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

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1. INTRODUCTION

The Rock River watershed experienced a significant precipitation event on June 15th, 2014 that initiated flood preparations in Rock Rapids, Iowa. A second intense precipitation event occurred on June 17th as sandbagging efforts were underway. Many of these temporary measures were overwhelmed as the Rock River reached a flowrate of 33,800 cubic feet per second (cfs). Approximately 700 residents, or one-third of the community's population, were evacuated. The river stage crested at 26.98 feet, nearly 8 feet above the major flood designation established by the National Weather Service (NWS). An aerial photo illustrating the extent of the 2014 flooding event is shown in Figure 1.

Figure 1. Flooding in Rock Rapids, Iowa on June 17, 2014. (From M-Kopter Aerials, mkopteraerials.com)

Rock Rapids 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 Rapids was completed in 1991. Since that time, more reliable topographic data has 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.

2. STUDY AREA

The Rock River, a tributary of the Big Sioux River, flows in a southeasterly direction though the eastern portion of Rock Rapids, as shown in Figure 2. There are three bridges and two weir structures within the study area. The 2.75 mile study reach extends approximately 1.75 miles upstream and 1.0 miles downstream of the United States Geological Survey (USGS) Rock River gaging station (06483290) located near Rock Rapids. The upstream drainage area is 853 square miles at this river gaging station.

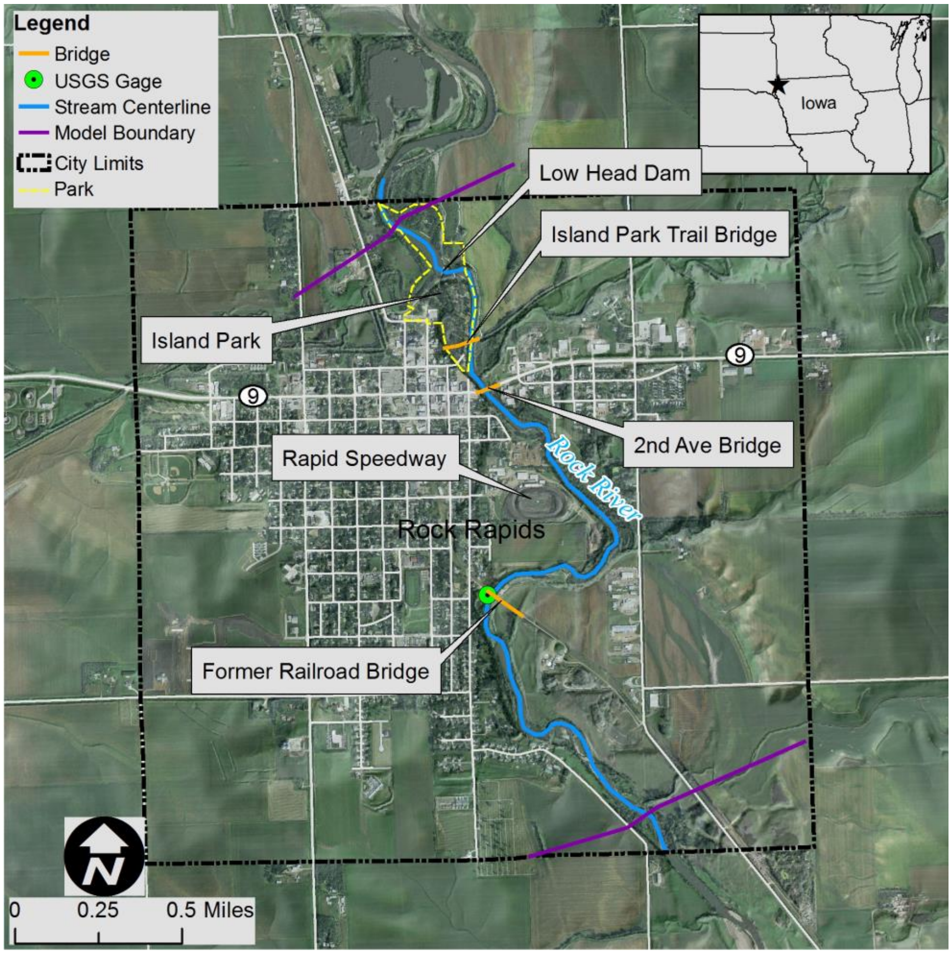

Figure 2. Rock River study area near Rock Rapids, IA.

3. HYDRAULIC MODEL DEVELOPMENT

3.1. General Approach

The Iowa Flood Center (IFC) developed a one-dimensional (1D) hydraulic model of the Rock River near Rock Rapids, 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 06483290.

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), and IIHR-Hydroscience and Engineering (IIHR) field measurements. The city and county were unable to locate as-built plan sets for Island Park Trail Bridge and the Former Railroad Bridge. Therefore, some geometric parameters were estimated using real time kinetic (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. Land cover data provided by the United States Department of Agriculture (USDA) land cover dataset were used to assign overland roughness values. USGS river gage data were used for calibration.

Table 1. Structures within the study area included in the HEC-RAS model.

Structure	Type	River Station	Elevations		Piers		Span
			High Chord	Low Chord	Spacing	Width	
Island Park Low Head Dam	Weir	4284	RTK Measurement (IIHR)	n/a	n/a	n/a	Google Earth
Island Park low water crossing	Weir	4230	RTK Measurement (IIHR)	n/a	n/a	n/a	RTK Measurement (IIHR)
Notched weir	Weir	3848	RTK Measurement (IIHR)	n/a	n/a	n/a	RTK Measurement (IIHR)
Island Park Trail Bridge	Bridge	3814	LiDAR	LiDAR	Bing Maps	Bing Maps	LiDAR
2nd Ave Bridge	Bridge	3552	Plan Set (IDOT)	Plan Set (IDOT)	Plan Set (IDOT)	Plan Set (IDOT)	Plan Set (IDOT)
Former Railroad Bridge	Bridge	1759	RTK Measurement (IIHR), LiDAR	RTK Measurement (IIHR), LiDAR	Bing Maps	Bing Maps	LiDAR

Model development required collection of bathymetric data, which was completed by IIHR personnel on November 5, 2014. Bathymetric measurements upstream of the Island Park Trail Bridge were collected using a SonTek RiverSureyor M9 acoustic Doppler profiler (ADP) deployed from an inflatable kayak. The face of the transducer was submerged 0.4 feet (0.12 meters) below the water surface, a depth sufficient to prevent entrained air interfering with measurements. The reported accuracy of the depth measured by the vertical echo-sounder is 1% of the measured depth with a resolution of 0.003 feet (0.001 meters). Horizontal and vertical positions were measured using a Trimble R8 RTK global navigation satellite system (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 Iowa Department of Transportation (IDOT).

SonTek RiverSurveyor Live software was used to integrate system components, and store measured data. Depth was recorded at each position along the boat's path at a rate of 1 Hz. Transects upstream of the Island Park dam were spaced approximately 200 feet apart, while transects downstream of the dam were spaced approximately 100 feet apart.

Bathymetric measurements downstream of the Island Park Trail Bridge, shown in Figure 2, were collected via wading using the Trimble R8 GNSS mounted on a range pole. The locations of wading transects were determined prior to the field visit based on inspection of LiDAR topography for cross-section placement.

3.2. Boundary Conditions

Upstream boundary conditions were based on flood frequency analyses, discussed in Section 4, and discharges derived from the current (at the time of model development) rating curve at USGS Gage 06483290. The downstream boundary conditions were based on a normal depth assumption. An energy slope of 0.0004 for the downstream reach was estimated using the water surface captured in LiDAR data.

3.3 Manning's Roughness Values

Land use classification areas were digitized based on 2014 National Agriculture Imagery Program (NAIP) data. Initial Manning's roughness values were selected based on typical values provided by reference material. Values were modified within acceptable ranges during the calibration process discussed in section 3.6.

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	0.130
Pasture	HEC-RAS Reference Manual	Pasture - High grass	0.03 - 0.050	0.040	0.050
Row Crops	HEC-RAS Reference Manual	Cultivated areas - Mature Row Crops	0.025 - 0.045	0.035	0.035
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

3.4 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 used 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

3.5 Ineffective Flow Areas

Ineffective flow areas were used to represent contractions and expansions of flow by removing conveyance areas near road and railroad 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.

3.6 Model Calibration

The peak river stage measured by USGS gage 06483290 during the 2014 Flood was the only high water mark available. Therefore the established rating curve at the gage station was used for calibration over a large range of flow rates. Initial Manning's roughness values were adjusted iteratively based on a series of steady flow simulations using discharges determined from half foot increments of the established rating curve at USGS gage 06483290. Final Manning's roughness values are shown in Figure 3, and are within ranges recommended by the HEC-RAS Hydraulic Reference Manual. A comparison of the simulated rating curve to the established rating curve is shown in Figure 4. The maximum difference was approximately 0.67 feet, and the standard deviation of the residuals was 0.29 feet. The 2014 Flood peak discharge of 33,800 cfs was also simulated using a steady flow analysis. The difference between the simulated and measured water surface elevation was 0.16 feet.

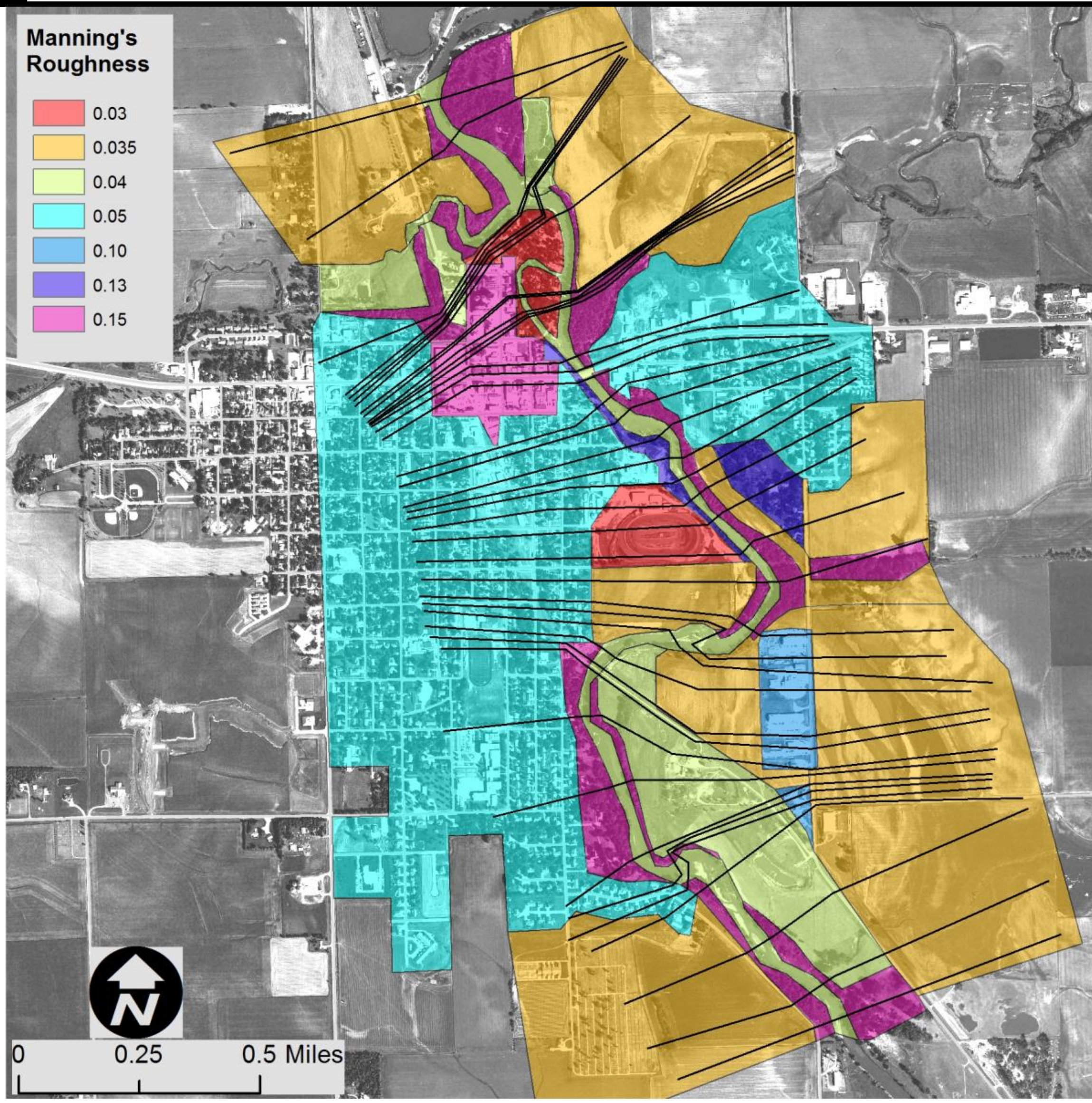

Figure 3. Final Manning's roughness values were based on land use classifications determined from 2014 aerial photos, and calibration results.

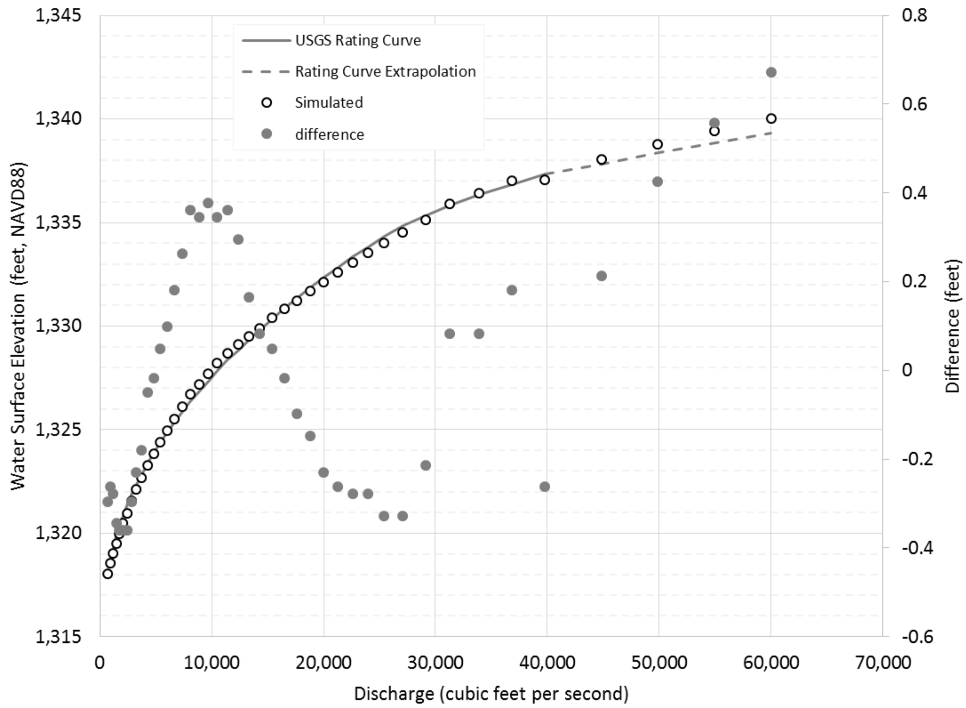

Figure 4. Comparison of simulated results to the rating curve established at USGS gage station 06483290.

3.7 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

There are 126 measured stage-discharge pairs for USGS gage 06483290. These stage-discharge pairs are plotted in Figure 5, 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 are both 0.26 feet.

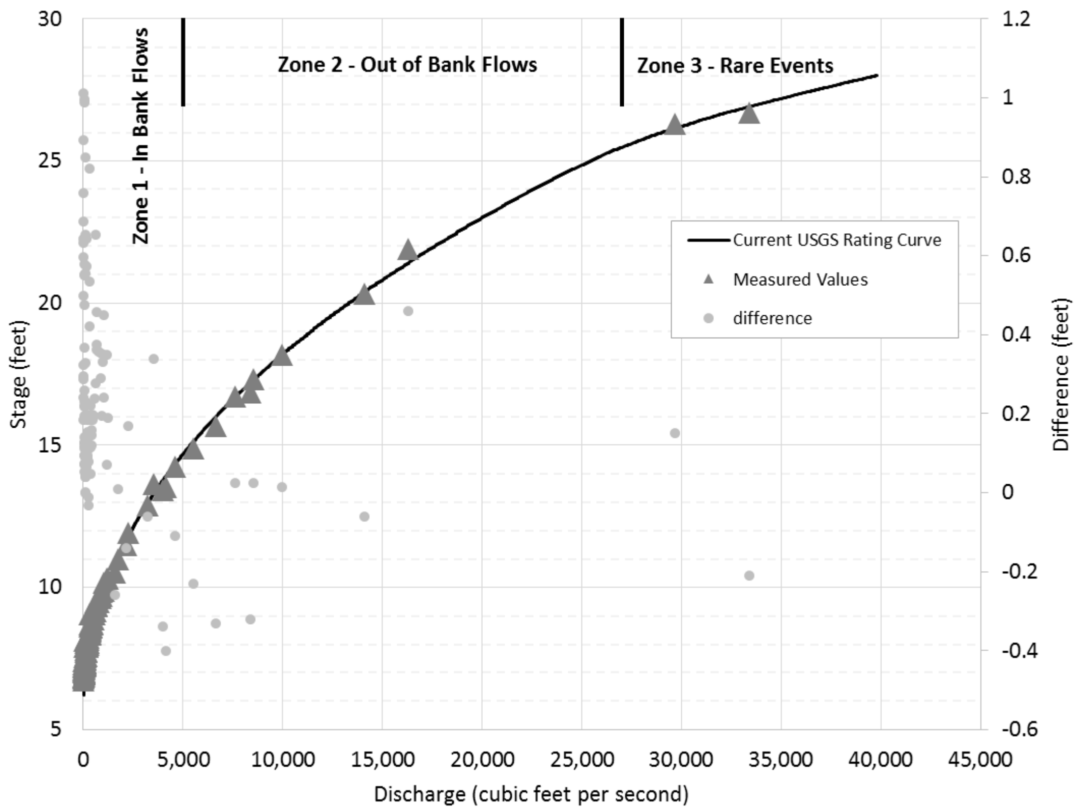

Figure 5. Measured stage-discharge pairs collected by the USGS at gage 06483290, plotted with the current rating curve. Zones for in bank and out of bank flows are also defined.

3.8 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 4. 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, no high water marks were available for model calibration and validation. Based on provided guidance, the estimated uncertainty of simulated stages is 0.7 feet.

Table 4. 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 06483290 ("Rock River below Tom Creek") with 13 years of continuous record (2002 – 2014). To supplement this information, peak flow data from USGS gage 06483500 located downstream on the Rock River in Rock Valley, Iowa was transferred to the Rock Rapids location using a direct drainage area ratio adjustment. The difference in drainage area between the two gage locations is 739 square miles, or 87% increase. The equivalent systematic record was increased to 67 years with the additional peak flow data, shown in . Using guidance provided by USACE EM 1110-2-1619, shown in , the Equivalent Record Length (ERL) of the dataset is approximately 50% to 90% of the record length due to the additional data from a long-period gage record within the same watershed. Using an average of 70%, an estimate of ERL is 47 years for this analysis.

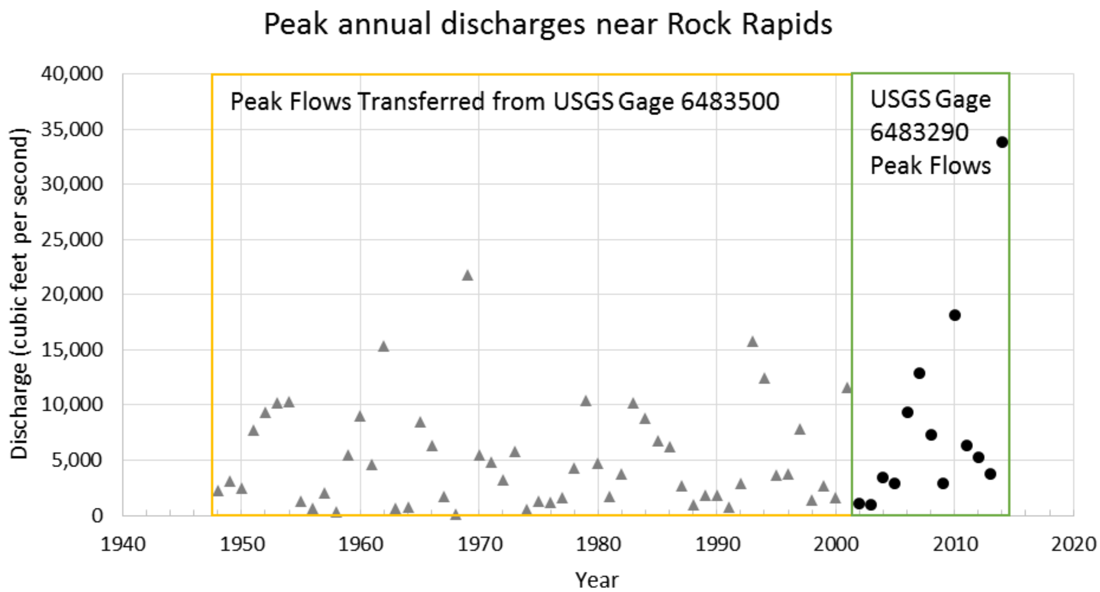

Figure 6. Peak annual discharge data on the Rock River near Rock Rapids. Data from 1948 – 2001 was collected downstream at USGS gage 06483500, and transferred using a drainage area adjustment. Data from 2002 – 2014 was collected at USGS gage 06483290, which is downstream of Tom's Creek.

Table 5. 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 with long-period gauged record available at site	Systematic record length
Estimated from analytical distribution fitted for long-period gauge on the same stream, with upstream drainage area within 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 within same watershed	50% to 90% of record length
Estimated with regional discharge-probability function parameters	Average length of record used in regional study
Estimated with rainfall-runoff-routing model calibrated to several events recorded at short-interval event gauge in watershed	20 to 30 years
Estimated with rainfall-runoff-routing model with regional model parameters (no rainfall-runoff-routing model calibration)	10 to 30 years
Estimated with rainfall-runoff-routing model with 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 with 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 7. Annual-Chance Probability estimates for the 0.2, 0.5, 1, 2, 4, and 10-percent discharges are shown in Table 6.

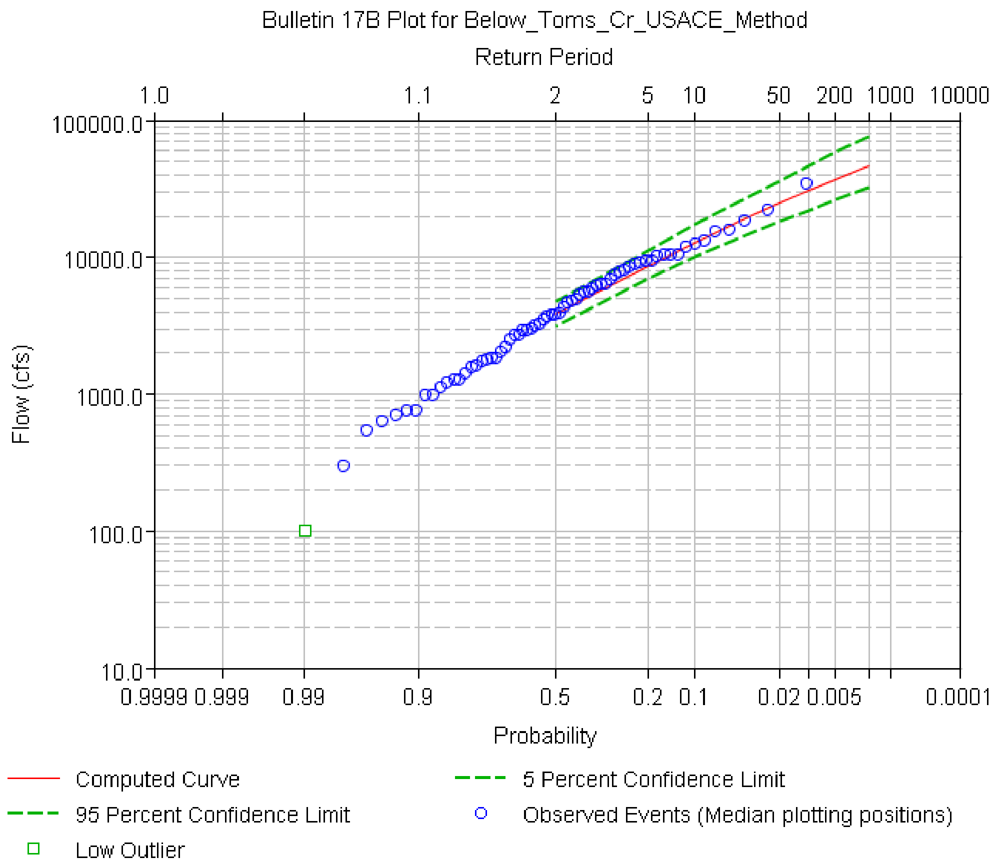

Figure 7. Results of the Bulletin 17B analysis completed using HEC-SSP.

Table 6. Percent-Annual-Chance Probability estimates developed using a Bulletin 17B analysis.

Return Year	Percent-Annual-Chance Probability	Bulletin 17B Estimate, cfs
10	10	12,830
25	4	19,210
50	2	24,680
100	1	30,710
200	0.5	37,290
500	0.2	46,860

Flood flow frequencies estimated using the Bulletin 17B analysis can be improved by weighting the estimates with estimates calculated using regional regression equations shown in Table 7 (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 7. 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 8.

Table 8. 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	12,830	47	16,789	13.5	13,710
25	4	19,210	47	21,967	20.5	20,050
50	2	24,680	47	25,697	24	25,020
100	1	30,710	47	29,622	25.9	30,320
200	0.5	37,290	47	33,388	26	35,900
500	0.2	46,860	47	38,525	26.5	43,850

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 8. Small interior dry islands larger 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 2nd Avenue (Highway 9), resulting in a local decrease in the water surface, but a corresponding equivalent 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 9.

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.

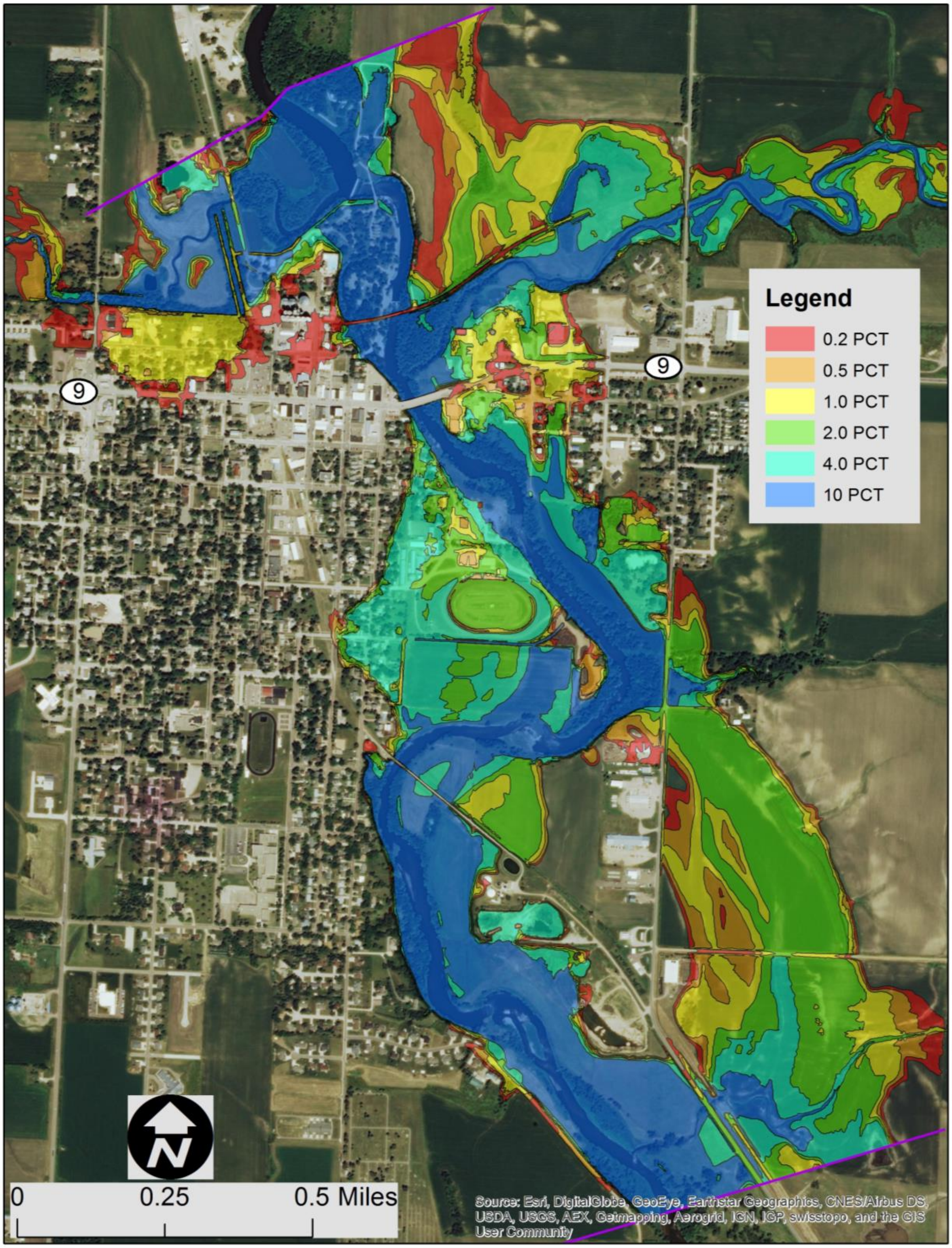

Figure 8. Flood extents based on steady flow analysis of the 0.2-, 0.5-, 1.0-, 2.0-, 4.0-, and 10-percent-annual-chance flows.

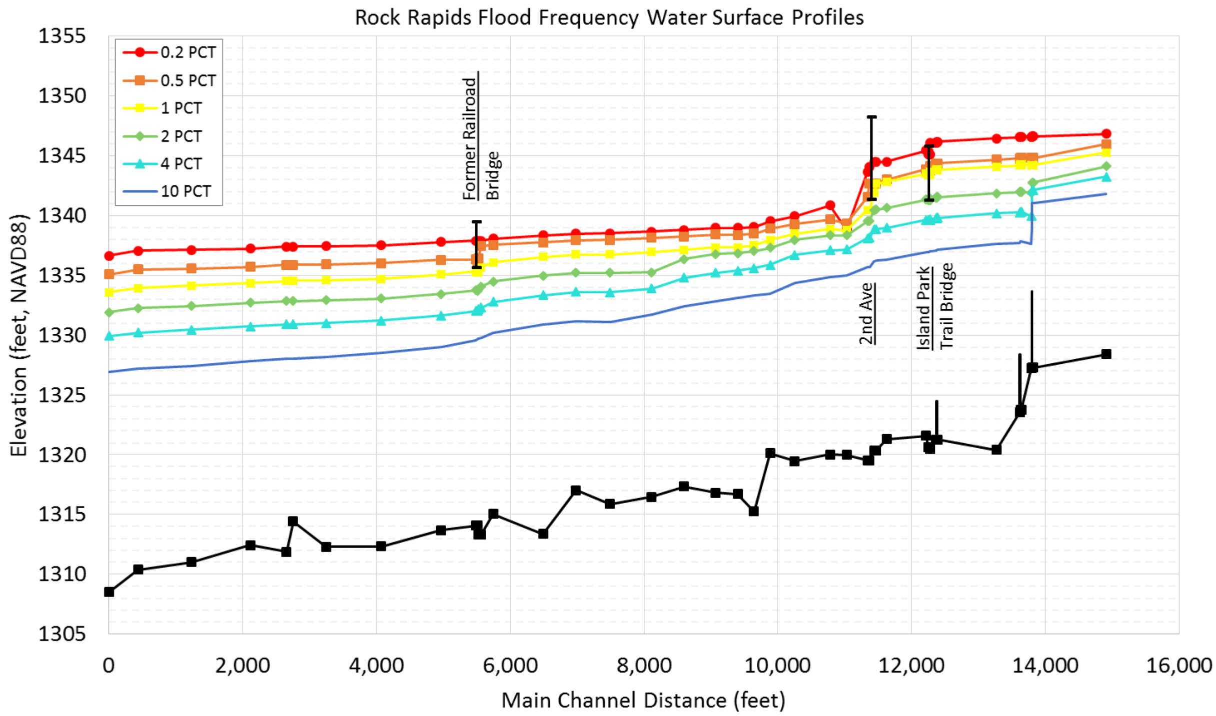

Figure 9. Simulated water surface profiles for each percent annual chance flow.

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