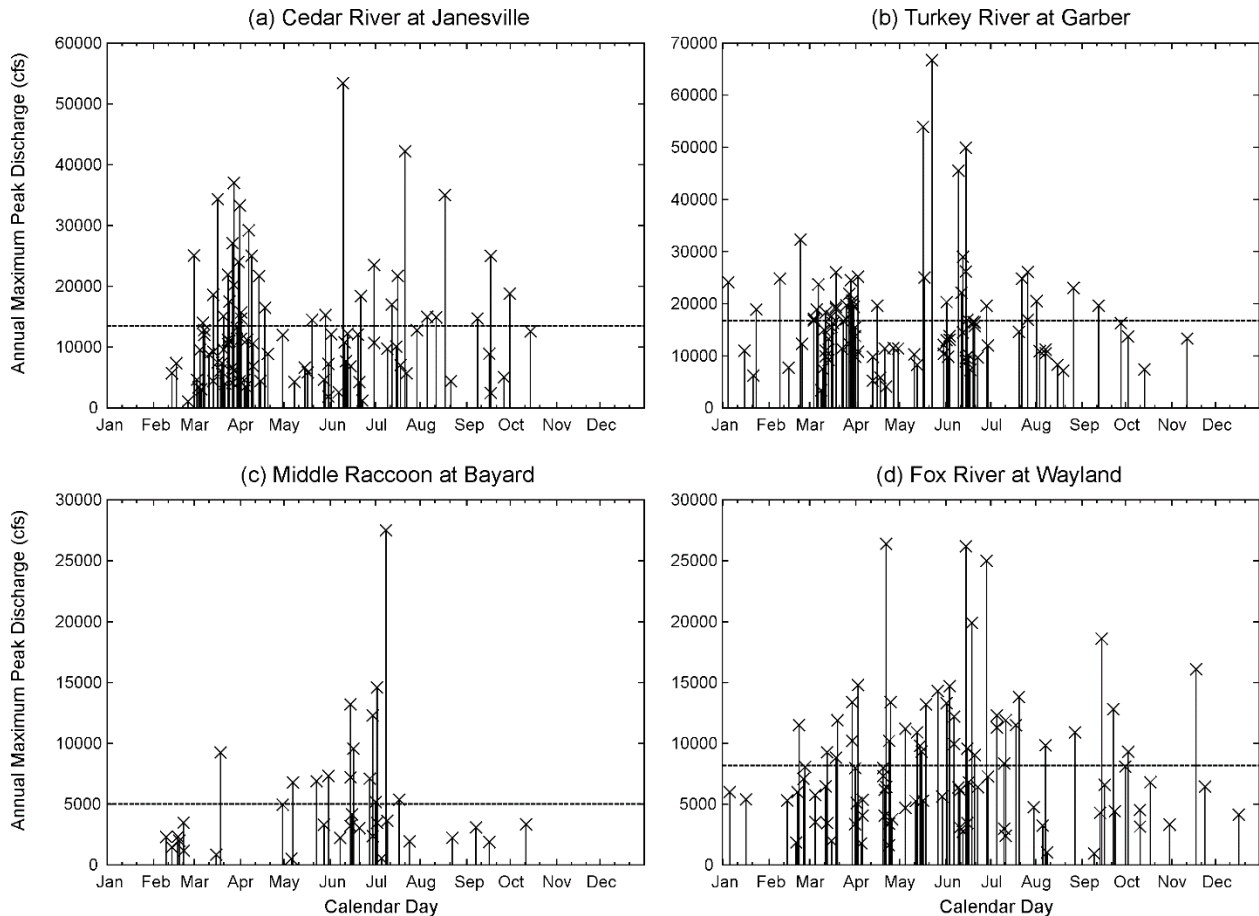
Figure 1.5. Annual maximum peak discharges and the calendar day of occurrence for four Iowa watersheds. The plots show all annual maximums for the period of record at four USGS stream-gage sites: (a) Cedar River at Janesville, (b) Turkey River at Garber, (c) Middle Raccoon at Bayard, and (d) Fox River at Wayland. The mean annual flood for each site is shown by the horizontal line.

For the northernmost watersheds (Cedar and Turkey), annual maximums often occur in March or April. These maximums may be associated with snow melt, rain on snow events, or heavy spring rains when soils are often near saturation. Still, the largest annual maximums all occurred in the summer season, when the heaviest rainstorms occur.

In contrast, the majority of all annual maximums occur in summer for the Middle Raccoon. For the Fox River, annual maximums are more evenly distributed throughout the year; as noted earlier, this river is surface flow dominated, and whenever heavy rainfall occurs during the year, large river flow can occur. Like the northernmost basins, both the Middle Raccoon and the Fox River see their largest annual maximums in the summer.

In addition to the annual maximums, Figure 1.5 also shows the mean annual flood for each river (the average of the annual maximums). For most rivers, the mean annual flood serves as a good

approximate threshold for flooding. As can be seen, there are many years when the annual maximum peak discharge is not large enough to produce a flood. Figure 1.6 shows an estimate of the occurrence frequency for flood events (annual maximums that exceed the mean annual flood).
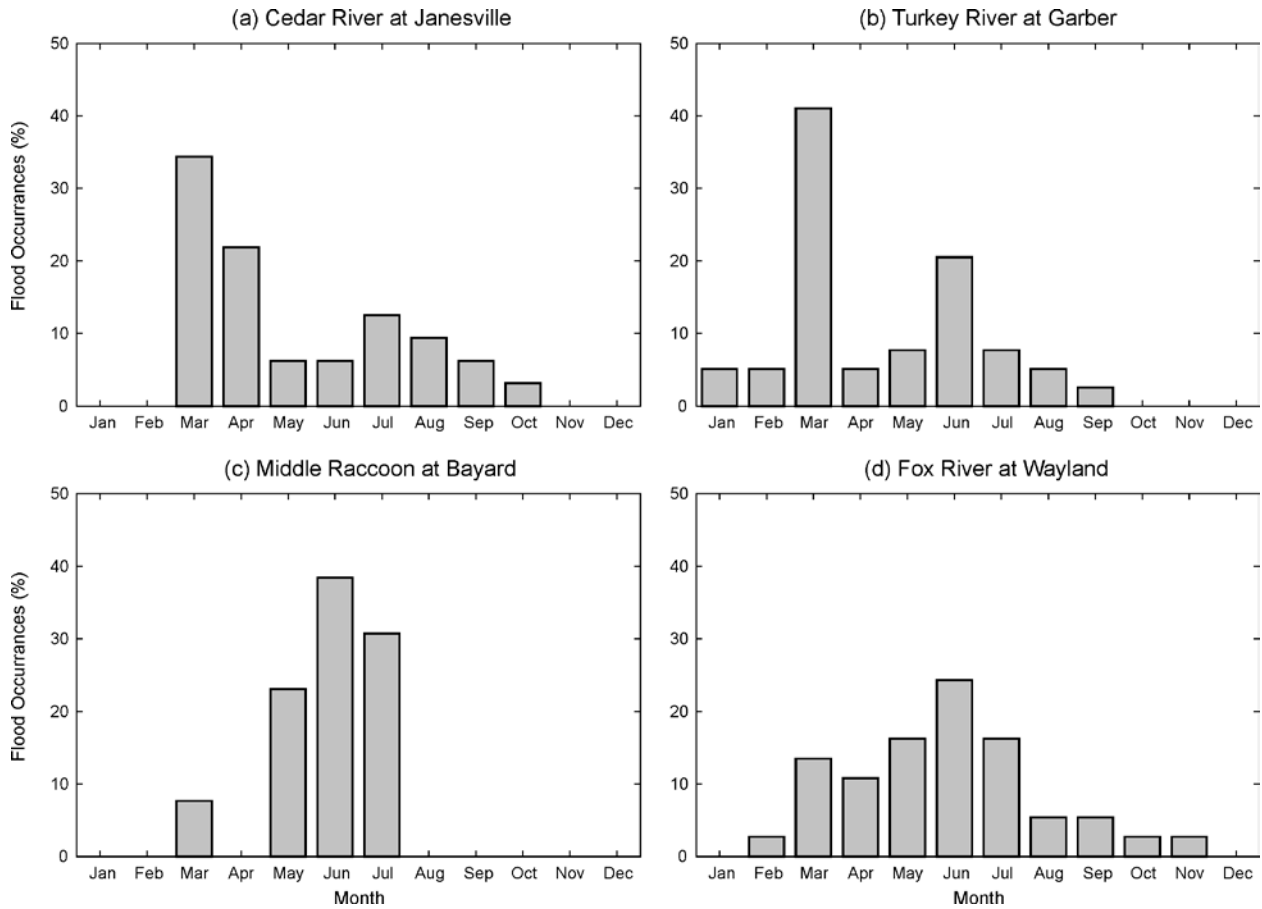
Figure 1.6. Flood occurrence frequency by month for four Iowa watersheds. The plots show the percent of peak annual discharges for a given month that exceed the mean annual flood at four USGS stream-gage sites: (a) Cedar River at Janesville, (b) Turkey River at Garber, (c) Middle Raccoon at Bayard, and (d) Fox River at Wayland.

For the northernmost watersheds (Cedar and Turkey), the peak of flood occurrences is March. Both have a smaller secondary peak in summer. For the Middle Raccoon, nearly all the flood flows have occurred in late spring to early summer (May to July). Floods have occurred in all months except December and January in the Fox River watershed, although the peak flood occurrence is also in the late spring to early summer.

b. Hydrological Alterations in Iowa and the Iowa Watersheds Project Study Areas

Although the hydrologic conditions presented for the Iowa Watersheds Project study areas illustrate the historical water cycle, the watersheds themselves are not static; historical changes have occurred that have altered the water cycle. In this section, we discuss the hydrological alterations of Iowa's watersheds, and look for evidence of these alterations in long-term streamflow records.

i. Hydrological Alterations from Agricultural-Related Land Use Changes

The Midwest, with its low-relief poorly-drained landscape, is one of the most intensively managed areas in the world (Pimentel, 2012). With European-descendent settlement, most of the land was transformed from low-runoff prairie and forest to higher-runoff farmland. Within Iowa, the land cover changes in the first decades of settlement occurred at an astonishing rate (Wehmeyer et al., 2011). Using land cover information obtained from well-documented studies in 1859, 1875, and 2001, Wehmeyer et al. (2011) estimated that the increase in runoff potential in the first thirty years of settlement represents the majority of predicted change in the 1832 to 2001 study period.

Still, other transformations associated with an agricultural landscape have also impacted runoff potential (see Table 1.2). For example, the introduction of conservation practices in the second half of the 20th century tend to reduce runoff, as suggested by a recent study of an Iowa watershed (Papanicolaou). The Conservation Reserve Program (CRP) originally began in the 1950s. Many programs were established in the 1970s to remove lands from agricultural production and establish native or alternative permanent vegetative cover; in an effort to reduce erosion and gulley formation, practices such as terraces, conservation tillages, and contour cropping were also encouraged. The Farm Bill of 1985 was the first act that officially established the CRP as we know it today, followed by expanded activities through the Bills of 1990, 1996, 2002, and 2008. The timeline of agriculture-driven land use changes and its impacts on local hydrology are summarized in Table 1.2.

Table 1.2. Agricultural-related alterations and hydrologic impacts.

Timeline	Land use status, change & interventions	Hydrologic effect(s)	Source
1830s - Prior	Native vegetation (tallgrass prairies and broad-leaved flowering plants) dominate the landscape	Baseflow dominated flows; slow response to precipitation events	Petersen (2010)
1830-1980	Continuous increase of agricultural production by replacement of perennial native vegetation with row crops 1940: <40% row crop (Raccoon) 1980: 75% row crop (statewide)	Elimination of water storage on the land; acceleration of the upland flow; expanded number of streams; increased stream velocity	Jones & Schilling (2011); Knox (2001)
1820-1930	Wetland drainage, stream channelization (straightening, deepening, relocation) leading to acceleration of the rate of change in channel positioning	Reduction of upland and in-stream water storage, acceleration of stream velocity	Winsor (1975); Thompson (2003); Urban & Rhoads (2003)
1890- 1960 2000-present	Reduction of natural ponds, potholes, wetlands; development of large-scale artificial drainage system (tile drains)	Decrease of water storage capacity, groundwater level fluctuations, river widening	Burkart (2010); Schottler et al. (2013)
1940-1980	Construction of impoundments and levees in Upper Mississippi Valley	Increased storage upland	Sayre (2010);
1950-present	Modernization/intensification of the cropping systems	Increased streamflow, wider streams	Zhang & Schilling (2006); Schottler et al. (2013)
1970-present	Conservation practices implementation: Conservation Reserve Program (CRP); Conservation Reserve Enhancement Program (CREP); Wetland Reserve Program (WRP)	Reduction of runoff and flooding; increase of upland water storage	Castle (2010); Schilling (2000); Schilling et al. (2008);
2002- present	62% of Iowa's land surface is intensively managed to grow crops (dominated by corn and soybeans up to 63% of total)	About 25% to 50% of precipitation converted to runoff (when tiling is present)	Burkart (2010)

ii. Hydrological Alterations Induced by Climate Change

Over periods ranging from decades to millions of years, Iowa has seen significant changes to its climate. Studies show that since the 1970s, Iowa and the Midwest have seen increases in annual and seasonal precipitation totals, and changes in the frequency of intense rain events and the seasonality of timing of precipitation (Takle, 2010). Large increases in runoff and flood magnitudes in the north central U.S. (including Iowa) have prompted scientific inquiries to unequivocally attribute these changes to driving factors (Ryberg et al., 2012). Although recent agricultural land use changes, such as the transition from perennial vegetation to seasonal crops, is an important driver (Schilling et al, 2008; Zhang and Schilling, 2006), other investigations show that climate-related drivers may be an equal or more significant contributor to recent hydrologic trends (Ryberg et al., 2012; Frans et al, 2013).

iii. Hydrological Alterations Induced by Urban Development

Although Iowa remains an agriculural state, a growing portion of its population resides in urban areas. The transition from agricultural to urban land uses has a profound impact on local hydrology, increasing the amount of runoff, the speed at which water moves through the landscape, and the magnitude of flood peaks. The factors that contribute to these increases (Meierdiercks et al., 2010) are the increase in the percentage of impervious areas within the drainage catchment and its location (Mejia et al., 2010), and the more efficient drainage of the landscape associated with the constructed drainage system — the surface, pipe, and roadway channels that add to the natural stream drainage system. Although traditional stormwater management practices aim to reduce increased flood peaks, urban areas have long periods of high flows that can erode its stream channels and degrade aquatic habitat.

iv. Detecting Streamflow Changes in Iowa's Rivers

Hydrologic alterations in Iowa watersheds were tested through the analysis of changes in the long-term flow at the stream-gaging sites. The identification of statistically significant shifts in the flow time series was made using the approach developed by Villarini et al. (2011). Figure 1.7 shows the results of the analysis for mean daily discharge for the four Iowa watersheds. Note the stream-gage record for the Middle Raccoon River at Bayard does not begin until 1980, so analysis results are shown for the downstream stream gage for the Raccoon River at Van Meter, where the record spans 96 years.
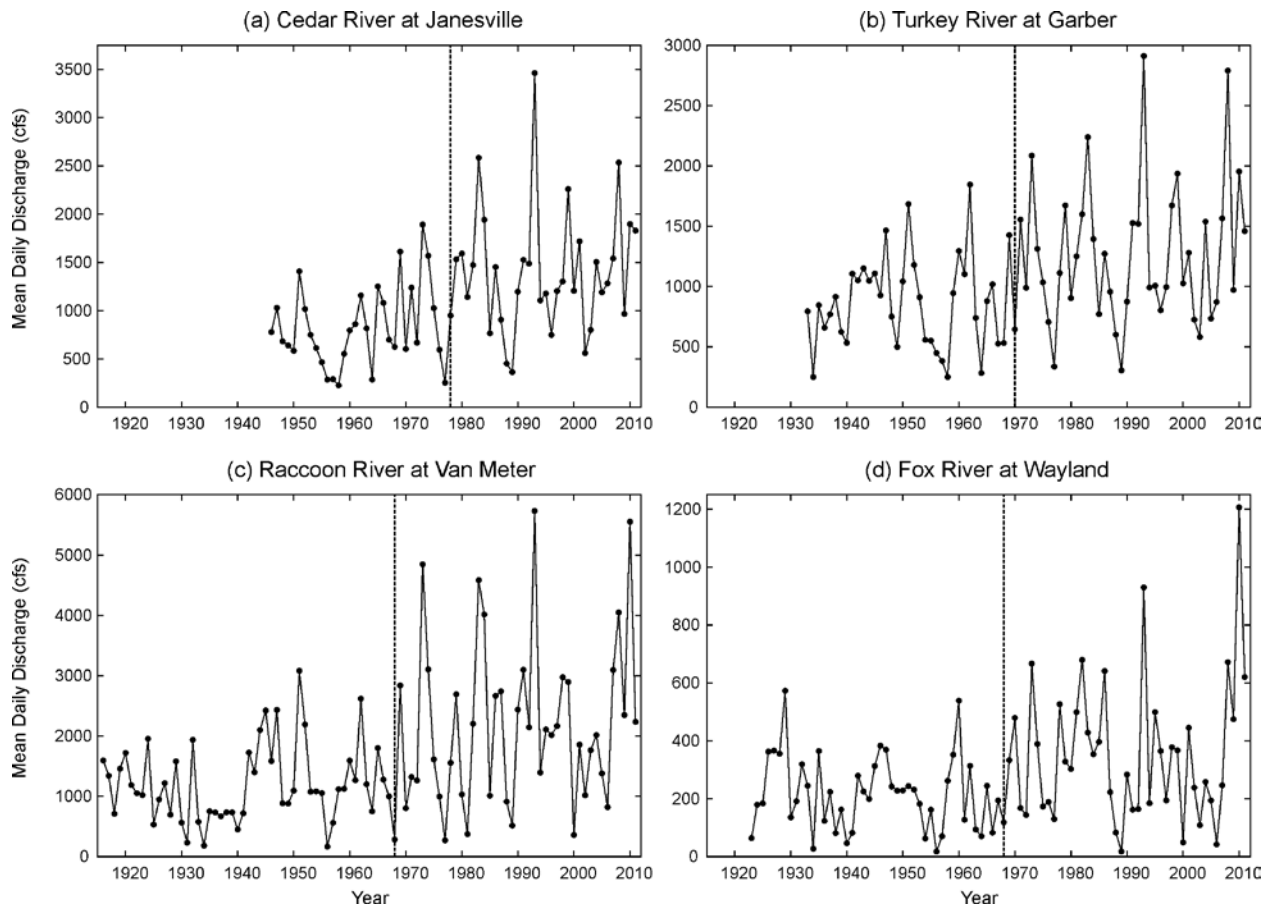
Figure 1.7. Time series of mean daily discharge for the period of record. An analysis was carried out to detect changes in the statistical characteristics of mean daily discharge; the vertical dashed lines indicate the location of any identified change point.

All four watersheds have statistically significant changes in mean daily discharge, occurring between 1968 and 1978. Streamflow since the 1970s is slightly higher than before, and its year-to-year variability has increased noticeably. The trends seen in the Iowa Watersheds Project study areas are common among many Iowa watersheds. Similar outcomes are observed for a measure of low flows (the 5% daily discharge for the year); all the detected changes occur within the narrow period between 1968 and 1972. Changes in a measure of high flows (the maximum daily discharge for the year) are not as clear. No statistically significant changes were detected for two watersheds (Cedar and Turkey); for the Raccoon, changes were detected in 1943, and in 1978 for the Fox River. Still, the general tendencies observed for mean and low flows — increased flow amounts and greater variability in the last 40 years — are also observed for high flows, even if the changes are not statistically significant.

Overall, the evidence suggests that Iowa (and elsewhere in the Midwest) has experienced long-term changes in the nature of streamflow (around 1970). The reasons for these changes is still the subject of intense on-going research (e.g., Mora et al., 2013; Frans et al, 2013; Shawn et al., 2013; Yiping et al., 2013). Still, Iowans have all seen the impacts of increased and more highly variable flows; the widespread flooding in 1993 and 2008 mark two visible examples.

c. Summary of Iowa's Flood Hydrology

The hydrologic assessment begins by looking at the historical conditions within Iowa watersheds, and moves on to predicting their flooding characteristics. Ultimately, for watersheds to prevent flooding, large- and small-scale mitigation projects directed towards damage reduction will be proposed and implemented. In many instances, projects aim to change the hydrologic response of the watershed, e.g., by storing water temporarily in ponds, enhancing infiltration and reducing runoff, etc. Such changes have (and are designed to have) significant local water cycle effects; cumulatively, the effects of many projects throughout the watershed can also have impacts further downstream.

Still, it is important to recognize that all Iowa watersheds are undergoing alterations — changes in land use, conservation practices, increases in urban development, and changes in weather with a changing climate. Therefore, a watershed-focused strategy, which considers local interventions and their impacts on the basin as a whole, within the historical context of a changing water cycle, is needed for sound water resources planning.

2. Conditions in the Upper Cedar River Watershed

This chapter provides an overview of the current Upper Cedar River Watershed conditions including hydrology, geology, topography, land use, hydrologic/meteorologic instrumentation, as well as a summary of previous floods of record. Detailed maps of related material can be found in Appendix A.

a. Hydrology

The Upper Cedar River Watershed begins in southeast Minnesota and flows south into northern Iowa where its confluence with the Shell Rock River forms the main stem of the Cedar River approximately four river miles downstream of Janesville, IA (see Figure 2.1). The watershed as defined by its eight-digit hydrologic unit code (HUC 8) 07080201 drains 1,685 mi². The watershed extends over 10 counties, four in Minnesota and six in Iowa. The Upper Cedar River has one main tributary, the Little Cedar River, which flows north to south and drains 311 mi² along the eastern part of the watershed.
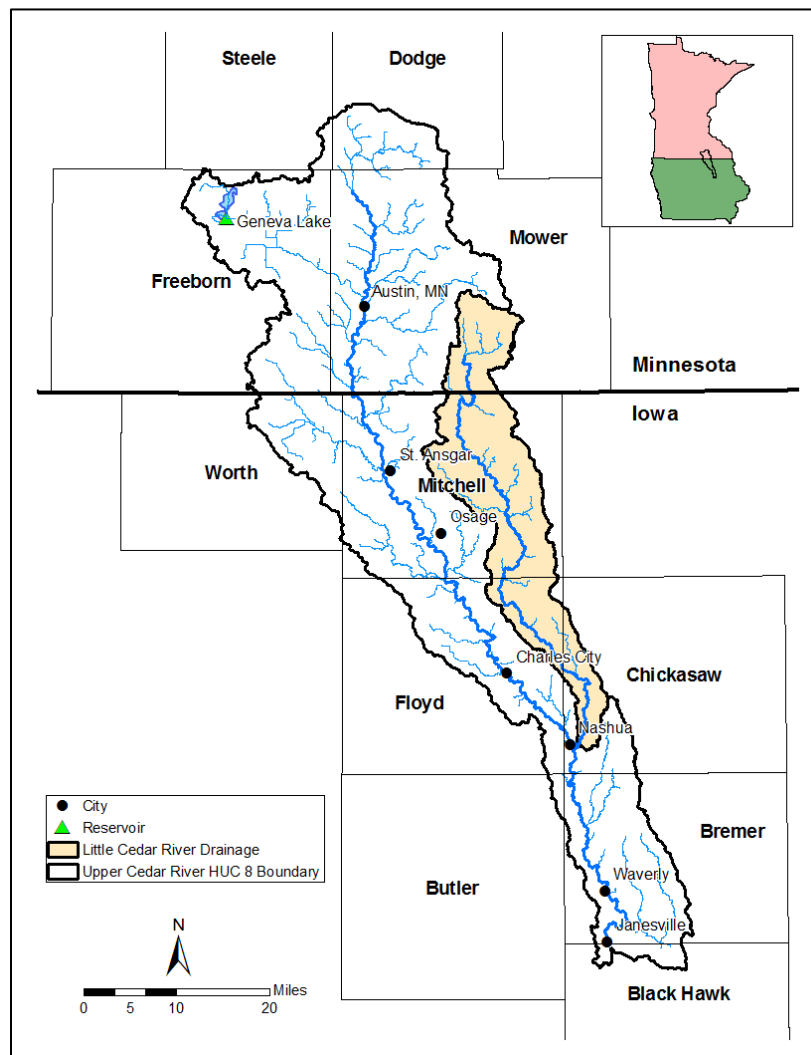
Figure 2.1. Overview of the Upper Cedar River Watershed (HUC 07080201). The watershed drains 1,685 mi² and the Little Cedar River is the largest tributary.

b. Geology and Soils

The watershed encompasses two primary geologic landform regions, the Iowan Surface and the Des Moines Lobe. The Iowan Surface, covering the majority of the watershed (80.2%) and composed primarily of glacial drift and loess, is characterized by low relief and well developed drainage. The Des Moines Lobe, composed primarily of glacial till, covers a smaller portion (19.8%) along the western part of the basin and is characterized by low relief and poorly drained soils (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 2,132 documented sinkholes in the watershed can have a major impact on both the basin hydrology and water quality. The geographic landform regions, sinkhole locations, and depth to bedrock in the Upper Cedar River Watershed are shown in Figure 2.2.
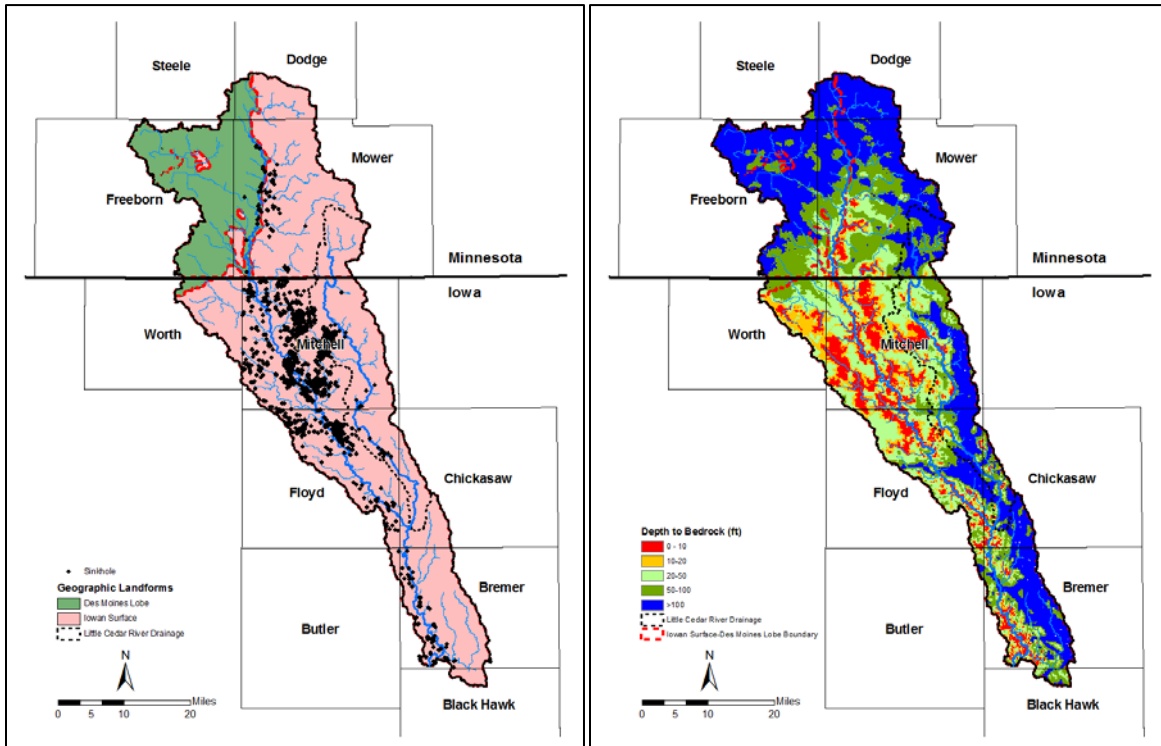
Figure 2.2. Geologic features of the Upper Cedar River Watershed. The left figure shows the two geographic landform regions the watershed encompasses (Iowan Surface and Des Moines Lobe) and the 2,132 sinkholes documented in the watershed. The right figure shows the depth to bedrock.

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 have the highest runoff potential (lowest infiltration capacity). A sand or gravel classifies as an A-type soil whereas a clay or silt is defined as a C or D-type soil. In addition, there are dual code soil classes A/D, B/D,

and C/D that are assigned to certain wet soils. In the case of these soil groups, even though the soil properties may be favorable to allow 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 water table were to be drained away or lowered, but the higher runoff potential of a D-type soil if it is not. Table 2.1 summarizes some of the properties generally true for each HSG A-D. This table is meant to provide a general description of each HSG and is not all inclusive. Complete descriptions of HSGs can be found in Chapter 7 of the National Engineering Handbook (Part 630).

Table 2.1. Soil properties and characteristics generally true for Hydrologic Soil Groups A-D.

Hydrologic Soil Group	Runoff Potential	Soil Texture(s)	Composition	Minimum Infiltration Rate[1] (in/hr)
A	Low	Sand, gravel	< 10% clay > 90% sand/gravel	> 5.67
B	Moderately low	Loamy sand, sandy loam	10-20% clay 50-90% sand	1.42-5.67
C	Moderately high	Loam containing silt and/or clay	20-40% clay <50% sand	0.14-1.42
D	High	Clay	>40% clay <50%	<0.14

[1] For HSG A-C, infiltration rates based on a minimum depth to any water impermeable layer and the ground water table of 20 and 24 inches, respectively.

Figure 2.3 shows the distribution of HSGs in the Upper Cedar River Watershed. The Iowan Surface portion of the watershed consists primarily of B, C, and C/D type soils, resulting in areas that range from moderate to high runoff potential. The soils in the northwestern portion of the watershed in the Des Moines Lobe region contain a greater amount of B/D type soils, reflecting a shallower ground water table which results in a lesser storage volume in the soil for infiltration and increased runoff potential. 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.
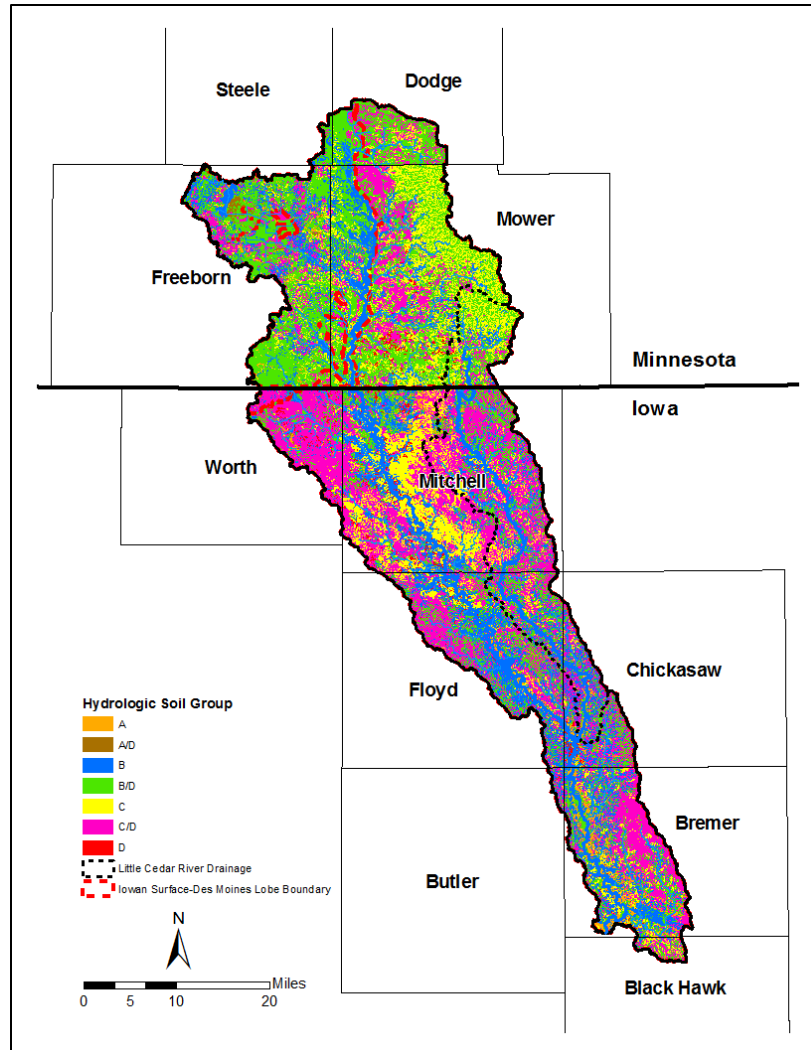
Figure 2.3. Hydrologic Soil Group distribution in the Upper Cedar River Watershed. Hydrologic Soil Groups reflect the degree of runoff potential a particular soil has, with Type A representing the lowest runoff potential and Type D representing the highest runoff potential.

The HSG composition in the watershed is tabulated in Table 2.2.

Table 2.2. Hydrologic Soil Group distribution in the Upper Cedar River Watershed.

Hydrologic Soil Group	Composition in Watershed (%)
A	3.8
A/D	0.3
B	19.5
B/D	30.2
C	15.2
C/D	29.5
D	1.4

c. Topography

The topography of the Upper Cedar River Watershed is relatively flat, more so in the Des Moines Lobe region, and consists primarily of rolling hills and farm ground. Elevations range from 1,438 feet in the uppermost part of the watershed to 814 feet at the outlet (624 feet of relief). The terrain tends to become slightly steeper moving from north to south out of the Des Moines Lobe region and into the Iowan Surface. Typical land slopes are between 0.6 and 5.6% (25th and 75th percentiles), with the steepest areas most common along the eastern portion of the watershed or near the river channel network; slopes lessen moving from south to northwest. Figure 2.4 shows the elevations and land slopes for the Upper Cedar River Watershed.
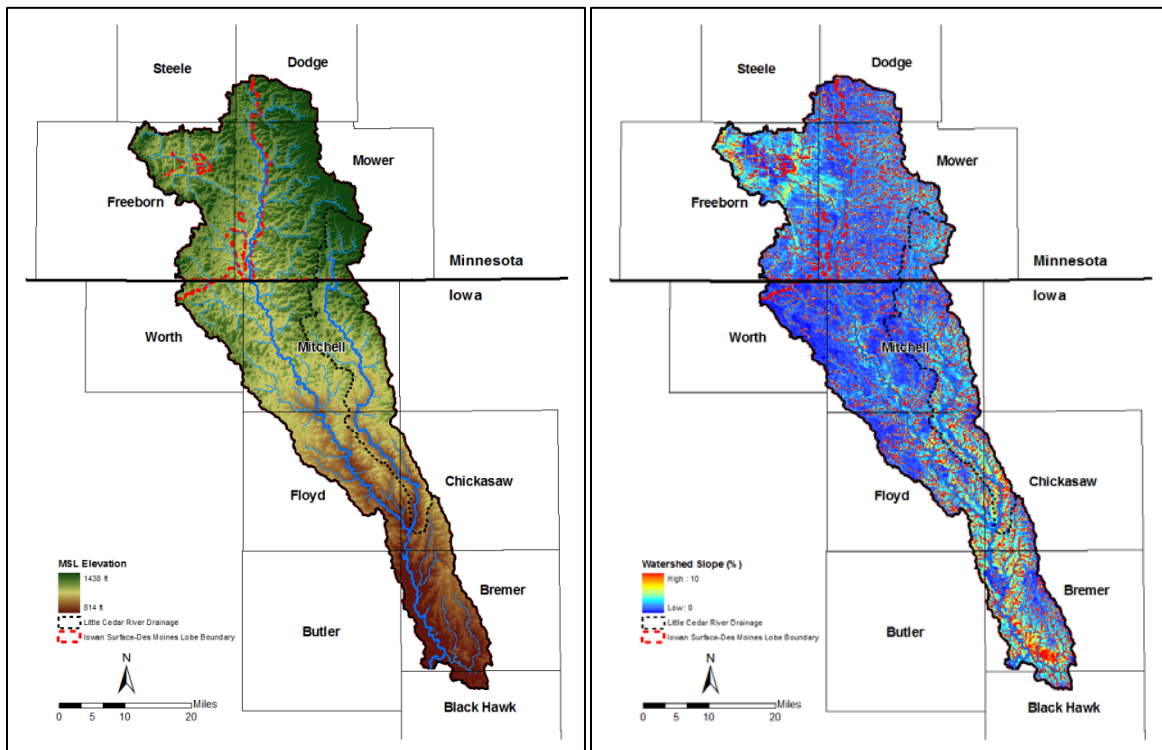
Figure 2.4. Topography of the Upper Cedar River Watershed. The left figure shows watershed elevations (in feet) and the right figure shows land slopes ranging from 0-10%.

d. Land Use

Land use in the Upper Cedar River Watershed is predominantly agricultural, dominated by cultivated crops (corn/soy beans) 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 2.5.
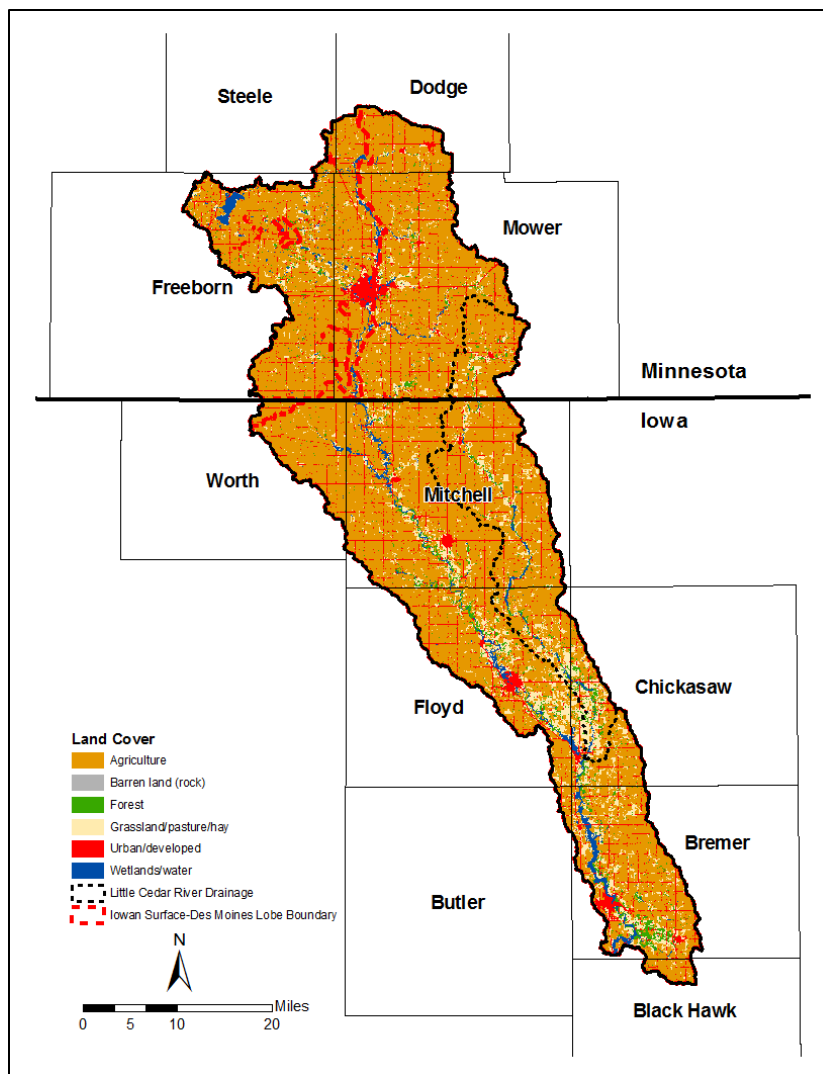
Figure 2.5. Land use in the Upper Cedar River Watershed. Agricultural land use is shown in orange.

e. Instrumentation/data records

The Upper Cedar River Watershed has instrumentation installed to collect and record stream stage, discharge, and precipitation measurements. There are six U.S. Geological Survey (USGS) operated 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. The operational period of record varies amongst each of the gages. Table 2.2 and Figure 2.6 below detail the periods of record and locations of the hydrologic/meteorologic instrumentation.

Table 2.3. 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
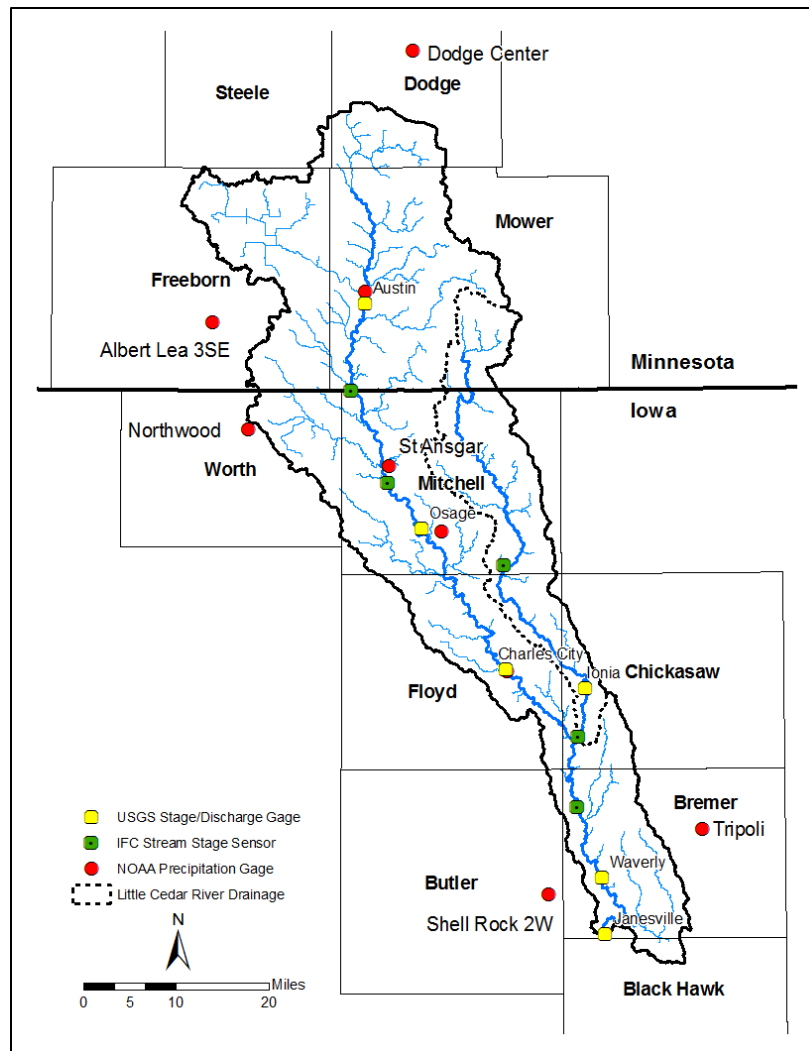
Figure 2.6. Hydrologic and meteorologic instrumentation in or near the Upper Cedar River Watershed. Stage/discharge gages (11) are shown in yellow or green while precipitation gages (9) are shown in red.

f. 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. On August 15, over 10 inches of rain were reported in southern Mower County, MN, resulting in severe flooding at Austin, MN 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.

The 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 rainfall 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 over the period June 3-12 totaled 8.3 inches, with nearly 50% of that falling on June 7-8. As a result, 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.

3. Upper Cedar River Hydrologic Model Development

This chapter summarizes the development of the model used in the Phase I Hydrologic Assessment 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.

HEC-HMS is designed to simulate rainfall-runoff processes of a watershed. It is applicable in a wide range of geographic areas and for watersheds ranging in size from very small (a few acres) to very large (the size of the Upper Cedar River Watershed or larger). Figure 3.1 reviews the water cycle and major hydrologic processes that occur in a watershed.
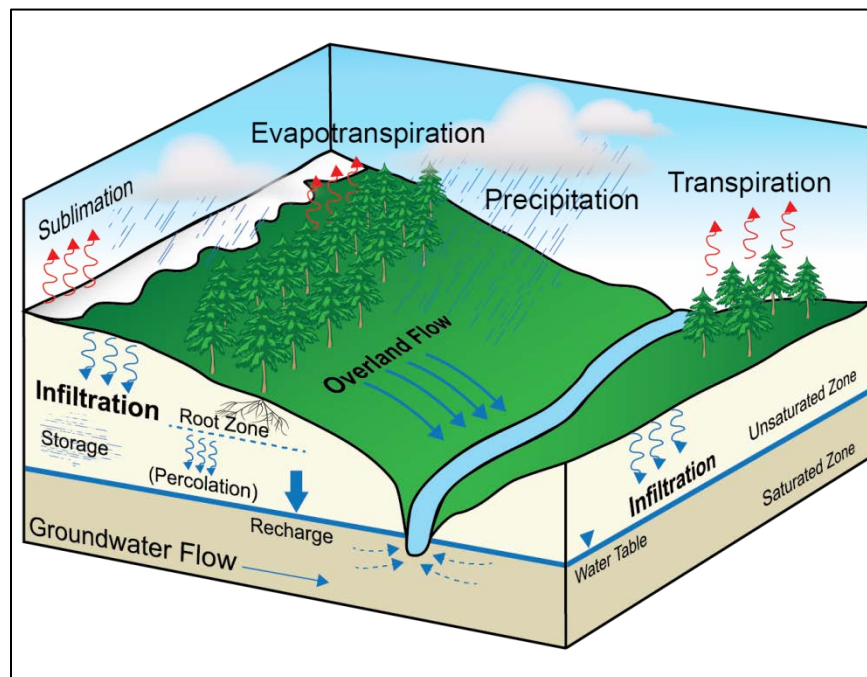
Figure 3.1. Hydrologic processes that occur in a watershed.

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 were often empirically developed to best describe observations or controlled experiments. HMS is also a lumped parameter model, meaning physical characteristics of the watershed, such as land use and soil type, are "lumped" together into a single representative value for a given land area. Once these averaged values are established within HMS, the value remains constant throughout the simulation 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 the ground is nearly saturated since overland flow is expected to dominate the partitioning of rainfall for both these cases.

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. Simulated hydrographs from HMS can be compared to discharge observations.

a. Model Development

The portion of the Upper Cedar River Watershed modeled and detailed herein is approximately 1663 mi². The watershed was divided into 320 smaller units, called subbasins in HMS, with an average area of about 5.2 mi² but as large as 19.5 mi². The subbasin delineation of the Upper Cedar River Watershed implemented into HMS is shown in Figure 3.2.
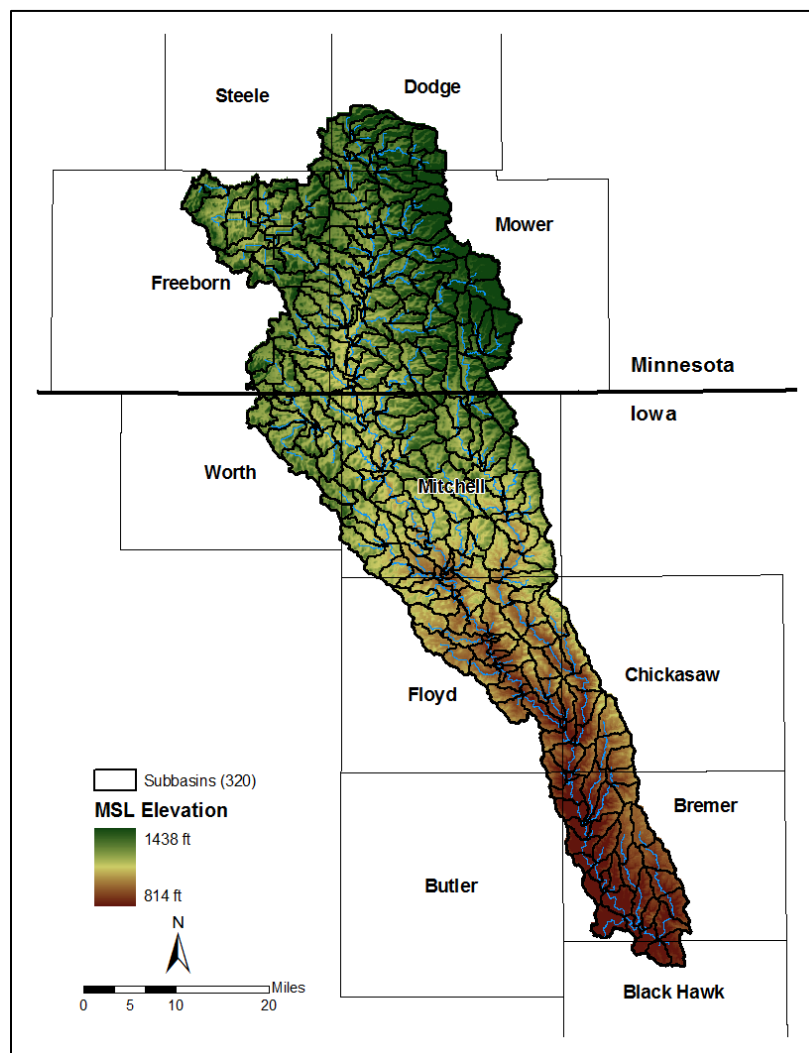
Figure 3.2. HEC-HMS delineated subbasins in the Upper Cedar River Watershed. The watershed was divided into 320 subbasins to better refine model parameters based on land use and soil type.

ArcGIS and Arc Hydro tools were used for terrain preprocessing, creating flow direction and flow accumulation grids, defining the stream network, and subbasin delineation. The stream

network was defined to begin when the upstream drainage area was 10 km^2 (3.86 mi^2), and subbasins were delineated such that a subbasin was defined upstream of all stream confluences. 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; see Figure 2.1). In HMS, area-weighted averaging is performed within the boundary of each subbasin to assign each subbasin a single value for the parameter being developed.

i. Incorporated Structures

One reservoir, Geneva Lake (see Figure 2.1), 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 1,955 acres, volume 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. No other existing water storage structures were incorporated into the HMS model.

ii. Development of Model Inputs and Parameters

This section provides an overview of data inputs used and assumptions made to develop the HMS model.

Rainfall (Meteorological Model)

Stage IV radar rainfall estimates were used as the precipitation input for simulation of actual (historical) rainfall 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 United States and combining them into a nationwide 4 km x 4 km (2.5 mile x 2.5 mile) gridded hourly precipitation estimate data set. Stage IV radar rainfall estimates are available from January 2002 – present.

Figure 3.3 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. This figure helps demonstrate the gridded nature of the radar rainfall estimates as well as the distributed nature of rainfall in time and space.
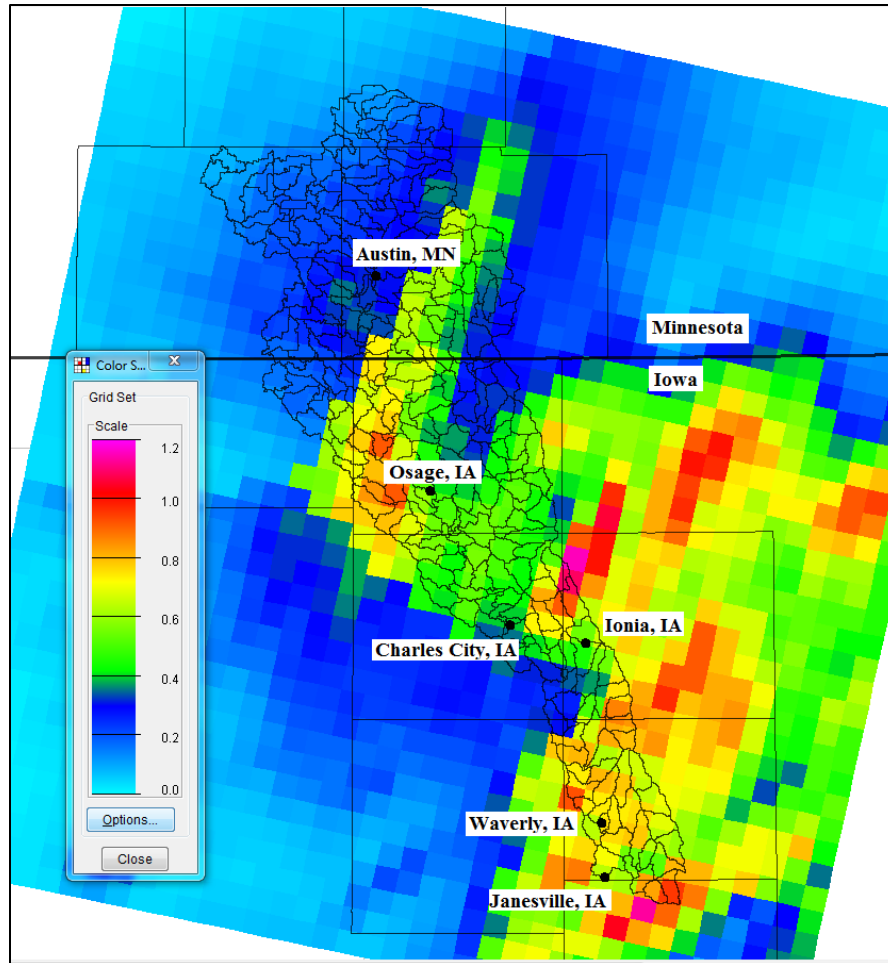
Figure 3.3. Demonstration of the gridded Stage IV radar rainfall product used as the precipitation input for historical storms in the Upper Cedar River Watershed HMS model. The Stage IV product provides hourly rainfall estimates for each 4 km x 4 km grid cell. The scale shown refers to the depth of rainfall (in inches) estimated for a one hour period.

Use of radar rainfall estimates provides increased accuracy 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. Actual storms using Stage IV data were the basis for model calibration and validation.

Hypothetical storms were developed for comparative analyses such as potential runoff generation, increased infiltration capacity through land use changes or soil improvements, and increased distributed storage within the watershed. These hypothetical storms apply a uniform depth of rainfall across the entire watershed with the same timing everywhere. NRCS Type-II distribution, 24-hour storms were used for all hypothetical storms. Point precipitation values (rainfall depths) for the 10-, 25-, 50-, and 100-year average recurrence interval, 24-hour design 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 basin centroid.

Studies have been performed on the spatial distribution characteristics of heavy rainstorms in the Midwest (Huff and Angel, 1992). Point precipitation frequency estimates are generally only applicable for drainage areas up to 10 mi² before the assumption of spatial uniformity is no longer valid. For drainage areas between 10 and 400 mi², relations have been established between point precipitation estimates and an areal mean precipitation approximation. Areal reduction factors based on storm duration and drainage area can be found in the *Rainfall Frequency Atlas of the Midwest* (Huff and Angel, 1992). NOAA does not recommend adjusting point estimates for watersheds much beyond 400 mi², as the dependence between the point and areal values breaks down for watersheds larger than this.

For the comparative analyses that were performed in this modeling effort, an extrapolation was performed to get an areal reduction factor beyond 400 mi². It is 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, the point rainfall estimates were multiplied by an areal reduction factor of 0.88 (the areal reduction factor for the 1663 mi² drainage area at Janesville). Table 3.1 summarizes the point precipitation frequency estimates collected 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 3.1. Rainfall frequency estimates used for hypothetical watershed scenario analyses. The areal reduced values were used in HMS as the cumulative rainfall depths for the 24-hour design storms of different return period.

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

Both the point and areal reduced precipitation estimates used in this modeling analysis should not be used for localized project design purposes. However, the process described for obtaining point estimates from NOAA Atlas 14 and applying the appropriate correction factor based on a specific project's drainage area (up to 400 mi²) is applicable.

Watershed (Basin Model)
Topography
Elevation data was obtained from the National Elevation Dataset (NED). Four blocks (n43w093, n43w094, n44w093, and n44w094) of approximately 10-meter resolution digital elevation models (DEMs) covering the extent of the Upper Cedar River Watershed were downloaded, clipped to the watershed extents using ArcGIS, and then joined into a seamless DEM. NED data are distributed in geographic coordinates in units of decimal degrees, in conformance with the North American Datum of 1983 (NAD 83). All elevation values are in meters and are referenced to the North American Vertical Datum of 1988 (NAVD 88).

Runoff Volume
Soil Conservation Service (SCS) Curve Number (CN) methodology was used to determine the rainfall-runoff partitioning in the Upper Cedar River Watershed HMS modeling. The CN serves

as a runoff index 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. CN values 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 U.S. Department of Agriculture – Natural Resource Conservation Service (USDA-NRCS), previously known as the SCS. The CNs assigned to each land use and soil type combination for the Upper Cedar River HMS model are shown in Table 3.2 below.

Table 3.2. 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

In cases where a land use code or soil type was unassigned to an area, soil type B or the row crop agriculture land use class was assigned (*Iowa DOT Design Manual*, 2013). 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 thought to be present throughout the watershed.

A CN grid was generated for the Upper Cedar River Watershed in ArcGIS by intersecting the 2006 National Land Cover Dataset (NLCD) with the digital Soil Survey Geographic Database (SSURGO) dataset. 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 methodology for rainfall-runoff partitioning accounts for precipitation losses due to initial abstraction and infiltration during the simulation. The initial abstraction refers to the initial amount of rainfall that must fall before any runoff begins (losses due to plant interception, soil wetting, and storage in surface depressions). The remaining precipitation is considered excess precipitation and a portion of this is converted to runoff. Evaporation and transpiration (evapotranspiration) were neglected in the modeling as the focus is to simulate

short duration, large rain events when evapotranspiration is thought to be a minimal component of the water balance. CN regeneration, in which the initial abstraction is reset after some time period, was not considered since short duration, event-based storms were the primary modeling focus.

Antecedent Moisture Conditions

Rainfall-runoff partitioning for an area is also dependent on the antecedent soil moisture conditions (how wet the soil is) at the time rain falls on the land surface. In essence, the wetter the soil is, the less water is able to infiltrate into the ground and more rain is converted to runoff. Therefore, a methodology was developed 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 moisture conditions (AMC) by classifying CNs into one of three classes based on the seasonal five-day antecedent rainfall total. While this method provides a simple way to adjust CNs to reflect AMC based on the seasonal five-day antecedent rainfall total, it is oversimplified in several ways. The five-day antecedent rainfall totals defining each AMC class are universal; they have not been adjusted to reflect differences in 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 CN classification; this can lead to drastic overestimations or underestimations of runoff volume for only a small change in the five-day antecedent rainfall total.

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 because of its added flexibility. The API is computed using a temporal decay constant to account for soil moisture losses which allows more or less weight to be applied to precipitation that fell closer in time to the event of interest. Using records from the nine NOAA hourly/daily precipitation stations listed in Table 2.2 and shown in Figure 2.6, basin average daily precipitation was computed for the Upper Cedar River Watershed over a 65-year period (1948-2013). 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 included in this analysis. Using this precipitation record, the API cumulative distribution function (CDF) was computed and related to CN by the percentiles defining the AMC I, II, and III classes (10th, 50th, and 90th percentiles, respectively). Linear interpolation was performed between each API percentile defining each AMC class and the percent change in the basin average AMC I and III CNs from the AMC II condition.

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. Figure 3.4

summarizes how AMC were accounted for in the Upper Cedar HMS model; the 0-100 percentiles (or 0-1 probabilities of non-exceedance) of API are plotted along the horizontal axis and the percent change in CN from the basin average AMC II CN is plotted along the vertical axis. The finer dashed line shows how CNs would be classified into one of three classes if traditional NRCS methodology were being used; based on the seasonal five-day antecedent rainfall totals defining each class, the Upper Cedar would fall into the AMC I (dry) condition over 85% of the time, which is unreasonable. The coarser dashed line represents the initial CN-API curve developed from the statistical definitions of the three AMC classes. This curve was adjusted accordingly (solid line) to better match the optimal CN adjustments determined for each calibration storm through trial and error (crosses).
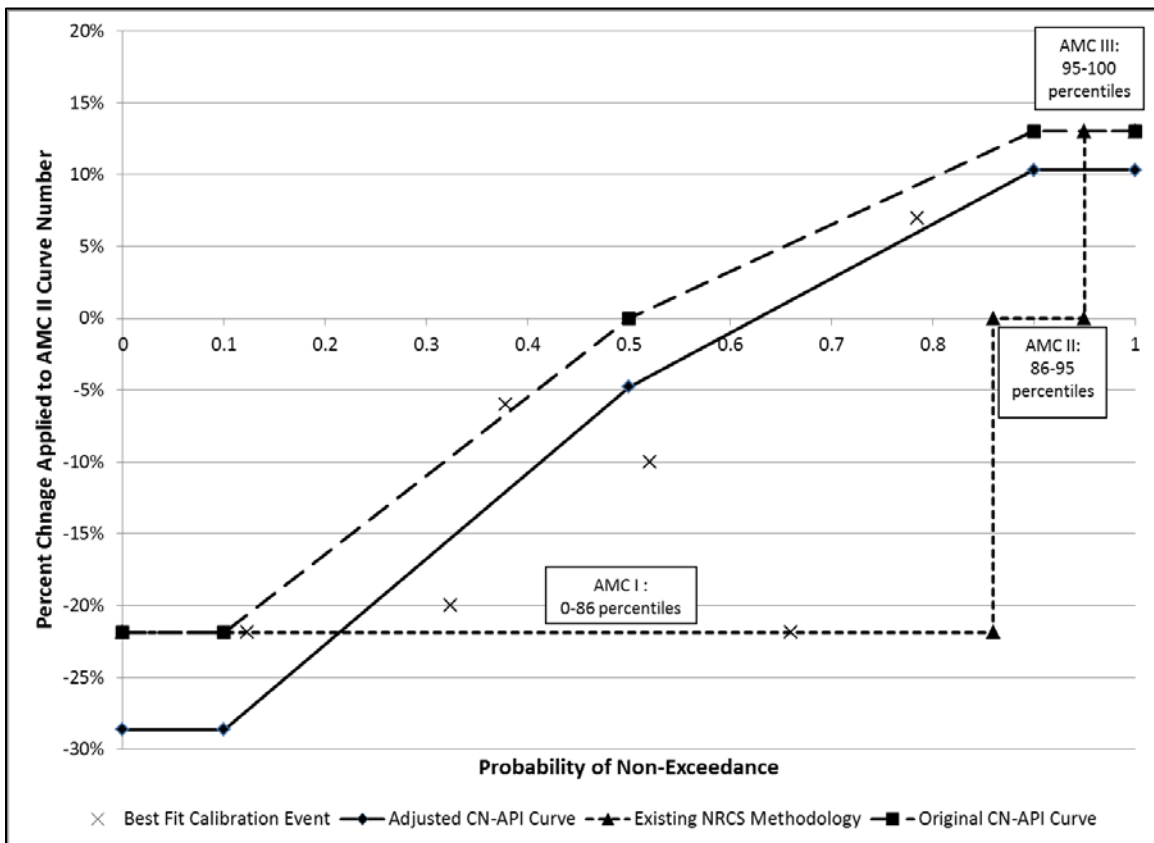
Figure 3.4. Accounting for antecedent moisture conditions in the Upper Cedar River Watershed HMS model through use of the antecedent precipitation index (API). Precipitation gage records were used to calculate the API prior to each historical storm event and a corresponding percent change in Curve Number was applied to each subbasin Curve Number to account for the initial soil moisture conditions.

Runoff Hydrographs

The Clark and ModClark Unit Hydrograph methods were used to convert excess precipitation to a direct runoff hydrograph for each subbasin. The Clark Method was selected to account for the hydrologic impacts of tile drainage. The ModClark method requires the same grid used for radar rainfall, so this method was used for simulating historical storms used for calibration and validation while the traditional Clark method was used for hypothetical design storm analysis. Both methods account for translation (delay) and attenuation (reduction) of the peak subbasin

hydrograph discharge due to travel time of the excess precipitation to the subbasin outlet and temporary surface storage effects. The primary difference between the two methods is the traditional Clark Unit Hydrograph method uses a pre-developed time-area histogram while the ModClark method uses a grid-based travel time model to account for translation (lag) of the subbasin hydrograph. Both methods route the hydrograph through a linear reservoir to account for temporary storage effects.

Both the ModClark and Clark unit hydrograph methods require two inputs – time of concentration and a time storage coefficient. The time of concentration is the time required for water to travel from the hydraulically most remote point in the subbasin to the subbasin outlet. This was estimated as 5/3 times the lag time, where lag time is the time difference between the center of mass of the excess precipitation and the peak of the runoff hydrograph. This is a reasonable approximation according to NRCS methodology (Woodward, 2010). Lag time is a function of land slope, longest flow path, and soil retention (represented through CN); these parameters were estimated for each subbasin in ArcGIS tools. While time of concentration is a measure of lag due to travel time effects as water moves through the watershed, the time storage coefficient is a measure of lag due to natural storage effects in the subbasin (Kull and Feldman, 1998). Based on the literature, it can be estimated as a multiple of the time of concentration. Figure 3.5 illustrates the NRCS methodologies for runoff depth estimation and how this runoff depth is converted to discharge (using one of the Clark unit hydrograph methods).
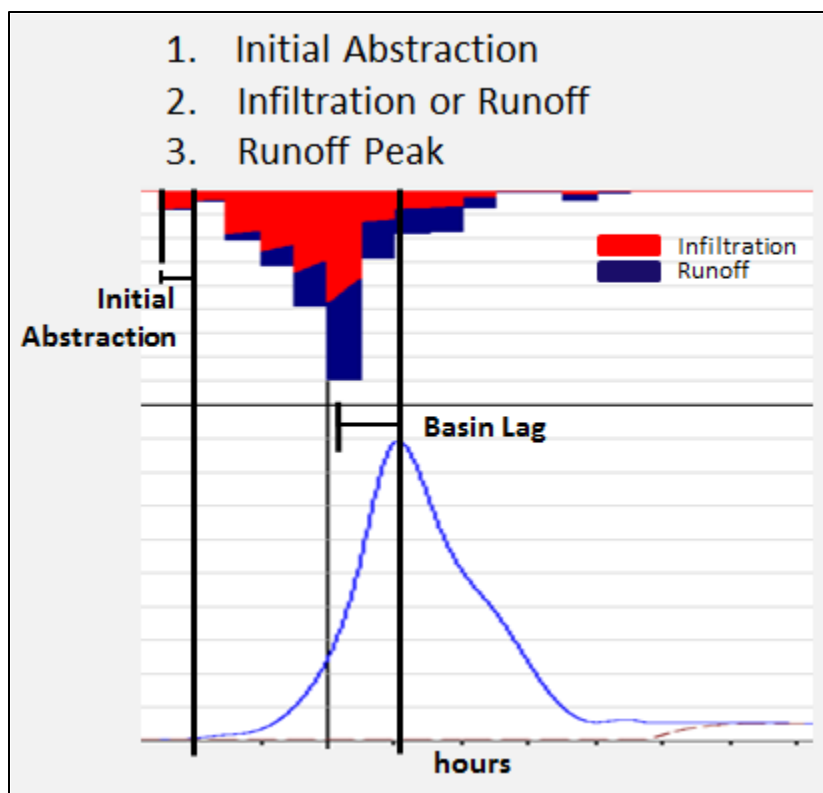
Figure 3.5. Subbasin runoff hydrograph conceptual model. This figure shows how rainfall is partitioned into a runoff depth using the NRCS Curve Number methodology which is then converted to discharge using either the ModClark or Clark unit hydrograph method.

ArcGIS to HEC-HMS

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

Parameters Assigned in HEC-HMS

Baseflow

Baseflow was approximated by a first order exponential decay relationship for all historical storms. The USGS stage/discharge gages for the Cedar River near Austin, Osage, Charles City, Waverly, Janesville, and the Little Cedar River near Ionia were used to develop discharge-drainage area (cfs/mi²) relationships to set initial conditions for streamflow prior to each actual storm event simulation. An initial baseflow condition was determined for the Cedar River and its tributaries upstream of the Austin, MN gage and for the Little Cedar River and its tributaries upstream of the Ionia gage. A third initial baseflow condition for all other river reaches was determined using a combination of the remaining USGS gages. These three sets of initial

conditions were applied to the appropriate subbasins in HMS for each historical storm event simulation. A baseflow recession constant describing the rate of decay of baseflow per day and a threshold indicating when baseflow should be reactivated were also specified.

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

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

Flow routing through Geneva Lake was executed using level pool routing. A level water surface is assumed and the methodology is derived from Conservation of Mass, similar to the Muskingum model. A 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 incorporated into the model were assumed to be filled to the normal pool level at the beginning of each simulation.

b. Calibration

Model calibration is a process of taking an initial set of parameters developed for the hydrologic model through GIS and other means and making adjustments to them so that simulated results produced by the model match as close as possible to an observed time series, typically stream discharge at a gaging station. However, 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 represent the watershed and it is upon the modeler to change methods used within the model or find what parameter(s) might be needed to better represent the watershed's hydrologic response.

The 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

runoff (CN) and timing (river routing and unit hydrograph) parameters to best match the simulated response to the observed discharge time series at each USGS discharge gage location. Greater detail on calibration and validation is provided in Appendix B.

c. Validation

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 storms were considered for model validation.

As with calibration, the HMS model validation results are not perfect. For the four validation storms considered, the HMS simulated responses consistently overestimate the USGS discharge observations, both in magnitude of the peak discharge and total runoff volume. While this likely reflects the model parameters responsible for runoff (namely the subbasin CNs) are overestimated, there are other reasons that could contribute to this disparity. One reason for difference may be due to the size of the storms considered. Due to a number of high runoff storms being selected for calibration, smaller storms yielding less runoff generally had to be selected for validation. These smaller storms tend to have a greater subsurface flow component than larger storms since the ground is likely to have a greater capacity to infiltrate water, depending on antecedent moisture conditions. Because HMS is a surface water model, it struggles to simulate these types of conditions where overland flow does not dominate the partitioning of rainfall. Secondly, all of the storms occurred in or near the peak of the growing season when precipitation losses due to evapotranspiration and plant root uptake are at a maximum. This is reflected in the observations as most of the storms produce a small amount of runoff despite a substantial amount of rain, even with some storms having wetter than normal antecedent conditions. Lack of accounting for evapotranspiration losses in the HMS model may also contribute to runoff overestimation.

Despite these discrepancies, the HMS model did acceptable simulation of the June 2008 flood that produced a record discharge of 53,400 cfs at Janesville. Simulated hydrographs at multiple locations throughout the watershed closely match observations, providing reassurance that the calibrated HMS model has some predictive capability, particularly in simulating high runoff events.

4. Analysis of Watershed Scenarios

The HEC-HMS model of the Upper Cedar River Watershed was used to identify areas in the watershed with high runoff potential and run simulations to help understand the potential impact of potential flood mitigation strategies in the watershed. Focus for the scenarios was placed on understanding the impacts of (1) increasing infiltration in the watershed and (2) implementing a system of distributed storage projects (ponds) across the landscape.

a. High Runoff Potential Areas

Identifying areas of the watershed with higher runoff potential is the first step in selecting mitigation project sites. High runoff areas offer the greatest opportunity for retaining more water from large rainstorms on the landscape and reducing downstream flood peaks.

In the HMS model of the Upper Cedar River Watershed, the runoff potential for each subbasin is defined by the SCS Curve Number (CN). The runoff CN assigned to a subbasin depends on its land use and the underlying soils. The fraction of rainfall that is converted to runoff — also known as the runoff coefficient — is a convenient way to illustrate runoff potential. Areas with higher runoff coefficients have higher runoff potential. To evaluate the runoff coefficient, the runoff from each subbasin area is simulated with the HMS model for the same rainstorm; we chose a rainstorm with a total accumulation of 5.05 inches in 24 hours (25-year average recurrence interval).

Figure 4.1 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 4.1. Area-weighted average runoff coefficients were determined for each of the 47 HUC 12 watersheds in the Upper Cedar basin. Areas in the basin with the highest runoff potential are primarily located in Mitchell, Worth, and parts of Floyd counties. Runoff coefficients exceed 50% in many areas. 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. 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.
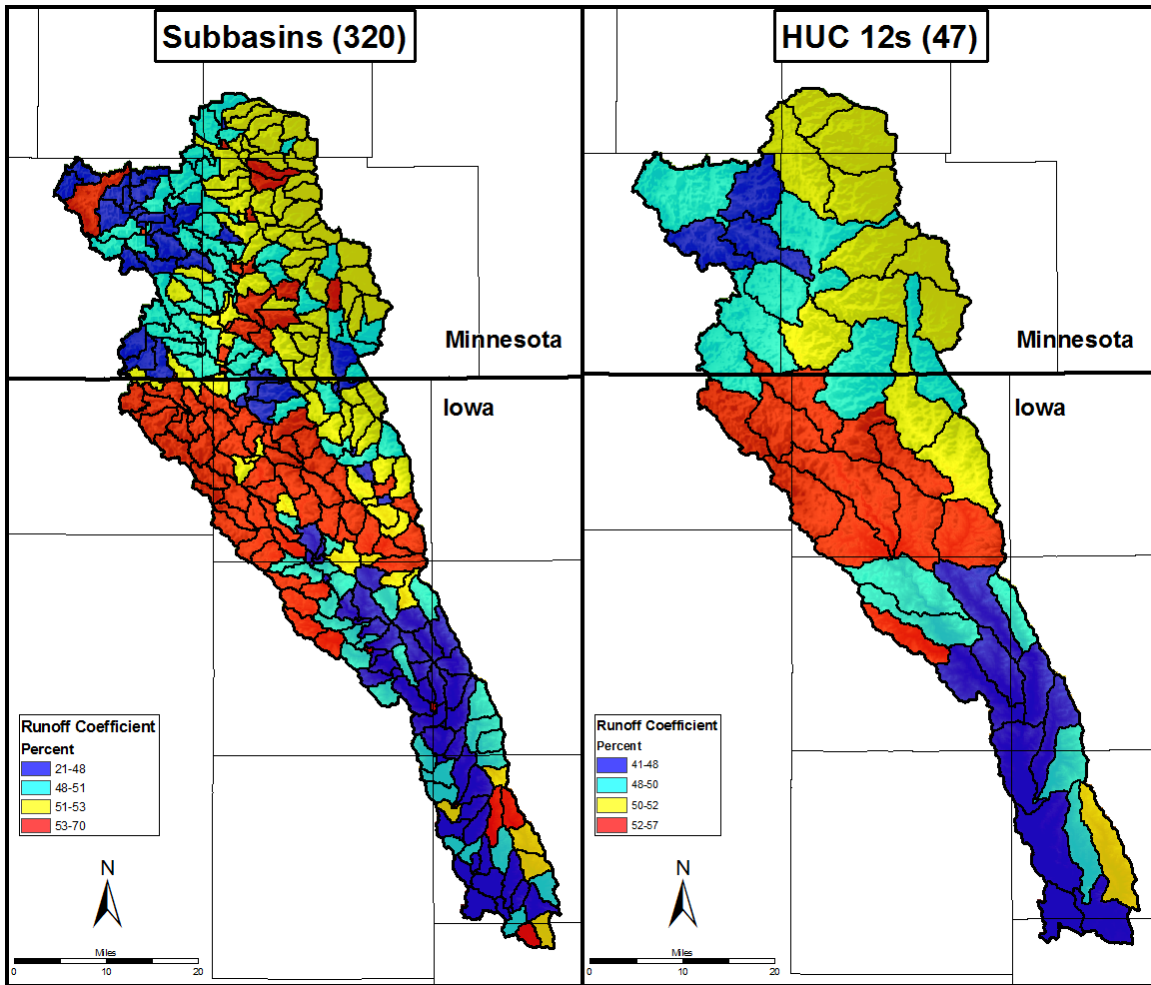
Figure 4.1. Runoff potential in the Upper Cedar River Watershed. Runoff coefficients computed for each subbasin (left) and HUC 12 watershed (right) for the 25-year, 24-hour design storm (5.05 inches of rain in 24 hours) are shown. Higher runoff coefficients are shown in red.

b. Mitigating the Effects of High Runoff with Increased Infiltration

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 section, we examine several different alternatives for reducing runoff through either land use changes or soil quality improvements. One hypothetical land use change is the conversion of row crop agriculture back to native tall-grass prairie. Another land use change is the improved agricultural condition compared to existing conditions that would result from planting cover crops during the dormant season. The third alternative is the effect that improving soil quality (soil infiltration characteristics) would have on runoff reduction. All are hypothetical examples;

they are meant to illustrate the potential effects on flood reduction. The examples are also not project proposals; they would neither be recommended or practically feasible. Still, the hypothetical examples do provide valuable benchmarks on the limits of flood reduction that are physically possible with runoff reduction.

Conversion of Row Crop Agriculture to Tall-Grass Prairie

Much has been documented about the historical water cycle of the native tall-grass prairie of the Midwest. Some evidence suggests that the tall-grass prairie could handle several 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.

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 this example, all current agricultural land use is converted to native tall-grass prairie with its much higher infiltration characteristics. 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.

To simulate the conversion to native tall-grass prairie with the HMS model, the model parameters affecting runoff potential across the landscape were adjusted to reflect the tall-grass prairie condition. Specifically, existing agricultural land use, which accounts for 77% of the watershed area, was redefined as tall-grass prairie. New CNs, reflecting the lower runoff potential of prairie, were assigned to each subbasin. 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 only considers the runoff reduction resulting from the much improved infiltrating characteristics of the native prairie 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. Comparisons were made between current and tall-grass prairie simulations for the 10-, 25-, 50-, and 100-year return period, 24-hour NRCS design storms. Using design storms of different severity illustrates how flooding characteristics change during more intense rainstorms.

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, 1.2 inches for a 50-year storm, and 1.3 inches for a 100-year storm. As a result of increased infiltration across the landscape, the river response is dampened.

Figure 4.2 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. 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.
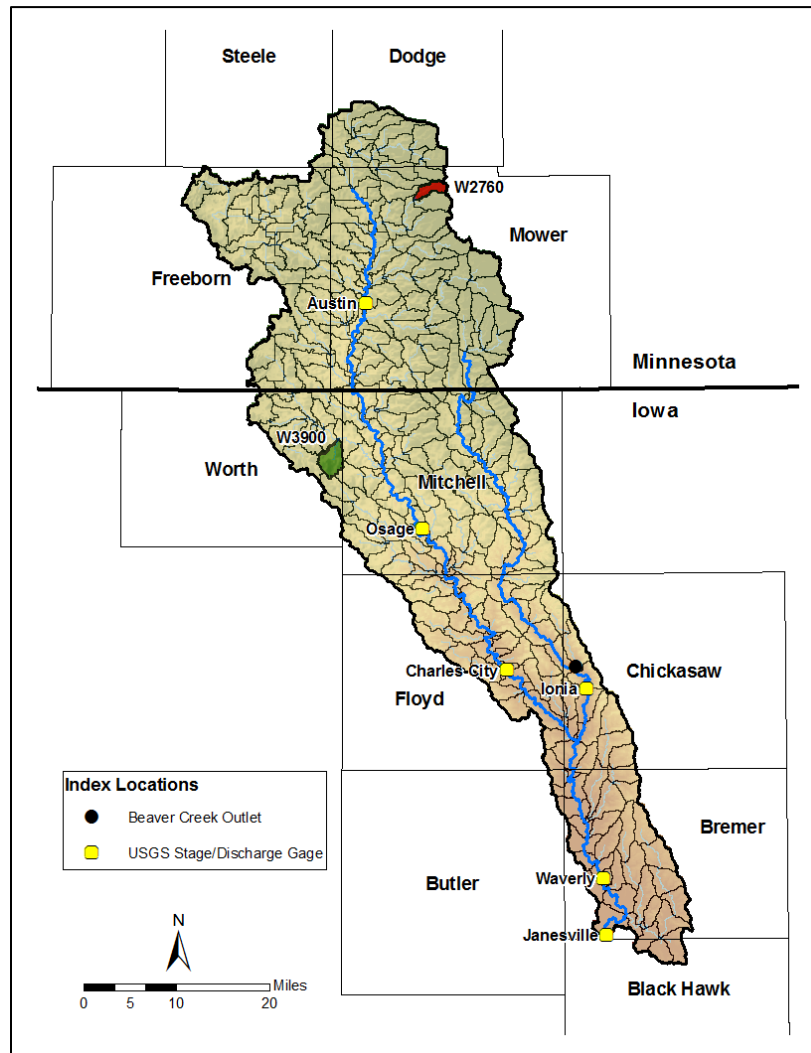
Figure 4.2. Index locations selected for comparing hypothetical flood mitigation scenarios to current conditions. The six USGS stage/discharge gages and the Beaver Creek outlet served as points of reference to compare scenario results to existing conditions. Subbasins W2760 and W3900 were also used to demonstrate the impact of a particular scenario at the subbasin scale.

Figure 4.3 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 mi²), 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 (30-40%), reflecting the relatively even distribution of agriculture throughout the watershed.
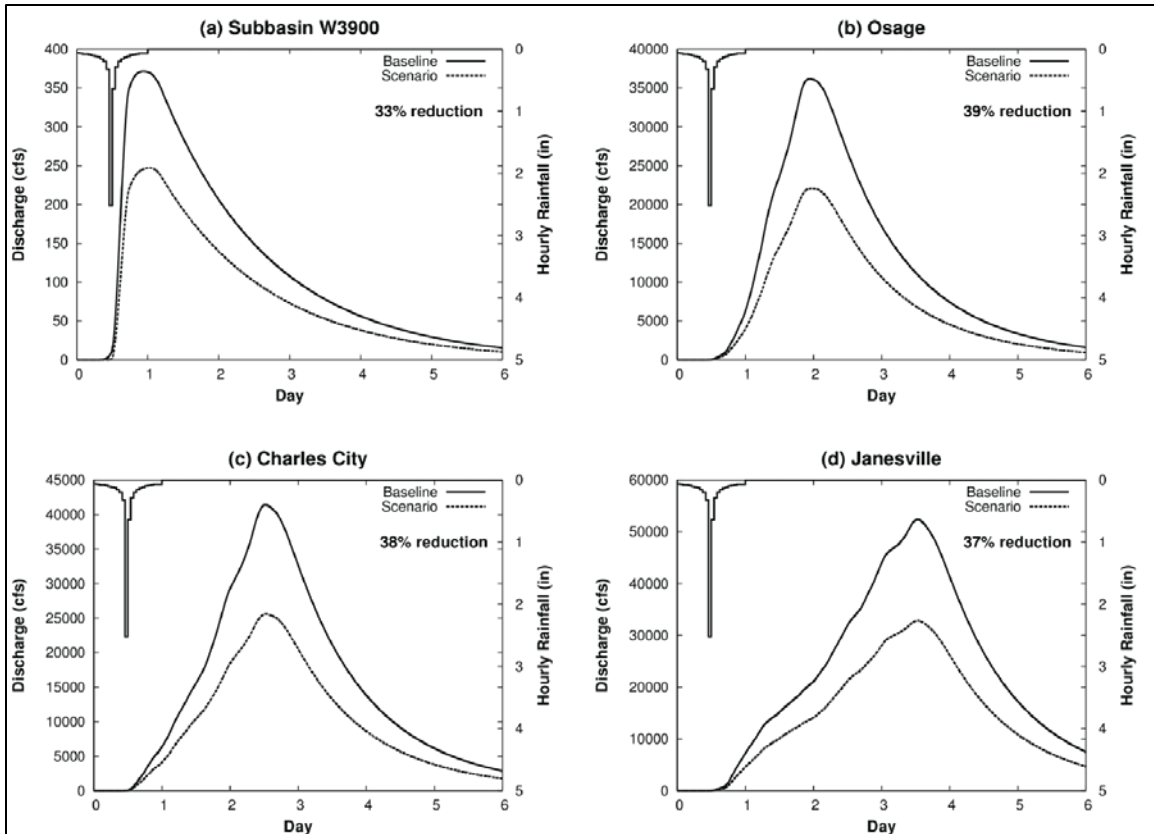
Figure 4.3. Hydrograph comparison: Conversion of row crop agriculture to native prairie for the 50-year, 24-hour design storm (5.89 inches of rain in 24 hours).

Figure 4.4 shows the peak discharge reduction 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.
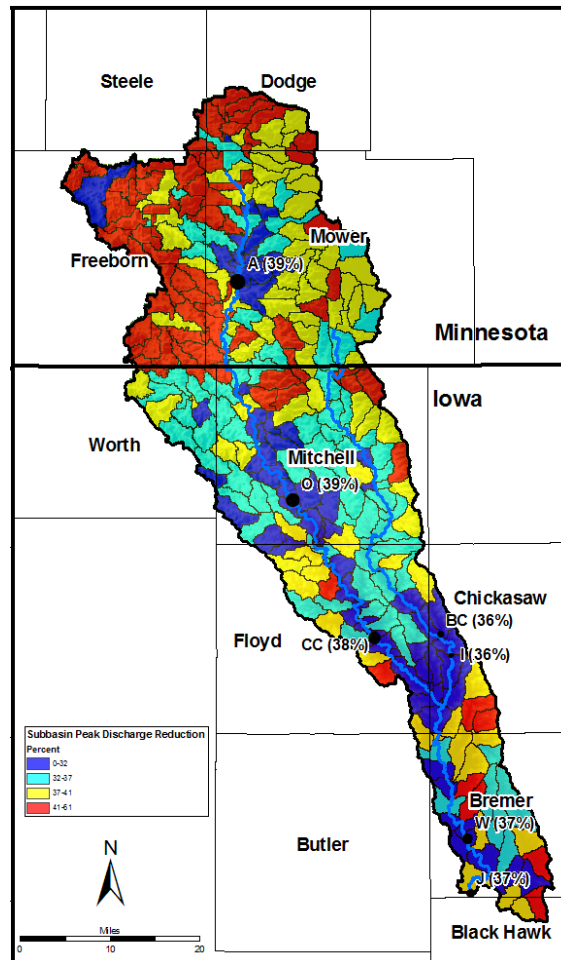
Figure 4.4. Subbasin peak discharge reductions: Conversion of row crop agriculture to native prairie for the 50-year, 24-hour design storm (5.89 inches of rain in 24 hours).

Figure 4.5 shows the peak discharge reductions resulting from the 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.
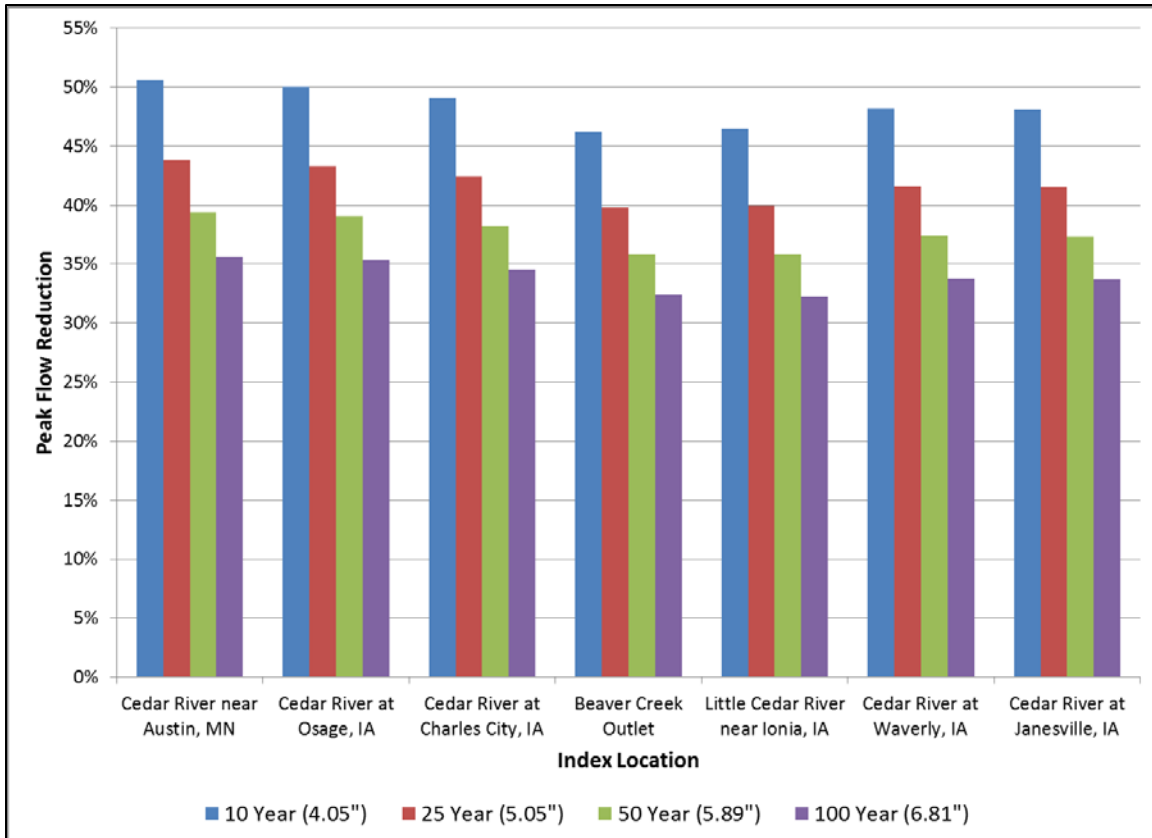
Figure 4.5. Peak discharge reductions: Conversion of row crop agriculture to native prairie for the 10-, 25-, 50-, and 100-year, 24-hour design storms (4.05, 5.05, 5.89, and 6.81 inches of rain in 24 hours).

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 4.5, the corresponding reduction in flood stage is between 2 and 7 feet. This reduction was estimated for the USGS stream 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 National Weather Service (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 of the size of the 25- (5.05 inches), 50- (5.89 inches), or 100-year (6.81 inches) return period design storm level. 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 agricultural to tall-grass prairie landscape does not eliminate flooding, but would reduce its severity and frequency.

Improved Agricultural Conditions from Planting Cover Crops

Cover crops can be an effective 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; afterwards, 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 content, 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 (e.g. from 2% to 3%), the soil can retain an additional 17,000-25,000 gallons of water per acre (Archuleta, 2014). Examples of cover crops include clovers, annual and cereal ryegrasses, winter wheat, and oilseed radish (Mutch, 2010).

The purpose of this hypothetical example is to investigate the impact improved agricultural management practices could have on reducing flood peak discharges throughout the watershed. Planting cover crops across all agricultural areas in the watershed during the dormant (winter) season is hypothesized to lower the runoff potential of these same areas during the growing season (spring and summer) due to increased soil health and fertility. To be clear, this scenario does not represent the conversion of the existing agricultural landscape (constituting row crops primarily) to cover crops. Rather, the existing agricultural landscape is still mostly intact, but its runoff potential during the growing season has been slightly reduced by planting cover crops during the dormant season. In a similar manner to the tall-grass prairie scenario, new runoff CNs, reflecting the landscape's lower runoff potential from improved agricultural management practices, were assigned to each subbasin. Comparisons were made between current and cover crop simulations for the 10-, 25-, 50-, and 100-year return period, 24-hour NRCS design storms.

Improved agricultural management practices, represented through cover crop planting during the dormant season, results in less reduction of runoff and peak discharges than the native tall-grass prairie simulation. On average for the basin, planting cover crops increases infiltration by 0.2-0.3 inches for the four design storms. Figure 4.6 shows the peak discharge reductions 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.
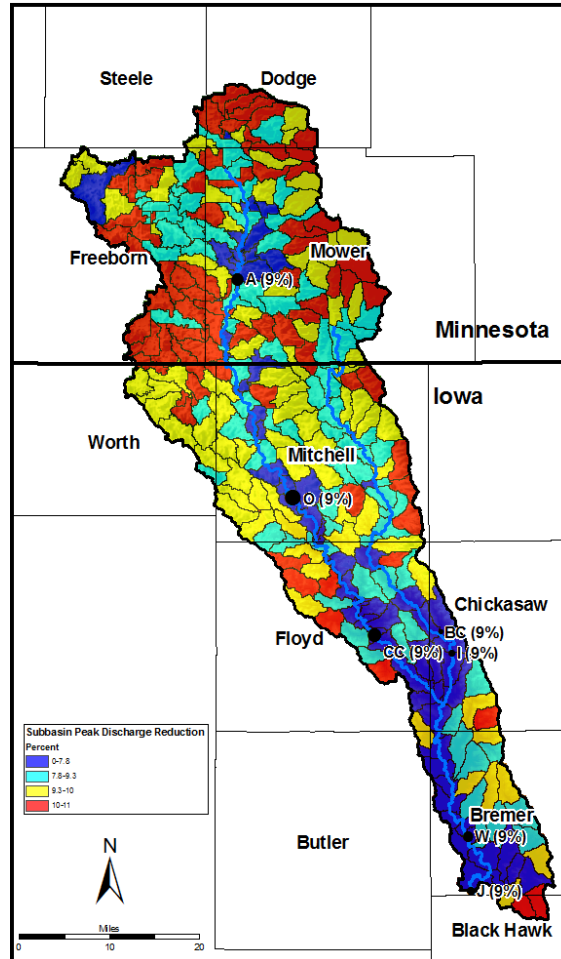
Figure 4.6. Subbasin peak discharge reductions: Effect of improved agricultural management practices (cover crops) for the 50-year, 24-hour design storm (5.89 inches of rain in 24 hours).

Figure 4.7 shows the peak discharge reductions 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 with 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.
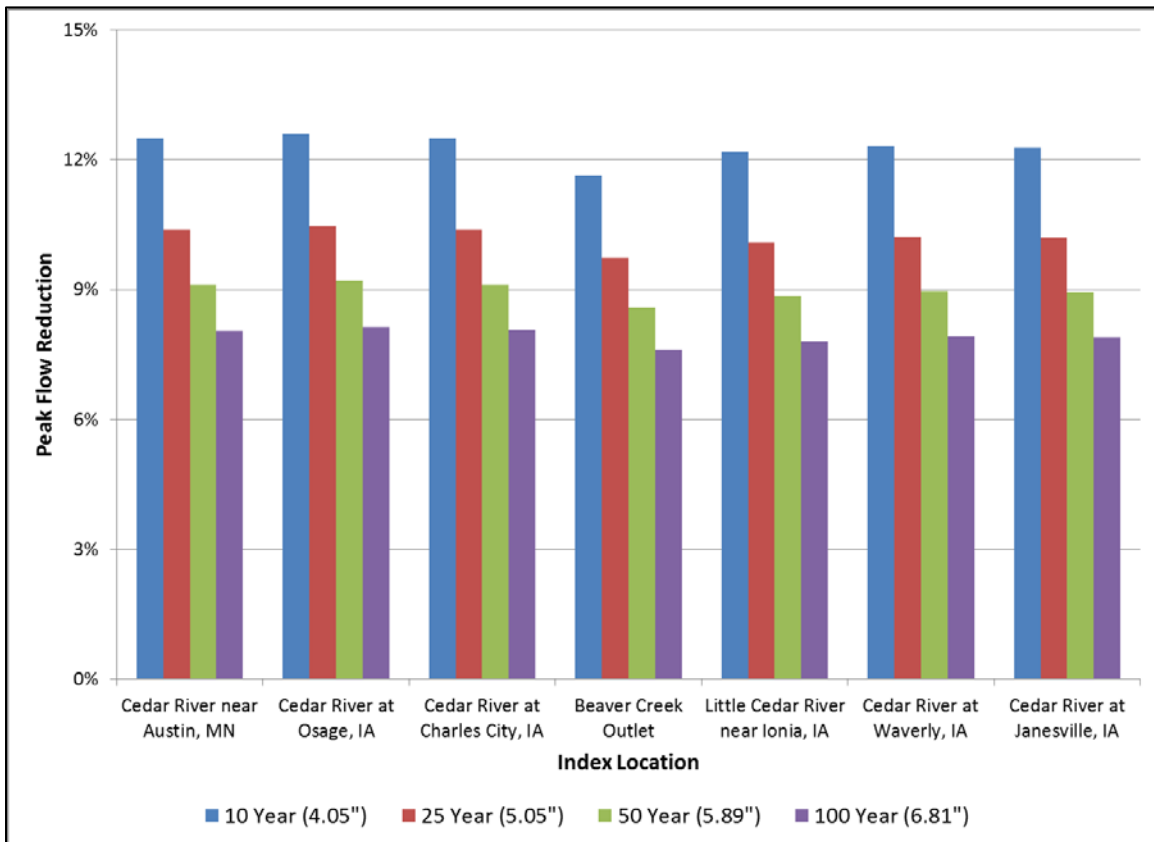
Figure 4.7. Peak discharge reductions: Effect of improved agricultural management practices (cover crops) for the 10-, 25-, 50-, and 100-year, 24-hour design storms (4.05, 5.05, 5.89, and 6.81 inches of rain in 24 hours).

Improving Soil Quality: Case 1

Another way to reduce runoff is to improve soil quality. Here, soil quality refers to the infiltration capacity of the soil. Better soil quality (increased soil infiltration characteristics) 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.

To simulate improved soil quality with the HMS model, we hypothesize that improvements translate to changes in the NRCS hydrologic soil group (HSG). As discussed previously, NRCS rates the runoff potential of soils with four HSGs (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 HSG – saturated hydraulic conductivity (the rate water flows

through the soil under saturated conditions, which corresponds to the minimum infiltration rate), 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 HSG 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 infiltration to some degree.

In the HMS model of the Upper Cedar River Watershed, the effects of improved soil quality through conservation and best management practices are represented by changes in the NRCS HSG. 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. This represents a change in soil texture from that of a sandy loam or loamy sand (average minimum infiltration rate of about 3.5 inches/hour) to that of a sand or gravel (minimum infiltration rate greater than 5.67 inches/hour). 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. This represents a change in soil texture from that of a loam containing silt or clay (average minimum infiltration rate of 0.78 inches/hour) to that of a sandy loam or loamy sand (average minimum infiltration rate of 3.5 inches/hour). For both cases, new CNs, reflecting the lower runoff potential with improved soil quality, were assigned to each subbasin. Then the model was run for a set of design storms. Comparisons were made between current and improved soil quality simulations for the 10-, 25-, 50-, and 100-year return period, 24-hour NRCS design storms.

The first soil improvement case — where all Type B soils (sandy loam or loamy sand) improve to Type A (sand or gravel)— results in approximately 0.4 inches more infiltration than current soil conditions for the 10-year return period design storm. Additional infiltration increases to about 0.5 inches for the 25-year storm, and levels off at about 0.6 inches for the 50-year and 100-year storms. Figure 4.8 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 (changes of Type B to A soils) 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%.
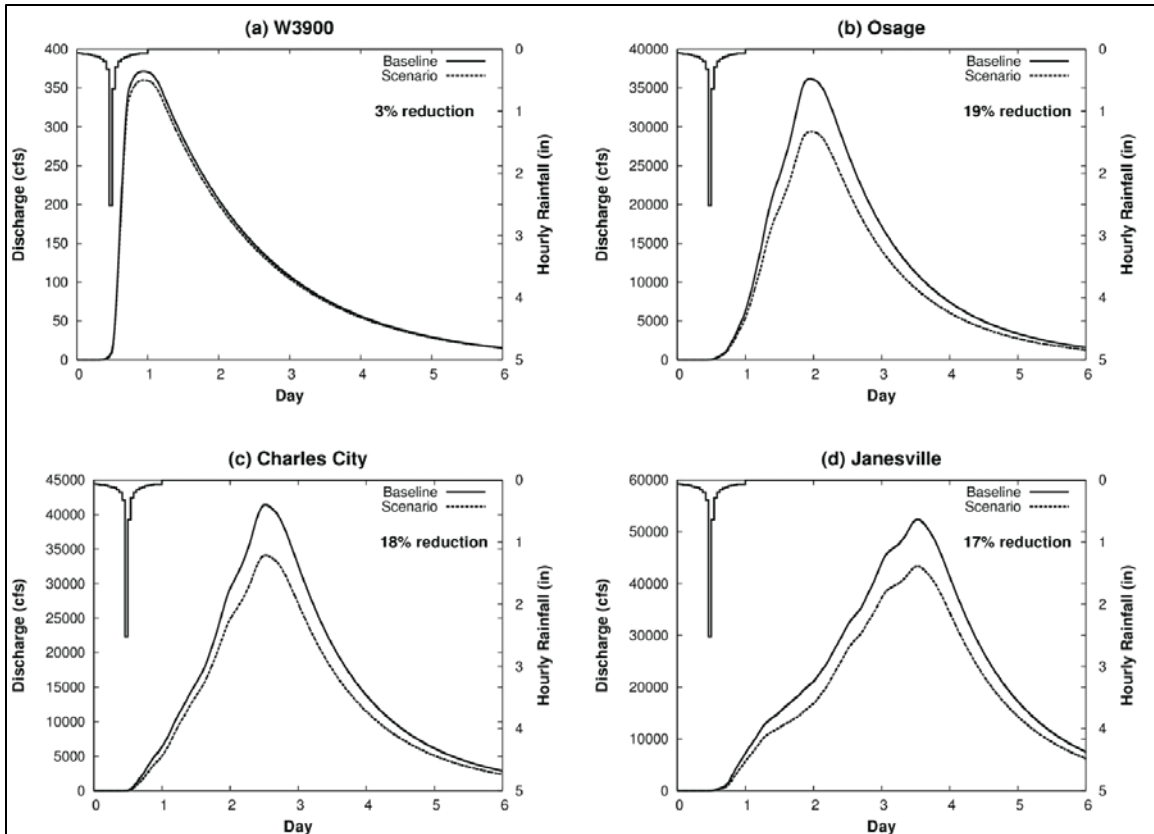
Figure 4.8. Hydrograph comparison: First soil improvement case (Type B or B/D to A) for the 50-year, 24-hour design storm (5.89 inches of rain in 24 hours).

Figure 4.9 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 greatest reductions in the Minnesota portion of the watershed occur in the Des Moines Lobe region, where Type B or B/D soils dominate. The greatest reductions in the lower third of the watershed occur where higher concentrations of Type B soils are found within the Iowan Surface. In contrast, the smallest peak discharge reductions occur in the middle part of the watershed, where Type C soils dominate. Peak discharge reductions are fairly uniform at the seven index locations (16-22%).
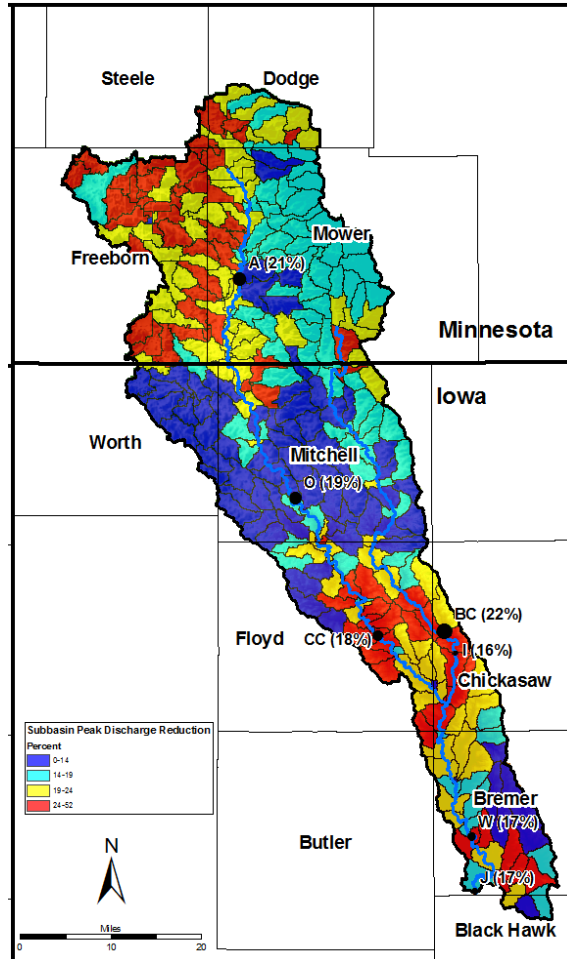
Figure 4.9. Subbasin peak discharge reductions: First soil improvement case (Type B or B/D to A) for the 50-year, 24-hour design storm (5.89 inches of rain in 24 hours).

Figure 4.10 shows the peak reductions resulting from the first soil improvement case at the seven index locations for the four design storms. Peak discharges are typically reduced by 15-25%. As a result, flood stages are reduced by 1-3 feet. As with the native 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). This outcome reflects the landscape's diminished capacity to infiltrate additional water as rain rates increase.

Figure 4.10. Peak discharge reductions: First soil improvement case (Type B or B/D to A) for the 10-, 25-, 50-, and 100-year, 24-hour design storms (4.05, 5.05, 5.89, and 6.81 inches of rain in 24 hours).

Improving Soil Quality: Case 2

The second soil improvement case — where all Type C soils (loam containing silt and/or clay) improve to Type B (sandy loam or loamy sand) — results in less reduction of runoff and peak discharges than the first case. On average for the basin, only 0.2 to 0.3 inches of additional infiltration occur for the four design storms.

Figure 4.11 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, reductions are fairly uniform at the seven index locations, ranging from 7-11%.

Figure 4.11. Subbasin peak discharge reductions: Second soil improvement case (Type C or C/D to B) for the 50-year, 24-hour design storm (5.89 inches of rain in 24 hours).

Figure 4.12 shows the peak discharge reductions resulting from this hypothetical scenario at the seven index locations for the four design storms. Peak discharges 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 the middle part of the watershed, particularly in Worth and Mitchell counties, the peak discharge reductions show an increasing trend moving downstream from Austin to Charles City. In the lower third of the watershed, 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.
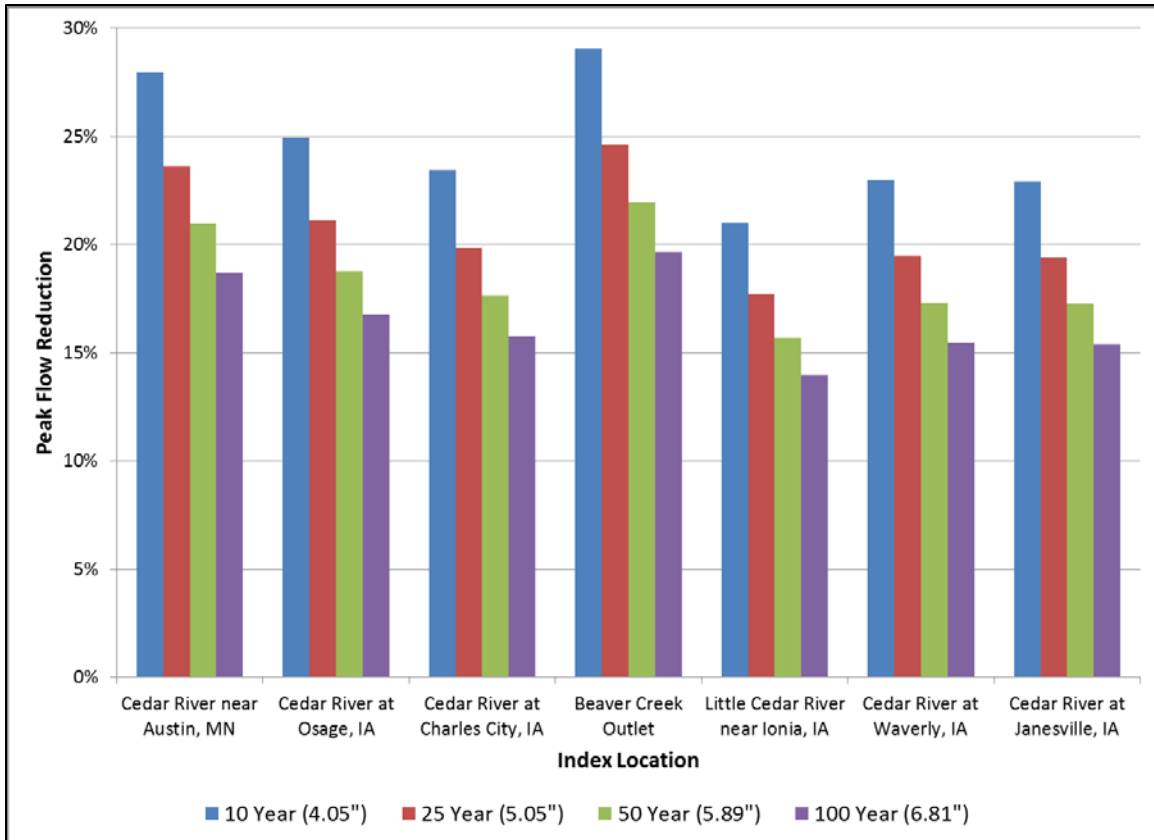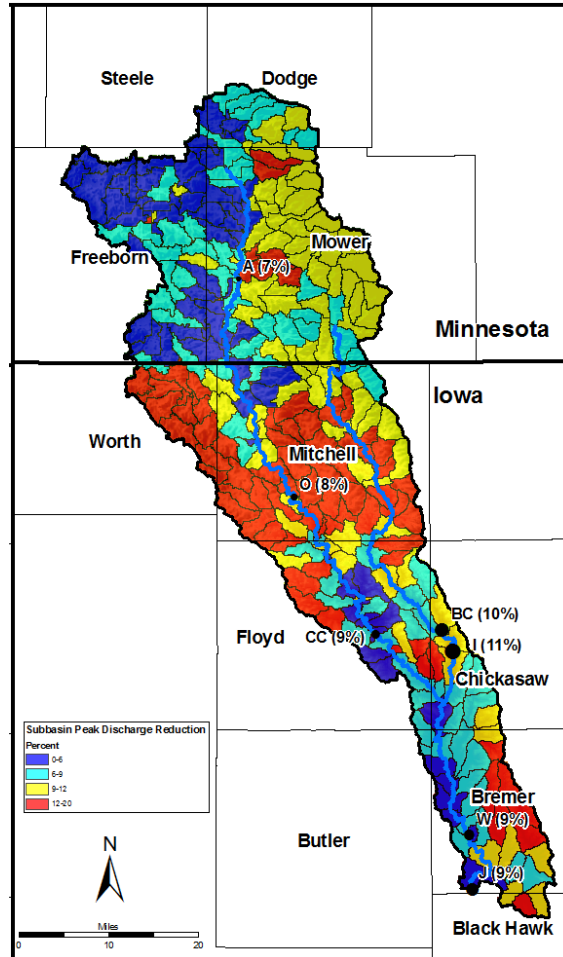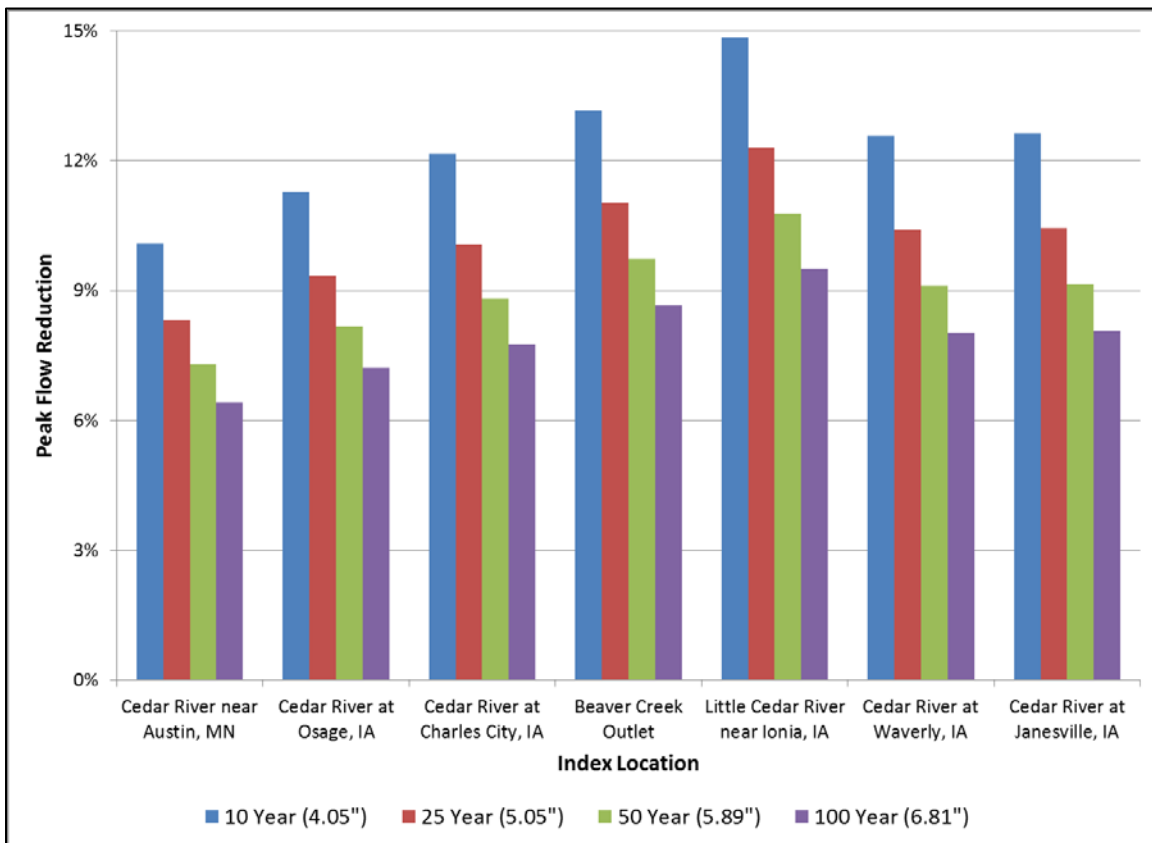

Figure 4.12. Peak discharge reductions: Second soil improvement case (Type C or C/D to B) for the 10-, 25-, 50-, and 100-year, 24-hour design storms (4.05, 5.05, 5.89, and 6.81 inches of rain in 24 hours).

c. Mitigating the Effects of High Runoff with Flood Storage

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. 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 built.

In this section, we use the HMS model 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, we instead use a prototype pond design that mimics the hydrologic impacts of flood storage. Therefore, this example is not a proposed plan for siting a system of storage ponds; we have 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.

i. Prototype Storage Pond Design

Many ponds in Iowa have been constructed to provide flood storage. A pond schematic is illustrated in Figure 4.13. The pond is created by constructing an earthen embankment across the 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. 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. The volume between the principal spillway elevation and emergency spillway elevation is called the flood storage.
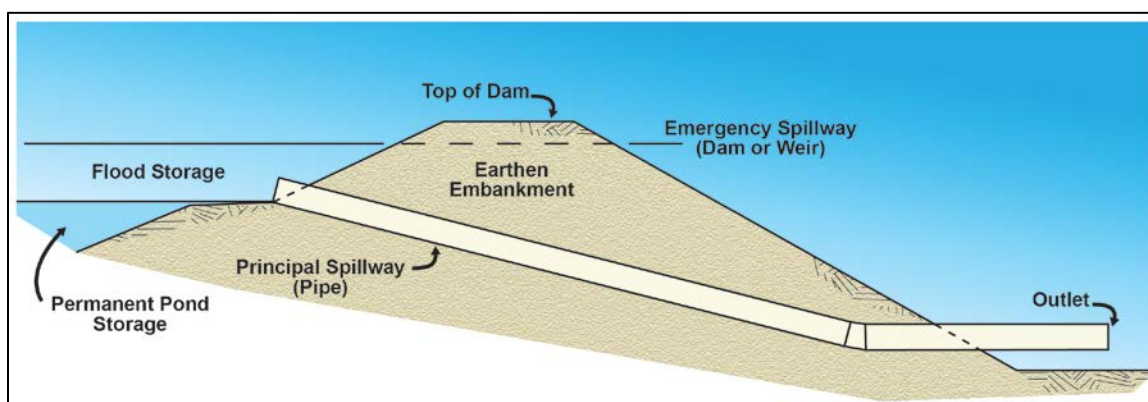
Figure 4.13. Prototype pond used for distributed flood storage analysis.

Prototype Pond Outlet and Emergency Spillway

Using information from ponds constructed in Soap Creek (Davis County, Iowa) 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. 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, releases also include contributions from the emergency spillway. Discharge of 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).

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. As a compromise, we analyzed the relationship between stage and storage at a few potential pond sites in the Upper Cedar River Watershed, and averaged the results 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 4.14 shows the subbasins in the HMS model. Of these, 121 are headwater basins. Headwater basins make good locations for flood storage ponds; they have relatively small drainage areas, and typical pond outlets (like the prototype above) can effectively reduce flood discharge at this scale. Hence, 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).

Figure 4.14. Subbasin locations selected for distributed flood storage analysis. Prototype ponds were placed in 121 headwater subbasins (colored beige) and seven of these subbasins (colored orange) were used as exploratory sites to develop a representative stage-storage relationship for the prototype pond.
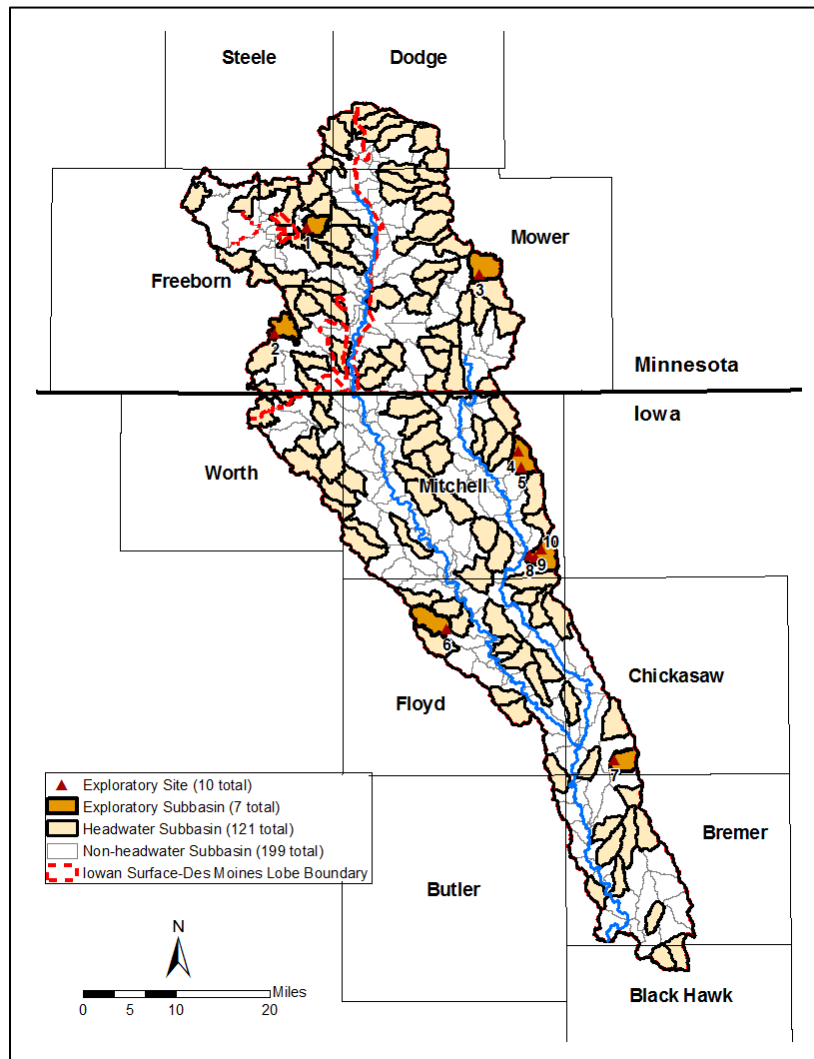
In each of the seven subbasins, a location for a pond embankment was selected. Each site was chosen where we felt there was sufficient topographic relief to support the construction of a pond. Then, for a given water level, we computed the volume of water that would be impounded behind the dam. This calculation was done by GIS analysis using the 10-meter resolution digital elevation model (DEM) of the local terrain. 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.

The last step was to compare the different stage-storage relationships developed for the seven pond locations. The stage-storage relationships for similar projects constructed in the Soap Creek watershed were also examined. 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 average fit to relationships from all seven sites — was selected for use in all the simulations.

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. 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. 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.

ii. 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, we placed prototype ponds throughout the headwater subbasins (see again Figure 4.14). In the Soap Creek Watershed, where flood storage is already used extensively, the average pond density is 1 built pond for every 1.9 mi² of drainage area. Therefore, for the flood storage simulations for the Upper Cedar River Watershed, we decided to place pond structures in headwater subbasins at a density of 1 pond for every 2 mi² of drainage area.

The 121 headwater subbasins range in size from 3.9-9.3 mi². Hence, all the subbasins contain more than one pond. For example, if a subbasin drainage area was 6 mi², it would have 3 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, we first assumed that half the subbasin area drains through a pond, and half does not. Next, for areas that drain through a pond, we 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. That is, if there were 3 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. 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 subbasins contained between 2 and 5 ponds. Figure 4.15 shows the 121 headwater subbasins, and the number of 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 mitigate flows from a total area of 375 mi²; in other words, 23% of the watershed area drains through the simulated prototype ponds.
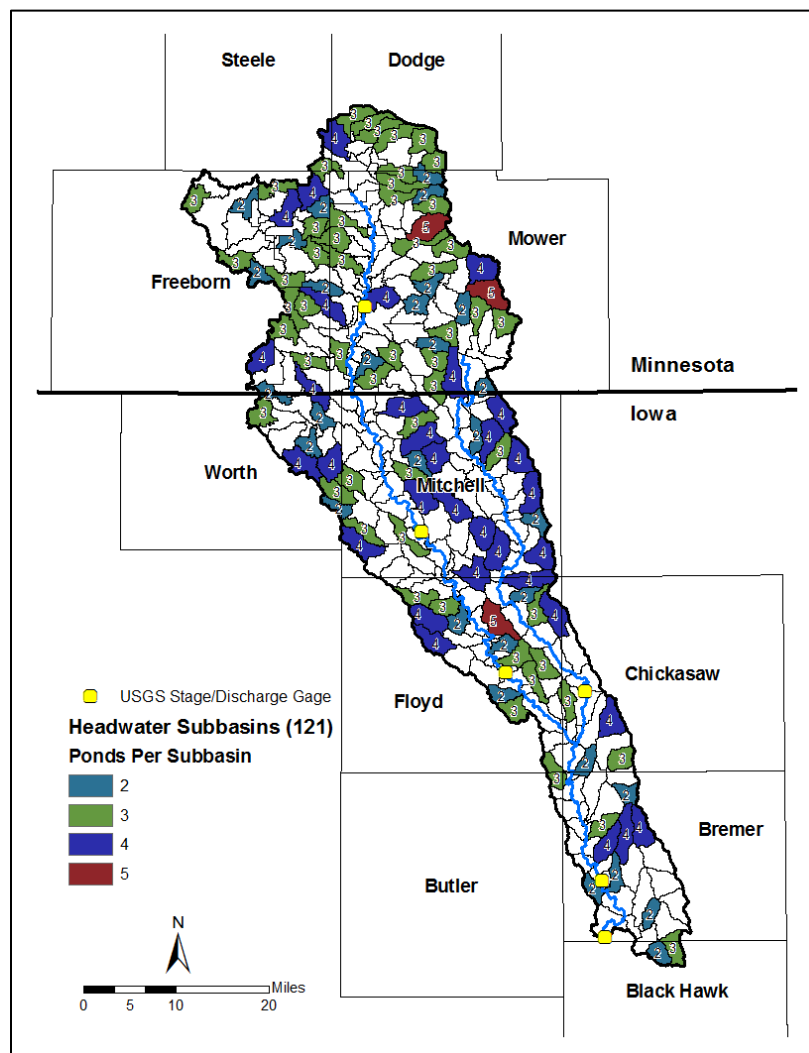
Figure 4.15. Headwater subbasins selected for distributed flood storage analysis and number of prototype ponds assigned to each subbasin. Each headwater subbasin contained 2-5 ponds, resulting in 372 prototype ponds that were aggregated into 121 ponds, one for each headwater subbasin.

The pond characteristics upstream of the seven index locations are characterized in Table 4.1. Overall, the percentage of area upstream of 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 4.1. Summary of pond characteristics for distributed flood storage analysis. The table summarizes the number of aggregated ponds, prototype ponds, and the upstream area at each index location.

Location	Drainage Area (mi²)	Aggregated Ponds Upstream	Prototype Ponds Upstream	Drainage Area Upstream from Ponds (mi²)
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 4.2 summarizes the flood storage available upstream of the locations. For small ponds, the total flood storage is 4,069 acre-feet; this amount of water placed over the upstream drainage area 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. These average storage depths are relatively consistent at the seven locations.

Table 4.2. Summary of the flood storage available upstream of the seven index locations for the small, medium, and large pond scenarios.

Pond Size Flood Storage/Prototype Pond		Small 10.9 acre-feet		Medium 26.8 acre-feet		Large 48.2 acre-feet	
Location	Total Flood Storage Available Upstream (%)	Flood Storage Upstream (acre-feet)	Equivalent Depth (inches)	Flood Storage Upstream (acre-feet)	Equivalent Depth (inches)	Flood Storage Upstream (acre-feet)	Equivalent 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

iii. Storage Pond Simulations

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 are in place. Each simulation started with all pond water levels at the primary spillway elevation; this assumes that the permanent storage is full as the storm begins. Comparisons were then made for the simulated discharges without ponds in place (the existing baseline condition). Flood hydrographs were compared for the 10-, 25-, 50-, and 100-year return period, 24-hour NRCS design storms.

Small Pond Scenario

Figure 4.16 shows the peak discharge reductions at the seven index locations for the small pond scenario (3 foot emergency spillway elevation). In this scenario, each 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 peak 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 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 area draining through 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.
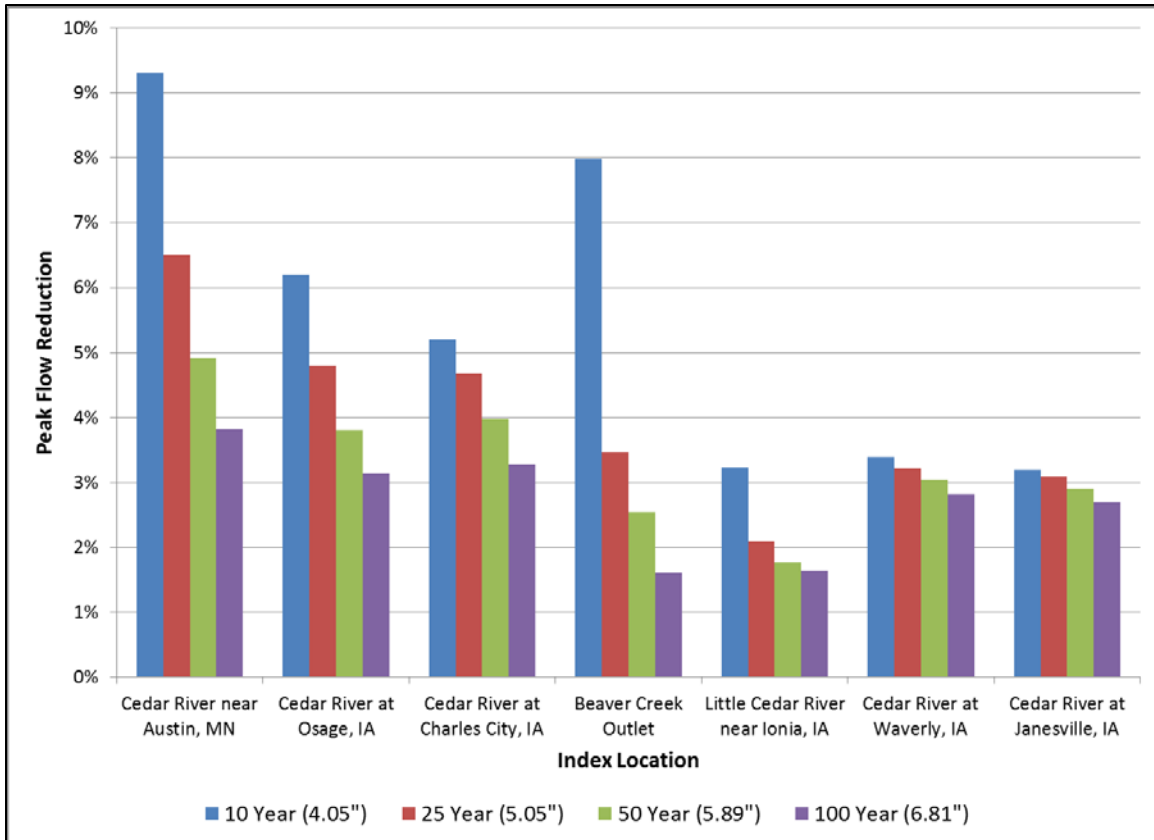
Figure 4.16. Peak discharge reductions: Effect of small ponds for the 10-, 25-, 50-, and 100-year, 24-hour design storms (4.05, 5.05, 5.89, and 6.81 inches of rain in 24 hours).

Medium-Sized Pond Scenario

Figure 4.17 compares the simulated flood hydrographs for the current no pond condition (baseline) to those with medium-sized 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 mi². 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 discharge reduction is the maximum observed at the seven sites (13%). Austin also has the maximum upstream area draining through ponds (25.8%). Even though the area is very similar downstream, the peak discharge reduction is not; the peak discharge 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.
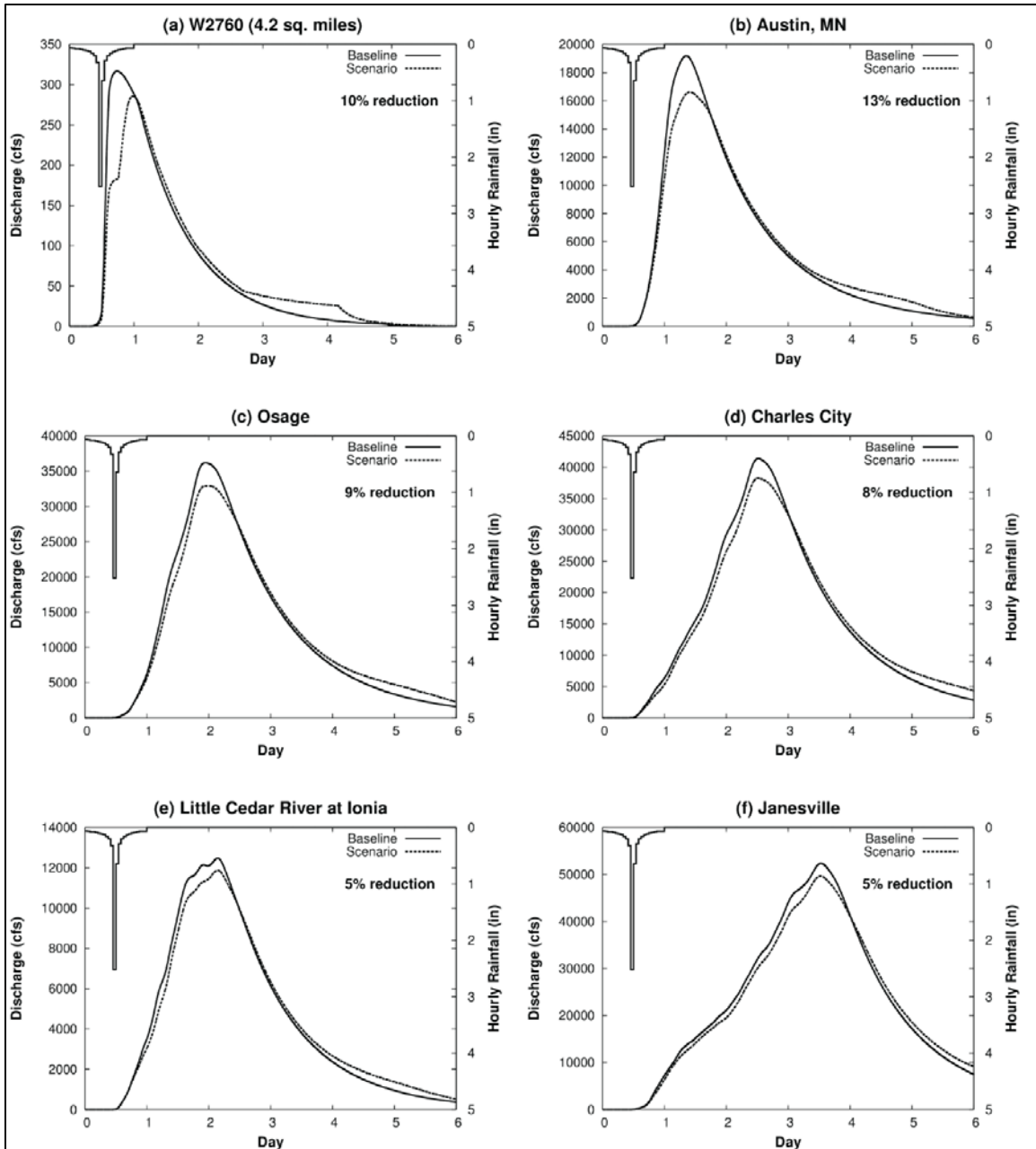
Figure 4.17. Hydrograph comparison: With and without medium-sized ponds for the 50-year, 24-hour design storm (5.89 inches of rain in 24 hours).

Figure 4.18 shows the peak discharge reductions for the medium-sized pond scenario (5 foot emergency spillway elevation). In this scenario, each 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).
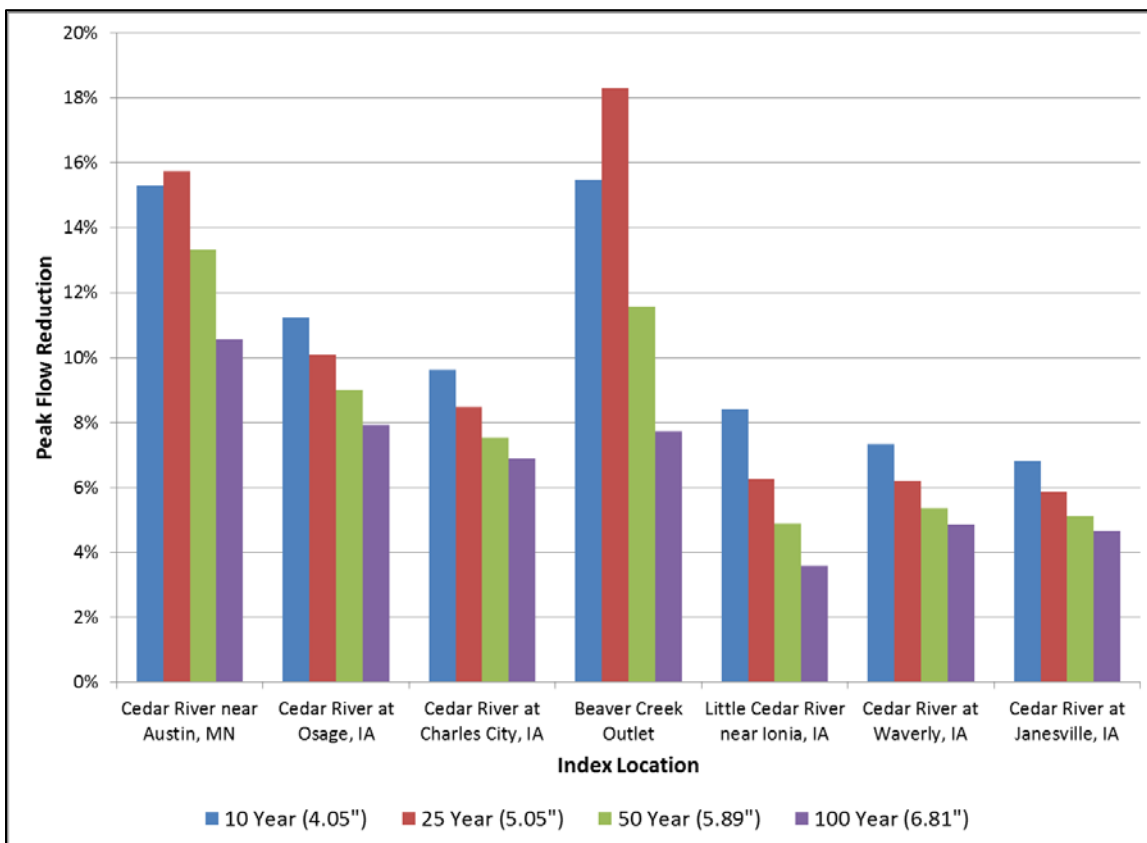

Figure 4.18. Peak discharge reductions: Effect of medium-sized ponds for the 10-, 25-, 50-, and 100-year, 24-hour design storms (4.05, 5.05, 5.89, and 6.81 inches of rain in 24 hours).

Large Pond Scenario

Figure 4.19 shows the peak discharge reductions for the large pond scenario (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 50-year 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).
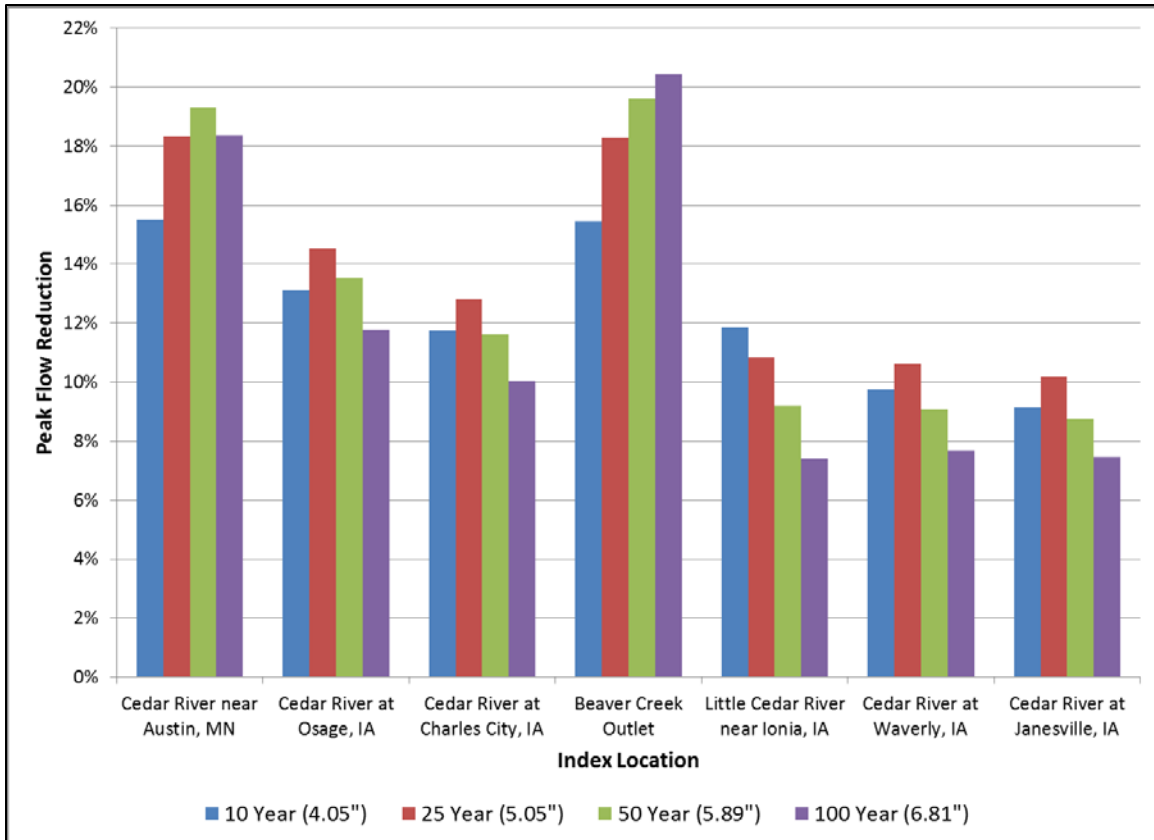
Figure 4.19. Peak discharge reductions: Effect of large ponds for the 10-, 25-, 50-, and 100-year, 24-hour design storms (4.05, 5.05, 5.89, and 6.81 inches of rain in 24 hours).

Summary of Pond Performance Characteristics

To illustrate how effectively the 121 aggregated ponds utilize their storage in the simulated flood events, the pond performance characteristics are summarized in Tables 4.3-4.6. Results are shown for the 10-, 25-, 50-, and 100-year return period, 24-hour NRCS design storms. Additional figures are provided in Appendix A. For the 10-year return period design flood (Table 4.3), the water level reaches the emergency spillway elevation for all 121 of the small ponds (3 foot emergency spillway elevation). In contrast, the water level reaches the emergency spillway for only 92 (76%) of the medium-sized ponds (5 foot emergency spillway elevation) and 15 (12%) of the large ponds (7 foot emergency spillway elevation). As a result, all (100%) of the flood storage is utilized in a 10-year flood for small ponds, compared to 97% for the medium-sized ponds and 70% for the large ponds.

Table 4.3. Aggregated pond performance characteristics for the 10-year, 24-hour design storm (4.05 inches of rain in 24 hours).

Pond Scenario	Ponds Activating Emergency Spillway (121 total)	Ponds with Peak Water Level One Foot Above Emergency Spillway	Flood Storage Exhausted (%)
Small	121	2	100
Medium	92	0	96.6
Large	15	0	70.2

For the 25-year design flood (Table 4.4), the water level reaches the emergency spillway elevation for all 121 small and medium-sized ponds, and 106 (88%) of the large ponds.

Table 4.4. Aggregated pond performance characteristics for the 25-year, 24-hour design storm (5.05 inches of rain in 24 hours).

Pond Scenario	Ponds Activating Emergency Spillway (121 total)	Ponds with Peak Water Level One Foot Above Emergency Spillway	Flood Storage Exhausted (%)
Small	121	26	100
Medium	121	9	100
Large	106	3	99.1

By the 50-year design flood (Table 4.5), the water level reaches the emergency spillway for all the large ponds as well. The available flood storage in the watershed is exhausted for all three pond scenarios.

Table 4.5. Aggregated pond performance characteristics for the 50-year, 24-hour design storm (5.89 inches of rain in 24 hours).

Pond Scenario	Ponds Activating Emergency Spillway (121 total)	Ponds with Peak Water Level One Foot Above Emergency Spillway	Flood Storage Exhausted (%)
Small	121	73	100
Medium	121	50	100
Large	121	21	100

For the 100-year design flood (Table 4.6), no overtopping of the dam occurs in any scenario, but the water level does reach at least one foot above the emergency spillway for 117 (97%) small ponds, 104 (86%) medium-sized ponds, and 66 (55%) large ponds.

Table 4.6. Aggregated pond performance characteristics for the 100-year, 24-hour design storm (6.81 inches of rain in 24 hours).

Pond Scenario	Ponds Activating Emergency Spillway (121 total)	Ponds with Peak Water Level One Foot Above Emergency Spillway	Flood Storage Exhausted (%)
Small	121	117	100
Medium	121	104	100
Large	121	66	100

Effect of Using Permanent Storage for Additional Flood Storage

An interesting exercise would be to examine the additional flood reduction benefit gained from using the permanent storage of each pond for additional flood storage. Recall from Figure 4.13 the permanent storage is the volume below the primary spillway elevation. Ponds possessing permanent storage are often called wet ponds since they hold water most of the time. In contrast, when an outlet (such as a pipe) is located at the bottom of the pond, the pond begins to discharge water as soon as it begins to fill and drains to empty between rain events; this type of pond is often referred to as a dry pond.

The prior pond analysis considered the impact a system of wet ponds could have on reducing flood peak discharges. The principal spillway was set at an elevation three feet above the pond bottom in order to achieve a reasonable surface area (2-3 acres) according to NRCS technical references. This resulted in each prototype pond having a permanent storage of 2.76 acre-feet. Utilizing the permanent storage of each pond for additional flood storage would add 1,028 acre-feet of flood storage to the watershed for each pond size scenario; this represents a 25% increase in the total flood storage for the small pond scenario, a 10% increase for the medium-sized pond scenario, and a 6% increase for the large pond scenario. Alternatively, the 1,028 acre-feet of flood storage gained from the permanent storage of each pond represents a water depth of 0.05 inches over the drainage area upstream of the ponds; hence, the ponds could temporarily store 0.05 inches of additional runoff from a storm event when the permanent storage is utilized for flood storage.

For each pond scenario, the effect that using the permanent storage for additional flood storage could have on reducing flood peak discharges was simulated for the 10-, 25-, 50-, and 100-year, 24-hour design storms. The same elevations and outflow characteristics were assumed for the primary and emergency spillways as before; in addition, a smaller outlet (a two-inch pipe) was set at the bottom of each pond so each pond would immediately begin discharging water as it filled from a particular storm event.

Figure 4.20 shows the additional peak discharge reduction gained at each index location from using the permanent storage for additional flood storage for the four design storms. A positive anomaly indicates a greater peak discharge reduction was achieved when the permanent storage was utilized for additional flood storage. For instance, the anomaly for the 10-year, 24-hour storm (upper left panel) at Austin is approximately 1.3% for small ponds, meaning the peak discharge reduction at Austin was about 10.6% when the permanent storage was used for additional flood storage and 9.3% when it was not (Figure 4.16).

Overall, using the permanent storage for additional flood storage has a negligible effect on increasing the peak discharge reduction at downstream sites. Figure 4.20 shows that the permanent storage has the greatest effect on increasing the peak discharge reduction at the seven index locations for the small pond scenario. The peak reduction benefit is greatest for the 10-year, 24-hour design storm and decreases for larger flood events. For a given design storm and index location, the additional peak reduction gained from the permanent storage diminishes as the pond size increases. This is because the permanent storage represents a smaller fraction of the total flood storage for larger ponds. For instance, for the 10-year event, using the permanent storage for additional flood storage increases the peak reduction at Austin by 1.3% with small ponds but less than 0.4% with large ponds. Finally, for a given pond size, the

added peak reduction from permanent storage diminishes at each location with increasing storm size. Peak reductions also generally decrease moving downstream for a particular flood event. This serves as another confirmation that ponds are best at reducing discharges directly downstream of their sites and reductions typically diminish moving downstream to sites with greater drainage areas.

Several conclusions can be drawn from this permanent storage analysis. While utilizing the permanent storage for additional flood storage does increase the peak discharge reductions at downstream sites, the effects are minimal and would be difficult to discern in practice. Using the permanent storage for additional flood storage may be a possibility with small ponds, but the additional peak reduction benefit diminshes as the storm size increases and is minimal for larger ponds. Ultimately, in order to increase peak discharge reductions with distributed flood storage by a noticeable amount, larger ponds must be built that provide substantially greater flood storage. This is achieved by increasing the elevation difference between the primary and emergency spillways. The flood storage added from the permanent storage of each pond is such a small contribution of the total flood storage, particularly for larger ponds, that the peak reduction effect is minimal.
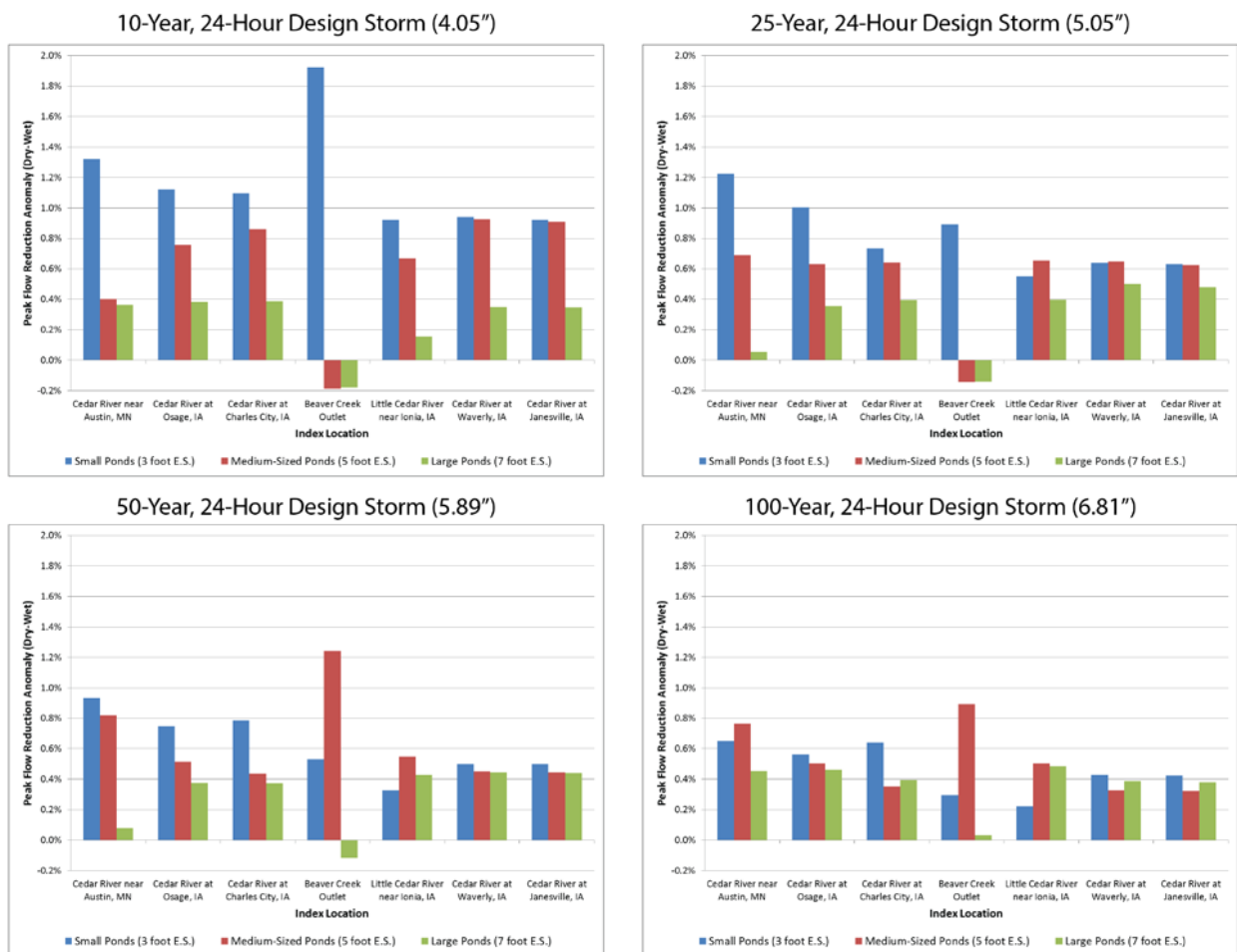

Figure 4.20. Peak discharge reduction anomalies: Effect of using permanent storage for additional flood storage for the 10-, 25-, 50-, and 100-year, 24-hour design storms (4.05, 5.05, 5.89, and 6.81 inches of rain in 24 hours). A positive anomaly represents the additional peak reduction benefit achieved from using the permanent storage for additional flood storage.

d. Mitigating the Effects of High Runoff with Increased Infiltration and Flood Storage

Projects to be constructed in the Upper Cedar are likely to rely on both enhanced infiltration practices and flood storage, so gaining a sense for the potential flood reduction benefit of both together is important. In this section, flood reduction resulting from distributed flood storage (storage ponds) and improved agricultural conditions due to cover crops is examined with the HMS model. Both strategies are applied in a more limited capacity than previously considered to represent a more realistic and feasible implementation scheme. Once again, this is a hypothetical and simplified example but does illustrate how multiple practices can work together to reduce flood peak discharges.

i. Distributed Flood Storage

The large pond design described in section 4c 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 emergency spillway set seven feet above the primary spillway. Each prototype large pond provides 48.2 acre-feet of flood storage.

The ponds were distributed in headwater areas with the greatest amount of topographic relief to reflect a more realistic implementation scheme. Of the 121 aggregated ponds located in headwater subbasins, half were retained in the steepest headwater subbasins. Figure 4.21 shows the 60 headwater subbasins selected for pond placement for the "blended" scenario. Not surprising, most of the ponds are located in the Iowan Surface 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.
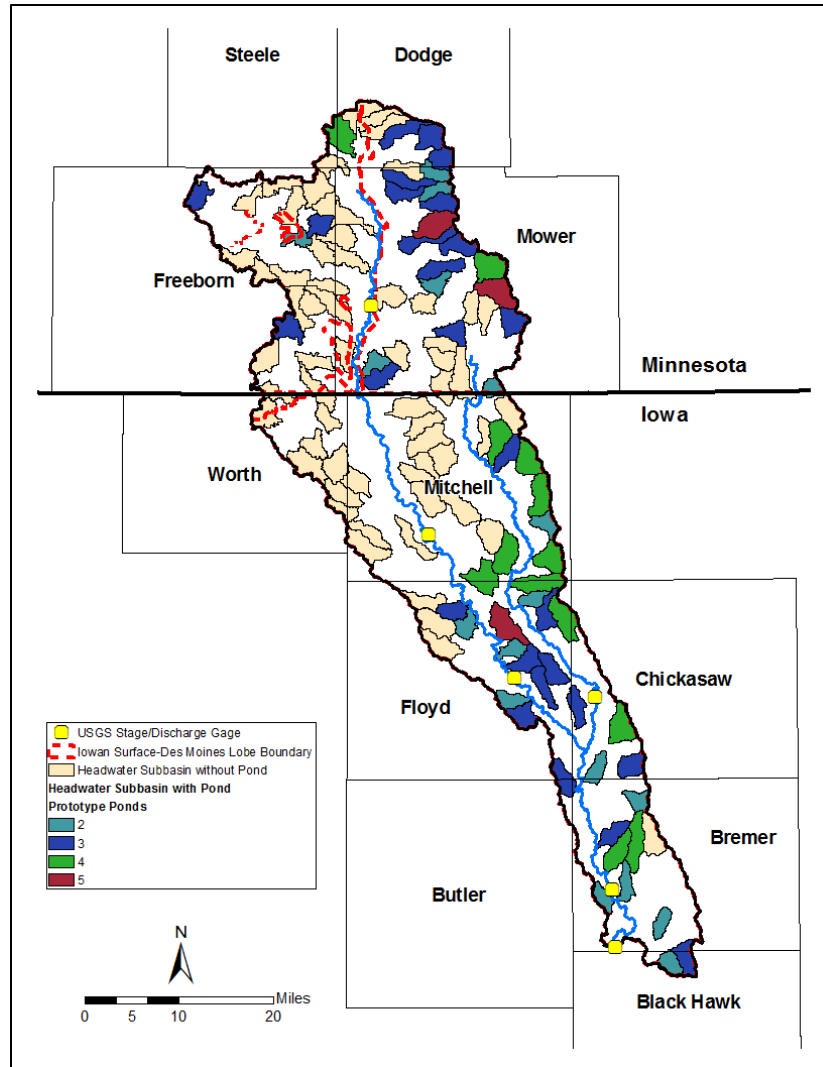
Figure 4.21. Headwater subbasins selected for pond placement for the blended scenario and number of prototype ponds assigned to each subbasin. Sixty ponds were placed in the steepest headwater subbasins, which tended to be located in the Iowan Surface region.

The pond characteristics upstream of the seven index locations are characterized in Table 4.7. The percentage of area upstream from the 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 area upstream from the ponds generally decreases moving downstream, as expected. However, a slight increase in the percentage of area 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 4.7. Summary of pond characteristics for the blended scenario. The table summarizes the number of aggregated ponds, prototype ponds, and area upstream from the ponds at each index location.

Location	Drainage Area (mi²)	Aggregated Ponds Upstream	Prototype Ponds Upstream	Drainage Area Upstream of Ponds (mi²)
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 4.8 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. This represents a uniform depth of about 0.9 inches over the upstream area draining through the ponds. As shown in Table 4.8, the average storage depths remain relatively equal at the seven locations.

Table 4.8. Summary of the flood storage available upstream of the seven index locations for the blended scenario.

Location	Total Flood Storage Available Upstream (%)	Flood Storage Upstream (acre-feet)	Equivalent Depth (inches)
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

ii. Enhanced Infiltration from Planting Cover Crops

In addition to simulating the effects of distributed flood storage, infiltration was also assumed to improve in the watershed by planting cover crops, though to a lesser extent than considered in section 4b. For the blended scenario, cover crops were implemented on 50% of the agricultural land on average, rather than 100% of the agricultural land considered in section 4b.

As with the previous land use change scenarios, improved agricultural management practices – planting cover crops during the dormant season to better soil quality and infiltration during the growing season – were represented in the HMS model through changes to CN. To represent cover crops being planted 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 section 4b (all agricultural area improved as a result of cover crops).

iii. Blended Scenario Simulations

The blended scenario – where half the agricultural land in the watershed is improved from planting cover crops and half (60) the number of aggregated large storage ponds are implemented – results in less flood reduction than either more extensive flood mitigation strategy. 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 4.22 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 mi², has two prototype ponds placed upstream, and is nearly 90% agriculture by area. About 0.2 additional inches of rainfall would infiltrate in this subbasin if 50% of the agricultural land were improved by planting cover crops during the dormant season. The flood storage in this subbasin and enhanced infiltration from cover crops reduces the peak discharge leaving this subbasin by 30%. This is greater than the reduction achieved through either 100% cover crops (10% reduction) or distributed storage with large, wet ponds (24% reduction). Moving downstream, reductions drastically decrease. The peak reduction at Austin is 14% and decreases to 7% at Janesville, which is less than was predicted from all 121 large, wet ponds or improvement of all (100%) agricultural areas from planting cover crops; the peak discharge reduction at Janesville for both these scenarios was around 9%.
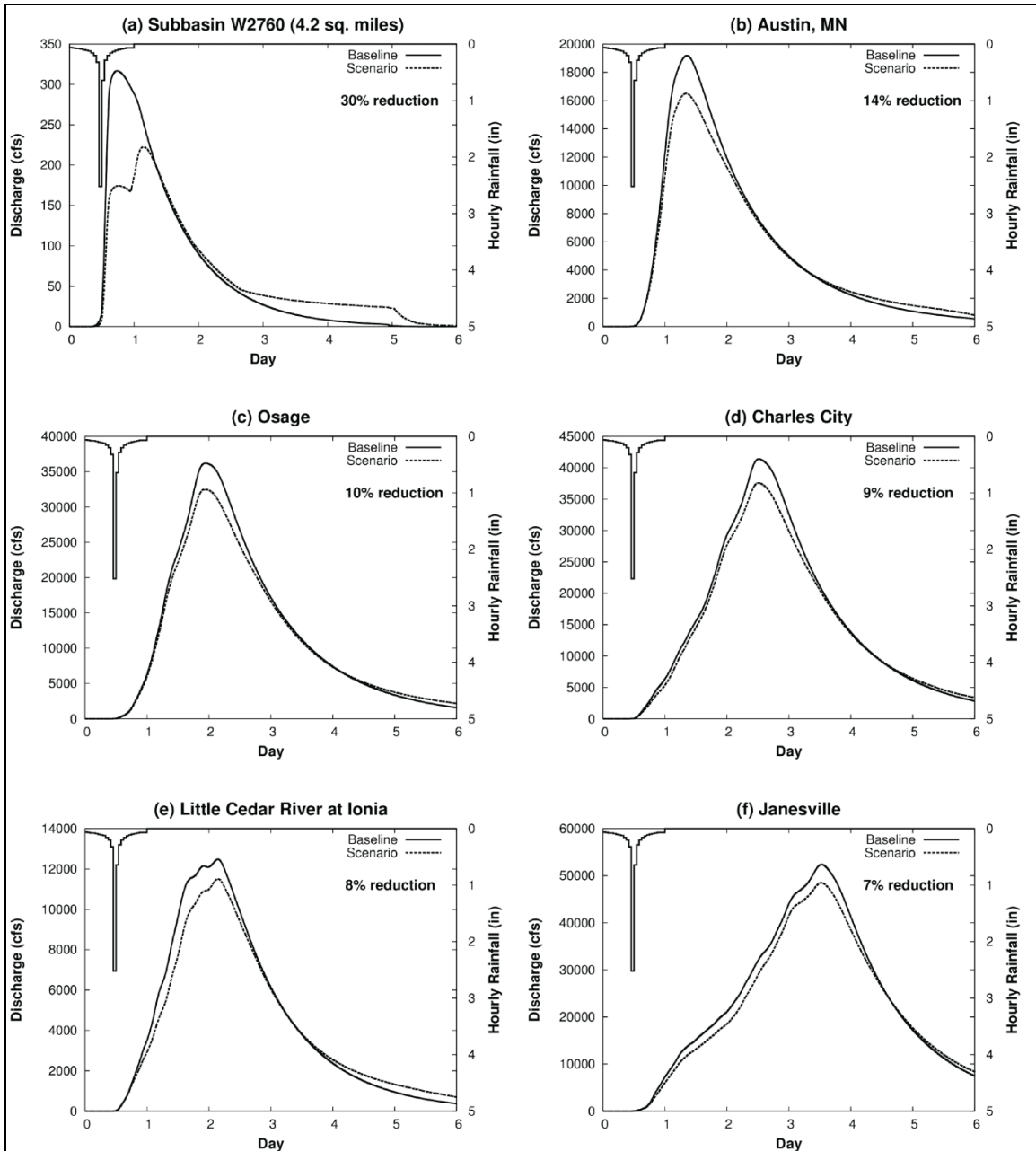
Figure 4.22. Hydrograph comparison: Blended scenario for the 50-year, 24-hour design storm (5.89 inches of rain in 24 hours).

Figure 4.23 shows the peak discharge reductions resulting from the blended scenario at the seven index locations for the four design storms. Reductions range from 6-24%; excluding the Beaver Creek outlet, peak reductions are less than 15%. Flood stages were reduced by 0.5-1.5 feet. The peak reductions at the index locations are similar to but generally less than either idealized hypothetical scenario; for the large pond scenario (121 aggregated ponds), peak reductions ranged from 7-20% and for the 100% cover crop scenario, reductions were between 8 and 12%.
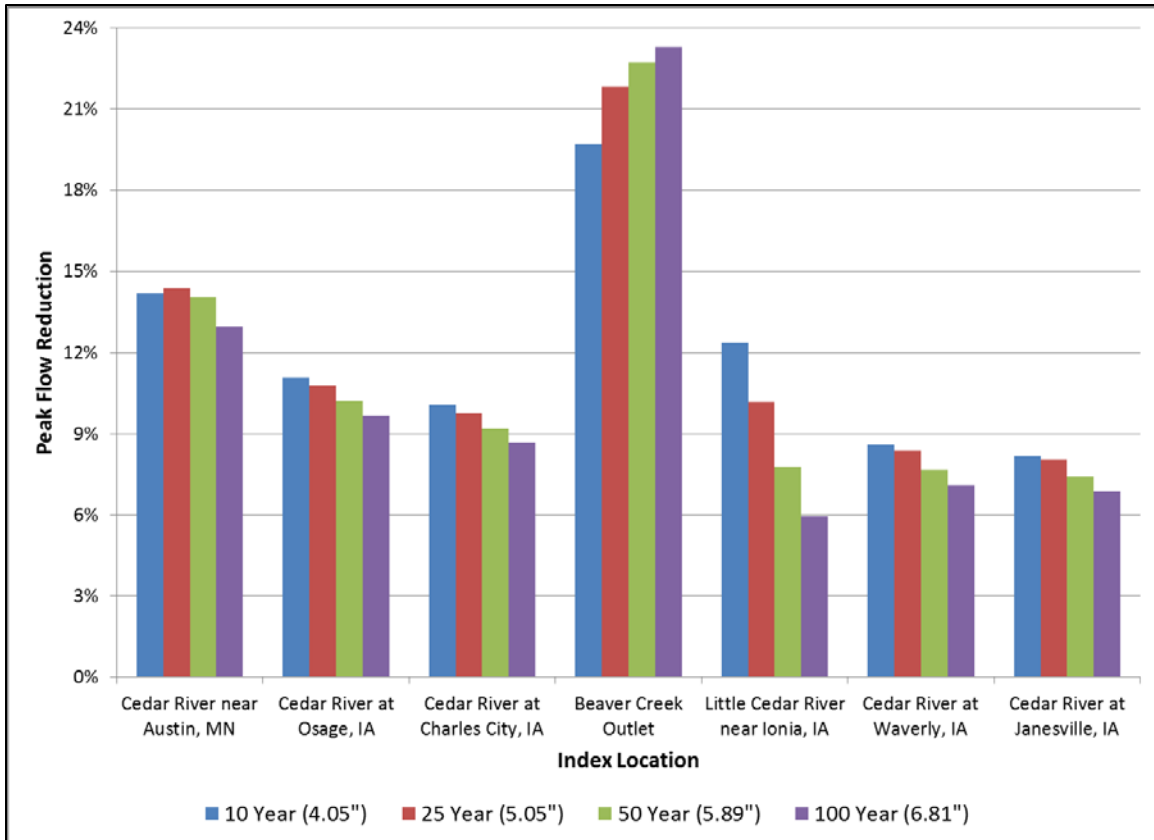
Figure 4.23. Peak discharge reductions: Blended scenario for the 10-, 25-, 50-, and 100-year, 24-hour design storms (4.05, 5.05, 5.89, and 6.81 inches of rain in 24 hours).

Pond performance characteristics for the blended scenario for the four design storms are summarized in Table 4.9. Additional figures showing the amount of flood storage utilized by each pond and the reductions predicted at the index locations for the four design storms are included in Appendix A. The percent 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 aggregated large ponds described in section 4c. 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 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 (same for the 121 large ponds). By the 50-year design flood, the water level reaches the emergency spillway of all ponds. 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 4.9. Summary of aggregated large pond performance characteristics for the blended scenario for the 10-, 25-, 50-, and 100-year, 24-hour design storms (4.05, 5.05, 5.89, and 6.81 inches of rain in 24 hours).

24-Hour Design Storm Return Period (years)	Ponds Activating Emergency Spillway (60 total)	Ponds with Peak Water Level One Foot Above Emergency Spillway	Flood Storage Exhausted
10	4	0	68%
25	54	1	99%
50	60	10	100%
100	60	33	100%

e. Evaluation of Flood Mitigation Strategies for the June 2008 Flood

In addition to evaluating flood mitigation strategies for reducing peak discharges throughout the watershed using NRCS design rain storm events, each flood mitigation scenario was also run for the June 2008 flood used as a validation storm event. 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 simulated responses of each flood mitigation scenario will be reasonable. In this section, the HMS model is used to simulate the effects the flood mitigation strategies considered in sections 4b-d could have had on reducing peak discharges for the June 2008 flood.

i. Overview of the June 2008 Flood

The HMS model was used to simulate the impact each flood mitigation strategy considered previously could have on reducing peak discharges throughout the watershed for the June 2008 flood. 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 4.24 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.
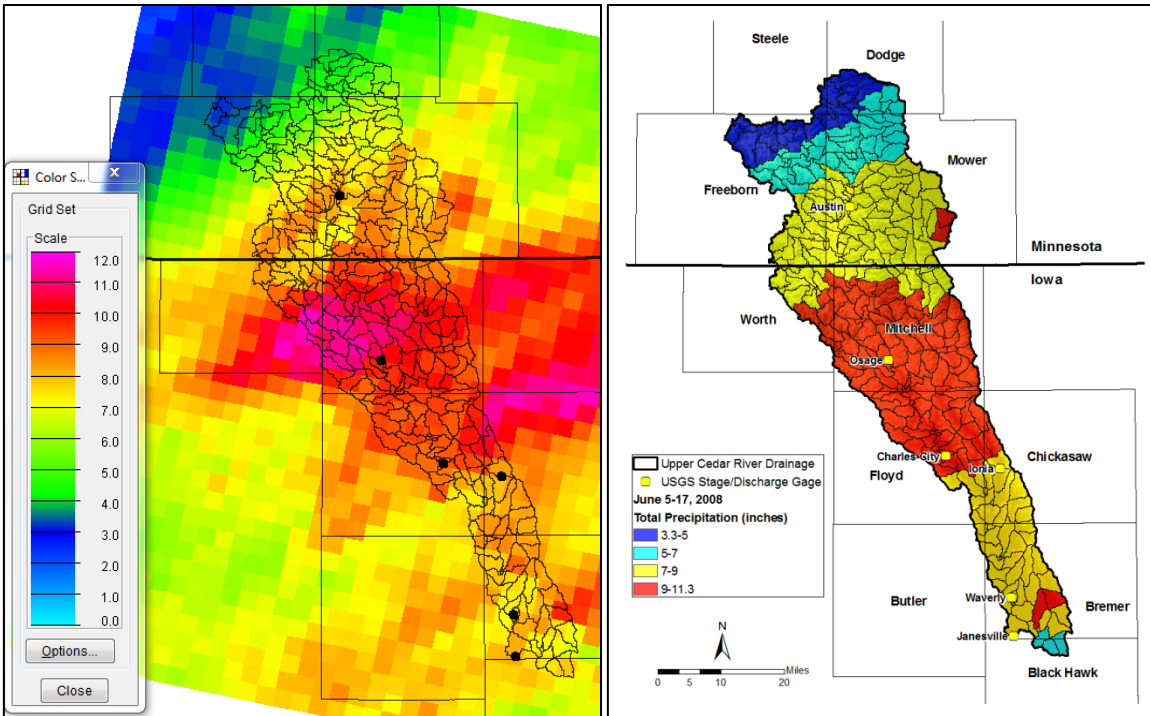
Figure 4.24. Cumulative rainfall estimated for the June 5-17, 2008 flood in the Upper Cedar River Watershed. Stage IV radar gridded estimates are shown on the left (in inches) and the cumulative rainfall estimated for each subbasin from the Stage IV estimates is shown on the right (in inches).

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. 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 reflect a higher runoff initial condition.

ii. Assessment of Flood Mitigation Strategies

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 flood.

Effects of Enhanced Infiltration Practices

Figure 4.25 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) involving a combination of distributed storage and cover crops. 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. Despite spatially variable rainfall for this historical event, peak 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 for 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. 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 the June 2008 flood, but no overtopping of the dam occurs for any pond. 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.
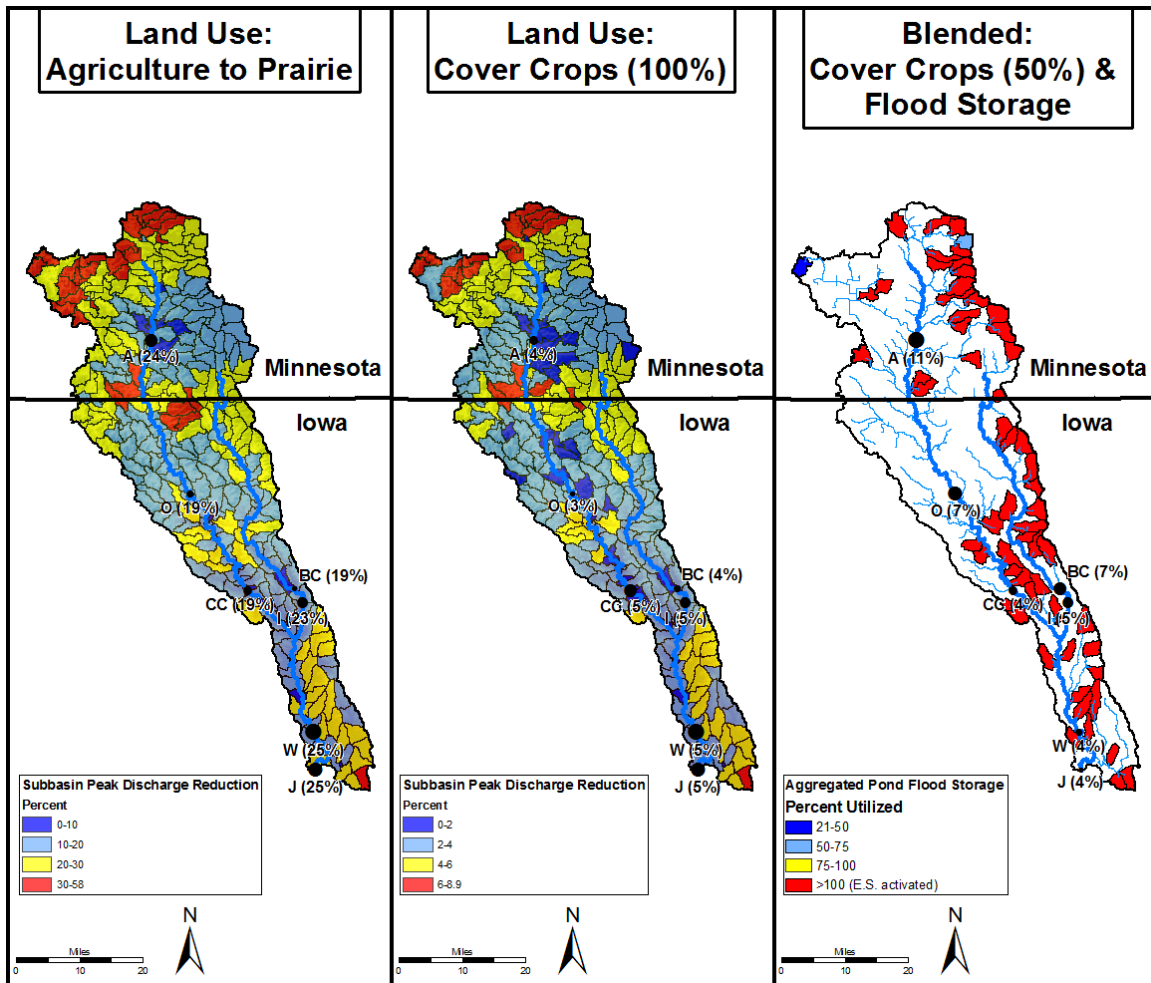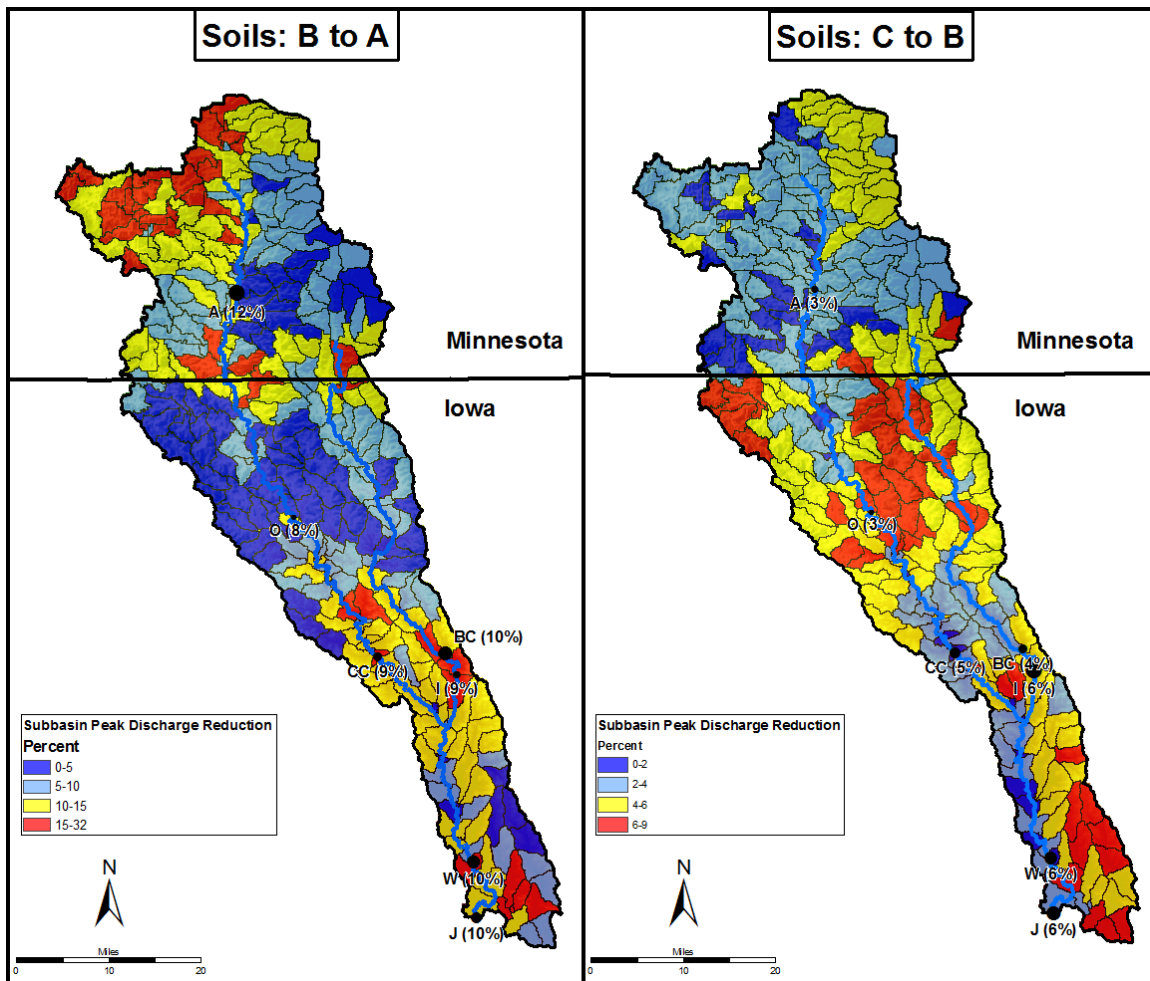
Figure 4.25. Subbasin peak discharge reductions: Enhanced infiltration from land use changes for the June 2008 flood simulation. Peak discharge reductions are shown for the tall-grass prairie (left), improved agricultural management practices (middle), and blended (right) scenarios.

Similarly, Figure 4.26 shows the peak discharge reduction for each subbasin and the seven index locations resulting from enhanced infiltration due to soil improvement scenarios. The first soil improvement case (Type B or B/D to A), a change in soil texture from a sandy loam or loamy sand to a sand or gravel, increases infiltration by about 0.8 inches (49% increase) and peak discharge reductions of 5-13% are common at the subbasin scale. Greatest peak discharge reductions are observed in the northern and southern thirds of the watershed where the greatest amount of Type B soils exist. Peak discharge reductions at the seven index locations throughout the watershed remain fairly even, ranging from 9-12%.

The second soil improvement case (Type C or C/D to B), a change in soil texture from a loam containing silt and/or clay to a sandy loam or loamy sand, increases infiltration in the watershed by about half as much as the first soil improvement case; infiltration is increased by about 0.4 inches (26% increase) and peak discharge reductions of 2-5% are common at the subbasin scale. 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 as much as observed for the first soil improvement case.

Figure 4.26. Subbasin peak discharge reductions: Enhanced infiltration from soil quality improvements for the June 2008 flood simulation. Peak discharge reductions are shown for the B to A (left, sandy loam/loamy sand to sand/gravel) and C to B (right, loam containing silt and/or clay to sandy loam/loamy sand) soil improvement cases.

Figure 4.27 compares the performance of each enhanced infiltration scenario (including the blended scenario which includes the effects of flood storage as well) for reducing peak discharges at the seven index locations for the June 2008 flood simulation. 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 second soil improvement (Type C or C/D to B) scenarios generally resulted in the least amount of flood reduction. In general, peak 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 (often Osage, Charles City, and the Beaver Creek outlet) where rainfall was most intense. 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.
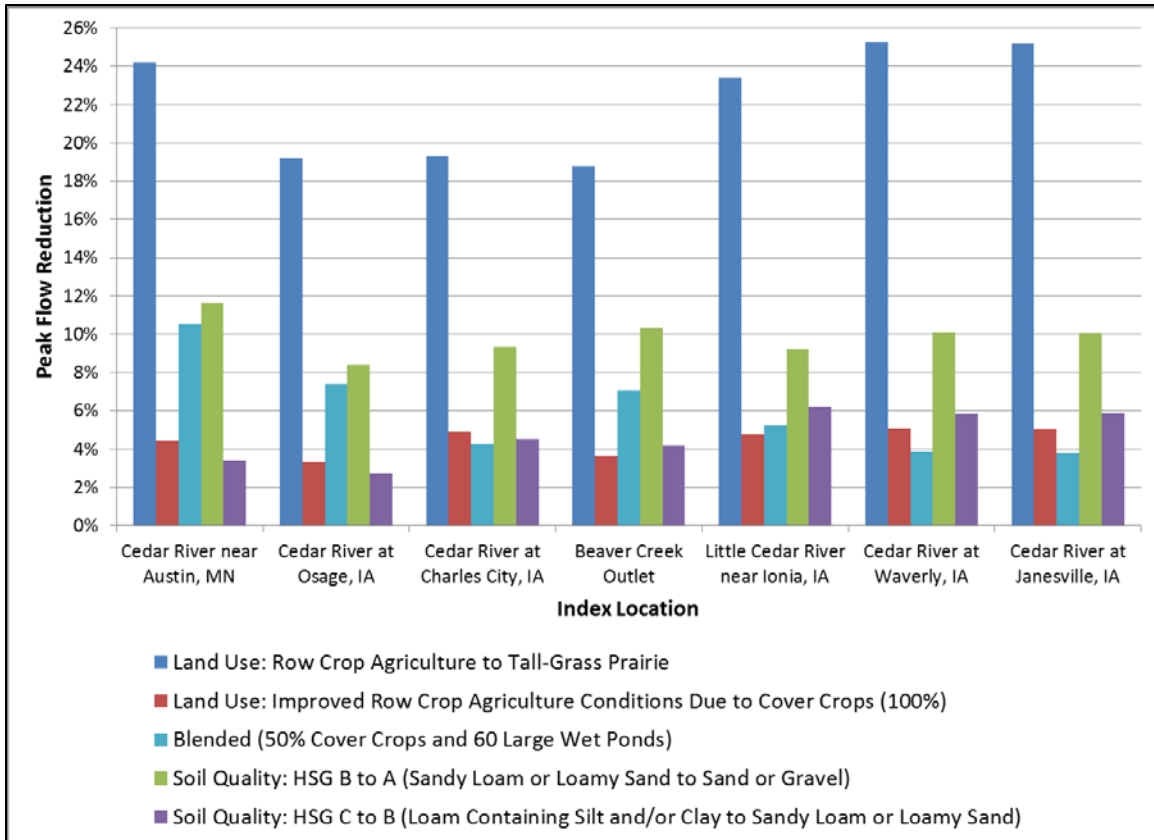
Figure 4.27. Peak discharge reductions: Effect of enhanced infiltration practices for the June 2008 flood simulation.

Effects of Distributed Flood Storage

The effects distributed flood storage could have had on reducing peak discharges for the June 2008 flood 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 section 4c.

Pond performance characteristics and peak discharge reductions at the seven index locations for the June 2008 flood simulation are summarized in Table 4.10 and Figure 4.28 below. For all pond scenarios, nearly 100% of the flood storage in the watershed was exhausted. The water level reaches the emergency spillway for all (100%) small ponds, 116 (96%) medium-sized ponds, and 110 (91%) large ponds. The water level reached the emergency spillway for 58 of 60 (97%) large ponds for the blended scenario, and nearly 98% of the flood storage was exhausted. The flood storage of each pond is exhausted except for a few of the medium and large ponds in the northern part of the watershed (where rain was less intense). 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 50 (41%) of the large ponds.

Table 4.10. Summary of pond performance characteristics for the June 2008 flood simulation.

Pond Scenario	Ponds Activating Emergency Spillway (121 total)	Ponds with Peak Water Level One Foot Above Emergency Spillway	Flood Storage Exhausted
Small	121	82	100%
Medium	116	68	99%
Large	110	51	96%
Blended (large ponds)	58 (60 total)	24	97.7%
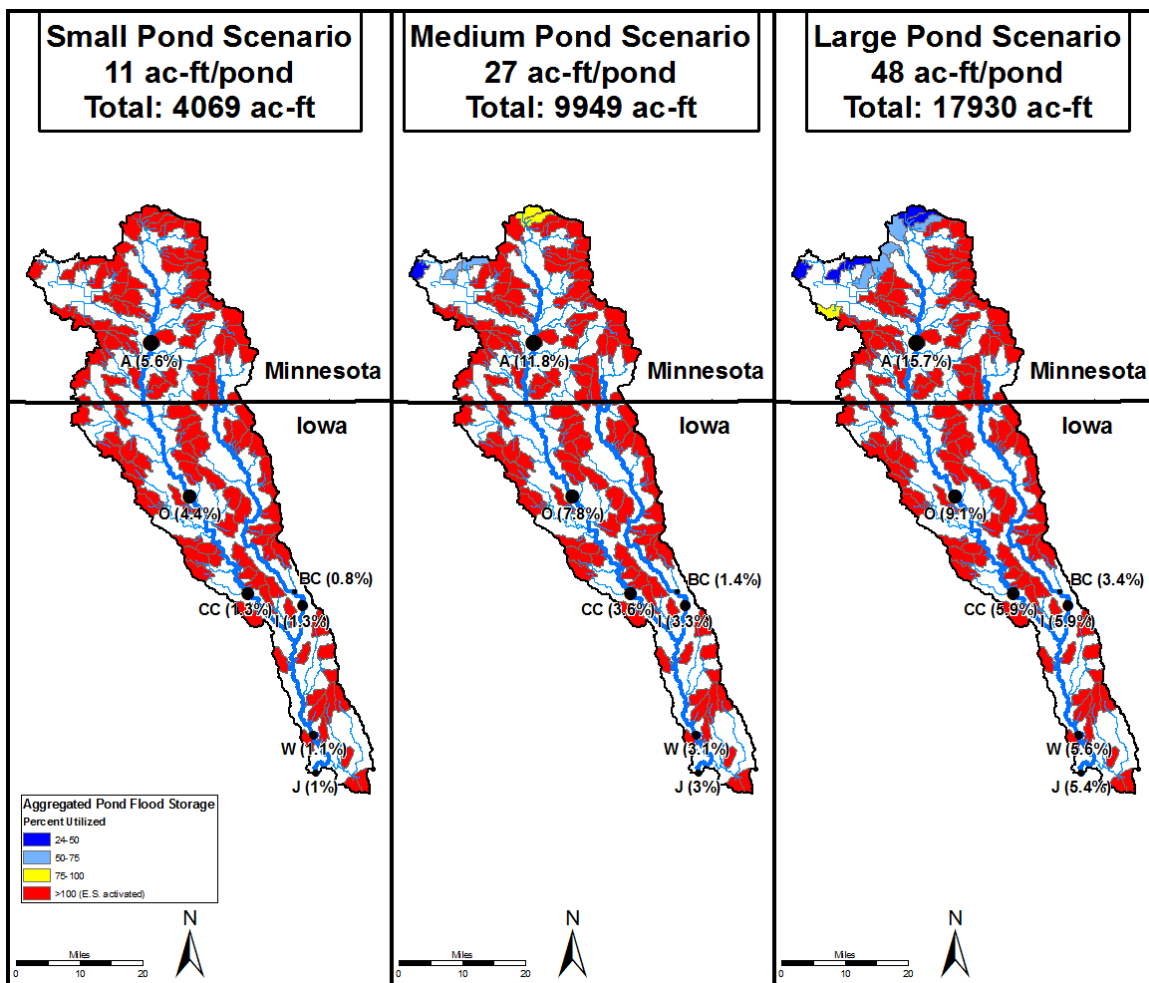
Figure 4.28. Comparison of small, medium, and large pond performance for the June 2008 flood simulation.

Figure 4.29 compares the peak discharge reductions estimated at the seven index locations for each pond scenario for the June 2008 flood simulation. As expected, the peak discharge reduction at each location increases with increasing pond size since a greater amount of flood 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 reasoning as described for

the design rain events – with distributed flood storage, the peak reduction effect is largest nearer to the ponds and decreases downstream. However, the spatial variability of rainfall for this historical event also impacts the peak discharge 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.
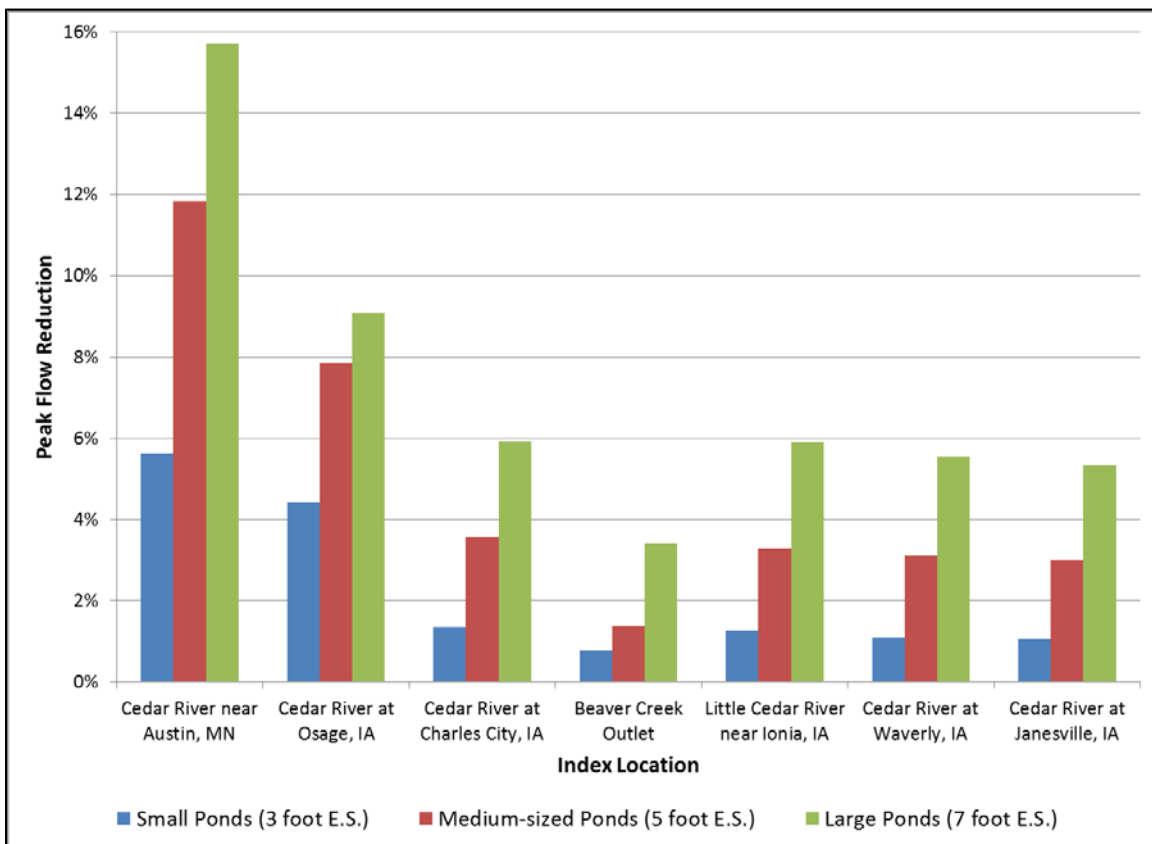
Figure 4.29. Peak discharge reductions: Effect of distributed flood storage (small, medium-sized, and large ponds) for the June 2008 flood simulation.

5. Summary and Conclusions

This hydrologic assessment of the Upper Cedar River Watershed is part of the Iowa Watersheds Project, a project being undertaken in four watersheds across Iowa by the Iowa Flood Center located at IIHR—Hydroscience & Engineering on the University of Iowa campus. The assessment is meant to provide local leaders, landowners and watershed residents in the Upper Cedar River Watershed an understanding of the hydrology – or movement of water – within the watershed, and the potential of various hypothetical flood mitigation strategies.

a. Upper Cedar Water Cycle and Watershed Conditions

The water cycle of the Upper Cedar River Watershed was examined using historical precipitation and streamflow records. The average annual precipitation for the Upper Cedar River Watershed is 35.1 inches. Of this precipitation amount, 68.5% (24.0 inches) evaporates back into the atmosphere and the remaining 31.5% (11.1 inches) runs off the landscape into the streams and river. The majority of the runoff amount is baseflow (68.9% or 7.6 inches), and the rest is surface flow (31.1% or 3.4 inches). Average monthly streamflow peaks in April, and decreases slowly through the summer growing season. In most years, the largest discharge observed during the year occurs in March or April, associated with snow melt, rain on snow events, or heavy spring rains. However, the largest floods on record tend to occur in the summer season (e.g., 2008), when the heaviest rainfall can occur.

The water cycle has changed due to land use and climate changes. The largest change occurred in the late 1800s when the landscape was transformed from low-runoff prairie and forest to higher-runoff farmland. Since the 1970s, Iowa has seen increases in precipitation, changes in timing of precipitation, and changes in the frequency of intense rain events. Streamflow records in Iowa (including those for the Upper Cedar) suggest that average flows, low flows, and perhaps high flows have all increased and become more variable since the late 1960s or 1970s; however, the relative contributions of land use and climate changes are difficult to sort out.

The majority of the Upper Cedar River Watershed lies in the Iowan Surface landform region and has well developed drainage; the northwest corner (mostly in Minnesota) lies in the Des Moines Lobe region and has poorly drained soils. Soils of the watershed have moderate to high runoff potential. The terrain is relatively flat — a quarter of the area has slopes less than 0.6%, and a quarter has slopes greater than 5.6%. The land use is predominantly agricultural; cultivated crops and pasture areas account for 86.4% of the area. Storms in recent decades produced record flood levels at stream-gages in August 1993, July 1999, September 2004, and June 2008.

b. Upper Cedar River Hydrologic Model

The U.S. Army Corp of Engineers' (USACE) Hydrologic Engineering Center's Hydrologic Modeling System (HEC-HMS) was used to develop a flood prediction model for the Upper Cedar River Watershed. First, the watershed was divided in 320 smaller units, called subbasins, with an average area of about 5.2 mi². For model calibration and validation with actual (historical) rainfall events, radar rainfall estimates were used as the precipitation input for simulation. For the analysis of watershed scenarios, 24-hour duration design storms (an NRCS Type-II

distribution) with rainfall accumulations equal to the 10-, 25-, 50-, and 100-year return period basin-average depths were used as the precipitation input.

The NRCS Curve Number (CN) methodology was used to determine the rainfall-runoff partitioning in the Upper Cedar River Watershed HMS modeling. The CN methodology accounts for precipitation losses due to initial abstractions and infiltration during the rainstorm. CN values are estimated based on land use and underlying soil type, and the areal-weighted average CN is assigned to each subbasin as an initial parameter estimate. Clark Unit Hydrograph methods were selected for converting excess precipitation into a direct runoff hydrograph for each subbasin to better account for the impacts of tile drainage. Baseflow was simulated for model calibration and validation with actual (historical) rainfall events, but was not simulated for the analysis of watershed scenarios with hypothetical design storms. Conveyance of runoff through the river network, or flood wave routing, was executed using the Muskingum routing method. Flow routing through Geneva Lake was executed using level pool routing.

Model calibration adjusts the initial set of model parameters so that simulated results match observed discharges at gaging stations more closely for historical events. The Upper Cedar River Watershed HEC-HMS model was calibrated with six storm events that occurred between September 2004 and June 2013 (significant runoff events where radar rainfall estimates are available). After calibration of model parameters, model validation assesses the predictive capability of the model to simulate discharge for other storm events (not used in calibration). Four storms were considered for model validation. Although the model tended to overestimate discharge for validation storm events that were smaller than those used in model calibration, its performance was good for the June 2008 flood; simulated hydrographs at multiple locations through the watershed closely match observations. The results show that the calibrated HEC-HMS model has predictive capability, particularly in simulating high runoff events (which is the focus of the analysis of watershed scenarios).

c. Watershed Scenarios for the Upper Cedar

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

To quantify the potential effects of flood mitigation strategies, the HEC-HMS model was used to simulate river flows throughout the Upper Cedar River Watershed. Several flood mitigation strategies were considered — enhancing local infiltration through changes in land use (from agricultural to native tall-grass prairie), enhancing local infiltration through improvements in

soil quality by cover crops and other practices, and storing floodwaters temporarily in ponds throughout the watershed to reduce downstream discharges. A blended scenario utilizing cover crops and storage ponds was also considered. The effects of these strategies were simulated for significant design flood events — those resulting from a 10-, 25-, 50-, and 100-year return period, 24-hour design rainfall. These events correspond to rainfall amounts of 4.05, 5.05, 5.89, and 6.81 inches in 24 hours over the entire Upper Cedar River Watershed. The effects were also simulated for the June 2008 flood event. The results for these strategies were compared to simulations of flows for the existing watershed condition. Although each scenario simulated is hypothetical and simplified, the results provide valuable insights on the relative performance of each strategy for flood mitigation planning.

Figure 5.1 summarizes the relative effectiveness of each flood mitigation strategy considered for reducing peak discharges. The Cedar River at Charles City was selected as the location for comparison, and the relative impact of each strategy from highest (restoration of native prairie) to lowest (distributed flood storage with the smallest pond size) is shown for both the 50-year, 24-hour design storm (5.89 inches of rain in 24 hours) and the June 2008 flood simulation. A brief summary of each flood mitigation strategy and concluding remarks are provided.
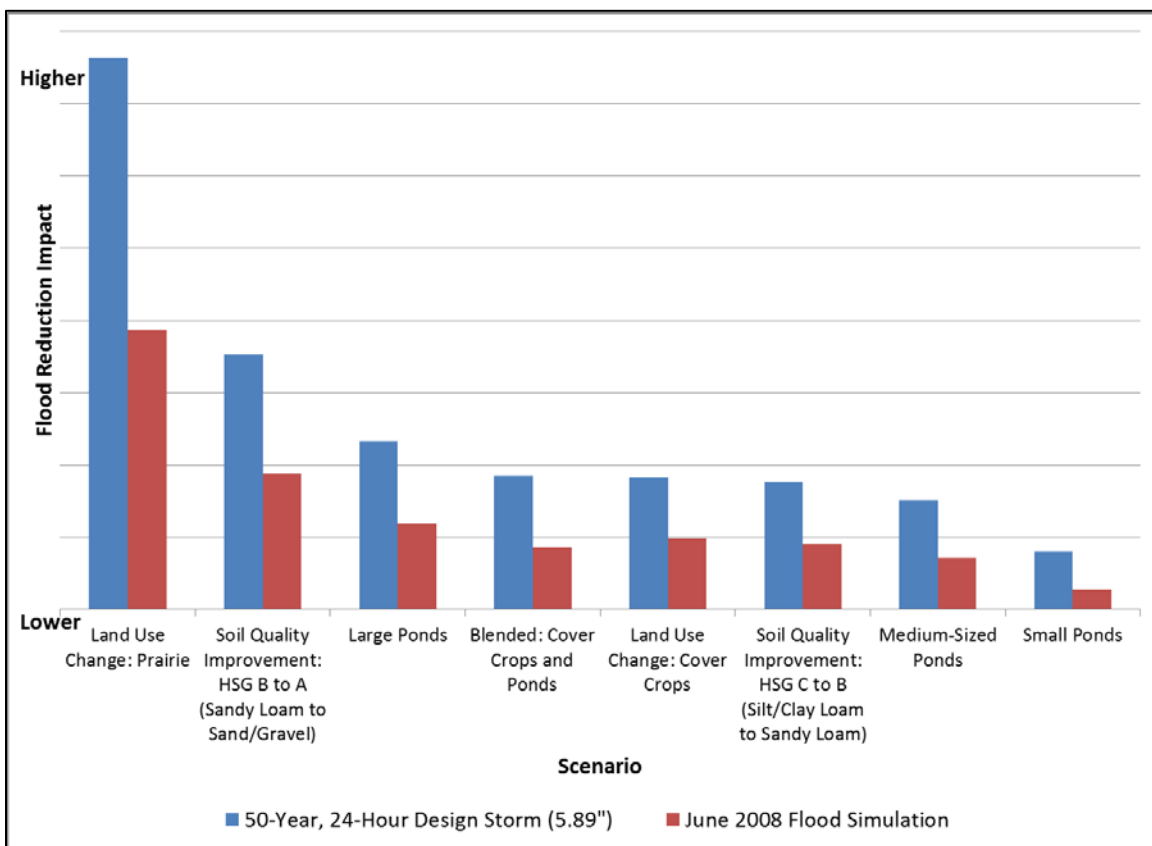
Figure 5.1. Comparison of the relative impact of the flood mitigation scenarios for reducing peak discharges on the Cedar River at Charles City. The restoration of native prairie had the greatest flood reduction impact while distributed storage with small ponds had the lowest impact for both the 50-year, 24-hour design storm (5.89 inches of rain in 24 hours) and the June 2008 flood simulation.

i. Increased Infiltration in the Watershed: Land Use Change

From the simulation results, enhancing local infiltration through changes in land use has the most significant impact on runoff. The model predicts that changes from an agricultural to a native tall-grass prairie landscape would increase infiltration during large storms by 0.8 inches (for the 10-year design event) to 1.3 inches (100-year event). Said another way, the conversion of native tall-grass prairie to agriculture land use has resulted in a significant reduction in the infiltration capacity of the landscape and more runoff. Obviously, converting the entire agricultural landscape back to tall-grass prairie (as was simulated) is not a practical or economically desirable strategy. Still, from a hydrologic point of view, targeted projects that enhance infiltration by land-use change could be an effective part of a watershed's flood mitigation efforts. Infiltrating more water is effective because potential floodwaters are instead stored within the landscape.

ii. Increased Infiltration in the Watershed: Cover Crop Agricultural Management Practice

Even without changes to land use, agricultural management practices could increase the infiltration of rain water. In particular, cover crop plantings during the dormant season can produce long-lasting improvements in soil quality. From the simulation results, enhancing local infiltration through soil quality improvement from cover crops has a small but significant effect. The model predicts that adopting cover crop management practices would increase infiltration during large storms by 0.2 inches (for the 10-year design event) to 0.3 inches (100-year event), about a quarter as much infiltration as simulated for a prairie landscape. Reductions in peak discharge throughout the watershed ranged from 12% (10-year event) to 8% (100-year event). Given the widespread agricultural land use in the Upper Cedar, and the growing interest in the use of cover crops as a part of Iowa's nutrient management strategy, implementation of such practices could play an important role as a watershed-wide flood mitigation strategy.

iii. Increased Infiltration in the Watershed: Improving Soil Quality

Crop covers are but one practice that can improve soil quality; the addition of other practices to improve soil quality could better utilize the storage capacity of the soils to enhance infiltration. Still, the hypothetical improved soil quality scenario suggests that it is a less effective strategy than land use change. The improved soil quality scenarios predicted an increased infiltration during large storms by as much as 0.4 inches (for the 10-year design event) to 0.6 inches (100-year event), about half or less than those simulated for changing land use. Still, locations where the land use remains the same, such increases in infiltration (and the resulting downstream reduction in flood discharges) are very significant. For the Upper Cedar River Watershed, where agricultural land use will continue to dominate for the foreseeable future, efforts to improve soil quality can also be an effective part of a watershed-wide flood mitigation strategy.

iv. Increased Storage on the Landscape

In some ways, using ponds to temporarily store floodwaters is an attempt to replace the loss of water that was stored in soils (in the pre-agricultural landscape). In the hypothetical scenarios involving pond storage, between 4,069 acre-feet and 17,930 acre-feet of storage was added to the Upper Cedar River Watershed with 372 prototype ponds. For the upstream areas that have runoff that drains through the ponds, this is equivalent to an added storage depth between 0.2

inches (using small ponds) and 0.9 inches (for large ponds) of runoff. However, for the Upper Cedar River Watershed as a whole (not just the area with ponds), the added storage depth ranges from 0.05 to 0.2 inches. Compared to the extra water that was stored by infiltration in the previous scenario simulations, the amount of storage replaced by ponds is much smaller. As a result, the overall flood peak reduction with storage ponds is less than predicted for the other scenarios. Still, compared to the other scenarios, the flood storage scenario is one that is more realistically achievable. As a flood mitigation strategy, ponds are very effective in reducing flood peaks immediately downstream of their headwater sites. Further downstream, floodwaters originating from locations throughout the watershed arrive at vastly different times; some areas are have ponds, others do not. The result is that the storage effect from ponded areas is spread out in time, instead of being concentrated at the time of highest flows. Hence, as one moves further downstream in the watershed, the flood peak reduction of storage ponds slowly diminishes.

v. Increased Infiltration and Increased Storage: A Blend of Cover Crops and Flood Storage Ponds

Projects to be constructed in the Upper Cedar are likely to rely on both enhanced infiltration practices and flood storage. A blended scenario was created — using cover crop agricultural management practices and ponds — to illustrate a flood mitigation approach that employs multiple strategies. This scenario assumes that half the agricultural land in the watershed is improved from planting cover crops, and that half the number of large ponds are implemented from the distributed flood storage scenario. From the simulation results, the blended scenario of enhancing infiltration and storing floodwaters would not be as effective as full implementation of cover crops or of large ponds (neither of which is likely to be fully achievable). Still, the model predicts that infiltration during large storms would increase by 0.1 inches (for the 10-year design event) to 0.2 inches (100-year event), and that peak discharge reductions of between 6 and 15% could be achieved at locations within the watershed.

vi. Watershed Scenarios for the June 2008 Flood

The performance of the different watershed scenarios was simulated for the June 2008 flood, the largest event on record. Over a thirteen day period, the watershed received an average of over 8 inches of rainfall, with some areas in the central portion of the watershed receiving as much as 11 inches. Before the storms began, the soils in the watershed were already unusually wet, which diminishes the landscape's ability to soak up additional rain water. Compared to the simulation for the existing (baseline) conditions, the tall-grass prairie landscape increased infiltration by 1.7 inches (a 109% increase), and peak discharge reductions at river locations ranged from 19% (Beaver Creek outlet) to 25% (Janesville). With cover crop agricultural management, infiltration increased by 0.4 inches (a 23% increase) and peak discharge reductions ranged from 3 to 5%. For improved soil quality scenarios, infiltration increased by as much as 0.8 inches (a 49% increase) and peak discharge reductions ranged from 9 to 12%. For scenarios involving flood storage ponds, the peak discharge reductions for small ponds ranged from 0.8% (Beaver Creek outlet) to 6% (Austin), and for large ponds ranged from 3.4% (Beaver Creek outlet) to 16% (Austin). For the blended scenario with cover crops and large ponds, infiltration increased by about 0.2 inches (15% increase) and peak discharge reductions ranged

from 11% (Austin) to 4% (Janesville). Note that peak reductions tended to be lowest in the middle part of the watershed, where the largest amount of rain fell.

d. Concluding Remarks

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

Appendix A – Maps

Depth to Bedrock

Karst Features

Minnesota

Iowa

Minnesota

Iowa

USGS Stage/Discharge Gage
Little Cedar River Drainage

Depth to Bedrock

Feet
- 0 - 10
- 10-20
- 20-50
- 50-100
- >100

N

Miles
0   10   20
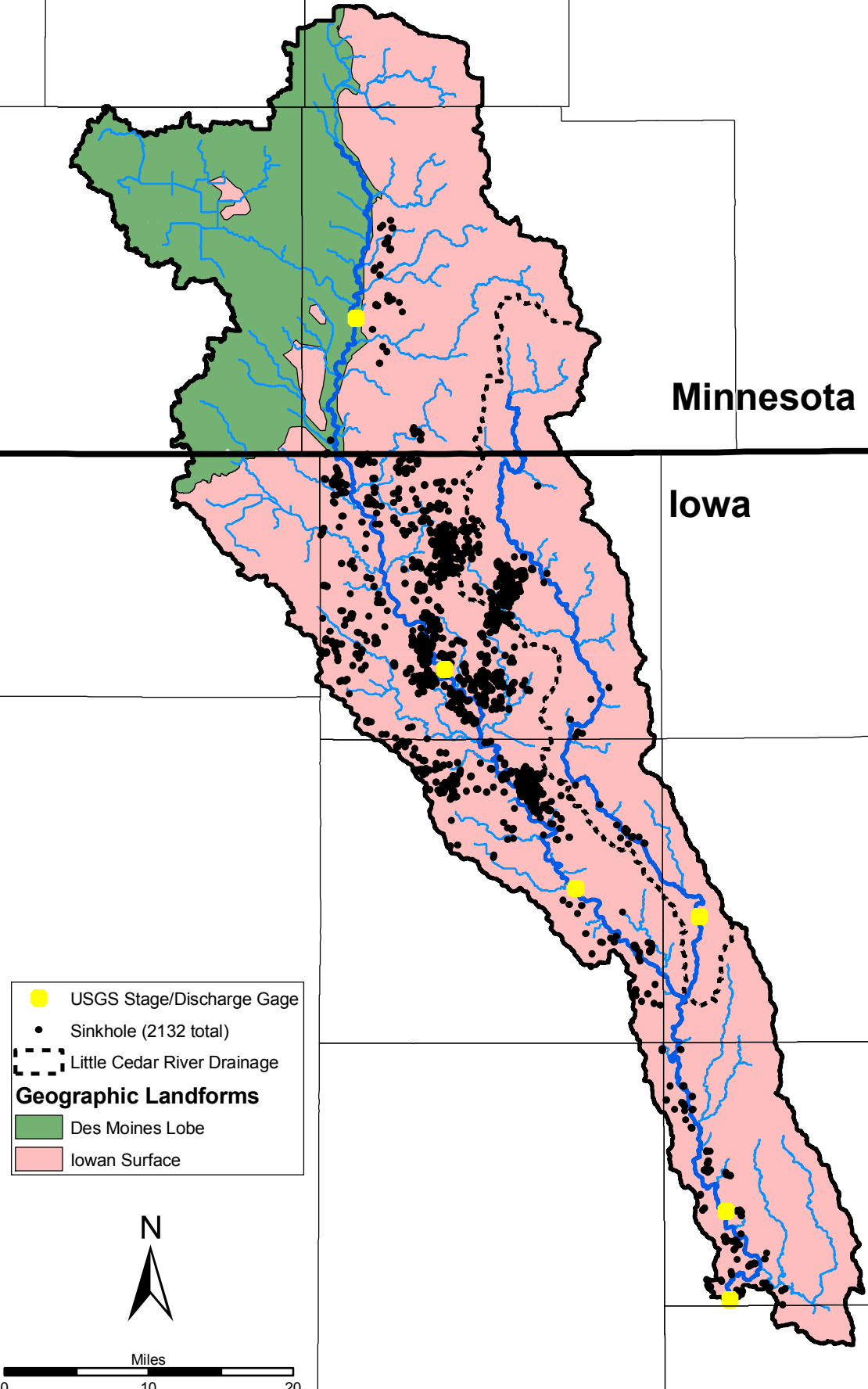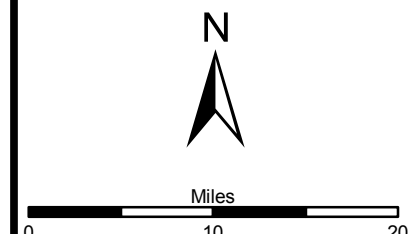
USGS Stage/Discharge Gage
Sinkhole (2132 total)
Little Cedar River Drainage

Geographic Landforms
- Des Moines Lobe
- Iowan Surface
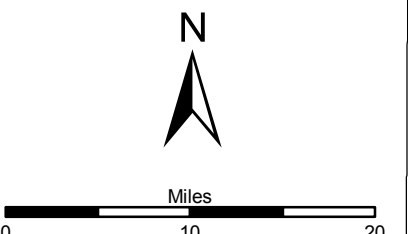
N

Miles
0   10   20

Iowa Flood Center

IIHR Hydroscience & Engineering
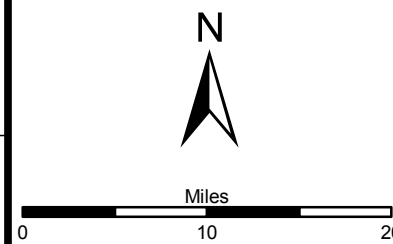
The University of Iowa
C. Maxwell Stanley Hydraulics Laboratory
Iowa City, Iowa 52246

Minnesota
Wisconsin
Iowa
Illinois

Upper Cedar River Watershed

Steele Dodge
Freeborn Mower
Worth Mitchell
Floyd Chickasaw
Butler Bremer
Black Hawk

Upper Cedar River
Watershed County
Delineation
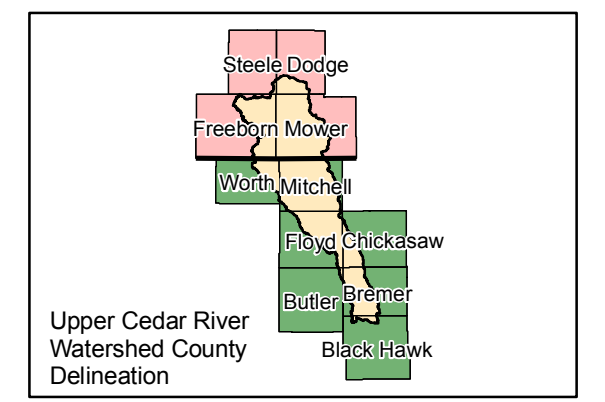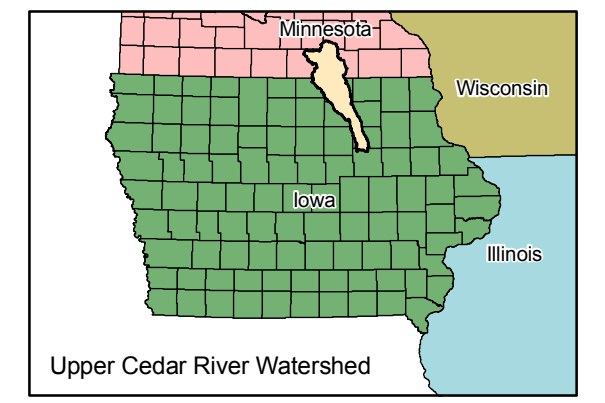
Upper Cedar River Watershed

Depth to Bedrock
Geographic Landform Regions & Sinkholes

Date: March 2014

By: Chad Drake

Data Sources:
Iowa Geological & Water Survey,
Iowa DNR (2009-2010).
Minnesota DNR & Univ. of
Minnesota Dept. of Geology &
Geophysics (2003, 2013).

Figure A-1

Land Cover

Hydrologic Soil Groups

Minnesota

Iowa

Minnesota

Iowa

N

N

Miles

0   10   20

Miles

0   10   20

USGS Stage/Discharge Gage
Little Cedar River Drainage
Iowan Surface - Des Moines Lobe Boundary
Land Cover (2006 NLCD)
Agriculture
Bare rock/sand/clay
Forest
Grassland/pasture/hay
Urban/developed
Wetlands/water

USGS Stage/Discharge Gage
Little Cedar River Drainage
Iowan Surface - Des Moines Lobe Boundary
Hydrologic Soil Group
A
A/D
B
B/D
C
C/D
D

Iowa Flood Center

IIHR
Hydroscience & Engineering

The University of Iowa
C. Maxwell Stanley Hydraulics Laboratory
Iowa City, Iowa 52246

Minnesota
Wisconsin
Iowa
Illinois

Upper Cedar River Watershed

Steele Dodge
Freeborn Mower
Worth Mitchell
Floyd Chickasaw
Butler Bremer
Black Hawk

Upper Cedar River
Watershed County
Delineation

Upper Cedar River Watershed

Land Cover
Hydrologic Soil Groups

Date: March 2014

By: Chad Drake

Data Sources:
National Land Cover Dataset,
USGS (2006). Soil Survey
(SSURGO) Geographic
Database, NRCS (2012).

Figure A-2

Elevations

Land Slopes

Minnesota

Iowa

Minnesota

Iowa

Iowa Flood Center

IIHR Hydroscience & Engineering

The University of Iowa
C. Maxwell Stanley Hydraulics Laboratory
Iowa City, Iowa 52246

Upper Cedar River Watershed

Upper Cedar River Watershed County Delineation
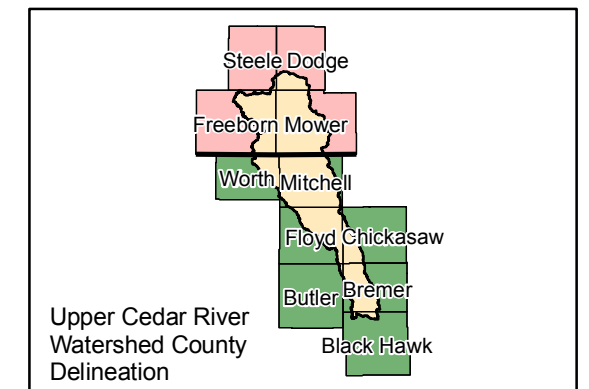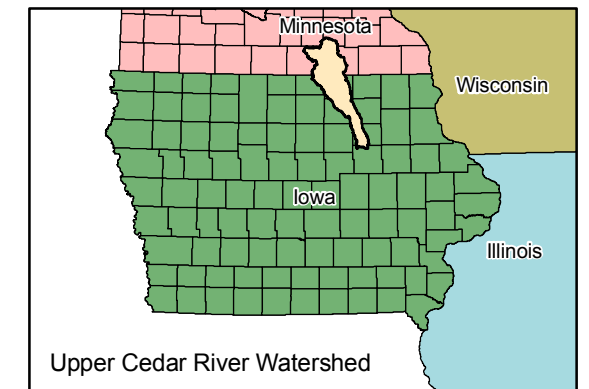
Upper Cedar River Watershed
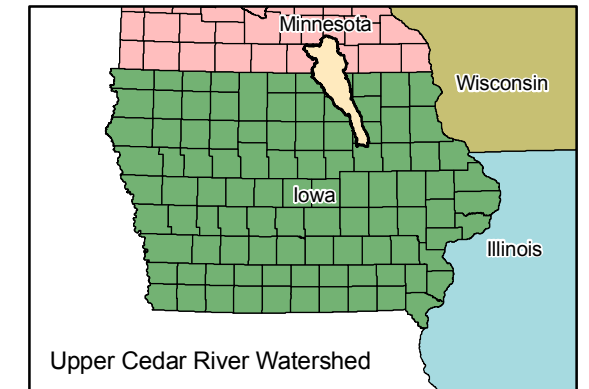
Elevations
Land Slopes

Date: March 2014

By: Chad Drake

Data Sources:
National Elevation Dataset 1/3
Arc Second (6.6 m x 6.6 m).
USGS (2009).

Figure A-3

USGS Stage/Discharge Gage
Little Cedar River Drainage
Iowan Surface - Des Moines Lobe Boundary
MSL Elevation
Value
High : 438.398
Low : 248

USGS Stage/Discharge Gage
Little Cedar River Drainage
Iowan Surface - Des Moines Lobe Boundary
Watershed Slope
Value
High : 10
Low : 0

N

N

Miles
0   10   20

Miles
0   10   20

Steele Dodge
Freeborn Mower
Worth Mitchell
Floyd Chickasaw
Butler Bremer
Black Hawk

Minnesota
Wisconsin
Iowa
Illinois

Instrumentation

Dodge Center

Albert Lea 3SE

Austin WWTP
Austin

Ortranto

Northwood

St. Ansgar

Osage

Orchard

Charles City

Ionia

Nashua

Plainfield

Tripoli

Waverly

Shell Rock 2W

Janesville

Minnesota

Iowa

N

Miles
0   10   20

Legend:
- USGS Stage/Discharge Gage
- IFC Stream Stage Sensor
- NOAA Precipitation Gage
- Little Cedar River Drainage

Subbasin Delineation

Minnesota

Iowa

USGS Stage/Discharge Gage

Subbasin Delineation (320 total)
Subbasin Curve Number
- 55-73
- 73-75
- 75-76
- 76-78
- 78-86

N

Miles
0   10   20

Iowa Flood Center

IIHR
Hydroscience & Engineering

The University of Iowa
C. Maxwell Stanley Hydraulics Laboratory
Iowa City, Iowa 52246

Minnesota
Wisconsin
Iowa
Illinois

Upper Cedar River Watershed

Steele Dodge
Freeborn Mower
Worth Mitchell
Floyd Chickasaw
Butler Bremer
Black Hawk

Upper Cedar River
Watershed County
Delineation

Upper Cedar River Watershed

Hydrologic/Meteorologic Instrumentation
Subbasin Delineation for HMS

Date: March 2014

By: Chad Drake

Data Sources:

Figure A-4
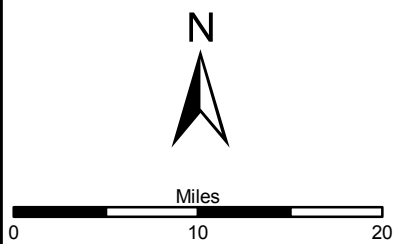
Subbasins (320)

HUC 12s (47)
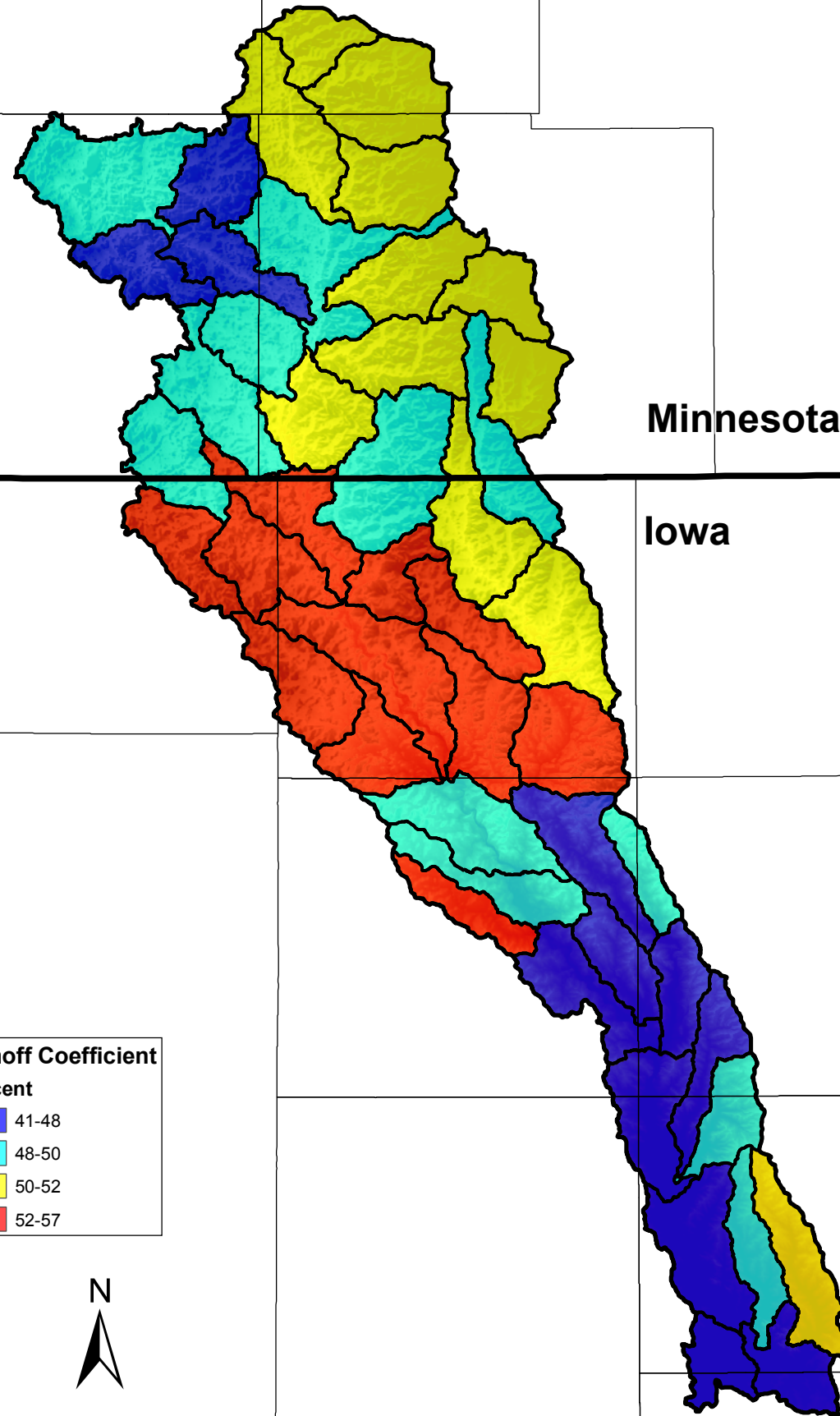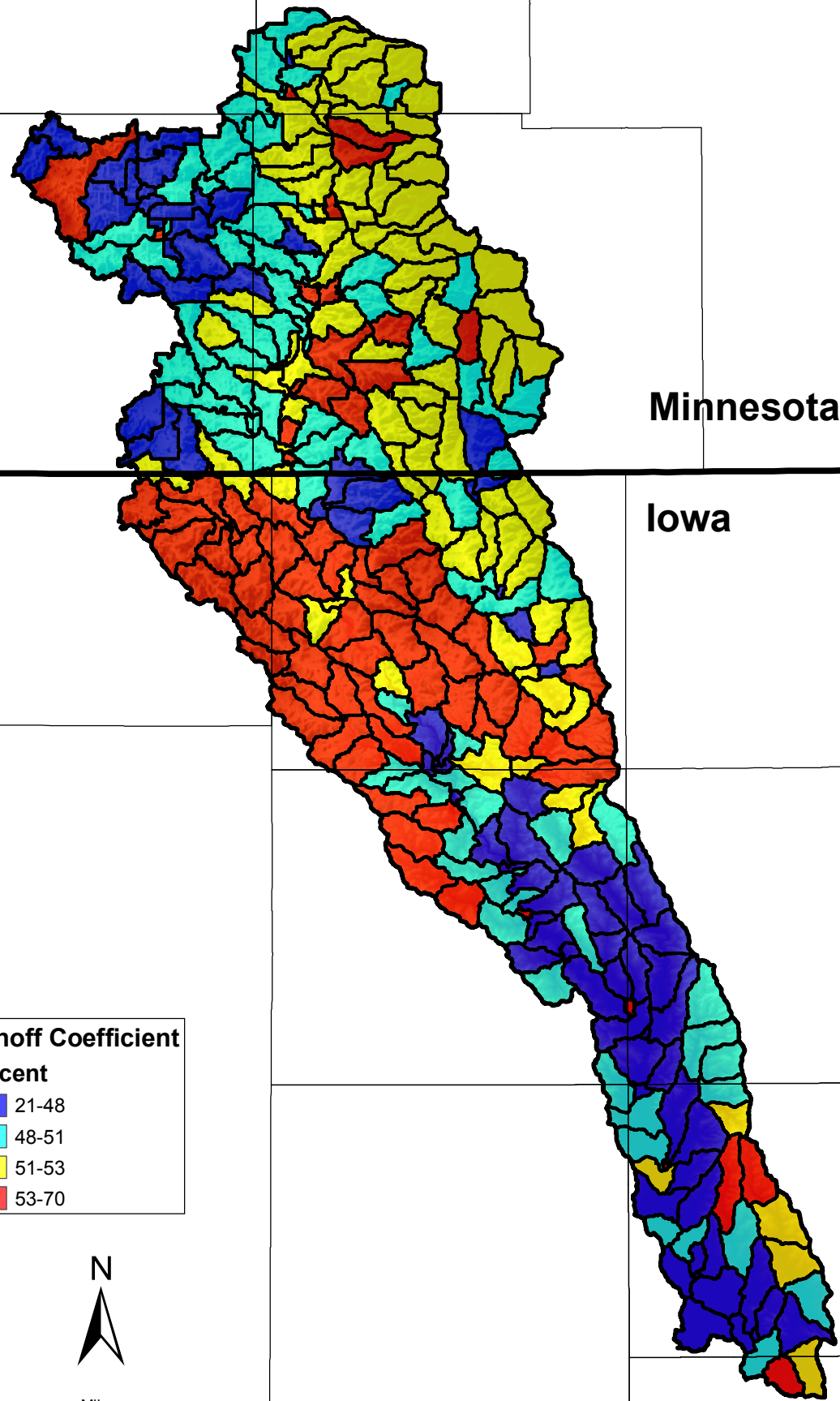
Minnesota

Iowa

Runoff Coefficient Percent
- 21-48
- 48-51
- 51-53
- 53-70

N

Miles
0   10   20

Minnesota

Iowa

Runoff Coefficient Percent
- 41-48
- 48-50
- 50-52
- 52-57

N

Miles
0   10   20

Iowa Flood Center

IIHR Hydroscience & Engineering

**The University of Iowa
C. Maxwell Stanley Hydraulics Laboratory
Iowa City, Iowa 52246**

Minnesota
Wisconsin
Iowa
Illinois

Upper Cedar River Watershed

Steele Dodge
Freeborn Mower
Worth Mitchell
Floyd Chickasaw
Butler Bremer
Black Hawk

Upper Cedar River Watershed County Delineation
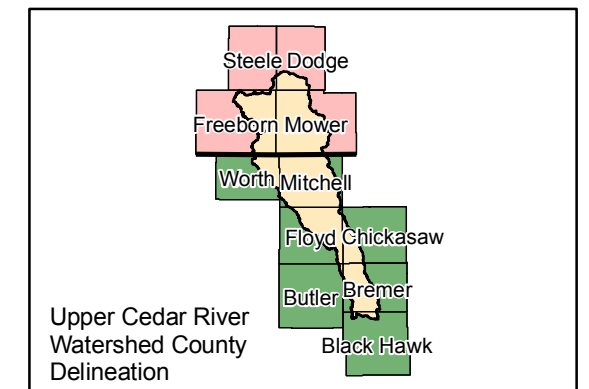
Upper Cedar River Watershed

Areas of High Runoff Potential
Subbasin and HUC 12 Scales

Notes:
1. Runoff coefficients (ratio of excess precipitation to total precipitation) computed for 25 year - 24 hour design storm (5.05 inches of rain in 24 hours)

2. Because of similar land use throughout the basin, runoff coefficients are all fairly similar despite changes in color scale. Care should be taken in selecting project locations based on this figure alone.

Date: March 2014

By: Chad Drake

Data Sources:

Figure A-5

Land Use Composition

Minnesota

Iowa

Land Use
- All Other Land Uses
- Agriculture

N

Miles
0   10   20

Hydrologic Impact

A (39%)

Minnesota

Iowa

O (39%)

BC (36%)

CC (38%)

I (36%)

W (37%)

J (37%)
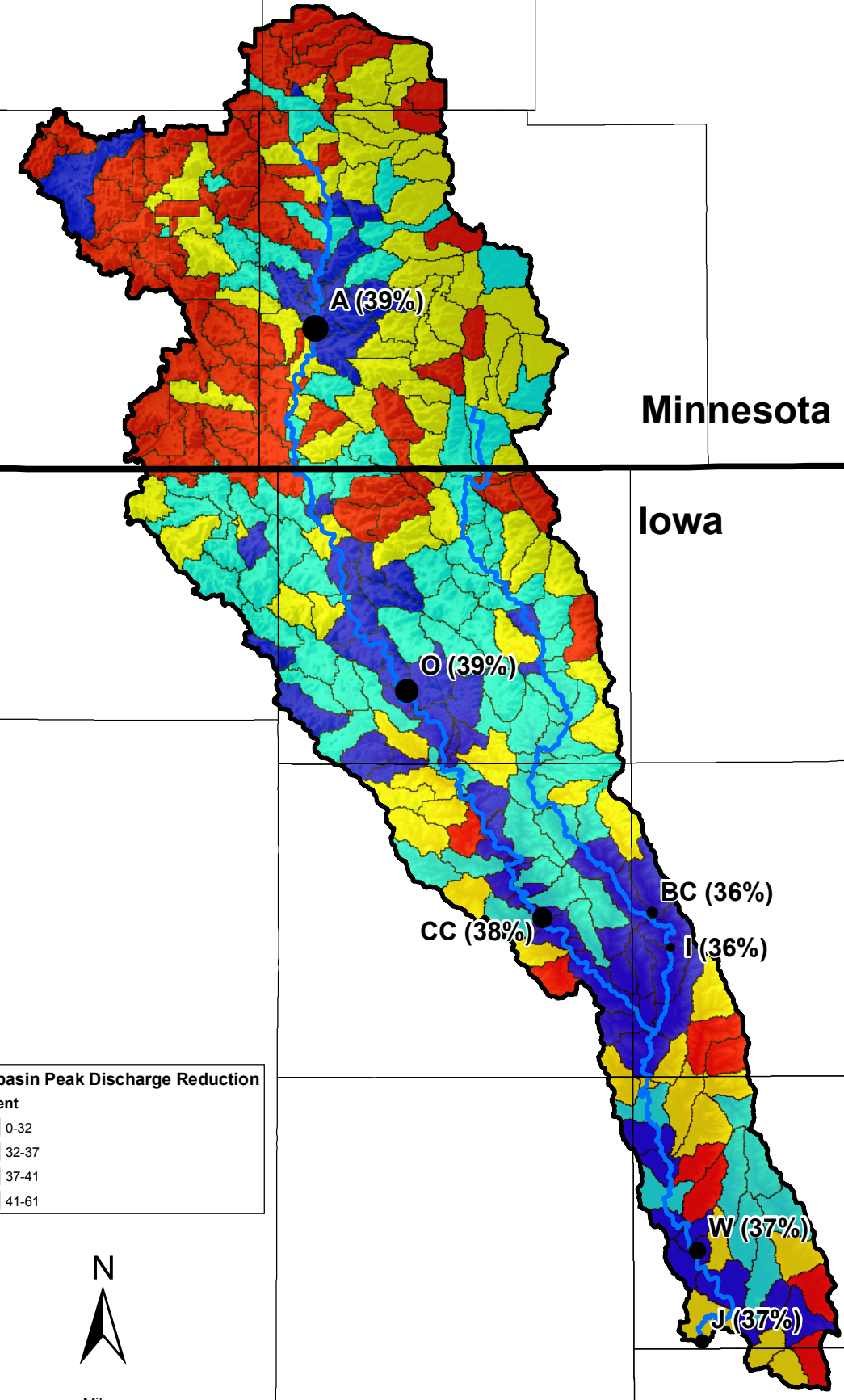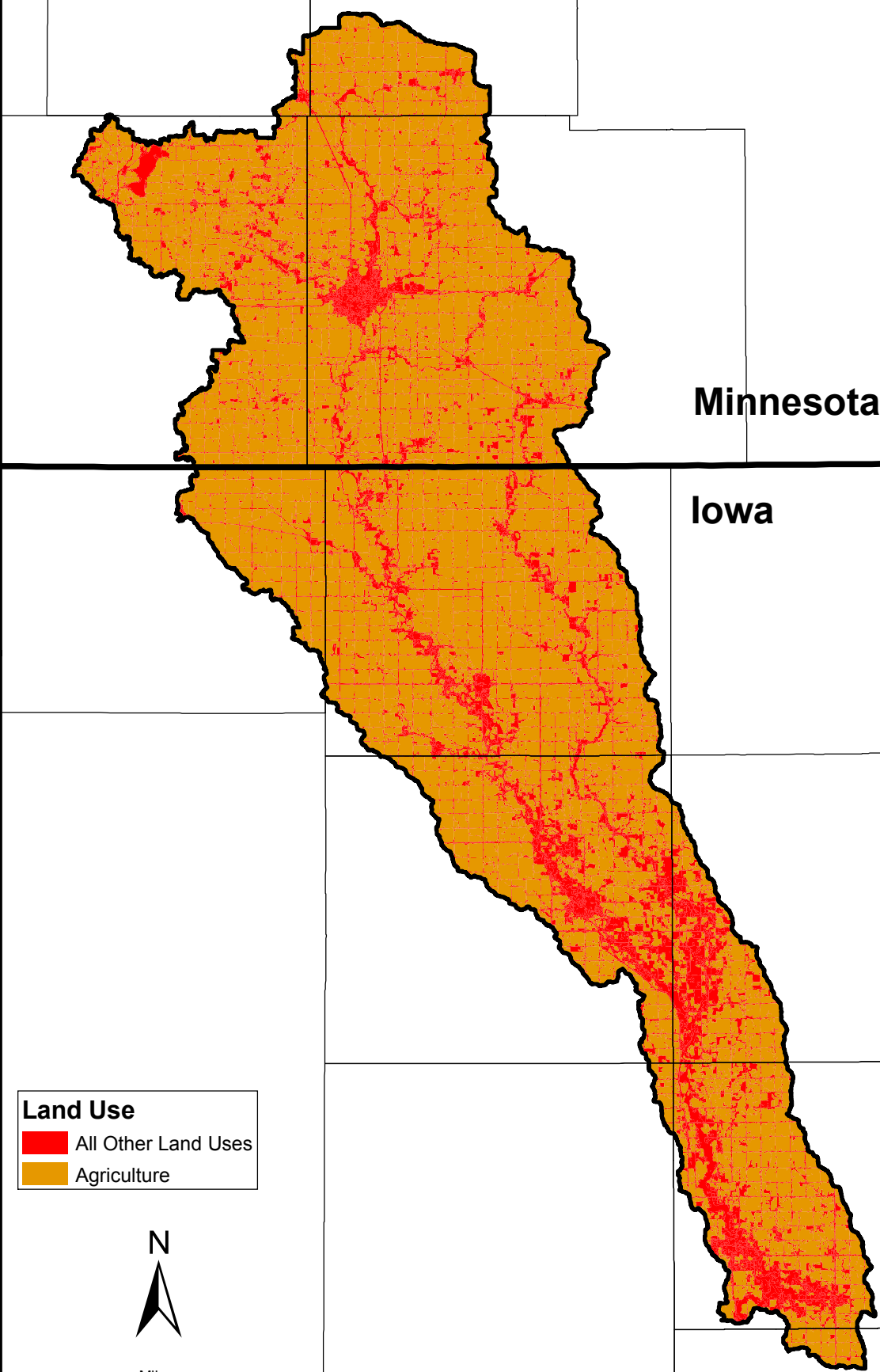
Subbasin Peak Discharge Reduction Percent
- 0-32
- 32-37
- 37-41
- 41-61
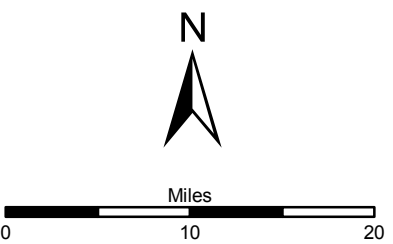
N

Miles
0   10   20

Iowa Flood Center

IIHR Hydroscience & Engineering

The University of Iowa
C. Maxwell Stanley Hydraulics Laboratory
Iowa City, Iowa 52246

Minnesota
Wisconsin
Iowa
Illinois

Upper Cedar River Watershed

Steele Dodge
Freeborn Mower
Worth Mitchell
Floyd Chickasaw
Butler Bremer
Black Hawk

Upper Cedar River
Watershed County
Delineation
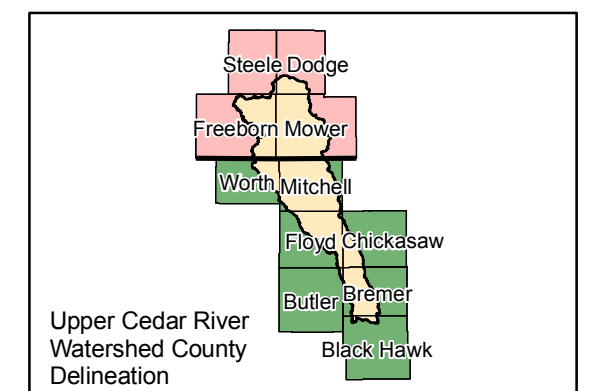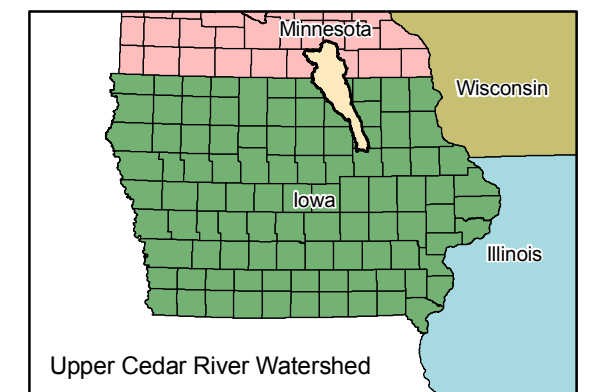
Upper Cedar River Watershed

Hypothetical Increase in Infiltration
Conversion of Row Crop Agriculture to Native Prairie
50 Year - 24 Hour Design Storm (5.89")

Index Locations:
A - Austin
O - Osage
CC - Charles City
BC - Beaver Creek Outlet
I - Ionia
W - Waverly
J - Janesville

Date: March 2014

By: Chad Drake

Data Sources:
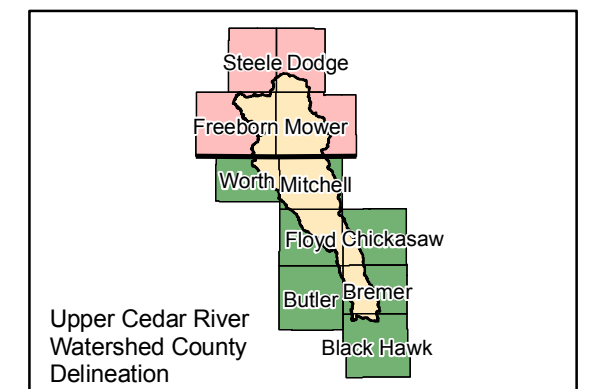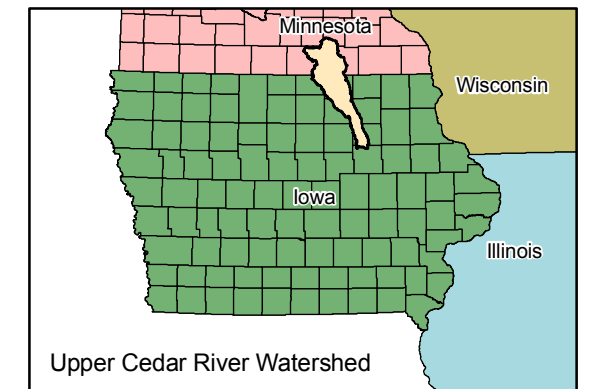National Land Cover Dataset,
USGS (2006).

Figure A-6

10-Year, 24-Hour Storm (4.05 inches)

25-Year, 24-Hour Storm (5.05 inches)

100-Year, 24-Hour Storm (6.81 inches)

Iowa Flood Center

IIHR
Hydroscience & Engineering

The University of Iowa
C. Maxwell Stanley Hydraulics Laboratory
Iowa City, Iowa 52246

Upper Cedar River Watershed

Upper Cedar River Watershed County Delineation
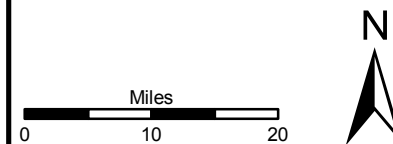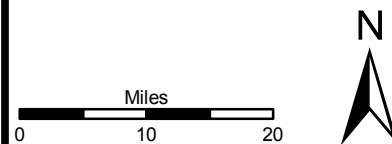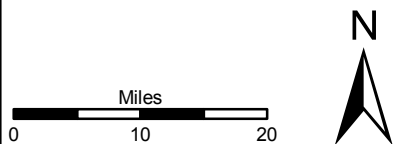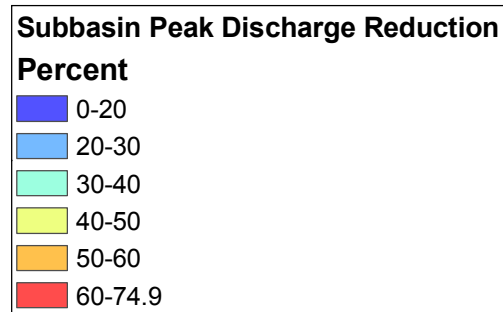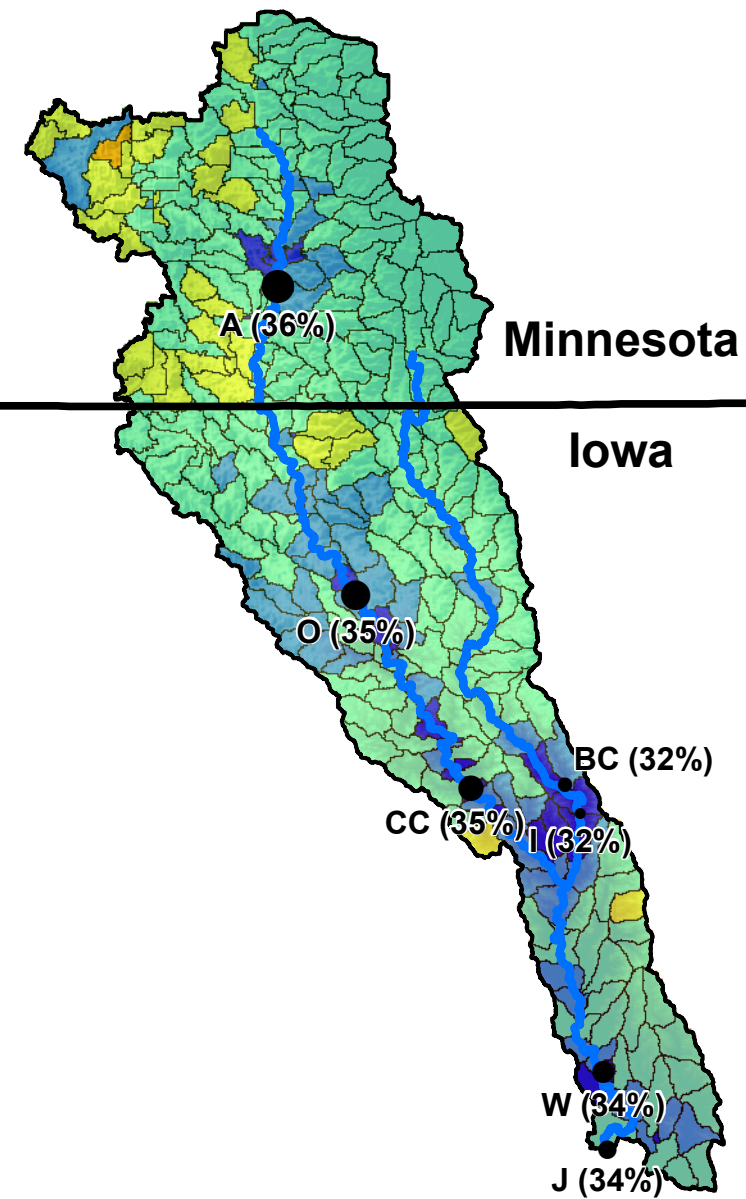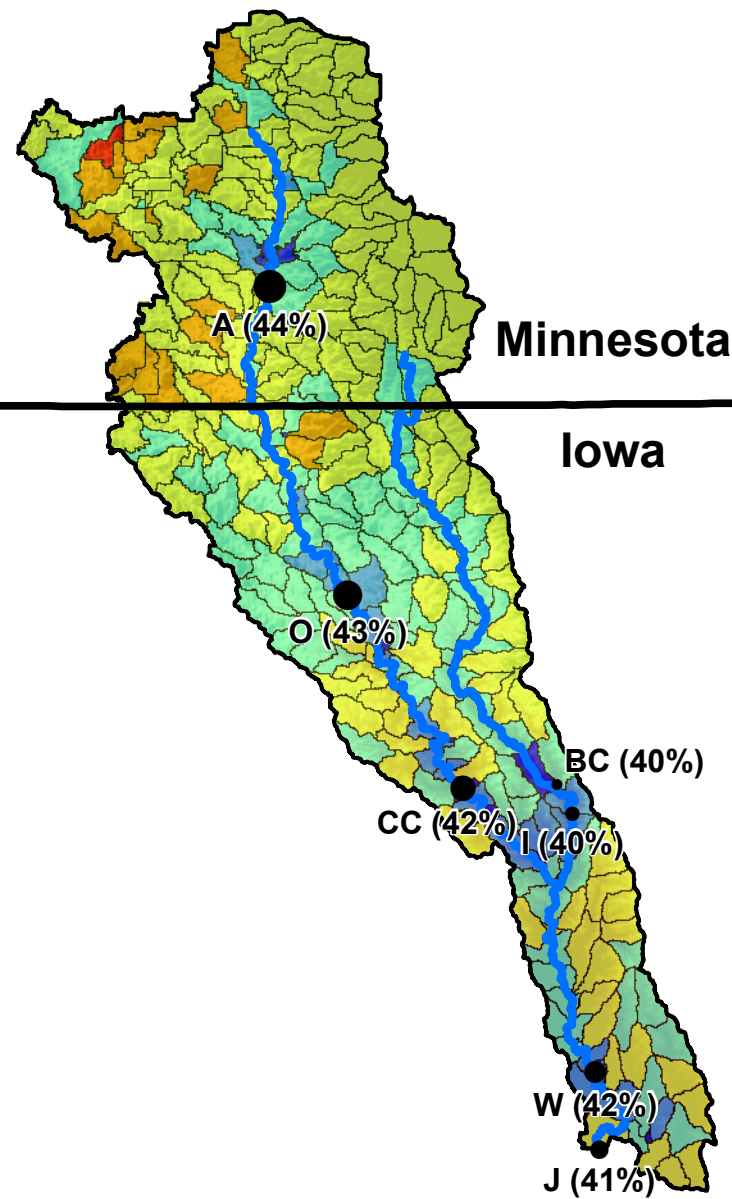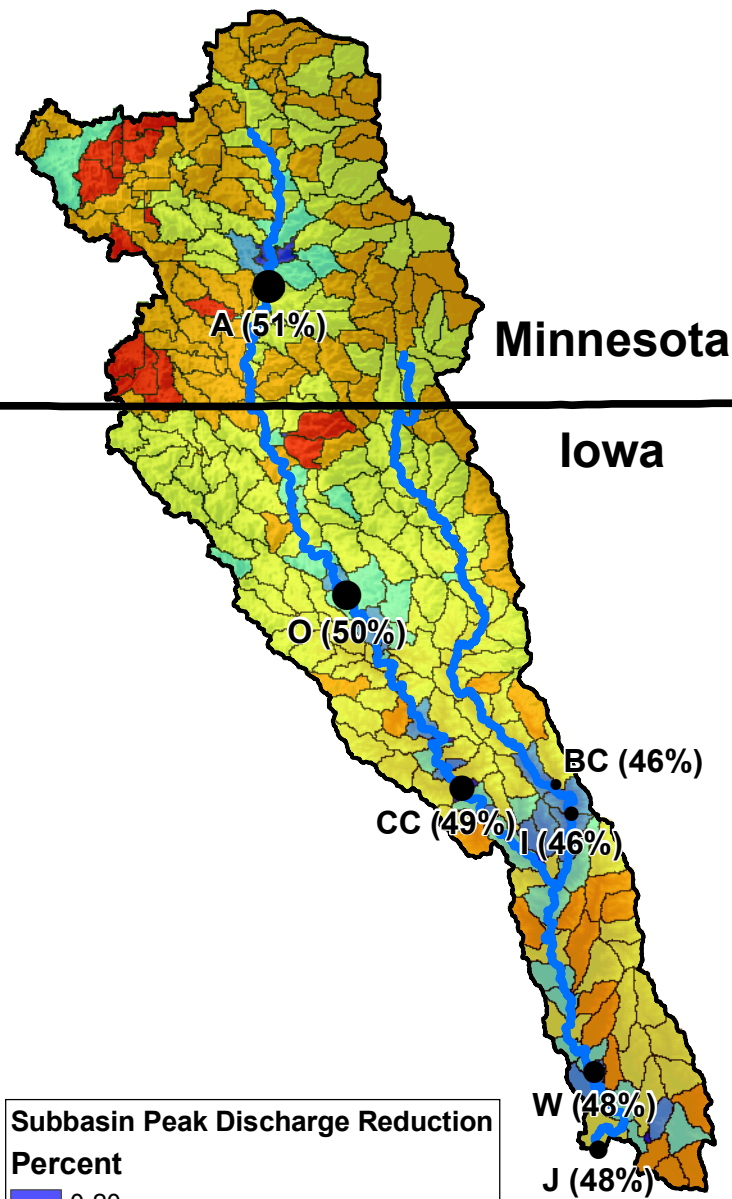
Upper Cedar River Watershed

Hypothetical Increase in Infiltration
Conversion of Row Crop Agriculture to Native Prairie
10-, 25-, and 100-Year, 24-Hour Design Storms

Subbasin Peak Discharge Reduction Percent
- 0-20
- 20-30
- 30-40
- 40-50
- 50-60
- 60-74.9

10-Year map labels: A (51%), O (50%), BC (46%), CC (49%), I (46%), W (48%), J (48%), Minnesota, Iowa

25-Year map labels: A (44%), O (43%), BC (40%), CC (42%), I (40%), W (42%), J (41%), Minnesota, Iowa

100-Year map labels: A (36%), O (35%), BC (32%), CC (35%), I (32%), W (34%), J (34%), Minnesota, Iowa

Index Locations:
- A - Austin
- O - Osage
- CC - Charles City
- BC - Beaver Creek Outlet
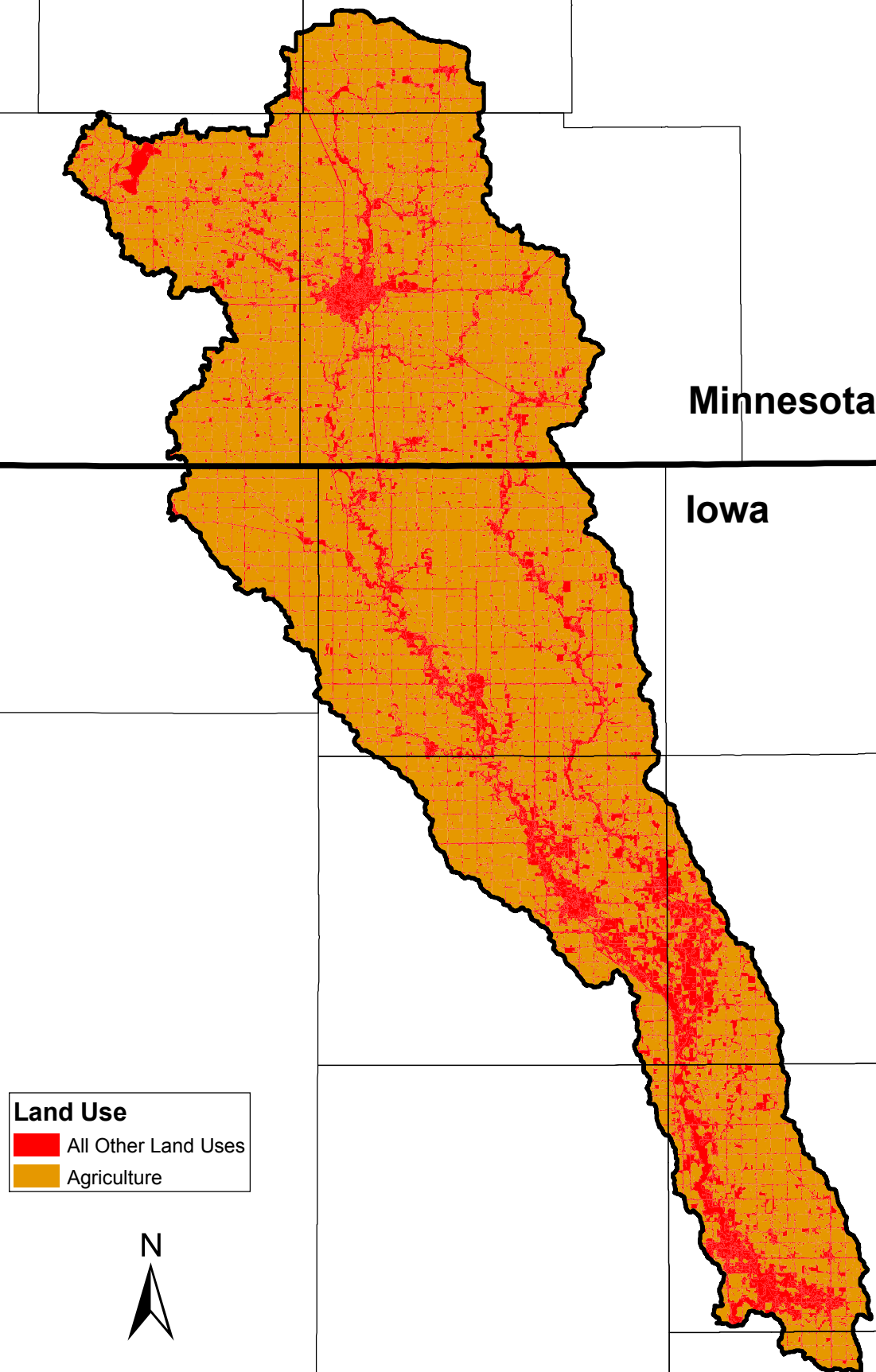- I - Ionia
- W - Waverly
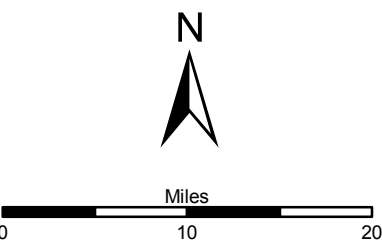- J - Janesville

Date: April 2014
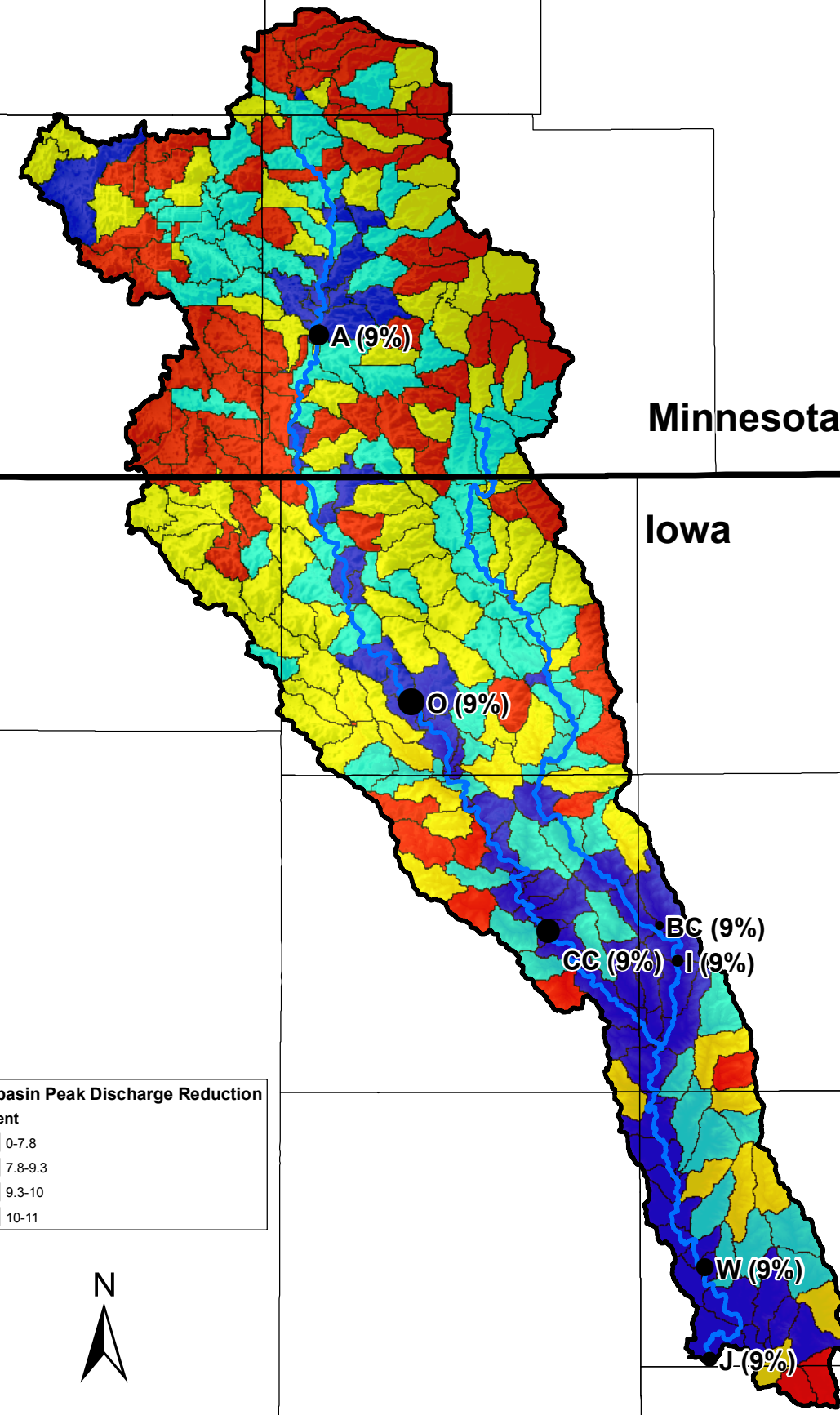
By: Chad Drake

Data Sources:

Figure A-7

Land Use Composition

Land Use
- All Other Land Uses
- Agriculture

Hydrologic Impact

Minnesota

Iowa

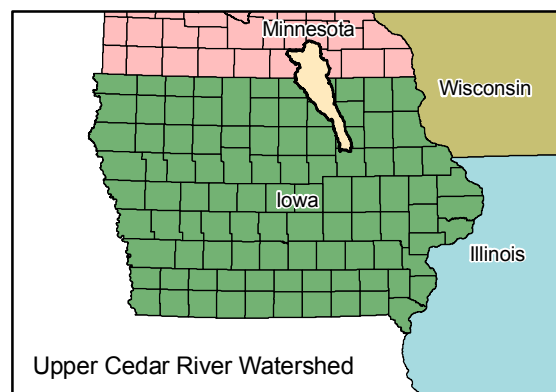**Subbasin Peak Discharge Reduction Percent**
- 0-7.8
- 7.8-9.3
- 9.3-10
- 10-11
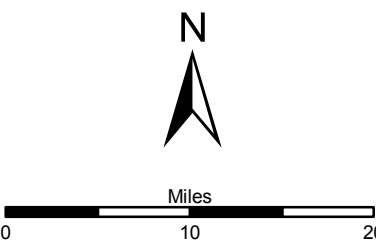
A (9%)

O (9%)

BC (9%)
CC (9%) I (9%)

W (9%)

J (9%)

Miles
0   10   20

Iowa Flood Center

IIHR Hydroscience & Engineering

The University of Iowa
C. Maxwell Stanley Hydraulics Laboratory
Iowa City, Iowa 52246

Upper Cedar River Watershed

Minnesota
Wisconsin
Iowa
Illinois

Steele Dodge
Freeborn Mower
Worth Mitchell
Floyd Chickasaw
Butler Bremer
Black Hawk

Upper Cedar River Watershed County Delineation
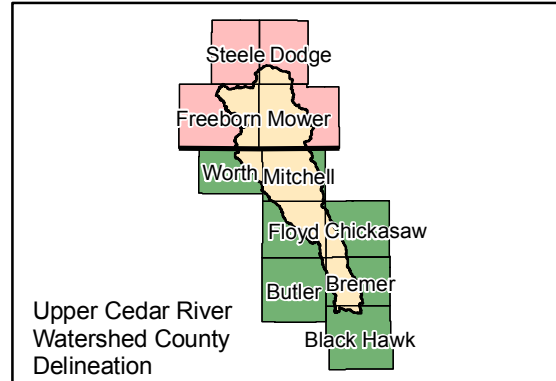
Upper Cedar River Watershed

Hypothetical Increase in Infiltration
Improved Condition from Cover Crops
50 Year - 24 Hour Design Storm (5.89")

Index Locations:
A - Austin
O - Osage
CC - Charles City
BC - Beaver Creek Outlet
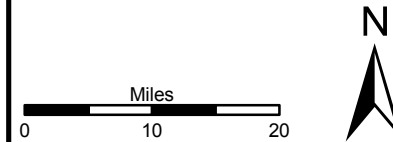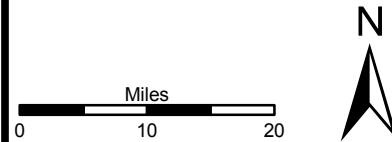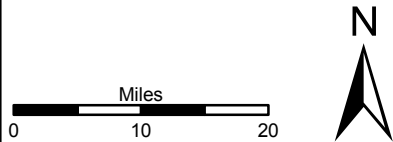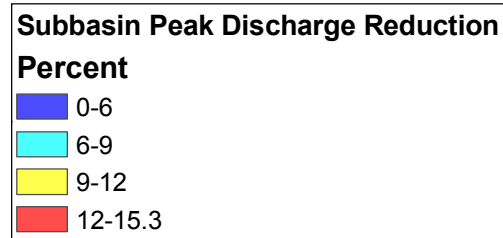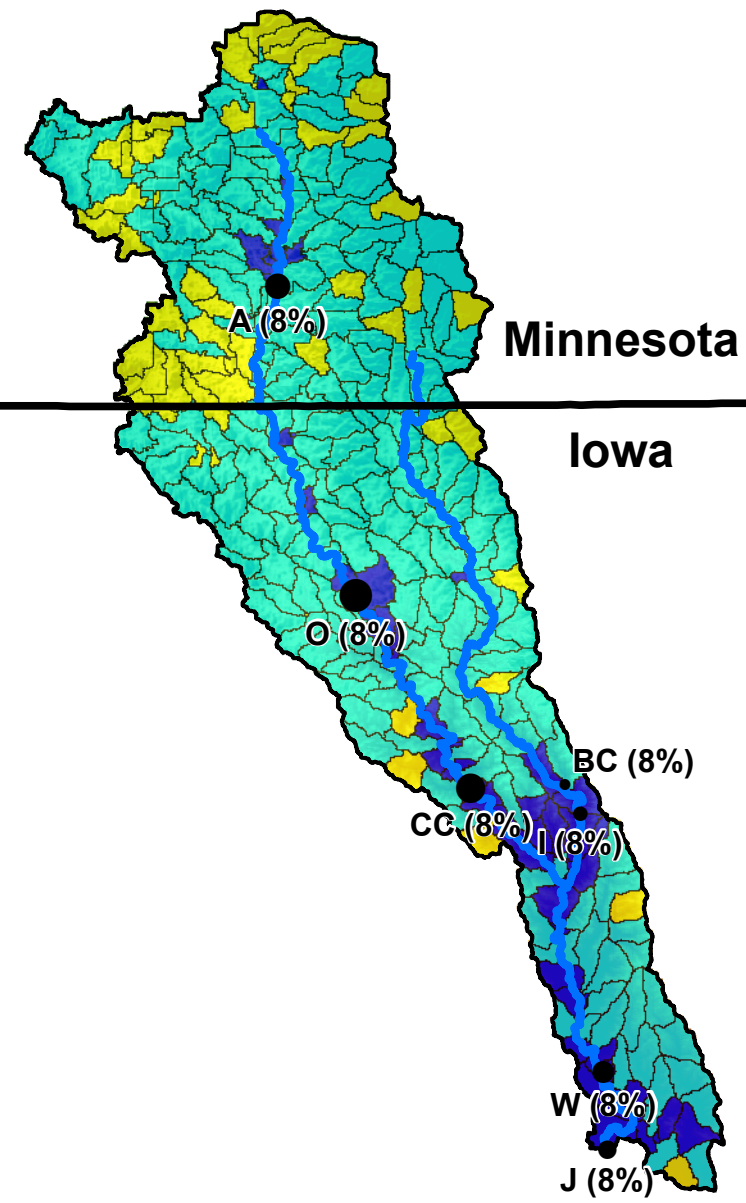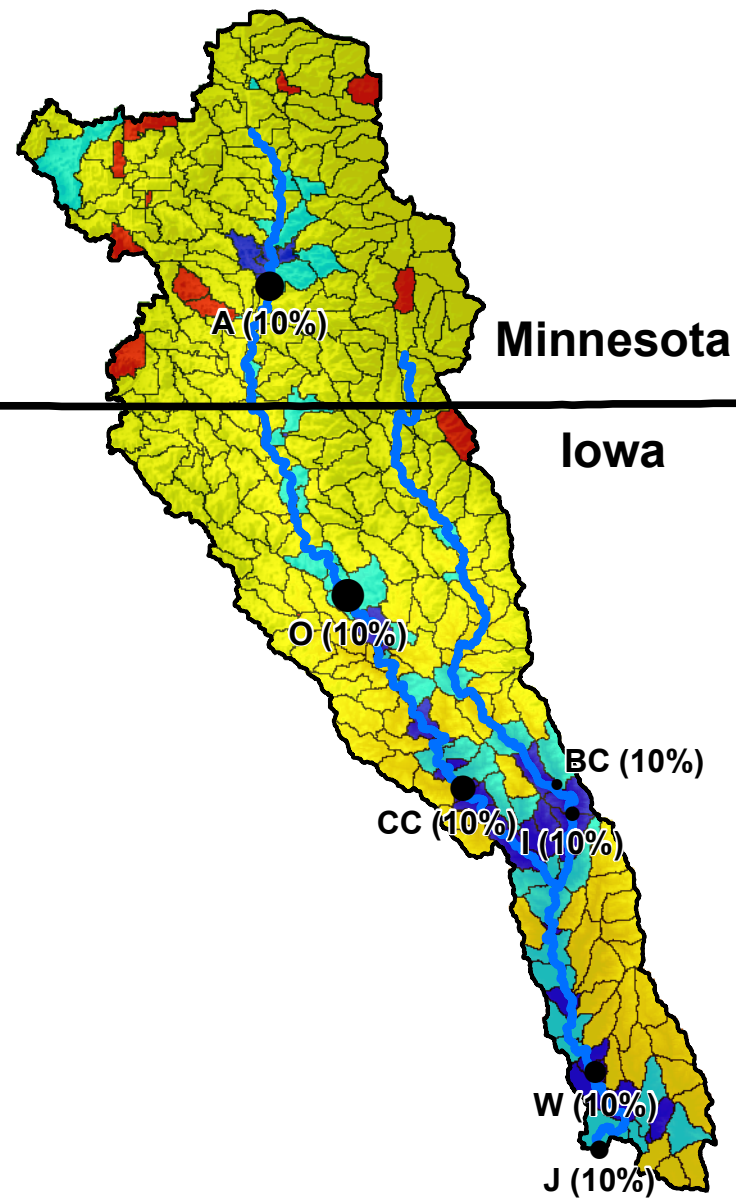I - Ionia
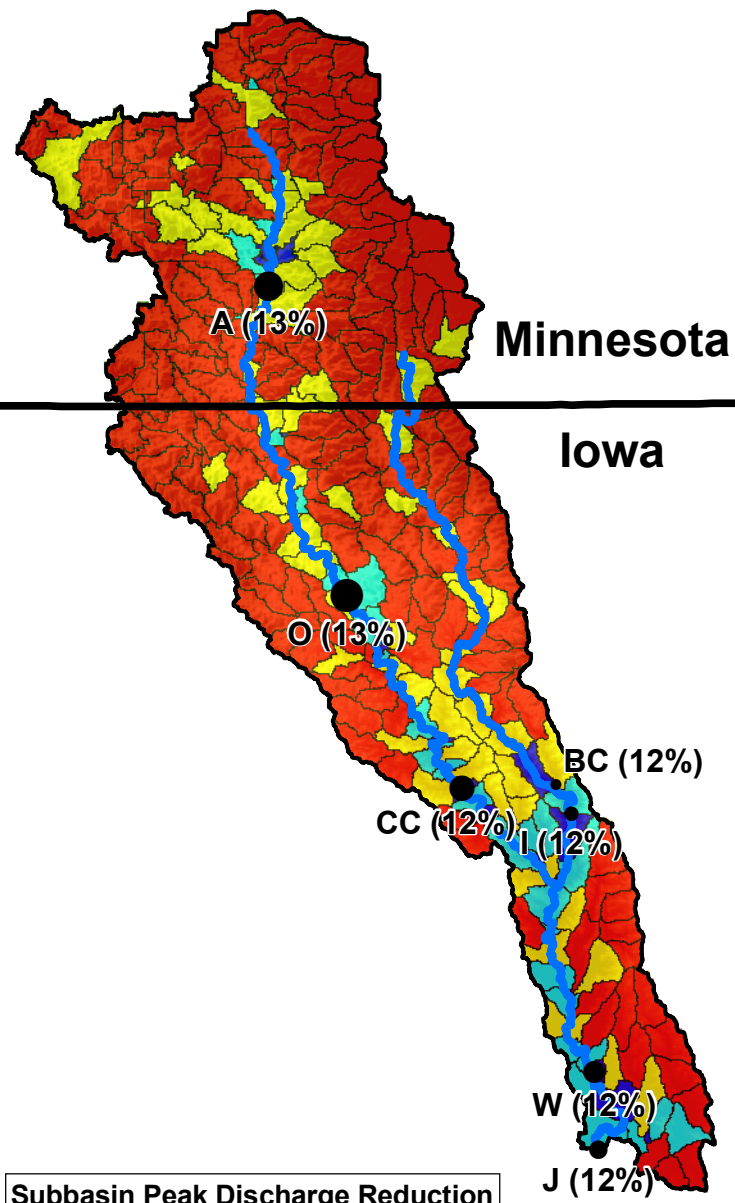W - Waverly
J - Janesville

Date: March 2014

By: Chad Drake
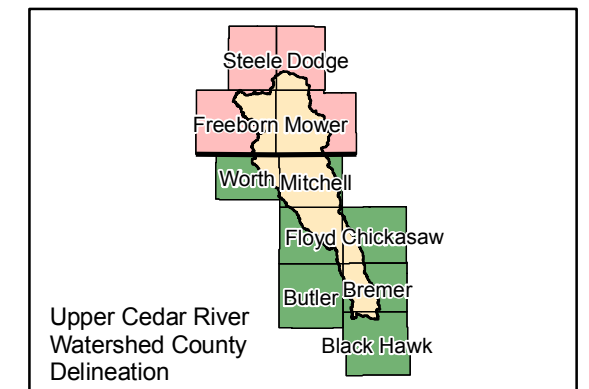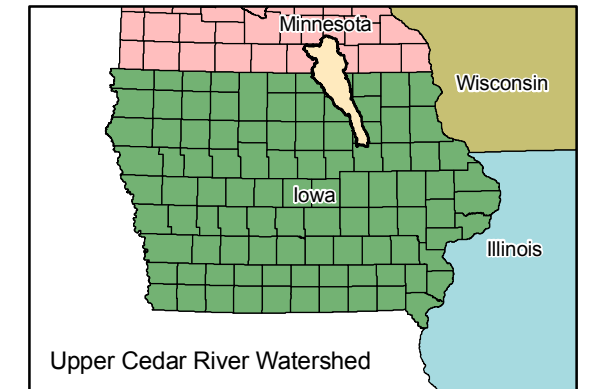
Data Sources:
National Land Cover Dataset,
USGS (2006).

Figure A-8

10-Year, 24-Hour Storm (4.05 inches)

25-Year, 24-Hour Storm (5.05 inches)

100-Year, 24-Hour Storm (6.81 inches)

Minnesota

Iowa

A (13%)
O (13%)
BC (12%)
CC (12%)
I (12%)
W (12%)
J (12%)

A (10%)
O (10%)
BC (10%)
CC (10%)
I (10%)
W (10%)
J (10%)

A (8%)
O (8%)
BC (8%)
CC (8%)
I (8%)
W (8%)
J (8%)

Subbasin Peak Discharge Reduction
Percent
- 0-6
- 6-9
- 9-12
- 12-15.3

N

Miles
0   10   20

Iowa
Flood
Center

IIHR
Hydroscience & Engineering

The University of Iowa
C. Maxwell Stanley Hydraulics Laboratory
Iowa City, Iowa 52246

Upper Cedar River Watershed

Minnesota
Wisconsin
Iowa
Illinois

Upper Cedar River Watershed County Delineation

Steele Dodge
Freeborn Mower
Worth Mitchell
Floyd Chickasaw
Butler Bremer
Black Hawk

Upper Cedar River Watershed

Hypothetical Increase in Infiltration
Improved Condition from Cover Crops
10-, 25-, and 100-Year, 24-Hour Design Storms

Index Locations:
 A - Austin
 O - Osage
 CC - Charles City
 BC - Beaver Creek Outlet
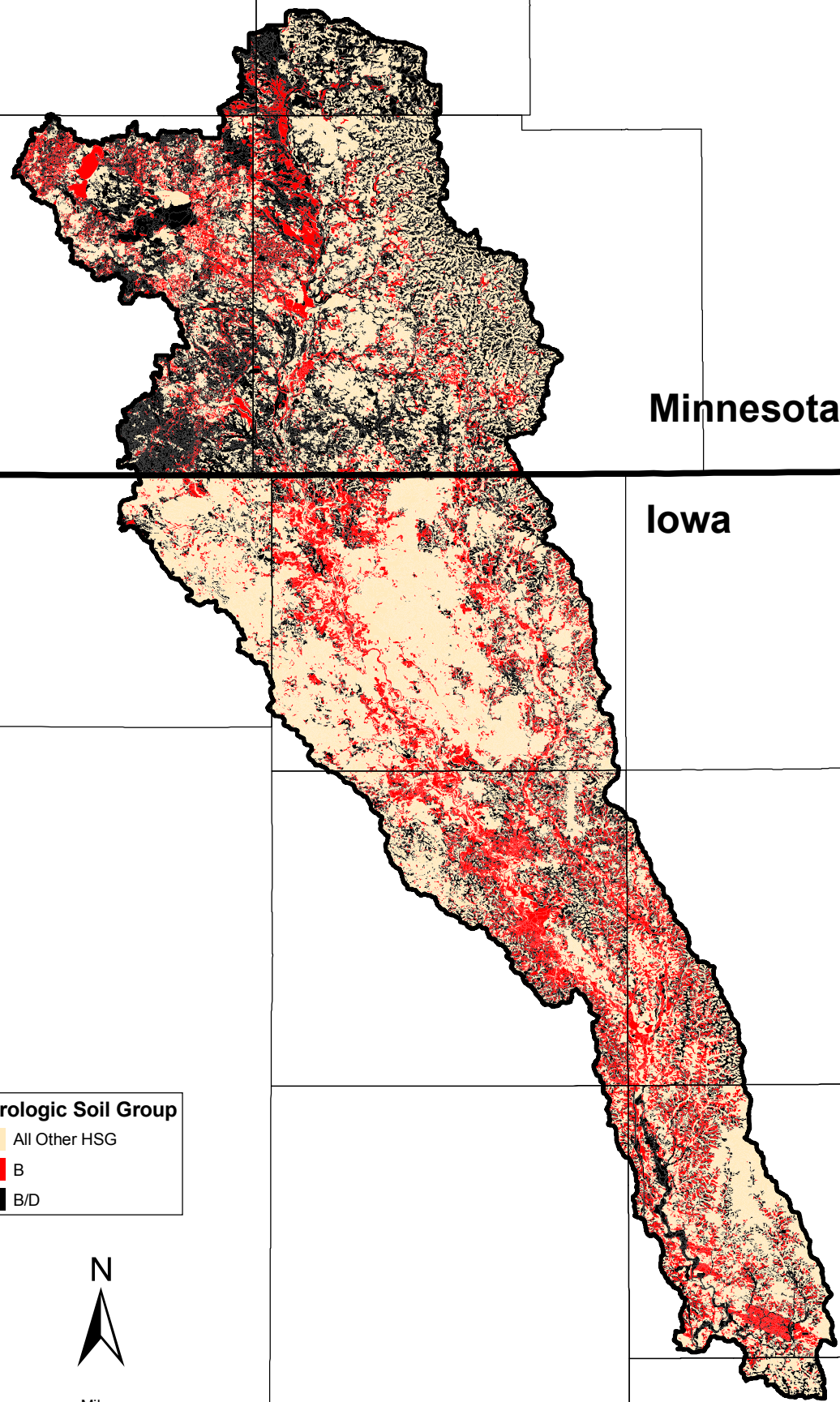 I - Ionia
 W - Waverly
 J - Janesville

Date: April 2014

By: Chad Drake

Data Sources:

Figure A-9

Soils Composition

Hydrologic Soil Group
- All Other HSG
- B
- B/D

N

Miles
0   10   20

Hydrologic Impact

Minnesota

Iowa

A (21%)

O (19%)

BC (22%)

CC (18%)

I (16%)

W (17%)

J (17%)

**Subbasin Peak Discharge Reduction
Percent**
- 0-14
- 14-19
- 19-24
- 24-52

N

Miles
0   10   20

Iowa Flood Center

IIHR Hydroscience & Engineering

**The University of Iowa
C. Maxwell Stanley Hydraulics Laboratory
Iowa City, Iowa 52246**

Minnesota
Wisconsin
Iowa
Illinois

Upper Cedar River Watershed

Steele Dodge
Freeborn Mower
Worth Mitchell
Floyd Chickasaw
Butler Bremer
Black Hawk

Upper Cedar River
Watershed County
Delineation
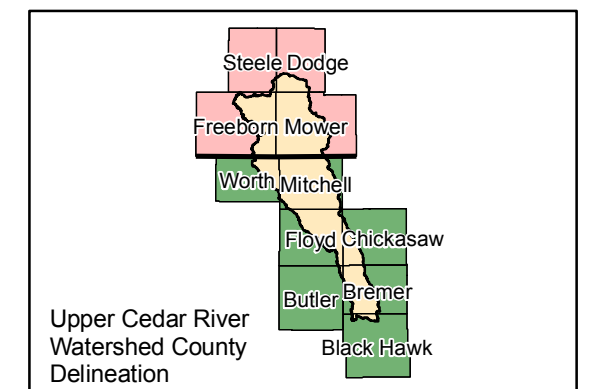
Upper Cedar River Watershed

Hypothetical Increase in Infiltration
Improved Soil Conditions (HSG B, B/D to A)
50 Year - 24 Hour Design Storm (5.89")

Index Locations:
A - Austin
O - Osage
CC - Charles City
BC - Beaver Creek Outlet
I - Ionia
W - Waverly
J - Janesville
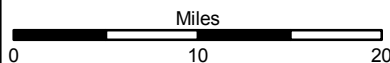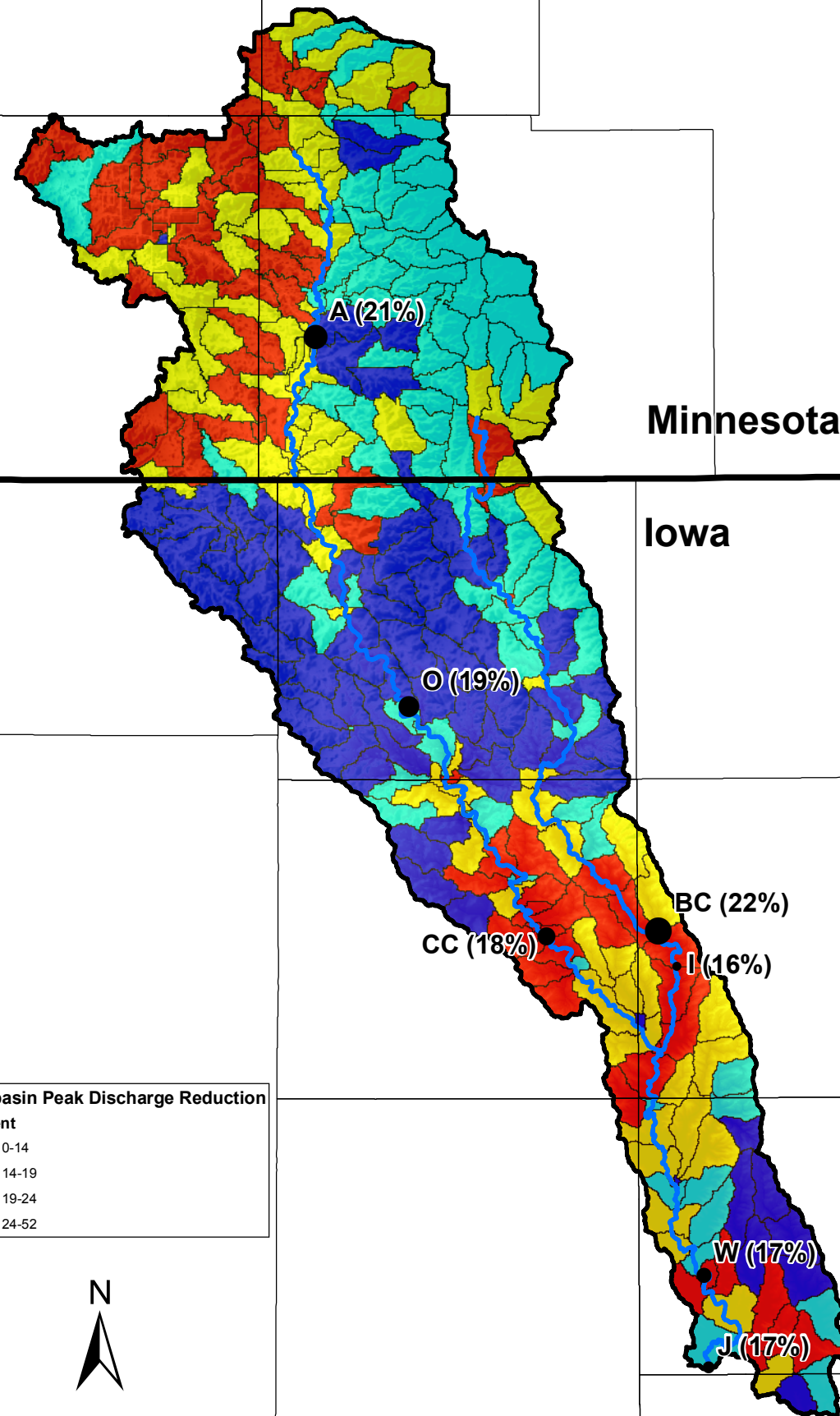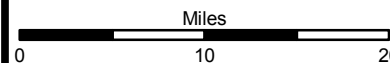
Date: March 2014

By: Chad Drake

Data Sources:
Soil Survey (SSURGO)
Geographic Database, NRCS
(2012).

Figure A-10

10-Year, 24-Hour Storm (4.05 inches)

25-Year, 24-Hour Storm (5.05 inches)

100-Year, 24-Hour Storm (6.81 inches)

Iowa Flood Center

IIHR Hydroscience & Engineering

**The University of Iowa
C. Maxwell Stanley Hydraulics Laboratory
Iowa City, Iowa 52246**

Upper Cedar River Watershed

Upper Cedar River Watershed County Delineation
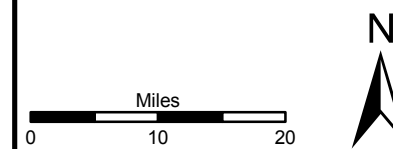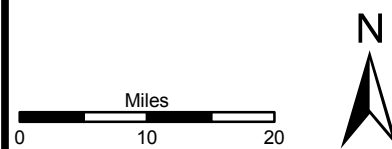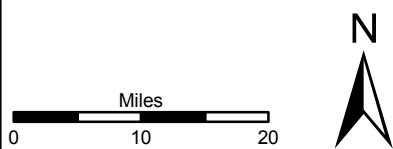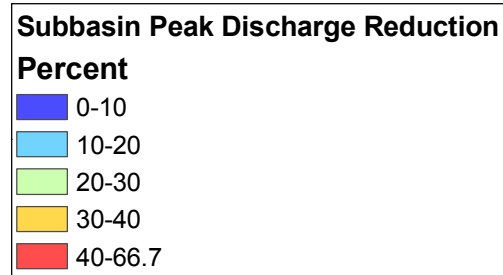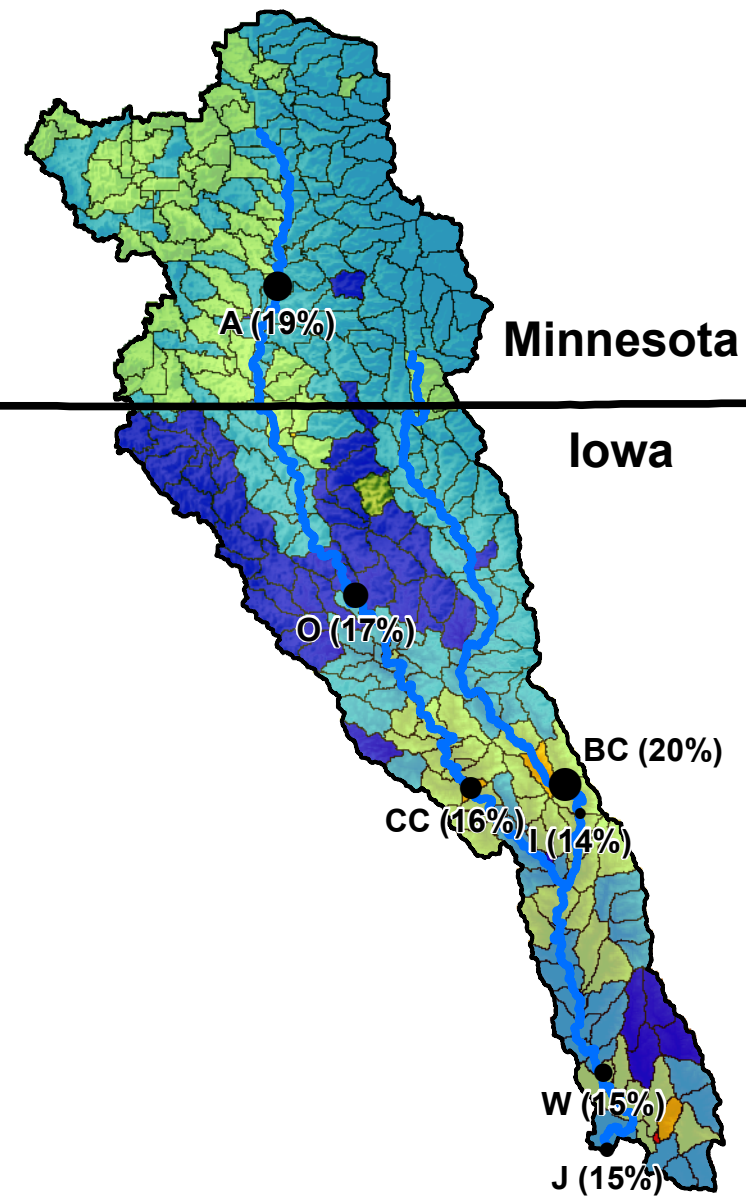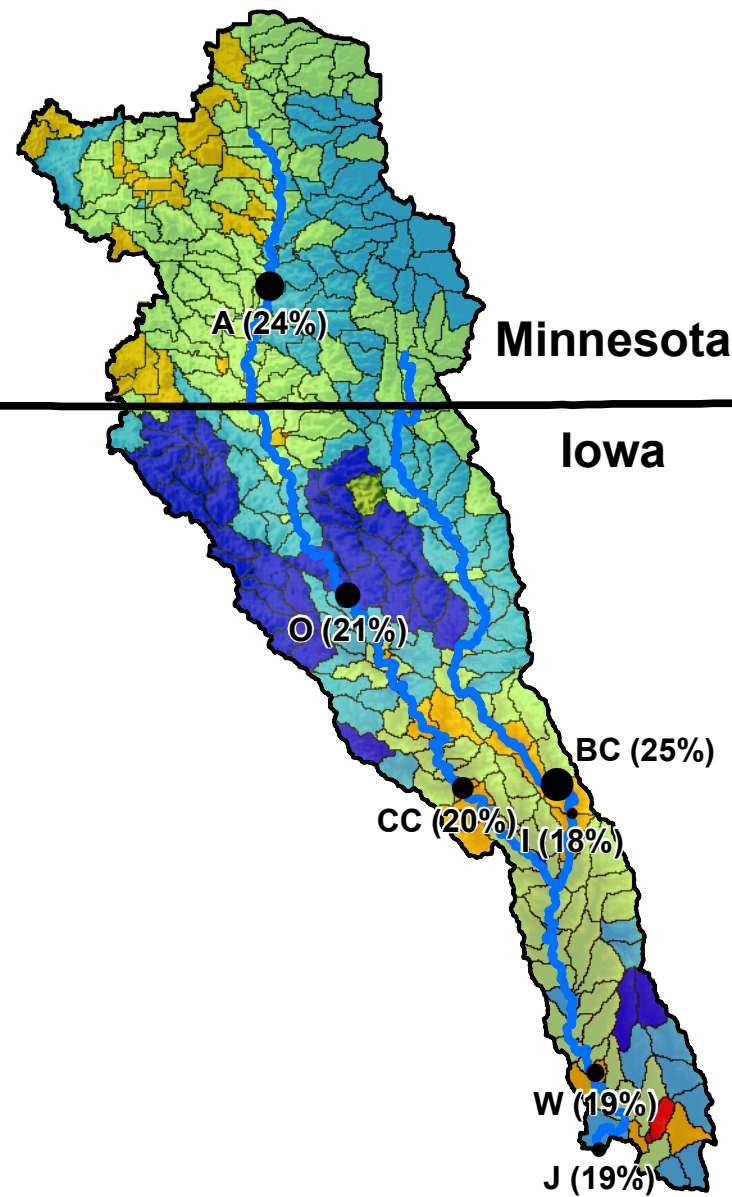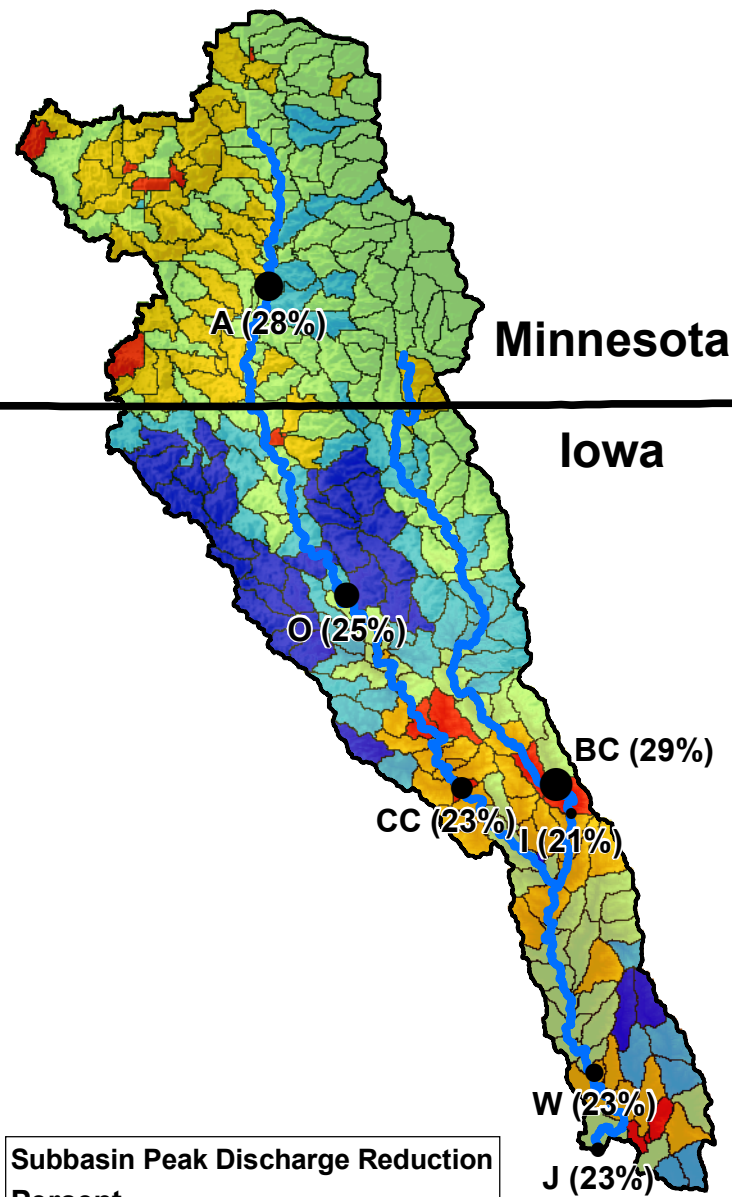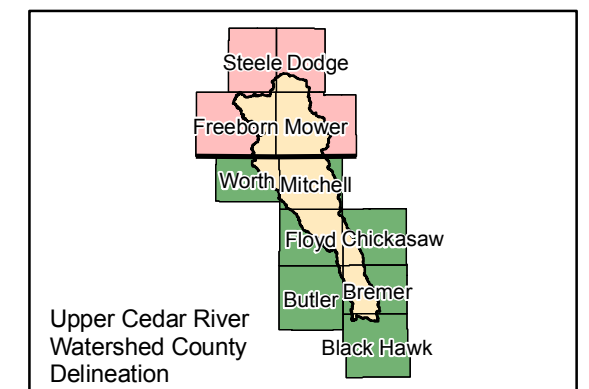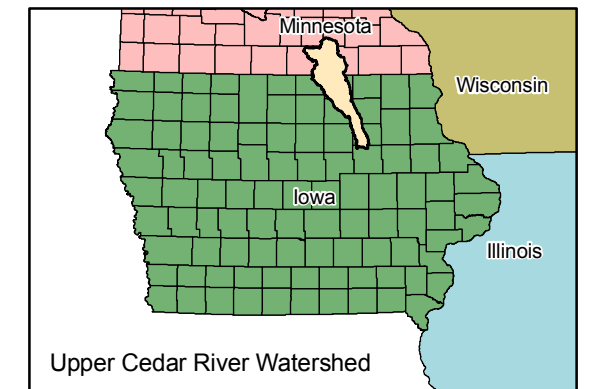
Upper Cedar River Watershed

Hypothetical Increase in Infiltration
Improved Soil Conditions (HSG B, B/D to A)
10-, 25-, and 100-Year, 24-Hour Design Storms

Index Locations:
 A - Austin
 O - Osage
 CC - Charles City
 BC - Beaver Creek Outlet
 I - Ionia
 W - Waverly
 J - Janesville

Date: April 2014

By: Chad Drake

Data Sources:

Figure A-11

Subbasin Peak Discharge Reduction Percent
- 0-10
- 10-20
- 20-30
- 30-40
- 40-66.7

10-Year map labels: A (28%), O (25%), BC (29%), CC (23%), I (21%), W (23%), J (23%), Minnesota, Iowa

25-Year map labels: A (24%), O (21%), BC (25%), CC (20%), I (18%), W (19%), J (19%), Minnesota, Iowa

100-Year map labels: A (19%), O (17%), BC (20%), CC (16%), I (14%), W (15%), J (15%), Minnesota, Iowa
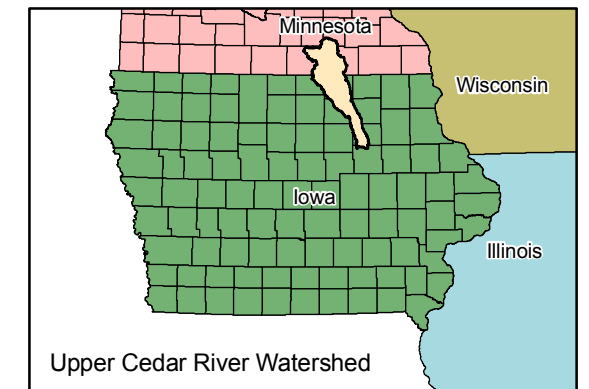
Soils Composition

Hydrologic Soil Group
- All Other HSG
- C
- C/D

Minnesota

Iowa

N

Miles
0   10   20

Hydrologic Impact

A (7%)

O (8%)

BC (10%)

CC (9%)

I (11%)

W (9%)

J (9%)

Minnesota

Iowa

Subbasin Peak Discharge Reduction Percent
- 0-6
- 6-9
- 9-12
- 12-20

N

Miles
0   10   20

Iowa Flood Center

IIHR Hydroscience & Engineering

The University of Iowa
C. Maxwell Stanley Hydraulics Laboratory
Iowa City, Iowa 52246

Minnesota | Wisconsin | Iowa | Illinois

Upper Cedar River Watershed

Steele Dodge
Freeborn Mower
Worth Mitchell
Floyd Chickasaw
Butler Bremer
Black Hawk

Upper Cedar River Watershed County Delineation

Upper Cedar River Watershed

Hypothetical Increase in Infiltration
Improved Soil Conditions (HSG C, C/D to B)
50 Year - 24 Hour Design Storm (5.89")

Index Locations:
A - Austin
O - Osage
CC - Charles City
BC - Beaver Creek Outlet
I - Ionia
W - Waverly
J - Janesville

Date: March 2014

By: Chad Drake

Data Sources:
Soil Survey (SSURGO)
Geographic Database, NRCS
(2012).

Figure A-12
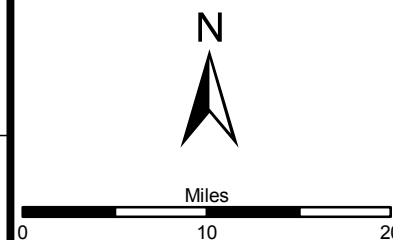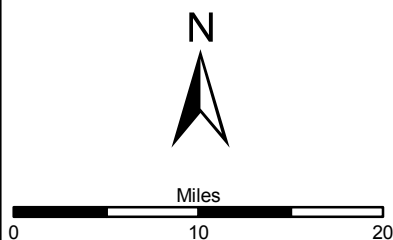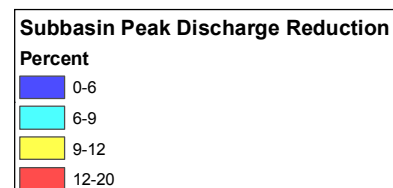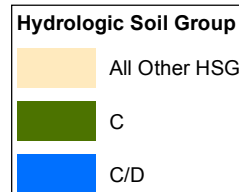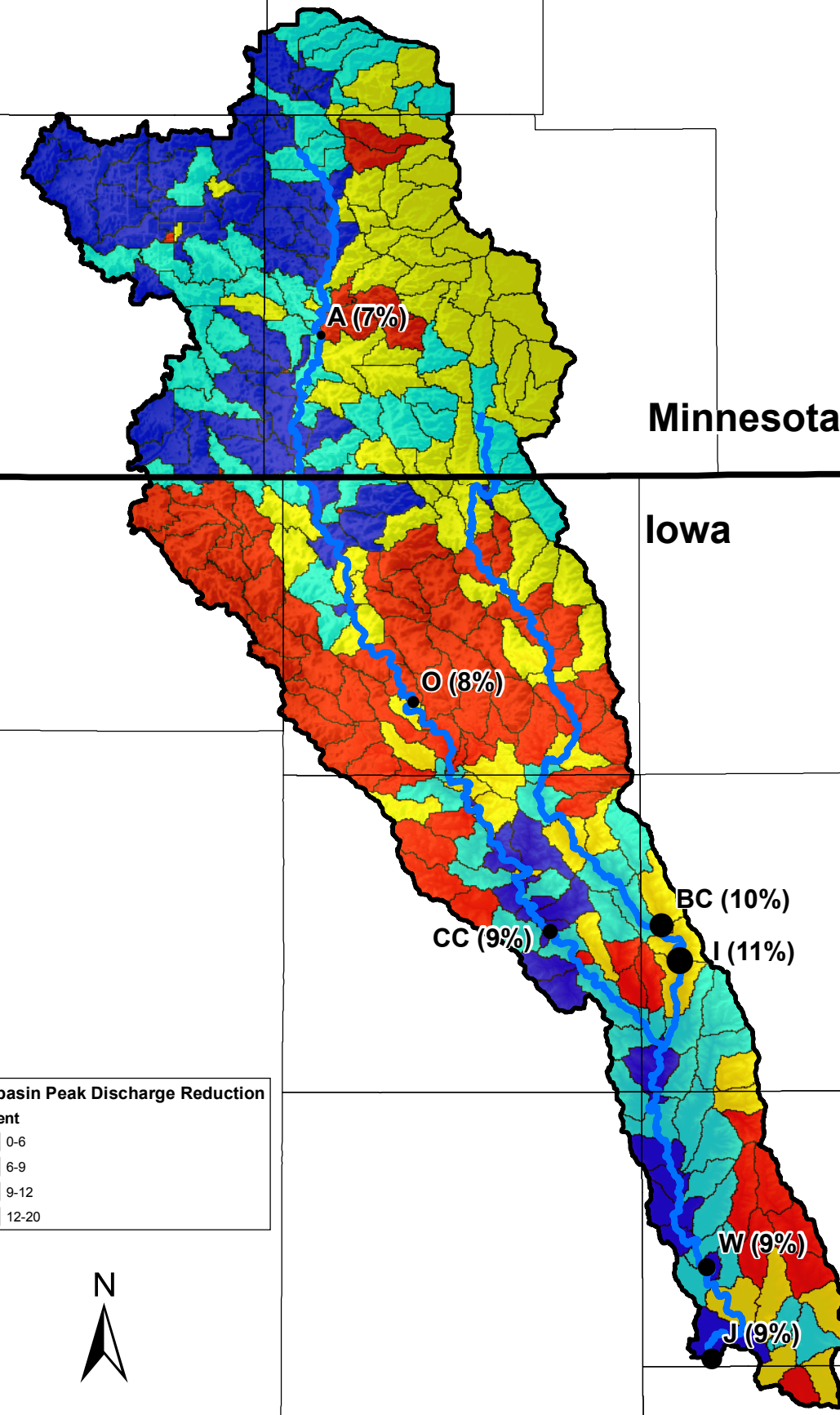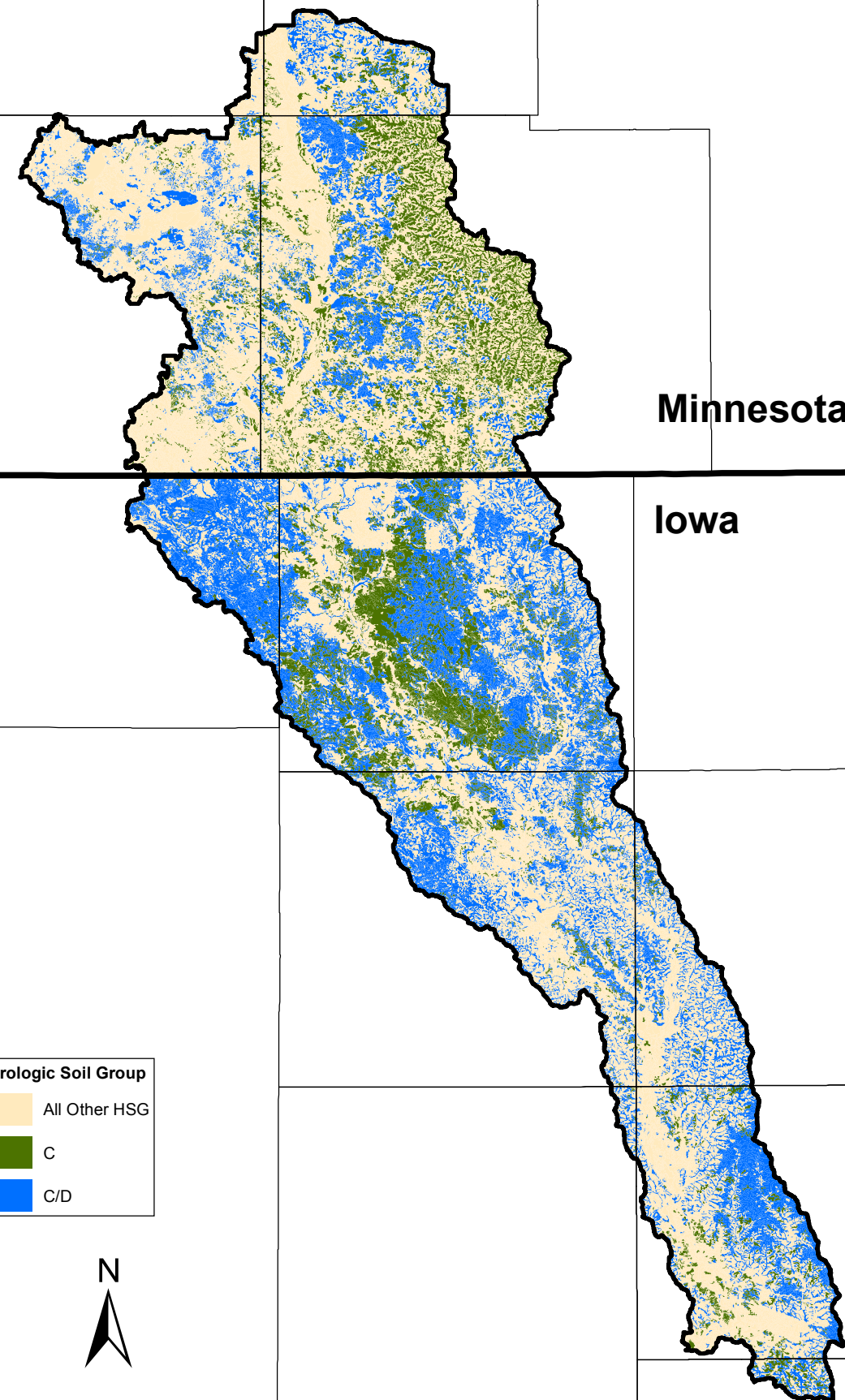
10-Year, 24-Hour Storm (4.05 inches)

25-Year, 24-Hour Storm (5.05 inches)

100-Year, 24-Hour Storm (6.81 inches)

Iowa Flood Center

IIHR
Hydroscience & Engineering

The University of Iowa
C. Maxwell Stanley Hydraulics Laboratory
Iowa City, Iowa 52246

Upper Cedar River Watershed

Upper Cedar River Watershed County Delineation
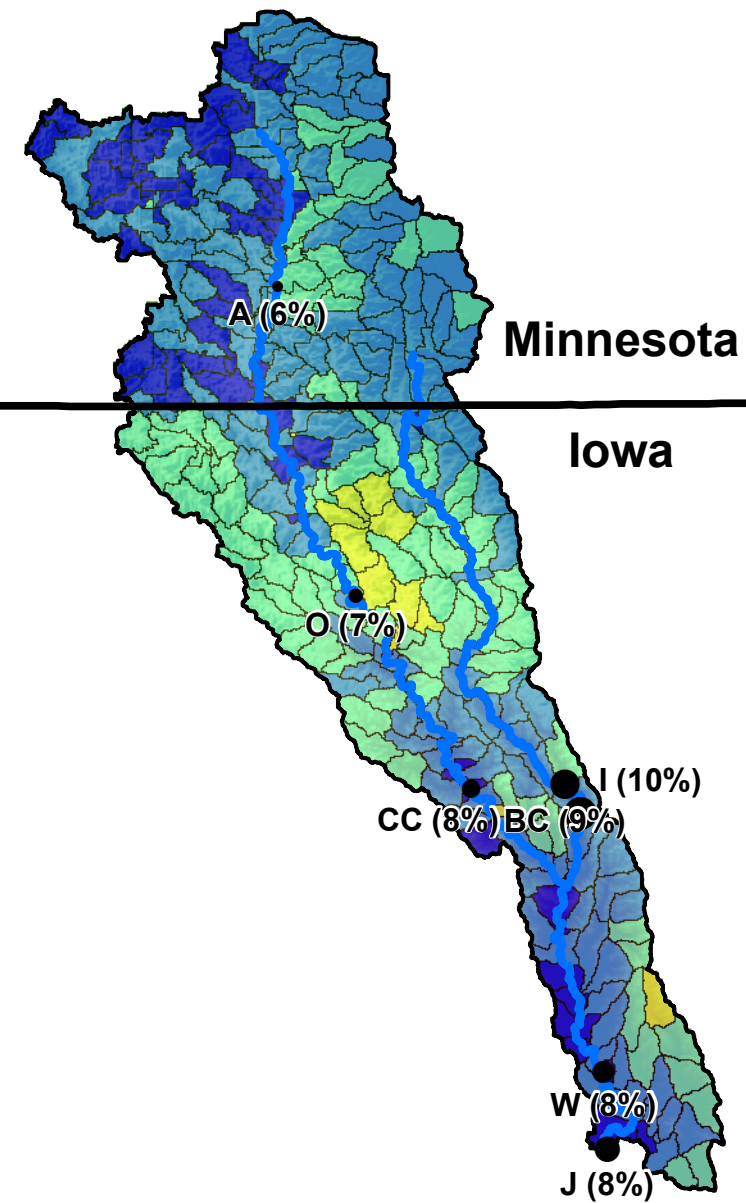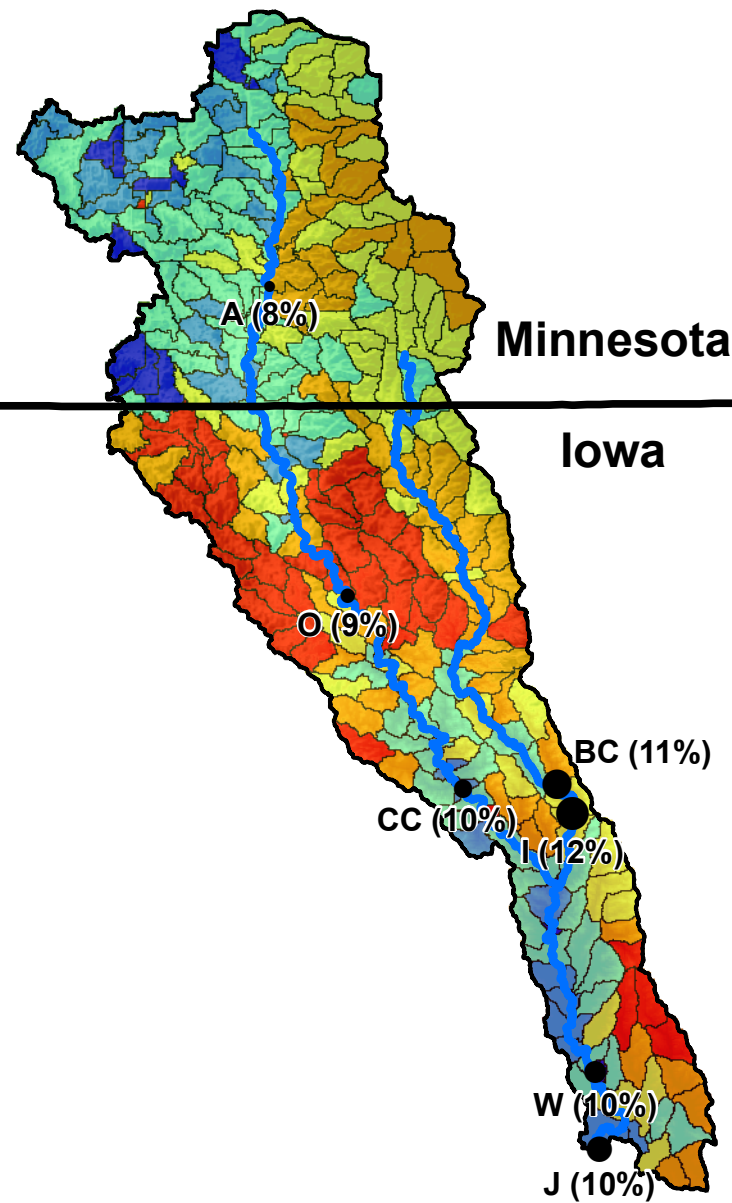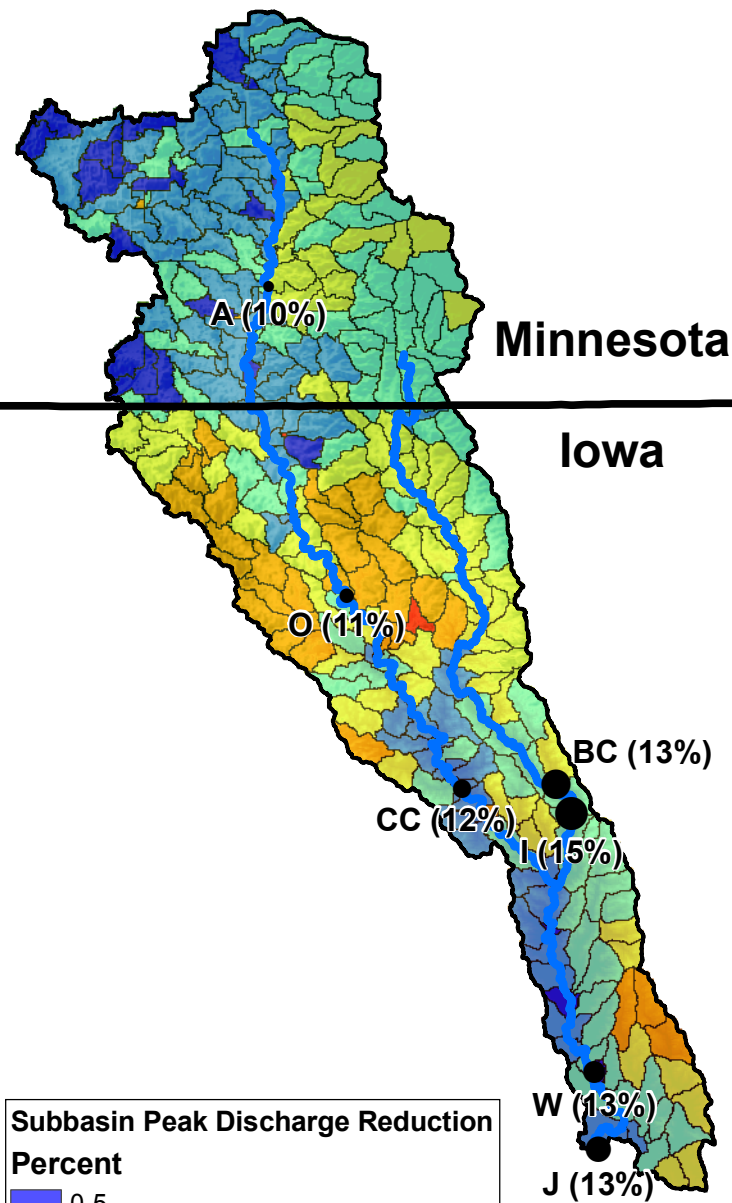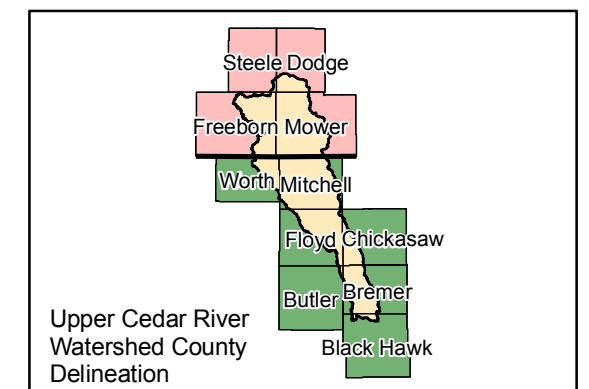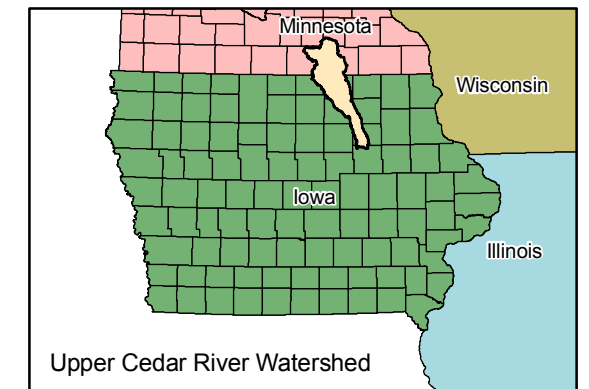
Upper Cedar River Watershed

Hypothetical Increase in Infiltration
Improved Soil Conditions (HSG C, C/D to B)
10-, 25-, and 100-Year, 24-Hour Design Storms

Index Locations:
 A - Austin
 O - Osage
 CC - Charles City
 BC - Beaver Creek Outlet
 I - Ionia
 W - Waverly
 J - Janesville
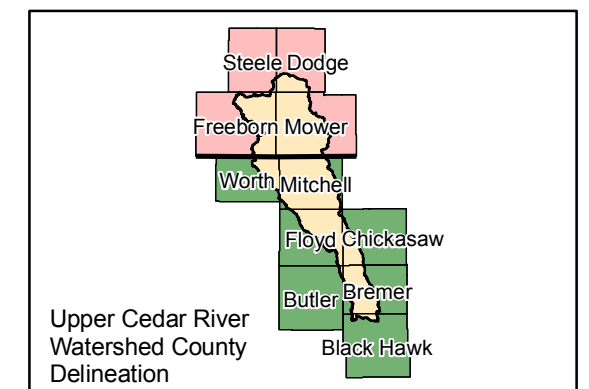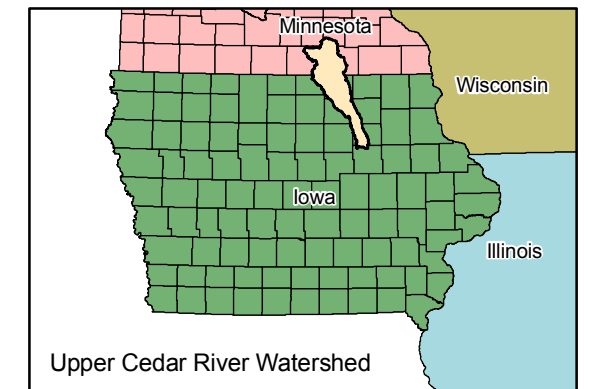
Date: April 2014

By: Chad Drake

Data Sources:

Figure A-13

Subbasin Peak Discharge Reduction Percent
- 0-5
- 5-10
- 10-15
- 15-20
- 20-25
- 25-33.7

Topographic Analysis

Minnesota

Iowa

Legend:
- USGS Stage/Discharge Gage
- Iowan Surface- Des Moines Lobe Boundary
- Exploratory Pond Site Location (10 total)
- Exploratory Headwater Subbasin (7 total)
- Headater Subbasin (121 total)
- Non-headwater subbasin (199 total)

N

Miles
0   10   20

Prototype Pond Assignments

Minnesota

Iowa

Legend:
- USGS Stage/Discharge Gage

Headwater Subbasins
Prototype Ponds Per Subbasin
- 2
- 3
- 4
- 5

N

Miles
0   10   20

logo: Iowa Flood Center

logo: IIHR Hydroscience & Engineering

The University of Iowa
C. Maxwell Stanley Hydraulics Laboratory
Iowa City, Iowa 52246

Minnesota — Wisconsin — Iowa — Illinois

Upper Cedar River Watershed

Steele Dodge
Freeborn Mower
Worth Mitchell
Floyd Chickasaw
Butler Bremer
Black Hawk

Upper Cedar River
Watershed County
Delineation
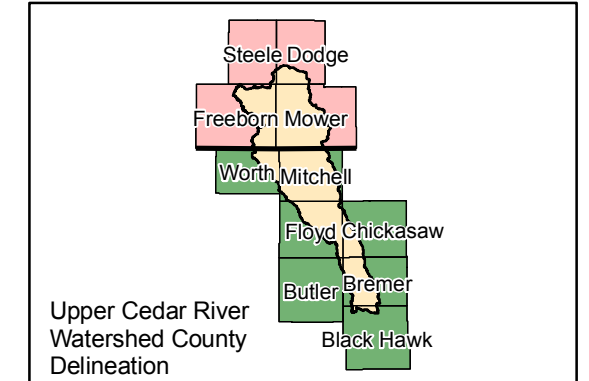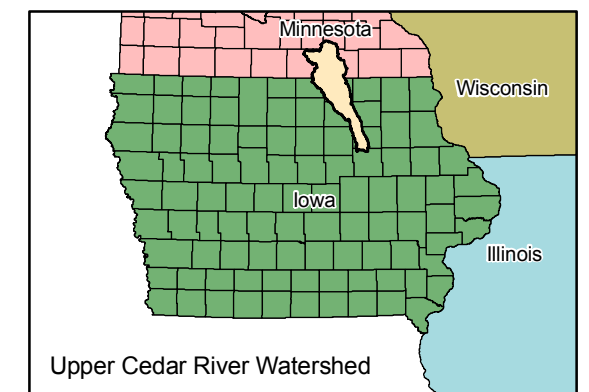
Upper Cedar River Watershed

Distributed Flood Storage Analysis
Topographic Analysis for Prototype Pond Development
Headwater Subbasin Prototype Pond Assignments

Notes:

1. GIS analysis performed at 10 exploratory pond site locations to develop a singe stage-storage relationship for hypothetical pond scenarios.

2. 372 prototype ponds aggregated into 121 ponds, one for each headwater subbasin.

Date: March 2014

By: Chad Drake

Data Sources:

Figure A-14

Small Pond Scenario
11 ac-ft/pond
Total: 4069 ac-ft

A (9%)

Minnesota

Iowa

O (6%)

BC (8%)
CC (5%)
I (3%)

W (3%)
J (3%)

Aggregated Pond Flood Storage
Percent Utilized
- 34-50
- 50-75
- 75-100
- >100 (E.S. activated)

N

Miles
0   10   20

Medium Pond Scenario
27 ac-ft/pond
Total: 9949 ac-ft

A (15%)

Minnesota

Iowa

O (11%)

BC (15%)
CC (10%)
I (8%)

W (7%)
J (7%)

N

Miles
0   10   20

Large Pond Scenario
48 ac-ft/pond
Total: 17930 ac-ft

A (15%)

Minnesota

Iowa

O (13%)

BC (15%)
CC (12%)
I (12%)

W (10%)
J (9%)

N

Miles
0   10   20

Iowa Flood Center

IIHR
Hydroscience & Engineering

The University of Iowa
C. Maxwell Stanley Hydraulics Laboratory
Iowa City, Iowa 52246

Minnesota
Wisconsin
Iowa
Illinois

Upper Cedar River Watershed

Steele Dodge
Freeborn Mower
Worth Mitchell
Floyd Chickasaw
Butler Bremer
Black Hawk

Upper Cedar River
Watershed County
Delineation
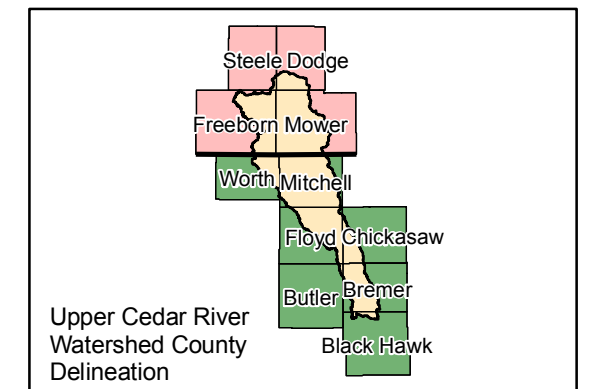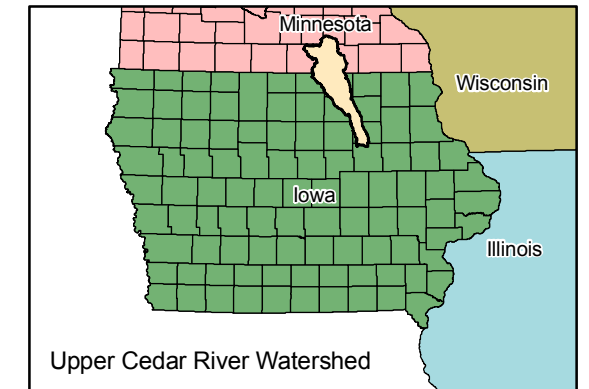
Upper Cedar River Watershed

Distributed Flood Storage Analysis
10 Year - 24 Hour Storm (4.05")
Wet Ponds

Index Locations:
 A - Austin
 O - Osage
 CC - Charles City
 BC - Beaver Creek Outlet
 I - Ionia
 W - Waverly
 J - Janesville

Date: March 2014
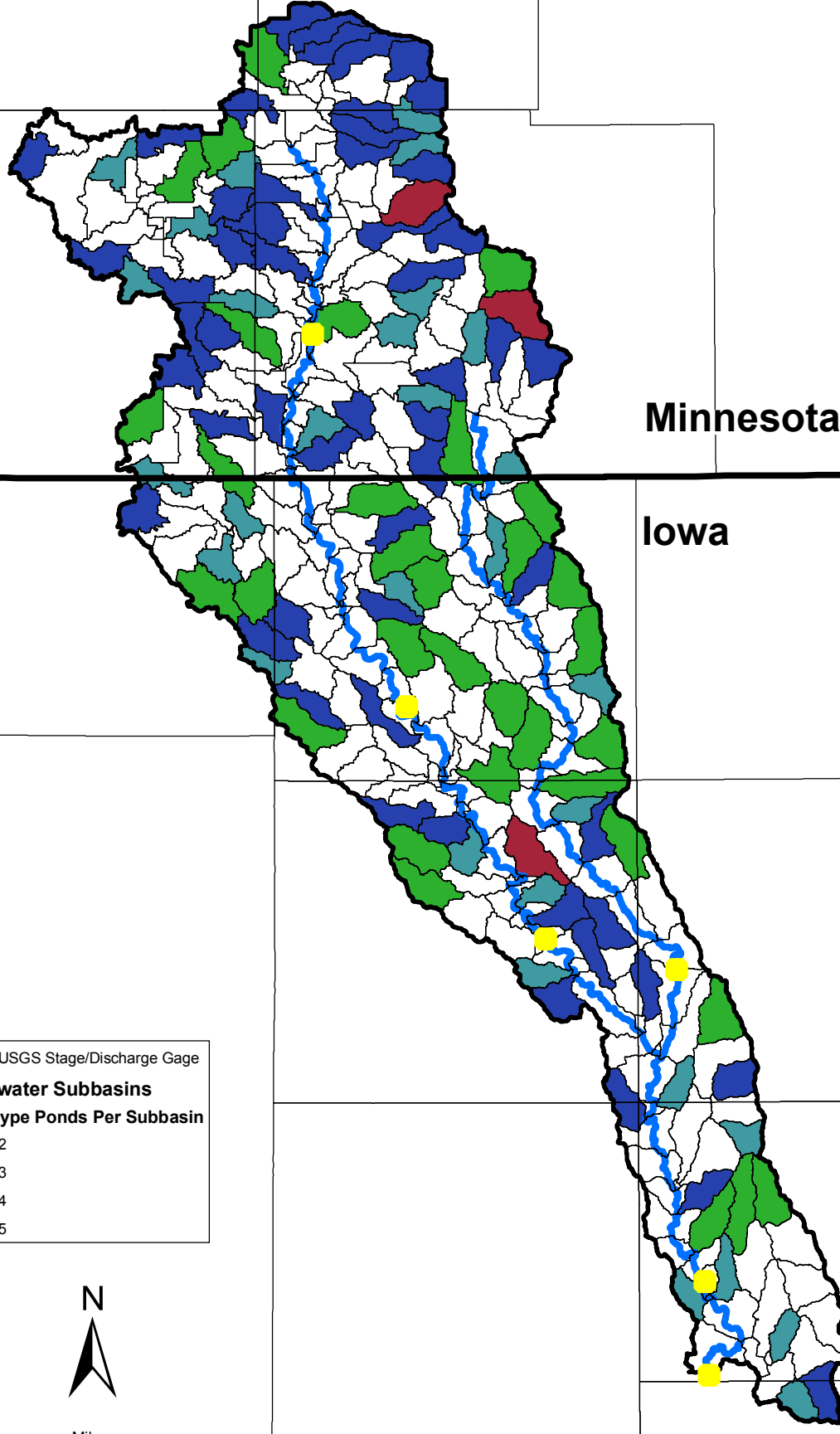
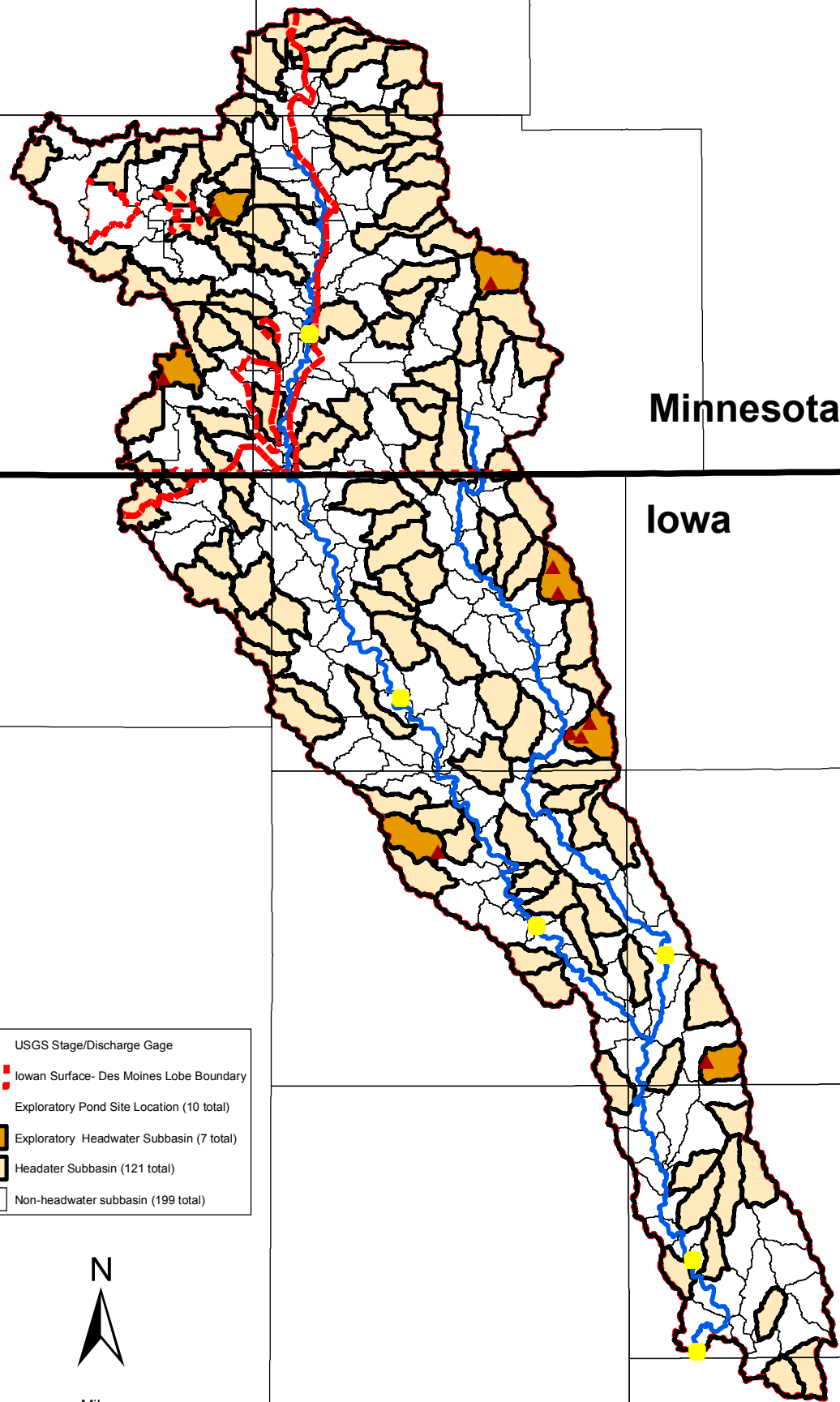By: Chad Drake

Data Sources:

Figure A-15

Figure A-16. Upper Cedar River Watershed — Distributed Flood Storage Analysis, 25 Year - 24 Hour Storm (5.05"), Wet Ponds.

Small Pond Scenario
11 ac-ft/pond
Total: 4069 ac-ft

A (6%)
Minnesota
Iowa
O (5%)
BC (3%)
CC (5%)
I (2%)
W (3%)
J (3%)

Aggregated Pond Flood Storage
Percent Utilized
- 34-50
- 50-75
- 75-100
- >100 (E.S. activated)

N

Miles
0   10   20

Medium Pond Scenario
27 ac-ft/pond
Total: 9949 ac-ft

A (16%)
Minnesota
Iowa
O (10%)
BC (18%)
CC (8%)
I (6%)
W (6%)
J (6%)

N

Miles
0   10   20

Large Pond Scenario
48 ac-ft/pond
Total: 17930 ac-ft

A (18%)
Minnesota
Iowa
O (15%)
BC (18%)
CC (13%)
I (11%)
W (11%)
J (10%)

N

Miles
0   10   20

Iowa Flood Center

IIHR
Hydroscience & Engineering

The University of Iowa
C. Maxwell Stanley Hydraulics Laboratory
Iowa City, Iowa 52246

Minnesota
Wisconsin
Iowa
Illinois

Upper Cedar River Watershed

Steele | Dodge
Freeborn | Mower
Worth | Mitchell
Floyd | Chickasaw
Butler | Bremer
Black Hawk

Upper Cedar River
Watershed County
Delineation

Upper Cedar River Watershed

Distributed Flood Storage Analysis
25 Year - 24 Hour Storm (5.05")
Wet Ponds

Index Locations:
A - Austin
O - Osage
CC - Charles City
BC - Beaver Creek Outlet
I - Ionia
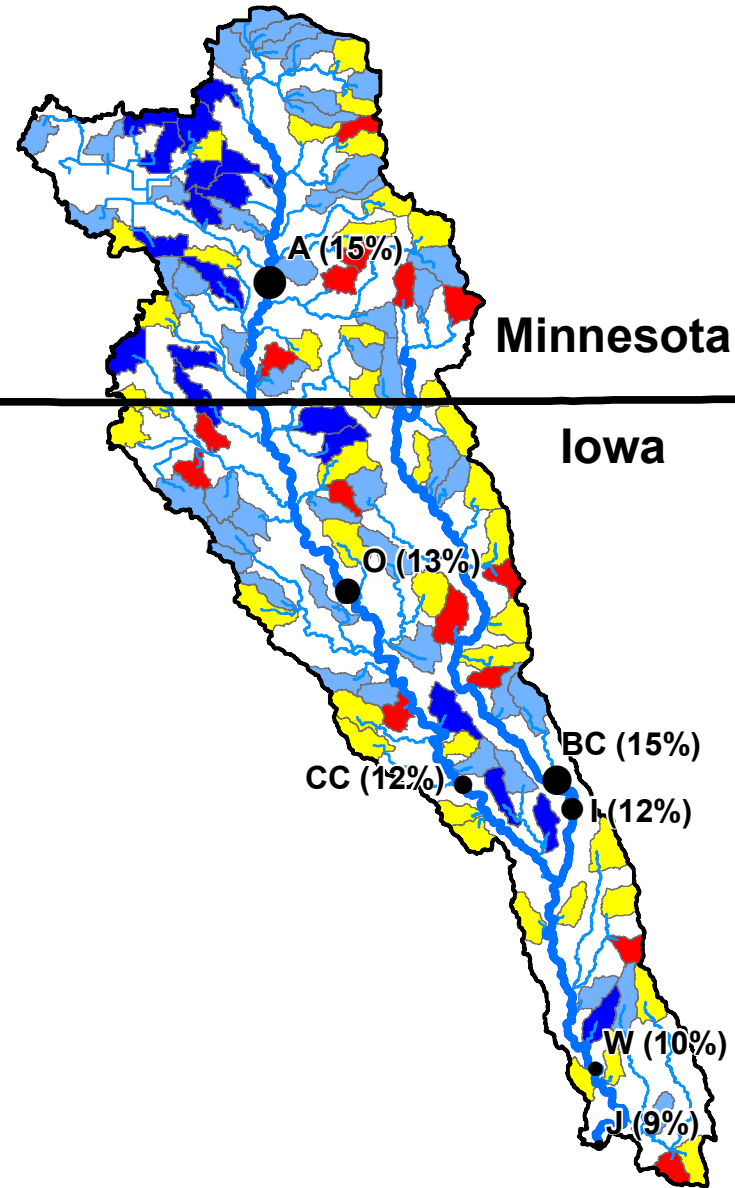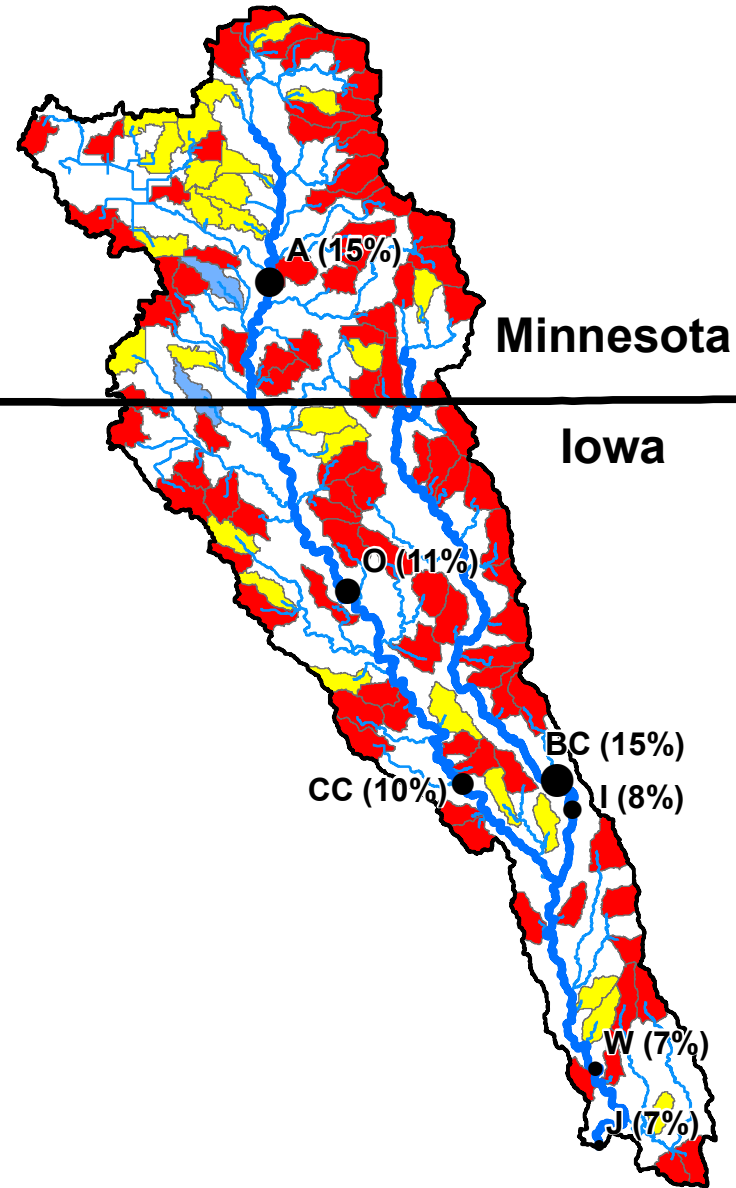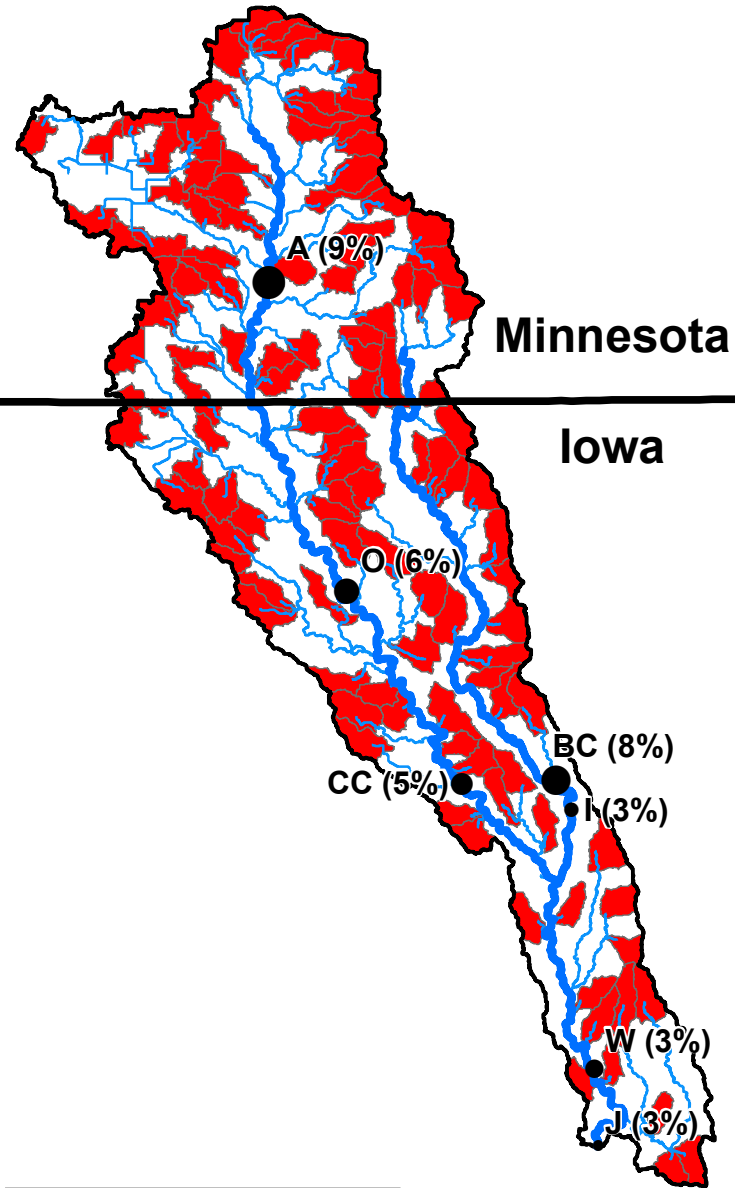W - Waverly
J - Janesville

Date: March 2014

By: Chad Drake

Data Sources:

Figure A-16

Small Pond Scenario
11 ac-ft/pond
Total: 4069 ac-ft
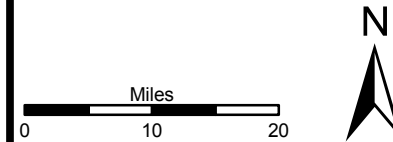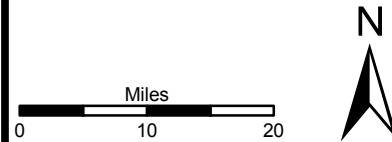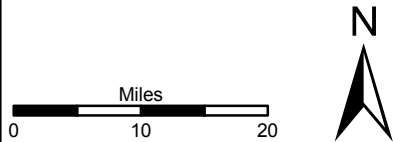
Medium Pond Scenario
27 ac-ft/pond
Total: 9949 ac-ft

Large Pond Scenario
48 ac-ft/pond
Total: 17930 ac-ft

Iowa
Flood
Center

IIHR
Hydroscience & Engineering

The University of Iowa
C. Maxwell Stanley Hydraulics Laboratory
Iowa City, Iowa 52246

Upper Cedar River Watershed

Upper Cedar River
Watershed County
Delineation

Steele Dodge
Freeborn Mower
Worth Mitchell
Floyd Chickasaw
Butler Bremer
Black Hawk
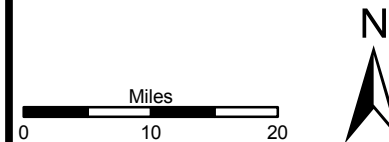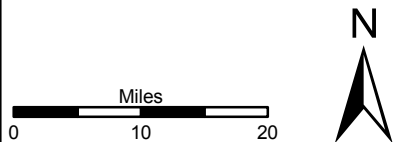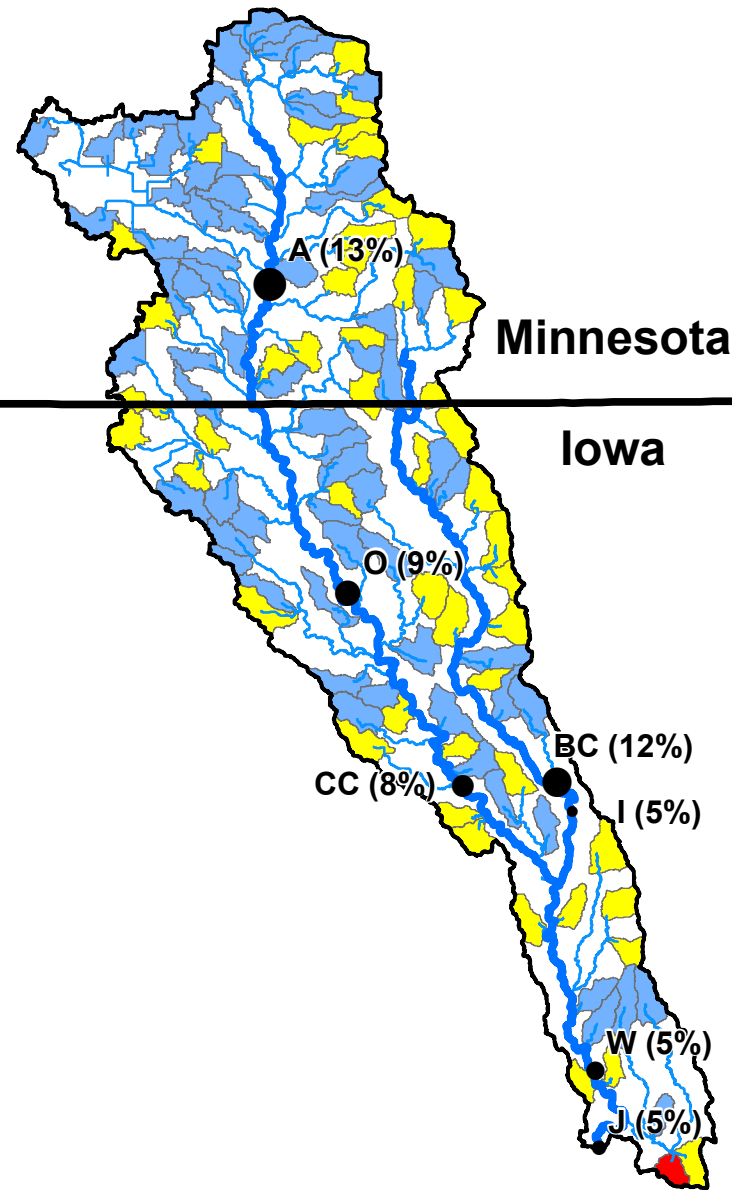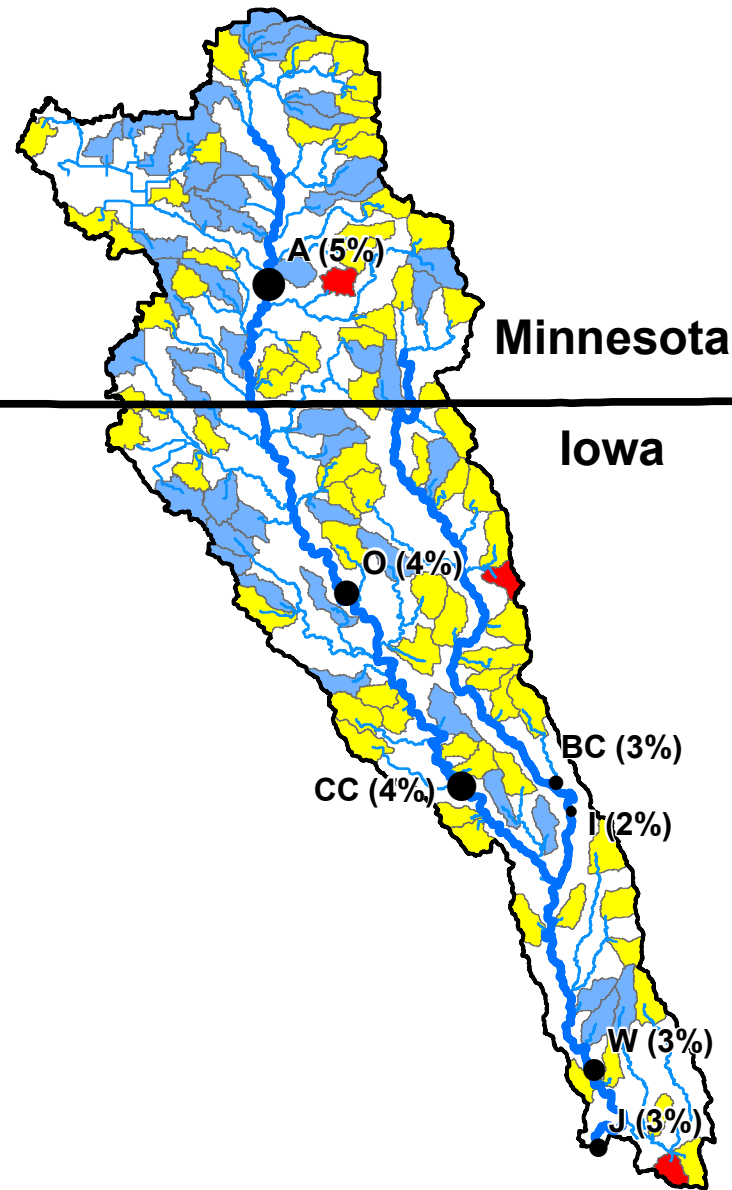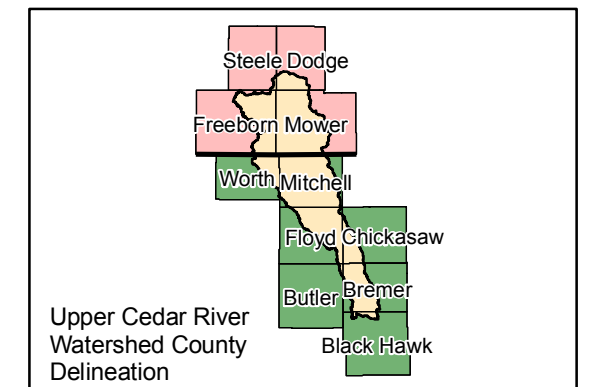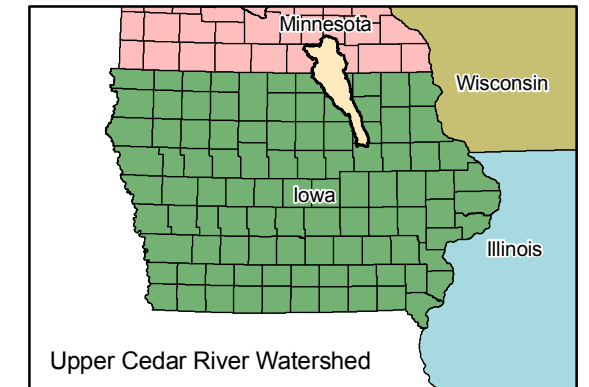
Upper Cedar River Watershed

Distributed Flood Storage Analysis
50 Year - 24 Hour Storm (5.89")
Wet Ponds

Index Locations:
A - Austin
O - Osage
CC - Charles City
BC - Beaver Creek Outlet
I - Ionia
W - Waverly
J - Janesville

Date: March 2014

By: Chad Drake

Data Sources:

Figure A-17

Small Pond Scenario: A (5%), O (4%), BC (3%), CC (4%), I (2%), W (3%), J (3%)

Medium Pond Scenario: A (13%), O (9%), BC (12%), CC (8%), I (5%), W (5%), J (5%)

Large Pond Scenario: A (19%), O (14%), BC (20%), CC (12%), I (9%), W (9%), J (9%)

Water Depth Over Emergency Spillway
Feet
- 0.2-0.5
- 0.5-1
- 1-1.5
- 1.5-1.6

Minnesota
Iowa

Minnesota
Wisconsin
Iowa
Illinois

Miles
0   10   20

N

Small Pond Scenario
11 ac-ft/pond
Total: 4069 ac-ft

A (4%)
Minnesota
Iowa
O (3%)
BC (2%)
CC (3%)
I (2%)
W (3%)
J (3%)

Water Depth Over Emergency Spillway
Feet
- 0.2-0.5
- 0.5-1
- 1-1.5
- 1.5-1.8

N
Miles
0 10 20

Medium Pond Scenario
27 ac-ft/pond
Total: 9949 ac-ft

A (11%)
Minnesota
Iowa
O (8%)
BC (8%)
CC (7%)
I (4%)
W (5%)
J (5%)

N
Miles
0 10 20

Large Pond Scenario
48 ac-ft/pond
Total: 17930 ac-ft

A (18%)
Minnesota
Iowa
O (12%)
BC (20%)
CC (10%)
I (7%)
W (8%)
J (7%)

N
Miles
0 10 20

Iowa Flood Center

IIHR
Hydroscience & Engineering

The University of Iowa
C. Maxwell Stanley Hydraulics Laboratory
Iowa City, Iowa 52246

Minnesota
Wisconsin
Iowa
Illinois
Upper Cedar River Watershed

Steele Dodge
Freeborn Mower
Worth Mitchell
Floyd Chickasaw
Butler Bremer
Black Hawk
Upper Cedar River Watershed County Delineation

Upper Cedar River Watershed

Distributed Flood Storage Analysis
100 Year - 24 Hour Storm (6.81")
Wet Ponds

Index Locations:
A - Austin
O - Osage
CC - Charles City
BC - Beaver Creek Outlet
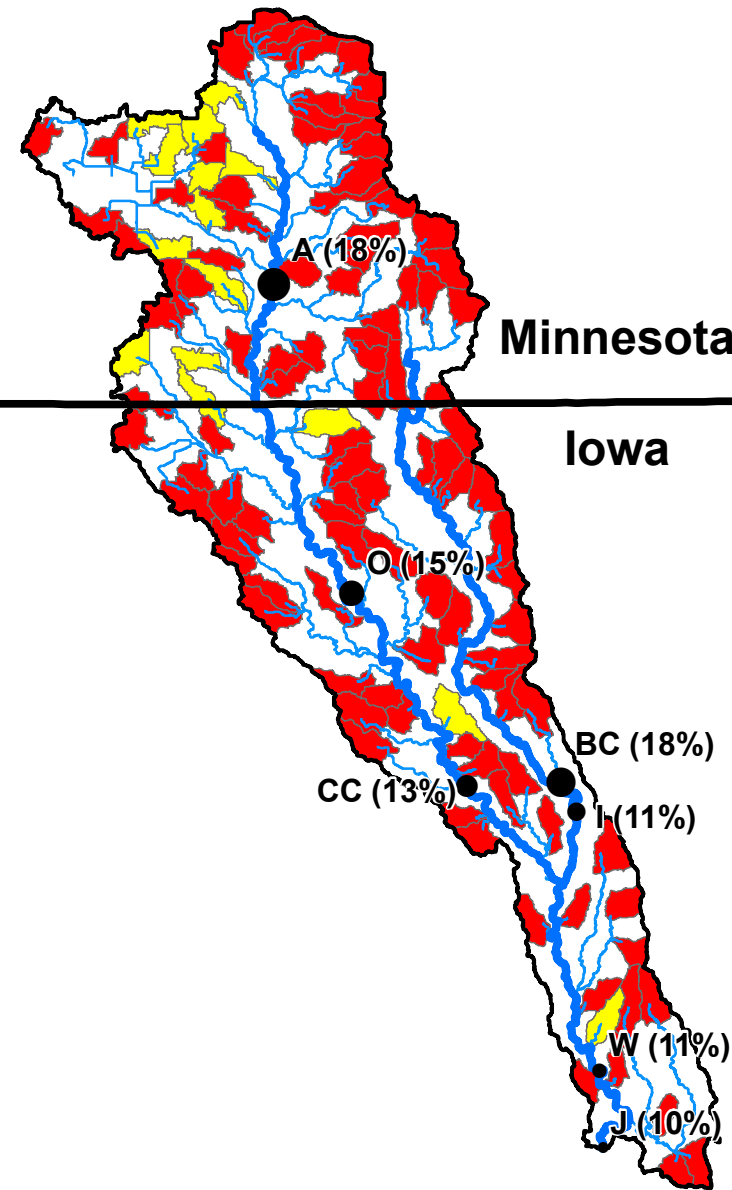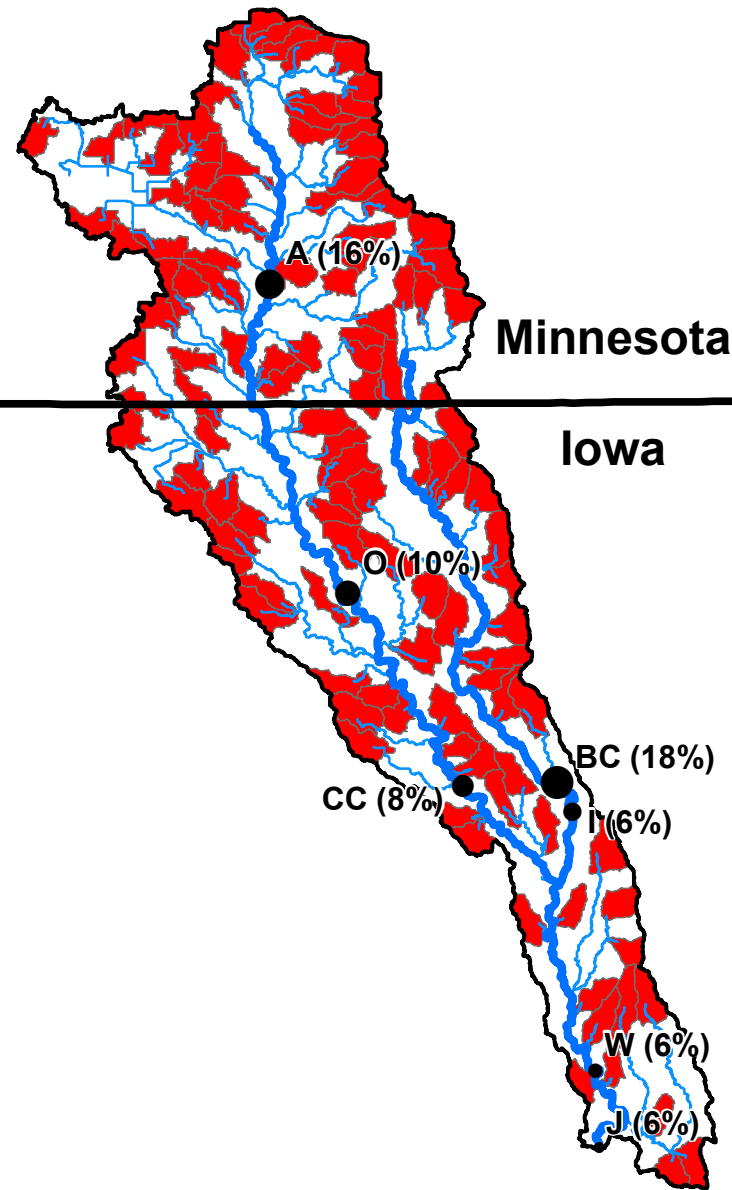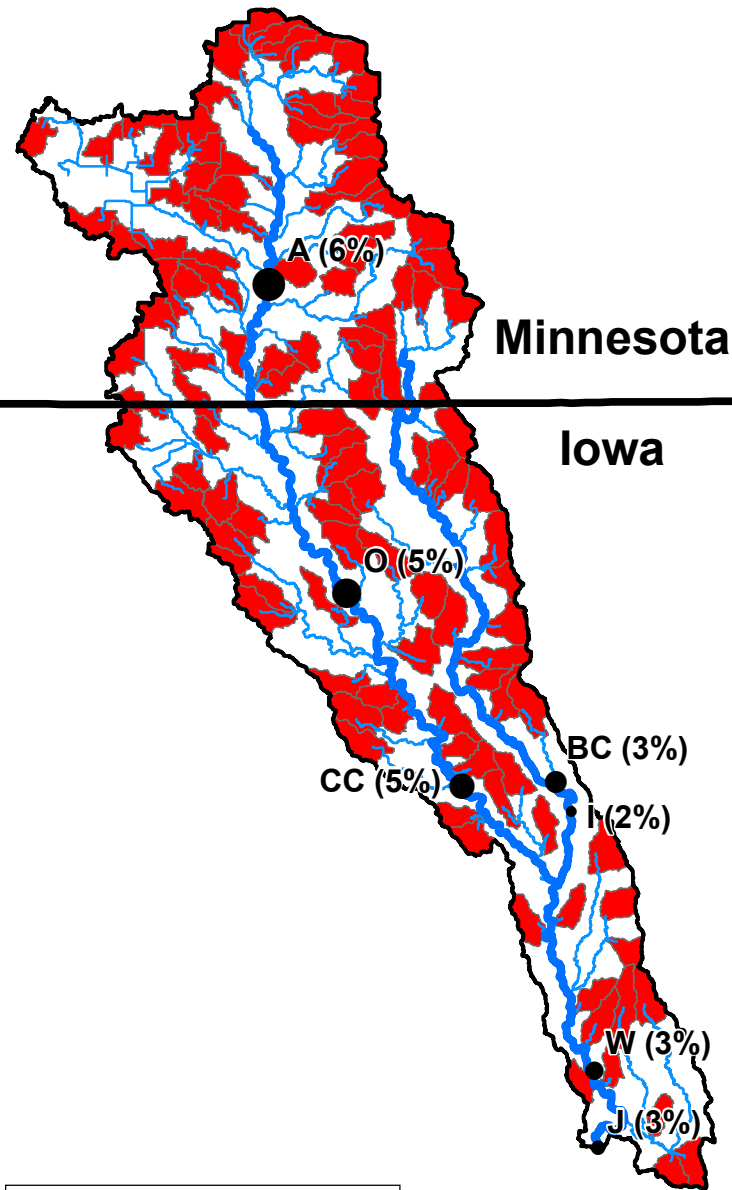I - Ionia
W - Waverly
J - Janesville

Date: March 2014

By: Chad Drake

Data Sources:

Figure A-18

10-Year, 24-Hour Storm (4.05 inches)

25-Year, 24-Hour Storm (5.05 inches)

Minnesota

Iowa

A (14%)

O (11%)

CC (10%)

BC (20%)

I (12%)
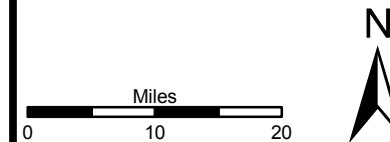
W (9%)

J (8%)
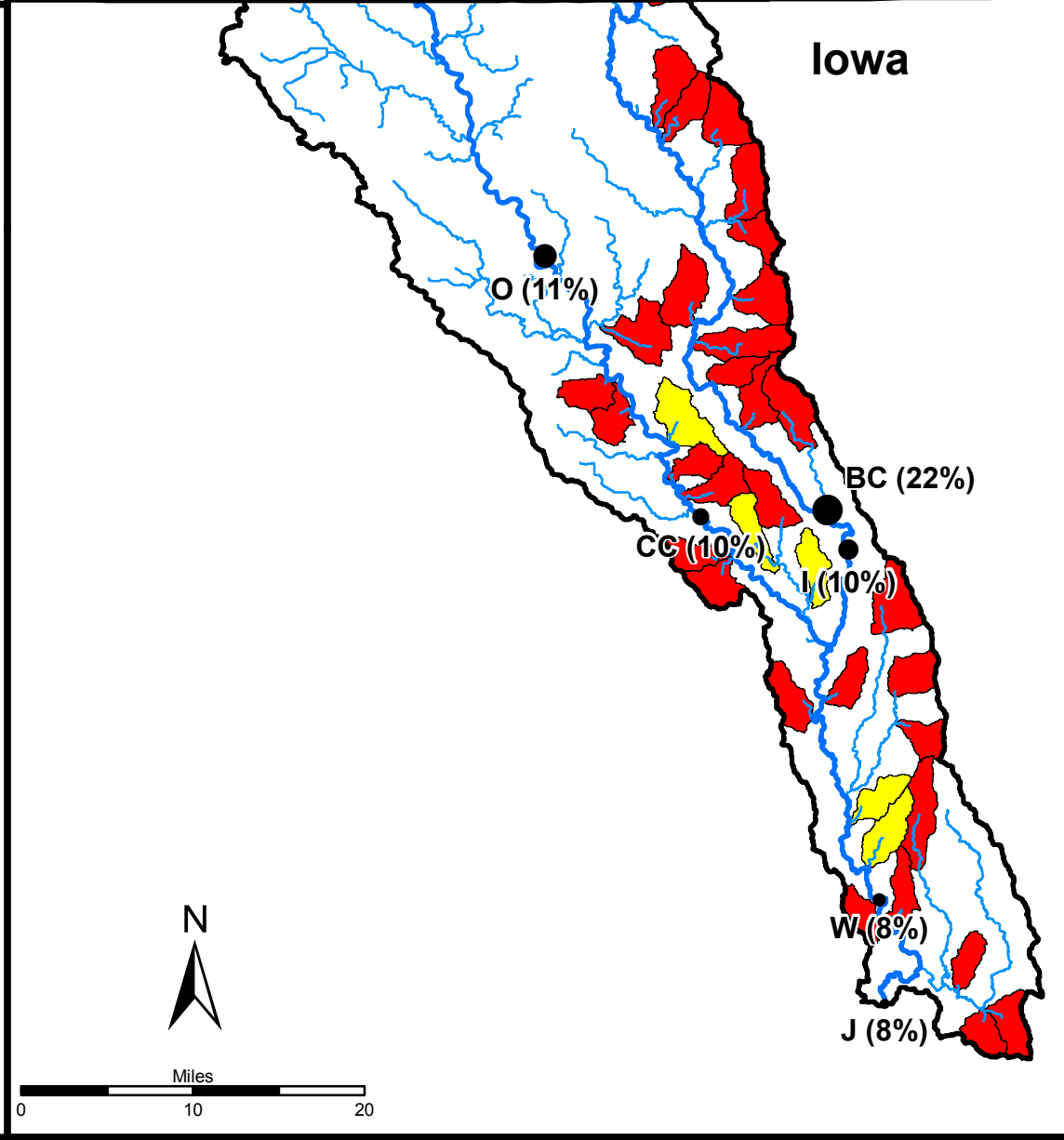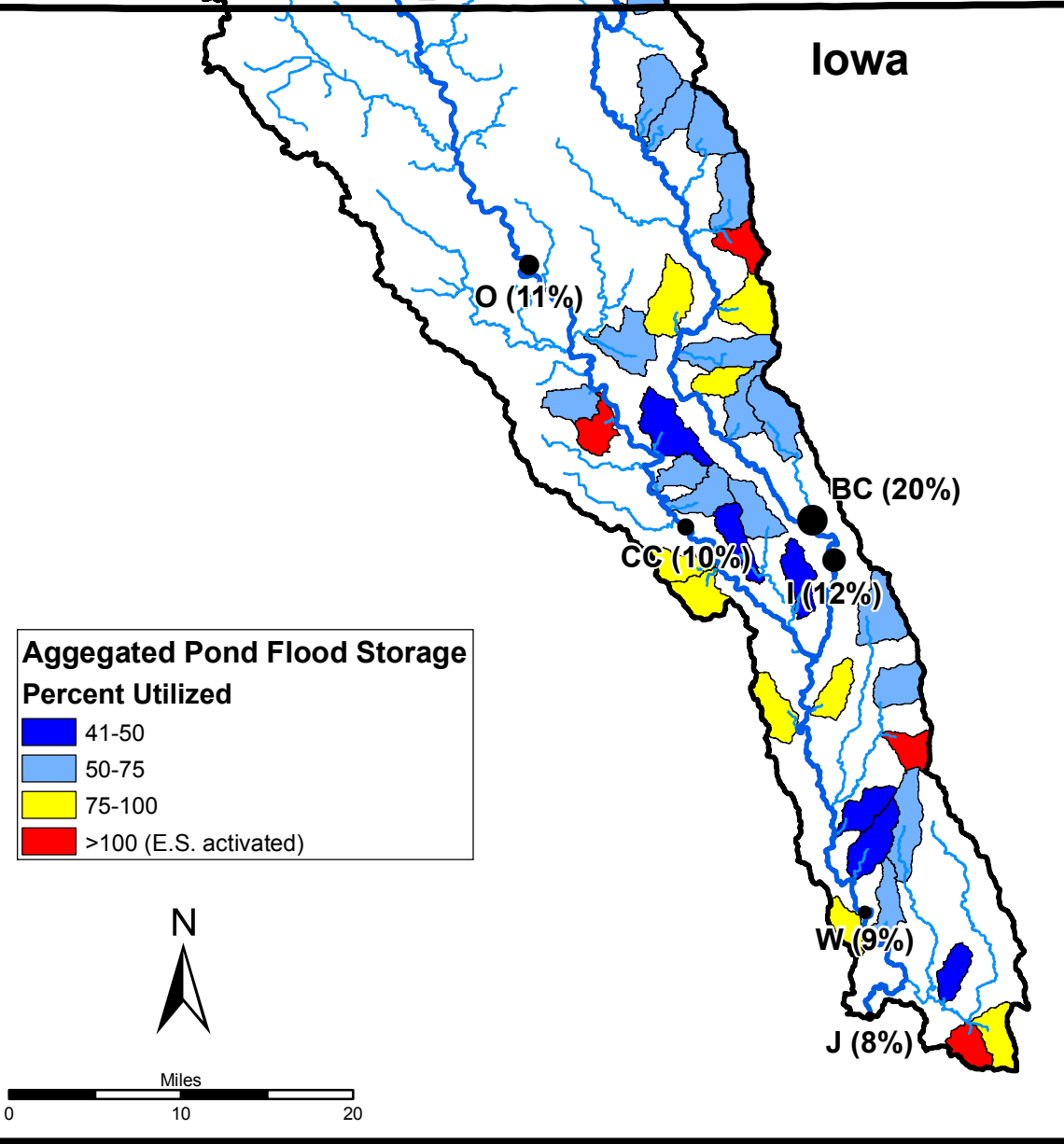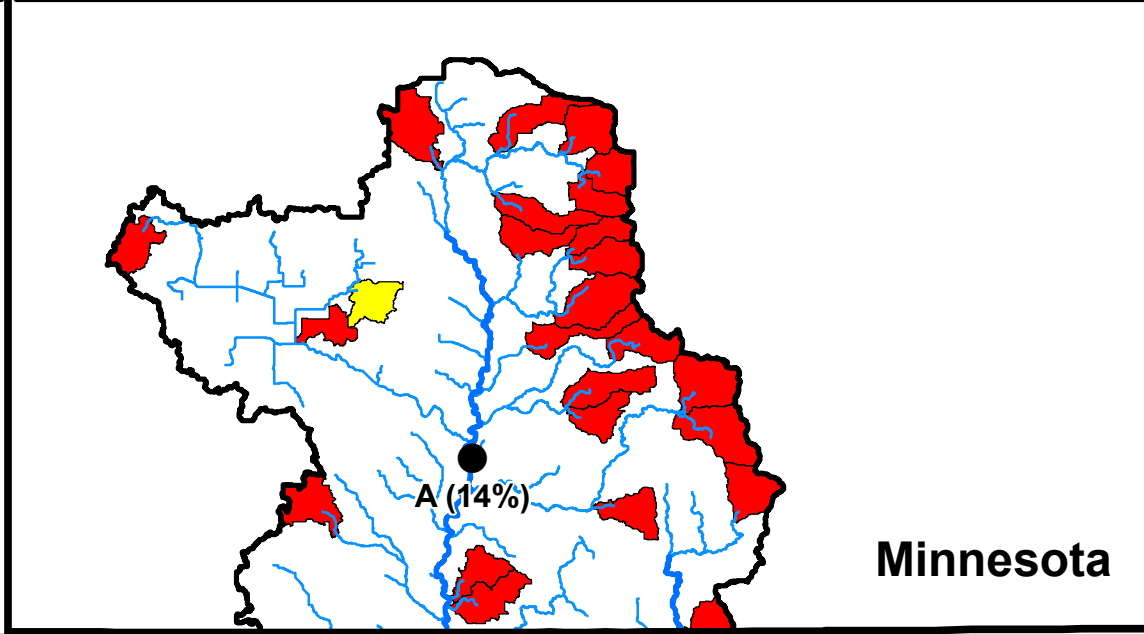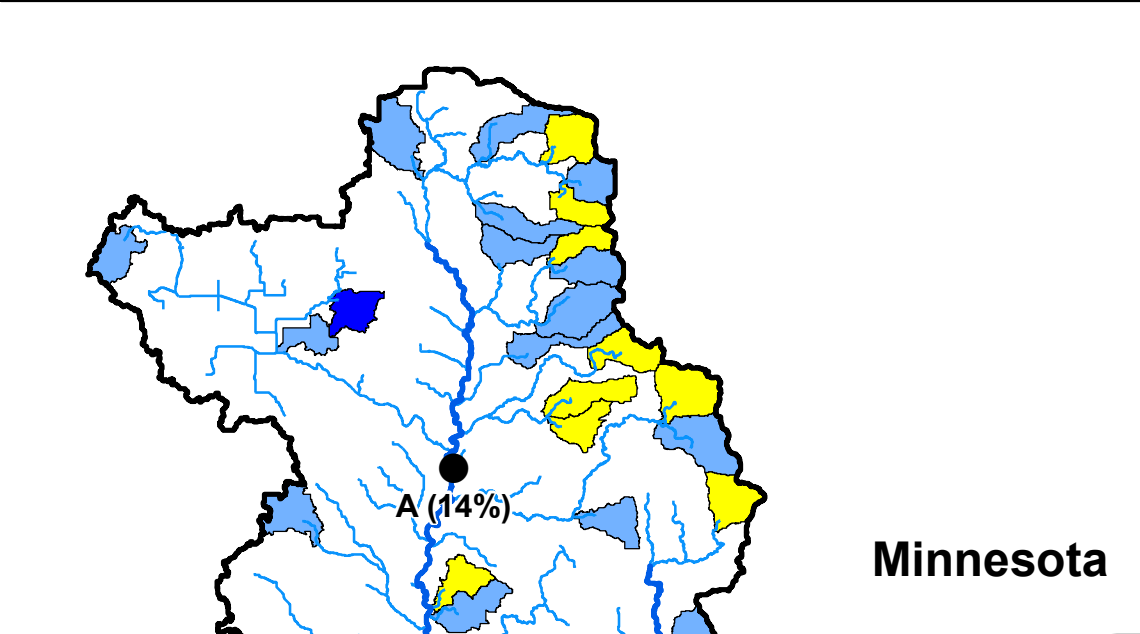
Aggegated Pond Flood Storage
Percent Utilized
- 41-50
- 50-75
- 75-100
- >100 (E.S. activated)
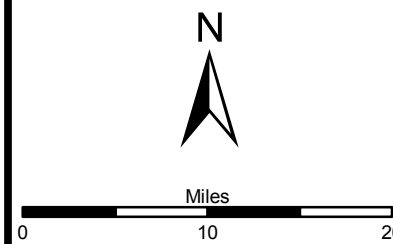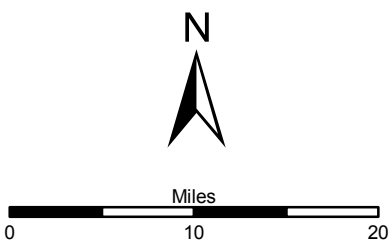
N

Miles
0   10   20

Minnesota

Iowa

A (14%)

O (11%)

CC (10%)

BC (22%)

I (10%)

W (8%)

J (8%)

N

Miles
0   10   20

Iowa
Flood
Center

IIHR
Hydroscience & Engineering

The University of Iowa
C. Maxwell Stanley Hydraulics Laboratory
Iowa City, Iowa 52246

Minnesota
Wisconsin
Iowa
Illinois

Upper Cedar River Watershed

Steele Dodge
Freeborn Mower
Worth Mitchell
Floyd Chickasaw
Butler Bremer
Black Hawk

Upper Cedar River
Watershed County
Delineation
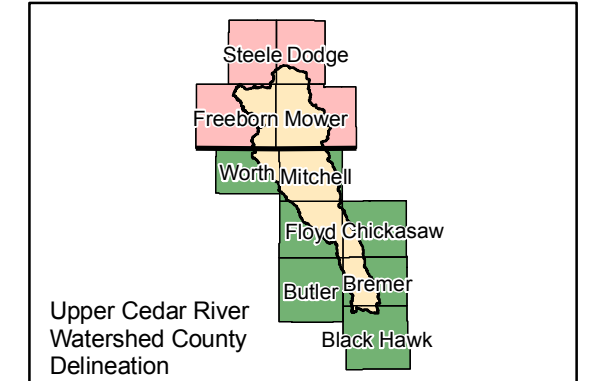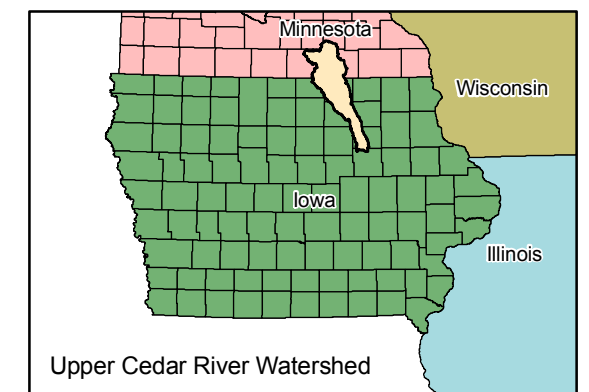
Upper Cedar River Watershed

Combination of Flood Mitigation Strategies
Distributed Storage and Cover Crops
10- and 25-Year, 24-Hour Design Storms

Index Locations:
A - Austin
O - Osage
CC - Charles City
BC - Beaver Creek Outlet
I - Ionia
W - Waverly
J - Janesville
NOTES:
1. Large, wet pond design
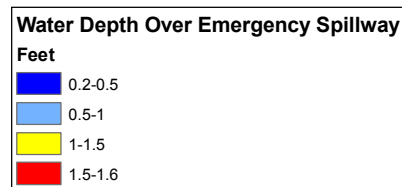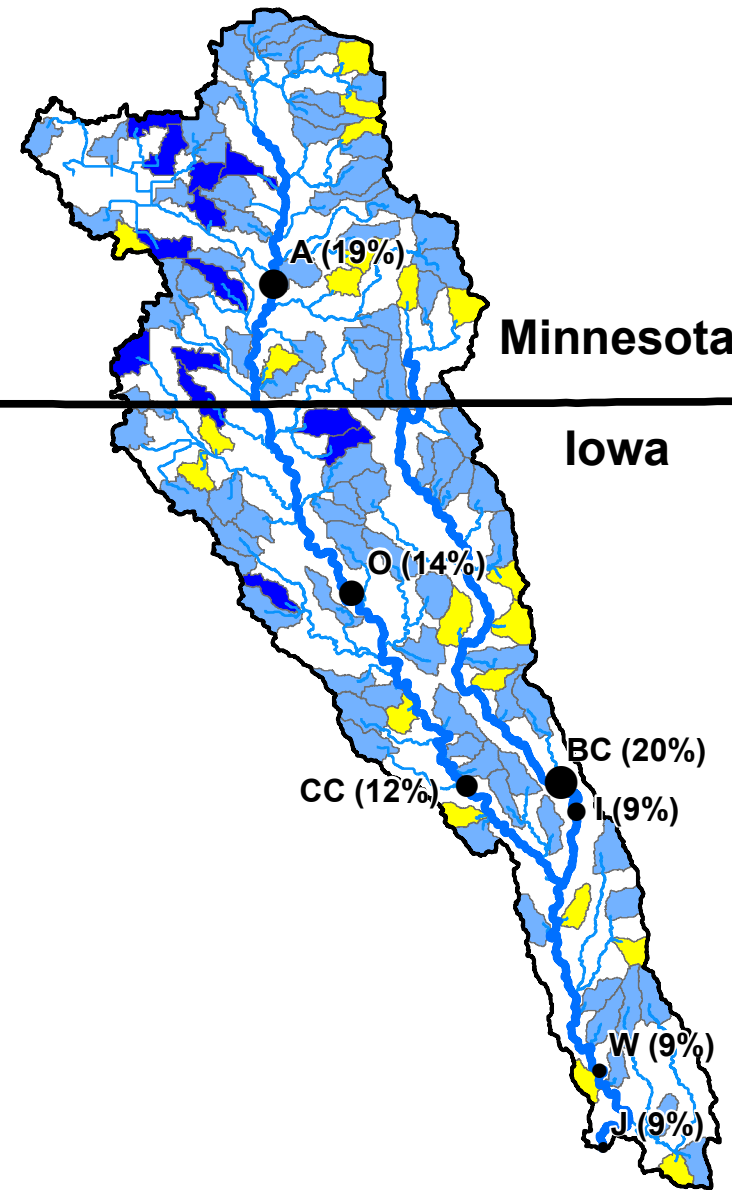(48.2 ac-ft/prototype pond)
2. Assume 50% of agricultural
areas plant cover crops

Date: April 2014

By: Chad Drake

Data Sources:

Figure A-19

50-Year, 24-Hour Storm (5.89 inches)

100-Year, 24-Hour Storm (6.81 inches)

Minnesota

Iowa

A (14%)

O (10%)

BC (23%)

CC (9%)

I (8%)

W (8%)

J (7%)

Minnesota

Iowa

A (13%)

O (10%)

BC (23%)

CC (9%)

I (6%)

W (7%)

J (7%)

Water Depth Over Emergency Spillway
Feet

- 0.4-0.5
- 0.5-1
- 1-1.5
- 1.5-1.6

N

Miles
0   10   20

N

Miles
0   10   20

Iowa
Flood
Center

IIHR
Hydroscience & Engineering

The University of Iowa
C. Maxwell Stanley Hydraulics Laboratory
Iowa City, Iowa 52246

Minnesota
Wisconsin
Iowa
Illinois

Upper Cedar River Watershed

Steele Dodge
Freeborn Mower
Worth Mitchell
Floyd Chickasaw
Butler Bremer
Black Hawk

Upper Cedar River
Watershed County
Delineation
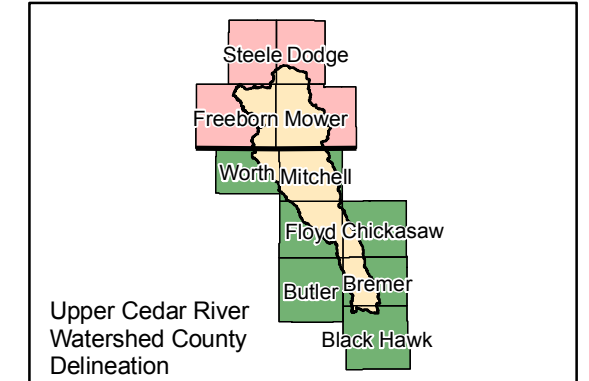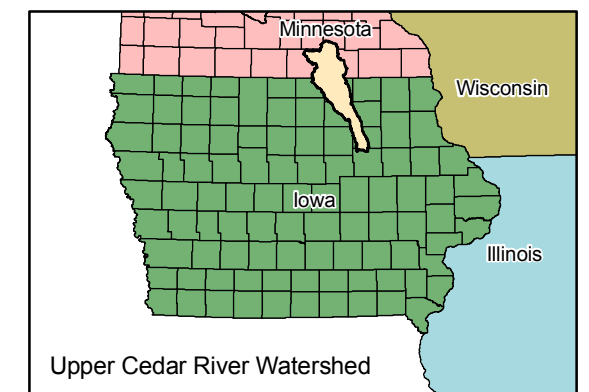
Upper Cedar River Watershed

Combination of Flood Mitigation Strategies
Distributed Storage and Cover Crops
50- and 100-Year, 24-Hour Design Storms

Index Locations:
A - Austin
O - Osage
CC - Charles City
BC - Beaver Creek Outlet
I - Ionia
W - Waverly
J - Janesville
NOTES:
1. Large, wet pond design
(48.2 ac-ft/prototype pond)
2. Assume 50% of agricultural
areas plant cover crops

Date: April 2014

By: Chad Drake

Data Sources:

Figure A-20

Land Use: Agriculture to Prairie

Minnesota

Iowa

A (24%)

O (19%)

BC (19%)

CC (19%)

I (23%)

W (25%)

J (25%)

Subbasin Peak Discharge Reduction
Percent
- 0-10
- 10-20
- 20-30
- 30-58

Miles
0   10   20

N

Land Use: Cover Crops (100%)

Minnesota

Iowa

A (4%)

O (3%)

BC (4%)

CC (5%)

I (5%)

W (5%)

J (5%)

Subbasin Peak Discharge Reduction
Percent
- 0-2
- 2-4
- 4-6
- 6-8.9

Miles
0   10   20

N
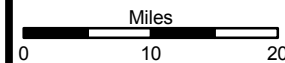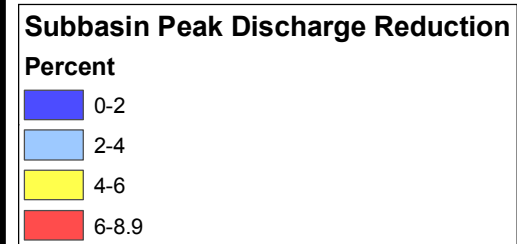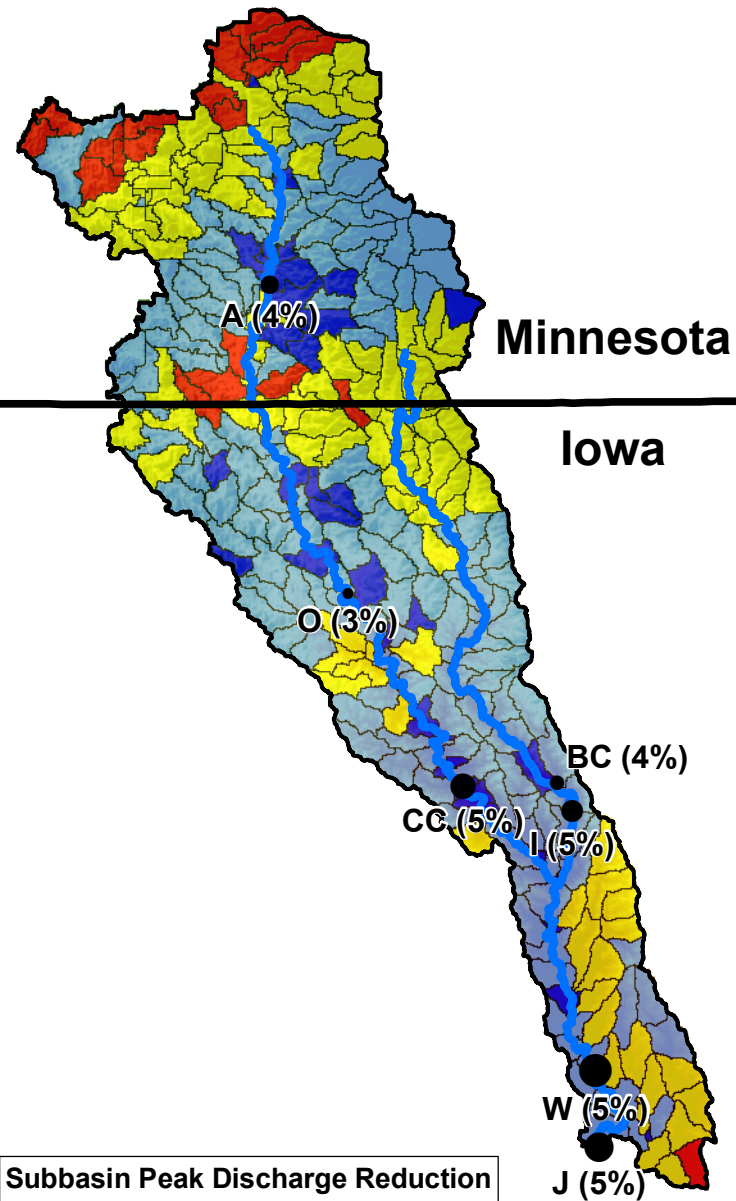
Blended: Cover Crops (50%) & Flood Storage
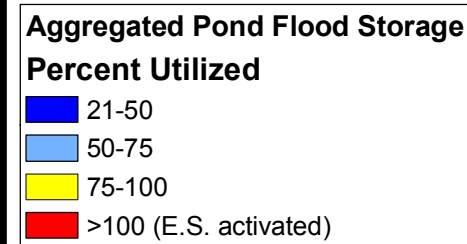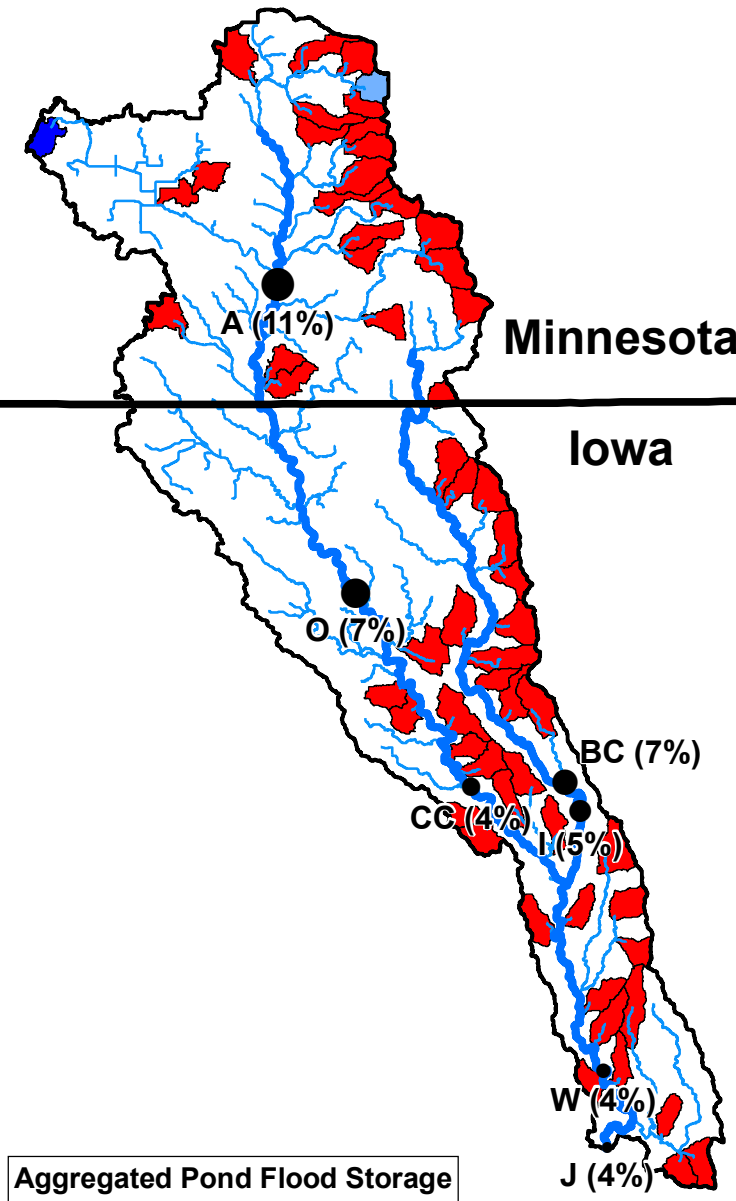
Minnesota

Iowa

A (11%)

O (7%)

BC (7%)

CC (4%)

I (5%)

W (4%)

J (4%)

Aggregated Pond Flood Storage
Percent Utilized
- 21-50
- 50-75
- 75-100
- >100 (E.S. activated)

Miles
0   10   20

N

Iowa Flood Center

IIHR
Hydroscience & Engineering

The University of Iowa
C. Maxwell Stanley Hydraulics Laboratory
Iowa City, Iowa 52246

Minnesota
Wisconsin
Iowa
Illinois

Upper Cedar River Watershed

Steele Dodge
Freeborn Mower
Worth Mitchell
Floyd Chickasaw
Butler Bremer
Black Hawk

Upper Cedar River Watershed County Delineation
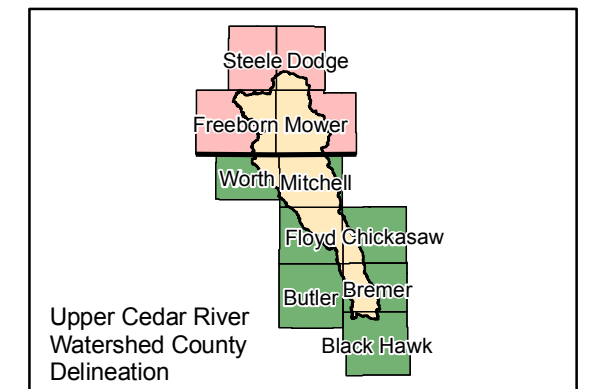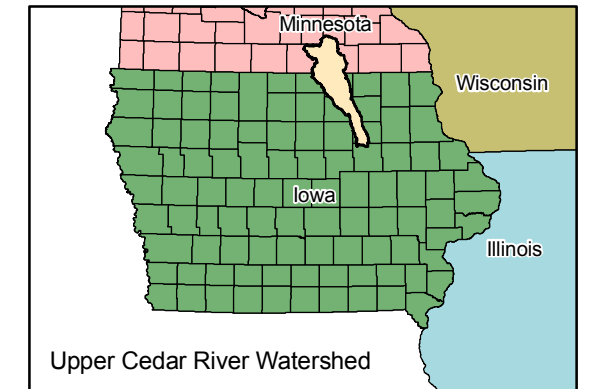
Upper Cedar River Watershed

Assessment of Flood Mitigation Strategies
June 5-17, 2008 Flood Simulation

Index Locations:
 A - Austin
 O - Osage
 CC - Charles City
 BC - Beaver Creek Outlet
 I - Ionia
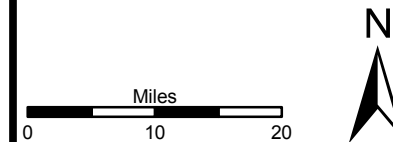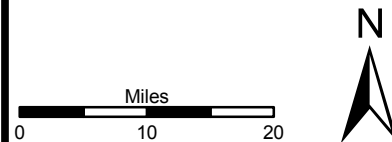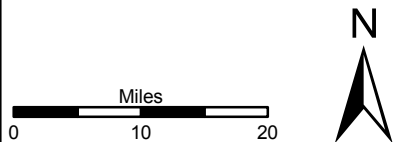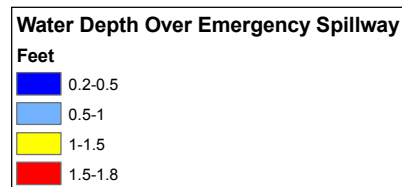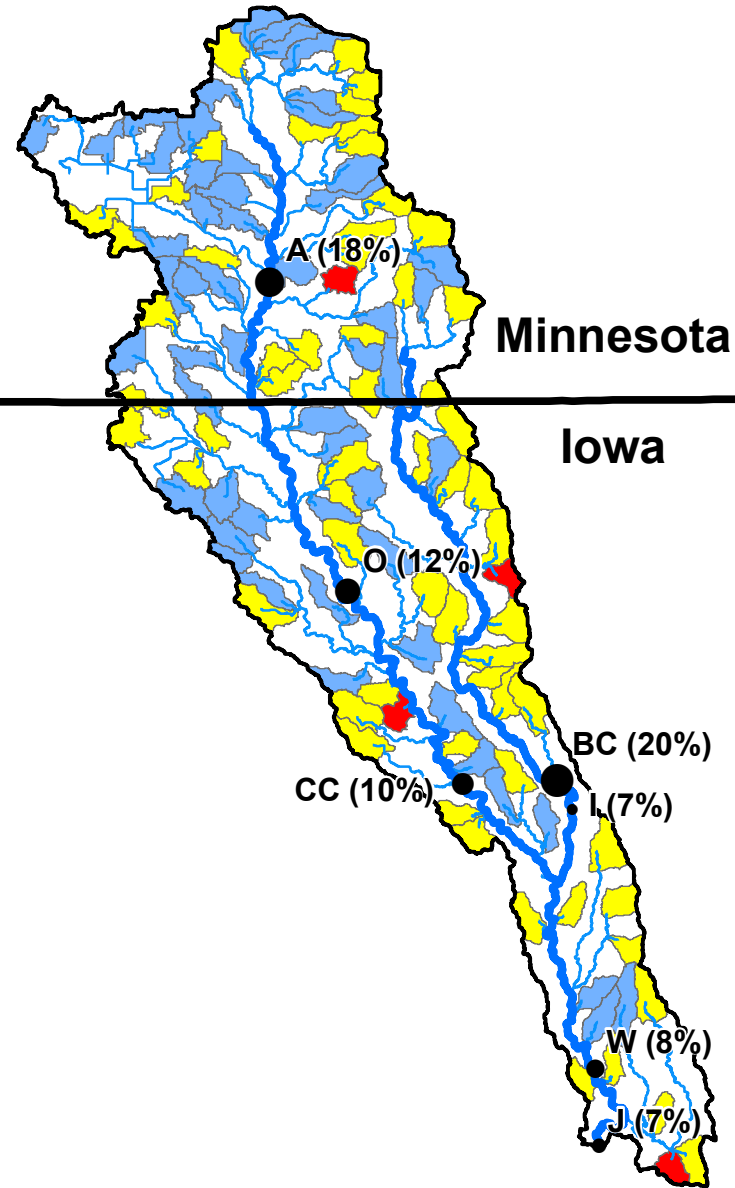 W - Waverly
 J - Janesville

Date: May 2014

By: Chad Drake

Data Sources:

Figure A-21

Soils: B to A

A (12%)

Minnesota

Iowa

O (8%)

BC (10%)

CC (9%)

I (9%)

Subbasin Peak Discharge Reduction Percent
- 0-5
- 5-10
- 10-15
- 15-32

N

Miles
0   10   20

W (10%)

J (10%)

Soils: C to B

A (3%)

Minnesota

Iowa

O (3%)

CC (5%)   BC (4%)

I (6%)

Subbasin Peak Discharge Reduction Percent
- 0-2
- 2-4
- 4-6
- 6-9

N

Miles
0   10   20

W (6%)

J (6%)

Iowa Flood Center

IIHR Hydroscience & Engineering

The University of Iowa
C. Maxwell Stanley Hydraulics Laboratory
Iowa City, Iowa 52246

Minnesota   Wisconsin

Iowa   Illinois

Upper Cedar River Watershed

Steele Dodge

Freeborn Mower

Worth Mitchell

Floyd Chickasaw

Butler Bremer

Black Hawk

Upper Cedar River Watershed County Delineation

Upper Cedar River Watershed

Assessment of Flood Mitigation Strategies
June 5-17, 2008 Flood Simulation

Index Locations:
A - Austin
O - Osage
CC - Charles City
BC - Beaver Creek Outlet
I - Ionia
W - Waverly
J - Janesville
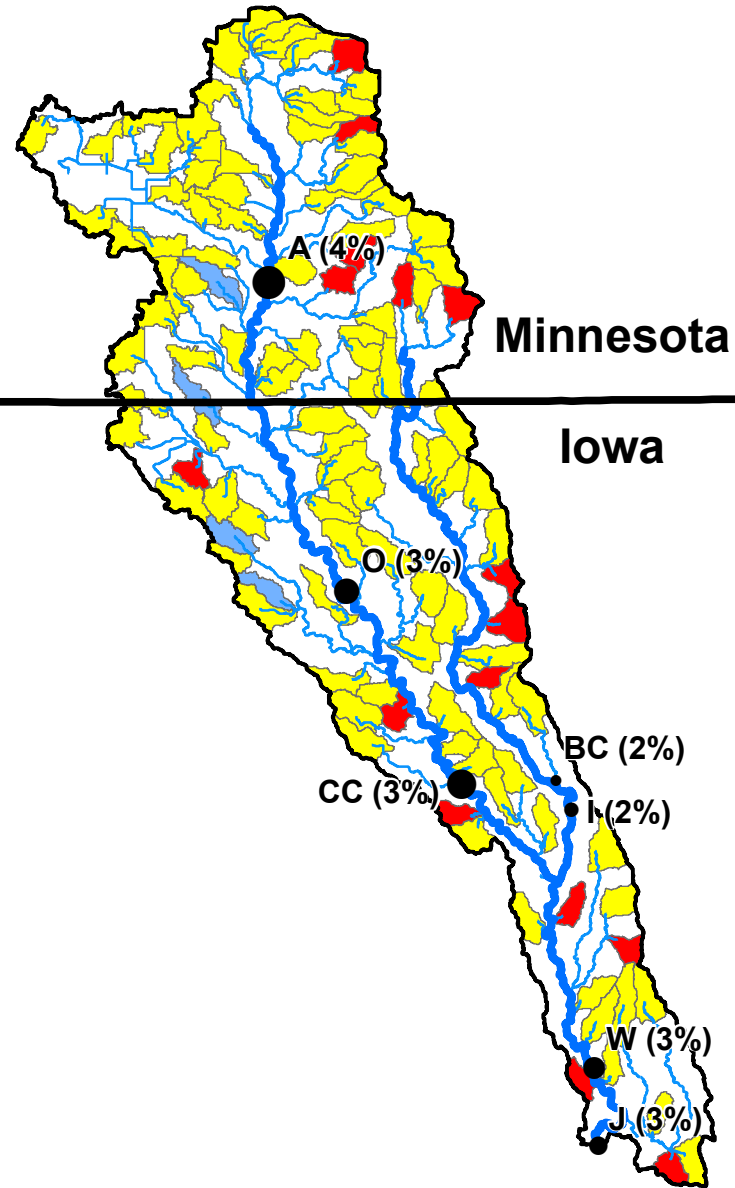
Date: May 2014

By: Chad Drake
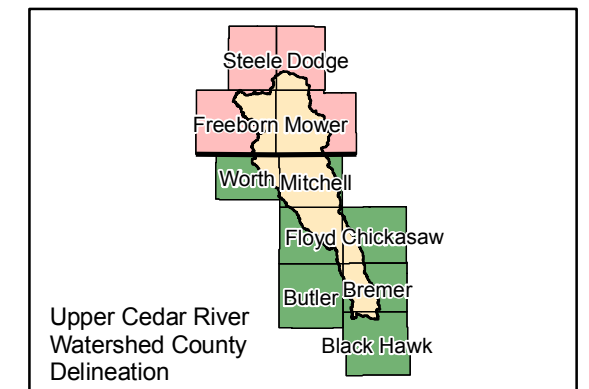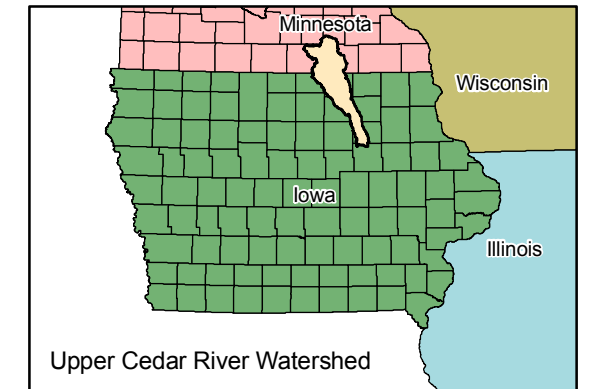
Data Sources:

Figure A-22

Small Pond Scenario
11 ac-ft/pond
Total: 4069 ac-ft

Medium Pond Scenario
27 ac-ft/pond
Total: 9949 ac-ft

Large Pond Scenario
48 ac-ft/pond
Total: 17930 ac-ft

Iowa
Flood
Center

IIHR
Hydroscience & Engineering

The University of Iowa
C. Maxwell Stanley Hydraulics Laboratory
Iowa City, Iowa 52246

Minnesota
Wisconsin
Iowa
Illinois

Upper Cedar River Watershed

Steele Dodge
Freeborn Mower
Worth Mitchell
Floyd Chickasaw
Butler Bremer
Black Hawk

Upper Cedar River
Watershed County
Delineation

Small Pond Scenario map:
A (5.6%)
Minnesota
Iowa
O (4.4%)
BC (0.8%)
CC (1.3%)
I (1.3%)
W (1.1%)
J (1%)

Medium Pond Scenario map:
A (11.8%)
Minnesota
Iowa
O (7.8%)
BC (1.4%)
CC (3.6%)
I (3.3%)
W (3.1%)
J (3%)

Large Pond Scenario map:
A (15.7%)
Minnesota
Iowa
O (9.1%)
BC (3.4%)
CC (5.9%)
I (5.9%)
W (5.6%)
J (5.4%)

Aggregated Pond Flood Storage
Percent Utilized
- 24-50
- 50-75
- 75-100
- >100 (E.S. activated)

N

Miles
0   10   20

Upper Cedar River Watershed

Distributed Flood Storage Analysis
June 5-17, 2008 Flood Simulation

Index Locations:
 A - Austin
 O - Osage
 CC - Charles City
 BC - Beaver Creek Outlet
 I - Ionia
 W - Waverly
 J - Janesville
NOTES:
 1. Wet and dry ponds performed
 similarly, so only wet pond
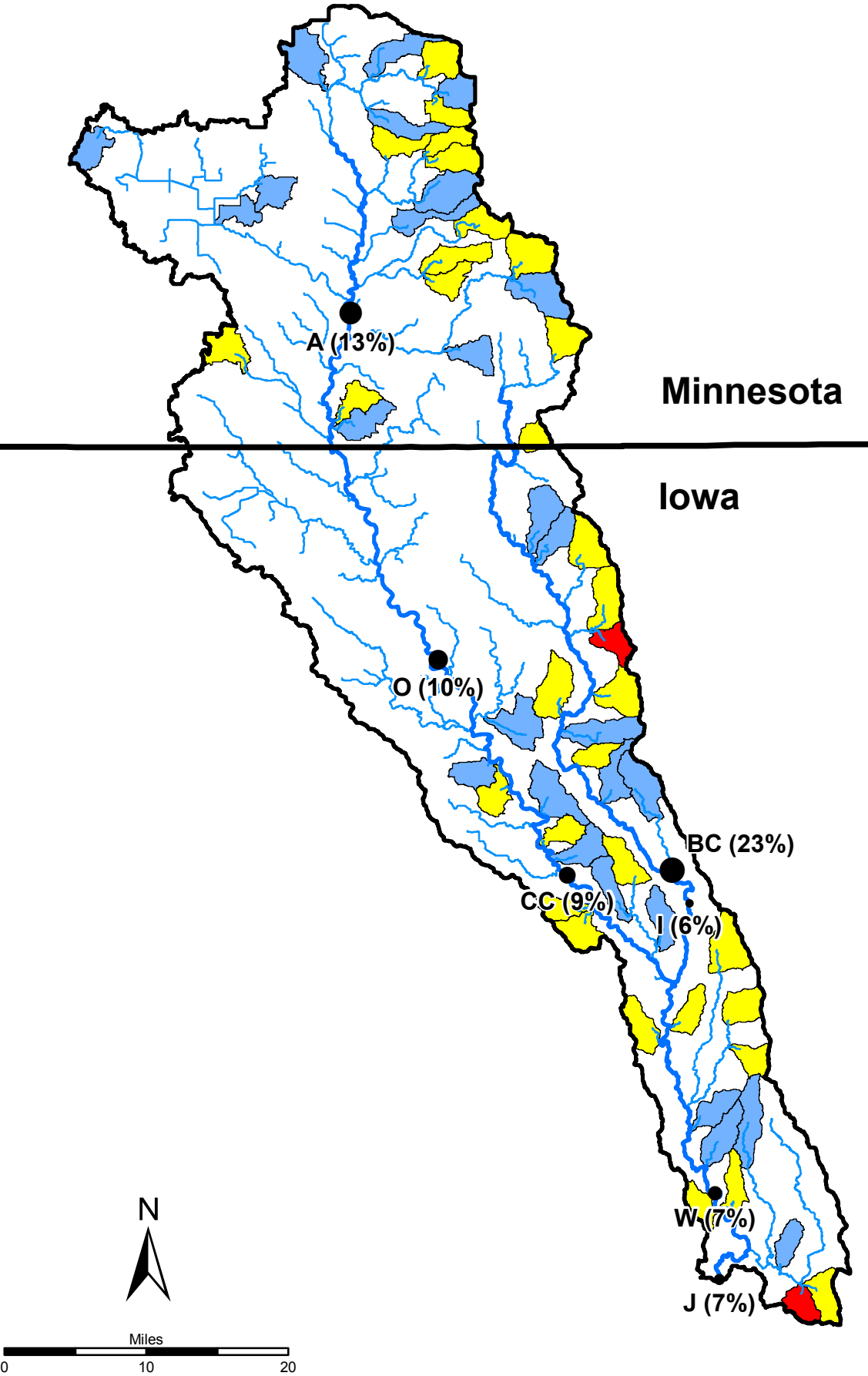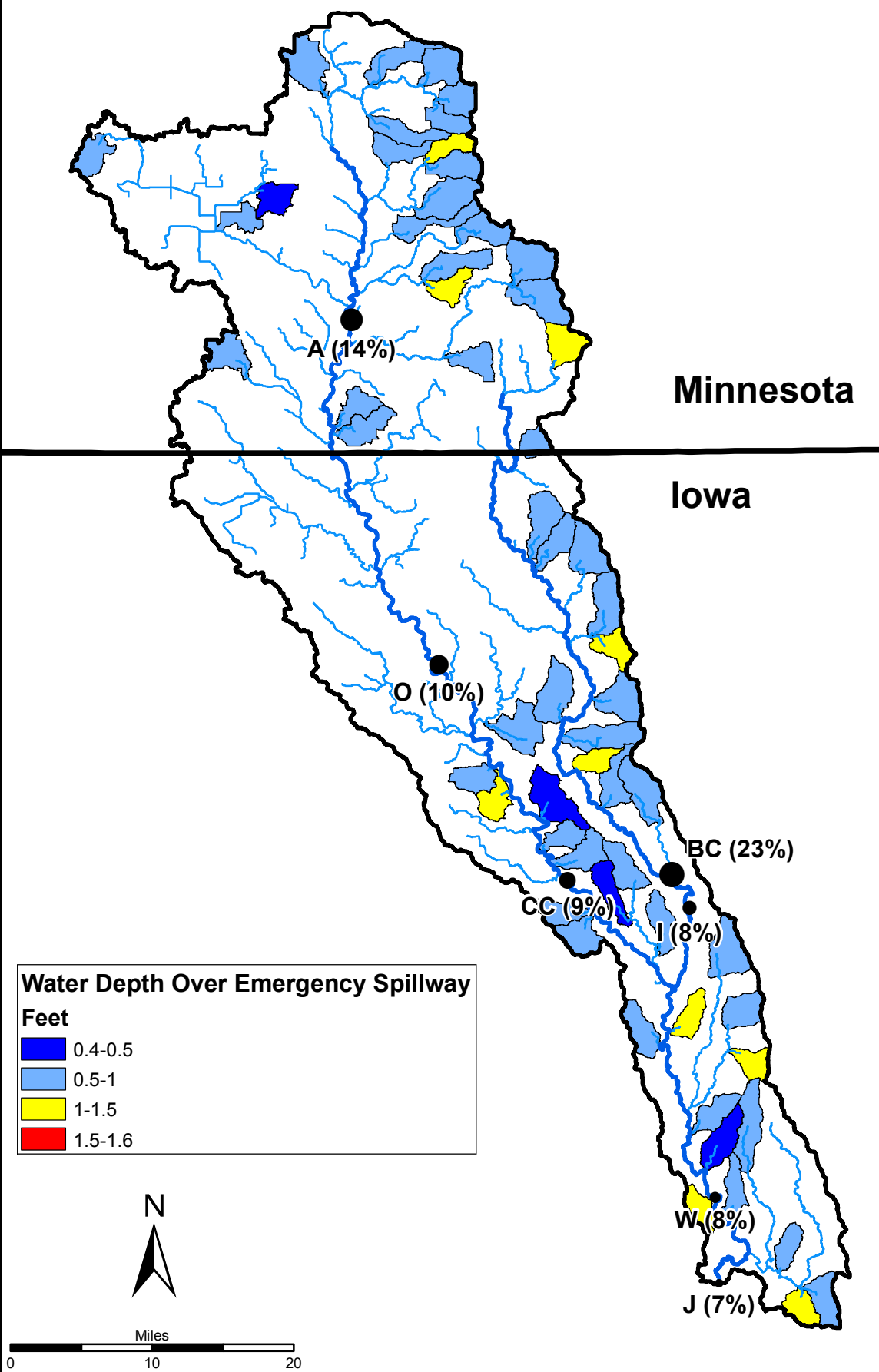 performance is shown.

Date: May 2014

By: Chad Drake

Data Sources:

Figure A-23

Appendix B – Calibration and Validation

Calibration

This appendix details the calibration and validation 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 calibration and validation historical storms.

i. 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 t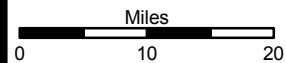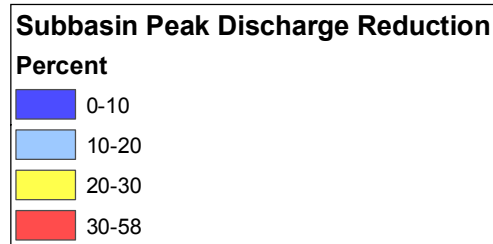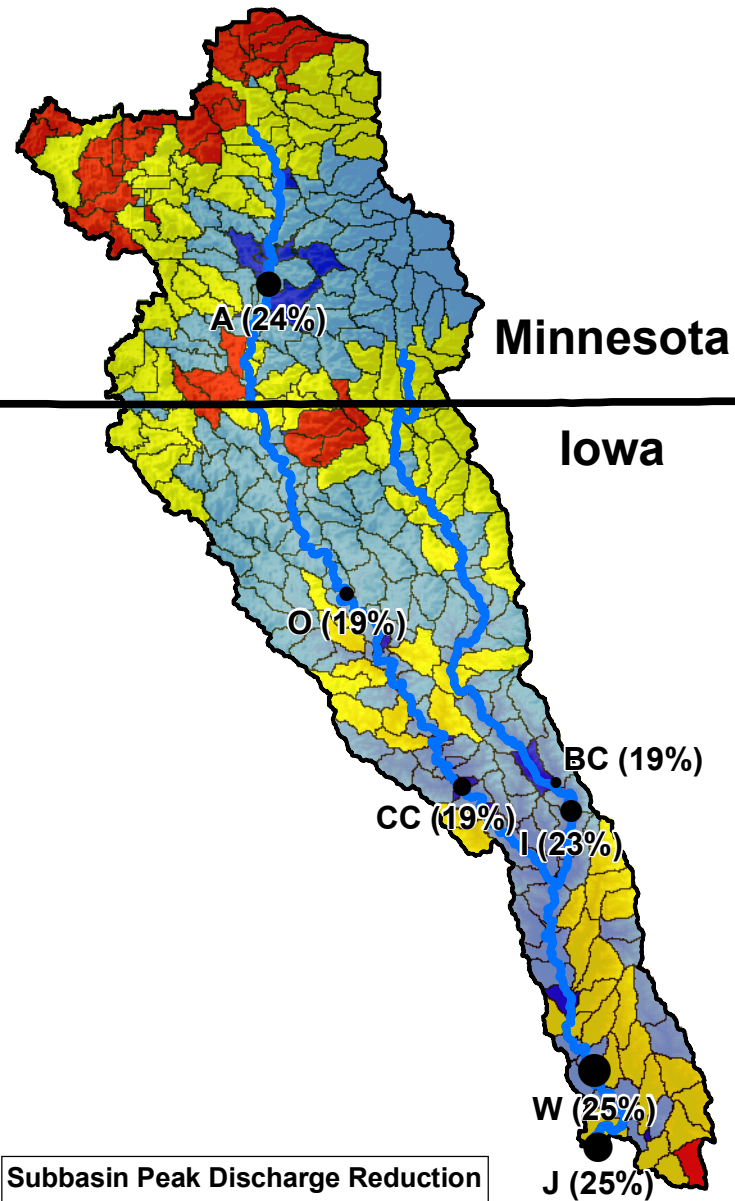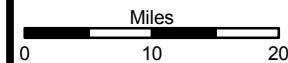he model parameters described in section 3b to best match the simulated hydrograph to the observed discharge time series at each USGS stage/discharge gage location. Model performance was evaluated based on how well the simulated hydrograph peak discharge, time of peak discharge, total runoff volume, 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 Curve Number (CN) methodology, and Clark Unit Hydrograph parameters were adjusted to account for the effects of tile drainage on the basin hydrology.

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 10th, 50th, and 90th 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 B.1.

Table B.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)	10th	< 1.4	$CN(I) = \dfrac{4.2CN(II)}{10 - 0.058CN(II)}$
II (normal)	50th	1.4-2.1	---
III (wet)	90th	> 2.1	$CN(III) = \dfrac{23CN(II)}{10 + 0.13CN(II)}$

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 B-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 2.2 and shown in Figure 2.6, 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 B.1). The basin average, five-day rainfall total CDF developed for the Upper Cedar River Watershed is shown in Figure B.1.

Evident from Figure B.1, using existing NRCS definitions for defining AMC classes (Table B.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.
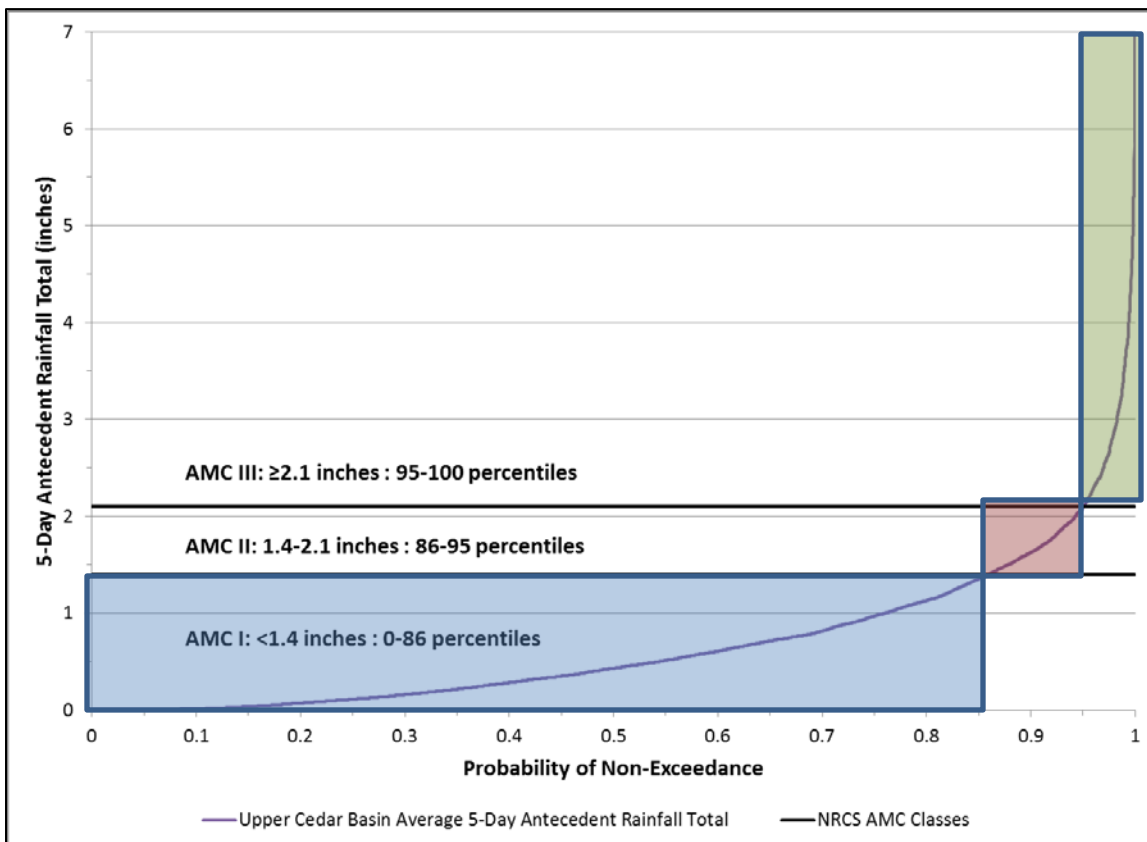
Figure B.1. Five-day rainfall total cumulative distribution function (CDF) for the Upper Cedar River Watershed. The CDF was computed using the daily basin average precipitation computed between May and September of each year over a 65 year period (1948-2013).

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 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 (10th, 50th, and 90th 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 B.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 B.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 B.2. The finer dashed line shows how CNs would be defined into one of three classes if the NRCS methodology (Table B.1) were being used (same piecewise curve shown in Figure B.1).
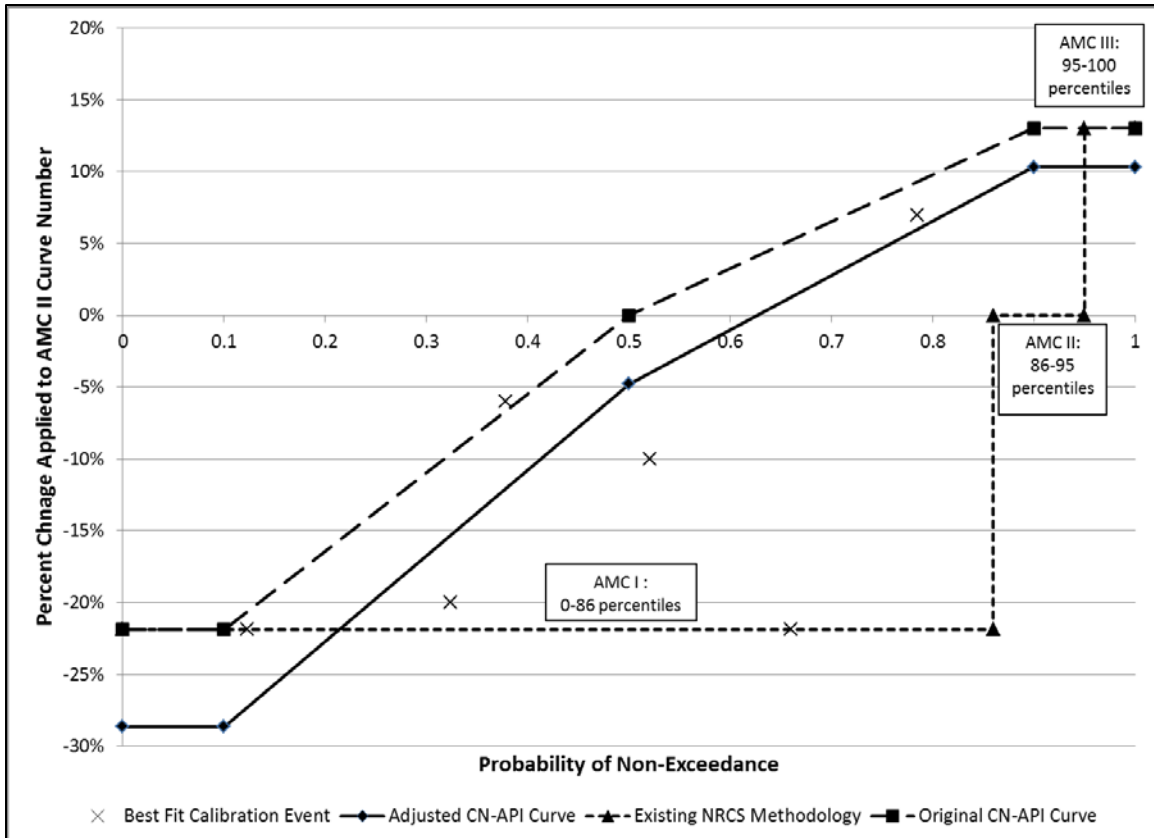
Figure B.2. Accounting for antecedent moisture conditions in the Upper Cedar River Watershed HMS model through use of the antecedent precipitation index (API). Precipitation gage records were used to calculate the API prior to each historical storm event and a corresponding percent change in Curve Number was applied to each subbasin Curve Number to account for the initial soil moisture conditions.

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.

ii. Summary of Calibrated HMS Model Parameters

Table B.2. summarizes the final set of HMS model parameters determined through calibration for the Upper Cedar River Watershed.

Table B.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 3.2	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	

iii. 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 1a). 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.

September 2004

The September 14-22, 2004 storm was characterized by a basin average 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 B.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 B.3.

Figure B.3. Hydrograph comparisons for the September 2004 calibration storm.

August 2007

The August 18-27, 2007 storm was characterized by a basin average 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 (66th 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 B.4.

Figure B.4. Hydrograph comparisons for the August 2007 calibration storm.

July 2010

The July 22-28, 2010 storm was characterized by a basin average 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 (32nd 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 B.5.

Figure B.5. Hydrograph comparisons for the July 2010 calibration storm.

May 1-8, 2013

The May 1-8, 2013 storm was characterized by a basin average 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%. 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 B.6.
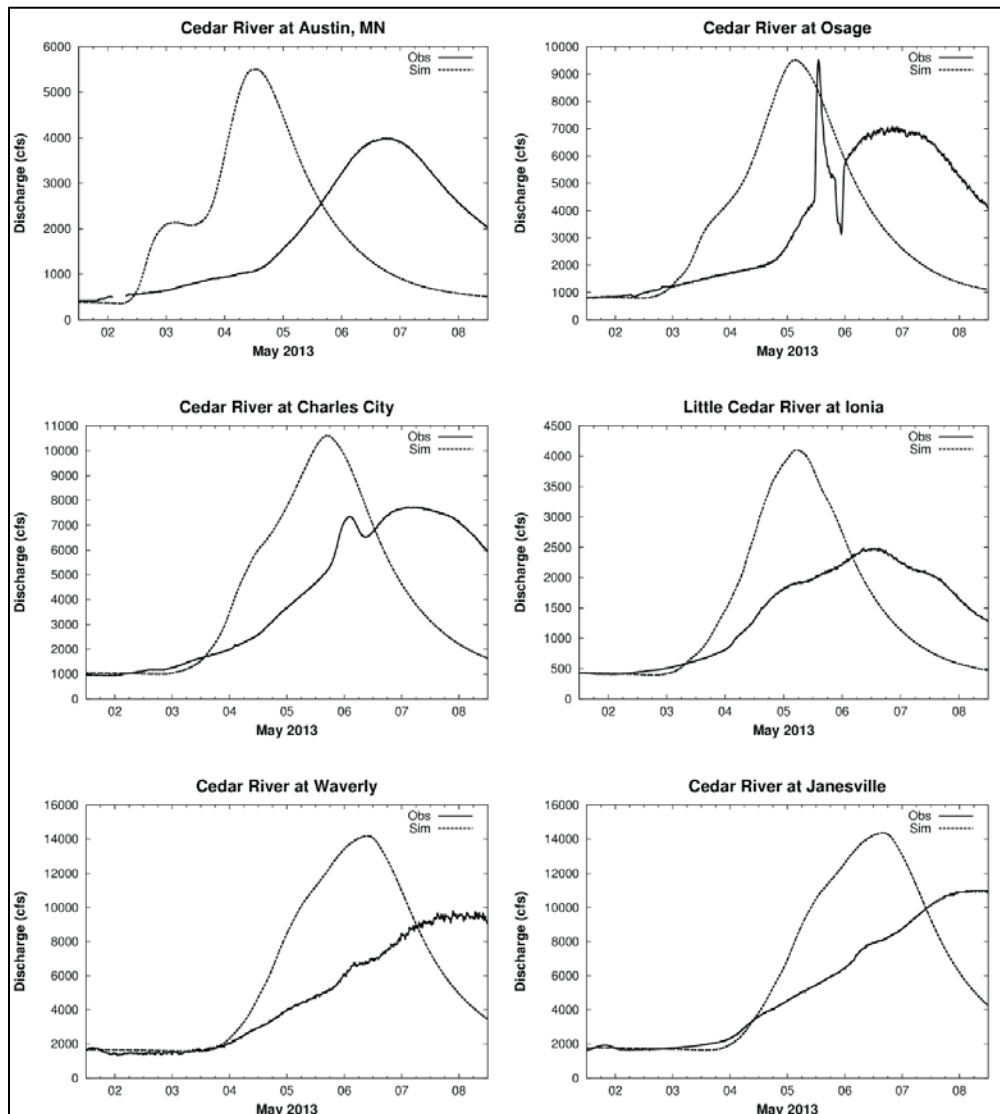
Figure B.6. Hydrograph comparisons for the May 1-8, 2013 calibration storm.

May 16-25, 2013

The May 16-25, 2013 storm was characterized by a basin average 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 B.7.
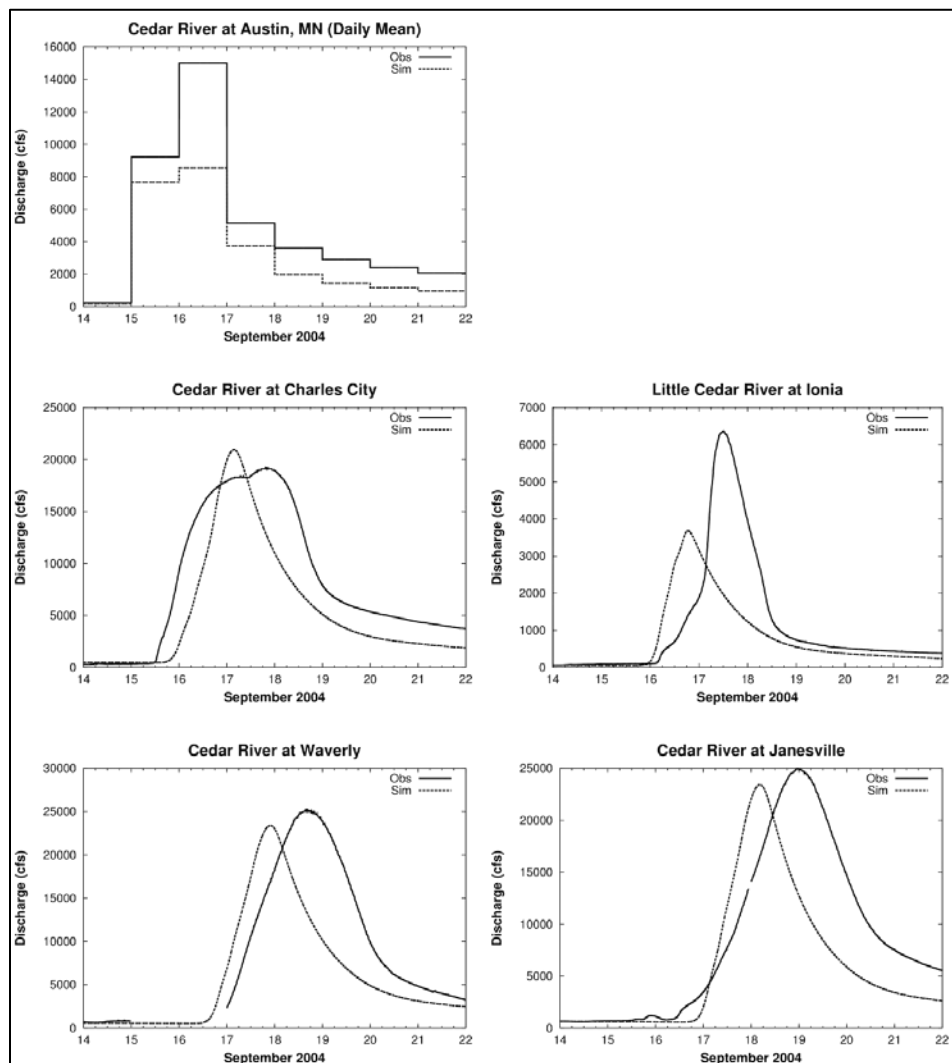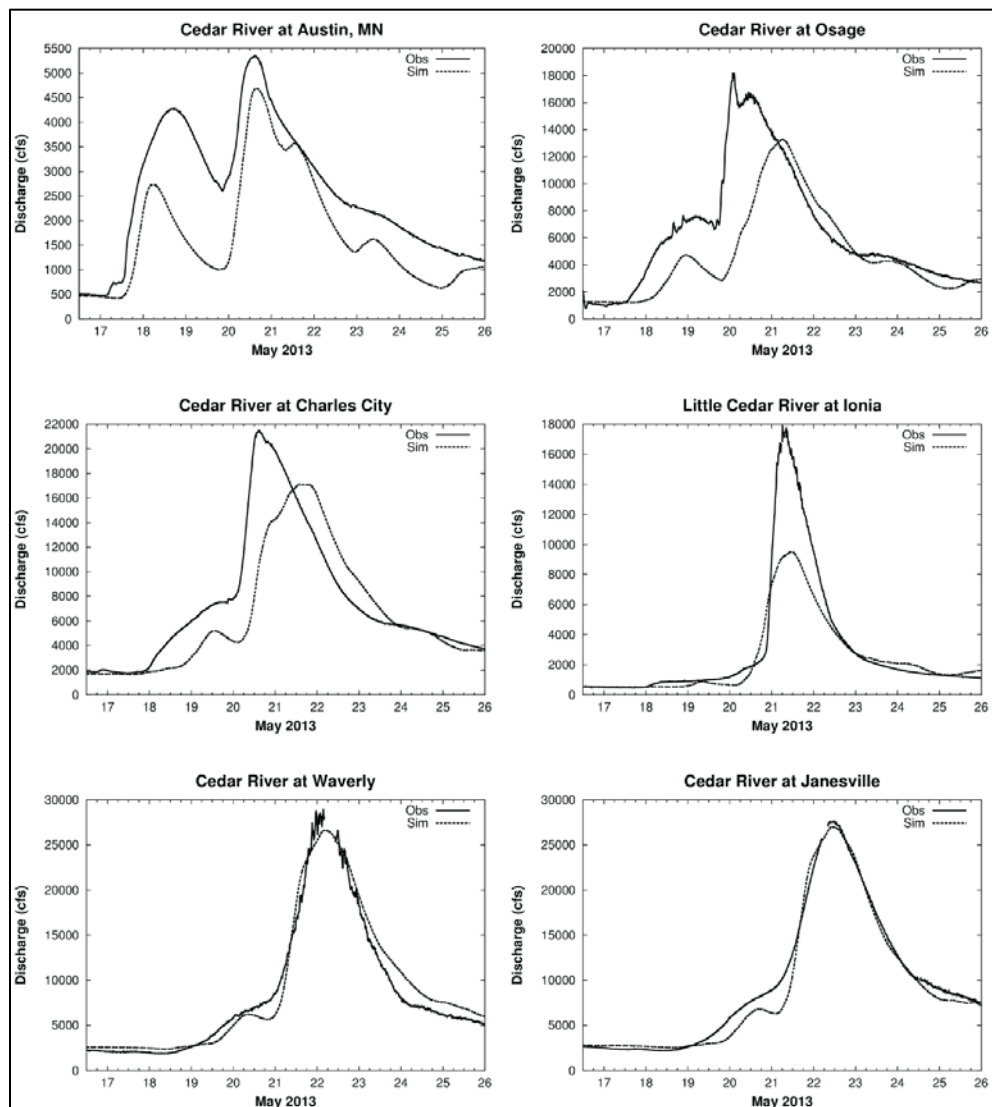

Figure B.7. Hydrograph comparisons for the May 16-25, 2013 calibration storm.

June 2013

The June 10-19, 2013 storm was characterized by a basin average 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 (79th 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 B.8.

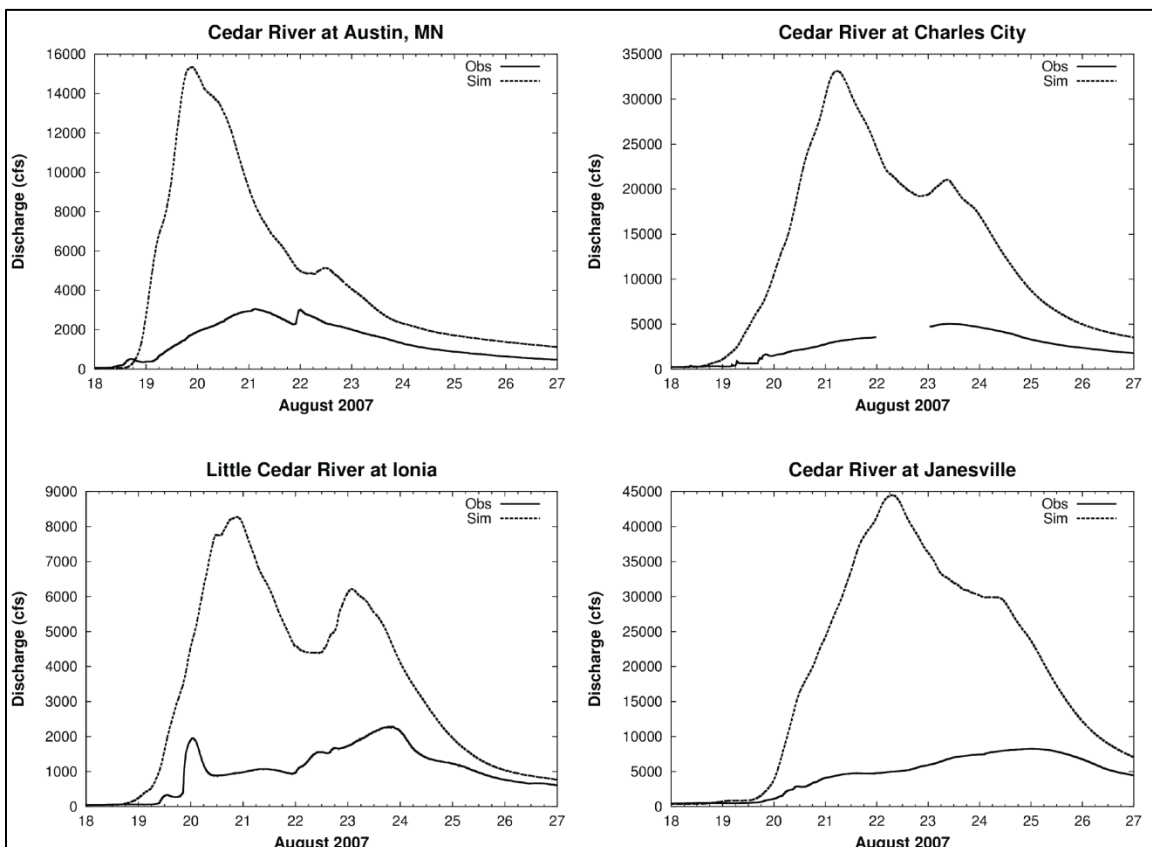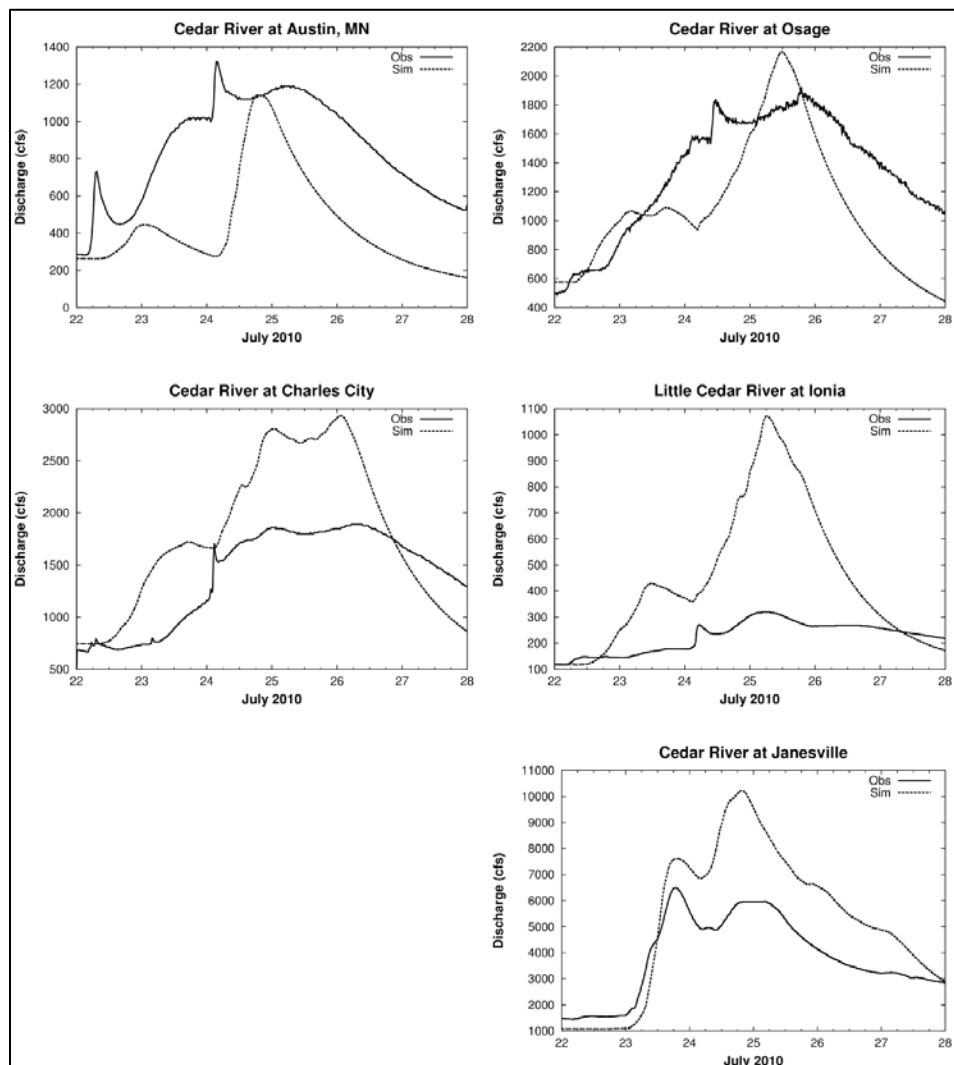Figure B.8. Hydrograph comparisons for the June 2013 calibration storm.

iv. 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 the previous section, model performance varied for the six calibration storms.

Figure B.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 B.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 B.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.
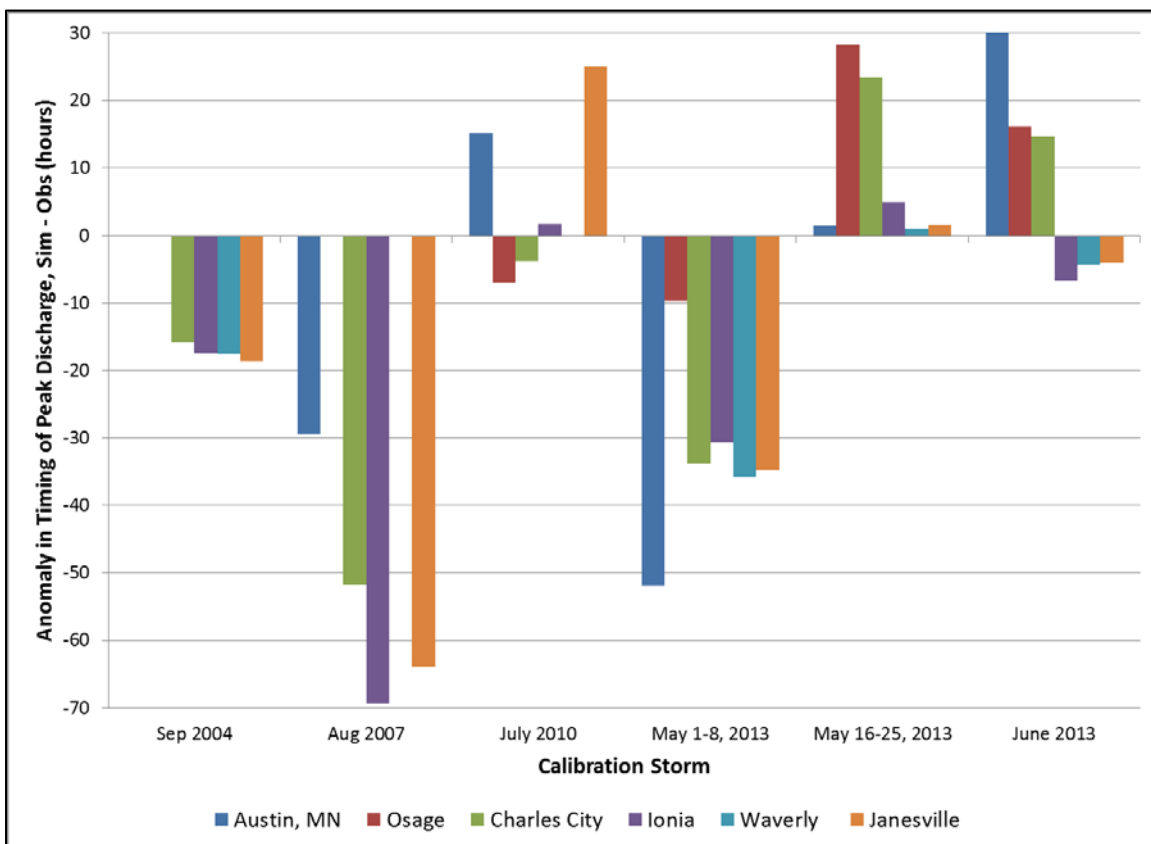
Figure B.10. Anomalies in time to peak discharge between simulated and observed hydrographs at the operational USGS stage/discharge gage locations in the Upper Cedar River Watershed for the six calibration storms. A positive anomaly indicates the simulated time to peak was later than observed (and vice versa).

Figure B.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 B.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.
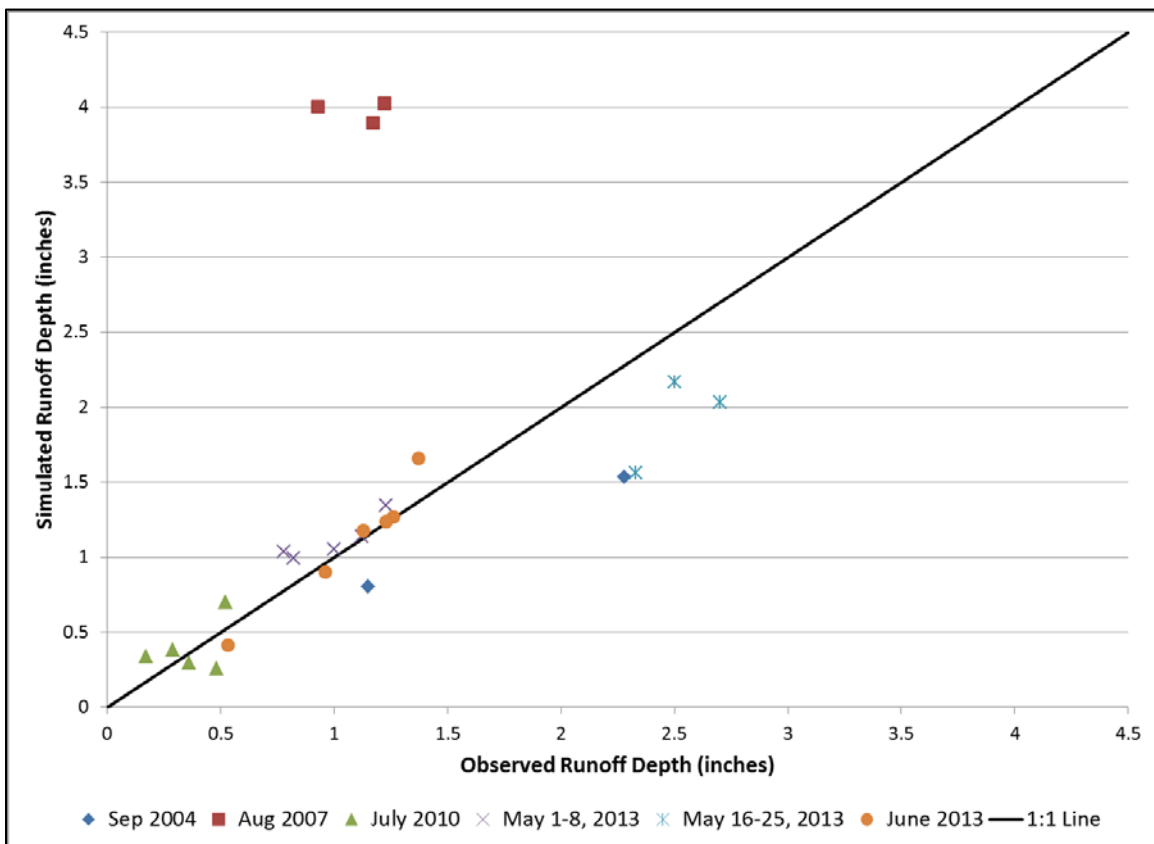
Figure B.11. Comparison of simulated and observed runoff depths at operational USGS stage/discharge gage locations in the Upper Cedar River Watershed for the six calibration storms.

v. Calibration Summary

This section 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 section.

Validation

Four historical storms were considered for model validation. Results for these storms are presented and discussed.

i. Validation Storm Results

May 2004

The May 20-28, 2004 validation storm was characterized by a basin average 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 B.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 B.12.

Figure B.12. Hydrograph comparisons for the May 2004 validation storm.

September 2010

The September 22 – October 1, 2010 validation storm was characterized by a basin average 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 B.13.

Figure B.13. Hydrograph comparisons for the September 2010 validation storm.

July 2011

The July 14-22, 2011 validation storm was characterized by a basin average 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 B.14.

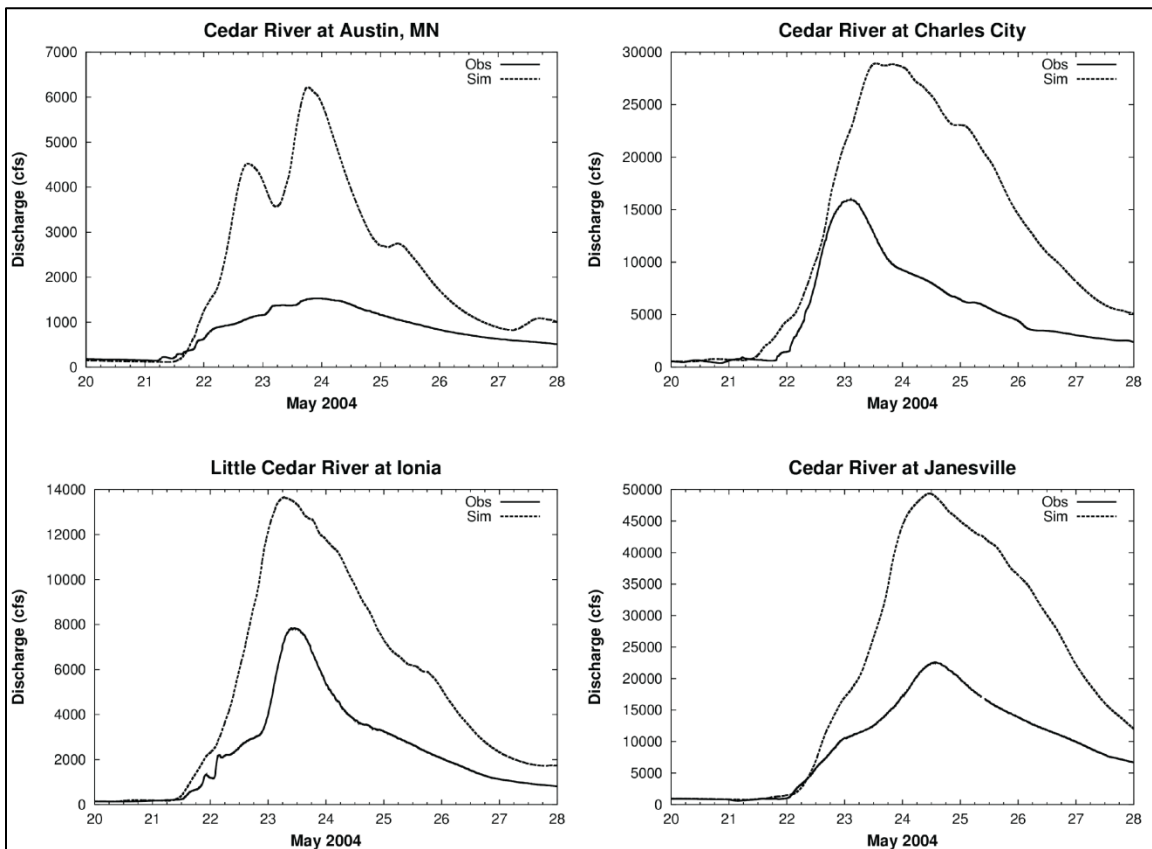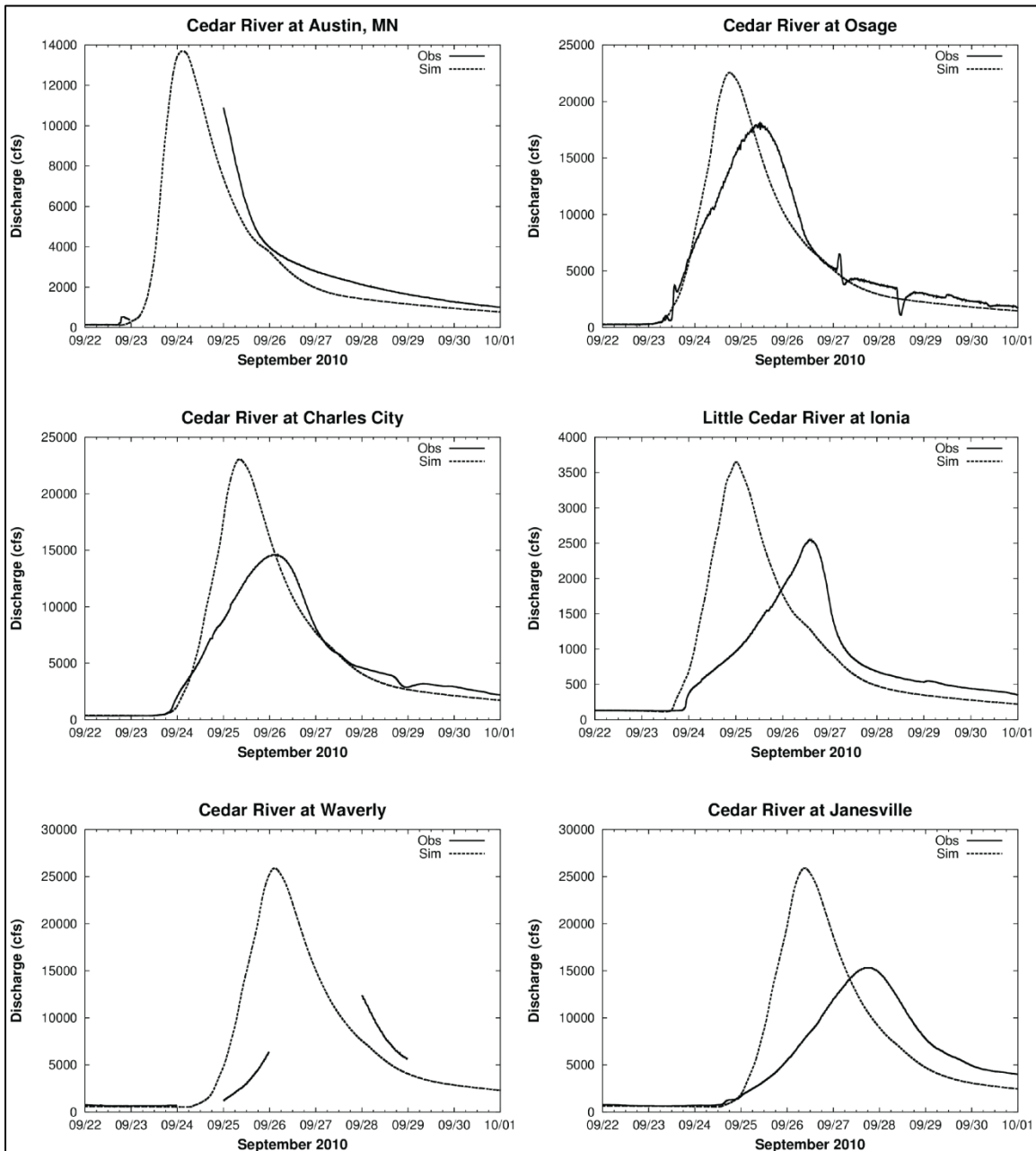Figure B.14. Hydrograph comparisons for the July 2011 validation storm.

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 B.15.
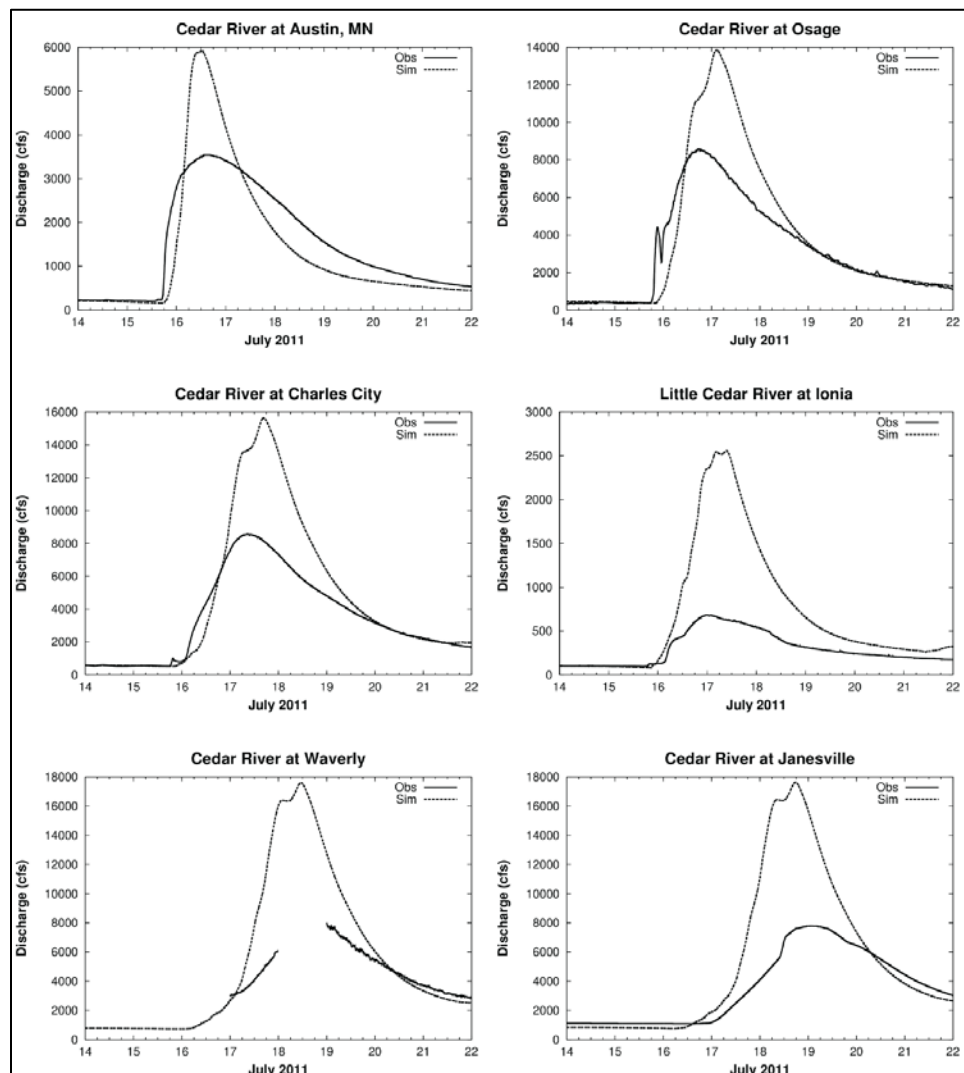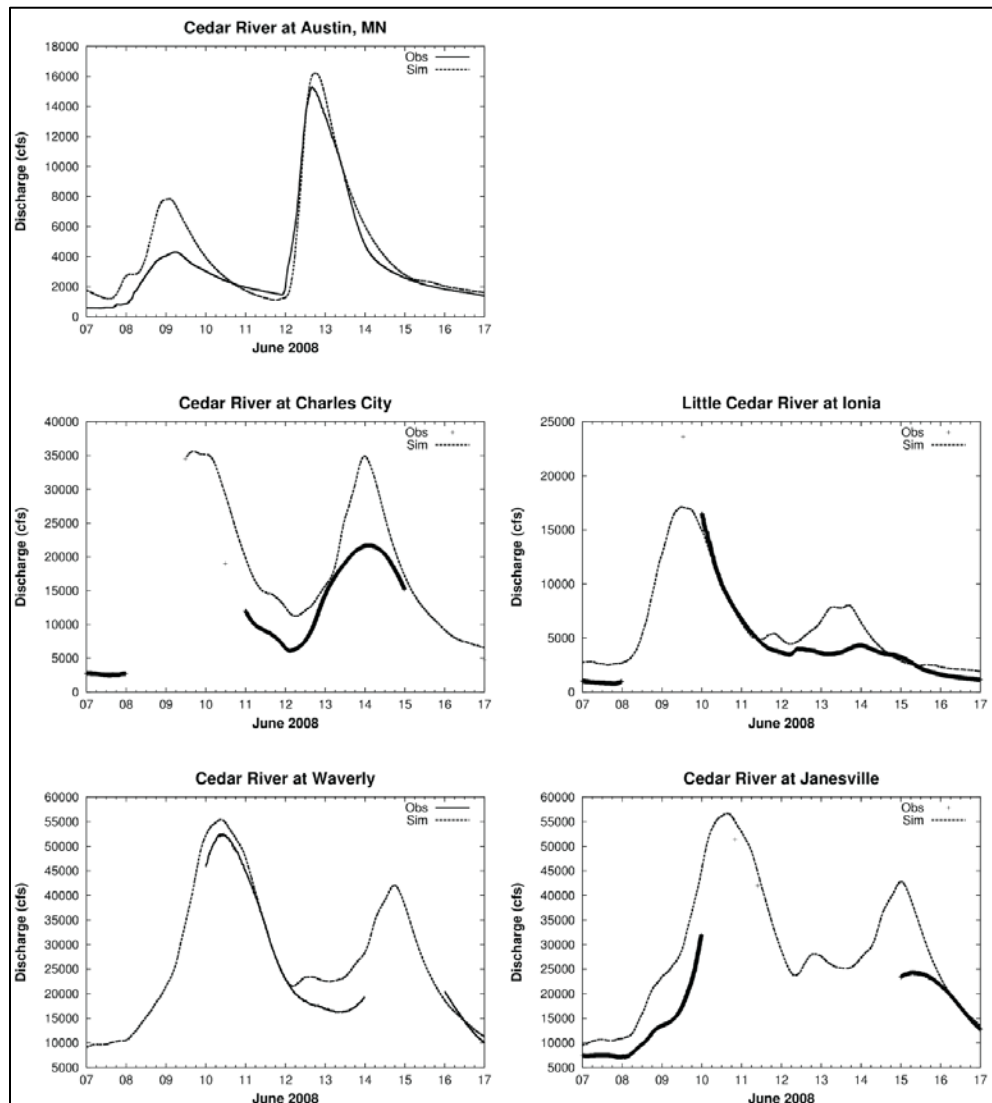

Figure B.15. Hydrograph comparisons for the June 2008 validation storm.

ii. 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 B.16 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 B.16. 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 B.17 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 B.16, 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.
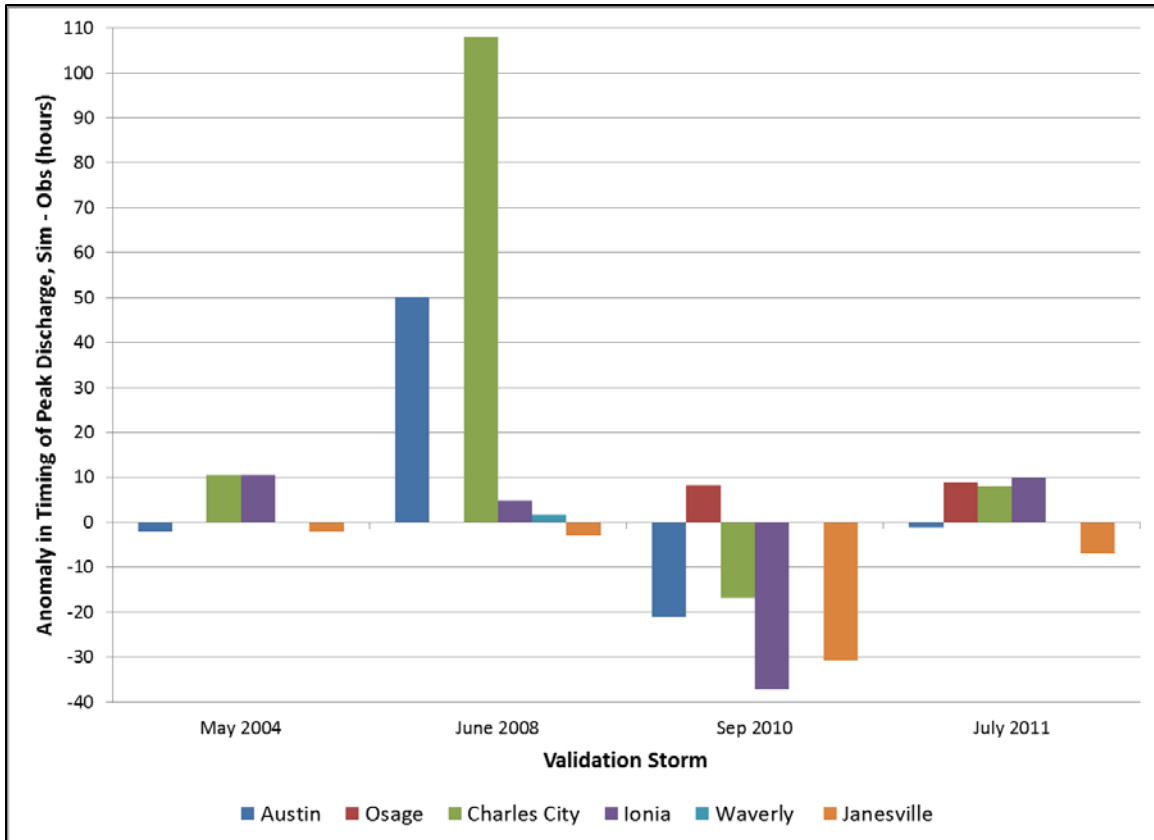
Figure B.17. Anomalies in time to peak discharge between simulated and observed hydrographs at the operational USGS stage/discharge gage locations in the Upper Cedar River Watershed for the four validation storms. A positive anomaly indicates the simulated time to peak was later than observed (and vice versa).

Figure B.18 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.
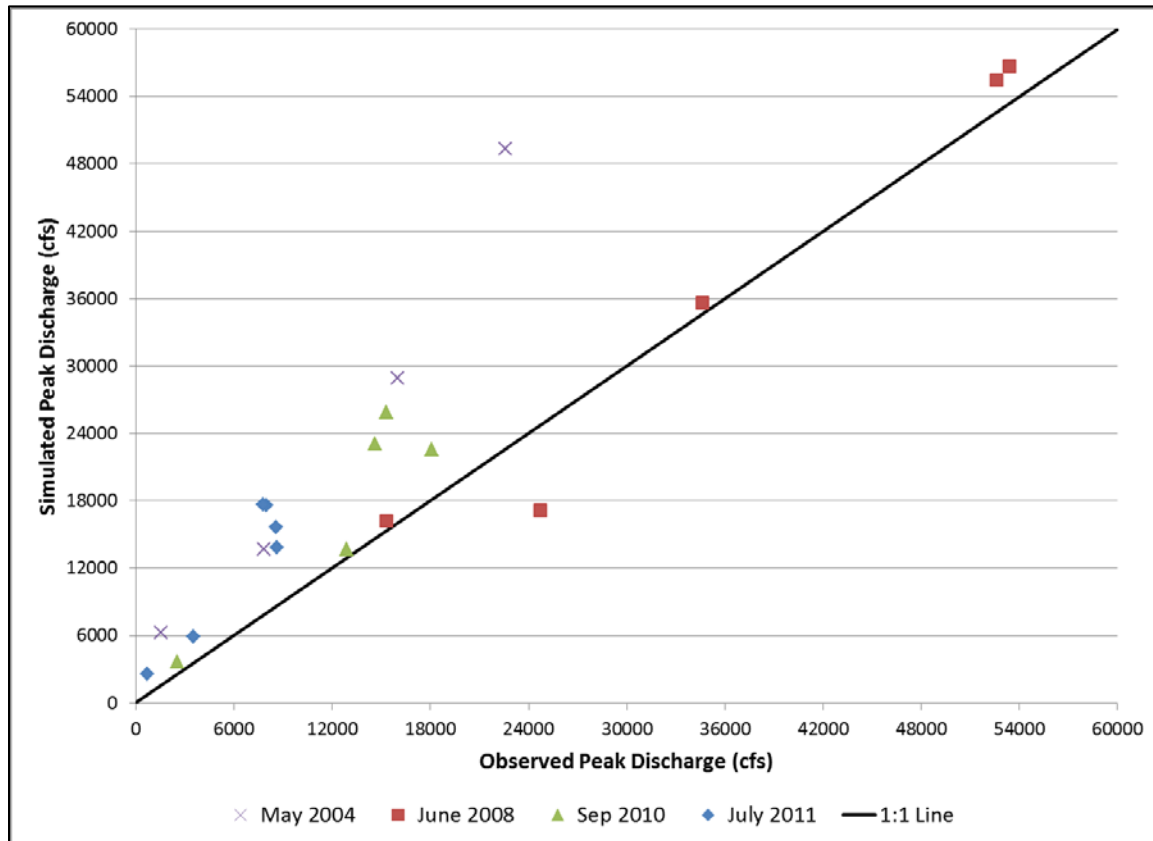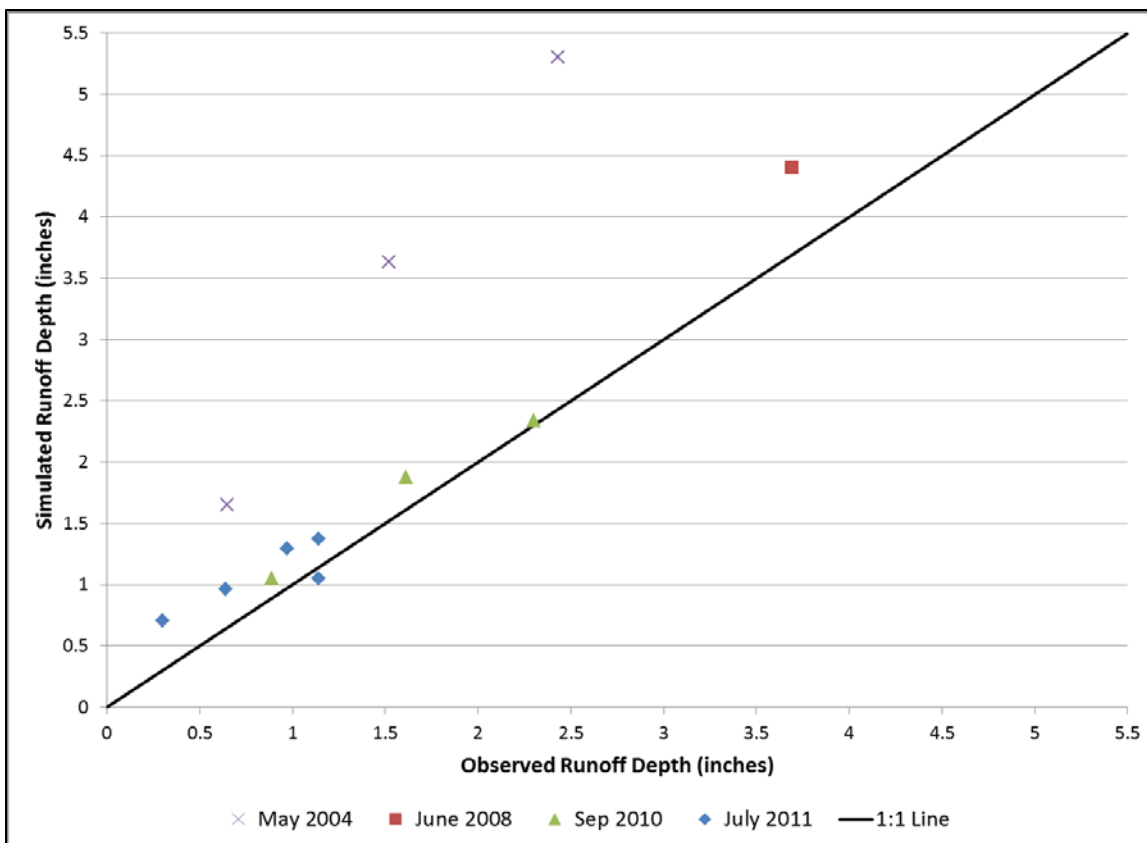
Figure B.18. Comparison of simulated and observed runoff depths at operational USGS stage/discharge gage locations in the Upper Cedar River Watershed for the four validation storms.

iii. Validation 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.

Appendix C – References

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

Beck, H.E., De Jeu, R.A., Schellekens, J., Van Dijk, A.I, and Bruijnzeel, L.A. "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: 250-259, 2009.

Burkart, M. "The Hydrologic Footprint of Annual Crops." A Watershed Year: Anatomy of the Iowa Floods of 2008, C. Mutel (editor), University of Iowa Press, Iowa City, 77-85, 2010.

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

Frans, C., Istanbulluoglu, E., Mishra, V., Munoz-Arriola, F., and Lettenmaier, D.P. "Are climatic or land cover changes the dominant cause of runoff trends in the Upper Mississippi River Basin?" Geophysical Research Letters, 40 (17), 2013.

Hjelmfelt, A.T. Jr, Kramer, K.A., and Burwell, R.E. "Curve numbers as random variables," In Proceedings International Symposium on Rainfall-Runoff Modeling, Singh VP (ed.). Water Resources Publication: Littleton, CO; 365-373, 1982.

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

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

Hutchinson, K.J., and Christiansen, D.E., "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," 5002, 36, 2013.

Iowa DOT. "Ch. 10: Stormwater Discharge Permits," Iowa DOT Design Manual, Iowa DOT, 5, 2013.

Jones, C.S. and Schilling, K.E. "From Agricultural Intensification to Conservation: Sediment Transport in the Raccoon River, Iowa, 1916-2009," Journal of Environmental Quality, 40, 2011.

Knox, J.C. "Agricultural Influence on Landscape Sensitivity in the Upper Mississippi River Valley," Catena 42, 193-224, 2001.

Kull, Daniel W., and Arlen D. Feldman. "Evolution of Clark's Unit Graph Method to Spatially Distribute Runoff." Journal of Hydrologic Engineering 3: 9-19, 1998.

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

Meierdiercks, K.L., Smith, J.A., Baeck, M.L. and Miller, A.J. "Analyses of Urban Drainage Network Structure and its Impacts on Hydrologic Response," Journal of American Water Resources Research, 14(4), 416-424, 2010.

Mejia, Ai.I. and Moglen, G.E. "Impact of the Spatial Distribution of Imperviousness on the Hydrologic Response of an Urbanizing Basin," Hydrologic Processes, 24, 3353-3373, 2010.

Mora, C., Abby G. Frazier, A.g., Longman, R.J., Dacks, R.S., Walton, M.M., Tong, E.J., Sanchez, J.J., Kaiser, L.R., Stender, Y.O., Anderson, J.M., Ambrosino, C.M., Fernandez-Silva, I., Giuseffi, L.M. and Giambelluca, T.M. "The projected timing of climate departure from recent variability," Nature, 502(7470), 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>.

Papanicolaou, A.N., Wacha, K.M., and Wilson, C.G. "Quantifying the collective effects of rainfall- and tillage-induced erosion on an SOC budget in Southeastern Iowa using WEPP and CENTURY," Soil & Tillage Research, submitted.

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.

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

Petersen W. "The Hydrology of Urban Landscape," Chapter in A Watershed Year: Anatomy of the Iowa Floods of 2008, C. Mutel (editor), University of Iowa Press, Iowa City, IA. 87-95, 2010.

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

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

Ryberg, K.R., Lin, W. and Vecchia, A.V. "Impact of Climate Variability on Runoff in the North Central United States," Journal of Hydrologic Engineering, doi: 10.1061/(ASCE) HE.1943-5584.00000775, 2012.

Sayre, R. "The Dam and the Flood: Cause of Cure," Chapter in A Watershed Year: Anatomy of the Iowa Floods of 2008, C. Mutel (editor), University of Iowa Press, Iowa City, 103- 109, 2010.

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

Schilling, K. "Patterns of Discharge and Suspended Sediment Transport in the Walnut and Squaw Creek Watersheds, Jasper County, Iowa: Water Years 1996-1998," Iowa Department of Natural Resources, Geological Survey Bureau, Iowa City, IA, 2000.

Schilling, K.E., Chan, K-S., Jha, M.K., Zhang, Y-K., and Gassman, P.W. "Impact of Land Use and Land Cover Change on the Water Balance of a Large Agricultural Watershed: Historical Effects and Future Directions," Water Resources Research, 44, doi: 10.1029/2007WE006644, 2008.

Schilling, K.E., Chan, K-S., Liu, K-S., and Zhang, Y-K., Gassman, P.W. "Quantifying the Effect of Land Use Land Cover Change on Increasing Discharge in the Upper Mississippi River," Journal of Hydrology 387, 343-345, 2008.

Schottler, S.P., Ulrich, J., Belmont P., Moore, R., Lauer, J.W., Engstrom, D.R., Almendinger, J.E. "Twentieth Century Agricultural Drainage Creates More Erosive Rivers," Hydrological Processes, doi:10.1002/hyp.9738, 2013.

"Storm Water Discharge Permits." Iowa DOT Design Manual; Chapter 10. Ames, IA: Iowa DOT, 4-5, 2013.

Takle, E.S. "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, 111-116, 2010.

Thompson, J. "Wetlands Drainage, River Modification, and Sectoral Conflict in the Lower Illinois Walley," Southern Illinois University, Carbondale, IL, 2003.

Urban, M.A. and Rhoads, B.L. "Catastrophic Human-Induced Change in Stream-Channel Planform and Geometry in an Agricultural Watershed, Illinois, USA", Annals of the Association of American Geographers, 93(4), 783-796, 2003.

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

Wehmeyer, L.L., Weirich, F.H., and Cuffney, T.F. "Effect of Land Cover Change on Runoff Curve Number Estimation in Iowa, 1832–2001," Ecohydrology. 4, 315-321, 2011.

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>.

Winsor, R. "Artificial Drainage of East Central Illinois 1820-1920," PhD Thesis, Department of Geography, University of Illinois, Urbana-Champaign, IL, 1975.

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

Yiping Wu, Y., Liu, S., Sohl, T.L., Young, C.J. "Projecting the land cover change and its environmental impacts in the Cedar River Basin in the Midwestern United States," Environmental Research Letters, 8(2), 2013.

Zhang, Y. K. and Schilling, K.E. "Effects of Land Cover on Water Table, Soil Moisture, Evapotranspiration, and Groundwater Recharge: a Field Observation and Analysis," Journal of Hydrology, 319(1-4), 328-338, 2006.

Zhang, Y. K. and Schilling, K.E. "Increasing Streamflow and Baseflow in Mississippi River since the 1940s:Effects of Land-use Change," Journal of Hydrology, 325, doi:10.1016/j.jhydrol. 200509.033, pp 412-422, 2006.