IIHR Report
No. 508

October, 2016

Iowa Watersheds Project
Phase II Report

# Otter Creek Watershed

## *Project Evaluation*

*Prepared by:*
Iowa Flood Center / IIHR—Hydroscience & Engineering

*Sponsored by:*
Turkey River Watershed Management Authority

IIHR—Hydroscience & Engineering
The University of Iowa
C. Maxwell Stanley Hydraulics Laboratory
Iowa City, Iowa 52242

# Iowa Watersheds Project Phase II: Otter Creek Watershed Evaluation of Project Performance

October 2016

IIHR Technical Report No. 508

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

## Acknowledgements

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

Iowa Department of Natural Resources
Natural Resources Conservation Service
Northeast Iowa Resource Conservation and Development
Turkey River Technical Committee
Turkey River Watershed Management Authority

**Cover photograph.** Otter Creek, Fayette County. Photograph by Iowa Flood Center.

# Table of Contents

# List of Figures

# List of Tables

# 1. 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 — formed to tackle local challenges with a unified watershed approach.

In 2010, Iowa received $8.8 million from the U.S. Department of Housing and Urban Development (HUD) to assist with ongoing disaster recovery programs following these devastating floods. The Iowa Flood Center (IFC), a unit of the University of Iowa's IIHR—Hydroscience & Engineering, led an effort called the Iowa Watersheds Project. Its goal was to evaluate and implement flood reduction methods in Iowa watersheds. The Turkey River Watershed, in collaboration with the Turkey River Watershed Management Authority and Northeast Iowa RC&D, was one of four watersheds (Figure 1.1) selected to demonstrate a watershed approach for flood risk reduction.

In Phase I of the project, the Iowa Flood Center carried out a hydrologic assessment of the Turkey River Watershed (IFC, 2014). The assessment characterized the water cycle of the Turkey River using historical observations. It also investigated trends observed for the Turkey River within the broader context of changes in land use and weather patterns. Researchers implemented a hydrologic model of the Turkey River, using the Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS), to identify areas in the watershed with high runoff potential and to run simulations to help understand the potential impact of alternative flood mitigation strategies in the watershed. For scenario development, researchers focused on understanding the impacts of: (1) increasing infiltration in the watershed; and (2) implementing a system of distributed storage projects (ponds) across the landscape.

We are adding modeling results and scenario simulations from the Phase I hydrologic assessments to the Iowa Watershed Decision Support System (IoWaDSS) as part of an Iowa Flood Center project funded by the U.S. Army Corps of Engineers Institute for Water Resources. The system aims to assemble data, tools, and models in one place to: (1) inform watershed stakeholders of the current status and forecasts in Iowa watersheds; (2) support the assessment of alternative strategies for sustainable watershed resources; (3) provide real-time integrated data and simulation models from multiple disciplines; and (4) facilitate collaboration and the sharing of resources and model results across agencies and communities. An IoWaDSS video tutorial can be found online (https://www.youtube.com/watch?v=-yIikldRrXA). Modeling results for the Soap Creek Watershed and the Turkey River Watershed are now available online (http://iowawatersheds.org/dev/dss_alpha/). Results for the Upper Cedar River Watershed may be added to the IoWaDSS in the future.

In Phase II of the project, researchers identified a smaller catchment (known as a HUC12 sub-watershed) for development and construction of flood mitigation projects. In collaboration with the Turkey River Watershed Management Authority, they selected the Otter Creek Watershed (Figure 1.1), where IFC researchers evaluated the flood mitigation performance of proposed projects through monitoring and detailed hydrologic modeling. The team developed small-scale hydrologic simulations for the Otter Creek Watershed using a more detailed representation of the watershed and flood mitigation strategies than that which was used in Phase I. This report describes the assessment results for Phase II of the project for the Otter Creek Watershed.

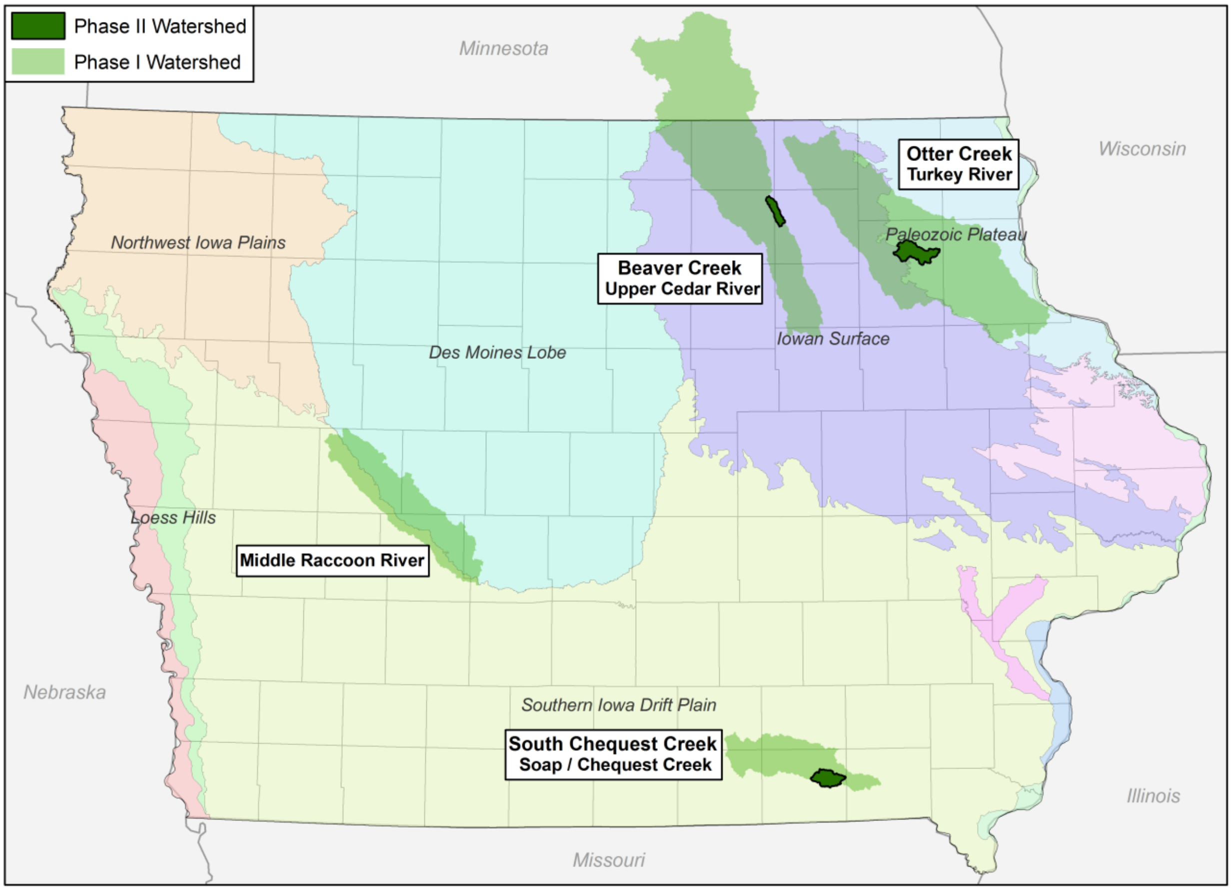


Figure 1.1. Iowa Watersheds Project, Phase I and Phase II selected watersheds.

# 2. Conditions in the Otter Creek Watershed

This chapter provides an overview of the current Otter Creek Watershed conditions, including hydrology, geology and soils, topography, and land use.

## a. Hydrology

The Otter Creek Watershed is a sub-watershed within the Turkey River Watershed as defined by the boundary of eight-digit Hydrologic Unit Code (HUC8) 07060004 (Figure 2.1). The Turkey River Watershed is located in Northeast Iowa and encompasses approximately 1,693 square miles ($mi^2$). The outlet discharges to the Mississippi River approximately six miles south of Guttenberg, Iowa. The Otter Creek Watershed, a drainage area of approximately 47 $mi^2$, is located in Fayette County, and Otter Creek's outlet discharges into the Turkey River at Elgin, Iowa.

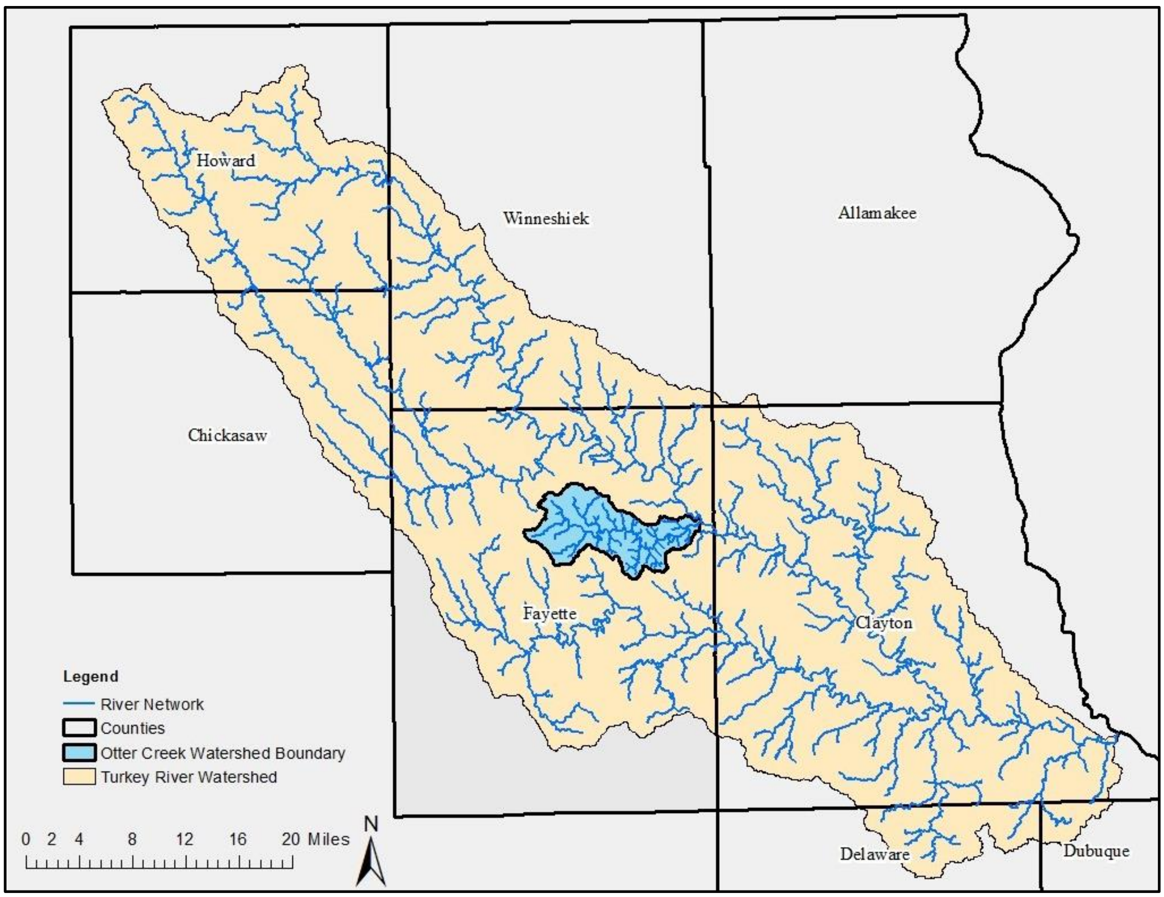


Figure 2.1. The Otter Creek Watershed drains 47.1 $mi^2$.

Average annual precipitation for this region of Northeast Iowa is roughly 36 inches (PRISM, 1981–2010), with about 70% of the annual precipitation falling as rain during the months of April–September. During this period, thunderstorms capable of producing torrential rains are possible, with the peak frequency of such storms occurring in June. Northeast Iowa has experienced increased variability in annual precipitation since 1975, along with a general increase in the amount of spring rainfall (U.S. Department of Agriculture — Iowa State University, 2011).

## b. Geology and Soils

The Otter Creek Watershed is located within two identified landform regions, the Iowan Surface and Paleozoic Plateau, each of which has a unique influence on the rainfall-runoff characterization of the watershed. The Iowan Surface of Northeast Iowa is dominated by gently rolling terrain created during the last period of intense glacial cold, 21,000–16,000 years ago. Hilly landscapes succumbed to vigorous episodes of weathering and leveling as materials were loosened and moved (Iowa Geological & Water Survey, Iowa Department of Natural Resources, 2013). Approximately 20% of the watershed, which is generally located south and west of West Union, falls into the Iowan Surface landform region.

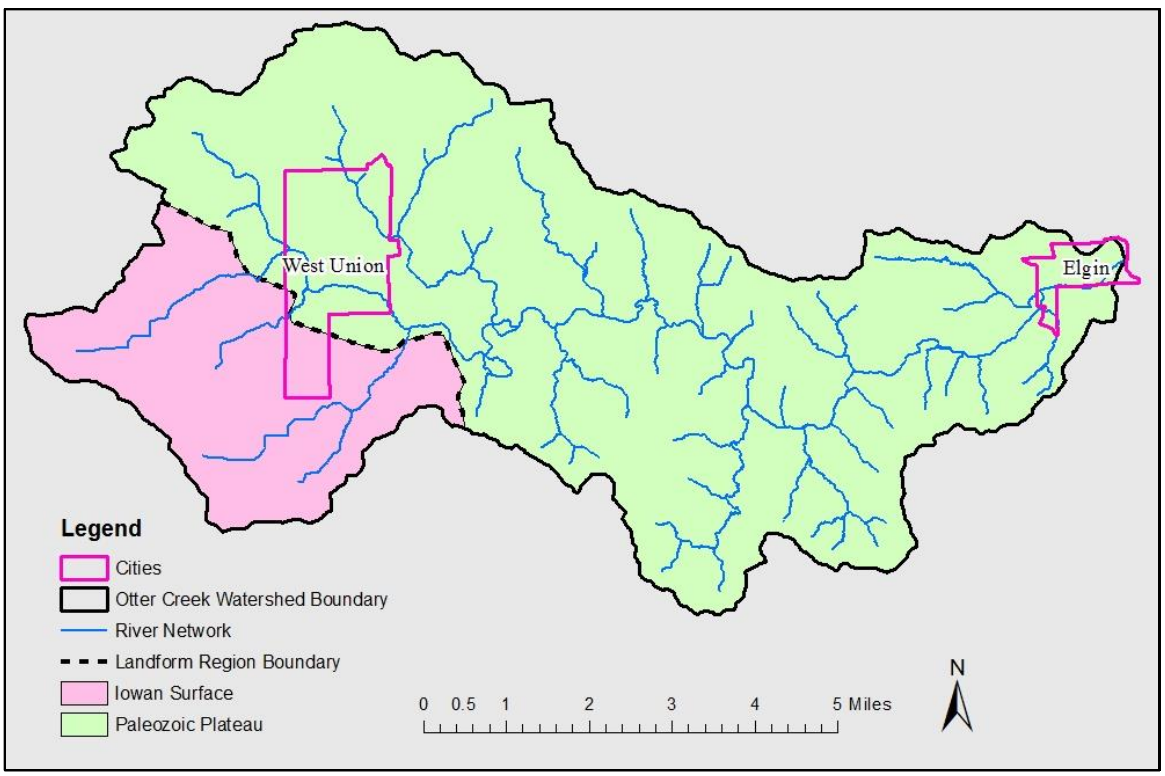


Figure 2.2. Defined Landform Regions of the Otter Creek Watershed.

In contrast, the Paleozoic Plateau is characterized by narrow valleys deeply carved into sedimentary rock. The rock layers vary in resistance to erosion, producing bluffs, waterfalls, and rapids. Shallow limestone coupled with the dissolving action of groundwater yields numerous caves, springs, and sinkholes (Iowa Geological & Water Survey, Iowa Department of Natural

Resources, 2013). The Iowa Department of Natural Resources (IDNR) has mapped the locations of nearly 250 sinkholes in the Otter Creek Watershed; these are shown in Figure 2.3.

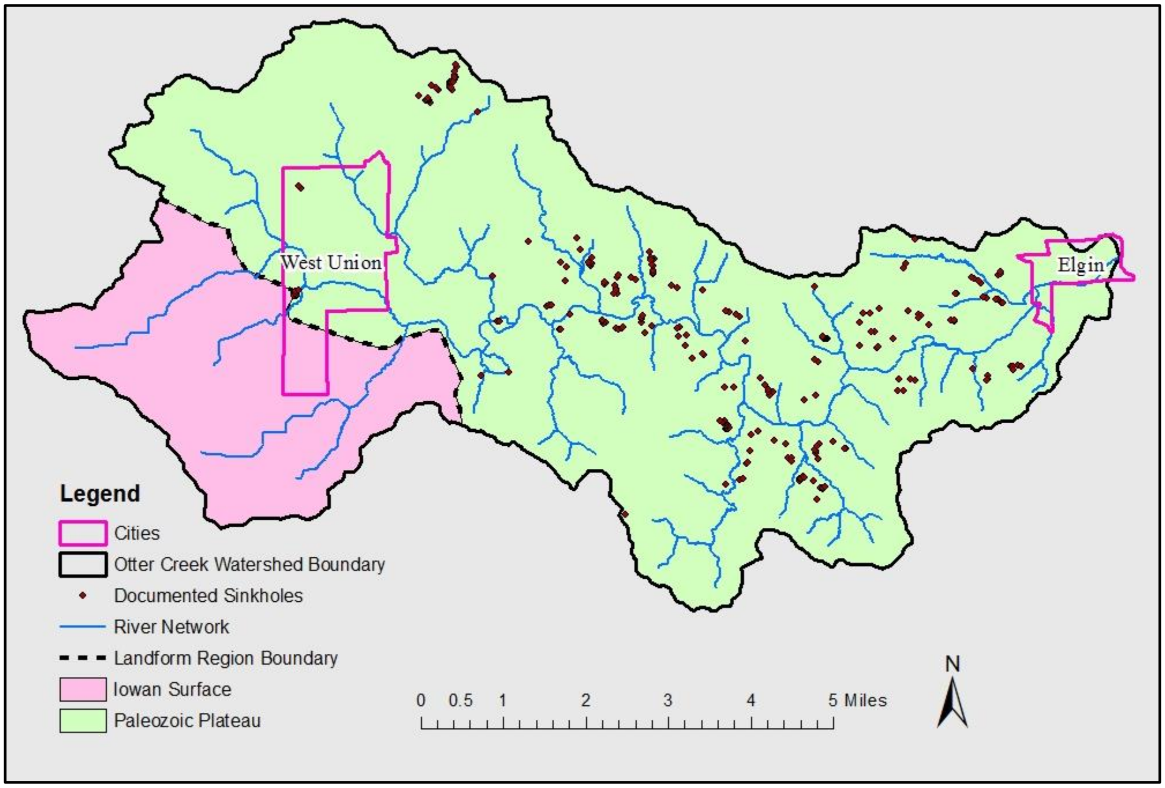


Figure 2.3. Location of sinkholes as mapped by Iowa Department of Natural Resources.

The Natural Resources Conservation Service (NRCS) classifies soils into four Hydrologic Soil Groups (HSG) 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 and D-type have the highest. In addition, dual code soil classes A/D, B/D, and C/D are assigned to certain wet soils. In the case of these soil groups, even though the soil properties may be favorable to infiltration (water passing from the surface into the ground), a shallow groundwater table (within 24 inches of the surface) typically prevents much water from doing so. 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, but the higher runoff potential of a D-type soil if it is not. Complete descriptions of the Hydrologic Soil Groups can be found in the *USDA-NRCS National Engineering Handbook*, Part 630 – Hydrology, Chapter 7.

The Iowan Surface consists primarily of a mix of HSG B, C, B/D, and C/D type soils, resulting in areas that range from moderate to higher runoff potential. The soils overlying the bedrock (limestone) of the Paleozoic Plateau are largely C-type soils with areas of exposed rock or very shallow soils classified as D-type over rock. These soils allow much less water to infiltrate into the ground, resulting in much higher runoff potential. Figure 2.4 shows the soil distribution of the Otter Creek Watershed per digital soils data (SSURGO) available from the USDA-NRCS Web Soil

Survey (WSS). Figure 2.5 shows the soil texture classification of the soils found within the watershed.

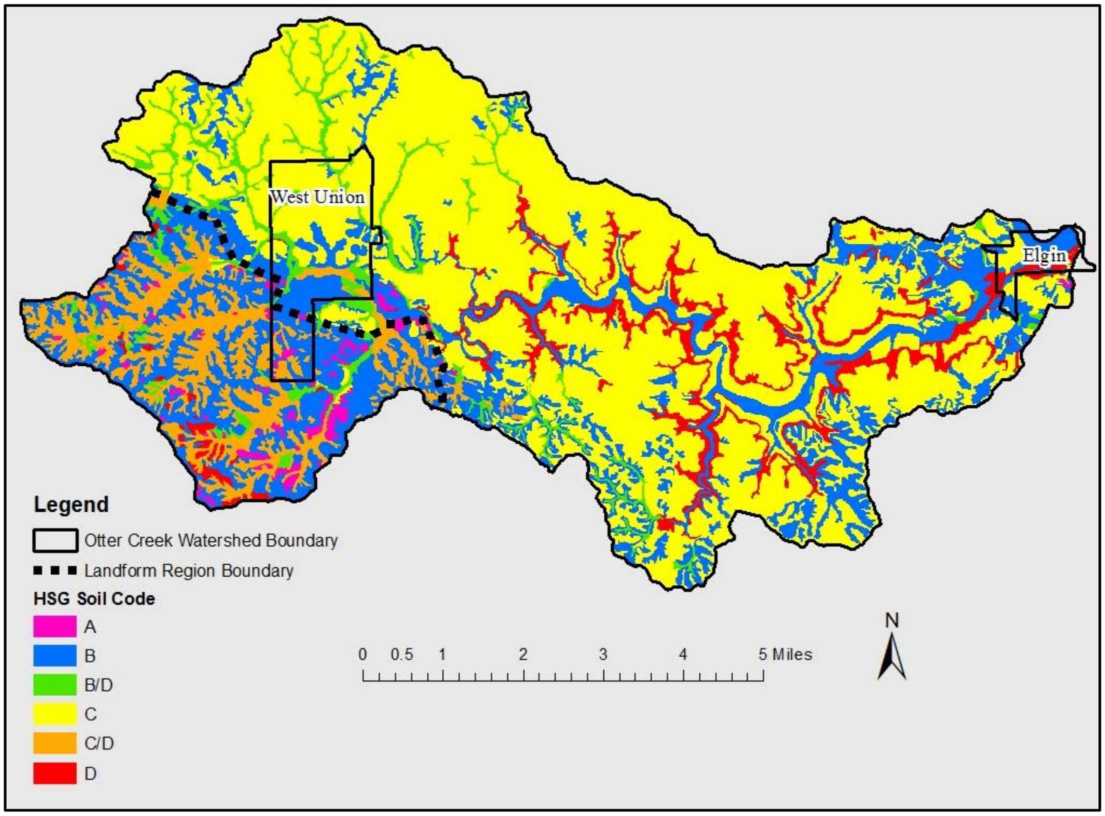


Figure 2.4. Distribution of Hydrologic Soil Groups in the Otter Creek Watershed. Hydrologic Soil Groups reflect the degree of runoff potential a particular soil has, with Type A representing the lowest runoff potential and Type D representing the highest runoff potential.

Viewing the soil distribution at this map scale is difficult, but the map does illustrate how much soils vary in space and the noticeable difference in soil types of the Iowan Surface compared to those of the Paleozoic Plateau. Table 2.1 shows the approximate percentages by area of each soil type for the Iowan Surface and the Paleozoic Plateau.

Table 2.1. Approximate Hydrologic Soil Group Percentages by Area of the Otter Creek Watershed.

| *Hydrologic Soil Group* | *Iowan Surface Approximate %* | *Paleozoic Plateau Approximate %* |
|---|---|---|
| A | 6.0 | 0.3 |
| A/D | 0 | 0 |
| B | 45.4 | 20.4 |
| B/D | 5.1 | 4.6 |
| C | 0.8 | 67.0 |
| C/D | 39.8 | 0.7 |
| D | 2.9 | 7.0 |

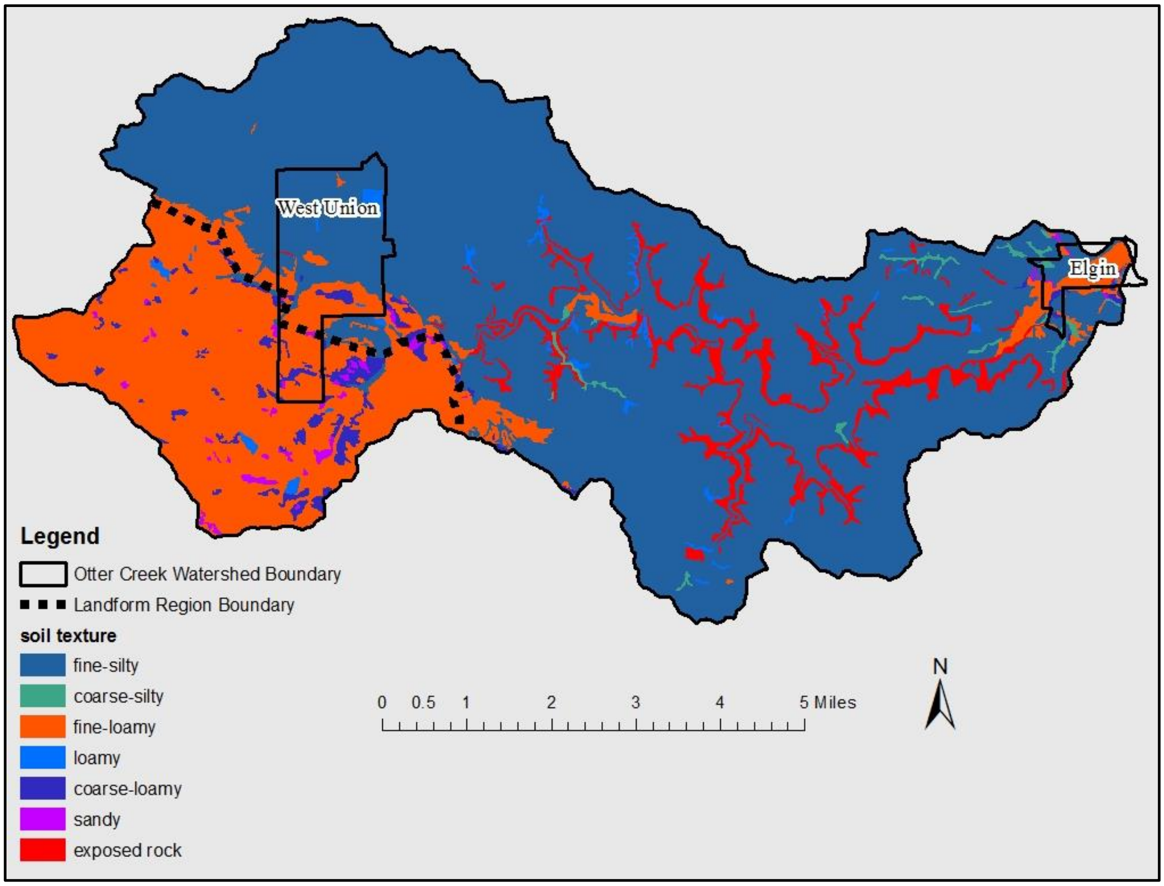


Figure 2.5. Soil texture within the Otter Creek Watershed.

## c. Topography

The topography (Figure 2.6) of the Otter Creek Watershed reflects its geologic past. The area of the watershed located south and west of West Union drains the low-relief, rolling terrain of the Iowan Surface. Streams in this area are well defined but with relatively low slopes. As the streams of the watershed continue in an easterly direction, they cross the boundary between the Iowan Surface and the Paleozoic Plateau. The bedrock-dominated topography of the Paleozoic Plateau is characterized by flatter upland regions with integrated drainages of deeply carved valleys and steep sloping streambeds. Elevations range from approximately 1,243 feet above sea level in the uppermost part of the watershed to 792 feet at the Otter Creek outlet. Figure 2.7 depicts the land surface slope in the Otter Creek Watershed. Slopes generally range from 1–9% in the headwater regions located in the Iowan Surface and quickly steepen as you move east into the Paleozoic Plateau, with nearly vertical exposed limestone bluffs.

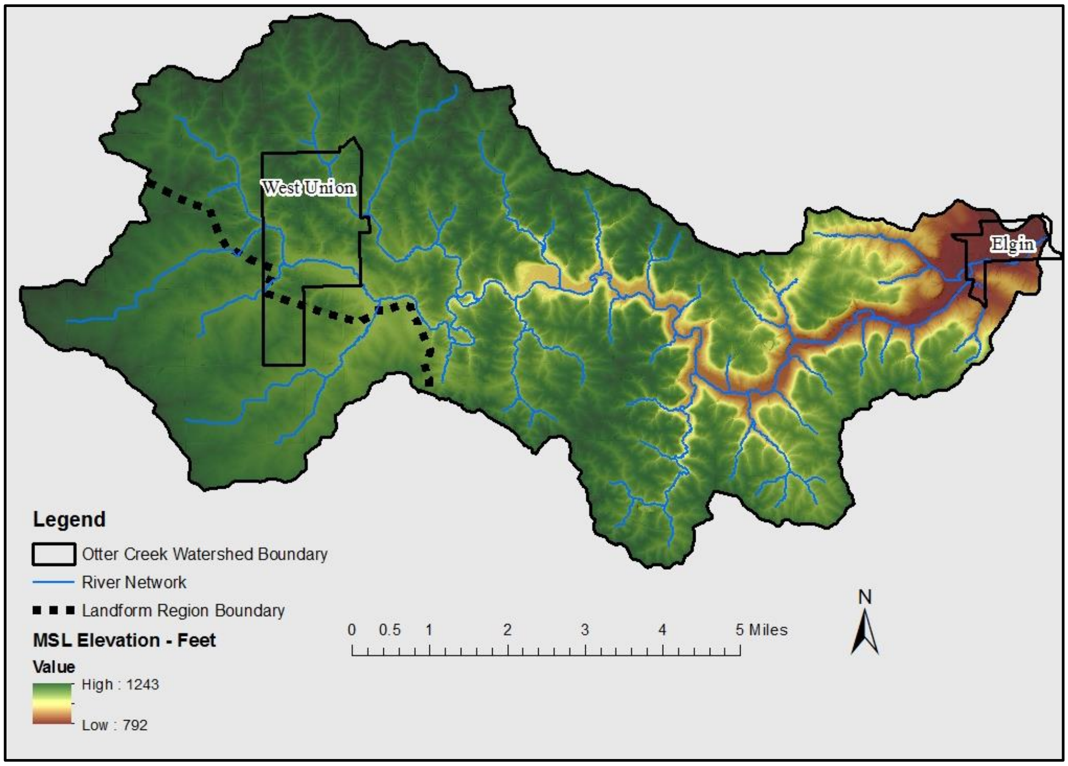


Figure 2.6. Topography of the Otter Creek Watershed.

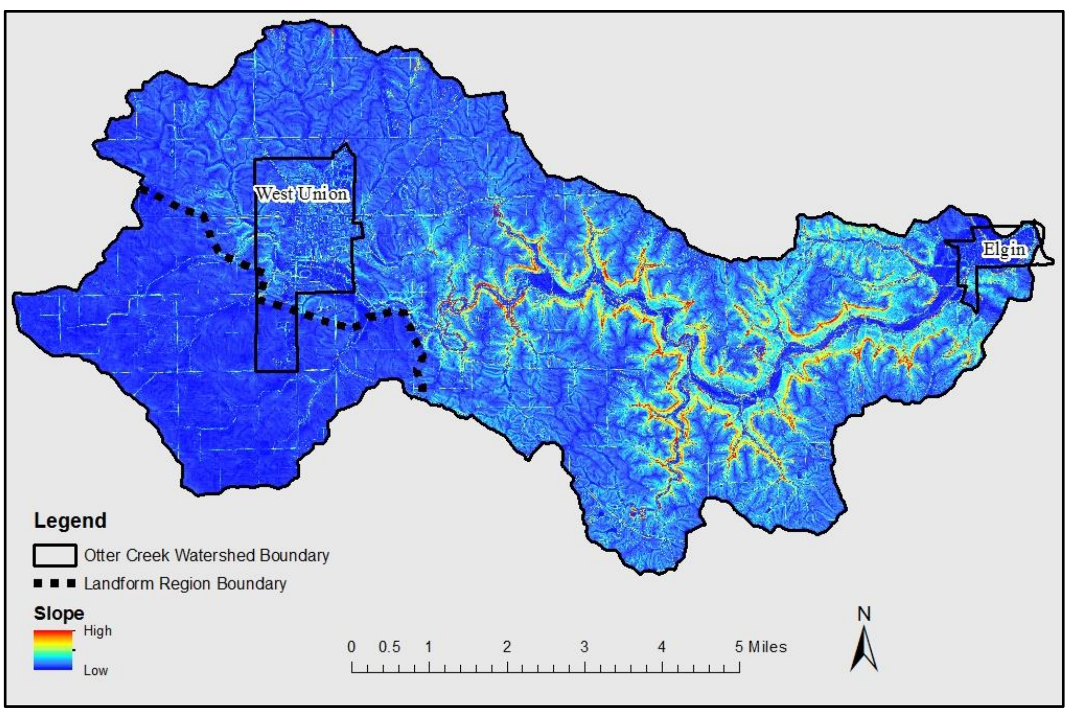


Figure 2.7. Land surface slopes within the Otter Creek Watershed.

## d. Land Use

Land use in the Otter Creek Watershed is predominantly agricultural, dominated by cultivated crops (corn/soybeans) on approximately 54% of the acreage, followed by grass/hay/pasture on approximately 29%. The remaining acreage in the watershed is about 12% forest (primarily deciduous forest), 4% developed land, and less than 1% open water and/or wetlands, per the 2009 High Resolution Land Cover (HRLC) Data Set (Figure 2.8). In excess of 90% of the land within the watershed is privately owned.

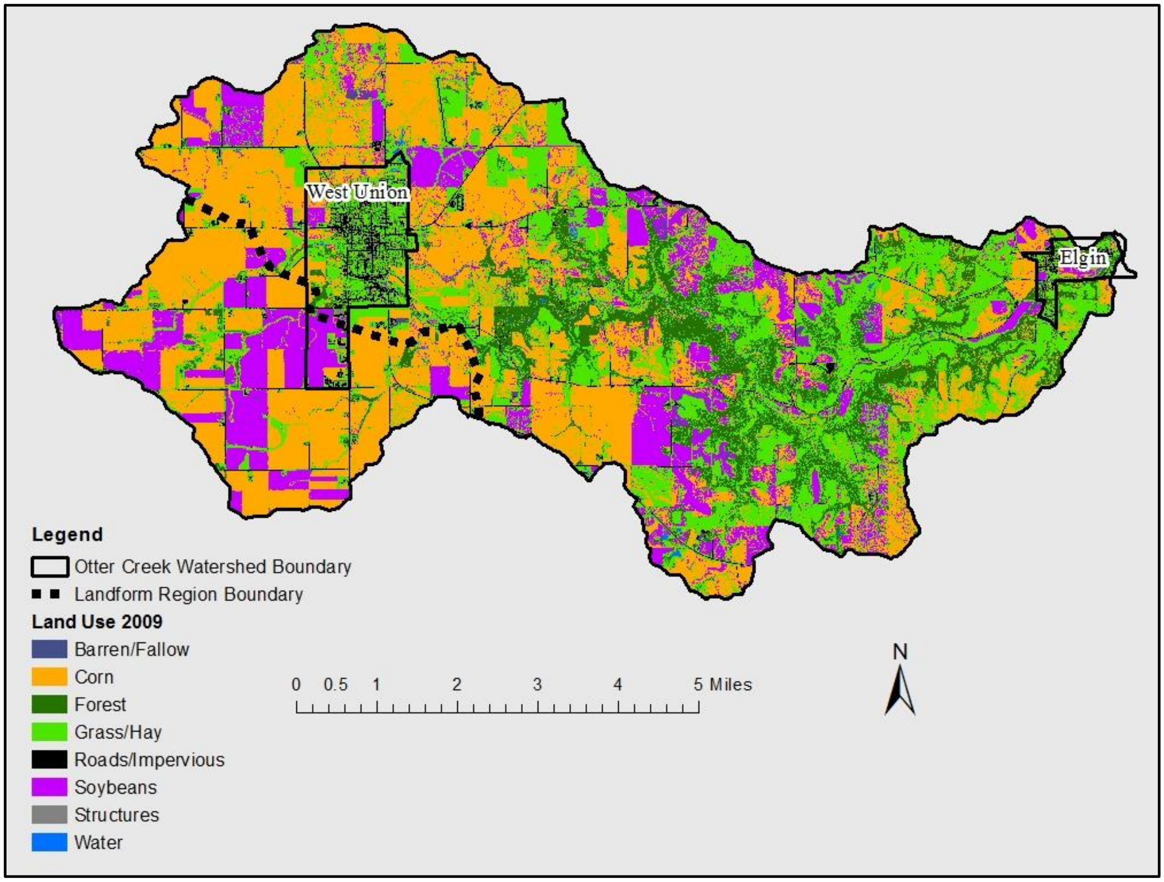


Figure 2.8. Land use composition in the Otter Creek Watershed per the 2009 HRLC. Cultivated crops are shown in orange.

# 3. Data Collection

As part of the Phase II work on the Iowa Watersheds Project, the Iowa Flood Center (IFC) and IIHR—Hydroscience & Engineering (IIHR) installed instruments within the Otter Creek Watershed to monitor hydrologic variables and water quality. This chapter describes the Phase II data collection effort in the Otter Creek Watershed.

## a. Water and Water-quality Measurement Locations

Beginning in spring 2014, we installed several sensors throughout the Otter Creek Watershed. The IFC deployed the sensors to monitor several hydrologic variables and IIHR led the water-quality monitoring. The instrumentation included five rain gauge and soil moisture (RGSM) platforms, four stage sensors, four shallow groundwater wells, and three water-quality sensors. Figure 3.1 shows the locations of the sensors; Table 3.1 shows the sensor station names and periods of record.

**Rain Gauge and Soil Moisture Platforms:** At each of the five rain gauge and soil moisture platform locations, instruments measure soil water content at 2-inch, 4-inch, 8-inch, and 20-inch depths with horizontally installed Campbell Scientific CS655 Water Content Reflectometers. Dual MetOne 380 precipitation gauges are co-located with the soil moisture sensors and measure 15-minute precipitation accumulations. When temperatures go below freezing in the late fall, the precipitation gauges are removed because soil moisture measurements are considered unreliable when moisture near the surface freezes. Each of the sensors is located in short grass open areas, and some of them are in areas adjacent to agricultural activity.

**Shallow Groundwater Wells:** The four shallow groundwater wells are co-located with rain gauge platforms. Each well is constructed from 2-inch PVC pipe drilled to a depth of up to 10.0 ft. Well screens are installed in three-meter increments, beginning at depth of 1.52 m down to 4.57 m. Each site is backfilled with bentonite and equipped with a Decagon CTD-10 water level transducer.

**Stream-stage Sensors:** The four stream-stage sensors are mounted at road crossings. The sensors acoustically measure the distance to the water surface. An approximation of the bed elevation enables the estimation of water depth.

**Water-quality Sensors:** The three IIHR water-quality stations are co-located with stream-stage sensors or streamflow stations (see Figure 3.1). Each sensor platform consists of a Hach Nitratax SC Nitrate Sensor, an FTS DTS-12 Turbidity Sensor, and an Ott-Hydromet Hydrolab DS5X Sonde. We configured the Hydrolab multiprobe sensors to measure water temperature, specific conductance, chlorophyll a, pH, and dissolved oxygen.

Each monitoring system consists of an IIHR-developed datalogger, battery, solar panel, and cellular modem. Data are collected, transmitted, and ingested into servers located at the University of Iowa on a 15-minute schedule.

There is also one United States Geological Survey (USGS) stream gauge within the Otter Creek Watershed. It is located about one mile upstream from the confluence with the Turkey River (see

Figure 3.1). The USGS stream gauge has been operational since March 2014 and continuously monitors stream stage and discharge.

The hydrologic and water-quality data collected by the sensors and presented in Figure 3.1 are publicly available on the internet. The Iowa Flood Information System (IFIS) online tool provides real-time information on watersheds, precipitation, and stream levels for more than 1,000 Iowa communities. Data collected from the rain gauge and soil moisture platforms, shallow groundwater wells, and stream sensors deployed in the Otter Creek Watershed can be accessed at http://ifis.iowafloodcenter.org/ifis/app.

The Iowa Water-Quality Information System (Iowa WQIS) online tool is built on the same user-friendly Google Maps platform that IFIS uses. The Iowa WQIS integrates data gathered by IIHR and the USGS and allows users to track water-quality conditions in real-time. Water-quality data for Otter Creek can be accessed from the site at http://iwqis.iowawis.org/app/.

IFIS and Iowa WQIS provide extensive and critical information needed by scientists, policy-makers, and other Iowans to make science-based decisions that will move us toward accomplishing Iowa's water-quality objectives.

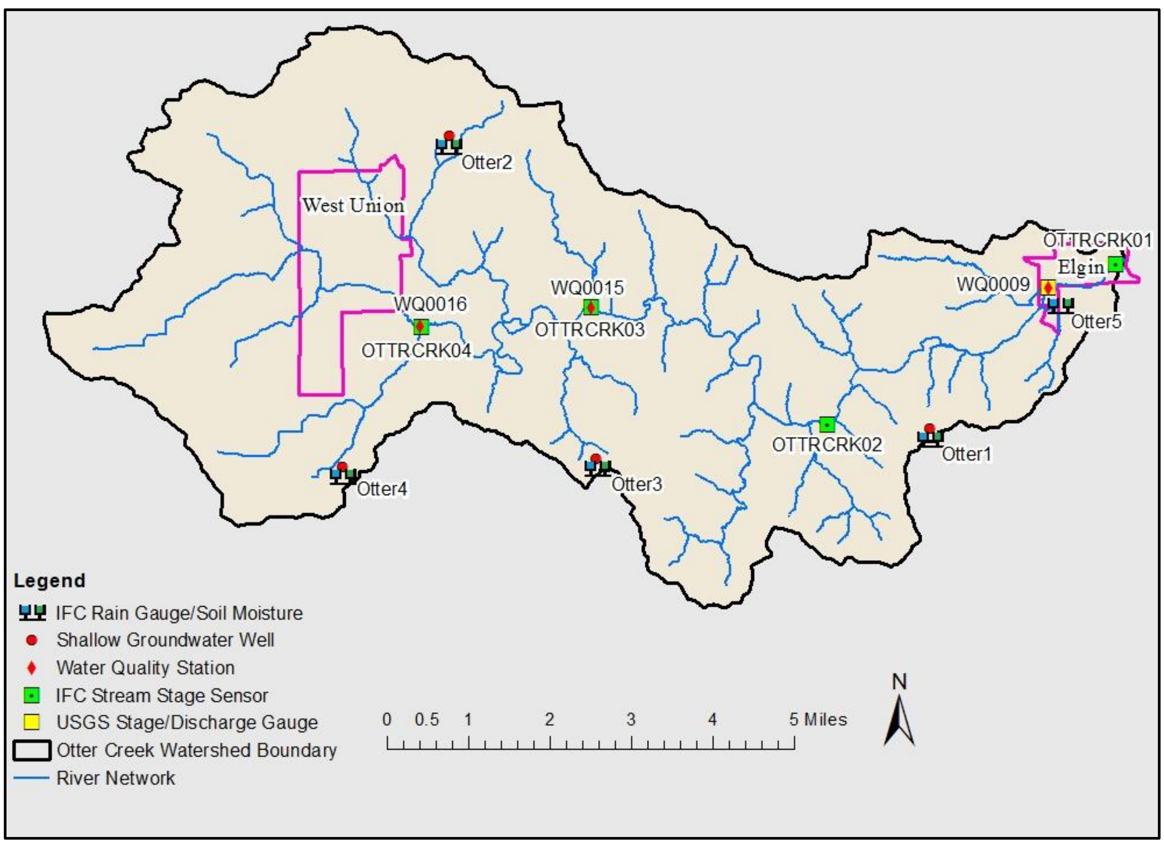


Figure 3.1. Water and water-quality monitoring stations in the Otter Creek Watershed.

Table 3.1. Stage/Discharge Gauges, Water-quality Stations, and Precipitation Gauges in the Otter Creek Watershed.

| *Gauge Type* | *Location* | *Period of Record* |
|---|---|---|
| USGS Stage/Discharge Station ID 05411900 | Otter Creek at Elgin, IA | 2014 – present |
| IFC Stream Sensor (Stage) OTTRCRK04 | Otter Creek, Echo Valley Road, West Union, IA | 2014 – present |
| IFC Stream Sensor (Stage) OTTRCRK03 | Otter Creek, Hornet Rd., near intersection of Hornet Rd. and Hazel Rd., Fayette County, | 2014 – present |
| IFC Stream Sensor (Stage) OTTRCRK02 | Otter Creek, Dove Rd., near intersection of Dove Rd. and Echo Valley Rd., Fayette County, | 2015 – present |
| IFC Stream Sensor (Stage) OTTRCRK01 | Otter Creek, Mill St., Elgin, IA | 2011 – present |
| IIHR Water-Quality Station WQ0016 | Otter Creek, Echo Valley Road, West Union, IA, co-located with OTTRCRK04 | 2014 – present |
| IIHR Water-Quality Station WQ0015 | Otter Creek, Hornet Rd., Fayette County, co-located with OTTRCRK03 | 2014 – present |
| IIHR Water-Quality Station WQ0009 | Co-located with USGS stage/discharge station 05411900 - Otter Creek at Elgin, IA | 2014 – present |
| IFC Rain Gauge/Soil Moisture/Soil Temperature/Groundwater Otter2 | 240$^{th}$ St., approx. 1 mile NE of West Union, IA | 2014 – present |
| IFC Rain Gauge/Soil Moisture/Soil Temperature/Groundwater Otter4 | HYW 150, approx. 2.5 miles S of West Union, IA | 2014 – present |
| IFC Rain Gauge/Soil Moisture/Soil Temperature/Groundwater Otter3 | 200$^{th}$ St., approx. 4 miles SE of West Union, IA | 2014 – present |
| IFC Rain Gauge/Soil Moisture/Soil Temperature/Groundwater Otter1 | Dove Rd., approx. 3 miles SW of Elgin, IA | 2014 – present |
| IFC Rain Gauge/Soil Moisture/Soil Temperature Otter5 | Cedar Rd., approx. 0.5 miles S of Elgin, IA | 2014 – present |

## b. Stream-stage Measurements

With the installation of the IFC sensors, we have collected continuous observations of hydrologic conditions at station locations. Figure 3.2 shows stream-stage and precipitation observations for the 2014 measurement season. The figure shows the hourly average precipitation rate (in inches per hour) for the five rain gauge platforms, and the 15-minute stream-stage observations (in feet above sea level) at two locations in the upper watershed (OTTRCRK03 and OTTRCRK04). As can be seen, the watershed responds quickly when it rains; the stream stages increase rapidly with heavy rain rates, then recede immediately after they reach their peak. In 2014, the heaviest rainfall occurred in late spring and early summer, producing three high-stage periods. After the storm runoff passes, the stages return to lower levels where streamflow is the result of groundwater inflow to the stream (known as baseflow). In general, baseflow levels are slightly higher in the spring when there is more soil moisture in the ground and decrease throughout the summer and fall as soil moisture is depleted and groundwater levels drop. Note, too, that baseflow measurements tend to oscillate daily; this artifact is most likely related to the acoustic sensors, which are affected by daily temperature variations, and not a real oscillation in water levels.

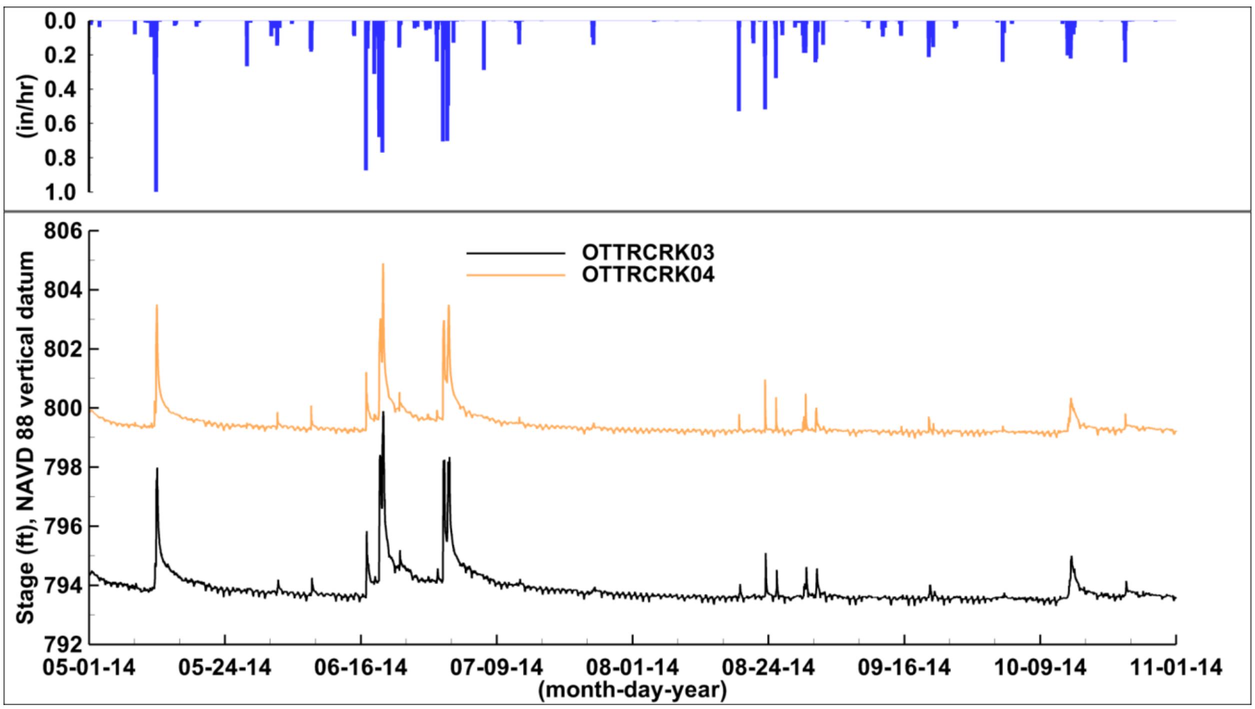


Figure 3.2. Stream-stage hydrographs and precipitation measurements for the 2014 season. The stream-stage elevation (in feet above sea level) is shown for two sites in Otter Creek: the OTTRCRK04 site in the upper watershed, and the downstream OTTRCRK03 site in the middle watershed. Precipitation (in inches per hour) is the average of the measurements at the five rain gauge platforms.

Figure 3.3 shows a nine-day period in June 2014 that includes the highest stream stages that occurred in 2014. Three distinct stage hydrograph peaks can be seen, associated with three rainstorms with more than 1 inch of accumulation. The first was the 1.4-inch rainstorm on June

16 and 17; the storm caused stream stages to rise quickly by nearly 2 feet, and then more slowly recede. After the storm, the baseflow stages are clearly higher than before; this suggests that a significant portion of the rainfall soaked into the ground and recharged the groundwater, increasing the baseflow to the stream. The next two peaks occurred on June 19, associated with two rainy periods. The first rainy period brought 1.9 inches of rain, causing the stream stages to rise by nearly 4 feet. Before the stages could recede back to baseflow, another 1.3 inches of rain fell, which pushed the stages about 2 feet higher than their previous peak. Even though the second rainy period produced less rain than the first (1.3 inches compared to 1.9 inches), its peak was higher than the one from the first rainy period. During the first rainy period, a portion of the water soaked into the ground, and another portion ran off quickly into the stream (causing stages to rise). When rains picked up again during the second rainy period, less water was able to soak into the wet soils; as a result, more water ran off and caused the higher peak stages. Again, baseflow stages after the two rainy periods are higher than before, suggesting that a portion of rainfall had recharged baseflows.

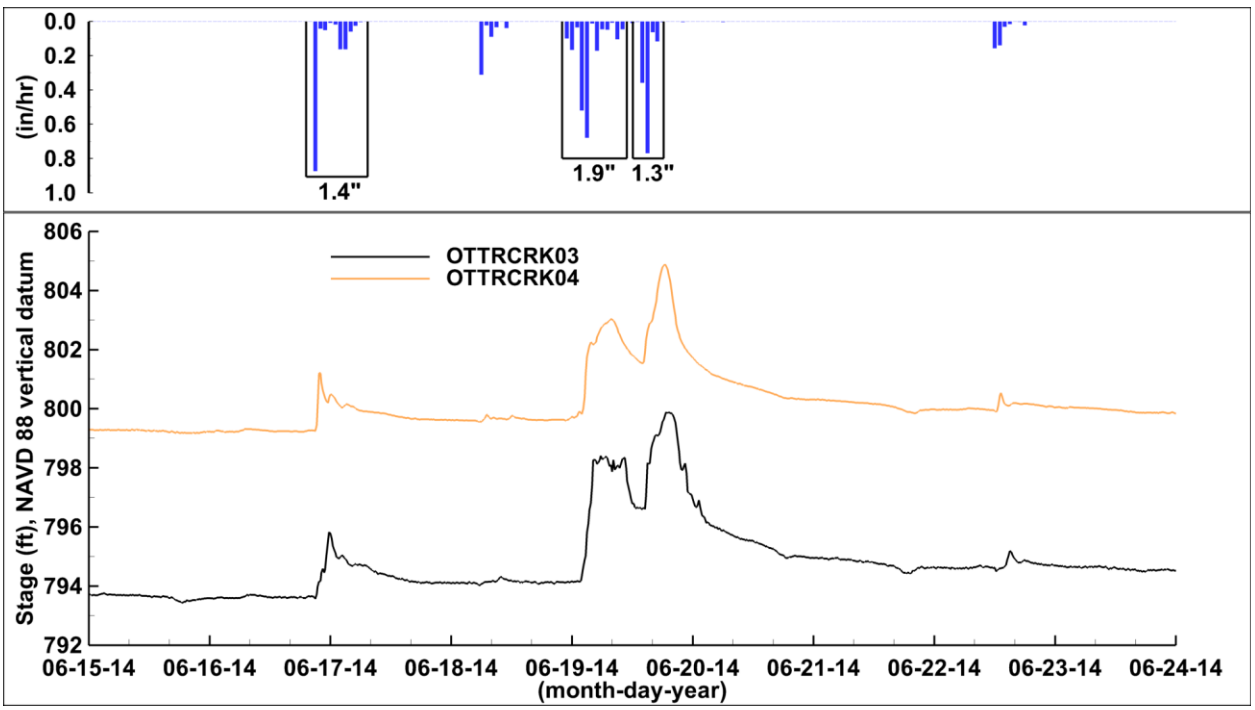


Figure 3.3. Stream-stage hydrographs and precipitation measurements for a nine-day period in June 2014. The stream-stage elevation (in feet above sea level) is shown for two sites in Otter Creek: the OTTRCRK04 site in the upper watershed, and the downstream OTTRCRK03 site in the middle watershed. Precipitation (in inches per hour) is the average of the measurements at the five rain gauge platforms.

## c. Soil Moisture Measurements

Figure 3.4 shows soil moisture and precipitation observations for the 2015 measurement season. The figure shows the soil moisture (in %) at 2-, 4-, 8-, and 20-inch depths; the observations are the average soil moisture at these depths at the five soil moisture platforms (see Figure 3.1). The precipitation is the hourly average precipitation rate for the five rain gauge platforms. Clearly, the soil moisture reacts differently at the different depths. Near the surface at a 2-inch depth, soil moisture content varies the most; it goes from near saturation (100%) to dry conditions (as low as 30%) many times over the season in response to rainfall. The variation at the 4-inch depth is similar, but not as extreme as at 2 inches; the variation is even less at the 8-inch depth. All the way down at a 20-inch depth, the soil moisture varies much more slowly and over a much narrower range.

Note that at depths from 2–8 inches, soil moisture increases rapidly when sufficient infiltrating rainfall occurs. Afterwards, the soil dries more quickly near the surface (2-inch depth). The drying is delayed at the 4-inch depth and even more so at the 8-inch depth. This occurs through a combination of evapotranspiration and percolation. The water nearest to the surface is most readily available for evaporation and transpiration (by plants and vegetation). The water that does not evaporate percolates downward through the soils, keeping the soil moisture at greater depths higher for longer. At the 20-inch depth, the soil moisture only increases rapidly during storms when the entire profile is near saturation, which occurs at times from April to June. Starting around July, the soil moisture at this depth slowly decreases through August, even though some rainstorms significantly increase soil moisture near the surface. Higher September and October rains (when evapotranspiration from plants is less than in the summer) reverses this trend, and soil moisture at 20 inches slowly increases. The depletion of soil moisture at this level (and lower) in the summer growing season helps explain why baseflow (stream inflow from saturated groundwater) typically decreases through the summer months.

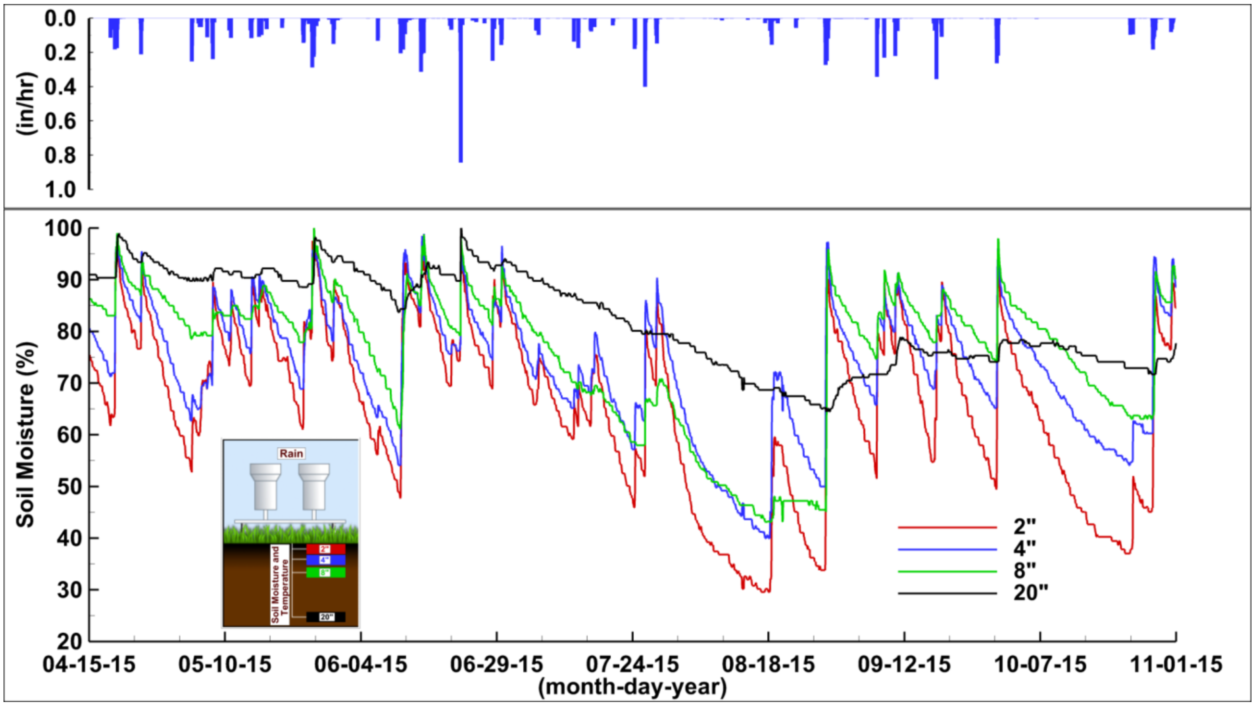


Figure 3.4. Soil moisture and precipitation measurements for the 2015 season. Soil moisture is reported at 2-, 4-, 8-, and 20-inch depths from the surface. The soil moisture values are the average from the five rain gauge/soil measurement platforms in the Otter Creek Watershed. Soil moisture is reported as a percentage; saturated conditions correspond to a soil moisture of 100%. Precipitation (in inches per hour) is the average of the measurements at the five rain gauge platforms.

Figure 3.5 shows a three-week period in May and June 2014 that included two heavy rain periods. Before the first heavy rain period on May 24, the soils were drying at all four levels; it dried quicker nearest the surface, and progressively slower at lower depths. When the 0.6-inch rain occurred on May 24, soil moisture increased at the 2-inch depth the quickest, going from about 61% to 82%. The soil moisture at the 4-inch depth also increased, but more slowly and by a lesser amount. At both the 8-inch and 20-inch depths, there was no significant increase in soil moisture (although the drying of soil has ceased there). After the storm ended, the soil moisture was similar at depths from 2- to 8-inch (around 80%). With the near-surface now wetter, the soil moisture at these three depths reacted differently to the 1.1-inch rain on the following days; all three depths saw soil moisture rapidly increase to near saturation. Soil moisture at the 20-inch depth also increased to near saturation, but the response was delayed. Afterwards, the typical drying progression commenced; soil near the surface dried faster than deeper soils after the rainfall ceased. Hence, the response of the soils to rainfall, and the partitioning of rainfall into infiltration and surface runoff, depends on the soil moisture near the surface and through the entire profile.

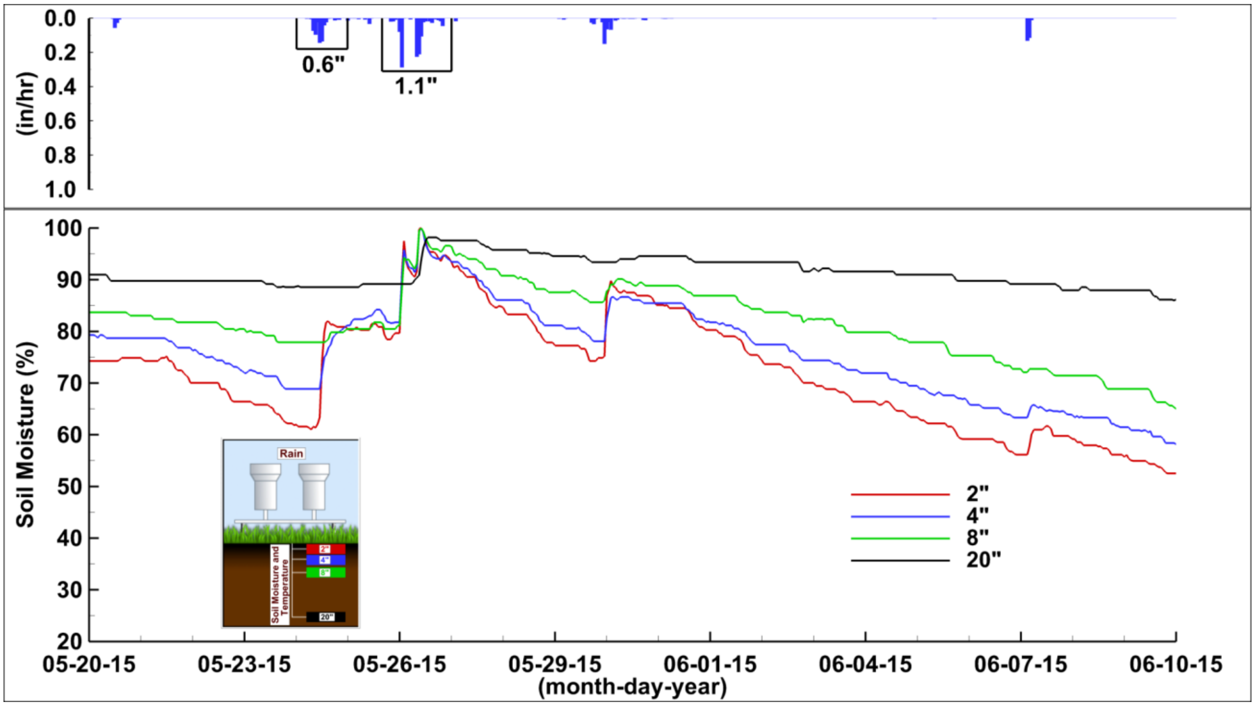


Figure 3.5. Soil moisture and precipitation measurements for a three-week period in late spring 2015. Soil moisture is reported at 2-, 4-, 8-, and 20-inch depths from the surface. The soil moisture values are the average from the five rain gauge/soil measurement platforms in the Otter Creek Watershed. Soil moisture is reported as a percentage; saturated conditions correspond to a soil moisture of 100%. Precipitation (in inches per hour) is the average of the measurements at the five rain gauge platforms.

## d. Soil Temperature Measurements

Figure 3.6 shows soil temperature and precipitation observations for the 2015 measurement season. The figure shows the soil temperature (in °F) at 2-, 4-, 8-, and 20-inch depths; the observations are the average temperature at these depths for the five soil moisture platforms (see Figure 3.1). The precipitation is the hourly average precipitation rate for the five rain gauge platforms. The variations in temperature are what one would expect; the largest diurnal range in temperature occurs nearest to the surface (at the 2-inch depth), where the ground heats during the day and cools rapidly at night. A smaller diurnal range is seen at lower depths. At the lowest depth (20-inch), daily fluctuations are very minor. Overall, the soil warms from April to mid-September (with some cooling in late-August); from mid-September to November, the soil cools. Note at the temperature at lowest depth (20-inch) lags behind that at the other stations, both during the warm-up in spring and summer and during the cool-down in fall.

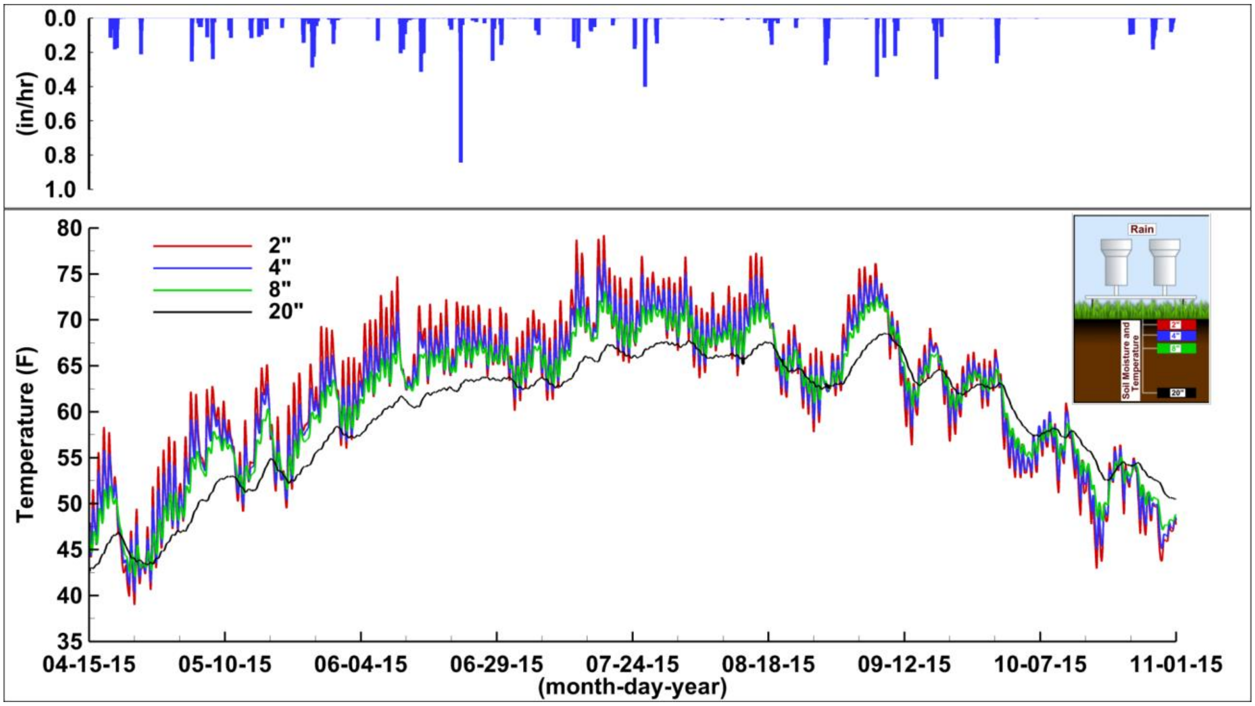


Figure 3.6. Soil temperature and precipitation measurements for the 2015 season. Soil temperature (in °F) is reported at 2-, 4-, 8-, and 20-inch depths from the surface. The soil temperature values are the average from the five rain gauge/soil measurement platforms in the Otter Creek Watershed. Precipitation (in inches per hour) is the average of the measurements at the five rain gauge platforms.

Many of these features are seen more clearly during a three-week period in May and June 2015, shown in Figure 3.7. The soil temperature is higher and has a larger daily range at the 2-inch depth (nearest to the surface). The temperature gets progressively lower, and has a smaller daily range in temperature, as one moves down to the 4-, 8-, and 20-inch depths. The effects of rainy periods on soil temperature is also clearly seen. On days with significant rain, the daily range of soil temperature tends to be lower than on dry days. Rainy days often have less sunshine to warm the soils. Furthermore, after it rains, the soil is heated less because more incoming solar radiation is used to evaporate soil moisture. These two factors explain why rainy days tend to have a lower daily range in temperatures.

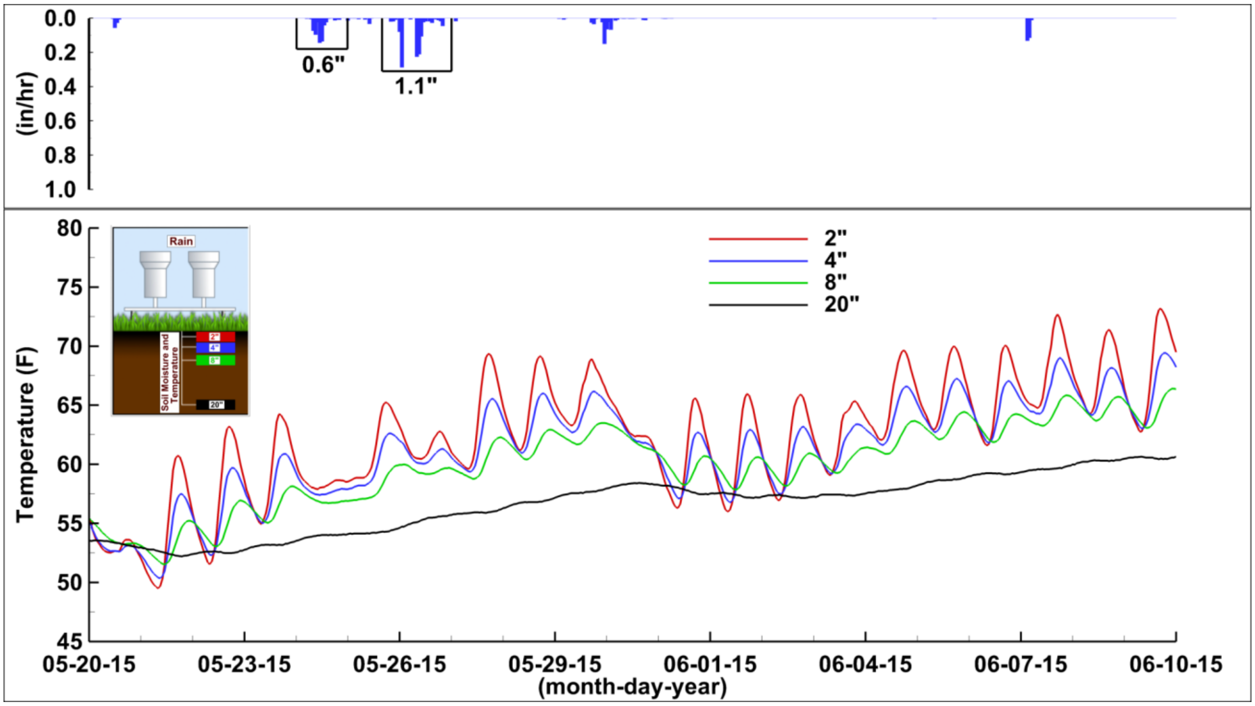


Figure 3.7. Soil moisture and precipitation measurements for a three-week period in late spring 2015. Soil temperature (in °F) is reported at 2-, 4-, 8-, and 20-inch depths from the surface. The soil temperature values are the average from the five rain gauge/soil measurement platforms in the Otter Creek Watershed. Precipitation (in inches per hour) is the average of the measurements at the five rain gauge platforms.

## e. Groundwater Measurements

Figure 3.8 shows groundwater levels and precipitation observations for the 2015 measurement season. The figure shows the groundwater below the surface at two of the five groundwater sites (co-located with the rain gauge/soil platforms); one site is located in the upper watershed (Otter2), and the other in the lower watershed (Otter1) (see Figure 3.1). The precipitation is the hourly average precipitation rate for the five rain gauge platforms. Both sites saw significant groundwater recharge in 2015, as indicated by the rising groundwater levels through spring and early summer. Both sites also saw isolated rapid increases (over a few days) at similar times, suggesting that recent rainfall and groundwater percolation were causing the groundwater rises. After the rises, groundwater levels slowly begin to fall until the groundwater is recharged again. Beginning in late June, the groundwater levels begin a fairly steady drop through August; less rainfall, combined with more water retention and evaporation at the surface during rainstorms, contribute to the seasonal drop in groundwater levels in late summer. Note that this trend is similar to the drying trend seen in the 20-inch depth soil moisture over the same period (see Figure 3.4).

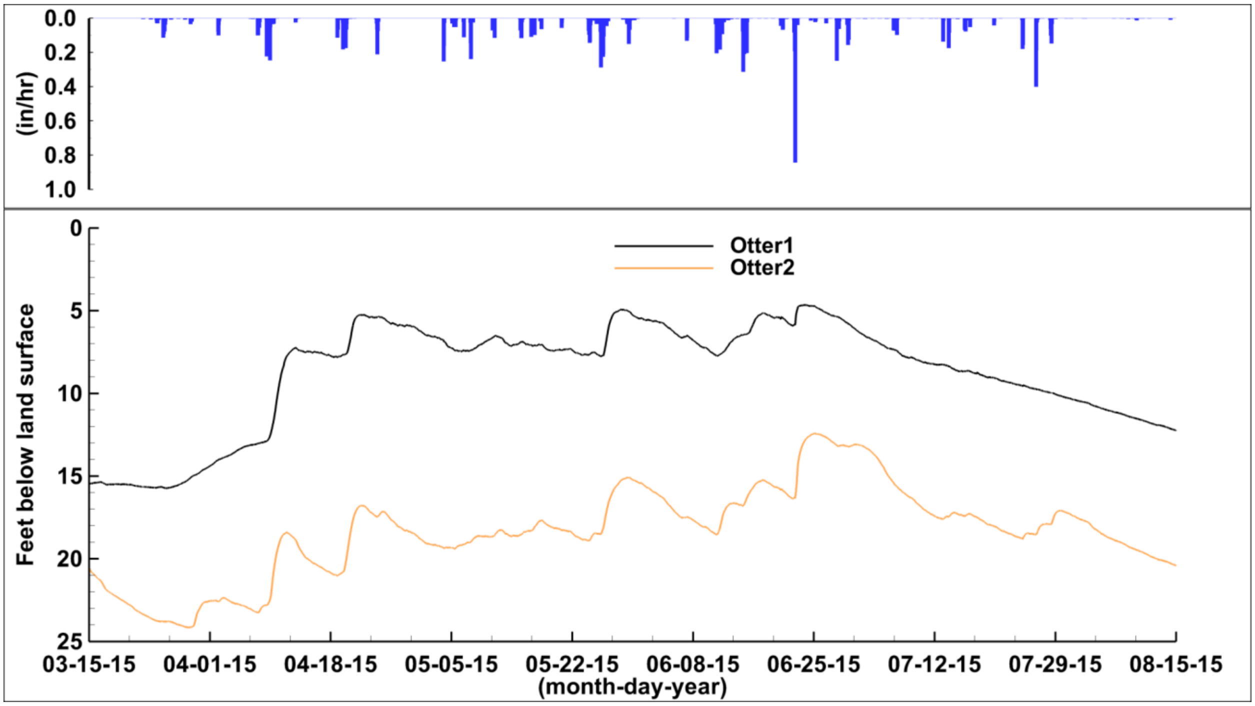


Figure 3.8. Groundwater levels and precipitation measurements for the 2015 season. The depth to the groundwater (in feet) is shown for two sites in Otter Creek: the Otter2 site in the upper watershed, and the Otter1 site in the lower watershed. Precipitation (in inches per hour) is the average of the measurements at the five rain gauge platforms.

Figure 3.9 shows a three-week period in May and June 2015 that includes two heavy rain periods. At both groundwater sites, the groundwater levels are slowly dropping before the first heavy rainfall period on May 24. As the second rainy period begins, both sites see groundwater levels raise several feet over a two–three day period. Note that the smaller rains in late May and early June do not dramatically change the groundwater levels or significantly halt the drawdown in levels that begins as the groundwater is recharged from the earlier rains.

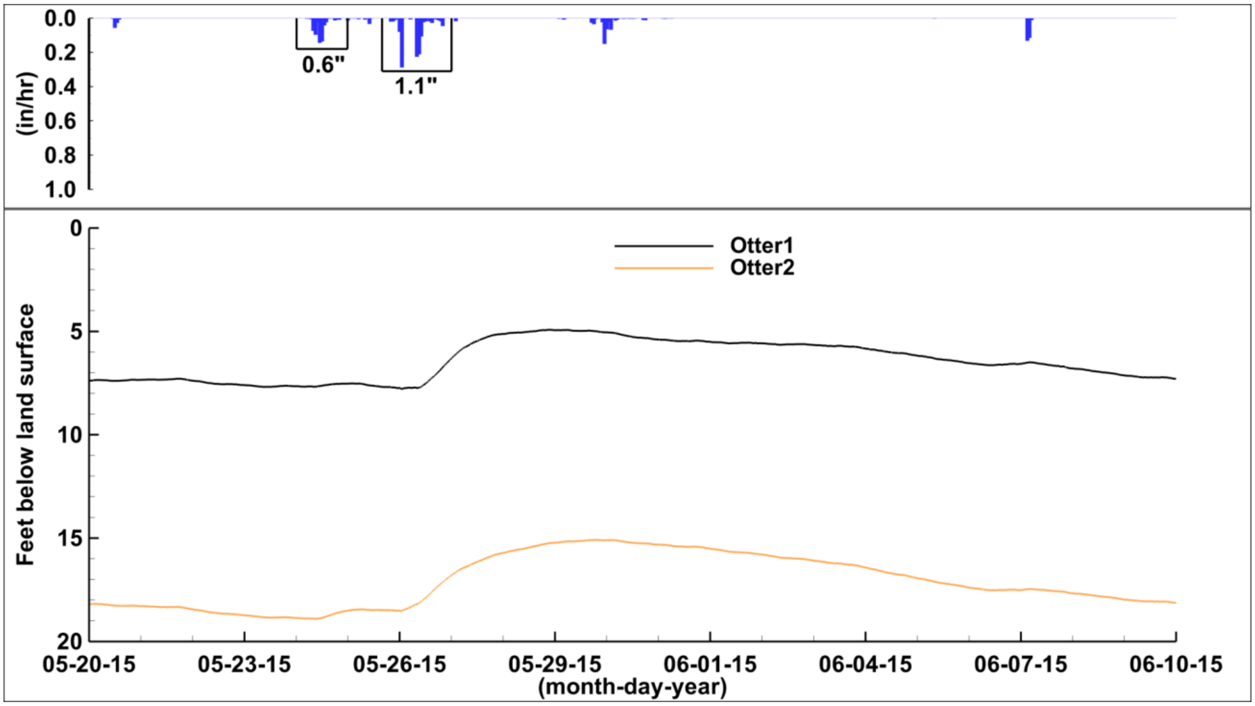


Figure 3.9. Groundwater levels and precipitation measurements for a three-week period in late spring 2015. The depth to the groundwater (in feet) is shown for two sites in Otter Creek: the Otter2 site in the upper watershed, and the Otter1 site in the lower watershed. Precipitation (in inches per hour) is the average of the measurements at the five rain gauge platforms.

## f. Water-quality Measurements

Figure 3.10 shows nitrate concentrations, discharge, and precipitation observations for the 2015 measurement season. The figure shows the nitrate concentrations (Nitrate-N in mg/L) at all three IIHR water-quality stations in Otter Creek (see Figure 3.1). The discharge is the hourly flow rate (in cubic feet per second) at the downstream USGS stream gauge. The precipitation is the hourly average precipitation rate for the five rain gauge platforms. Nitrate concentrations are consistently the highest at the upstream site (WQ0016). Concentrations are lower in the middle watershed (WQ0015), and lowest near the outlet (WQ0009). These trends are related to geology and land use. The upper watershed, which resides in the Iowan Surface region, has a much higher fraction of agricultural land use; corn crops and application of nitrogen fertilizer contribute to higher nitrate concentrations. The middle and lower watershed, which reside in the Paleozoic Plateau region, have a lower fraction of agricultural land use, and water inflows here decrease the overall nitrate concentration in the creek. The variations in nitrate concentration at all three sites follow a similar pattern, which is related to the Otter Creek discharge; concentrations increase after high runoff periods, and slowly recede afterwards. However, nitrate concentrations have abrupt temporary reductions during peak runoff periods. During heavy rainfalls, surface runoff volumes briefly dilute nitrate concentrations.

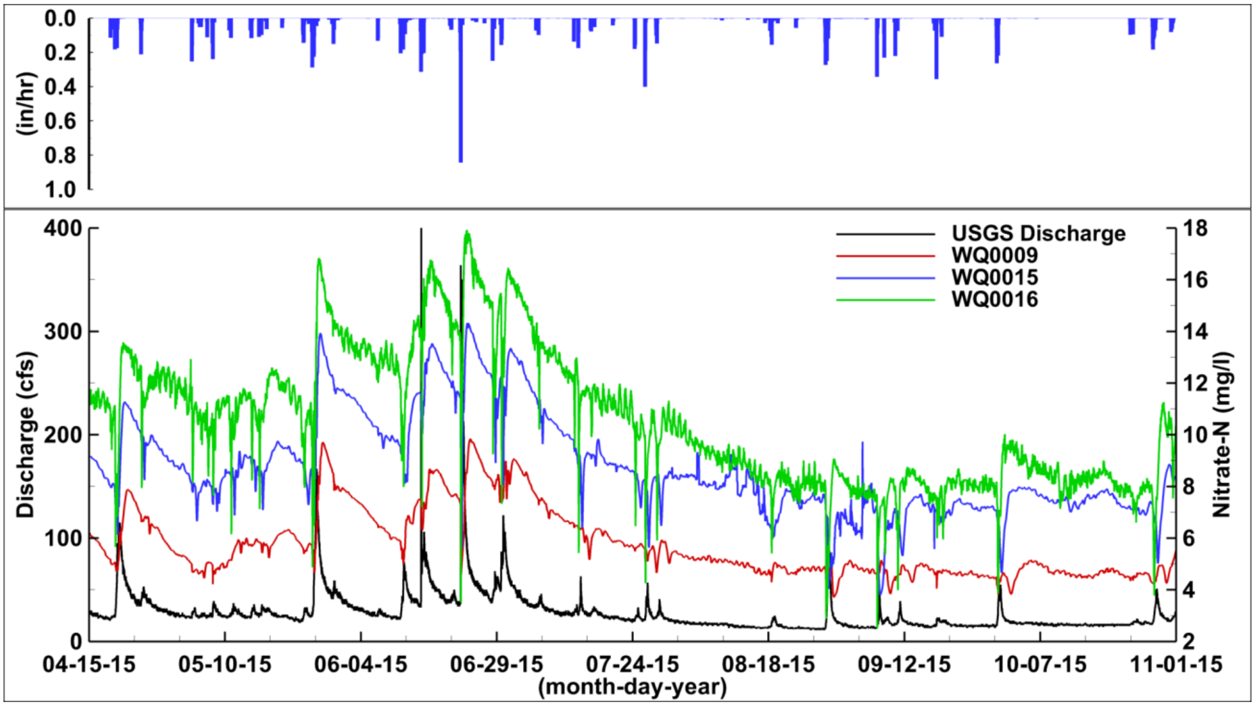


Figure 3.10. Nitrate concentrations, discharge, and precipitation measurements for the 2015 season. The nitrate concentrations (Nitrate-N in mg/L) are shown for three sites in Otter Creek: WQ0016 in the upper watershed, WQ0015 in the middle watershed, and WQ0009 near the outlet. Discharge is the volumetric flow rate (cubic feet per second) at the Otter Creek USGS station (USGS 05411900). Precipitation (in inches per hour) is the average of the measurements at the five rain gauge platforms.

This effect is seen more clearly in Figure 3.11, which shows a three-week period in May and June 2015 that includes two heavy rain periods. During the heavy rainfall periods between May 24 and 27, nitrate concentrations decrease at all three sites. The decrease occurs during the rising limb of the discharge hydrograph, but we see a rapid rebound to higher concentration levels afterwards. The decrease is more pronounced upstream (WQ0016), where there is a higher fraction of land in corn production, and muted downstream. After the storm ends, runoff from groundwater sources (baseflow) continues, but at a higher rate than before the storm. The increased baseflow leaches and transports more of the nitrate stored in the soils, resulting in the higher nitrate concentration after the storm.

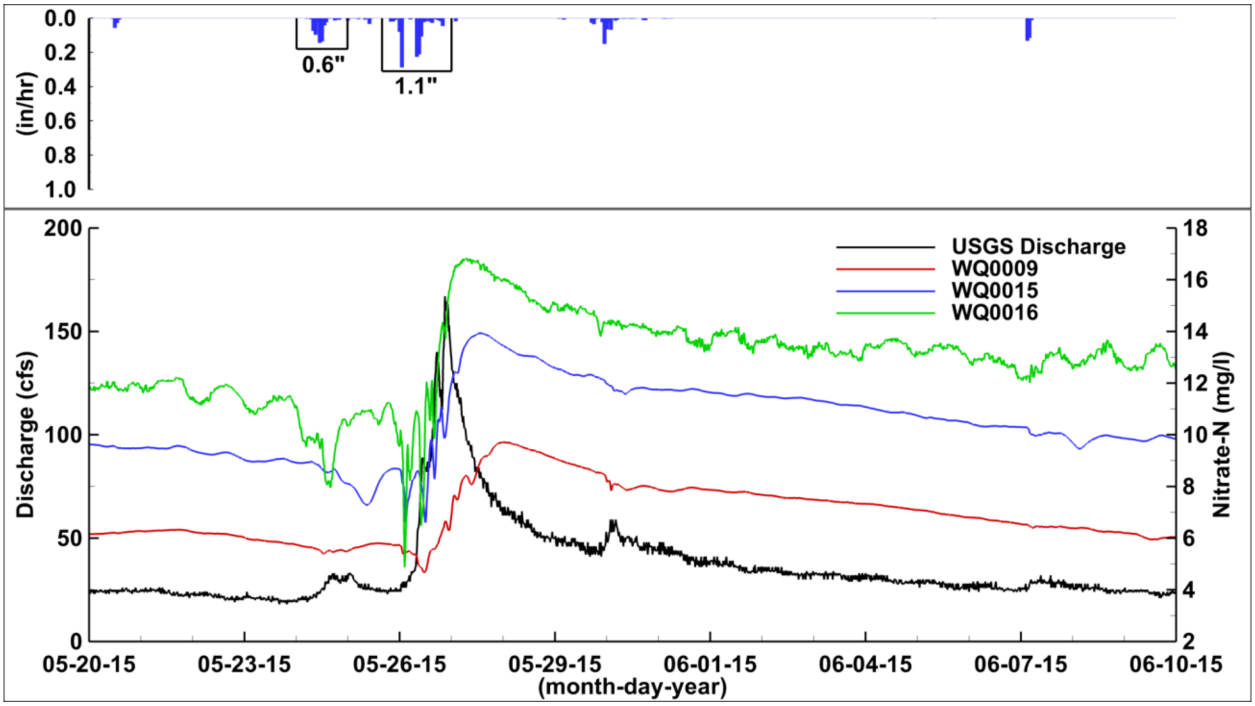


Figure 3.11. Nitrate concentrations, discharge, and precipitation measurements for a three-week period in late spring 2015. The nitrate concentrations (Nitrate-N in mg/L) are shown for three sites in Otter Creek: WQ0016 in the upper watershed, WQ0015 in the middle watershed, and WQ0009 near the outlet. Discharge is the volumetric flow rate (cubic feet per second) at the Otter Creek USGS station (USGS 05411900). Precipitation (in inches per hour) is the average of the measurements at the five rain gauge platforms.

## g. Monitoring Summary

Beginning in 2014, the Iowa Flood Center and IIHR–Hydroscience & Engineering started intensive monitoring of water and water quality in the Otter Creek Watershed. Instrumentation deployed there measures precipitation, discharge, stream stage, soil moisture, soil temperature, groundwater levels, and water quality. The data collected during the 2014 and 2015 seasons record changes in the water and water quality within the watershed. This data collection effort guides our work to develop detailed hydrologic models that mimic observed watershed processes. The network of instruments will also be used to monitor changes in the watershed as project activities are implemented.

# 4. Project Inventory

To meet the primary goal of the Iowa Watersheds Project, we allocated a total $1,500,000 to the Otter Creek Watershed to plan, implement, and construct watershed improvement projects directed at reducing flood damage. Project locations were selected based on volunteer landowner interest and recommendations from the Fayette County Natural Resource Conservation Service (NRCS) staff.

We intended the projects built in the Otter Creek Watershed to serve as demonstration projects so landowners and others can visit to better understand the projects. The Turkey River Watershed Management Authority and the county Soil and Water Conservation Districts hope to implement practices in other locations across the entire watershed. This chapter describes the Iowa Watersheds Project Phase II projects built in the Otter Creek Watershed.

## a. Iowa Watersheds Project Phase II Flood Mitigation Projects

Many ponds in Iowa have been constructed to provide flood storage. Figure 4.1 is a schematic of a typical flood storage pond. An earthen embankment constructed across the stream creates the pond. The pond holds some water nearly all the time (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. Typically, this principal spillway consists of a pipe passing through the embankment and discharging water downstream of the embankment. As the water level rises during a flood, the pond temporarily stores more water. Eventually, the water level reaches the auxiliary spillway elevation. The auxiliary spillway is constructed as a means to release water rapidly so the flow does not damage or overtop the earthen embankment. The volume of water stored between the principal spillway elevation and the auxiliary spillway elevation is called the flood storage.

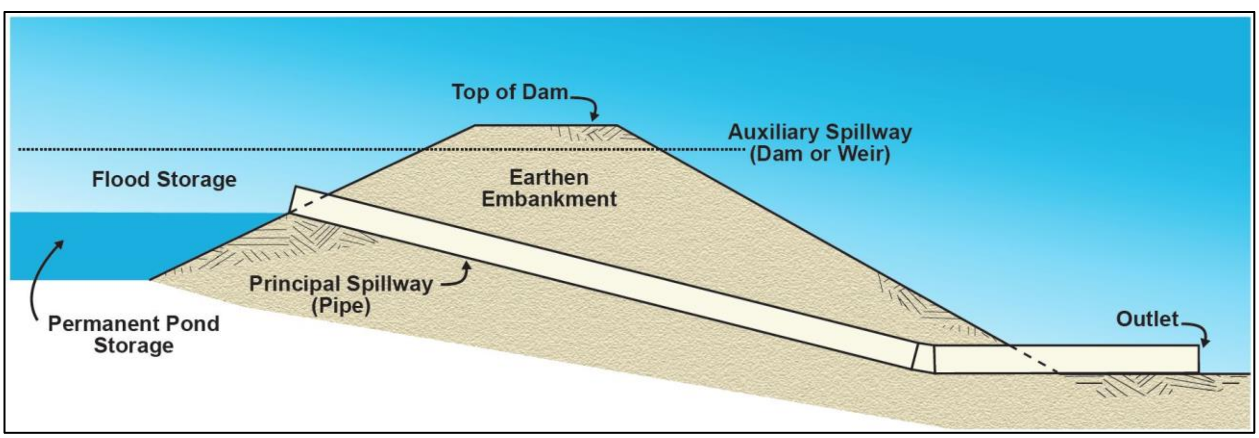


Figure 4.1. Schematic of pond constructed to provide flood storage.

The Iowa Watersheds Project Phase II provided support for the construction of five on-road detention structures, five terrace water and sediment control structures, and 19 ponds in the Otter Creek Watershed. The on-road detention structures use a baffle type installation on a roadway culvert to raise the invert elevation of the culvert. Thus, the structures impound water upstream of the roadway, slowing flood runoff. A terrace water and sediment control basin is an earth embankment or a combination ridge and channel constructed across the slope of minor watercourses to form a sediment trap and water detention basin with a stable outlet. This practice may be applied as part of a resource management system for one or more of the following purposes: to reduce watercourse and gully erosion; to trap sediment; and/or to reduce and manage local onsite and downstream runoff (NRCS Code No. 638).

These constructed flood mitigation structures will provide approximately 460 acre-feet of flood storage. Additional storage is provided as the water level on the ponds rises higher than the elevation of the auxiliary spillway up to the top of the dam. The storage from the principal spillway elevation to the top of dam is often called total storage.

A private consulting engineering firm and the Fayette County NRCS staff designed the projects , which were built to NRCS Practice Codes No. 410 (NRCS 1985), No. 378 (NRCS 2011), and Iowa Department of Natural Resources (IDNR) Technical Bulletin No. 16 (IDNR 1990). Figure 4.2 shows the project locations, which have been numbered for IFC tracking purposes. Table 4.1 provides the IFC pond ID #, the property owner, and the name given as the pond identifier on the design documentation.

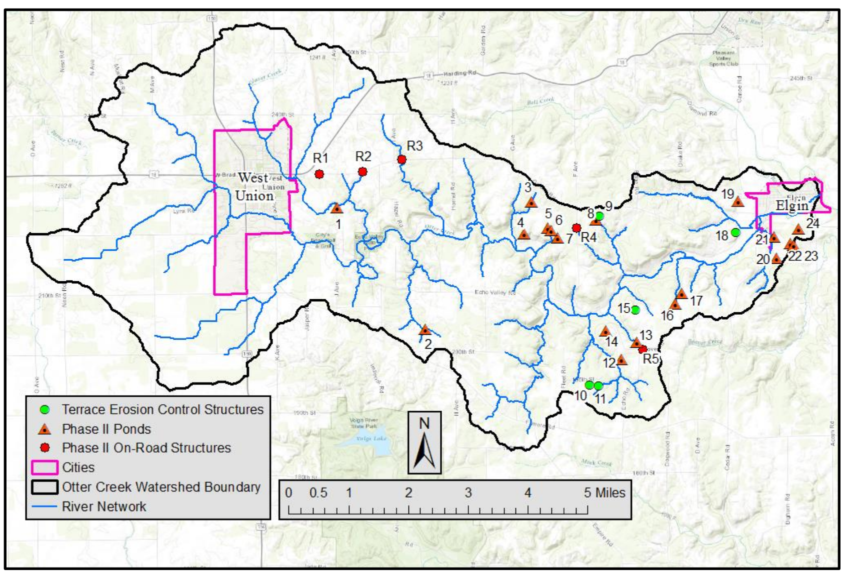


Figure 4.2. Iowa Watersheds Project Phase II project locations in the Otter Creek Watershed.

Table 4.1. Iowa Watersheds Project Phase II flood mitigation projects in the Otter Creek Watershed.

| ***Pond ID #*** | ***Property Owner*** | ***Design Documentation ID*** | ***Type*** | ***Flood Storage (acre-feet)*** |
|---|---|---|---|---|
| R1 | Fayette County | Golden On-Road #1 | On-Road | 46.1 |
| R2 | Fayette County | Golden On-Road #2 | On-Road | 29.2 |
| R3 | Fayette County | Golden On-Road #3 | On-Road | 243.1 |
| R4 | Fayette County | F Avenue On- Road | On-Road | 10.6 |
| R5 | Fayette County | Dove Road On-Road | On-Road | 46.1[1] |
| 1 | Iowa DNR | DNR | Pond | 0.7 |
| 2 | Open Range Farms | Open Range Farms | Pond | 16.2 |
| 3 | McMillan | McMillan #2 | Pond | 11.0 |
| 4 | McMillan | McMillan #1 | Pond | 0.9 |
| 5 | McMillan | McMillan #3 | Pond | 3.5 |
| 6 | McMillan | McMillan #4 | Pond | 6.6 |
| 7 | McMillan | McMillan #5 | Pond | 3.1 |
| 8 | Helgerson | Helgerson 410 | Pond | 0.8 |
| 9 | Helgerson | Helgerson 638 | Terrace Water & Sediment Control | 2.1 |
| 10 | Howard | Howard 638 #2 | Terrace Water & Sediment Control | 0.9 |
| 11 | Howard | Howard 638 #1 | Terrace Water & Sediment Control | 5.6 |
| 12 | Bennett | Bennett #1 | Pond | 2.8 |
| 13 | Bennett | Bennett #2 | Pond | 5.3 |
| 14 | Woltz | Woltz | Pond | 0.3 |
| 15 | Bennett | Bennett #3 | Terrace Water & Sediment Control | 4.1 |
| 16 | Bennett | Bennett #4 | Pond | 4.6 |
| 17 | Bennett | Bennett #5 | Pond | 4.8 |
| 18 | Medberry | Medberry 638 | Terrace Water & Sediment Control | 1.7 |
| 19 | Frieden | Frieden | Pond | 1.0 |
| 20 | Helms | Helms #1 | Pond | 0.4 |
| 21 | Helms | Helms #5 | Pond | 3.5 |
| 22 | Helms | Helms #3 | Pond | 0.3 |
| 23 | Helms | Helms #2 | Pond | 0.6 |
| 24 | Helms | Helms #4 | Pond | 0.2 |

[1] Design documentation was unavailable for the Dove Road On-Road structure, so the flood storage reported for Golden On-Road #1 (46.1 acre-feet) was assumed for modeling purposes.

Figure 4.3 shows the earthen embankment of one of the Iowa Watersheds Project flood mitigation structures after the completion of construction, reseeding of the area, and filling of the pond (taken 4/12/2016).

Figure 4.3. Earthen embankment of one of the Iowa Watersheds Project ponds constructed to provide flood storage.

## b. Hydraulics of Flood Mitigation (Pond) Projects

Pond and on-road structure projects can reduce flood damages by storing water during high runoff periods. That is, storage ponds hold floodwaters temporarily, and release water at a slower rate. Therefore, the peak flood discharge downstream of a storage pond is lowered. The effectiveness of any one storage pond depends on its size (storage volume) and how quickly water is released. Ponds are engineered to efficiently use their available storage for large floods (typically in the 10- to 50-year return period range). Figure 4.4 shows two hydrographs for one of the Phase II pond locations. The larger magnitude hydrograph represents the inflow to the pond (or what would pass downstream if the pond wasn't there), and the smaller magnitude hydrograph shows what is coming out of the pond. The solid black line would be exceeded in magnitude by the outflow hydrograph if the auxiliary spillway was activated during this storm event. For this event, the auxiliary spillway was activated and the pond stored a significant volume of water during the event.

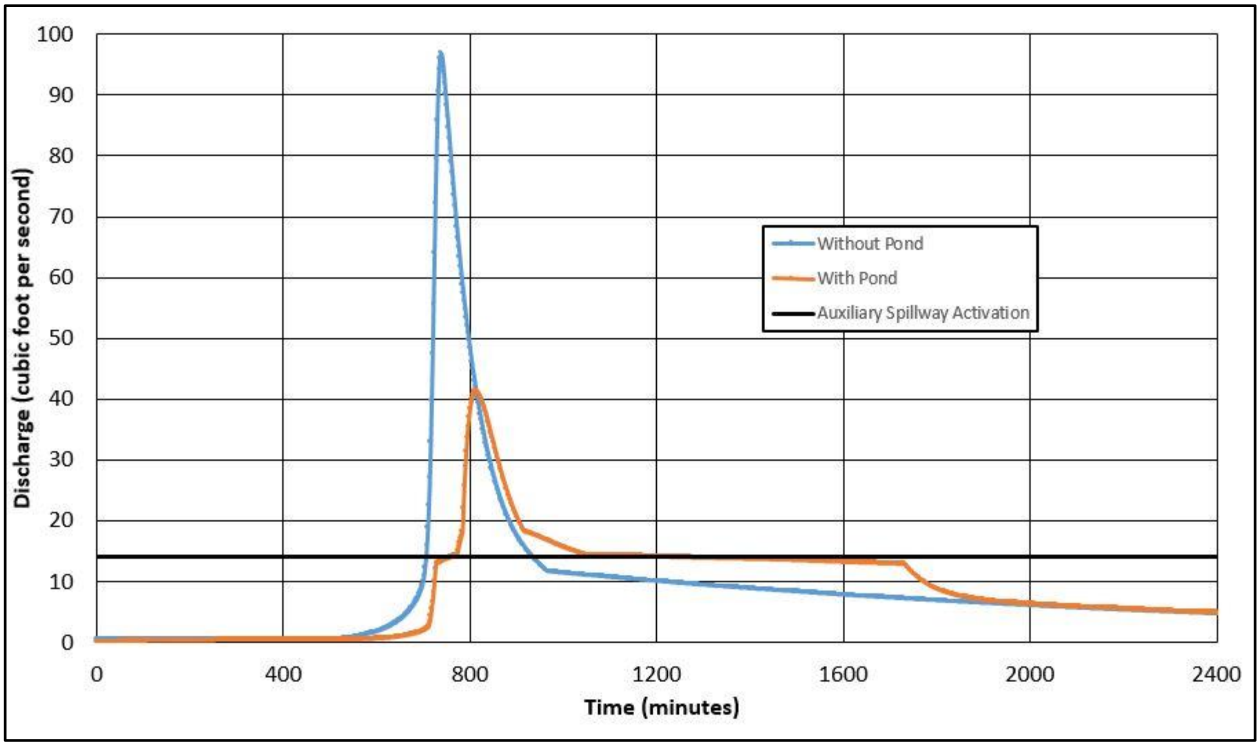


Figure 4.4. Inflow and outflow hydrographs for one of the Iowa Watersheds Project Phase II pond projects.

To determine the pond volume and outflow characteristics of the Iowa Watersheds Project flood mitigation structures, we obtained design documentation from the design engineer(s) and/or Fayette County NRCS staff. This included the project plans, which describe how the project was built, as well as any hydrologic design information used to select the principal and auxiliary spillway outflow structures. Engineers determined each pond's stage (elevation)-storage relationship as a part of the predesign topographic analysis included in a table in the design plans. For hydrologic modeling purposes, the pond's stage-discharge table is needed to route rainfall runoff through the pond at the appropriate magnitude throughout the simulation. Engineers determined the stage-discharge relationship for each project based on the final design specifications for the principal spillway (pipe) size and slope, as well as the width and retardance class of the auxiliary spillway. They used NRCS's WinPond hydrologic routing software to verify the stage-discharge relationship derived for discharges associated with elevations ranging between the principal spillway and the top of dam. Discharge in the event of dam overtopping was estimated based on the additional depth of water in the emergency spillway.

Figure 4.5 shows an example of a stage-storage relationship of one of the ponds and the developed stage-discharge relationship for the same pond. Stage-storage tables as provided by the consulting engineer and stage-storage-discharge tables as used for hydrologic modeling for each of the 29 Phase II flood mitigation projects have been included in Appendix A of this report.

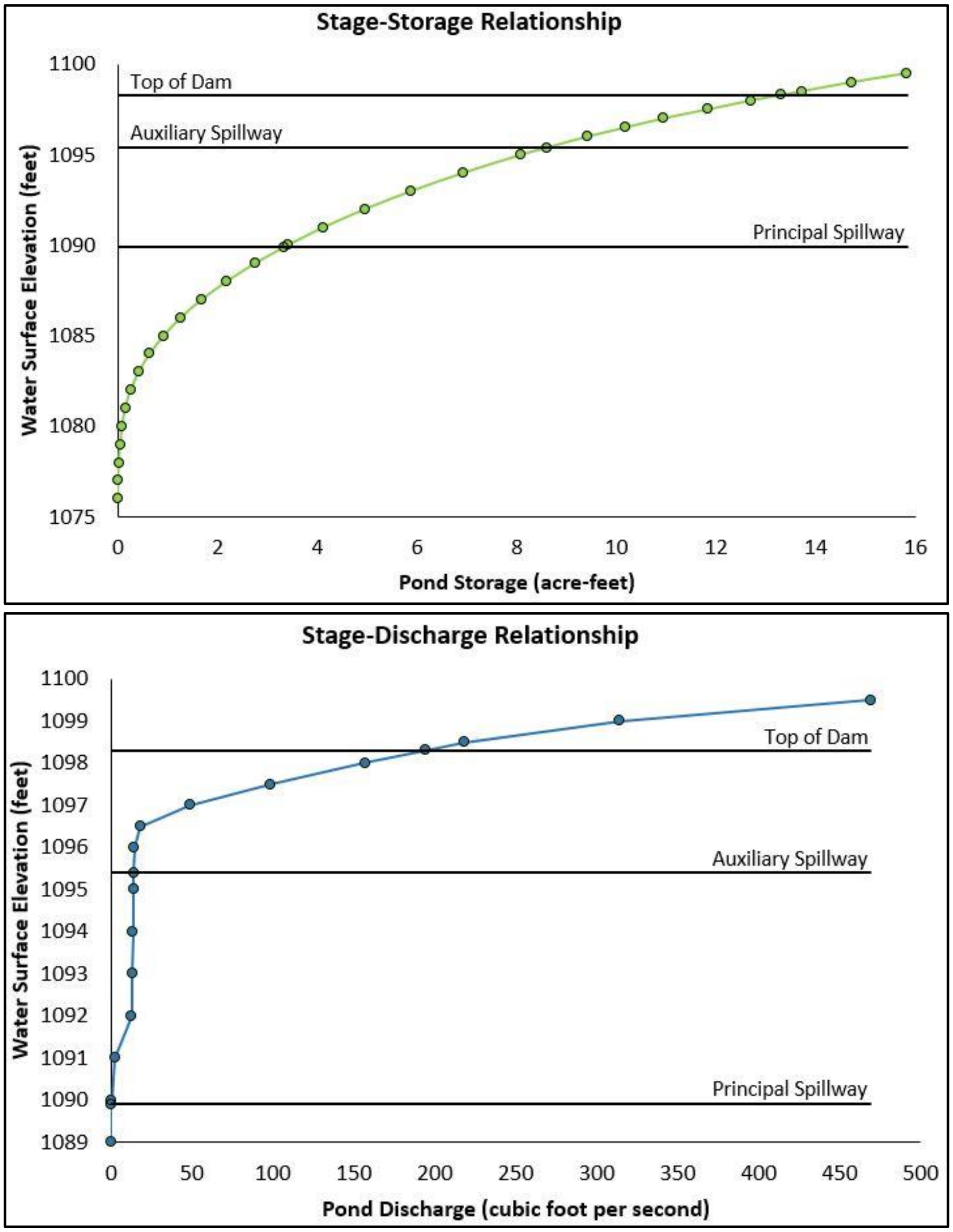


Figure 4.5. Pond hydraulic relationships for one of the Iowa Watersheds Project Phase II flood mitigation projects: (top) Stage (elevation) – Storage relationship, and (bottom) Stage – Discharge relationship. The figure indicates the elevations of the principal spillway, the auxiliary spillway, and the top of dam.

## c. Project Summary

The projects constructed through the Iowa Watersheds Project provide multiple benefits both on- and off-site. Landowners enjoy the farm ponds on their property for the aesthetic beauty, recreation, and the wildlife they attract. In addition, landowners can use the ponds to water livestock and control erosion on their land. The project placed pond structures based on input from volunteer landowners and the guidance of the Fayette County NRCS staff to fit the landowners' overall working plan for the ground. The flood mitigation projects create water storage on the landscape that reduces downstream flooding, protecting both people and infrastructure. The pond structures are able to provide significant savings in federal, state, and local road and bridge maintenance costs by managing runoff to reduce and mitigate structural and nonstructural flood damage.

Figure 4.6. Iowa Watersheds Project Phase II project showing a water and sediment control basin.

# 5. Detailed Predictions of Hydrologic Alterations

This section offers a comprehensive analysis of the fine-scale impacts of the flood mitigation structures. To quantify the effects of human-induced hydrologic alterations on the Phase II watersheds, researchers built a numerical model, which was calibrated and validated with monitoring data. They also used design storm analysis to investigate project performance for flood conditions. This chapter continues with a description and construction of the numerical model, calibration, validation, and a design storm assessment.

## a. Numerical Model Description – HydroGeoSphere

Researchers selected the numerical model HydroGeoSphere (HGS) to investigate the detailed aspects of integrated watershed response to flood mitigation practices. HGS takes into account all of the key components of the hydrologic cycle, applying the most physically realistic representation of water movement (see Figure 5.1). Within the model domain, rainfall is partitioned between overland surface flow, evaporation, transpiration, and infiltration, enabling discharge through the surface or subsurface into downstream water bodies or aquifer flows (Brunner and Simmons, 2012). The software can implement wells, tile drains, subsurface fractures, and channelized flow. Rainfall is applied to the surface of the domain. We modeled interception, evaporation, and transpiration using the Kristensen and Jensen approach (Brunner and Simmons, 2012), in which evapotranspiration is a function of soil water availability and vegetation growth characteristics. HGS quantifies and illustrates the micro- and macro-scale effects of each project on the water balance and overall fluxes.

In direct comparison to the Hydrologic Assessment of the Turkey River Watershed, HGS is a mathematical, physically-based, distributed, coupled, surface-subsurface hydrologic model. We will briefly discuss each of these items. The fact that HGS is a mathematical model implies that the different hydrologic processes are represented by mathematical expressions based on the fundamentals of fluid mechanics or based in physics. HGS is a distributed parameter model, meaning that physical characteristics of the watershed, such as land use and soil type, can vary from one location to the next. HGS is a coupled model, meaning that the different hydrologic processes are solved jointly rather than independently. In reality, surface and subsurface processes are dependent on one another, and their governing equations should be solved simultaneously. Finally, HGS is a surface-subsurface hydrologic model, meaning that it is applicable to almost every hydrologic simulation.

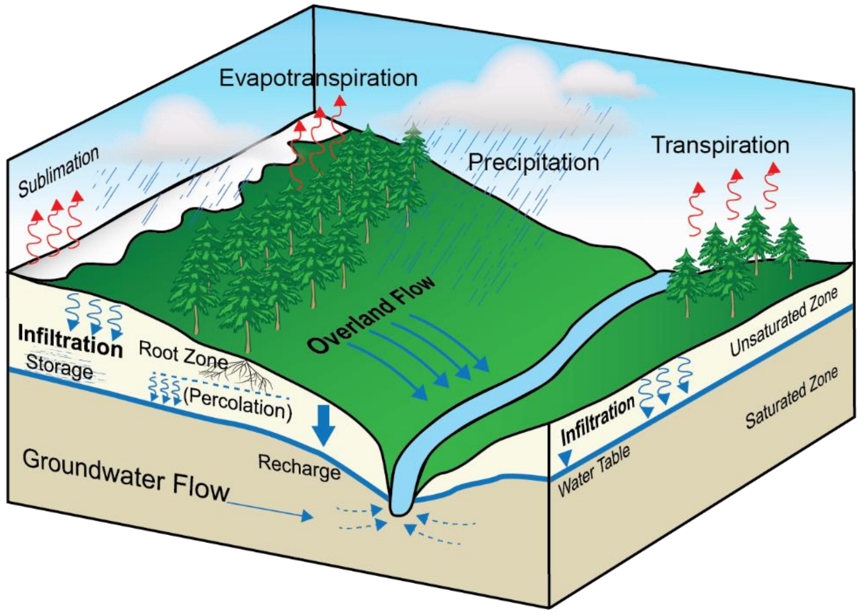


Figure 5.1. The numerical model HydroGeoSphere's simulation of hydrologic processes.

## b. Mesh Generation

The objectives of this study required the investigation of surface and near-surface water flow processes. The automatic generation of variably sized triangular elements created a two-dimensional representation of the land surface. For this study, HGS produced a mesh from the watershed boundary, stream centerlines, roadways, and hydraulic structure locations. We identified the watershed boundary as the local topographic high, draining all internal areas to a single outlet location. This boundary acts as the lateral edge of both the surface and subsurface domains. A majority of Iowa is typified by mildly sloped agricultural expanses, divided by elevated roadways and incised stream channels. During heavy rainfall events, elevated roadways act as topographic divides, forcing rainfall into nearby drainage ditches and then into stream channels. HGS extracts elevation information from element edges. By allocating element edges along topographic features, the elevation at that location is enforced. We deemed roadways and stream centerlines as topographically significant features and included them as mesh generation boundaries. We delineated stream centerlines and incorporated them to ensure continuous flow to the catchment outlet, maintaining travel times and realistically capturing surface-subsurface interactions (Li et al., 2008). To increase the efficiency of numerical simulations, we coarsened mesh elements to 600 feet across mildly sloped areas, and refined near streams and constructed projects to 100 feet. The final two-dimensional surface grid contained 36,638 triangular elements (see Figure 5.2).

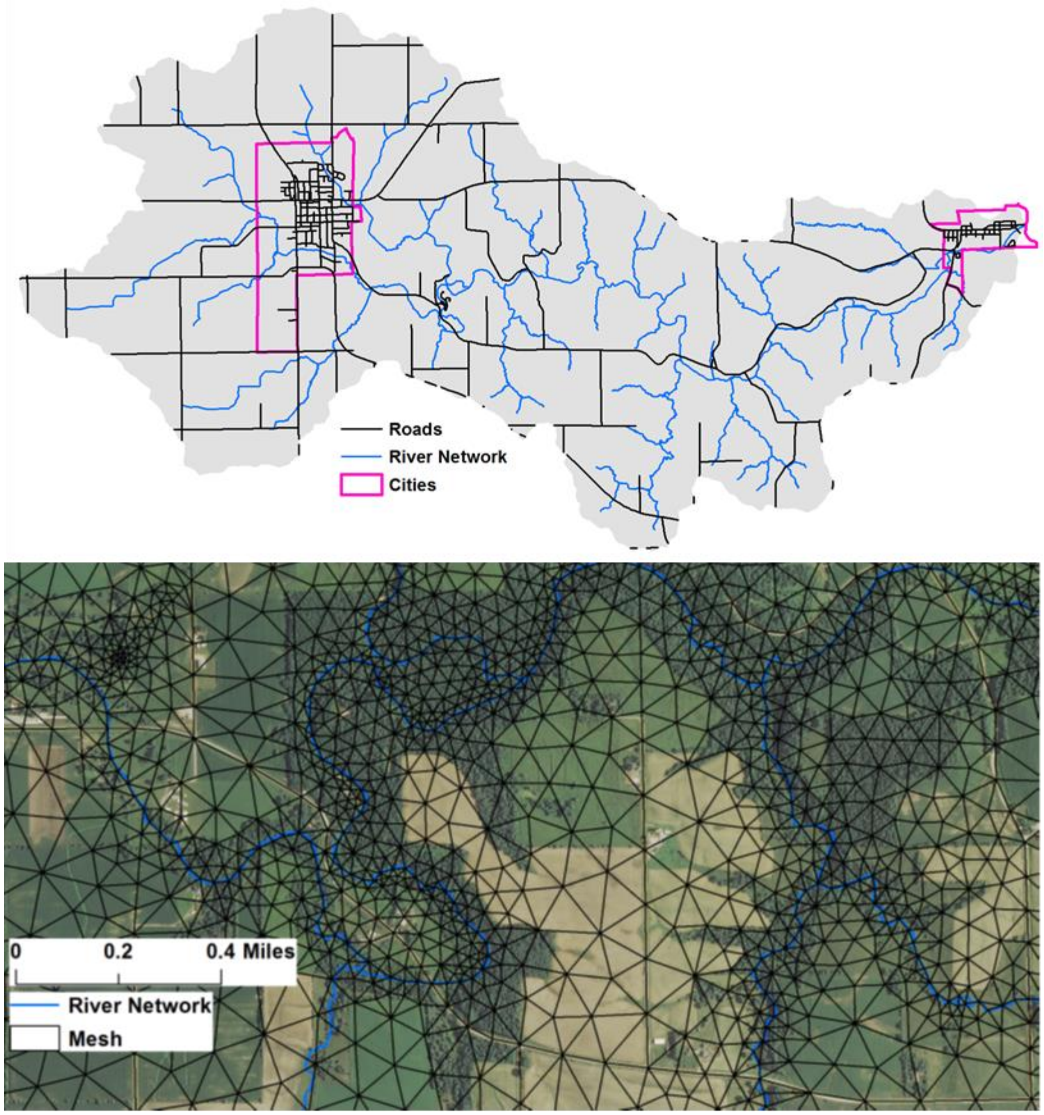


Figure 5.2. Otter Creek Watershed surface domain grid generation. Top: Boundaries for mesh generation. Bottom: Example location of the completed 2-D finite element grid.

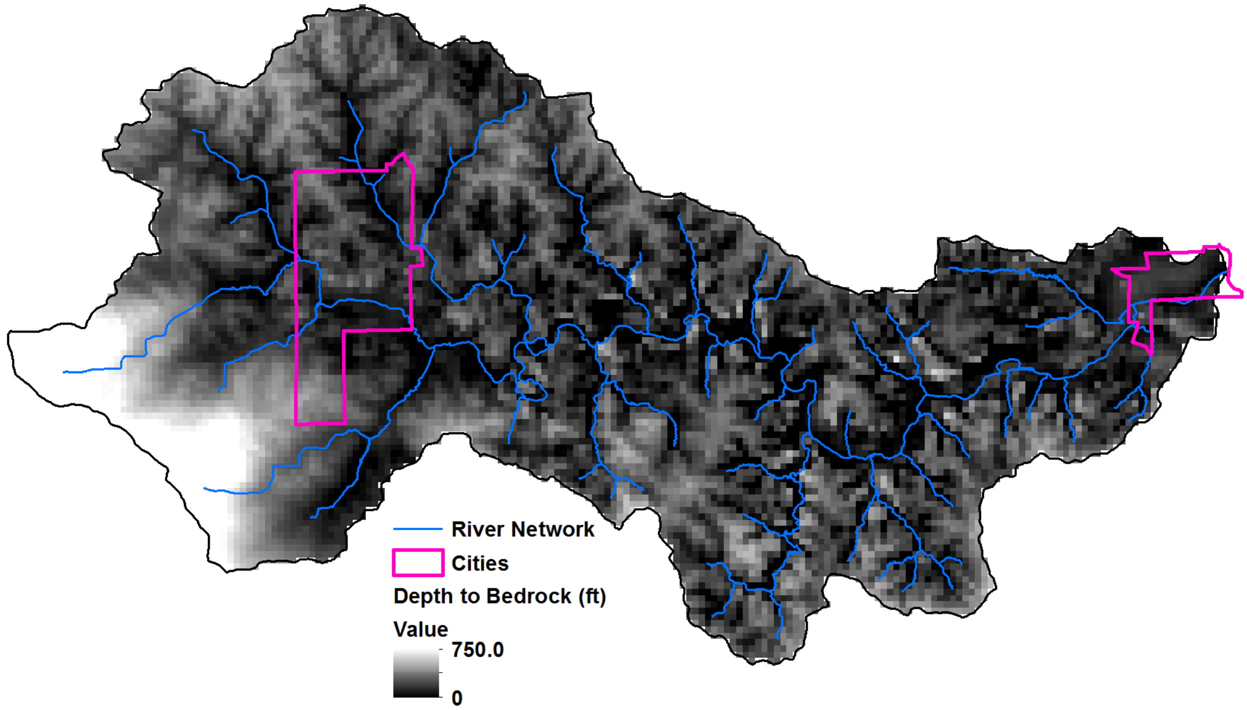


Figure 5.3. Estimated depth to bedrock as defined by the Iowa Geologic Survey (2010).

We projected the completed two-dimensional surface mesh downward to the estimated bedrock depth (see Figure 5.3), to form three-dimensional subsurface elements (Witzke et al., 2010). The subsurface was divided into two zones: three feet below the surface, and from the three-foot depth to the bedrock. Ten elements were spaced vertically through the top three feet of soil, such that the depths of the soil moisture sensors were explicitly included (2 in., 4 in., 8 in., 20 in.). The remaining element depths varied in increasing thickness from 2 feet to 6 feet near the impermeable layer. The increased number of numerical elements near the surface allowed for a more accurate representation of the interactions between the surface and subsurface domains (see Figure 5.4). The product of mesh generation was a 732,760 element three-dimensional modeling domain.

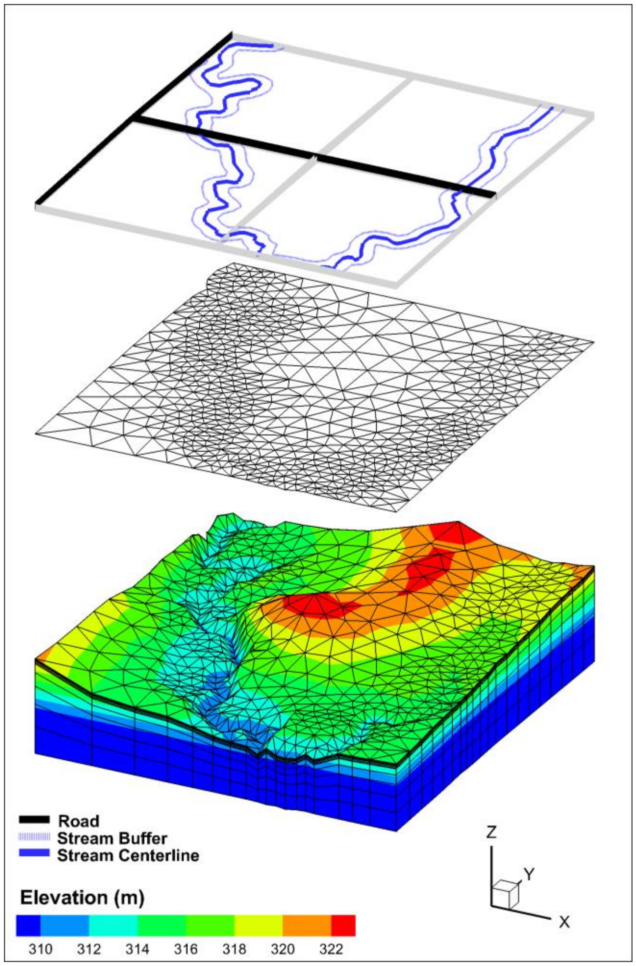


Figure 5.4. Generation of 2-D and 3-D mesh. Conceptual mesh generation though incorporation of important boundaries (top) to produce a 2-D mesh (middle), which was projected downwards to create a 3-D tetrahedral mesh (bottom). Vertical axis at 10:1 ratio.

## c. Attributing the Model

We used publicly available land use, soil type, and well log data to spatially describe surface and subsurface classifications.

### Surface

We assigned spatially variable land use and topographic information to each triangular surface element, relating the location to overland roughness, evapotranspiration properties, and land surface slopes, respectively.

The National Land Cover Database 2006 (Fry et al., 2011) provided spatially variable land use classifications. We simplified land classifications into five categories: agriculture, grassland, forest, developed, and water. We then assigned these classifications to the appropriate elemental area (see Figure 2.8). The five surface land use classifications relate surface elements to overland flow resistance parameters and vegetation properties. Li et al. (2008) thoroughly described the parameters used to calculate the actual evapotranspiration (Kristensen and Jensen, 1975).

The Iowa Geological and Water Survey aggregated Light Detection and Ranging (LiDAR) datasets for the entire state of Iowa between 2007 and 2010 (Iowa Geological and Water Survey, 2010). We used this LiDAR data to describe the landscape topography. We derived one-meter Digital Elevation Models of bare ground surface data from the LiDAR products. A high spatial resolution topography enabled accurate identification of stream, roadway centerlines, watershed boundaries, and culvert locations for mesh generation. We extracted element elevation data representing the land surface directly from the one-meter resolution elevation model. Mesh generation boundaries ensured that the extracted elevation data coincided with roadways and stream centerlines.

## Subsurface

We divided subsurface stratigraphy into surficial soils and deeper geologic soils. We described the surficial three feet of subsurface depth as spatially variable, vertically uniform to soil data. We used an aggregation of well log data creating a homogeneous deeper soil layer to represent the deeper subsurface.

Researchers used the Soil Survey Geographic (SSURGO) database (Soil Survey Staff, 2014) (Figure 2.5) to describe the top three feet of the subsurface. They allocated the flow properties based on soil texture classification and assigned the mean textural properties.

The remaining deeper geology below the top three feet of soil was described by historical well logs at 133 sites across the watershed and surrounding area (IGS 2015) (Figure 5.5). General trends in the geologic interpolation indicated that the deeper geologic materials were comprised of clay, loess, sandy clay, and shale. We volume weighted the soil properties in this deeper region to produce an aggregated representation of geologic properties. The homogeneous representation of hydraulic properties (described above) from one meter deep to the estimated depth to bedrock represented the deeper subsurface.

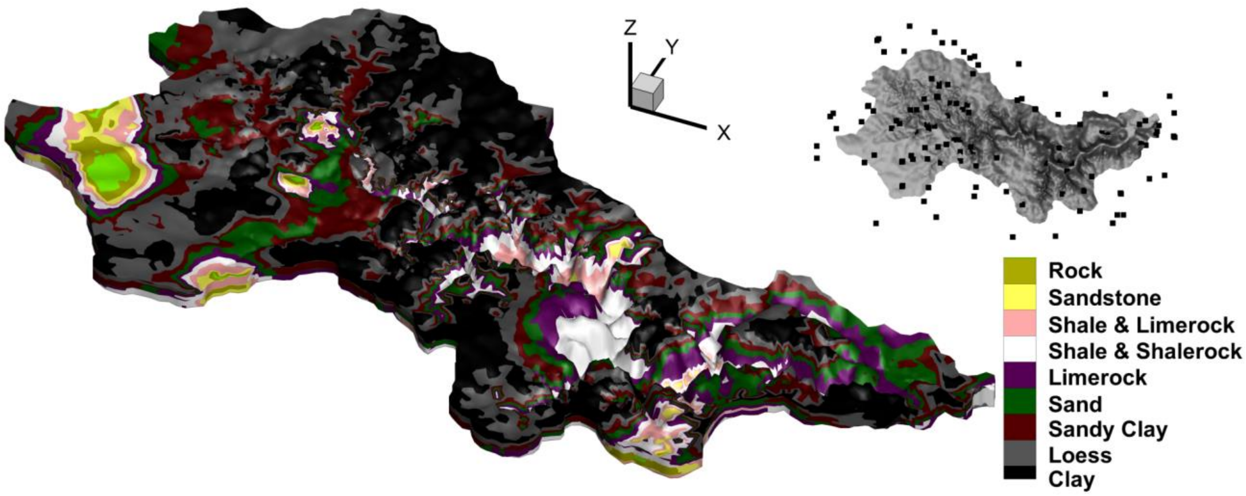

Figure 5.5. Inverse distance interpolation of well log points onto the mesh up to 150 ft. deep. The inset shows geologic well log locations (133) within OC and the surrounding area.

## Meteorological Input for Hydrologic Simulation

We applied measured meteorological data for 2014 and 2015 from the Otter Creek Watershed for all annual simulations. This section describes the exact alterations to the raw data for input into numerical simulations.

Precipitation was measured at five locations within the watershed at 15-minute increments beginning April 19, 2014 (Figure 4.1). We aggregated the raw data to the hourly time step to produce a uniformly distributed rainfall at hourly time steps. We further altered the input precipitation time series by incorporating solid form snow storage when temperatures dropped below freezing (32 F). We aggregated PRISM daily average temperature data ( PRISM Climate Group, 2016) for the 2014 and 2015 time periods at the centroid of the Otter Creek Watershed. When temperatures were below freezing, we assumed snow would accumulate on the land and be stored until temperatures rose above freezing. A degree day method (Natural Resource Conservation Service, 2004a) allowed us to use temperature as an index for a wide range of energy fluxes affecting the melting process. A difference of temperature to base temperature (freezing) allows us to calculate daily melt depths until the storage of snow has been depleted. For modeling purposes, we completed this analysis prior to simulation, whereby the daily melt flux was input as a rainfall rate into the domain. This process shifts the introduction of frozen precipitation into the early spring months, saturating near-surface soils and causing higher runoff potential. This process shifts the introduction of frozen precipitation into the early spring months, saturating near-surface soils and causing higher runoff potential.

We acquired daily potential evapotranspiration (PET) based on the Penman-Monteith approach and downloaded from the Iowa State AgClimate station at Nashua, Iowa (Iowa State University, 2015). A gap in PET data from April 1, 2014, to August 18, 2014, required supplemental PET data. Using time series on air temperature, dew point temperature, and cloud cover from Charles City, we estimated daily PET using a Penman approach (Shuttleworth, 1993). We combined the Charles City and Nashua PET data for further preprocessing. (Figure 5.6).

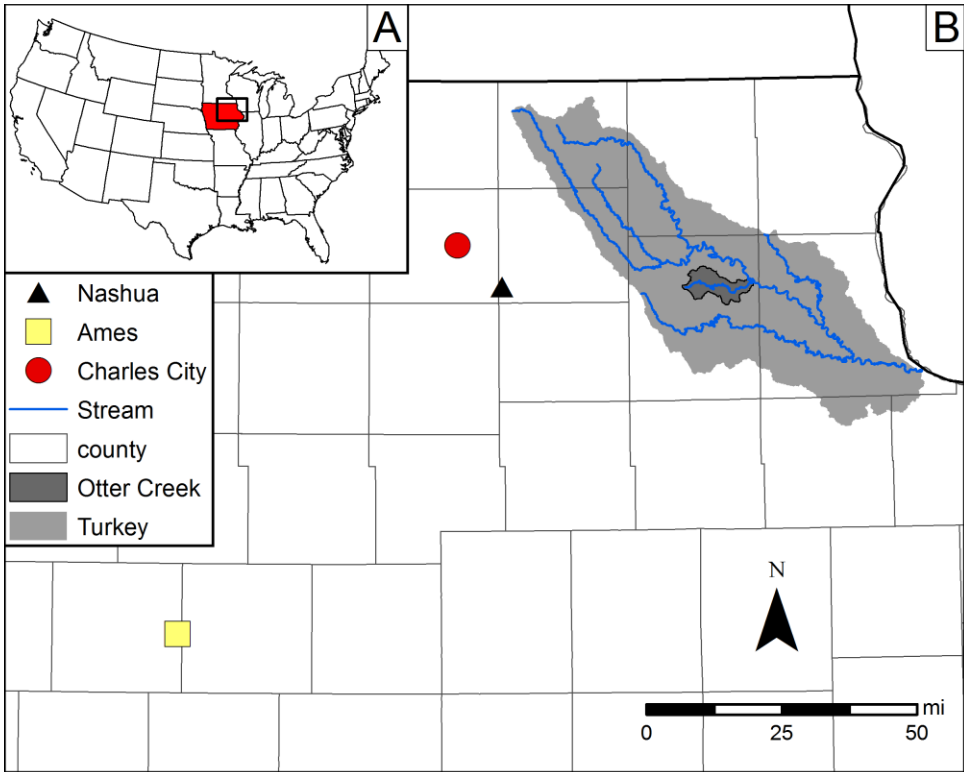


Figure 5.6. Data collection sites in Iowa: Ames SCAN – long-term water content (yellow); Charles City – supplemental meteorological data for PET calculation (red); and Nashua – PET data and supplemental hourly rainfall (black).

## Long-term Soil Water Content Record

Long-term measured soil water content data were available at only a few locations in the state of Iowa. The Soil Climate Analysis Network (SCAN) was developed to gain insight into the soil-climate dynamics through the NRCS (Natural Resource Conservation Service, 2004b, 2015; Schaefer et al., 2007). A nearby SCAN site in Ames, Iowa, measured continuous soil water content data from 2002 to 2012. Soil water content was measured at 2 in., 4 in., 8 in., 20 in., and 40 in. depths using a dielectric measuring device (Natural Resource Conservation Service, 2004b). We used the data to identify long-term soil moisture trends and as initial conditions to investigate antecedent moisture controls.

SCAN soil water content data were shown to vary with depth and time. We noted that shallower soils had increased soil moisture variability with lower median soil moisture values. As measurement depth increased, median soil moisture increased, and variability decreased. The highest median soil water values and lowest variability occurred in the months of March, April, and May, due to spring snowmelt and rainfall. June, July, and August were attributed with the highest variability and lowest median moisture values due to high evapotranspiration. Temporal trends held true at each depth.

## d. Calibration

We adjusted model parameters so that simulated results matched known annual ratios between components of the hydrologic cycle as closely as possible. We used the following target ratios: discharge to precipitation (*Q/P*), evapotranspiration to precipitation (*ET/P*), evaporation to evapotranspiration (*E/ET*), transpiration to evapotranspiration (*T/ET*), and baseflow to discharge ($Q_b/Q$). Table 5.1 presents the targets for the ratios. When evaluating the existing literature for these ratios, we gave preference to studies performed in Iowa or other agriculturally dominated Midwestern landscapes, but in some cases we used ratios from other locations.

Precipitation and potential evapotranspiration are the major meteorological drivers in physically-based coupled simulations. We used meteorological data measured in 2014 to run recursive simulations and ultimately determine model parameters (Ajami et al., 2015). A comparison of surface, near-surface, and groundwater storages from one year to the next indicated a convergence to a 1.0% change threshold after four years of model simulation. Figures 5.7 and 5.8 display results from the last year (4) of this recursive simulation.

In general, the calibrated water balance components adequately matched the calibration targets. Q/P was 36%, with ET/P representing the remaining 64%, slightly higher and lower than the respective target ratios. This indicates that the watershed over this period tended toward a “wet” condition. This was a reasonable result, as 2014 was wetter than normal (higher precipitation). An iterative cycle of a wet year pushes the model into a wet equilibrium. E/ET (36%) and T/ET (64%) allocated more water toward the evaporation component than the calibration targets. This is representative of a wet watershed condition. Evaporation is not limited near saturation, but transpiration is. Furthermore, evaporation acts in the upper part of the soil column, whereas transpiration throughout the top three feet. In a wet condition, more water is closer to the surface and available for evaporation. Baseflow accounted for approximately 65% of the total outflow, which is close to the upper limit of the calibration target range. The partitioning of water balance components over the iteratively run wet year responds in a logical pattern tending toward a wetter condition.

We further used data from the USGS discharge gauge (Figure 3.1) at the outlet of the Otter Creek Watershed to evaluate model performance. We selected a series of events occurring in June 2014 as an example time period in which three heavy rainfall events produced peaks in simulated and measured flow time series (Figure 5.8). In each of the three events, the initiation of streamflow at the outlet or the start of the hydrograph increase was consistent with measured data. Overall, the timing and magnitude of simulated hydrographs were adequate when compared to measured streamflow peaks.

Table 5.1. Annual ratios of hydrologic components used in the calibration and evaluation of the model. Q is total flow, P is precipitation, ET is evapotranspiration, E is evaporation, T is transpiration, and Qb is base flow.

| Ratio | Values | Sources |
|---|---|---|
| Q/P | 0.24 | Schilling et al. (2008) |
| | 0.27 | McDonald (1961) |
| | 0.24 | Hoyt (1936) |
| | 0.29 | Estimated with measured data |
| ET/P | 0.76 | Schilling et al. (2008) |
| | 0.73 | McDonald (1961) |
| | 0.76 | Hoyt (1936) |
| E/ET | 0.26, 0.33 | Kang et al. (2003) |
| | 0.23, 0.35 | Wang et al. (2013) |
| T/ET | 0.67, 0.74 | Kang et al. (2003) |
| | 0.65, 0.77 | Wang et al. (2013) |
| | 0.61±0.15 | Schlesinger and Jasechko (2014) |
| $Q_b/Q$ | 0.56 | Schilling et al. (2008); Schilling and Libra (2003) |
| | 0.45-0.66 | Schilling (2005) |

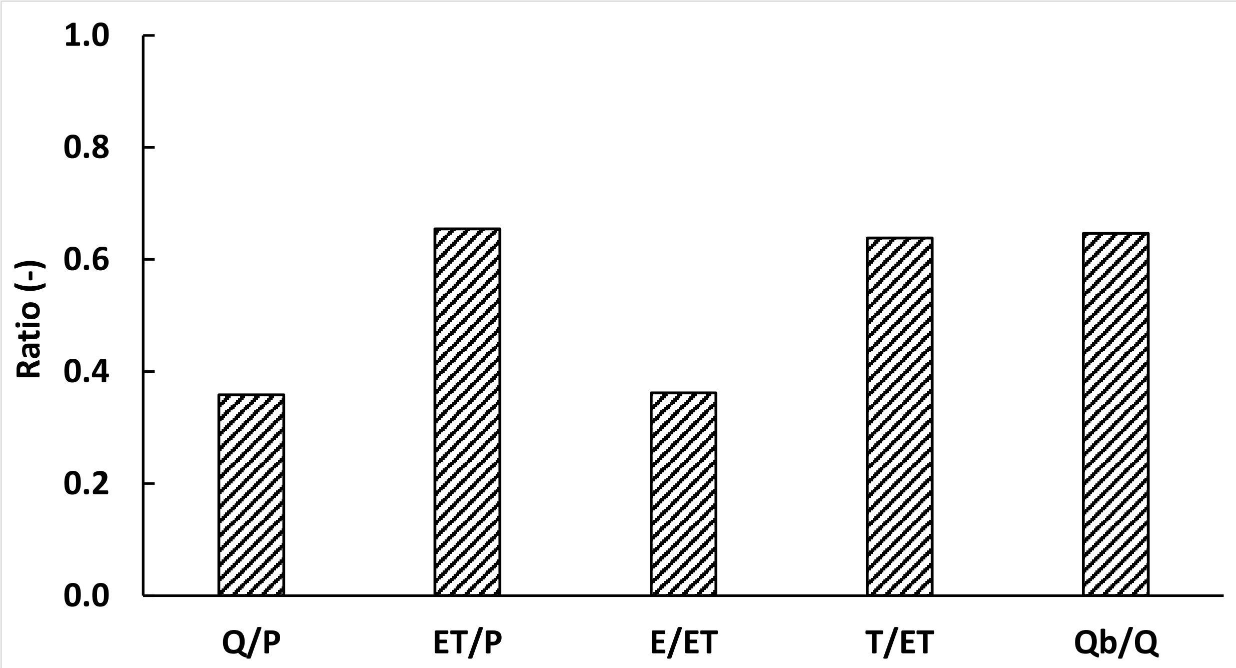


Figure 5.7. Calibration. Estimated annual ratios of hydrologic components obtained with the final set of parameters (fourth year).

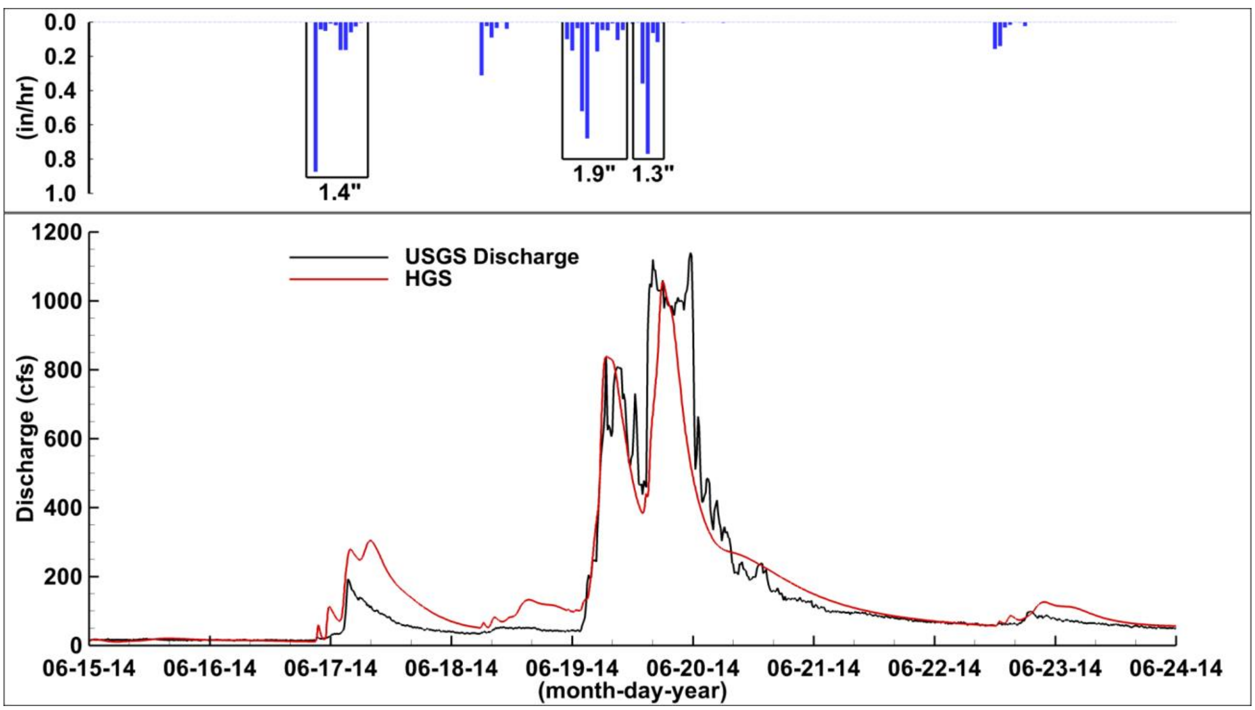


Figure 5.8. Discharge (volumetric flow rate in cubic feet per second), measured at the Otter Creek USGS station (USGS 05411900) and simulated (HGS). Precipitation (in inches per hour) is the average of the measurements at the five rain gauge platforms.

## e. Validation

The purpose of model validation is to assess the model's capacity to match field observations over periods that differ from the calibration time window. We used rainfall and potential evapotranspiration data collected in 2015 to drive the model, while we kept constant all the model parameters determined in the calibration phase. Figures 5.9 and 5.10 display results of the validation period.

Over the validation period, we divided precipitation into 38% stream flow and 62% ET. These water balance components trend well from a wet state (2014) initial condition toward the calibration target of 30% Q, 70% ET. We divided ET into 36% E, 64% T, consistent with calibration results. Baseflow slightly decreased to 64%, fitting the water balance metrics well.

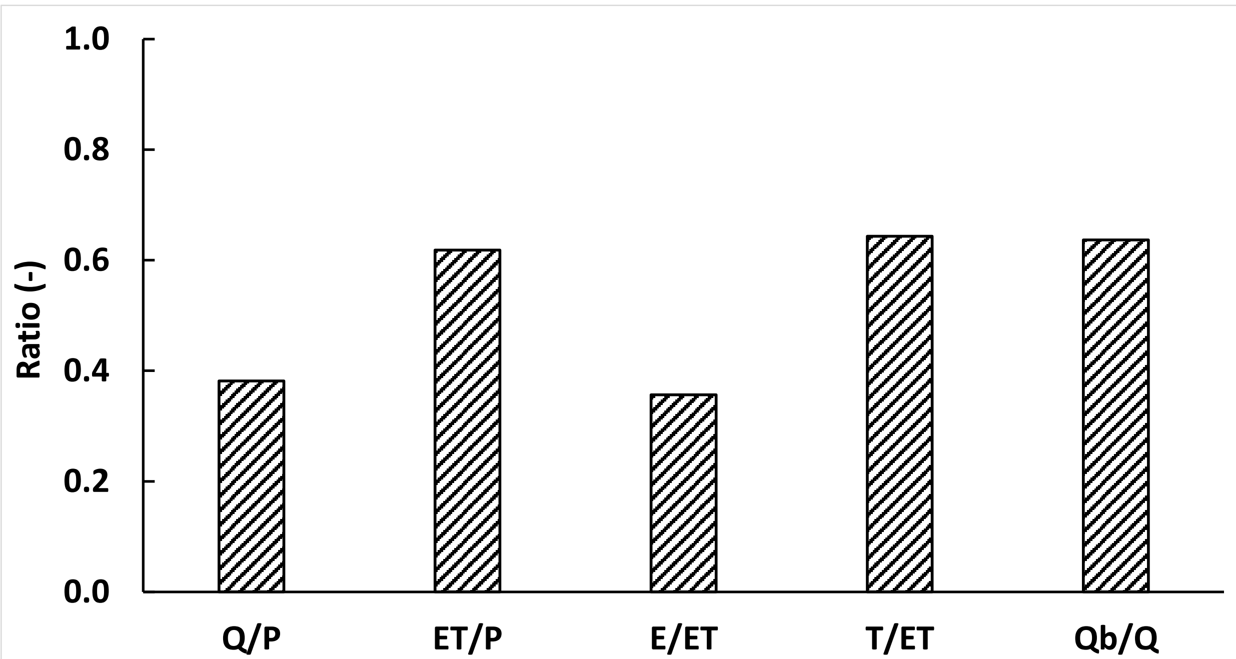


Figure 5.9. Validation. Estimated annual ratios of hydrologic components obtained with the final set of parameters.

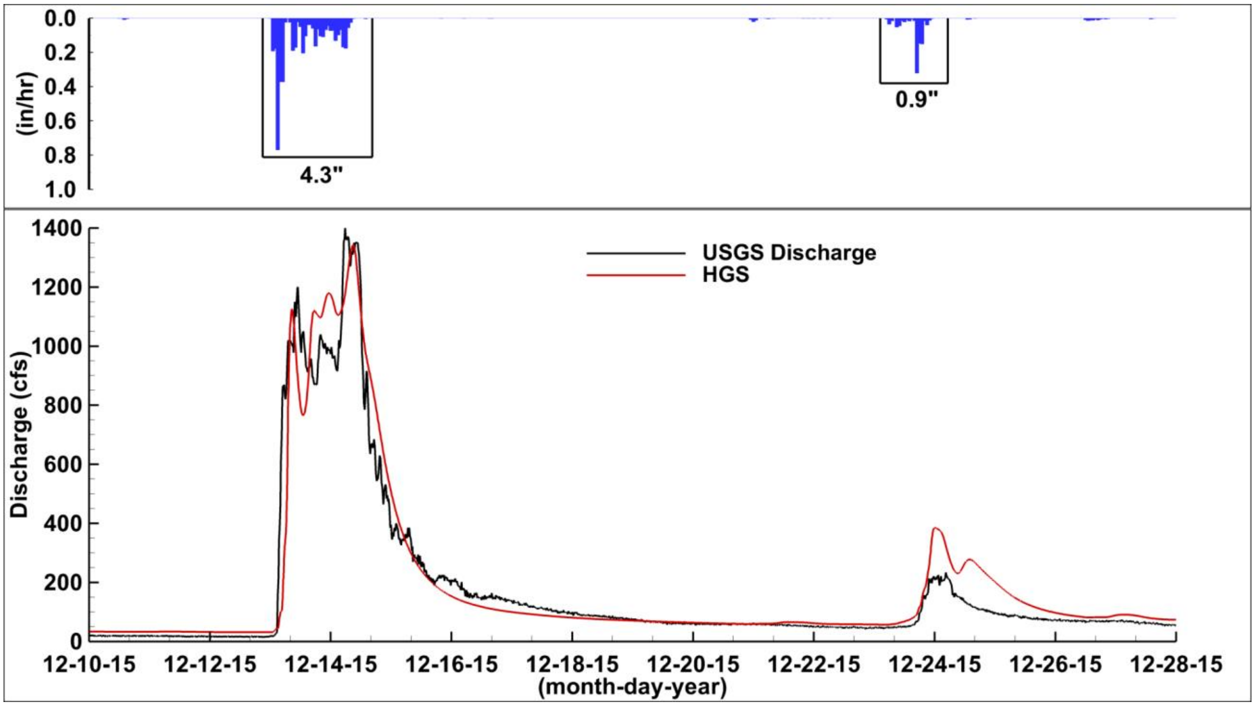


Figure 5.10. Discharge (volumetric flow rate in cubic feet per second), measured at the Otter Creek USGS station (USGS 05411900) and simulated (HGS). Precipitation (in inches per hour) is the average of the measurements at the five rain gauge platforms.

We can attribute differences in calibration and validation results to model complexities and calibration time period. The Otter Creek Watershed has a relatively fast time of concentration; the basin responds to rainfall within hours. Although we collected rainfall data locally, refining the space and time distributions of rainfall can greatly impact a watershed response. The largest differences were due to the 2014 calibration period. This period was selected for iterative calibration because there were only two years of local data. The year 2014 represented a wet year; starting the watershed off in a wet condition shifted water balance components and stream flow response in a manner to produce more flow per unit precipitation annually. This caused the surface soils to be wetter. Overall, the calibration and validation periods succeeded in depicting the expected variation from the calibration targets; they represent overall watershed processes well.

## f. Localized Impact of the Projects

We used the HydroGeoSphere numerical model of the Otter Creek Watershed to analyze localized project impacts, providing a comprehensive numerical depiction of water dynamics. In this section, we will describe the project incorporation into the HGS model and the addition of flood mitigation projects, which we tested under high and low synthetic potential peak flow reduction scenarios.

## Project Inclusion (Mesh and Elevation)

We incorporated The Open Range project (Table 4.1 and Figure 4.2) into the mesh through two components: the structural embankment centerline, and the estimated inundation limits of the emergency spillway (Figure 5.11). We extracted elevation contours at the emergency spillway from the 10 ft. resolution DEM and incorporated them into the mesh. We refined the mesh in proximity to the detention structures, ensuring the appropriate representation of inundation, flow, and storage. We assigned the following per design specification: the elevation of the top of dam, the normal pool outlet, and the emergency spillway (Figure 5.11). The remaining nodal elevations remained consistent with the original LiDAR derived elevation data.

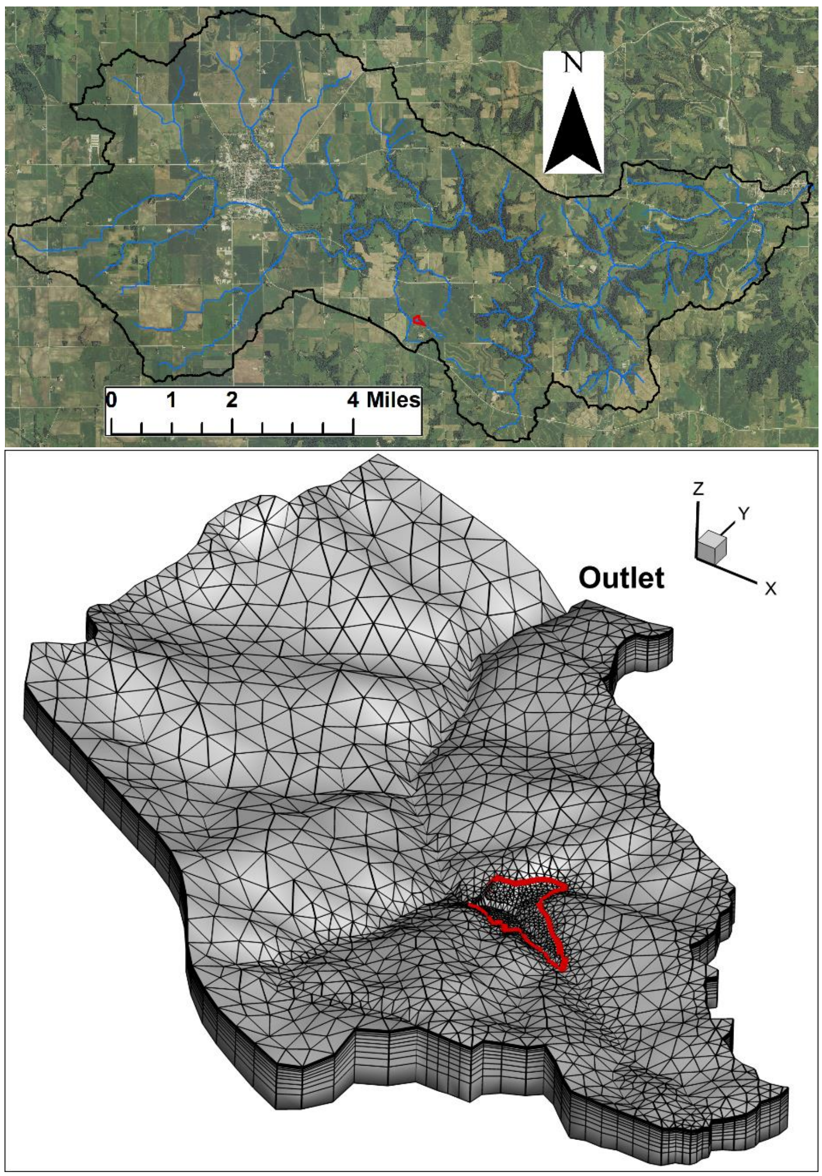


Figure 5.11. We incorporated the Open Range project into the numerical mesh through embankment centerline and estimated maximum inundation extent (red line). *Note the additional refinement of the numerical mesh near the project embankment and estimated inundation area.

## Synthetic Analysis of Watershed Response with Incorporated Flood Mitigation Measures

We analyzed the Open Range project for an array of antecedent wetness conditions and pre-event project storage conditions, as well as for a given design storm precipitation event. Antecedent soil wetness refers to how wet the soil was prior to precipitation; the wetter the soil, the greater the basin's peak flow response. Pre-event storage refers to the amount of surface water contained behind a flood mitigation detention structure prior to a precipitation event. An empty project condition provides a reduced peak flow when compared to a pool storage condition. Similarly, as the depth of rainfall increases, the watershed response increases in a nonlinear manner. This section describes the range of watershed responses to precipitation depth, antecedent soil moisture, and pre-event project storage.

### *Synthetic Precipitation*

We developed hypothetical storms for comparative analysis of the Open Range project. The hypothetical storm applies a uniform depth of rainfall across the entire model domain with the same timing everywhere. We used a NRCS Type-II distribution, 24-hour storms for all the simulations. We derived point precipitation values (rainfall depths) for the 50-year average recurrence interval (5.7 in.), 24-hour design storms using the online version of NOAA Atlas 14 – Point Precipitation Frequency Estimates (Perica et al., 2013). We determined point precipitation frequency estimates at the basin centroid.

### *Antecedent Soil Moisture*

Numerous methods are available to incorporate antecedent moisture into hydrologic models, but they are not directly applicable to a coupled surface-subsurface model, which dynamically varies soil moisture spatially and with depth. For this study, we aggregated soil moisture data for a 10-year period beginning January 1, 2002, from the Soil Climate Analysis Network (SCAN, Ames location) (Natural Resource Conservation Service, 2015). Without prior knowledge of a vertical distribution to represent soil moisture variability, we applied a non-parametric approach. This study treated initial soil moisture as an independent variable over a range of exceedance probabilities based on an estimated cumulative distribution function (CDF) of measured soil moisture.

We normalized, ranked, and plotted the hourly soil moisture data with measured soil porosity at each depth (Figure 5.12) (Natural Resource Conservation Service, 2004b, 2015). We extracted the 98% and 50% exceedance probability soil moisture contents at each measurement depth, representing very wet (98%) and normal soil wetness (50%) conditions. We defined initial soil moisture conditions in the top three feet of the model subsurface to match, on average, the profiles presented in Figure 5.12. Near stream channels, we assumed the soil would have a saturation value of 1.0 for the profile depth.

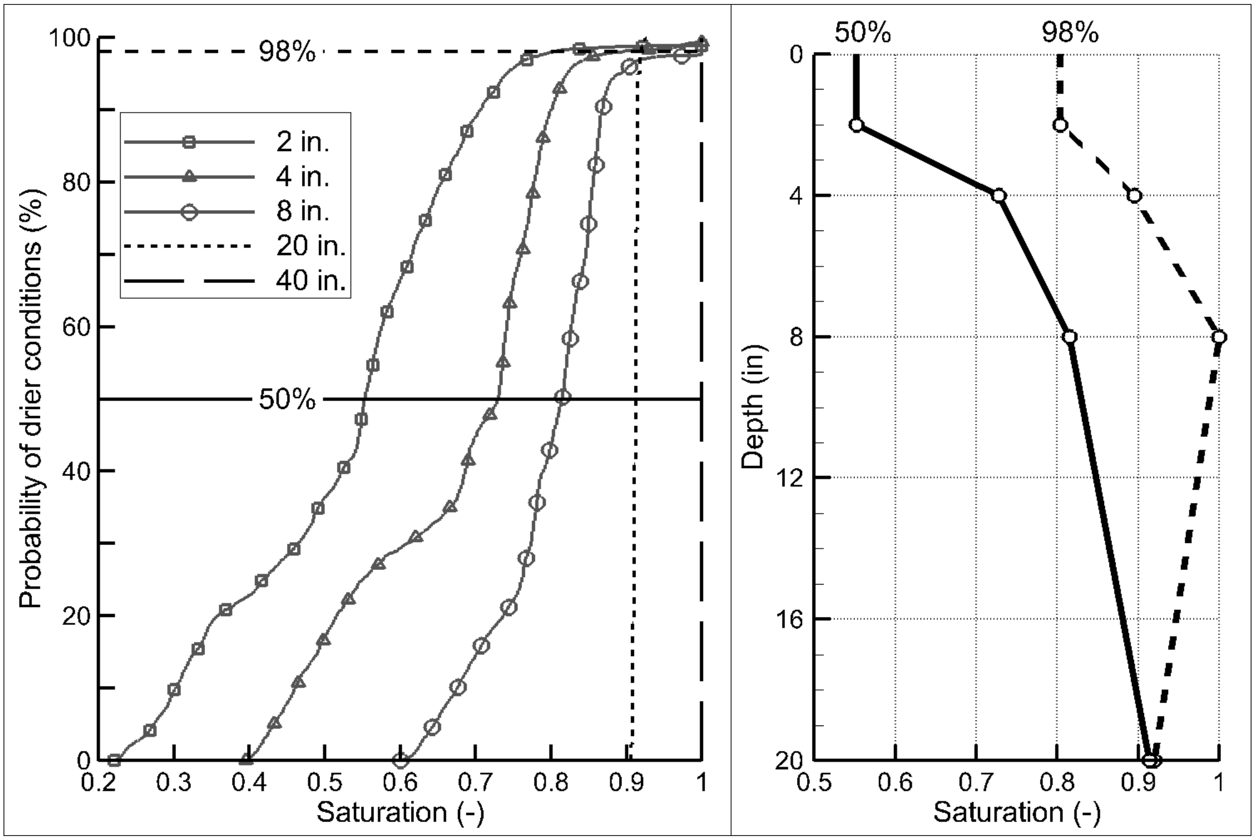


Figure 5.12. (Left) Ranked saturation values at five measured depths. Horizontal lines represent the initial conditions for event simulation. (Right) Soil water initialization saturation for over the first 20 in. The 40 in. initialization state was equal to 1.0 for all chosen exceedance probabilities. Circles indicate soil measurement location; lines indicate linearly interpolated HGS input values.

### *Detention Basin Storage Initial Condition*

We have previously noted that peak flow alterations from flood control structures are dependent upon the initial storage. We chose three project conditions to adequately encompass the detention basin initial conditions: no projects, full projects, and empty projects. These conditions represent a control (no projects), a maximum peak flow reduction potential (empty projects), and a minimum peak flow reduction potential (full projects). We initialized full project simulations with water up to the emergency spillway. We initialized empty project scenarios without surface water stored behind the structures. This amount of storage capacity is unlikely, as a combination of low precipitation, high evapotranspiration, and/or high infiltration over a prolonged duration would be required to empty the surface storage. The empty project scenario captures the maximum magnitude of peak flow reduction this suite of practices is capable of.

### *Synthetic Storm Analysis*

We performed an analysis to quantify the impact of the Open Range project. We selected the 50-year average recurrence interval rainfall event for comparison of pre- and post-project construction basin response under heavy rainfall. We isolated the local area containing project, representing the location of maximum project influence, for further analysis. We extracted hydrographs from the outlet of the model domain (Figure 5.11).

We selected soil moisture antecedent conditions from the 10-year aggregation described in the *Antecedent Soil Moisture* to represent a normal wetness condition (50%) and a high wetness condition (98%). This range encompasses a reasonable range of flood-producing soil moisture conditions. We simulated three event initial conditions. (1) We defined the normal wetness condition as no pre-event water held behind the structures. This situation provides the maximum storage capacity available, representing an upper bound on flood mitigation potential. (2) We defined the normal wetness condition as a full project condition. This was representative of the most likely circumstance. (3) We defined the high wetness condition as a full project initial condition, which represents a lower bound for peak flow reduction, as neither the soil moisture nor the projects have a large remaining holding capacity for this incoming heavy rainfall. In the simulations, water was allowed to flow downstream through the emergency spillway and the effect of the principal spillway (12" pipe) was neglected.

Figure 5.13 describes the varied peak flow response from initial normal wetness conditions, with and without the flood detention structure. The input design storm temporal distribution was such that about 40% of the precipitation depth occurred over 1.0 hour. Peak flows in each simulation always occurred after simulation hour 12. Without the flood detention structure, the peak flow was approximately 400 cfs. In comparison, maximum discharge for the empty project simulations was close to 300 cfs. The three hydrographs display a single peaked response. The maximum peak flow reduction (no-project vs. empty project) under initial normal wetness conditions is approximately 26% at the outlet of the model domain. Assuming full project initial conditions, the reduction was 22%. Over the first 24 hours, approximately 2.4", 1.8", and 2.2" of precipitation were transformed into runoff for the simulations with no project, empty project, and full project, respectively.

As expected, for a high soil wetness initial condition, predicted peak flows were higher than those experienced under initial normal wetness conditions (Figures 5.13 and 5.14). This is a result of the soil's limited capacity to store water under high saturation levels. Figures 5.13 and 5.14 show a comparison of the results; under high soil wetness, peak flows are approximately twice as much for both the no project and full project simulations. Under initial high soil wetness conditions, comparison of the no project and full project simulations shows a peak flow reduction of approximately 24% (Figure 5.14). This value is similar to that found under normal wetness conditions (Figure 5.13). It is worth noting that only 25% of the model domain (Figure 5.11, bottom) is located upstream of the Open Range project, and therefore hydrographs presented in Figures 5.13 and 5.14 are chiefly controlled by runoff generation processes that occur downstream of the pond.

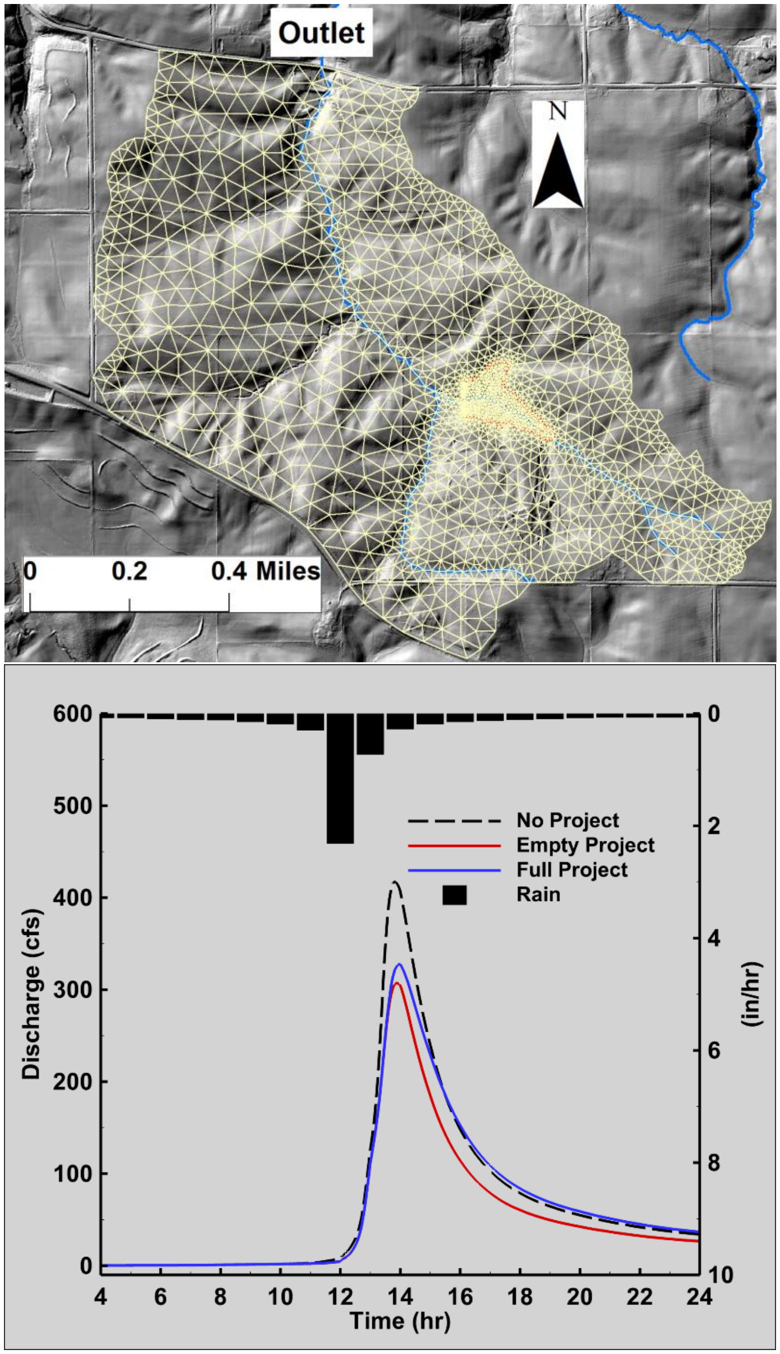


Figure 5.13. Hydrographs at the outlet of the model domain: a 50-year design storm under initial normal wetness condition.

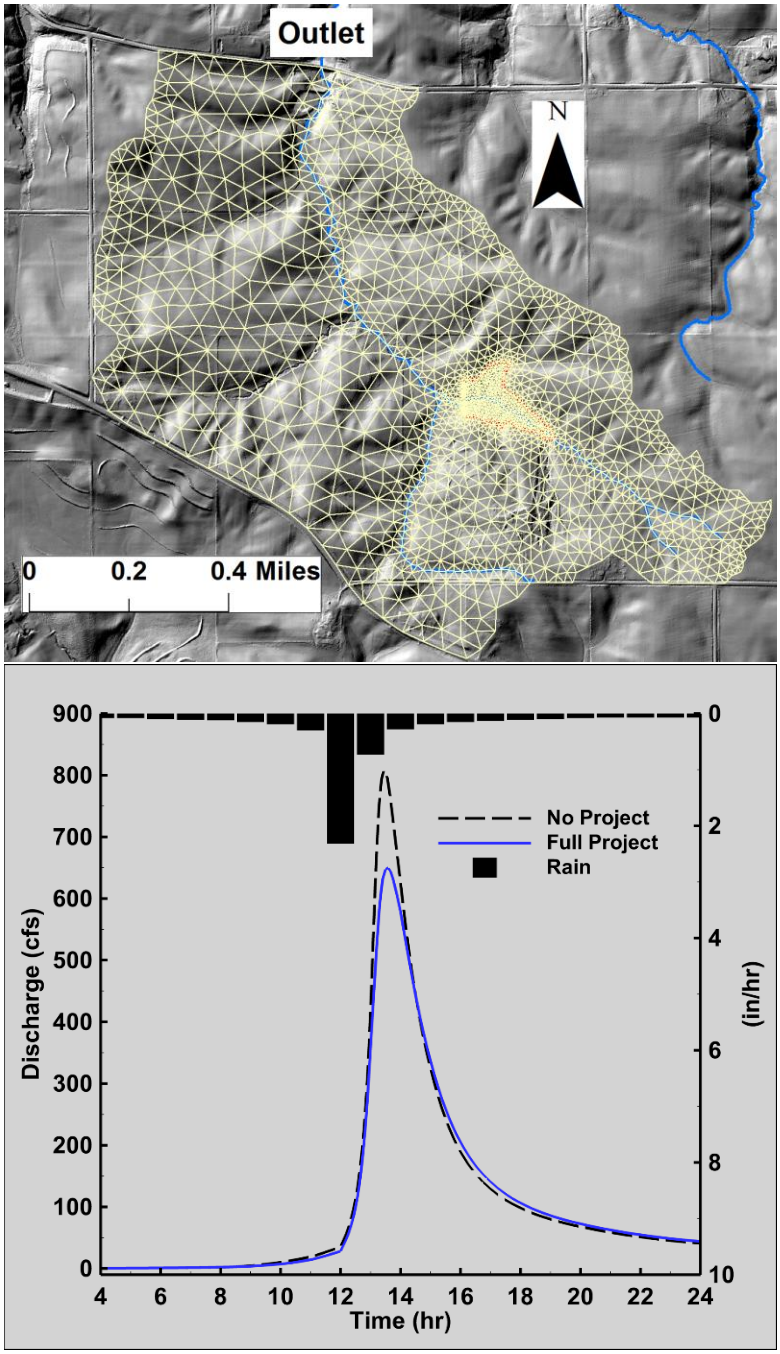


Figure 5.14. Hydrographs at the outlet of the model domain: 50-year design storm under initial high wetness condition.

The combined results of the normal to high wetness and empty to full project conditions adequately bounded the local effectiveness of the Open Range project. As wetness increased, peak flow reduction decreased; similarly, as the pre-event storage was filled, peak reductions were reduced at the outlet of the model domain. Project influence is expected to decrease as the model domain area increases. This implies that peak flow reductions at the outlet of Otter Creek due to the Open Range project are expected to be significantly smaller than those presented in Figures 5.13 and 5.14.

## g. June 2008 Flood Event

As documented in the Hydrologic Assessment of the Turkey River Watershed (IIHR 2015), the June 2008 flood produced some of the greatest discharges and stages on record throughout the Turkey River Watershed. USGS stations in the Turkey River Watershed reported large discharges, including the following: Eldorado (50,100 cfs., June 9); Elkader (40,500 cfs., June 10); and Garber (45,500 cfs. June 10). The flooding that occurred in June 2008 was set up by the wet fall of 2007, followed by abundant snowfall over the winter of 2007–08, and then a wet spring in 2008. Precipitation from December 2007 through May 2008 was the second wettest on record from 1895–2008. NEXRAD Stage IV radar rainfall data show Otter Creek received up to 3.5" in a period of 24 hours in June 2008 (Figure 5.15).

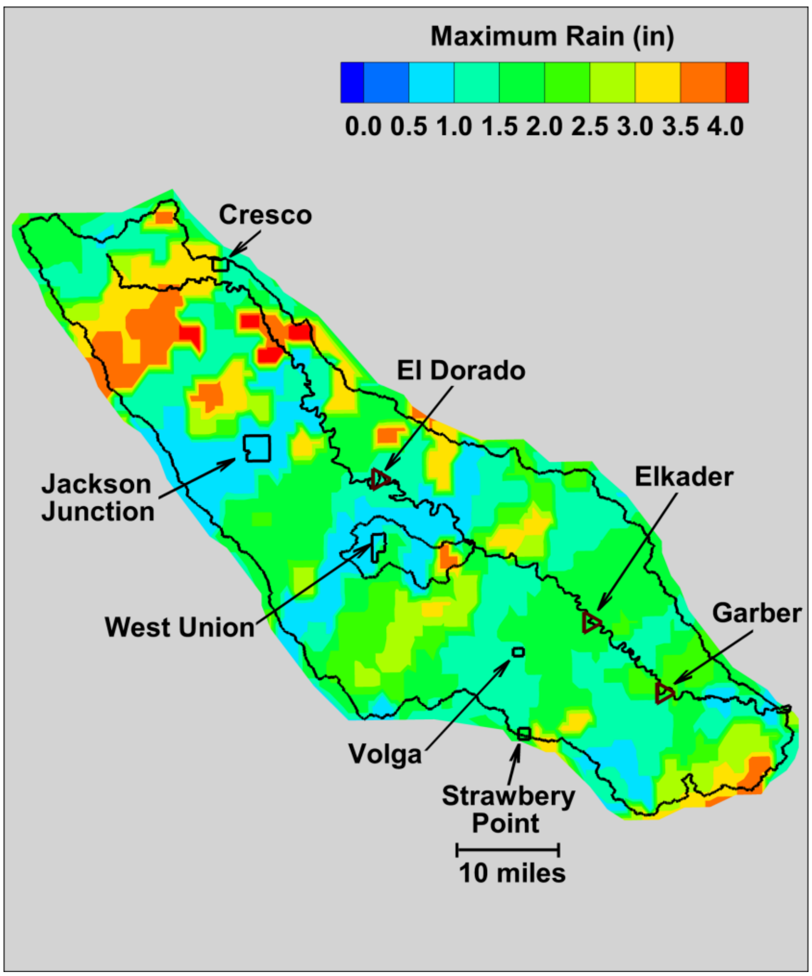


Figure 5.15. Turkey River Watershed, June 2008: NEXRAD Stage IV radar rainfall with maximum accumulated rain after 24 consecutive hours.

### *2008 Rainfall*

We initialized the simulation for June 2008 assuming relatively wet conditions in the subsurface. We assumed that this provided a good representation of the watershed on June 1, 2008. Beginning on June 1, 2008, through June 30, 2008, we used Stage IV radar rainfall as the precipitation input (Figures 5.15 and 5.16). The National Center for Environmental Prediction (NCEP) produced the Stage IV dataset by combining Stage III radar rainfall estimates produced by the 12 National Weather Service (NWS) River Forecast Centers across the continental United States into a nationwide 4 km x 4 km (2.5 mile x 2.5 mile) gridded hourly precipitation estimate dataset. Stage IV radar rainfall estimates are available from January 2002 – present. Use of radar rainfall estimates provides increased accuracy of the spatial and temporal distribution of precipitation over the watershed; Stage IV estimates provide a level of manual quality control performed by the NWS that incorporates available rain gauge measurements into the rainfall estimates. We resampled NEXRAD Stage IV data onto a grid with approximately 2-km spatial resolution to run the June 2008 simulations with numerical stability (Figure 5.16).

### *Watershed-wide Response*

We applied spatially variable Stage IV precipitation to the HGS Otter Creek numerical watershed model. Heavy precipitation on the evening of June 7 and through June 8, 2008, induced a peak flow rate of 1,840 cfs. Figure 5.16 shows precipitation and surface water for June 07 at 7 pm (frames a) and b)) and June 08 at 6 am (frames c) and d)). In addition, the bottom frame in that figure displays two predicted hydrographs. The red line shows the hydrograph at the outlet of the watershed, and the blue line displays the hydrograph in the middle of the watershed (see white dashed line in frame b). Heavy rainfall varied in intensity across the watershed, forcing streams with the most intense rainfall rates to expand and exit their banks, causing overland flooding. The Otter Creek Watershed responds rapidly to rainfall, routing water across the landscape into stream channels and out of the system in a matter of hours. NEXRAD Stage IV precipitation data recorded 9.4 inches in the simulated window. HGS simulations predict that approximately 31% of that precipitation was transformed into streamflow in the same time window.

Model results suggest that the eastern half of the watershed plays a larger role in streamflow generation than the western half. The streamflow volume predicted in the middle of the watershed (Figure 5.16, blue line in bottom frame) accounts for 44% of the total volume predicted at the outlet. This is consistent with the land surface slopes presented in Figure 2.7, which shows steeper slopes, and therefore higher runoff generation potential, in the eastern part of Otter Creek.

## h. Summary and Conclusion

This section described the local project influence on a synthetic heavy rainfall event under a range of soil and project storage initial conditions. The results indicated that in the modelled area, the Open Range project (see Table 4.1) could locally provide a peak flow reduction of approximately 25%. Section 6 will investigate the additive flow reduction effects of all the projects at the larger basin.

Physically-based coupled surface-subsurface modeling offers many capabilities important for the investigation of flood mitigation strategies. Physics-based modeling offers a fundamental approach to fluid movement though the surface and subsurface domains. We can parameterize surface and subsurface domains by measurable known quantities. We were able to realistically incorporate the Open Range project into the model with altered elevations that directly mimicked the natural case. We dynamically formed inundation extents and stream channels without explicit numerical representation. Baseflow is physically represented through subsurface surface exchange. Furthermore, these incorporations allow the investigation of antecedent moisture and pre-event storage in a realistic manner.

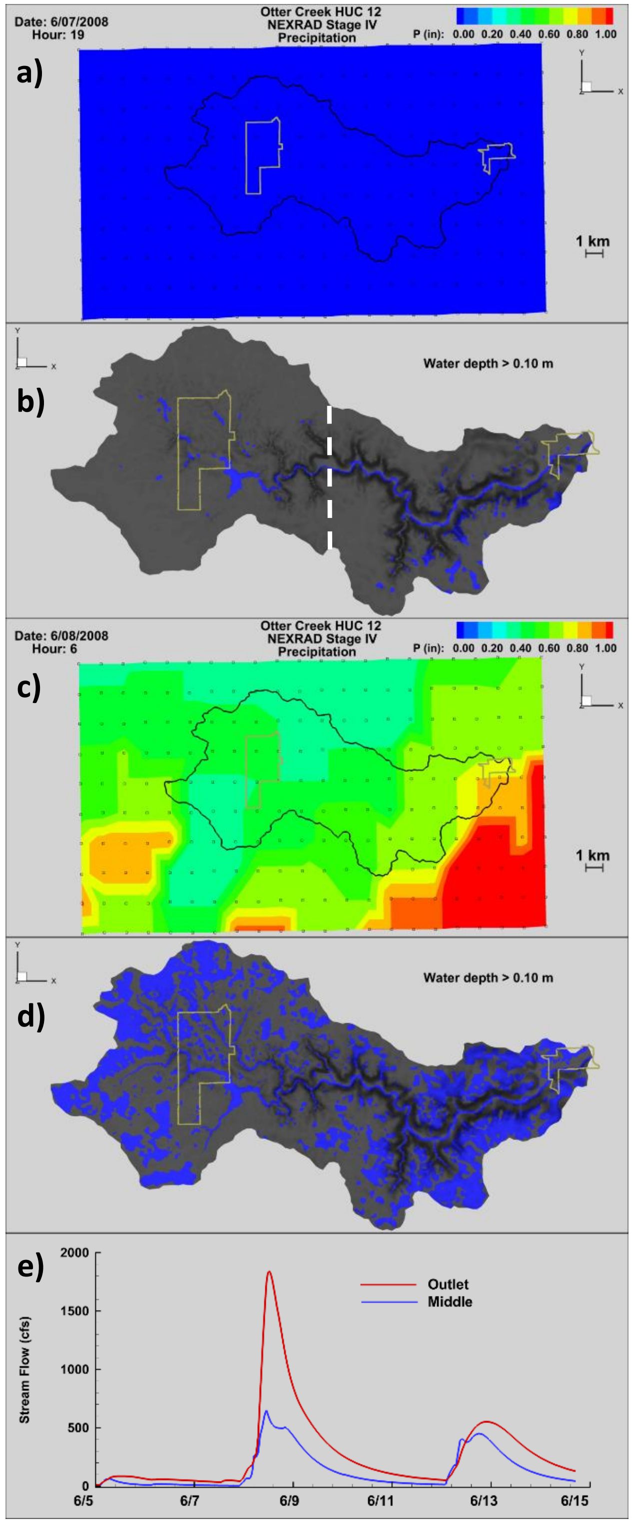


Figure 5.16. Otter Creek Watershed response to NEXRAD Stage IV radar rainfall for the June 2008 flood: a) and b) Rainfall and stream inundation extent on June 7 at 7 pm.; c) and d) Rainfall and stream inundation extent on June 8 at 6 am.; e) Predicted hydrographs at the outlet and in the middle of the watershed (see dashed line in frame b).

The drawbacks of this style of modeling are the extensive time required to set up, calibrate, and validate the model. Simulation run times often exceed 72 hours for a year of simulation time, reducing the model's capability to handle long-term datasets with accuracy. Another style of modeling can better complete a realistic evaluation of the structures over long-term historical meteorological forcing. The next section describes a simplified approach to the incorporation of realistic fluid dynamics without comprehensively solving the fundamental equations of fluid mechanics. This approach allows for reduced computational time, increased historical simulation, and a comprehensive view of peak flow reduction over a long period of historical events.

# 6. Analysis of Watershed Scenarios

This section summarizes the development of a long-term continuous simulation computer model for the Otter Creek Watershed. We performed the modeling using the Environmental Protection Agency (EPA) Hydrological Simulation Program–FORTRAN (HSPF) Version 12.2 (Bicknell et al., 2005). HSPF is designed to make long-term continuous simulations of hydrologic (rainfall-runoff) and water-quality (e.g., nutrient) processes of a watershed. The model has been used for water quantity and quality simulation for large and small watersheds across Iowa (Donigian et al., 1983a, 1984; Bradley et al., 2015) and the United States. For instance, a community effort has used the Chesapeake Bay Watershed HSPF model for many years to study water management and restoration options for inflows to the threatened Chesapeake Bay. The remaining sections describe the model representation of the Otter Creek Watershed.

## i. Otter Creek HSPF Model Development

The Otter Creek HSPF model is based on the Turkey River HSPF model developed by the Iowa Flood Center (Leach, 2015). The model simulates the entire 1,693 square mile Turkey River watershed, and makes predictions at 710 river locations. It simulates runoff from the land surface for seven different land use types, and routes the runoff downstream through the river network. The developers calibrated the model over a twenty year period, and verified its predictive ability for the 44-year period not used in calibration. They performed the simulations for a 64-year period (1948 to 2012) (Leach, 2015).

For this study, we extracted portions of the calibrated Turkey River HSPF model for simulation of Otter Creek. The original model used 22 sub-basins to represent the Otter Creek watershed (at 47 square miles). This resolution is too coarse to evaluate the effects of projects on flooding. Therefore, we created a more detailed representation of the Otter Creek watershed to represent its stream network and the movement of water through the watershed. To represent runoff processes in the Otter Creek HSPF model for individual land use types, we used the same estimated land surface model parameters as in the Turkey River HSPF model. The following sections describe the data and model set up we employed for the Otter Creek HSPF model.

### Historical Weather Inputs

Historical weather information is the main time series input driving an HSPF watershed simulation. Figure 6.1 shows the weather stations with long-term records used to construct hourly weather inputs for the Otter Creek HSPF model.

The closest long-term weather stations to Otter Creek are at Fayette (Cooperative Observer ID IA132864) and Postville (Cooperative Observer ID IA136766). The stations collect daily precipitation and air temperature data; we extended the record for the Otter Creek HSPF model to cover observations from October 1948 through September 2013 at these sites. There are gaps in the records (observations are missing or incomplete). We filled these gaps by interpolating data from nearby daily weather stations. Then we created continuous hourly precipitation and temperature time series from the daily data to represent the conditions at Otter Creek. We disaggregated the daily precipitation into hourly time steps using the precipitation pattern at nearby hourly stations: Strawberry Point (IA138009), Calmar NE (IA131126), Spillville

(IA137855), McGregor (IA135315), Lynxville Dam 9 (WI474937), and Shell Rock 2W (IA137602). We generated hourly temperature time series from daily records of maximum and minimum temperature using a fixed daily cycle.

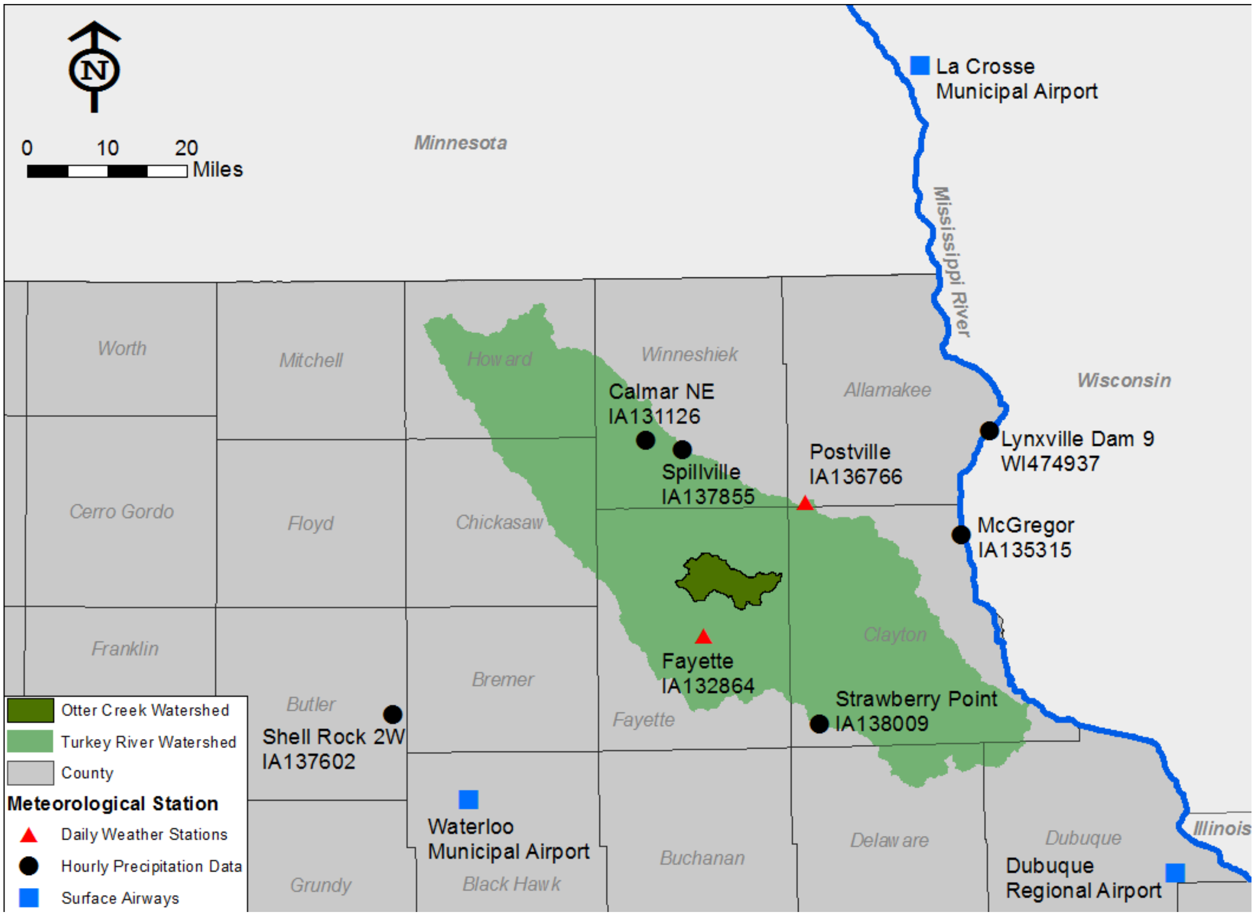


Figure 6.1. Weather stations used in the HSPF model of Otter Creek. The Fayette and Postville stations provides long-term daily precipitation and temperature data. We used hourly precipitation data at nearby stations to disaggregate the daily precipitation to an hourly time step. Other weather inputs, such as cloud cover, wind speed, and dew point temperature, came from the surface airways stations located at airports.

HSPF also requires time series inputs on cloud cover, wind speed, and dew point temperature. These data are used primarily in the cold season to predict snowfall and snow accumulation and melt. Cloud cover, wind speed, and dew point temperature are measured at surface airways stations, located a certain nearby airports. The closest stations are at Decorah Municipal Airport, Oelwein Municipal Airport, and Prairie Du Chien Municipal Airport. Unfortunately, their records begin in 1995 or later. However, the long-term stations at Dubuque Regional Airport, Waterloo Municipal Airport, and La Crosse Municipal Airport surround the watershed; their records begin in October 1948. We represented conditions for Otter Creek with the average of the observations at the three sites. Even though these site are located some distance from the watershed, cloud cover, wind speed, and dew point temperature will be similar at Otter Creek.

Finally, HSPF requires time series inputs on potential evapotranspiration and solar radiation. These variables are rarely measured directly. However, methods based on weather inputs can provide reliable estimates for hydrologic modeling. Using time series on air temperature, dew point temperature, and cloud cover, we estimated daily time series of potential evapotranspiration and solar radiation using a Penman approach (Shuttleworth, 1993). Potential evapotranspiration is the more critical variable. Along with precipitation, it predicts the overall water balance and storage of water in the subsurface (soils) for the simulation. Solar radiation is used only to predict snow melt during the cold season. Still, this approach provides consistent estimates of the two (related) variables for both uses of the data. Hourly time series are then generated from the daily values using a fixed daily cycle.

## River Reach Delineation

Figure 6.2 shows the subdivision of the Otter Creek watershed into 135 sub-basin areas. These areas define the drainage areas to a portion of the river network of streams (shown as the blue lines in Figure 6.2). Within HSPF, these areas are known as river reaches; runoff from the surrounding drainage area, as well as flow from upstream river reaches, combines to predict the resulting flow at the river reach outlet using an HSPF RCHRES operation. Hence, we made predictions at the outlet of the river reaches. To create these sub-basin areas, we begin with the 22 sub-basins from in the Turkey River HSPF model. Then we identified the locations of all the planned projects in Otter Creek, and the location of all water and water quality monitoring stations. Finally, we re-subdivided the watershed to enable predictions at all these locations. For the Otter Creek HSPF model, the average river reach drainage area is 0.35 square miles (233 acres). Note however that in the sixteen tributaries with projects, the network has higher resolution than in tributaries without projects.

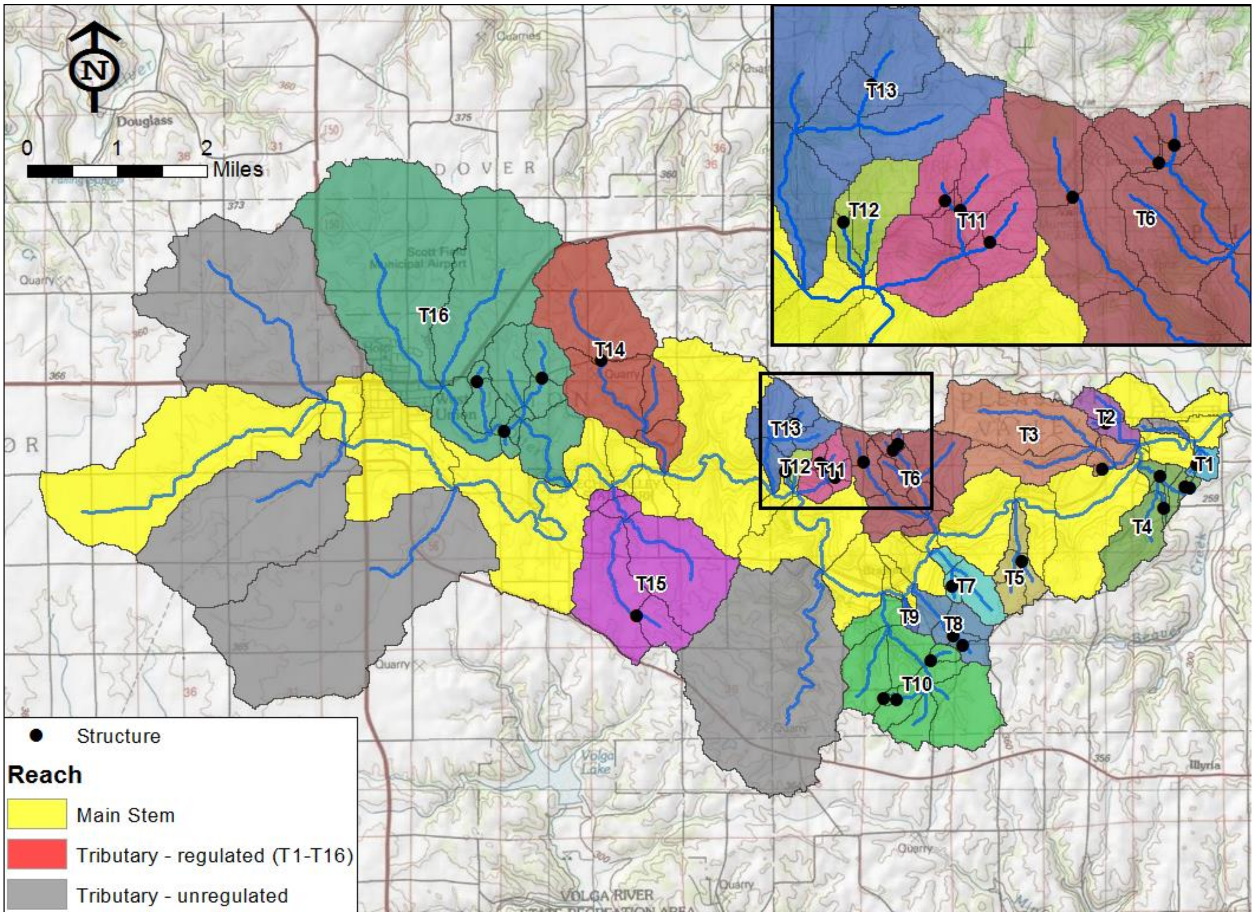


Figure 6.2. Subdivision of the Otter Creek watershed into HSPF RCHRES river reaches. The blue lines indicate the Otter Creek network of streams; the black lines outline the drainage area of the river reaches. The locations pond flood storage projects are indicated by the black circles. We labeled the sixteen tributaries with projects from downstream to upstream (T1 to T16). Note that HSPF RCHRES river reaches are sub-basin areas, and the runoff from these areas is combined with flows from upstream river reaches to make predictions at the outlet of the reach.

For each river reach segment, HSPF RCHRES requires river channel hydraulic information to determine how quickly water moves through the reach. The storage-discharge relationship summarizes this information. It defines the discharge at the outlet for a given amount of water stored within the channel of the river reach. For locations with a stream gauge, this information is straightforward to estimate. A stream-gauge provides direct measurements of the discharge and the channel cross-section flow area. By multiplying the area by the HSPF river reach length, we can also obtain the reach storage. Unfortunately, the new U.S. Geological Survey stream-gauge in Otter Creek has few direct observations. A standard approach for estimating channel reach information uses a scaling relationship between channel reach dimensions and drainage area. Using a relationship fitted to measurements from five U.S. Geological Survey stream-gauge sites in the Turkey River watershed (Leach 2015), we estimated the channel reach dimensions for all 135 HSPF RCHRES segments. Combining the dimensions with the reach lengths and using estimates of the hydraulic roughness for the channel and floodplain area, we estimated a storage-discharge relationship for all the segments for the Otter Creek HSPF model.

## Land Segment Definition

HSPF uses land segments to represent the hydrologic response at different locations. Pervious land segments (PLSs) represent the response from most areas. Impervious land segments (ILSs) represent the response from roads and urban areas where water cannot infiltrate into the ground.

Land segments are not meant to represent the hydrology of any one specific point in the watershed. Instead, they represent the average response from locations with similar characteristics (soils and land use) given the input weather time series. Therefore, land segments are defined by identifying areas with similar characteristics. In the Turkey River HSPF model (Leach, 2015), land segments were defined for the following land uses: corn, soybeans, forest, grassland, pasture, wetlands, and urban. We used the land use map for Otter Creek (see Figure 2.8) to reclassify areas to these seven distinct land uses. Table 6.1 shows the percentage of the Otter Creek Watershed assigned to each land use classification.

Table 6.1. Watershed area (in %) by land use classification for the Otter Creek HSPF model.

| Land Use | Watershed Area (%) |
|---|---:|
| Corn | 42.7 |
| Soybeans | 12.8 |
| Forest | 10.8 |
| Grassland | 10.2 |
| Pasture | 14.5 |
| Wetlands | <0.1 |
| Urban | 8.9 |

Each land use classification is represented by a unique pervious land segment. However, a portion of the urban and grassland areas are also represented by an impervious land segment. The impervious land segment is used to represent roads and paved areas, where water cannot infiltrate the ground. Based on this representation, there are nine different land segment types simulated for the Otter Creek watershed.

## HSPF Continuous Simulation

With the river reach and land segment definitions established for Otter Creek, we used the HSPF model to do a long-term continuous simulation for water years 1949—2013. A water year begins in October (when flows tend to be low) and continues through September, so the simulation period runs from October 1948 through September 2013. HSPF first computes runoff from the nine land segments at an hourly time step. It then routes the runoff through the river reach network at a five-minute time step.

Figure 6.3 shows the daily time series at the Otter Creek watershed outlet for a 10-year period. The results shows how runoff responds to the hourly weather inputs. Because the model continuously tracks the amount of water on the land surface after precipitation, runoff, and evaporation occur, its moisture conditions will reflect the effects of drought spells (e.g., dry soil conditions) or extended rainy periods (e.g., wet soil conditions).

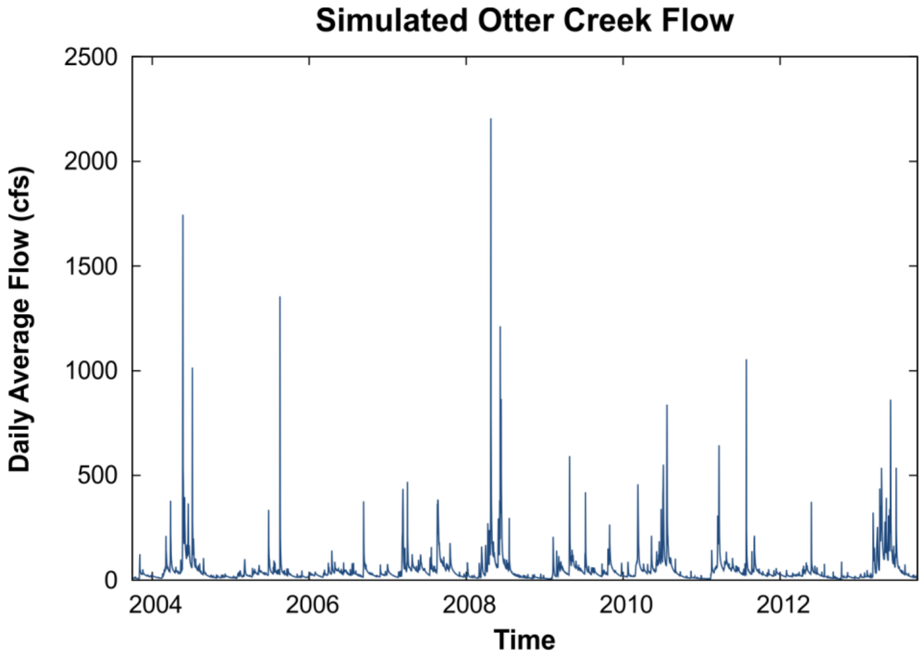


Figure 6.3. Simulated daily flow time series for the Otter Creek outlet (OTTRCRK01) for water years from 2004—2013.

Given the inherent limitations of hydrologic modeling, one should not expect simulated flows to exactly match what actually occurred over the past 64 years. The model uses nearby weather inputs (not those that actually occurred), and its employs a simplified representation of the rainfall-runoff process. Furthermore, the land use conditions are based on recent observations, and may not represent the changing conditions over the simulation period. However, despite some expected mismatch with actual flows, over the long-term, the model is expected to give a reasonable representation of the components of the water cycle. Results from the Turkey River HSPF simulation provide evidence that this is so.

One example of this is illustrated in Figure 6.4, which shows simulated and observed monthly water cycle results are shown for the Volga River at Littleport (USGS 05412400). The Volga River drains an area directly south of the Otter Creek watershed. This stream-gage was not used as part of the Turkey River HSPF model calibration, so the results demonstrate the calibrated model's predictive ability for sites within its watershed. The average monthly water depths are for water years 2000—2013, the overlapping period when stream-gauge observations are available. Overall, there is a pronounced seasonal cycle in runoff, and the simulated and observed monthly water cycles are similar. However, the observed depths are slightly higher during the peak runoff months of May, June, and July. For many other months, the simulated depths are slightly higher. Still, the long-term average flows simulated by the Turkey River HSPF model are a reasonable approximation at this site near Otter Creek.

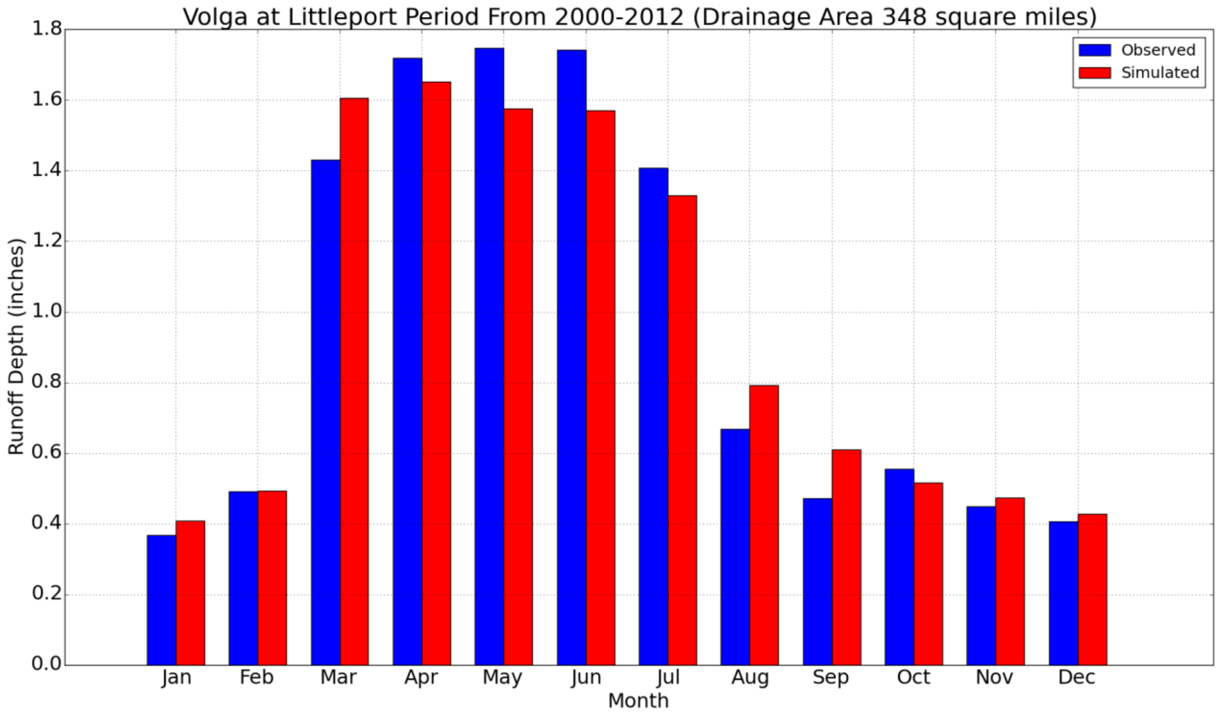


Figure 6.4. Simulated and observed average monthly runoff depth (in inches) for the Volga River at Littleport (USGS 05412400). The simulated depths are from the Turkey River HSPF model. Both results are based on the same period (water years 2000—2012).

Even though the model predictions of one flood may be too low, and another may be too high, what is most important for flood assessment is that the model can reproduce the statistical characteristics of flood peaks over the historical record. Figure 6.5 shows a flood frequency analysis of simulated and observed annual maximum peak discharge for the Volga River at Littleport. For the 13 years of overlapping records, the annual maximum peak discharges are ranked from smallest to largest, and then plotted versus a sample estimate of their exceedance probability. Note that to estimate flood magnitudes for large events (e.g., the 100-year flood, which has a 1% exceedance probability), engineers typically fit a mathematical model (known as a probability distribution) to these sample data. As the plot illustrates, the sample probability distributions for simulated and observed flows match well. Therefore, we can conclude that the Otter Creek HSPF model, which we created from the calibrated Turkey River HSPF model, provides a reliable basis for assessing flood characteristics.

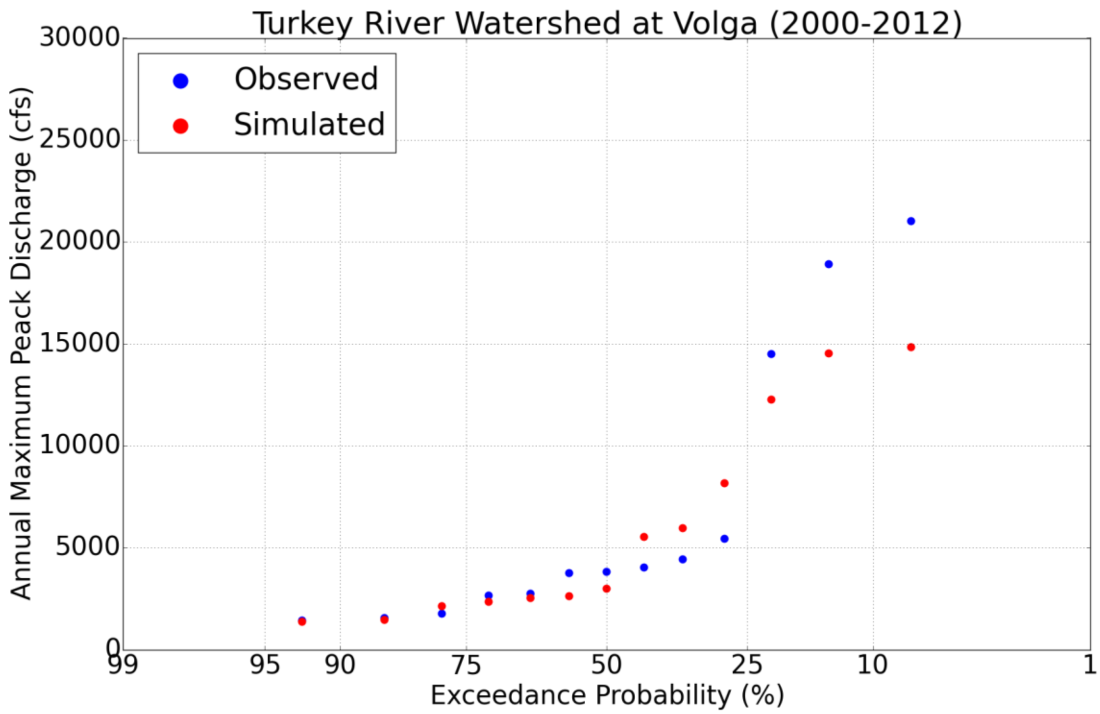


Figure 6.5. Flood frequency analysis of annual maximum peak discharges for simulated and observed flows for the Volga River at Littleport (USGS 05412400). The annual maximums are for the water years 2000—2012.

## j. Flood Characteristics of the Otter Creek Watershed

Before evaluating the performance of the Iowa Watersheds Project flood control ponds, we first examine the flood characteristics of the Otter Creek watershed. We made our baseline evaluation on the 65-year continuous simulation of the watershed without pond projects using the Otter Creek HSPF model. We will later use this baseline to examine the changes in flood characteristics with the constructed ponds.

Using the simulated peak discharges at the subbasins outlets, we can examine what individual extreme floods are like in the watershed. Peak discharge is an insufficient measure to identify extreme floods. Peak discharges for large drainage areas are usually much larger than for small drainage areas, even in cases when a flood is "more severe" at small drainage locations. Hence, we will use a flood severity index to characterize flood peak discharge at all locations. Our flood severity index is simply the ratio of the peak discharge to the mean annual flood at a location. Since the mean annual flood is a rough measure of the bankfull discharge, a flood severity of 1 or greater is an indicator of a flood. By determining the flood severity index for the annual maximum peak discharge at all sites for each year, we can rank the outcomes to identify times with extreme flooding. Table 6.2 shows the ranking of the top five years.

Table 6.2. Ranking of the top simulated floods in the Otter Creek watershed based on a flood severity index. The index is the ratio of peak discharge for the event and the mean annual flood. The flood events are ranked below based on the average index at all main stem locations and tributary outlets. The maximum and minimum index values at these locations within the watershed are also shown.

| Rank | Event | Average | Maximum | Minimum |
|---|---|---|---|---|
| 1 | May 1962 | 5.5 | 7.9 | 2.8 |
| 2 | May 1999 | 5.3 | 7.0 | 3.1 |
| 3 | August 1981 | 4.6 | 5.4 | 2.6 |
| 4 | April 2008 | 3.7 | 4.2 | 1.5 |
| 5 | May 2004 | 3.6 | 4.1 | 1.5 |

All of the top five simulated floods are spring or summertime events. In the spring, soils tend to be wet from recent snowmelt and low evapotranspiration, and strong storm systems can bring heavy rainfall with warmer weather. In the late spring and summer, rainfall intensities tend to be the highest (with thunderstorms). For Otter Creek, rainfall accumulations over a few hours are sufficient to cause streams to quickly rise out of their banks. For these top events, the rainfall accumulations for different durations is shown in Table 6.3. Notice that the most severe simulated flood event (May 1962) has the largest rainfall accumulation for 2- and 3-hour durations; its 1-hour accumulation is only slightly less than others. Given the basin's size, accumulations over these durations are most closely related to flood response. However, the August 1981 simulated flood event has the second largest accumulations over 2- and 3-hour duration, but had less severe flooding than the May 1999 simulated event. The soil moisture conditions also play a role in flooding. Since the model continuously tracks soil moisture conditions, it represents the soil conditions that would exist at the time of the storm (e.g., wet conditions from a series of rainy periods, or dry conditions after a long period without rain).

Table 6.3. Maximum rainfall accumulations for 1-hour, 2-hour, and 3-hour durations for the top simulated floods in the Otter Creek watershed.

| | | Accumulation (inches) | | | | |
|---|---|---|---|---|---|---|
| Rank | Event | 1-hour | 2-hour | 3-hour | 6-hour | 24-hour |
| 1 | May 1962 | 2.02 | 3.79 | 4.22 | 4.41 | 4.41 |
| 2 | May 1999 | 2.07 | 2.48 | 3.17 | 3.31 | 6.30 |
| 3 | August 1981 | 2.11 | 3.02 | 3.78 | 4.73 | 5.65 |
| 4 | April 2008 | 1.03 | 1.91 | 2.35 | 2.65 | 4.70 |
| 5 | May 2004 | 1.16 | 2.02 | 2.12 | 2.31 | 3.67 |

Based on the average flood severity index across all locations, the May 1962 event is the top simulated flood. The average index value is 5.5. On average, the peak discharge was 5.5 times the mean annual flood at main stem locations and tributary outlets. Figure 6.6 maps out the flood severity index for sub-basins for 1962. The flood severity index shows that the simulated flooding was intense in smaller tributary areas, but still significant along the Otter Creek main stem.

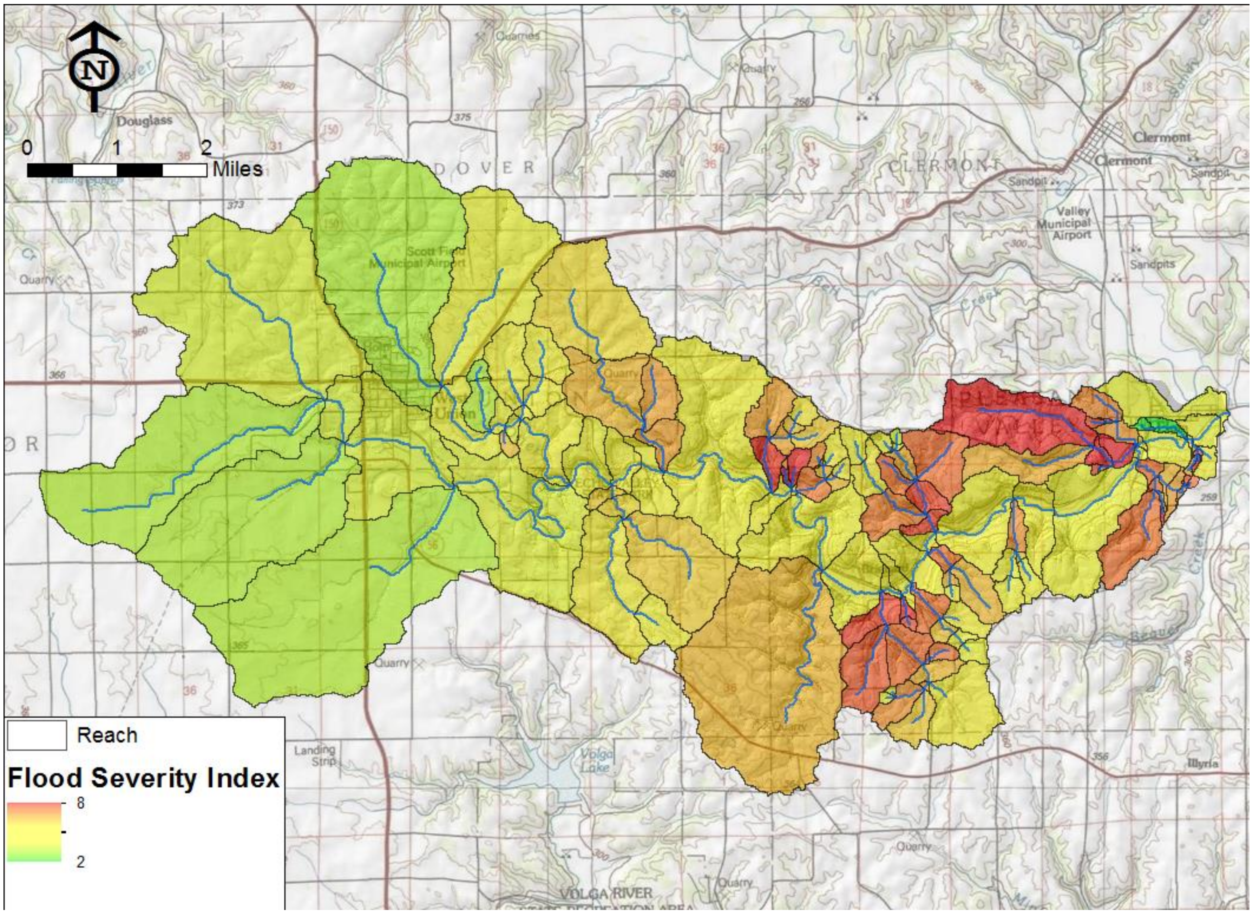


Figure 6.6. Flooding intensity and extent for the May 1962 flood. The map shows the estimated flood severity index at each sub-basin outlet.

The severity of the second largest simulated flood event of May 1999 is close to that of the largest event (May 1962). The average index value is 5.3 (compared to 5.5 for May 1962). Figure 6.7 maps out the flood severity index for sub-basins for the May 1999 event. Unlike the top May 1962 flood, the May 1999 event has the more intense simulated flooding in the tributaries. In contrast, the simulated flooding along the Otter Creek main stem is much less severe, especially in the middle and lower reaches.

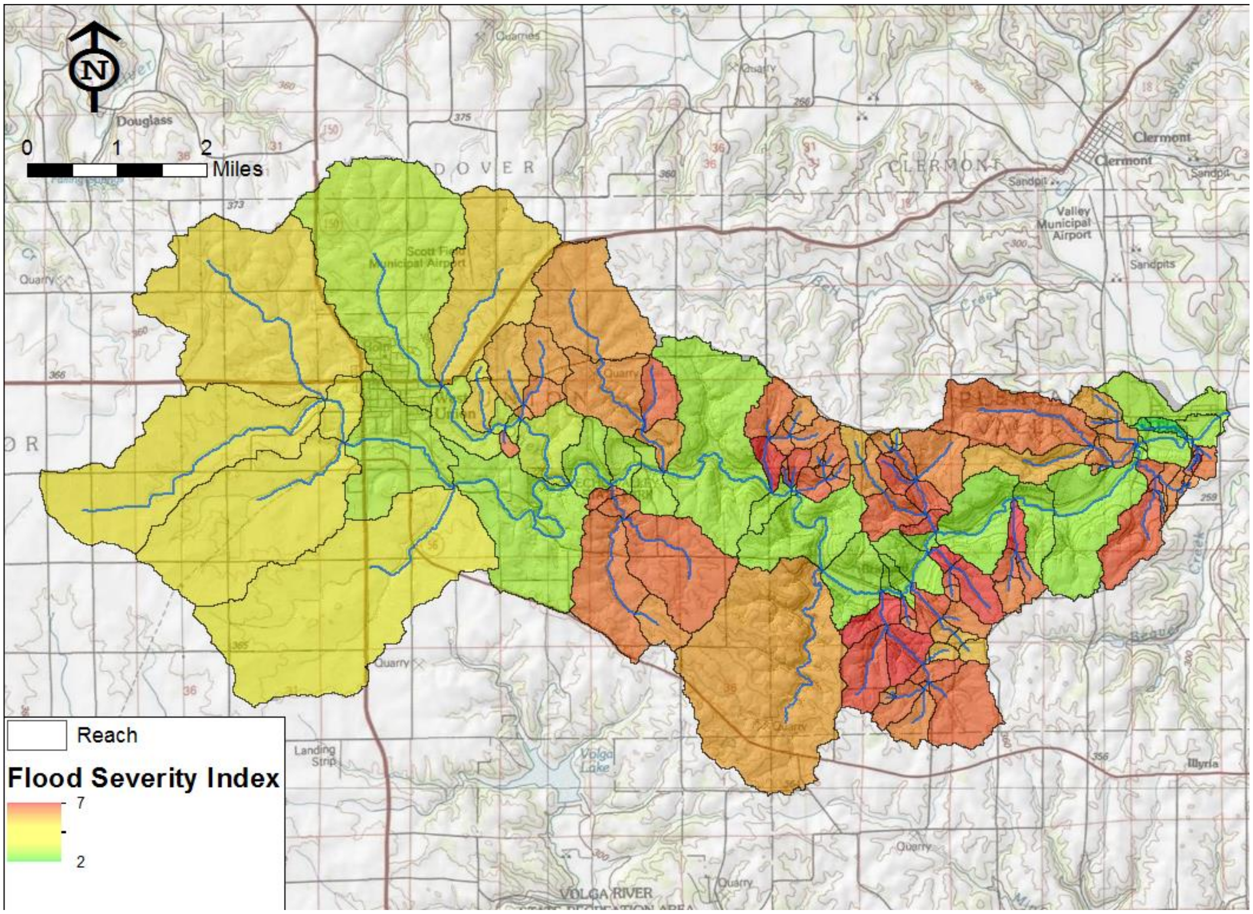


Figure 6.7. Flooding intensity and extent for the May 1999 flood. The map shows the estimated flood severity index at each subbasin outlet.

The examination of extreme flooding from the 65-year Otter Creek HSPF model simulations provides a better understanding of the nature of these events in the watershed. The largest floods in Otter Creek tend to be spring and summertime events. During one of these events, significant flooding occurs throughout the watershed. (Note that widespread flooding is due in part to the use of uniform rainfall from a single gauge at the input to the entire watershed; however, given the small size of the watershed, the spatial variability of extreme rainfall accumulation across the watershed should be relatively small.) All parts of the basin react rather quickly to storm rainfall, so intense rainfall over durations up to 3 to 6 hours is sufficient to cause a flood. Although flooding is widespread during these events, its severity is not uniform. Events tend to either be more severe in the tributary areas (in reaction to short duration high-intensity rainfall), or along the main stem (in reaction to the steady accumulation of runoff from the tributaries over longer durations).

From a flood mitigation planning perspective, it is important to recognize how different individual flood extremes can be. One advantage of using a continuous simulation model (like HSPF) for evaluation is that the performance of flood mitigation ponds over a range of potential flood conditions can be simulated and evaluated. In the remaining sections of this chapter, we will use this approach to evaluate the effect of pond projects on reducing peak discharges for flood events.

## k. Evaluation of Flood Mitigation from Pond Projects

In this section, we will use the Otter Creek HSPF model to simulate the effect of pond storage on flood peaks. First, we inserted the twenty-nine Iowa Watersheds Project ponds into the HSPF model, routing flow from upstream reaches through the pond storage. We determined the outflow from each pond based on its elevation-storage-discharge relationship, as shown in Appendix A. We simulated the pond performance continuously for the 65-year simulation period. The following sections will compare the simulated flows with the twenty-nine ponds to the baseline simulated flows without ponds for the 65-year period.

### Hydrographs for Top Flood Events

We examined the simulated flood hydrographs for the top flood events at ten locations. Six locations are at Otter Creek tributary outlets; another four locations are on the Otter Creek main stem. Figure 6.8 shows the four locations on the Otter Creek main stem that we chose as index points. At all the locations, we compare the simulated floods with ponds to the baseline case.

Figure 6.9 shows flood hydrographs for the May 1962 flood at six tributary outlets. Results are shown for baseline simulation with no ponds, and with the pond projects. The ponds significantly reduce flood peaks in all six tributaries for this event (the largest simulated flood in the 65-year period). Tributary #5, which has two flood control ponds with a total storage capacity of 9.4 acre-feet, has the largest peak reduction at 41.3%. Tributary #14, which has the largest on-road pond with 243 acre-feet of storage, has a peak reduction of 32.7%. Tributary #13, which has a single pond (11.0 acre-feet of storage capacity) has the lowest peak reduction shown at 8.9%. Most of the other ten tributaries not shown in Figure 6.9 have even lower peak reductions.

Figure 6.10 shows flood hydrographs for the May 1962 flood at four Otter Creek main stem locations. Even though all the pond projects are in upstream tributary areas, they still reduce flood peaks at downstream main stem locations. On the main stem above Tributary #14, where the drainage area is 26.1 $mi^2$ and there are four ponds (9.4 acre-feet storage capacity), the peak reduction is only 1.9%. The impact of the large on-road pond in Tributary #14 is seen on the main stem above the confluence with Tributary #13; the peak reduction for this flood event increases to 6.3%. At further downstream main stem locations, the peak reduction diminishes slightly. Near the Otter Creek outlet at the OTTRCRK01 stream sensor, the peak reduction is 5.1%.

We mapped the peak reduction effect at all locations for the May 1962 flood in Figure 6.11 and 6.12. The peak reduction is shown at all the pond project outlets (circles) and at the outlet of each sub-basin (colored sub-basin areas). Obviously, upstream of all the project locations, the ponds do not regulate flow (so no peak reduction mapping is shown). Not surprisingly, high peak reductions occur at the project outlets. The peak reduction effect diminishes as one moves downstream from the projects and as additional runoff enters from contributing drainage areas. The highest peak reductions occur in the tributary with the Iowa Watersheds Project ponds.

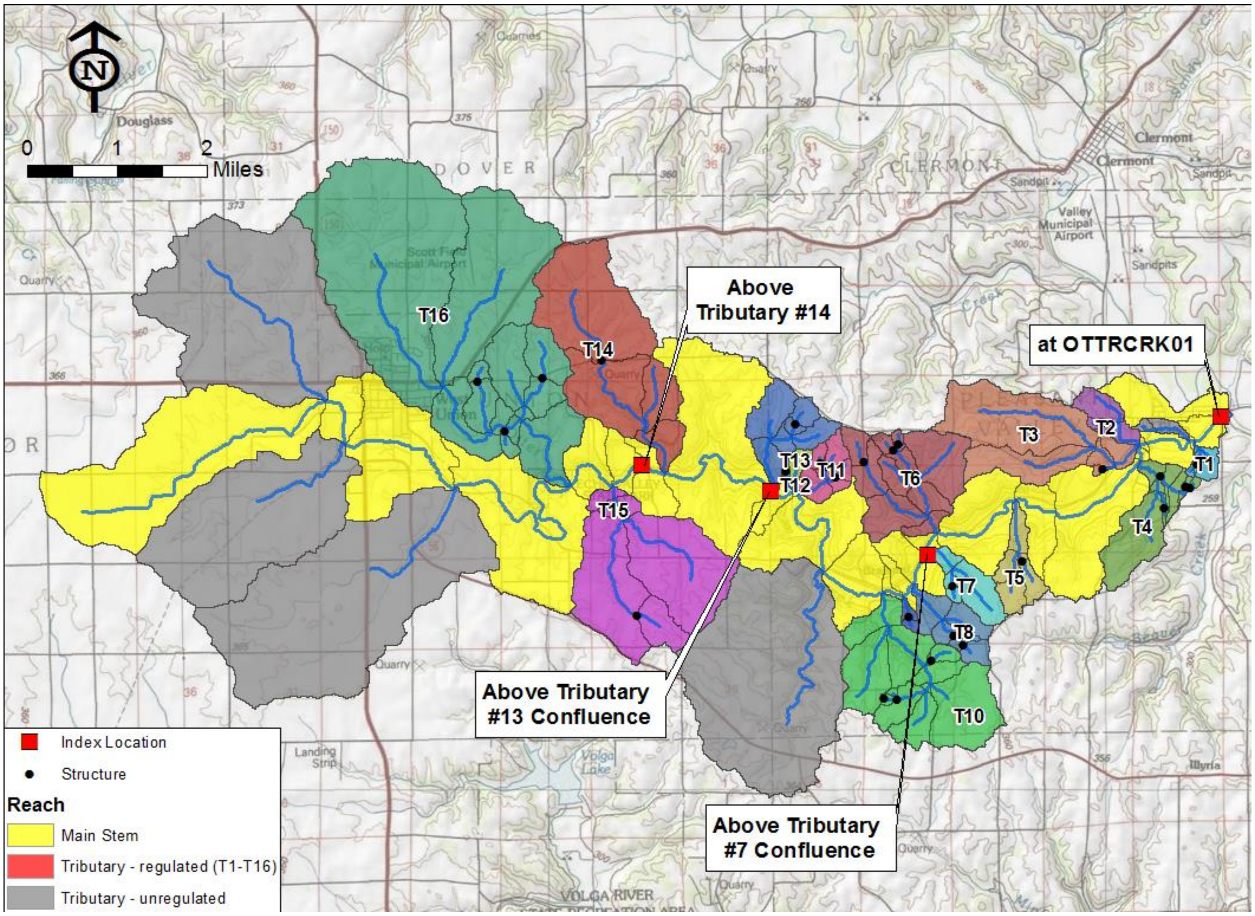


Figure 6.8. Index point locations.

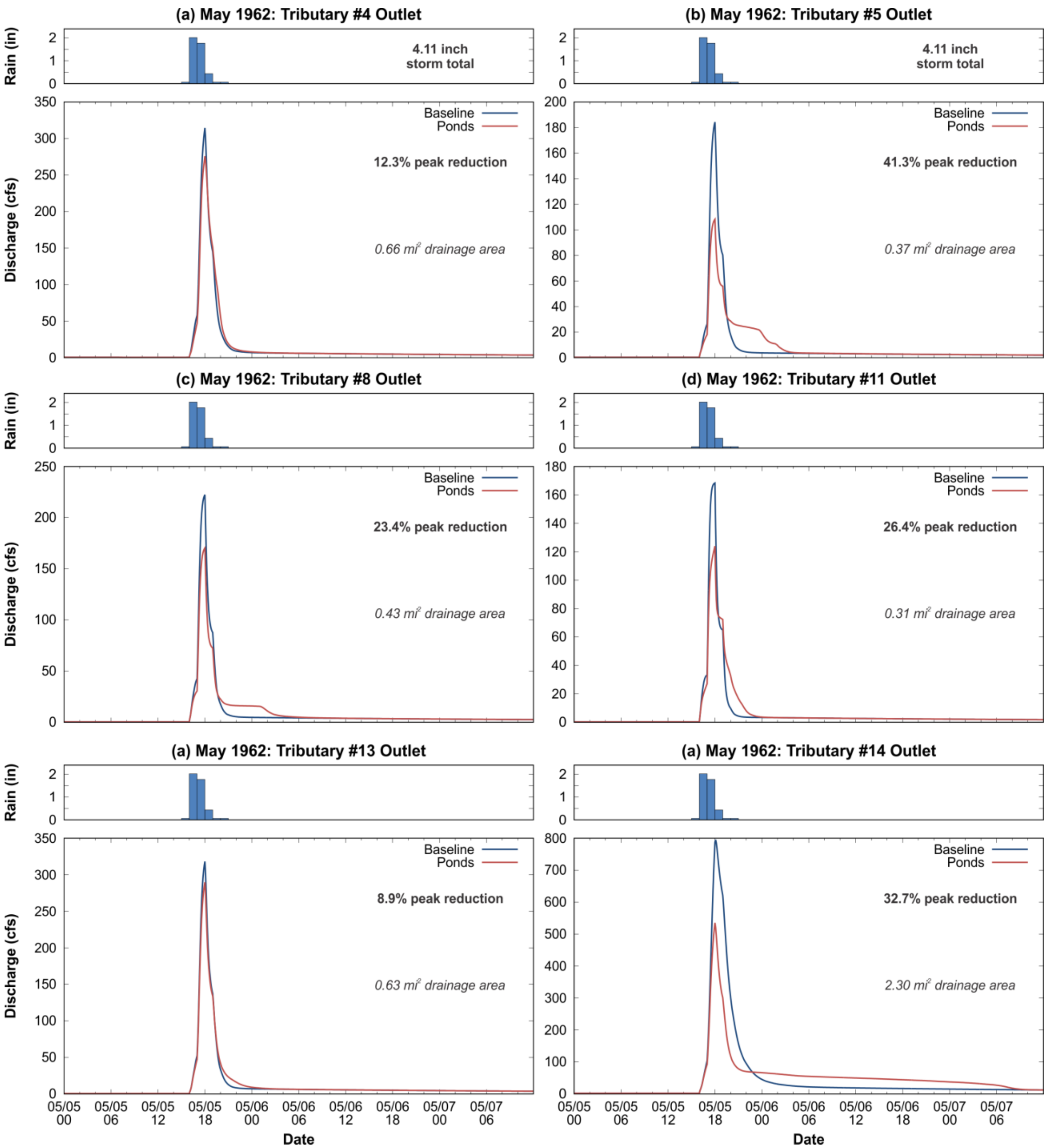


Figure 6.9. Flood hydrographs for the May 1962 event at six tributary locations: (a) Tributary #4 outlet; (b) Tributary #5 outlet; (c) Tributary #8 outlet; (d) Tributary #11 outlet; (e) Tributary #13 outlet; and (f) Tributary #14 outlet (see Figure 6.8 for locations).

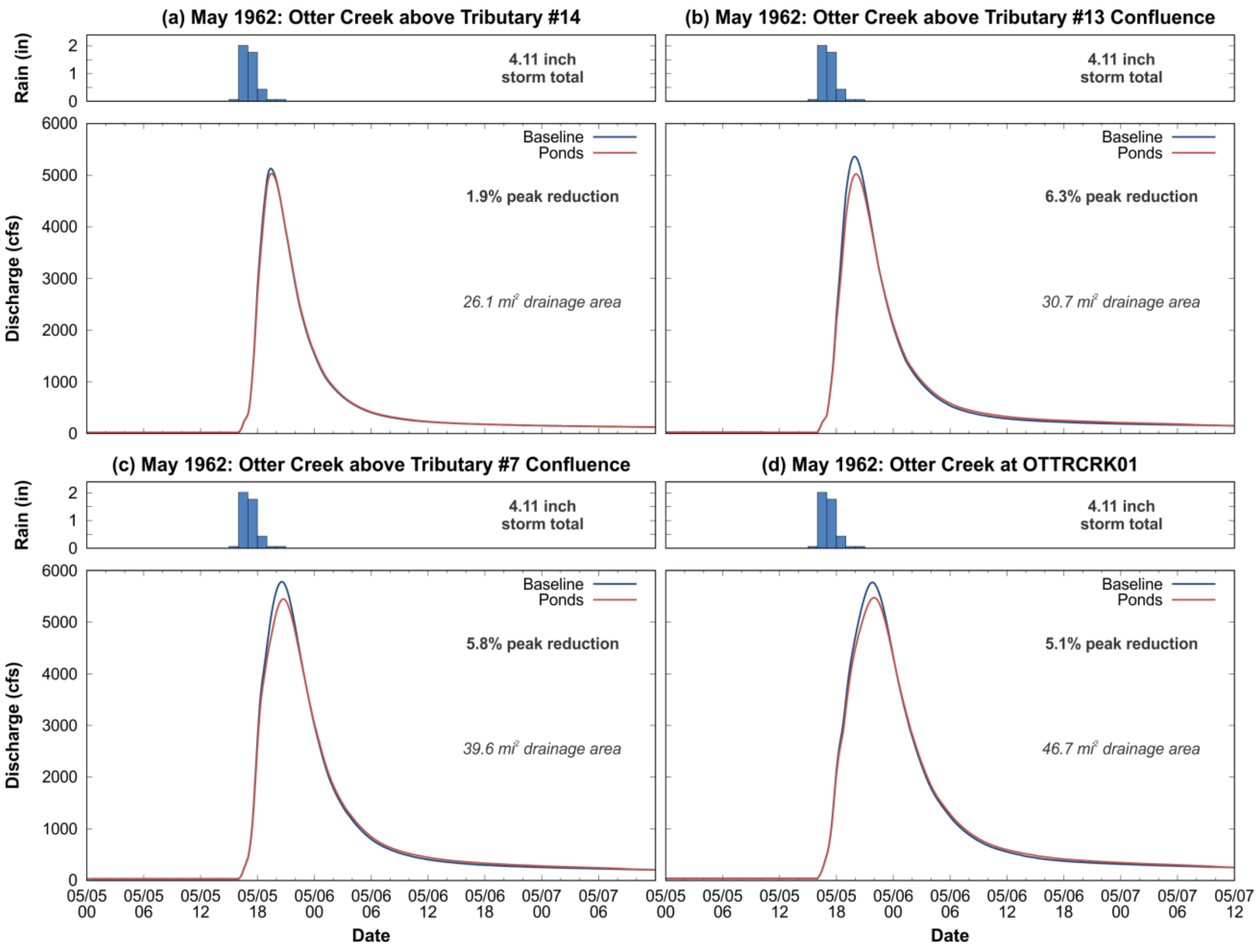


Figure 6.10. Flood hydrographs for the May 1962 event at Otter Creek main stem locations: (a) above Tributary #14; (b) above the confluence with Tributary #13; (c) above the confluence with Tributary #7; and (d) at the OTTRCRK01 stage sensor (see Figure 6.8 for locations).

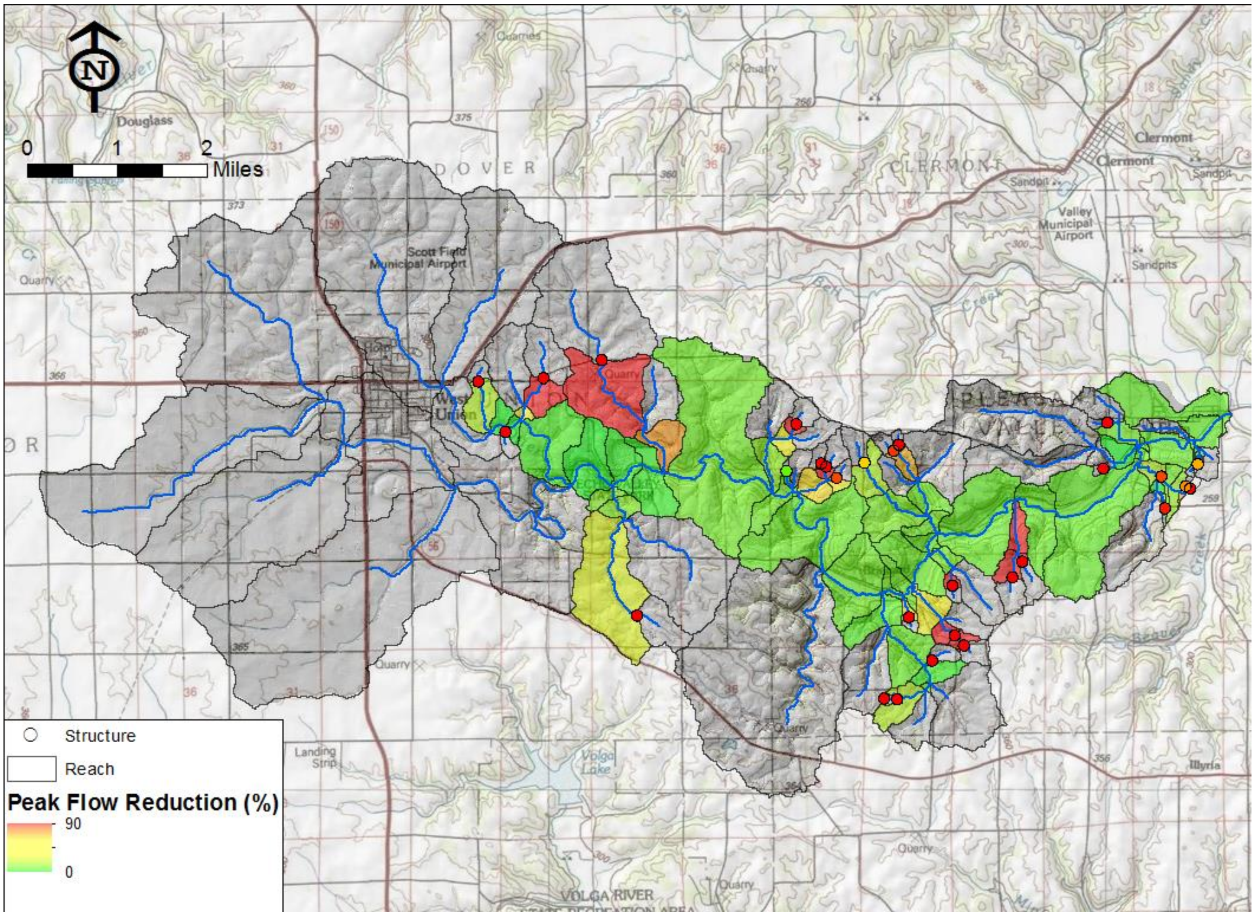


Figure 6.11. Peak reduction (%) for the May 1962 flood with twenty-nine pond projects for the Otter Creek Watershed. The map shows the estimated peak reduction at each sub-basin outlet compared to the baseline simulation without ponds.

The May 1999 event is the second largest simulated event in the 65-year record. It is notable in that three heavy rainfall periods occurred within about 12 hours. Figure 6.13 shows flood hydrographs for the May 1999 flood at six tributary locations. At all locations, there are three peaks associated with the three heavy rainfall periods. The flood control ponds store runoff from the first (and largest) heavy rainfall period, resulting in significant peak reductions. The peak reductions at most locations are slightly higher than for the May 1962 flood. However, the ponds have less storage available when the second and third heavy rainfall periods occur. As a result, the peak reduction for the two later peaks is generally less. The ponds still provide some additional storage for the later rains, which helps reduce flood peaks on the main stem.

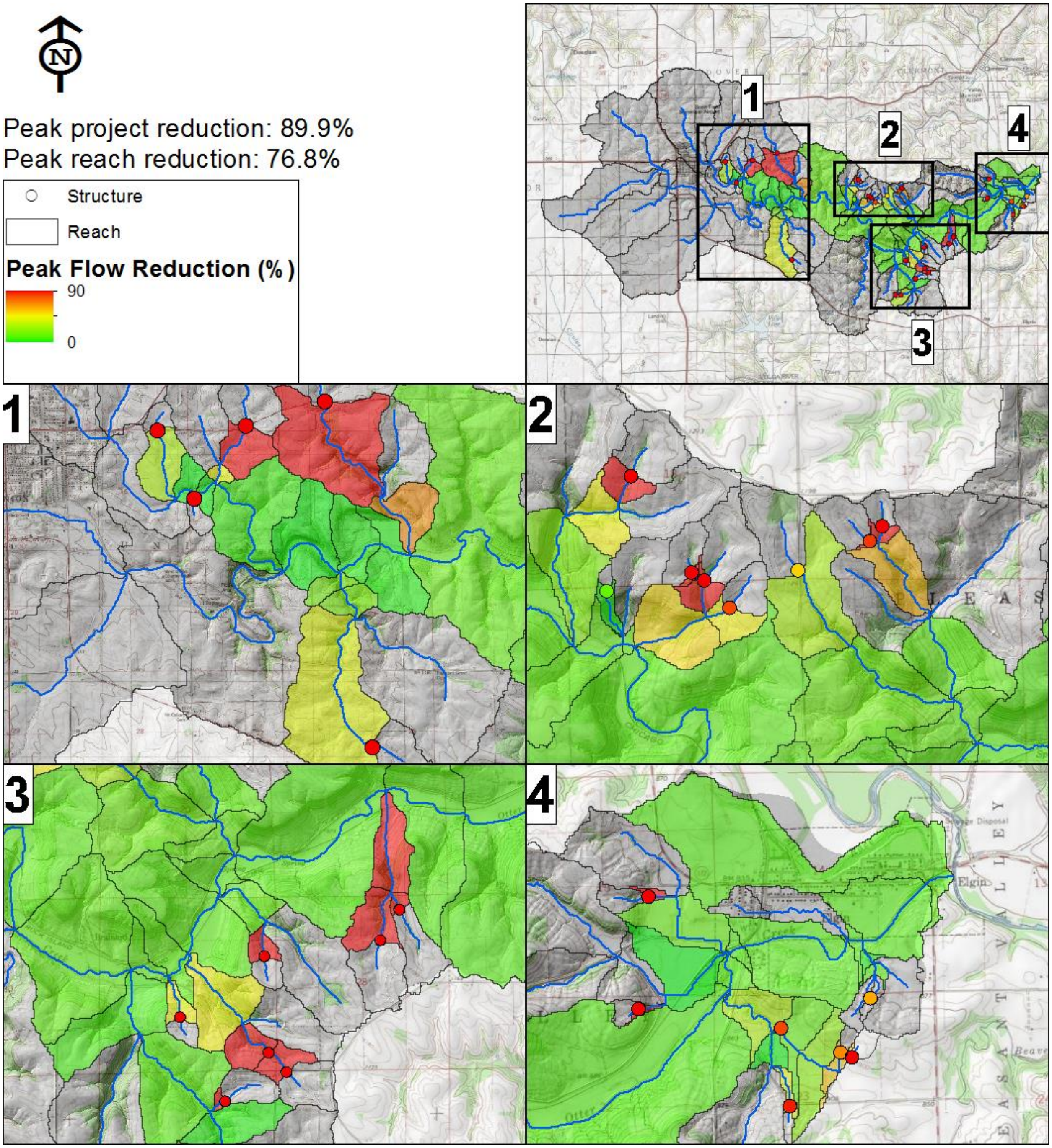


Figure 6.12. Peak reduction (%) for the May 1962 flood with twenty-nine pond projects for the Otter Creek tributaries. The map shows the estimated peak reduction at each sub-basin outlet compared to the baseline simulation without ponds.

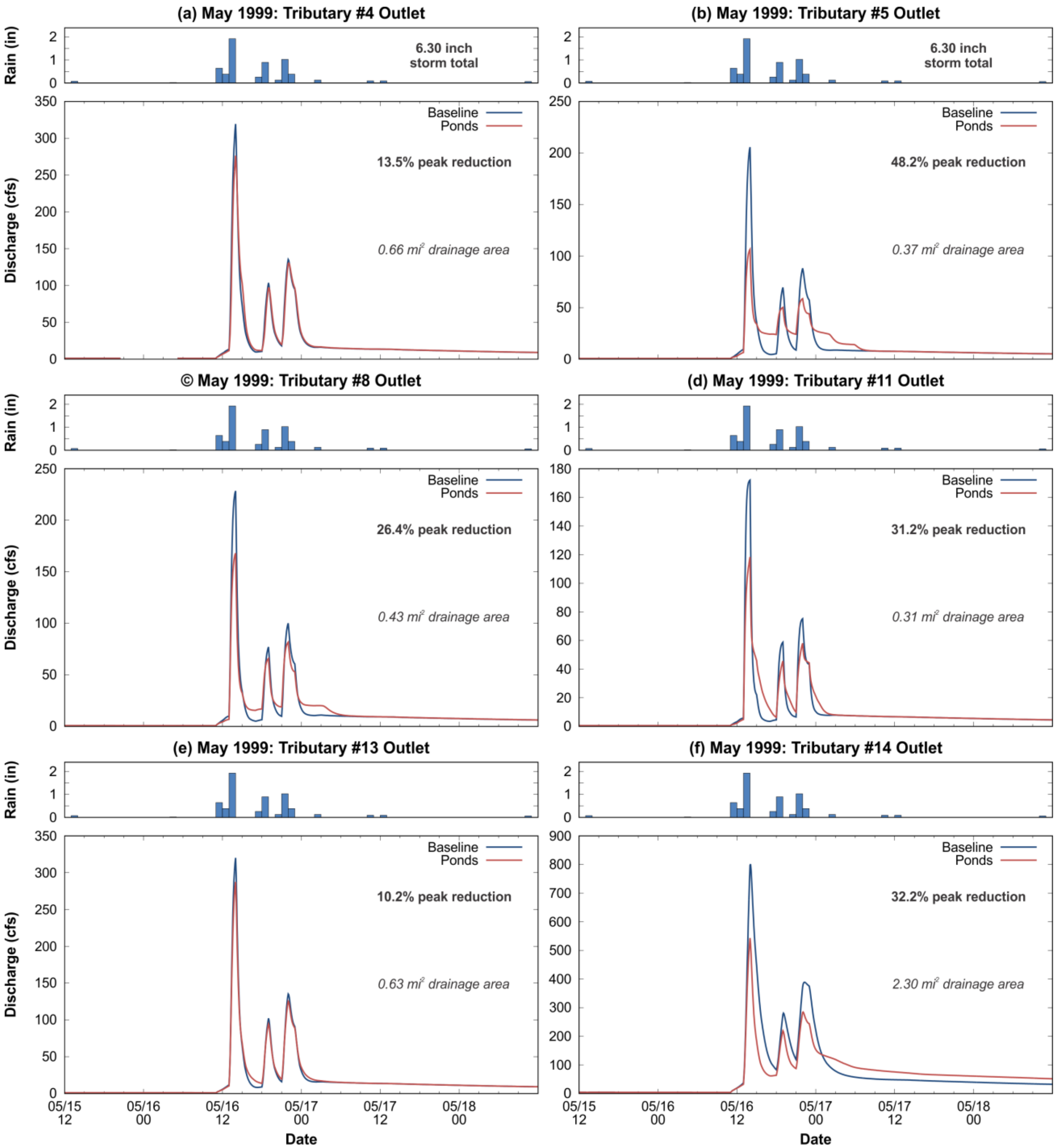


Figure 6.13. Flood hydrographs for the May 1962 event at six tributary locations: (a) Tributary #4 outlet; (b) Tributary #5 outlet; (c) Tributary #8 outlet; (d) Tributary #11 outlet; (e) Tributary #13 outlet; and (f) Tributary #14 outlet (see Figure 6.8 for locations).

Figure 6.14 shows flood hydrographs for the May 1999 flood at the four Otter Creek main stem locations. As with the May 1962 flood, the ponds reduced flood peaks along the Otter Creek main stem downstream of the pond sites. However, unlike in the tributaries, the peak reduction is slightly less in than May 1999 flood than in the May 1962 flood. The three peaks from the three heavy rainfall periods occur at three of the main stem locations. However, near the Otter Creek outlet at the OTTRCRK01 stream sensor, only two peaks occur due to the travel of water through the watershed. Again, the largest peak reduction occurs above the confluence with Tributary #13, downstream of the large on-road pond in Tributary #14. The peak reduction diminishes slows moving downstream to the outlet.

We mapped the peak reduction effect at all locations for the May 1999 flood in Figure 6.15 and 6.16. The peak reduction effect is similar in magnitude and pattern as the May 1962 event. High peak reductions occurred at the project outlets. The peak reduction effect diminished downstream from the project. The highest peak reductions occurred in the tributary with the twenty-nine Iowa Watersheds Project wetlands.

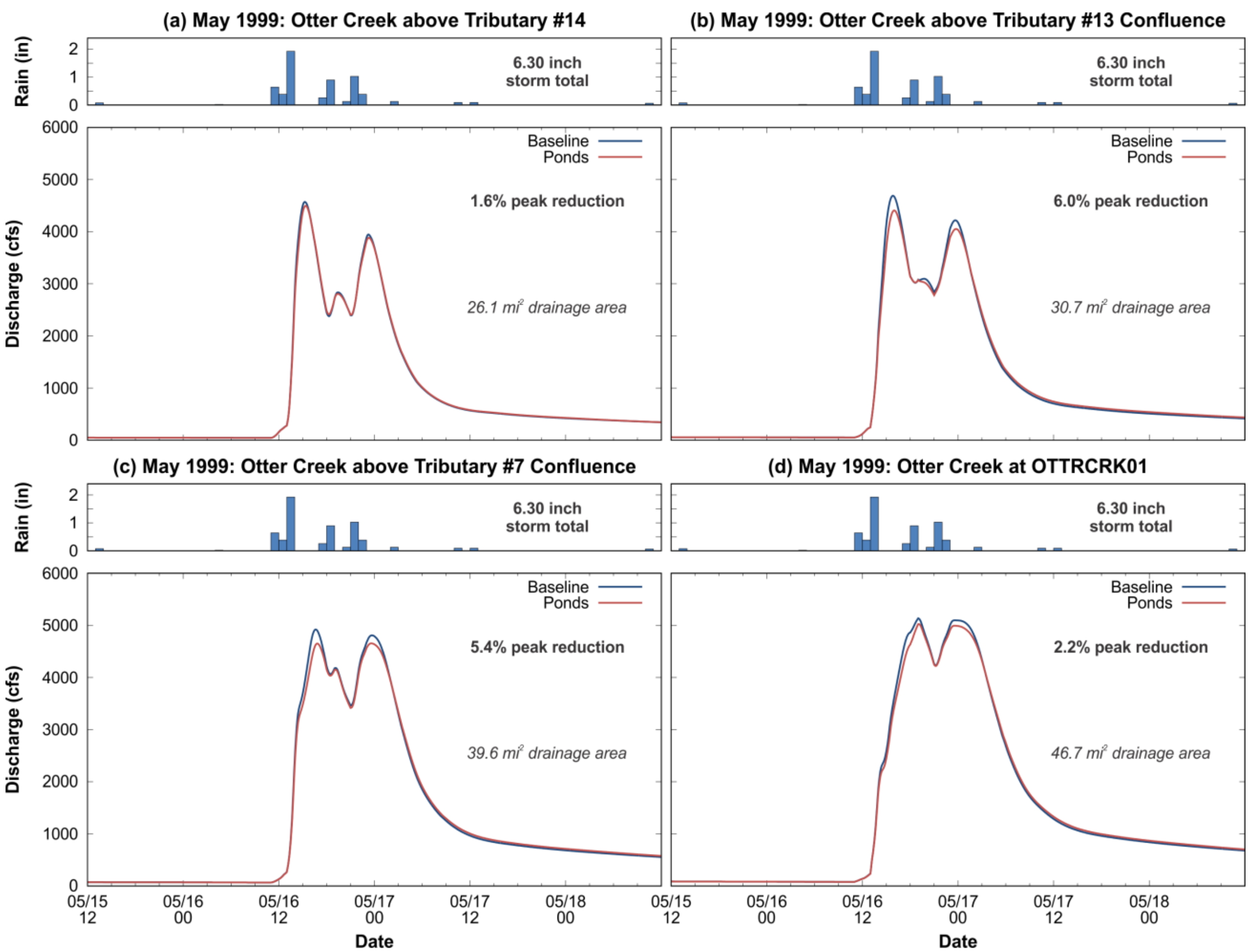


Figure 6.14. Flood hydrographs for the May 1962 event at Otter Creek main stem locations: (a) above Tributary #14; (b) above the confluence with Tributary #13; (c) above the confluence with Tributary #7; and (d) at the OTTRCRK01 stage sensor (see Figure 6.8 for locations).

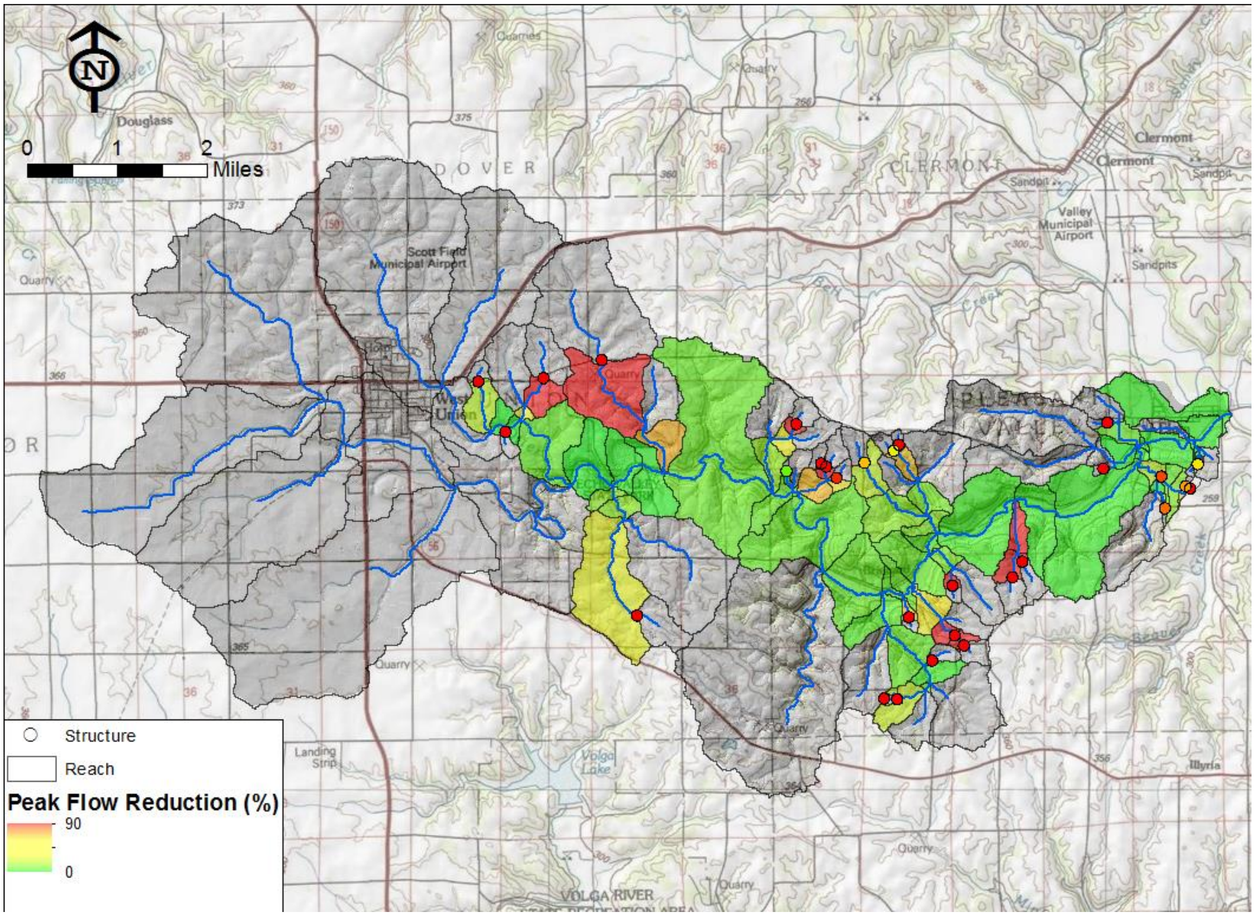


Figure 6.15. Peak reduction (%) for the May 1999 flood with twenty-nine wetland projects for the Otter Creek Watershed. The map shows the estimated peak reduction at each sub-basin outlet compared to the baseline simulation without wetlands.

## Flood Frequency Analysis

To study how the pond projects perform over a range of possible flood events, we analyzed flood frequencies. Figure 6.17 shows the flood frequency analysis of simulated baseline condition (without ponds) and the pond simulation at the six tributary locations. For each year in the 65-year simulation, the annual maximum peak discharges (i.e., the largest discharge in a given year) are found at each location. Then, we ranked them from smallest to largest and plotted versus a sample estimate of their exceedance probability. Each plot also shows exceedance probabilities corresponding to the 2-year, 10-year, 25-year, and 50-year return periods (dashed vertical lines). We computed the average peak reduction based on all 65 pairs of annual maximums.

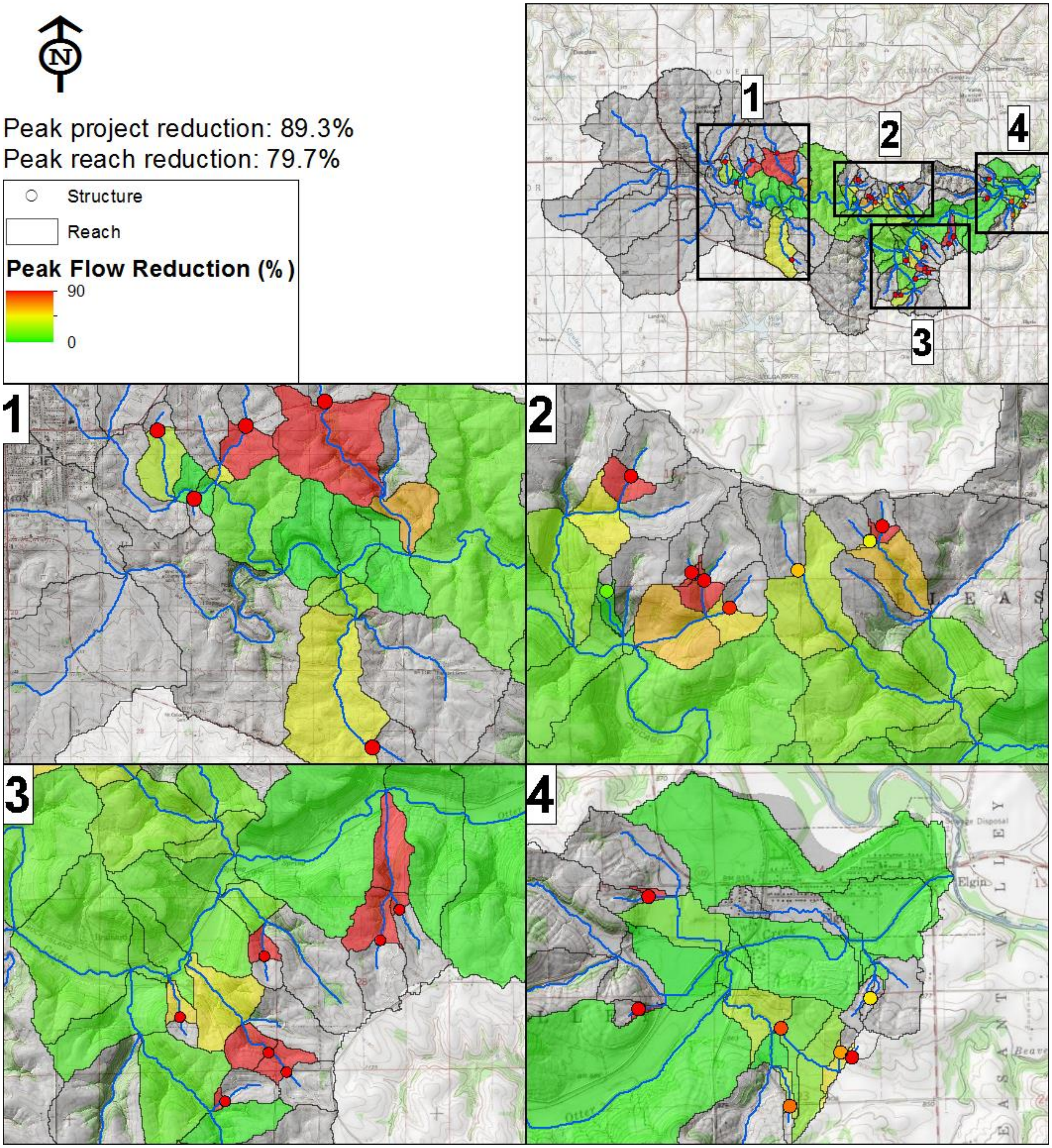


Figure 6.16. Peak reduction (%) for the May 1999 flood with twenty-nine wetland projects for the Otter Creek tributaries. The map shows the estimated peak reduction at each sub-basin outlet compared to the baseline simulation without wetlands.

Similar to the two individual flood events in the previous section, the simulated peak discharge flood frequencies are lower for the ponds simulation. The largest average peak reduction occurs in Tributary #5 (35.4%). Tributary #14 with the large on-road pond also has a large average peak reduction (29.4%). Tributaries #4 and #13 have the lowest average peak reduction shown (9.1%). Most of the other ten tributaries not shown in Figure 6.15 have even lower peak reductions.

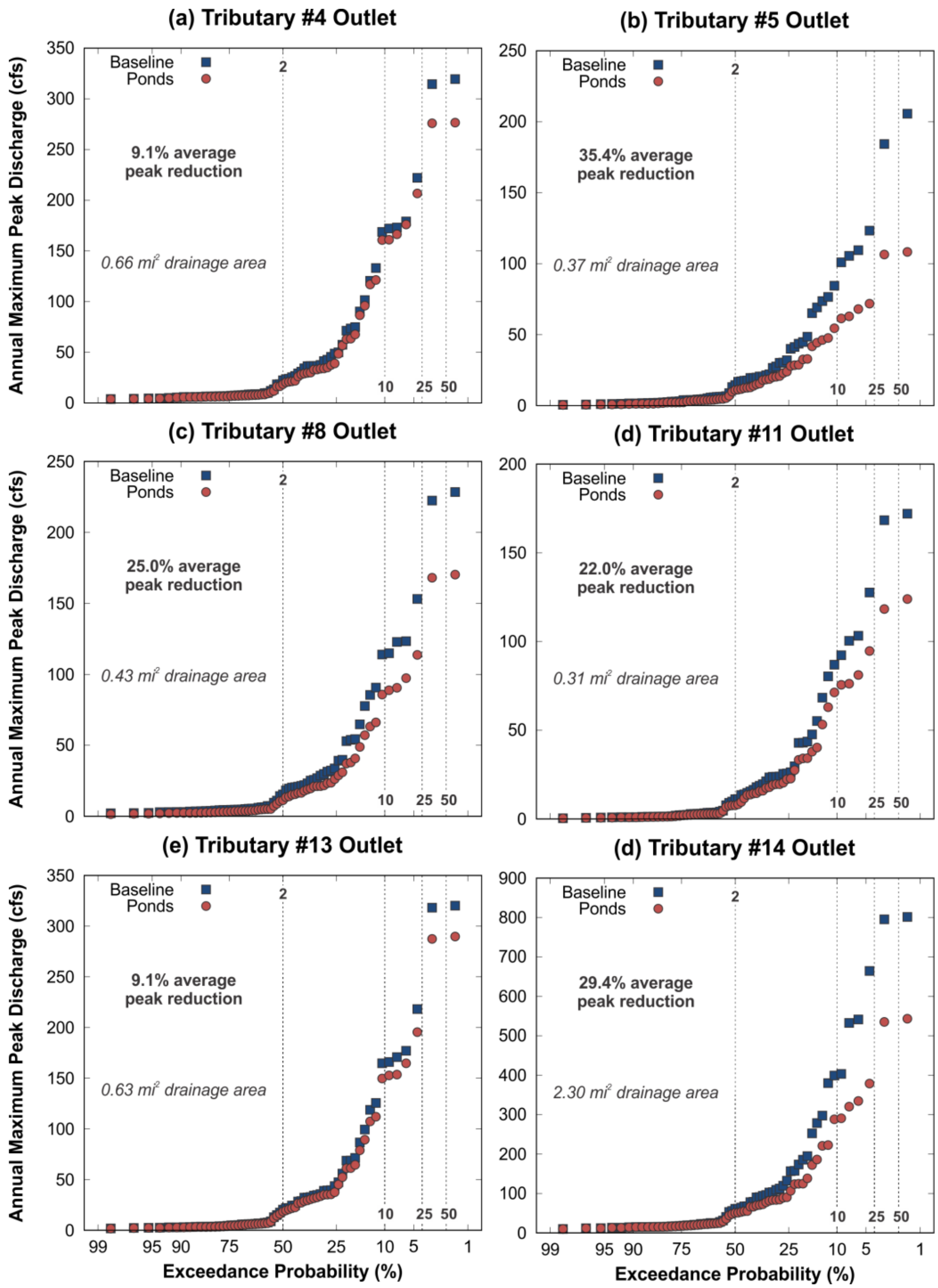


Figure 6.17. Sample probability distribution of annual maximum peak discharges for the baseline and pond simulations at four tributary locations: (a) Tributary #4 outlet; (b) Tributary #5 outlet; (c) Tributary #8 outlet; (d) Tributary #11 outlet; (e) Tributary #13 outlet; and (f) Tributary #14 outlet (see Figure 6.8 for locations). Vertical dashed lines show the 2-, 10-, 25-, and 50-year return periods. We computed the average peak reduction based on all 65 pairs of annual maximums.

Figure 6.18 shows the flood frequency analysis of simulated baseline condition (without ponds) and the pond simulation at the four main stem locations. The trends observed for the two individual flood events are also evident with the flood frequencies. On the main stem above Tributary #14, where there are four ponds in upstream tributaries, the average peak reduction is only 0.9%. The average peak reduction is highest (3.6%) above the confluence with Tributary #13 because of the large on-road pond upstream. Further downstream, the average peak reduction diminishes towards the Otter Creek outlet. The average is 2.5% at the OTTRCRK01 stream sensor.

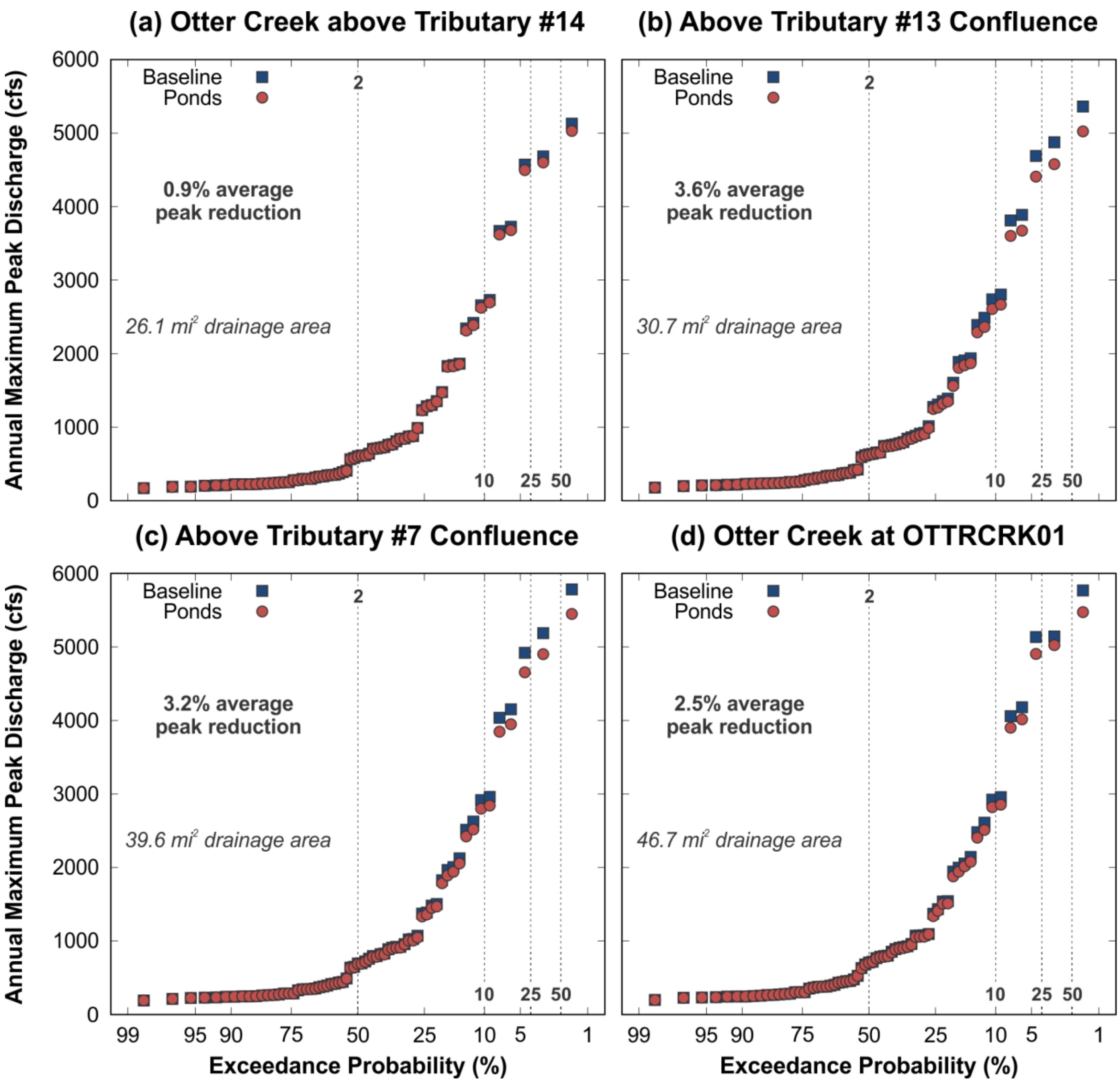


Figure 6.18. Sample probability distribution of annual maximum peak discharges for the baseline and wetland simulations at four main stem locations: (a) above Tributary #14; (b) above the confluence with Tributary #13; (c) above the confluence with Tributary #7; and (d) at the OTTRCRK01 stage sensor (see Figure 6.8 for locations). Vertical dashed lines show the 2-, 10-, 25-, and 50-year return periods. We computed the average peak reduction based on all 65 pairs of annual maximums.

By design, ponds store a greater volume of water as flows increase. After the water level rises above the auxiliary spillway elevation, the flood storage volume is exhausted and the peak reduction diminishes. Therefore, the flood storage is most effective in reducing peak discharges for a targeted range of flows. The effects are illustrated in Table 6.4, which shows the peak reduction for different return periods at the outlet of the pond projects. The peak reduction is large at almost all the pond outlets for all return periods. For most of the Iowa Watersheds Project ponds, the peak reduction is largest for the 50-year return period. Peak reduction increases from the 2-year to 10-year flood, from the 10-year to 25-year flood, and from the 25-year to the 50-year flood. This occurs because the ponds are using their storage for rarer large events, and not for smaller, more common high flow periods.

Table 6.5 shows the peak reduction for different return periods at the tributary outlets. At the tributary outlets, the peak reduction is less than at the pond outlets. For some tributaries, the peak reduction is only a few percent. Tributaries that have a larger fraction of their drainage area regulated by the ponds tend to have larger peak reductions. Unlike at the pond outlets, the peak reduction is not consistently the largest for the 50-year return period flood. Half the tributaries see their largest peak reduction for the 25- or 50-year return period, and the other have for the 2- or 10-year return period.

Table 6.6 shows the peak reduction for different return periods at main stem locations. At the main stem locations, the pattern is similar to that at the pond outlets. The peak reduction increases from the 2-year to 10-year flood, from the 10-year to 25-year flood, and from the 25-year to the 50-year flood. However, the peak reduction on the main stem is much lower, reaching a maximum of 6.3% above the confluence with Tributary #13. The peak reduction diminishes moving downstream to 4.3% (for the 50-year flood) at the OTTRCRK01 stream sensor location.

Table 6.4. Peak reduction effect for the pond project outlets (relative to the baseline simulation). Reductions (%) for the 2-, 10-, 25-, and 50-year return periods. The average (%) is the average reduction based on all 65 ranked annual maximum events.

| Location | Tributary | Average | 2-year | 10-year | 25-year | 50-year |
|---|---|---|---|---|---|---|
| Helms #4 | 1 | 1.5 | 0.0 | 1.8 | 18.5 | 27.4 |
| Frieden | 2 | 14.3 | 2.0 | 54.5 | 67.5 | 70.3 |
| Medberry 638 | 3 | 10.6 | 0.4 | 54.6 | 68.9 | 72.0 |
| Helms #5 | 4 | 35.2 | 49.7 | 5.7 | 29.0 | 40.7 |
| Helms #3 | 4 | 2.6 | 0.0 | 8.0 | 27.5 | 32.4 |
| Helms #1 | 4 | 3.3 | 0.8 | 12.6 | 31.4 | 40.8 |
| Helms #2 | 4 | 5.8 | 0.0 | 27.1 | 45.8 | 52.0 |
| Bennett #5 | 5 | 52.8 | 58.2 | 67.0 | 77.2 | 83.3 |
| Bennett #4 | 5 | 22.3 | 15.5 | 58.7 | 68.2 | 76.9 |
| F Avenue On-Road | 6 | 7.1 | 8.0 | 18.5 | 23.3 | 27.3 |
| Helgerson 410 | 6 | 51.4 | 61.3 | 59.6 | 34.0 | 26.6 |
| Helgerson 638 | 6 | 17.2 | 1.3 | 60.7 | 69.5 | 76.8 |
| Bennett #3 | 7 | 25.8 | 11.8 | 65.9 | 71.9 | 78.3 |
| Bennett #2 | 8 | 62.1 | 52.6 | 68.2 | 77.8 | 82.3 |
| Dove Road On-Road | 8 | 16.3 | 6.8 | 59.1 | 67.1 | 74.2 |
| Woltz | 9 | 44.7 | 66.7 | 21.9 | 45.2 | 46.1 |
| Bennett #1 | 10 | 76.8 | 76.2 | 69.5 | 80.3 | 82.9 |
| Howard 638 #2 | 10 | 33.7 | 28.1 | 75.5 | 82.9 | 87.4 |
| Howard 638 #1 | 10 | 32.3 | 27.2 | 65.2 | 71.3 | 78.2 |
| McMillan #5 | 11 | 15.2 | 15.5 | 27.4 | 33.0 | 42.6 |
| McMillan #4 | 11 | 23.4 | 22.9 | 48.4 | 45.9 | 55.7 |
| McMillan #3 | 11 | 19.8 | 23.3 | 22.3 | 38.2 | 44.4 |
| McMillan #1 | 12 | -1.1 | -1.0 | 0.3 | 2.8 | 6.1 |
| McMillan #2 | 13 | 41.3 | 35.7 | 52.3 | 57.0 | 63.9 |
| Golden On-Road #3 | 14 | 37.3 | 35.9 | 83.5 | 86.9 | 80.5 |
| Open Range Farms | 15 | 74.9 | 82.2 | 83.9 | 88.3 | 89.5 |
| Golden On-Road #2 | 16 | 18.7 | 1.8 | 62.3 | 63.5 | 69.2 |
| DNR | 16 | 40.7 | 31.1 | 79.7 | 83.6 | 86.6 |
| Golden On-Road #1 | 16 | 10.5 | 0.4 | 41.8 | 53.4 | 63.5 |

Table 6.5. Peak reduction effect for the pond simulation (relative to the baseline simulation) at tributary outlet locations. Reductions (%) for the 2-, 10-, 25-, and 50-year return periods. The average (%) is the average reduction based on all 65 ranked annual maximum events.

| Tributary | Average | 2-year | 10-year | 25-year | 50-year |
|---|---|---|---|---|---|
| 1 | 0.1 | 0.1 | 0.2 | 1.1 | 1.1 |
| 2 | 0.3 | 0.1 | 1.3 | 1.5 | 1.6 |
| 3 | 0.1 | 0.0 | 0.5 | 0.6 | 0.6 |
| 4 | 9.1 | 17.2 | 5.4 | 9.3 | 13.1 |
| 5 | 27.7 | 24.6 | 37.1 | 41.9 | 45.8 |
| 6 | 4.2 | 7.5 | 5.0 | 7.3 | 6.8 |
| 7 | 3.0 | 1.9 | 7.3 | 7.0 | 7.1 |
| 8 | 25.8 | 26.4 | 23.9 | 25.2 | 25.1 |
| 9 | 30.6 | 44.1 | 10.8 | 28.2 | 24.9 |
| 10 | 3.3 | 2.6 | 5.1 | 4.4 | 4.1 |
| 11 | 16.7 | 31.0 | 18.0 | 27.5 | 28.5 |
| 12 | 0.0 | 0.0 | 0.0 | 0.4 | 0.6 |
| 13 | 8.3 | 9.6 | 8.7 | 10.1 | 9.6 |
| 14 | 15.7 | 17.1 | 27.8 | 38.8 | 32.4 |
| 15 | 7.4 | 9.2 | 7.8 | 8.4 | 8.7 |
| 16 | 0.6 | -0.7 | 3.2 | 4.9 | 3.7 |

Table 6.6. Peak reduction effect for the pond simulation (relative to the baseline simulation) at main stem locations. Reductions (%) for the 2-, 10-, 25-, and 50-year return periods. The average (%) is the average reduction based on all 65 ranked annual maximum events.

| Location | Average | 2-year | 10-year | 25-year | 50-year |
|---|---|---|---|---|---|
| Otter Creek @ OTTRCRK01 | 1.2 | 0.8 | 3.5 | 3.7 | 4.3 |
| Otter Creek above Trib #4 Confluence | 1.2 | 0.8 | 3.5 | 3.3 | 5.1 |
| Otter Creek above Trib #5 Confluence | 1.3 | 0.9 | 3.8 | 3.6 | 5.4 |
| Otter Creek above Trib #7 Confluence | 1.4 | 0.7 | 4.0 | 5.4 | 5.7 |
| Otter Creek above Trib #10 | 1.5 | 0.8 | 4.3 | 5.5 | 5.7 |
| Otter Creek above Trib #13 Confluence | 1.7 | 1.2 | 4.8 | 6.0 | 6.3 |
| Otter Creek above Trib #14 | 0.5 | 0.3 | 1.2 | 1.7 | 1.9 |
| Otter Creek above Trib #15 Confluence | 0.0 | 0.0 | 0.3 | 0.6 | 0.7 |

To illustrate how the pond projects change simulated flood peaks, Figure 6.19 and 6.20 map the peak reduction at sub-basin outlets throughout the watershed for the 50-year return period. Note that some pond outlets (circles) have very high peak reductions, while others are much lower. The peak reduction at pond outlets range from about 6% to almost 90%. Flood storage is most effective immediately downstream of a pond. As one moves downstream from a structure, the peak reduction effect diminishes rapidly. The effect continues to diminish along the main stem below all 29 pond locations, to a minimum of just 4.3% at the Otter Creek outlet.

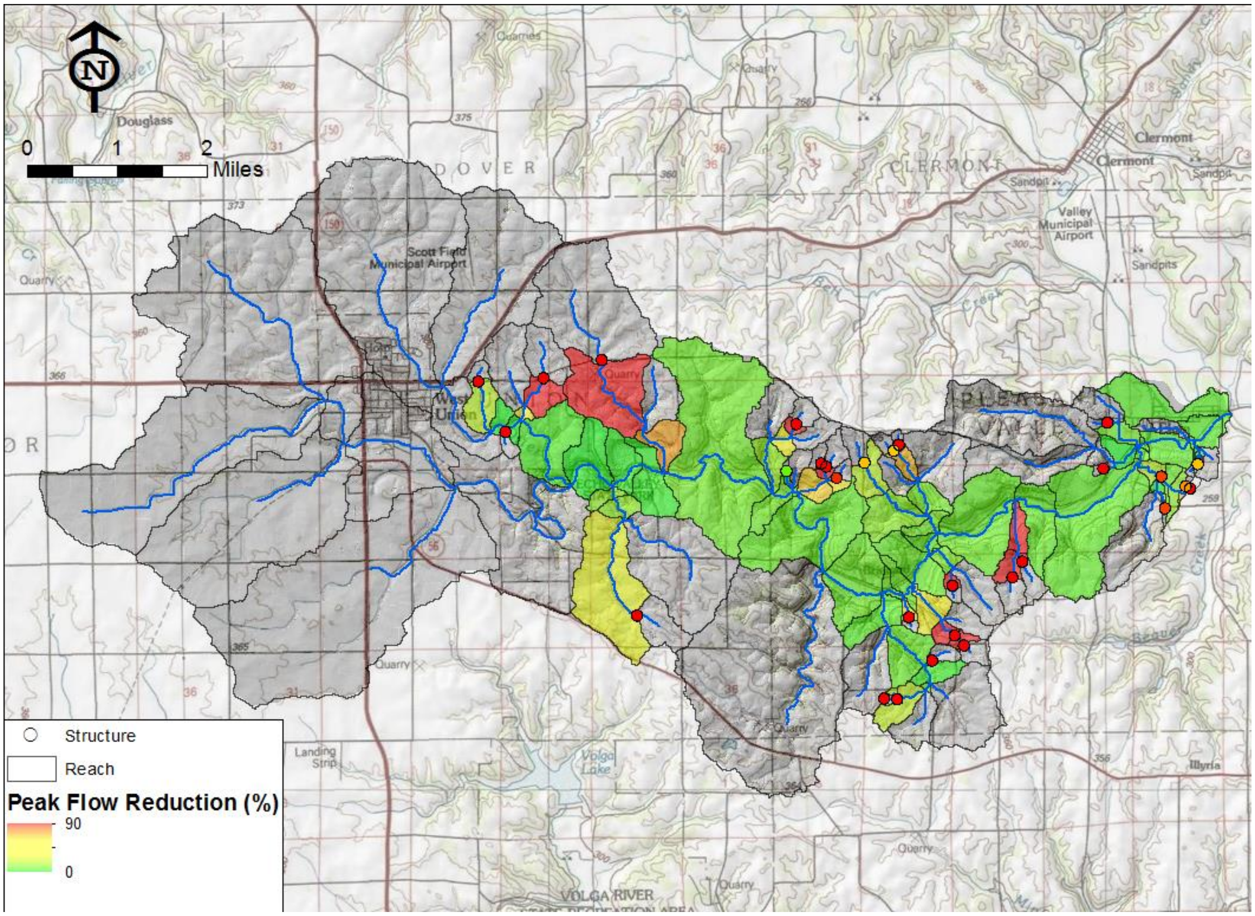


Figure 6.19. Average peak discharge reduction (%) for locations in the Otter Creek Watershed for the 50-year return period flood. We computed the peak reduction effect from the 65 ranked annual events.

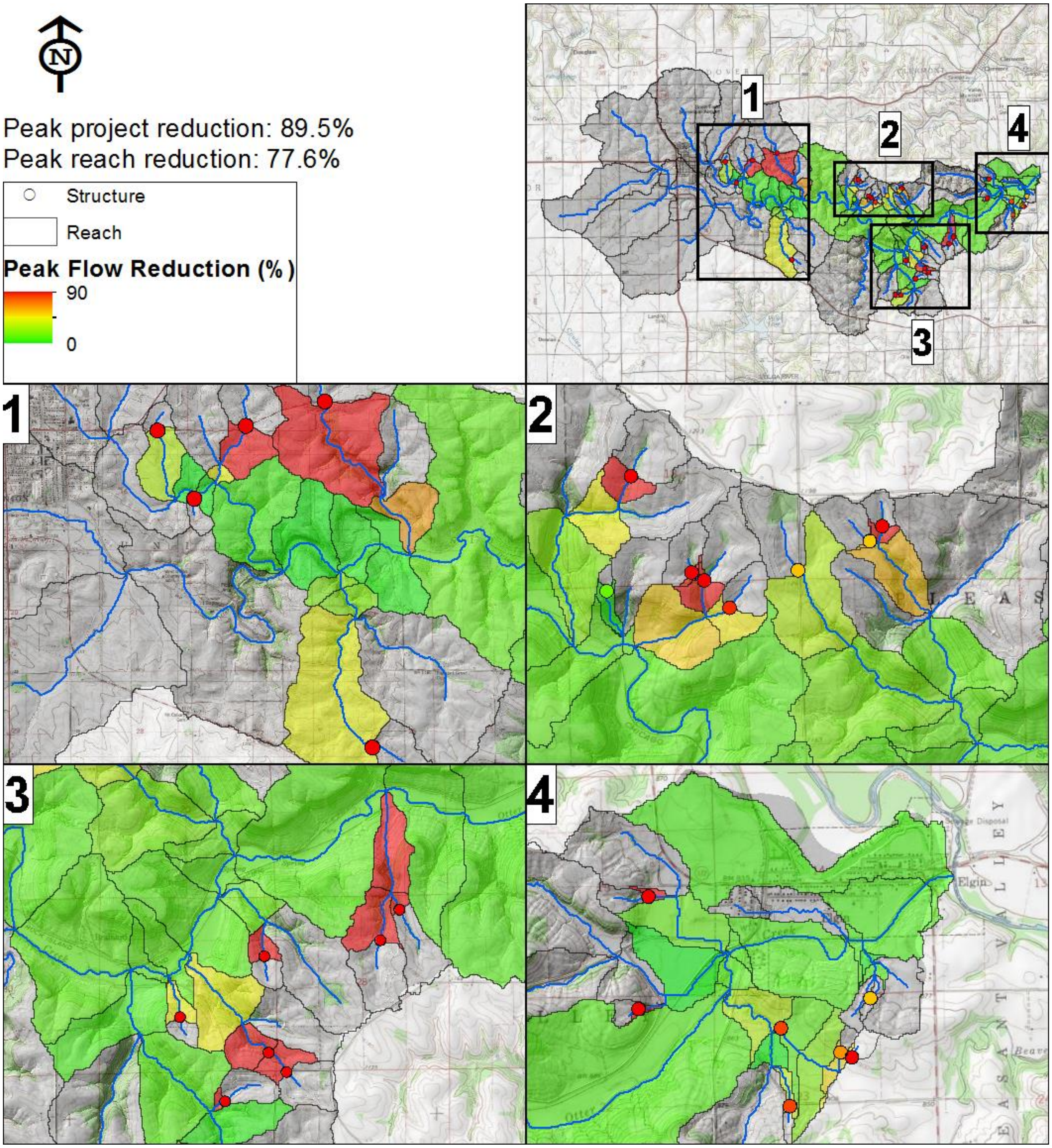


Figure 6.20. Average peak discharge reduction (%) for locations in the Otter Creek tributaries for the 50-year return period flood. We computed the peak reduction effect from the 65 ranked annual events.

## Effects of Additional Hypothetical Ponds

The Iowa Watersheds Project pond projects are effective in reducing peak discharges in Otter Creek tributaries. However, the peak reduction is modest in the main stem reaches. Because of the siting of ponds within the watershed, no peak reduction occurs in the upper main stem reaches. Additional investments in pond projects could improve peak reduction in these areas. We can use the Otter Creek HSPF model to explore potential options for future project investments. In this section, we investigate several hypothetical pond scenarios.

For these hypothetical scenarios, additional ponds are distributed in tributary areas of the Otter Creek Watershed. Because an actual pond design requires detailed site-specific information, we used a prototype pond design mimicking the hydrologic impacts of flood storage. Therefore, these hypothetical examples are not a proposed plan for siting additional ponds. We have not determined whether suitable sites are available in the simulated locations. Still, these examples do provide a quantitative benchmark on the effectiveness of additional flood storage and the flood reduction benefits that are physically possible.

For this analysis, we developed prototype ponds based on the stage-storage relationship of the constructed Iowa Watersheds Project ponds. For the first scenario, we created a medium-sized prototype pond based on those with drainage areas of about 50 acres. After plotting their stage-discharge relationship, we found that averaging the stage-storage-discharge for two ponds (McMillan #2 and #4 sites) results in a realistic average relationship. We assumed that hypothetical ponds would be sited at locations with a 50-acre drainage area. For the second scenario, we created a large-sized prototype pond based on the largest pond (Open Range Farms Pond), which drains an area of 140 acres. We used the stage-storage-discharge for the Open Range Farms Pond for the large-sized hypothetical ponds. We assumed that hypothetical ponds would be sited at locations with a 140-acre drainage area. For the third scenario, we assumed that three on-road detention areas were added at road crossings in upstream tributaries. We used the stage-storage-discharge for the Golden On-Road #3 pond for the on-road hypothetical ponds.

Figure 6.21 shows the locations of pond sites for the three hypothetical scenarios. For the first two scenarios, the tributary areas we selected for ponds are shown. Most of the hypothetical ponds are sited in unregulated tributaries in the upper watershed. A few others are sited in Tributaries #16 and #3, which have large unregulated portions and a small peak reduction with the existing Iowa Watershed Project ponds (see Table 6.5). The number of ponds in each of these tributaries depends on its size. We assumed that roughly 10% of their drainage area passes through the hypothetical ponds. Based on the drainage area assumed for the ponds (50 acres for ponds in the first scenario and 140 acres for ponds in the second scenario), we determined the number of ponds in each tributary. For the first scenario, this results in 30 hypothetical medium-sized ponds. For the second scenario, this results in 14 hypothetical large-sized ponds. Figure 6.21 also shows the location of the three road crossings for the third hypothetical scenario. We assumed that on-road ponds could be used at these sites. Engineers would need to do additional work to determine whether they are actually feasible sites. Our analysis is meant to illustrate the potential effects that additional on-road ponds might have.

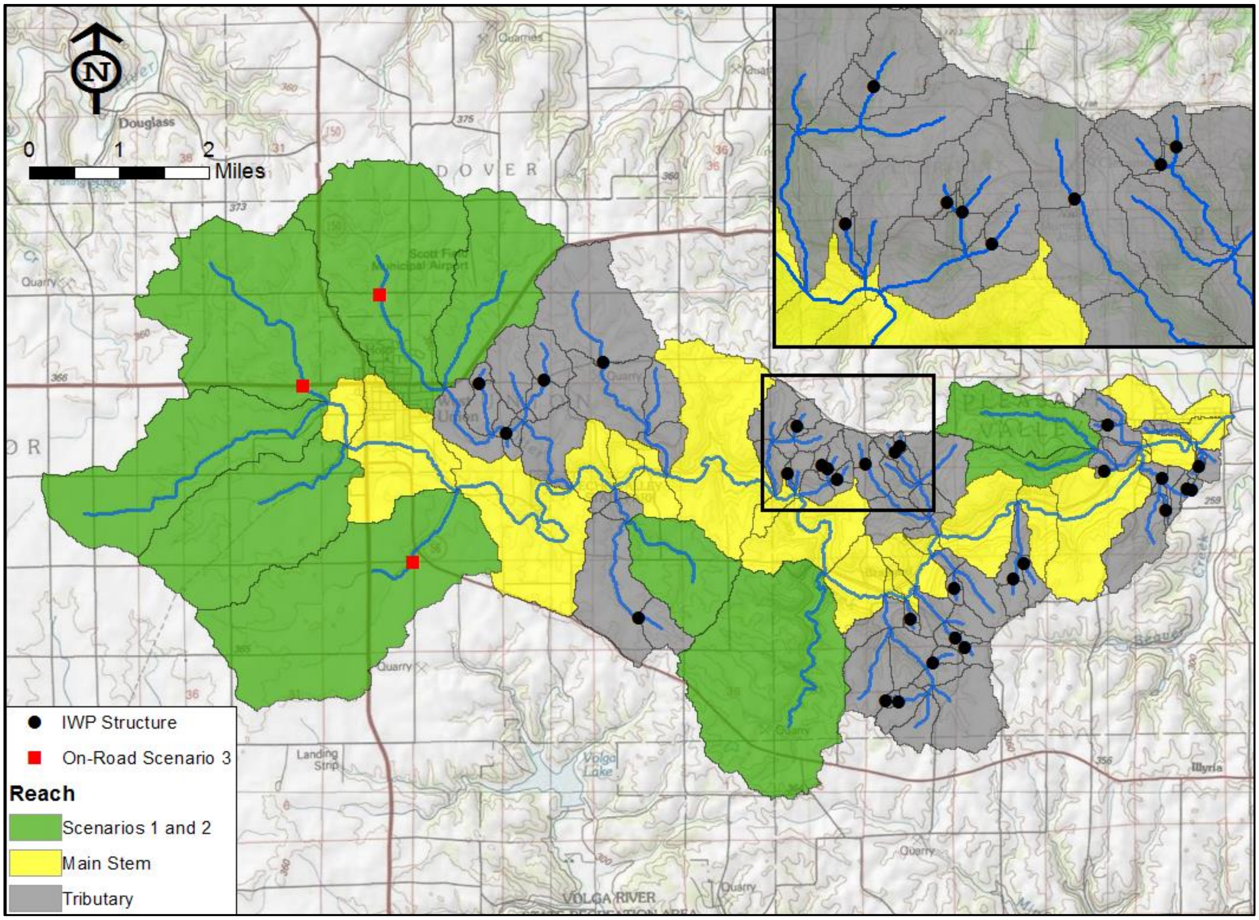


Figure 6.21. Pond site locations for the three hypothetical pond scenarios. The green shaded areas are tributaries where hypothetical ponds are located for Scenarios 1 and 2. The red squares are approximate locations for on-road ponds for Scenario 3. We simulated the three hypothetical pond scenarios to assess potential future flood reduction beyond what was achieved with the Iowa Watersheds Project Phase II flood mitigation structures.

Figure 6.22 shows simulated hydrographs for the May 1962 flood for the hypothetical pond scenarios on the main stem above Tributary #14. With the constructed Iowa Watersheds Project ponds, the peak reduction at this location is only 1.9%. However, with the medium-sized ponds added in the tributaries, the peak reduction is 6.1% (Scenario 1). With the large-sized ponds added in the tributaries, the peak reduction increases to 10.5% (Scenario 2). And with just three on-road ponds added in upstream tributaries, the peak reduction increases to 14.6% (Scenario 3).

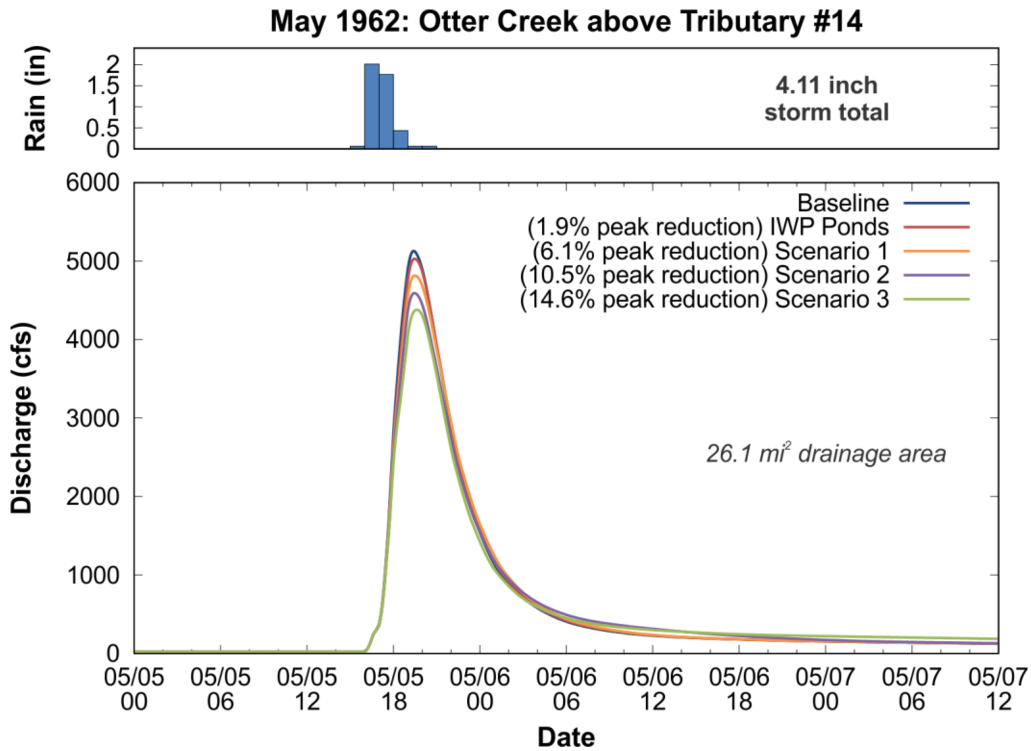


Figure 6.22. Flood hydrographs for the May 1962 event for the Otter Creek main stem above Tributary #14. Results are shown for the baseline (no ponds) and the Iowa Watersheds Project pond simulation, as well as the three additional hypothetical wetland scenarios.

Based on a flood frequency analysis of the hypothetical pond scenarios, Table 6.7 summarizes the peak reduction at different return periods along the main stem locations. Figure 6.23 shows the 50-year return period flood peak reduction at all main stem locations. With the Iowa Watersheds Project ponds, there is no peak reduction on the main stem upstream of Tributary #16. Then the 50-year peak reduction rises to a maximum of near 6.3% downstream of Tributary #14. Moving further downstream, the peak reduction slowly decreases to 4.3% at OTTCRK01. Adding the medium-sized ponds in Scenario 1 increases the peak reduction along the entire main stem. The minimum 50-year peak reduction is 4.3% just upstream of Tributary #16. The maximum reaches 9.9% downstream of Tributary #14. And the peak reduction slowly decreases to 5.9% at OTTCRK01. Adding the large-sized ponds in Scenario 2 increases the peak reduction even more. The maximum 50-year peak reduction is now 13.3% downstream of Tributary #14, and reduces to 9.9% at OTTCRK01. Adding the three on-road ponds at upstream locations has the largest peak reductions. The maximum 50-year peak reduction is 17.0% downstream of Tributary #14, and reduces to 11.3% at OTTCRK01.

Table 6.7. Peak reduction effect for the hypothetical pond simulation scenarios (relative to the baseline simulation) at main stem locations. Reductions (%) for the 2-, 10-, 25-, and 50-year return periods. The average (%) is the average reduction based on all 65 ranked annual maximum events.

| Location | Average | 2-year | 10-year | 25-year | 50-year |
|---|---|---|---|---|---|
| | | | *Hypothetical Scenario 1* | | |
| Otter Creek @ OTTRCRK01 | 1.9 | 1.5 | 5.4 | 5.2 | 5.9 |
| Otter Creek above Trib #4 Confluence | 2.0 | 1.6 | 5.6 | 4.5 | 7.2 |
| Otter Creek above Trib #5 Confluence | 2.1 | 1.7 | 5.9 | 4.7 | 7.7 |
| Otter Creek above Trib #7 Confluence | 2.3 | 1.4 | 6.2 | 6.2 | 8.4 |
| Otter Creek above Trib #10 | 2.6 | 1.5 | 7.1 | 9.2 | 9.2 |
| Otter Creek above Trib #13 Confluence | 2.9 | 1.7 | 7.8 | 9.9 | 9.9 |
| Otter Creek above Trib #14 | 2.0 | 0.8 | 5.0 | 6.1 | 6.0 |
| Otter Creek above Trib #15 Confluence | 1.5 | 0.5 | 4.2 | 5.2 | 5.1 |
| | | | *Hypothetical Scenario 2* | | |
| Otter Creek @ OTTRCRK01 | 5.3 | 6.0 | 8.9 | 9.3 | 9.9 |
| Otter Creek above Trib #4 Confluence | 5.6 | 6.4 | 9.5 | 8.8 | 11.3 |
| Otter Creek above Trib #5 Confluence | 5.7 | 6.5 | 9.8 | 9.0 | 11.8 |
| Otter Creek above Trib #7 Confluence | 6.0 | 6.3 | 10.2 | 10.5 | 12.8 |
| Otter Creek above Trib #10 | 6.5 | 6.3 | 11.2 | 13.4 | 13.2 |
| Otter Creek above Trib #13 Confluence | 6.8 | 5.1 | 11.9 | 14.1 | 13.9 |
| Otter Creek above Trib #14 | 6.4 | 6.1 | 9.3 | 10.5 | 10.4 |
| Otter Creek above Trib #15 Confluence | 6.0 | 5.8 | 8.8 | 9.7 | 9.5 |
| | | | *Hypothetical Scenario 3* | | |
| Otter Creek @ OTTRCRK01 | 12.3 | 17.7 | 20.0 | 11.4 | 11.3 |
| Otter Creek above Trib #4 Confluence | 12.9 | 20.1 | 20.6 | 10.5 | 12.5 |
| Otter Creek above Trib #5 Confluence | 13.1 | 20.3 | 21.1 | 10.8 | 13.0 |
| Otter Creek above Trib #7 Confluence | 13.9 | 20.6 | 22.2 | 11.9 | 14.3 |
| Otter Creek above Trib #10 | 16.3 | 22.5 | 24.8 | 14.1 | 15.9 |
| Otter Creek above Trib #13 Confluence | 17.0 | 21.5 | 25.9 | 15.5 | 17.0 |
| Otter Creek above Trib #14 | 17.9 | 24.1 | 25.2 | 14.0 | 14.8 |
| Otter Creek above Trib #15 Confluence | 19.4 | 26.0 | 26.1 | 14.4 | 13.8 |

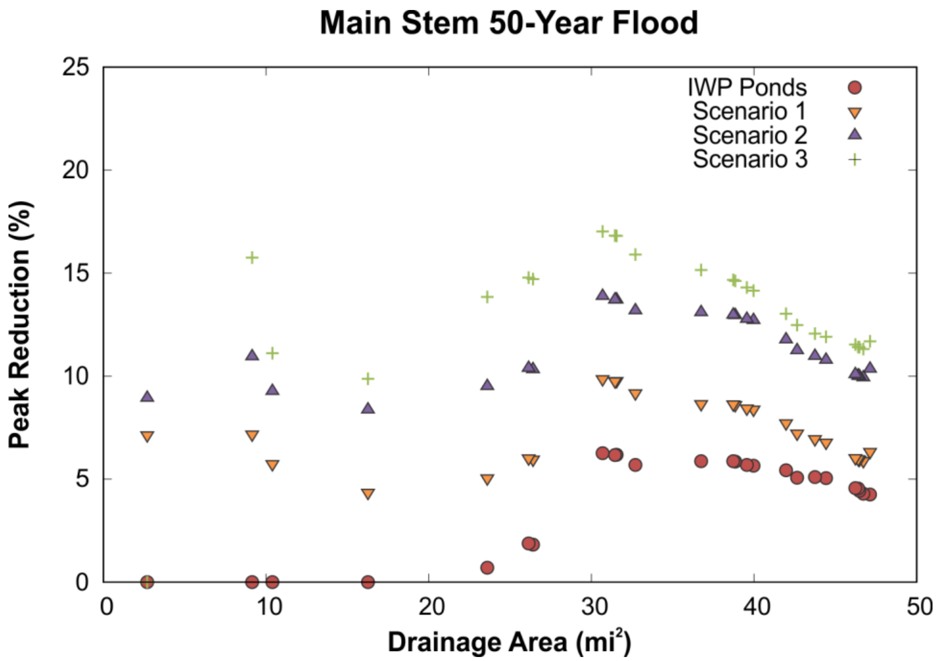


Figure 6.23. Peak reduction (%) for the 50-year return period peak discharge for all main stem locations in the Otter Creek Watershed. We show the peak reductions as a function of the location's drainage area (in mi²). The peak reductions are for the Iowa Watersheds Project ponds and for the three hypothetical pond project scenarios: Scenario 1 (medium-sized ponds), Scenario 2 (large-sized ponds), and Scenarios 3 (on-road ponds).

## l. Summary

We used the Otter Creek HSPF model to examine flooding characteristics in the Otter Creek Watershed. We created the model from the Turkey River HSPF model developed by the Iowa Flood Center (Leach, 2015). We extracted the model parameters representing different land uses and used them to simulate runoff in Otter Creek. A network of stream reaches was then created to route water and simulate flows throughout the Otter Creek Watershed. For simulations, we assembled weather inputs for a 65-year period. Based on the results from the 65-year simulation, we identified the top simulated flood events. Each was seen to produce widespread flooding throughout the watershed.

We then used the Otter Creek HSPF model to assess the performance of pond projects in the watershed. We simulated the operation of the twenty-nine Iowa Watersheds Project constructed ponds for the 65-year historical period. We found that the pond projects significantly reduced flood peak discharges within the tributaries. Figure 6.24 summarizes the peak reduction for the 50-year return period discharge (used here as a measure of a significant flood event) for tributary locations. Peak reduction is large at the project outlets, ranging from about 6% at the McMillian #1 pond (with the smallest upstream drainage area) to almost 90% at Open Range Farms pond (the largest pond with the largest upstream drainage area). Downstream of the pond projects, the peak reduction diminishes (green diamonds on Figure 6.24). The peak reduction tends to be higher for tributaries where a larger fraction of the area drains through ponds. Along the Otter

Creek main stem (not shown), the flood peak reduction is less. For locations downstream of tributaries with pond projects, the peak reduction ranges from 1 to about 6%.

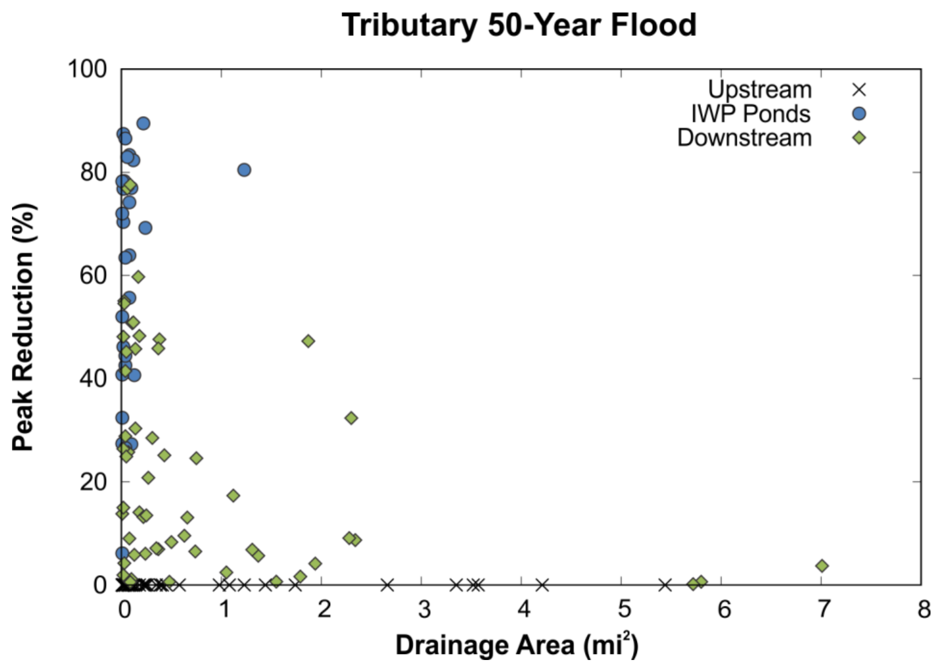


Figure 6.24. Peak reduction (%) for the 50-year return period peak discharge for all simulated locations in the Otter Creek tributaries. Locations upstream of projects (shown with an ×) are unregulated and have no peak reduction. Peak reductions at the ponds (blue circles) and downstream (green diamonds) are shown as a function of their drainage area (in $mi^2$).

To further reduce peak discharges on the Otter Creek main stem, we explored three hypothetical pond project scenarios. Figure 6.23 summarizes the results for the 50-year return period discharge. The scenarios show a clear preference for future project investments in flood mitigation. If road crossings are available for on-road detention, these sites should be the first choice. Because they regulate flows from large drainage areas and have the largest storage volumes, they are the most effective in reducing peak discharges. With only three ponds, this hypothetical scenario produced lower peak discharges than the other two scenarios. After on-road detention, large-sized ponds should be favored over medium-sized ponds. By design, both the large-sized and medium-sized hypothetical pond scenarios regulated the same percentage of the tributary drainage areas (about 10%). However, adding a fewer number of large-sized ponds was more effective than adding a larger number of medium-sized ponds.

# 7. Summary and Conclusions

The Iowa Flood Center (IFC), a unit of the University of Iowa's IIHR—Hydroscience & Engineering (IIHR), has collaborated with the Turkey River Watershed Management Authority (WMA) on Phase II of the Iowa Watersheds Project. Phase II involved the development and construction of flood mitigation projects in the Otter Creek Watershed, a sub-watershed of the Turkey River. In this report, IFC researchers evaluated the flood mitigation performance of proposed projects through monitoring and detailed hydrologic modeling. The team developed small-scale hydrologic simulations for the Otter Creek Watershed using a more detailed representation of the watershed and flood mitigation strategies than was used in the Phase I study of the entire Turkey River Watershed.

## a. Monitoring Stations and Data Collection

Data collection before and after implementation of the watershed projects was especially critical for the Iowa Watersheds Project. In the Otter Creek Watershed, we used monitoring equipment to quantify the benefits of constructed projects and to provide critical information to help Iowans make better informed decisions about the implementation, design, size, cost, and impact of additional watershed projects.

The Iowa Flood Center has been collecting data from four stream-stage sensors and five rainfall/soil moisture platforms deployed in the Otter Creek Watershed. The information from this deployed instrumentation network is made available to the public in real-time on the Iowa Flood Information System (IFIS) (http://ifis.iowafloodcenter.org/ifis), a user-friendly Google Maps interface.

In addition, IIHR has three water-quality sensors in the watershed to monitor the nutrient-reduction benefits of constructed projects. Sensors collect data in real-time, which is made available to the public through the Iowa Water-Quality Information System (Iowa WQIS) (http://iwqis.iowawis.org/). By incorporating hydrologic information with water-quality data, scientists, policy-makers, and interested stakeholders will be able to better understand how various hydrologic drivers impact the fate and transport of nutrients in Iowa's waterways.

## b. Constructed Projects

In 2014 and 2015, the Iowa Watersheds Project spent a total of $1,500,000 to design and construct 29 projects in the Otter Creek Watershed: five on-road structures, 19 ponds, and five terrace and sediment control structures. All projects were designed to reduce flooding by increasing the storage capacity on the landscape; some of them provide a secondary benefit of improving water quality through nutrient processing. Some projects also increase the aesthetic beauty of the land and have the potential of creating habitat for wildlife. The constructed structures act as demonstration projects to promote the adoption of best management practices and provide education and outreach opportunities.

Volunteer landowners received 75% cost-share assistance on constructed projects. The project designs followed Natural Resources Conservation Service (NRCS) specifications and guidelines.

The projects come with a 20-year maintenance agreement. Project locations were selected based on recommendations from the Fayette Soil and Water Conservation District staff, input from the Turkey River Watershed Management Authority, and with consultation with the Iowa Flood Center.

## c. Evaluation of Project Performance

We evaluated the performance of the constructed projects with two hydrologic models. The Otter Creek HydroGeoSphere (HGS) model is a high-resolution physics-based model that simulates water storage and movement at almost 37,000 grid elements within the watershed. It can track the movement of water at the land surface and in the sub-surface (soils). The model was calibrated and validated in hydrologic time series collected in 2014 and 2015, respectively. In addition, the HGS model was used to evaluate the performance of a selected project for design rainfall events the entire watershed for the June 2008 event. Since such a detailed model can take several days of computer time to simulate a year's worth of conditions, we also used a simpler model to evaluate project performance over a long historical period. The Otter Creek Hydrological Simulation Program-FORTRAN (HSPF) model lumps the watershed area into seven distinct land uses and makes predictions at 136 stream locations in the watershed. It accounts for water on the landscape continuously in time through a 65-year simulation period. We used historical weather records from nearby stations as input to drive the simulation.

Both hydrologic models demonstrate the effectiveness of the twenty-nine Iowa Watersheds Project wetlands in reducing downstream flood peaks. Just downstream from the projects themselves, peak discharge reduction for design or historical events is significant, even for large flood events. The ponds perform very well at around the 50-year return period, where peak reductions range from 6% to 90%. As one moves downstream from the projects, the peak reduction effect diminishes. Flood peaks at Otter Creek main stem locations see less peak reduction. For the 50-year return period, the peak reduction ranges from about 1% to 6%. Simulations in which hypothetical ponds are added to tributary locations illustrate how additional investments in flood mitigation could enhance flood peak reduction on the main stem. On-road structure are most effective in reducing main stem flood peaks. A smaller number of large-size ponds are more effective than are greater number of small or medium-sized ponds.

## d. Concluding Comments

These watershed demonstration projects have taught us much about the movement of water through the landscape and serve as an essential step toward long-term recovery to improve Iowa's future flood resiliency. The hydrologic assessment, watershed plan, and project evaluation will guide future decision making to expand project implementation to other sub-watersheds in the Turkey River Watershed. The watershed planning and project implementation conducted through the Iowa Watersheds Project will serve as leverage for the Turkey River WMA to seek additional funding for continued work toward its long-term goals.

In January 2016, the U.S. Department of Housing and Urban Development (HUD) awarded $96.9 million to Iowa for a statewide watershed improvement program, the Iowa Watersheds Approach

(IWA). The IWA will address issues associated with the devastating and dangerous floods Iowa communities experience year after year. The foundation of the IWA was built on the framework and success of the IWP, which served as a significant source of leverage for the state of Iowa to receive another round of HUD funding for a new five-year project.

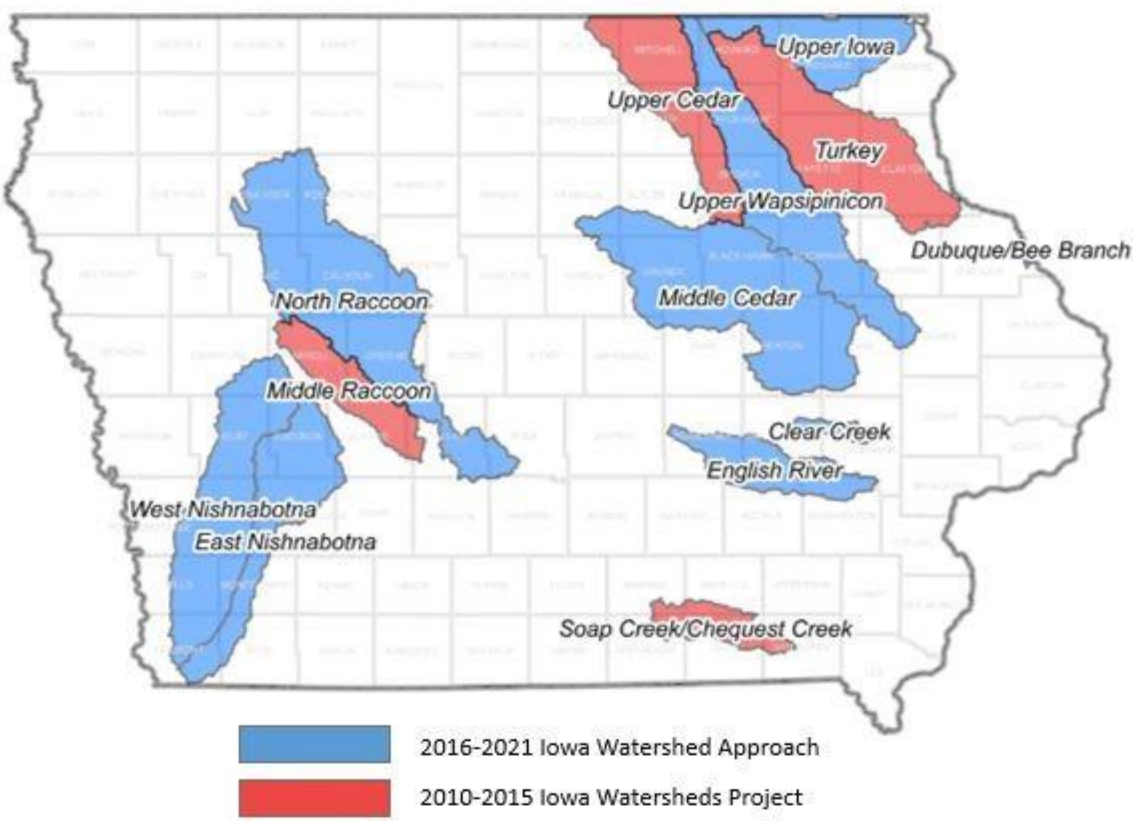


Figure 7.1. Location of watersheds selected for the Iowa Watersheds Project and the Iowa Watershed Approach.

The IWA project will work in nine new watersheds across the state: Bee Branch Creek in Dubuque, Upper Iowa River, Upper Wapsipinicon River, Middle Cedar River, Clear Creek, English River, North Raccoon River, West Nishnabotna River, and East Nishnabotna River. Each will have the opportunity to form a Watershed Management Authority (WMA), develop a hydrologic assessment and watershed plan, and implement projects to reduce the magnitude of downstream flooding and to improve water quality during and after flood events.

A video explaining the Iowa Watersheds Project and Iowa Watershed Approach can be accessed at https://www.youtube.com/watch?v=tODPRvs4ycU.

# Appendix A – Iowa Watersheds Project Phase II Pond Stage-Storage-Discharge Relationships

**Project:** **Golden On-Road #1** **Pond ID #R1**
Drainage Area: 38 acres (0.06 square miles)
Description: This is an on-road structure adaptation on an existing 3' x 3' box culvert. It consists of 6' diameter manhole. The principal spillway is a 15" reinforced concrete pipe (RCP), invert elevation of 1151.0 feet MSL. The auxiliary spillway consists of a 12" diameter orifice with invert of 1157.0 feet MSL and then top inlet with elevation 1159.5 feet MSL. Top of road is approximately 1166.3 feet MSL.
Hydraulic Design: Fehr Graham, West Union, Iowa

Golden On-Road #1: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0 | 1151.0 | 0 | 0 | principal spillway: 1151.0 |
| 1 | 1152.0 | 0.02 | 0.01 | |
| 2 | 1153.0 | 0.18 | 0.11 | |
| 3 | 1154.0 | 0.78 | 0.59 | |
| 4 | 1155.0 | 1.34 | 1.65 | |
| 5 | 1156.0 | 1.90 | 3.27 | |
| 6 | 1157.0 | 2.41 | 5.42 | auxiliary spillway: 1157.0 |
| 7 | 1158.0 | 2.95 | 8.10 | |
| 8 | 1159.0 | 3.45 | 11.30 | |
| 9 | 1160.0 | 3.92 | 14.99 | |
| 10 | 1161.0 | 4.61 | 19.26 | |
| 11 | 1162.0 | 5.30 | 23.83 | |
| 12 | 1163.0 | 5.98 | 28.41 | |
| 13 | 1164.0 | 6.67 | 32.98 | |
| 14 | 1165.0 | 7.36 | 37.56 | |
| 15 | 1166.0 | 8.05 | 42.13 | top of road: 1166.3 |
| 16 | 1167.0 | 8.73 | 46.71 | |

Golden On-Road #1: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0 | 1151.0 | 0 | 0 | principal spillway: 1151.0 |
| 1 | 1152.0 | 0.01 | 3.58 | |
| 2 | 1153.0 | 0.11 | 6.93 | |
| 3 | 1154.0 | 0.59 | 9.11 | |
| 4 | 1155.0 | 1.65 | 10.86 | |
| 5 | 1156.0 | 3.27 | 12.36 | |
| 6 | 1157.0 | 5.42 | 13.70 | auxiliary spillway: 1157.0 |
| 7 | 1158.0 | 8.10 | 17.59 | |
| 8 | 1159.0 | 11.30 | 20.68 | |
| 10 | 1161.0 | 19.26 | 123.68 | |
| 11 | 1162.0 | 23.83 | 129.45 | continued on next page |

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 12 | 1163.0 | 28.41 | 134.96 | |
| 13 | 1164.0 | 32.98 | 140.24 | |
| 14 | 1165.0 | 37.56 | 145.31 | |
| 15 | 1166.0 | 42.13 | 150.19 | top of road: 1166.3 |
| 16 | 1167.0 | 46.71 | 154.91 | |

**Project:** **Golden On-Road #2** **Pond ID #R2**
Drainage Area: 144 acres (0.23 square miles)
Description: This is an on-road structure adaptation on an existing 42” reinforced concrete pipe (RCP) culvert. It consists of 6’ diameter manhole. The principal spillway is an 18” reinforced concrete pipe, invert elevation of 1132.19 feet MSL. The auxiliary spillway consists of a 15” diameter orifice with invert of 1139.0 feet MSL and then top inlet with elevation 1142.5 feet MSL. Top of road is approximately 1153.0 feet MSL.
Hydraulic Design: Fehr Graham, West Union, Iowa

Golden On-Road #2: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0 | 1132.19 | 0 | 0 | principal spillway: 1132.19 |
| 0.8 | 1133.0 | 0.002 | 0.001 | |
| 1.8 | 1134.0 | 0.01 | 0.01 | |
| 2.8 | 1135.0 | 0.02 | 0.02 | |
| 3.8 | 1136.0 | 0.06 | 0.06 | |
| 4.8 | 1137.0 | 0.45 | 0.32 | |
| 5.8 | 1138.0 | 0.92 | 1.00 | |
| 6.8 | 1139.0 | 1.36 | 2.14 | auxiliary spillway: 1139.0 |
| 7.8 | 1140.0 | 1.93 | 3.79 | |
| 8.8 | 1141.0 | 2.52 | 6.01 | |
| 9.8 | 1142.0 | 3.15 | 8.85 | |
| 10.8 | 1143.0 | 3.90 | 12.37 | |
| 11.8 | 1144.0 | 4.64 | 16.64 | |
| 12.8 | 1145.0 | 5.37 | 21.65 | |
| 13.8 | 1146.0 | 5.74 | 27.15 | |
| 14.8 | 1147.0 | 5.75 | 27.44 | |
| 15.8 | 1148.0 | 5.77 | 27.72 | |
| 16.8 | 1149.0 | 5.79 | 28.01 | |
| 17.8 | 1150.0 | 5.81 | 28.30 | |
| 18.8 | 1151.0 | 5.83 | 28.58 | |
| 19.8 | 1152.0 | 5.85 | 28.87 | |
| 20.8 | 1153.0 | 5.86 | 29.16 | top of road: 1153.0 |

Golden On-Road #2: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0 | 1132.19 | 0 | 0 | principal spillway: 1151.0 |
| 0.8 | 1133.0 | 0.001 | 2.99 | |
| 1.8 | 1134.0 | 0.01 | 8.76 | |
| 2.8 | 1135.0 | 0.02 | 12.21 | |
| 3.8 | 1136.0 | 0.06 | 14.88 | |
| 4.8 | 1137.0 | 0.32 | 17.14 | |
| 5.8 | 1138.0 | 1.00 | 19.14 | auxiliary spillway: 1157.0 |
| 6.8 | 1139.0 | 2.14 | 20.96 | |
| 7.8 | 1140.0 | 3.79 | 26.18 | |
| 8.8 | 1141.0 | 6.01 | 31.09 | continued on next page |

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 10.8 | 1143.0 | 12.37 | 70.41 | |
| 11.8 | 1144.0 | 16.64 | 146.93 | |
| 12.8 | 1145.0 | 21.65 | 154.06 | |
| 13.8 | 1146.0 | 27.15 | 160.88 | |
| 14.8 | 1147.0 | 27.44 | 167.41 | |
| 15.8 | 1148.0 | 27.72 | 173.70 | |
| 16.8 | 1149.0 | 28.01 | 179.78 | |
| 17.8 | 1150.0 | 28.30 | 185.66 | |
| 18.8 | 1151.0 | 28.58 | 191.69 | |
| 19.8 | 1152.0 | 28.87 | 197.42 | |
| 20.8 | 1153.0 | 29.16 | 203.04 | top of road: 1153.0 |

**Project:** **Golden On-Road #3** **Pond ID #R3**
Drainage Area: 800 acres (1.25 square miles)
Description: This is an on-road structure adaptation on an existing 10' x 10' box culvert. It consists of 10' x 10' manhole. The principal spillway is a 24" reinforced concrete pipe, invert elevation of 1123.60 feet MSL. The auxiliary spillway consists of a 20" x 20" orifice with invert of 1135.8 feet MSL and then top inlet with elevation 1138.5 feet MSL. Top of road is approximately 1151.0 feet MSL.
Hydraulic Design: Fehr Graham, West Union, Iowa

Golden On-Road #3: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0 | 1123.37 | 0 | 0 | principal spillway: 1123.60 |
| 0.63 | 1124.0 | 0.01 | 0.002 | |
| 1.63 | 1125.0 | 0.13 | 0.05 | |
| 2.63 | 1126.0 | 0.71 | 0.45 | |
| 3.63 | 1127.0 | 1.74 | 1.66 | |
| 4.63 | 1128.0 | 2.75 | 3.91 | |
| 5.63 | 1129.0 | 3.74 | 7.15 | |
| 7.63 | 1131.0 | 5.82 | 16.69 | |
| 9.63 | 1133.0 | 7.53 | 30.05 | |
| 11.63 | 1135.0 | 9.42 | 46.93 | auxiliary spillway: 1135.8 |
| 12.63 | 1136.0 | 10.64 | 56.95 | |
| 13.63 | 1137.0 | 12.14 | 68.35 | |
| 14.63 | 1138.0 | 13.72 | 81.26 | |
| 15.63 | 1139.0 | 15.59 | 95.90 | |
| 16.63 | 1140.0 | 17.55 | 112.48 | |
| 17.63 | 1141.0 | 19.50 | 131.00 | |
| 18.63 | 1142.0 | 21.58 | 151.53 | |
| 19.63 | 1143.0 | 23.32 | 172.33 | |
| 20.63 | 1144.0 | 24.14 | 182.44 | |
| 21.63 | 1145.0 | 24.95 | 192.56 | |
| 23.63 | 1147.0 | 26.58 | 212.78 | |
| 26.63 | 1150.0 | 29.02 | 243.13 | top of road: 1151.0 |

Golden On-Road #3: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0 | 1123.37 | 0 | 0 | principal spillway: 1151.0 |
| 0.63 | 1124.0 | 0.002 | 0.87 | |
| 1.63 | 1125.0 | 0.05 | 7.59 | |
| 2.63 | 1126.0 | 0.45 | 15.75 | |
| 3.63 | 1127.0 | 1.66 | 20.68 | |
| 4.63 | 1128.0 | 3.91 | 24.61 | |
| 5.63 | 1129.0 | 7.15 | 28.00 | |
| 7.63 | 1131.0 | 16.69 | 33.76 | |
| 9.63 | 1133.0 | 30.05 | 38.68 | |
| 11.63 | 1135.0 | 46.93 | 43.04 | auxiliary spillway: 1135.8 |
| 12.63 | 1136.0 | 56.95 | 45.57 | continued on next page |

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 15.63 | 1139.0 | 95.90 | 106.06 | |
| 16.63 | 1140.0 | 112.48 | 255.89 | |
| 17.63 | 1141.0 | 131.00 | 463.62 | |
| 18.63 | 1142.0 | 151.53 | 714.12 | |
| 19.63 | 1143.0 | 172.33 | 950.69 | |
| 20.63 | 1144.0 | 182.44 | 979.63 | |
| 21.63 | 1145.0 | 192.56 | 1007.74 | |
| 23.63 | 1147.0 | 212.78 | 1061.69 | |
| 26.63 | 1150.0 | 243.13 | 1138.63 | top of road: 1151.0 |

**Project:** **F. Avenue On-Road** **Pond ID #R4**
Drainage Area: 67.5 acres (0.11 square miles)
Description: This is an on-road structure connected to a new 42" reinforced concrete pipe (RCP) culvert. It consists of 6' diameter manhole. The principal spillway is a 12" reinforced concrete pipe, invert elevation of 1117.0 feet MSL. The auxiliary spillway consists of a 24" diameter orifice with invert of 1119.0 feet MSL, a second 36" diameter orifice with invert of 1122.0 feet MSL and then top inlet with elevation 1125.75 feet MSL. Top of road is approximately 1128.0 feet MSL.
Hydraulic Design: Fehr Graham, West Union, Iowa

F Avenue On-Road: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0 | 1117.0 | 0 | 0 | principal spillway: 1117.0 |
| 1.0 | 1118.0 | 0.02 | 0.008 | |
| 2.0 | 1119.0 | 0.09 | 0.06 | auxiliary spillway: 1119.0 |
| 3.0 | 1120.0 | 0.23 | 0.22 | |
| 4.0 | 1121.0 | 0.42 | 0.54 | |
| 5.0 | 1122.0 | 0.67 | 1.09 | |
| 6.0 | 1123.0 | 0.93 | 1.88 | |
| 7.0 | 1124.0 | 1.22 | 2.95 | |
| 8.0 | 1125.0 | 1.55 | 4.34 | |
| 9.0 | 1126.0 | 1.89 | 6.06 | |
| 10.0 | 1127.0 | 2.25 | 8.13 | |
| 11.0 | 1128.0 | 2.58 | 10.33 | |
| 11.6 | 1128.6 | 2.61 | 10.55 | top of road: 1128.0 |

F Avenue On-Road: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0 | 1117.0 | 0 | 0 | principal spillway: 1117.0 |
| 1.0 | 1118.0 | 0.008 | 2.64 | |
| 2.0 | 1119.0 | 0.06 | 4.67 | auxiliary spillway: 1119.0 |
| 3.0 | 1120.0 | 0.22 | 11.41 | |
| 4.0 | 1121.0 | 0.54 | 22.20 | |
| 5.0 | 1122.0 | 1.09 | 29.53 | |
| 6.0 | 1123.0 | 1.88 | 42.16 | |
| 7.0 | 1124.0 | 2.95 | 64.00 | |
| 8.0 | 1125.0 | 4.34 | 85.86 | |
| 9.0 | 1126.0 | 6.06 | 115.47 | |
| 10.0 | 1127.0 | 8.13 | 156.08 | |
| 11.0 | 1128.0 | 10.33 | 160.64 | |
| 11.6 | 1128.6 | 10.55 | 163.32 | top of road: 1128.0 |

**Project:** **Iowa DNR Pond** **Pond ID #1**
Drainage Area: 11.4 acres (0.02 square miles)
Description: The principal spillway is a 10" corrugated metal pipe (CMP), invert elevation of 1099.0 feet MSL. The auxiliary spillway is 40 feet wide, crest elevation at 1107.0 feet MSL. Top of dam at 1108.0 feet MSL.
Hydraulic Design: Fehr Graham, West Union, Iowa

Iowa DNR Pond: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0 | 1099.0 | 0 | 0 | principal spillway:1099.0 |
| 1.0 | 1100.0 | 0.17 | 0.09 | |
| 2.0 | 1101.0 | 0.24 | 0.29 | |
| 3.0 | 1102.0 | 0.30 | 0.56 | |
| 4.0 | 1103.0 | 0.37 | 0.90 | |
| 5.0 | 1104.0 | 0.44 | 1.30 | |
| 6.0 | 1105.0 | 0.50 | 1.77 | |
| 7.0 | 1106.0 | 0.57 | 2.31 | |
| 8.0 | 1107.0 | 0.63 | 2.90 | auxiliary spillway: 1107.0 |
| 8.5 | 1107.5 | 0.72 | 3.24 | |
| 9.0 | 1108.0 | 0.81 | 3.63 | |
| 9.5 | 1108.5 | 0.90 | 4.06 | top of dam: 1108.0 |
| 10.0 | 1109.0 | 0.99 | 4.53 | |

Iowa DNR Pond: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0 | 1099.0 | 0 | 0 | principal spillway:1099.0 |
| 1.0 | 1100.0 | 0.09 | 1.24 | |
| 2.0 | 1101.0 | 0.29 | 1.73 | |
| 3.0 | 1102.0 | 0.56 | 2.10 | |
| 4.0 | 1103.0 | 0.90 | 2.42 | |
| 5.0 | 1104.0 | 1.30 | 2.70 | |
| 6.0 | 1105.0 | 1.77 | 2.95 | |
| 7.0 | 1106.0 | 2.31 | 3.18 | |
| 8.0 | 1107.0 | 2.90 | 3.40 | auxiliary spillway: 1107.0 |
| 8.5 | 1107.5 | 3.24 | 9.71 | |
| 9.0 | 1108.0 | 3.63 | 33.09 | top of dam: 1108.0 |
| 9.5 | 1108.5 | 4.06 | 125.91 | |
| 10.0 | 1109.0 | 4.53 | 253.43 | |

**Project:** **Open Range Farms Pond** **Pond ID #2**
Drainage Area: 134 acres (0.21 square miles)
Description: The principal spillway is a 12" plastic pipe (PVC), invert elevation of 1133.1 feet MSL. The auxiliary spillway is 12 feet wide, crest elevation at 1137.8 feet MSL. Top of dam at 1140.1 feet MSL.
Hydraulic Design: NRCS, Elkader, Iowa

Open Range Farms Pond: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0 | 1133.1 | 2.17 | 0 | principal spillway:1133.1 |
| 0.9 | 1134.0 | 2.56 | 2.13 | |
| 1.9 | 1135.0 | 3.09 | 4.95 | |
| 2.9 | 1136.0 | 3.70 | 8.35 | |
| 3.9 | 1137.0 | 4.41 | 12.41 | |
| 4.7 | 1137.8 | 4.96 | 16.15 | auxiliary spillway: 1137.8 |
| 4.9 | 1138.0 | 5.10 | 17.16 | |
| 5.4 | 1138.5 | 5.44 | 19.80 | |
| 5.9 | 1139.0 | 5.78 | 22.60 | |
| 6.4 | 1139.5 | 6.15 | 25.58 | |
| 6.9 | 1140.0 | 6.52 | 28.75 | |
| 7 | 1140.1 | 6.60 | 29.41 | top of dam: 1140.1 |
| 7.9 | 1141.0 | 7.31 | 35.67 | |
| 8.9 | 1142.0 | 8.34 | 43.49 | |
| 9.9 | 1143.0 | 9.30 | 52.31 | |

Open Range Farms Pond: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0 | 1133.1 | 0 | 0 | principal spillway:1133.1 |
| 0.9 | 1134.0 | 2.13 | 1.83 | |
| 1.9 | 1135.0 | 4.95 | 11.35 | |
| 2.9 | 1136.0 | 8.35 | 11.71 | |
| 3.9 | 1137.0 | 12.41 | 12.06 | |
| 4.7 | 1137.8 | 16.15 | 12.33 | auxiliary spillway: 1137.8 |
| 4.9 | 1138.0 | 17.16 | 12.40 | |
| 5.4 | 1138.5 | 19.80 | 12.78 | |
| 5.9 | 1139.0 | 22.60 | 18.76 | |
| 6.4 | 1139.5 | 25.58 | 47.39 | |
| 6.9 | 1140.0 | 28.75 | 97.91 | |
| 7 | 1140.1 | 29.41 | 108.02 | top of dam: 1140.1 |
| 7.9 | 1141.0 | 35.67 | 230.50 | |
| 8.9 | 1142.0 | 43.49 | 405.56 | |
| 9.9 | 1143.0 | 52.31 | 630.64 | |

**Project:** **McMillian Pond #2 Pond ID #3**
Drainage Area: Not Specified in Design Documents
Description: The principal spillway is a 24" corrugated metal pipe (CMP), invert elevation of 1150.0 feet MSL. The auxiliary spillway is 24 feet wide, crest elevation at 1156.4 feet MSL. Top of dam at 1159.9 feet MSL.
Hydraulic Design: Fehr Graham, West Union, Iowa

McMillian Pond #2: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0 | 1150.0 | 1.09 | 0 | principal spillway:1150.0 |
| 1.0 | 1151.0 | 1.29 | 1.19 | |
| 2.0 | 1152.0 | 1.49 | 2.58 | |
| 3.0 | 1153.0 | 1.68 | 4.17 | |
| 4.0 | 1154.0 | 1.87 | 5.94 | |
| 5.0 | 1155.0 | 2.07 | 7.91 | |
| 6.0 | 1156.0 | 2.27 | 10.08 | auxiliary spillway: 1156.4 |
| 6.4 | 1156.4 | 2.35 | 11.00 | |
| 7.0 | 1157.0 | 2.48 | 12.46 | |
| 7.5 | 1157.5 | 2.59 | 13.72 | |
| 8.0 | 1158.0 | 2.70 | 15.04 | |
| 8.5 | 1158.5 | 2.80 | 16.42 | |
| 9.0 | 1159.0 | 2.91 | 17.85 | top of dam: 1159.9 |

McMillian Pond #2: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0 | 1150.0 | 0 | 0 | principal spillway:1150.0 |
| 1.0 | 1151.0 | 1.19 | 10.69 | |
| 2.0 | 1152.0 | 2.58 | 15.12 | |
| 3.0 | 1153.0 | 4.17 | 21.38 | |
| 4.0 | 1154.0 | 5.94 | 26.19 | |
| 5.0 | 1155.0 | 7.91 | 30.24 | |
| 6.0 | 1156.0 | 10.08 | 33.81 | |
| 6.4 | 1156.4 | 11.00 | 35.14 | auxiliary spillway: 1156.4 |
| 7.0 | 1157.0 | 12.46 | 41.85 | |
| 7.5 | 1157.5 | 13.72 | 57.37 | |
| 8.0 | 1158.0 | 15.04 | 130.10 | |
| 8.5 | 1158.5 | 16.42 | 221.04 | |
| 9.0 | 1159.0 | 17.85 | 332.24 | top of dam: 1159.9 |

**Project:** **McMillian Pond #1 Pond ID #4**
Drainage Area: Not Specified in Design Documents
Description: The principal spillway is a 10" corrugated metal pipe (CMP), invert elevation of 1124.75 feet MSL. The auxiliary spillway is 30.5 feet wide, crest elevation at 1133.2 feet MSL. Top of dam at 1135.5 feet MSL.
Hydraulic Design: Fehr Graham, West Union, Iowa

McMillian Pond #1: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0 | 1124.75 | 0 | 0 | principal spillway: 1124.75 |
| 0.25 | 1125.0 | 0.003 | 0.001 | |
| 1.25 | 1126.0 | 0.05 | 0.026 | |
| 2.25 | 1127.0 | 0.06 | 0.082 | |
| 4.25 | 1129.0 | 0.10 | 0.244 | |
| 6.25 | 1131.0 | 0.14 | 0.485 | |
| 8.45 | 1133.2 | 0.20 | 0.859 | auxiliary spillway: 1133.2 |
| 8.75 | 1133.5 | 0.20 | 0.919 | |
| 9.25 | 1134.0 | 0.21 | 1.023 | |
| 9.75 | 1134.5 | 0.23 | 1.133 | |
| 10.25 | 1135.0 | 0.24 | 1.250 | |
| 11.25 | 1136.0 | 0.26 | 1.504 | top of dam: 1135.5 |

McMillian Pond #1: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0 | 1124.75 | 0 | 0 | principal spillway: 1124.75 |
| 0.25 | 1125.0 | 0.001 | 0.93 | |
| 1.25 | 1126.0 | 0.026 | 2.40 | |
| 2.25 | 1127.0 | 0.082 | 3.55 | |
| 4.25 | 1129.0 | 0.244 | 5.14 | |
| 6.25 | 1131.0 | 0.485 | 6.34 | |
| 8.45 | 1133.2 | 0.859 | 7.44 | auxiliary spillway: 1133.2 |
| 8.75 | 1133.5 | 0.919 | 10.00 | |
| 9.25 | 1134.0 | 1.023 | 16.77 | |
| 9.75 | 1134.5 | 1.133 | 54.82 | |
| 10.25 | 1135.0 | 1.250 | 140.16 | |
| 11.25 | 1136.0 | 1.504 | 351.59 | top of dam: 1135.5 |

**Project:** **McMillian Pond #3 Pond ID #5**
Drainage Area: Not Specified in Design Documents
Description: The principal spillway is a 24" corrugated metal pipe (CMP), invert elevation of 1132.0 feet MSL. The auxiliary spillway is 17.5 feet wide, crest elevation at 1135.65 feet MSL. Top of dam at 1138.95 feet MSL.
Hydraulic Design: Fehr Graham, West Union, Iowa

McMillian Pond #3: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0 | 1132.0 | 0.74 | 0 | principal spillway: 1132.0 |
| 0.5 | 1132.5 | 0.80 | 0.38 | |
| 1.0 | 1133.0 | 0.87 | 0.80 | |
| 2.0 | 1134.0 | 0.97 | 1.72 | |
| 3.65 | 1135.65 | 1.15 | 3.47 | |
| 4.0 | 1136.0 | 1.19 | 3.88 | |
| 4.5 | 1136.5 | 1.23 | 4.48 | auxiliary spillway: 1136.65 |
| 5.0 | 1137.0 | 1.28 | 5.11 | |
| 5.5 | 1137.5 | 1.34 | 5.77 | |
| 6.0 | 1138.0 | 1.41 | 6.46 | |
| 6.95 | 1138.95 | 1.50 | 7.84 | |
| 7.65 | 1139.65 | 1.58 | 8.92 | top of dam: 1138.95 |

McMillian Pond #3: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0 | 1132.0 | 0 | 0 | principal spillway: 1132.0 |
| 0.5 | 1132.5 | 0.38 | 7.56 | |
| 1.0 | 1133.0 | 0.80 | 10.69 | |
| 2.0 | 1134.0 | 1.72 | 15.12 | |
| 3.65 | 1135.65 | 3.47 | 24.61 | |
| 4.0 | 1136.0 | 3.88 | 27.39 | |
| 4.5 | 1136.5 | 4.48 | 31.76 | auxiliary spillway: 1136.65 |
| 5.0 | 1137.0 | 5.11 | 48.41 | |
| 5.5 | 1137.5 | 5.77 | 88.92 | |
| 6.0 | 1138.0 | 6.46 | 143.62 | |
| 6.95 | 1138.95 | 7.84 | 273.93 | |
| 7.65 | 1139.65 | 8.92 | 393.67 | top of dam: 1138.95 |

**Project:** **McMillian Pond #4 Pond ID #6**
Drainage Area: Not Specified in Design Documents
Description: The principal spillway is a 24" corrugated metal pipe (CMP), invert elevation of 1117.0 feet MSL. The auxiliary spillway is 31.5 feet wide, crest elevation at 1123.5 feet MSL. Top of dam at 1127.0 feet MSL.
Hydraulic Design: Fehr Graham, West Union, Iowa

McMillian Pond #4: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0 | 1117.0 | 0.78 | 0 | principal spillway: 1117.0 |
| 0.5 | 1117.5 | 0.81 | 0.40 | |
| 1.0 | 1118.0 | 0.85 | 0.81 | |
| 2.0 | 1119.0 | 0.92 | 1.70 | |
| 3.0 | 1120.0 | 1.00 | 2.65 | |
| 4.0 | 1121.0 | 1.07 | 3.69 | |
| 5.0 | 1122.0 | 1.15 | 4.80 | |
| 6.0 | 1123.0 | 1.23 | 5.99 | |
| 6.5 | 1123.5 | 1.26 | 6.61 | auxiliary spillway: 1123.5 |
| 7.0 | 1124.0 | 1.28 | 7.24 | |
| 7.5 | 1124.5 | 1.31 | 7.89 | |
| 8.0 | 1125.0 | 1.34 | 8.55 | |
| 9.0 | 1126.0 | 1.51 | 9.98 | |
| 10.0 | 1127.0 | 1.60 | 11.53 | top of dam: 1127.0 |

McMillian Pond #4: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0 | 1117.0 | 0 | 0 | principal spillway: 1117.0 |
| 0.5 | 1117.5 | 0.40 | 7.56 | |
| 1.0 | 1118.0 | 0.81 | 10.69 | |
| 2.0 | 1119.0 | 1.70 | 15.12 | |
| 3.0 | 1120.0 | 2.65 | 21.38 | |
| 4.0 | 1121.0 | 3.69 | 26.19 | |
| 5.0 | 1122.0 | 4.80 | 30.24 | |
| 6.0 | 1123.0 | 5.99 | 33.81 | |
| 6.5 | 1123.5 | 6.61 | 35.46 | auxiliary spillway: 1123.5 |
| 7.0 | 1124.0 | 7.24 | 40.85 | |
| 7.5 | 1124.5 | 7.89 | 51.32 | |
| 8.0 | 1125.0 | 8.55 | 108.99 | |
| 9.0 | 1126.0 | 9.98 | 304.85 | |
| 10.0 | 1127.0 | 11.53 | 559.49 | top of dam: 1127.0 |

**Project:** **McMillian Pond #5 Pond ID #7**
Drainage Area: Not Specified in Design Documents
Description: The principal spillway is a 24” corrugated metal pipe (CMP), invert elevation of 1120.0 feet MSL. The auxiliary spillway is 21 feet wide, crest elevation at 1124.0 feet MSL. Top of dam at 1127.0 feet MSL.
Hydraulic Design: Fehr Graham, West Union, Iowa

McMillian Pond #5: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0 | 1120.0 | 0.64 | 0 | principal spillway: 1117.0 |
| 0.5 | 1120.5 | 0.67 | 0.33 | |
| 1.0 | 1121.0 | 0.70 | 0.67 | |
| 2.0 | 1122.0 | 0.77 | 1.40 | |
| 3.0 | 1123.0 | 0.84 | 2.21 | |
| 4.0 | 1124.0 | 0.92 | 3.08 | auxiliary spillway: 1124.0 |
| 4.5 | 1124.5 | 0.96 | 3.55 | |
| 5.0 | 1125.0 | 1.00 | 4.05 | |
| 6.0 | 1126.0 | 1.12 | 5.11 | |
| 6.5 | 1126.5 | 1.17 | 5.68 | |
| 7.0 | 1127.0 | 1.23 | 6.28 | top of dam: 1127.0 |

McMillian Pond #5: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0 | 1120.0 | 0 | 0 | principal spillway: 1117.0 |
| 0.5 | 1120.5 | 0.33 | 7.56 | |
| 1.0 | 1121.0 | 0.67 | 10.69 | |
| 2.0 | 1122.0 | 1.40 | 15.12 | |
| 3.0 | 1123.0 | 2.21 | 21.38 | |
| 4.0 | 1124.0 | 3.08 | 26.19 | auxiliary spillway: 1124.0 |
| 4.5 | 1124.5 | 3.55 | 30.74 | |
| 5.0 | 1125.0 | 4.05 | 37.76 | |
| 6.0 | 1126.0 | 5.11 | 132.48 | |
| 6.5 | 1126.5 | 5.68 | 210.07 | |
| 7.0 | 1127.0 | 6.28 | 287.58 | top of dam: 1127.0 |

**Project:** **Helgerson 410 Pond** **Pond ID #8**

Drainage Area: 13 acres (0.02 square miles)

Description: The principal spillway is a 6” plastic pipe (PVC), invert elevation of 1134.4 feet MSL. The auxiliary spillway is 10 feet wide, crest elevation at 1136.2 feet MSL. Top of dam at 1139.0 feet MSL.

Hydraulic Design: NRCS, Elkader, Iowa

Helgerson 410 Pond: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0 | 1134.4 | 0.34 | 0 | principal spillway: 1134.4 |
| 0.4 | 1134.8 | 0.37 | 0.14 | |
| 0.8 | 1135.2 | 0.41 | 0.30 | |
| 1.1 | 1135.5 | 0.44 | 0.43 | |
| 1.6 | 1136.0 | 0.48 | 0.65 | |
| 1.8 | 1136.2 | 0.50 | 0.75 | auxiliary spillway: 1136.2 |
| 1.9 | 1136.3 | 0.51 | 0.80 | |
| 2.6 | 1137.0 | 0.57 | 1.18 | |
| 3.1 | 1137.5 | 0.62 | 1.48 | |
| 3.6 | 1138.0 | 0.67 | 1.80 | |
| 4.1 | 1138.5 | 0.73 | 2.15 | |
| 4.6 | 1139.0 | 0.79 | 2.53 | top of dam: 1139.0 |
| 5.1 | 1139.5 | 0.85 | 2.94 | |
| 5.6 | 1140.0 | 0.91 | 3.38 | |

Helgerson 410 Pond: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0 | 1134.4 | 0 | 0 | principal spillway: 1134.4 |
| 0.4 | 1134.8 | 0.14 | 0.32 | |
| 0.8 | 1135.2 | 0.30 | 1.28 | |
| 1.1 | 1135.5 | 0.43 | 4.83 | |
| 1.6 | 1136.0 | 0.65 | 5.03 | |
| 1.8 | 1136.2 | 0.75 | 5.05 | auxiliary spillway: 1136.2 |
| 1.9 | 1136.3 | 0.80 | 5.06 | |
| 2.6 | 1137.0 | 1.18 | 5.57 | |
| 3.1 | 1137.5 | 1.48 | 14.15 | |
| 3.6 | 1138.0 | 1.80 | 49.14 | |
| 4.1 | 1138.5 | 2.15 | 98.27 | |
| 4.6 | 1139.0 | 2.53 | 152.29 | top of dam: 1139.0 |
| 5.1 | 1139.5 | 2.94 | 227.31 | |
| 5.6 | 1140.0 | 3.38 | 304.67 | |

**Project:** **Helgerson 638 Pond** **Pond ID #9**

Drainage Area: 13 acres (0.02 square miles)

Description: The principal spillway is a 6” plastic pipe (PVC), invert elevation of 1125.1 feet MSL. The auxiliary spillway is not specified in design documents, crest elevation at 1136.9 feet MSL. Top of dam at 1139.1 feet MSL.

Hydraulic Design: NRCS, Elkader, Iowa

Helgerson 638 Pond: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0 | 1125.1 | 0 | 0 | principal spillway: 1125.1 |
| 0.9 | 1126.0 | 0.003 | 0.001 | |
| 1.9 | 1127.0 | 0.01 | 0.01 | |
| 2.9 | 1128.0 | 0.03 | 0.03 | |
| 3.9 | 1129.0 | 0.05 | 0.07 | |
| 4.9 | 1130.0 | 0.09 | 0.15 | |
| 5.9 | 1131.0 | 0.13 | 0.26 | |
| 6.9 | 1132.0 | 0.18 | 0.41 | |
| 7.9 | 1133.0 | 0.24 | 0.62 | |
| 8.9 | 1134.0 | 0.31 | 0.90 | |
| 9.9 | 1135.0 | 0.39 | 1.24 | |
| 10.9 | 1136.0 | 0.48 | 1.68 | |
| 11.8 | 1136.9 | 0.56 | 2.14 | auxiliary spillway: 1136.9 |
| 12.3 | 1137.4 | 0.61 | 2.44 | |
| 12.8 | 1137.9 | 0.66 | 2.75 | |
| 13.3 | 1138.4 | 0.72 | 3.10 | |
| 13.8 | 1138.9 | 0.78 | 3.47 | |
| 14 | 1139.1 | 0.80 | 3.63 | top of dam: 1139.1 |

Helgerson 638 Pond: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0 | 1125.1 | 0 | 0 | principal spillway: 1125.1 |
| 0.9 | 1126.0 | 0.001 | 0.76 | |
| 1.9 | 1127.0 | 0.01 | 1.21 | |
| 2.9 | 1128.0 | 0.03 | 1.54 | |
| 3.9 | 1129.0 | 0.07 | 1.81 | |
| 4.9 | 1130.0 | 0.15 | 2.04 | |
| 5.9 | 1131.0 | 0.26 | 2.25 | |
| 6.9 | 1132.0 | 0.41 | 2.44 | |
| 7.9 | 1133.0 | 0.62 | 2.61 | |
| 8.9 | 1134.0 | 0.90 | 2.78 | |
| 9.9 | 1135.0 | 1.24 | 2.94 | |
| 10.9 | 1136.0 | 1.68 | 3.08 | |
| 11.8 | 1136.9 | 2.14 | 3.21 | auxiliary spillway: 1136.9 |
| 12.3 | 1137.4 | 2.44 | 5.05 | |
| 12.8 | 1137.9 | 2.75 | 8.33 | |
| 13.3 | 1138.4 | 3.10 | 33.73 | continued on next page |

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 13.8 | 1138.9 | 3.47 | 72.94 | |
| 14 | 1139.1 | 3.63 | 96.64 | top of dam: 1139.1 |

**Project:** **Howard 638 #2 Pond** **Pond ID #10**
Drainage Area: 2.8 acres (0.004 square miles)
Description: The principal spillway is a 4" plastic pipe (PVC), invert elevation of 1156.5 feet MSL. No auxiliary spillway is specified in design documents. Top of dam at 1163.0 feet MSL.
Hydraulic Design: NRCS, Elkader, Iowa

Howard 638 #2 Pond: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0.0 | 1156.5 | 0 | 0 | principal spillway: 1156.5 |
| 1.0 | 1157.5 | 0.047 | 0.02 | |
| 2.0 | 1158.5 | 0.09 | 0.09 | |
| 3.0 | 1159.5 | 0.14 | 0.21 | |
| 4.0 | 1160.5 | 0.19 | 0.38 | |
| 5.0 | 1161.5 | 0.24 | 0.59 | |
| 6.0 | 1162.5 | 0.28 | 0.85 | |
| 6.5 | 1163.0 | 0.41 | 1.03 | top of dam: 1163.0 |

Howard 638 #2 Pond: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0.0 | 1156.5 | 0 | 0 | principal spillway: 1156.5 |
| 1.0 | 1157.5 | 0.02 | 0.33 | |
| 2.0 | 1158.5 | 0.09 | 0.34 | |
| 3.0 | 1159.5 | 0.21 | 0.34 | |
| 4.0 | 1160.5 | 0.38 | 0.35 | |
| 5.0 | 1161.5 | 0.59 | 0.35 | |
| 6.0 | 1162.5 | 0.85 | 0.36 | |
| 6.5 | 1163.0 | 1.03 | 0.36 | top of dam: 1163.0 |

**Project:** **Howard 638 #1 Pond** **Pond ID #11**

Drainage Area: 18 acres (0.03 square miles)

Description: The principal spillway is an 8” plastic pipe (PVC), invert elevation of 1116.0 feet MSL. The auxiliary spillway is 6 feet wide, crest elevation at 1124.7 feet MSL. Top of dam at 1125.7 feet MSL.

Hydraulic Design: NRCS, Elkader, Iowa

Howard 638 #1 Pond: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0 | 1116.0 | 0 | 0 | principal spillway: 1116.0 |
| 0.6 | 1116.6 | 0.10 | 0.04 | |
| 1.6 | 1117.6 | 0.22 | 0.22 | |
| 2.6 | 1118.6 | 0.33 | 0.50 | |
| 3.6 | 1119.6 | 0.44 | 0.88 | |
| 4.6 | 1120.6 | 0.55 | 1.38 | |
| 5.6 | 1121.6 | 0.66 | 1.98 | |
| 6.6 | 1122.6 | 0.77 | 2.70 | |
| 7.6 | 1123.6 | 0.88 | 3.53 | |
| 8.7 | 1124.7 | 0.99 | 4.46 | auxiliary spillway: 1124.7 |
| 9.1 | 1125.1 | 1.11 | 5.62 | |
| 9.6 | 1125.6 | 1.16 | 6.08 | top of dam: 1125.7 |
| 10.1 | 1126.1 | 1.21 | 6.67 | |
| 10.6 | 1126.6 | 1.27 | 7.29 | |

Howard 638 #1 Pond: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0 | 1116 | 0 | 0 | principal spillway: 1116.0 |
| 0.6 | 1116.6 | 0.04 | 1.14 | |
| 1.6 | 1117.6 | 0.22 | 2.34 | |
| 2.6 | 1118.6 | 0.50 | 3.03 | |
| 3.6 | 1119.6 | 0.88 | 3.59 | |
| 4.6 | 1120.6 | 1.38 | 4.07 | |
| 5.6 | 1121.6 | 1.98 | 4.50 | |
| 6.6 | 1122.6 | 2.70 | 4.90 | |
| 7.6 | 1123.6 | 3.53 | 5.26 | |
| 8.7 | 1124.7 | 4.46 | 5.60 | auxiliary spillway: 1124.7 |
| 9.1 | 1125.1 | 5.62 | 5.95 | |
| 9.6 | 1125.6 | 6.08 | 7.45 | top of dam: 1125.7 |
| 10.1 | 1126.1 | 6.67 | 10.29 | |
| 10.6 | 1126.6 | 7.29 | 28.86 | |

**Project:** **Bennett #1 Pond** **Pond ID #12**
Drainage Area: 37 acres (0.06 square miles)
Description: The principal spillway is an 8” plastic pipe (PVC), invert elevation of 1085.5 feet MSL. The auxiliary spillway is 10 feet wide, crest elevation at 1087.5 feet MSL. Top of dam at 1090.0 feet MSL.
Hydraulic Design: NRCS, Elkader, Iowa

Bennett #1 Pond: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0 | 1085.5 | 1.25 | 0 | principal spillway: 1085.5 |
| 0.5 | 1086.0 | 1.34 | 0.65 | |
| 1.0 | 1086.5 | 1.43 | 1.34 | |
| 1.5 | 1087.0 | 1.51 | 2.07 | |
| 2.0 | 1087.5 | 1.60 | 2.85 | auxiliary spillway: 1087.5 |
| 2.5 | 1088.0 | 1.68 | 3.67 | |
| 3.0 | 1088.5 | 1.78 | 4.54 | |
| 3.5 | 1089.0 | 1.89 | 5.45 | |
| 4.0 | 1089.5 | 2.00 | 6.43 | |
| 4.5 | 1090.0 | 2.11 | 7.45 | top of dam: 1090.0 |
| 5.0 | 1090.5 | 2.22 | 8.54 | |
| 5.5 | 1091.0 | 2.34 | 9.68 | |

Bennett #1 Pond: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0 | 1085.5 | 0 | 0 | principal spillway: 1085.5 |
| 0.5 | 1086.0 | 0.65 | 0.50 | |
| 1.0 | 1086.5 | 1.34 | 4.59 | |
| 1.5 | 1087.0 | 2.07 | 5.06 | |
| 2.0 | 1087.5 | 2.85 | 5.11 | auxiliary spillway: 1087.5 |
| 2.5 | 1088.0 | 3.67 | 5.20 | |
| 3.0 | 1088.5 | 4.54 | 6.91 | |
| 3.5 | 1089.0 | 5.45 | 24.28 | |
| 4.0 | 1089.5 | 6.43 | 66.89 | |
| 4.5 | 1090.0 | 7.45 | 116.72 | top of dam: 1090.0 |
| 5.0 | 1090.5 | 8.54 | 181.74 | |
| 5.5 | 1091.0 | 9.68 | 251.55 | |

**Project:** **Bennett #2 Pond** **Pond ID #13**
Drainage Area: 76 acres (0.12 square miles)
Description: The principal spillway is a 12" plastic pipe (PVC), invert elevation of 1089.9 feet MSL. The auxiliary spillway is 14 feet wide, crest elevation at 1095.4 feet MSL. Top of dam at 1098.3 feet MSL.
Hydraulic Design: NRCS, Elkader, Iowa

Bennett #2 Pond: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0.0 | 1089.9 | 0.68 | 0 | principal spillway: 1089.9 |
| 0.1 | 1090.0 | 0.69 | 0.07 | |
| 0.6 | 1090.5 | 0.74 | 0.43 | |
| 1.1 | 1091.0 | 0.78 | 0.80 | |
| 2.1 | 1092.0 | 0.88 | 1.63 | |
| 3.1 | 1093.0 | 0.98 | 2.56 | |
| 4.1 | 1094.0 | 1.10 | 3.60 | |
| 5.1 | 1095.0 | 1.22 | 4.76 | |
| 5.5 | 1095.4 | 1.30 | 5.27 | auxiliary spillway: 1095.4 |
| 6.1 | 1096.0 | 1.42 | 6.08 | |
| 7.1 | 1097.0 | 1.64 | 7.61 | |
| 8.0 | 1097.9 | 1.86 | 9.19 | |
| 8.1 | 1098.0 | 1.89 | 9.38 | top of dam: 1098.3 |
| 9.1 | 1099.0 | 2.11 | 11.38 | |
| 10.1 | 1100.0 | 2.34 | 13.60 | |
| 11.1 | 1101.0 | 2.59 | 16.07 | |
| 12.1 | 1102.0 | 2.89 | 18.81 | |
| 13.1 | 1103.0 | 3.18 | 21.84 | |
| 14.1 | 1104.0 | 3.47 | 25.17 | |
| 15.1 | 1105.0 | 3.61 | 28.71 | |
| 16.1 | 1106.0 | 3.71 | 32.37 | |

Bennett #2 Pond: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0.0 | 1089.9 | 0 | 0 | principal spillway: 1089.9 |
| 0.1 | 1090.0 | 0.07 | 0.08 | |
| 0.6 | 1090.5 | 0.43 | 0.90 | |
| 1.1 | 1091.0 | 0.80 | 2.96 | |
| 2.1 | 1092.0 | 1.63 | 11.88 | |
| 3.1 | 1093.0 | 2.56 | 12.20 | |
| 4.1 | 1094.0 | 3.60 | 12.51 | |
| 5.1 | 1095.0 | 4.76 | 12.82 | |
| 5.5 | 1095.4 | 5.27 | 12.93 | auxiliary spillway: 1095.4 |
| 6.1 | 1096.0 | 6.08 | 13.21 | |
| 7.1 | 1097.0 | 7.61 | 45.65 | |
| 8.0 | 1097.9 | 9.19 | 165.42 | continued on next page |

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 8.1 | 1098.0 | 9.38 | 180.03 | top of dam: 1098.3 |
| 9.1 | 1099.0 | 11.38 | 375.40 | |
| 10.1 | 1100.0 | 13.60 | 638.95 | |
| 11.1 | 1101.0 | 16.07 | 945.74 | |
| 12.1 | 1102.0 | 18.81 | 1353.45 | 12.1 |
| 13.1 | 1103.0 | 21.84 | 1761.16 | 13.1 |
| 14.1 | 1104.0 | 25.17 | 2297.68 | 14.1 |
| 15.1 | 1105.0 | 28.71 | 2903.55 | 15.1 |
| 16.1 | 1106.0 | 32.37 | 3509.42 | 16.1 |

**Project:** **Woltz Pond Pond ID #14**
Drainage Area: 18 acres (0.03 square miles)
Description: The principal spillway is an 8” plastic pipe (PVC), invert elevation of 1080.0 feet MSL. The auxiliary spillway is 10 feet wide, crest elevation at 1081.0 feet MSL. Top of dam at 1083.0 feet MSL.
Hydraulic Design: NRCS, Elkader, Iowa

Woltz Pond: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0 | 1080.0 | 0.30 | 0 | principal spillway: 1080.0 |
| 0.2 | 1080.2 | 0.31 | 0.06 | |
| 0.4 | 1080.4 | 0.33 | 0.13 | |
| 0.6 | 1080.6 | 0.34 | 0.19 | |
| 0.8 | 1080.8 | 0.36 | 0.26 | |
| 1.0 | 1081.0 | 0.37 | 0.34 | auxiliary spillway: 1081.0 |
| 1.5 | 1081.5 | 0.41 | 0.53 | |
| 2.0 | 1082.0 | 0.45 | 0.74 | |
| 2.5 | 1082.5 | 0.49 | 0.98 | |
| 3.0 | 1083.0 | 0.52 | 1.23 | top of dam: 1083.0 |
| 3.5 | 1083.5 | 0.57 | 1.50 | |
| 4.0 | 1084.0 | 0.61 | 1.80 | |
| 5.0 | 1085.0 | 0.71 | 2.46 | |
| 6.0 | 1086.0 | 0.81 | 3.22 | |
| 7.0 | 1087.0 | 0.92 | 4.08 | |

Woltz Pond: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0 | 1080.0 | 0 | 0 | principal spillway: 1080.0 |
| 0.2 | 1080.2 | 0.06 | 0.11 | |
| 0.4 | 1080.4 | 0.13 | 0.34 | |
| 0.6 | 1080.6 | 0.19 | 0.68 | |
| 0.8 | 1080.8 | 0.26 | 2.15 | |
| 1.0 | 1081.0 | 0.34 | 4.59 | auxiliary spillway: 1081.0 |
| 1.5 | 1081.5 | 0.53 | 4.70 | |
| 2.0 | 1082.0 | 0.74 | 6.40 | |
| 2.5 | 1082.5 | 0.98 | 27.72 | |
| 3.0 | 1083.0 | 1.23 | 66.26 | top of dam: 1083.0 |
| 3.5 | 1083.5 | 1.50 | 125.30 | |
| 4.0 | 1084.0 | 1.80 | 184.34 | |
| 5.0 | 1085.0 | 2.46 | 364.60 | |
| 6.0 | 1086.0 | 3.22 | 589.14 | |
| 7.0 | 1087.0 | 4.08 | 857.96 | |

**Project:** **Bennett #3 638 Pond** **Pond ID #15**
Drainage Area: 26 acres (0.04 square miles)
Description: The principal spillway is an 8” plastic pipe (PVC), invert elevation of 1107.8 feet MSL. The auxiliary spillway is 8 feet wide, crest elevation at 1118.0 feet MSL. Top of dam at 1119.0 feet MSL.
Hydraulic Design: NRCS, Elkader, Iowa

Bennett #3 638 Pond: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0 | 1107.8 | 0 | 0 | principal spillway: 1107.8 |
| 1.0 | 1108.8 | 0.08 | 0.04 | |
| 2.0 | 1109.8 | 0.16 | 0.16 | |
| 3.0 | 1110.8 | 0.24 | 0.35 | |
| 4.0 | 1111.8 | 0.31 | 0.63 | |
| 5.0 | 1112.8 | 0.39 | 0.98 | |
| 6.0 | 1113.8 | 0.47 | 1.41 | |
| 7.0 | 1114.8 | 0.55 | 1.92 | |
| 8.0 | 1115.8 | 0.63 | 2.51 | |
| 9.0 | 1116.8 | 0.71 | 3.18 | |
| 10.2 | 1118.0 | 0.80 | 4.08 | auxiliary spillway: 1118.0 |
| 10.7 | 1118.5 | 0.84 | 4.49 | |
| 11.2 | 1119.0 | 0.88 | 4.92 | top of dam: 1119.0 |
| 11.7 | 1119.5 | 0.92 | 5.37 | |
| 12.3 | 1120.1 | 0.96 | 5.93 | |

Bennett #3 638 Pond: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0 | 1107.8 | 0 | 0 | principal spillway: 1107.8 |
| 1.0 | 1108.8 | 0.04 | 1.26 | |
| 2.0 | 1109.8 | 0.16 | 1.99 | |
| 3.0 | 1110.8 | 0.35 | 2.51 | |
| 4.0 | 1111.8 | 0.63 | 2.95 | |
| 5.0 | 1112.8 | 0.98 | 3.33 | |
| 6.0 | 1113.8 | 1.41 | 3.67 | |
| 7.0 | 1114.8 | 1.92 | 3.98 | |
| 8.0 | 1115.8 | 2.51 | 4.26 | |
| 9.0 | 1116.8 | 3.18 | 4.53 | |
| 10.2 | 1118.0 | 4.08 | 4.84 | auxiliary spillway: 1118.0 |
| 10.7 | 1118.5 | 4.49 | 6.72 | |
| 11.2 | 1119.0 | 4.92 | 10.06 | top of dam: 1119.0 |
| 11.7 | 1119.5 | 5.37 | 35.51 | |
| 12.3 | 1120.1 | 5.93 | 86.62 | |

**Project:** **Bennett #4 Pond** **Pond ID #16**
Drainage Area: 26 acres (0.04 square miles)
Description: The principal spillway is an 18" corrugated metal pipe (CMP), invert elevation of 1110.0 feet MSL. The auxiliary spillway is 12 feet wide, crest elevation at 1115.1 feet MSL. Top of dam at 1117.6 feet MSL.
Hydraulic Design: NRCS, Elkader, Iowa

Bennett #4 Pond: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0 | 1110.0 | 0.26 | 0 | principal spillway: 1110.0 |
| 1.0 | 1111.0 | 0.52 | 0.39 | |
| 2.0 | 1112.0 | 0.76 | 1.03 | |
| 3.0 | 1113.0 | 1.00 | 1.91 | |
| 4.0 | 1114.0 | 1.28 | 3.05 | |
| 4.5 | 1114.5 | 1.38 | 3.72 | |
| 5.0 | 1115.0 | 1.48 | 4.43 | |
| 5.1 | 1115.1 | 1.51 | 4.58 | auxiliary spillway: 1115.1 |
| 5.5 | 1115.5 | 1.61 | 5.20 | |
| 6.0 | 1116.0 | 1.73 | 6.04 | |
| 6.5 | 1116.5 | 1.88 | 6.94 | |
| 7.0 | 1117.0 | 2.02 | 7.91 | |
| 7.5 | 1117.5 | 2.17 | 8.96 | top of dam: 1117.6 |
| 8.0 | 1118.0 | 2.32 | 10.08 | |
| 8.9 | 1118.9 | 2.64 | 12.32 | |
| 9.0 | 1119.0 | 2.68 | 12.58 | |
| 10.0 | 1120.0 | 3.06 | 15.45 | |
| 11.0 | 1121.0 | 3.42 | 18.69 | |
| 12.0 | 1122.0 | 3.83 | 22.32 | |
| 13.0 | 1123.0 | 4.26 | 26.36 | |
| 14.0 | 1124.0 | 4.67 | 30.83 | |
| 15.0 | 1125.0 | 5.07 | 35.70 | |
| 16.0 | 1126.0 | 5.48 | 40.97 | |

Bennett #4 Pond: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0 | 1110.0 | 0 | 0 | principal spillway: 1110.0 |
| 1.0 | 1111.0 | 0.39 | 11.22 | |
| 2.0 | 1112.0 | 1.03 | 12.34 | |
| 3.0 | 1113.0 | 1.91 | 13.37 | |
| 4.0 | 1114.0 | 3.05 | 14.32 | |
| 4.5 | 1114.5 | 3.72 | 14.77 | |
| 5.0 | 1115.0 | 4.43 | 15.21 | |
| 5.1 | 1115.1 | 4.58 | 15.30 | auxiliary spillway: 1115.1 |
| 5.5 | 1115.5 | 5.20 | 15.66 | |
| 6.0 | 1116.0 | 6.04 | 17.10 | |
| 6.5 | 1116.5 | 6.94 | 32.86 | continued on next page |

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 7.0 | 1117.0 | 7.91 | 70.25 | |
| 7.5 | 1117.5 | 8.96 | 136.39 | top of dam: 1117.6 |
| 8.0 | 1118.0 | 10.08 | 204.84 | |
| 9.0 | 1119.0 | 12.58 | 415.68 | |
| 10.0 | 1120.0 | 15.45 | 669.94 | |
| 11.0 | 1121.0 | 18.69 | 989.58 | |
| 12.0 | 1122.0 | 22.32 | 1389.11 | |
| 13.0 | 1123.0 | 26.36 | 1788.64 | |
| 14.0 | 1124.0 | 30.83 | 2335.28 | |
| 15.0 | 1125.0 | 35.70 | 2939.13 | |
| 16.0 | 1126.0 | 40.97 | 3542.97 | |

**Project:** **Bennett #5 Pond** **Pond ID #17**

Drainage Area: 52 acres (0.08 square miles)

Description: The principal spillway is a 10" plastic pipe (PVC), invert elevation of 1098.8 feet MSL. The auxiliary spillway is 12 feet wide, crest elevation at 1104.3 feet MSL. Top of dam at 1107.0 feet MSL.

Hydraulic Design: NRCS, Elkader, Iowa

Bennett #5 Pond: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0 | 1098.8 | 0.59 | 0 | principal spillway: 1098.8 |
| 0.2 | 1099.0 | 0.61 | 0.12 | |
| 0.7 | 1099.5 | 0.66 | 0.44 | |
| 1.2 | 1100.0 | 0.71 | 0.78 | |
| 1.7 | 1100.5 | 0.76 | 1.15 | |
| 2.2 | 1101.0 | 0.81 | 1.54 | |
| 3.2 | 1102.0 | 0.92 | 2.41 | |
| 4.2 | 1103.0 | 1.03 | 3.38 | |
| 5.2 | 1104.0 | 1.15 | 4.47 | |
| 5.5 | 1104.3 | 1.19 | 4.82 | auxiliary spillway: 1104.3 |
| 6.2 | 1105.0 | 1.28 | 5.69 | |
| 7.2 | 1106.0 | 1.41 | 7.03 | |
| 8.2 | 1107.0 | 1.57 | 8.52 | top of dam: 1107.0 |
| 8.7 | 1107.5 | 1.65 | 9.33 | |
| 9.2 | 1108.0 | 1.73 | 10.17 | |
| 10.2 | 1109.0 | 1.92 | 12.00 | |
| 11.2 | 1110.0 | 2.13 | 14.02 | |
| 12.2 | 1111.0 | 2.34 | 16.26 | |
| 13.2 | 1112.0 | 2.64 | 18.75 | |
| 14.2 | 1113.0 | 2.90 | 21.52 | |
| 15.2 | 1114.0 | 3.20 | 24.57 | |
| 16.2 | 1115.0 | 3.52 | 27.93 | |
| 17.2 | 1116.0 | 3.87 | 31.62 | |
| 18.2 | 1117.0 | 3.71 | 35.41 | |

Bennett #5 Pond: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0 | 1098.8 | 0 | 0 | principal spillway: 1098.8 |
| 0.2 | 1099.0 | 0.12 | 0.14 | |
| 0.7 | 1099.5 | 0.44 | 1.03 | |
| 1.2 | 1100.0 | 0.78 | 7.57 | |
| 1.7 | 1100.5 | 1.15 | 7.72 | |
| 2.2 | 1101.0 | 1.54 | 7.83 | |
| 3.2 | 1102.0 | 2.41 | 8.04 | |
| 4.2 | 1103.0 | 3.38 | 8.25 | |
| 5.2 | 1104.0 | 4.47 | 8.46 | continued on next page |

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 6.2 | 1105.0 | 5.69 | 8.86 | |
| 7.2 | 1106.0 | 7.03 | 47.77 | top of dam: 1107.0 |
| 8.2 | 1107.0 | 8.52 | 169.71 | |
| 8.7 | 1107.5 | 9.33 | 249.67 | |
| 9.2 | 1108.0 | 10.17 | 329.63 | |
| 10.2 | 1109.0 | 12.00 | 618.40 | |
| 11.2 | 1110.0 | 14.02 | 923.10 | |
| 12.2 | 1111.0 | 16.26 | 1290.63 | |
| 13.2 | 1112.0 | 18.75 | 1774.87 | |
| 14.2 | 1113.0 | 21.52 | 2259.11 | |
| 15.2 | 1114.0 | 24.57 | 2788.93 | |
| 16.2 | 1115.0 | 27.93 | 3526.42 | |
| 17.2 | 1116.0 | 31.62 | 4263.92 | |
| 18.2 | 1117.0 | 35.41 | 5001.41 | |

**Project:** **Medberry 638 Pond** **Pond ID #18**
Drainage Area: 10 acres (0.015 square miles)
Description: The principal spillway is a 6” plastic pipe (PVC), invert elevation of 918.7 feet MSL. There is no auxiliary spillway. Top of dam at 927.6 feet MSL.
Hydraulic Design: NRCS, Elkader, Iowa

Medberry 638 Pond: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0 | 918.7 | 0 | 0 | principal spillway: 918.7 |
| 0.3 | 919.0 | 0.0002 | 0.00002 | |
| 0.9 | 919.6 | 0.003 | 0.001 | |
| 1.3 | 920.0 | 0.005 | 0.003 | |
| 2.3 | 921.0 | 0.03 | 0.02 | |
| 3.3 | 922.0 | 0.08 | 0.08 | |
| 4.3 | 923.0 | 0.13 | 0.18 | |
| 5.3 | 924.0 | 0.20 | 0.35 | |
| 6.3 | 925.0 | 0.28 | 0.59 | |
| 7.3 | 926.0 | 0.38 | 0.92 | |
| 8.3 | 927.0 | 0.49 | 1.36 | |
| 8.5 | 927.2 | 0.54 | 1.46 | |
| 8.9 | 927.6 | 0.64 | 1.70 | top of dam: 927.6 |
| 9.9 | 928.6 | 0.80 | 2.42 | |
| 12.3 | 931.0 | 1.02 | 4.61 | |
| 12.8 | 931.5 | 1.08 | 5.14 | |
| 13.3 | 932.0 | 1.15 | 5.70 | |
| 13.8 | 932.5 | 1.20 | 6.28 | |
| 14 | 932.7 | 1.22 | 6.53 | |

Medberry 638 Pond: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0 | 918.7 | 0 | 0 | principal spillway: 918.7 |
| 0.3 | 919.0 | 0.00002 | 0.14 | |
| 0.9 | 919.6 | 0.001 | 0.30 | |
| 1.3 | 920.0 | 0.003 | 0.53 | |
| 2.3 | 921.0 | 0.02 | 0.91 | |
| 3.3 | 922.0 | 0.08 | 1.16 | |
| 4.3 | 923.0 | 0.18 | 1.36 | |
| 5.3 | 924.0 | 0.35 | 1.54 | |
| 6.3 | 925.0 | 0.59 | 1.70 | |
| 7.3 | 926.0 | 0.92 | 1.84 | |
| 8.3 | 927.0 | 1.36 | 1.98 | |
| 8.5 | 927.2 | 1.46 | 2.00 | |
| 8.9 | 927.6 | 1.70 | 2.05 | top of dam: 927.6 |
| 9.9 | 928.6 | 2.42 | 2.17 | |
| 12.3 | 931.0 | 4.61 | 2.44 | |
| 12.8 | 931.5 | 5.14 | 2.49 | |
| 13.3 | 932.0 | 5.70 | 2.54 | continued on next page |

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) |
|---|---|---|---|
| 13.8 | 932.5 | 6.28 | 2.59 |
| 14 | 932.7 | 6.53 | 2.61 |

**Project:** **Frieden Pond** **Pond ID #19**

Drainage Area: 11 acres (0.017 square miles)

Description: The principal spillway is an 8” corrugated metal pipe (CMP), invert elevation of 863.0 feet MSL. The auxiliary spillway is 40 feet wide, crest elevation at 871.0 feet MSL. Top of dam at 872.0 feet MSL.

Hydraulic Design: Fehr Graham, West Union, Iowa

Frieden Pond: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0 | 863.0 | 0 | 0 | principal spillway: 863.0 |
| 1.0 | 864.0 | 0.05 | 0.02 | |
| 2.0 | 865.0 | 0.07 | 0.08 | |
| 3.0 | 866.0 | 0.10 | 0.17 | |
| 4.0 | 867.0 | 0.12 | 0.28 | |
| 5.0 | 868.0 | 0.15 | 0.41 | |
| 6.0 | 869.0 | 0.18 | 0.58 | |
| 7.0 | 870.0 | 0.20 | 0.77 | |
| 8.0 | 871.0 | 0.24 | 0.99 | auxiliary spillway: 871.0 |
| 8.5 | 871.5 | 0.25 | 1.11 | |
| 9.0 | 872.0 | 0.26 | 1.24 | top of dam: 872.0 |
| 9.5 | 872.5 | 0.27 | 1.37 | |
| 10.0 | 873.0 | 0.28 | 1.51 | |

Frieden Pond: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0 | 863.0 | 0 | 0 | principal spillway: 863.0 |
| 1.0 | 864.0 | 0.02 | 1.24 | |
| 2.0 | 865.0 | 0.08 | 1.73 | |
| 3.0 | 866.0 | 0.17 | 2.10 | |
| 4.0 | 867.0 | 0.28 | 2.42 | |
| 5.0 | 868.0 | 0.41 | 2.70 | |
| 6.0 | 869.0 | 0.58 | 2.95 | |
| 7.0 | 870.0 | 0.77 | 3.18 | |
| 8.0 | 871.0 | 0.99 | 3.40 | auxiliary spillway: 871.0 |
| 8.5 | 871.5 | 1.11 | 9.71 | |
| 9.0 | 872.0 | 1.24 | 33.09 | top of dam: 872.0 |
| 9.5 | 872.5 | 1.37 | 125.91 | |
| 10.0 | 873.0 | 1.51 | 253.43 | |

**Project:** **Helms #1 Pond** **Pond ID #20**
Drainage Area: 6 acres (0.01 square miles)
Description: The principal spillway is an 8” corrugated metal pipe (CMP), invert elevation of 932.0 feet MSL. The auxiliary spillway is 40 feet wide, crest elevation at 936.0 feet MSL. Top of dam at 937.0 feet MSL.
Hydraulic Design: Fehr Graham, West Union, Iowa

Helms #1 Pond: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0 | 932.0 | 0 | 0 | principal spillway: 932.0 |
| 1.0 | 933.0 | 0.04 | 0.02 | |
| 2.0 | 934.0 | 0.10 | 0.09 | |
| 3.0 | 935.0 | 0.13 | 0.20 | |
| 4.0 | 936.0 | 0.17 | 0.35 | auxiliary spillway: 936.0 |
| 4.3 | 936.3 | 0.18 | 0.40 | |
| 4.5 | 936.5 | 0.18 | 0.44 | |
| 4.8 | 936.8 | 0.19 | 0.49 | |
| 5.0 | 937.0 | 0.20 | 0.54 | top of dam: 937.0 |
| 5.5 | 937.5 | 0.22 | 0.64 | |
| 6.0 | 938.0 | 0.23 | 0.76 | |
| 6.5 | 938.5 | 0.25 | 0.88 | |
| 7.0 | 939.0 | 0.27 | 1.01 | |

Helms #1 Pond: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0 | 932.0 | 0 | 0 | principal spillway: 932.0 |
| 1.0 | 933.0 | 0.02 | 1.37 | |
| 2.0 | 934.0 | 0.09 | 1.96 | |
| 3.0 | 935.0 | 0.20 | 2.23 | |
| 4.0 | 936.0 | 0.35 | 2.48 | auxiliary spillway: 936.0 |
| 4.3 | 936.3 | 0.40 | 5.51 | |
| 4.5 | 936.5 | 0.44 | 8.80 | |
| 4.8 | 936.8 | 0.49 | 12.47 | |
| 5.0 | 937.0 | 0.54 | 32.19 | top of dam: 937.0 |
| 5.5 | 937.5 | 0.64 | 125.01 | |
| 6.0 | 938.0 | 0.76 | 252.54 | |
| 6.5 | 938.5 | 0.88 | 414.08 | |
| 7.0 | 939.0 | 1.01 | 575.63 | |

**Project:** **Helms #5 Pond** **Pond ID #21**

Drainage Area: 91 acres (0.14 square miles)

Description: The principal spillway is a 12" corrugated metal pipe (CMP), invert elevation of 859.0 feet MSL. The auxiliary spillway is 50 feet wide, crest elevation at 863.0 feet MSL. Top of dam at 864.0 feet MSL.

Hydraulic Design: Fehr Graham, West Union, Iowa

Helms #5 Pond: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0 | 859.0 | 0.73 | 0 | principal spillway: 859.0 |
| 1.0 | 860.0 | 0.85 | 0.79 | |
| 2.0 | 861.0 | 0.99 | 1.71 | |
| 3.0 | 862.0 | 1.14 | 2.78 | |
| 4.0 | 863.0 | 0.24 | 3.46 | auxiliary spillway: 863.0 |
| 4.3 | 863.3 | 0.28 | 3.53 | |
| 4.5 | 863.5 | 0.32 | 3.60 | |
| 4.8 | 863.8 | 0.36 | 3.68 | |
| 5.0 | 864.0 | 0.39 | 3.78 | top of dam: 864.0 |
| 5.5 | 864.5 | 0.47 | 4.00 | |
| 6.0 | 865.0 | 0.55 | 4.25 | |
| 6.5 | 865.5 | 0.63 | 4.55 | |
| 7.0 | 866.0 | 0.71 | 4.88 | |

Helms #5 Pond: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0 | 859.0 | 0 | 0 | principal spillway: 859.0 |
| 1.0 | 860.0 | 0.79 | 5.36 | |
| 2.0 | 861.0 | 1.71 | 46.59 | |
| 3.0 | 862.0 | 2.78 | 34.39 | |
| 4.0 | 863.0 | 3.46 | 52.53 | auxiliary spillway: 863.0 |
| 4.3 | 863.3 | 3.53 | 59.60 | |
| 4.5 | 863.5 | 3.60 | 66.53 | |
| 4.8 | 863.8 | 3.68 | 73.41 | |
| 5.0 | 864.0 | 3.78 | 92.71 | top of dam: 864.0 |
| 5.5 | 864.5 | 4.00 | 204.75 | |
| 6.0 | 865.0 | 4.25 | 364.89 | |
| 6.5 | 865.5 | 4.55 | 562.54 | |
| 7.0 | 866.0 | 4.88 | 759.91 | |

**Project:** **Helms #3 Pond** **Pond ID #22**
Drainage Area: 4.5 acres (0.007 square miles)
Description: The principal spillway is an 8" corrugated metal pipe (CMP), invert elevation of 901.0 feet MSL. The auxiliary spillway is 25 feet wide, crest elevation at 907.0 feet MSL. Top of dam at 908.0 feet MSL.
Hydraulic Design: Fehr Graham, West Union, Iowa

Helms #3 Pond: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0 | 901.0 | 0 | 0 | principal spillway: 901.0 |
| 1.0 | 902.0 | 0.02 | 0.01 | |
| 2.0 | 903.0 | 0.03 | 0.03 | |
| 3.0 | 904.0 | 0.05 | 0.07 | |
| 4.0 | 905.0 | 0.06 | 0.13 | |
| 5.0 | 906.0 | 0.08 | 0.20 | |
| 6.0 | 907.0 | 0.11 | 0.29 | auxiliary spillway: 907.0 |
| 6.5 | 907.5 | 0.12 | 0.35 | |
| 7.0 | 908.0 | 0.13 | 0.41 | top of dam: 908.0 |
| 7.5 | 908.5 | 0.14 | 0.48 | |
| 8.0 | 909.0 | 0.16 | 0.56 | |
| 8.5 | 909.5 | 0.17 | 0.64 | |
| 9.0 | 910.0 | 0.18 | 0.72 | |

Helms #3 Pond: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0 | 901.0 | 0 | 0 | principal spillway: 901.0 |
| 1.0 | 902.0 | 0.01 | 1.24 | |
| 2.0 | 903.0 | 0.03 | 1.73 | |
| 3.0 | 904.0 | 0.07 | 2.10 | |
| 4.0 | 905.0 | 0.13 | 2.42 | |
| 5.0 | 906.0 | 0.20 | 2.70 | |
| 6.0 | 907.0 | 0.29 | 2.95 | auxiliary spillway: 907.0 |
| 6.5 | 907.5 | 0.35 | 7.03 | |
| 7.0 | 908.0 | 0.41 | 19.17 | top of dam: 908.0 |
| 7.5 | 908.5 | 0.48 | 77.83 | |
| 8.0 | 909.0 | 0.56 | 162.12 | |
| 8.5 | 909.5 | 0.64 | 272.30 | |
| 9.0 | 910.0 | 0.72 | 382.48 | |

**Project:** **Helms #2 Pond** **Pond ID #23**
Drainage Area: 7.5 acres (0.01 square miles)
Description: The principal spillway is an 8" corrugated metal pipe (CMP), invert elevation of 901.0 feet MSL. The auxiliary spillway is 40 feet wide, crest elevation at 909.0 feet MSL. Top of dam at 910.0 feet MSL.
Hydraulic Design: Fehr Graham, West Union, Iowa

Helms #2 Pond: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0 | 901.0 | 0 | 0 | principal spillway: 901.0 |
| 1.0 | 902.0 | 0.02 | 0.01 | |
| 2.0 | 903.0 | 0.03 | 0.03 | |
| 3.0 | 904.0 | 0.05 | 0.07 | |
| 4.0 | 905.0 | 0.06 | 0.13 | |
| 5.0 | 906.0 | 0.08 | 0.20 | |
| 6.0 | 907.0 | 0.10 | 0.29 | |
| 7.0 | 908.0 | 0.12 | 0.40 | |
| 8.0 | 909.0 | 0.24 | 0.58 | auxiliary spillway: 909.0 |
| 8.5 | 909.5 | 0.25 | 0.70 | |
| 9.0 | 910.0 | 0.26 | 0.83 | top of dam: 910.0 |
| 9.5 | 910.5 | 0.27 | 0.96 | |
| 10.0 | 911.0 | 0.28 | 1.10 | |

Helms #2 Pond: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0 | 901.0 | 0 | 0 | principal spillway: 901.0 |
| 1.0 | 902.0 | 0.01 | 1.24 | |
| 2.0 | 903.0 | 0.03 | 1.73 | |
| 3.0 | 904.0 | 0.07 | 2.10 | |
| 4.0 | 905.0 | 0.13 | 2.42 | |
| 5.0 | 906.0 | 0.20 | 2.70 | |
| 6.0 | 907.0 | 0.29 | 2.95 | |
| 7.0 | 908.0 | 0.40 | 3.18 | |
| 8.0 | 909.0 | 0.58 | 3.40 | auxiliary spillway: 909.0 |
| 8.5 | 909.5 | 0.70 | 9.71 | |
| 9.0 | 910.0 | 0.83 | 33.09 | top of dam: 910.0 |
| 9.5 | 910.5 | 0.96 | 125.91 | |
| 10.0 | 911.0 | 1.10 | 253.43 | |

**Project:** **Helms #4 Pond** **Pond ID #24**

Drainage Area: 3.6 acres (0.006 square miles)

Description: The principal spillway is an 8" corrugated metal pipe (CMP), invert elevation of 916.0 feet MSL. The auxiliary spillway is 40 feet wide, crest elevation at 920.0 feet MSL. Top of dam at 921.0 feet MSL.

Hydraulic Design: Fehr Graham, West Union, Iowa

Helms #4 Pond: Elevation (Stage) – Pool Area – Storage relationships from design documentation.

| Stage (feet) | Elevation (feet) | Pool Area (acres) | Accumulated Storage (acre-feet) | |
|---|---|---|---|---|
| 0 | 916.0 | 0 | 0 | principal spillway: 916.0 |
| 1.0 | 917.0 | 0.03 | 0.02 | |
| 2.0 | 918.0 | 0.05 | 0.05 | |
| 3.0 | 919.0 | 0.07 | 0.11 | |
| 4.0 | 920.0 | 0.08 | 0.19 | auxiliary spillway: 920.0 |
| 4.3 | 920.3 | 0.09 | 0.21 | |
| 4.5 | 920.5 | 0.09 | 0.23 | |
| 4.8 | 920.8 | 0.09 | 0.25 | |
| 5.0 | 921.0 | 0.10 | 0.28 | top of dam: 921.0 |
| 5.5 | 921.5 | 0.10 | 0.33 | |
| 6.0 | 922.0 | 0.11 | 0.38 | |
| 6.5 | 922.5 | 0.12 | 0.44 | |
| 7.0 | 923.0 | 0.12 | 0.50 | |

Helms #4 Pond: Elevation (Stage) – Storage – Discharge relationships developed by IFC for hydrologic models.

| Stage (feet) | Elevation (feet) | Accumulated Storage (acre-feet) | Discharge (cfs) | |
|---|---|---|---|---|
| 0 | 916.0 | 0 | 0 | principal spillway: 916.0 |
| 1.0 | 917.0 | 0.02 | 1.24 | |
| 2.0 | 918.0 | 0.05 | 1.73 | |
| 3.0 | 919.0 | 0.11 | 2.10 | |
| 4.0 | 920.0 | 0.19 | 2.42 | auxiliary spillway: 920.0 |
| 4.3 | 920.3 | 0.21 | 5.46 | |
| 4.5 | 920.5 | 0.23 | 8.77 | |
| 4.8 | 920.8 | 0.25 | 12.45 | |
| 5.0 | 921.0 | 0.28 | 32.18 | top of dam: 921.0 |
| 5.5 | 921.5 | 0.33 | 125.03 | |
| 6.0 | 922.0 | 0.38 | 252.58 | |
| 6.5 | 922.5 | 0.44 | 414.15 | |
| 7.0 | 923.0 | 0.50 | 575.71 | |

# Appendix B – References


Ajami, H., M. F. McCabe, and J. P. Evans, 2015: Impacts of model initialization on an integrated surface water-groundwater model. *Hydrological Processes*, 29 (17), 3790–3801, doi:10.1002/hyp.10478.

Benning, J., D. Herzmann, C. Ingels, and A. Wilke, 2013: Agriculture and Weather Variability in the Corn Belt: Northeast Iowa. SustainableCorn.org: Crops, Climate, Culture, and Change, Iowa State University, USDA-NIFA Award No. 2011-68002-30190, URL http://sustainablecorn.org/Weather-and-Ag/Pubs/IA-Northeast.pdf.

Bicknell, B., J. Imhoff, J. Kittle, Jr., T. Jobes, and A. Donigian, Jr., 2005: HSPF Version 12.2 User's Manual. AQUA TERRA Consultants, Mountain View, Calif., in cooperation with the Office of Surface Water, U.S. Geological Survey, Reston, Virginia, and the National Exposure Research Laboratory, U.S. Environmental Protection Agency, Athens, Georgia.

Bradley, A. A., A. KC, N. P. Leach, and R. Tokuhisa, 2015: Hydrologic Modeling of the English River Watershed. IIHR Technical Report No. 494, Iowa Flood Center, IIHR–Hydroscience & Engineering.

Brunner, P. and C. T. Simmons, 2012: HydroGeoSphere: A fully integrated, physically based hydrological model. *Ground Water*, 50 (2), 170–176, doi: 10.1111/j.1745-6584.2011.00882.x.

Donigian, A., Jr., D. Baker, D. Haith, and M. Walter, 1983: HSPF Parameter Adjustments to Evaluate the Effects of Agricultural Nest Management Practices. U.S. Environmental Protections Agency, Environmental Research Laboratory, Athens, Georgia, EPA Contract No. 68-03-2895, PB- 83-247171.

Donigian, A., Jr., J. Imhoff, B. Bicknell, and J. Kittle, Jr., 1984: Application Guide for the Hydrological Simulation Program - FORTRAN. U.S. Environmental Protection Agency, Environmental Research Laboratory, Athens, Georgia, EPA/600/3-84-066.

Environmental Protection Agency, 2010: Cedar River Watershed Total Maximum Daily Load. Report, EPA Region 7.

Fry, J., et al., 2011: Completion of the 2006 National Land Cover Database for the conterminous United States. *Photogrammetric Engineering and Remote Sensing*, 77 (9), 858–864.

High Plains Regional Climate Center, 2015: Crop coefficients for corn. URL http://www.hprcc.unl.edu/awdn.php.

Hoyt, W. G., and others, 1936: Studies of relations of rainfall and run-off in the United States. U.S. Geological Survey, Water-Supply Paper 772, Washington, D.C.

Hutchinson, K. and D. Christiansen, 2013: Use of the Soil and Water Assessment Tool (SWAT) for Simulating Hydrology and Water Quality in the Cedar River Basin, Iowa, 2000-10. U.S. Geological Survey Scientific Investigations Report 2013-5002, 36 pp.

Iowa Flood Center, 2014: Hydrologic Assessment of the Upper Cedar River Watershed. IIHR Technical Report No. 489, IIHR–Hydroscience & Engineering.

Iowa Geological and Water Survey, 2010: One meter digital elevation model of Iowa. Iowa Department of Natural Resources, URL https://programs.iowadnr.gov/nrgislibx/.

Iowa Geological and Water Survey, 2013: Landform Regions of Iowa. Iowa Department of Natural Resources, URL http://www.igsb.uiowa.edu/browse/landform.htm.

Iowa Geological and Water Survey, 2015: Geosam. URL https://geosam.iihr.uiowa.edu/home.

Iowa State University, 2015: Legacy ISU AgClimate Network. URL https://mesonet.agron.iastate.edu/agclimate/.

Kang, S. Z., B. J. Gu, T. S. Du, and J. H. Zhang, 2003: Crop coefficient and ratio of transpiration to evapotranspiration of winter wheat and maize in a semi-humid region. *Agricultural Water Management*, 59 (3), 239–254, doi:10.1016/s0378-3774(02)00150-6.

Kristensen, K. J. and S. E. Jensen, 1975: A model for estimating actual evapotranspiration from potential evapotranspiration. *Nordic Hydrology*, 6 (3), 170–188.

Li, Q., A. J. A. Unger, E. A. Sudicky, D. Kassenaar, E. J. Wexler, and S. Shikaze, 2008: Simulating the multi-seasonal response of a large-scale watershed with a 3D physically-based hydrologic model. *Journal of Hydrology*, 357 (3-4), 317–336, doi:10.1016/j.jhydrol.2008.05.024.

Love, J. T. and T. P. Whitney, 2008: The Cedar River basin pathogen total maximum daily load – A case study for watershed-scale total maximum daily loads. American Society of Civil Engineers, World Environmental and Water Resources Congress 2008 Ahupua'a, CC-325A TMDLs and Non-Point Sources.

McDonald, J., 1961: On the ratio of evaporation to precipitation. *American Meteorological Society Bulletin*, 42 (3), 185–189.

Natural Resource Conservation Service, 2004a: Chapter 11 Snowmelt. Part 630 Hydrology, National Engineering Handbook. U.S. Department of Agriculture. URL http://www.wcc.nrcs.usda.gov/ftpref/wntsc/H&H/NEHhydrology/ch11.pdf.

Natural Resource Conservation Service, 2004b: Soil moisture/soil temperature pilot project. U.S. Department of Agriculture.

Natural Resource Conservation Service, 2009: Chapter 7 Hydrology. Part 630 Hydrology, National Engineering Handbook. U.S. Department of Agriculture. URL http://www.wcc.nrcs.usda.gov/ftpref/wntsc/H&H/NEHhydrology/ch7.pdf.

Natural Resource Conservation Service, 2015: Soil Climate Analysis Network (SCAN). U.S. Department of Agriculture, URL http://www.wcc.nrcs.usda.gov/scan/.

Perica, S., D. Martin, D., S. Pavlovic, I. Roy, M. St. Larent, C. Trypaluk, D. Unruh, M. Yekta, and G. Bonnin, 2013: NOAA Atlas 14, Precipitation- Frequency Atlas of the United States. National Weather Service, URL http://hdsc.nws.noaa.gov/hdsc/pfds/index.html, 297 pp.

PRISM Climate Group, 2016: Oregon State University, URL http://prism.oregonstate.edu.

RESPEC, 2007: Work plan for the Cedar River Basin Project. RSI-WP-01, Prepared by RESPEC, Rapid City, SD, for Iowa Department of Natural Resources, Des Moines, IA.

Sanford, W. E. and D. L. Selnick, 2013: Estimation of evapotranspiration across the conterminous United States using a regression with climate and land-cover data 1. *Journal of the American Water Resources Association*, 49 (1), 217–230, doi:10.1111/jawr.12010.

Schaefer, G. L., M. H. Cosh, and T. J. Jackson, 2007: The USDA Natural Re- sources Conservation Service Soil Climate Analysis Network (SCAN). *Journal of Atmospheric and Oceanic Technology*, 24 (12), 2073–2077, doi:10.1175/2007JTECHA930.1.

Schilling, K. E., 2005: Relation of baseflow to row crop intensity in Iowa. *Agriculture Ecosystems & Environment*, 105 (1-2), 433–438, doi:10.1016/j. agee.2004.02.008.

Schilling, K. E., M. K. Jha, Y.-K. Zhang, P. W. Gassman, and C. F. Wolter, 2008: 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 (7), W00A09, doi:10.1029/2007WR006644.

Schilling, K. E. and R. D. Libra, 2003: Increased baseflow in Iowa over the second half of the 20th century. *Journal of the American Water Resources Association*, 39 (4), 851–860, doi:10.1111/j.1752-1688.2003.tb04410.x.

Schlesinger, W. H. and S. Jasechko, 2014: Transpiration in the global water cycle. *Agricultural and Forest Meteorology*, 189, 115–117, doi:10.1016/j. agrformet.2014.01.011.

Shuttleworth, W., 1993: *Evaporation*, Chapter 4, Handbook of Hydrology, D.R. Maidment (editor), 4.1–4.53. McGraw-Hill, New York.

Soil Survey Staff, 2014: Gridded Soil Survey Geographic (gSSURGO) Database for the United States of America and the Territories, Commonwealths, and Island Nations served by the USDA-NRCS. U.S. Department of Agriculture, Natural Resources Conservation Service, URL https://gdg.sc.egov.usda.gov/.

Wang, L. X., et al., 2013: The effect of warming on grassland evapotranspiration partitioning using laser-based isotope monitoring techniques. *Geochimica Et Cosmochimica Acta*, 111, 28–38, doi:10.1016/j.gca.2012.12.047.

Witzke, B. J., J. L. Anderson, and J. P. Pope, 2010: Estimated depth to bedrock of Iowa as a 110-meter pixel, 32-bit imagine format raster dataset. Iowa Department of Natural Resources, OFM-2010-1.