ONE-DIMENSIONAL HYDRAULIC MODEL DEVELOPMENT FOR THE ROCK RIVER NEAR ROCK VALLEY, IOWA

by

Daniel Gilles

IIHR – Hydroscience & Engineering
College of Engineering
The University of Iowa
Iowa City, Iowa 52242-1585

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1. INTRODUCTION

The Rock River watershed experienced significant precipitation events on June 15th and 17th, 2014 after a series of smaller events saturated the area. The June 15th and 17th events are depicted in Figure 1, using Stage IV radar rainfall products produced by the National Weather Service (NWS). The cumulative rainfall of both events ranged from 5 – 8 inches across the watershed. The resulting flood wave inflicted damages in several communities as it traveled down the Rock River. The most severe flooding was experienced in the community of Rock Valley as the Rock River peaked at 22.72 feet, nearly three feet above the major flood designation defined by the NWS. An aerial photo illustrating the extent of the 2014 flooding in Rock Valley is shown in Figure 2.

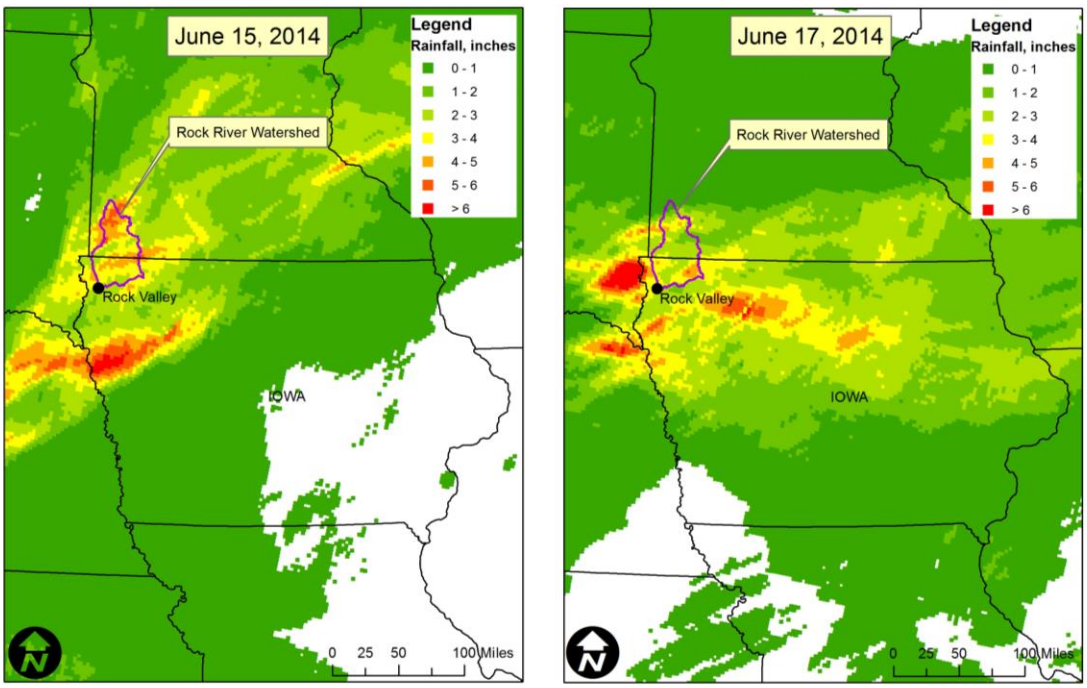

Figure 1. Observed radar rainfall totals from June 15th and 17th, 2014, which resulted in extreme flooding in Rock Valley, Iowa. (Source: National Weather Service's Advanced Hydrologic Prediction Service)

Rock Valley was selected by the Iowa Flood Center (IFC) for development of a flood inundation map library. IFC flood inundation map libraries provide information valuable to the community in evaluating its flood risk, responding to ongoing flood events, and augmenting long-term planning. The effective Federal Emergency Management Agency (FEMA) Flood Insurance

Study (FIS) for Rock Valley was completed in 1985. Since that time, more reliable topographic data have been collected for this area that will improve delineation of flood boundaries. This community is an ideal candidate for an IFC flood inundation map library due to its dated FIS and need for response and planning following these recent flood events.

Figure 2. Rock Valley, Iowa during the June 2014 flood. View is to the Northeast. (Source: Charlie Litchfield, Des Moines Register)

2. STUDY AREA

The Rock River, a tributary of the Big Sioux River, flows in a southwesterly direction just north of Rock Valley, as shown in Figure 3. The 6.2 mile study reach extends approximately 2.2 miles upstream and 4.0 miles downstream of the United States Geological Survey (USGS) Rock River gaging station (06483500) located near Rock Valley. The upstream drainage area is 1592 square miles at this the gaging station. There are three bridges within the study area: Elmwood Ave, Kiwanis Trail, and Highway 18.

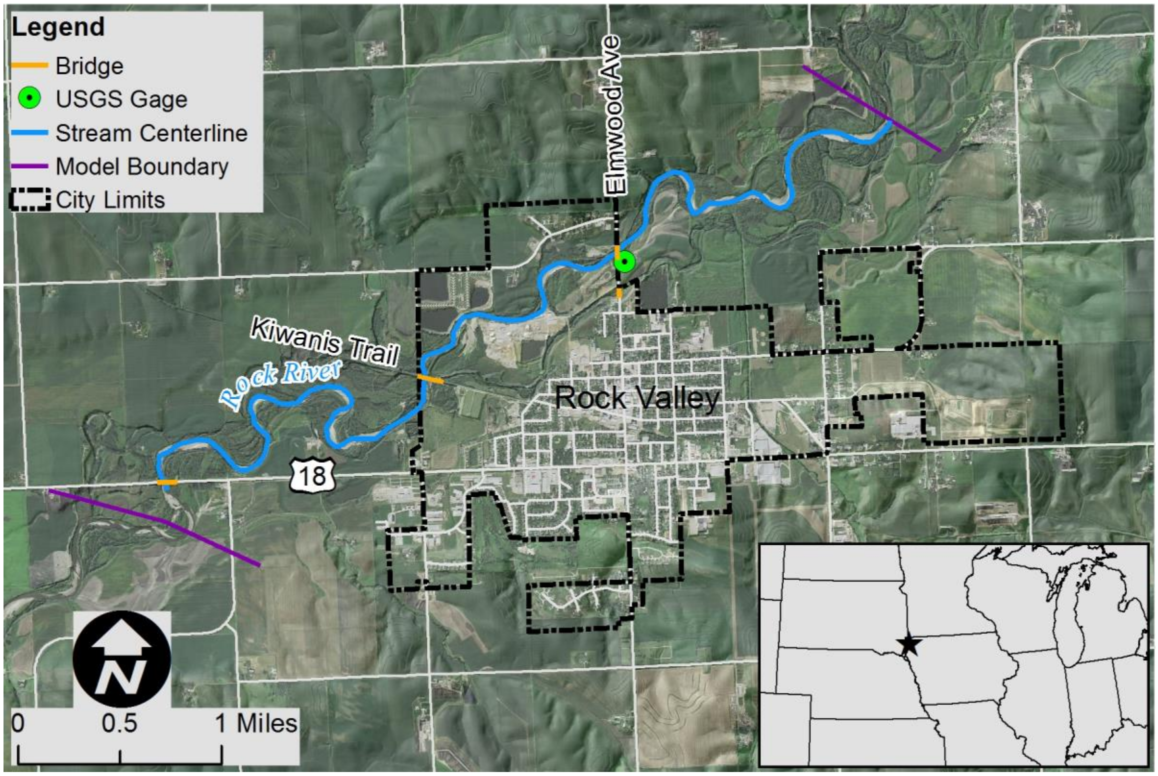

Figure 3. Rock River study area near Rock Valley, IA.

3. HYDRAULIC MODEL DEVELOPMENT

3.1. General Approach

IFC developed a one-dimensional (1D) hydraulic model of the Rock River near Rock Valley, Iowa in support of the IFC's community based mapping initiative. The purpose of this report is to describe the model development, including data sources and assumptions. The 1D hydraulic model was developed using the United States Army Corps of Engineers (USACE) Hydrologic Engineering Center's River Analysis System (HEC-RAS). HEC-RAS is capable of performing one-dimensional water surface profile calculations for steady gradually varied flow. Water surface profiles are computed between cross-sections by solving the energy equation with the standard step method. The HEC-RAS model was calibrated to information available at USGS gage 06483500 and high water marks from the 2014 Flood.

3.2. Data Sources

Topographic information was provided by the Iowa Department of Natural Resources' (IaDNR) statewide ground surface mapping project in the form of Light Detection and Ranging (LiDAR) data. Bridge geometry was provided by the Iowa Department of Transportation (IDOT), Sioux County, and IIHR-Hydroscience and Engineering (IIHR) field measurements. Some geometric parameters were estimated using real time kinematic (RTK) global navigation satellite system (GNSS) measurements, raw LiDAR points, and isometric photography provided by Microsoft Bing Maps and Google Maps. Sources of structural information for various geometric parameters are shown in Table 1. National Agriculture Imagery Program (NAIP) 2014 ortho-imagery produced by the U.S. Department of Agriculture (USDA) were used to assign overland roughness values. High water marks from the 2014 Flood were photographed by USACE and approximate elevations were measured by IIHR. At the time of measurement, many high water marks were no longer visible. Therefore, heights were estimated from photographs and elevations were measured using the RTK GNSS system. USGS also provided river gage data used for calibration. Aerial photographs just prior to the flood peak were provided by Brent Koops Photography, and aerial photographs following the flood peak were provided by the Des Moines Register.

Table 1. Structures within the study area included in the HEC-RAS model. Stationing is relative to the downstream study limit.

Structure	Type	River Station (meters)	Elevations	Piers		Span
				Spacing	Width	
Elmwood Ave Bridge (Main/Side Channels)	Bridge	6898.1	high/low chord: plan sets, RTK Measurement (Sioux Co.)	Plan Set (Sioux Co.)	Plan Set (Sioux Co.)	Plan Set (Sioux Co.)
Kiwanis Trail Bridge	Bridge	4608.2	high/low chord: RTK Measurement (IIHR)	RTK Measurement (IIHR)	Bing Maps	Google Earth
310th St (Hwy 18)	Bridge	484.6	high/low chord: plan sets (IDOT)	Plan Set (IDOT)	Plan Set (IDOT)	Plan Set (IDOT)

Model development required collection of bathymetric data, which was completed by IIHR personnel on November 5, 2014. Bathymetric measurements were completed using a Trimble R8 RTK GNSS. The Trimble R8 is rated with horizontal and vertical accuracy of ± 0.03 feet and ± 0.07 feet, respectively, with real-time corrections from a ground-based reference station. Real-time corrections were provided via cellular modem by the Iowa Real Time Network (IaRTN), a statewide system of reference stations operated by the IDOT. Measurements were collected via wading using the Trimble R8 GNSS mounted on a range pole. The majority of transect locations were determined prior to the field visit based on inspection of LiDAR topography for cross-section placement.

3.3. Boundary Conditions

Upstream boundary conditions were based on flood frequency analyses, discussed in Section 4, and discharges derived from the rating curve at USGS Gage 06483500. Downstream boundary conditions were based on a normal depth assumption using an energy surface slope of 0.0005. The energy slope was estimated using the water surface profile captured in LiDAR data near the downstream study limit.

3.4 Manning's Roughness Values

NAIP 2014 ortho-imagery produced by the USDA were used to assign initial overland roughness values. Initial Manning's roughness values were selected based on typical values provided by the HEC-RAS Hydraulic Reference Manual Version 4.1 (USACE 2010) and Mattocks and Forbes (2008). Values were modified within acceptable ranges during the calibration process discussed in Section 3.7. Initial and calibrated Manning's roughness values are shown in Table 2.

Table 2. Manning's roughness values were selected from reference materials based on land use classification. Initial roughness values were modified during the calibration process.

Land Use Description	Reference	Reference Description	Reference Range	Initial Value	Calibrated Value
Stream Channel	HEC-RAS Reference Manual	Clean, winding, some pools and shoals	0.033 - 0.045	0.035	0.040
Forest	HEC-RAS Reference Manual	Trees - Dense willows, summer, straight	0.110 - 0.200	0.150	0.150
Brush	HEC-RAS Reference Manual	Medium to dense brush, in summer	0.070 - 0.160	0.120	0.100
Pasture	HEC-RAS Reference Manual	Pasture - High grass	0.03 - 0.050	0.040	0.040
Row Crops	HEC-RAS Reference Manual	Cultivated areas - Mature Row Crops	0.025 - 0.045	0.035	0.045
Park	HEC-RAS Reference Manual	Pasture - short grass	0.025 - 0.035	0.030	0.030
Low Intensity Development	Mattocks and Forbes, 2008	Developed - Low Intensity	0.050	0.050	0.050
Med Intensity Development	Mattocks and Forbes, 2008	Developed - Med Intensity	0.100	0.100	0.100
High Intensity Development	Mattocks and Forbes, 2008	Developed - High Intensity	0.150	0.150	0.150

3.5 Contraction and Expansion Coefficients

Energy losses due to contraction or expansion of flow were captured using contraction and expansion coefficients. The absolute difference in velocity head between two cross-sections are multiplied by coefficients to estimate the energy loss due to change in flow area. Typical transitions in this model and the corresponding coefficients are shown in Table 3. These coefficients are recommended by the HEC-RAS Hydraulic Reference Manual.

Table 3. Contraction and Expansion coefficients

Transition	Contraction	Expansion
No transition loss computed	0	0
Gradual transitions	0.1	0.3
Typical bridge sections	0.3	0.5

3.6 Ineffective Flow Areas

Ineffective flow areas were used to represent contractions and expansions of flow by removing conveyance areas near road and trail embankments. The majority of ineffective flow areas are non-permanent, such that the conveyance area becomes active when the ineffective flow area is overtopped. Some ineffective flow areas are permanent to prevent over-estimation of conveyance for topographic features like ponds or sewage lagoons.

3.7 Model Calibration

The model was calibrated by adjusting Manning's roughness values until water surface elevations matched observations. The model was initially calibrated for in-bank flows using a series of steady flow simulations determined from half foot increments of the established rating curve at USGS gage 06483500. After an appropriate Manning's roughness value for the channel was selected, high flow calibration was completed using high water marks from the 2014 flood.

Locations and elevations of high water marks are shown in Figure 4. Although some high water marks are in close proximity, there are inconsistencies in elevations. It is likely that the height estimate from the photograph of high water mark “F” was incorrect based on the consistency of nearby measurements. There are also inconsistencies in slopes along the reach. This is apparent when plotting high water marks along with the simulated water surface profile, as shown in Figure 5. This likely resulted from incorrect estimates of high water mark heights, but could also be affected by many factors: incorrect assignment of corresponding river stations, temporary sandbag levees, or complex overland flow paths. The accuracy of some high water mark measurement outliers can be evaluated by comparing aerial photography near the flood peak with LiDAR terrain data. Based on these comparisons, it is likely that high water marks “A” and “C” were under-estimates of the actual elevations at those sites. It is also likely that high water mark “G” was measured at a location that didn’t flood. A comparison of high water mark measurements and simulated values for the 2014 Flood are shown in Table 4. Measurements used in high flow calibration are also indicated.

Final Manning’s roughness values, shown in Figure 6, and are within acceptable ranges defined by the HEC-RAS Hydraulic Reference Manual. A comparison of the simulated rating curve to the full range of flows provided by the rating curve at USGS gage 06483500 is shown in Figure 7. The maximum difference was approximately 0.6 feet, and the standard deviation of the residuals was 0.29 feet.

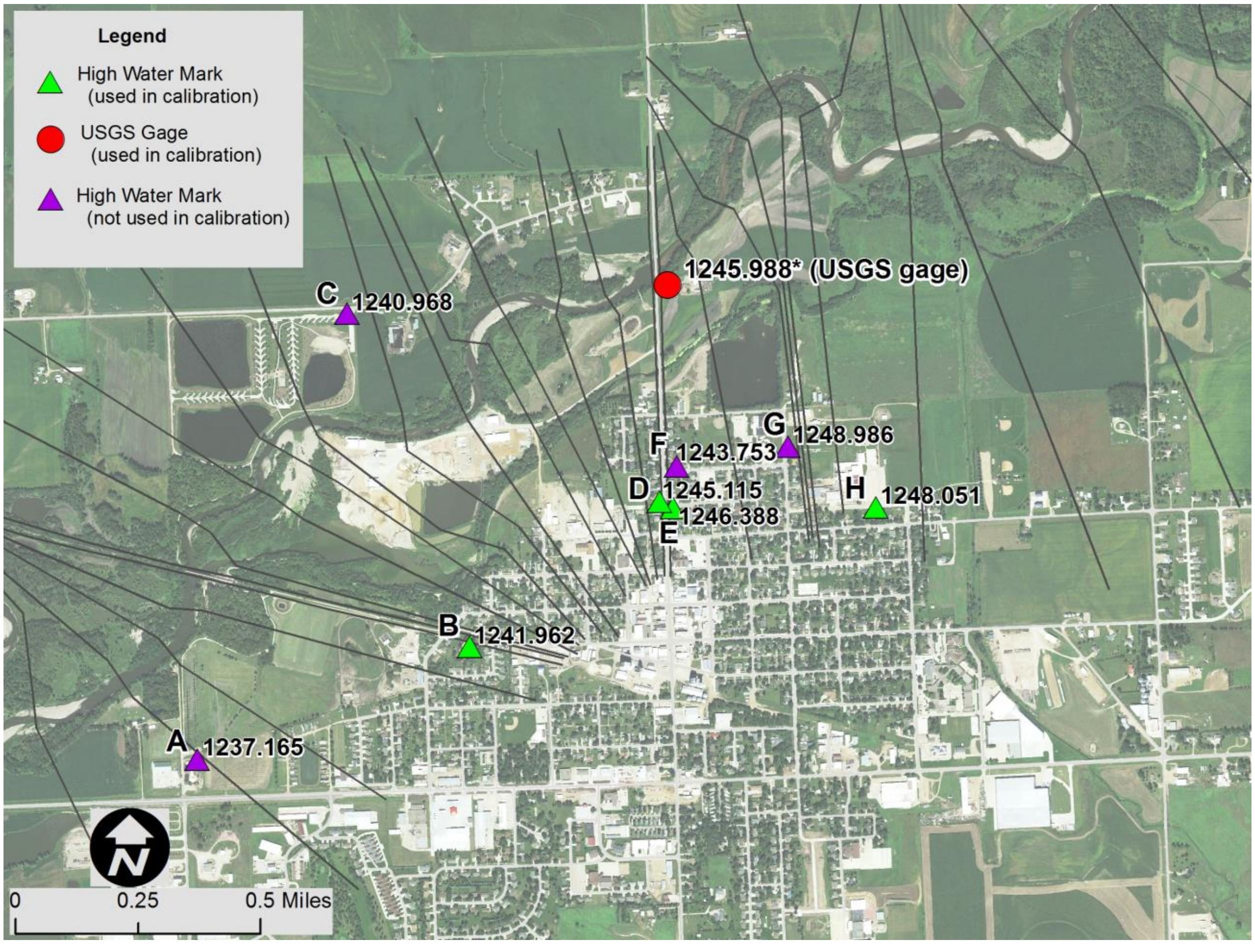

Figure 4. Elevations of high water marks (shown) were measured using RTK GNSS equipment, with the exception of a measurement provided by nearby USGS gage 06483500. In locations where high water mark lines were no longer visible, the approximate height was estimated from photographs provided by USACE.

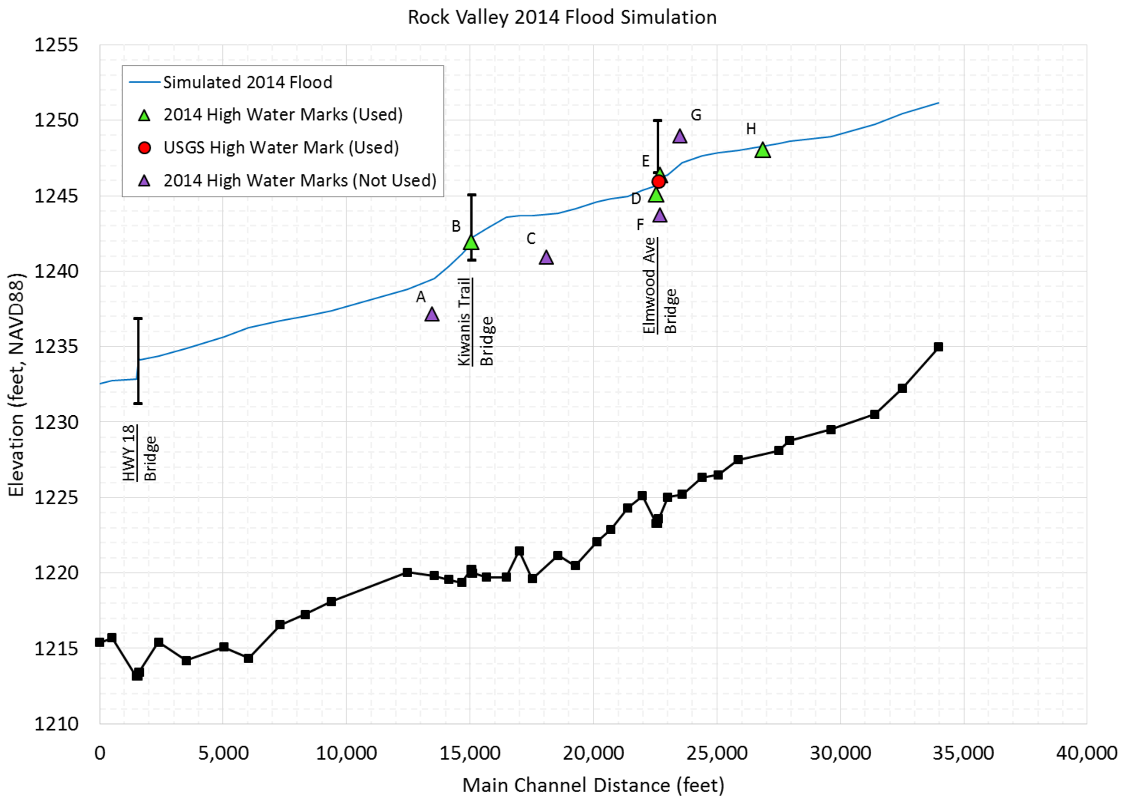

Figure 5. This plot shows the calibrated simulation of the 2014 Flood water surface profile with measured high water marks. Some high water mark elevations are inconsistent along the reach. The simulated water surface elevation at USGS gage 06483500, located at Elmwood Avenue, was 0.2 feet higher than the observed elevation.

Table 4. High water mark measurements compared to simulated results. Some high water marks were not used in calibration due to uncertainty in the measurement value and/or difficulty in reproducing the high water mark elevation.

Label	River Station (feet)	Measured Elevation (feet, NAVD88)	Simulated Elevation (feet, NAVD88)	Difference (feet)	Used in Calibration
A	13454	1237.17	1239.44	-2.27	No
B	15035	1241.96	1241.80	0.16	Yes
C	18099	1240.94	1243.80	-2.85	No
D	22543	1245.11	1245.67	-0.55	Yes
USGS	22641	1245.99	1246.19	-0.21	Yes
E	22684	1246.39	1246.39	0.00	Yes
F	22703	1243.75	1246.39	-2.64	No
G	23483	1248.98	1247.01	1.97	No
H	26842	1248.05	1248.29	-0.24	Yes

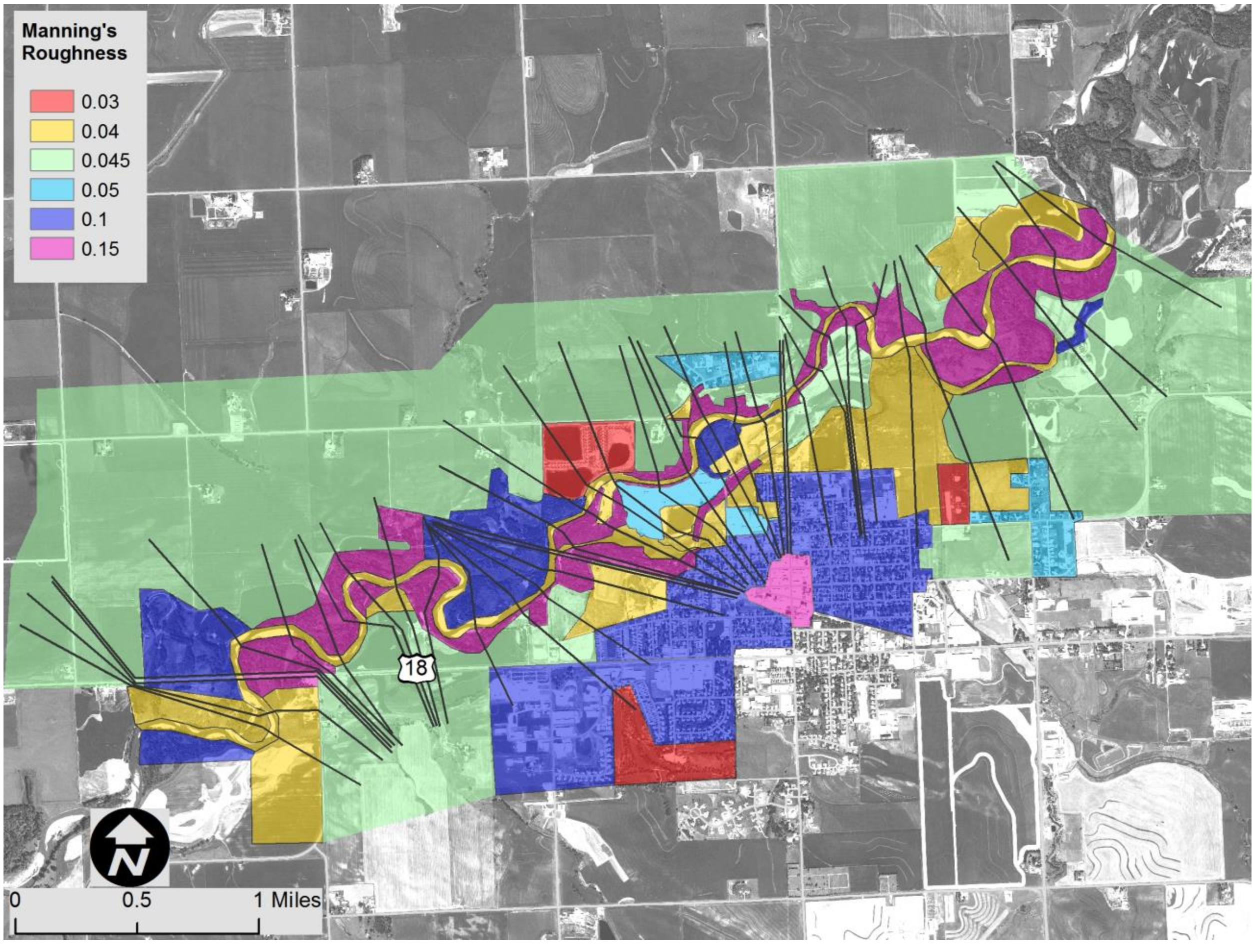

Figure 6. Final Manning's roughness values were based on 2014 NAIP imagery provided by the USDA, and calibration results.

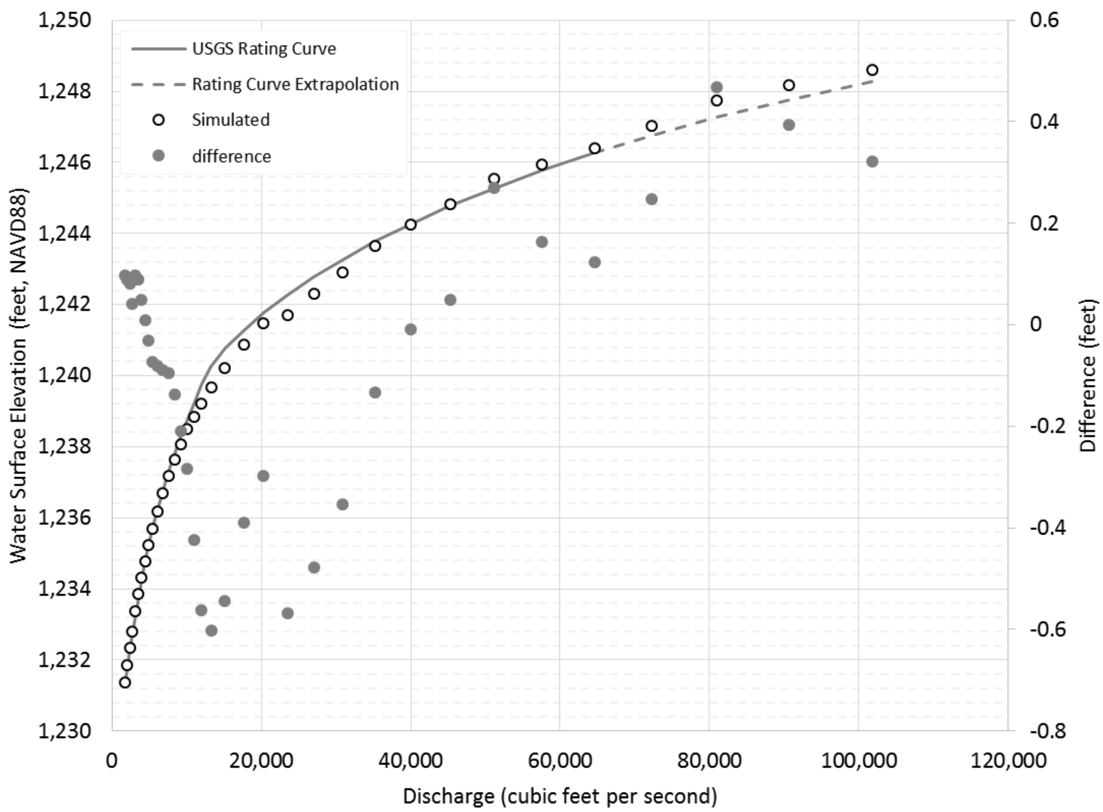

Figure 7. Comparison of simulated results to the rating curve established at USGS gage station 06483500.

3.8 Uncertainty of Stage-Discharge Relationship

The measure to define uncertainty of the stage-discharge relationship is standard deviation of the stage residuals as defined by USACE EM 1110-2-1619 (USACE 1996). This is computed using the difference between the observed and predicted stage as follows:

$$S = \sqrt{\frac{\sum_{i=1}^{N}(X_i - M)^2}{N-1}}$$

Where X_i is the stage for observation i corresponding to discharge Q_i,
M is rating curve estimation of stage corresponding to discharge Q_i,
N is the number of stage-discharge observations in the zone

A selection of the last 25 years of field measurements included 228 measured stage-discharge pairs for USGS gage 06483500. These stage-discharge pairs are plotted in Figure 8, along with the most recent rating curve and the difference between the measured and predicted stage. The approximate zones of in bank, out of bank, and rare events are shown. The estimated stage-discharge standard deviations of Zone 2 – Out of Bank Flows and Zone 3 – Rare Events were 0.99 and 0.10 feet, respectively.

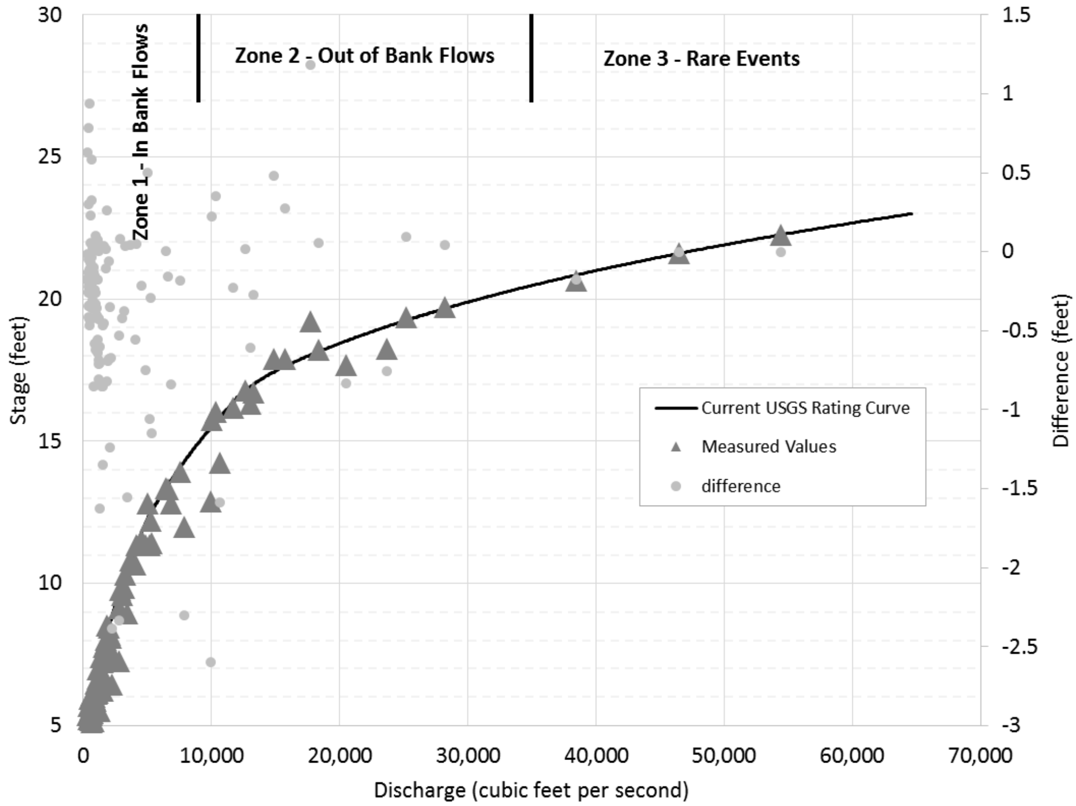

Figure 8. Measured stage-discharge pairs collected by the USGS at gage 06483500, plotted with the current rating curve. Zones for in bank and out of bank flows are also defined.

3.9 Uncertainty of Simulated Stages

Research conducted by USACE's Hydrologic Engineering Center and the U.S. Army Engineer Waterways Experiment Station (WES) provides guidance for estimating the uncertainty in simulated water surface profiles using a gradually varied flow model (USACE 1986; Freeman, Copeland, and Cowan 1996). The studies found that the uncertainty can be estimated based on the quality of topographic data and confidence in estimated Manning's roughness values during calibration, as shown in Table 5. The measurement of uncertainty is quantified by standard deviation of the errors of predicted stages. While cross-sections used in this study were based on field surveys and high-resolution LiDAR terrain data, the collection of high water marks were not consistently reliable for model calibration and validation. Based on provided guidance, the estimated uncertainty of simulated stages is 0.7 feet.

Table 5. Guidance provided by USACE EM 1110-2-1619 for estimating uncertainty in water surface profiles obtained when using a gradually varied flow model.

***Minimum* Standard Deviation of Error in Stage**		
	Standard Deviation (in feet)	
Manning's n Value Reliability[1]	**Cross Section Based on Field Survey or Aerial Spot Elevation**	**Cross Section Based on Topographic Map with 2-5' Contours**
Good	0.3	0.6
Fair	0.7	0.9
Poor	1.3	1.5

[1] Where good reliability of Manning's *n* value equates to excellent to very good model adjustment/validation to a stream gauge, a set of high water marks in the project effective size range, and other data. Fair reliability relates to fair to good model adjustment/ validation for which some, but limited, high-water mark data are available. Poor reliability equates to poor model adjustment/validation or essentially no data for model adjustment/validation.

4. FLOOD FREQUENCY ANALYSIS

The United States Geological Survey currently maintains river gaging station 06483500 ("Rock River near Rock Valley") with 67 years of continuous record (1948 – 2014). Peak discharges during this period of record are plotted in Figure 9. Using guidance provided by USACE EM 1110-2-1619, shown in Table 6, the Equivalent Record Length (ERL) of the dataset is equivalent to the systematic record length, 67 years.

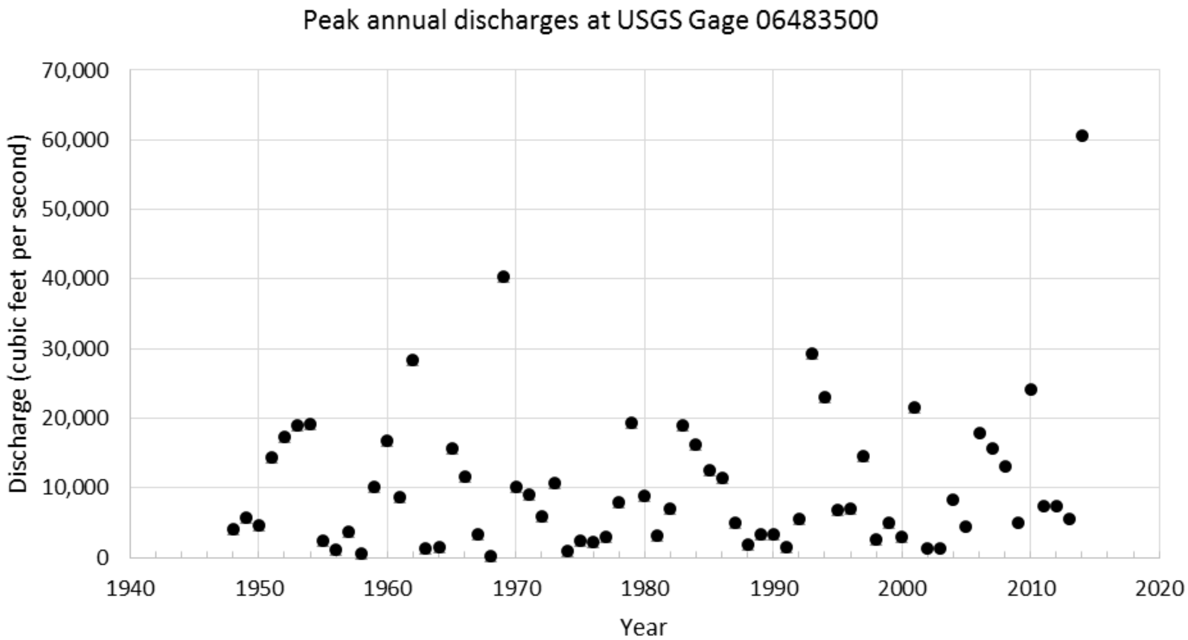

Figure 9. Peak annual discharge measurements from USGS gage 06483500, Rock River near Rock Valley.

Table 6. Guidance for estimating equivalent record lengths from USACE EM 1110-1-1619.

Equivalent Record Length Guidelines

Method of Frequency Function Estimation	Equivalent Record Length[1]
Analytical distribution fitted w ith long-period gauged record available at site	Systematic record length
Estimated from analytical distribution fitted for long-period gauge on the same stream, w ith upstream drainage area w ithin 20% of that of point of interest	90% to 100% of record length of gauged location
Estimated from analytical distribution fitted for long-period gauge w ithin same w atershed	50% to 90% of record length
Estimated w ith regional discharge-probability function parameters	Average length of record used in regional study
Estimated w ith rainfall-runoff-routing model calibrated to several events recorded at short-interval event gauge in w atershed	20 to 30 years
Estimated w ith rainfall-runoff-routing model w ith regional model parameters (no rainfall-runoff-routing model calibration)	10 to 30 years
Estimated w ith rainfall-runoff-routing model w ith handbook or textbook model parameters	10 to 15 years

1 Based on jundgement to account for the quality of any data used in the analysis, for the degree of confidence in models, and for previous experience w ith similar studies.

Flood flow frequencies were estimated using procedures described in Bulletin 17B guidelines created by the Hydrology Subcommittee of the Interagency Advisory Committee on Water Data (Interagency Advisory Committee on Water Data, 1982). A Bulletin 17B analysis was completed using USACE Hydrologic Engineering Center's Statistical Software Package (HEC-SSP) Software to estimate the discharges at selected exceedance probabilities. A regional skew value of -0.4 and a regional skew mean-square error (MSE) of 0.16 were used as regional skew parameters based on Eash (2013). The station and regional skew coefficients can be combined to form a better estimate of skew (Interagency Advisory Committee on Water Data, 1982). A weighted skew was determined by weighting the station skew and the regional skew as shown in the following equation (Interagency Advisory Committee on Water Data, 1982):

$$G_W = \frac{MSE_{\bar{G}}(G) + MSE_G(\bar{G})}{(MSE_{\bar{G}} + MSE_G)}$$

Where: G_W = weighted skew coefficient

G = station skew

$\bar{G}$ = generalized skew

$MSE_{\bar{G}}$ = mean-square error of generalized skew

MSE_G = mean-square error of station skew

A plot showing the results of the Bulletin 17B analysis is shown in Figure 10. Annual-Chance Probability estimates for the 0.2, 0.5, 1, 2, 4, and 10-percent discharges are shown in Table 7.

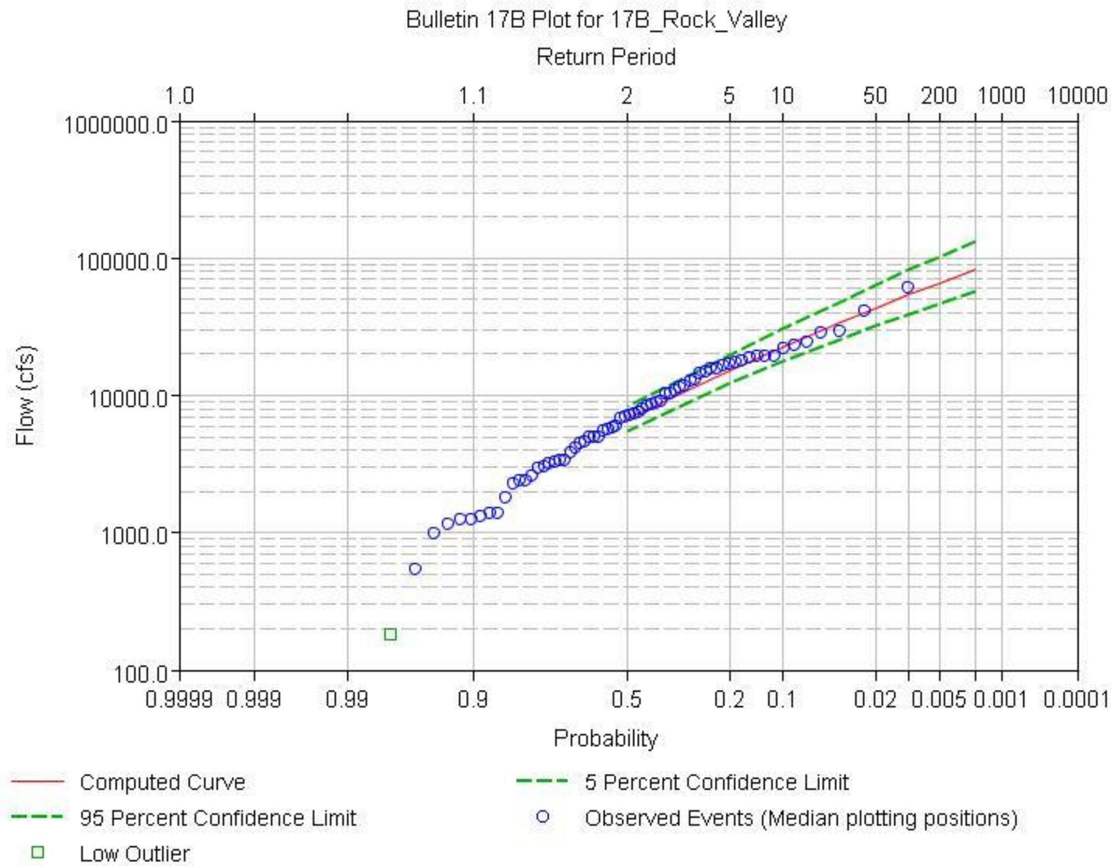

Figure 10. Results of the Bulletin 17B analysis completed using HEC-SSP.

Table 7. Percent-Annual-Chance Probability estimates developed using a Bulletin 17B analysis.

Return Year	Percent-Annual-Chance Probability	Bulletin 17B Estimate, cfs
10	10	22,740
25	4	34,030
50	2	43,660
100	1	54,210
200	0.5	65,680
500	0.2	82,240

Flood flow frequencies estimated using the Bulletin 17B analysis can be improved by weighting the estimates using regional regression equations shown in Table 8 (Eash, 2001). Weighted discharge estimates were calculated using the following equation (Eash, 2001):

$$Q_{t(wg)} = \frac{(Q_{t(pg)})(ERL) + \left(Q_{t(rg)}\right)(EYR)}{(ERL + EYR)}$$

Where: $Q_{t(wg)}$ = weighted discharge estimate for recurrence interval t

$Q_{t(pg)}$ = discharge estimate using log-Pearson Type III (Bulletin 17B)

ERL = effective record length

$Q_{t(rg)}$ = regional regression discharge estimate using Eash (2001)

EYR = equivalent years of record for the regional regression equations

Table 8. Eash (2001) Single-Parameter USGS Regional Regression Equations for the State of Iowa. (Equivalent years of record associated with the equations are shown in parentheses).

Single Parameter Regression Equations
$Q_{10} = 728 \times A^{0.465}$ (13.5 years)
$Q_{25} = 1120 \times A^{0.441}$ (20.5 years)
$Q_{50} = 1440 \times A^{0.427}$ (24.0 years)
$Q_{100} = 1800 \times A^{0.415}$ (25.9 years)
$Q_{200} = 2200 \times A^{0.403}$ (26.5 years)
$Q_{500} = 2790 \times A^{0.389}$ (26.0 years)

Final weighted discharge estimates along with weighting parameters are shown in Table 9.

Table 9. Parameters used to calculate final weighted discharge estimates using Eash (2001).

Return Year	Percent-Annual-Chance Probability	Bulletin 17B Estimate, $Q_{t(pg)}$, cfs	Equivalent Record Length, *ERL*, years	Regional-Regression, $Q_{t(rg)}$, cfs	Equivalent Years of Record, *EYR*, years	Final Weighted Discharge, $Q_{t(wg)}$, cfs
10	10	22,740	67	22,441	13.5	22,700
25	4	34,030	67	28,925	20.5	32,850
50	2	43,660	67	33,542	24	41,000
100	1	54,210	67	38,378	25.9	49,800
200	0.5	65,680	67	42,934	26	59,300
500	0.2	82,240	67	49,109	26.5	72,850

5. SIMULATION RESULTS

Flood frequency discharge estimates for 0.2-, 0.5-, 1.0-, 2.0-, 4.0-, and 10-percent-annual-chance events were simulated using a HEC-RAS steady flow analysis. The flood inundation extents were estimated using Geographic Information System (GIS) techniques and are shown in Figure 1. Interior dry islands smaller than 100 square meters were disregarded. Inundated areas in the initial preliminary inundation extents were removed if they appeared to be hydraulically disconnected from the main channel.

These flood inundation extents were created using the simulated energy grade line rather than the water surface elevation in an effort to be more conservative. This is especially relevant in areas that simulated main channel velocities are high, resulting in a localized decrease in water surface elevation, but an increase in the velocity head. A reach with very high velocities occurs near the Kiwanis Trail Bridge, resulting in a local decrease in the water surface, but a corresponding increase in velocity. Theoretically, the inundated areas away from the channel, with zero velocity, will be inundated to the elevation of the energy grade line. Simulated water surface elevation profiles along the main channel corresponding to each percent-annual-chance flow are shown in Figure 12.

Potential uses of this HEC-RAS model might include development of a flood inundation map library to be hosted on the Iowa Flood Center's Iowa Flood Information System (IFIS). Other uses include risk analysis for evaluating flood risk management projects with secondary programs, such as USACE Hydrologic Engineering Center's Flood Damage Reduction Analysis (HEC-FDA) software or FEMA's HAZUS software.

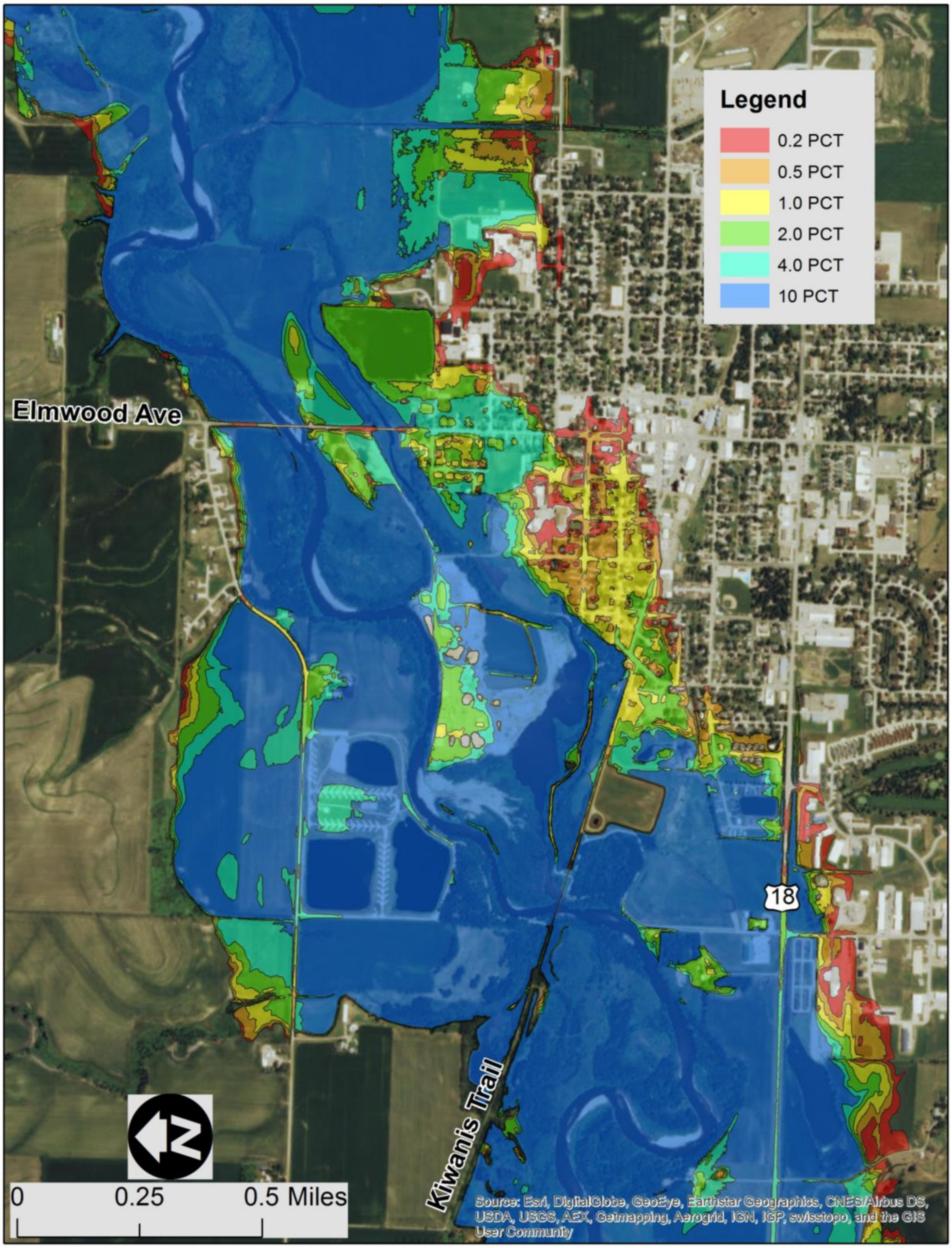

Figure 11. Flood extents based on steady flow analysis of the 0.2-, 0.5-, 1.0-, 2.0-, 4.0-, and 10-percent-annual-chance flows

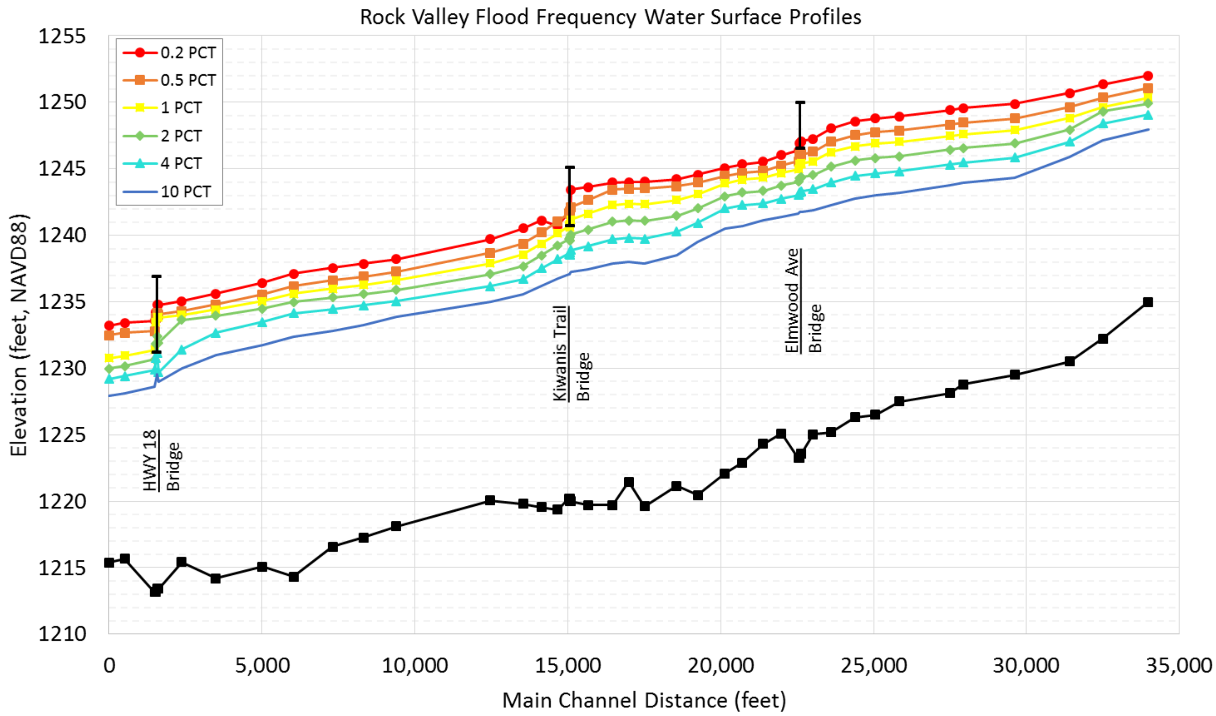

Figure 12. Simulated water surface profiles for each percent annual chance flow.

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