HYDRAULIC ANALYSIS – IDA GROVE, IOWA

Hydraulics Report for Ida Grove, IA
Case No. 16-07-2280S
by
The Iowa Flood Center

Submitted to
FEMA Region VII
Kansas City, Missouri
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1. INTRODUCTION

1.1 Study Area

Streams draining to Ida Grove, Iowa, include the Maple River, Odebolt Creek, and Badger Creek. The study area is located within Hydrologic Unit Code (HUC) eight digit identifier (HUC8) 10230005, Ida County, Iowa. The downstream limit of the study is approximately 2.2 miles downstream of the confluence of Maple River and Badger Creek. The upstream study limit of Badger Creek begins at 260th Street. The upstream study limit of Odebolt Creek begins near the Ida Grove Municipal Airport. The upstream study limit of Maple River begins at the northern corporate limit of Ida Grove. Highway 175 lies along the northern portion of the community, crossing the Maple River just upstream of the confluence with Odebolt Creek.

1.2 Purpose and Type of Study

The current flood insurance study (FIS) for Ida Grove is dated, becoming effective in March 1979. This study utilizes the latest hydraulic modeling software capable of both one- and two-dimensional (1D/2D) hydraulic modeling. The effective study was developed using standard-step backwater solvers like HEC-1 and CH20A. The CH20A solver was proprietary software developed by Stanley Consultants that is no longer widely used. Topographic data are now available that have greater spatial resolution and vertical accuracy than data used to develop the effective study.

This study utilizes longer periods of peak annual flow records and improved hydrologic analysis methods. In addition, two bridges crossing Badger Creek have been replaced in recent years.

Streams with high flood risk include Maple River and Odebolt Creek. Streams with moderate flood risk include Badger Creek.

1.3 Type of Flooding

The entire study area is riverine without any tidal influences.

1.4 Flooding History

Ida Grove has experienced flooding several times in its history, most notably in 1891, 1962, several times in recent decades, most recently in 2013. During the 1962 Flood, shown in Figure 1, the homes of at least ninety-three families were damaged when Badger and Odebolt Creeks overflowed their banks (Otjen, 2018). The aftermath of this event spurred public demand for citywide flood control. Three years after the 1962 Flood, the U.S. Army Corps of Engineers modified the channel of Odebolt Creek and constructed a series of dikes on both the Maple River and Odebolt Creek (Otjen, 2018) at the project cost of $350,000. Flood dikes were also constructed along Badger Creek in the 1980s, through independent funding and implementation.

The most recent flooding occurred in 2013, shown in Figure 2, and was the result of heavy rainfall in the upper portion of the Maple River watershed that traveled downstream along the Maple River. Figure 2 shows inundation at the confluence of Odebolt Creek and Maple River, looking upstream Odebolt Creek.

Figure 1. Photograph featured in Ida County Pioneer Record on September 6, 1962. (The Peoples' Weather Map)

Figure 2. Flooding near Pizza Hut in Ida Grove, Iowa, located near the confluence of Odebolt Creek and Maple River on May 28, 2013. Photo by Bethany Jones, KTIV.com.

1.5 Other General Information

In March 2016, the Iowa Flood Center (IFC) received a request for technical assistance from the City of Ida Grove's city clerk. The city was hopeful its flood insurance rate maps (FIRMs) could be updated to reflect any changes in flood risk after recent replacement of two Badger Creek bridges. Due to the changes in stream reach geometry, an updated hydraulic study of Badger Creek was warranted. IFC agreed to develop a new hydraulic and hydrologic (H&H) study that could be leveraged by the Iowa Department of Natural Resources (IDNR) for its ongoing floodplain mapping activities. Ida Grove contracted with JEO Consulting Group Inc. (JEO) to survey channel bathymetry and hydraulic structure information within the study area.

The new study will be provided to IDNR's FEMA contractor for incorporation in a paper FEMA map reduction project, wherein, Ida County's paper FEMA FIRMs will be converted to digital format.

The HUC8 – 10230005 watershed is currently being studied by USACE and Iowa DNR with approximate methods as part of Section 202 effort. The mapping products will become part of the Iowa DNR's statewide draft flood hazard maps. In addition to this effort, Ida County is also undergoing a FEMA physical map revision and paper map reduction effort.

2. METHODOLOGY AND MODELING

2.1 Methodology

The hydraulic model was developed using the U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center's River Analysis System (HEC-RAS) version 5.0.5. The newest software version is capable of both one- and two-dimensional simulation of flood flow, and has quickly become widely used in the engineering community. HEC-RAS is a powerful computational and visualization tool, with the ability to rapidly analyze multiple flow and geometry scenarios.

The main river channels of Badger Creek, Odebolt Creek, and Maple River were modeled separately using a one-dimensional hydraulic model, coupled to two-dimensional hydraulic models of the overbank areas. Typical computational cells were square with face dimensions of 60 feet, an example is shown in Figure 3. Breaklines defining the top of roadway embankments and berms were used within the mesh. The exchange of flow between each model domain is modeled using a lateral weir, unless noted otherwise.

Figure 3. Typical computational cells in the two-dimensional flow area.

2.2 Assumptions

2.3 Topography

Topographic information was provided by IDNR in the form of one-meter resolution bare-earth LiDAR data collected in spring 2009. Elevations reference the North American Vertical Datum of 1988 (NAVD88). The original units were meters, with a horizontal coordinate system of Universal Transverse Mercator, Zone 15 North. This coordinate system was projected to Iowa State Plane North (1401), feet. The elevations were also converted to feet.

Bathymetry and other relevant survey data was collected by JEO, adhering to FEMA guidelines and specifications.

2.4 Survey

The city of Ida Grove retained JEO to complete the survey work for this study. Survey data were collected using guidance issued by FEMA in "Guidance for Flood Risk Analysis and Mapping, Data Capture-Workflow Details: November, 2016." Locations of data collected were based on a memorandum prepared by IFC dated August 3, 2016. The scope included collecting elevation data along 90 cross-sections along the study streams, survey of 10 bridges and one weir structure, survey of the top of levee of 7,700 linear feet of levee along the Maple River. JEO also collected pictures of each cross-section location, bridges, and weir. JEO developed sketches of the upstream face of each bridge.

Data was collected in Iowa State Plane North (1401), feet. Elevations reference the North American Vertical Datum of 1988 (NAVD88). A memorandum summarizing data collection and a certification of completeness provided by JEO is included in the documentation submittal.

2.5 Boundary Conditions and Tie-ins

The downstream boundary condition of the one- and two-dimensional domains was a normal depth assumption. The inflow hydrograph for Maple River at the upstream study limit was developed using an observed hydrograph that occurred at the Mapleton, Iowa USGS gaging station 06607200 in May 2013. The lateral inflow hydrographs were developed using an SCS unit hydrograph methodology discussed in further detail in the hydrology report. A summary of boundary conditions for each return period is shown in Table 2.1.

The timing of lateral inflow hydrographs from Odebolt and Badger Creeks were calibrated using the simulated Maple River peak flow at each confluence compared to the computed Maple River discharges. The magnitude of lateral inflow hydrographs was dictated by the flow quantiles calculated for Odebolt and Badger Creeks. The timing of the lateral inflow hydrographs were adjusted until the simulated Maple River peak flows were within 20 cfs of the computed discharges. Comparisons of calculated Maple River peak flow versus simulated peak flow using calibrated lateral inflows just downstream of Odebolt and Badger Creeks are shown in Table 2.2 and Table 2.3, respectively.

Table 2.1. Summary of boundary conditions.

Study Reach	Percent AEP	Return Year	Discharge (cfs)	Source	Downstream Boundary
Badger Cr	50	2	501	StreamStats	Normal Depth
	20	5	1040	StreamStats	Normal Depth
	10	10	1580	StreamStats	Normal Depth
	4	25	2400	StreamStats	Normal Depth

	2	50	2950	StreamStats	Normal Depth
	1	100	3500	StreamStats	Normal Depth
	0.5	200	4540	StreamStats	Normal Depth
	0.2	500	4980	StreamStats	Normal Depth
	1-Plus	100-Plus	4281	StreamStats	Normal Depth
Odebolt Cr	50	2	1840	StreamStats	Normal Depth
	20	5	3570	StreamStats	Normal Depth
	10	10	5220	StreamStats	Normal Depth
	4	25	7690	StreamStats	Normal Depth
	2	50	9320	StreamStats	Normal Depth
	1	100	10900	StreamStats	Normal Depth
	0.5	200	14000	StreamStats	Normal Depth
	0.2	500	15100	StreamStats	Normal Depth
	1-Plus	100-Plus	13330.7	StreamStats	Normal Depth
Maple R (U/S Odebolt)	50	2	5134	17C	n/a
	20	5	8641	17C	n/a
	10	10	11023	17C	n/a
	4	25	13996	17C	n/a
	2	50	16152	17C	n/a
	1	100	18241	17C	n/a
	0.5	200	20271	17C	n/a
	0.2	500	22869	17C	n/a
	1-Plus	100-Plus	21859	17C	n/a
Maple R (D/S Odebolt)	50	2	5518	17C	Odebolt Cr unsteady inflow with peak of 1840 cfs, calibrated timing
	20	5	9288	17C	Odebolt Cr unsteady inflow with peak of 3570 cfs, calibrated timing
	10	10	11849	17C	Odebolt Cr unsteady inflow with peak of 5220 cfs, calibrated timing
	4	25	15045	17C	Odebolt Cr unsteady inflow with peak of 7690 cfs, calibrated timing
	2	50	17362	17C	Odebolt Cr unsteady inflow with peak of 9320 cfs, calibrated timing
	1	100	19607	17C	Odebolt Cr unsteady inflow with peak of 10900 cfs, calibrated timing
	0.5	200	21789	17C	Odebolt Cr unsteady inflow with peak of 14000 cfs, calibrated timing
	0.2	500	24582	17C	Odebolt Cr unsteady inflow with peak of 15100 cfs, calibrated timing

	1-Plus	100-Plus	23497	17C	Odebolt Cr unsteady inflow with peak of 13330.7 cfs, calibrated timing
Maple R (D/S Badger)	50	2	5558	17C	Badger Cr unsteady inflow with peak of 501 cfs, calibrated timing. Normal Depth at DS Boundary.
	20	5	9356	17C	Badger Cr unsteady inflow with peak of 1040 cfs, calibrated timing. Normal Depth at DS Boundary.
	10	10	11935	17C	Badger Cr unsteady inflow with peak of 1580 cfs, calibrated timing. Normal Depth at DS Boundary.
	4	25	15154	17C	Badger Cr unsteady inflow with peak of 2400 cfs, calibrated timing. Normal Depth at DS Boundary.
	2	50	17488	17C	Badger Cr unsteady inflow with peak of 2950 cfs, calibrated timing. Normal Depth at DS Boundary.
	1	100	19750	17C	Badger Cr unsteady inflow with peak of 3500 cfs, calibrated timing. Normal Depth at DS Boundary.
	0.5	200	21948	17C	Badger Cr unsteady inflow with peak of 4540 cfs, calibrated timing. Normal Depth at DS Boundary.
	0.2	500	24761	17C	Badger Cr unsteady inflow with peak of 4980 cfs, calibrated timing. Normal Depth at DS Boundary.
	1-Plus	100-Plus	23667	17C	Badger Cr unsteady inflow with peak of 4281 cfs, calibrated timing. Normal Depth at DS Boundary.

Table 2.2. Comparison of calculated Maple River peak flow versus simulated peak flow using calibrated lateral inflow just downstream of Odebolt Creek.

Percent AEP	Maple River Calculated Peak Q d/s Odebolt Creek (cfs)	Maple River Simulated Peak Q d/s Odebolt Creek (cfs)	Absolute Error (cfs)	Percent Relative Error	Odebolt Peak Time (hrs)
50	5518	5524.3	6.297	-0.11%	28.700

20	9288	9282.0	5.991	0.06%	28.000
10	11849	11841.8	7.178	0.06%	27.600
4	15045	15038.9	6.136	0.04%	27.500
2	17362	17368.2	6.244	-0.04%	27.400
1	19607	19620.2	13.246	-0.07%	27.800
0.5	21789	21805.7	16.715	-0.08%	27.000
0.2	24582	24587.9	5.904	-0.02%	26.800
1-plus	23497	23502.5	5.531	-0.02%	27.100

Table 2.3. Comparison of calculated Maple River peak flow versus simulated peak flow using calibrated lateral inflow just downstream of Badger Creek.

Percent AEP	Maple River Calculated Peak Q d/s Badger Creek (cfs)	Maple River Simulated Peak Q d/s Badger Creek (cfs)	Absolute Error (cfs)	Percent Relative Error	Badger Creek Peak Timing (hrs)
50	5558	5570.1	12.111	-0.22%	31.100
20	9356	9354.5	1.479	0.02%	31.400
10	11935	11919.4	15.564	0.13%	31.400
4	15154	15160.4	6.421	-0.04%	31.200
2	17488	17502.0	14.022	-0.08%	31.200
1	19750	19760.9	10.881	-0.06%	30.600
0.5	21948	21931.6	16.435	0.07%	31.100
0.2	24761	24766.3	5.273	-0.02%	31.300
1-plus	23667	23676.3	9.307	-0.04%	31.400

2.6 Cross Sections

Cross-sections were placed at surveyed transects along each reach, as shown in Figure 4 . Typical cross-section spacing was 300 – 500 feet, with more closely spaced cross-sections near bridges to capture any head loss through openings. Maple River cross-section spacing downstream of the study area was coarser, but sufficient to establish a downstream normal depth boundary condition. Additional interpolated cross-sections were generated in the downstream reach of Maple River, with elevations extracted from LiDAR data. The channel inverts of these additional cross-sections were interpolated from surveyed cross-sections.

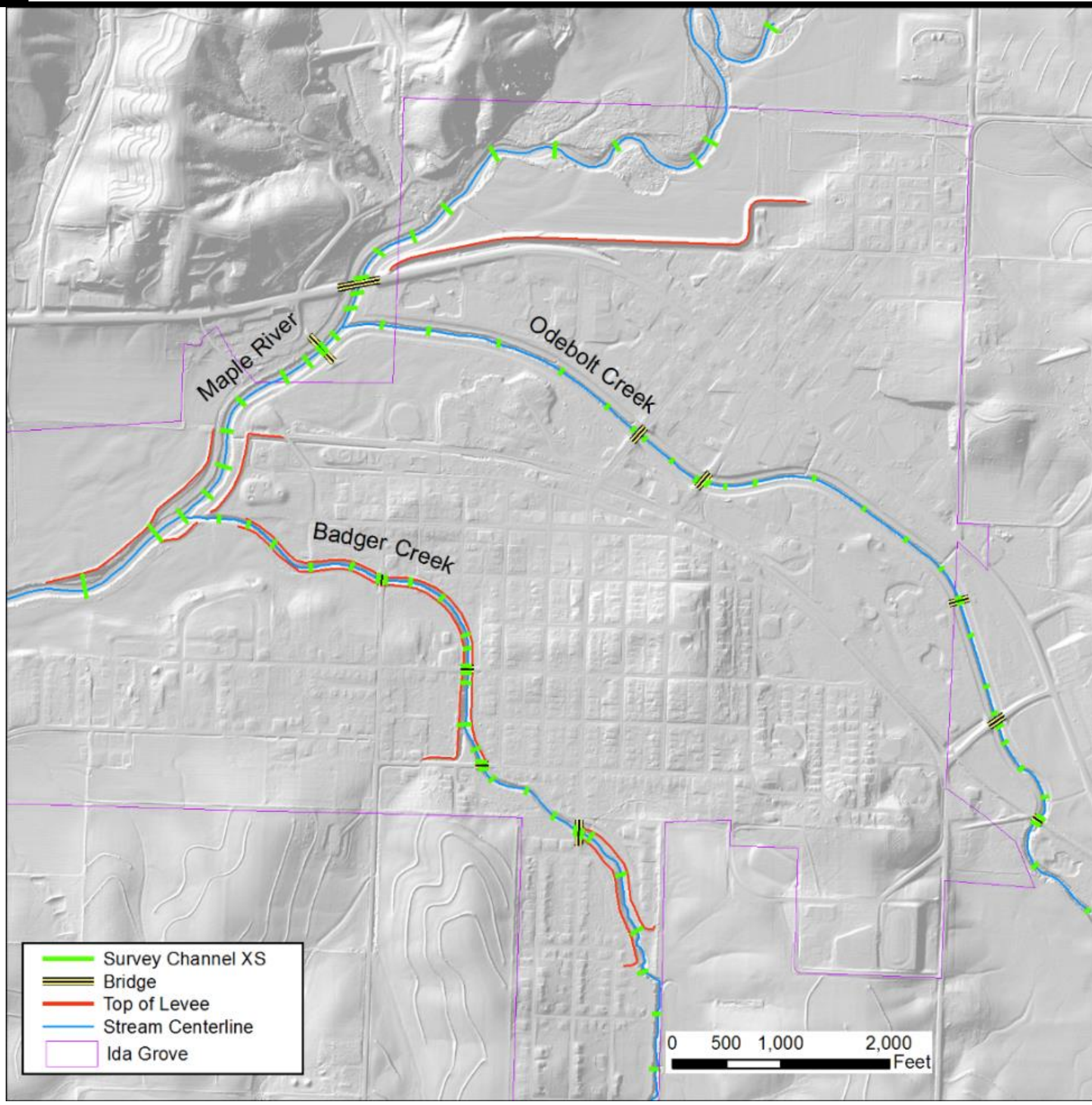

Figure 4. Ida Grove study area, some surveyed cross-sections are not shown.

2.7 Structures

There are eleven bridges within the study area; four on Badger Creek, five on Odebolt Creek, and two on Maple River, shown in Figure 4. Bridge geometries were developed from survey data and photographs collected by JEO. The structures' piers, low and high chords were incorporated into a one-dimensional hydraulic model using HEC-RAS bridge routines. Modeling methods for low and high flow scenarios are shown in Table 2.4. For low flow, the highest head loss of either the Energy or Momentum methods were selected when piers were present. Energy methods were the default method for high flow. Pressure/Weir methods were selected for high flow if the water surface profile reached the bridge deck for the 1-percent-annual-chance flow simulation. Ineffective flow areas were used to account for flow contraction and expansion through the structures.

Table 2.4. Bridge modeling methods for low and high flow.

				Modeling Method	
Stream	Reach	Bridge Name	Bridge Station	Low Flow	High Flow
Badger_Cr	Badger_Cr	Main St	5431.919	Highest (Energy, Momentum)	Energy Only
		7th St	4302.237	Highest (Energy, Momentum)	Energy Only
		5th St	3374.836	Highest (Energy, Momentum)	Energy Only
		Rohwer St	2050.718	Energy	Pressure/weir
Odebolt_Cr	Odebolt_Cr	Railroad	8848.541	Highest (Energy, Momentum)	Energy Only
		Harold Godberson Dr	7768.646	Highest (Energy, Momentum)	Energy Only
		Golf Course	6603.354	Highest (Energy, Momentum)	Energy Only
		Washington St	3761.387	Highest (Energy, Momentum)	Energy Only
Maple_R	Maple_R	Hwy 175	441.9974	Highest (Energy, Momentum)	Energy Only
Maple_R_2	Maple_R_2	Pleasant Valley Trail	2127.21	Highest (Energy, Momentum)	Energy Only

Levee Considerations

Levee structures are located along each stream, and some are documented in the USACE National Levee Database (NLD) and/or FEMA's midterm levee inventory, which can be seen in Figure 5. Each levee structure or berm was modeled as a lateral weir in the hydraulic model. Additionally, the exchange of flow between the 1D and 2D models required lateral weir structures along each stream, even when berms or embankments were not present. In these cases, the "Normal 2D Equation Domain" option was selected for the overflow computation method rather than the "Use Weir Equation" option. This was done to avoid sharp changes in water surface across the lateral connection.

The Maple River levee segment located upstream of Ida Grove is currently shown as providing protection from the 1-percent-annual-chance flood in the effective FIRM. As part of the paper map reduction project, the accreditation status of Ida Grove's levee system will be likely be evaluated by FEMA. The accreditation process would require documentation of design criteria demonstrating adequate freeboard, closure structures, embankment protection, embankment and foundation stability, settlement, and interior drainage. Additional information regarding operation plans for closures, interior drainage systems and maintenance plans must all be certified by a registered professional engineer.

In anticipation of FEMA mapping procedures for non-accredited levees, several HEC-RAS geometry files were created. These geometry files are intended to provide scenarios for existing conditions and hypothetical scenarios such as the removal of levee embankments for Natural Valley procedures. The "existing_conditions" scenario is a non-conservative estimate of inundation, as it allows all documented (FEMA and National Levee Database) levees and undocumented berms to provide protection to the embankment crest with no freeboard requirement. The documentation status of each levee or berm is shown in Figure 6. Natural Valley procedures allow flow to occur on the landward sides of levees. Base Flood Elevations (BFEs) on the left-landward side, right-landward side, and riverside of the levee are established using separate geometry scenarios. Figure 6 shows levee structure names, some of which were systematically removed for Natural Valley treatment. Table 2.5 describes the changes to each geometry file. Levee or berm structures were removed from the terrain model by interpolating from toe of the embankment on the wet side to the toe of the dry side. The resulting terrain elevation profile along the structure alignment was used as the weir elevation profile for the lateral or 2D flow area connection. While Natural Valley procedure guidance states that the analysis should be done leaving the topographic features of the levee in place, this guidance is intended for a strictly one-dimensional model. Since this these levee features are represented using a 1D/2D model, the levees or berms have to be removed from the terrain, rather than just ignored, and conveyance allowed on the dry side of the feature for Natural Valley analyses.

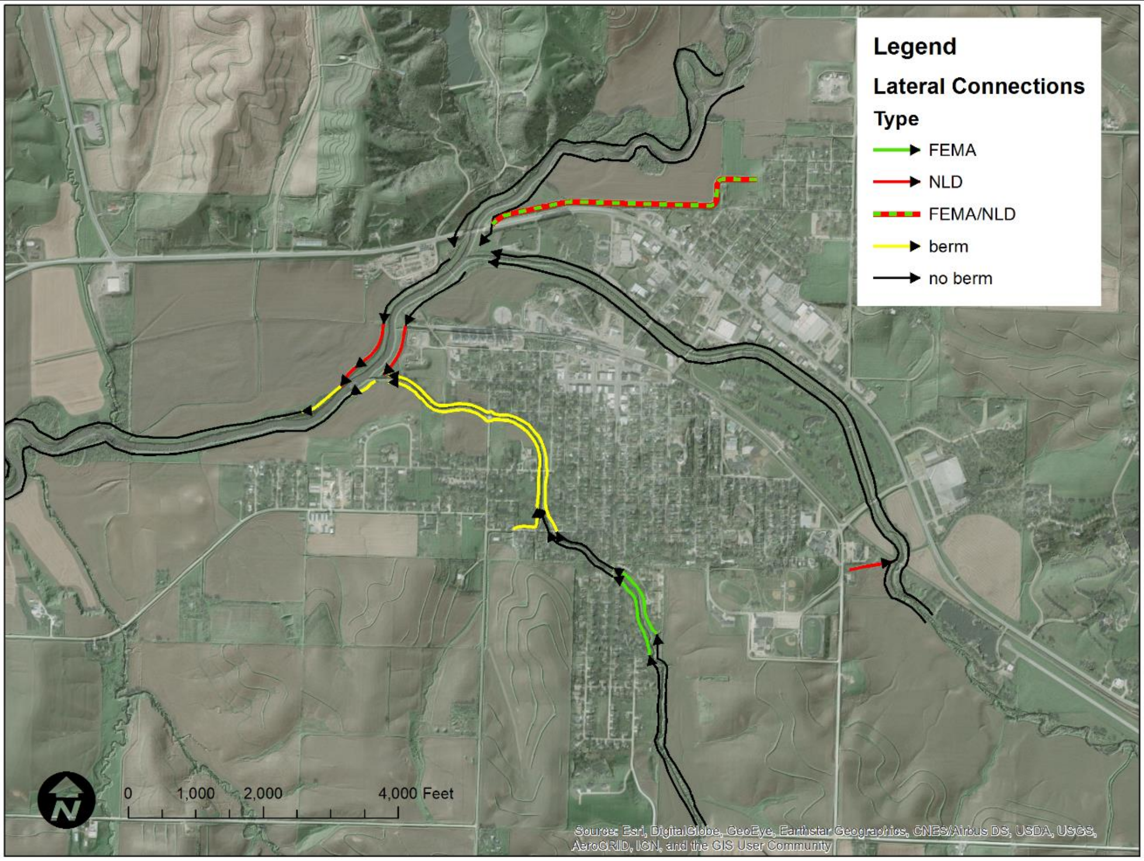

Figure 5. Lateral weir structures with the hydraulic model. Some levee structures are documented in the NLD and/or FEMA databases.

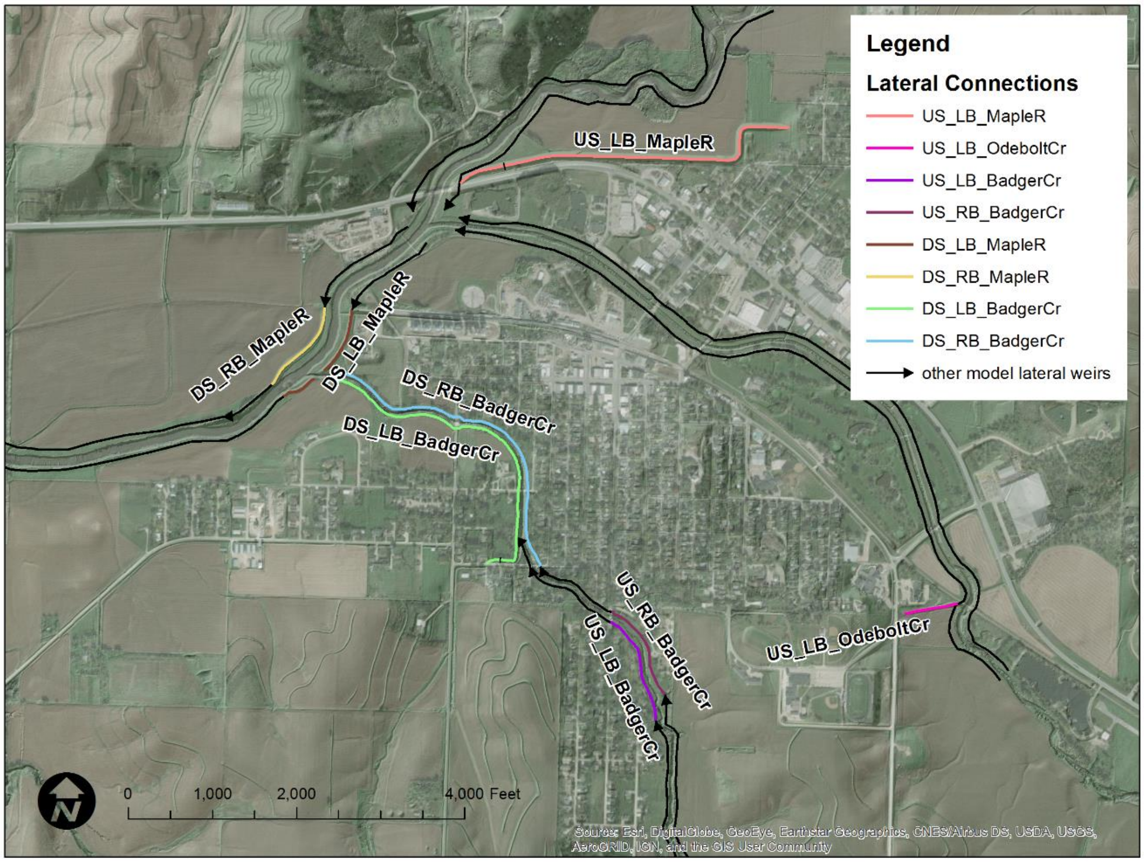

Figure 6. Levee structures receiving Natural Valley treatment in separate HEC-RAS geometry files.

An additional consideration for "Maple_River_LB_O" (FEMA ID – 1701001547, NLD FC ID – 4705000161), was the treatment of Highway 59 that runs parallel to the levee structure, as shown in Figure 7. The Highway 59 embankment was likely not designed and constructed as to provide flood protection, therefore, allowing this feature to remain in the Natural Valley scenario for "Maple_River_LB_O" will inadvertently represent the highway embankment as providing flood hazard reduction. This would indicate a lesser flood hazard extent and corresponding risk than what actually exists. Therefore, the Highway 59 embankment was also removed in addition to "Maple_River_LB_O" (FEMA ID – 1701001547, NLD FC ID – 4705000161) during Natural Valley procedures. The embankments were removed by interpolating from the toe of the wet side of the embankment to the toe of the dry side of the embankment, as shown in Figure 8.

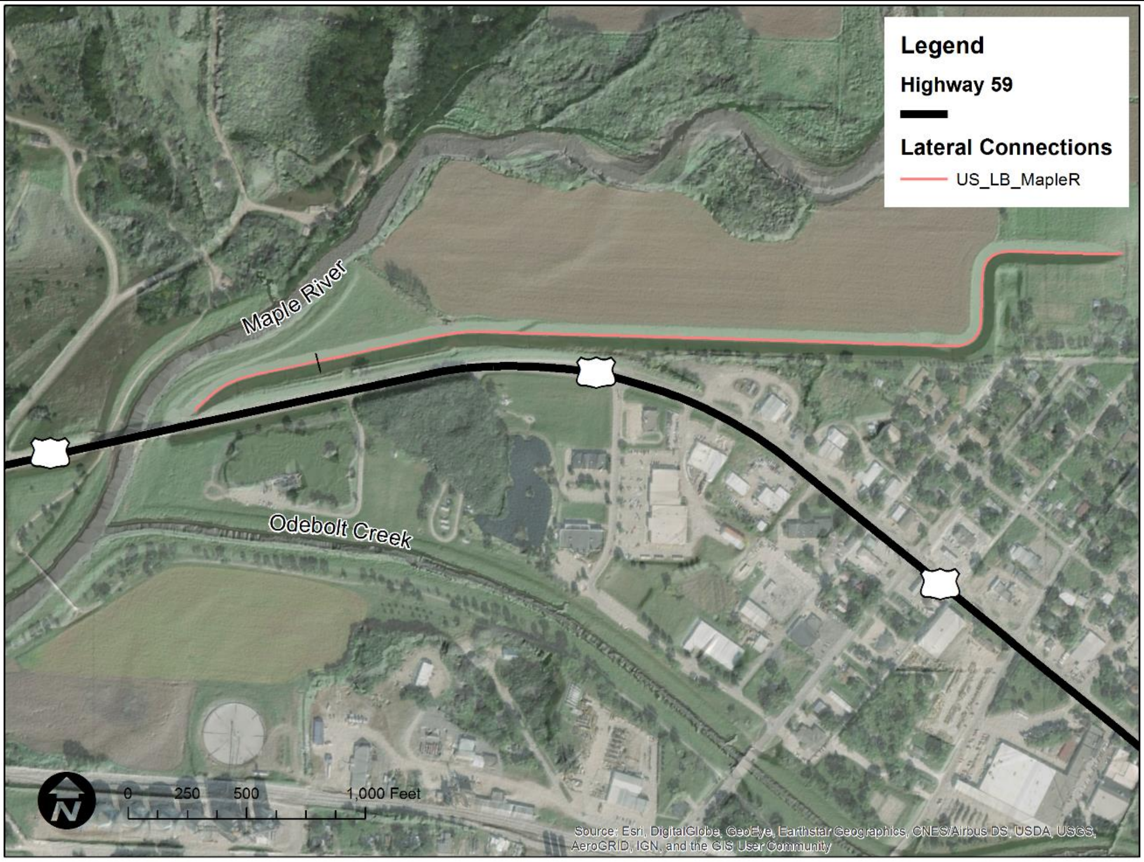

Figure 7. Location of Highway 59 embankment relative to the upstream Maple River levee system.

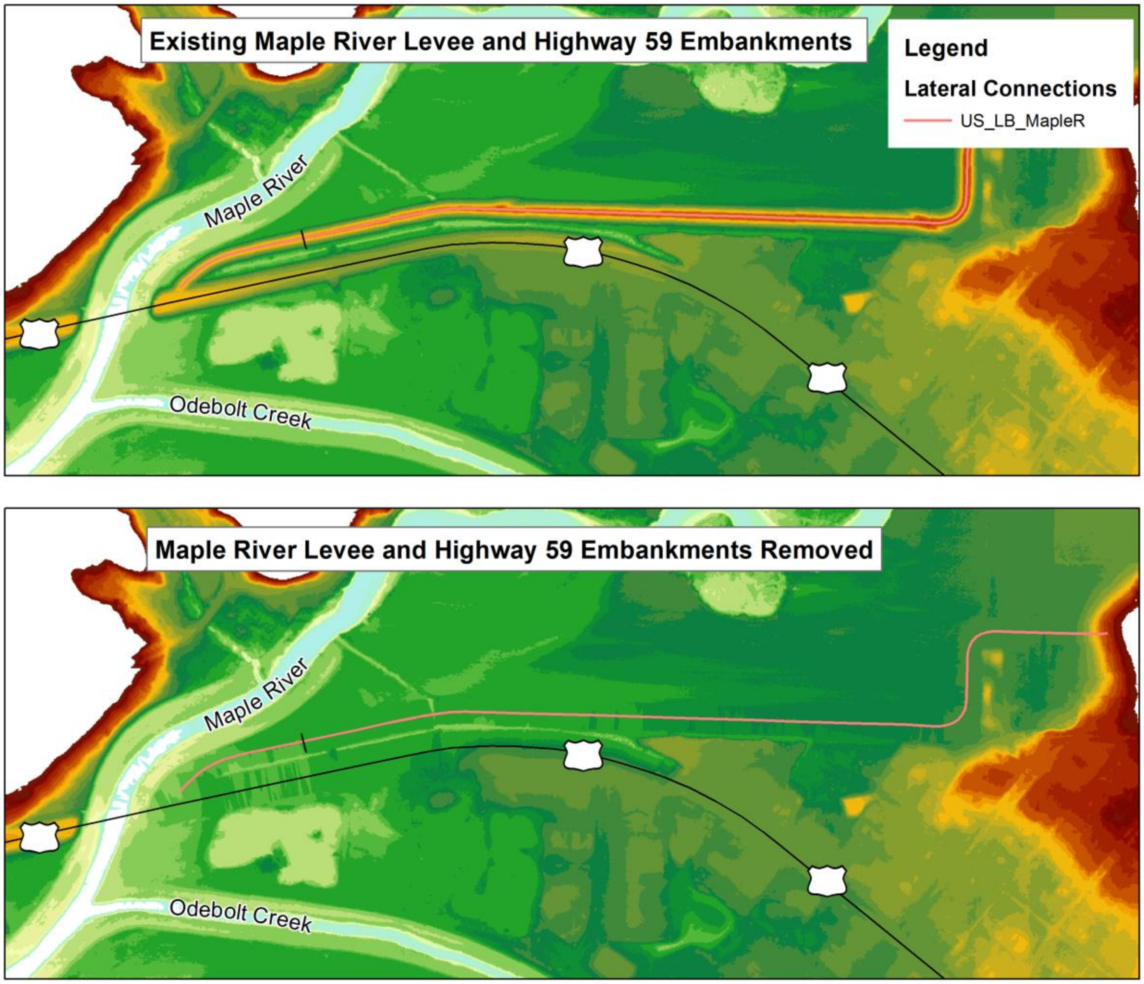

Figure 8. With and without the left bank, upstream Maple River levee and Highway 59 embankment removed.

An additional consideration for Badger Creek was the treatment of undocumented berms along the main channel from the mouth to 7th Street. These berms are not shown on the effective FIRM, and therefore, do not require a Natural Valley analysis. However, when these berms are left in the model as they are represented in the bare earth LiDAR data, they appear to provide some protection to areas along the left and right overbank areas. Similar to the Highway 59 embankment, these berm features were likely not designed and constructed as to provide flood protection. This was communicated to the city and its consulting engineer (John Callen, JEO Consulting Inc.). The city's preference, communicated through John Callen, was to treat the downstream Badger Creek undocumented berms as unaccredited levees and perform a similar Natural Valley analysis for the downstream reach to be consistent with other documented FEMA levee reaches. This will more accurately communicate flood risk along the downstream Badger Creek reach.

Existing top of levee elevations were surveyed by JEO in late 2016 and early 2017. Elevations of a training levee located on the left bank in the upstream reach of Odebolt Creek were extracted from LiDAR. Sources of top of levee elevations are summarized in Table 2.6.

Table 2.5. HEC-RAS geometry scenarios and descriptions

River Model	HEC-RAS Geometry File	Description
Badger Creek	existing_Badger_Cr	Surveyed top of levee elevations have been incorporated into corresponding lateral weir elevations for “US_LB_BadgerCr” (FEMA – 1701001727) and “US_RB_BadgerCr” (FEMA – 1701001726). Undocumented berms, “DS_RB_BadgerCr” and “DS_LB_BadgerCr” have are present in the model using the LiDAR elevation
Badger Creek	LB_Levees_removed_Badger_Cr	Lateral weir elevations of “US_LB_BadgerCr” (FEMA – 1701001727) lowered to near grade elevations by interpolating from embankment toe to toe. Weir coefficient lowered from 2 to 0.35.
Badger Creek	RB_Levees_removed_Badger_Cr	Lateral weir elevations “US_RB_BadgerCr” (FEMA – 1701001726) lowered to near grade elevations by interpolating from embankment toe to toe. Weir coefficient lowered from 2 to 0.35.
Badger Creek	DS_LB_Levees_removed_Badger_Cr	Lateral weir elevations “DS_LB_BadgerCr” lowered to near grade elevations by interpolating from embankment toe to toe. Berm is being treated as non-accredited levee. Weir coefficient lowered from 2 to 0.35.
Badger Creek	DS_RB_Levees_removed_Badger_Cr	Lateral weir elevations “DS_RB_BadgerCr” lowered to near grade elevations by interpolating from embankment toe to toe. Berm is being treated as non-accredited levee. Weir coefficient lowered from 2 to 0.35.
Odebolt Creek	existing_Odebolt_Cr	Top of levee elevations have been extracted from LiDAR data and incorporated into corresponding lateral weir elevations for “US_LB_OdeboltCr” (NLD FC ID – 4705000018)

Odebolt Creek	LB_Levees_removed_Odebolt_Cr	Lateral weir elevations "US_LB_OdeboltCr" (NLD FC ID – 4705000018) lowered to near grade elevations by interpolating from embankment toe to toe. Weir coefficient lowered from 2 to 0.35.
Maple River	existing_Maple_R	Surveyed top of levee elevations have been incorporated into corresponding lateral weir elevations for "US_LB_MapleR" (FEMA ID – 1701001547, NLD FC ID – 4705000161), "DS_LB_MapleR" (NLD FC ID – 44705000161), of "DS_RB_MapleR" (NLD FC ID – 44705000161).
Maple River	LB_US_Levees_removed_Maple_R	Lateral weir elevations of "Maple_River_LB_O" (FEMA ID – 1701001547, NLD FC ID – 4705000161), lowered to near grade elevations by interpolating from embankment toe to toe. Highway 59 roadway embankment was also removed within the 2D grid. Weir coefficient lowered from 2 to 0.35.
Maple River	LB_DS_Levees_removed_Maple_R	Lateral weir elevations of "DS_LB_MapleR" (NLD FC ID – 44705000161) lowered to near grade elevations by interpolating from embankment toe to toe. Weir coefficient lowered from 2 to 0.35.
Maple River	RB_Levees_removed_Maple_R	Lateral weir elevations of "DS_RB_MapleR" (NLD FC ID – 44705000161) lowered to near grade elevations by interpolating from embankment toe to toe. Weir coefficient lowered from 2 to 0.35.

Table 2.6. Sources for existing top of levee elevations

Levee Structure	Top of Levee Elevation Source	Point Spacing
US_LB_MapleR	Surveyed Points by JEO Consulting (April 2017)	25 feet
DS_LB_MapleR	Surveyed Points by JEO Consulting (April 2017)	25 feet
DS_RB_MapleR	Surveyed Points by JEO Consulting (April 2017)	25 feet
US_LB_OdeboltCr	Bare-earth LiDAR provided by Iowa DNR (2009)	1 meter
US_LB_BadgerCr	Surveyed XS End Points by JEO Consulting (April 2017)	300-500 feet, 30-50 feet at levee tie-back
US_RB_BadgerCr	Surveyed XS End Points by JEO Consulting (April 2017)	300-500 feet, 30-50 feet at levee tie-back
DS_LB_BadgerCr	Bare-earth LiDAR provided by Iowa DNR (2009)	1 meter
DS_RB_BadgerCr	Bare-earth LiDAR provided by Iowa DNR (2009)	1 meter

2.8 Ineffective and Storage Areas

Ineffective flow areas were used to account for flow contraction and expansion through the structures. Contraction and expansion coefficients are typically adjusted at bridges to account for any sudden changes in the floodplain conveyance. Since the software doesn't use these coefficients during unsteady simulations they were not defined. Additionally, contraction and expansion losses in the floodplain are simulated using the two-dimensional model.

2.9 Channel Roughness Values

Channel roughness values were selected based on typical values recommended by Chow (1959), and were informed by photographs collected by JEO at each cross-section location. Channel roughness values for Badger Creek ranged from 0.04 to 0.05, with the highest roughness values in the upper reach due to flow obstructions, an example is shown in Figure 9.

Channel roughness values for Odebolt Creek ranged from 0.025 to 0.05, with the highest roughness values occurring in the upper reach, also due to poor channel conditions. Most of the Odebolt Creek reach was straightened and enlarged with a compound shape by USACE Omaha District as part of a 1960's era Section 205 flood control project. The characteristics of the engineered channel, shown in Figure 10, result in a relatively low roughness value of 0.025.

Channel roughness values for Maple River ranged from 0.025 to 0.035. Portions of Maple River were also straightened and enlarged by USACE, resulting in relatively low roughness values.

Spatially-varied Manning's n roughness values for overbank areas were developed based on typical values recommended by Chow (1959), and parameterized by HRLC classifications, shown in Figure 11. Roughness values for each HRLC classification are shown in Table 2.7. Roughness values are typically adjusted in order to match any measured water surface elevations. While there are no observations available, selected roughness values are within typical ranges.

Table 2.7. Roughness coefficients corresponding to high-resolution land cover classifications.

Land Cover Description	Manning's n Roughness
Barren / Fallow	0.02
Roads /Impervious	0.02
Shadow / No data	0.02
Soybeans	0.045
Structures	0.5
Water	0.035
Wetland	0.1
Coniferous Forest	0.15
Corn	0.045
Deciduous Medium	0.1
Deciduous Short	0.1
Deciduous Tall	0.1
Grass 1	0.03
Grass 2	0.03

Figure 9. Badger Creek channel conditions in the upper reach. (photo provided by JEO)

Figure 10. Typical Odebolt Creek channel conditions in downstream reach. (photo provided by JEO)

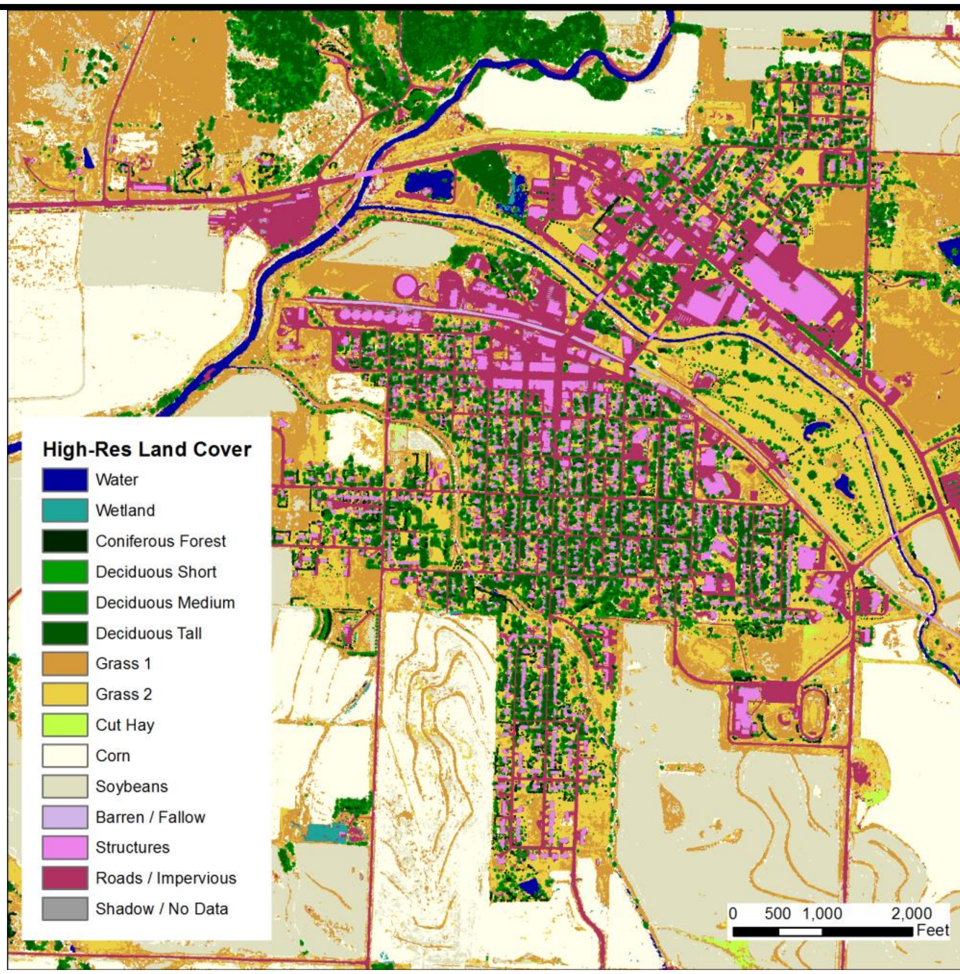

Figure 11. High resolution land cover data provided by Iowa DNR.

2.10 Other Model Input

Downstream reach lengths of the 1D model cross-sections were estimated using the HEC-GeoRAS extension for ArcGIS 10.1. Bank elevations were set based on visual inspection of the cross-section station elevation data and aerial imagery.

Culvert dimensions and invert elevations were assumed in some locations along Odebolt Creek. Geometries for these structures were assumed in order to incorporate a generalized culvert structure to allow backwater to fill ditches behind roadway embankments.

2.11 Floodway Analysis and Mapping

FEMA guidance does not current specify a floodway analysis methodology for 1D/2D unsteady state hydraulic models. To develop the floodway boundaries, a separate 1D, steady state model was produced by JEO to perform a floodway analysis for each stream.

The 1D/2D models were used to develop the floodplain, while the 1D models were utilized for the floodway delineation. It is important to note the 1D floodway models were calibrated within a tolerance of the varying water surface elevation produced by 1D/2D models, following guidance provided on behalf of FEMA. The cross-sections included in the DFIRM spatial database originate from the 1D floodway models, and do not represent a single water surface elevation across their length, nor do they represent the regulatory water surface elevation.

To complete the floodway analysis, a baseline HEC-RAS version 5.0.5 model was created for each stream and calibrated to the water surface profile for the 1% annual chance event from the existing conditions 1D/2D hydraulic model. Guidance was provided on behalf of FEMA that set guidelines for the calibration targets that must be met by the calibrated base model; in general the expectation was the water surface elevations calculated by the calibrated 1D model should be within 0.1 feet of the water surface elevations determined by the 1D/2D model. Details of this guidance are outlined in a guidance memo provided as an Appendix to the technical memos developed describing the approach to base model calibration and floodway development. The technical memos are provided as attachments to this report.

Using Arc-GIS and HEC-GeoRAS software a base geometry file was created which included the 1D portion of the existing 1D/2D model cross sections with the geometry of each cross section extended to high ground resulting in a complete 1D geometry for the 1D steady state model. For circumstances where a stream has non-accredited but hydraulically significant levee segments, for the purposes of the base model calibration the 'with levee' existing conditions run was used. However, for the floodway analysis the levee segment topography was included in the geometry, but it was assumed floodplain area landward of the levee embankments is effective flow as is required for a 1D natural valley analysis according to Section 6.12.2 of the February 2019 FEMA document "Guidance for Flood Risk Analysis and Mapping – Levees". This guidance is provided in Appendix A of the detailed floodway analysis technical memos attached.

The floodway analysis was completed using the guidelines provided in the February 2019 FEMA document "Guidance for Flood Risk Analysis and Mapping – Levees". Using the calibrated 1D geometry model as the base model, a natural valley floodway analysis was completed to determine an equal conveyance reduction floodway. Floodway encroachments were placed riverward of the levee system segments, where applicable and feasible within standard surcharge requirements of the floodway analysis. Floodway mapping was developed based on RAS Mapper floodway outputs coordinated with floodplain boundaries developed based on the 1D/2D models.

For a more detailed description of the floodway analysis including base model calibration approach and floodway results, see the technical memorandum for each stream provided as an attachment to this report.

2.12 Floodplain Boundaries

Floodplain boundaries were developed by exporting maximum water surface elevation rasters from 1D/2D hydraulic model for the 1- and 0.2-percent annual chance flows, and post-processed using ArcGIS. Additional maximum water surface elevation rasters were exported for the 1-percent annual chance flow for each Natural Valley configuration, discussed previously.

The LiDAR terrain for the 1-percent annual chance floodplain development incorporated terrain models for Natural Valley configurations, which had the corresponding levee embankments removed from the terrain. The LiDAR terrain for the 0.2-percent annual chance floodplain development was the base LiDAR data.

The water surface elevation rasters for the 1- and 0.2-percent annual chance flows were intersected with the corresponding LiDAR terrains and reclassified to produce a binary raster of dry versus wet areas. Wet areas were converted to a simplified polygon in ArcGIS. Further post-processing was completed to remove dry islands with areas less than 2,500 square feet, and small disconnected polygons (STARR II, 2019). Examples of these products are shown in Section 4.

2.13 Calibration

There were no surveyed high water marks available for calibration.

Simulations of the Maple River model required calibration of peak discharges just downstream of Odebolt and Badger Creeks by iteratively adjusting the timing of inflow hydrographs at each tributary confluence.

3. OTHER SUPPORTING INFORMATION

4. RESULTS

Several model simulations were created for each flood frequency quantile and natural valley geometry configuration, and are summarized in Table 4.1.

The base products produced by the 1D/2D models were used to develop floodplain mapping, base flood elevation (BFE) polylines, and profiles. Due to the 2D gridded model, water surface elevations in the overbank areas may be different than nearby main channel water surface elevations. In addition to this consideration, the 1D/2D model simulations using natural valley configurations result in abrupt transitions in water surface elevations from the main channel to the overbank areas landward of the levees. The cross-sections included in the spatial database were derived from the 1D floodway model and not the 1D/2D model, and do not represent a single water surface elevation across the floodplain, nor do they represent the regulatory water surface elevation. It is important to evaluate the corresponding BFEs and the particular modeling approach along with the floodplain mapping while interpreting the FIRM.

Floodplain boundaries were developed by exporting maximum water surface elevation rasters from the 1D/2D hydraulic model for the 1- and 0.2-percent annual chance flows, and post-processed using ArcGIS. Additional maximum water surface elevation rasters were exported for the 1-percent annual chance flow for each Natural Valley configuration, discussed previously, and used to create a mosaic of maximum water surface elevations for the 1-percent annual chance flows. A mosaic of these water surface elevations is shown in Figure 12. Each stream's maximum water surface elevation raster for each 1-percent annual chance flow simulation is located in corresponding [Stream]\Supplemental_Data folders. The composite mosaic of maximum water surface elevation for all streams and model simulations is located in Spatial_Files\Supplemental_Data.

The LiDAR terrain for the 1-percent annual chance floodplain development incorporated terrain models for Natural Valley configurations, which had the corresponding levee embankments removed from the terrain. The LiDAR terrain for the 0.2-percent annual chance floodplain development was the base LiDAR data.

The maximum water surface elevation rasters for the 1- and 0.2-percent annual chance flows were intersected with the corresponding LiDAR terrains and reclassified to produce a binary raster of dry versus wet areas. Wet areas were converted to a simplified polygon in ArcGIS. Further post-processing was completed to remove dry islands with areas less than 2,500 square feet, and small disconnected polygons (STARR II, 2019). Additional narrow dry islands along the main channels were present due to the LiDAR terrain, and were manually removed from the floodplain polygon if they overlapped the floodway. The boundaries of the inundation polygons were further smoothed using the "Smooth Polygon" ArcGIS toolbox that utilizes a PAEK algorithm with a tolerance of 25 feet. Examples of these products are shown in Section 4.

Inundation extent after post-processing to fill small holes and remove small disconnected polygons is shown overlain on a rectified paper FIRM in Figure 13. This same post-processed extent is shown overlain on an aerial photo in Figure 14.

BFEs were developed in ArcGIS using the same maximum water surface elevation mosaic raster of 1-percent annual chance flow 1D/2D model simulations. Water surface contour lines were generated at 0.5 foot intervals using the ArcGIS Spatial Analyst toolbox. Locations where dry islands were removed from the 1-percent annual chance polygon required manual editing to create a continuous contour line. Some portions of the water surface contour lines were simplified by manually removing polyline vertices, but leaving the general alignment of the contour line intact. The contour lines were snapped to the boundary of the proposed Zone AE polygon. The edited contour lines were then dissolved to create multipart features for each BFE and imported into the DFIRM spatial database.

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Table 4.1. Simulation runs for each stream

Stream	Plan Name	Geometry File	Flow File	Description
Badger Creek	Badger_Proposed1pct_existing	existing_Badger_Cr	Badger_proposed1pct	Base model run for 1% AEP
	Badger_Proposed0.2pct_existing	existing_Badger_Cr	Badger_proposed0.2pct	Base model run for 0.2% AEP
	Badger_Proposed0.5pct_existing	existing_Badger_Cr	Badger_proposed0.5pct	Base model run for 0.5% AEP
	Badger_Proposed2pct_existing	existing_Badger_Cr	Badger_proposed2pct	Base model run for 2% AEP
	Badger_Proposed4pct_existing	existing_Badger_Cr	Badger_proposed4pct	Base model run for 4% AEP
	Badger_Proposed10pct_existing	existing_Badger_Cr	Badger_proposed10pct	Base model run for 10% AEP
	Badger_Proposed20pct_existing	existing_Badger_Cr	Badger_proposed20pct	Base model run for 20% AEP
	Badger_Proposed50pct_existing	existing_Badger_Cr	Badger_proposed50pct	Base model run for 50% AEP
	Badger_Proposed1pctPLUS_existing	existing_Badger_Cr	Badger_proposed1pctPLUS	Base model run for 1%Plus AEP
	Badger_proposed_1pct_LB_Levees_removed	LB_Levees_removed_Badger_Cr	Badger_proposed1pct	Natural valley run removing US_LB_BadgerCr from the geometry
	Badger_proposed_1pct_RB_Levees_removed	RB_Levees_removed_Badger_Cr	Badger_proposed1pct	Natural valley run removing US_RB_BadgerCr from the geometry
	Badger_proposed_1pct_DS_LB_Levees_removed	DS_LB_Levees_removed_Badger_Cr	Badger_proposed1pct	Natural valley run removing DS_LB_BadgerCr from the geometry
	Badger_proposed_1pct_DS_RB_Levees_removed	DS_RB_Levees_removed_Badger_Cr	Badger_proposed1pct	Natural valley run removing DS_RB_BadgerCr from the geometry
	Badger_Proposed1pct_existing_FULLMOM	existing_Badger_Cr	Badger_proposed1pct	Sensitivity run of base model for 1% AEP, using full momentum equations rather than diffusive wave

	Badger_1D_1pct	existing_Badger_Cr_1D	Existing	1D model baseline run to compare with 1D/2D hydraulic model baseline
	Badger_1D_1pct_FWY	existing_Badger_Cr_FWY	FWY	1D model baseline run calibrated to 1D/2D hydraulic model baseline. Also includes Floodway encroachments

Table 4.1. Continued - Simulation runs for each stream

Stream	Plan Name	Geometry File	Flow File	Description
Odebolt Creek	Odebolt_proposed1pct_existing	existing_Odebolt_Cr	Odebolt_proposed1pct	Base model run for 1% AEP
	Odebolt_proposed0.2pct_existing	existing_Odebolt_Cr	Odebolt_proposed0.2pct	Base model run for 0.2% AEP
	Odebolt_proposed0.5pct_existing	existing_Odebolt_Cr	Odebolt_proposed0.5pct	Base model run for 0.5% AEP
	Odebolt_proposed2pct_existing	existing_Odebolt_Cr	Odebolt_proposed2pct	Base model run for 2% AEP
	Odebolt_proposed4pct_existing	existing_Odebolt_Cr	Odebolt_proposed4pct	Base model run for 4% AEP
	Odebolt_proposed10pct_existing	existing_Odebolt_Cr	Odebolt_proposed10pct	Base model run for 10% AEP
	Odebolt_proposed20pct_existing	existing_Odebolt_Cr	Odebolt_proposed20pct	Base model run for 20% AEP
	Odebolt_proposed50pct_existing	existing_Odebolt_Cr	Odebolt_proposed50pct	Base model run for 50% AEP
	Odebolt_proposed1pctPLUS_existing	existing_Odebolt_Cr	Odebolt_proposed1pctPLUS	Base model run for 1%Plus AEP
	Odebolt_proposed_1pct_LB_Levees_removed	LB_Levees_removed_Odebolt_Cr	Odebolt_proposed1pct	Natural valley run removing US_LB_OdeboltCr from the geometry

	Odebolt_proposed1pct_existing_FullMom	existing_Odebolt_Cr	Odebolt_proposed1pct	Sensitivity run of base model for 1% AEP, using full momentum equations rather than diffusive wave
	Existing_Odebolt_1D_053119	Existing_Odebolt_Cr_1D_053119	Existing_1_pct	1D model baseline run to compare with 1D/2D hydraulic model baseline
	Existing_Odebolt_1D_FWY_0610	Existing_Odebolt_Cr_1D_FWY	Existing_1_pct_FWY	1D model baseline run calibrated to 1D/2D hydraulic model baseline. Also includes Floodway encroachments

Table 4.1. Continued - Simulation runs for each stream

Stream	Plan Name	Geometry File	Flow File	Description
Maple Creek	Maple_proposed1pct_existing	existing_Maple_Cr	Maple_proposed1pct	Base model run for 1% AEP
	Maple_proposed0.2pct_existing	existing_Maple_Cr	Maple_proposed0.2pct	Base model run for 0.2% AEP
	Maple_proposed0.5pct_existing	existing_Maple_Cr	Maple_proposed0.5pct	Base model run for 0.5% AEP
	Maple_proposed2pct_existing	existing_Maple_Cr	Maple_proposed2pct	Base model run for 2% AEP
	Maple_proposed4pct_existing	existing_Maple_Cr	Maple_proposed4pct	Base model run for 4% AEP
	Maple_proposed10pct_existing	existing_Maple_Cr	Maple_proposed10pct	Base model run for 10% AEP
	Maple_proposed20pct_existing	existing_Maple_Cr	Maple_proposed20pct	Base model run for 20% AEP
	Maple_proposed50pct_existing	existing_Maple_Cr	Maple_proposed50pct	Base model run for 50% AEP
	Maple_proposed1pctPLUS_existing	existing_Maple_Cr	Maple_proposed1pctPLUS	Base model run for 1%Plus AEP
	Maple_proposed_1pct_LB_DS_Levees_removed	LB_DS_Levee_removed_Maple_R	Maple_proposed1pct	Natural valley run removing DS_LB_MapleR from the geometry

	Maple_proposed_1pct_LB_US_Levees_removed	LB_US_Levee_removed_Maple_R	Maple_proposed1pct	Natural valley run removing Maple_River_LB_O from the geometry
	Maple_proposed_1pct_RB_DS_Levees_removed	RB_Levees_removed_Maple_R	Maple_proposed1pct	Natural valley run removing DS_RB_MapleR from the geometry
	Maple_proposed1pct_existing_FULLMOM	existing_Maple_Cr	Maple_proposed1pct	Sensitivity run of base model for 1% AEP, using full momentum equations rather than diffusive wave
	Existing_1D	Exisintg_Maple_1D	Existing	1D model baseline run to compare with 1D/2D hydraulic model baseline
	Existing_1D_FWY	Existing_Maple_1D_FWY	FWY	1D model baseline run calibrated to 1D/2D hydraulic model baseline. Also includes Floodway encroachments

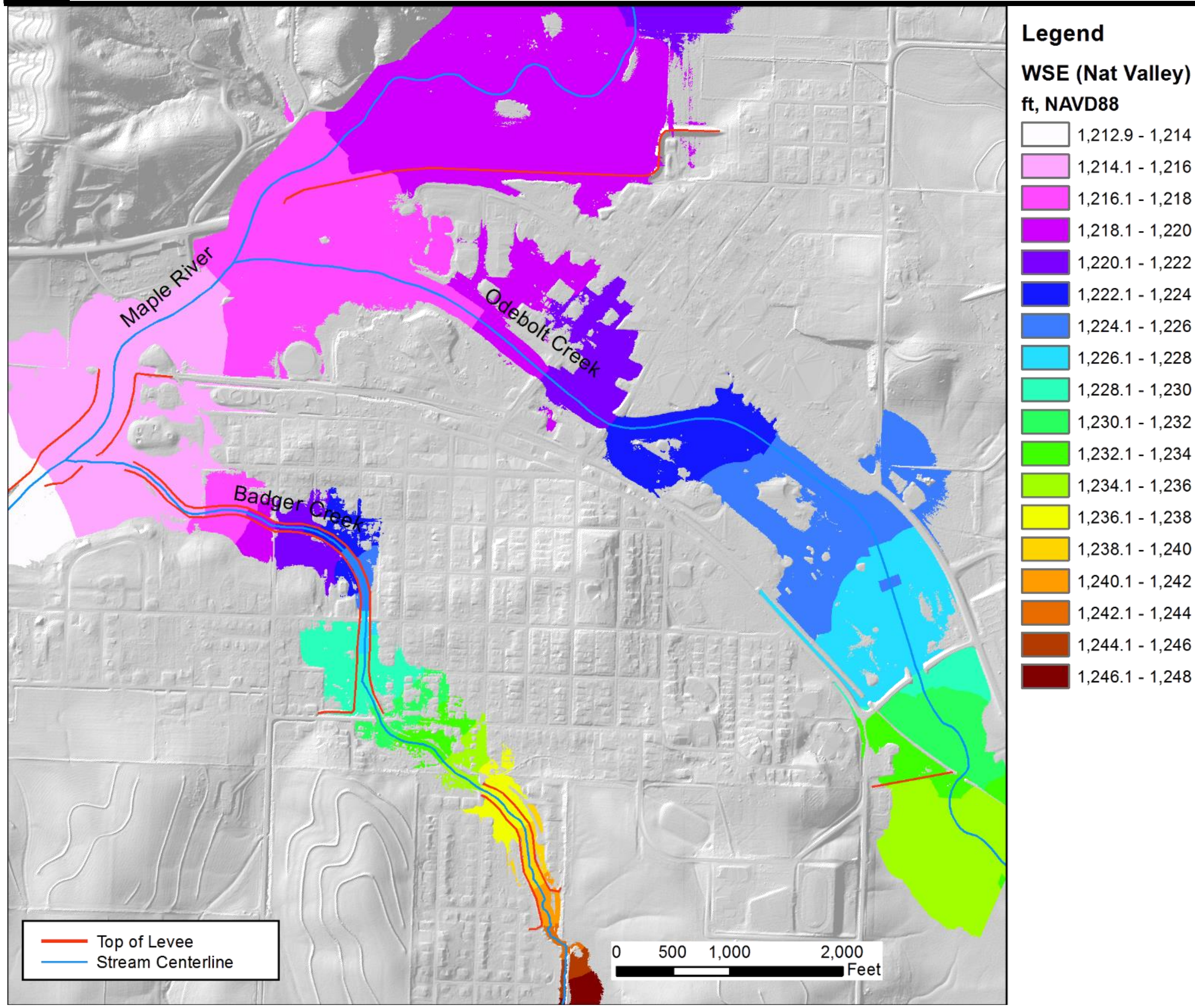

Figure 12. Maximum simulated water surface elevations for each 1percent annual chance flow simulation.

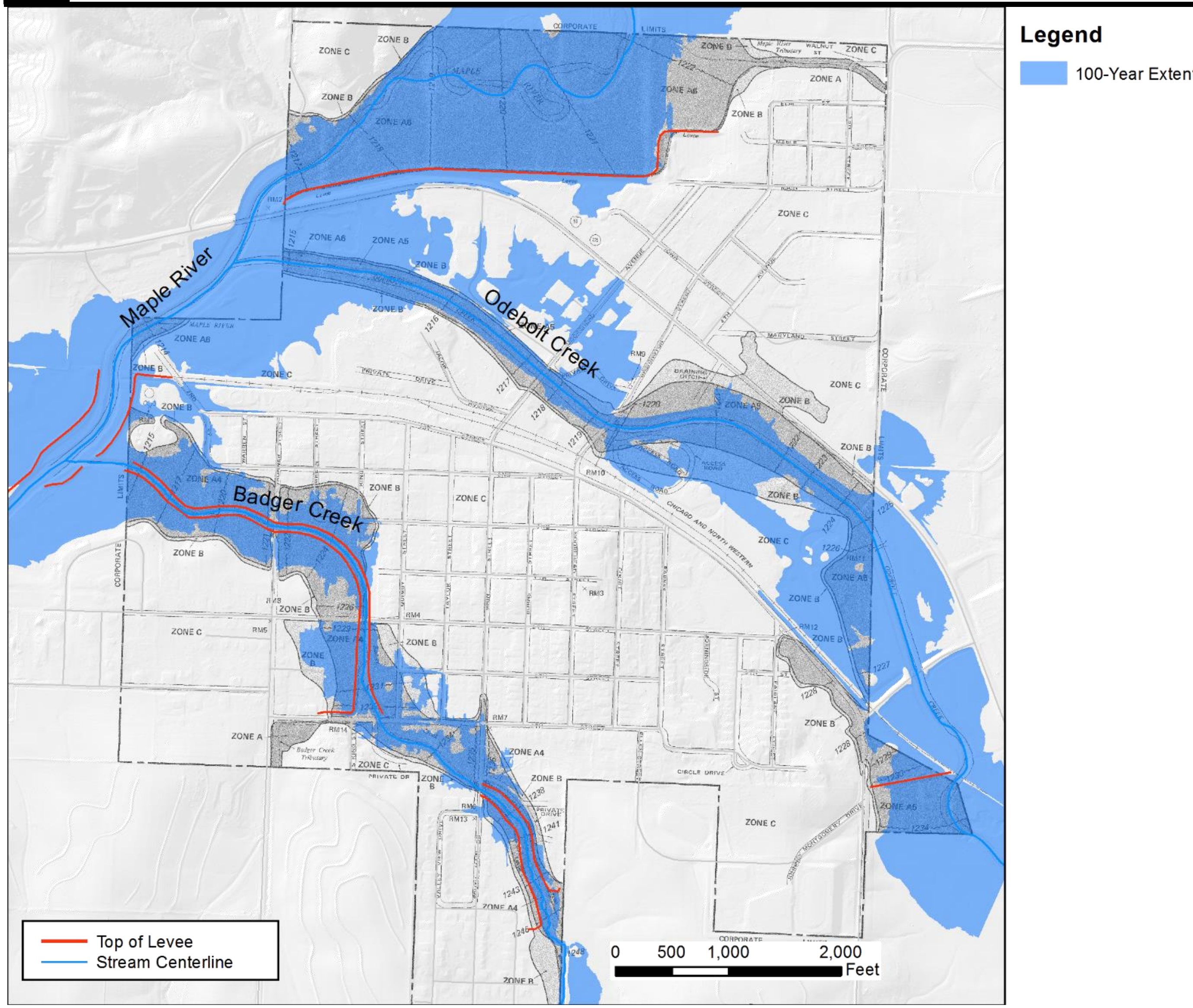

Figure 13. Inundation extent for the 1-percent annual chance simulations.

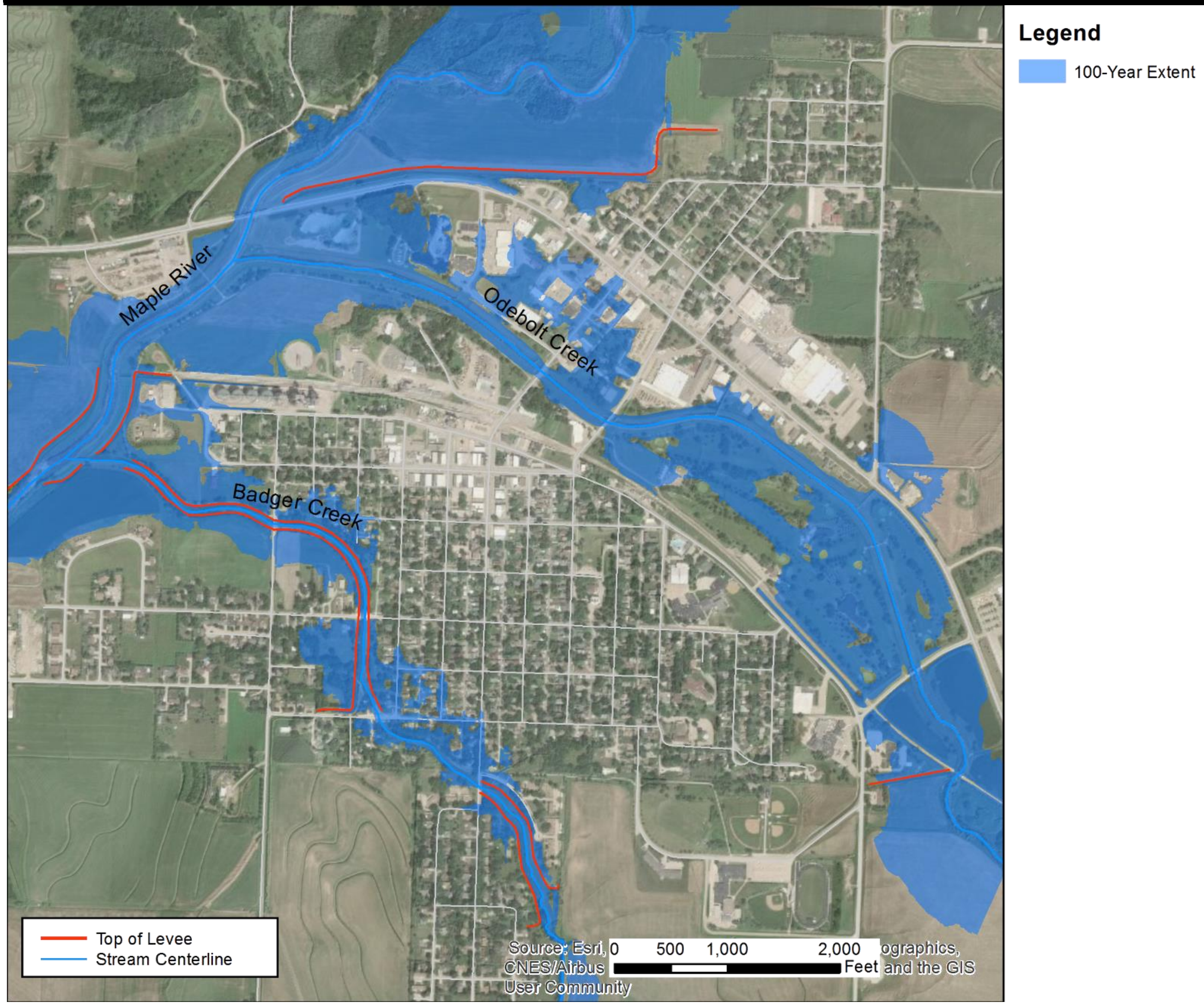

Figure 14. Inundation extent for the 1-percent annual chance simulations overlain on an aerial photograph.

5. EFFECTIVE ELEVATION COMPARISON

A comparison of the new 1-percent annual chance water surface profile with the effective profile at lettered cross sections and base flood elevation locations derived from the effective FIRM are shown for Badger Creek in Figure 15. Similar comparison plots for Maple River and Odebolt Creek are shown in Figure 16 and Figure 17, respectively. Overall, the simulated profile decreased in elevation compared to the effective study for Badger Creek This is likely due to a combination of updated regional regression equations and changes in channel geometry. The simulated profile for Odebolt Creek was slightly higher along most of the reach. This is likely a result of increased flows due to updated regional regression equations. The simulated profile for Maple River was slightly lower than the effective study along most of the reach within the corporate limits of Ida Grove. It appears the previous study had a steeper water surface slope relative this this new study.

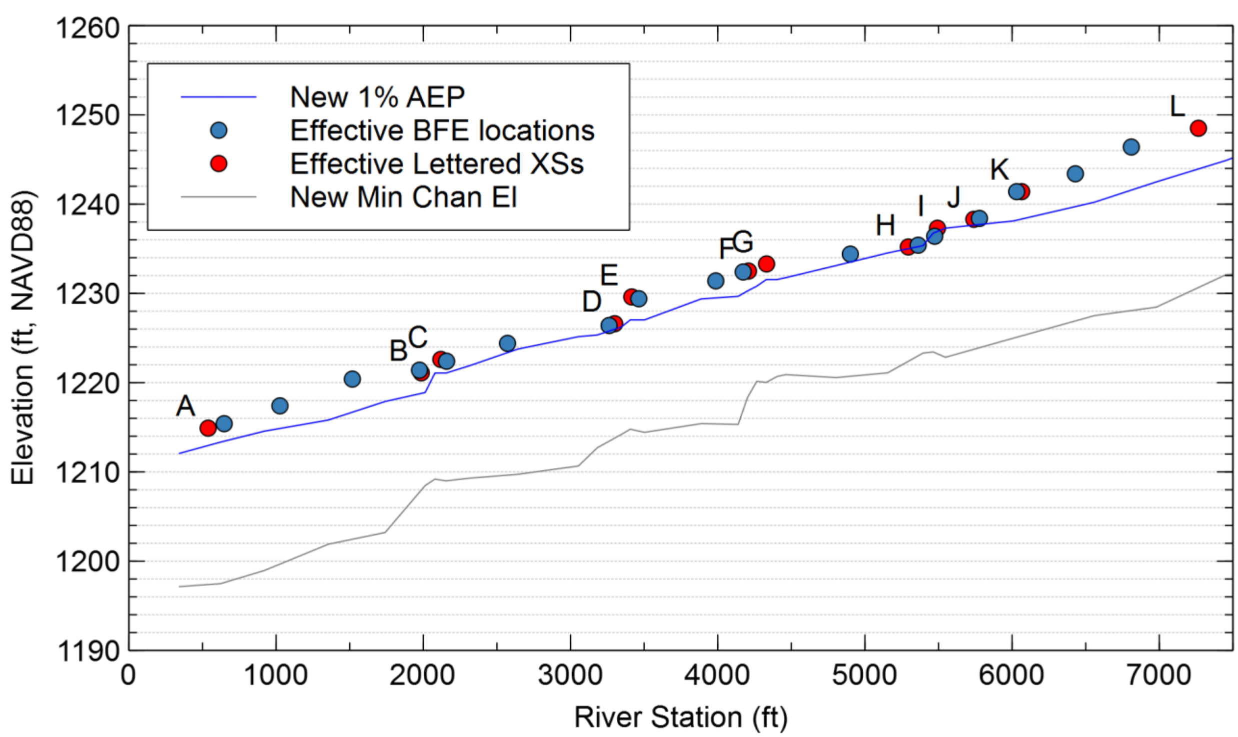

Figure 15. Comparison of effective and proposed 1-percent annual chance water surface elevations for Badger Creek.

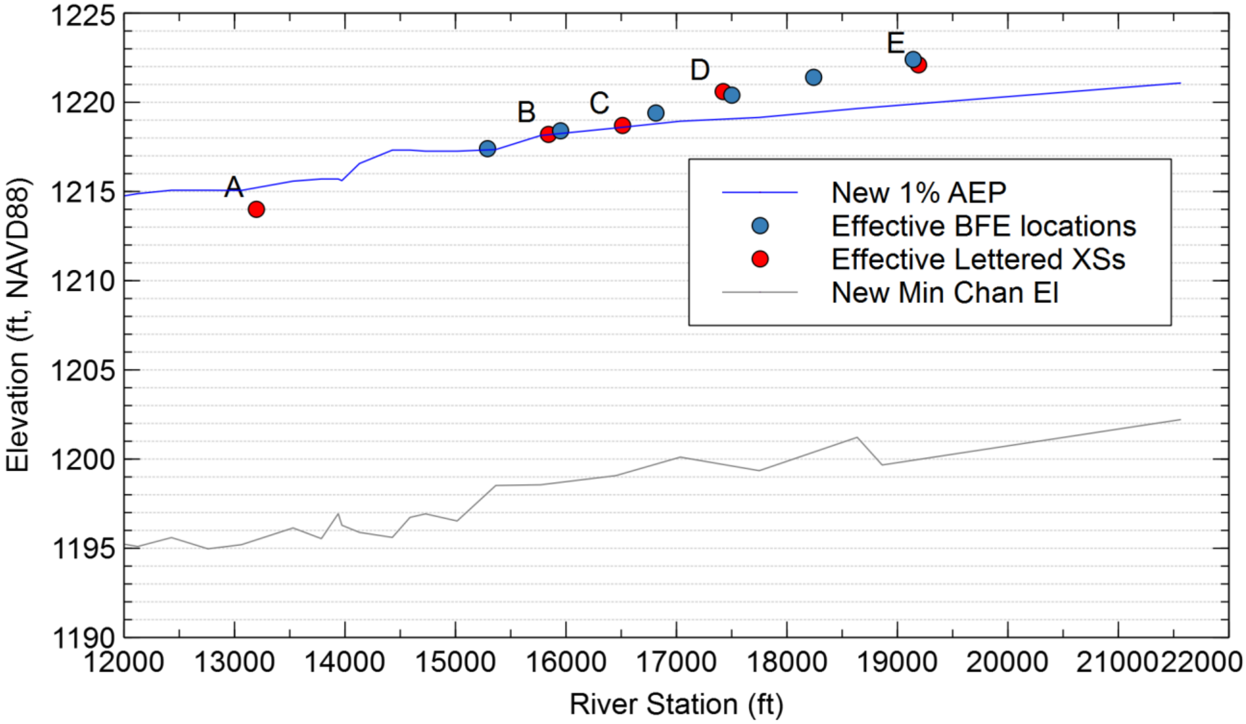

Figure 16. Comparison of effective and proposed 1-percent annual chance water surface elevations for Maple River.

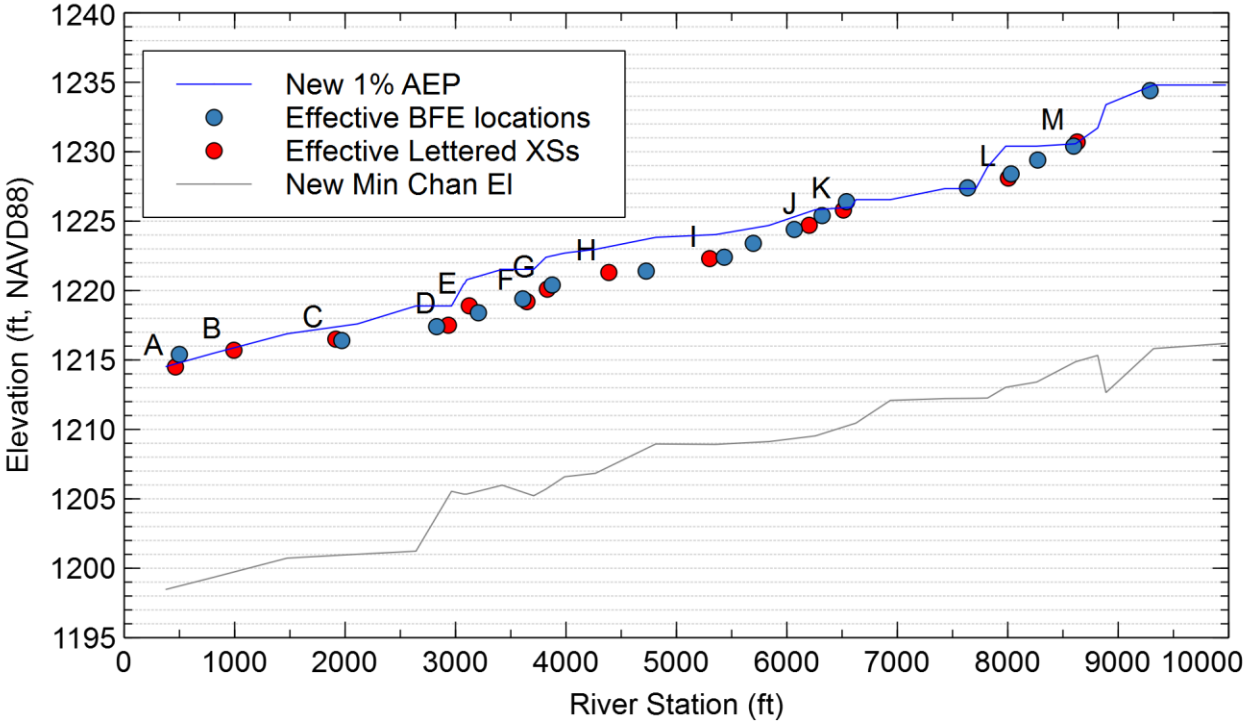

Figure 17. Comparison of effective and proposed 1-percent annual chance water surface elevations for Odebolt Creek.

6. REFERENCES

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7. ATTACHMENTS – FLOODWAY ANALYSES BY JEO CONSULTING GROUP INC.

7.1 Badger Creek Floodway Analysis

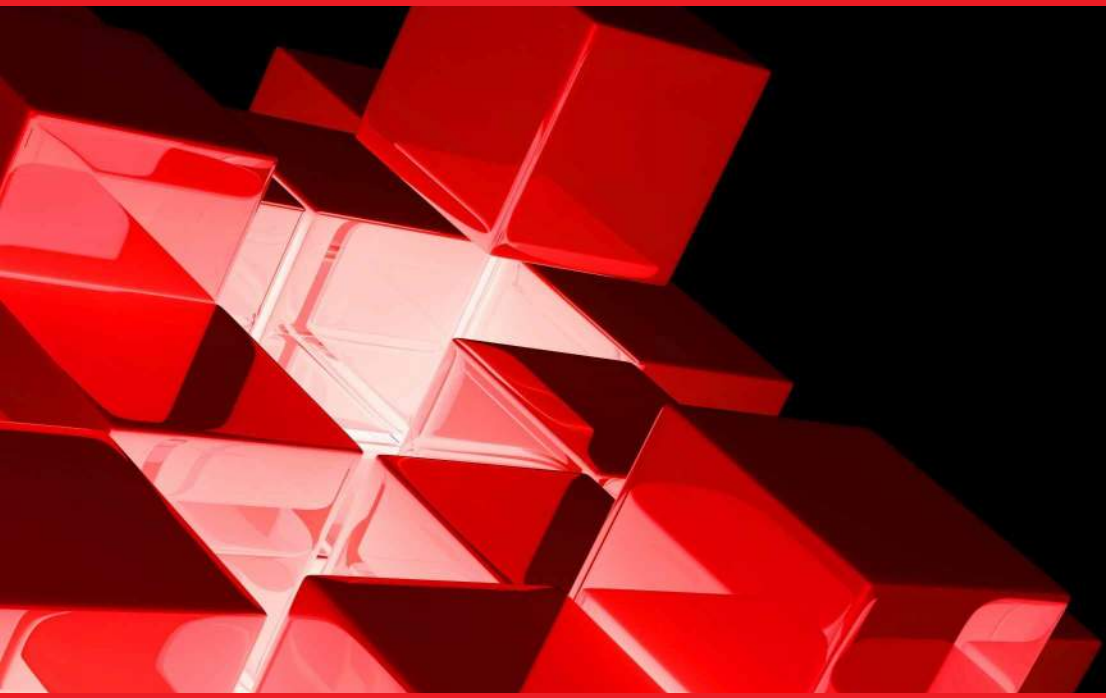
7.2 Odebolt Creek Floodway Analysis

7.3 Maple River Floodway Analysis

JEO CONSULTING GROUP INC

Badger Creek Floodway Analysis

Ida Grove, Iowa



June 2019

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Appendices

Appendix A –

- STARR II Guidance Memo
- FEMA guidance document "Guidance for Flood Risk Analysis and Mapping – Levees", February, 2019 (digital only)
- cHECk-RAS Floodway Check

Appendix B – HEC-RAS Model (digital only)

1.0 PROJECT BACKGROUND

The City of Ida Grove, Iowa has been coordinating with Iowa Flood Center (IFC), the Iowa Department of Natural Resources (IDNR) and FEMA Region VII (FEMA) to complete a revised flood study for Maple River, Odebolt Creek, and Badger Creek in the City of Ida Grove as part of a countywide Digital Flood Insurance Rate Map (DFIRM) update for Ida County, Iowa. As part of this process, Iowa Flood Center has developed a detailed hydraulic model for Ida Grove using a 1D/2D modeling approach. This hydraulic model has been finalized through FEMA's independent technical review process. A separate 1D, Steady State model is being produced by JEO to perform a floodway analysis. The purpose of this technical memo is to describe the technical procedures used for the development of the floodway analysis for Badger Creek.

2.0 METHODOLOGY AND MODELING

2.1 Base Model Development and Calibration

A baseline 1D, steady state HEC-RAS version 5.0.5 model was created for Badger Creek and calibrated to the water surface profile for the 1% annual chance event from the existing conditions 1D/2D hydraulic model. Using Arc-GIS and HEC-GeoRAS software a base geometry file was created which included the 1D portion of the existing 1D/2D model cross sections with the geometry of each cross section extended to high ground resulting in a complete 1D geometry for the 1D steady state model. Badger Creek has multiple non-accredited but hydraulically significant levee segments on both banks; for calibration purposes the 'with levee' geometry was used with ineffective flow areas landward of the levees. For the purposes of the floodway analysis the levee segment topography was included in the geometry, but it was assumed floodplain area landward of the levee embankments is effective flow as is required for a 1D natural valley analysis according to Section 6.12.2 of the February 2019 FEMA document "Guidance for Flood Risk Analysis and Mapping – Levees". This guidance is provided in Appendix A. Alignment and location of the levee embankment locations are show on Figures 1 and 2.

Analysis was then completed to determine an appropriate calibration tolerance between the 1% annual chance water surface elevation (WSE) from the equivalent 1D model and the existing 1D/2D floodplain model using guidance provided by STARR II which is provided in Appendix A. The analysis compared the WSE of all secondary flow areas in the floodplain to the main channel and the portion of the flood volume conveyed by the secondary floodplain flow paths. Floodplain flow area water surface elevations were calculated using tools within Arc-GIS. It was determined all cross-sections fall into the categories of Case 2a and Case 2b and therefore should be calibrated to a tolerance of 0.1 feet of the main channel average WSE. See appendix A for the complete STARR II memo and case descriptions.

A 1D steady state run was completed using the IFC reported peak flow of 3,500 cfs and the same downstream normal depth boundary condition of 0.004 ft/ft used in the 1D/2D hydraulic model. Model calibration was then achieved through adjustments to manning's n values and ineffective flow area locations on a cross-section by cross-section basis. Results from the calibration effort are shown in Table 1.

2.2 Floodway Analysis

A floodway analysis was completed based on the guidelines provided in the February 2019 FEMA document "Guidance for Flood Risk Analysis and Mapping – Levees" and further guidance from STARR II. Using the calibrated 1D geometry model as a starting base model, a floodway analysis was completed to determine an equal conveyance reduction floodway. Due to the multiple levee embankments involved and the approach to the natural valley analysis completed by IFC for the 1D/2D model, completing the sequencing required by the FEMA guidance was impractical. After coordination with STARR II, the approach taken was to remove ineffective flow locations which were used to calibrate the 1D model to the existing conditions 1D/2D model to create a natural valley base model for the floodway for the entire reach of Badger Creek. To facilitate completing the analysis in accordance with standard floodway encroachment practices, bank stations were moved from the locations used for the 1D/2D hydraulic model. In some cases, with the 1D/2D model bank stations were placed on top of the levee embankments; for the 1D steady flow model these were moved into the channel flow region and lowered in elevation to provide flexibility with placement of encroachment stations. Analysis results included floodway surcharges ranging from 0.00 feet to 1.00 feet. Results of the analysis are shown in Table 2.

The floodway check was run in cHECk-RAS. Results of the floodway check are provided in Appendix A. No changes to the floodway analysis were made in response to cHECk-RAS comments from the floodway check.

Table 1 - Model Calibration Results

River Station	LOB WSE	ROB WSE	Target WSE from 1D/2D Model	Max WSE Difference*	Main Channel Peak Flow (cfs)	% Peak Flow Conveyed by Secondary Channel	STARR II Memo Case	Calibrated 1D Max WSE	1D WSE Difference**
9781.776	1256.47	0.00	1256.47	0.00	3492	0.00	2a	1256.56	-0.09
9169.787	1254.86	1254.80	1255.03	-0.23	3322	0.05	2a	1255.07	-0.04
8577.904	1252.15	1251.09	1250.69	1.46	3400	0.03	2a	1250.60	0.09
7971.212	0.00	1248.15	1248.19	-0.04	3203	0.08	2a	1248.27	-0.08
7452.984	0.00	1245.95	1244.89	1.06	3204	0.08	2a	1244.89	0.00
6979.874	0.00	1242.33	1242.49	-0.16	3417	0.02	2a	1242.41	0.08
6560.169	0.00	1240.76	1240.22	0.54	3432	0.02	2a	1240.19	0.03
6010.188	0.00	1238.05	1238.12	-0.07	3410	0.03	2a	1238.08	0.04
5546.898	0.00	1237.16	1237.31	-0.15	3346	0.04	2a	1237.36	-0.05
5463.547	0.00	1236.92	1236.75	0.17	3345	0.04	2a	1236.75	0.00
5394.885	0.00	1236.17	1235.35	0.82	3344	0.04	2a	1235.41	-0.06
5154.368	1234.80	1234.42	1234.53	0.27	3342	0.05	2a	1234.54	-0.01
4805.449	1233.02	1232.53	1233.12	-0.59	3311	0.05	2a	1233.04	0.08
4462.235	1231.69	1231.63	1231.76	-0.13	3274	0.06	2a	1231.69	0.07
4402.619	1231.54	1231.35	1231.46	-0.11	3279	0.06	2a	1231.48	-0.02
4330.115	1231.53	1230.63	1231.55	-0.92	3294	0.06	2a	1231.55	0.00
4265.133	1229.55	1230.23	1230.81	-0.58	3294	0.06	2a	1230.78	0.03
4202.005	1229.58	1229.68	1230.27	-0.59	3282	0.06	2a	1230.24	0.03
4139.126	1229.52	1229.42	1229.67	-0.25	3262	0.07	2a	1229.70	-0.03
3887.453	1227.71	1228.99	1229.38	-1.67	3306	0.06	2a	1229.31	0.07
3502.323	0.00	1227.79	1226.77	1.02	3299	0.06	2a	1226.87	-0.10
3406.087	0.00	1227.44	1227.03	0.41	3292	0.06	2a	1227.02	0.01
3344.186	0.00	0.00	1226.23	0.00	3292	0.06	2a	1226.27	-0.04
3181.879	0.00	0.00	1225.34	0.00	3291	0.06	2a	1225.28	0.06
3052.657	0.00	0.00	1225.15	0.00	3289	0.06	2a	1225.10	0.05
2641.649	1220.50	1222.37	1223.76	-3.26	3288	0.06	2a	1223.74	0.02
2310.271	1220.50	1220.68	1221.87	-1.37	3281	0.06	2a	1221.80	0.07
2155.534	1220.50	1220.82	1220.66	-0.16	3074	0.12	2a	1220.59	0.07
2078.819	1220.50	1220.62	1221.08	-0.58	2918	0.17	2a	1221.18	-0.10
2012.315	1218.52	1218.52	1218.89	-0.37	3161	0.10	2a	1218.87	0.02
1740.38	1217.29	1216.68	1217.89	-1.21	3056	0.13	2a	1217.83	0.06
1354.907	1216.18	1214.75	1215.81	-1.06	3021	0.14	2a	1215.88	-0.07
919.6187	1214.68	1214.30	1214.56	-0.26	2699	0.23	2b	1214.46	0.10
622.9671	1213.43	1214.29	1213.34	0.95	2970	0.15	2a	1213.39	-0.05
343.8126	1213.09	0.00	1212.08	1.01	3031	0.13	2a	1212.14	-0.06

*WSE difference between the target WSE from the 1D/2D model and the left overbank (LOB) or right overbank (ROB). Reported value is the greater of the two.

**WSE difference between the target WSE from the 1D/2D model and the calibrated 1D maximum WSE. WSE Tolerance for all cross sections was +/- 0.1 feet

Table 2 - Floodway Results

River Sta	Profile	W.S. Elev	Prof Delta WS	E.G. Elev	Top Wdth Act	Q Left	Q Channel	Q Right	Enc Sta L	Ch Sta L	Ch Sta R	Enc Sta R
		(ft)	(ft)	(ft)	(ft)	(cfs)	(cfs)	(cfs)	(ft)	(ft)	(ft)	(ft)
9781.776	Floodway	1257.4	0.81	1257.8	70.0	263.1	2949.7	287.2	655.0	666.6	715.5	725.0
9169.787	Floodway	1256.0	0.95	1256.6	75.0	200.3	3269.7	30.0	475.0	490.7	538.0	550.0
8577.904	Floodway	1250.8	0.21	1252.4	37.1	0.1	3499.9		468.9	469.0	506.0	506.0
7971.212	Floodway	1248.3	0.02	1249.0	85.0	129.0	2696.5	674.5	375.0	382.2	416.2	460.0
7452.984	Floodway	1244.9	0.00	1246.4	43.0	0.0	3500.0	0.0	398.0	398.0	441.0	441.0
6979.874	Floodway	1242.4	0.01	1243.1	100.9	97.5	3196.0	206.5	331.5	343.3	398.4	432.4
6560.169	Floodway	1240.1	0.05	1241.1	103.9	383.2	2764.9	351.9	420.0	445.3	491.8	523.9
6010.188	Floodway	1238.2	0.01	1238.8	147.9	305.3	2102.6	1092.1	525.0	602.5	632.1	705.0
5546.898	Floodway	1237.5	0.00	1238.1	165.2	269.7	2449.4	780.9	445.0	506.5	540.7	630.0
5463.547	Floodway	1236.7	0.04	1237.8	60.0	136.4	3162.5	201.1	478.6	491.3	530.8	538.6
5431.919BR U	Floodway	1236.2	0.07	1237.6	54.3	159.5	3067.9	272.6	478.6	491.3	530.8	538.6
5431.919BR D	Floodway	1235.3	0.10	1236.9	50.6	136.4	3228.0	135.6	472.0	476.5	514.4	527.2
5394.885	Floodway	1235.4	0.08	1236.8	55.2	77.1	3263.6	159.3	472.0	476.5	514.4	527.2
5154.368	Floodway	1234.5	0.08	1235.6	68.7	157.1	3160.9	182.1	467.0	480.3	522.4	535.7
4805.449	Floodway	1233.1	0.01	1234.1	77.1	544.6	2715.2	240.3	400.0	431.6	462.2	477.1
4462.235	Floodway	1231.5	0.22	1232.7	101.4	127.2	3204.2	168.6	225.0	268.6	312.9	327.6
4402.619	Floodway	1231.3	0.18	1232.5	107.0	88.9	2811.8	599.3	264.0	308.4	344.8	371.0
4330.115	Floodway	1231.4	0.25	1232.0	89.0	18.0	3400.0	82.0	552.0	555.5	629.1	641.0
4302.237BR U	Floodway	1231.2	0.28	1231.9	82.0	16.3	3381.9	101.8	552.0	555.5	629.1	641.0
4302.237BR D	Floodway	1230.5	0.81	1231.7	76.0	236.5	3006.1	257.4	522.0	540.7	586.6	603.0
4265.133	Floodway	1230.6	0.78	1231.6	81.0	231.0	3084.6	184.4	522.0	540.7	586.6	603.0
4202.005	Floodway	1230.0	0.31	1231.3	89.3	29.4	3469.9	0.7	480.0	537.2	583.7	584.0
4139.126	Floodway	1229.9	0.03	1231.1	137.1	248.4	3139.0	112.6	670.0	780.2	820.2	829.0
3887.453	Floodway	1229.3	0.00	1229.9	100.8	110.9	3362.6	26.4	733.7	749.8	822.4	860.0
3502.323	Floodway	1226.6	0.23	1228.3	64.0	166.9	3284.0	49.2	775.0	790.7	834.0	839.0
3406.087	Floodway	1226.8	0.21	1227.5	80.0	127.3	3257.5	115.2	802.0	815.4	868.5	882.0
3374.836BR U	Floodway	1226.5	0.24	1227.4	75.0	165.0	3187.2	147.8	802.0	815.4	868.5	882.0
3374.836BR D	Floodway	1225.9	0.26	1226.8	71.0	36.5	3361.7	101.8	804.0	810.3	869.7	880.0
3344.186	Floodway	1226.0	0.25	1226.7	76.0	26.5	3402.0	71.6	804.0	810.3	869.7	880.0
3181.879	Floodway	1224.6	0.98	1226.1	58.8	139.7	2973.5	386.9	537.0	545.2	579.7	595.8
3052.657	Floodway	1224.2	1.00	1225.4	89.2	209.6	3288.3	2.1	470.0	544.3	581.2	581.5
2641.649	Floodway	1222.9	0.89	1223.7	140.0	595.5	2675.4	229.2	420.0	523.3	562.3	602.9
2310.271	Floodway	1221.7	0.46	1222.5	183.3	848.7	2335.1	316.1	460.0	555.6	586.9	680.0
2155.534	Floodway	1221.2	0.08	1222.1	164.6	666.9	2805.7	27.4	530.0	613.7	653.8	720.0
2078.819	Floodway	1221.1	0.49	1222.0	111.9	463.4	2762.3	274.4	550.0	609.4	642.2	710.0
2050.718BR U	Floodway	1221.1	0.49	1222.0	12.9	349.5	2878.8	272.0	550.0	609.4	642.2	710.0
2050.718BR D	Floodway	1221.1	0.49	1221.4	42.5	124.1	3135.0	241.3	550.0	616.8	658.5	730.0
2012.315	Floodway	1218.7	0.23	1220.2	95.9	235.9	3011.9	252.2	550.0	616.8	658.5	730.0
1740.38	Floodway	1217.8	0.06	1218.8	151.6	35.3	3161.9	302.9	436.0	445.0	494.0	600.0
1354.907	Floodway	1216.0	0.06	1217.3	111.7	164.6	3235.8	99.6	258.0	309.7	351.1	400.0
919.6187	Floodway	1214.6	0.11	1215.5	174.5	65.7	3372.9	61.4	362.0	424.4	475.3	550.0
622.9671	Floodway	1213.5	0.09	1214.5	123.8	101.0	3299.9	99.1	467.0	567.8	608.3	650.0
343.8126	Floodway	1212.2	0.09	1213.5	43.0	0.4	3498.8	0.8	718.0	718.2	760.7	761.0

3.0 FLOODWAY MAPPING

The resulting proposed final floodway delineations are shown in Figures 1 and 2. In some locations, floodway delineations are shown at the landward levee toe as requested by the City. See Section 6.19 of "Guidance for Flood Risk Analysis and Mapping – Levees" for justification of placement. Figures 1 and 2 also show the proposed draft floodplain delineations, provided by Iowa Flood Center.

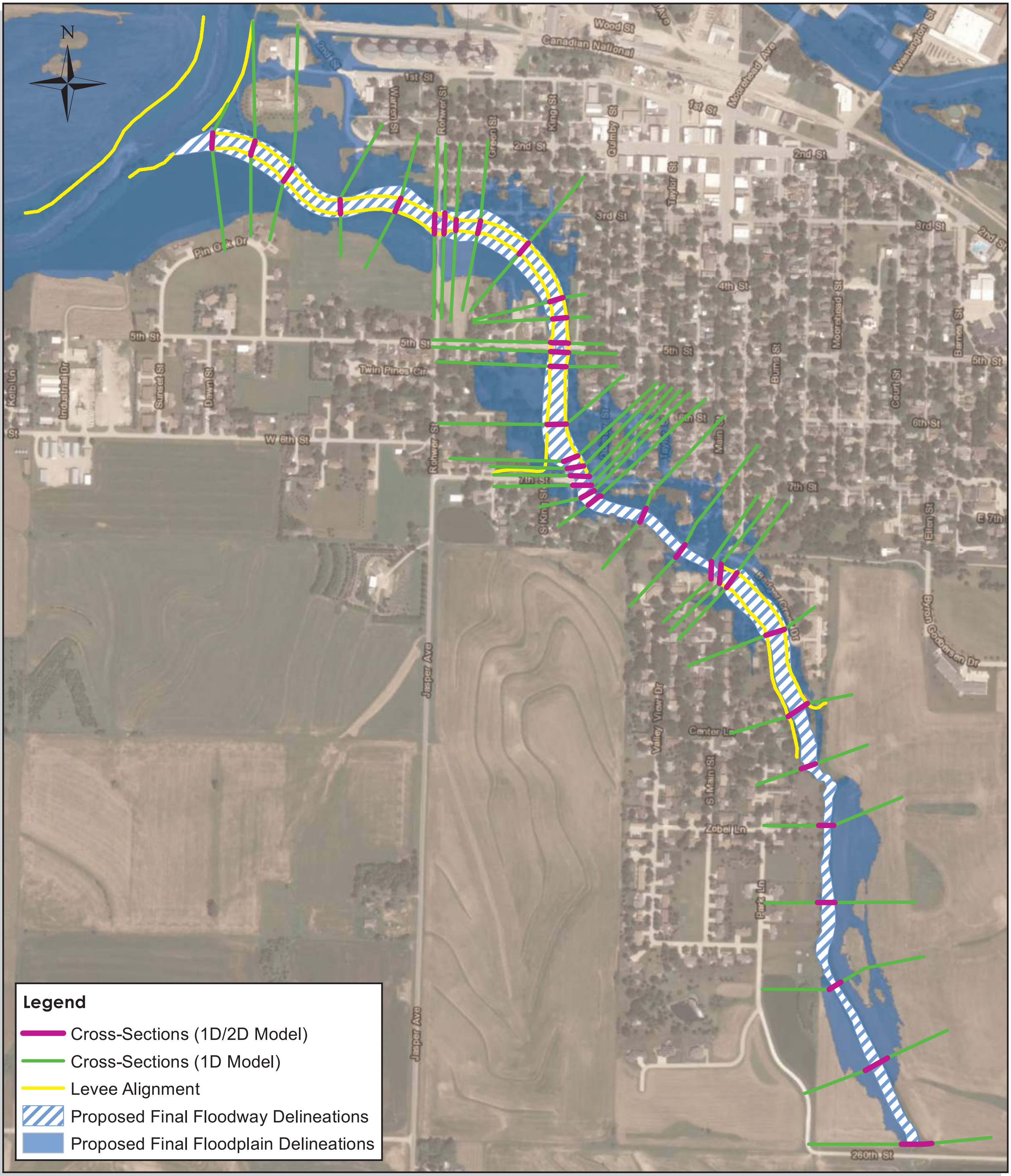

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Figure 1: Floodway/Floodplain Proposed Delineations

Badger Creek
Ida Grove, Iowa

0 500 1,000 Feet

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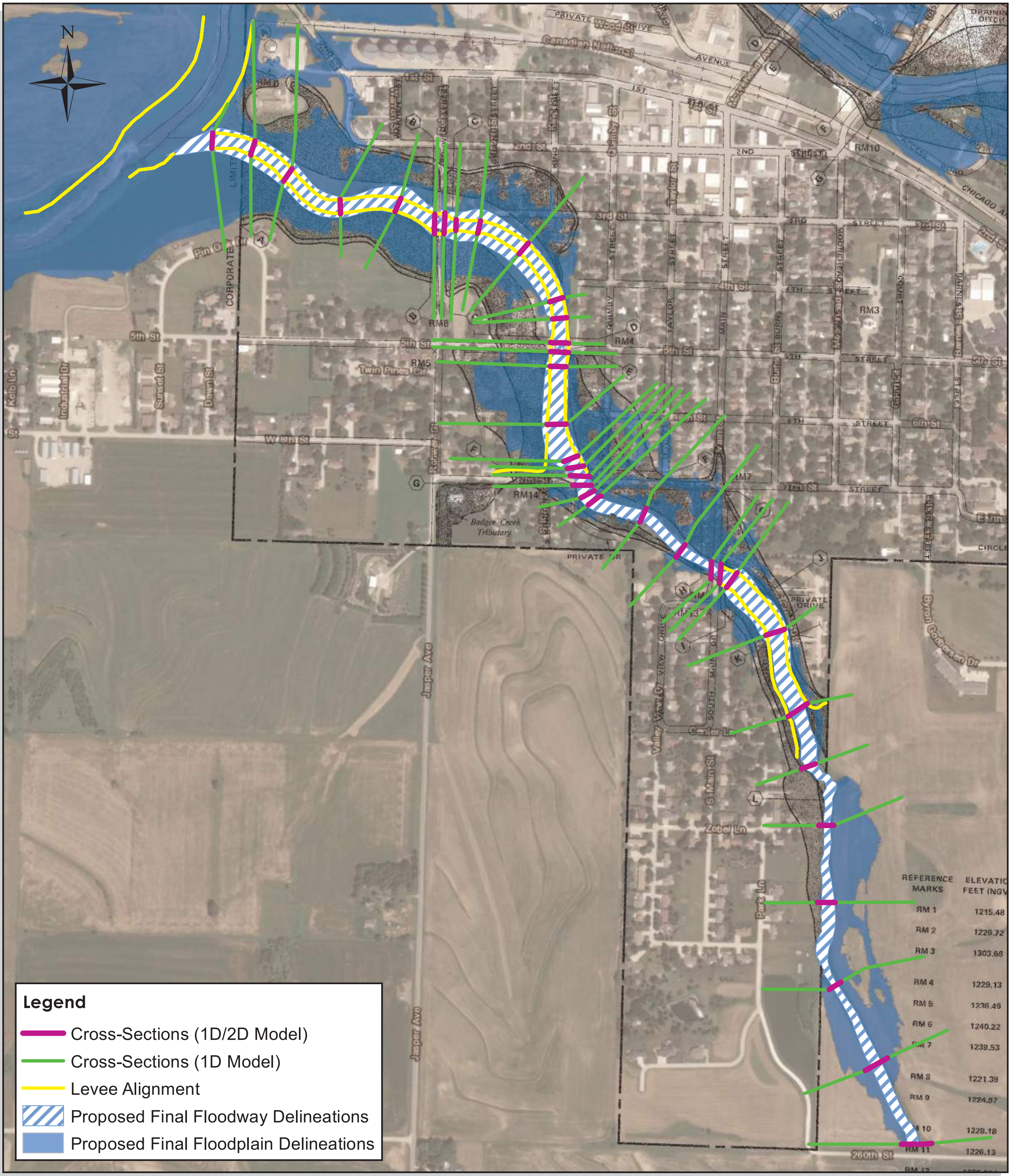

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Figure 2: Floodway/Floodplain Delineations vs. Effective FIRM

Badger Creek
Ida Grove, Iowa

0 500 1,000 Feet

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Appendix A

Included -

- STARR II Guidance Memo
- FEMA guidance document "Guidance for Flood Risk Analysis and Mapping – Levees", February, 2019 (digital only)
- cHECk-RAS Floodway Check

Memo

To:	Rick Nusz / Dane Bailey FEMA Region VII	From:	Anish Pradhananga STARR II
File:	1D floodway on 2D modeled and mapped area	Date:	May 7, 2019

Reference: **Tolerance in discrepancy between water surface elevation from the equivalent 1D model for floodway analysis for areas modeled and mapped using 2D methodology**

Iowa Department of Water Resources (IDNR) is producing a floodway analysis in one of IDNR's projects. The project used 1D/2D analysis methodology to model and map flood hazard in the area under question. Per the Region's current guidelines IDNR is using 1D analysis approach to produce a floodway in this area.

IDNR is inquiring about the calibration tolerance in discrepancy between the 1D/2D model water surface elevation and the equivalent 1D model water surface elevation.

<u>Issue:</u>

IDNR is finding with relatively small effort it is possible to get the 1D water surface elevations to match 1D/2D water surface elevations within +/-0.5ft at the representative 1D cross-sections. However, it requires significant additional effort and in cases unreasonable manipulation in the 1D model parameters to get the discrepancy within a smaller tolerance, close to +/- 0.1 ft.

Though +/- 0.5 ft is generally used best practice tolerance in producing equivalent models for a variety of FEMA Flood Risk studies, we think +/-0.5 feet is too wide of the tolerance in this case. The primary purpose of the model is to identify reasonable encroachment stations to establish a floodway extent and produce a floodway data table. The floodway extents established by a model with an error tolerance half of a typical floodway surcharge has high uncertainty in reliability of floodway extents and the resulting floodway surcharge.

The problem is compounded by the fact that the water surface elevation estimates from a 2D model can be different across a width of a single cross-section. Thus, we think this situation requires the tolerance established based on the 2D model results and topographic condition of the area under study.

The following are our suggested solutions:

<u>Suggested calibration tolerance approach:</u>

A model can have a single tolerance for the entire model (all cross-sections) or the tolerance can vary at each cross-section. This will depend on uniqueness of the model. In general, the calibration approaches pointed below are used for all models

1. Adjust manning's n values and ineffective areas in 1D cross-section to calibrate 1D results within the established tolerance at the cross-section
2. In case of steady state 1D analyses match the peak flow at each cross-section to the peak flow at the cross-section location from 2D routed model. Use the same discharge values in equivalent 1D model and with floodway model.

Follow the following to establish the tolerance in discrepancy between 1D and 2D model results:

Case1: Connected flooding between the main channel and floodplain

In the majority of cases, flooding in the main channel and floodplain are connected across a cross-section in rare flood events like the 1% annual chance event. In these cases, it is likely the maximum water surface elevation is static across the 2D cells represented by a 1D cross-section. If the variation in maximum water surface elevation across representative 2D cells are within +/-0.1 ft, use the average of the water surface elevation across 2D cells as a target water surface elevation. The tolerance in discrepancy between the 2D model and 1D floodway model is the target water surface elevation +/- 0.1 ft.

If the variation in maximum water surface elevation in 2D cells are higher than +/- 0.1 ft across the representative 1D cross-section, follow Case 2.

Case 2: Disconnected flooding in the main channel and floodplain

In many instances depending on the terrain of the 2D modeled area, it is likely the main channel flooding is disconnected from the floodplain flooding. In cases where the 2D model is indicating there are possible split flow conditions and the flood risk is better represented by keeping the flow paths separate, it is suggested to develop a separate floodway model for each flowpath.

However, it is often more likely these split flows are relatively very short and/or conveys only small proportion of the flood wave being modeled. In these situations, it is reasonable to assume the main channel and floodplain flow and water surface elevations are represented by a single flow and single elevations in the model. In these situations, follow the steps below.

In all cases calibrate to meet the target water surface elevation +/-0.1ft except in case 2c.

Case 2a

1. Check the 2D model water surface elevation difference between the main channel and secondary flow area(s). If it is less than 0.5 ft. Use the average water surface elevation in the main channel as the target water surface elevation. If it is higher than 0.5 ft, follow case 2b.

Case 2b

2. Check the proportion of the flood volume conveyed by the secondary flow area. If the secondary flow area conveys less than 20% of the peak, use the main channel average water surface elevation as target elevation. If the secondary channel flow is higher than 20% of the peak flow, then follow case 2c.

Case 2c

3. If the difference between main channel average water surface elevation and secondary channel average water surface elevation is greater than 0.5 ft and the secondary channel accounts for more than 20% of the total flow, then

    a. Calculate a weighted water surface elevation

    $WSelev_{WT}$ = {(% flow conveyed by primary channel X average water surface elev in the primary channel) + (% flow conveyed in secondary channel X average water surface elev in the secondary channel)+……….}/(total flow across the channels)

$$Tolerence_{WT} = \text{WSelev at the main channel} +/- (\text{Wsel elev main channel} - WSelev_{WT})$$

4. If $Tolerance_{WT}$ is higher than 0.5ft, the model is indicating it is not a good assumption to use a single elevation for both main and the secondary channels. In these situations, use engineering judgement to evaluate whether the floodway encroachment is likely to encroach into the entire secondary flowpath. If yes, use main channel water surface elevation as a target elevation. If the floodway is likely going to only partially encroach the secondary channel, then use +/- 0.5 ft tolerance.

Case 3 Calibration within the target elevation is not achieved

5. In some cases, the calibration to the targeted tolerance may not be achievable within reasonable adjustment to the model parameters. In these situations, document the calibration process and coordinate with the FEMA Region. FEMA Region may approve using a smaller floodway surcharge tolerance at the cross-sections where calibration could not be achieved. Below is the suggested floodway data table documentation to report the model calibration and adjustment to surcharge tolerance.

The approaches outlined above should not be used for braided streams.

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE [1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A	203	92	253	2.2	1,048.9	1,047.5[2]	1,048.0	0.5
B	378	110	116	4.7	1,048.9	1,048.7[2]	1,048.8	0.1
C	480	147	203	2.7	1,049.9	1,049.9	1,050.5	0.6
D	709	65	120	4.6	1,052.2	1,052.3[3]	1,052.7	0.4
E	1,025	61	98	5.6	1,055.3	1,055.3	1,055.7	0.4
F	1,219	113	228	2.4	1,056.9	1,056.9	1,057.8	0.9
G	1,496	50	131	4.2	1,059.0	1,059.0	1,059.2	0.2
H	1,818	54	86	6.4	1,062.8	1,062.7[3]	1,062.9	0.2
I	1,987	61	91	6.0	1,064.1	1,064.1	1,064.3	0.2
J	2,268	55	81	6.7	1,067.9	1,067.9	1,068.0	0.1
K	2,500	97	671	0.8	1,074.3	1,074.4[3]	1,074.7	0.3
L	3,001	59	214	2.5	1,074.4	1,074.4	1,074.7	0.3
M	3,489	39	85	6.4	1,076.0	1,076.5[3]	1,077.0	0.5[4]
N	3,747	44	83	6.5	1,078.9	1,078.9	1,078.9	0.0
					2D model Result WSEL	1D Calibrated model WSEL	1D Calibrated FW WSEL	difference based on calibrated WSEL and calibrated FW

[1] Feet above mouth

[2] Elevation computed without consideration of backwater effects from Inundation River

[3] Elevation computed based on 1D calibrated Floodway Model

[4] Due to floodway calibration, maximum allowable surcharge

<< Additional footnotes based on calibration tolerance

TABLE 24

FEDERAL EMERGENCY MANAGEMENT AGENCY

EXAMPLE COUNTY, NE

AND INCORPORATED AREAS

FLOODWAY DATA

FLOODING SOURCE: EXAMPLE STREAM

Floodway Check

River	Reach	RS	Method	Surcharge	EncStaL	EncStaR	LStaEff	RStaEff	LeftSlope	RightSlope	Structure	LateralWeir Station
Badger_Cr	Badger_Cr	9781.776					585.62	771.18				
		9781.776	1	0.81	655	725	655	725	0.01	0		
		9169.787					277.73	819.05				
		9169.787	1	0.95	475	550	475	550	-0.04	-0.03		
		8577.904					251.17	789.95				
		8577.904	1	0.21	468.9	506	468.9	506	0.01	0.07		
		7971.212					373.43	599.71				
		7971.212	1	0.02	375	460	375	460	-0.01	-0.08		
		7452.984					313.71	621.5				
		7452.984	1	0	397.99	441.01	397.99	441.01	0.04	0.08		
		6979.874					331.14	432.48				
		6979.874	1	0.01	331.5	432.4	331.5	432.4	0.02	-0.01		
		6560.169					413.06	524.17				
		6560.169	1	0.05	420	523.93	420	523.93	0.08	0.06		
		6010.188					345.12	793.75				
		6010.188	1	0.01	525	705	525	705	-0.03	0.04		
		5546.898					309.13	862.88				
		5546.898	1	0	445	630	445	630	-0.56	-0.95		
		5463.547					478.76	540.2				
		5463.547	1	0.04	478.78	538.61	478.78	538.61	-0.13	0.06		
		5431.919					480.52	538.61			Bridge-UP	
		5431.919					472.52	526.82			Bridge-DN	
		5431.919		0.07	478.78	538.61	480.29	538.61			Bridge-UP	
		5431.919		0.1	472.01	527.19	472.52	527.11			Bridge-DN	
		5394.885					472	527.21				
		5394.885	1	0.08	472.01	527.19	472.01	527.19	0.05	0.01		
		5154.368					466.45	982.05				
		5154.368	1	0.08	467	535.7	467	535.7	0.04	-0.01		
		4805.449					307.81	915.29				
		4805.449	1	0.01	400	477.1	400	477.1	0.06	0.02		
		4462.235					151.87	948.82				
		4462.235	1	0.22	225	327.6	225	327.6	-0.05	0.13		
		4402.619					124.91	1044.34				
		4402.619	1	0.18	264	371	264	371	-0.31	0.06		
		4330.115					552	641				
		4330.115	1	0.25	552.01	640.99	552.01	640.99	0.01	-0.14		
		4302.237					553	640			Bridge-UP	
		4302.237					522.7	603			Bridge-DN	
		4302.237		0.28	552.01	640.99	553	640			Bridge-UP	
		4302.237		0.81	522.55	602.99	522.55	602.99			Bridge-DN	
		4265.133					522.51	603				
		4265.133	1	0.78	522.55	602.99	522.55	602.99	0.62	-0.25		
		4202.005					190.09	1127.18				
		4202.005	1	0.31	480	584	480	584	0.79	0.08		
		4139.126					361.37	1303.18				
		4139.126	1	0.03	670	829	670	829	-0.31	0.18		
		3887.453					446.14	1195.66				
		3887.453	1	0	733.7	860	733.7	860	-0.04	-0.12		
		3502.323					774.54	840.37				
		3502.323	1	0.23	775	839	775	839	0.03	0.14		
		3406.087					801.59	882.48				
		3406.087	1	0.21	802	882	802	882	-0.06	0		
		3374.836					802.45	881.6			Bridge-UP	
		3374.836					803.52	881.71			Bridge-DN	
		3374.836		0.24	802	882	802	882			Bridge-UP	
		3374.836		0.27	804	880	804	880			Bridge-DN	

		3344.186					803.19	882.02				
		3344.186	1	0.25	804	880	804	880	-0.06	-0.04		
		3181.879					201.03	595.21				
		3181.879	1	0.98	537	595.2	537	595.2	0.52	-0.11		
		3052.657					196.86	590.96				
		3052.657	1	1	470	581.5	470	581.5	0.07	0.1		
		2641.649					232.8	850.57				
		2641.649	1	0.89	420	602.88	420	602.88	-0.03	0.15		
		2310.271					221.38	751.8				
		2310.271	1	0.46	460	680	460	680	-0.05	-0.15		
		2155.534					262.64	765.92				
		2155.534	1	0.08	530	720	530	720	-0.36	-0.03		
		2078.819					243.66	862.14				
		2078.819	1	0.49	550	710	550	710	0.18	0.12		
		2050.718					359.79	862.15			Bridge-UP	
		2050.718					372.79	879.23			Bridge-DN	
		2050.718		0.49	550	710	550	710			Bridge-UP	
		2050.718		0.49	550	730	550	730			Bridge-DN	
		2012.315					366.11	792.38				
		2012.315	1	0.23	550	730	550	730	-0.2	0.14		
		1740.38					199.97	794.64				
		1740.38	1	0.06	436	600	436	600	0.1	-0.16		
		1354.907					180.07	827.06				
		1354.907	1	0.06	258	400	258	400	0.04	0.07		
		919.6187					203.72	1426.55				
		919.6187	1	0.11	362	550	362	550	0.11	-0.13		
		622.9671					154.02	839.17				
		622.9671	1	0.09	467	650	467	650	-0.36	-0.14		
		343.8126					241.45	772.85				
		343.8126	1	0.09	718	761	718	761				

If the Left Slope or Right Slope is more than 1, the change in floodway boundary between the two River Stations is equal to or more than 45 degrees. The Left Slope or Right Slope with the angle equal to or more than 45 degrees is shown in red. The floodway widths at these River Stations should be smoothed.

Appendix B

Included -

- HEC-RAS Model (digital only)

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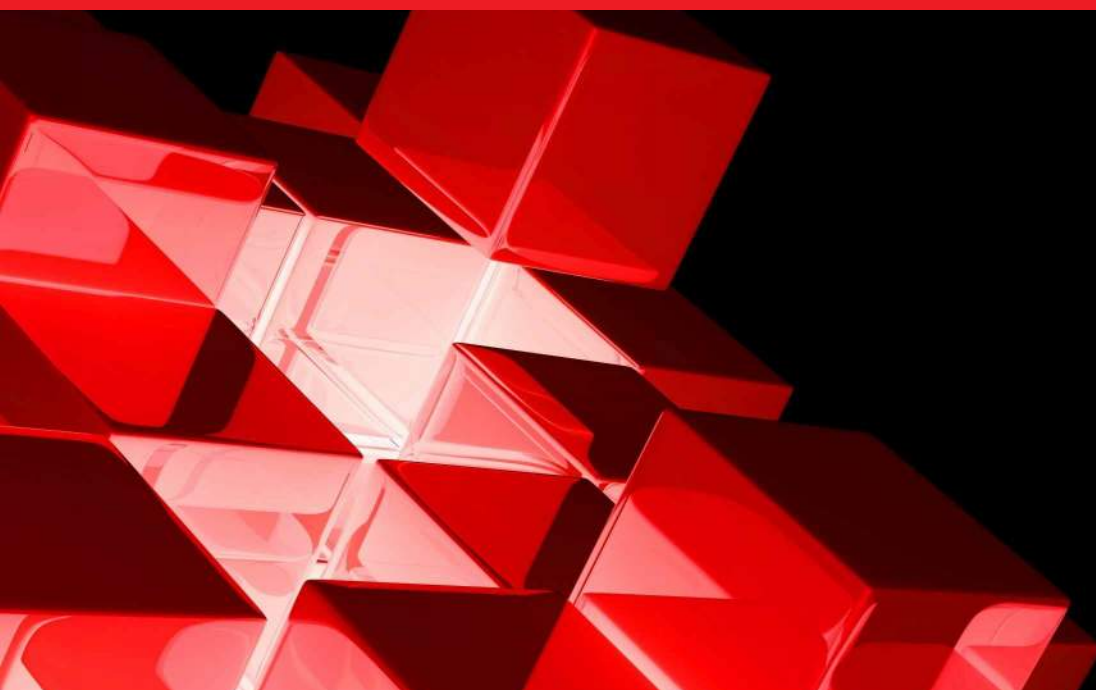
Odebolt Creek Floodway Analysis

Ida Grove, Iowa



June 2019

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Appendix A –

- STARR II Guidance Memo
- FEMA guidance document "Guidance for Flood Risk Analysis and Mapping – Levees", February, 2019 (digital only)
- cHECk-RAS Floodway Check

Appendix B – HEC-RAS Model (digital only)

1.0 PROJECT BACKGROUND

The City of Ida Grove, Iowa has been coordinating with Iowa Flood Center (IFC), the Iowa Department of Natural Resources (IDNR) and FEMA Region VII (FEMA) to complete a revised flood study for Maple River, Odebolt Creek, and Badger Creek in the City of Ida Grove as part of a countywide Digital Flood Insurance Rate Map (DFIRM) update for Ida County, Iowa. As part of this process, Iowa Flood Center has developed a detailed hydraulic model for Ida Grove using a 1D/2D modeling approach. This hydraulic model has been finalized through FEMA's independent technical review process. A separate 1D, Steady State model is being produced by JEO to perform a floodway analysis. The purpose of this technical memo is to describe the technical procedures used for the development of the floodway analysis for Odebolt Creek.

2.0 METHODOLOGY AND MODELING

2.1 Base Model Development and Calibration

A baseline 1D, steady state HEC-RAS version 5.0.5 model was created for Odebolt Creek and calibrated to the water surface profile for the 1% annual chance event from the existing conditions 1D/2D hydraulic model. Using Arc-GIS and HEC-GeoRAS software a base geometry file was created which included the 1D portion of the existing 1D/2D model cross sections with the geometry of each cross section extended to high ground resulting in a complete 1D geometry for the 1D steady state model. Odebolt Creek has one non-accredited but hydraulically significant training levee on the left bank at the upstream end of the Odebolt Creek Flood Risk Reduction Project (FRRP) channel built by the U.S. Army Corps of Engineers; for calibration purposes the 'with levee' geometry was used within this region. For the purposes of the floodway analysis the levee segment topography was included in the geometry, but it was assumed floodplain area landward of the levee embankments is effective flow as is required for a 1D natural valley analysis according to Section 6.12.2 of the February 2019 FEMA document "Guidance for Flood Risk Analysis and Mapping – Levees". This guidance is provided in Appendix A. Alignment and location of the levee embankment is shown on Figures 1 and 2.

Analysis was then completed to determine an appropriate calibration tolerance between the 1% annual chance water surface elevation (WSE) from the equivalent 1D model and the existing 1D/2D floodplain model using guidance provided by STARR II which is provided in Appendix A. The analysis compared the WSE of all secondary flow areas in the floodplain to the main channel and the portion of the flood volume conveyed by the secondary floodplain flow paths. Floodplain flow area water surface elevations were calculated using tools within Arc-GIS. It was determined most cross-sections fall into the categories of Case 2a and Case 2b and therefore should be calibrated to a tolerance of 0.1 feet of the main channel average WSE. Cross-sections 8261.741 falls into the category of Case 2c. Target WSE and tolerance for this cross-section was determined following the STARR II guidance and is reported in Table 1. See appendix A for the complete STARR II memo and case descriptions.

A 1D steady state run was completed using the IFC reported peak flow of 10,900 cfs and the same downstream normal depth boundary condition of 0.002 ft/ft used in the 1D/2D hydraulic model. Model calibration was then achieved through adjustments to manning's n values and ineffective flow area locations on a cross-section by cross-section basis. Results from the calibration effort are shown in Table 1.

2.2 Floodway Analysis

A floodway analysis was completed based on the guidelines provided in the February 2019 FEMA document "Guidance for Flood Risk Analysis and Mapping – Levees" and further guidance from STARR II. Using the calibrated 1D geometry model as a starting base model, a floodway analysis was completed to determine an equal conveyance reduction floodway. After coordination with STARR II regarding approach to treatment of the levee embankment within the floodway analysis, the approach taken was to remove ineffective flow locations which were used to calibrate the 1D model to the existing conditions 1D/2D model to create a natural valley base model for the floodway for the entire reach of Odebolt Creek. Analysis results included floodway surcharges ranging from 0.00 feet to 0.70 feet. Results of the analysis are shown in Table 2.

The floodway check was run in cHECk-RAS. Results of the floodway check are provided in Appendix A. No changes to the floodway analysis were made in response to cHECk-RAS comments from the floodway check.

Table 1 - Model Calibration Results

River Station	LOB WSE	ROB WSE	Channel WSE	Max WSE Difference*	Main Channel Peak Flow (cfs)	IFC Reported Peak Flow	% Peak Flow Conveyed by Secondary Channel	STARR II Memo Case	Target WSE	Calibrated 1D Max WSE	1D WSE Difference**
9970.4	1234.5	1234.1	1234.6	-0.45	10864	10900	0.00	2a	1234.56	1234.61	-0.05
9320.199	1234.5	1233.9	1234.8	-0.90	9196		0.16	2b	1234.80	1234.75	0.05
8889.332	0.0	1234.1	1233.4	0.66	9196		0.16	2b	1233.39	1233.49	-0.1
8848.541	1231.0	1231.5	1231.7	-0.68	9195		0.16	2b	1231.71	1231.69	0.02
8814.518	1230.6	1230.7	1230.6	0.09	9713		0.11	2a	1230.57	1230.47	0.1
8613.595	1230.2	1230.0	1230.0	0.20	9868		0.09	2a	1229.97	1229.99	-0.02
8261.741	1230.1	1229.9	1230.4	-0.51	8620		0.21	2c	1230.29	1230.47	-0.07
7981.541	1229.9	1229.8	1228.9	1.02	10851		0.00	2b	1228.90	1228.88	0.02
7817.194	1226.9	1227.2	1227.3	-0.34	10439		0.04	2a	1227.28	1227.19	0.09
7768.646	1227.0	1227.1	1227.3	-0.37	10326		0.05	2a	1227.34	1227.29	0.05
7710.629	1226.1	1226.1	1225.9	0.22	10022		0.08	2a	1225.87	1225.91	-0.04
7431.011	1225.9	1226.0	1226.6	-0.63	9006		0.17	2b	1226.55	1226.5	0.05
6936.969	1225.9	1225.9	1226.0	-0.12	8998		0.17	2a	1226.00	1226.1	-0.1
6624.792	1225.4	1225.4	1225.8	-0.44	8794		0.19	2a	1225.82	1225.86	-0.04
6603.354	1225.1	0.0	1224.7	0.37	9568		0.12	2a	1224.69	1224.76	-0.07
6583.601	1224.4	1224.1	1224.0	0.36	9531		0.13	2a	1224.03	1224.11	-0.08
6257.355	1223.9	1223.8	1223.8	0.05	9377		0.14	2a	1223.83	1223.89	-0.06
5837.107	1222.9	0.0	1223.0	-0.08	10621		0.03	2a	1222.97	1222.95	0.02
5354.235	1222.6	0.0	1222.7	-0.11	10595		0.03	2a	1222.70	1222.69	0.01
4812.35	0.0	1222.4	1222.4	-0.01	10733		0.02	2a	1222.40	1222.48	-0.08
4267.027	0.0	1221.4	1221.3	0.05	10730		0.02	2a	1221.30	1221.37	-0.07
3987.029	0.0	1221.3	1221.5	-0.23	10366		0.05	2a	1221.53	1221.56	-0.03
3818.981	0.0	1221.1	1220.8	0.37	10547		0.03	2a	1220.78	1220.78	0
3761.387	0.0	1221.1	1220.5	0.62	10547		0.03	2b	1220.48	1220.47	0.01
3707.206	0.0	1221.1	1220.5	0.60	10547		0.03	2b	1220.47	1220.48	-0.01
3422.357	0.0	1221.0	1218.9	2.13	10456		0.04	2b	1218.86	1218.95	-0.09
3101.807	1218.8	1219.7	1218.9	0.81	10467		0.04	2b	1218.89	1218.96	-0.07
3086.707	0.0	1219.0	1217.6	1.38	10557		0.03	2b	1217.60	1217.54	0.06
3080.867	1216.9	1216.3	1216.9	-0.62	10539		0.03	2b	1216.89	1216.9	-0.01
3073.793	1215.7	1213.9	1215.5	-1.63	10063		0.08	2b	1215.49	1215.48	0.01
3029.09	1212.7	1214.3	1214.5	-1.85	10370		0.05	2b	1214.53	1214.5	0.03

*WSE difference between the target WSE from the 1D/2D model and the left overbank (LOB) or right overbank (ROB). Reported value is the greater of the two.

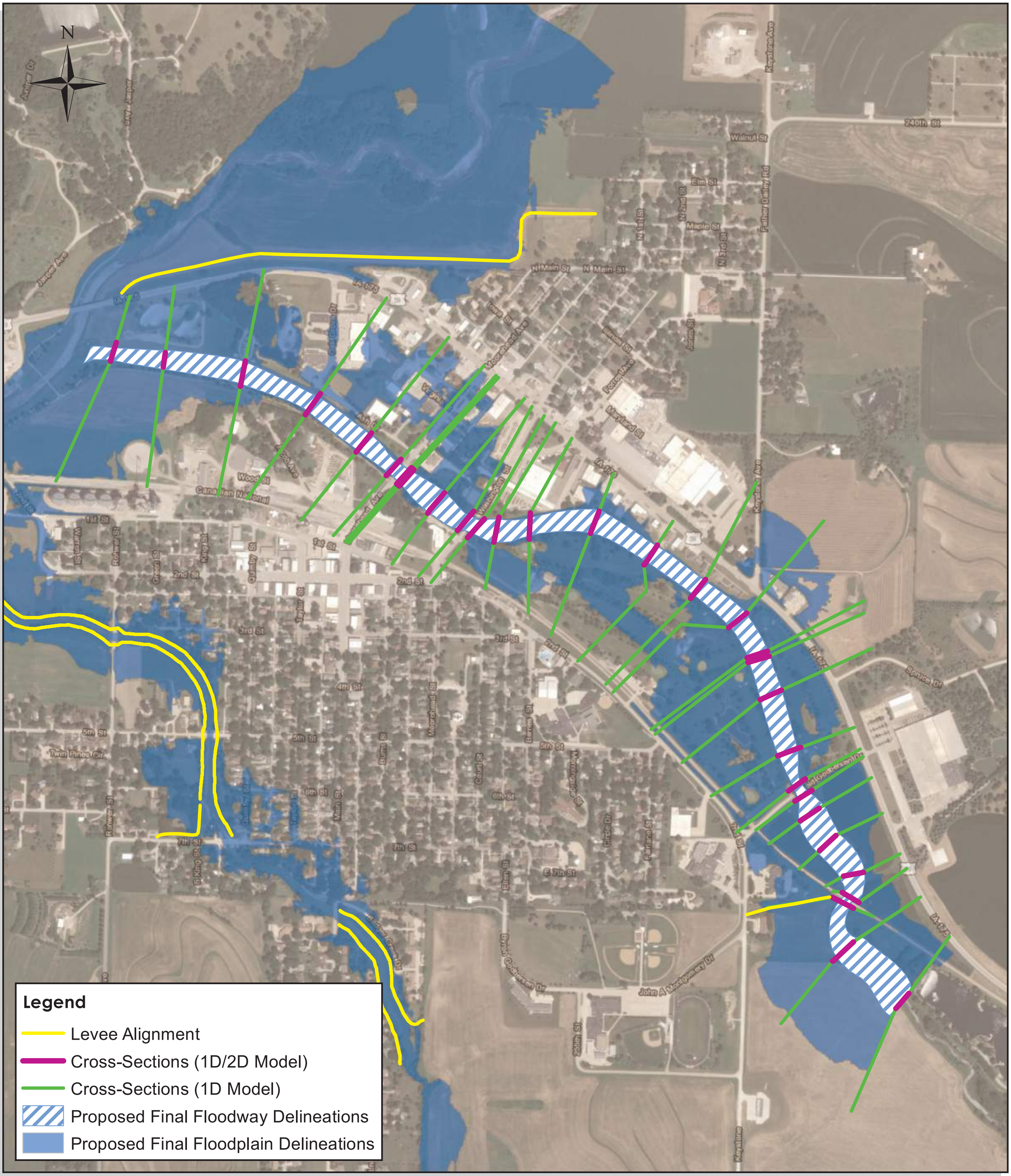
** WSE difference between the target WSE from the 1D/2D model and the calibrated 1D maximum WSE. WSE Tolerance for all cross sections was +/- 0.1 feet with the exception of cross sections 8261.741 which followed Case 2c. Tolerance for this cross section was determined to be +/- 0.11 feet.

Table 2 - Floodway Results

River Sta	Profile	W.S. Elev	Prof Delta WS	E.G. Elev	Top Wdth Act	Q Left	Q Channel	Q Right	Enc Sta L	Ch Sta L	Ch Sta R	Enc Sta R
		(ft)	(ft)	(ft)	(ft)	(cfs)	(cfs)	(cfs)	(ft)	(ft)	(ft)	(ft)
9970.4	Floodway	1234.85	0.29	1236.81	260.00	801.11	9804.24	294.65	860.00	990.76	1046.91	1120.00
9320.199	Floodway	1234.71	0.01	1235.38	265.00	301.87	8196.56	2401.58	715.00	753.46	840.76	980.00
8889.332	Floodway	1233.61	0.19	1234.75	91.00	2.69	10891.20	6.11	609.00	609.93	698.69	700.00
8848.541BR U	Floodway	1232.04	0.36	1234.57			10882.49	17.51	609.00	609.93	698.69	700.00
8848.541BR D	Floodway	1231.03	0.65	1234.35	38.00	311.62	10434.95	153.44	100.00	112.46	187.99	210.00
8814.518	Floodway	1232.17	0.57	1233.73	110.00	170.94	10092.87	636.19	100.00	112.46	187.99	210.00
8613.595	Floodway	1230.53	0.00	1233.13	215.00	863.23	9494.83	541.94	135.00	222.11	283.90	350.00
8261.741	Floodway	1229.95	0.00	1232.37	249.00	770.06	9336.69	793.26	201.00	290.53	353.91	450.00
7981.541	Floodway	1230.46	0.15	1231.34	245.00	1940.05	6672.38	2287.57	310.00	383.24	445.12	555.00
7817.194	Floodway	1228.8	0.03	1230.76	110.00	299.24	10263.68	337.08	435.00	454.43	522.99	545.00
7768.646BR U	Floodway	1228.68	0.06	1230.68	107.50	337.58	10099.92	462.50	435.00	454.43	522.99	545.00
7768.646BR D	Floodway	1227.41	0.44	1230.36	71.75	0.70	10895.71	3.58	597.00	597.46	667.60	670.00
7710.629	Floodway	1227.39	0.45	1230.26	73.00	0.16	10898.03	1.81	597.00	597.46	667.60	670.00
7431.011	Floodway	1227.57	0.04	1229.61	170.00	1175.34	8376.33	1348.33	650.00	708.68	767.75	820.00
6936.969	Floodway	1227.07	0.15	1228.97	200.00	1253.44	7797.23	1849.33	830.00	896.85	951.87	1030.00
6624.792	Floodway	1227.38	0.28	1228.51	180.01	1667.43	7173.46	2059.11	981.69	1035.98	1095.49	1161.70
6603.354BR U	Floodway	1226.44	0.70	1228.42	155.47	1036.79	8506.59	1356.62	981.69	1035.98	1095.49	1161.70
6603.354BR D	Floodway	1226.42	0.07	1228.40	155.78	1304.01	8165.55	1430.44	1000.00	1058.15	1109.93	1180.00
6583.601	Floodway	1226.89	0.30	1228.18	180.00	1954.73	7016.19	1929.08	1000.00	1058.15	1109.93	1180.00
6257.355	Floodway	1226.21	0.02	1227.79	175.00	1589.77	7455.11	1855.12	1205.00	1258.35	1310.02	1380.00
5837.107	Floodway	1224.79	0.06	1227.08	151.00	783.91	8497.81	1618.27	1041.00	1070.97	1126.64	1192.00
5354.235	Floodway	1224.05	0.02	1226.15	159.00	1335.61	7907.83	1656.56	1062.00	1107.66	1155.86	1221.00
4812.35	Floodway	1223.91	0.00	1225.08	205.00	1890.22	6812.10	2197.68	910.00	981.31	1034.18	1115.00
4267.027	Floodway	1223.06	0.00	1224.42	152.00	1836.13	7553.11	1510.76	638.00	691.92	749.16	790.00
3987.029	Floodway	1222.98	0.19	1224.06	200.00	2137.08	6838.44	1924.48	390.00	470.15	523.64	590.00
3818.981	Floodway	1222.52	0.02	1223.88	145.00	2500.48	6681.23	1718.29	530.00	575.89	621.95	675.00
3761.387BR U	Floodway	1221.82	0.01	1223.69	139.00	2189.32	7348.86	1361.82	530.00	575.89	621.95	675.00
3761.387BR D	Floodway	1221.92	0.06	1223.51	136.00	1280.69	7873.91	1745.41	518.00	555.32	609.29	660.00
3707.206	Floodway	1221.4	0.00	1223.34	142.00	947.14	8429.59	1523.28	518.00	555.32	609.29	660.00
3422.357	Floodway	1221.67	0.00	1222.81	221.00	2146.92	6601.96	2151.12	519.00	600.87	650.42	740.00
3101.807	Floodway	1220.89	0.11	1222.44	137.40	1588.29	7830.52	1481.19	648.30	690.34	740.86	785.70
3086.707		Inl Struct										
3080.867	Floodway	1220.59	0.12	1222.25	136.60	1552.40	7907.44	1440.16	655.70	697.42	747.95	792.30
3073.793	Floodway	1220.6	0.12	1222.22	130.00	1606.02	7804.48	1489.51	660.00	699.82	750.35	790.00
3029.09 BR U	Floodway	1220.37	0.22	1222.15	127.60	1555.54	7994.49	1349.97	660.00	699.82	750.35	790.00
3029.09 BR D	Floodway	1219.05	0.11	1221.78	117.60	1222.53	8741.71	935.76	690.00	726.90	778.49	810.00
2963.447	Floodway	1219.03	0.08	1221.60	120.00	1371.30	8508.50	1020.20	690.00	726.90	778.49	810.00
2641.589	Floodway	1219.17	0.21	1220.79	170.00	1266.13	8147.82	1486.05	740.00	795.33	848.99	910.00
2112.79	Floodway	1217.7	0.17	1219.87	150.00	1134.21	8645.11	1120.68	880.00	930.96	988.56	1030.00
1475.789	Floodway	1217.23	0.33	1218.73	190.00	697.78	9290.74	911.48	940.00	1001.51	1074.33	1130.00
810.1682	Floodway	1216.04	0.56	1217.85	160.00	517.03	9657.17	725.80	980.00	1024.52	1097.36	1140.00
380.2754	Floodway	1214.72	0.23	1217.06	110.00	126.59	10701.53	71.88	1100.00	1118.44	1201.07	1210.00

3.0 FLOODWAY MAPPING

The resulting proposed final floodway delineations are shown in Figure 1. Figure 1 also shows the proposed final floodplain delineations, provided by Iowa Flood Center.



Created By: Ann Nissen
Date: 05-16-2019
Revised: 06-11-2019
File: 190345.00

Figure 1: Floodway/Floodplain Proposed Delineations

Odebolt Creek
Ida Grove, Iowa

0 600 1,200 Feet

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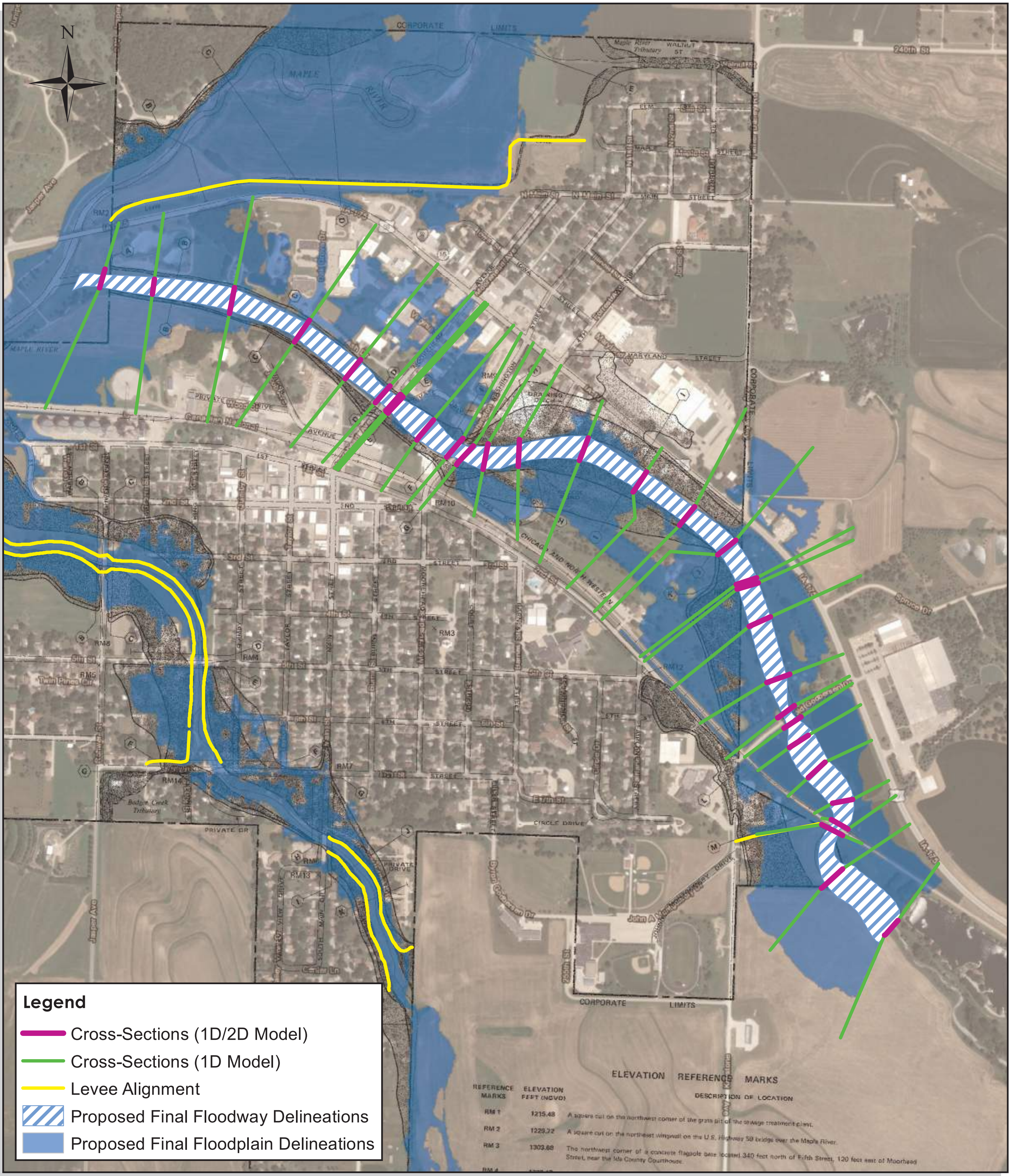

Created By: Ann Nissen
Date: 05-16-2019
Revised: 06-10-2019
File: 190345.00

Figure 2: Floodway/Floodplain Delineations vs. Effective FIRM

Odebolt Creek
Ida Grove, Iowa

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Appendix A

Included -

- STARR II Guidance Memo
- FEMA guidance document “Guidance for Flood Risk Analysis and Mapping – Levees”, February, 2019 (digital only)
- cHECk-RAS Floodway Check

STARR II Strategic Alliance for Risk Reduction

Memo

To:	Rick Nusz / Dane Bailey FEMA Region VII	From:	Anish Pradhananga STARR II
File:	1D floodway on 2D modeled and mapped area	Date:	May 7, 2019

Reference: **Tolerance in discrepancy between water surface elevation from the equivalent 1D model for floodway analysis for areas modeled and mapped using 2D methodology**

Iowa Department of Water Resources (IDNR) is producing a floodway analysis in one of IDNR's projects. The project used 1D/2D analysis methodology to model and map flood hazard in the area under question. Per the Region's current guidelines IDNR is using 1D analysis approach to produce a floodway in this area.

IDNR is inquiring about the calibration tolerance in discrepancy between the 1D/2D model water surface elevation and the equivalent 1D model water surface elevation.

Issue:

IDNR is finding with relatively small effort it is possible to get the 1D water surface elevations to match 1D/2D water surface elevations within +/-0.5ft at the representative 1D cross-sections. However, it requires significant additional effort and in cases unreasonable manipulation in the 1D model parameters to get the discrepancy within a smaller tolerance, close to +/- 0.1 ft.

Though +/- 0.5 ft is generally used best practice tolerance in producing equivalent models for a variety of FEMA Flood Risk studies, we think +/-0.5 feet is too wide of the tolerance in this case. The primary purpose of the model is to identify reasonable encroachment stations to establish a floodway extent and produce a floodway data table. The floodway extents established by a model with an error tolerance half of a typical floodway surcharge has high uncertainty in reliability of floodway extents and the resulting floodway surcharge.

The problem is compounded by the fact that the water surface elevation estimates from a 2D model can be different across a width of a single cross-section. Thus, we think this situation requires the tolerance established based on the 2D model results and topographic condition of the area under study.

The following are our suggested solutions:

Suggested calibration tolerance approach:

A model can have a single tolerance for the entire model (all cross-sections) or the tolerance can vary at each cross-section. This will depend on uniqueness of the model. In general, the calibration approaches pointed below are used for all models

1. Adjust manning's n values and ineffective areas in 1D cross-section to calibrate 1D results within the established tolerance at the cross-section
2. In case of steady state 1D analyses match the peak flow at each cross-section to the peak flow at the cross-section location from 2D routed model. Use the same discharge values in equivalent 1D model and with floodway model.

Follow the following to establish the tolerance in discrepancy between 1D and 2D model results:

Case1: Connected flooding between the main channel and floodplain

In the majority of cases, flooding in the main channel and floodplain are connected across a cross-section in rare flood events like the 1% annual chance event. In these cases, it is likely the maximum water surface elevation is static across the 2D cells represented by a 1D cross-section. If the variation in maximum water surface elevation across representative 2D cells are within +/-0.1 ft, use the average of the water surface elevation across 2D cells as a target water surface elevation. The tolerance in discrepancy between the 2D model and 1D floodway model is the target water surface elevation +/- 0.1 ft.

If the variation in maximum water surface elevation in 2D cells are higher than +/- 0.1 ft across the representative 1D cross-section, follow Case 2.

Case 2: Disconnected flooding in the main channel and floodplain

In many instances depending on the terrain of the 2D modeled area, it is likely the main channel flooding is disconnected from the floodplain flooding. In cases where the 2D model is indicating there are possible split flow conditions and the flood risk is better represented by keeping the flow paths separate, it is suggested to develop a separate floodway model for each flowpath.

However, it is often more likely these split flows are relatively very short and/or conveys only small proportion of the flood wave being modeled. In these situations, it is reasonable to assume the main channel and floodplain flow and water surface elevations are represented by a single flow and single elevations in the model. In these situations, follow the steps below.

In all cases calibrate to meet the target water surface elevation +/-0.1ft except in case 2c.

Case 2a

1. Check the 2D model water surface elevation difference between the main channel and secondary flow area(s). If it is less than 0.5 ft. Use the average water surface elevation in the main channel as the target water surface elevation. If it is higher than 0.5 ft, follow case 2b.

Case 2b

2. Check the proportion of the flood volume conveyed by the secondary flow area. If the secondary flow area conveys less than 20% of the peak, use the main channel average water surface elevation as target elevation. If the secondary channel flow is higher than 20% of the peak flow, then follow case 2c.

Case 2c

3. If the difference between main channel average water surface elevation and secondary channel average water surface elevation is greater than 0.5 ft and the secondary channel accounts for more than 20% of the total flow, then

    a. Calculate a weighted water surface elevation

       $WSelev_{WT}$ = {(% flow conveyed by primary channel X average water surface elev in the primary channel) + (% flow conveyed in secondary channel X average water surface elev in the secondary channel)+……….}/(total flow across the channels)

$$\text{Tolerence}_{WT} = \text{WSelev at the main channel +/- (Wsel elev main channel- WSelev}_{WT})$$

4. If Tolerance$_{WT}$ is higher than 0.5ft, the model is indicating it is not a good assumption to use a single elevation for both main and the secondary channels. In these situations, use engineering judgement to evaluate whether the floodway encroachment is likely to encroach into the entire secondary flowpath. If yes, use main channel water surface elevation as a target elevation. If the floodway is likely going to only partially encroach the secondary channel, then use +/- 0.5 ft tolerance.

Case 3 Calibration within the target elevation is not achieved

5. In some cases, the calibration to the targeted tolerance may not be achievable within reasonable adjustment to the model parameters. In these situations, document the calibration process and coordinate with the FEMA Region. FEMA Region may approve using a smaller floodway surcharge tolerance at the cross-sections where calibration could not be achieved. Below is the suggested floodway data table documentation to report the model calibration and adjustment to surcharge tolerance.

The approaches outlined above should not be used for braided streams.

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE [1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A	203	92	253	2.2	1,048.9	1,047.5[2]	1,048.0	0.5
B	378	110	116	4.7	1,048.9	1,048.7[2]	1,048.8	0.1
C	480	147	203	2.7	1,049.9	1,049.9	1,050.5	0.6
D	709	65	120	4.6	1,052.2	1,052.3[3]	1,052.7	0.4
E	1,025	61	98	5.6	1,055.3	1,055.3	1,055.7	0.4
F	1,219	113	228	2.4	1,056.9	1,056.9	1,057.8	0.9
G	1,496	50	131	4.2	1,059.0	1,059.0	1,059.2	0.2
H	1,818	54	86	6.4	1,062.8	1,062.7[3]	1,062.9	0.2
I	1,987	61	91	6.0	1,064.1	1,064.1	1,064.3	0.2
J	2,268	55	81	6.7	1,067.9	1,067.9	1,068.0	0.1
K	2,500	97	671	0.8	1,074.3	1,074.4[3]	1,074.7	0.3
L	3,001	59	214	2.5	1,074.4	1,074.4	1,074.7	0.3
M	3,489	39	85	6.4	1,076.0	1,076.5[3]	1,077.0	0.5[4]
N	3,747	44	83	6.5	1,078.9	1,078.9	1,078.9	0.0
					2D model Result WSEL	1D Calibrated model WSEL	1D Calibrated FW WSEL	difference based on calibrated WSEL and calibrated FW

[1] Feet above mouth

[2] Elevation computed without consideration of backwater effects from Inundation River

[3] Elevation computed based on 1D calibrated Floodway Model

[4] Due to floodway calibration, maximum allowable surcharge

<< Additional footnotes based on calibration tolerance

TABLE 24

FEDERAL EMERGENCY MANAGEMENT AGENCY

EXAMPLE COUNTY, NE

AND INCORPORATED AREAS

FLOODWAY DATA

FLOODING SOURCE: EXAMPLE STREAM

Floodway Check

River	Reach	RS	Method	Surcharge	EncStaL	EncStaR	LStaEff	RStaEff	LeftSlope	RightSlope	Structure	LateralWeir Station
Odebolt_Cr	Odebolt_Cr	9970.4					338.75	1139.84				
		9970.4	1	0.29	860	1120	860	1120	-0.12	0.13		
		9320.199					85.43	1187.95				
		9320.199	1	0.01	715	980	715	980	-0.09	-0.32		
		8889.332					555.37	796.35				
		8889.332	1	0.19	609	700	609	700	0.07	0.19		
		8848.541					611.69	703.69			Bridge-UP	
		8848.541					100.38	190.38			Bridge-DN	
		8848.541		0.36	609	700	611.69	700			Bridge-UP	
		8848.541		0.65	100	210	100.38	190.38			Bridge-DN	
		8814.518					99.38	211.38				
		8814.518	1	0.57	100	210	100	210	0.34	0.19		
		8613.595					15.79	419.52				
		8613.595	1	0	135	350	135	350	0.01	0.09		
		8261.741					200	600				
		8261.741	1	0	201	450	201	450	-0.06	0.05		
		7981.541					300	575				
		7981.541	1	0.15	310	555	310	555	-0.31	-0.51		
		7817.194					423.62	557.79				
		7817.194	1	0.03	435	545	435	545	-0.17	-0.18		
		7768.646					423.94	557.53			Bridge-UP	
		7768.646					569.67	697.12			Bridge-DN	
		7768.646		0.06	435	545	435	545			Bridge-UP	
		7768.646		0.44	597	670	597	670			Bridge-DN	
		7710.629					569.72	697.08				
		7710.629	1	0.45	597	670	597	670	0.19	0.16		
		7431.011					33.02	1267.45				
		7431.011	1	0.04	650	820	650	820	0.01	0.05		
		6936.969					11.94	1580.8				
		6936.969	1	0.15	830	1030	830	1030	-0.03	-0.03		
		6624.792					11.14	1944.04				
		6624.792	1	0.28	981.69	1161.7	981.69	1161.7	0	0		
		6603.354					15.25	1606.25			Bridge-UP	
		6603.354					12.89	1912.39			Bridge-DN	
		6603.354		0.7	981.69	1161.7	981.69	1161.7			Bridge-UP	
		6603.354		0.07	1000	1180	1000	1180			Bridge-DN	
		6583.601					12.22	1924.75				
		6583.601	1	0.3	1000	1180	1000	1180	-0.01	0		
		6257.355					52.53	2206.76				
		6257.355	1	0.02	1205	1380	1205	1380	-0.05	-0.01		
		5837.107					425.16	1193				
		5837.107	1	0.06	1041	1192	1041	1192	0.02	-0.01		
		5354.235					426.97	1225.27				
		5354.235	1	0.02	1062	1221	1062	1221	0.05	0.03		
		4812.35					434.54	1265.52				
		4812.35	1	0	910	1115	910	1115	-0.03	-0.07		
		4267.027					54.22	915.57				
		4267.027	1	0	638	790	638	790	0.09	0.08		
		3987.029					200	590.44				
		3987.029	1	0.19	390	590	390	590	-0.23	-0.1		
		3818.981					508.21	677.49				
		3818.981	1	0.02	530	675	530	675	-0.04	0.01		
		3761.387					510.26	675.66			Bridge-UP	
		3761.387					511.5	678.06			Bridge-DN	
		3761.387		0.01	530	675	530	675			Bridge-UP	
		3761.387		0.06	518	660	518	660			Bridge-DN	

		3707.206					512.94	676.68				
		3707.206	1	0	518	660	518	660	0.15	0.13		
		3422.357					499.63	800				
		3422.357	1	0	519	740	519	740	-0.12	-0.14		
		3101.807					648.2	785.9				
		3101.807	1	0.11	648.3	785.7	648.3	785.7	-0.01	-0.02		
		3086.707					648.2	785.9			InlineWeir-UP	
		3086.707					655.67	792.38			InlineWeir-DN	
		3086.707			648.3	785.7	648.3	785.7			InlineWeir-UP	
		3086.707			655.7	792.3	655.7	792.3			InlineWeir-DN	
		3080.867					655.67	792.38				
		3080.867	1	0.12	655.7	792.3	655.7	792.3	-0.31	-0.76		
		3073.793					658.05	794.8				
		3073.793	1	0.12	660	790	660	790	-0.02	-0.07		
		3029.09					658.46	794.15			Bridge-UP	
		3029.09					687.32	820.53			Bridge-DN	
		3029.09		0.22	660	790	660	790			Bridge-UP	
		3029.09		0.11	690	810	690	810			Bridge-DN	
		2963.447					687.29	820.57				
		2963.447	1	0.08	690	810	690	810	0.06	0.09		
		2641.589					690.62	925.86				
		2641.589	1	0.21	740	910	740	910	0	-0.03		
		2112.79					861.17	1051.2				
		2112.79	1	0.17	880	1030	880	1030	0.03	0.03		
		1475.789					784.54	1245.35				
		1475.789	1	0.33	940	1130	940	1130	-0.03	-0.02		
		810.1682					961.09	1160.37				
		810.1682	1	0.56	980	1140	980	1140	-0.05	-0.07		
		380.2754					1059.08	1254.11				
		380.2754	1	0.23	1100	1210	1100	1210				

If the Left Slope or Right Slope is more than 1, the change in floodway boundary between the two River Stations is equal to or more than 45 degrees. The Left Slope or Right Slope with the angle equal to or more than 45 degrees is shown in red. The floodway widths at these River Stations should be smoothed.

FW ST 03BDL SECNO: 3029.09

This is (Bridge-DN) Downstream Internal Section. The left encroachment station is within the structure opening area. The left station effective of 687.32 for the 1-percent-annual-chance profile is less than the left abutment station of 689.3084. The 1-percent-annual-chance floodplain is outside the structure opening. The left encroachment station of 690 is greater than the left abutment station of 689.3084. Enc_Sta_L should be relocated outside of the structure opening area.

FW ST 03BUR SECNO: 7768.646

This is (Bridge-UP) Upstream Internal Section. The right encroachment station is within the structure opening area. The right station effective of 697.12 for the 1%-annual-chance profile is greater than the right abutment station of 558.695. The 1%-annual-chance floodplain is outside the structure opening. The right encroachment station of 545 is less than the right abutment station of 558.695. Enc_Sta_R should be relocated outside of the structure opening area.

FW ST 03BUR SECNO: 3761.387

This is (Bridge-UP) Upstream Internal Section. The right encroachment station is within the structure opening area. The right station effective of 678.06 for the 1%-annual-chance profile is greater than the right abutment station of 675.59. The 1%-annual-chance floodplain is outside the structure opening. The right encroachment station of 675 is less than the right abutment station of 675.59. Enc_Sta_R should be relocated outside of the structure opening area.

FW ST 03BUR SECNO: 3029.09

This is (Bridge-UP) Upstream Internal Section. The right encroachment station is within the structure opening area. The right station effective of 820.53 for the 1%-annual-chance profile is greater than the right abutment station of 806.428. The 1%-annual-chance floodplain is outside the structure opening. The right encroachment station of 790 is less than the right abutment station of 806.428. Enc_Sta_R should be relocated outside of the structure opening area.

FW ST 03S2L SECNO: 2963.447

This is Section 2 of a hydraulic structure. The left encroachment station is within the structure opening area. The left station effective of 687.29 for the 1%-annual-chance profile is less than the left most abutment station of 689.3084. The 1%-annual-chance floodplain is outside the structure opening. The left encroachment station of 690 is greater than the left most abutment station of 689.3084. Enc_Sta_L should be relocated outside of the structure opening area.

FW ST 03S3L SECNO: 3818.981

TThis is Section 3 of a hydraulic structure. The left encroachment station is within the structure opening area. The left station effective of 508.21 for the 1%-annual-chance profile is less than the left most abutment station of 509.93. The 1%-annual-chance floodplain is outside the structure opening. The left encroachment station of 530 is greater than the left most abutment station of 509.93. Enc_Sta_L should be relocated outside of the structure opening area.

FW ST 03S3R SECNO: 8889.332

This is Section 3 of a hydraulic structure. The right encroachment station is within the structure opening area. The right station effective of 796.35 for the 1%-annual-chance profile is greater than the right most abutment station of703.688. The 1%-annual-chance floodplain is outside the structure opening. The right encroachment station of 700 is less than the right most abutment station of 703.688. Enc_Sta_R should be relocated outside of the structure opening area.

FW ST 03S3R SECNO: 3818.981

This is Section 3 of a hydraulic structure. The right encroachment station is within the structure opening area. The right station effective of 677.49 for the 1%-annual-chance profile is greater than the right most abutment station of675.59. The 1%-annual-chance floodplain is outside the structure opening. The right encroachment station of 675 is less than the right most abutment station of 675.59. Enc_Sta_R should be relocated outside of the structure opening area.

Appendix B

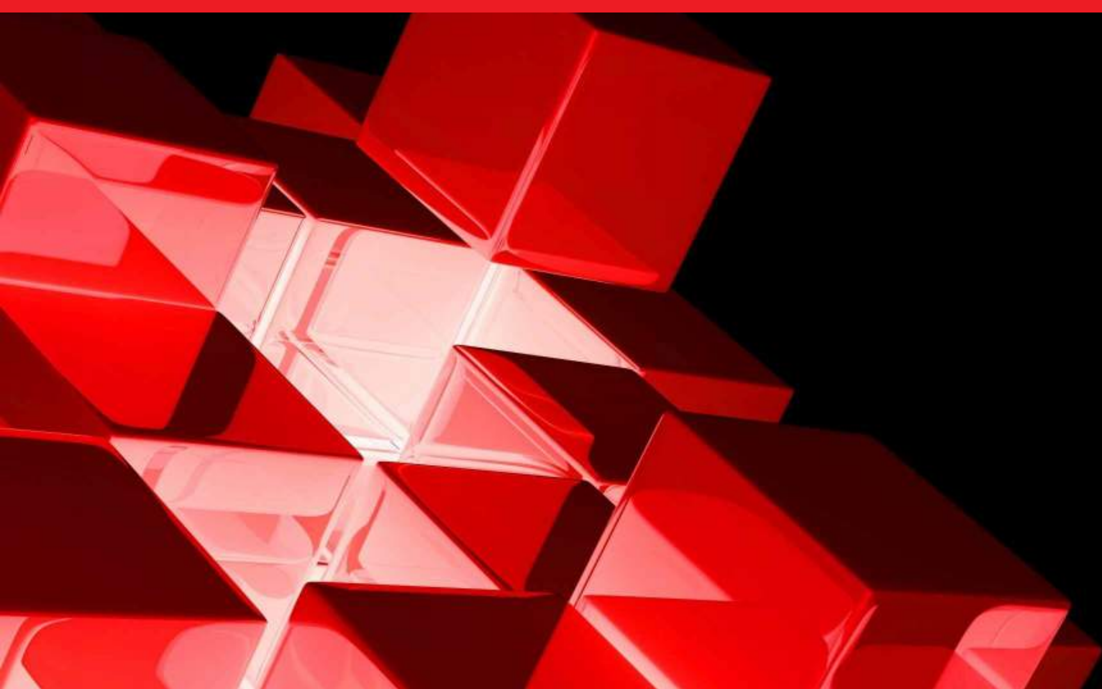
Included -

- HEC-RAS Model (digital only)

JEO CONSULTING GROUP INC

Maple River Floodway Analysis

Ida Grove, Iowa



June 2019

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Appendix A –
- STARR II Guidance Memo
- FEMA guidance document "Guidance for Flood Risk Analysis and Mapping – Levees", February, 2019 (digital only)
- cHECk-RAS Floodway Check

Appendix B – HEC-RAS Model (digital only)

1.0 PROJECT BACKGROUND

The City of Ida Grove, Iowa has been coordinating with Iowa Flood Center (IFC), the Iowa Department of Natural Resources (IDNR) and FEMA Region VII (FEMA) to complete a revised flood study for Maple River, Odebolt Creek, and Badger Creek in the City of Ida Grove as part of a countywide Digital Flood Insurance Rate Map (DFIRM) update for Ida County, Iowa. As part of this process, Iowa Flood Center has developed a detailed hydraulic model for Ida Grove using a 1D/2D modeling approach. This hydraulic model has been finalized through FEMA's independent technical review process. A separate 1D, Steady State model is being produced by JEO to perform a floodway analysis. The purpose of this technical memo is to describe the technical procedures used for the development of the floodway analysis for Maple River.

2.0 METHODOLOGY AND MODELING

2.1 Base Model Development and Calibration

A baseline 1D, steady state HEC-RAS version 5.0.5 model was created for Maple River and calibrated to the water surface profile for the 1% annual chance event from the existing conditions 1D/2D hydraulic model. Using Arc-GIS and HEC-GeoRAS software a base geometry file was created which included the 1D portion of the existing 1D/2D model cross sections with the geometry of each cross section extended to high ground resulting in a complete 1D geometry for the 1D steady state model. Maple River has multiple non-accredited but hydraulically significant levee segments on both banks; for the purposes of the floodway analysis the levee segment topography was included in the geometry, but it was assumed floodplain area landward of the levee embankments is effective flow as is required for a 1D natural valley analysis according to Section 6.12.2 of the February 2019 FEMA document "Guidance for Flood Risk Analysis and Mapping – Levees". This guidance is provided in Appendix A. Alignment and location of the levee embankment locations are show on Figures 1 and 2.

Analysis was then completed to determine an appropriate calibration tolerance between the 1% annual chance water surface elevation (WSE) from the equivalent 1D model and the existing 1D/2D floodplain model using guidance provided by STARR II which is provided in Appendix A. The analysis compared the WSE of all secondary flow areas in the floodplain to the main channel and the portion of the flood volume conveyed by the secondary floodplain flow paths. Floodplain flow area water surface elevations were calculated using tools within Arc-GIS. It was determined most cross-sections fall into the categories of Case 2a and Case 2b and therefore should be calibrated to a tolerance of 0.1 feet of the main channel average WSE. Cross-sections 14131.08, 13971.75, 13938.8 and 675.475 fall into the category of Case 2c. Target WSE and tolerances for these cross-sections were determined following the STARR II guidance and are reported in Table 1. See appendix A for the complete STARR II memo and case descriptions.

A 1D steady state run was completed using the IFC reported peak flows as shown in Table 1 and the same downstream normal depth boundary condition of 0.0007 ft/ft used in the 1D/2D hydraulic model. Model calibration was then achieved through adjustments to manning's n values and ineffective flow area locations on a cross-section by cross-section basis. Results from the calibration effort are shown in Table 1.

2.2 Floodway Analysis

A floodway analysis was completed using the guidelines provided in the February 2019 FEMA document “Guidance for Flood Risk Analysis and Mapping – Levees”. Using the calibrated 1D geometry model as the base model, a natural valley floodway analysis was completed to determine an equal conveyance reduction floodway. Floodway encroachments were placed riverward of the levee system segments, where applicable and feasible within standard surcharge requirements of the floodway analysis. Analysis results included floodway surcharges ranging from 0.44 feet to 1.00 feet. Results of the analysis are shown in Table 2.

The floodway check was run in cHECk-RAS. Results of the floodway check are provided in Appendix A. The majority of comments for the floodway check related to bank station placement and encroachments within the opening area of bridges. For the bridge locations, due to the physical layout of the stream and the relationship of the encroachments at these locations to the upstream and downstream floodway encroachments, it is JEO’s opinion that the encroachments are in appropriate locations. No changes to the floodway analysis were made in response to cHECk-RAS comments from the floodway check.

Table 1 - Model Calibration Results

River Station	LOB WSE	ROB WSE	Channel WSE	Max WSE Difference*	Main Channel Peak Flow (cfs)	IFC Reported Peak Flow	% Peak Flow Conveyed by Secondary Channel	STARR II Memo Case	Target WSE	Calibrated 1D Max WSE	1D WSE Difference**
21560.2	1220.8	1221.0	1221.3	0.44	18241	18241	0.00	2a	1221.28	1221.25	0.03
18861.89	1219.9	1219.9	1219.4	0.47	16178		0.11	2a	1219.42	1219.52	-0.10
18636.03	1219.5	1219.8	1219.7	-0.18	15106		0.17	2a	1219.70	1219.71	-0.01
17749.33	1219.0	1219.4	1218.7	0.73	15150		0.17	2b	1218.68	1218.66	0.02
17034.9	1218.6	1218.8	1218.8	-0.13	15210		0.17	2a	1218.78	1218.72	0.06
16450.42	1218.4	0.0	1218.3	0.13	14806		0.19	2a	1218.31	1218.23	0.08
15771.08	1217.8	1217.7	1217.4	0.34	17358		0.05	2a	1217.43	1217.53	-0.10
15364.57	1217.0	1217.3	1216.6	0.70	18011		0.01	2b	1216.63	1216.64	-0.01
15014.78	1216.7	1216.5	1216.4	0.29	17979		0.01	2a	1216.37	1216.46	-0.09
14728.49	1216.5	1216.5	1216.6	0.00	17991		0.01	2a	1216.55	1216.52	0.03
14588.08	1216.2	1216.2	1216.2	0.00	17973		0.01	2a	1216.23	1216.33	-0.10
14428.61	1216.0	1216.2	1216.2	-0.15	16813		0.08	2a	1216.20	1216.24	-0.04
14131.08	1215.2	1216.2	1215.8	-0.92	13312	19607	0.32	2c	1215.78	1215.87	-0.09
13971.75	1215.1	1216.2	1215.8	-1.16	11860		0.40	2c	1215.79	1215.78	0.01
13938.8	1215.1	1216.2	1215.8	-1.17	11860		0.40	2c	1215.79	1215.49	0.30
13786.25	1215.1	1216.2	1215.3	0.89	16686		0.15	2b	1215.30	1215.39	-0.09
13529.67	1215.0	1216.2	1215.2	0.92	16633		0.15	2b	1215.24	1215.32	-0.08
13062.01	0.0	1216.2	1215.0	1.20	16496		0.16	2b	1214.97	1214.97	0.00
12758.65	0.0	1216.1	1214.8	1.30	16895		0.14	2b	1214.81	1214.76	0.05
12429.2	1213.9	1215.0	1214.9	-0.92	16895		0.14	2b	1214.86	1214.81	0.05
12123.8	1214.4	1214.7	1214.7	-0.28	14009		0.29	2a	1214.69	1214.6	0.09
11819.62	1214.2	1214.4	1214.4	-0.27	13178	19750	0.33	2a	1214.44	1214.35	0.09
11521.6	1213.9	1213.9	1213.8	0.17	16647		0.16	2a	1213.75	1213.84	-0.09
10678.25	1213.5	1213.4	1213.5	-0.08	16501		0.16	2a	1213.48	1213.57	-0.09
9998	1212.9	1213.1	1212.8	0.25	16610		0.16	2a	1212.83	1212.91	-0.08
9374.935	1212.6	1212.7	1212.6	0.08	16281		0.18	2a	1212.64	1212.6	0.04
8775	1211.9	1212.2	1211.6	0.62	17228		0.13	2b	1211.57	1211.61	-0.04
8317.427	1211.4	1211.8	1211.6	-0.22	17271		0.13	2a	1211.60	1211.62	-0.02
7485	1210.1	1210.4	1210.3	-0.21	18671		0.05	2a	1210.34	1210.26	0.08
6896.634	1209.9	1210.1	1210.2	-0.23	18691		0.05	2a	1210.17	1210.11	0.06
6424	1209.8	1210.0	1210.2	-0.40	16492		0.16	2a	1210.18	1210.12	0.06
5959	1209.7	1209.5	1209.6	0.13	16599		0.16	2a	1209.56	1209.48	0.08
5439	1209.5	1209.3	1209.4	0.13	16769		0.15	2a	1209.36	1209.27	0.09
4931	1209.2	1208.9	1208.7	0.49	15666		0.21	2a	1208.68	1208.76	-0.08
4344.629	1208.8	1208.5	1208.7	0.13	12941		0.34	2a	1208.65	1208.56	0.09
3643	1208.5	1208.4	1208.3	0.23	13574		0.31	2a	1208.30	1208.38	-0.08
2949	1208.4	1208.2	1208.4	-0.19	10204		0.48	2a	1208.42	1208.35	0.07
2184	1208.3	1208.2	1208.1	0.17	10425		0.47	2a	1208.10	1208	0.10
1902	1208.1	1208.1	1207.9	0.19	11476		0.42	2a	1207.94	1207.97	-0.03
1526	1208.0	1208.1	1207.7	0.42	12365		0.37	2a	1207.70	1207.78	-0.08
675.475	1207.9	1207.5	1207.3	0.96	12620		0.36	2c	1207.27	1206.93	0.34

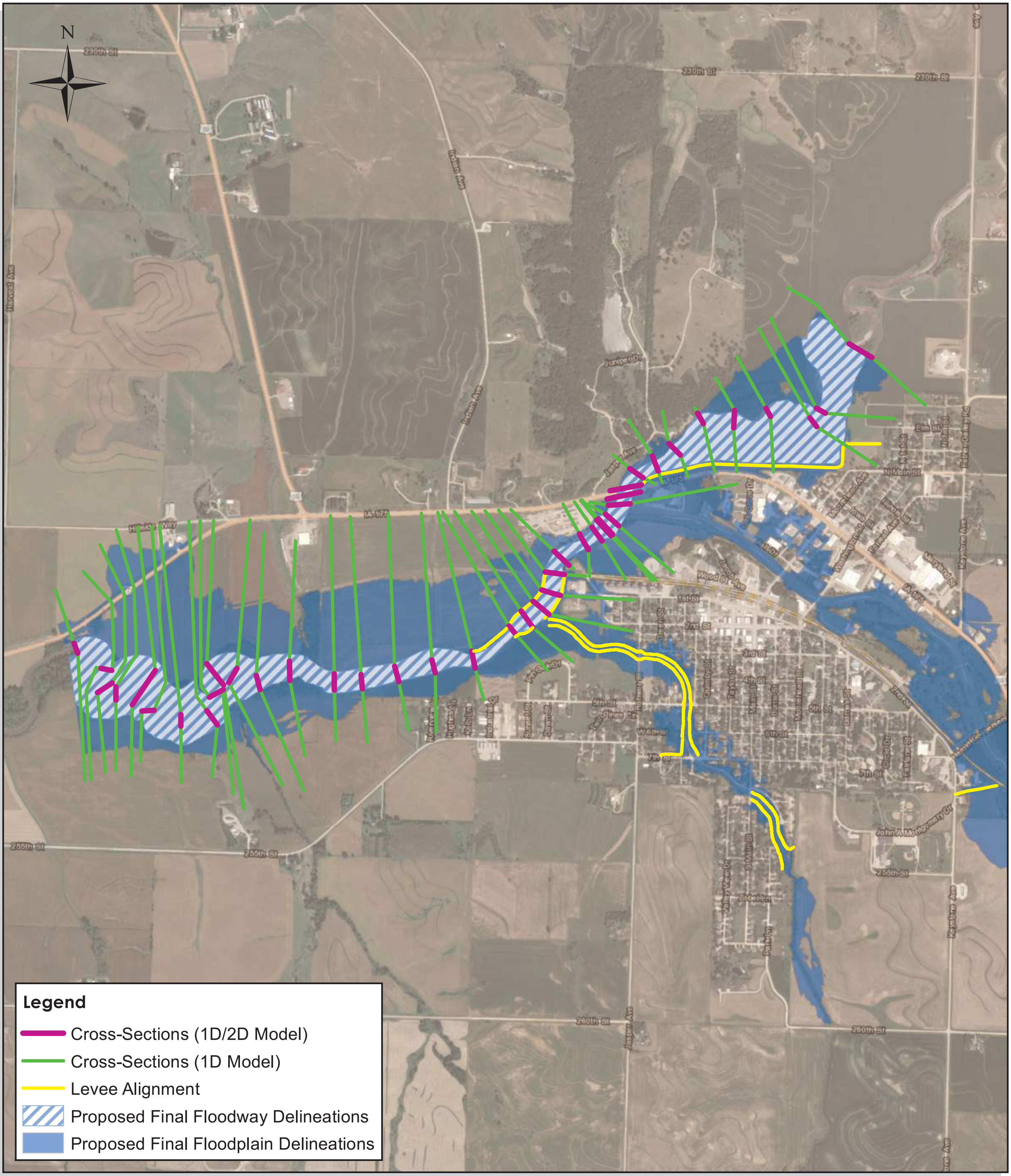
*WSE difference between the target WSE from the 1D/2D model and the left overbank (LOB) or right overbank (ROB). Reported value is the greater of the two.
**WSE difference between the target WSE from the 1D/2D model and the calibrated 1D maximum WSE. WSE Tolerance for all cross sections was +/- 0.1 feet with the exception of cross sections 14131.08, 13971.75, 13938.8 and 675.475 which followed Case 2c. Tolerance for these cross sections was determined to be +/- 0.29, 0.46, 0.46 and 0.35 feet, respectively.

Table 2 - Floodway Results

River Sta	Profile	W.S. Elev	Prof Delta WS	E.G. Elev	Top Wdth Act	Q Left	Q Channel	Q Right	Enc Sta L	Ch Sta L	Ch Sta R	Enc Sta R
		(ft)	(ft)	(ft)	(ft)	(cfs)	(cfs)	(cfs)	(ft)	(ft)	(ft)	(ft)
21560	Floodway	1221.94	0.71	1222.30	785.47	4314.17	12257.66	1669.17	1315.26	1465.33	1591.49	2100.73
18862	Floodway	1219.92	0.44	1221.07	482.37	194.58	18045.68	0.74	800.00	1168.93	1308.03	1309.03
18636	Floodway	1220.24	0.56	1220.75	982.51	1242.85	14085.58	2912.56	670.00	1208.56	1331.79	1652.51
17749	Floodway	1219.21	0.60	1220.23	810.17	3234.06	15006.55	0.39	154.00	957.26	1060.14	1061.14
17035	Floodway	1219.35	0.66	1219.83	823.14	2237.85	16002.73	0.41	170.00	837.04	992.14	993.14
16450	Floodway	1219.03	0.80	1219.65	818.01	3106.84	15131.96	2.19	185.00	869.24	1002.01	1003.01
15771	Floodway	1218.44	0.91	1219.34	419.08	1985.69	16253.79	1.52	178.00	469.41	596.08	597.08
15365	Floodway	1217.47	0.80	1219.03	204.20	851.94	17385.80	3.26	175.00	259.64	378.20	379.20
15015	Floodway	1217.29	0.83	1218.77	206.16	1547.62	16690.26	3.13	195.00	298.95	400.16	401.16
14728	Floodway	1217.43	0.91	1218.35	250.00	1251.01	16162.34	827.64	120.00	187.28	321.10	370.00
14669.76BR U	Floodway	1217.34	0.92	1218.30	238.00	1417.29	15907.45	916.27	120.00	187.28	321.10	370.00
14669.76BR D	Floodway	1217.31	0.98	1218.25	231.00	1188.94	16150.57	901.49	130.00	194.90	334.64	370.00
14588.08	Floodway	1217.3	0.97	1218.23	240.00	1059.54	16530.20	651.27	130.00	194.90	334.64	370.00
14429	Floodway	1217.24	1.00	1218.15	345.00	1045.01	15411.58	1784.41	1795.00	1895.24	2001.19	2140.00
14131	Floodway	1216.84	0.97	1217.94	310.00	984.99	16900.92	1721.09	1170.00	1267.60	1381.39	1480.00
13972	Floodway	1216.55	0.77	1217.77	309.48	1885.43	15306.94	2414.63	1009.00	1172.46	1266.85	1460.00
13956.76BR U	Floodway	1216.37	0.86	1217.73	301.13	1665.20	15784.56	2157.24	1009.00	1172.46	1266.85	1460.00
13956.76BR D	Floodway	1216.41	0.85	1217.65	312.52	1882.39	15589.94	2134.67	980.00	1138.95	1241.65	1392.00
13939	Floodway	1216.4	0.92	1217.63	316.42	1734.50	15666.12	2206.38	980.00	1138.95	1241.65	1392.00
13786	Floodway	1216.38	0.98	1217.47	210.00	438.38	18580.91	587.71	790.00	825.44	955.91	1000.00
13530	Floodway	1216.26	0.95	1217.36	170.00	281.33	19186.62	139.05	490.00	507.11	646.56	660.00
13062	Floodway	1215.97	1.00	1217.14	237.72	1135.28	17660.33	811.39	185.00	240.66	356.23	422.72
12759	Floodway	1215.74	0.98	1216.93	210.00	1112.94	17458.97	1035.09	450.00	498.23	611.16	660.00
12432	Floodway	1215.78	0.98	1216.54	260.00	419.17	18117.87	1069.96	1260.00	1317.58	1470.40	1520.00
12124	Floodway	1215.44	0.84	1216.25	290.00	1114.77	17269.15	1223.09	1415.00	1475.68	1635.01	1705.00
11820	Floodway	1215.15	0.80	1215.92	300.00	572.52	18222.53	954.95	880.00	939.90	1087.26	1180.00
11522	Floodway	1214.54	0.70	1215.67	142.25	0.09	19749.38	0.53	806.75	807.00	948.25	949.00
10678	Floodway	1214.11	0.54	1215.13	201.71	0.59	19404.90	344.51	578.29	579.29	736.19	780.00
9998	Floodway	1213.44	0.53	1214.74	402.88	17.40	19378.25	354.35	977.40	1007.40	1152.70	1389.27
9375	Floodway	1213.22	0.62	1214.44	311.44	22.02	19607.40	120.58	833.56	863.56	1004.74	1145.00
8775	Floodway	1212.38	0.77	1214.06	261.77	7.53	19720.09	22.39	886.57	916.57	1059.47	1243.00
8317	Floodway	1212.27	0.65	1213.53	310.06	3.98	19724.15	21.86	792.00	886.18	1031.60	1151.00
7485	Floodway	1211.07	0.81	1212.51	328.09	412.12	19326.84	11.04	1291.00	1391.35	1539.05	1710.00
6897	Floodway	1210.81	0.70	1211.78	458.14	421.10	19079.46	249.44	1821.65	1896.52	2046.18	2353.03
6424	Floodway	1210.82	0.70	1211.39	858.00	820.75	16108.70	2820.55	1592.00	1959.11	2109.51	2450.00
5959	Floodway	1210.27	0.78	1211.06	890.00	2450.54	17277.10	22.36	1650.00	2311.35	2509.55	2540.00
5439	Floodway	1210.11	0.83	1210.73	720.99	1469.94	18273.44	6.62	1330.00	1770.10	1964.20	2060.00
4931	Floodway	1209.63	0.87	1210.45	806.62	75.06	17872.95	1802.00	840.00	893.05	1070.85	1710.00
4345	Floodway	1209.48	0.92	1209.99	942.52	2804.32	15965.13	980.55	616.00	944.75	1119.76	1560.00
3643	Floodway	1209.25	0.88	1209.79	898.00	303.51	16980.99	2465.50	712.00	873.21	1041.01	1610.00
2949	Floodway	1209.19	0.84	1209.58	1044.15	4139.43	15247.39	363.18	1006.46	1656.32	1853.92	2050.61
2185	Floodway	1208.77	0.77	1209.31	1110.33	1489.24	16228.28	2032.49	855.81	1182.55	1343.35	1966.14
1903	Floodway	1208.74	0.77	1209.17	1221.55	2377.98	15260.85	2111.17	889.93	1336.92	1503.02	2111.48
1526	Floodway	1208.52	0.74	1209.07	1203.84	2552.94	16898.73	298.33	1054.39	1979.20	2136.50	2262.97
675	Floodway	1207.64	0.70	1208.73	1097.41	2145.56	17545.17	59.28	1230.00	2134.52	2254.80	2330.00

3.0 FLOODWAY MAPPING

The resulting proposed final floodway delineations are shown in Figure 1. In some locations, floodway delineations are shown at the landward levee toe as requested by the City. See Section 6.19 of "Guidance for Flood Risk Analysis and Mapping – Levees" for justification of placement. Figure 1 also shows the proposed draft floodplain delineations, provided by Iowa Flood Center.



Created By: Ann Nissen
Date: 05-16-2019
File: 190345.00

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Figure 1: Floodway/Floodplain Proposed Final Delineations

Maple River
Ida Grove, Iowa

0 1,250 2,500 Feet

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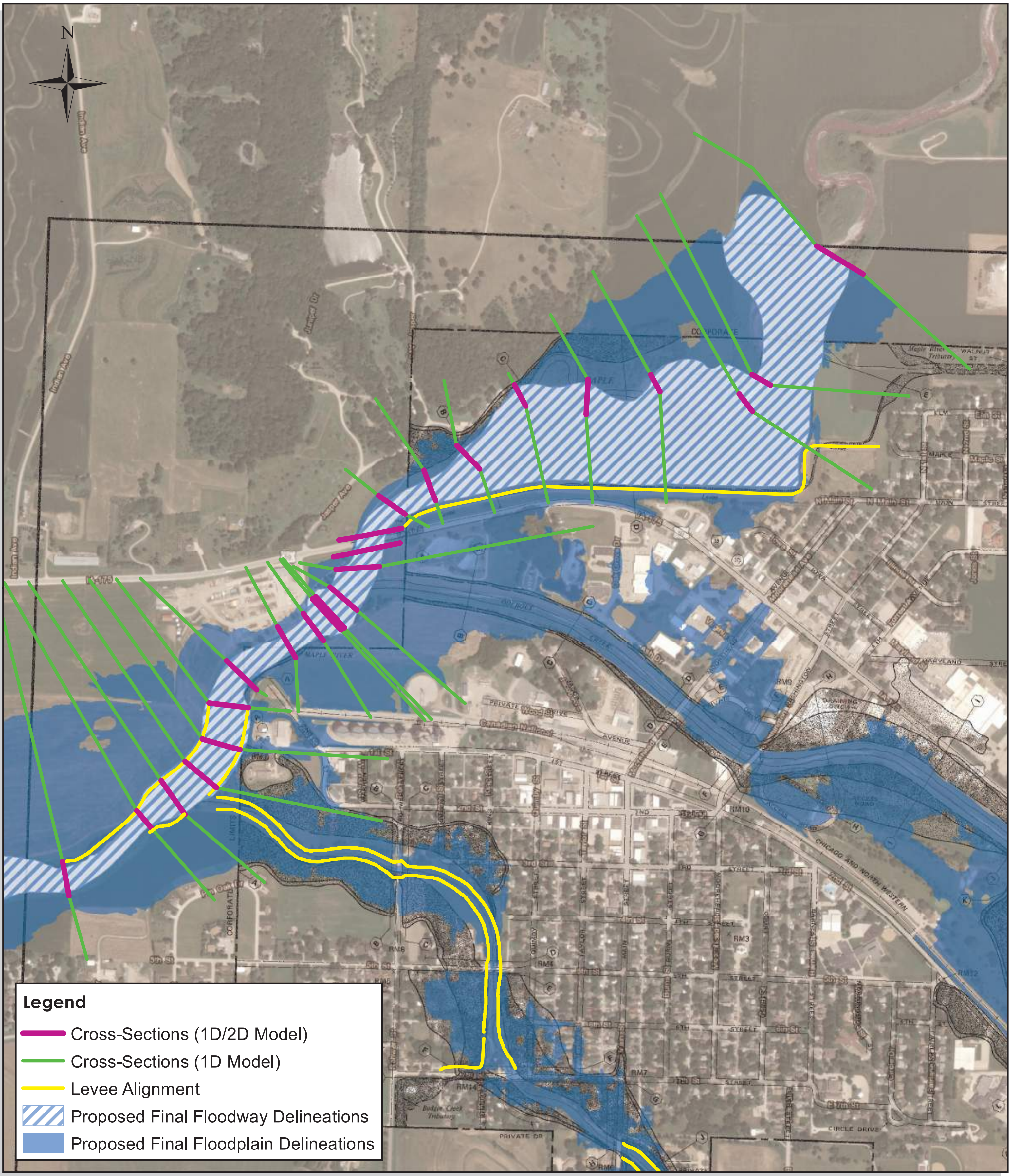

Created By: Ann Nissen
Date: 05-16-2019
File: 190345.00

Figure 2: Floodway/Floodplain Delineations vs. Effective FIRM

Maple River
Ida Grove, Iowa

0 600 1,200 Feet

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Appendix A

Included -

- STARR II Guidance Memo
- FEMA guidance document "Guidance for Flood Risk Analysis and Mapping – Levees", February, 2019 (digital only)
- cHECk-RAS Floodway Check

STARR II Strategic Alliance for Risk Reduction

Memo

To:	Rick Nusz / Dane Bailey FEMA Region VII	From:	Anish Pradhananga STARR II
File:	1D floodway on 2D modeled and mapped area	Date:	May 7, 2019

Reference: **Tolerance in discrepancy between water surface elevation from the equivalent 1D model for floodway analysis for areas modeled and mapped using 2D methodology**

Iowa Department of Water Resources (IDNR) is producing a floodway analysis in one of IDNR's projects. The project used 1D/2D analysis methodology to model and map flood hazard in the area under question. Per the Region's current guidelines IDNR is using 1D analysis approach to produce a floodway in this area.

IDNR is inquiring about the calibration tolerance in discrepancy between the 1D/2D model water surface elevation and the equivalent 1D model water surface elevation.

Issue:

IDNR is finding with relatively small effort it is possible to get the 1D water surface elevations to match 1D/2D water surface elevations within +/-0.5ft at the representative 1D cross-sections. However, it requires significant additional effort and in cases unreasonable manipulation in the 1D model parameters to get the discrepancy within a smaller tolerance, close to +/- 0.1 ft.

Though +/- 0.5 ft is generally used best practice tolerance in producing equivalent models for a variety of FEMA Flood Risk studies, we think +/-0.5 feet is too wide of the tolerance in this case. The primary purpose of the model is to identify reasonable encroachment stations to establish a floodway extent and produce a floodway data table. The floodway extents established by a model with an error tolerance half of a typical floodway surcharge has high uncertainty in reliability of floodway extents and the resulting floodway surcharge.

The problem is compounded by the fact that the water surface elevation estimates from a 2D model can be different across a width of a single cross-section. Thus, we think this situation requires the tolerance established based on the 2D model results and topographic condition of the area under study.

The following are our suggested solutions:

Suggested calibration tolerance approach:

A model can have a single tolerance for the entire model (all cross-sections) or the tolerance can vary at each cross-section. This will depend on uniqueness of the model. In general, the calibration approaches pointed below are used for all models

1. Adjust manning's n values and ineffective areas in 1D cross-section to calibrate 1D results within the established tolerance at the cross-section
2. In case of steady state 1D analyses match the peak flow at each cross-section to the peak flow at the cross-section location from 2D routed model. Use the same discharge values in equivalent 1D model and with floodway model.

Follow the following to establish the tolerance in discrepancy between 1D and 2D model results:

Case1: Connected flooding between the main channel and floodplain

In the majority of cases, flooding in the main channel and floodplain are connected across a cross-section in rare flood events like the 1% annual chance event. In these cases, it is likely the maximum water surface elevation is static across the 2D cells represented by a 1D cross-section. If the variation in maximum water surface elevation across representative 2D cells are within +/-0.1 ft, use the average of the water surface elevation across 2D cells as a target water surface elevation. The tolerance in discrepancy between the 2D model and 1D floodway model is the target water surface elevation +/- 0.1 ft.

If the variation in maximum water surface elevation in 2D cells are higher than +/- 0.1 ft across the representative 1D cross-section, follow Case 2.

Case 2: Disconnected flooding in the main channel and floodplain

In many instances depending on the terrain of the 2D modeled area, it is likely the main channel flooding is disconnected from the floodplain flooding. In cases where the 2D model is indicating there are possible split flow conditions and the flood risk is better represented by keeping the flow paths separate, it is suggested to develop a separate floodway model for each flowpath.

However, it is often more likely these split flows are relatively very short and/or conveys only small proportion of the flood wave being modeled. In these situations, it is reasonable to assume the main channel and floodplain flow and water surface elevations are represented by a single flow and single elevations in the model. In these situations, follow the steps below.

In all cases calibrate to meet the target water surface elevation +/-0.1ft except in case 2c.

Case 2a

1. Check the 2D model water surface elevation difference between the main channel and secondary flow area(s). If it is less than 0.5 ft. Use the average water surface elevation in the main channel as the target water surface elevation. If it is higher than 0.5 ft, follow case 2b.

Case 2b

2. Check the proportion of the flood volume conveyed by the secondary flow area. If the secondary flow area conveys less than 20% of the peak, use the main channel average water surface elevation as target elevation. If the secondary channel flow is higher than 20% of the peak flow, then follow case 2c.

Case 2c

3. If the difference between main channel average water surface elevation and secondary channel average water surface elevation is greater than 0.5 ft and the secondary channel accounts for more than 20% of the total flow, then

    a. Calculate a weighted water surface elevation

       $WSelev_{WT}$ = {(% flow conveyed by primary channel X average water surface elev in the primary channel) + (% flow conveyed in secondary channel X average water surface elev in the secondary channel)+……….}/(total flow across the channels)

$$\text{Tolerence}_{WT} = \text{WSelev at the main channel +/- (Wsel elev main channel- WSelev}_{WT})$$

4. If Tolerance_{WT} is higher than 0.5ft, the model is indicating it is not a good assumption to use a single elevation for both main and the secondary channels. In these situations, use engineering judgement to evaluate whether the floodway encroachment is likely to encroach into the entire secondary flowpath. If yes, use main channel water surface elevation as a target elevation. If the floodway is likely going to only partially encroach the secondary channel, then use +/- 0.5 ft tolerance.

Case 3 Calibration within the target elevation is not achieved

5. In some cases, the calibration to the targeted tolerance may not be achievable within reasonable adjustment to the model parameters. In these situations, document the calibration process and coordinate with the FEMA Region. FEMA Region may approve using a smaller floodway surcharge tolerance at the cross-sections where calibration could not be achieved. Below is the suggested floodway data table documentation to report the model calibration and adjustment to surcharge tolerance.

The approaches outlined above should not be used for braided streams.

LOCATION		FLOODWAY			1% ANNUAL CHANCE FLOOD WATER SURFACE ELEVATION (FEET NAVD88)			
CROSS SECTION	DISTANCE [1]	WIDTH (FEET)	SECTION AREA (SQ. FEET)	MEAN VELOCITY (FEET/SEC)	REGULATORY	WITHOUT FLOODWAY	WITH FLOODWAY	INCREASE
A	203	92	253	2.2	1,048.9	1,047.5[2]	1,048.0	0.5
B	378	110	116	4.7	1,048.9	1,048.7[2]	1,048.8	0.1
C	480	147	203	2.7	1,049.9	1,049.9	1,050.5	0.6
D	709	65	120	4.6	1,052.2	1,052.3[3]	1,052.7	0.4
E	1,025	61	98	5.6	1,055.3	1,055.3	1,055.7	0.4
F	1,219	113	228	2.4	1,056.9	1,056.9	1,057.8	0.9
G	1,496	50	131	4.2	1,059.0	1,059.0	1,059.2	0.2
H	1,818	54	86	6.4	1,062.8	1,062.7[3]	1,062.9	0.2
I	1,987	61	91	6.0	1,064.1	1,064.1	1,064.3	0.2
J	2,268	55	81	6.7	1,067.9	1,067.9	1,068.0	0.1
K	2,500	97	671	0.8	1,074.3	1,074.4[3]	1,074.7	0.3
L	3,001	59	214	2.5	1,074.4	1,074.4	1,074.7	0.3
M	3,489	39	85	6.4	1,076.0	1,076.5[3]	1,077.0	0.5[4]
N	3,747	44	83	6.5	1,078.9	1,078.9	1,078.9	0.0
					2D model Result WSEL	1D Calibrated model WSEL	1D Calibrated FW WSEL	difference based on calibrated WSEL and calibrated FW

[1] Feet above mouth
[2] Elevation computed without consideration of backwater effects from Inundation River
[3] Elevation computed based on 1D calibrated Floodway Model
[4] Due to floodway calibration, maximum allowable surcharge

<< Additional footnotes based on calibration tolerance

TABLE 24

FEDERAL EMERGENCY MANAGEMENT AGENCY

EXAMPLE COUNTY, NE

AND INCORPORATED AREAS

FLOODWAY DATA

FLOODING SOURCE: EXAMPLE STREAM

Floodway Check

River	Reach	RS	Method	Surcharge	EncStaL	EncStaR	LStaEff	RStaEff	LeftSlope	RightSlope	Structure	LateralWeir Station
Maple_R	Maple_R	21560					696.08	2428.88				
		21560	1	0.71	1315.26	2100.73	1315.26	2100.73	0.08	-0.19		
		18862					795.45	2485.84				
		18862	1	0.44	800	1309.03	800	1309.03	0.72	1.38		
		18636					203.52	2562.41				
		18636	1	0.56	670	1652.51	670	1652.51	0.29	-0.37		
		17749					20.99	1763.06				
		17749	1	0.6	154	1061.14	154	1061.14	-0.15	0.04		
		17035					33.8	1424.39				
		17035	1	0.66	170	993.14	170	993.14	0.01	-0.02		
		16450					39.17	1014.96				
		16450	1	0.8	185	1003.01	185	1003.01	-0.58	0		
		15771					38.62	707.99				
		15771	1	0.91	178	597.08	178	597.08	-0.52	-0.01		
		15365					44.54	510.37				
		15365	1	0.8	175	379.2	175	379.2	0.03	-0.02		
		15015					82.34	505.42				
		15015	1	0.83	195	401.16	195	401.16	-0.07	0.22		
		14728					106.53	391.89				
		14728	1	0.91	120	370	120	370	0	-0.08		
		14669.76					106.78	391.61			Bridge-UP	
		14669.76					112.1	395.77			Bridge-DN	
		14669.76		0.92	120	370	120	370			Bridge-UP	
		14669.76		0.98	130	370	130	370			Bridge-DN	
		14588.08					112.1	395.77				
		14588.08	1	0.97	130	370	130	370	0.12	0.54		
		14429					1650	2327.24				
		14429	1	1	1795	2140	1795	2140	0	-0.12		
		14131					383.46	1670.48				
		14131	1	0.97	1170	1480	1170	1480	0.35	0.53		
		13972					411.94	1488.62				
		13972	1	0.77	1009	1460	1009	1460	-0.01	-1.17		
		13956.76					416.81	1483.54			Bridge-UP	
		13956.76					376.15	1428.64			Bridge-DN	
		13956.76		0.86	1009	1460	1009	1460			Bridge-UP	
		13956.76		0.85	980	1392	980	1392			Bridge-DN	
		13939					377.83	1428.07				
		13939	1	0.92	980	1392	980	1392	-0.72	-0.61		
		13786					330	1103.92				
		13786	1	0.98	790	1000	790	1000	-0.05	-0.1		
		13530					240	780.05				
		13530	1	0.95	490	660	490	660	0.06	0.09		
		13062					133.81	788.55				
		13062	1	1	185	422.72	185	422.72	-0.03	-0.06		
		12759					374.89	1095.12				
		12759	1	0.98	450	660	450	660	0.09	0.06		
		12432					656.56	2368.49				
		12432	1	0.98	1260	1520	1260	1520	0.02	0.08		
		12124					618.13	2691.5				
		12124	1	0.84	1415	1705	1415	1705	-0.02	0.06		
		11820					244.77	2201.57				
		11820	1	0.8	880	1180	880	1180	-0.21	-0.32		
		11522					213	2026.45				
		11522	1	0.7	806.75	949	806.75	949	0.01	0.06		

		10678					430.65	2129.01				
		10678	1	0.54	578.29	780	578.29	780	0.03	0.27		
		9998					587.88	2482.36				
		9998	1	0.53	977.4	1389.27	977.4	1389.27	0	-0.16		
		9375					572.39	2536.43				
		9375	1	0.62	833.56	1145	833.56	1145	0	0.07		
		8775					705.87	2616.59				
		8775	1	0.77	886.57	1243	886.57	1243	0.14	-0.14		
		8317					325.53	2492.33				
		8317	1	0.65	792	1151	792	1151	0.01	0.06		
		7485					376.21	2350.54				
		7485	1	0.81	1291	1710	1291	1710	-0.04	0.23		
		6897					860.83	3266.66				
		6897	1	0.7	1821.65	2353.03	1821.65	2353.03	0.62	0.07		
		6424					699.6	3206.75				
		6424	1	0.7	1592	2450	1592	2450	0.68	-0.62		
		5959					839.47	3701.83				
		5959	1	0.78	1650	2540	1650	2540	-0.43	0.12		
		5439					635.28	4751.18				
		5439	1	0.83	1330	2060	1330	2060	-0.78	1.05		
		4931					323.7	3881.47				
		4931	1	0.87	840	1710	840	1710	0.47	-0.34		
		4345					478.33	3587.88				
		4345	1	0.92	616	1560	616	1560	-0.24	0.18		
		3643					197.3	3365.41				
		3643	1	0.88	712	1610	712	1610	0.73	-0.52		
		2949					503.87	3357.41				
		2949	1	0.84	1006.46	2050.61	1006.46	2050.61	-0.45	0.53		
		2185					346.51	3085.46				
		2185	1	0.77	855.81	1966.14	855.81	1966.14	0.44	-0.04		
		1903					353.72	3107.35				
		1903	1	0.77	889.93	2111.48	889.93	2111.48	1.26	-1.29		
		1526					428.89	3176.03				
		1526	1	0.74	1054.39	2262.97	1054.39	2262.97	-0.05	-0.08		
		675					464.98	2363.58				
		675	1	0.7	1230	2330	1230	2330				

If the Left Slope or Right Slope is more than 1, the change in floodway boundary between the two River Stations is equal to or more than 45 degrees. The Left Slope or Right Slope with the angle equal to or more than 45 degrees is shown in red. The floodway widths at these River Stations should be smoothed.

FW FW 03L SECNO: 9375

The left channel bank elevation of 1213.04 is higher than the 1-percent-annual-chance WSEL of 1212.6. Relocate the left channel bank station at or below the 1-percent-annual-chance WSEL. Do not place the bank stations at the bottom of the channel. Do not place the bank stations at the low flow channel. Use the Horizontal Variation in "n" Values option in HEC-RAS to assign different "n" values to the left bank slope, low flow channel, and the right bank slope. Let HEC-RAS compute the composite "n" value based on the depth of flow.

FW FW 03L SECNO: 8317

The left channel bank elevation of 1213.3 is higher than the 1-percent-annual-chance WSEL of 1211.62. Relocate the left channel bank station at or below the 1-percent-annual-chance WSEL. Do not place the bank stations at the bottom of the channel. Do not place the bank stations at the low flow channel. Use the Horizontal Variation in "n" Values option in HEC-RAS to assign different "n" values to the left bank slope, low flow channel, and the right bank slope. Let HEC-RAS compute the composite "n" value based on the depth of

	flow.
FW FW 03R	SECNO: 8317 The right channel bank elevation of 1211.81 is higher than the 1-percent annual chance WSEL of 1211.62. Relocate the right channel bank station at or below the 1-percent annual chance WSEL. Do not place the bank stations at the bottom of the channel. Do not place the bank stations at the low flow channel. Use the Horizontal Variation in "n" Values option in HEC-RAS to assign different "n" values to the left bank slope, low flow channel, and the right bank slope. Let HEC-RAS compute the composite "n" value based on the depth of flow.
FW FW 03R	SECNO: 7485 The right channel bank elevation of 1210.28 is higher than the 1-percent annual chance WSEL of 1210.26. Relocate the right channel bank station at or below the 1-percent annual chance WSEL. Do not place the bank stations at the bottom of the channel. Do not place the bank stations at the low flow channel. Use the Horizontal Variation in "n" Values option in HEC-RAS to assign different "n" values to the left bank slope, low flow channel, and the right bank slope. Let HEC-RAS compute the composite "n" value based on the depth of flow.
FW FW 03R	SECNO: 6897 The right channel bank elevation of 1210.44 is higher than the 1-percent annual chance WSEL of 1210.11. Relocate the right channel bank station at or below the 1-percent annual chance WSEL. Do not place the bank stations at the bottom of the channel. Do not place the bank stations at the low flow channel. Use the Horizontal Variation in "n" Values option in HEC-RAS to assign different "n" values to the left bank slope, low flow channel, and the right bank slope. Let HEC-RAS compute the composite "n" value based on the depth of flow.
FW FW 03R	SECNO: 5439 The right channel bank elevation of 1209.53 is higher than the 1-percent annual chance WSEL of 1209.27. Relocate the right channel bank station at or below the 1-percent annual chance WSEL. Do not place the bank stations at the bottom of the channel. Do not place the bank stations at the low flow channel. Use the Horizontal Variation in "n" Values option in HEC-RAS to assign different "n" values to the left bank slope, low flow channel, and the right bank slope. Let HEC-RAS compute the composite "n" value based on the depth of flow.
FW FW 03R	SECNO: 4345 The right channel bank elevation of 1208.6 is higher than the 1-percent annual chance WSEL of 1208.56. Relocate the right channel bank station at or below the 1-percent annual chance WSEL. Do not place the bank stations at the bottom of the channel. Do not place the bank stations at the low flow channel. Use the Horizontal Variation in "n" Values option in HEC-RAS to assign different "n" values to the left bank slope, low flow channel, and the right bank slope. Let HEC-RAS compute the composite "n" value based on the depth of flow.
FW ST 03BUR	SECNO: 14669.76 This is (Bridge-UP) Upstream Internal Section. The right encroachment station is within the structure opening area. The right station effective of 395.77 for the 1%-annual-chance profile is greater than the right abutment station of 394.688. The 1%-annual-chance floodplain is outside the structure opening. The right encroachment station of 370 is less than the right abutment station of 394.688. Enc_Sta_R should be relocated outside of the structure opening area.

Appendix B

Included -

- HEC-RAS Model (digital only)