US Army Corps of Engineers

Lower Missouri River Jefferson City L-142 Flood Risk Management Feasibility Study

Final Feasibility Report and Integrated Environmental Assessment

Appendix A1 – Hydrology and Hydraulics

July 2026

MISSOURI DEPARTMENT OF NATURAL RESOURCES

Lower Missouri Jefferson City L-142 FRM Study

Appendix A1

Part 1: Existing Conditions & Hydrology

Capital View Drainage District, looking southeast toward inundated airport May 28, 2019

U.S. Army Corps of Engineers
Kansas City District

Final Report, July 2026

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1 INTRODUCTION

This report summarizes project background, data development, and hydrologic input sources for the evaluation of the existing condition, future without project, and plan alternatives for the Missouri River Levee L-142 Feasibility Study in Jefferson City, Missouri. Included is a discussion of representative flows used in hydraulic modeling and flood frequency analysis as related to the Jefferson City left bank study area. Additional reports pertaining to the full hydrologic and hydraulic analysis for the Missouri River Levee L-142 Feasibility Study are provided in **Table 1**.

Table 1. Summary of Reports Pertaining to Hydrologic and Hydraulic Analysis

Identifier	Report Name	Description
Appendix A1 – Part 1	Existing Conditions & Hydrology	Project Introduction, Data Development, and Hydrology Background
Appendix A1 – Part 2	Hydraulic Model Development	Existing Conditions Model Development
Appendix A1 – Part 3	Alternatives Hydraulic Analysis	FWOP and Alternatives Analysis
Appendix A1 – Part 4	Hydrologic and Hydraulic Federal Interest Plan Refinement	Further refinement to be completed post-public review.
Appendix A2	Climate Change Assessment	Assessment of potential climate change vulnerabilities to flood risk reduction alternatives in accordance with ECB 2018-14

2 BACKGROUND

This study focuses on addressing impact to the Capital View leveed area in Jefferson City, Missouri under the authorization of the 1941 and 1944 Flood Control Acts which included unconstructed levee unit L-142.

2.1 Modeled Area

The modeled area includes the Missouri River from Boonville to Hermann, spanning river miles 98 to 197, through central Missouri. The drainage area of the Missouri River at Jefferson City is approximately 507,500 square miles. The immediate study area is located within the Hydrologic Unit Code 10 (HUC-10) Jefferson City - Missouri River watershed 1030010213, outlined in **Figure 1**.

The Missouri River watershed within the HUC-10 upstream of Jefferson City is generally rural and used for agriculture. The Missouri River overbank through central Missouri is protected by locally constructed and operated levees. The Moreau River and Osage River confluences with the Missouri River are six and fourteen miles downstream of the Jefferson City United States Geological Survey (USGS) gage, respectively. The Osage River has multiple mainstem dams including Bagnell Dam, privately owned by Ameren, and the Harry S. Truman Dam, a USACE operated structure utilized for flood control. The Osage River confluence with the Missouri River is downstream of Jefferson City and is regulated for a flow target on the Osage River at St. Thomas, MO and the Missouri River at Hermann, MO.

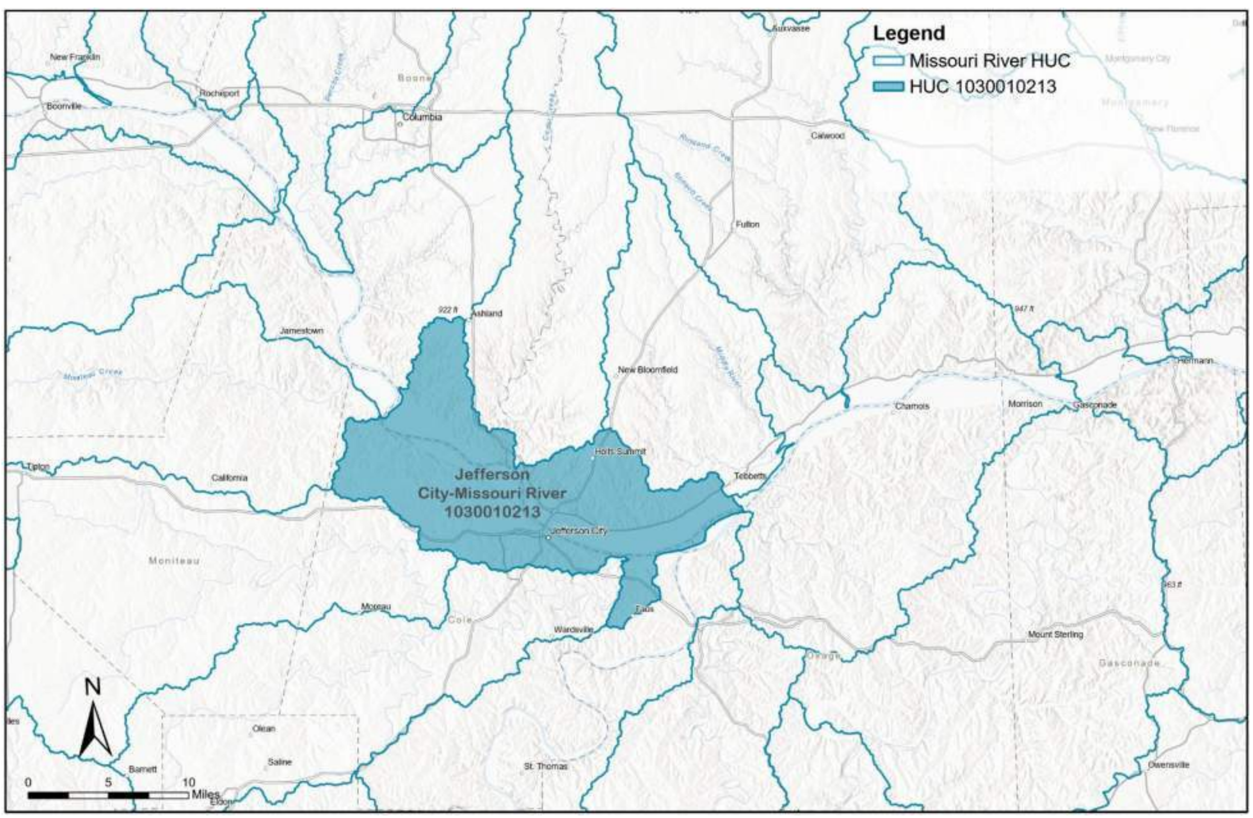

Figure 1. Jefferson City - Missouri River Watershed (HUC 1030010213)

2.2 Study Area

The study area for the Missouri River Levee L-142 Feasibility Study is located within Callaway County, bordered by Cole County south of the river. Many non-federal and private levees occupy the area, with locally operated levees generally paralleling closely to the riverbank.

A majority of Jefferson City, including the Missouri State Capitol Building, sits above the Missouri River on the right-bank bluff. The U.S. Highway 54/63 bridge connects the city core to the commercial, industrial, and public infrastructure located on the lower lying left bank study area. On the right bank, Wears Creek and Boggs Creek tributaries flow through Jefferson City. The left bank tributaries are less developed and include Turkey Creek and Niemans Creek.

Also of interest for this study is the Church Farm Levee, a private levee located at the Church Farm Conservation Area, upstream of Jefferson City from river miles 152 to 154.5. This bottomland area in Cole County, is managed by the Missouri Department of Conservation and utilized for recreational hunting, hiking, and wildlife viewing.

2.3 Capital View Drainage District

Shown in **Figure 2**, the Capital View (CV) Drainage District is bounded by the Missouri River (River Mile (RM) 139.8-144.5) to the south, Turkey Creek to the west, bluffs to the north, and Niemans Creek to the east. Existing flood risk management measures for the Capital View Drainage District consist of 8.5 miles of earthen levee and nine gravity drains. Construction documentation of the original private levee is not available, as construction of non-engineered private levees in this area typically consisted of years of gradual placement of earthen material.

The Capital View levee begins at high ground northeast of the US 54/63 interchange, parallels the left bank of an unnamed tributary to its confluence with Turkey Creek, then continues along the left bank of Turkey Creek to the confluence with the Missouri River. The levee follows the Missouri River incorporating high ground of the Capital Sand dredging operation plant and continues to Niemans Creek, where it follows the right bank and ties into high ground at Highway 94.

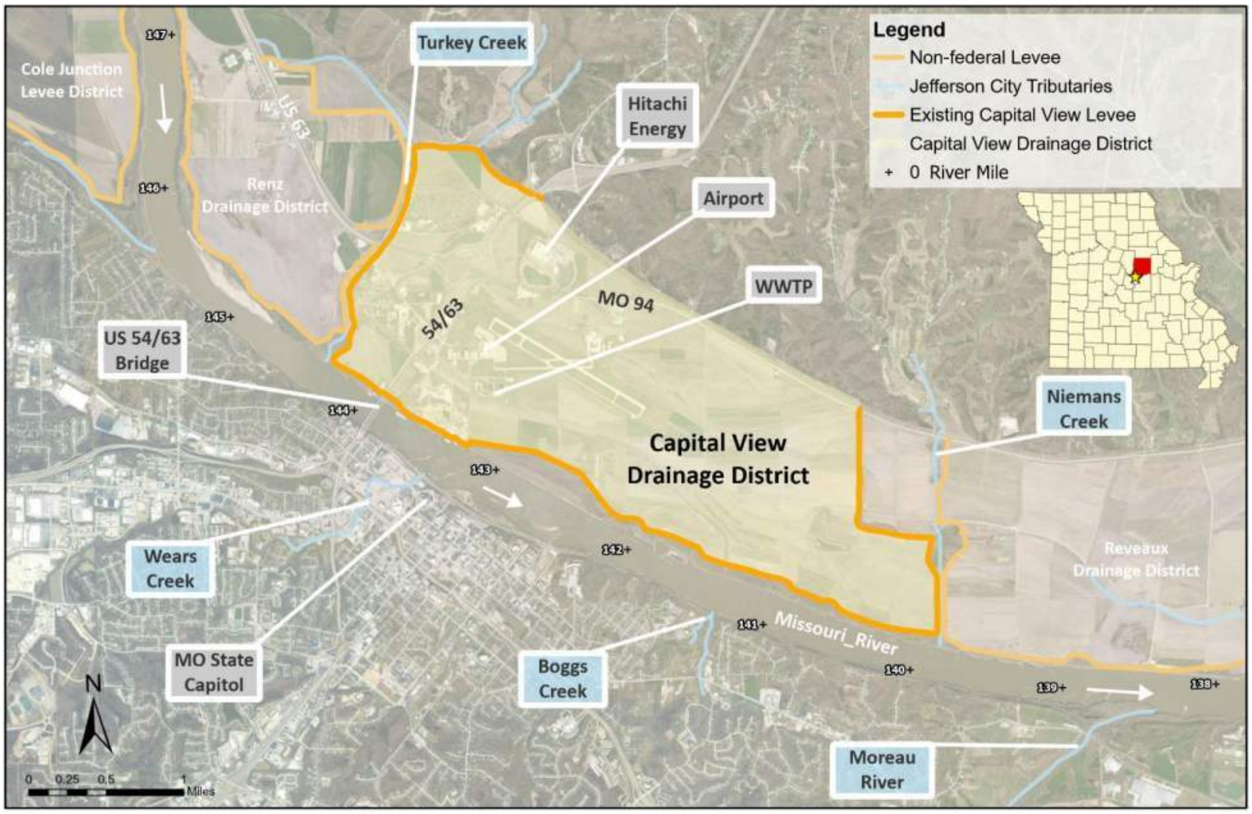

Figure 2. Capital View Levee Drainage District Delineation

Land use within the Capital View leveed area is varied and includes the Jefferson City Memorial Airport, the city's regional wastewater treatment plant, Air National Guard facility, commercial development, recreational facilities, and agricultural land. The U.S. Highway 54/63 bridge crosses the Missouri River at river mile 144, with the northern bridge abutment and highway interchange located within the Capital View Drainage District.

Public stormwater infrastructure within the leveed area includes enclosed storm sewers throughout the airport property. Missouri Department of Transportation owned bridges are located through the area, serving Highways 63, 54, and 94. Missouri River historical records show the presence of pile dikes, since buried with accretion, west of Niemans Creek that extend well into the leveed area.

Turkey Creek has been identified during public meetings as an area of regular flood fight within the existing Capital View levee system. The Turkey Creek channel is restricted on both sides by non-federal and private levees situated on the top of the creek bank, with little opportunity for overbank flow. Turkey Creek drains approximately 11 square miles including both floodplain and bluff, as delineated in **Figure 3**. Backwater conditions with the Missouri River is the driving condition for floods at the Capital View levee Turkey Creek tieback.

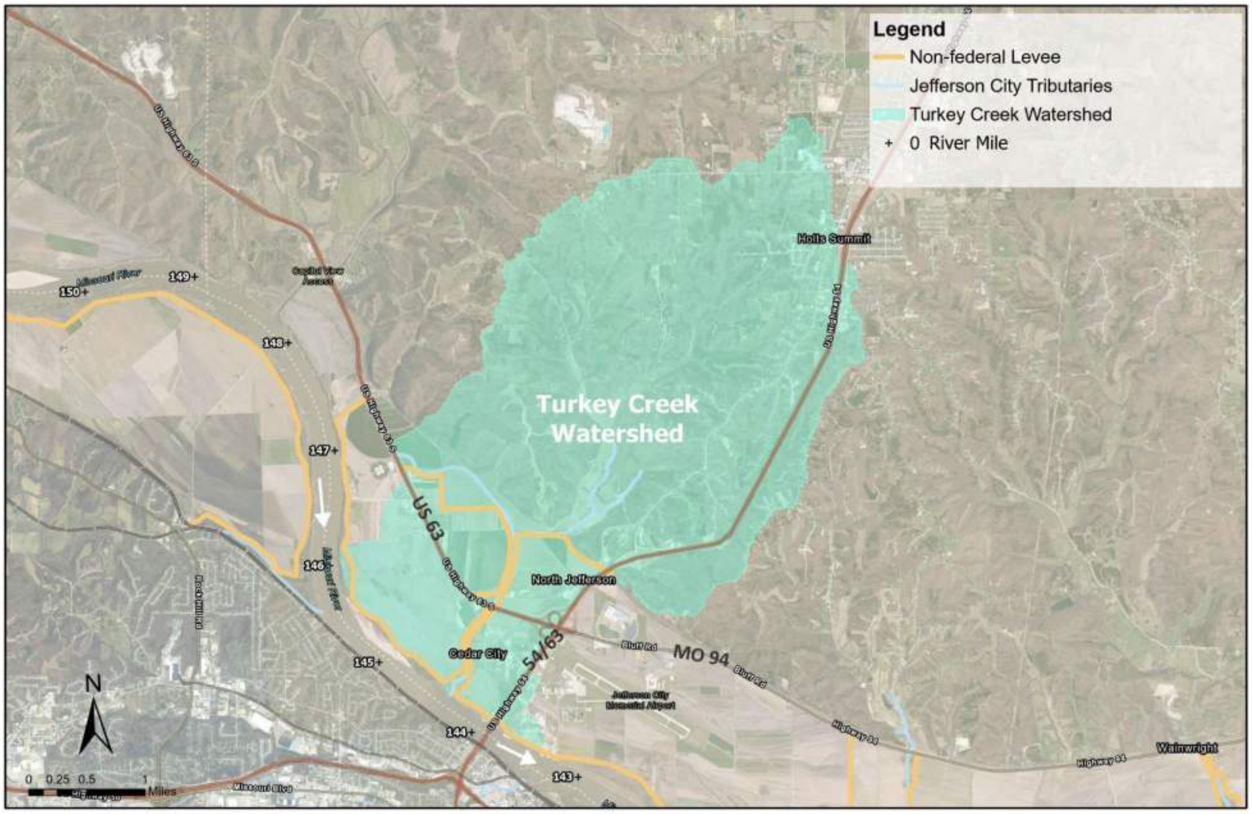

Figure 3. Turkey Creek Watershed Delineation

Flood fighting efforts for the existing Capital View levee typically focus on the Turkey Creek corridor where upstream levee failures have been observed at Renz North levee, allowing the Missouri River to connect with Turkey Creek, pressuring the Capital View levee near the western intersection with the Katy Trail. The Cedar City Road bridge is another common location requiring flood fight. This bridge sits low in the floodplain and is 85-feet long with an overtopping elevation of 550 feet, similar in elevation to the overtopping crest of the Capital View levee. Reference **Figure 4** for bridge plans. When the Missouri River creates a backwater condition through this bridge, the bridge hydraulically controls, creating an increase in water surface elevation upstream. During current floodfight operations, additional fill material is brought in to armor the left wingwalls against undermining and to raise the channel overbank to delay overtopping of Capital View levee at this location.

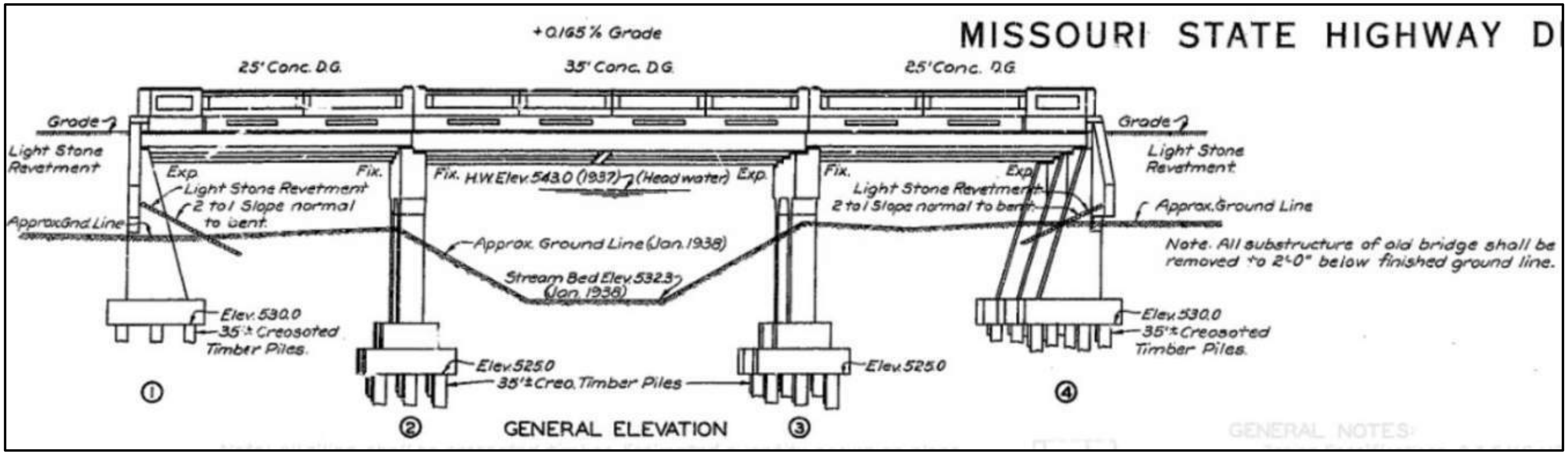

Figure 4. Cedar City Road Cross Section over Turkey Creek

Source: MoDOT Bridge G0915 As-Builts

Evidence of streambank repair and armoring were observed onsite at Turkey Creek at Oilwell Road, as depicted in **Figure 5**.

Figure 5. Turkey Creek at Oilwell Road, looking upstream (June 18, 2024)

2.4 Project History

Missouri River bank-full discharge at Jefferson City is approximately 250,000 cfs at the Jefferson City gage, which corresponds to a flow event between 50% AEP and 20% AEP. The average bed slope through the project reach is 1 foot per mile.

The 2019 flood caused extensive damages to agriculture and infrastructure in communities in the states of Iowa, Nebraska, Kansas, and Missouri. In the aftermath of the flood, the four states formed the Flood Recovery Advisory Working Group to identify actions that could reduce systemwide flood risk, reduce reoccurring damages, and improve system resilience. In addition to the system-wide evaluations as described below, Jefferson City was identified as a priority area for flood risk evaluation, prompting this site-specific feasibility study.

The Lower Missouri River has also been of interest to USACE in the past. A summary of key current and past studies pertaining to the hydrology and hydraulics of the study area are presented in the following sections.

2.4.1 Missouri River Levee System Definite Project Report (1947)

The Missouri River Levee System Definite Project Report, also known as the Pick-Sloan Plan, is the original authorization for many agricultural levee systems on the Missouri River and proposed a minimum floodway width of 4,500 feet for the Missouri River from the Grand River to Osage River confluences. This plan was not fully constructed.

2.4.2 General Reevaluation Report & Environmental Assessment (2001)

The Jefferson City L-142 study area previously underwent analysis resulting in a recommendation for construction of a federal levee. The project moved forward to design but was paused in 2007 when the sponsor declined to sign the Project Partnership Agreement.

2.4.3 Lower Missouri River Flood Risk Resiliency Plan Section 22 PAS (2022)

The *Considerations and Analysis for Flood Risk Resiliency for the Lower Missouri River (Nebraska, Iowa, Kansas, and Missouri) Section 22 Planning Assistance to States (PAS)* study investigated conceptual measures and strategies to lessen flood risk vulnerability and improve flood risk resilience throughout the Lower Missouri River Basin. The four state partners assisted in prioritizing areas between Sioux City, Iowa and St. Louis, Missouri for potential action. The PAS study analyzed vulnerable locations on the Lower Missouri River and conducted stakeholder outreach to better understand recurrent problem areas and impacts. Jefferson City was identified as one of the critical areas to be considered for feasibility-level study with this analysis.

2.4.4 Lower Missouri River Flow Frequency Study (2023)

The *Missouri River Flow Frequency Study – Yankton, South Dakota to Hermann, Missouri* was completed in June 2023 and incorporates new data and statistical analysis and applies state of the practice methodologies to update the Lower Missouri River flow frequencies, including flow and annual exceedance probabilities, at 10 gages. This study extends the period of record and replaces the Missouri River flow frequencies developed with the 2004 Upper Mississippi River System Flow Frequency Study (UMRSFFS).

2.4.5 Lower Missouri River Stage Frequency Analysis (Expected in 2025)

The Lower Missouri River Stage Frequency analysis will update hydraulic modeling and associate 2023 Flow Frequency study flows to determine stage frequencies at select river locations. The stage analysis of the Lower Missouri River was not complete at the time of this study and is anticipated to be complete in 2025.

2.4.6 Lower Missouri River Flood Risk & Resiliency Study (Expected in 2027)

This feasibility study will evaluate the Lower Missouri River system over 735 river miles from Sioux City, Iowa to the mouth near St. Louis, Missouri. In partnership with Missouri, Kansas, Iowa, and Nebraska, the study will create a vision for a more resilient future for the Lower Missouri River, with a focus on flood risk management. Outcomes of the study will include system analysis providing context to the initial studies, identify and recommend any additional spin-off feasibility studies and their locations, and identify potential policy recommendations that would support the system and potential actions for local stakeholders. The study is scheduled to be complete in March 2027. The Jefferson City study is not directly tied to the findings of the system plan, but many of the same resources will be used in the development of both analyses.

2.4.7 Wears Creek Studies (1953, 1974, 1980, 1986, 2003)

Directly across the river from the Capital View levee, Wears Creek is a right-bank tributary to the Missouri River with a confluence located 0.5-mile downstream of the U.S. Highway 54/63 bridge. The drainage area encompasses 14.1 square miles, delineated in **Figure 6**, and is routed through the city of Jefferson City as an open channel, with many bridges and a couple of enclosed sections. The Wears Creek system is subject to two very different types of flooding including flash runoff floods from the upstream watershed and backwater floods from the Missouri River (USACE 1974). The scope of the current Jefferson City study does not extend to the right bank of the Missouri River but will document the effects of proposed left-bank alternatives.

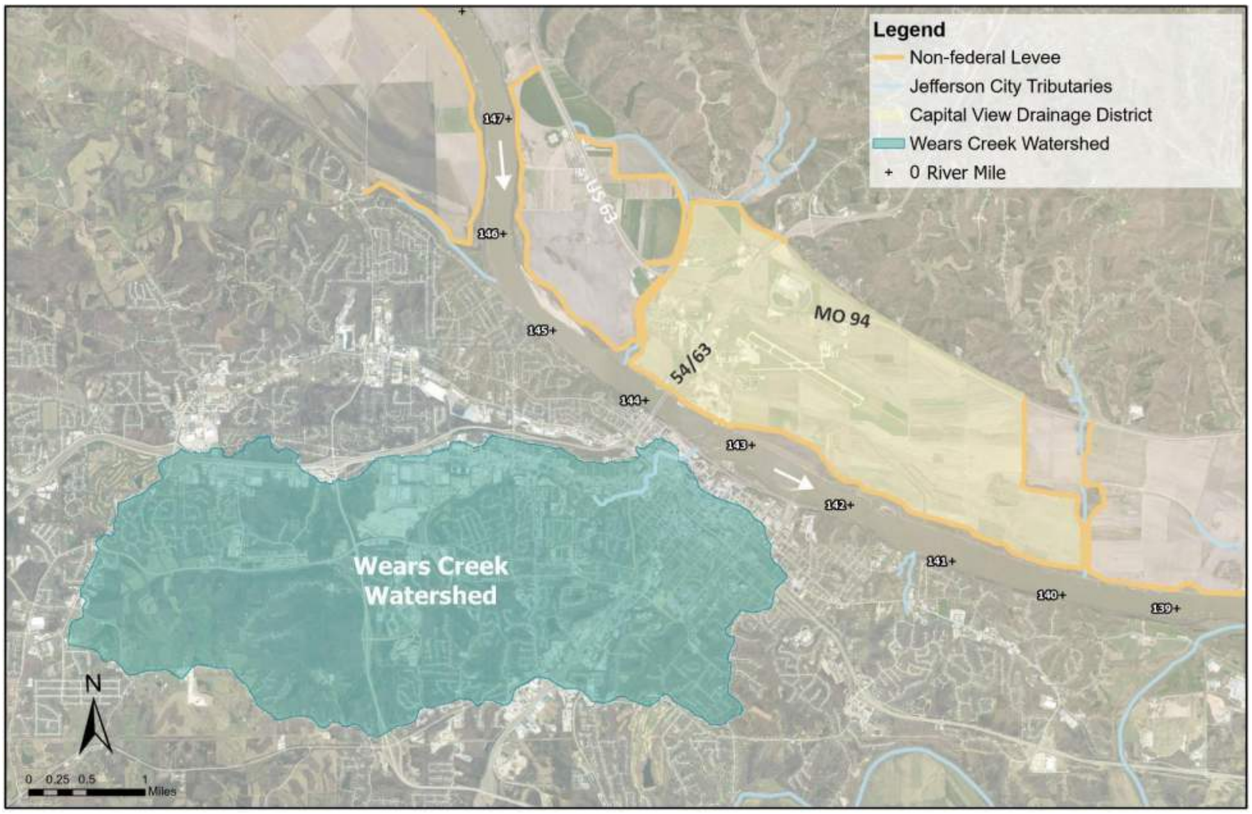

Figure 6. Wears Creek Watershed Delineation

Many studies, conducted by both USACE and the State of Missouri, have been produced to assess feasibility of reducing flooding in the Wears Creek basin since 1928. The most recent USACE findings are documented in a General Design Memorandum issued in 1986 by USACE, Kansas City District. The report concluded that feasible alternatives for the reduction or elimination of flood damages in the developed Wears Creek basin do not exist. Considered alternatives for additional past USACE studies are summarized in **Table 2**.

Table 2. Summary of USACE Wears Creek Flood Risk Management Study Efforts

Date	Recommendation	Outcome
January 1953	Interception and diversion of Wears Creek North and East Branches to the Missouri River and provision of a gated outlet at the mouth of Wears Creek.	Not submitted to Congress for authorization due to lack of local support
September 1974	Series of closed conduits for Wears Creek and portions of the Frog Hollow and East and North Branch tributaries to pass the 1% AEP Wears Creek flood. Filling the floodplain downstream of the Whitten Expressway to the Missouri River Urban Design Flood elevation and filling the floodplain between the Witten Expressway and Highway 54 to above the 1% AEP flood elevation. Other considered alternatives included Zoning and Building Code Requirements, Floodproofing, Evacuation of the Floodplain, Pressurized Conduit, Upstream Impoundments, and Open channel alternatives. ($48M 1979 cost)	Plan was not economically justified using standard USACE benefit/cost analysis due to removal of existing infrastructure to make way for new. Project defined as an urban renewal effort and authorized for further evaluation for federal interest instead of construction.
March 1980	Recommendation of a closed conduit.	Project not economically justifiable due to land use changes in the floodplain. Jefferson City action to convert floodplain to compatible uses.
January 1986	Study to determine if federal interest could be found for targeted flood protection measures for the most developed reach of Wears Creek including Closed Conduit, Earthen Levees, Channel Improvements, Floodwall, and Floodproofing.	The only alternative that would meet project goals of flood management for both Wears Creek and Missouri River flooding was the floodwall alternative, which would be cost prohibitive in relation to the flood damages reduced. The City removed much of the flood-prone development prior to the study, and USACE recommendation is to continue flood damage reduction efforts.

The most recent report lead by the State of Missouri was completed by HDR in 2003, titled *Wears Creek flood Mitigation Study Phase II Proj. No. 00025-01*. This study considered alternatives for providing flood protection to key infrastructure within the Capitol Complex area and performed hydrologic and hydraulic evaluations to define risk of flooding from Missouri River backwater, Wears Creek runoff, and coincident flooding. Two conceptual designs were considered with both providing 1-ft freeboard above the 0.2% AEP Missouri River flood event and 1% AEP interior pumping facility for the Wears Creek drainage area. One conceptual design included an alternative for a levee, flood gate, and pumping facilities at the mouth of Wears Creek totaling $76 Million. The second conceptual design included site-specific floodwalls, gates, and pumping facilities at the Truman Building, State Health Laboratory, and Capital Plaza Hotel totaling $5 Million. Neither alternative was implemented.

2.4.8 Flood Insurance Studies

Available Flood Insurance Study (FIS) reports for the Missouri River at Jefferson City include the jurisdictions of Callaway County (29027CV000B, 2012) and Cole County (29051CV000B, 2012). The FEMA defined floodway adjacent to the Capital View levee averages 4,850 ft wide, with the floodplain expanding bluff to bluff at a consistent 9,000 ft width. Through the study area, the floodway is restricted through the existing US 54/63 overpass embankments, necking down from a 7,000 ft width to 2,950 ft through the bridge as shown in **Figure 7**.

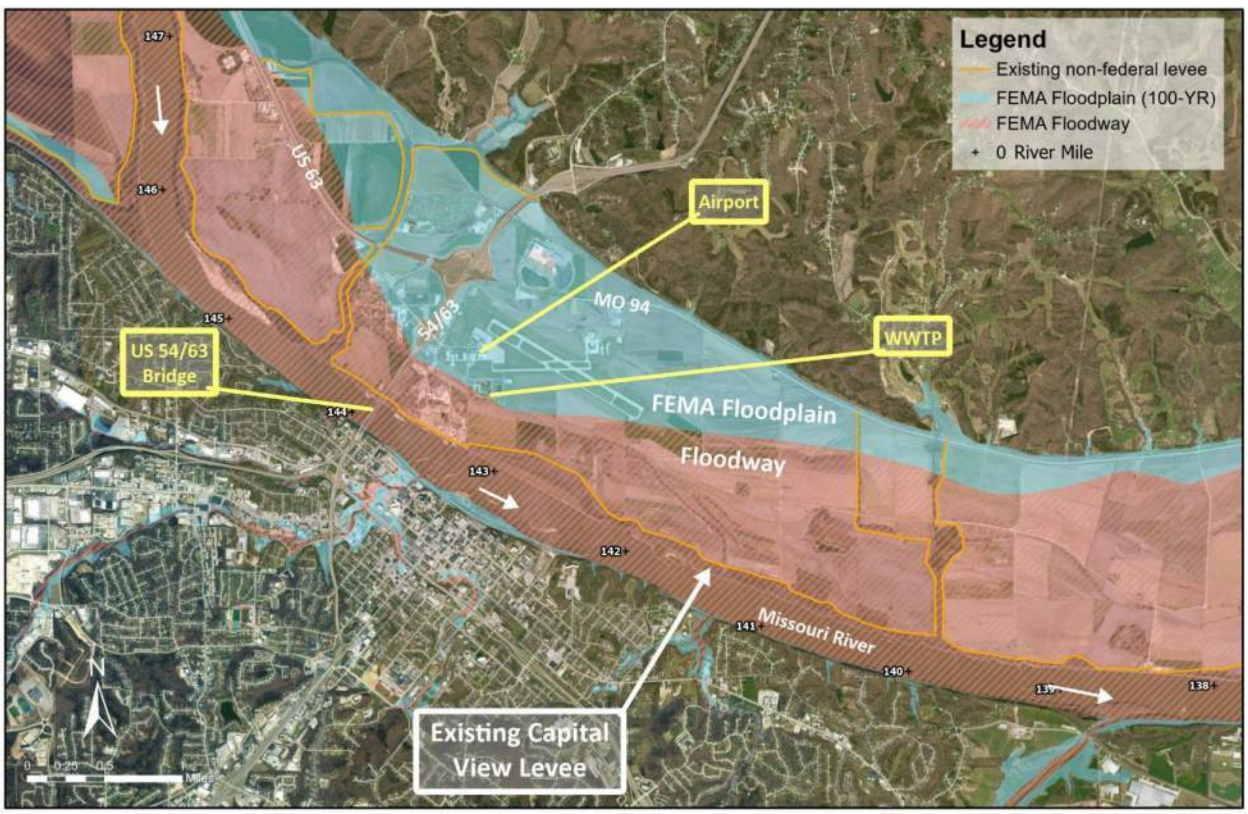

Figure 7. FEMA Mapped Floodway and Floodplain

Table 3 summarizes the peak discharges for each frequency event utilized to develop the FEMA mapped flood risk areas. These flows were developed from the 2004 UMRSFFS.

Table 3. Missouri River at Jefferson City FIS Flows (FEMA, 2012)

AEP (%)	Return Period (years)	Peak Discharge (cfs) RM 149 Upstream of Jeff City	Peak Discharge (cfs) RM 130 Osage River Confluence
10	10	358,000	427,000
2	50	511,000	593,000
1	100	582,000	665,000
0.2	500	763,000	836,000

2.4.9 Levee Screenings

Levee screenings were performed on the non-federal levee systems throughout the Jefferson City area from 2014-2016. No federal levees are located within the study area, but many of the private levees are enrolled in the USACE non-federal levee program, referred to as P.L. 84-99. **Figure 8** provides delineation of adjacent levee districts as identified by the National Levee Database (NLD). Overtopping for all levees through the study area is driven by Missouri River flood events. Overtopping frequency events, listed in **Table 4**, were developed from Upper Mississippi River System Flow Frequency Study (2004 UMRSFFS) frequency flows.

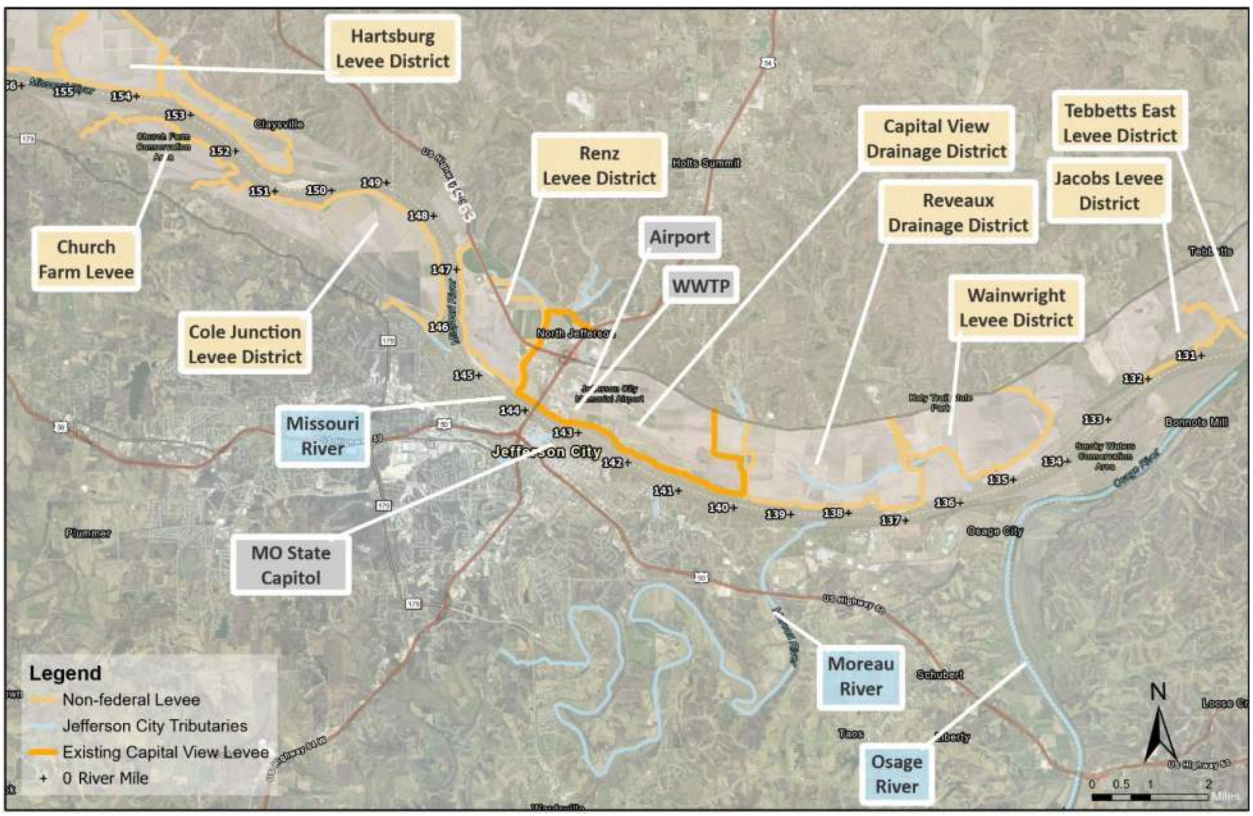

Figure 8. Capital View Levee and Adjacent Levee Districts

Table 4. Summary of Adjacent Levees

Adjacent Non-Federal Levee	Overtopping Stage (ft)	AEP	Year of Latest USACE Levee Screening
Church Farm (Prison Farm)	30.0	12%	2014
Cole Junction	33.5	4.5%	2014
Renz	30.5	12%	2015
Capital View	30.5	17%	
Reveaux	32.0	9%	2014
Wainwright	31.0	12%	2014

The Capital View levee segment's latest routine inspection by USACE occurred in April 2021. Findings of this report concluded that the levee system was rated as "minimally acceptable" due to a cracked flap gate, a silted-in drainage structure, evidence of animal burrows, and instances of large trees and woody vegetation within riprap embankment and levee slope.

2.5 Observed Flood History

The Capital View Drainage District and greater Jefferson City riverfront has a history of flooding. Recent flood events have resulted in notable damage to public infrastructure, access issues, and sustained impacts to inundated property. **Table 5** summarizes seven historic flood events in the project area and the following report sections provide additional discussion pertaining to the events. **Figure 9** compiles observed stage hydrographs of historic flood events. Continuous stage records for Jefferson City began in 2015 and are therefore not available for the 1993 and 1995 events. All records of stage and flow data were obtained from National Weather Service (NWS) and USGS (United States Geological Survey) databases.

Table 5. Summary of Historic Floods at Jefferson City gage

Event	Max. River Stage[1] (ft)	Flow (cfs)	AEP[2]	Capital View Levee Impacts[3]
2019	33.41	366,000	7% (1/14 yr)	Overtopped & Breached (7 locations)
2017	26.97	288,000	20% (1/5 yr)	Loaded Levee
2015	27.81	293,000	19% (1/5 yr)	Loaded Levee
2013	30.83	317,000[4]	14% (1/7 yr)	Loaded Levee
2011	27.23	NA	NA	Loaded Levee
1995	33.05	356,000[4]	9% (1/12 yr)	Overtopped & Breached (13 locations)
1993	38.65	750,000[5]	0.2% (1/500 yr)	Overtopped & Breached (13 locations)

[1] *Flood stage at Jefferson City, USGS gage = 23 ft*

[2] *As calculated with 2023 Missouri River Flow Frequency Study as adopted for this study (Section 4.3.1)*

[3] *Capital View levee AEP Overtopping Event = 17% (1/6 yr) - 30.5ft stage*

[4] *Estimated from current Jefferson City rating curve, no USGS flow data available*

[5] *Flow estimated from reported flows at Boonville and Hermann gages*

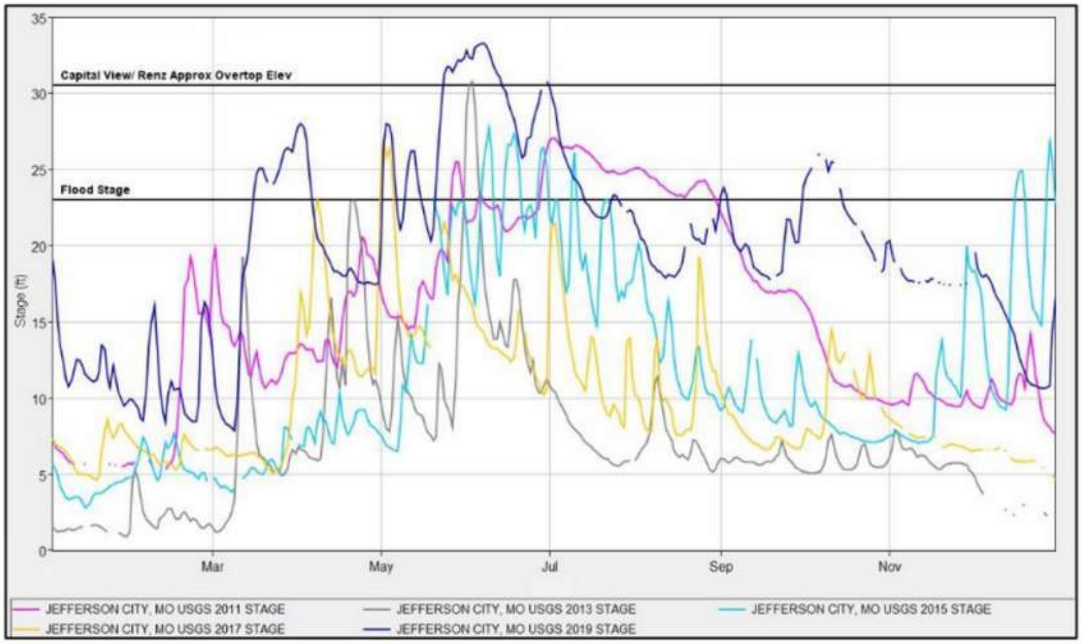

Figure 9. Recorded USGS Stages for Recent Historical Flood Event Years

The Flood of 1993 remains the flood of record at Jefferson City, with a recorded maximum stage of 38.7 feet and discharge of 750,000 cfs at the Jefferson City Missouri River gage, corresponding to an approximate 0.2% (1/500 yr) AEP flood event. The 2019 flood recorded a maximum river stage of 33.4 feet and discharge of 366,000 cfs, an approximate 10% (1/10 yr) AEP flood event. **Figure 10** shows the difference in inundation between the 2019 and 1993 flood events in the Capital View Drainage District leveed area.

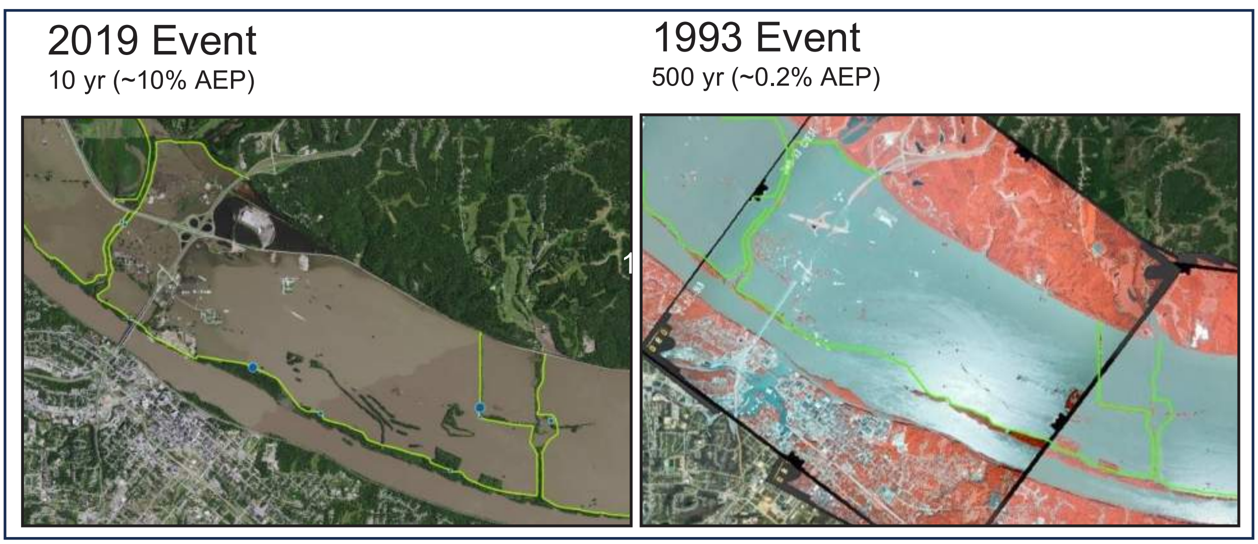

Figure 10. Inundation Extent Comparison at Capital View Levee for 2019 and 1993 Flood Events

2.5.1 Flood of 2019

The 2019 flood had a historically long duration where the Missouri River was above flood stage for at least one gage from early March to December 2019. There were two separate events occurring first in March then in May to June. The first event was due to a combination of snowmelt and a sudden, warm rain event in Nebraska and the Dakotas. At the Jefferson City gage, peak stages (NWS) of 28.1 feet were recorded April 1st and May 3rd with peak flows of 294,000 cfs. Flood consequences downstream of Kansas City were not as impactful. The second event compounded largely saturated soil conditions and already high upper Missouri River basin flows with record May and June rainfall totals across the Kansas River Basin and 15 to 20 inches of precipitation within the Grand River Basin. The May – June flood event more heavily impacted the lower basin between Kansas City and St. Charles and included multiple flood peaks, with notable flows from multiple major tributaries. At the Jefferson City gage, a peak stage (NWS) of 33.4 feet was recorded June 7th with a peak flow of 366,000 cfs. **Figure 11** shows an aerial extent of the flooding impacts at Jefferson City near the highway bridge over the Missouri River floodplain.

The Missouri River stage at Jefferson City was greater than or equal to the NWS Flood Stage a total of 106 days in 2019. Due to the prolonged high river stage and flood duration experienced by many levee systems, considerable damage occurred. Capital View levee experienced sod damage and erosion on both riverside and landside levee side slopes in addition to seven levee breaches. One breach out of the seven included foundation and landside scour. Per the Capital View levee Project Information Report dated August 26, 2019, all levees with minimal foundation scour were repaired in place with the addition of fill material. The single breach with foundation impacts (Station 211+00 to 221+00) was recommended to be repaired with a landward setback, however current aerial imagery suggests the levee was repaired in place and does not indicate a setback was constructed.

During the 2019 flood, multiple non-federal levees in the Jefferson City area overtopped and breached including the Church Farm Levee, Renz Levee District, Capital View Drainage District, and Reveaux Drainage District. The Wainwright levee was overtopped, but not breached. The Cole Junction levee was not overtopped or breached.

Figure 11. Capital View Levee from the Missouri River North of US-54 Bridge, Looking East – 2019 Flood

A post flood report was assembled by USACE to preserve perishable flood data. High water marks were collected, and rated based on quality, by the USACE Kansas City District utilizing GPS survey equipment referencing a Virtual Reference Station (VRS) with a vertical accuracy within 0.2 feet. High water marks were utilized for calibration of the hydraulic model at Jefferson City and a summary is included in Appendix A1 - Part 2.

2.5.2 Flood of 2013

The 2013 flood loaded levees through the Jefferson City area to their limits with active flood fighting efforts. Levee overtopping was generally observed downstream of river mile 132. River stages did not lead to levee overtopping within the direct Jefferson City corridor. Jacobs Levee and Tebetts East Levee District just downstream of the study area were overtopped.

2.5.3 Flood of 1995

The 1995 flood spanned 55 days beginning May 9^{th} and was a 9% (1/12 yr) AEP event at the Capital View Drainage District with a peak flow of 356,000 cfs. This flood resulted in levee overtopping and 13 breaches throughout the Capital View levee district, all located downstream of the US 54/63 bridge.

During discussion with existing Jefferson City public works engineers, it was mentioned that part of the reason the 1995 flood was impactful to the city was because of a lack of recovery time from the 1993 flood. There were still repairs being made to restore conditions to the pre-1993 condition.

2.5.4 Flood of 1993

The flood of 1993 documented the greatest peak flows on the Missouri River from St. Joseph to the mouth, post construction of regulation dams upstream on the Missouri mainstem. High flows resulted due to extended, widespread rain in the Missouri River basin downstream of the mainstem dams. With these record flows, all non-federal levee units and many federal units downstream of Rulo, Nebraska were overtopped.

In Jefferson City, water reached the US 54/63 bridge, limiting traffic into Jefferson City from the east. The floodwaters through Jefferson City spread bluff-to-bluff.

3 STREAM GAGES AND HYDROLOGIC RECORD DEVELOPMENT

Primary data sources for the hydrologic analysis include observed gage data and compiled period of records from previous flow frequency studies. The hydrologic inflow data used in the hydraulic model was developed from the Missouri River Flow Frequency Study (2023 MRFFS).

3.1 Gage Data

Three Missouri River mainstem USGS gages (Boonville, Jefferson City, and Hermann) are located within the model extents. An additional mainstem gage, owned by the NWS, is located near Chamois, Missouri. USGS flow and stream gage data was utilized for configuration and calibration of models. Of the many tributaries in the study area, the only two gaged tributaries include the Moreau River and the Osage River.

3.1.1 Observed Data

The USGS gage at Jefferson City was the primary data used in the calibration of the hydraulic model. The additional mainstem gages at Boonville and Hermann, Missouri were utilized as flow boundary conditions. A summary of gage location, USGS ID, period of record (POR), and datums is located in **Table 6**.

Table 6. Modeled Area Stream Gages

Gage No.	Operator	Gage Name	River Mile	Data Type	Historic / Peak Data Availability	Daily Flow / Stage Data Availability	Gage Zero Datum (ft) (NAVD88)
6909000	USGS	Boonville	196.62	stage / flow / rating curve	1844 - present	1925 - present	565.58
6910450	USGS	Jefferson City	143.86	stage / flow / rating curve	2011 - present	2015 – present (flow from 2015)	520.18
CMSM7	NWS	Chamois	117.03	rating curve / historic crests	1929 - present	none	502.76
6934500	USGS	Hermann	97.93	stage / flow / rating curve	1844 - present	1928 - present	481.50

3.2 Ungaged Streams

Flow entering the Missouri River between the Boonville and Jefferson City gages is largely ungaged, as shown in **Figure 12**. In the visual, yellow areas represent origination of ungaged flow, or flow that is not measured by a gage location until it reaches the next downstream gage.

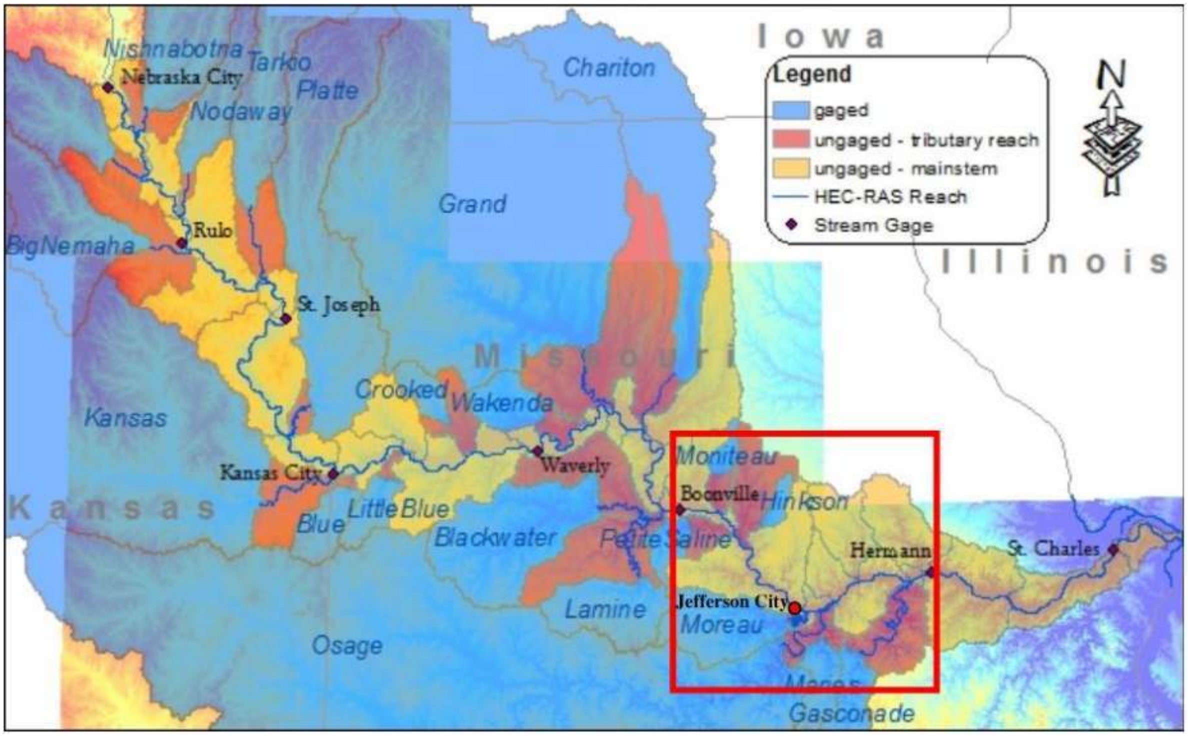

Figure 12. Ungaged Flow to the Missouri River

4 HYDROLOGIC ANALYSIS

The Missouri River is the source of flooding for the existing Capital View Drainage District. Targeted peak flow frequencies scaled from the 2023 MRFFS were utilized to create input hydrographs for the hydraulic model. Input hydrographs were developed by scaling the October 2018 Missouri River flow hydrographs. The upstream mainstem flow hydrograph boundary condition at the Boonville gage and lateral inflow hydrographs were scaled to best meet routed frequency flow targets at the Jefferson City and Hermann gages. As Jefferson City was not a location calculated in the 2023 MRFFS analysis, gage target flows were developed by balancing flow ratios between Boonville and Jefferson City and Jefferson City and Hermann, informed by MRSFFS 2004 tributary flow ratios. As each storm is different, scaled tributary flows were compared to peak flows for a range of historical storm events to ensure reasonable targets.

4.1 Flow Frequency Study Summary

Hydrology for this model was developed with reference to discharge frequency relationships as determined during the 2023 MRFFS. The 2023 MRFFS is a standalone component within the greater effort to update the Lower Missouri River modeling throughout the USACE Kansas City District and replaces the 2004 UMRSFFS. The 2023 MRFFS focused on expanding the period of record used to develop the flow frequency estimates from 1930 to 2019 and updating the statistical analysis to use current methodologies and computing techniques. A total of ten gages were studied through the Lower Missouri Basin as shown in **Figure 13**.

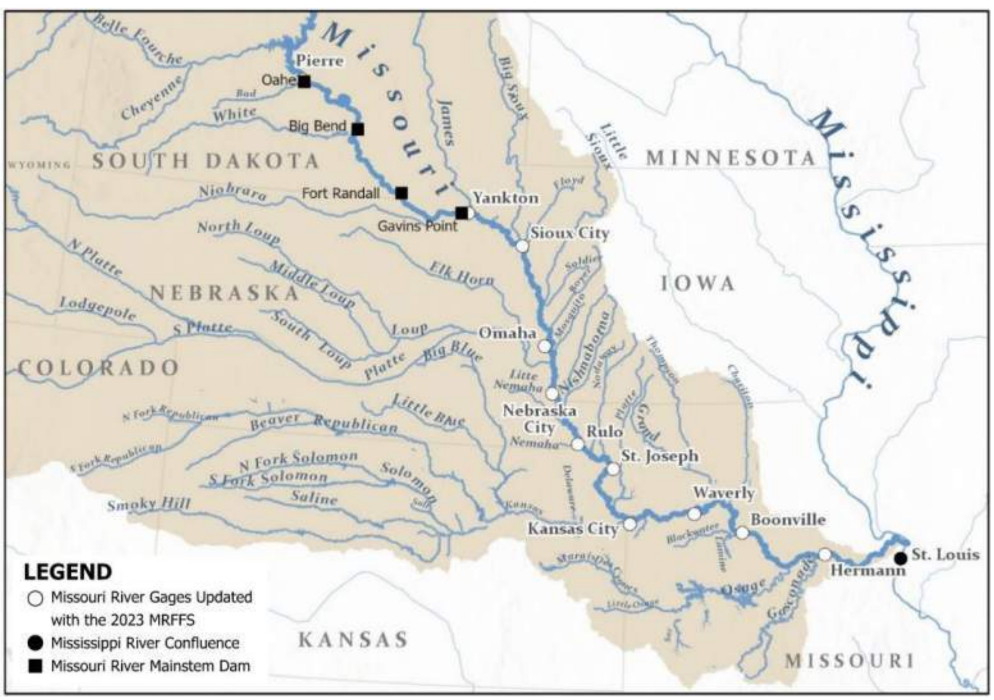

Figure 13. Stream Gages used in the Lower Missouri River Flow Frequency Analysis

Source: 2023 MRFFS, Figure 1-1

The 2023 MRFFS recommends the adoption of Monte Carlo simulation results for estimation of regulated flow frequency on the Lower Missouri River in current conditions for floods between a 99% and 0.2% annual exceedance probability (AEP). Adopted flow frequencies for all ten study gages can be found in **Figure 14**. While the 2023 MRFFS examines both regulated and unregulated Missouri River flows, this analysis at Jefferson City will solely rely on regulated upstream flow. The Jefferson City model area is bound by the Boonville and Hermann gages, upstream and downstream of the study area respectively.

AEP (%)	Yankton	Sioux City	Omaha	Nebraska City	Rulo	St Joseph	Kansas City	Waverly	Boonville	Hermann
0.2	213,000	285,000	351,000	480,000	510,000	526,000	640,000	674,000	731,000	933,000
0.4	169,000	268,000	312,000	399,000	432,000	444,000	555,000	588,000	702,000	742,000
0.5	164,000	266,000	293,000	382,000	422,000	433,000	546,000	573,000	672,000	722,000
1	164,000	218,000	232,000	329,000	336,000	349,000	467,000	503,000	572,000	666,000
2	104,000	156,000	187,000	244,000	294,000	296,000	393,000	412,000	531,000	571,000
4	81,000	121,000	154,000	220,000	250,000	255,000	312,000	323,000	417,000	506,000
5	77,000	111,000	151,000	212,000	233,000	239,000	293,000	294,000	393,000	473,000
10	64,000	89,000	118,000	171,000	187,000	197,000	247,000	251,000	334,000	416,000
20	54,000	71,000	99,000	132,000	148,000	157,000	197,000	214,000	280,000	345,000
50	44,000	47,000	62,000	88,000	101,000	107,000	136,000	142,000	204,000	262,000
80	38,000	41,000	47,000	61,000	65,000	75,000	97,000	101,000	134,000	175,000
90	35,000	38,000	43,000	54,000	57,000	66,000	82,000	86,000	109,000	142,000
95	33,000	36,000	40,000	49,000	51,000	59,000	72,000	75,000	97,000	123,000
99	28,000	32,000	37,000	42,000	44,000	52,000	58,000	59,000	78,000	100,000

Figure 14. Summary of Regulated Flow Frequency Annual Exceedance % Probability (Flow in CFS), Expected Probability Flows

Source: 2023 MRFFS, Table 6-18

The adopted, regulated flow frequencies calculated by the Monte Carlo method are compared to the 2004 UMRSFFS results in **Figure 15, Figure 16,** and **Figure 17** representing the 10%, 1%, and 0.2% AEP. In these exhibits, the Regulated WAT Flow represents the adopted 2023 MRFFS flows. The 2004 and 2023 flow frequency study results indicated little to no change in the 1% AEP regulated flows at the Boonville and Hermann gages. The 10% AEP 2023 MRFFS flows reflect a 5% decrease in flow at both the Boonville and Hermann gages, which suggests a similar proportionality to the Jefferson City gage. For the 0.2% AEP regulated flow, adopted flows increase 3% at Boonville and 11% at Hermann gage between the 2004 study and 2023 MRFFS.

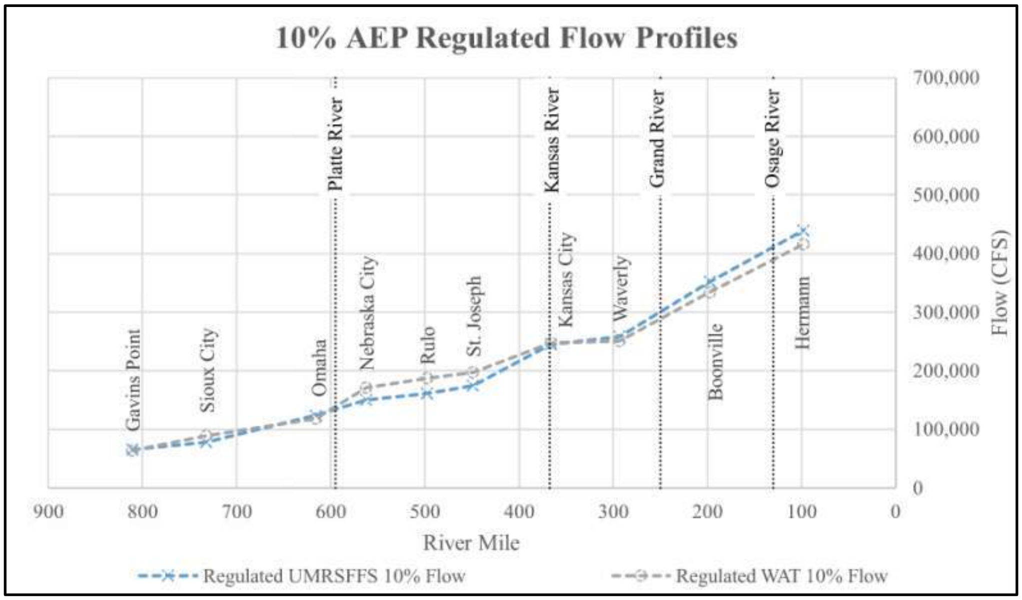

Figure 15. 10% AEP Flow- Adopted 2023 MRFFS Flow (Regulated WAT) Compared to 2004 UMRSFFS Flow

Source: 2023 MRFFS, Figure ES-3

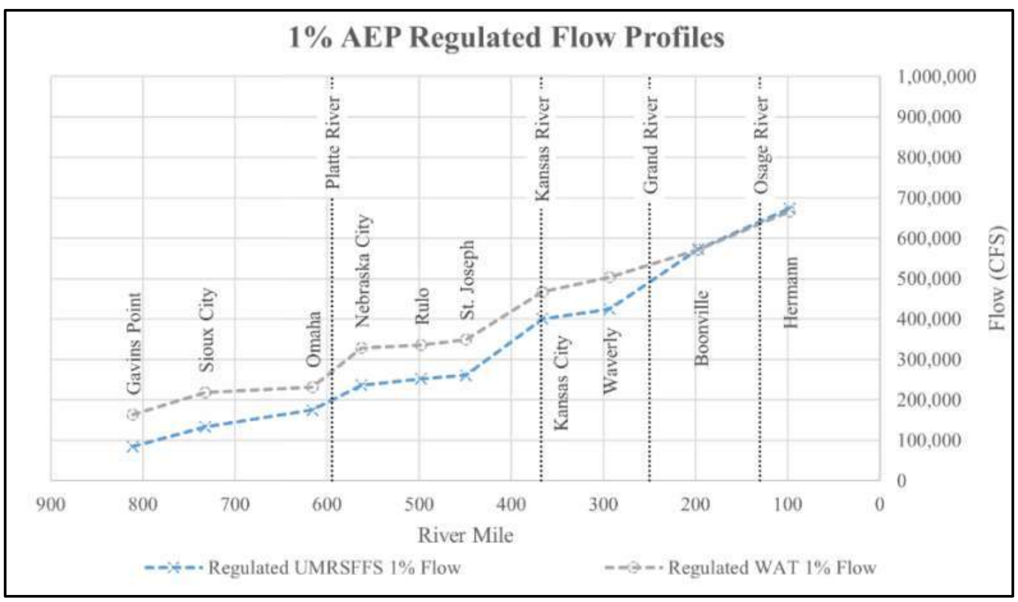

Figure 16. 1% AEP Flow- Adopted 2023 MRFFS Flow (Regulated WAT) Compared to 2004 UMRSFFS Flow (Adopted)

Source: 2023 MRFFS, Figure ES-4

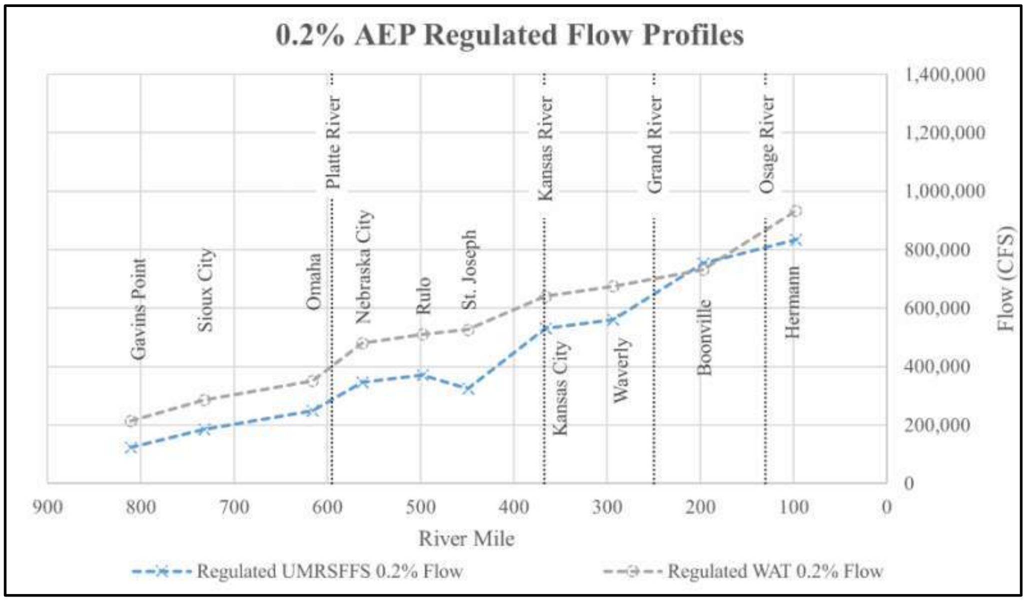

Figure 17. 0.2% AEP Flow- Adopted 2023 MRFFS Flow (Regulated WAT) Compared to 2004 UMRSFFS Flow

Source: 2023 MRFFS, Figure ES-5

Due to the consistency in flow frequency data between the 2023 MRFFS and 2004 UMRSFFS flow frequency studies, the 2023 expected flow frequencies were adopted for use in the Jefferson City Feasibility Study. Although the Jefferson City gage was not a gage with published flows in the 2023 MRFFS, the upstream and downstream Jefferson City Study model boundary gages were included and could be scaled to interpolate peak flows at the Jefferson City gage within acceptable magnitude for alternative formulation and comparison.

Stage frequency results are being developed as a continuation of the 2023 MRFFS at the time of this study. After publication, the results will contain stage and flow exceedance probabilities at the Jefferson City gage, developed through the routing of adopted frequency flows through the Lower Missouri River system. Stage and flow frequency results will be adopted for final Preconstruction Engineering Design (PED) of the recommended plan at Jefferson City.

4.2 Missouri River Dominated Event – Annual Exceedance Probability Flows

The hydrologic inflow data used in the Jefferson City Study hydraulic model was developed from the 2023 MRFFS published flows at the model boundary conditions of the Boonville and Hermann gages. Peak regulated expected frequency flows for the 50% through 0.2% AEP were adopted as target flows for the Boonville and Hermann gage locations.

Target flows at Jefferson City were developed by calculating flow ratios between both the Boonville and Jefferson City gages and the Jefferson City and Hermann gages as published with the 2004 UMRSFFS. The resulting flow ratios were utilized to develop percentage increase in flow from Boonville to the Jefferson City gage, with slight refinement to ensure resulting downstream tributary flows were within a plausible range to meet the 2023 MRFFS Hermann flow target. Flow ratios between the Boonville gage and Jefferson City gage that were applied to the 2023 MRFFS adopted flows to calculate Jefferson City Feasibility Study peak flow targets are summarized in **Table 7**. For additional context, the 2004 UMRSFFS flows utilized to develop the FEMA FIS through the study area are provided for reference in **Table 8**.

Table 7. Peak Gage Flow AEP – Developed with 2023 MRFFS flows

AEP (%)	Boonville (cfs)	% Increase from Boonville	Feasibility Study Jefferson City Gage Peak Flow Target (cfs)	Hermann (cfs)
0.2	731,000	2.30%	748,000	933,000
0.5	672,000	2.60%	689,000	722,000
1	572,000	2.80%	588,000	666,000
2	531,000	2.80%	546,000	571,000
4	417,000	3.30%	431,000	506,000
10	334,000	3.60%	346,000	416,000
20	280,000	3.60%	290,000	345,000
50	204,000	3.80%	212,000	262,000
99	78,000	4.00%	81,000	100,000

Table 8. Peak Gage Flow Annual Exceedance Probability – 2004 UMRSFFS* Flows

AEP (%)	Boonville (cfs)	Jefferson City (cfs)	Hermann (cfs)
0.2	753,000	774,000	833,000
0.5	648,000	662,000	742,000
1	573,000	591,000	673,000
2	503,000	520,000	604,000
4	NA	NA	NA
10	352,000	365,000	439,000
20	289,000	297,000	363,000
50	203,000	208,000	248,000
99	84,500	85,800	87,000

** Superseded by 2023 MRFFS data.*

Flow between the upstream Boonville gage and Jefferson City is ungaged without major tributary confluences. All Jefferson City Study modeled tributaries with independent inflow hydrographs enter the Missouri River downstream of the Jefferson City gage. After setting the Jefferson City peak flow target, flows were routed and tributary flow targets were developed, as listed in **Table 9**, to match the downstream Hermann gage peak flow target. Tributary peak flows were estimated using 2004 UMRSFFS flow ratios as well as historic gage data for similarly sized events, summarized in **Table 10**. Sensitivity of tributary flows in relation to the final top of levee profile will be assessed with final design optimization.

Table 9. Modeled Tributary Gage Flow Targets

AEP (%)	Moreau Gage Flow (cfs)	Osage Gage Flow (cfs)	Gasconade Gage Flow (cfs)
0.2	19,874	78,650	119,040
0.5	15,690	71,500	48,000
1	28,765	68,640	54,288
2	15,690	69,212	20,160
4	15,167	57,200	61,440
10	13,075	54,340	48,240
20	13,000	44,902	19,200
50	4,707	28,600	13,440

Table 10. Historical Peak Flow Comparison for Missouri River and Major Tributaries

Event	Est. AEP (%) at Jefferson City	Gage Flow at Boonville (cfs)	Gage Flow at Jefferson City (cfs)	Gage Flow at Hermann (cfs)	Gage Flow at Moreau (cfs)	Gage Flow at Osage (cfs)	Gage Flow at Gasconade (cfs)
Jul-93	0.2	755,000	NA	750,000	NA	NA	4,330
May-19	10	363,000	358,000	400,000	16,700	52,900	40,600
Apr-73	10	334,000	NA	500,000	14,500	NA	NA
Oct-18	20	273,000	275,000	294,400	5,230	28,500	1,920
Jun-15	20	281,000	293,000	321,000	24,800	84,300	52,300
Jun-21	50	252,000	254,000	316,000	NA	36,600	2,910
Jun-22	50	189,000	193,000	256,000	15,300	38,700	1,560

Note: Cells labeled with an "NA" indicate no available historical data.

4.3 Scaled Observed Flood Hydrograph to Study Target Flows

Representative hydrographs were patterned after the October 2018 flow hydrograph. This event represents Missouri River flows with a singular peak, without an extended event duration. This event was determined to be the cleanest study hydrograph to route the targeted peak flows. Flow data has been collected for the Jefferson City gage since 2015 as documented in **Figure 18**.

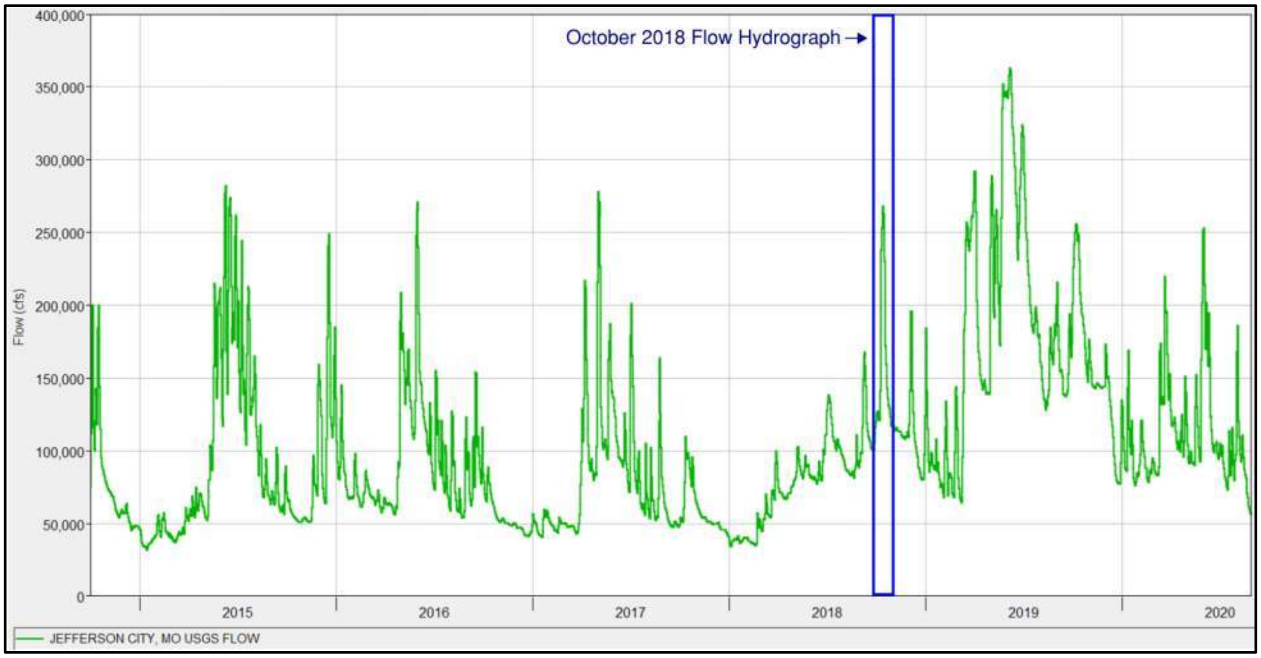

Figure 18. USGS Instantaneous Flow (cfs) at Jefferson City Missouri River Gage (2015-2020)

Figure 19 compares observed stage readings at the Jefferson City gage to the flood stage. Historic Missouri River flooding varies at this site and is susceptible to annual variabilities in the large upstream drainage area. The 2018 storm event was the single event that allowed for a single peak hydrograph to be scaled with an average scale factor between 2 and 3 to match the peak flow target through the study area.

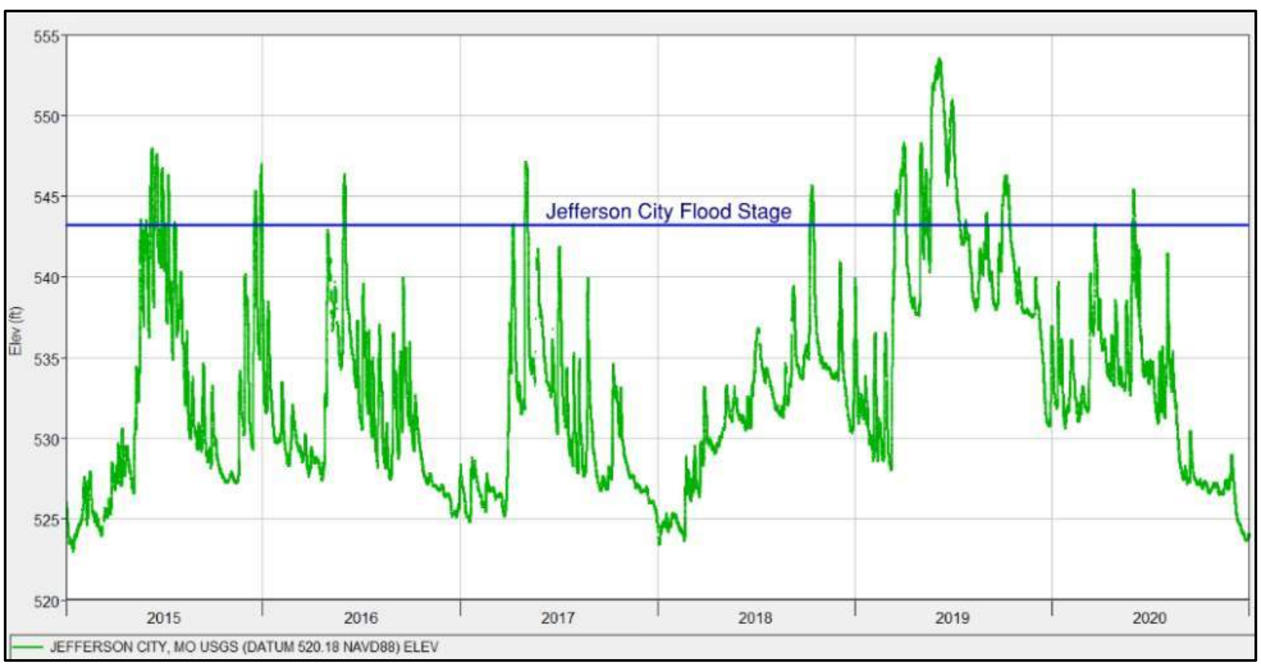

Figure 19. USGS Stage (elevation) at Jefferson City Missouri River Gage (2015-2020)

The 2018 hydrograph was an isolated storm event with defined rising and falling hydrograph limbs. The peak observed instantaneous flow during the October 2018 event was 269,000 cfs. The largest target flow for the 0.2% AEP event for the Jefferson City Feasibility Study is 748,000 cfs, or a factor of 2.8 greater than the observed hydrograph peak. Representative boundary and inflow hydrographs from the 2018 storm for the Missouri River mainstem and modeled major tributaries were scaled to mimic the peak flow targets at Boonville, Jefferson City, and Hermann Missouri River Gages. Hypothetical flood routing methodology from EM 1110-2-1415, Paragraph 3-9.d.2, allows for scaling of observed hydrographs by multiples up to 2 or 3 . Special attention was used to target peak flows at the Jefferson City Missouri River gage at the project site for the development and comparison of feasibility alternatives. The singular peak could underestimate flood volume for a final design, however comparison of alternatives for feasibility analysis is focused on determining levee height for feasibility determination and estimating economic considerations. Volume sensitivity will be completed for final design.

4.3.1 Resulting Discharge Frequency at Jefferson City Gage

In summary, the upstream boundary condition at Boonville was scaled to match the target 2023 flow and routed downstream. The flow at the river mile 149.3 lateral inflow was then scaled to input the required flow needed to meet 2023 flow target at the Jefferson City gage. From there, as mentioned in Section 4.2, downstream inflow hydrographs and tributary flows were scaled to roughly match the mainstem to tributary flow ratios developed with the 2004 UMRSFFS. Differences in total peak flow at the Hermann gage between the 2004 and 2023 flow frequency studies required some adjustment of flow ratios within the bounds of historic storms and exceedance probabilities for major tributary watersheds to produce reasonable values and meet the Hermann gage flow target.

The scaled upstream boundary condition hydrograph at Boonville is included as **Figure 20**. Routed hydrograph outputs, including calculations through both the 1D channel and 2D flow area, at the mainstem river gages are included in **Figure 21** and **Figure 22**. All hypothetical hydrographs were produced with arbitrary modeling times beginning in January of 2099 to distinguish them from observed flood events. The hydrograph shape downstream of Boonville is susceptible to change due to routing conditions and floodplain interaction between the Boonville, Jefferson City, and Hermann gages. Notably, the flattening of the 10% AEP hydrograph at peak flow for gages downstream of Boonville is due to overtopping of most agricultural levees within the modeled corridor, resulting in measurable water volume being stored until overtopping depths continue to rise to levels where flow conveyance activates over the levee systems.

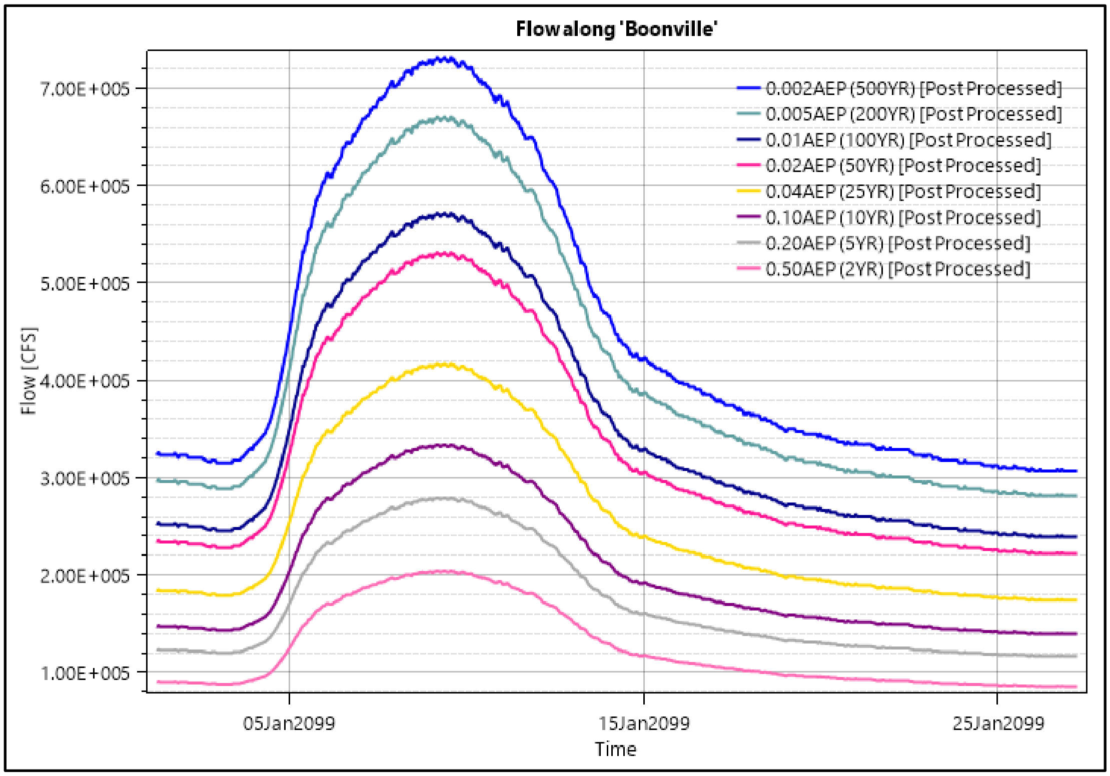

Figure 20. Boonville Gage Upstream Boundary Condition Hydrographs

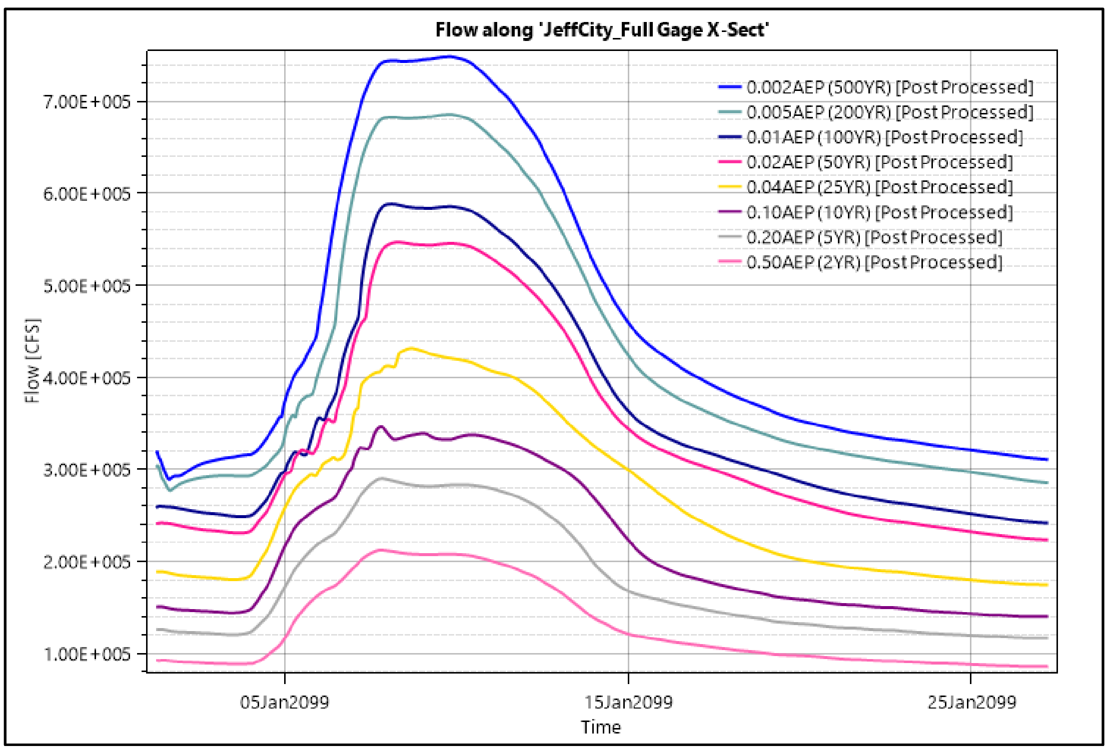

Figure 21. Jefferson City Gage Routed Hydrographs

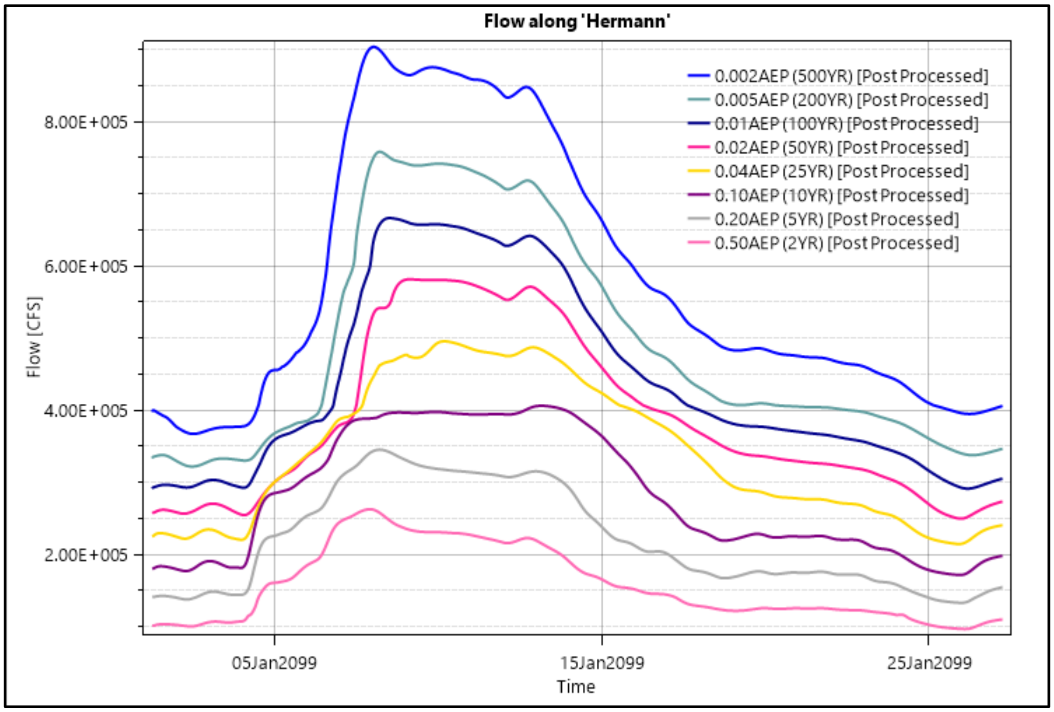

Figure 22. Hermann Gage Routed Hydrographs

Targeted Jefferson City flows are compared to those routed in the hydraulic model for each adopted frequency condition for the Jefferson City study in **Table 11**.

Table 11. Adopted Discharge Frequency Events at Jefferson City Gage

AEP (%)	Flow Target	Adopted Routed Model Flow	Deviation from Target (%)
0.2	748,000	748,553	-0.1
0.5	689,000	685,623	0.5
1	588,000	588,525	-0.1
2	546,000	546,855	-0.2
4	431,000	431,735	-0.2
10	346,000	346,201	-0.1
20	290,000	289,888	0.0
50	212,000	212,259	-0.1

The base year / current year peak flow frequency condition is equivalent to the Future Without Project (FWOP) condition for this study, which spans a 50-year period of analysis beginning in 2027. Refer to Appendix A1 Part 3 Section 2.2 for further information on how the FWOP condition was developed. In summary, no storage or significant regulation changes to the Missouri River are anticipated upstream of the study within the FWOP timeframe. Minimal effects have been observed in recent and historic flow trends for the river. Climate change analysis does not identify statistically significant trends in peak flow rates, as discussed in Appendix A1.3 Section 11. Flows for the FWOP condition will, therefore, duplicate the most recent existing conditions hydrology.

5 ECONOMIC MODEL INPUTS

Hydrology and hydraulics inputs are required for economic modeling for this flood risk management study. The following sections highlight how hydrology and hydraulic inputs were developed and assumptions utilized.

5.1 H&H Factors for Economic Model Development and Assumptions

The base year for the existing conditions model and analyses is 2027. The period of analysis of this study spans 50 years. Additional context regarding the economic analysis for this study, including full economic model development and output are located in Appendix E.

Three index points through the study area were defined and utilized as the location of risk equation calculations and comparison, calculated by the economists for both FWOP and alternative conditions. The index point locations are positioned at three differing hydraulic conditions through the study reach including one upstream of the U.S. Highway bridge, one downstream of the bridge within the hydraulic influence of the structure, and one downstream of the bridge beyond the hydraulic influence of the structure. These locations correlate with bluff-to-bluff cross sections of river miles 143.93, 143.14, and 141.73 as shown in **Figure 23**.

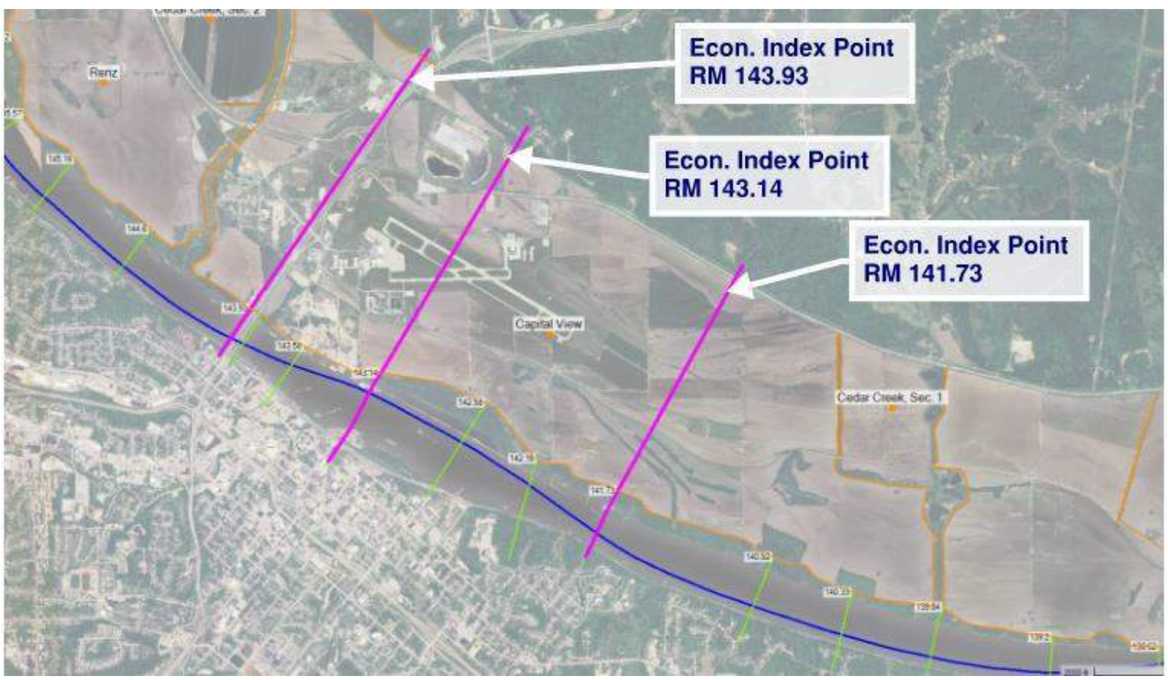

Figure 23. Economic Model Index Point Locations

A baseline flow for each index point was assigned to be 81,000 cfs (99% AEP). All index points are close enough in vicinity to use the same base flow input since the flow is confined to the channel. For FWOP and all plan alternatives, hydraulic output values including maximum water surface elevations, peak flows, and maximum water depths for each index point cross section were provided for input into economic modeling and evaluation.

5.2 Uncertainty Analysis

Uncertainty values were developed for the hydrology and hydraulics data utilized in the economic modeling to aid in the output of feasible bounds of the economic model output. Uncertainty is inherent in calculation of flow frequency due to data availability and record length. The equivalent record length adopted by the Jefferson City feasibility study is equivalent to an 81-year period of record. This value was developed by calculating the Boonville gage daily flows period of record utilized in development of the 2023 MRFFS, spanning 90 years from 1930 through 2019. A 90 percent weight was adopted to scale the utilized period of record to an equivalent record length of 81-years per guidance on applying the order statistics method from EM 1110-2-1619 Table 4-5, included as **Figure 24**.

Table 4-5
Equivalent Record Length Guidelines

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

[1] Based on judgment to account for the quality of any data used in the analysis, for the degree of confidence in models, and for previous experience with similar studies.

Figure 24. Equivalent Record Length Guidelines

Source: EM 1110-2-1619, Table 4 – 5

A 90% weight was selected to reflect inherent uncertainty with scaling Jefferson City flows from upstream and downstream flow targets. The flows at the Jefferson City gage are directly scaled from the Boonville gage for this feasibility study and are routed through the model to the downstream boundary of the Hermann gage. The Jefferson City flow targets were developed based on the previous 2004 UMRSFFS flow ratios because it was the best information available at the time of the study. The 90%-100% equivalent record length weight reflects the method of frequency function estimated from analytical distribution fitted for long-period gage on the same stream, with an upstream drainage area within 20% of that or the point of interest. The Boonville Missouri River gage has an upstream drainage area of 500,700 square miles, and the Jefferson City Missouri River gage has an upstream drainage area of 507,500 square miles resulting in a 1.4% increase in drainage area between the study site and upstream gage. However, the more conversative weighting factor of 90% was utilized due to reliance of the bounding gage flow frequency results upstream and downstream of the project and the large ungaged drainage area directly upstream of the study site.

5.3 Reliability Analysis

The reliability of frequency-discharge estimates is directly linked to the historical record of stream gage data available. In cases where records are small or incomplete, the associated uncertainty is greater than in circumstances where lengthy records of observed data are available. Reliability informs the economic analysis through HEC-FDA modeling as discussed in Appendix E.

5.3.1 Discharge Uncertainty

Uncertainty of adopted discharges was developed with the Monte Carlo analysis computed with the 2023 MRFFS. Confidence limits were defined by the 95 and 5 percent limits. The adopted flows for the Jefferson City feasibility study upstream (Boonville) and downstream (Hermann) bounding gages are denoted as WAT adjusted regulated flows in **Figure 25** and **Figure 26**.

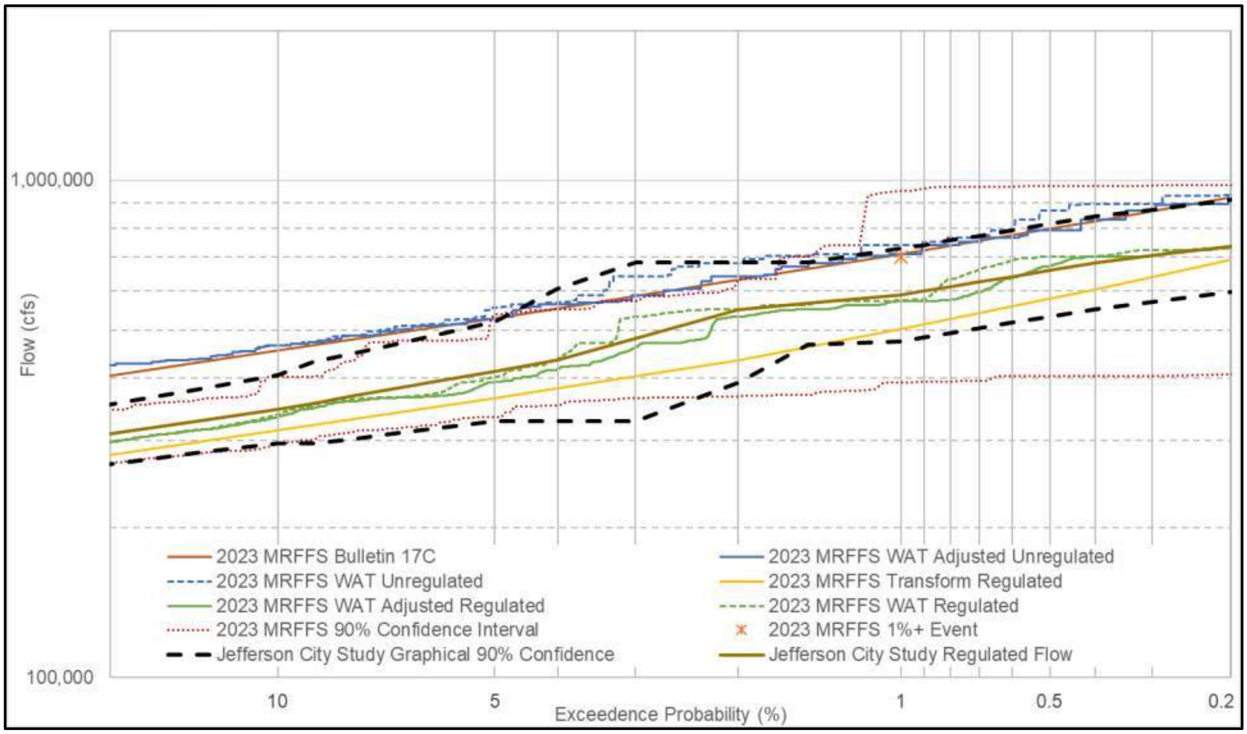

Figure 25. Study Flow Values at Jefferson City Gage over Comparison of Boonville Unregulated and Regulated Flow Frequency Curves

Source: 2023 MRFFS, Figure 6-31

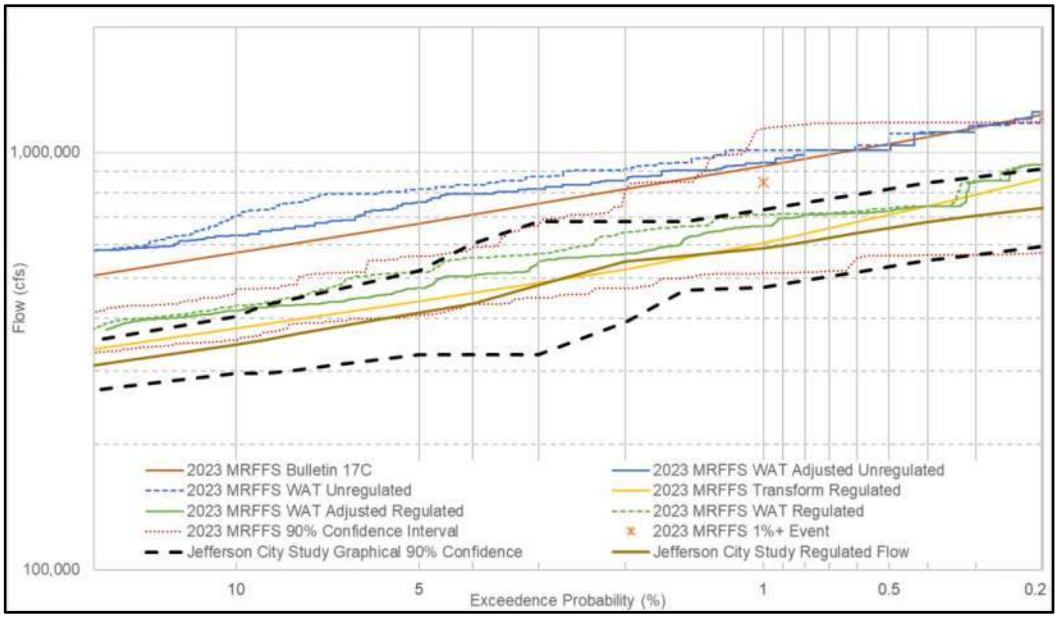

Figure 26. Study Flow Values at Jefferson City Gage over Comparison of Hermann Unregulated and Regulated Flow Frequency Curves

Source: 2023 MRFFS, Figure 6-32

5.3.2 Stage Error Standard Deviation

The stage error standard deviation at each of 8 events was calculated per EM 1110-2-1619, Chapter 5. At the Jefferson City gaged site, the uncertainty is defined as the standard deviation of the difference between observed and the USGS rating curve used to calibrate the model. Recent gage history, defined as the past ten years, includes enough data to use this to determine uncertainty up to the 10% AEP. The calculated values met the minimum standards presented in EM 1110-2-1619, Table 5- for “good” to “fair” Manning’s n alue reliability. Good reliability equates to model validation to a stream gage and calibration to high water marks for the effective model flow.

For flows above 10% AEP, we rely on data available from historic USACE studies. EM 1110-2-1619, Table 5-2 recommends a minimum standard deviation of 0.7-ft for a “fair” estimate of manning’s n alues, cross section data collection, and modeling inputs This was deemed representative of this study location where we have Light Detection and Ranging (LiDAR) overbank data, bathymetric survey data for the channel, and are using a historic MO River model that has been calibrated and tested as our model basis.

The FWOP and with project condition utilize the same uncertainty parameters as the flow regime remains largely the same and we are not drastically increasing or decreasing predictability with the bluff-to-bluff restriction.

The computed values in **Table 12** are representative of conditions experienced on the river and through calibration modeling. Lower flows, within the channel, are responsive to the ever-changing nature of the channel. More-frequent overbank flows exit the channel but interact with generally predictable features, such as levees and bluffs, which results in predictable flow. Less frequent, major flood flows through Jefferson City are also predictable once all the nearby levee systems are underwater, at an approximate 10% AEP flow, and the river expands bluff-to-bluff.

Table 12. Stage Error Standard Deviation

Flow Frequency AEP (%)	Stage Error Standard Deviation (ft)
50	0.9
20	0.9
10	0.6
4	0.7
2	0.7
1	0.7
0.5	0.7
0.2	0.7

5.3.3 Stage Error Uncertainty due to Sediment and Bed Changes

A Memorandum for Record documenting Missouri River bed changes from 2009 to 2021, published January 19, 2022, was consulted to inform uncertainty that could influence this analysis due to bed changes in the Missouri River system. The Missouri River is a self-scouring engineered navigation channel and is monitored for bed change consistencies. Survey data collected of Missouri River cross sections in 2014, 2018, 2019, and 2021 identified bed changes in comparison with a 2009 survey basis of comparison, as summarized in **Table 13**. Overall, the river has recently been in a trend of losing bed material through the Jefferson City study area, however loss and sedimentation patterns vary year to year. In total, nearly 114 million cubic yards of bed material was lost from 2009 to 2021, of which 78 million cubic yards occurred between 2019 and 2021.

Table 13. Average Bed Change in Feet over Missouri River Dredging Environmental Impact Statement 2011 Segments (USACE MFR, 2022)

Segment Name	St. Joseph	Kansas City	Waverly	Jefferson City	St. Charles
River Miles	**391 to 498**	**357 to 391**	**250 to 357**	**130 to 250**	**0 to 130**
2009 to 2014 (ft)	-1.40	-0.36	0.30	0.28	-0.04
2009 to 2018 (ft)	-1.62	0.18	-0.31	-0.17	-0.65
2009 to 2019 (ft)	-1.79	-0.38	-0.14	-0.04	0.06
2009 to 2021 (ft)	-2.97	-1.00	-0.93	-0.61	-0.96

Values of bed change at Jefferson City and St. Charles were used to compute average rates of bed change for the Jefferson City feasibility study modeling extents. As discussed in Appendix A1 Part 2, the Jefferson City study hydraulic model utilizes 2013 bathymetry data within the Missouri River channel. Documentation of river bed degradation rates as discussed above are variable with observed annual ranges resulting in +0.4 feet deposition to -0.4 feet degradation per year. Averaging rates since the original 2009 survey results in an average of -0.06 feet degradation per year. Maintaining the average of -0.06 feet / year of bed change would result in a bed change of -3.0 feet mean error of total degradation over the study span of 50 years, resulting in a standard deviation value of 3.0 ft / 4 = 0.8-feet (EM 1110-2-1619, equation 5-7). This standard deviation informs the uncertainty derived from developing the hydraulic model from a bathymetric survey snapshot of an ever-evolving channel. Any change in channel bed would most affect in-channel flows ranging from 50%-20% AEP and is not impactful to this feasibility study's comparison of alternatives. Therefore stage error standard deviation calculated using historic gage data as detailed in Section 5.3.2 is also sufficient for capturing uncertainty due to sediment and bed changes.

6 Climate Change and Non-Stationarity Analysis

Climate data is available for the full Lower Missouri River basin as part of the Missouri River Flow Frequency Study (2023 MRFFS, Section 2.6) in accordance with USACE policy (ECB 2018-14). A qualitative climate assessment was conducted in parallel with flow frequency development to identify the relative severity of climate change and its effects on precipitation, temperatures, and flows within the basin. This study utilized Climate Preparedness and Resilience tools to evaluate non-stationarity as well as identify trends in peak streamflow, monthly peak flow, and future hydrology.

A Tier 1, qualitative climate change analysis was conducted for the Jefferson City study site in accordance with USACE ECB 2018-14 and the complete report can be found in Appendix A2. In summary, climate change forecasts indicate the potential for likely increased precipitation and streamflow which would exacerbate flooding in the future. However, historical and projected trends in streamflows are not robust and are assumed to be gradual. As a result, flow frequencies between the existing and FWOP conditions are assumed to be stationary for this study. A quantitative, Tier 2, analysis is not necessary for this study. A risk analysis summary of future climatic factors on evaluated alternatives is included in Appendix A1, Part 3.

7 REFERENCES

7.1 USACE Primary Reference Documents

MISSOURI RIVER SYSTEM-WIDE STUDIES AND MODELS	
Considerations and Analysis for Flood Risk Resiliency for the Lower Missouri River Section 22 Planning Assistance to States	February 2022
Missouri River Flow Frequency Study – Yankton, SD to Hermann, MO	June 2023
Upper Mississippi River System Flow Frequency Study	January 2004
Missouri River Levee System Definite Project Report (1947)	1947
USACE POST FLOOD REPORTS	
2019 Post Flood Report – Missouri River: Rulo, Nebraska to the Mouth, including Key Tributaries	February 2020
2011 Missouri River Flood – Post Flood Report	May 2013
The Great Flood of 1993 Post-Flood Report – Lower Missouri River Basin	September 1994
JEFFERSON CITY PAST STUDIES	
Wears Creek flood Mitigation Study Phase II Proj. No. 00025-01 (HDR)	2003
Limited Reevaluation Report – Missouri River Levee System Unit L-142	April 2003
General Reevaluation Report & Environmental Assessment – Missouri River Levee System Unit L-142	April 2001
Wears Creek, Jefferson City, MO – General Design Memorandum Phase 1	January 1986
Wears Creek Feasibility Report for Flood Control Jefferson City, Missouri	September 1974
MODELING REFERENCES	
Missouri River – Rulo, NE to St. Charles, MO Corps Water Management System Report	March 2022
HEC-RAS User's Manual	2024
Memorandum: Federal Emergency Management Agency (FEMA) / U.S. Army Corps of Engineers (USACE) Joint Actions on Planning for Flood Risk Management Projects	2012

7.2 Flood Insurance Studies

FIS No.	Title	Date
29027CV000B	*Callaway County, MO and Incorporated Areas*	September 5, 2012
29051CV000B	*Cole County, MO and Incorporated Areas*	November 2, 2012
29151CV000B	*Osage County, MO and Incorporated Areas*	September 19, 2012

7.3 Design Criteria Source Documents

Number	Code / Criteria
ER 1105-2-100	Planning Guidance Notebook (Appendix E)
ER 1105-2-101	Risk Assessment for Flood Risk Management Studies
ER-1165-2-26	Implementation of Executive Order 11988 on Floodplain Management
EM 1110-2-1415	Hydrologic Frequency Analysis
EM 1110-2-1416	River Hydraulics
EM 1110-2-1419	Hydrologic Engineering Requirements for FDR Studies
EM 1110-2-1619	Risk-Based Analysis for Flood Damage Reduction Studies
EM 1110-2-1913	Design and Construction of Levees
ECB 2018-014, Rev 2	Guidance for Incorporating Climate Change Impacts to Inland Hydrology in Civil Works Studies, Designs, and Projects
IWR Report 88-R-2	Urban Flood Damage

Lower Missouri Jefferson City L-142 FRM Study

Appendix A1
Part 2: Hydraulic Model Development

Capital View Drainage District, looking southeast toward inundated airport May 28, 2019

U.S. Army Corps of Engineers
Kansas City District

Final Report, July 2026

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1 INTRODUCTION

This report summarizes the hydraulic analysis performed by the U.S. Army Corps of Engineers (USACE) Kansas City District (NWK) to analyze existing conditions and alternatives for the Jefferson City Levee L-142 Feasibility Study on the Lower Missouri River. Hydraulic computations are conducted using Hydrologic Engineering Center – River Analysis (HEC-RAS) software (Version RAS 6.4.1) and the model was calibrated for consistency with observed data from two recent flood events.

The objective of the Jefferson City L-142 Spin-Off HEC-RAS model is to establish a calibrated existing conditions hydraulic model in accordance with EM 1110-2-1416 as a basis for the development of hydraulic models for feasibility-level assessment of measures and alternatives.

2 BACKGROUND

The Jefferson City hydraulic model was developed utilizing reference models currently utilized by or being constructed by the USACE NWK office. Model data outside the immediate project area, as delineated by the 2D model area, has been maintained as the 1D CWMS model. Refinement of the 2D model study area included adoption of relevant geometry features from the Lower Missouri (LoMO) River System Plan model being developed in parallel and scheduled for completion after the Jefferson City study. Efforts were made to coordinate study features such as lateral structures and 2D connections across the spin-off study model and system plan models as relevant to model scale. Due to schedule offset between the Jefferson City study and Lower Missouri River System Plan, areas outside of the immediate project area reflect 2022 CWMS model sources for feasibility phase modeling. The summary of the existing and working lower Missouri River HEC-RAS models is shown in **Table 1**.

Table 1. Summary of Existing and Working Lower Missouri River HEC-RAS Models (USACE NWK)

Model Name	Description
2022 CWMS Missouri River – Rulo, NE to St. Charles, MO (2022)	Fully 1D model with channel cross sections connecting to 1D storage areas. Most updated Missouri River model downstream of Nebraska that has been fully calibrated, stress tested, and reviewed.
LoMO River System Plan (2025 Target)	Combined 1D/2D model spanning from Yankton, SD to St. Charles, MO. This model is being developed as part of a wide-reaching planning study of the Lower Missouri River to assess system wide alternatives and will be utilized to develop stage-frequency with updated 2023 flow-frequency values.
Jefferson City L-142 Spin-Off Model	This model spans from Boonville to Hermann (100 RM), however the project area is focused on the Jefferson City left-bank, specifically Capital View Drainage District. Existing information was leveraged from the 2022 CWMS model outside of the project extents and updated site data was coordinated between the Jefferson City study and LoMO System Plan, which are being developed concurrently.

2.1 Reach Characteristics

2.1.1 Missouri River

The stretch of the Missouri River between Boonville and Hermann, MO receives direct runoff from the surrounding watershed and is not immediately influenced by reservoirs. The Missouri River overbank through central Missouri is restricted by locally constructed and operated levees. Direct runoff from adjacent subbasins is further discussed in Section 4.2 where development of lateral inflow hydrographs is addressed. The levee heights in the area of study generally pass the 10-15% AEP without overtopping.

2.1.2 Major Tributaries

Two of three major tributaries modeled as reaches within the HEC-RAS model are located within 15 river miles of Jefferson City. The Moreau and Osage Rivers are located 6 and 14 river miles downstream of the Jefferson City gage respectively. The Osage River is regulated by Bagnell Dam and Harry S Truman Dam. Bagnell Dam is operated by Ameren and the Harry S

Truman Dam is operated by USACE. The Gasconade River is the third modeled tributary within the RAS model with a confluence with the Missouri River 40 miles downstream of the Jefferson City gage.

2.1.3 Minor Tributaries

Turkey Creek, Niemans Creek, Boggs Creek, and Wears Creek were not modeled as reaches due to being heavily influenced by backwater on the Missouri River during flood events. Wears Creek, although outside of the study area for this feasibility study, is located on the right bank of the Missouri River and is of special interest to the city of Jefferson City as it has historically flooded. Wears Creek was modeled as a 1D storage area. Backwater effects on Wears Creek were estimated in reference to the change in Missouri River water surface elevation at it's the mouth of Wears Creek.

2.2 Model Extent

The Jefferson City L-142 Spin-Off HEC-RAS model extends from the Boonville Missouri River gage at the upstream extent (RM 197) to the Hermann Missouri River gage at (RM 98), shown in **Figure 1**. The project area for the L-142 Feasibility Study is limited to the left-bank through Jefferson City near RM 144. The selected extents provided easily identified start and end points in relation to existing Missouri River models, encompassed major tributaries near the project area, and had available gage data.

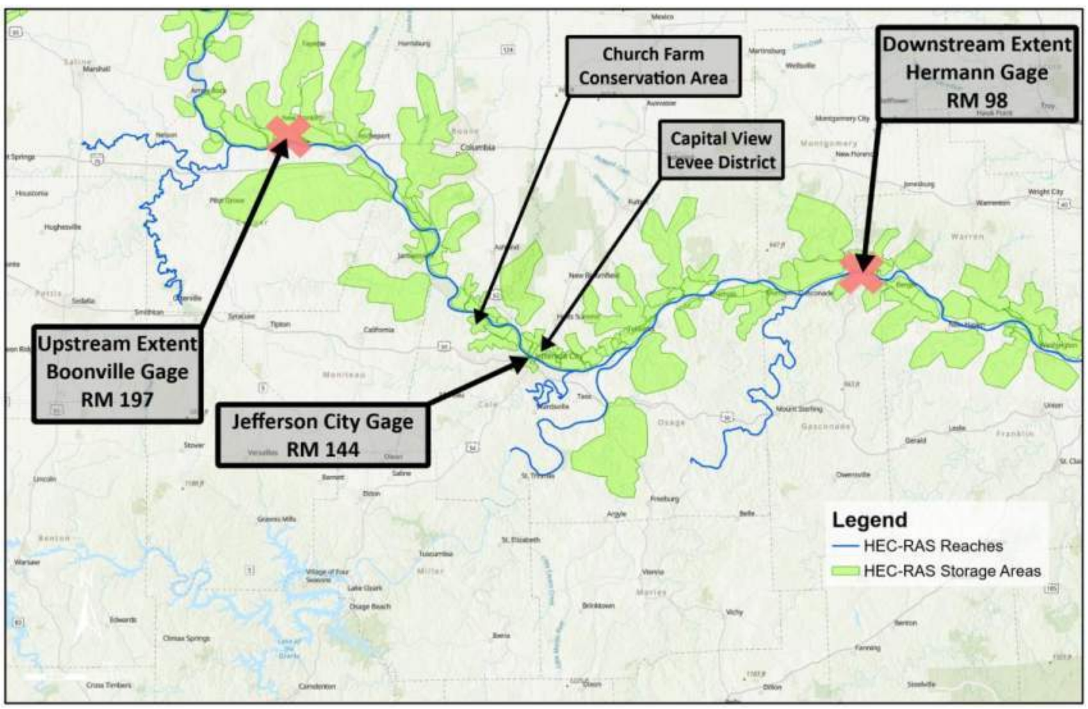

Figure 1. Hydraulic Model Extents

3 DATA SOURCES

The existing conditions Jefferson City L-142 model was developed in conjunction with the LoMO System Plan model. Data sources utilized for the LoMO System Plan were utilized within the immediate project area of Jefferson City. Due to development schedule offset, areas outside of the immediate project area reflect 2022 CWMS model sources.

3.1 Vertical Datum and Projection

Elevation data used in this study adopts the North American Vertical Datum of 1988 (NAVD88) in units of feet. The horizontal projection is Albers Equal Area Conic USGS Version (Foot US).

3.2 Bathymetry

Channel bed bathymetry and riverbank elevations within the Missouri River and major tributaries were sourced from the CWMS 2022 model. Sources for this model included 2013 bathymetry and winter 2013/2014 low water LiDAR. Updated hydrographic surveys of the Missouri River from 2021, and major tributaries from 2022 were developed for the 2024 System Plan Model. The 2013 and 2021 bathymetry data were compared and no significant changes to the overall cross-sectional area or channel shifts were observed within the Missouri River. The 2013 data was adopted for development of the Jefferson City model 1D cross-sections because the CWMS 2022 model cross sections have previously undergone full review by USACE, whereas the newer System plan model and calibration is preliminary.

3.3 Terrain

Terrain within the project area and modeled 2D storage area is sourced from 2021 Kansas City USACE Missouri River Corridor LiDAR. The 1-m terrain mesh was developed by Stantec, a FEMA contractor. The mosaic compiles the 2021 USACE low water corridor LiDAR (2-ft ERDAS Imagine hydro-flattened DEM) with 2020 USGS LiDAR (1-meter 3DEP hydro-flattened DEM), as delineated in **Figure 2**. The final combined terrain file used in RAS Mapper is consistent with the LoMO system plan model and is named “LoMo_1m_NWK.hdf”.

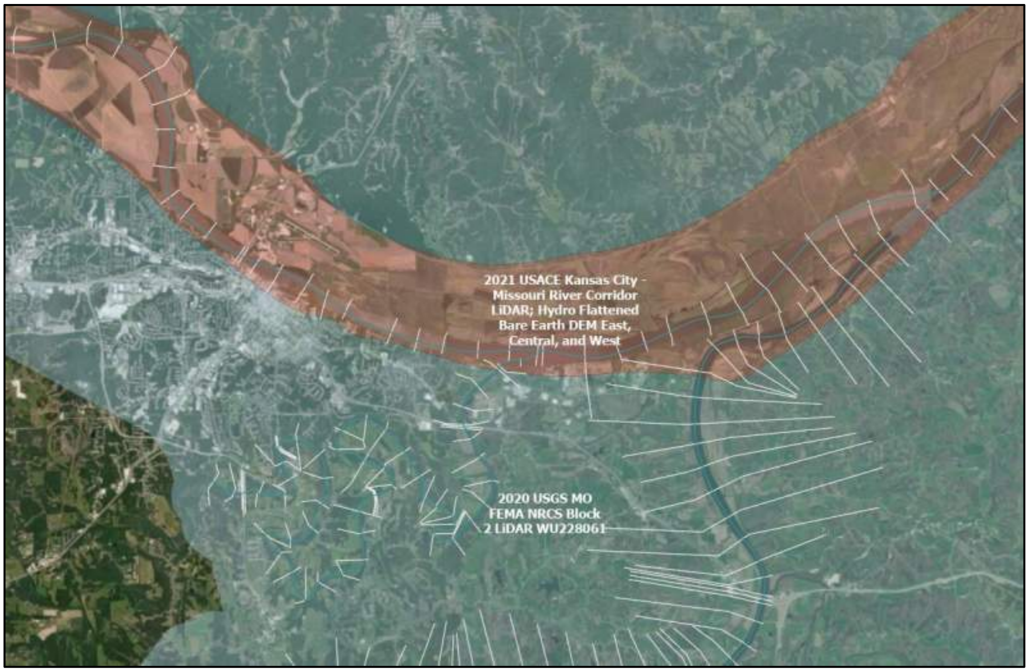

Figure 2. Jefferson City Terrain Sources

Updated terrain was utilized within the 2D project area, however the 1D cross sections reflect the 2013-2014 detailed bathymetry terrain developed for the 2022 CWMS model, named “rsc_lidar_3m_bathy2.hdf”. Cross sections depicting older terrain and bathymetric data were utilized to provide a starting point for model development, as the LoMO system plan 1D channel cross-sections were in development at the time of model construction. The CWMS model was calibrated for the 2019 event, which is the most recent flood in the model reach. Utilization of the CWMS model channel cross sections and storage area-elevation volume curves do not appear to provide large inconsistencies when compared to the most recent terrain within the project area.

The large model area utilized various sources of elevation data as summarized in **Figure 2**. Knitting together these sources lead to some discrepancies at the boundaries of more detailed bathymetry data compiled with standard USGS hydro-flattened DEM data, notably creating the appearance of a vertical drop where one does not exist where the hydro-flattened data meets the bathymetric data. For this study, overtopping of levees is driven by Missouri River backwater, and elevation difference from tributaries to the Missouri River bathymetric data does not impact results.

3.4 Survey Efforts

Survey efforts for the Jefferson City Levee L-142 Feasibility Study included collection of top elevations of the Capital View levee, Cedar Creek Drainage Dist. Sec. 2 levee, and the southern 4,000 linear feet of the western Reveaux levee tieback (**Figure 3**). This data was collected via a minimum of two GPS receivers for Real Time Kinematic (RTK) GPS positioning and has been incorporated into the National Levee Database (NLD).

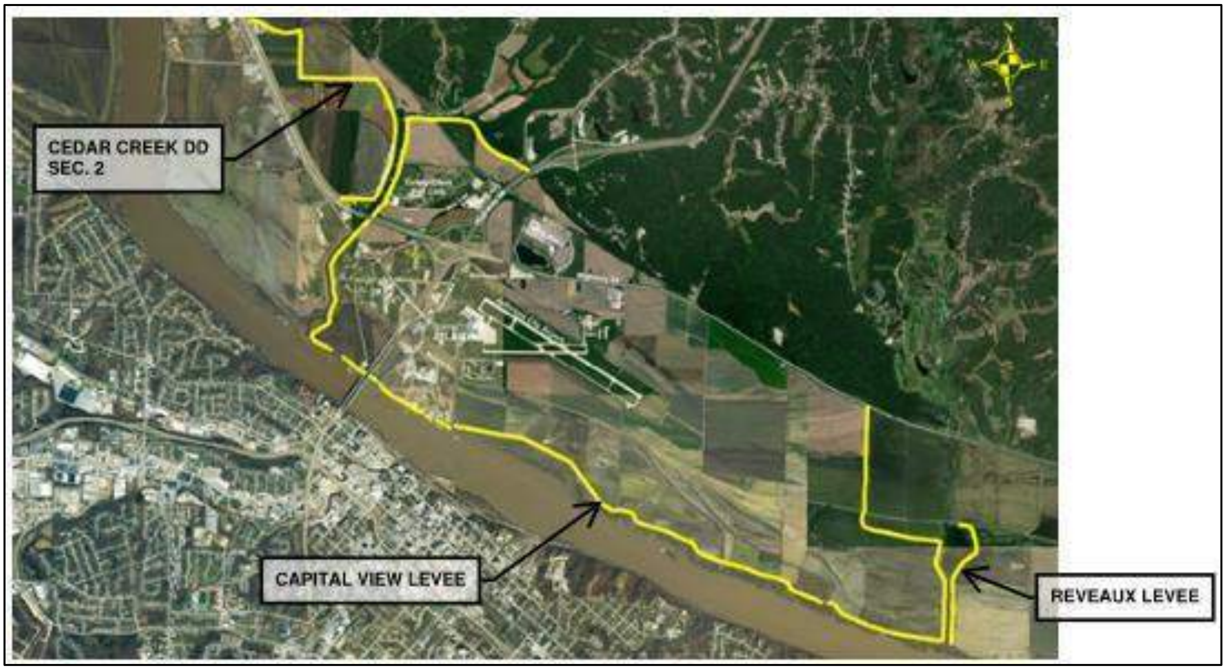

Figure 3. Survey Extents

3.5 Gage Data

A summary of the gage data and the ungaged stream approach for this model are included in Section 3 of Appendix A1 – Part 1 *Existing Conditions & Hydrology*. All gage data is reported in Coordinated Universal Time (UTC), as such, the HEC-RAS model is as well.

3.6 Land Cover

The land cover layer for the 2D area in RAS Mapper was built from the 2021 National Land Cover Database (NLCD) maintained by the USGS as shown in **Figure 4**. The 2021 NLCD compiles data from multiple federal agencies to provide information on the Nation's land cover in 30-meter pixel size density.

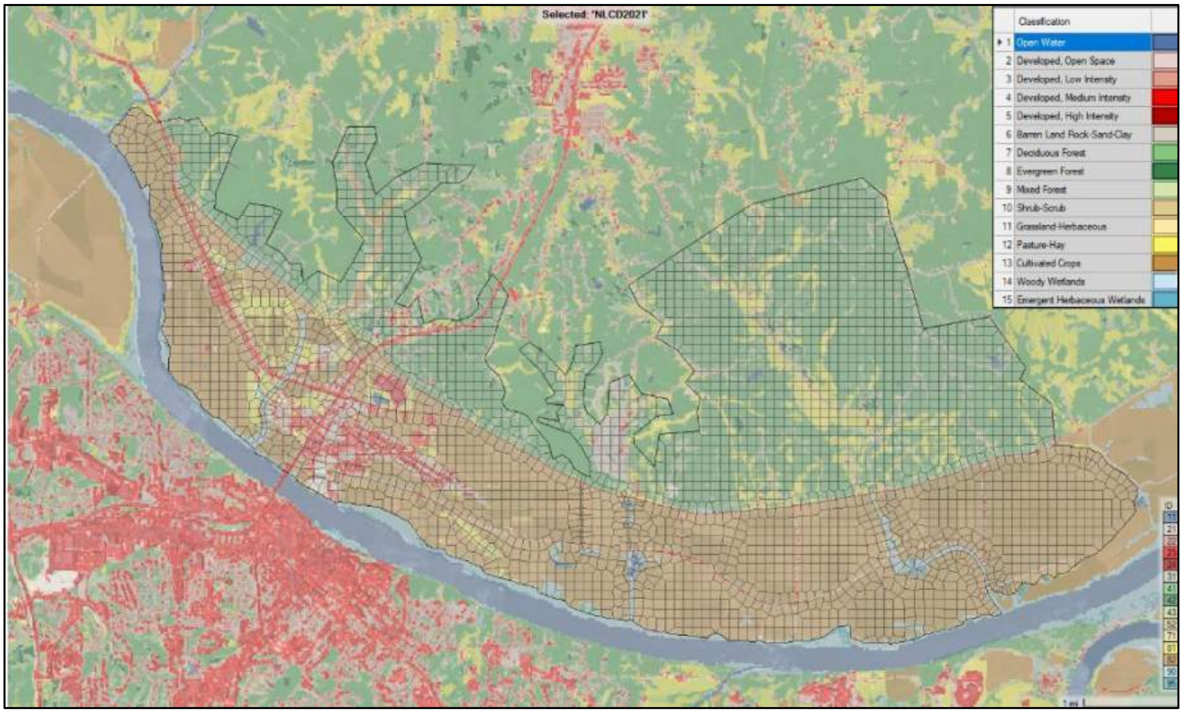

Figure 4. NLCD 2021 in 2D Flow Area

3.7 River Centerline

The centerline of the Missouri River was adopted from the recommended route line maintained by the Kansas City District Geospatial Branch (ED-S), which generally coincides with the river thalweg. As with the 2022 CWMS and LoMO System Plan models, cross sections are named to match the accepted 1960 river miles, which are the current convention for referencing river location.

4 FINAL MODEL DEVELOPMENT

4.1 Boundary Conditions

For calibration, the upstream boundary conditions at the Boonville gage and major tributaries (Moreau, Osage, and Gasconade Rivers) were established with observed flow hydrographs obtained from USGS. The USGS gage information is summarized in **Table 2**. Singular missing flow values within the DSS file were interpolated to provide a complete 15-minute time interval record for RAS.

Table 2. Upstream Boundary Condition Observed Inflow Hydrograph Sources

Reach	River Mile	USGS Gage ID	Location
Missouri	196.62	06909000	Boonville, MO
Gasconade	51.64	06934000	Rich Fountain, MO
Moreau	21.04	06910750	Jefferson City, MO
Osage	33.49	06926510	St. Thomas, MO

The downstream boundary condition is set as a rating curve at the Hermann gage. Setting the downstream boundary to normal depth was also tested, however the rating curve results were more representative of the observed gage data.

4.2 Lateral Inflow Hydrographs

Lateral inflow hydrographs, as developed with the calibrated 2022 CWMS model, were input at seven locations along the Missouri River reach, one location on the Gasconade River, and one location on the Osage River to represent flow received from minor tributaries and local runoff downstream of gage locations. The lateral inflow locations for the HEC-RAS model are denoted as yellow nodes in **Figure 5** and summarized in **Table 3**. Red hatched subbasins include mainstem flows originating below tributary gage stations and from ungaged mainstem drainage areas and are represented in the HEC-RAS model as lateral inflows. Blue hatched subbasins are accounted for in HEC-RAS boundary conditions developed from observed USGS tributary flow. The lateral inflow hydrographs were developed from a calibrated HEC-HMS model as detailed in Section 6 of the Missouri River CWMS model development report. Model results regarding stage and timing at Jefferson City did not appear sensitive to the “S_MissouriR_09” subbasin inflow location or separation into multiple upstream subbasins, as detailed in Section 6.2 of this report.

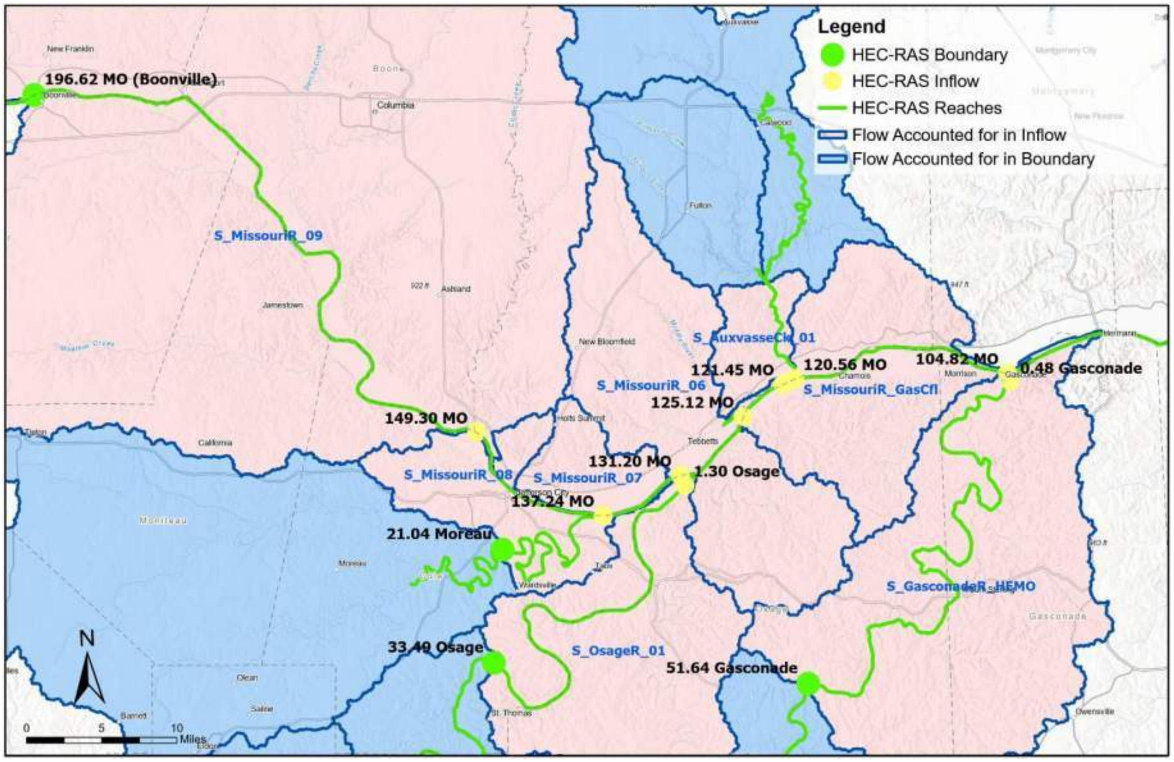

Figure 5. Inflow Hydrograph Input Locations

Table 3. Lateral Inflow Hydrograph Sub-Basin Areas (Adapted from CWMS 2022)

River	River Station	HMS Sub-Basin Name	Sub-Basin Description	Drainage Area (sq. mi.)
Missouri	149.3	S_MissouriR_09	Missouri River direct inflow – Left and Right bank Boonville to Jeff City gage locations	1767
Missouri	137.24	S_MissouriR_08	Missouri River direct inflow - Downstream Moreau River Gage (RT bank) and Turkey Creek (LT bank)	92
Missouri	131.2	S_MissouriR_07	Missouri River direct inflow - Jefferson City Left Bank	45
Missouri	125.12	S_MissouriR_06	Missouri River direct inflow – Downstream of Osage confluence to Auxvasse subbasin (LT and RT bank)	240
Missouri	121.45	S_Auxvasseck_01	Auxvasseck Creek tributary flow - Downstream of creek branch north of County Road 428 (MO River LT bank)	39
Missouri	120.56	R_Auxvasseck_01	Routed Auxvasseck Creek tributary flow – Upstream of S_Auxvasseck_01 subbasin	290
Missouri	104.82	S_MissouriR_GasCfl	Missouri River direct inflow – Left Upstream of Missouri River Gasconade Confluence	180
Osage	1.30	S_OsageR_01	Osage River tributary flow - Downstream of St. Thomas Gage	403
Gasconade	0.48	S_GasconadeR_HEMO	Gasconade tributary flow - Downstream of Rich Fountain Gage	392

Model results regarding stage and timing at Jefferson City did not appear sensitive to the inflow location of the ungaged subbasin directly upstream of Jefferson City as discussed in Section 6.2. Therefore, the location was selected to be located upstream of the study area, outside of the 2D area footprint.

4.3 Geometry

The model geometry was adapted from the 2022 CWMS Missouri River 1D HEC-RAS model; the most recently calibrated and reviewed existing Missouri River model. Modifications were made to the existing 1D CWMS model to increase level of detail for the feasibility study. Changes included:

a) Shortening the upstream model extents from the Rulo, NE gage to Boonville, MO to reduce inherent compounding computational error downstream and increase efficiency in total model runtime,

b) Replacing existing 1D storage areas in the Jefferson City left-bank project area with a single 2D area. This includes updating 2D connections and breaklines within the project area encompassing the Renz, Capital View, Reaveux, and Wainwright drainage districts,

c) Adding the U.S. Highway 54 / 63 bridge over the Missouri River in Jefferson City, and

d) Updating levee elevation data within the project area with recent top of levee elevations gathered by survey or LiDAR.

Model data outside the project area, as delineated by the 2D model area, has been maintained as the 1D CWMS model. Edits within and on the boundary of the 2D model area have been updated to reflect the most recent information available and will more closely reflect the final Lower Missouri River System Plan model being developed in 2024.

Table 4 summarizes reference model features and overview of the model geometry is shown in **Figure 6**.

Table 4. Reference Model Summary and Variances

Model Feature	Jefferson City L-142 (this effort)	CWMS (2022)	System Plan Model (2024)
River Reach Alignments and Junctions	X	X	X
1D Channel Cross sections	X	X	X
1D Storage Areas	X	X	
2D Storage Areas (Cell Size)	X (500x500)		X (2500x2500)
Lateral Structures & SA/2D Connections	X	X	X
Bridge Data	X		
Bank Stabilization and Navigation Project (BSNP) Data	X	X	X
Terrain	X	X (1D X-Sect)	X (2D)

Note: Green cells represent components utilized in development of the Jefferson City model.

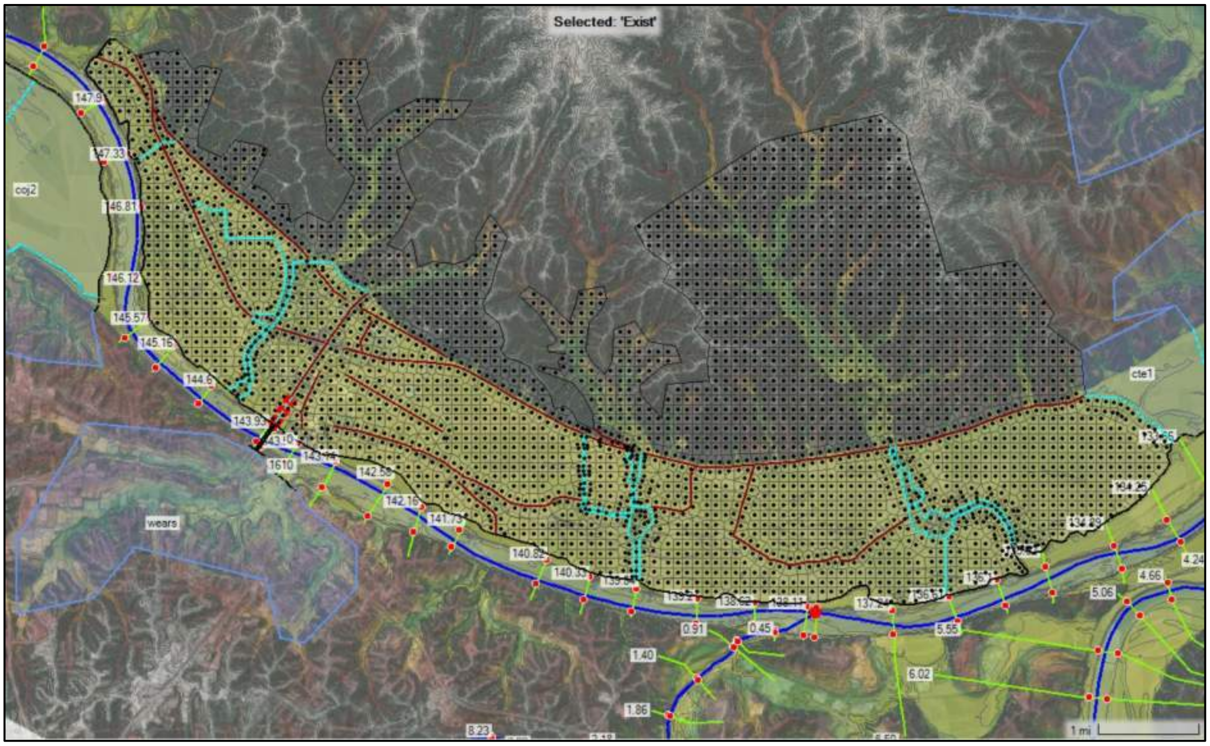

Figure 6. Existing HEC-RAS Geometry within Project Area

4.3.1 Channel Cross Sections

Channel 1D cross sections for the Missouri River and major tributaries including the Moreau, Osage, and Gasconade Rivers are adopted from the 2022 CWMS model bathymetry and winter low water LiDAR (2013). The 1D extents include the channel to the top of the adjacent levee or bluff. Where bounded by a levee, the cross section connects to a 1D or 2D storage area using lateral structures to facilitate levee overtopping events. Cross section spacing is approximately 2,500 feet on both the Missouri River and major tributaries. HTab vertical discretization parameters are set to 1-foot intervals on the Missouri River and 0.5-foot intervals on tributaries.

Manning's n-values for the channels were set referencing summer-season aerial imagery. Flow roughness factors were applied by gage reach in the model geometry, initially duplicated from the 2022 CWMS model and recalibrated upstream of the study area. Final n-values for the channel geometry are summarized in **Table 5**.

Table 5. Channel Cross Section Manning's n-Values

Land Cover	Model n-Value Range Adopted with Flow Roughness Factors
Channel – Mainstem Missouri River	0.018 – 0.025
Channel – Tributaries	0.028 – 0.035
Channel – Chutes	0.022 – 0.031
Overbank – Grass/Pasture/Crops	0.040 – 0.066
Overbank – Light to Dense Trees	0.056 – 0.165

Bank Stabilization and Navigation Project (BSNP) structures such as rock dikes, sills, and revetement structures are represented in the channel cross sections as permanent ineffective flow to the full length of the actual structures. The BSNP structures were adopted from the 2022 CWMS model and are sufficiently detailed for routing of channel flows to be modeled with this study.

4.3.2 Storage Areas

1D storage areas were maintained from the 2022 CWMS model upstream and downstream of the Jefferson City project area. These storage areas represent areas internal to levees and tributaries influenced by Missouri River backwater.

4.3.3 2D Flow Area

A 2D flow area was developed for the project area that captures the Renz, Capital View, Reveaux, and Wainwright levee districts. The total 2D area encompasses 29.9 square miles of the Missouri River left-bank and defines the region behind the levees continuing to the bluff line or tributary extents, shown in **Figure 7**. The computation point mesh for the 2D area reflects a 500-foot by 500-foot grid. Breaklines were incorporated to follow high-ground and low-ground peaks in the terrain that are critical to flow patterns. Interior levee systems are modeled as 2D connections.

Figure 7. Jefferson City 2D Flow Area Delineation

Land cover layer Manning's n-values were assigned values per the published *Manning's n-values for Various Land Covers to Use for Dam Breach Analyses by NRCS in Kansas*. **Table 6** summarizes the Manning's n-values and corresponding impervious surface percentages assigned to each NLCD land cover type.

Table 6. Manning's n-Values as Assigned to NLCD for 2D Flow Areas

NLCD ID	Name	Manning's n	Percent Impervious (%)
0	NoData	0.040	65
41	Deciduous Forest	0.160	0
43	Mixed Forest	0.160	0
81	Pasture-Hay	0.030	0
20	Developed, Open Space	0.040	0
11	Open Water	0.040	100
82	Cultivated Crops	0.035	0
31	Barren Land Rock-Sand-Clay	0.025	50
22	Developed, Low Intensity	0.100	35
23	Developed, Medium Intensity	0.080	60
24	Developed, High Intensity	0.150	75
42	Evergreen Forest	0.160	0
71	Grassland-Herbaceous	0.035	0
90	Woody Wetlands	0.120	0
52	Shrub-Scrub	0.100	0
95	Emergent Herbaceous Wetlands	0.070	0

4.3.4 Lateral Structures, Storage Area Connections and Internal 2D Area Connections

Lateral structures, storage area connections, and internal 2D area connections were utilized to represent the existing levees for specific use cases within the model. A consistent approach was utilized to define the levee top elevation for levees within the project area with survey, NLD, and LiDAR elevation comparisons to facilitate judgement of the most representative data for the structures post-2019 flood.

Lateral structures were utilized to allow the interchange of flow between the 1D channel and storage areas or 2D flow areas. Lateral structures represent both levee structures and at-grade structures. The at-grade structures allow the model to store backwater in smaller tributaries. Lateral structures modified within the project area and as listed below utilize the Normal 2D Equation option for the overtopping computation method. This was selected because levees within the project area overtop at a relatively low river flow (approximately 10% AEP) when compared to the flow events that will be modeled for the feasibility study alternative comparison. Lateral structures within the 1D model areas maintain the weir coefficients assigned from the 2022 CWMS model and range from 2.0-1.5 for levees and 0.3-0.1 for at-grade backwater connections.

The downstream tie-back at the private Renz North levee (lateral structure 148.1), was eliminated due to limitations in mesh size that would be needed to represent the channel between levees, as demonstrated in **Figure 8**. As a result, the creek at the tie-back will not inundate until the upstream levee overtops. This is an acceptable solution for this location because the Renz North levee is frequently inundated and is the first levee in this reach to inundate according to stakeholders (no USACE documentation of inundation is available due to its private levee status).

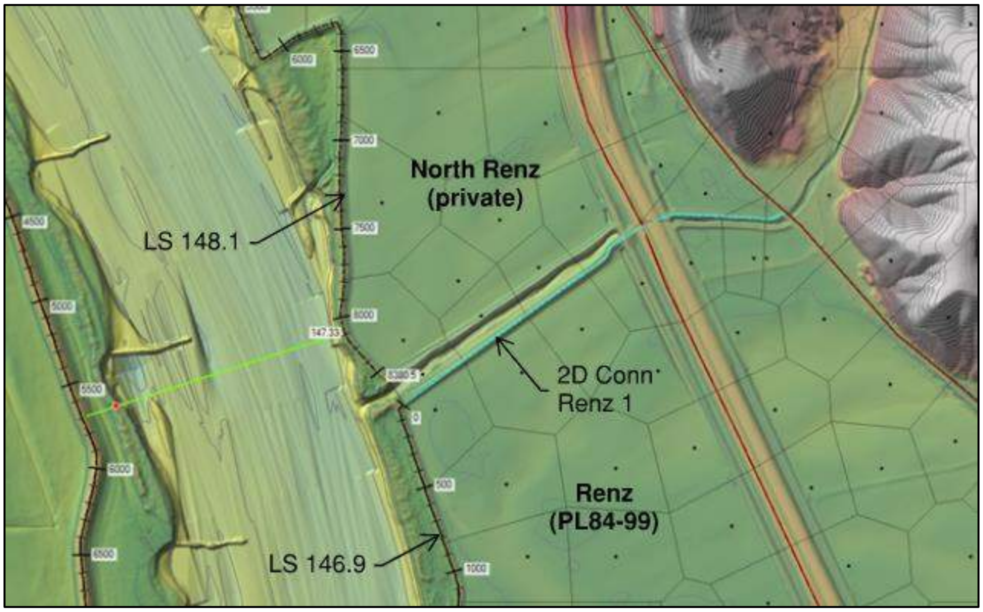

Figure 8. Renz North levee Lateral Structure and 2D Connection Layout

A summary of lateral structures and data sources relevant to the Jefferson City L-142 project area is summarized in **Table 7**.

Table 7. Jefferson City L-142 Lateral Structure Summary

Structure Name	Description	Data Source
148.1	Renz North	LiDAR (2021 USACE)
146.9	Renz	NLD survey (2010) Stations 3000-3575 no NLD data, supplemented with LiDAR, but removed irregularities.
144.4	Trib. Turkey Creek	LiDAR (2021 USACE)
144.3	Capital View	NLD survey (2023)
140	Trib. Niemans Creek	LiDAR (2021 USACE)
139.8	Reveaux	NLD survey (2010)
138	Reveaux	NLD survey (2010) Sta. 3450-4300 supplemented with LiDAR (2021 USACE) for setback repair
136.63	Rievaux DD East	LiDAR (2021 USACE)
135.8	Wainwright	NLD survey (2010). LiDAR (2021 USACE) indicates a raise of 8-12in through LS Sta. 5000-5500, however this did not appear to be a breach or repair location in 2019 and is wooded. Therefore NLD elev maintained.
135.78	Trib. Rivaux Creek	LiDAR (2021 USACE)

Storage area connections were imported from the 2022 CWMS model and correlate with the 1D geometry upstream and downstream of the project area. These structures facilitate movement of water between leveed areas during overbank flow conditions.

2D connections were utilized internal to the Jefferson City 2D project area to represent levee structures and bridges. In the HEC-RAS model, 2D connections were set to utilize the Normal 2D Equation Domain computation method. A summary of 2D connections and data sources is summarized in **Table 8**.

Table 8. Jefferson City L-142 2D Connection Summary

Structure Name	Description	Data Source
JeffCity_Renz1	Renz	LiDAR (2021 USACE) Levee setback from NLD levee location post 2019 flood repairs, NLD elevations average 11in lower than 2021 LiDAR
JeffCity_Renz2	Renz	NLD survey (2010) Sta. 0-890 supplemented with LiDAR (2021 USACE) for area without survey to follow high ground of highway embankment. Sta. 1300-1560 closed during flow, low points removed.
JeffCity_Renz3	Renz	LiDAR (2021 USACE)
JeffCity_CdrCrS2	Cedar Cr DD Sec2	NLD survey (2023). Elevations supplemented with LiDAR within Hwy 63 right-of-way Sta. 0-130
JeffCity_Cap1	Capital View	NLD survey (2023) Oilwell Rd Sta. 7600-7800 levee gap gets closed with flood action plan, low points removed and linear interpolation between
JeffCity_Cap2	Capital View	NLD survey (2023). Sta. 750-820 levee gap where Cedar City Drive crosses Turkey Creek gets closed by drainage district, low points removed and linear interpolation between.
JeffCity_Cap3	Capital View	NLD survey (2023)
JeffCity_NiemCr1	Cedar Cr DD Sec1	LiDAR (2021 USACE)
JeffCity_NiemCr2	Private Non-NLD	LiDAR (2021 USACE)
JeffCity_Rvx1	Reveaux	NLD survey (2023)
JeffCity_Rvx2	Reveaux	NLD survey (2010)
JeffCity_Rvx3	Reveaux	NLD survey (2010) Sta. 3350-3825 setback reflected in geometry, flat line assumption for elevs
JeffCity_RvxCr	Rievaux DD East	LiDAR (2021 USACE)
JeffCity_Wnw1	Wainwright	NLD survey (2010)
JeffCity_Wnw2	Wainwright	NLD survey (2010)

4.3.5 Bridges

Bridge plans were provided by MODOT for the study area. Local road bridges crossing tributaries within the 2D area are represented in the 2D terrain mesh and closely approximate the cross-sectional area of the drainage structures. The U.S. Highway 54/63 bridges crossing the Missouri River were modeled in HEC-RAS. These structures are both MODOT owned. Bridge L0550, designed in 1953, is located upstream of bridge A4497, designed in 1987. There is an extensive pier and steel pile network for the bridges on the left-overbank. The bridges are both set at bluff height of the Missouri River right bank. During historical flood events there has been minimal hydraulic interaction between the Missouri River and bridge deck structure. **Figure 9** shows the impact of the 2019 flood event on the bridge structure.

Figure 9. Missouri River Bridges (2019 Flood)

Top: Looking West from Airport, Bottom: Looking Southwest

Due to the close proximity of the bridges, they were modeled as a single structure, with the most restrictive structural elements modeled. Bridges were input as Bridge/Culvert Data within 1D cross section limits and were continued into 2D cross section with a SA/2D Connection. Elevations of the US 54 over Missouri River bridges were directly translated from MODOT bridge plans from 1953. No vertical datum was provided with bridge plans and no recent plans with verification of vertical datum were provided. With no information on the vertical datum, elevations were scaled to match existing LiDAR features. A maximum girder depth of 8-feet, 6.5-inches and a bridge deck curb of 10.5-inches were assumed as consistent depth for the modeled girder (total of 9.4-feet). Bridge plans as provided by MODOT are included as **Figure 10** and **Figure 11**.

Figure 10. Bridge General Layouts Overlaid on Aerial to Confirm Pier Location

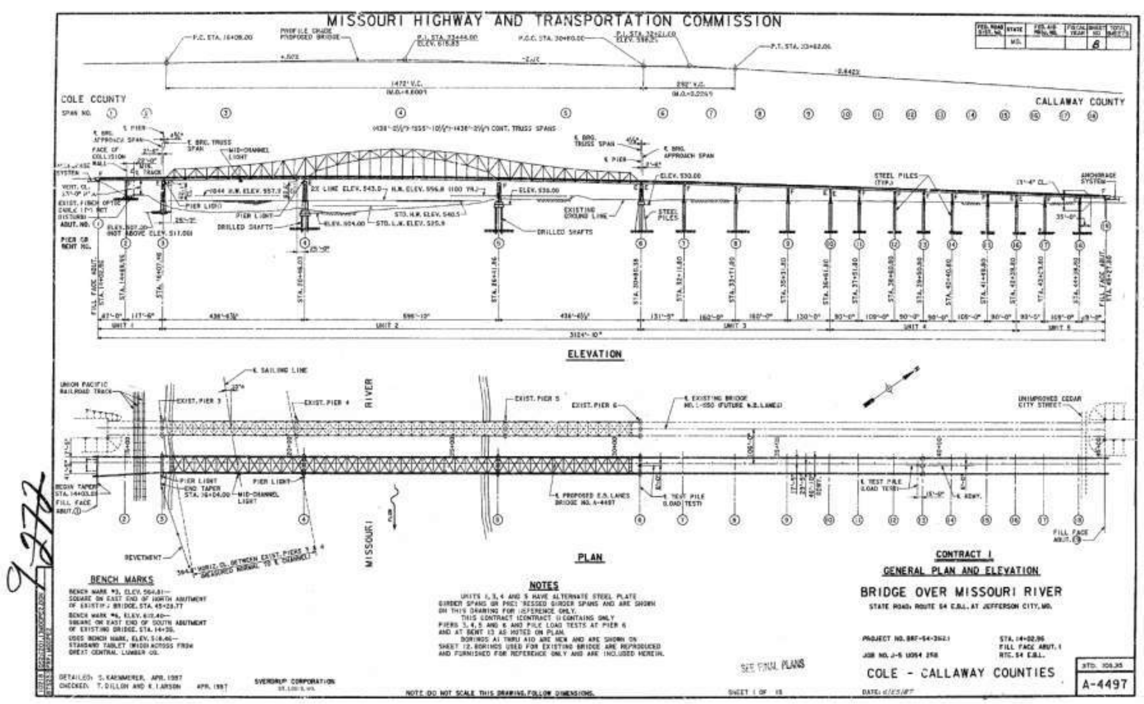

Figure 11. A4497 Bridge General Layout (MODOT 1987)

The bridge abutments are set high through this area, so gradual contraction/expansion is accurately represented in the channel cross sections. Any ineffective flow areas due to abutments/bridge embankment grading on the right bank are perched very high on the bluff line, above project features, and a 1:1 expansion/contraction for the adjacent cross sections resolves landside of the bluff. Therefore ineffective flow areas were not modeled for the 1D bridge. The left bank abutment is included in the 2D area and is represented as a 2D connection.

A pedestrian walkway column connecting the boat ramp parking lot to the downstream side of the southern highway bridge was added to the bridge structure in 2010, shown in **Figure 12**. This walkway is three levels, with 18-ft clearance from deck surface to deck surface and is entirely fenced with 5-ft chainlink (**Figure 13** and **Figure 14**). The walkway structure is 137-ft square. Due to the surface area of the chainlink fence and likelihood of collecting debris, this area was included as a pier in the bridge geometry.

Figure 12. Missouri River Bridge Pedestrian Access Ramp Location

Figure 13. Pedestrian Access Ramp

Note: Looking Southeast Past Northeast-bound Vehicular Lanes

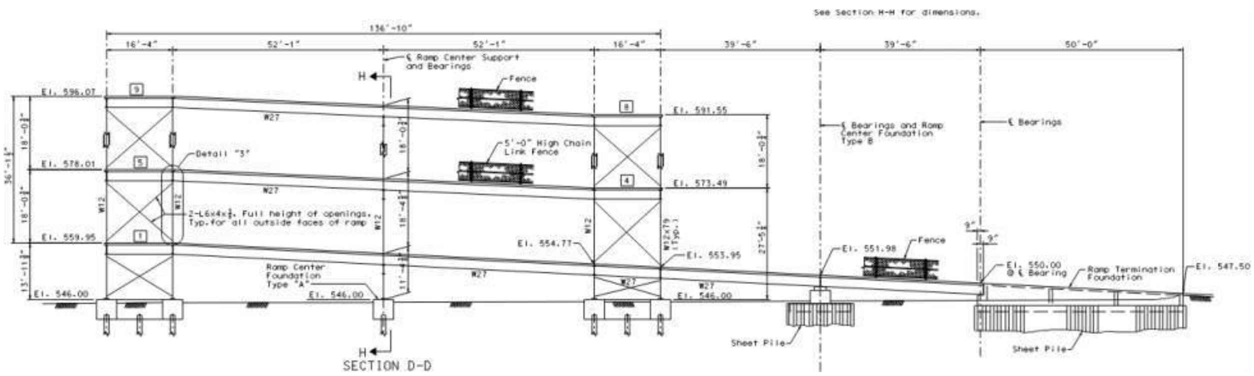

Figure 14. Pedestrian Access Plan Section
Source: MODOT 2012

4.4 Model Computation

Diffusion wave computation method was used for calculations. Full momentum computation (SWE – ELM) was also tested, however it was not as stable as diffusion wave with many cells within the 2D area calculating at maximum iterations, even with the timestep reduced to 1-minute. **Figure 15** depicts a comparison of maximum water surface elevation (WSE) results for the 2019 flood event calculated with both diffusion wave and shallow water equation methods. Computation methods produced identical results at most locations. Diffusion wave results were generally 0.1-ft higher north of U.S. Highway 63 with minor variation at the limits of inundation. The shallow water equation results were subtracted from the diffusion wave results to report differences. Comparison of the results do not indicate a need to further pursue stabilizing the model for shallow water equations.

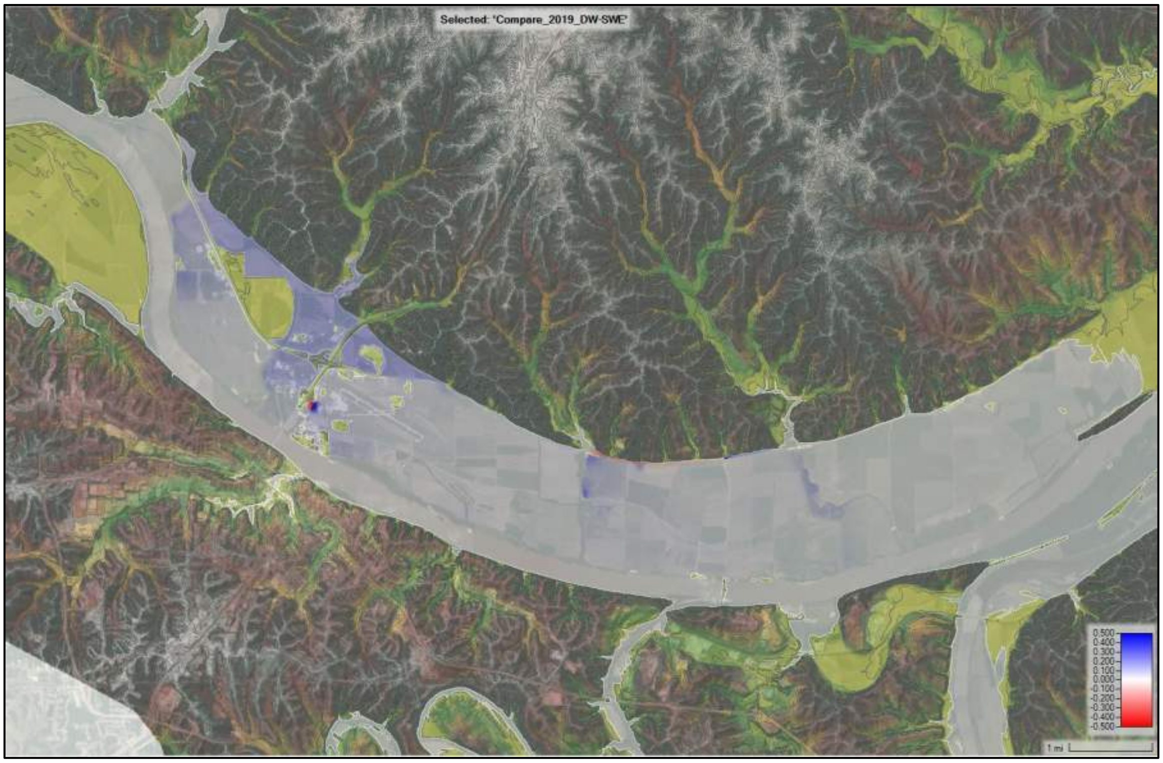

Figure 15. Comparison of Diffusion Wave and Shallow Water Equation WSE Results, Plotted over Terrain

A 48-hour warmup period (1 hour time step) was added to ensure connectivity of the 1D and 2D interface at startup. Computation settings were set to a computation interval of 30-seconds, with 30-minute mapping, 30-minute hydrograph, and 8-hour detailed output intervals. Computation interval limitations were set for all plans to accommodate the smallest computation interval required to effectively model levee breaches during the higher flow events. Mapping output sensitivity was conducted during development for the hydraulic model and again after alternatives were being developed to ensure calculated maps were reflecting model calculations. This was completed by dropping the 30-minute mapping output to 1-minute and comparing mapping results. It was determined 30-minute mapping output was sufficient for this model.

4.5 Summary of Model Organization

Geometries were developed for calibration to the May 2019 flood event and adjusted for verification of the October 2018 storm event in channel. Geometries utilized to develop the existing conditions model are listed in **Table 9**.

Table 9. HEC-RAS Existing Conditions Geometries Summary

Geometry	Name	Notes
.g09	Exist	Existing conditions calibration to the May 2019 flood event. Reach extends from Boonville, MO to Hermann, MO gage.
.g08	Exist_FlowRough_CalibOct18	Existing conditions geometry revised utilizing roughness coefficients to recreate October 2018 storm event stages.

Unsteady flow data was developed utilizing the 2022 CWMS model. Unsteady flow data files utilized to develop the calibrated HEC-RAS model are summarized in **Table 10**.

Table 10. HEC-RAS Existing Conditions Unsteady Flow Data Summary

Unsteady Flow Data	Name	Notes
.u01	CWMS_May2019	May 2019 flood event flow, developed and calibrated with the 2022 CWMS model
.u02	CWMS_Oct2018	October 2018 storm event flow, developed and calibrated with the 2022 CWMS model

All plan files utilized for HEC-RAS existing conditions models are included in **Table 11**.

Table 11. HEC-RAS Existing Conditions Plan File Summary

Plan File	Name	Notes
.p01	Calib_May2019_breaches	Calibration to the May 2019 flood event, breaches incorporated
.p04	Calib_May2019	Calibration to the May 2019 flood event, breaches not incorporated
.p03	Calib_Oct2018	Verification of the October 2018 storm event

5 CALIBRATION AND VALIDATION

Calibration of the model was conducted referencing river gages and documented flood high-water marks. The 2019 flood event is the most recent flood event overtopping the Capital View levee and has the most observed data available for calibration. Therefore, the 2019 event was utilized as the main calibration event to ensure efforts include overland flow and levee overtopping. The October 2018 event was utilized for verification that the model calibration is reasonable for lower flows, however in-bank flow is not the focus of this study. A summary of mainstem Missouri River gages and available information, including rating curves, flow hydrographs, and stage hydrographs, is provided in **Table 12.** High-water marks were collected by NWK USACE during the May 2019 flood and documented in the 2019 Post Flood Report. The main calibration parameter for this model was adjustment of flow roughness factors.

Table 12. Summary of Observed Gage Data

ID [Operator]	Cross Section	Location	Datum (NAVD88)	Data Availability
6909000 [USGS]	196.62	Boonville	565.58	Stage / Flow (2015) / Rating Curve
6910450 [USGS]	143.86	Jefferson City	520.18	Stage / Flow / Rating Curve
CMSM7 [NWS]	117.03	Chamois	502.76	Stage / Flow / NWS Rating Curve
6934500 [USGS]	97.93	Hermann	481.50	Stage / Flow / Rating Curve

With the focus on replicating river stages from the 2019 flood event, model results were calibrated to 2021 USGS rating curves utilized for the 2022 CWMS model. The NWS rating curve at Chamois is included for comparison, but the methods utilized to develop this curve are not field measured and were not prioritized for calibration.

5.1 Observed Events

Two observed events were utilized to refine model parameters, see **Table 13**. The base model outside of the immediate project area was calibrated with the 2022 CWMS model, therefore channel n-values of the 1D channel were maintained, but flow roughness factors were adjusted to better match calibration targets.

Table 13. Summary of Observed Events

Event Name	Event Duration	Description	Modeling Focus
May 2019	May 17 – June 15	Largest recent flood event through project area with many levee breaches and overtopping instances.	Overall model calibration. Flow roughness factors for model extents.
October 2018	October 5 – 31	Non-overtopping event for Missouri River levees. Single peak hydrograph.	Confirmation of 1D channel n-values and flow roughness factors for in-bank flows.

There are challenges to defining additional calibration events for this reach of the Missouri River. The 1993 event greatly impacted the river corridor, resulting in changes to physical conditions. The river bathymetry was likely quite different than today's existing condition. Human response to flood management, including levee construction, has impacts to overbank flow. The

Jefferson City gage has only routinely measured instantaneous flow since 2015, so all storms prior would rely on field measured flow for calibration.

5.1.1 May 2019 Event

The May 2019 Missouri River flood event at Jefferson City was defined by high-sustained peak flows and river stages. The peak flow of 366,000 cfs was measured by the USGS on June 6th. The first wave of levee overtopping near Jefferson City began on May 24, with levees continuing to overtop/breach throughout the flood. Levee overtopping times presented in the Post 2019 Flood Report are reported in Central Standard Time (CST). The HEC-RAS model is set to UTC, which is 6-hours ahead of CST.

Calibration efforts prioritized the May 2019 event as it is the most recent flood in the study area. It represents the flood condition to be modeled in the alternative analysis stage of this study.

5.1.2 October 2018 Event

The October 2018 event was confined by levees on the Missouri River. This event was used to test stability and inconsistencies in connectivity/low points of lateral structures, the 1D channel, and the 2D flow area in the HEC-RAS model. This event was used to validate and check consistency in the 1D channel Manning's n-values and flow roughness factors assigned by reach. Peak flow for this event generally correlates with bank-full for the length of the Missouri River mainstem.

5.2 2019 Event

5.2.1 Flow Comparisons at the Jefferson City Gage

The calibrated flow hydrograph at the Jefferson City gage is generally low and slow to arrive for the May 2019 flood event, however the second and third peak flow occurring June 1st and June 7th are consistent with observed flows, as seen in **Figure 16**. The maximum recorded USGS instantaneous peak flow was 366,000 cfs for the May 2019 event. The modeled peak flow results are 359,900 cfs with and without levee breaches.

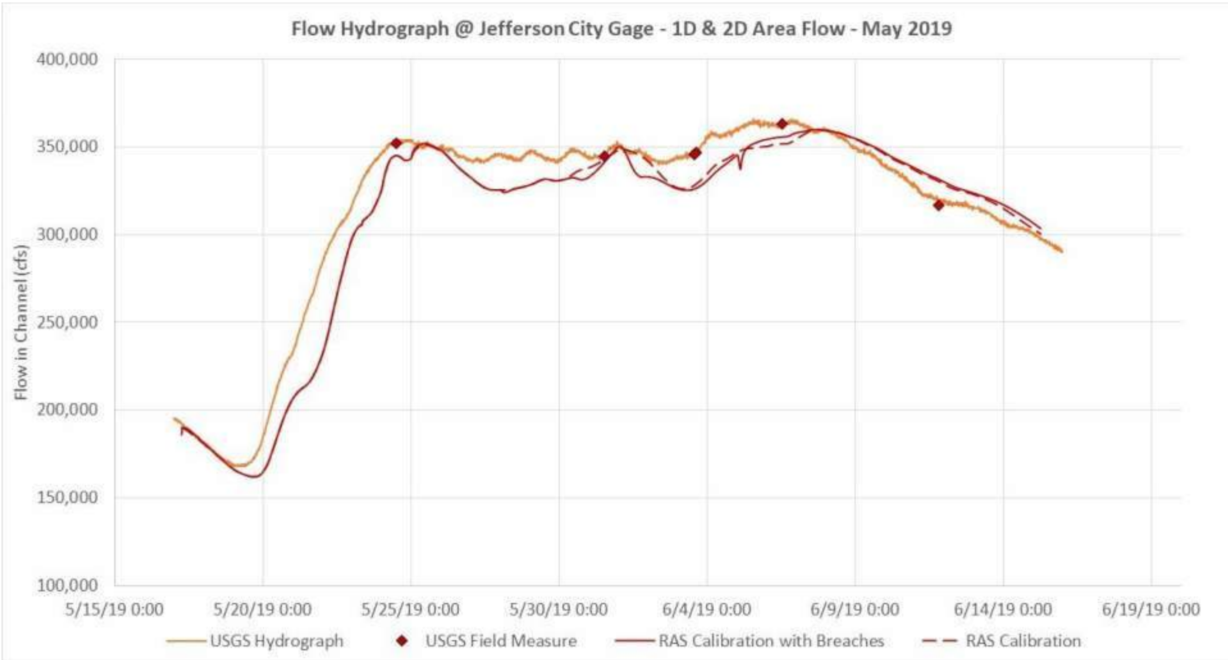

Figure 16. Flow Hydrograph at Jefferson City - May 2019

The difference in flow could be due to the developed hydrology of ungaged streams. The lateral inflow hydrograph for all ungaged flow upstream of Jefferson City inputs into the model immediately upstream of the study area. Additional data for tuning of the ungaged flow is unavailable, and the existing hydrology is the best estimated for the May 2019 event between the Boonville and Jefferson City gage. Levee breaches did not have a large impact on the overall flow hydrograph observed at Jefferson City.

Sensitivity of the model to the internal hydrograph input location was tested, including scaling the internal hydrograph upstream of Jefferson City into two subbasins. With this test, the *S_MissouriR_09* CWMS hydrograph was scaled to 57.7% and input at Missouri River Station 170.88 and the remaining 42.3% was input at RS 149.3. The maximum difference in WSE observed in the May 2019 breach condition utilizing two hydrograph inputs upstream was 0.04-feet, which resolved between river stations 172 and 152. There was also no impact to timing or peak flow to the Jefferson City gage flow hydrograph. Thus, the 2022 CWMS internal hydrograph input at RS 149.3 was maintained.

5.2.2 Maximum Water Surface Elevation Comparisons

Water surface elevations closely match the observed stage hydrograph at Jefferson City (**Figure 17**) and observed data through the length of the modeled Missouri River (**Figure 18**). Some outliers in reporting are noted, specifically at the Jefferson City gage where the USACE collected high water mark (HWM) is much higher than the USGS measured stages. The USACE collected HWM through this stretch were not classified as high-confidence values in the 2019 Post Flood Report. The USGS reported measurements were favored for calibration.

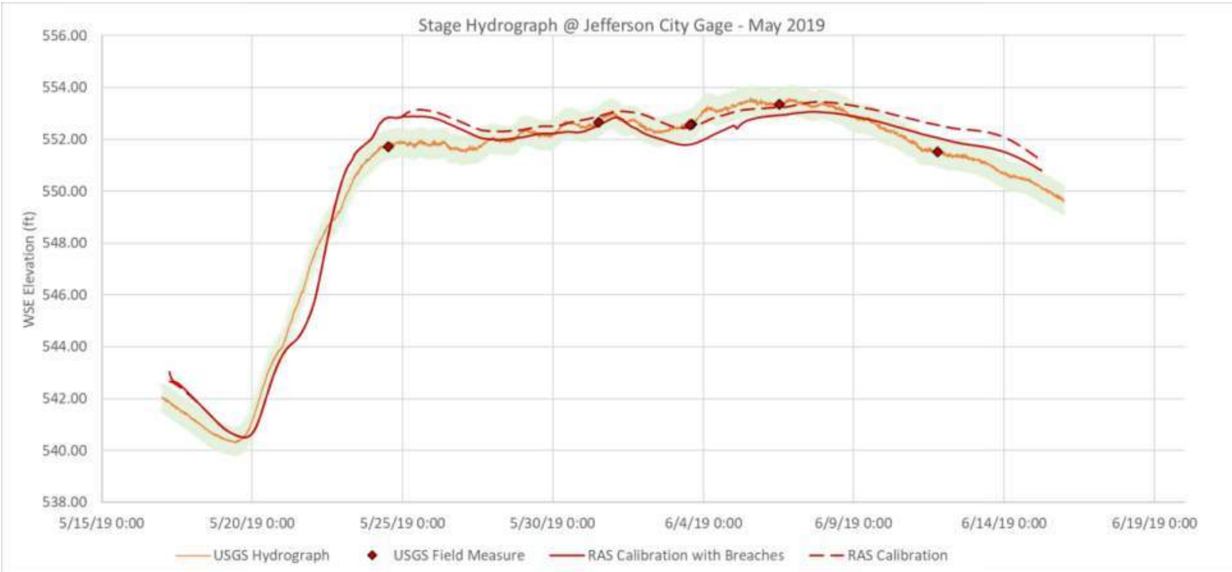

Figure 17. Stage Hydrograph at Jefferson City - May 2019

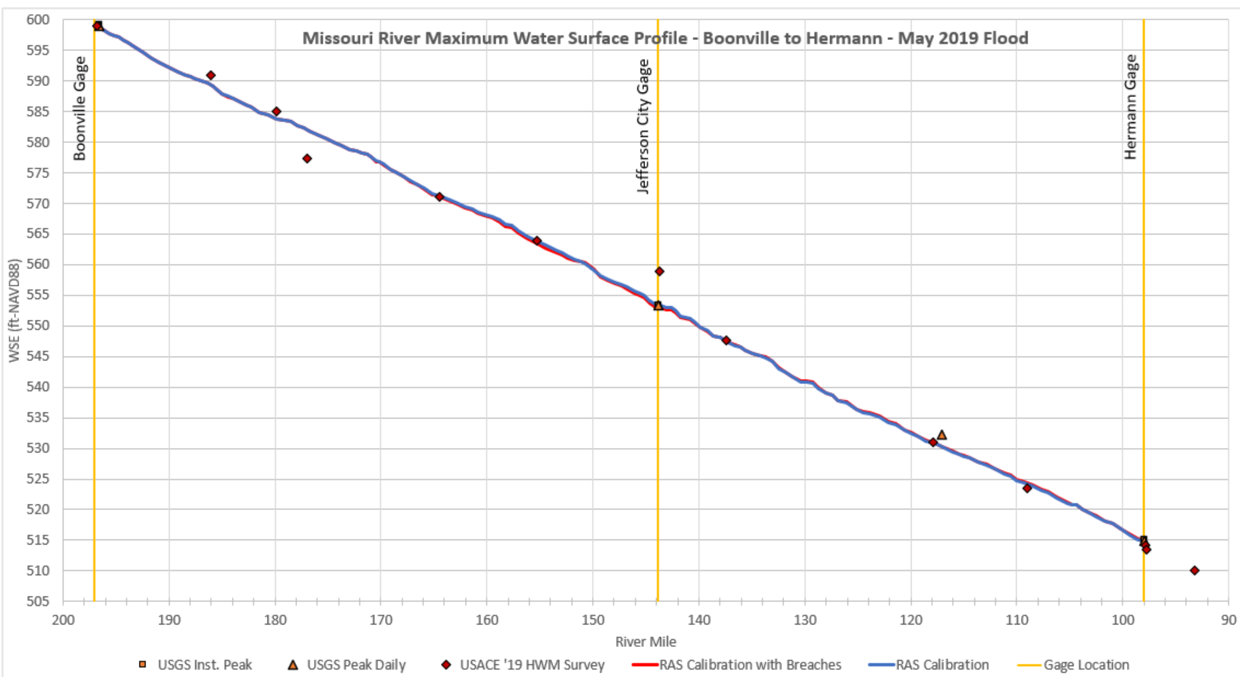

Figure 18. Missouri River WSE Profile - May 2019

USACE field personnel collected HWM after the 2019 flood. Available data for the study extents are shown in **Figure 19**. It is notable that the data through this reach is labeled Fair to Very Subjective due to the availability of evidence and limitation of equipment at the time of collection. Therefore, these marks inform the model calibration efforts, but should not be utilized as concrete evidence.

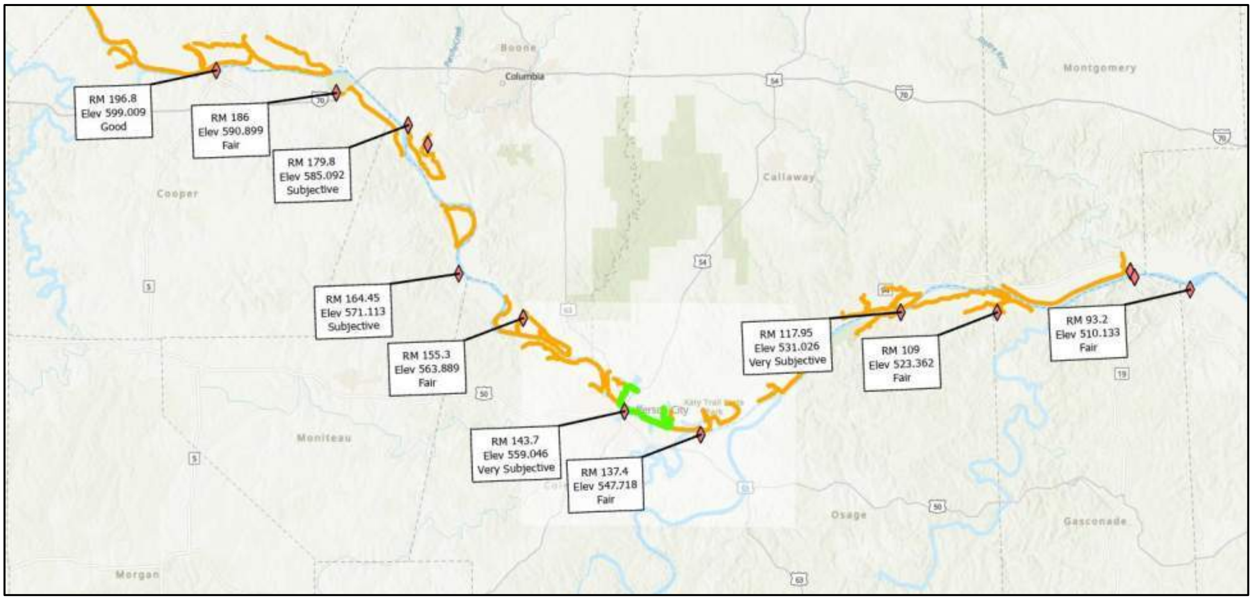

Figure 19. USACE Collected HWM for May 2019 Flood

5.2.3 Rating Curve Comparison

Comparison of the calibrated model to the 2021 USGS rating curve at Jefferson City is within 1.0-foot of the observed USGS curve between flows ranging from 150kcfs to 350kcfs during the 2019 flood event, as shown in **Figure 20**.

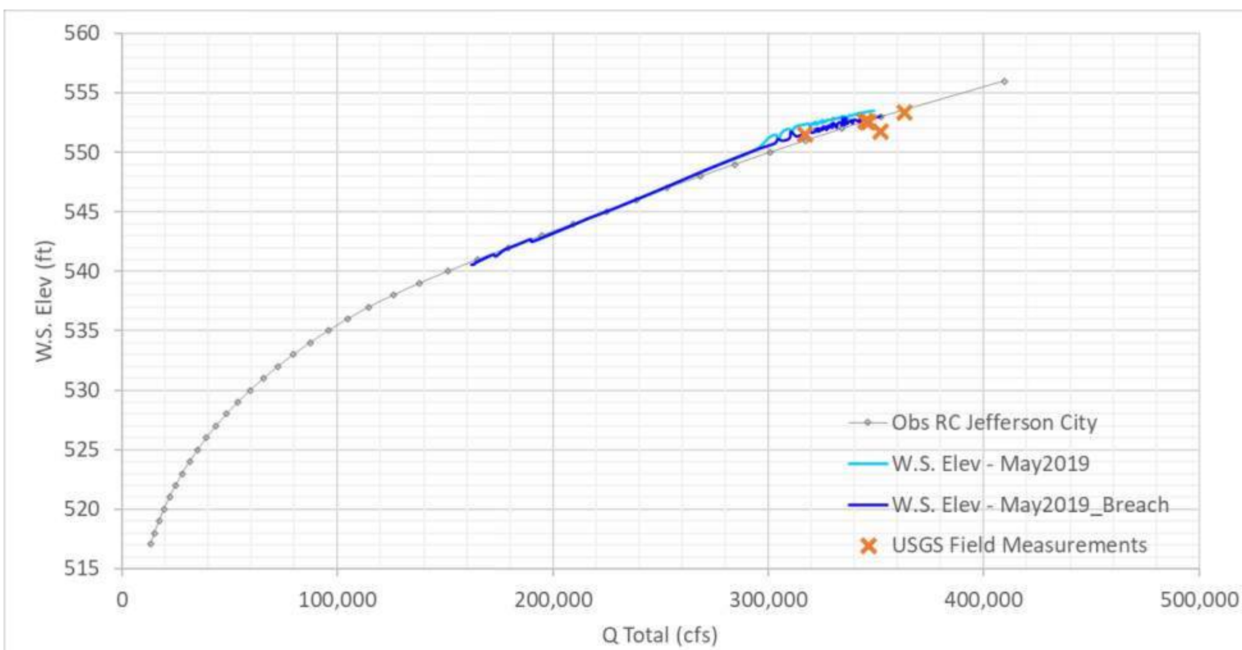

Figure 20. Jefferson City Rating Curve Model Output Comparison – May 2019

5.2.4 Inundation Extents

Inundation in the 2019 event was generally bluff-to-bluff through Jefferson City except for the Cedar Creek Drainage District Section 2, a private levee located north of U.S. Highway 63, and minimal ponding behind the Cole Junction levee, directly upstream of Jefferson City on the right bank. An aerial view of observed flooding through Jefferson City is presented as **Figure 21**. **Figure 22** presents the modeled inundation extents for the May 2019 HEC-RAS model.

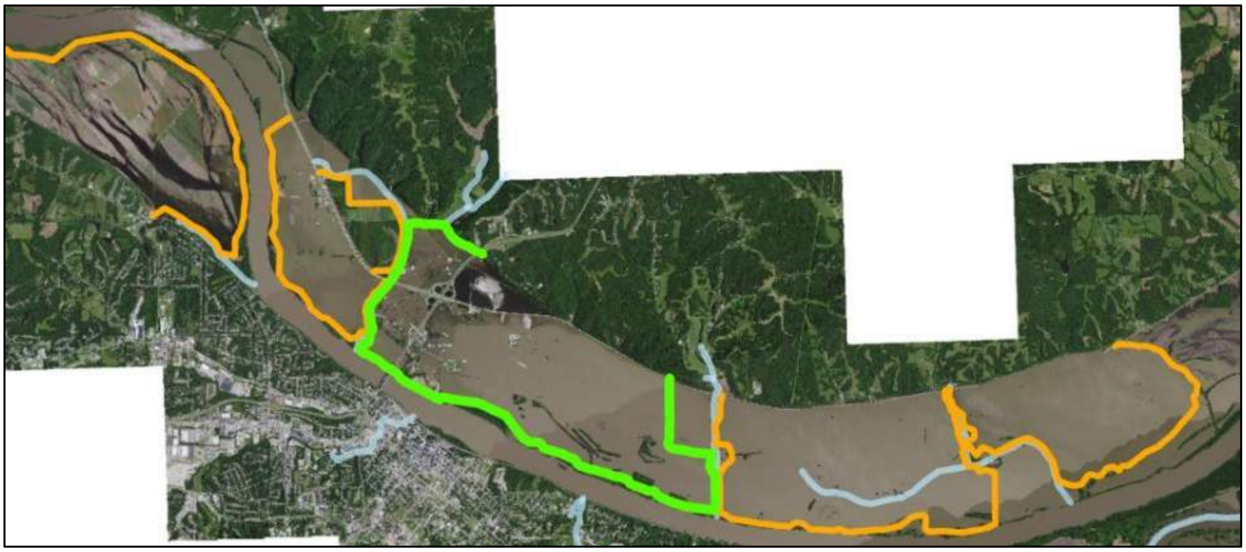

Figure 21. USACE Imagery - Jefferson City 06/13/2019

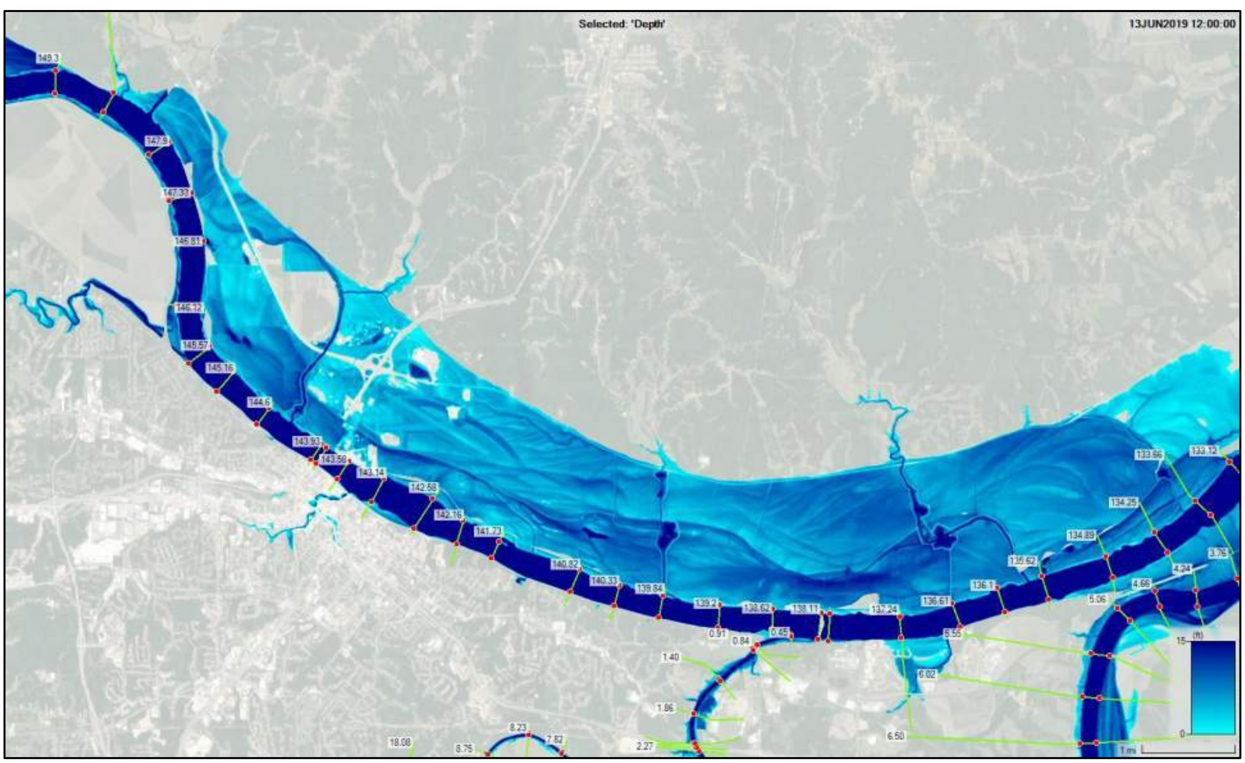

Figure 22. Computed Inundation for May 2019 Flood Event – 06/13/2019

5.3 October 2018

The October 2018 event was contained within the Missouri River levees through the study reach. This event was run to ensure the Manning's n-values and flow roughness factors within the 1D channel, prior to existing levees overtopping, are reasonable.

Overall, the maximum water surface profile follows the observed USGS peak daily flows through the reach, with slight overestimation of the maximum water surface elevations (**Figure 23**).

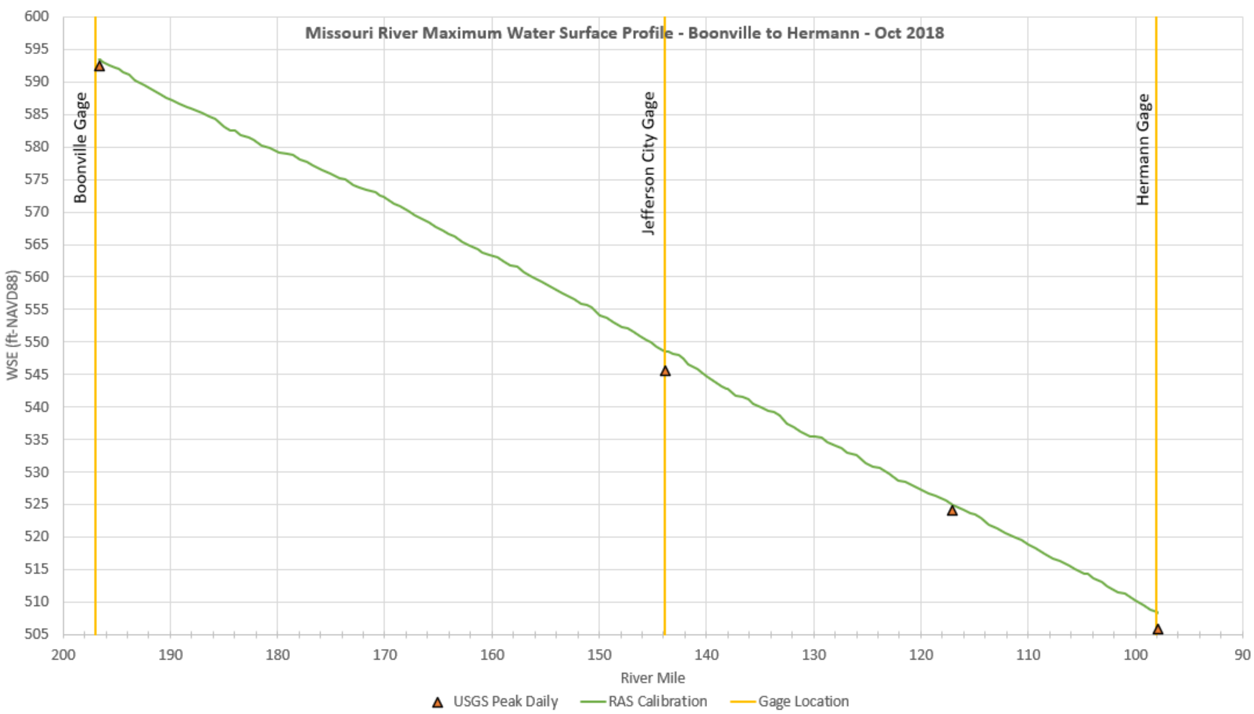

Figure 23. Maximum Water Surface Elevation through Study Reach - October 2018

The trendlines for the 2018 event are representative of the observed USGS instantaneous flow (**Figure 24**) and conservatively estimate stage (**Figure 25**) through the study limits, therefore the 2019 calibration values are acceptable. This model is not intended to be utilized for flow events contained within the existing levee systems on the Missouri River. Differences in stage between those observed and the model are influenced by the slight overestimation in flow at the Jefferson City gage. Other data limitations influencing in-bank stages could be due to 1D channel cross-sections being developed from 2013-2014 detailed terrain and not fully representing the 2018 condition and/or the modeling approach of navigation structures, both of which would most impact results of lower flow events.

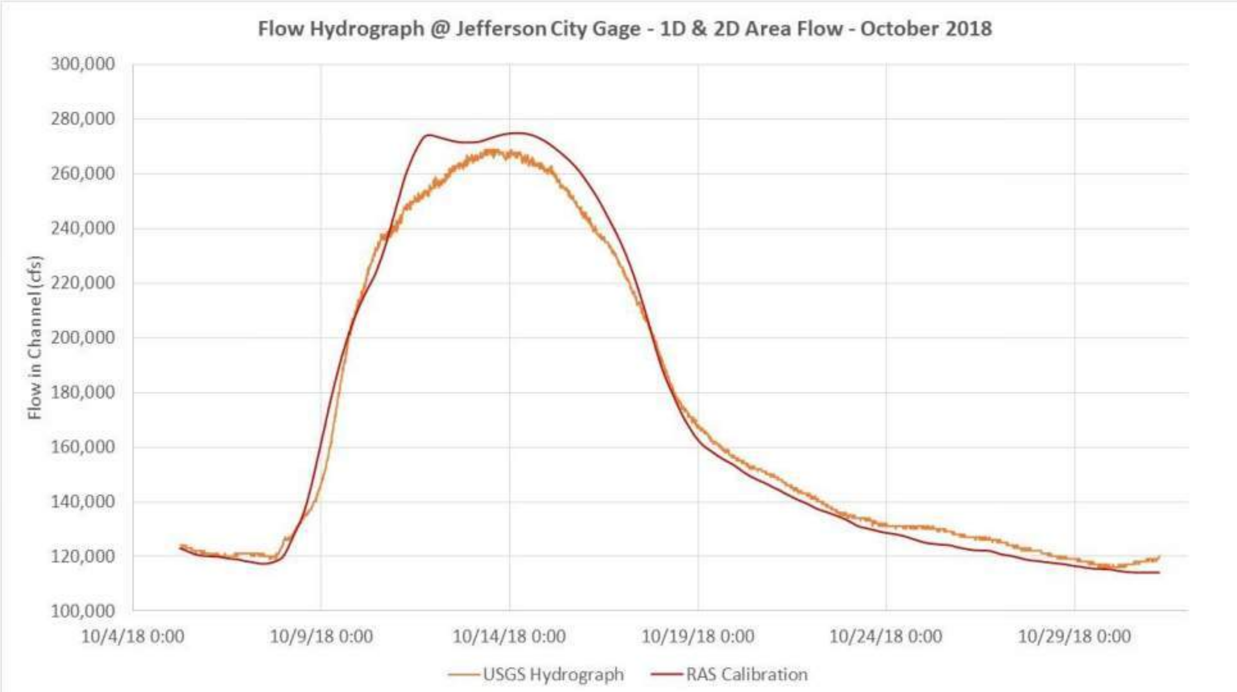

Figure 24. Flow Hydrograph at Jefferson City Gage - October 2018

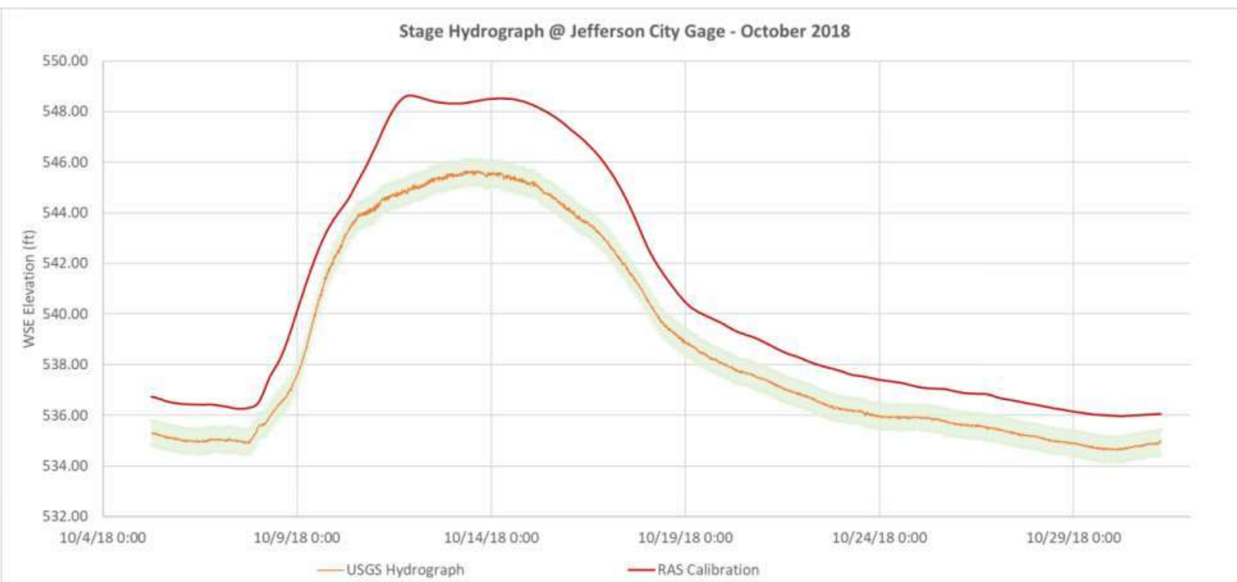

Figure 25. Stage Hydrograph at Jefferson City Gage - October 2018

5.3.1 Rating Curve Comparison

Comparison of the calibrated model to the USGS rating curve at Jefferson City is within 1.0-foot of the observed USGS curve between flows of 115kcfs – 275kcfs during the October 2018 flood event as shown in **Figure 26**. The observed USGS rating curve from 2017 was also included in the plot for comparison. Due to the ever-changing river dynamic, the observed rating curve in 2017 was lower than the 2019 calibration. Therefore, a model calibrated to the 2019 event is anticipated to have higher water surface elevation outputs at Jefferson City than for the same flow observed between 2017 and 2019.

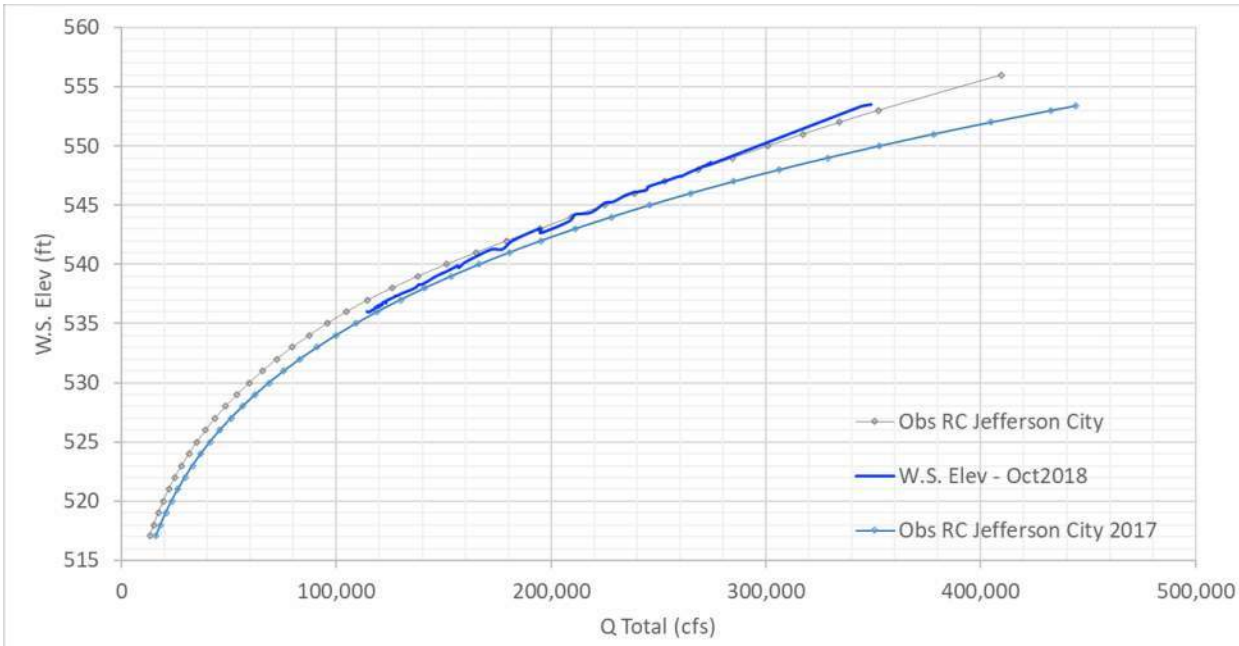

Figure 26. Jefferson City Rating Curve Model Output Comparison – October 2018

5.3.2 Sensitivity of Recalibration of Model to October 2018 Event

An effort to match stages for both the lower flow 2018 event and higher flow 2019 event was made. Results of matching the October 2018 Jefferson City gage stage hydrograph utilizing in-bank roughness factors is shown in **Figure 27**. The difference in channel n-values when matching the 2018 event in comparison to the 2019 event is on average 0.005 lower.

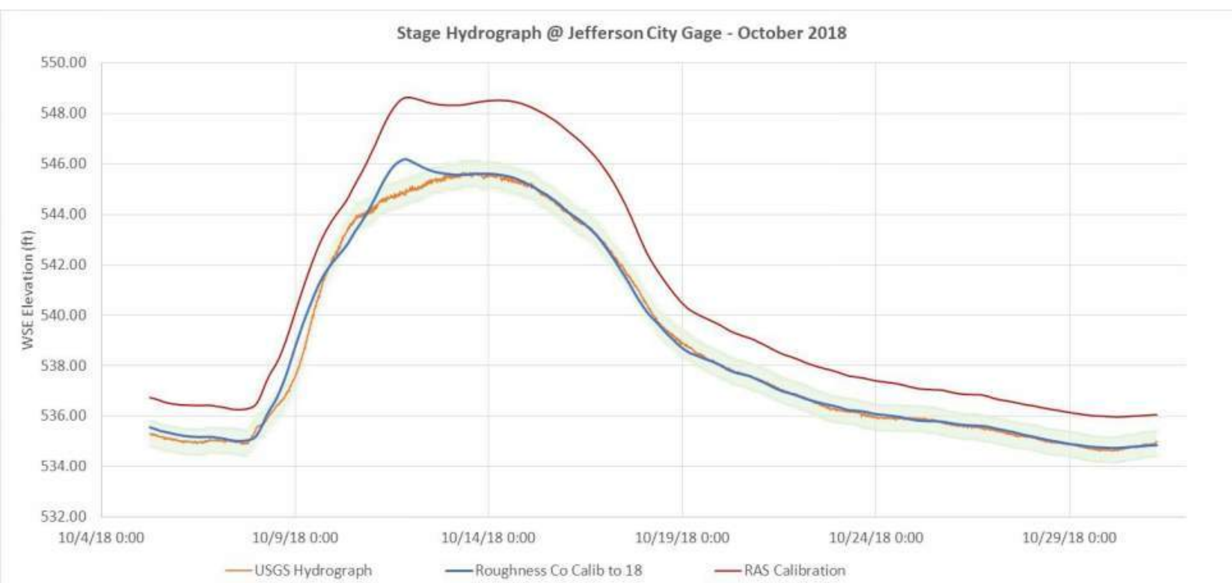

Figure 27. October 2018 Sensitivity Recalibration to Jefferson City Stage-Hydrograph

As seen in **Figure 28**, when the 2019 storm event is run with the 2018 in-bank factors, maximum water surface elevation results are on average 0.5-ft lower for the Jefferson City gage hydrograph. The peak maximum water surface elevation is within 0.2-ft.

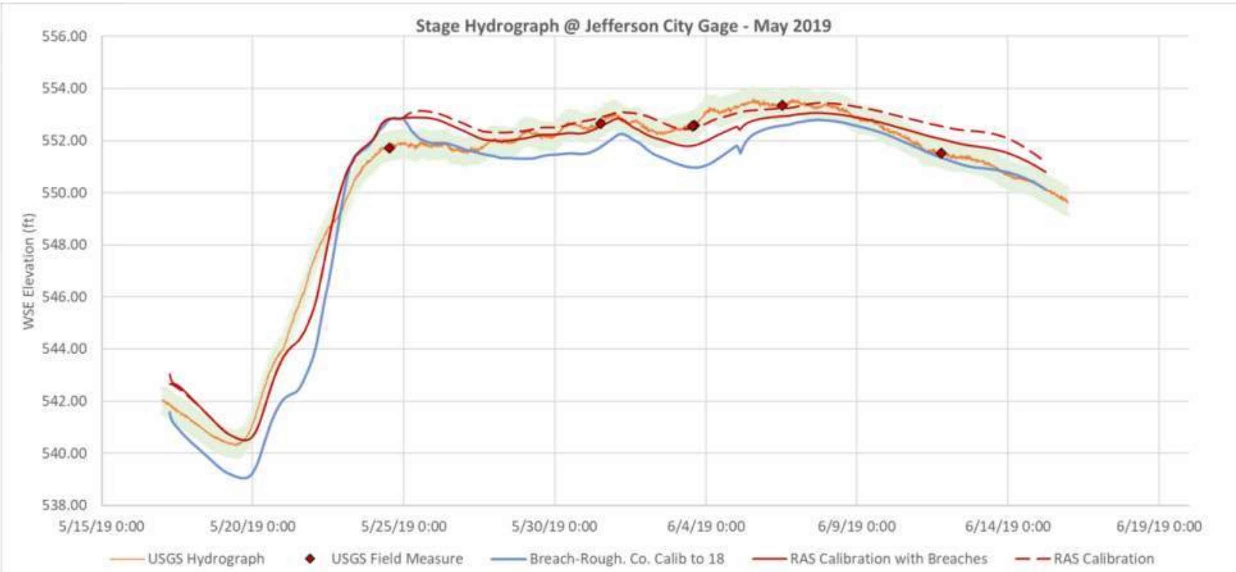

Figure 28. Jefferson City Stage-Hydrograph Comparing 2018 In-Bank Roughness Factors

The 2019 n-values are more conservative for the feasibility study concerned with overtopping events for estimating preliminary levee height alternatives and costs. The 2019 roughness coefficients are also a closer match to the current rating curve. The AEP events modeled for the Jefferson City study alternative analysis will be higher-flow events, closer to the 2019 event, rather than in-bank flows observed in 2018. There is also greater documentation and measurements for the 2019 event used for calibration. Therefore, calibration to 2019 was prioritized with the acceptance that stages for in-channel flow stages may be conservative as indicated by routing the 2018 event.

6 MODEL SENSITIVITY

6.1 Boundary Condition Sensitivity

Downstream boundary condition sensitivity was tested for both a normal depth friction slope of 0.0002 and a rating curve condition. Both options were stable. The rating curve method was chosen as the most representative of observed USGS data, shown in **Figure 29**.

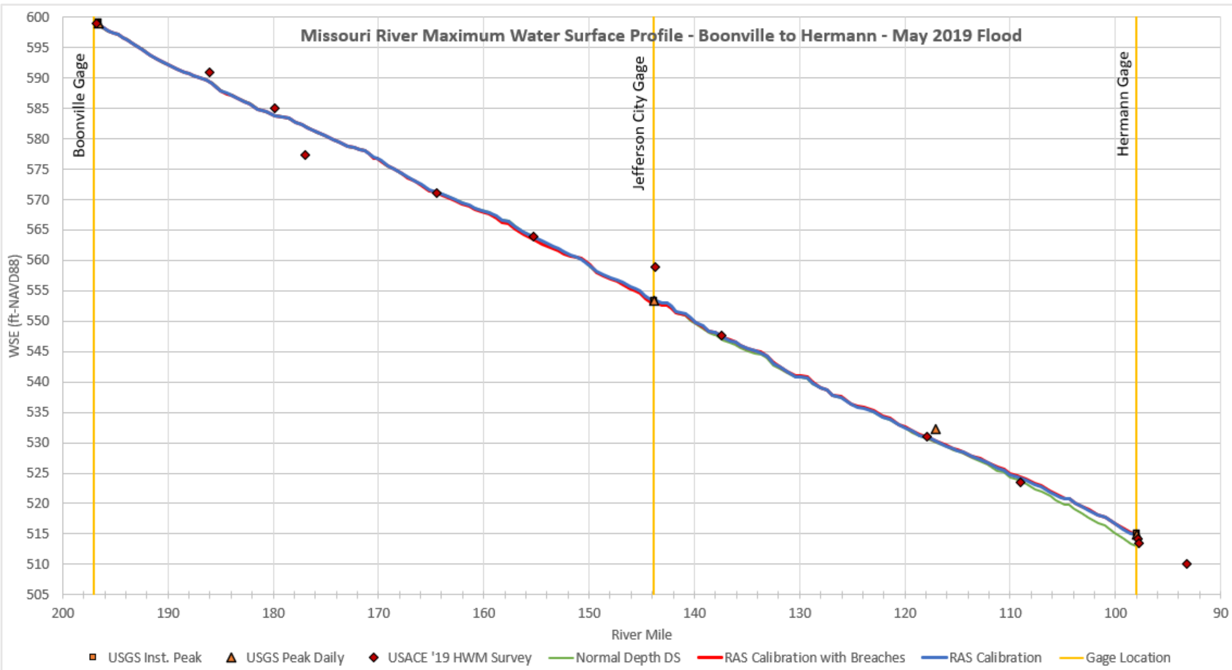

Figure 29. Downstream Boundary Sensitivity on Maximum WSE

6.2 Lateral Inflow Upstream of Study Area

Model results regarding stage and timing at Jefferson City did not appear sensitive to the "S_MissouriR_09" subbasin inflow location or separation into multiple upstream subbasins. **Figure 30** compares the stage hydrograph outputs at the Jefferson City gage for multiple lateral inflow locations. **Figure 31** compares the flow hydrograph outputs at the Jefferson City gage for multiple inflow locations. Sensitivity was tested by adding mainstream river flow between Boonville and Jefferson City at multiple lateral inflow locations. The original model inflow location was located at river mile 144.6. This location was not ideal, as it intercepted the 2D flow area developed for study. Sensitivity inflow locations were located at 144.6, 149.3, and 155.37. An additional sensitivity was conducted to split the total mainstem inflow with half entering at river mile 149.3 and half at 162.04. All sensitivity runs for lateral inflow were conducted without modeled levee breaches and compared to the "RAS Calibration without Breaches" run of the May 2019 event. The stage hydrograph and flow hydrographs at the Jefferson City gage showed no variability to the peak result values or general shape of the output hydrographs for the various lateral inflow configurations in comparison to the calibration event without breaches.

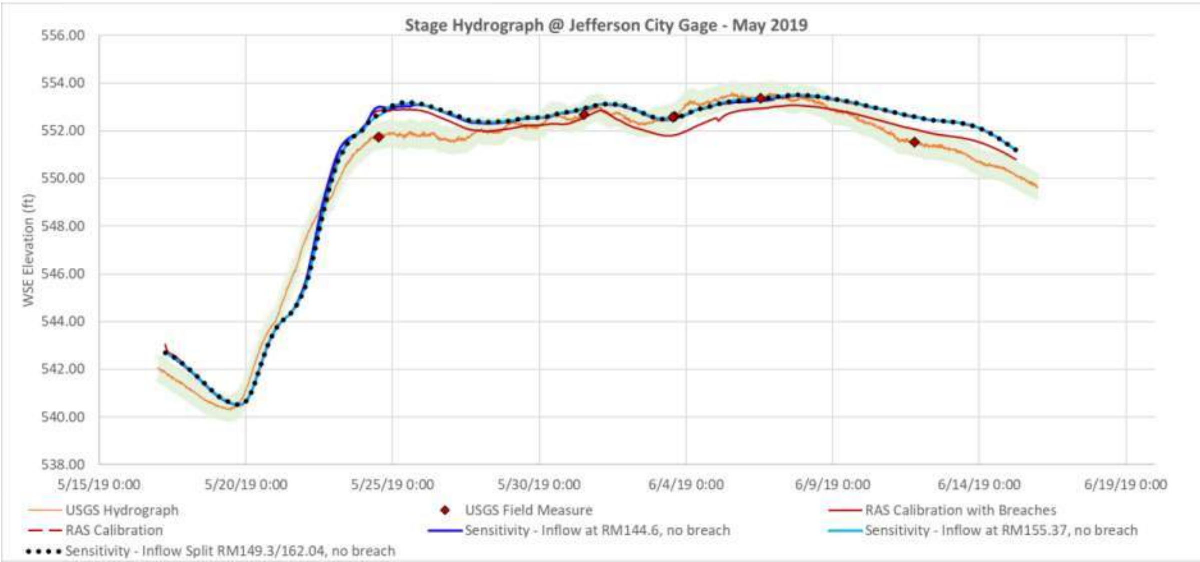

Figure 30. Stage Hydrograph at Jefferson City Gage - May 2019, Lateral Inflow Sensitivity Test

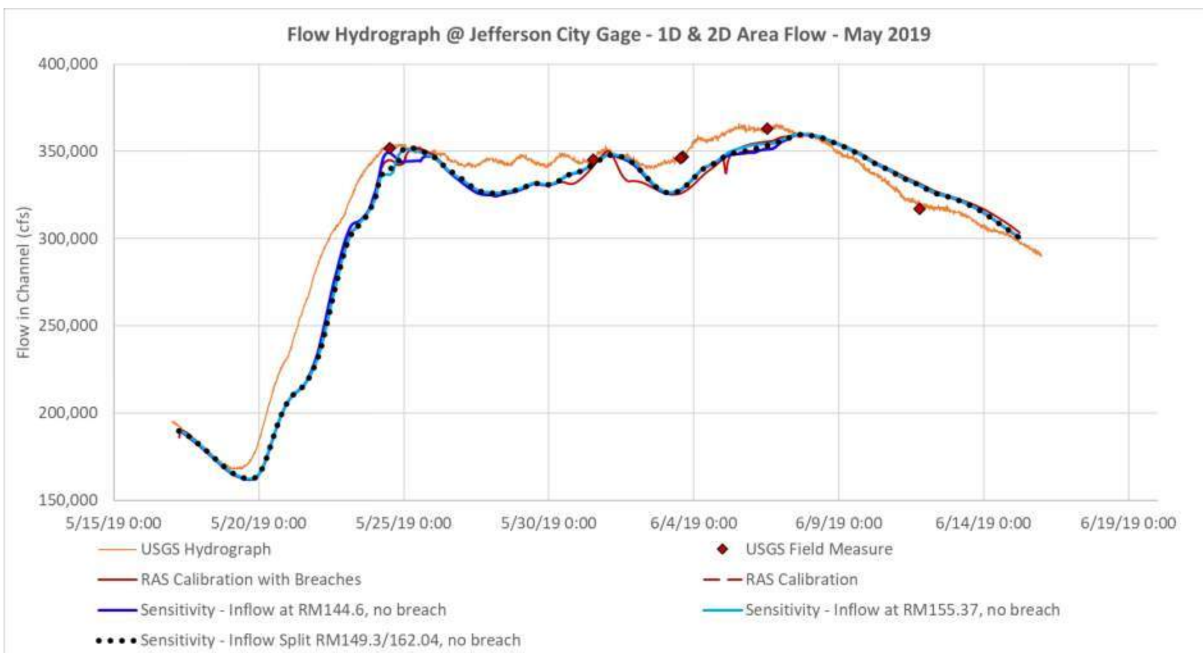

Figure 31. Flow Hydrograph at Jefferson City Gage - May 2019, Lateral Inflow Sensitivity Test

The mainstem inflow location upstream of Jefferson City was selected to be a single location at river mile 149.3, which situates the increase in inflow directly upstream of the study area, but outside of the 2D area footprint. This configuration routes the total target river flow at the Jefferson City gage through the complete study area with limited interaction by non-federal levees that could add inconsistency.

6.3 2D Area Manning's n-Value Modification

The Manning's n-values for the 2D flow area were adjusted to add additional friction to the 2D overbanks through the leveed area to test sensitivity. As a result, no change to surface water elevations was observed. Therefore, default Manning's n-values as detailed in **Table 6** were maintained.

6.4 Roughness Factors within the 1D Channel

Flow roughness factors were utilized to calibrate computed stages to those observed. For Boonville, Jefferson City, and Hermann, this was verified through comparison of observed stage and flow data, rating curve comparison, and levee overtopping times documented during the 2019 flood. Final roughness factors were applied by geometry reach and are summarized in **Figure 32**, and were set to generally correspond with base-flow, in-bank flow, and overbank flows.

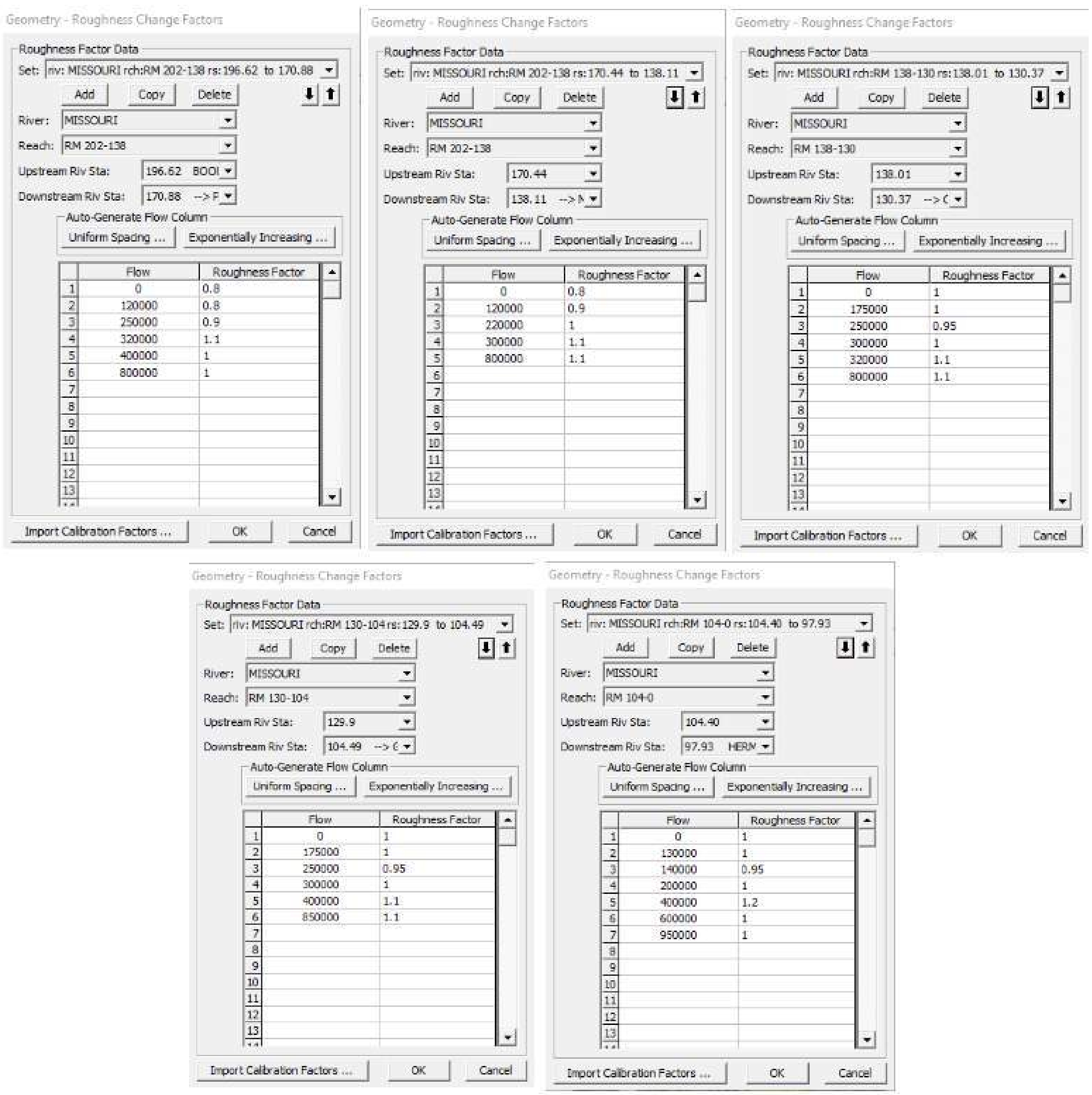

Figure 32. Flow Roughness Factors applied to the Missouri River Reach

6.5 Incorporation of Upstream Levee Breaches

Documented levee breaches from the May 2019 event were added to the model for final calibration of the 2019 event. For each lateral structure or 2D connection, the largest breach documented in the USACE 2019 Post Flood Report was utilized for the modeling. Breach documentation was reported over 24-hour time periods, so timing of the breach entered into the model is approximate. Model sensitivity to breach timing decreases as flood flows increase for this specific corridor of the Missouri River, so the timing of the breaches was not critical to overall calibration efforts for the timing of peak flow through Jefferson City, seen in **Figure 16**. **Figure 33** depicts all USACE reported levee breaches that occurred during the 2019 flood, with node sizes scaled to represent the magnitude of reported breach length. **Figure 34** provides a summary of approximate breach times for the lower Missouri River levee systems as documented in the USACE 2019 Post Flood Report.

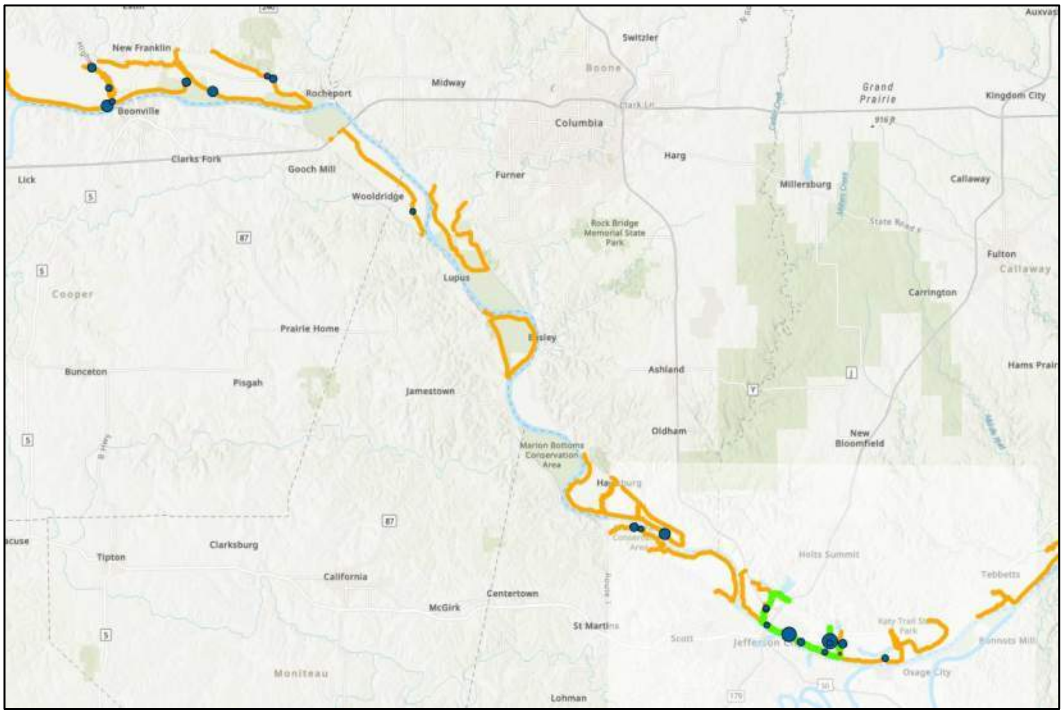

Figure 33. Levee Breach Locations Upstream of Capital View Levee

Note: Size identified by point size.

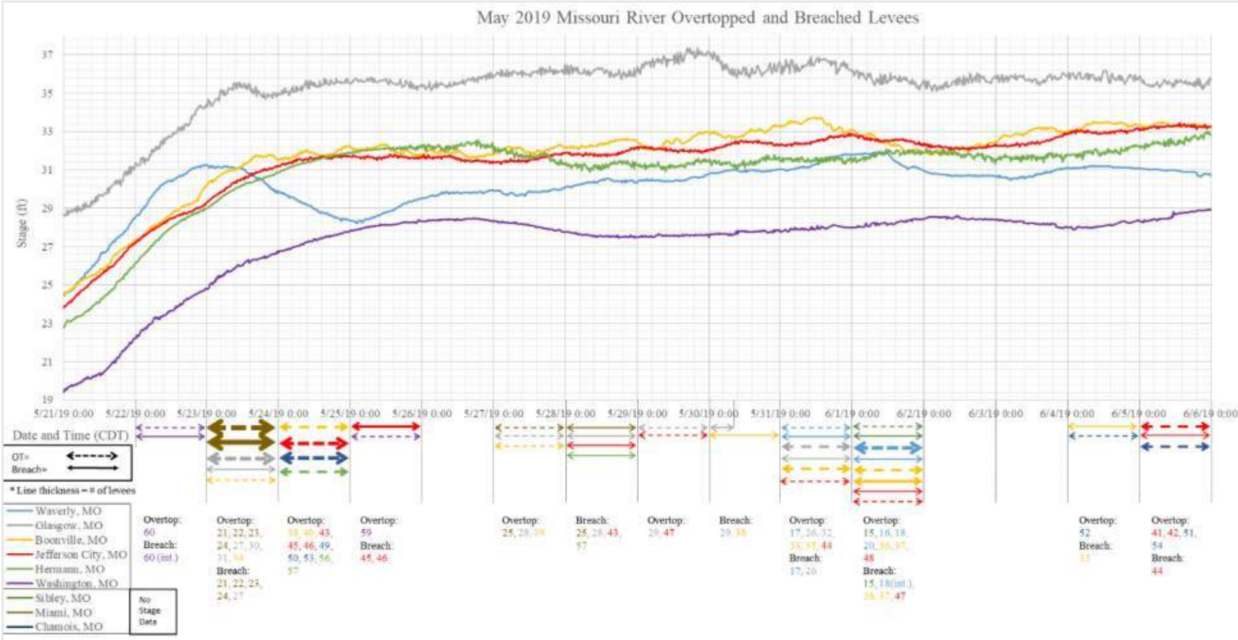

Figure 34. Overtopping Timing Estimates and Stage Hydrograph

Note: From 2019 Post Flood Report.

In addition to the USACE recorded breaches in P.L. 84-99 program levees, an additional breach was added to this model at the Renz North private levee tieback along Cedar Creek near U.S. Highway 63. This levee is known to breach first in the Jefferson City area during most flood events according to local stakeholders. This is also evidenced by aerial imagery.

Table 14 shows all breach assumptions incorporated into the HEC-RAS model. All breaches were modeled with 1:1 side slopes, overtopping failure mode, 12 hour breach formation, and were triggered at a specified time. Failure times and maximum widths were developed from data documented in the USACE 2019 Post Flood Report.

Table 14. Summary of Levee Breach Model Inputs

Lateral Structure / 2D Connection Name	Levee Name	Cen. Sta.	Max. Width	Min. Elev.	Date	Time (UTC)
how9b-frkl	LD No. 7 of Howard Co. No 4.	08+00	320	583	5/31/2019	19:00
howt-bfe	Howard Co. No. 4	162+50	450	580	5/31/2019	19:00
bfe-salt	Bonne Femme	180+00	250	575	5/31/2019	19:00
182.39	District 1 Cooper Co.	164+00	250	573	5/30/2019	0:00
154.63	Church Farm	73+50	275	548	5/28/2019	0:00
153.52	Hartsburg 3	83+00	575	551	6/05/2019	0:00
JeffCity_Renz2	Renz	13+50	50	544	5/25/2019	0:00
144.3	Capital View	83+00	925	544	5/25/2019	0:00
JeffCity_Cap2	Capital View	07+20	220	546	5/25/2019	6:00
JeffCity_Cap3	Capital View	37+50	1025	539	5/25/2019	6:00
138	Reveaux	25+00	150	540	6/01/2019	12:00
JeffCity_Rvx1	Reveaux	38+00	350	540	6/01/2019	18:00

Through Jefferson City, the condition incorporating observed breaches results in a lower maximum WSE at the Jefferson City gage, which was accounted for with the calibration of the model as shown in the stage hydrograph in **Figure 17**.

7 Model Intent and Use

Overall, the channel and overbanks are well calibrated, utilizing the May 2019 event, for a flood event equal to and greater than the 10% AEP, in which levees through the modeling corridor begin overtopping. Based on comparison with the October 2018 event, stages for flows contained within the levee systems prior to overtopping are likely conservative. The adopted calibration is well suited for a feasibility level analysis for a flood risk study where the assessed design flows range from 10% to 0.2% AEP.

7.1 Model Uncertainty

It is important to document model uncertainty for a feasibility-level model that may be further refined as the study progresses. Known contributors to model uncertainty at this time include data availability, model setup with an emphasis on model run-time at various flow regimes, and model level-of-detail in areas where future refinement will be conducted upon selection of a plan.

7.1.1 Calibration Limits

Limitations of this model due to data availability and/or intended model use include:

1. Areas outside the immediate Jefferson City study area as defined by the 2D flow area were developed from the reviewed and calibrated 1D RAS 2022 CWMS model. These data sources include 2013/2014 bathymetry and LiDAR. Although bathymetry and terrain data used to develop cross-sections and storage area volume curves are consistent with overall trends when compared to the preliminary 2021/2022 bathymetry for the Missouri River, the terrain and channel cross-sections may reflect localized differences. Levee data outside of the immediate study area was also adopted from the 2022 CWMS model and may not be the most representative of post-2019 flood conditions. This limitation is acceptable for the development of this model to aid in initial model creation timelines knowing the level of detail. This model is sufficient for planning level decisions.

2. Ungaged flow upstream of Jefferson City is largely unknown. The ungaged drainage between Boonville and Jefferson City inflows to the Missouri River from many small, unmodeled tributaries not utilized for CWMS modeling and not historically monitored. These ungaged tributaries are not a likely source of flooding, however they may influence the hydrograph shape at the Jefferson City gage.

7.1.2 Areas of Uncertainty

Areas of uncertainty that should be considered when utilizing the model for alternative comparison include:

1. Effects of Manning's n-values in 2D flow areas for areas that are actively conveying flow have not been calibrated because there has not been a recent flood event that both overtops the non-federal levees and continues to rise to actively convey flow above the top of levee elevations. Sensitivity to overbank Manning's n-values for the calibration event, which generally results in pooling behind the levee systems with minimal velocities, may not be representative of higher flow conditions where conveyance occurs behind the levees.

2. Local inflows of ungaged tributaries were not assessed for feasibility level analysis. The source of flooding through this river stretch is the Missouri River with 507,500 square miles of drainage area upstream, compared to the left bank tributaries at Jefferson City including Turkey Creek at RM 144.5 (drainage area of 11 sq. mi.), Niemans Creek at 139.84 (drainage area of 7 sq. mi.), and Rivaux Creek at 135.62 (drainage area of 21 sq. mi.). For this phase of feasibility, assumption of a Missouri River flood is valid. Upon selection of a selected plan, bounding tributary coincident flows will be assessed to refine the top of levee tiebacks.

3. Private levees outside of the 2D flow area are unknown and are not incorporated into the model. This model largely depends on levees included in the National Levee Database (NLD) upstream and downstream of Jefferson City. Any private levees not included in the NLD will not be reflected in the model.

4. Exact timing of levee overtopping and failure during the 2019 flood calibration event is largely unknown, with documentation of overtopping and failure rounded to the nearest 24-hour period. Overtopping times were adjusted in the model within 24-hours of observed to calibrate water surface elevations and timing of failure of upstream and downstream levees. Modeled breach formation time, side slopes, and weir coefficients were assumed to be consistent across all breach locations, as no field data is available for these parameters.

7.2 Model Limitation

Calibration to the May 2019 flood limits the model application to a flood event ranging from 700 to 1,050 kcfs in accordance with EM 1110-2-1415, which recommends a high-flow limit of 2 to 3 times the calibrated flow.

This model is not intended to create detailed inundation mapping at the right bank of the Missouri River through Jefferson City. Right bank comparisons between model runs should be compared at the Missouri River profile at the mouth of right bank tributaries or with the modeled storage areas.

This model was calibrated for a Missouri River high-flow event. Detailed analysis of smaller tributaries was not conducted, due to order of magnitude of the streams and focus on Missouri River backwater effects.

The Jefferson City model level-of-detail is intended to represent high-flow conditions as a basis of comparison for levee alternatives. Interior drainage including culverts and tributary bridges within the 2D Area are not modeled. This results in a ponding effect behind existing low-lying levees where internal drainage structures may not be effectively reflected. Ponding behind existing levees may be reflected as a reduction of volume in the receding hydrograph. For the efforts of this study, the peak flows and resulting peak stages are accurately represented.

Lower Missouri Jefferson City L-142 FRM Study

Appendix A1
Part 3: Alternatives Hydraulic Analysis

Capital View Drainage District, looking southeast toward inundated airport May 28, 2019

U.S. Army Corps of Engineers
Kansas City District

Final Report, July 2026

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1 INTRODUCTION

Development of measures and alternatives in accordance with USACE Planning Policies and Procedures is discussed in the main-body feasibility report. This appendix documents hydraulic model development, assessment, considerations, and results relating to measures and alternatives considered for the Jefferson City Feasibility Study. All measures and alternatives were compared to the Future Without Project (FWOP) hydraulic model to determine impacts and effectiveness in relation to a singular basis of comparison. For additional context regarding hydrology development refer to Appendix A1, Part 1. For additional context regarding development and constraints of the base hydraulic model refer to Appendix A1, Part 2.

2 FUTURE WITHOUT PROJECT CONDITION (FWOP)

The Future without Project Condition alternative is largely unchanged from the existing condition. In the FWOP, continued overtopping of Capital View levee is anticipated at 7.5-ft above flood stage (17% AEP) with ongoing reconstruction efforts to maintain the existing level of protection. Inundation of critical infrastructure in events greater than 17% AEP, or with the breach of Capital View levee will continue to impact critical services to the city of Jefferson City.

2.1 Land Use

Since the 1993 flood event in Jefferson City, the city has pursued additional restrictions for development within floodplain areas including the buyout of property and deed restrictions within flood affected areas. The left bank of the Missouri River is specifically outlined in the City's future land use map, **Figure 1**, as a "River Transition Overlay Annexation" which is defined as "areas that need additional scrutiny in regard to development in and around the floodplain, flood prone areas, or environmentally sensitive features" (Comprehensive Plan, Activate Jefferson City 2040). Therefore, modification to the Manning's n values within the 2D flow area of the hydraulic model for the future condition is not necessary.

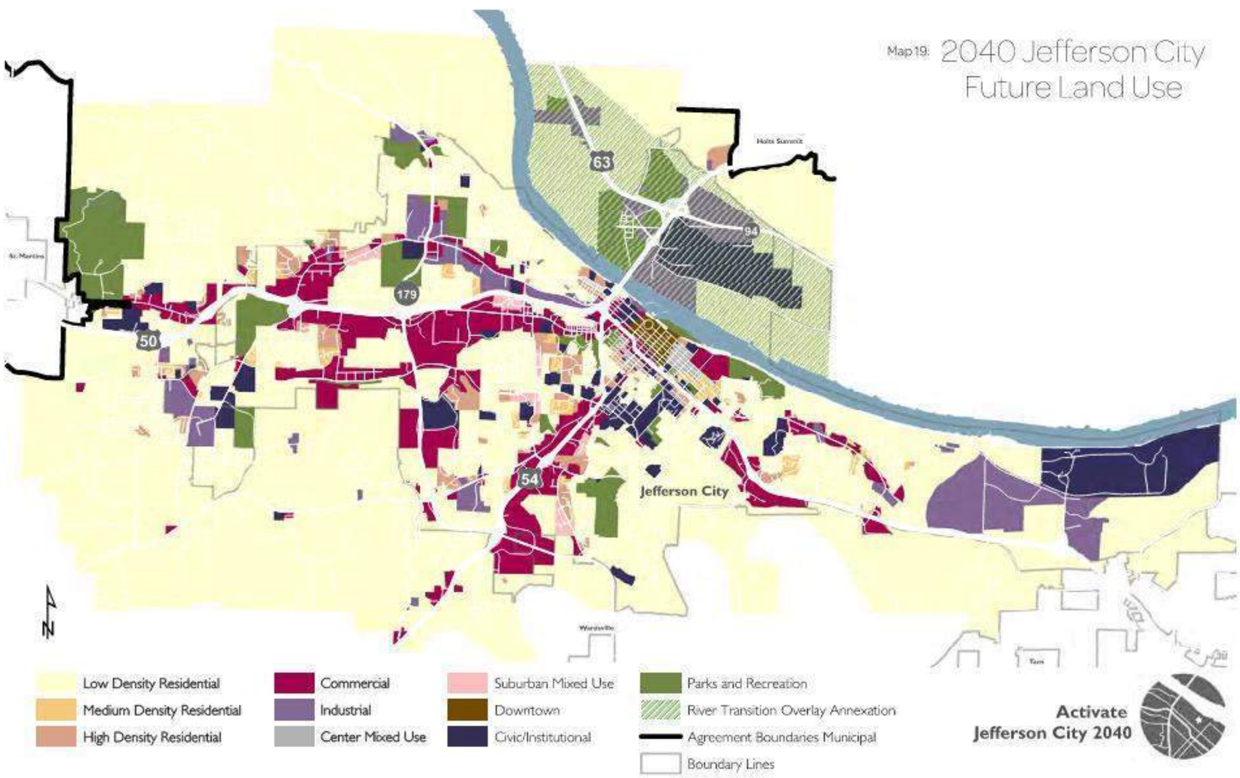

Figure 1. Jefferson City Future Land Use Map

Source: Activate Jefferson City 2040 Plan

The city and surrounding counties are generally rural farmland and have active stormwater management requirements that require stormwater quality and stormwater discharge control for development. Therefore, impactful changes to the unregulated/ungaged subbasin runoff to the

Missouri River between the Boonville and Jefferson City Missouri River gages are not anticipated.

2.2 Flow Frequency

No storage or significant regulation changes to the Missouri River are anticipated upstream of the study are within the FWOP timeframe. Flow Frequency updates to Lower Missouri River Basin saw minimal effects to the gages bounding the Jefferson City study area (Boonville and Hermann) over the 20-year update span. Therefore, the impacts of climate change are not reasonably expected to impact flow frequency outside of the range of uncertainty for the most current flow frequencies. Flows for the FWOP condition will duplicate the most recent existing conditions hydrology. This approach is confirmed with the qualitative climate change assessment conducted included as Appendix A2.

2.3 Levee Performance

The Capital View levee will remain in place at the current level of performance over the FWOP timeframe. All other levees within the study reach will be maintained to their existing level of performance.

2.4 Stage – Discharge Relationships

Stage-discharge relationships at the Jefferson City gage are summarized in **Figure 2** for the FWOP conditions and are compared to recent USGS rating curves and peak annual flows since 2015. The 2021 USGS observed rating curve was utilized for calibration of this model as discussed in Appendix A1 Part 2.

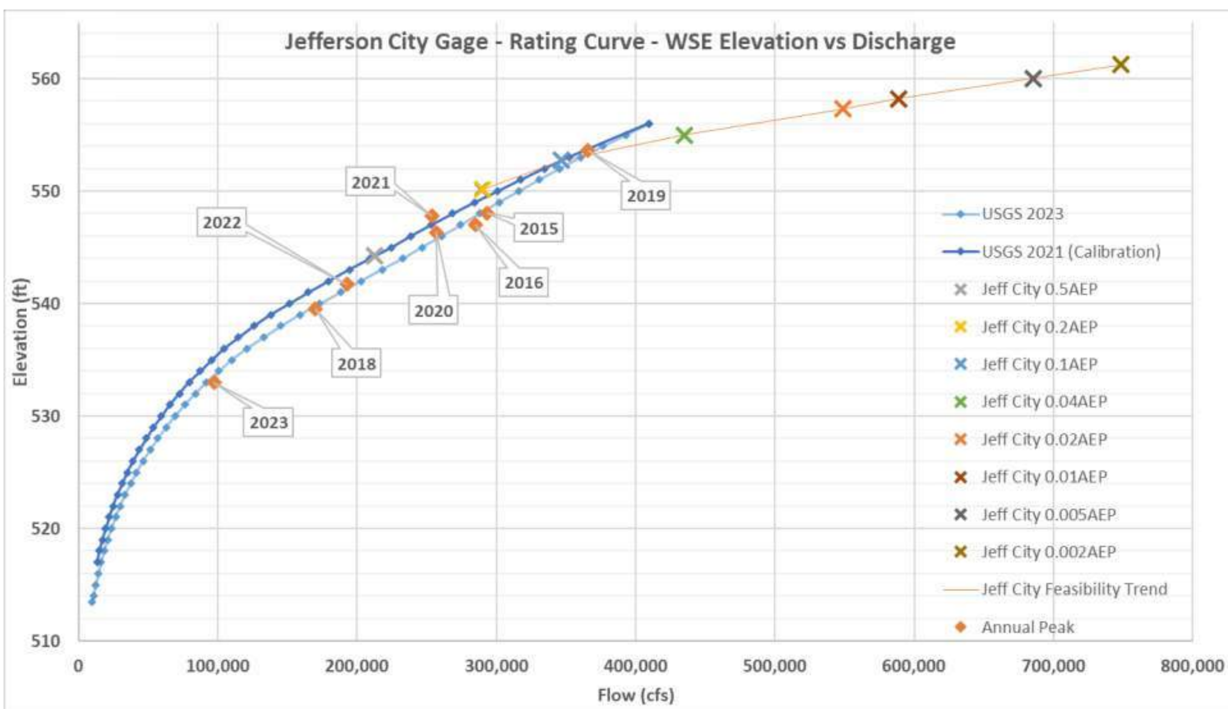

Figure 2. Stage - Discharge Relationship at Jefferson City Gage

USGS rating curves are developed from observed data and are therefore limited for less common storm frequencies. Updates occur to the rating curves over time as the river system experiences physical changes and with the accumulation of stage and flow data. **Figure 2** compares the 2021 calibration rating curve to the current 2023 rating curve for context only. Calibration to the 2021 curve may result in slightly higher water surface elevations for in channel flow than current conditions. This is acceptable for this feasibility study as it may result in more conservative design elevations for flood mitigation measures. Lower volume flows are the most revised between the rating curve versions and relate to in-channel flow, which is not the focus of this study. The higher flow values converge for higher volume flows, so recalibration was not deemed necessary. The USACE is developing separate stage-discharge relationships coordinating with the 2023 MRFFS results as a separate study, however results are not available for comparison with this model at the time of publishing.

3 EVALUATION PROCEDURES

To address the project problem of reoccurring economic and infrastructure damages to the north bank of the Missouri River in Jefferson City due to repetitive flooding, three planning objectives for the Jefferson City Feasibility Study were identified during the planning process.

1) Reduce the risk of flood damage and/or improve resilience for property, businesses, agriculture, and infrastructure (including critical facilities) near Jefferson City for the 50-year period of analysis beginning in 2035 (base year).

2) Reduce the risk of flooding on human life and safety for the 50-year period of analysis beginning in 2035 (base year).

3) Consider the project as part of the overall Lower Missouri River Flood Risk and Resiliency Study (System Plan) planning objectives for system conveyance and flood risk planning in the future.

Study constraints and considerations include:

- Avoid increasing bird/wildlife air strike hazard in accordance with USACE Memorandum of Agreement with the Federal Aviation Administration.
- Avoid impacts to the authorized purposes of the Bank Stabilization Navigation Project (BSNP).
- Avoid, minimize, or mitigate impacts to cultural sites in the project area.
- Avoid, minimize, or mitigate impacts to wetlands in the project area.
- Status of Capital View levee in P.L. 84-99 program (ER 500-1-1).
- Cannot increase flood profiles upstream/downstream or induce significant damages on opposite bank of project site unless mitigation as appropriate is included to offset.

4 PRELIMINARY FORMULATION OF MEASURES

A measure is a feature or an activity that can be implemented at a specific geographic site to address one or more planning objectives (ER 1105-2-100). These are the building blocks of alternative plans, which is a set of one or more management measures functioning together to address planning objectives. Measures were developed with insight from the full PDT, including engineers, stakeholders, the vertical team, and the study sponsor to address study objectives within the project area of the left-bank of the Missouri River through Jefferson City, as shown in **Figure 3**. All proposed measures were then preliminarily studied and screened or refined as warranted based on the effectiveness of specific measures to meet the planning objectives as discussed in the main body of the report, Section 3.3.

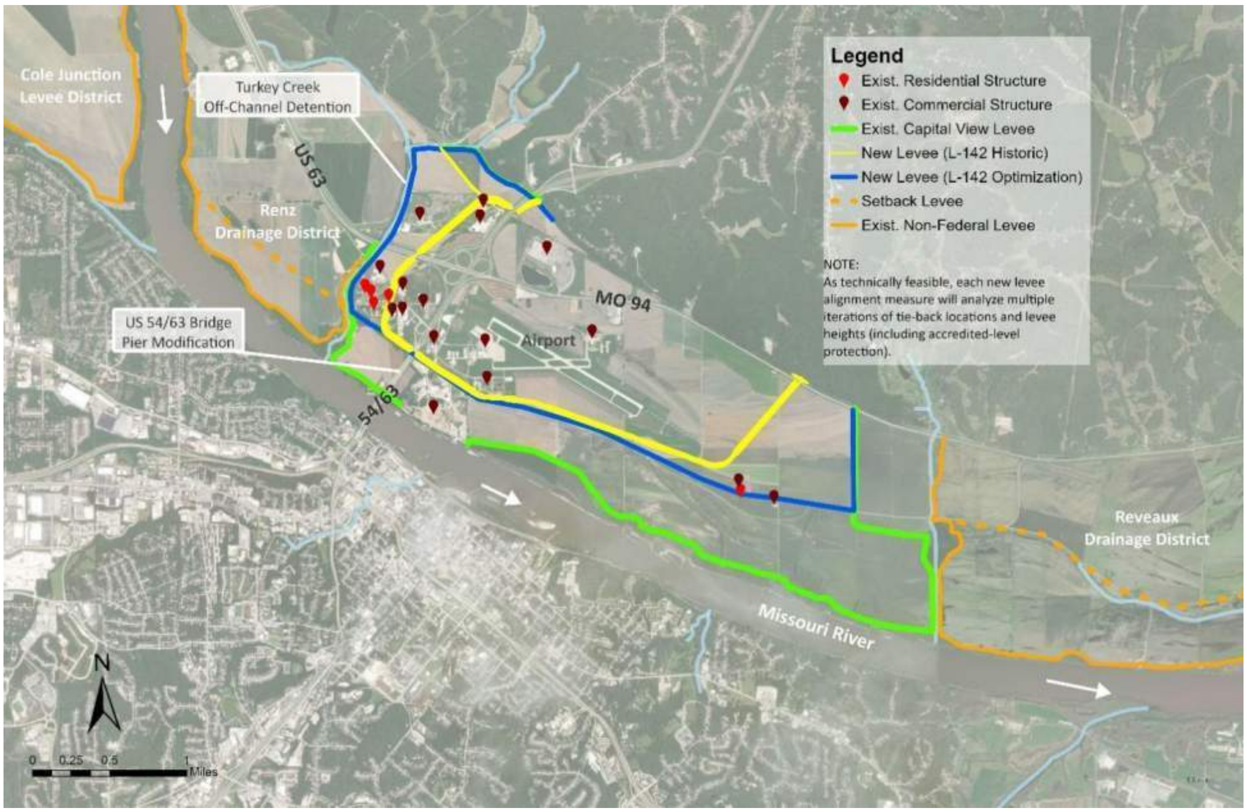

Figure 3. Identification of Preliminary Measures for the Jefferson City Flood Risk Management Study

Identified measures were classified in the general categories of no action (measure theme 1), nonstructural measures (measure theme 2), natural and nature-based solutions (measure theme 3), channel modification (measure theme 4), infrastructure improvement (measure theme 5), resilience improvements (measure theme 6), existing levee modifications (measure theme 7), or new structure measures (measure theme 8).

4.1 Nonstructural Measures

Utilization of nonstructural measures – defined as actions that do not revise the flood regime itself, but adapts floodplain features to reduce damages incurred by flooding - to minimize flood impacts through a combination of measures such as floodproofing, structure modifications, relocation, flood warning systems, and/or increased flood preparedness.

Nonstructural measures screened during the preliminary analysis of measures include a flood warning system. This measure was screened due to ample warning in the development of a flood at this location on the Missouri River, stakeholders that are very informed and connected to existing resources for Missouri River stage predictions, and a published City flood action plan that is frequently updated. Additional warning devices would likely not meaningfully contribute to existing practices in place at this location.

Nonstructural measures carried forward to the development of alternatives include wet/dry floodproofing existing structures, buyouts, relocation of existing structures, and elevation of existing structures in place. Determination of nonstructural measure applicability was conducted by assessing each structure's position in the floodplain (as it relates to flooding depth) and considering the occupancy type. As detailed in Appendix E, for this study, nonstructural measures applied to nonresidential structures include elevation or wet/dry floodproofing in place. The use of dry floodproofing throughout the study area is limited due to expected flood depths. Nonstructural measures applied to residential structures, including outbuildings on the same parcel, include elevation in-place, buyout, or relocation. Hydraulic analysis of these nonstructural measures includes determining flood depths and flow velocity at structures to determine constructability and effectiveness.

4.2 Natural and Nature Based Solutions

Utilization of natural approach measures to minimize flood impacts and increase positive environmental impact through a combination of measures such as wetland/ecosystem restoration, levee setbacks, and/or acquisition to restore floodplain.

Nature based measures screened during the preliminary analysis include wetlands, off-channel detention basins, and removal of the Church Farm levee (also referred to as Prison Farm levee in the USACE National Levee Database). Wetlands and off-channel detention basins are not hydraulically effective for this site due to the origination of the flood upstream of the study site on the mainstem Missouri River and bluff-to-bluff floodplain activation through this stretch of the Lower Missouri River. The Federal Aviation Administration also restricts the use of wetlands and/or ponded water features in the proximity of the airport located behind the existing Capital View levee. A discussion of the preliminary analysis conducted for the Church Farm levee removal measure is included in Section 7.11.

Nature based measures carried forward to the development of alternatives include levee setbacks and buyouts, which would increase riverward capacity and connectivity of the floodplain in combination with other measures. Hydraulic analysis of these natural and nature-based features includes determining the impact, if any, of the measures to depths, velocities, duration of inundation, and frequency of inundation to the project area and adjacent properties. Alternative refinement will also include the optimization of any levee setbacks to minimize project footprint while maintaining project benefits.

4.3 Channel Modifications

Channel modification measures incorporate direct actions to the river channel to accommodate or modify stage and flows and may include in-stream detention (reservoir), and/or use of dredged material for deepening and widening of channel.

All channel modification measures were screened during preliminary analysis of measures including a Missouri River reservoir, widening the Missouri River, deepening the Missouri River, and dredging the Missouri River for construction material and additional river capacity. The measures were screened on a basis of high level of environmental impacts, the measures not meeting study objectives, hydraulic ineffectiveness for large-scale flood events, and existing permitted dredging operations in the area.

4.4 Infrastructure Improvements

Infrastructure improvement measures directly impact levee performance or infrastructure that interacts with floodwaters through a combination of measures, such as bridge modifications, reverse loading, and/or raise utility infrastructure.

Infrastructure improvement measures screened during the preliminary analysis of measures included modifications to the U.S. Highway 54 / 63 bridge pier structure, raising the elevation of existing critical infrastructure, incorporation of adjacent private levees (North Renz and/or the Sod Farm levee) into the P.L. 84-99 program, and a closure structure for Renz Farm Road. The U.S. Highway 54 / 63 bridge modification measure was screened due to the lack of hydraulic influence resulting from the existing bridge pier structure, which was not evident prior to development of the existing conditions model. All other measures were screened due to their ineffectiveness of reducing the risk of flooding to meet study objectives.

4.5 Resilience Improvements

Improved resilience measures include modifications to levee structures and/or facilities impacted by floodwaters to reduce efforts to restore to pre-flood conditions after a flood recedes. This can be accomplished through a combination of measures such as reinforcement, designated overtopping, and seepage and stability berms.

Resilience measures screened during the preliminary analysis of measures include a designated breach of Capital View levee and addition of drainage structures. These items were screened due to the high inundation frequency of the existing levee, reducing the effectiveness of the features to only the most frequent/least damaging storm events. No stakeholder concerns regarding internal drainage of the existing levee system were documented through stakeholder conversations, public meetings, or sponsor meetings so it is assumed drainage structures existing today are not a leading concern for the existing leveed area.

Improved resilience measures carried forward to the development of alternatives include armoring of the existing levee, designated overtopping, reverse loading designs, the addition of seepage and stability berms, consolidation of existing drainage structures for improved efficiency, and the addition of pumping facilities. Hydraulic analysis of resilience measures includes ensuring designs are resilient to a full range of velocities and inundation durations. In addition, where inundation duration can be reduced, those opportunities are explored.

4.6 Existing Levee Modifications

Existing levee modification measures include changes to an existing levee structure to reduce flood impact. This can be accomplished through a combination of measures such as levee setbacks, raising the existing levee, or removing the levee.

No existing levee measures were screened during the preliminary analysis of measures due to local interest in maintaining the existing footprint and understanding of induced impacts of levee modifications. Preliminary screening provided insight that raises to the existing Capital View levee would likely cause induced impacts to adjacent properties and tributaries in more frequent flooding events, such as the 10% AEP (1/10-YR) storm event.

Existing levee measures carried forward to the development of alternatives include levee setbacks, raising the existing levee, or removing the existing levee. Hydraulic analysis of these existing levee measures includes assessment of effectiveness of proposed overtopping elevations on meeting the planning objectives, induced damages to adjacent stakeholders, performing iterations of alternatives to meet project goals and include stakeholder input, and understanding impacts of the project to the river system hydraulics as a whole.

4.7 New Structural Measures

New structural measures include construction of a new federal levee system, independent of existing levee systems. New floodwall measures were screened during the preliminary analysis of measures due to the increased cost of this structural alternative in comparison to earthen levee alternatives. In areas where an earthen levee is a constructable measure, an engineered floodwall would not be an economically justifiable option for construction.

New earthen levee measures were carried forward to the development of alternatives. Hydraulic analysis of new levee measures includes assessment of effectiveness of the proposed overtopping elevations and levee alignment on meeting the planning objectives, induced damages to adjacent stakeholders, and understanding impacts of the project to the river system hydraulics as a whole. Iterations of alternatives were developed to meet project goals and include stakeholder input.

5 IDENTIFICATION OF STUDY ALTERNATIVES

Flood risk management measures were developed to reduce susceptibility to flood damage within the project area. Measures resulting in a reduction in discharge are not viable at this location of the lower Missouri River, as the source of flooding is the mainstem river. Measures targeting a reduction of stage were generally not impactful due to the bluff-to-bluff nature of the Missouri River floodplain. The Missouri River corridor through Jefferson City is bounded by bluffs on both overbanks and the floodplain is fully inundated at a 10% AEP event, prior to reaching a 1% AEP flow event.

A summary of combined alternatives considered for the Jefferson City L-142 Missouri River Flood Risk Management Study is included as **Table 1**.

All alternatives are expressed in river stages based on the existing Jefferson City river gage at the U.S. Highway 54 / 63, as this is the standard language of local stakeholders. The gage datum is 520.2 feet NAVD88. The existing Capital View levee overtopping stage is 30.5 foot.

Table 1. Summary of Jefferson City Study Modeled Alternatives

Combined Alternative	Description
CA0	No Action
CA1	Existing Capital View Alignment, 34-ft Stage
CA2	Turkey Creek & Riverside Setback, 34-ft Stage, Capital View Removed
CA3	Existing Capital View Alignment with Resiliency
CA4	Optimization Alignment, 37-ft Stage, Capital View Remains
CA5	Optimization Alignment, 37-ft Stage, Capital View Removed
CA6	Historic L-142 Alignment, 45-ft Stage, Capital View Remains
CA7	Historic L-142 Alignment, 45-ft Stage, Capital View Removed
CA8	Highway Alignment, 37-ft Stage, Capital View Remains
CA9	Highway Alignment, 37-ft Stage, Capital View Removed
CA10	Limited Resiliency Measures Outside of Capital View Footprint
CA11	Nonstructural
CA12	Remove Church Farm Levee
CA13	Hybrid Alignment, 41-ft Stage, Capital View Removed
CA13	Hybrid Alignment, 41-ft Stage, Capital View Remains
CA13	Hybrid Alignment, 41-ft Stage, Partial Capital View Removal

Figure 4 summarizes the modeled alternatives in relation to the annual exceedance probability (AEP) of the proposed plan at the project site, measured at the Jefferson City gage. The array of alternatives includes a range of targeted overtopping AEPs with the lowest overtopping stage at the existing Capital View levee height and the maximum overtopping stage at the elevation suggested by the previous L-142 studied in the early 2000s, dubbed “Historic L-142”. USACE does not have a standard levee height or AEP design standard dictated by guidance. CA6 and CA7 would meet the freeboard requirements for the 1% AEP storm required by a FEMA accredited levee system. Overall, a FEMA accredited levee system was not a goal for this project.

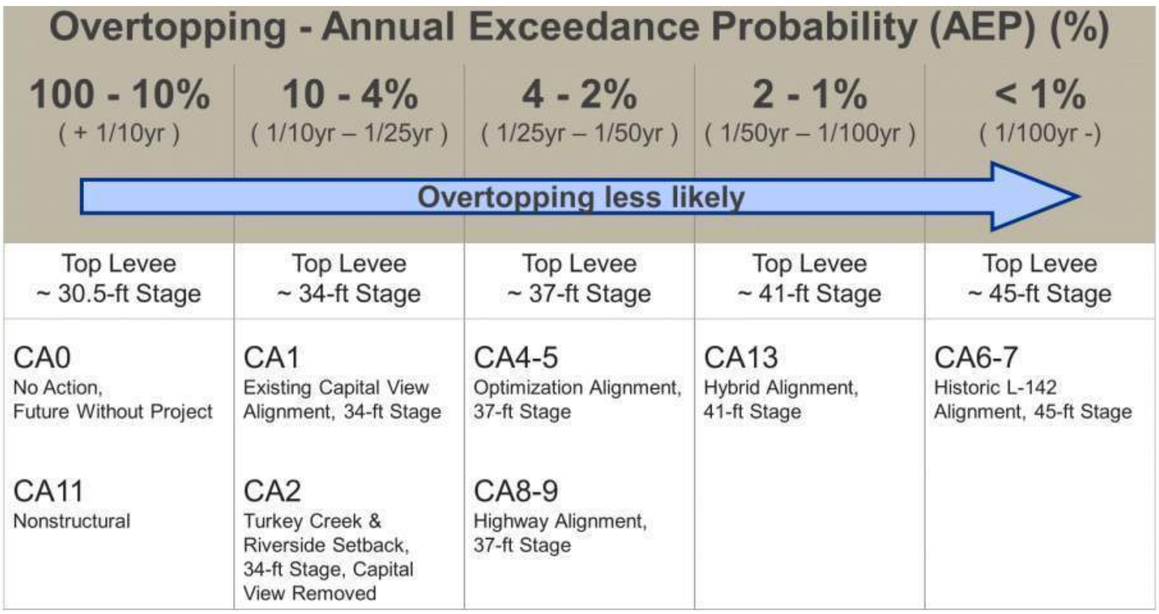

Figure 4. Summary of Alternatives in Comparison with Overtopping AEP

6 Hydraulic Modeling of Alternatives

Hydraulic modeling of proposed alternatives was conducted for those that would influence hydraulics throughout the project area. Hydrology and model geometry outside of the study area remained the same as the Future Without Project Condition (FWOP). Land use within the study area was assumed to remain the same as the FWOP for all alternatives. Model outputs showing Missouri River water surface profiles for the eight frequency profiles are attached to this report as Attachment B.

6.1 Existing Capital View Levee

In scenarios where the existing Capital View levee is to remain, the FWOP model was utilized to define the terrain, lateral structures, and 2D connection structures at the existing Capital View levee limits. The terrain file "LoMO_1m_NWK_JeffCity" reflects this condition.

In scenarios where the existing Capital View levee was to be removed, it was assumed to be completely removed. These edits were completed in the terrain file by modifying the levee crest alignment to follow the bank elevations, ensuring the cross sections were graded to drain. The terrain file "LoMO_1m_NWK_JeffCity_RemovedCV" reflects this condition. The lateral structure used to define the Capital View levee alignment was then edited so the top elevations matched the modified terrain. 2D equations using velocity were used as the overflow computation method. Following the same approach on the tiebacks, 2D connections were edited to also utilize the lowered terrain elevations as the structure top elevation. In cases where the geometry would have resulted in cells smaller than the average study area cell size, such as the removal of the Capital View levee and a very slight setback of a proposed levee, 2D connections were omitted completely.

6.2 Proposed Levee Modeling

Proposed levees were modeled utilizing 2D Connections when fully encompassed within the 2D Flow Area. For levee alternatives where the existing Capital View alignment was utilized, a combination of lateral structures and 2D Connections were utilized to create the 2D flow area geometry restrictions. For each condition, the 2D flow area grid was minimally adapted to maintain flowlines and enforce the proposed levee crest within the 2D area. For preliminary analysis and alternative comparison, proposed levees are assumed to act as a vertical wall within the model, which allows height iterations to optimize performance as alternatives were being developed. This is completed by setting 2D Connection top elevations to the desired levee height without modification to the underlying terrain. Levee top elevations were assumed to be straight-graded at 0.02% downstream, generally the water surface elevation slope through Jefferson City, unless variability in the Missouri River profile dictated intermediate elevation inflection points to provide a targeted AEP overtopping frequency. New levees greater in height than the existing Capital View levee utilize a weir equation overflow computation method with a broad-crest weir coefficient of 2.0 to reflect their higher positioning within the floodplain. Levee elevations were set to optimize the riverward levee profile to the desired estimated overtopping frequency. Tributary tieback elevations were preliminarily set to the energy grade line at the given river station, and only refined in elevation to achieve the desired overtopping frequency for the given alternative.

6.3 Levee Breach Assumptions

This stretch of the Lower Missouri River model is not sensitive to levee breaches, as the leveed areas are relatively small and the floodplain is more restricted than other Missouri River locations. All modeled levee units upstream and downstream levee units are assumed to breach in conditions of over 1-foot of overtopping at existing low points or documented reoccurring breach locations. For consistency in modeling for the wide range overtopping annual exceedance probabilities considered as alternatives, proposed levees were not assigned a breaching parameter for initial alternative comparison. For alternatives where the existing Capital View levee was maintained, the existing levee independent of the federal levee alignment, was assumed to breach at 1-foot of overtopping as modeled in the FWOP condition.

6.4 Summary of Model Organization

Geometries for each combined alternative were developed, as listed in **Table 2**. All geometries were run with unsteady flow analysis for all eight flow frequency conditions analyzed. Unsteady flow data are consistent with existing conditions development as detailed in Appendix A1, Part 1. Plan names are denoted as “Alternative#_Description_0.###AEP.”

Table 2. HEC-RAS Geometries Summary

Geometry	Name	Notes
.g09	Exist	Existing Conditions = FWOP Condition, CA0 No Action
.g16	01_CVAlign34ft	CA1 Existing Capital View Alignment, 34-ft Stage
.g26	02_CVAlign34ft_Setback	CA2 Turkey Creek & Riverside Setback, 34-ft Stage, Capital View Removed
.g19	04_OptAlign37ft_ExistCV	CA4 Optimization Alignment, 37-ft Stage, Capital View Remains
.g17	05_OptAlign37ft_NoCV	CA5 Optimization Alignment, 37-ft Stage, Capital View Removed
.g25	06_HistoricAlign_ExistCV	CA6 Historic L-142 Alignment, 45-ft Stage, Capital View Remains
.g27	07_HistoricAlign_NoCV	CA7 Historic L-142 Alignment, 45-ft Stage, Capital View Removed
.g21	08_HwyAlign37ft_ExistCV	CA8 Highway Alignment, 37-ft Stage, Capital View Remains
.g13	09_HwyAlign37ft_NoCV	CA 9 Highway Alignment, 37-ft Stage, Capital View Removed
.g03	12_ChurchFarmRemove	CA12 Remove Church Farm Levee
.g32	13_HybridAlign100YR_NoCV	CA13 Hybrid Alignment, 41-ft Stage, Capital View Removed
.g01	13A_HybridAlign100YR_ExistCV	CA13 Hybrid Alignment, 41-ft Stage, Capital View Remains
.g05	13C_HybridAlign100YR_PartialCV	CA13 Hybrid Alignment, 41-ft Stage, Partial Capital View Removal

7 COMBINED ALTERNATIVES EVALUATION

Developed alternatives are compared to the Future Without Project Condition (FWOP) model developed for this study. The FWOP model was developed and calibrated utilizing updated software, hydrologic frequency data, and the best physical information available at the time of the study. Therefore, model outputs are not directly comparable to existing models and will differ to some degree from previous reports and studies including the FEMA base-flood and existing stage-discharge relationships established on the river. The basis of comparison for all alternative impacts evaluated with this study is the differential from the FWOP model developed herein.

Multiple levee iterations of structural levee alternatives were considered varying both levee height and levee setback alignments. Iterations of both these factors were considered early in identifying alternatives to determine the most economic levee overtopping frequency and reduced flood area behind the proposed levee. Iterations began along the existing Capital View levee footprint as requested by public input, then iterated with setbacks to reduce induced impacts observed with levee raises within the floodplain.

7.1 Hydraulic Engineering Performance

This section summarizes the hydraulic performance of each plan alternative. Hydraulic performance factors are classified into local, community, and river-system considerations. Hydraulic engineering considerations were developed in reference to planning objectives, in addition to public questions and interests expressed during public outreach.

Local considerations include items of interest for stakeholders within the existing footprint regarding each formulated alternative, such as flood-fighting considerations, management of flood risk to stakeholders within the leveed area, and increased predictability in levee performance. Community considerations include changes in access to the roadway networks and access to critical facilities. River-system considerations include floodplain / floodway impacts, adjacent levee impacts, consideration of the Lower Missouri System Plan objectives, difference in conveyance, and impacts to river velocities in the channel and at port locations.

Table 3 summarizes each combined alternative's alignment with the defined hydraulic engineering performance standards.

Table 3. Summary of CA Alignment with Hydraulic Engineering Performance Standards

Combined Alternative	Reduce Risk of Flood Damage and/or Improve Resilience	Consideration of Lower Missouri River system conveyance and flood planning	Increase Riverward Conveyance for Reoccurring Flood Events	Increased Predictability in Levee Performance	Minimal Encroachment into the FEMA-Mapped Floodway	Maintain Existing Velocities in the Channel and at Ports	Increased Turkey Creek Conveyance
CA0 No Action	N	N	N	N	N	Y	N
CA1 Existing Capital View Alignment, 34-ft Stage	Y	N	N	Y	N	Y	N
CA2 Turkey Creek & Riverside Setback, 34-ft Stage, Capital View Removed	Y	Y	Y	Y	N	Y	Y
CA3 Existing Capital View Alignment with Resilience	N	N	N	Y	N	Y	N
CA4 Optimization Alignment, 37-ft Stage, Capital View Remains	Y	Y	N	Y	Y	Y	N
CA5 Optimization Alignment, 37-ft Stage, Capital View Removed	Y	Y	Y	Y	Y	Y	N
CA6 Historic L-142 Alignment, 45-ft Stage, Capital View Remains	Y	Y	N	Y	Y	Y	N
CA7 Historic L-142 Alignment, 45-ft Stage, Capital View Removed	Y	Y	Y	Y	Y	Y	Y
CA8 Highway Alignment, 37-ft Stage, Capital View Remains	Y	Y	N	Y	Y	Y	N
CA9 Highway Alignment, 37-ft Stage, Capital View Removed	Y	Y	Y	Y	Y	Y	Y
CA10 Limited Resilience Measures Outside of Capital View Footprint	N	N	N	N	Y	Y	N
CA11 Nonstructural	Y	N	N	N	Y	Y	N
CA12 Remove Church Farm Levee	N	Y	N	N	Y	Y	N
CA13 Hybrid Alignment, 41-ft Stage, Capital View Removed	Y	Y	Y	Y	Y	Y	Y
CA13 Hybrid Alignment, 41-ft Stage, Capital View Remains	Y	Y	N	Y	Y	Y	N
CA13 Hybrid Alignment, 41-ft Stage, Partial Capital View Removal	Y	Y	Y	Y	Y	Y	Y

7.2 CA0 No Action / Future Without Project

Adoption of the No Action / FWOP alternative would maintain the existing condition at the northern bank through Jefferson City, assuming the existing Capital View levee remains in place and maintained to the original performance standard of 17% AEP of overtopping. An aerial map of CA0 is provided as **Figure 5**.

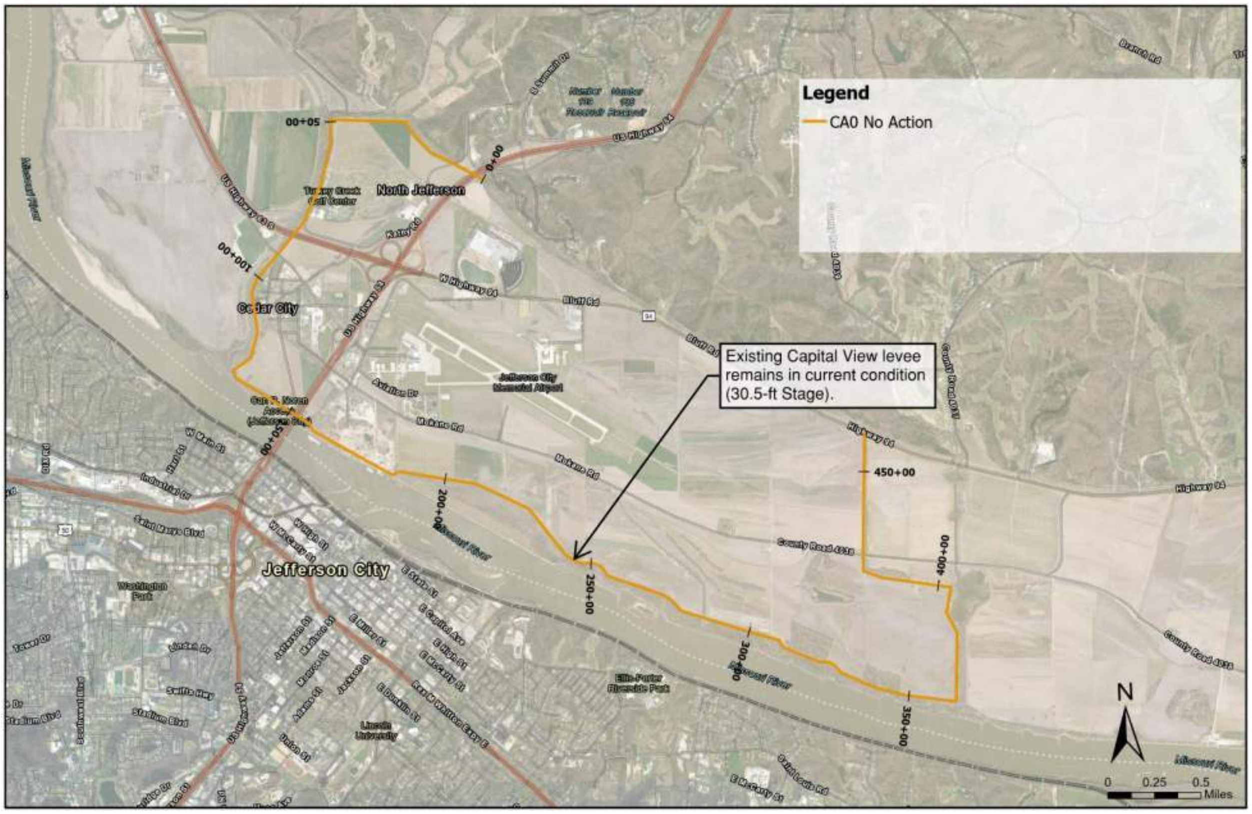

Figure 5. CA0 No Action Alignment

CA0 does not provide benefit to local, community, or river-system goals. Local considerations for CA0 include the continuation of repetitive breaches as experienced in past floods and flood fighting beginning at Missouri River stage 28-feet. There is no reduced risk for stakeholders or traveling public within the leveed area with this alternative and no change in levee performance.

Community considerations include limited road network access including access to Capital Sand and the regional wastewater treatment plant during a flood at a 31-ft stage corresponding to a 17% AEP of overtopping. The airport runways are also flooded at a 31-ft stage. The U.S. Highway 54 embankment is at an approximate 37-ft stage and then becomes overtopped.

Although there are no measurable impacts to the existing river-system with this alternative, the existing levee system is at the top bank of the Missouri River and within the mapped FEMA floodway shown in **Figure 6**. The existing alignment would continue to be exposed to repetitive toe loading. The existing Capital View levee alignment experiences repetitive breaches corresponding to locations where water is restricted, such as the Cedar City Road bridge on the western tieback and at the riverward midpoint where the Missouri River flow expands after exiting the bridge cross section. The no action alternative does not meet the Lower Missouri

System Plan objectives, which encourages implementation of a buffer at the riverside to minimize restrictions in conveyance.

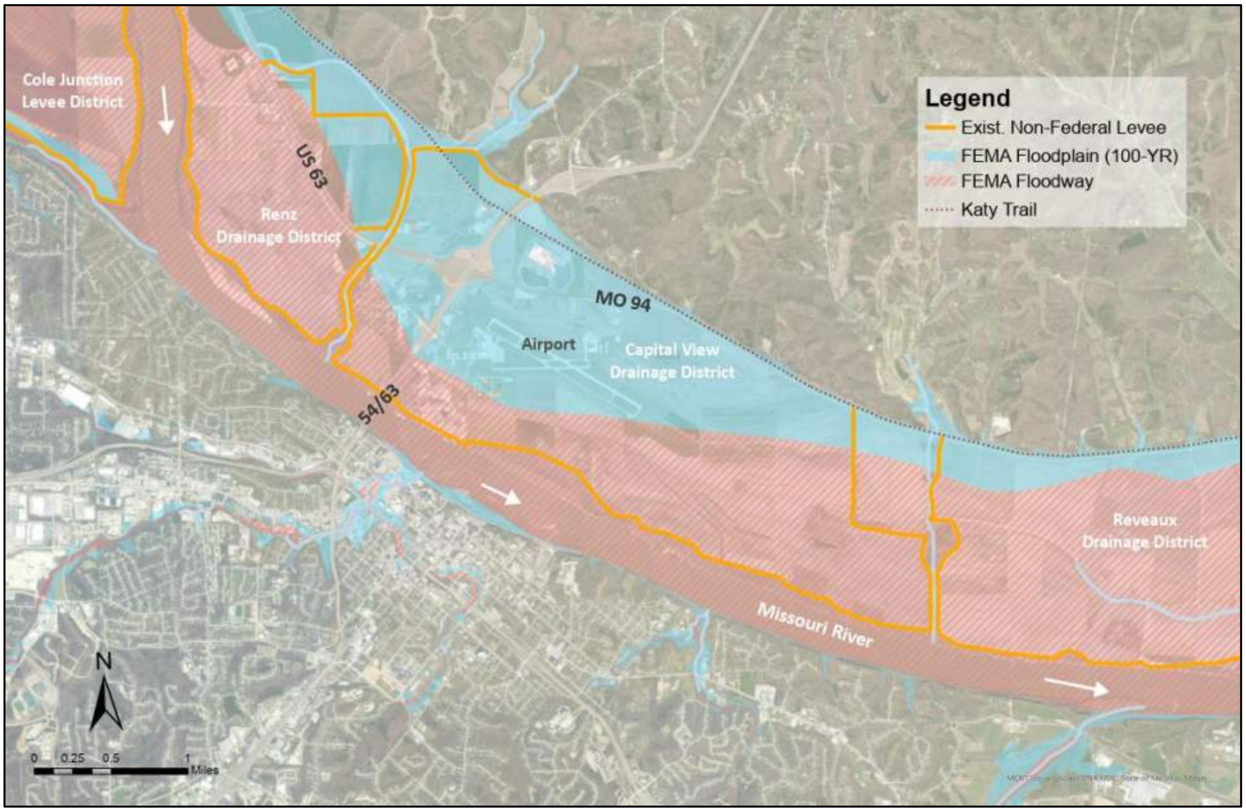

Figure 6. Existing Capital View Levee Location within the FEMA Mapped Floodway and Floodplain

CA0, No Action, is carried forward to the final array of alternatives for consideration and public input.

7.3 CA1 Existing Capital View Alignment, 34-ft Stage

CA1, shown in **Figure 7**, was presented as a locally desired alternative at the first public meeting. The methodology of assigning the 34-foot stage height for the top of levee was to provide a top elevation equivalent to adjacent levee systems in the area, notably Cole Junction and Cedar Creek Section 2 along Turkey Creek. The raise of an existing levee system requires the levee to be built to current federal engineering standards and incorporated as a federal levee, resulting in removal and rebuilding of an equivalent height levee. The cross section of a federal levee is not equivalent to a non-federal levee, resulting in real estate needs.

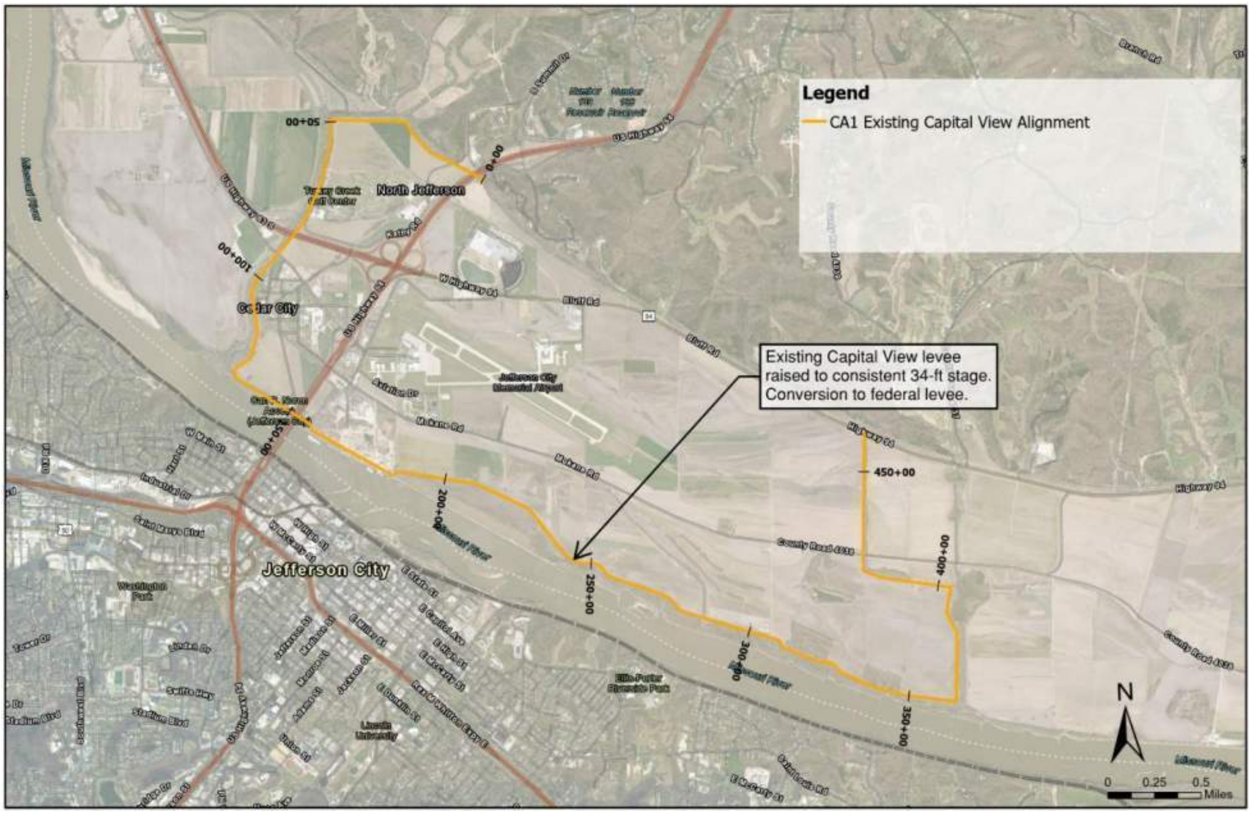

Figure 7. CA1 Existing Capital View Alignment, 34-ft Stage

CA1 modeling parameters are provided in **Table 4**. Adaptation of the FWOP hydraulic model to reflect the alternative conditions was completed as outlined in Section 6.

Table 4. CA1 - Summary of Key Parameters

Model Parameter	Input
Targeted Design Performance	34-ft Stage overtopping. 34-ft stage = 554.2 ft NAVD88
Terrain File	LoMO_1m_NWK_JeffCity
Riverward Levee US Top Elev. (ft)	554.2
Riverward Levee DS Top Elev. (ft)	550.5

Regarding local considerations, raising the existing levee height 3.5-feet above the existing overtopping elevation results in decreased overtopping frequency for all properties within the existing Capital View Drainage District footprint. Reduced flood fighting and increased predictability in levee performance with a federal levee design would be anticipated, as the levee would be constructed to federal standards (as outlined in Appendix A3 references), include structural closure structures at roadways, and eliminate low points that require sandbagging in FWOP conditions.

Community considerations include limited road network access including access to Capital Sand and the regional wastewater treatment plant up to a flood at a 34-ft stage, corresponding to an 8% AEP of overtopping. The airport runways are also flooded at a 34-ft stage. The U.S. Highway 54 embankment overtops at an approximate 37-ft stage.

The existing levee system is at the top bank of the Missouri River and within the mapped FEMA floodway shown in **Figure 6**. Maintaining the existing alignment would continue to be exposed to repetitive toe loading. The existing Capital View levee alignment experiences repetitive hydraulic interactions, including increased velocities and loading, at locations where water is restricted, such as the Cedar City Road bridge on the western tieback and at the riverward midpoint where the Missouri River flow expands after exiting the bridge cross section. Raising the existing Capital View levee does not meet the Lower Missouri System Plan objectives, which encourages implementation of a buffer at the riverside to minimize restrictions in conveyance. It is notable that raising levees “one-for-one” as a means of matching a neighboring levee system height without incorporating levee setbacks reduces channel conveyance and is impactful to peak water surface elevations upstream and downstream of the levee and is not a long-term solution.

CA1, Existing Capital View Alignment, 34-ft Stage, was not carried forward as a viable alternative after engineering analysis. Justification for screening was due to failure to meet overall goals of conveyance or resilience in the system plan and negative net benefits. Induced impacts and impacts to the mapped FEMA floodway are also a concern with this alternative.

7.4 CA2 Turkey Creek & Riverside Setback, 34-ft Stage, Capital View Removed

CA2, shown in **Figure 8**, iterated from the existing Capital View alignment to test setback locations that would allow for a 34-ft stage top of levee without causing a significant rise in the Missouri River water surface profile for all eight modeled flow frequencies.

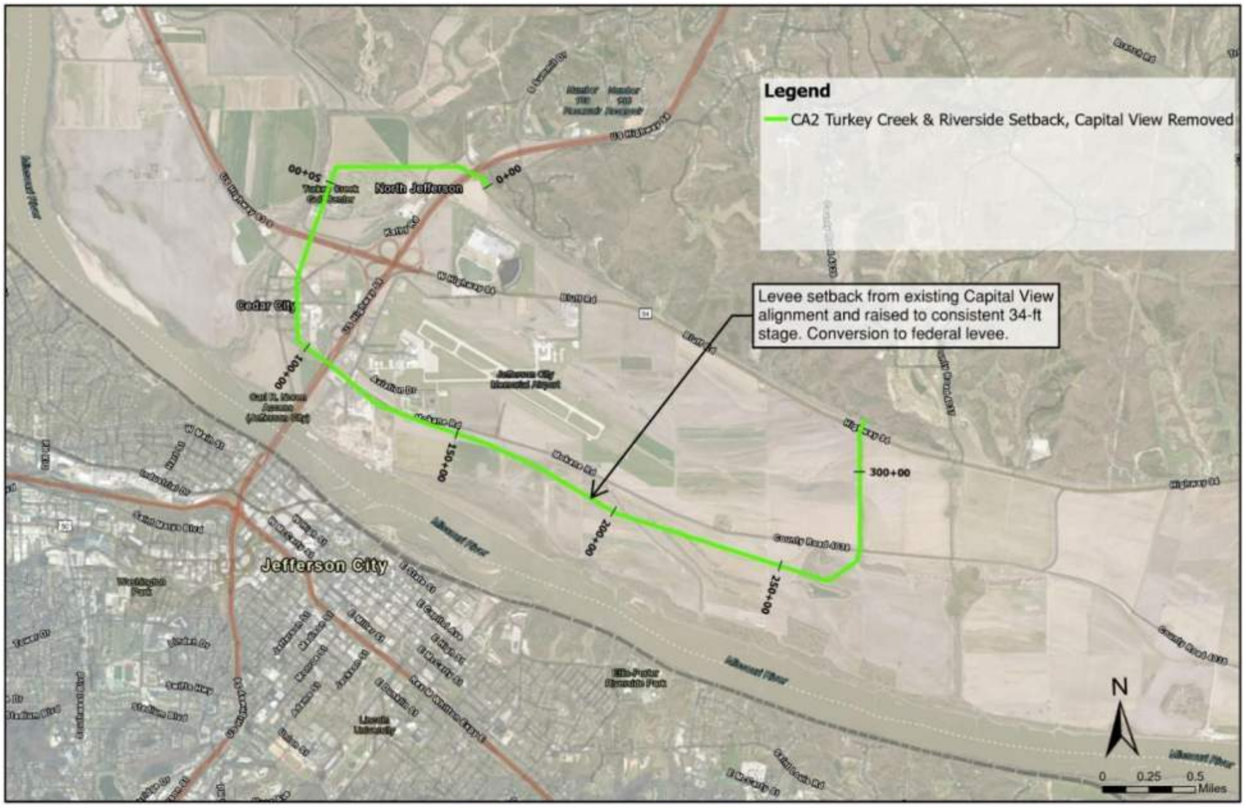

Figure 8. CA2 Turkey Creek & Riverside Setback, 34-ft Stage, Capital View Removed

CA2 modeling parameters are provided in **Table 5**. Adaptation of the FWOP hydraulic model to reflect the alternative conditions was completed as outlined in Section 6.

Table 5. CA2 - Summary of Key Parameters

Model Parameter	Input
Targeted Design Performance	34-ft Stage overtopping. 34-ft stage = 554.2 ft NAVD88.
Terrain File	LoMO_1m_NWK_JeffCity_RemovedCV
Riverward Levee US Top Elev. (ft)	554.2
Riverward Levee DS Top Elev. (ft)	551.5

Raising the existing levee height 3.5-feet above the existing overtopping elevation results in decreased overtopping frequency for properties within setback footprint. The levee setback would provide the Missouri River and Turkey Creek with added conveyance area and relieve hydraulic pinch points that cause hydraulic stacking and localized water surface elevation increase at Turkey Creek. Providing additional conveyance for the Missouri River allows hydraulic expansion to occur after exiting the bridge and would relocate infrastructure out of an area expected to require continuous maintenance due to repetitive stress. Reduced flood fighting and increased predictability in levee performance with a federal levee design would be anticipated, as the levee would be constructed to federal standards, include structural closure structures at roadways, and eliminate low points that require sandbagging in FWOP conditions. Incorporating a setback at Turkey Creek has an added benefit of moving flood fight efforts away from the main channel and higher velocity flows during the development of a flood.

Community considerations include limited road network access including access to Capital Sand and the regional wastewater treatment plant up to a flood at a 34-ft stage, corresponding to an 8% AEP of overtopping. The airport runways are also flooded at a 34-ft stage. The U.S. Highway 54 embankment is at an approximate 37-ft stage and then becomes overtopped. The setback footprint results in more frequent inundation for lands riverward of the levee alignment.

A levee setback and removal of the existing Capital View levee would increase Missouri River conveyance prior to overtopping and reduce constriction of the river in flood conditions through Jefferson City, in alignment with Lower Missouri System Plan objectives. Reduced constriction would allow the river volume to spread in the overbanks, resulting in a lower peak water surface elevation on the Missouri River for more frequent storm events. Providing a setback at Turkey Creek relieves channel constriction and eliminates the choke point of the Cedar City Road bridge, allowing for additional backwater storage in the channel. Setting back the levee along Turkey Creek also reduces the water surface elevation along adjacent levees through the Turkey Creek corridor. Impacts to river velocities in the channel and at existing port location are negligible, with velocity outputs within 0.1-ft/s of FWOP for all flow events.

CA2, Turkey Creek & Riverside Setback, 34-ft Stage, Capital View Removed, was not carried forward as a viable alternative after engineering analysis. Justification for screening was due to negative net benefits and a recurrent overtopping frequency that would not justify the cost of a federal levee segment.

7.5 CA3 Existing Capital View Alignment with Resilience

CA3, shown in **Figure 9**, incorporated resilience measures typical of federal levee design standards that would optimize the levee district's ability to flood fight and recover from a flood. Resilience measures applicable to the existing Capital View levee reduce breach potential including reinforcing the levee, constructing a designated overtopping location, adding seepage berms, and adding stability berms. Hydraulic modeling is equivalent to the Future Without Project condition for CA3.

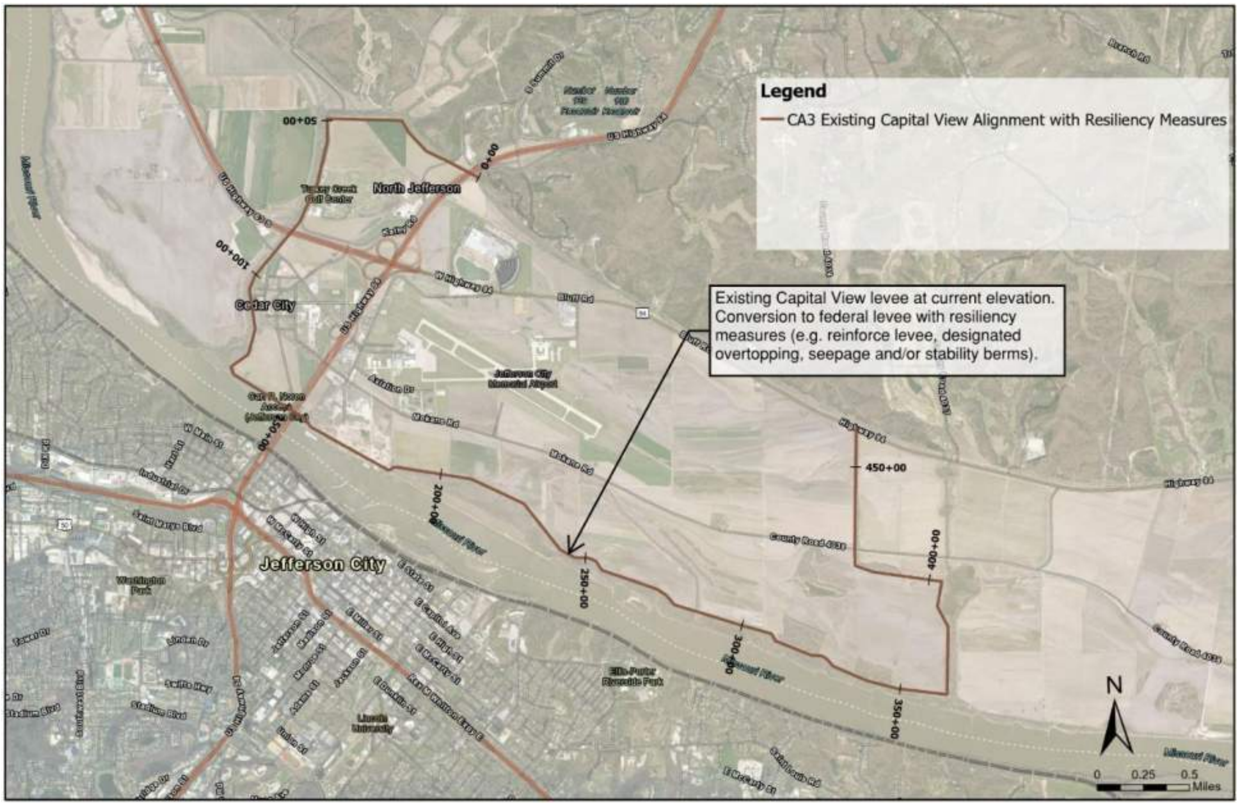

Figure 9. CA3 Existing Capital View Alignment with Resilience

Similar to CA0, CA3 does not provide benefit to local, community, or river-system goals. In addition, the existing Capital View levee location and elevations within the floodplain do not make this system a good candidate for resilience measures as a standalone alternative. The levee would continue to overtop with an AEP of 17% AEP, and less frequent floods would exceed the engineering limitations of preventing levee breaches or designating overtopping locations.

CA3, Existing Capital View Alignment with Resilience, was not carried forward as a viable alternative after the initial screening of alternatives in reference to the study objectives. Justification for screening was due to planning efficiency and no additional benefits gained from adoption.

7.6 CA4 & CA5 Optimization Alignment

CA4 and CA5, shown in **Figure 10** and **Figure 11**, both analyze a new levee alignment setback from the riverbank with the existing Capital View levee removed in CA5. This alignment was developed by local stakeholders to utilize available existing public right of way and existing easements to reduce real estate needs. The alternative was analyzed in conditions where the existing Capital View remains and where the existing Capital View levee is removed. Considerations for keeping or removing the existing Capital View levee are discussed in Section 8. This section focuses on analysis of the new federal levee alignment alternative.

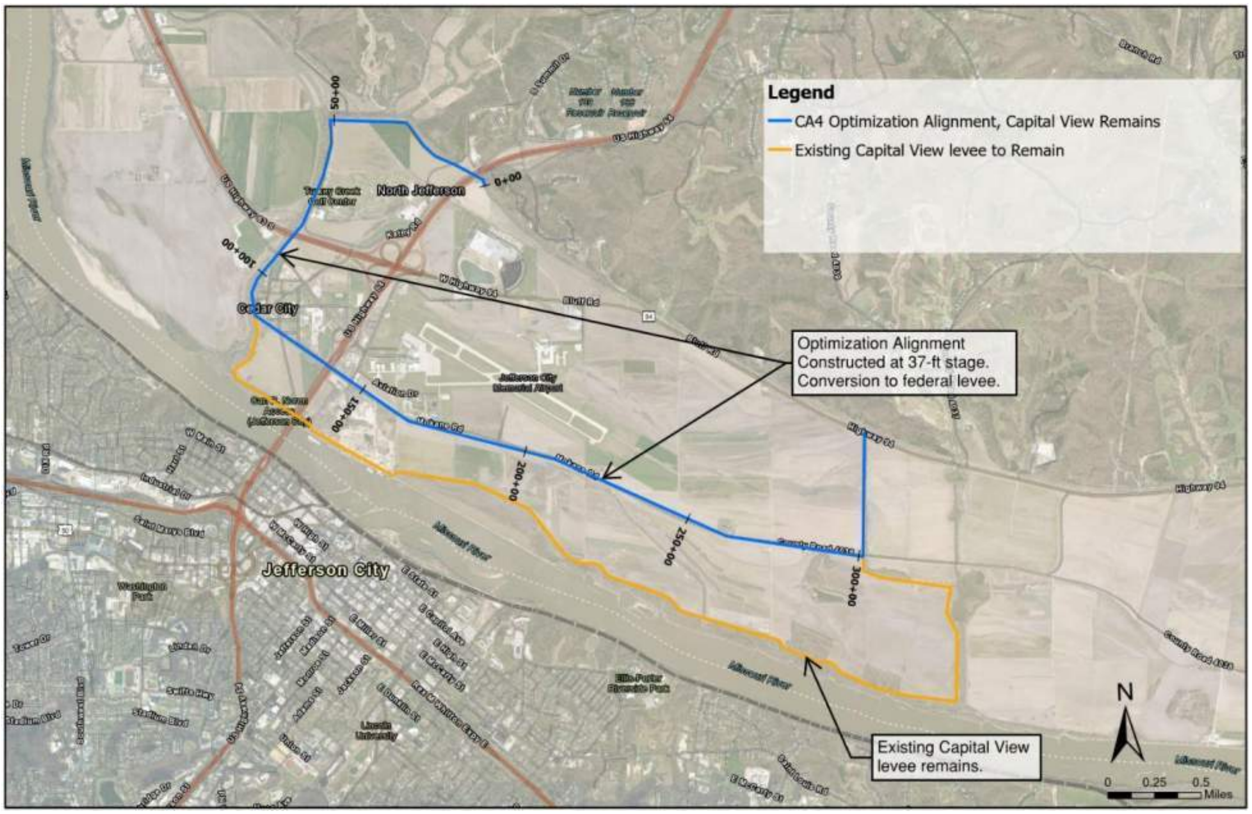

Figure 10. CA4 Optimization Alignment, 37-ft Stage, Capital View Remains

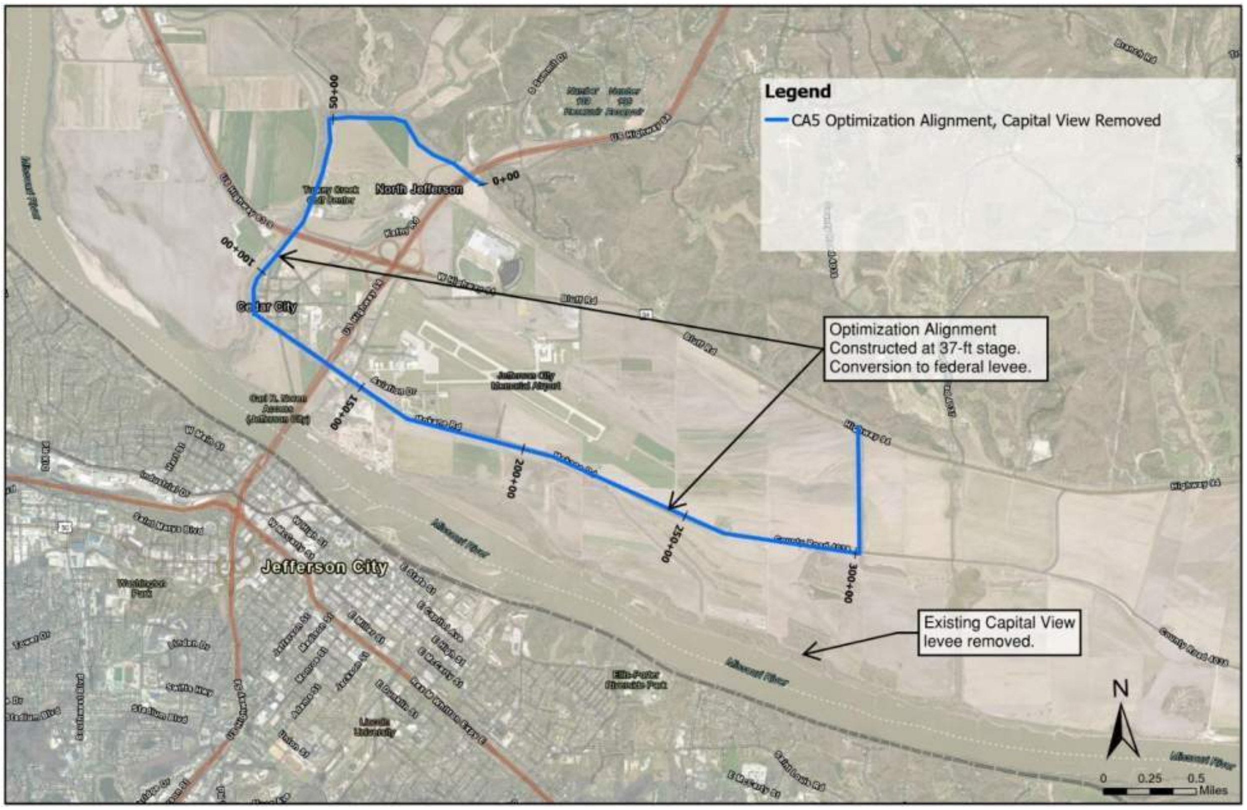

Figure 11. CA5 Optimization Alignment, 37-ft Stage, Capital View Removed

CA4 and CA5 modeling parameters are provided in **Table 6**. Adaptation of the FWOP hydraulic model to reflect the alternative conditions was completed as outlined in Section 6.

Table 6. CA4 & CA5 - Summary of Key Parameters

Model Parameter	Input
Targeted Design Performance	37-ft Stage overtopping. 37-ft stage = 557.2 ft NAVD88.
Terrain File	LoMO_1m_NWK_JeffCity LoMO_1m_NWK_JeffCity_RemovedCV
Riverward Levee US Top Elev. (ft)	557.5
Riverward Levee DS Top Elev. (ft)	553.8

Constructing a levee with a height 6.5-feet above the existing Capital View levee elevation results in decreased overtopping frequency for properties within setback footprint. Reduced flood fighting and increased predictability in levee performance with a federal levee design would be anticipated, as the levee would be constructed to federal standards, include closure structures at roadways, and eliminate low points that require sandbagging in FWOP conditions.

The removal of Capital View levee would provide the Missouri River additional conveyance area, especially downstream of the bridge where flow actively expands after being constricted. Providing additional conveyance for the Missouri River allows hydraulic expansion to occur after exiting the bridge and would relocate infrastructure out of an area expected to require continuous maintenance due to repetitive stress. The setback footprint results in more frequent inundation for lands riverward of the levee alignment. No setback on Turkey Creek is included with this alternative and existing concerns regarding conveyance and stresses to the western levee tieback would remain.

Community considerations include limited road network access including access to Capital Sand and the regional wastewater treatment plant up to a flood at a 37-ft stage, corresponding to a 3.6% AEP overtopping level if the existing Capital View levee is maintained or 3.4% AEP overtopping level if the existing Capital View levee is removed.

Without removal of the existing Capital View levee, additional Missouri River conveyance would not be realized to align with the Lower Missouri System Plan objectives. Impacts to adjacent levee systems upstream of the Optimization Alignment are expected without a Turkey Creek levee setback. Impacts to the Missouri River profile at the confluence of Wears Creek would be 0.1-foot rise with the existing Capital View levee in place, and 0.0-foot without the existing Capital View levee for the 10% AEP flow. Impacts of the 1% AEP flow would be a 0.5-foot rise with the existing Capital View levee and a 0.4-foot rise without the existing Capital View levee. Impacts to river velocities in the channel and at existing port location are negligible, with velocity outputs within 0.1 to 0.4-ft/s of FWOP for all flow events.

CA4 and CA5, Optimization Alignment at 37-foot stage, were not carried forward as a final alternative after engineering analysis. Justification for screening was due to negative net benefits, limited comprehensive benefits, and moderate overtopping frequency.

7.7 CA6 & CA7 Historic L-142 Alignment

CA6 and CA7, shown in **Figure 12** and **Figure 13**, both analyze a new levee alignment setback from the riverbank with the existing Capital View levee removed in CA7. This alignment was developed by USACE in 2001 as part of a General Reevaluation Report to provide flood risk management to properties downstream of U.S. Highway 54 and minimize the levee footprint within the regulated floodway. The alternative was analyzed in conditions where the existing Capital View remains and where the existing Capital View levee is removed. Considerations for keeping or removing the existing Capital View levee are discussed in Section 8. This section focuses on analysis of the new federal levee alignment alternative.

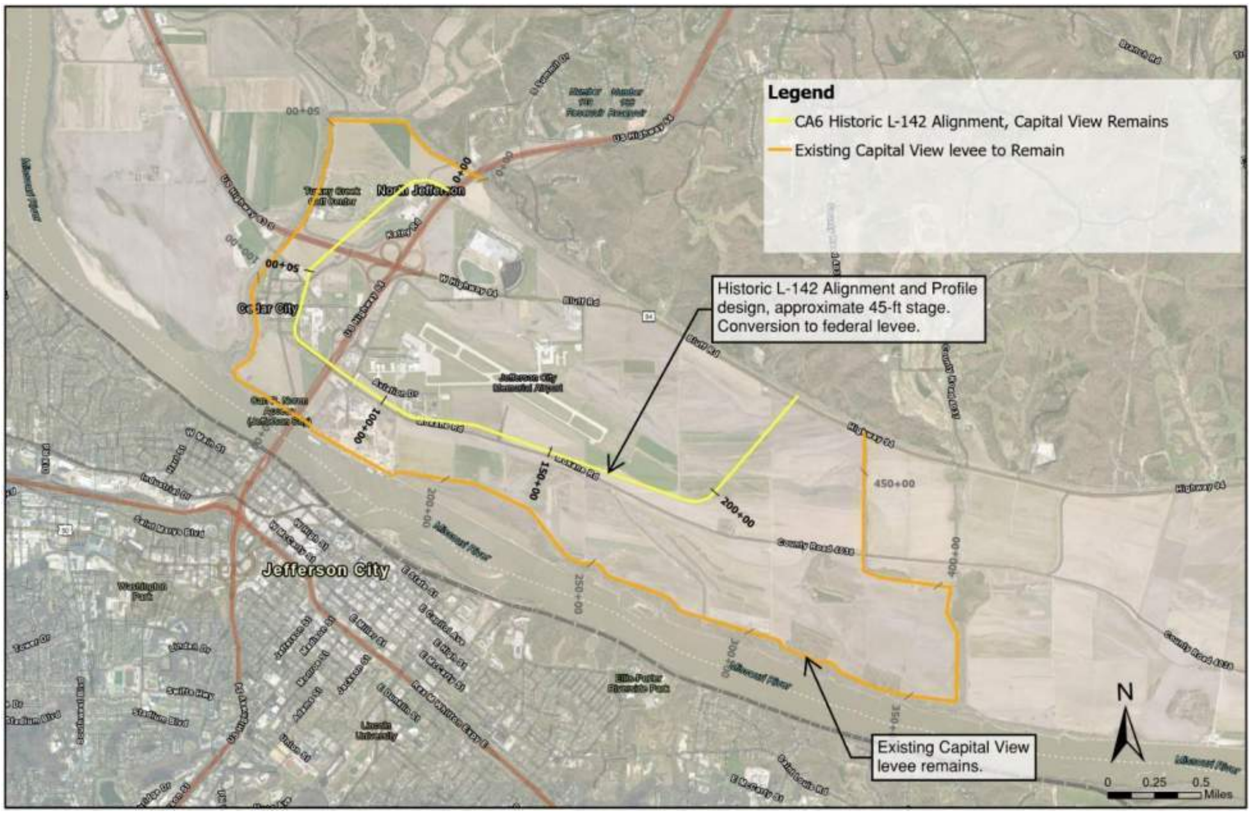

Figure 12. CA6 Historic L-142 Alignment, 45-ft Stage, Capital View Remains

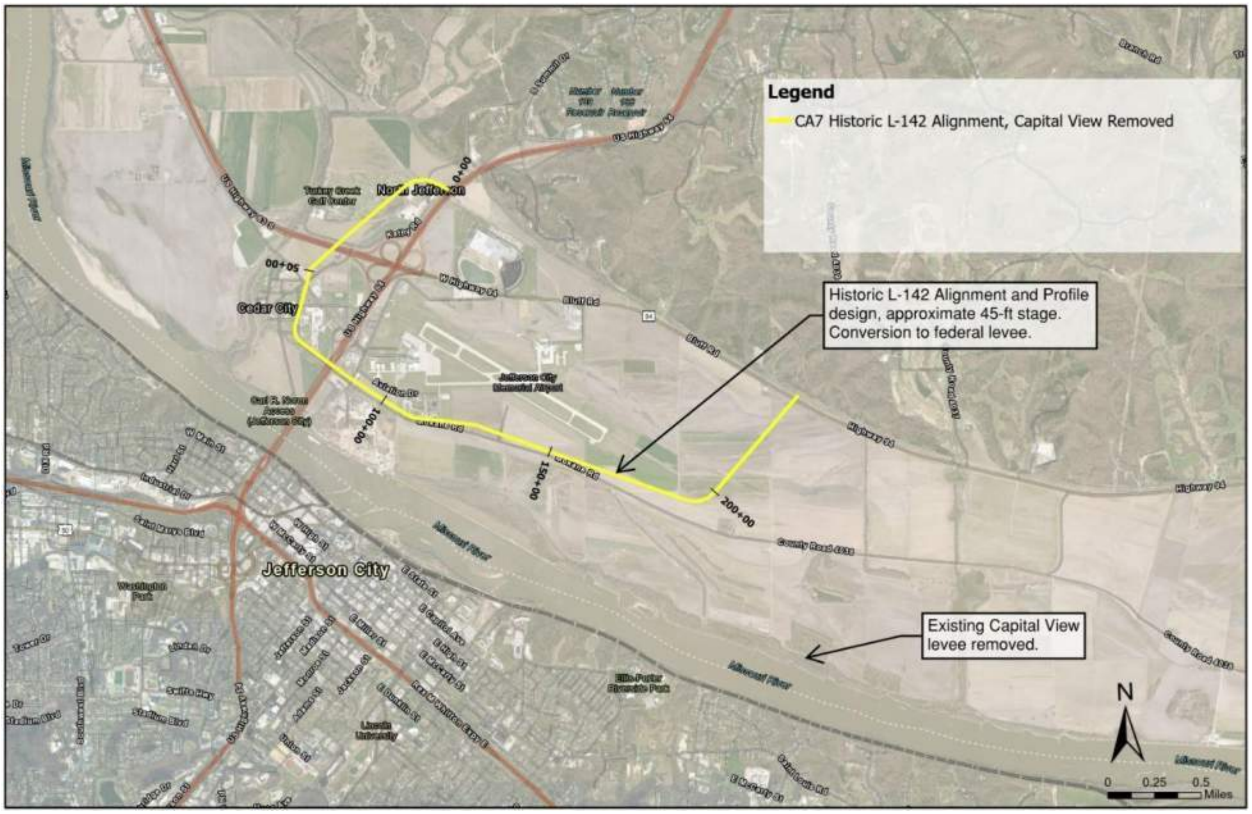

Figure 13. CA7 Historic L-142 Alignment, 45-ft Stage, Capital View Removed

CA6 and CA7 modeling parameters are provided in **Table 7**. Adaptation of the FWOP hydraulic model to reflect the alternative conditions was completed as outlined in Section 6.

Table 7: CA6 & CA7 - Summary of Key Parameters

Model Parameter	Input
Targeted Design Performance	Optimized benefits elevation from 2001 Criteria
Terrain File	LoMO_1m_NWK_JeffCity LoMO_1m_NWK_JeffCity_RemovedCV
Riverward Levee US Top Elev. (ft)	565.2
Riverward Levee DS Top Elev. (ft)	560.3

The proposed top of levee elevations would be designed to not overtop in a 0.2% AEP flood event, greatly reducing the overtopping frequency of properties within the setback footprint. Reduced flood fighting and increased predictability in levee performance with a federal levee design would be anticipated, as the levee would be constructed to federal standards, include closure structures at roadways, and eliminate low points that require sandbagging in FWOP conditions.

The removal of Capital View levee would provide the Missouri River additional conveyance area, especially downstream of the bridge where flow actively expands after being constricted. Providing additional conveyance for the Missouri River allows hydraulic expansion to occur after exiting the bridge and would relocate infrastructure out of an area expected to require continuous maintenance due to repetitive stress. The levee setback would provide Turkey Creek with added conveyance area and minimize hydraulic pinch points that cause hydraulic stacking and localized water surface elevation increase at Turkey Creek in the FWOP condition. The setback footprint results in more frequent inundation for lands riverward of the levee alignment.

Without removal of the existing Capital View levee, additional Missouri River conveyance would not be realized to align with the Lower Missouri System Plan objectives. The levee alignment avoids the mapped regulatory floodway. A levee at the proposed height would be much taller than the surrounding levee systems up and downstream of the system. Impacts to river velocities in the channel and at existing port location are negligible, with velocity outputs within 0.1 to 0.4-ft/s of FWOP for all flow events.

CA6 and CA7 (Historic L-142 Alignment, 45-ft Stage) were not carried forward as final alternatives after preliminary engineering analysis. Justification for screening was due to negative net benefits and community and non-federal sponsor opposition.

7.8 CA8 & CA9 Highway Embankment Alignment

CA8 and CA9, shown in **Figure 14** and **Figure 15**, both analyze a new levee alignment setback from the riverbank with the existing Capital View levee removed in CA9. This alignment was developed to utilize available existing public right of way and high ground to reduce real estate needs and total project costs. The alternative was analyzed in conditions where the existing Capital View remains and where the existing Capital View levee is removed. Considerations for keeping or removing the existing Capital View levee are discussed in Section 8. This section focuses on analysis of the new federal levee alignment alternative.

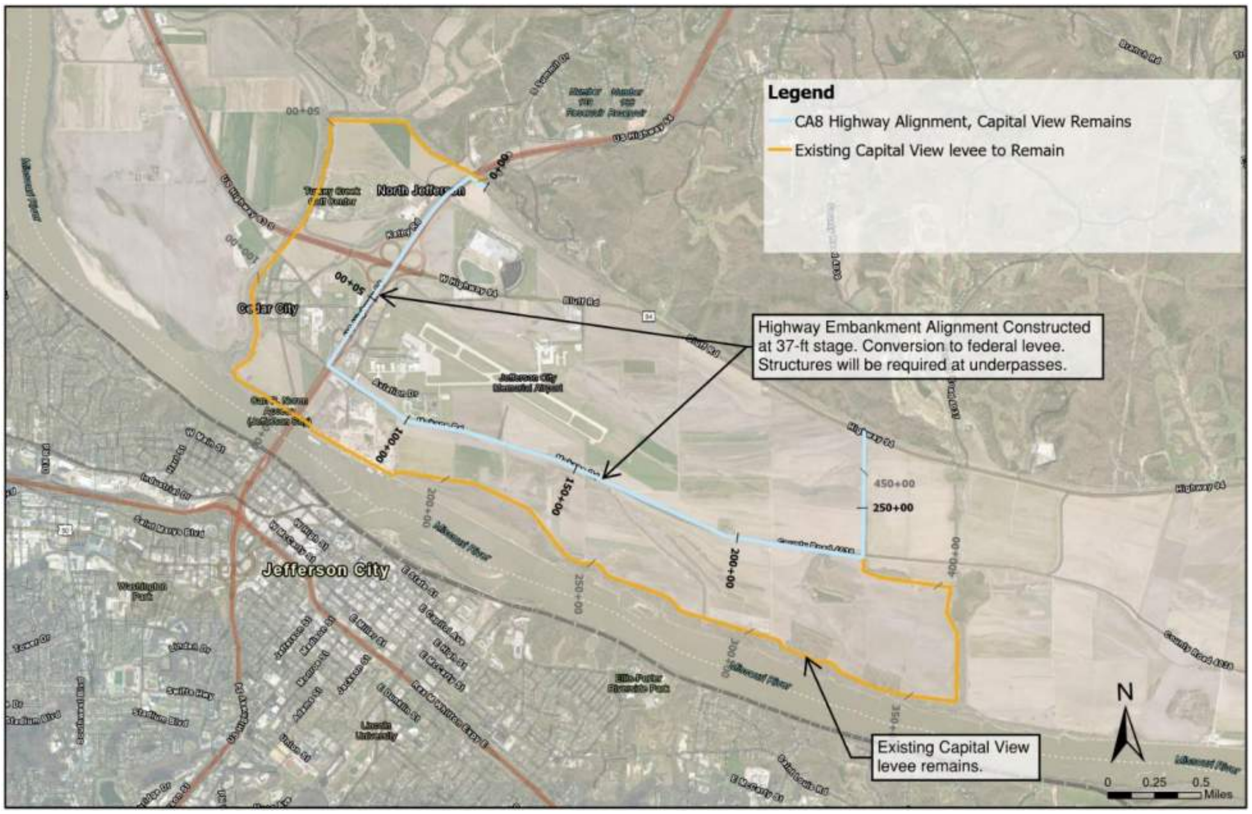

Figure 14. CA8 Highway Alignment, 37-ft Stage, Capital View Remains

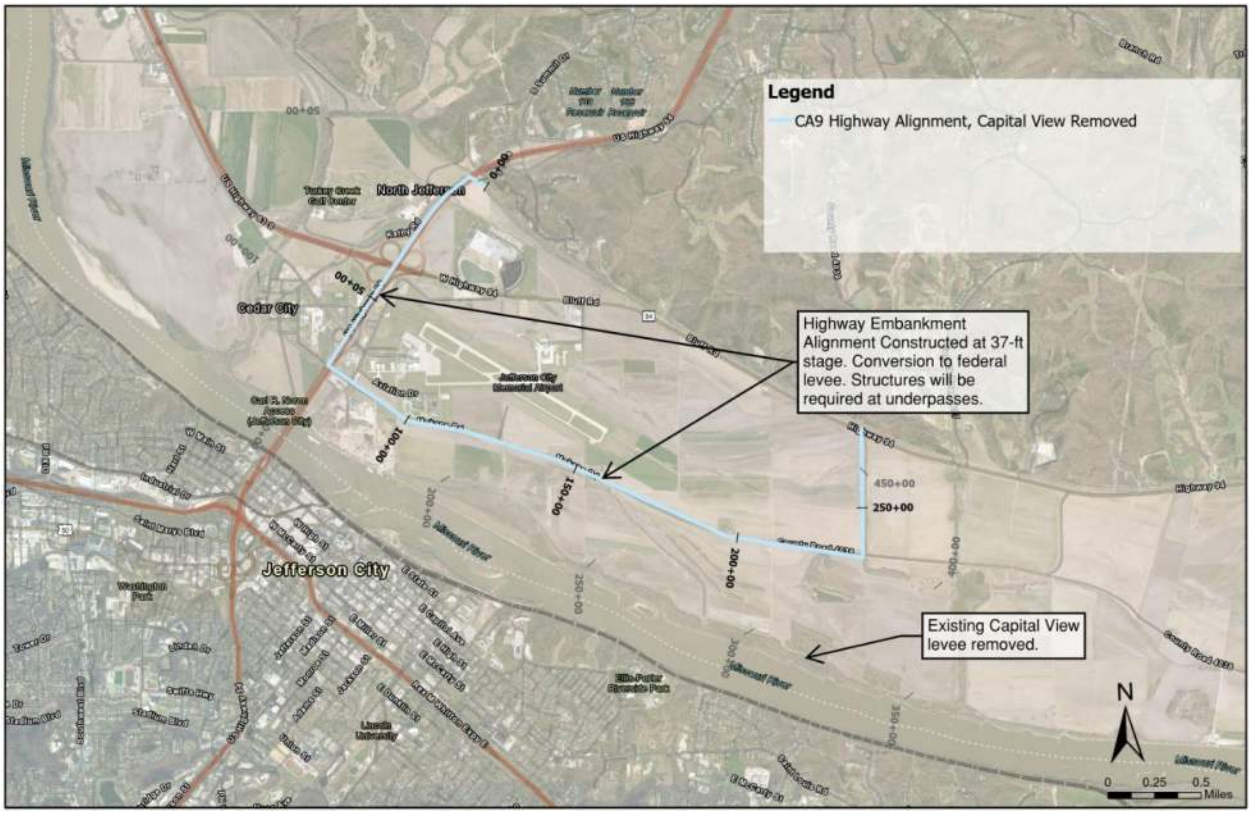

Figure 15. CA9 Highway Alignment, 37-ft Stage, Capital View Removed

CA8 and CA9 modeling parameters are provided in **Table 8**. Adaptation of the FWOP hydraulic model to reflect the alternative conditions was completed as outlined in Section 6.

Table 8. CA8 & CA9 - Summary of Key Parameters

Model Parameter	Input
Targeted Design Performance	Maximize levee height considering highway embankment lowest profile grade
Terrain File	LoMO_1m_NWK_JeffCity LoMO_1m_NWK_JeffCity_RemovedCV
Riverward Levee US Top Elev. (ft)	557.5
Riverward Levee DS Top Elev. (ft)	553.8

The proposed top of levee elevation utilizing the highway embankment is limited by the existing low point in the highway embankment. The resulting levee would be designed to overtop during a 2.4% AEP flood event, reducing the overtopping frequency for properties within the setback footprint. Reduced flood fighting and increased predictability in levee performance with a federal levee design would be anticipated, as the levee would be constructed to federal standards, include closure structures at roadways, and eliminate low points that require sandbagging in FWOP conditions.

The traffic conveyance at the highway would remain similar to the FWOP condition, with additional conveyance before overtopping for the leveed area. This plan would require additional consideration from the Missouri Department of Transportation and a closure structure and ring levee for the Katy Trail underpass below U.S. Highway 54.

The removal of Capital View levee would provide the Missouri River additional conveyance area, especially downstream of the bridge where flow actively expands after being constricted. Providing additional conveyance for the Missouri River allows hydraulic expansion to occur after exiting the bridge and would relocate infrastructure out of an area expected to require continuous maintenance due to repetitive stress. The levee setback would provide Turkey Creek with added conveyance area and minimize hydraulic pinch points that cause hydraulic stacking and localized water surface elevation increase at Turkey Creek in the FWOP condition. The setback footprint results in more frequent inundation for lands riverward of the levee alignment.

Without removal of the existing Capital View levee, additional Missouri River conveyance would not be realized to align with the Lower Missouri System Plan objectives. Impacts to river velocities in the channel and at existing port locations are negligible, with velocity outputs within 0.1 to 0.3-ft/s of FWOP for all flow events.

CA8 and CA9, Highway Embankment Alignment, were not carried forward as a final alternative after engineering analysis. Justification for screening was due to negative net benefits and stakeholder opposition.

7.9 CA10 Limited Resiliency Measures Outside of Capital View Footprint

CA10, shown in **Figure 16**, combined measures off-site from the Capital View Drainage District to analyze solutions that could influence watershed patterns that were repeatably identified as influences to the existing Capital View levee's failure modes.

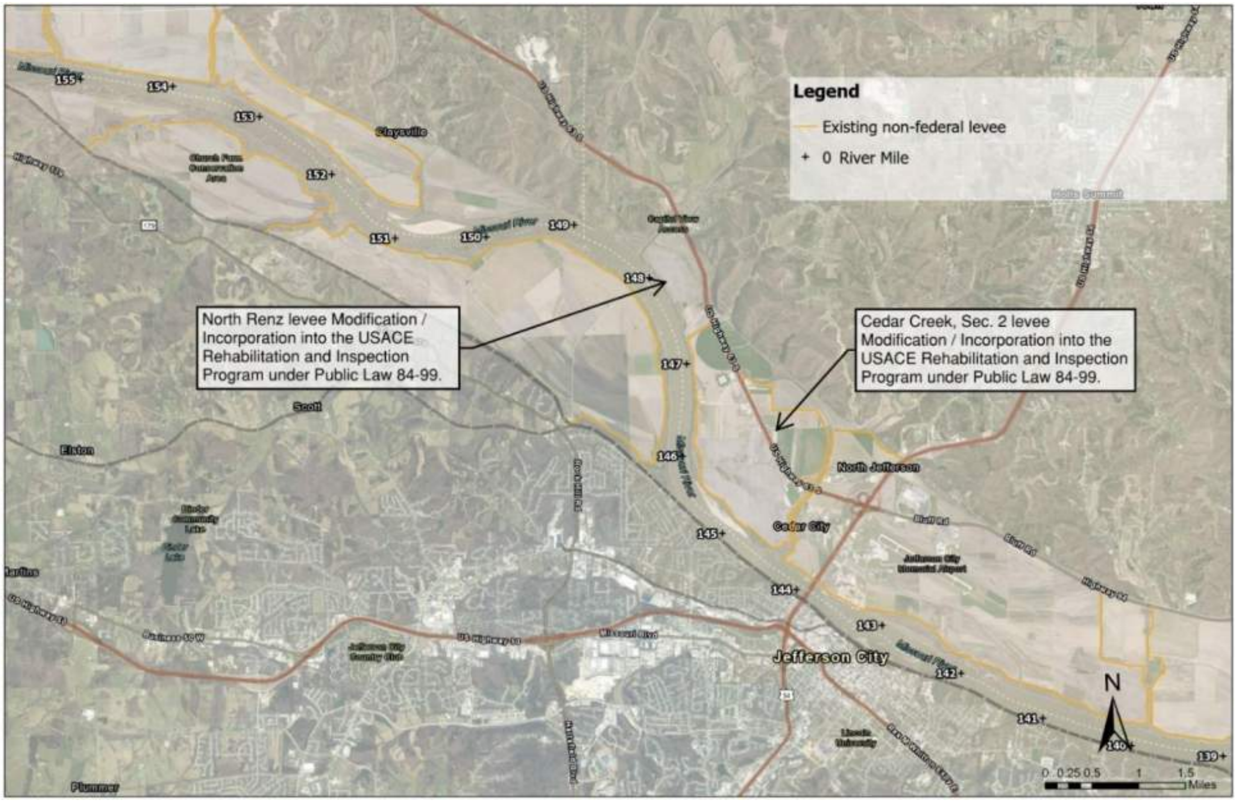

Figure 16. CA10 Limited Resiliency Measures Outside of Capital View Footprint

CA10 does not provide benefit to local, community, or river-system goal as a standalone alternative. No watershed measures are influential on the existing Capital View levee overtopping rate, and therefore ineffective in reducing overall flood risk. These measures were therefore, not hydraulically modeled and CA10 was not carried forward as a viable alternative after initial screening because it does not meet the project objective and is not effective as a standalone alternative.

7.10 CA11 Nonstructural

CA11 identifies nonstructural measures that could be implemented in the Capital View Drainage District. After the Flood of 1993, Jefferson City implemented ongoing floodplain management efforts including a flood action plan, floodplain zoning restrictions, and a floodplain buyout program. These efforts have been successful at reducing risk associated with flooding in areas experiencing repetitive inundation. Many of the properties within the Capital View Drainage District, especially within the focused area of Old Cedar City, now referred to as north Jefferson City, were acquired and converted to park land or other floodplain uses. In addition, many buildings that were not acquired have been raised to above the base flood elevation, or otherwise wet-floodproofed, to meet the City’s floodplain management standards. Unmodified buildings remaining in the floodplain at the time of this study are generally not good candidates for floodproofing due to inundation depths greater than 3-feet for events greater than the 10% AEP. Other considerations for determining constructability and engineering feasibility of floodproofing approaches included applying standard FEMA design methodology to determine candidates. This included assessing building condition, flood characteristics (depth, duration, velocity) at the building site, limitations to dry floodproofing over 3-feet for commercial properties that could not be elevated due to size and use, egress considerations, and level of business interruption.

CA11 is not selected as a final alternative due to not meeting project objectives and not having positive economic benefits.

7.11 CA12 Remove Church Farm Levee

Removal of the Church Farm, known as Prison Farm Levee in the USACE National Levee Database, was assessed as requested by the study sponsor to evaluate any impacts downstream of connecting the existing leveed area to the floodplain. The location is shown in **Figure 17**. The State of Missouri owns the 3.8 mile earthen levee and 998 acres within the leveed area. The levee was breached in the 2019 flood event but not fully restored at the time of this study. In modeling of the FWOP, this levee was assumed to be functioning as designed for full comparison of alternatives.

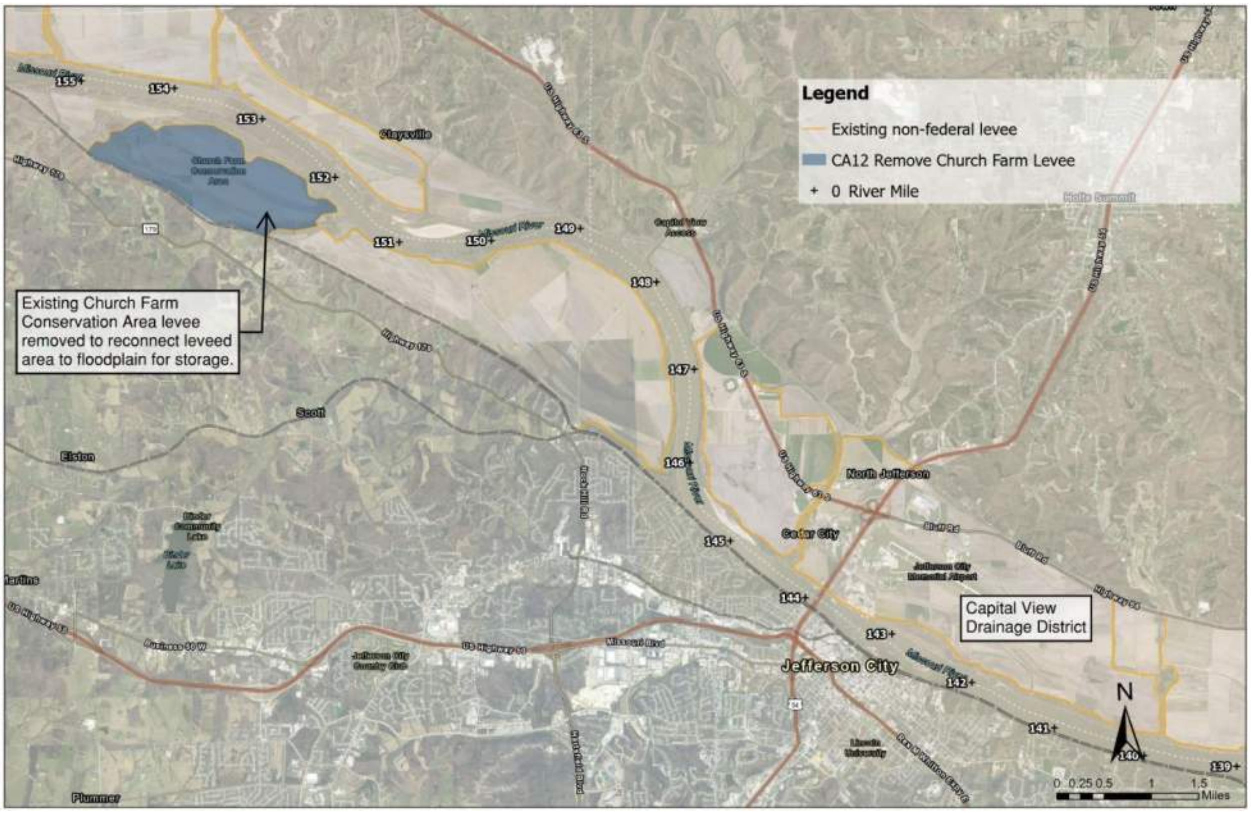

Figure 17. CA12 Remove Church Farm Levee

CA12 modeling parameters are provided in **Table 9**. Adaptation of the FWOP hydraulic model to reflect the alternative conditions was completed by altering the terrain file to lower the Church Farm levee 12-feet in vertical elevation, setting the terrain at the overbank elevation.

Table 9. CA12- Summary of Key Parameters

Model Parameter	Input
Targeted Design Performance	Remove Church Farm levee in entirety to reconnect interior area to overbank
Terrain File	LoMO_1m_NWK_JeffCity_RemovedCFarm
Riverward Levee US Top Elev. (ft)	Capital View Levee as FWOP Condition
Riverward Levee DS Top Elev. (ft)	Capital View Levee as FWOP Condition

This alternative was preliminarily assessed to determine if there would be a measurable, beneficial impact to water surface elevations through the Jefferson City Missouri River corridor with the removal of the Church Farm levee and reconnection of the floodplain. The existing levee overtops during a 12% AEP event in current condition, similar to other levees through the modeled area. Removing the levee resulted in a slight decrease in maximum water surface elevation (0.1-ft) and total peak flow (0.3%) at the Jefferson City gage during a 10% AEP event and effectively no change in the Missouri River profile or Jefferson City hydrographs during a 1% AEP event, summarized in **Table 10**.

Table 10. Church Farm Levee Removal Impacts at Jefferson City Gage

Condition	10% AEP Max WSE (ft)	10% AEP Max Peak Flow (cfs)	1% AEP Max WSE (ft)	1% AEP Max Peak Flow (cfs)
Existing Conditions	552.9	348,500	561.5	754,000
Church Farm Levee Removed	552.8	347,700	561.5	753,800

To assess sensitivity of the Missouri River water surface elevation profile to upstream storage, the Church Farm leveed area was also utilized to assess the impacts of using the interior leveed area as a storage basin by notching the downstream tieback to allow Missouri River backwater into the leveed area during a flood. The storage area was also not effective at reducing downstream water surface elevations during the 10% or 1% AEP events. **Figure 18** plots the Missouri River profiles comparing the FWOP condition to the alternative condition in which Church Farm levee is completely removed and Church Farm leveed area is utilized as floodplain storage.

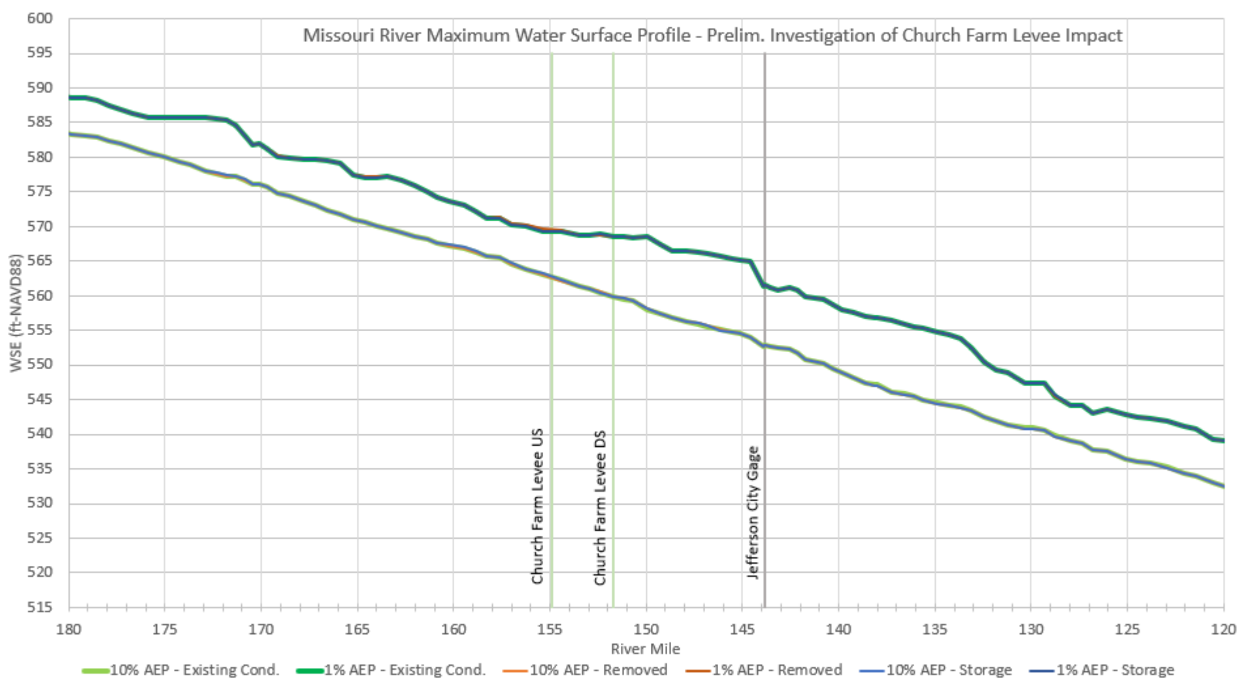

Figure 18. Missouri River Max. WSE Profile - Church Farm Levee Removal Investigation

CA12, Remove Church Farm Levee, was screened early in plan evaluation as a standalone alternative due to the overall effort required to implement the alternative with negligible impact to the Missouri River water surface elevation profile at the project area footprint. Although removal would align with overall goals considering system conveyance and flood planning as well as increasing riverward conveyance for reoccurring flood events, the impact at this study's project location is not an efficient standalone solution for Jefferson City.

7.12 CA13 Hybrid Alignment, 41-ft Stage

CA13, shown in **Figure 19**, **Figure 20**, and **Figure 21**, alignment was developed by maximizing the positive qualities of the optimization alignment and L-142 alignment footprints. The elevation was set at a 1% AEP level to provide a meaningful increase compared to the existing levee overtopping elevation, without designing to FEMA accreditation standards which could result in additional development behind the levee system. The 1% AEP overtopping elevation bridges the gap between the previously analyzed 37-ft and 45-ft stage overtopping heights that resulted in negative net benefits. The alternative was analyzed in multiple conditions including maintaining the existing Capital View levee, removing the existing Capital View levee, and partially removing the existing Capital View levee. Considerations for keeping or removing the existing Capital View levee are discussed in Section 8. This section focuses on analysis of the new federal levee alignment alternative.

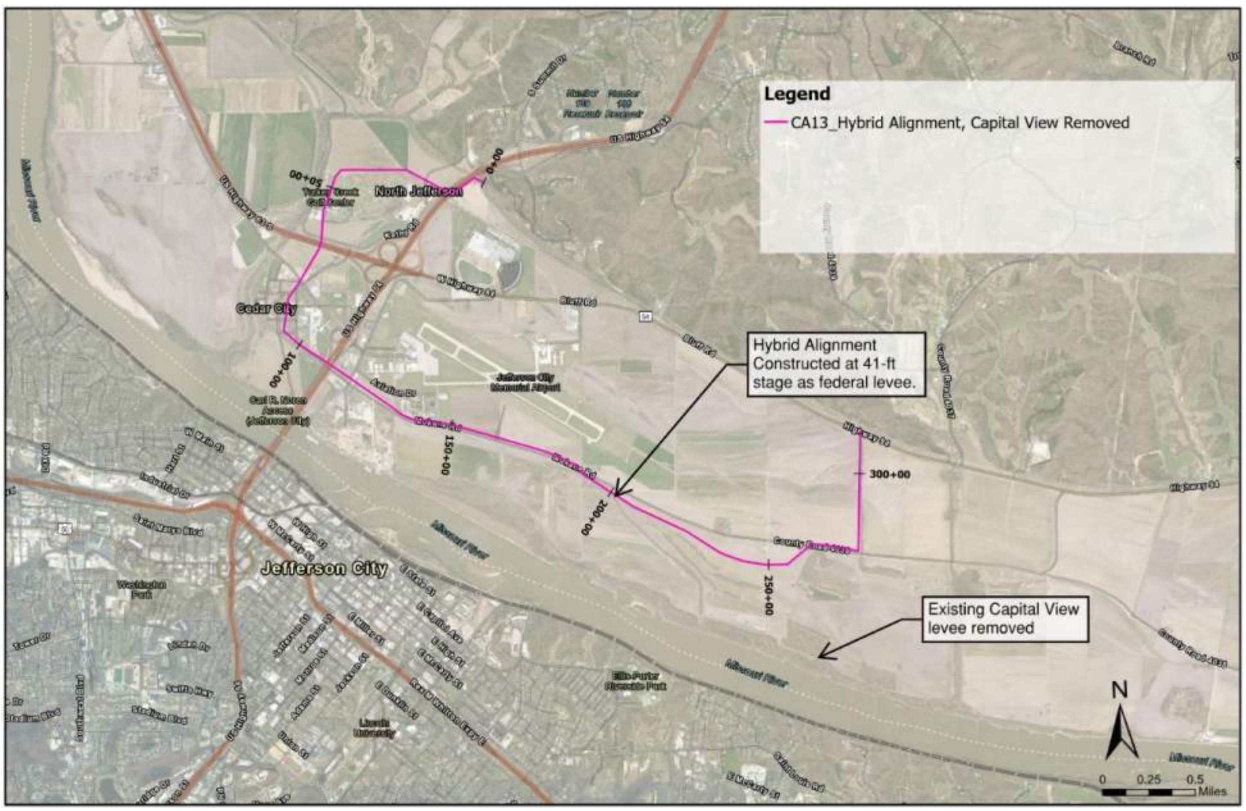

Figure 19. CA13 Hybrid Alignment, 41-ft Stage, Capital View Removed

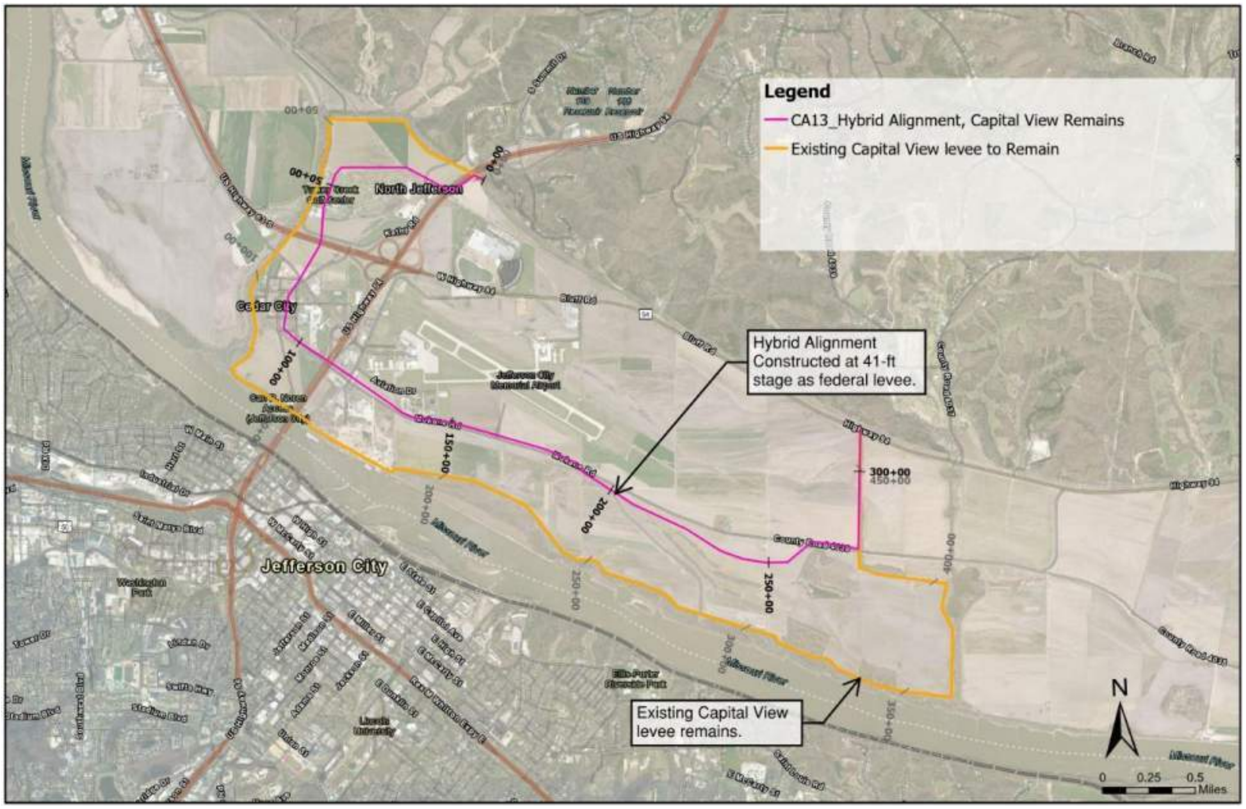

Figure 20. CA13 Hybrid Alignment, 41-ft Stage, Capital View Remains

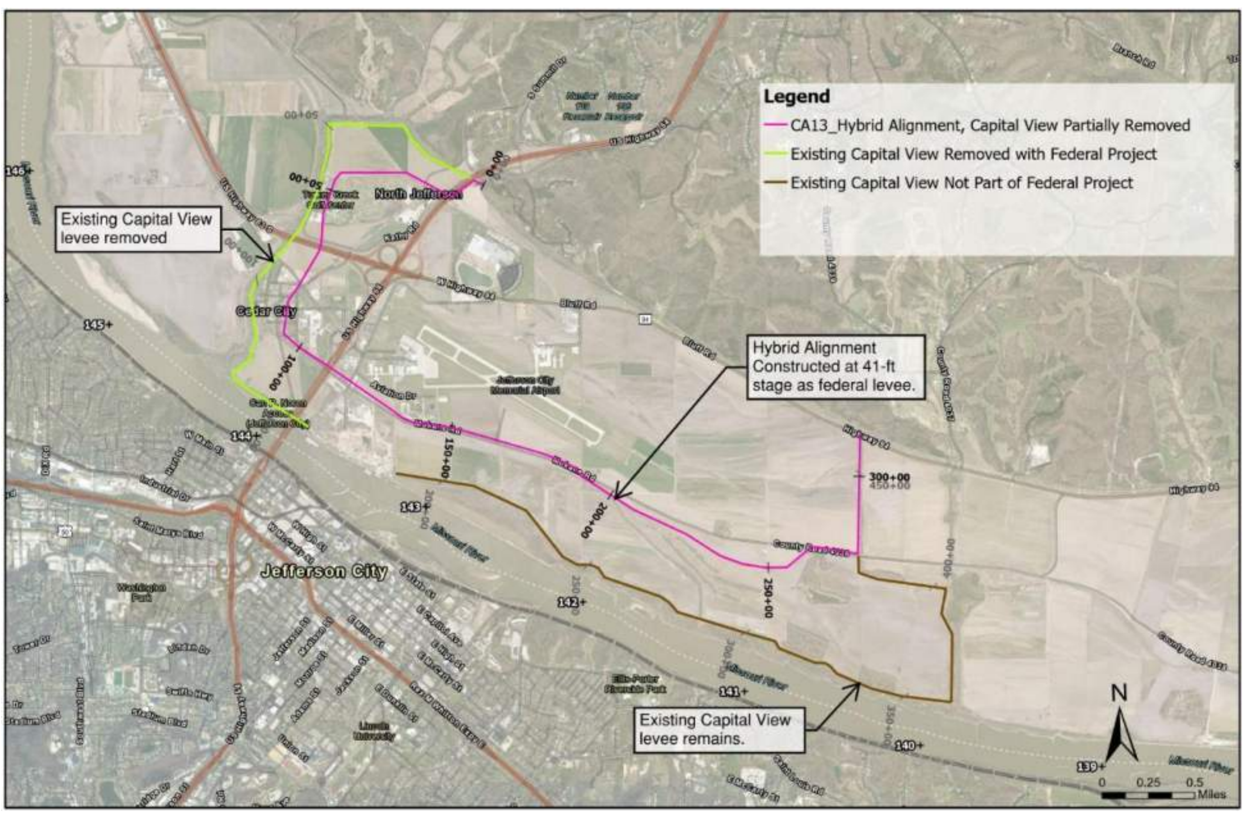

Figure 21. CA13 Hybrid Alignment, 41-ft Stage, Partial Capital View Removal

CA13 modeling parameters are provided in **Table 11**. Adaptation of the FWOP hydraulic model to reflect the alternative conditions was completed as outlined in Section 6.

Table 11. CA13 - Summary of Key Parameters

Model Parameter	Input
Targeted Design Performance	1% AEP overtopping, maximize levee height regarding economics while minimizing induced impacts with setback
Terrain File	LoMO_1m_NWK_JeffCity LoMO_1m_NWK_JeffCity_RemovedCV LoMO_1m_NWK_JeffCity_PartCV
Riverward Levee US Top Elev. (ft)	561.5
Riverward Levee DS Top Elev. (ft)	557.0

Constructing a levee with a 1% AEP event overtopping frequency results in decreased overtopping frequency for properties within the setback footprint. Reduced flood fighting and increased predictability in levee performance with a federal levee design would be anticipated, as the levee would be constructed to federal standards, include closure structures at roadways, and eliminate low points that require sandbagging in FWOP conditions. The levee setback would provide Turkey Creek with added conveyance area and minimize hydraulic pinch points that cause hydraulic stacking and localized water surface elevation increase at Turkey Creek in the FWOP condition.

The removal of Capital View levee would provide the Missouri River additional conveyance area, especially downstream of the bridge where flow actively expands after being constricted. Providing additional conveyance for the Missouri River allows hydraulic expansion to occur after exiting the bridge and would relocate infrastructure out of an area expected to require continuous maintenance due to repetitive stress. The setback footprint results in more frequent inundation for lands riverward of the levee alignment.

Community considerations include increased connectivity between the Jefferson City north and south banks during a wider range of flooding conditions, as low points in U.S. Highway 54 at the left bridge abutment, currently overtopping at 37-ft, would be located behind the levee. Levee overtopping would occur at a flood stage of 41-ft stage, corresponding to a 1% AEP overtopping level.

The alternative would not seek a levee design to meet minimum National Flood Insurance Program (NFIP) regulatory standards. Therefore, this project would not result in a request to reduce the leveed area’s flood risks associated with the 1-percent-annual-chance flood as delineated by the Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map (FIRM). Additional levee height would be needed to meet NFIP accredited levee standards, which is a concern of the local community and would have significant cost impact.

Without removal of the existing Capital View levee, additional Missouri River conveyance would not be realized to align with the Lower Missouri System Plan objectives. The levee setback at the Missouri River and Turkey Creek would mitigate increase in water surface elevation at the confluence of Wears Creek at the Missouri River, leading to a slight reduction in water surface elevation with the removal of the existing Capital View levee. Impacts to river velocities in the channel and at existing port locations are negligible, with velocity outputs within 0.1 to 0.3-ft/s of FWOP for all flow events.

CA13, Hybrid Alignment, 41-ft Stage with full or partial removal of the existing Capital View levee is carried forward to the final array of alternatives for consideration and public input. CA13 maintaining the existing Capital View levee in full is not carried forward as a final alternative after engineering analysis due to environmental, constructability, and hydraulic induced impact considerations.

8 CHANGES IN AREA INUNDATION PATTERNS AND HYDRAULICS

Changes in study area inundation patterns and hydraulics are assessed to understand the scale of impacts and/or betterments for the final array of alternatives in comparison with the No Action Alternative. Alternatives screened out due to reasons not including induced impact concerns are not evaluated below, including CA3 – Existing Capital View Alignment with Resiliency, CA10 – Limited Resiliency Measures Outside of Capital View Footprint, CA11 – Nonstructural, and CA12 – Remove Church Farm Levee.

This section summarizes preliminary analysis of changes in water surface elevation for comparison of alternatives and development of relative magnitude of impacts to compare alternatives. All modeled alternatives utilize the same assumptions as described in Section 6 to ensure a standard basis of comparison. A summary of findings is included below, further exhibits are included in Attachment D for all modeled frequency events.

8.1 Changes in Inundation Patterns for Alternatives Not Carried Forward to Final Array

Structural alternatives screened prior to the final study array include CA 1, 2, 4, 5, 6, 7, 8, and 9. These alternatives result in similar changes in inundation patterns as the structural measures are very similar. Each alternative proposes a federal levee setback from the Missouri River. Iterations include height, setbacks from Turkey Creek, and intention of Capital View levee.

Generally, alternatives with greater setbacks and removal of smaller levee systems result in less hydraulic impacts to adjacent properties and tributary backwater conditions. **Table 12** summarizes changes in maximum water surface elevation of the Missouri River profile at the Wears Creek confluence.

Table 12. Difference in Maximum WSE at Wears Creek Confluence for Screened Alternatives

	10% (1/10 YR)	1% (1/100 YR)
Max WSE FWOP Elevation (ft NAVD88)	552.6	557.9
Δ CA1 Existing Capital View Alignment, 34-ft Stage (Δ ft)	0.4	0.2
Δ CA2 Turkey Creek & Riverside Setback, 34-ft Stage, Capital View Removed (Δ ft)	0.0	0.1
Δ CA4 Optimization Alignment, 37-ft Stage, Capital View Remains (Δ ft)	0.1	0.5
Δ CA5 Optimization Alignment, 37-ft Stage, Capital View Removed (Δ ft)	0.0	0.4
Δ CA6 Historic L-142 Alignment, 45-ft Stage, Capital View Remains (Δ ft)	0.1	0.5
Δ CA7 Historic L-142 Alignment, 45-ft Stage, Capital View Removed (Δ ft)	-0.2	0.4
Δ CA8 Highway Alignment, 37-ft Stage, Capital View Remains (Δ ft)	0.1	0.6
Δ CA9 Highway Alignment, 37-ft Stage, Capital View Removed (Δ ft)	-0.1	0.5

8.2 CA0 No Action

The no action alternative has impacts equivalent to existing conditions. No impacts, negative nor positive, would be observed with adoption of this alternative. The evaluation assumes that the existing levee is maintained to current standards at its current height and location, the surrounding floodplain remains hydraulically the same, and the existing Flood Action Plan is implemented as adopted to close low points at Oilwell Road and Cedar Creek Drive at Turkey Creek. **Figure 22** and **Figure 23** show the inundation in the FWOP condition through the study area for the 10% AEP storm and 1% AEP storm respectively.

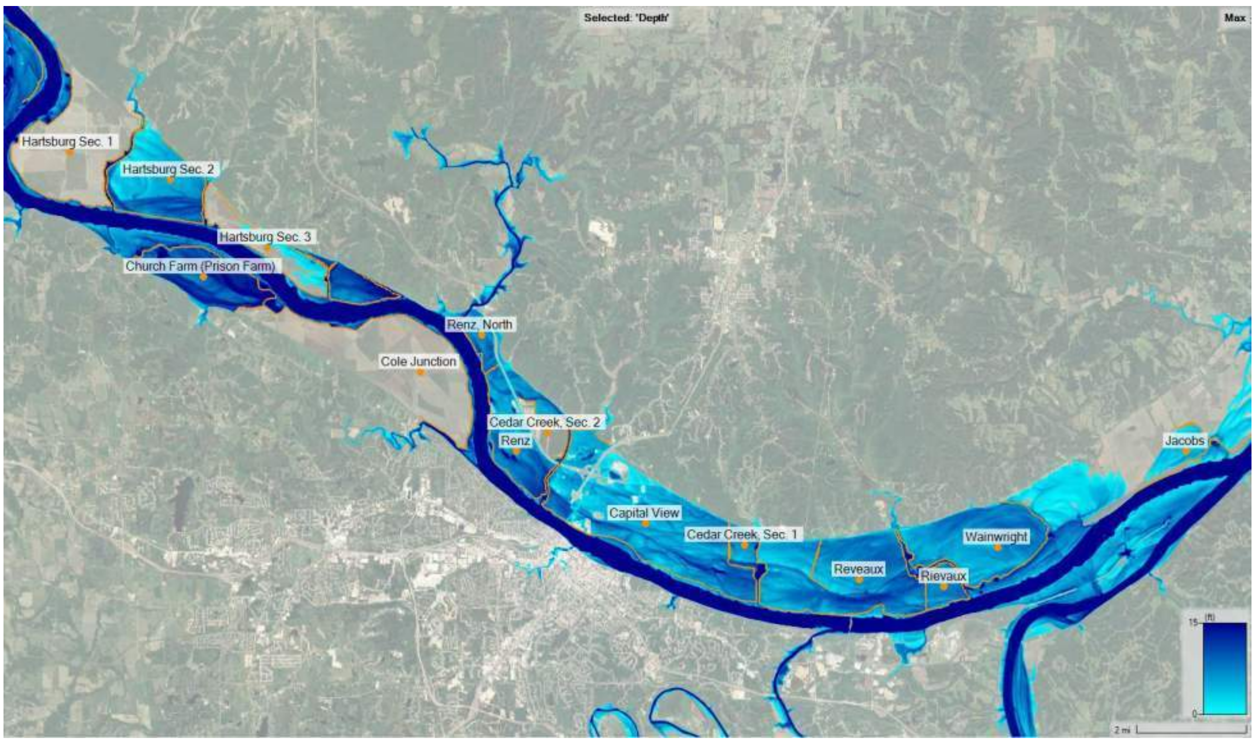

Figure 22. CA0 No Action, Inundation Limits and Depth – 10% AEP

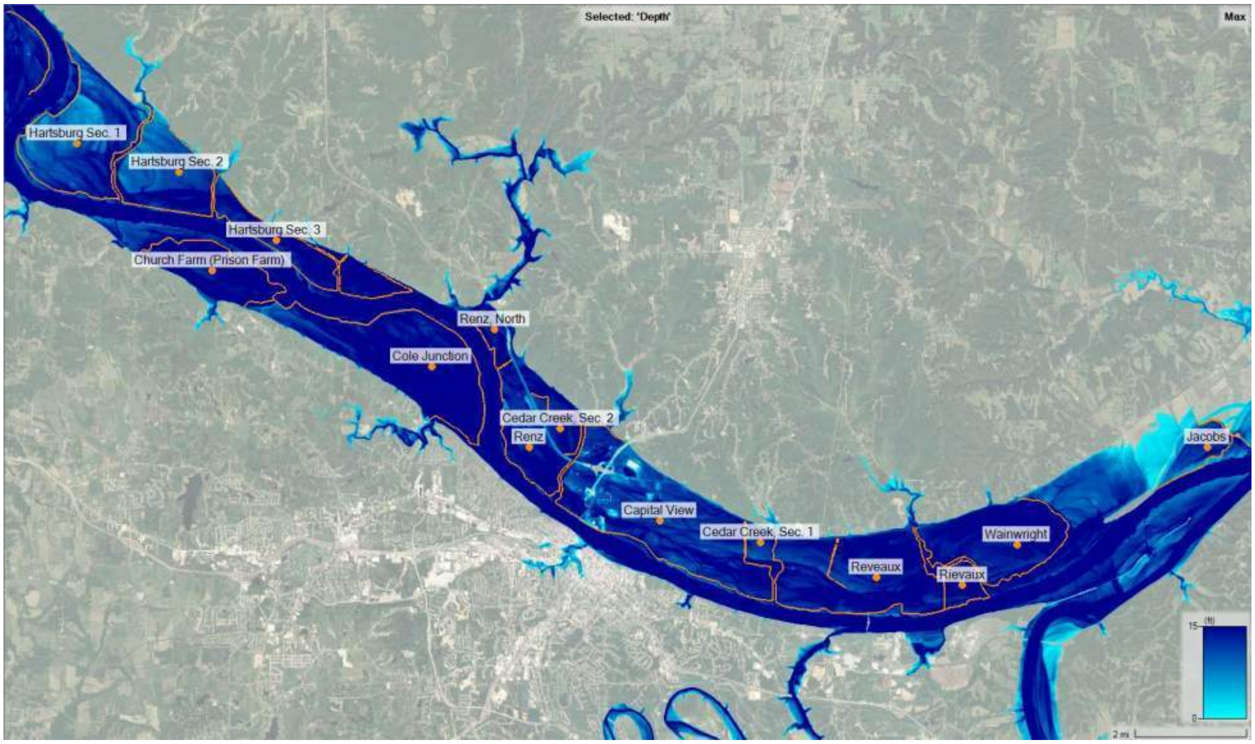

Figure 23. CA0 No Action, Inundation Limits and Depth – 1% AEP

In the future without project condition, full-bank capacity of the Missouri River is reached during a 50% AEP event. The non-federal levees within the study extents withstand a 20% AEP event, and nearly all are overtopped and inundated during a 10% AEP event with the exception of Cedar Creek Section 2, Cole Junction, and Hartsburg Section 1. This corridor of levees is generally constructed at the bank of the Missouri River, resulting in greater magnitude flows activating full bluff-to-bluff flow through this Lower Missouri River stretch. Due to the bluff restriction on each side, greater flow volumes generally result in deeper inundation of the floodplain. Upon first inundation of leveed areas, velocities are low due to the levee causing a ponding effect. As the water depth increases, the effect of the levee on the velocity in the channel cross section diminishes, as water can freely flow above the levee elevation, leading to activation of more and more of the cross-sectional area for active conveyance (non-zero velocity) capacity.

8.3 CA13 Hybrid Alignment, 41-ft Stage, Capital View Removed

CA13 was carried to the final array of alternatives for analysis. The CA13 alignment is considered without the existing Capital View levee and with partial removal of the existing Capital View levee.

CA13 includes a levee height set at 1% AEP of overtopping, setback from the Missouri River channel and Turkey Creek tributary. The L-142 levee would remain elevated above adjacent levees which overtop prior to the 10% AEP event, observed in **Figure 24**. With the incorporation of setbacks, upstream adjacent levees maintain an AEP of overtopping equivalent to FWOP conditions within 0.2%; or slightly improve their FWOP condition with improved conveyance. Detailed impacts to adjacent levees are included in Section 8.5.1. Adjacent agricultural levee systems downstream would maintain AEP of overtopping similar to FWOP conditions, with Cedar Creek Section 1 AEP increasing from 21.4% to 24.0% AEP, Reveaux increasing from 14.2% to 15.4% AEP, Wainwright increasing from 20.8% to 21.2% AEP, and Rievaux seeing a slight decrease in AEP of overtopping.

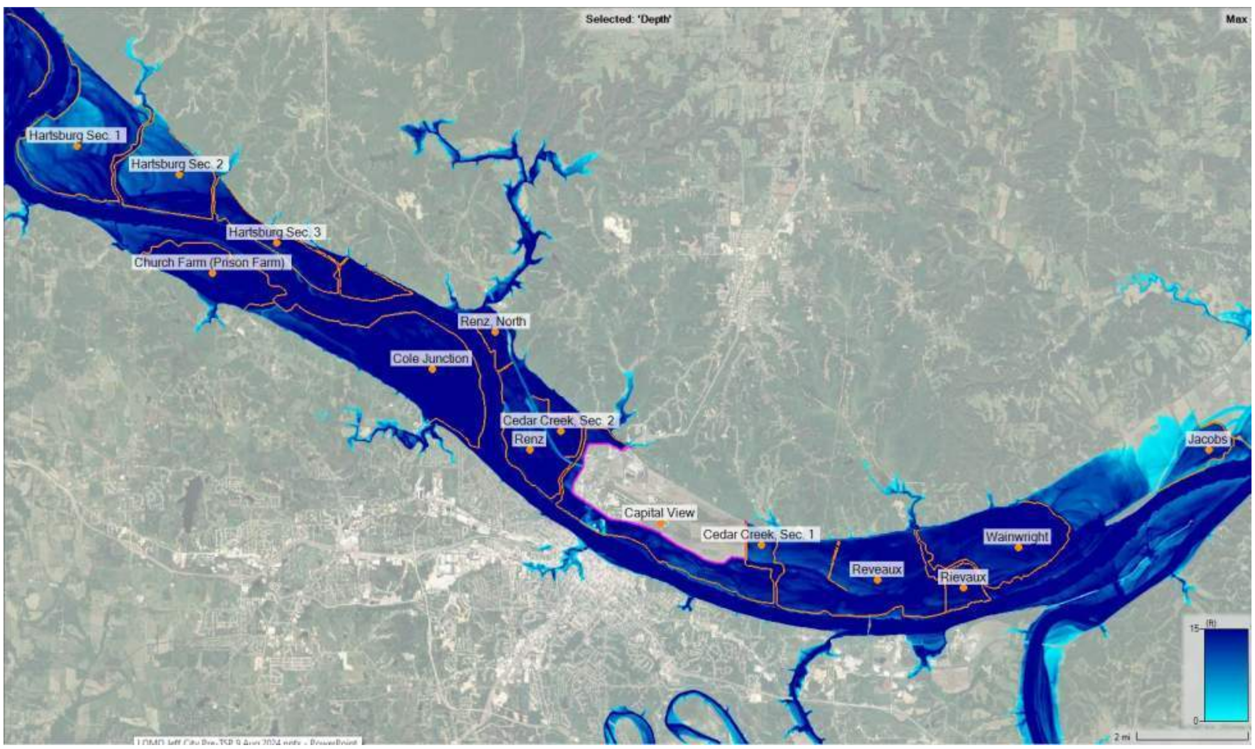

Figure 24. CA13 Hybrid Alignment, Inundation Limits and Depth – 1% AEP

All modeled frequency events were simulated to analyze changes in hydraulic patterns, inundation depths, and inundation duration. All results comparing CA13 to the FWOP condition are included in Attachment D. **Figure 25** and **Figure 26** detail changes in maximum water surface elevation for the 10% AEP and 1% AEP events respectively. With CA13 and removal of the existing Capital View levee system, additional connection of the overbank to the Missouri River channel allows for conveyance improvements, decreasing water surface elevations at Turkey Creek and maintaining Missouri River WSE at Wears Creek in the 10% AEP event. During the 1% AEP event, the Missouri River floodplain is inundated bluff-to-bluff in FWOP conditions. Due to the vertical nature of the floodplain boundary through Jefferson City, while there are local increases in water surface elevations during the 1% AEP event with CA13, there is limited inundation extent expansion in total acreage.

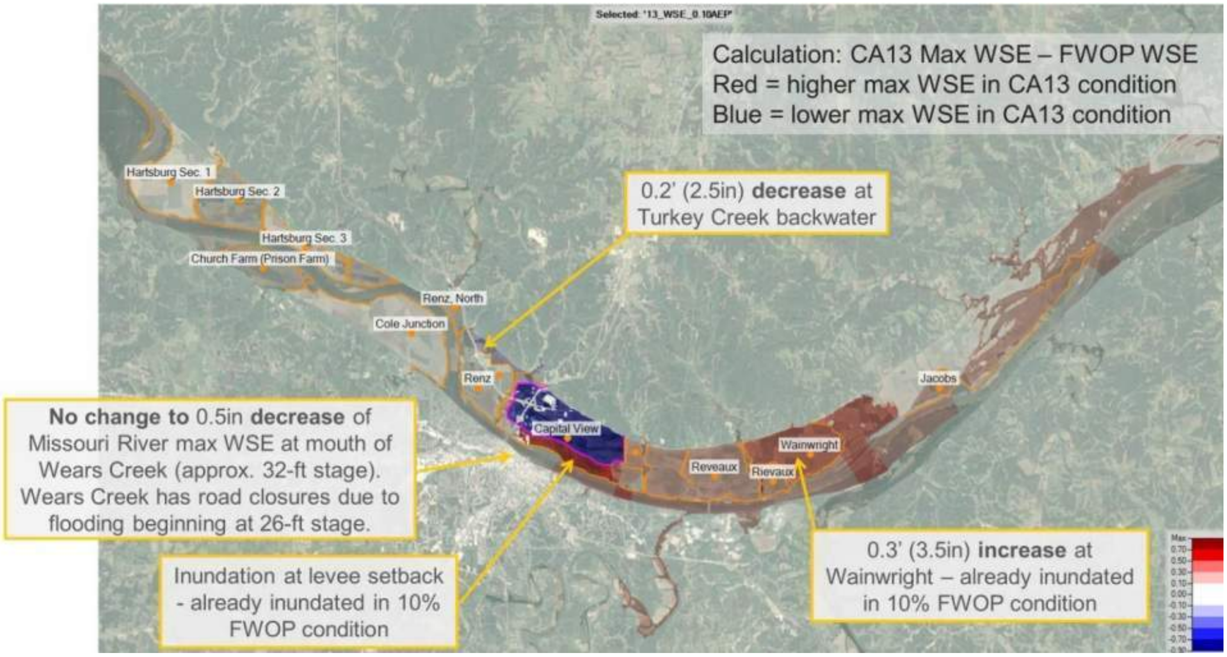

Figure 25. Change in Max. WSE Comparing CA13 to FWOP Condition – 10% AEP

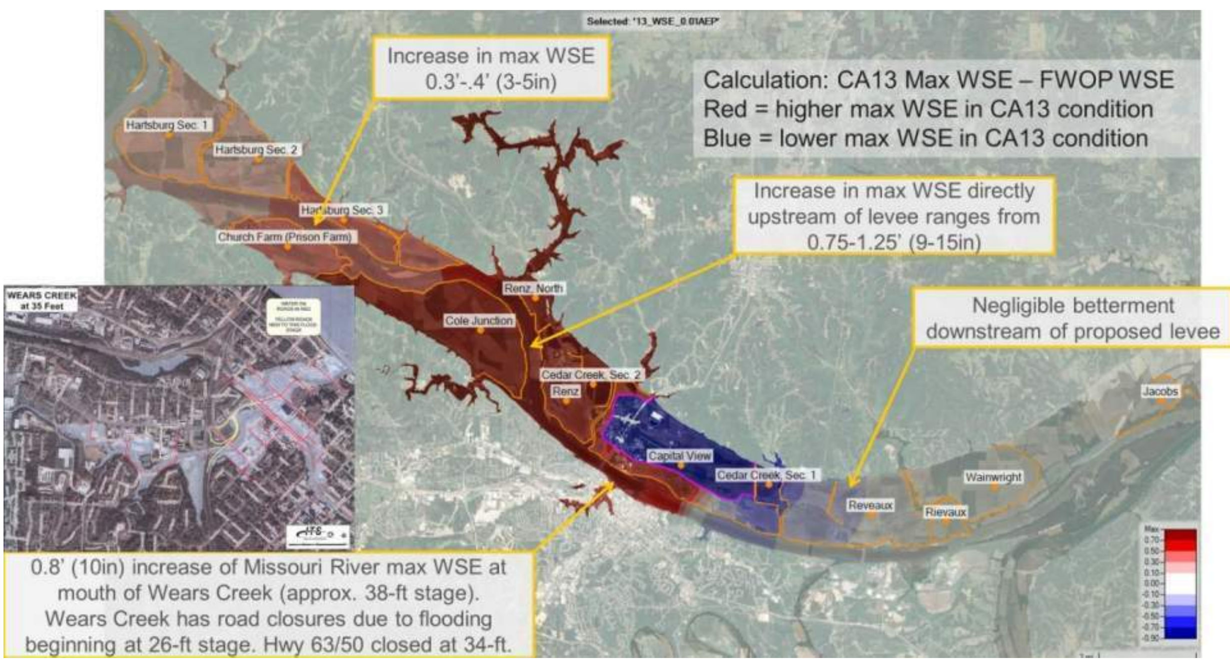

Figure 26. Change in Max. WSE Comparing CA13 to FWOP Condition – 1% AEP

Due to the depths of inundation in the FWOP conditions, a more descriptive impact indicator for change in water surface elevation is the percentage of change in water depth, mapped in **Figure 27**. For example, during a 1% AEP event, inundation depths at adjacent levees average 15 to 16 feet. The increase in water surface elevation with CA13 results in a 9% increase in depth directly upstream of the L-142 levee, with increases in maximum water surface elevation resolving upstream of Hartsburg levee district.

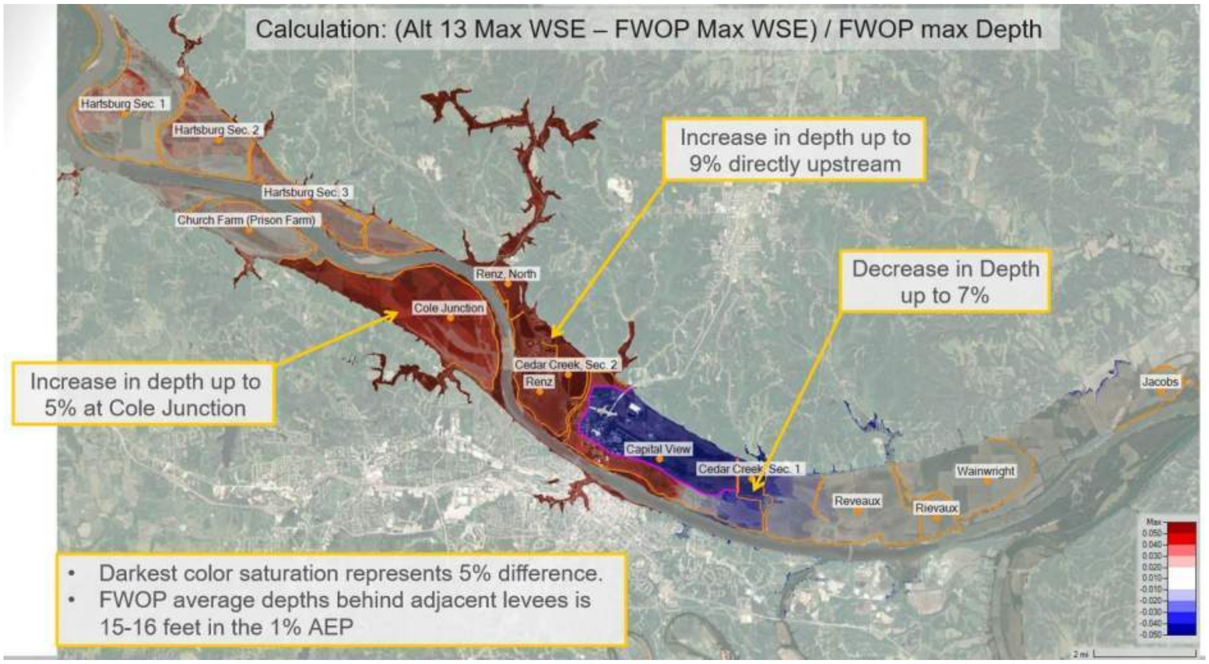

Figure 27. Percent Change in Maximum Depth Comparing CA13 to FWOP Condition – 1% AEP

A summary of Missouri River channel water surface profiles comparing CA13 with full removal of the existing Capital View levee to the FWOP condition is included in **Figure 28**.

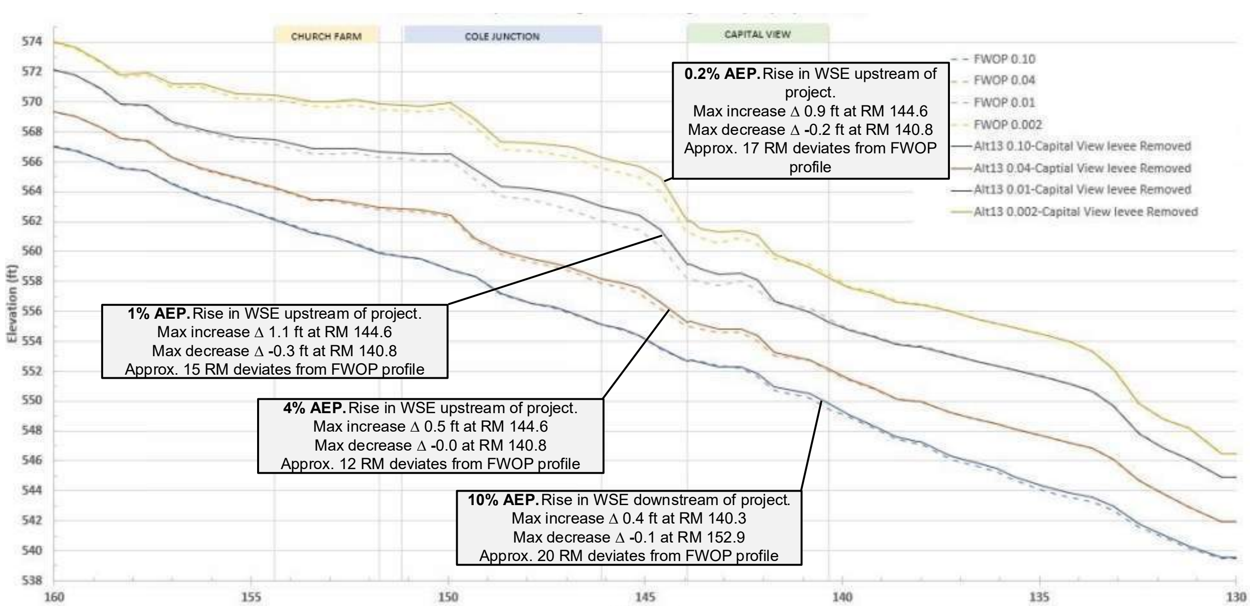

Figure 28. Missouri River Channel Maximum WSE Profile – CA13 with Full Removal of Capital View levee

8.4 CA13 Hybrid Alignment, 41-ft Stage, Partial Capital View Removal

Partial removal of the existing Capital View levee including full removal of the existing levee upstream of the U.S. Highway 54 / 63 overpass as part of the federal project has similar impacts to the full removal of the levee. However, beneficial reductions in water surface elevation during frequent flow events are not realized.

Figure 29 and **Figure 30** detail changes in maximum water surface elevation compared to the FWOP condition for the 10% AEP and 1% AEP events respectively. With CA13 and partial removal of the existing Capital View levee system, a setback relief at Turkey Creek reduces impacts of the taller levee structure to adjacent levee systems. An increase in the Missouri River WSE at Wears Creek is seen in the 10% and 1% AEP events. During the 1% AEP event, the Missouri River floodplain is inundated bluff-to-bluff in FWOP conditions. Due to the vertical nature of the floodplain boundary through Jefferson City, while there are local increases in water surface elevations during the 1% AEP event with CA13, there is limited inundation extent expansion in total acreage. Due to the depths of inundation in the FWOP conditions, a more descriptive impact indicator for change in water surface elevation is the percentage of change in water depth, mapped in **Figure 31**.

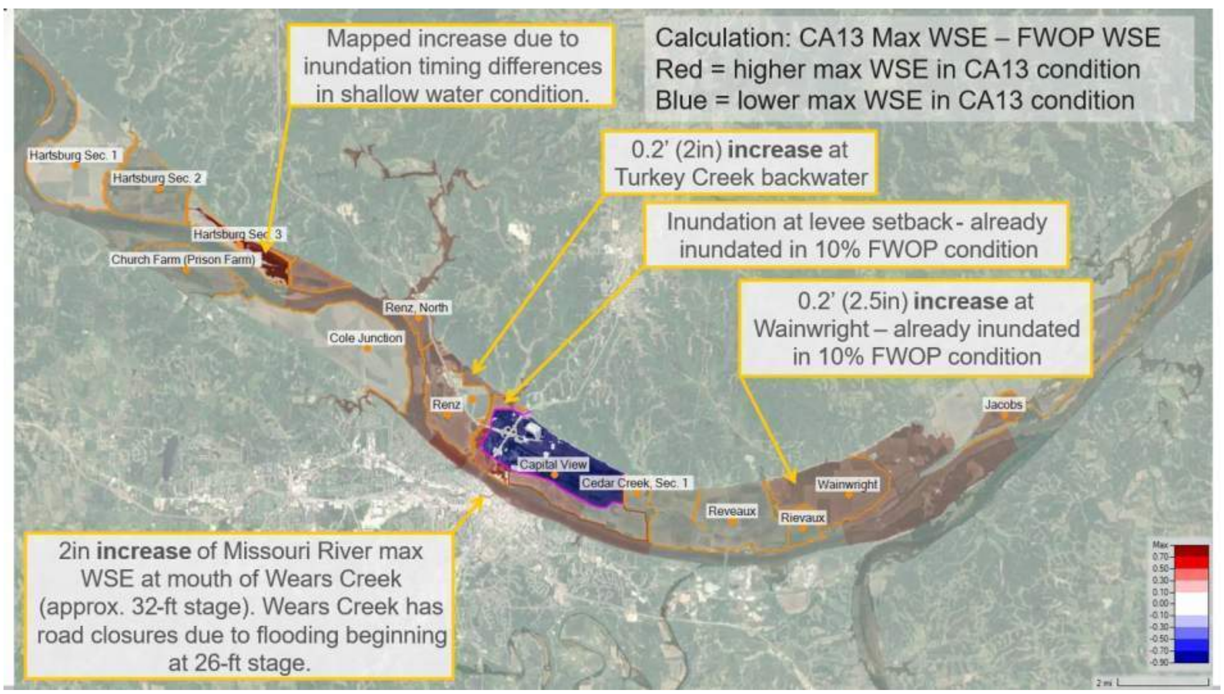

Figure 29. Change in Max. WSE Comparing CA13 with Partial Removal of Capital View levee to FWOP Condition - 10% AEP

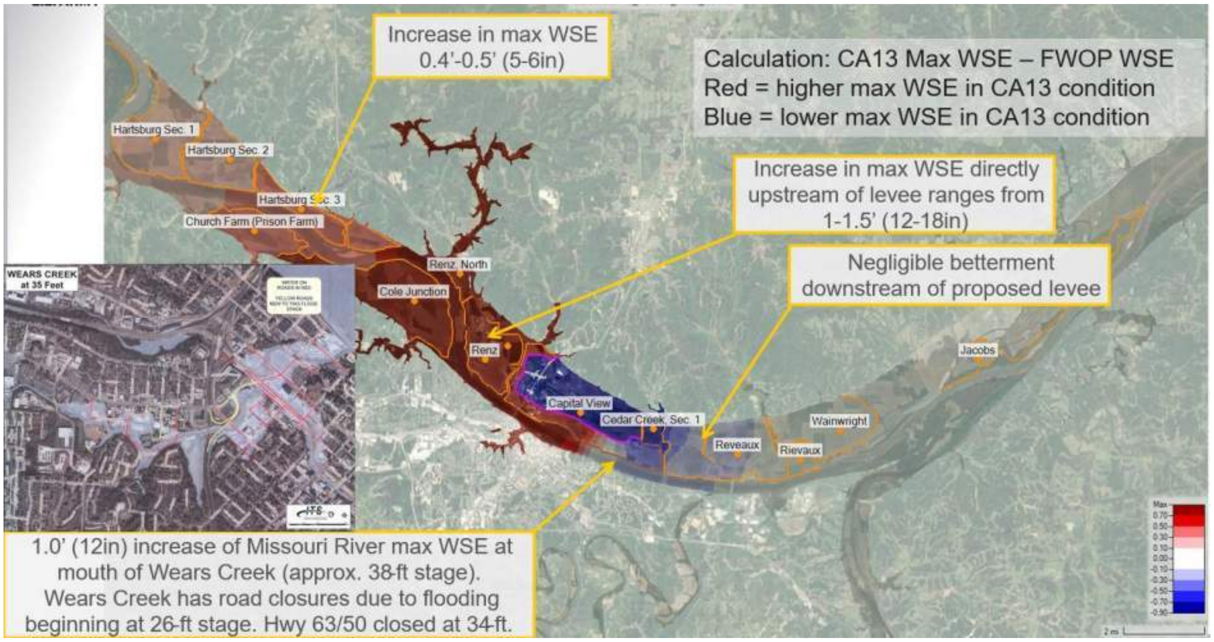

Figure 30. Change in Max. WSE Comparing CA13 with Partial Removal of Capital View levee to FWOP Condition – 1% AEP

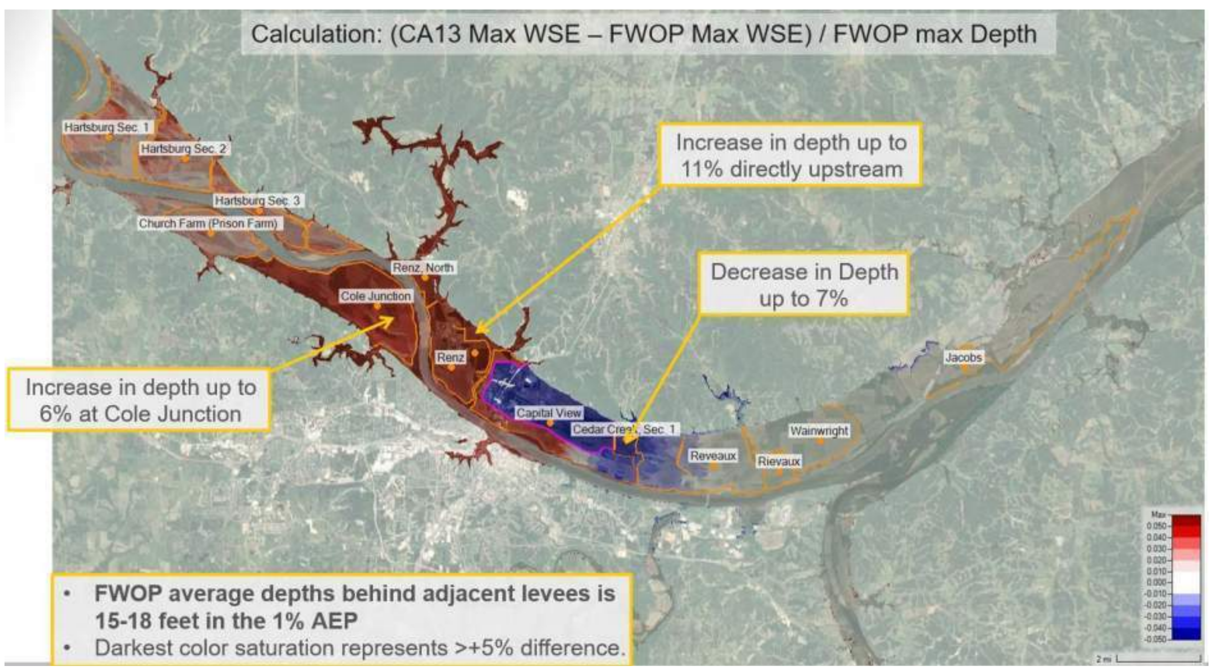

Figure 31. Percent Change in Maximum Depth Comparing CA13 with Partial Removal of Capital View levee to FWOP Condition – 1% AEP

A summary of Missouri River channel water surface profiles comparing both CA13 with full removal of the existing Capital View levee and partial removal of the existing Capital View levee to the FWOP condition is included in **Figure 32**.

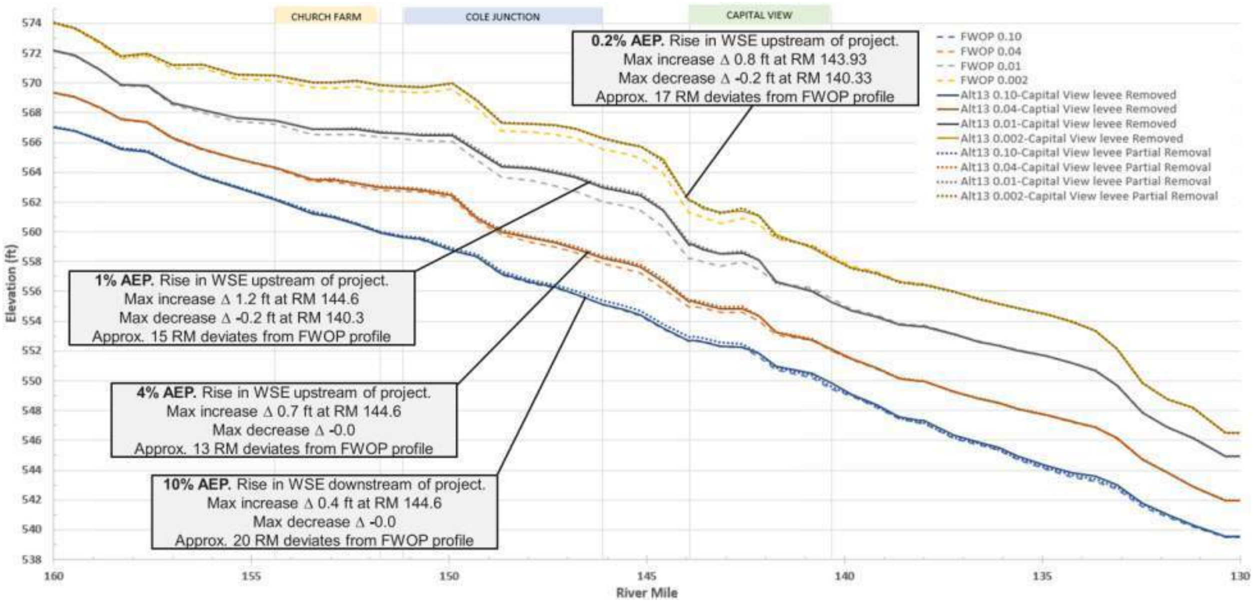

Figure 32. Missouri River Channel Maximum WSE Profile – CA13 with Partial Removal of Capital View Levee

8.5 Changes in Project Area Inundation Patterns Additional Discussion

Additional analysis was considered with the full development of project alternatives including special attention to adjacent stakeholders including adjacent levee systems, the Jefferson City right-bank, and river velocities. In all alternatives analyzed, Missouri River velocities are unchanged in order of magnitude from FWOP conditions, increasing 0.1-0.4 ft/s locally. Additional attention was given to ensuring changes in velocities around the Capital Sand port were consistent or below differences in main channel velocities.

8.5.1 Adjacent Levees

Adjacent levee system initial overtopping times developed with the FWOP model were documented for and compared to the CA13 modeling conditions, included as **Table 13**. These values were then utilized to determine the AEP flow at the Jefferson City gage to compare the effects of the alternative conditions on adjacent levee systems. The general trend, as anticipated, is removal of the existing Capital View levee provides additional conveyance riverward of levees, resulting delayed in overtopping in comparison to the FWOP condition.

Table 13. Adjacent Levee AEP of Overtopping Comparison between FWOP and CA13 Conditions

UPSTREAM LEVEE OVERTOPPING FLOWS AT JEFF CITY GAGE AND ASSOCIATED AEP												
	FWOP			Alt 13 Capital View Removed				Alt 13 Capital View Partial Removal				
	% AEP	(1/# YR) AEP	cfs	% AEP	(1/# YR) AEP	cfs	Δ from FWOP	% AEP	(1/# YR) AEP	cfs	Δ from FWOP
RENZ	17.3	5.8	299,000	16.4	6.1	302,000	-0.9	17.3	5.8	299,000	0
CEDAR CREEK SEC. 2	5.3	18.9	397,000	5.6	17.9	391,000	0.3	5.8	17.2	387,000	0.5
RENZ NORTH	21.6	4.6	281,000	21.1	4.7	283,000	-0.5	21.6	4.6	281,000	0
COLE JUNCTION	6.1	16.4	383,000	6.2	16.1	381,000	0.1	6.3	15.9	379,000	0.2
BOONE CO, MO 1	21.4	4.7	281,000	21	4.8	284,000	-0.4	21.4	4.7	281,000	0
CHURCH FARM (PRISON FARM)	14.4	6.9	312,000	14.6	6.8	311,000	0.2	15.0	6.7	309,000	0.6
HARTSBURG	17.6	5.7	298,000	18.8	5.3	298,000	1.2	17.7	5.6	297,000	0.1
DOWNSTREAM LEVEE OVERTOPPING FLOWS AT JEFF CITY GAGE AND ASSOCIATED AEP												
	FWOP			Alt 13 Capital View Removed				Alt 13 Capital View Partial Removal				
	% AEP	YR AEP	cfs	% AEP	YR AEP	cfs	Δ from FWOP	% AEP	YR AEP	cfs	Δ from FWOP
CEDAR CREEK SEC. 1	21.8	4.6	279,000	24.7	4.0	265,000	2.9	22.5	4.4	275,000	0.7
REVEAUX	14.4	6.9	312,000	15.6	6.4	306,000	1.2	15.3	6.5	307,000	0.9
RIEVAUX	20.7	4.8	286,000	20.4	4.9	287,000	-0.3	20.7	4.8	286,000	0
WAINWRIGHT	21.4	4.7	281,000	21.2	4.7	283,000	-0.2	21.5	4.7	281,000	0.1

** Green highlighted cells are an improvement (less frequent) in overtopping frequency with Alternative construction vs FWOP condition.*

8.5.2 Wears Creek

Impacts to Wears Creek, on the right bank of the Missouri River, are expressed as differences in maximum water surface elevation of the Missouri River at the confluence of Wears Creek. Appendix A1, Part 1 summarizes findings of previous studies conducted to investigate solutions to flooding within the Wears Creek watershed. The City of Jefferson City has conducted efforts to reduce repetitive losses due to flooding in the Wears Creek watershed, however public input and the City's Flood Action Plan confirm flooding is an ongoing concern. **Table 14** summarizes the changes in water surface elevation at the mouth of Wears Creek for various Missouri River flood frequencies with and without project implementation.

Table 14. Change in Max. WSE from FWOP Condition with CA13

AEP (%) Flow Event	50% (1/2 YR)	20% (1/5 YR)	10% (1/10 YR)	4% (1/25 YR)	2% (1/50 YR)	1% (1/100 YR)	0.5% (1/200 YR)	0.2% (1/500 YR)
Max WSE FWOP Elevation (ft NAVD88)	544.0	549.9	552.6	554.8	557.1	557.9	559.7	560.9
Change from FWOP with CA13 Capital View Removed (Δ ft)	0.0	-0.3	0.0	0.3	0.8	0.8	0.8	0.6
Change from FWOP with CA13 Capital View Partial Removal (Δ ft)	0.0	0.0	0.2	0.4	0.9	1.0	1.0	0.8

In existing conditions, road closures in the Wears Creek floodplain due to Missouri River flooding begin at the 26-foot river stage, corresponding to an annual exceedance probability between 50% and 20%. The U.S. Highway 63 / 50 corridor through the Wears Creek floodplain is closed at a 34-foot Missouri River stage, corresponding to an annual exceedance probability of roughly 2%.

Due to the frequent inundation of the Wears Creek floodplain, alternative development focused on relieving Missouri River maximum water surface elevation profiles at the confluence of Wears Creek for the most frequent events. This can be accomplished with setting existing levees back on the left-bank. Increases in water surface elevations at the mouth of Wears Creek for CA13 do occur for less frequent, higher volume storm frequencies. For the same storm events in the FWOP condition, there is already extensive local flooding. Storm exceedance probabilities exceeding a 2% AEP fully implement the city's existing flood action plan.

A separate 1D steady state HEC-RAS model for Wears Creek, utilized for the development of the Jefferson City flood insurance study, was used for the Jefferson City L-142 study to determine how far upstream effects from the Missouri River backwater would likely percolate with changes to the Missouri River profile. Wears Creek flooding originates from both Missouri River backwater and local drainage, with a coincident low frequency storm event on Wears Creek very unlikely to occur. Water surface elevations at the downstream boundary condition were set to mimic Missouri River a maximum WSE for the FWOP and CA13 – Partial Capital View Removal conditions for both a Missouri River driven flood and a coincident flood scenario where both the Missouri River and Wears Creek experience flood event peaks at similar times. This modeling allowed for comparison of the Wears Creek backwater elevations for both the FWOP condition and the CA13 – Partial Capital View Removal condition to determine where impacts would resolve. As observed in **Figure 33**, the increased water surface observed at the mouth of Wears Creek will resolve to the FWOP condition elevations within 2.2 miles of the mouth of Wears Creek, near the intersection of Southwest Boulevard, in the worst flow condition case. Attachment D provides further context for hydraulic changes anticipated with implementation of CA13 and comparison with FWOP Wears Creek conditions.

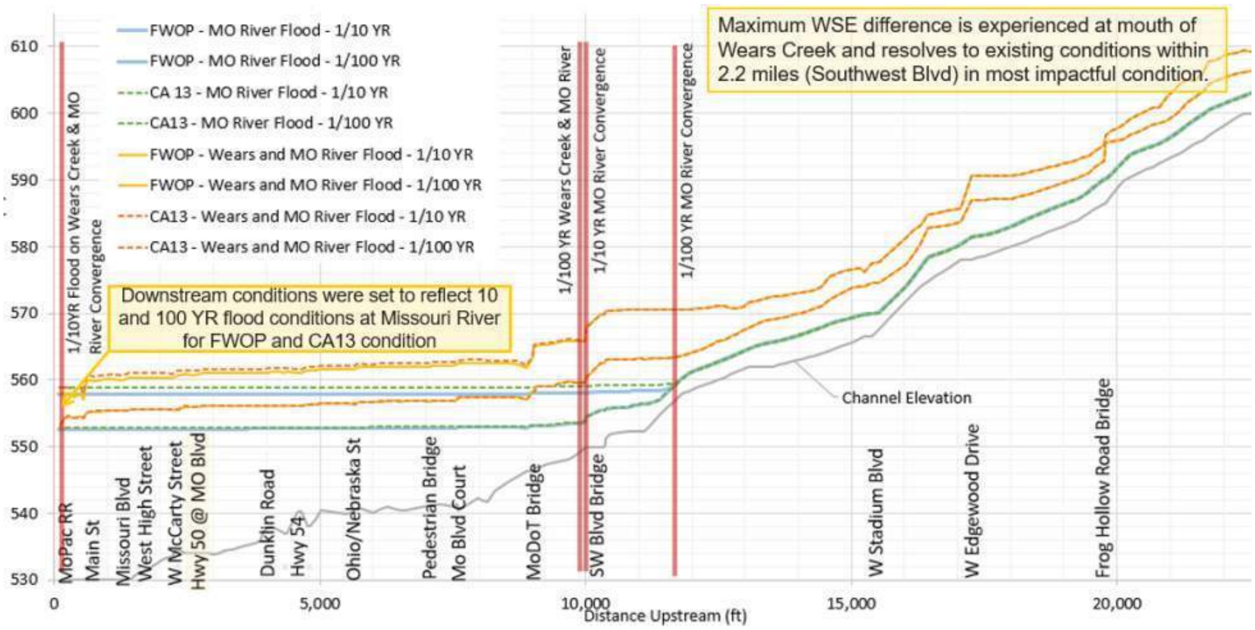

Figure 33. Wears Creek Upstream Impacts – CA13 Partial Capital View Removal

8.5.3 Flow Velocities

Flows are most restricted in the Jefferson City corridor through the U.S. Highway 54 / 63 bridge abutments and adjacent high ground at the Capital Sand plant. A cross section of this location at river mile 143.58 is shown in **Figure 34** with the resulting velocity distribution observed for the 0.002 AEP flow event. Other flow events have similar velocity distributions, with various activation of the overbanks for conveyance. As seen in the figure, velocities of the Missouri River through the study area are greatest in the center of the channel where they have minimal friction influence. The velocity through the overbanks is generally quite slow due to friction, variability in the terrain, and much shallower flow profiles.

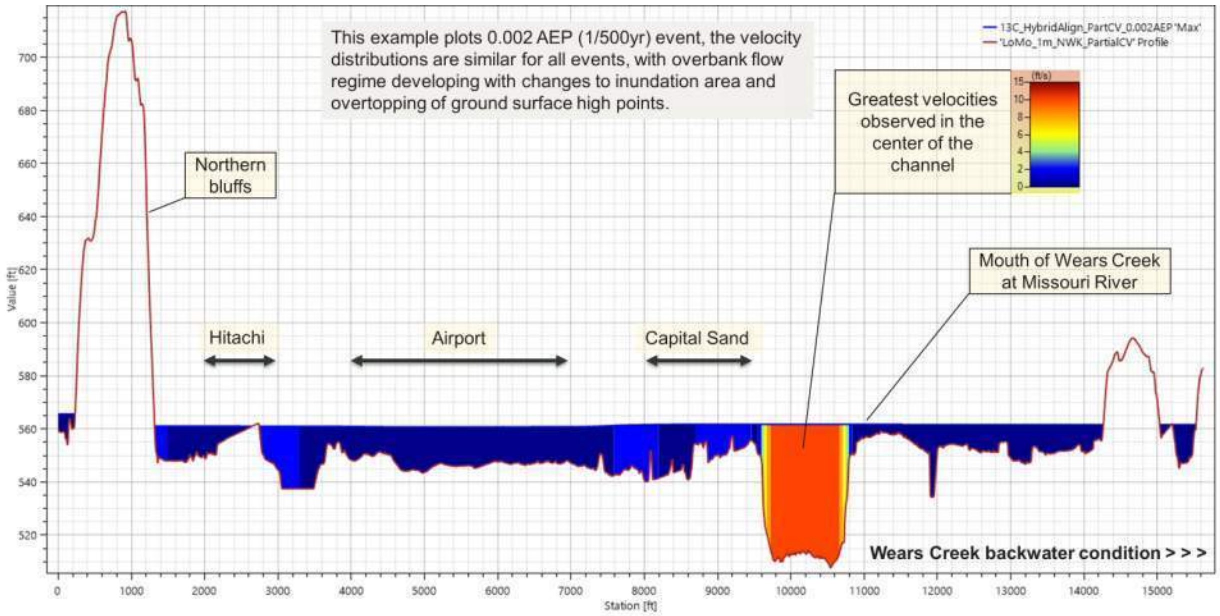

Figure 34. Velocity Cross Section at River Mile 143.58

A summary of changes to the maximum main-channel velocities with implementation of CA13, in comparison to FWOP conditions, are provided in **Figure 35** and **Figure 36**. Trends are similar across all storm event frequencies, with the 10% and 1% AEP storms provided below. In general, the addition of a federal levee with CA13 would minorly increase velocities in the study area, with changes still within the range observed in the FWOP condition and resolving near the Wainwright levee. Upstream of the federal levee, maximum velocities would be expected to negligibly decrease compared to FWOP conditions, resolving near the Hartsburg levees.

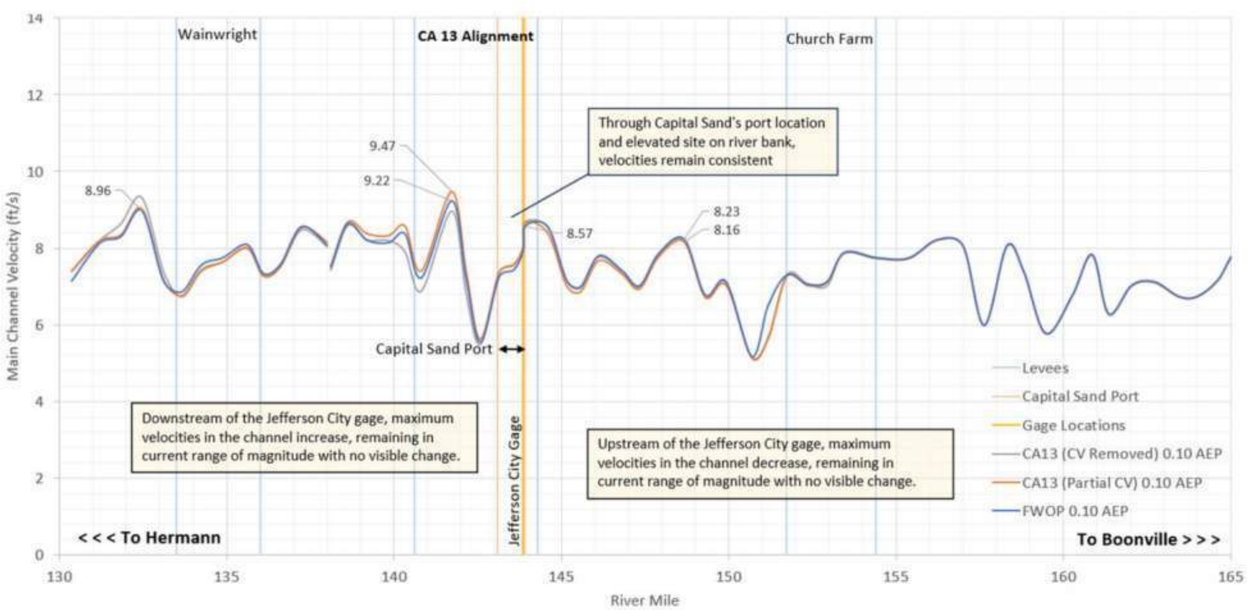

Figure 35. Velocity in Channel at Maximum Water Surface Elevation – 10% AEP

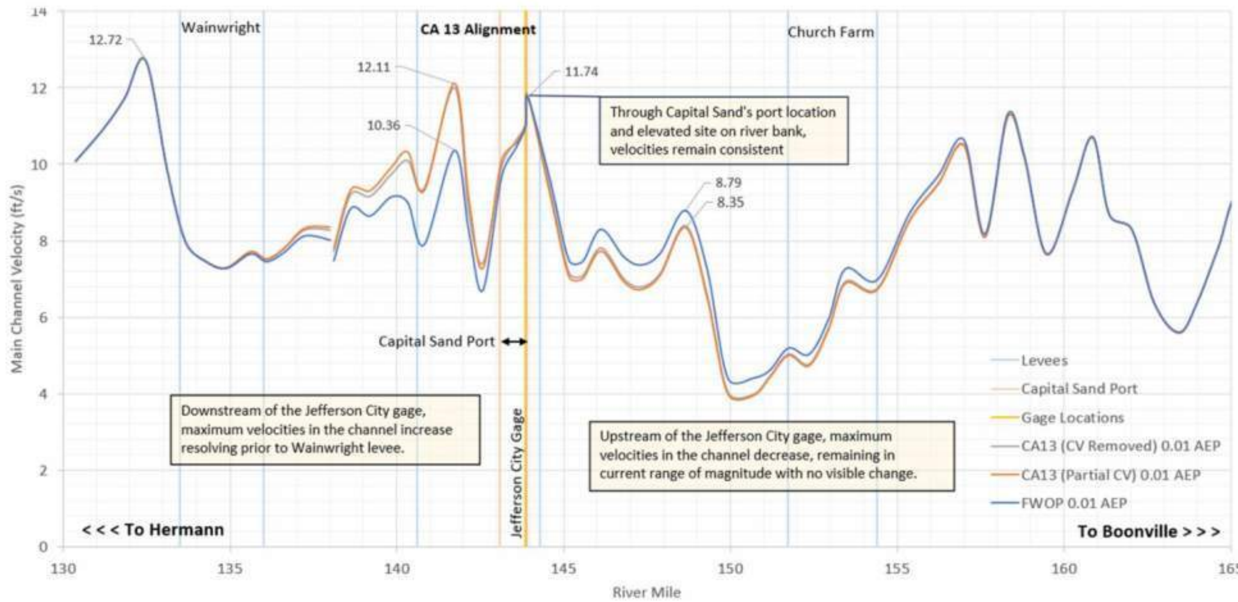

Figure 36. Velocity in Channel at Maximum Water Surface Elevation – 1% AEP

A comparison of FWOP and CA13 velocities in the main channel between Wainwright levee and Cole Junction levee is shown in **Figure 37**. This box and whisker plot represents the median and quartile velocity ranges experienced in both the FWOP and CA13 condition, confirming the range of high velocity flows in the immediate study area would be similar with implementation of CA13 when compared to the FWOP condition for the full range of flood flows.

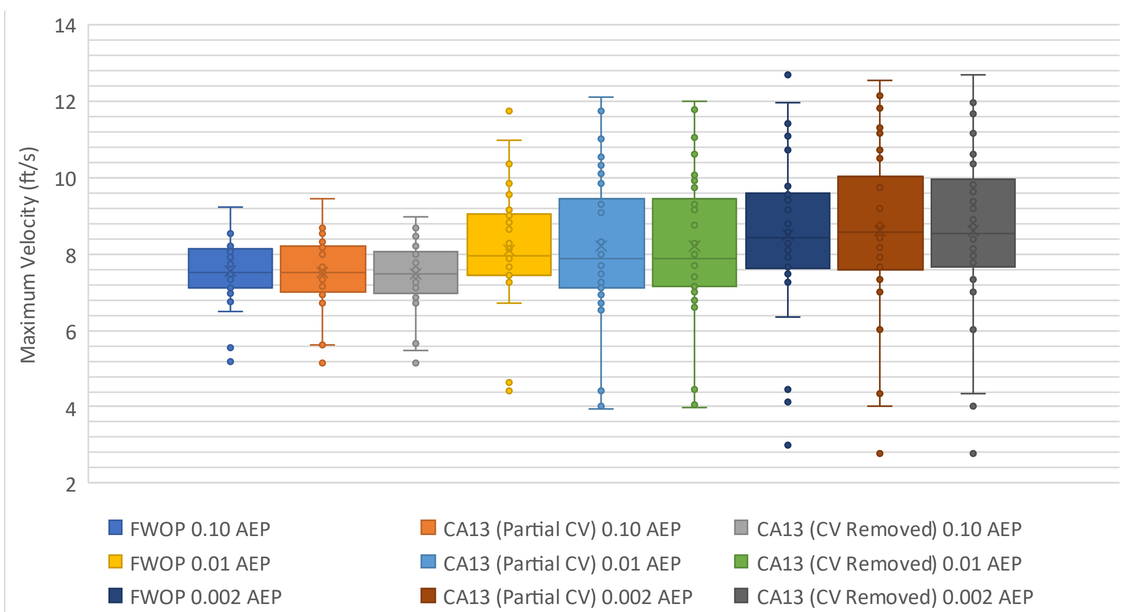

Figure 37. Comparison of Maximum In-Channel Velocity Outputs between River Miles 133 and 151

With the implementation of CA13, there would be no change in day-to-day operational velocities, including for events similar in magnitude to the 2019 flood (10% AEP). The Missouri River experiences a range of velocities throughout its full river profile, with pockets of variation as geographic features restrict and expand the conveyance cross section. With the adoption of a structural alternative, velocities remain within the range of velocities experienced through the study site and the proposed velocities still fit within the existing system dynamics experienced in the FWOP condition.

8.6 Impacts Discussion

Modeling impacts using an unsteady flow analysis is dependent upon assumptions of flow hydrograph shape and levee performance. Hydrographs are based on the shape of the 2018 flood and multiplying flows up or down to meet the peak flow rates of various frequency floods based on methodology from EM 1110-2-1415. Hydrograph shapes from other events could result in longer or shorter duration of high flows than as assumed and could potentially alter results by filling more or less of the leveed areas in smaller events that do not fully inundate the floodplain. Levee breach was assumed with 1 foot of overtopping, whereas the previous 2004 UMRSFFS assumed breach occurs at or below top of levee which would tend to estimate lower stages than this study. However, breach could occur earlier, later, or not occur during a given flood event.

For duration analysis, areas landward of levees are complex to model. After overtopping, as the flood recedes, water would flow out through the levee breaches (if formed) or be trapped within the leveed area at the elevation of the lowest point on the top of levee and would only be able to flow out through small gravity drains with their sluice gates left open. Many levee districts intentionally breach levees to drain in this scenario after the river recedes below top of levee. As the time to drain the levees after a flood is based on the volume of water below top of levee inside of each levee, which is not impacted by the project, coupled with actions and levee performance outside of USACE control, the best assumption is that there is no difference in ponding duration for levee protected areas. For riverside areas, assumptions applied for hydrograph shape previously discussed will be used to assess duration impacts.

Aside from the Osage, Moreau, and Gasconade Rivers, all other tributaries shown are modeled for Missouri River backwater impacts only. Flows from these streams will result in more frequent flooding due to independent watershed flood events but would usually occur at different timing than Missouri River floods.

Analysis conducted here is not using the FEMA effective HEC-RAS model, which is a steady-state model of only the 1% AEP event and would not be capable of computing downstream impacts from changes to levee overtopping volumes. For a new levee system that does not overtop in a 1% AEP event, it should be assumed that separate modeling coordination with FEMA and remapping will be required during design and post-project. New levee alignments are on the edge of the mapped floodway to the extent feasible.

8.7 Coincident Flow at Bounding Tributaries

Coincident tributary flow at the Turkey Creek levee tieback was assessed for sensitivity of maximum water surface elevations. Turkey Creek was not included in the FEMA FIS as a flooding source and therefore had no flow frequencies developed. The USGS StreamStats regression equations were consulted to determine a conservative 1% AEP flow at Turkey Creek. To test sensitivity to inflow from Turkey Creek, the lateral inflow hydrograph on the Missouri River directly upstream of Jefferson City was scaled to have a peak flow of 5,000 cfs. This inflow hydrograph was input into the model as a boundary condition on Turkey Creek upstream of the proposed CA13 tieback alignment. Duplicating the hydrograph shape input onto the Missouri River results in the most conservative condition in which the Turkey Creek and Missouri River peaks arrive at the same time. The 1% AEP flow at Turkey Creek was then run as a coincident flow with the 1% AEP (588,000 cfs) Missouri River mainstem flow. Other combinations such as a 1% AEP event on Turkey Creek and 10% AEP flow on the Missouri River mainstem are not influential to the final alignment or profile of Alternative CA13 which has an overtopping AEP equivalent to the Missouri River 1% AEP event water surface elevation.

Figure 38 shows the boundary condition location for the sensitivity test of coincident flow on Turkey Creek for the CA13 alternative. Also shown is the Turkey Creek alignment with stationing. **Figure 39** compares the maximum water surface elevation profiles along Turkey Creek for CA13 for a Missouri River event with and without coincident Turkey Creek flow. Maximum water surface elevations were within 0.05-foot (0.6 inches) between the backwater only and coincident flow events. This difference in WSE is negligible for selection of an alternative and preliminary cost development.

Figure 38. Turkey Creek Coincident Flow Model Boundary Condition and Profile Location

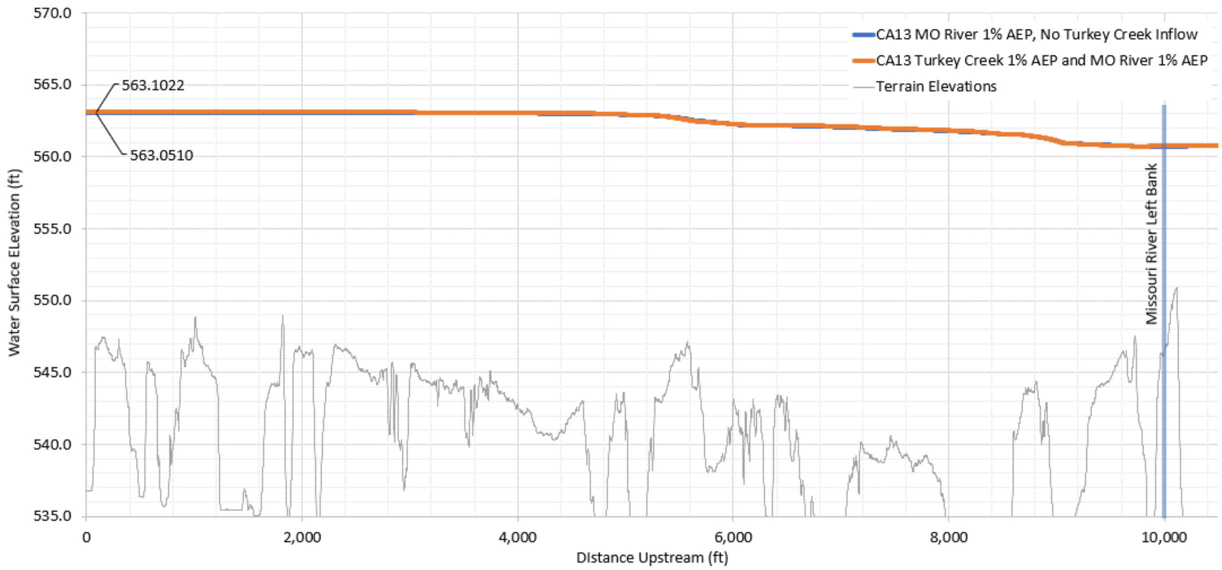

Figure 39. Turkey Creek Coincident Flow Maximum Water Surface Elevation Profile

A similar analysis was conducted for Niemans Creek, where the 1% AEP event is 4,100 cfs as developed by USGS StreamStats regression equations. **Figure 40** shows the boundary condition location for the sensitivity test of coincident flow on Niemans Creek for the CA13 alternative. **Figure 41** compares the maximum water surface elevation profiles along Niemans Creek for CA13 for a Missouri River event with and without coincident Niemans Creek flow. Maximum water surface elevations were equivalent for the backwater only and coincident flow events.

Figure 40. Niemans Creek Coincident Flow Model Boundary Condition and Profile Location

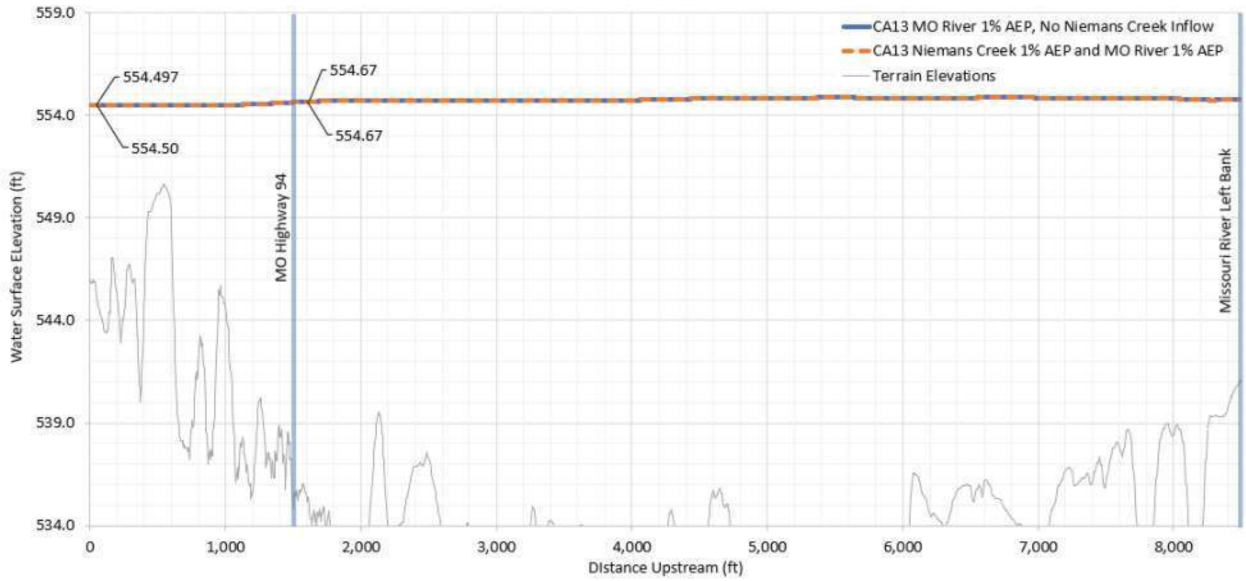

Figure 41. Niemans Creek Coincident Flow Maximum Water Surface Elevation Profile

9 SELECTED PLAN

Combined Alternative 13 is the selected plan determined to have federal interest. This plan provides an overtopping frequency equivalent to the 1% AEP event, which would reduce flooding of the left-bank through Jefferson City for the most frequently occurring storms, reduce the risk of flooding for critical facilities from an existing 17% AEP to a 1% AEP event, and provide an engineered federal levee system for much of the left-bank. **Table 15** summarizes elevations and corresponding Missouri River gage stage at Jefferson City for existing levee systems and adjacent notable features.

Table 15. Summary of Stages in reference to Jefferson City Missouri River Gage

Feature / Landmark	Elevation (NAVD88)	Stage (ft)	AEP	Probability Over 50 Years of Reaching or Exceeding Stage
Jefferson City Gage Datum	520.2	0.0		
Flood Action Stage	541.2	21.0	53% (1/2yr)	>99.9%
Minor Flood Stage	543.2	23.0	52% (1/2yr)	>99.9%
Moderate Flood Stage	545.2	25.0	39% (1/3yr)	>99.9%
Jacobs Levee		26.0		
Major Flood Stage	550.2	30.0	19% (1/5yr)	>99.9%
Hartsburg / Church Farm Levees		30.0		
Existing Capital View / Renz Levees	550.7	30.5	**17% (1/6yr)**	>99.9%
Wainwright Levee		31.0		
Reveaux Levee		32.0		
Flood of Similar Magnitude to 2019	553.2	33.0	7% (1/14yr)	98%
Cole Junction Levee	553.7	33.5	6% (1/18yr)	96%
Railroad (Low Point at Wears Creek)	554.4	34.2		
Capital Sand (eastern building)	556	35.8		
Wastewater Treatment Plant (west)	556	35.8		
U.S. Highway 54 Embankment	557.7	37.5	1.3% (1/78yr)	48%
Railroad (at U.S. Highway 54 / 63 bridge)	558.5	38.3		
Wastewater Treatment Plant (east)	560	39.8		
L-142 Top Elevation (CA13)	561	40.8	**1% (1/100yr)**	**39%**
Flood of Similar Magnitude to 1993	562	41.8	0.2% (1/500yr)	10%

9.1 Capital View Levee

CA13 was evaluated in three conditions due to public and study sponsor desire to understand the impact the existing Capital View levee would have hydraulically if it were to remain in place. With this analysis, it was determined that two feasible options regarding the future of the Capital View levee would be acceptable with the implementation of CA13. The existing levee could be removed in its entirety with the benefit of added river and stream conveyance, especially along Turkey Creek and at the Missouri River bank downstream of the U.S. Highway 54 / 63 bridge where the existing levee system sees repetitive breaches. Removal of the existing levee would also allow for on-site random borrow for construction activities related to the proposed federal levee. Alternatively, the existing levee could be partially removed west of the U.S. Highway 54 / 63 bridge and Capital Sand high ground. This would result in the benefit of added conveyance at Turkey Creek and reduce water surface elevation increases on adjacent upstream levee systems with construction of the federal levee. The remainder of the Capital View levee, east of the highway bridge would remain in place as a separate levee system and within the responsibility of the local drainage district. This levee would not remain in the P.L. 84-99 system.

10 USACE Feasibility Study Scope

Any revisions to the existing Capital View levee, beyond removal, are outside the federal interest for the Jefferson City Feasibility Study. However, please note that any revision to volume and/or changes in hydraulics within a floodway will have impacts to adjacent properties. The USACE process for feasibility studies includes detailed consideration to induced impacts and documentation of anticipated changes in hydraulics due to proposed changes within the study area. These impacts are assessed to determine impacts to adjacent landowners considering a range of storm events (50% - 0.2% AEP). The consideration of multiple storm events is critical to ensuring a full understanding of impacts in a variety of storm flows. It is beyond the scope of the Feasibility Study to consider revisions to the existing Capital View levee outside of the defined range of study alternatives.

The existing Capital View levee footprint is within both Jefferson City and Callaway County floodplain management jurisidictions, genearlly within the adopted FEMA Floodway. Ordinances for these jurisdictions should be consulted to understand regulatory requirements within the floodplain and floodway. For Jefferson City's Floodplain Management ordiance, please refer to Jefferson City's Code of Ordinances, Chapter 31, Article 3.

A floodplain fill scenario that does not have an overall impact to the FEMA Base Flood Elevation could result in impacts to adjacent properties during more frequent flood events. Consideration should be taken on any revisions to the existing Capital View levee, as more frequent flood events, specifically within the 10%-4% AEP range, are impactful to many stakeholders within Jefferson City as observed in recent years.

11 CLIMATE CHANGE

The assessed alternatives will have no direct impact on the regional climate as they do not propose a holistic change to drainage patterns or climate drivers. Further analysis of the effects of climate change on the proposed alternatives is not warranted for this site. Additional information regarding climate change patterns for the Lower Missouri River Basin can be found in Appendix A2.

12 EO 11988, ER 1165-2-26, AND 44 CFR 65.10

Floodplain Management requirements were reviewed in Executive Order (EO) 11988 "Floodplain Management" (1   7). Implementation guidance was reviewed in USACE Engineering Regulation (ER) 1165-2-26 (1984) "Implementation of Executive Order 11   on Flood Plain Management."

The left bank of the Missouri River is specifically outlined in the city's future land use map as a "River Transition Overlay Annexation" which is defined as "areas that need additional scrutiny in regard to development in and around the floodplain, flood prone areas, or environmentally sensitive features." The City of Jefferson City has extensive experience in floodplain management and has taken broad risk reduction acts since the Flood of 1993. In addition, the city does not desire a FEMA accredited levee to be constructed, reducing the perceived accessibility to development with the proposed L-142 alignment. There is added interest for the city to maintain the area east of the airport as undeveloped because it is their sludge disposal location for the adjacent wastewater treatment facility. During floods, there is considerable cost incurred by the City to find an alternative disposal location. The eastern boundary of the existing Capital View Drainage District is maintained in the CA13 footprint due to real estate concerns and to ensure a proposed levee would not impact airport runway operations or height restrictions.

No additional development in the floodplain is expected by constructing any of the features in the tentatively selected plan due to the unique site considerations discussed above. Life safety risk is not increased with the proposed CA13 as discussed in Appendix E - Economics. No practicable alternative has been identified to relocate facilities in the floodplain although considerations were made to relocate facilities in the nonstructural alternative (buyouts) that is documented in the economic analysis and the main feasibility report.

Jefferson City is an existing participant in FEMA's NFIP. The development of the CA13 alignment and profile included incorporating levee setbacks to optimize levee height, incorporate critical infrastructure into the leveed area, and minimize changes to the base flood elevation. Construction of the CA13 alignment would require a Conditional Letter of Map Revision (CLOMR) to document changes in floodplain hydraulics and BFE through the project area and mitigation to any impacted structures. The floodplain management plan currently enforced by the city is sufficient for continued implementation if CA13 were to be constructed.

13 RISK ANALYSIS

Semi-quantitative analysis of risk informed decision was conducted for the federal interest plan, CA13. The hydraulic modeling for the risk analysis included a non-breach, fully loaded levee condition at a 1% AEP storm, breach condition at a 1% AEP storm and breach condition at a 0.2% AEP storm. The modeled breach was assumed to occur at the historic location of Capital View levee breaches on the riverside levee midpoint south of the highway bridge. Breach width and development time were assumed to be comparable breach developments that occurred on federal levees along the Missouri River during the 2019 flood. Other breach factor inputs utilized USACE Modeling, Mapping, and Consequence (MMC) Production Center modeling standards, with breach timing assumed to occur at the hydrograph peak at River Station 142.58. Breach runs were conducted for the 1% AEP and 0.2% AEP storms. The risk assessment is included in Appendix G.

13.1 Risk Analysis Hydraulic Hazards

13.1.1 Expected Annual Exceedance Probability

The expected AEP is a measure of the likelihood of exceeding a specified target in any year.

13.1.2 Long Term Risk

Risk of one or more floods with an annual exceedance probability of 1-percent is approximately 39 percent for the 50-year study assessment.

13.2 Climate Change

Future changes in streamflow and/or precipitation patterns due to climate change may affect the long-term efficacy of the proposed alternatives. Most notably, the long-term rate of overtopping is anticipated to become more frequent due to increased precipitation within the watershed, especially in the spring when Missouri River flows are generally high. A climate assessment was conducted with this study and is included as Appendix A2. This could result in an increased AEP of the levee system. Resilience measures to account for climate change effects could include designing the selected levee alternative for a higher flow condition for both the Missouri River and internal drainage flows. **Table 16** summarizes anticipated residual risk due to climate change.

Table 16. Residual Risk due to Climate Change

Project Feature	Trigger	Hazard	Harm	Qualitative Likelihood
Levees & Berms	Increased seasonal precipitation	Future flows may be larger than present Large flood volumes may occur more frequently	Levees could overtop more frequently or be loaded for longer durations, resulting in increased floodfighting and O&M costs.	Likely
Internal Drainage	Increased seasonal precipitation	Increased flows behind the leveed area Longer drawn down times along the Missouri River	Flow volume internal to the leveed area could increase, requiring additional pumps or an extended time to relief Longer river draw down times could result in longer internal drainage ponding conditions	Likely
Riprap Bank Stabilization	Increased seasonal precipitation	Future flows may be larger than present Large flood volumes may occur more frequently	Higher flows and velocities would result in increased O&M costs	Likely
Floodproofing Measures	Increased seasonal precipitation	Future flows may be larger than present Large flood volumes may occur more frequently	Floodproofing measures could overtopped during more frequent storm events, leading to additional repair costs	Likely

All project features driving residual risk due to climate change have a likely qualitative likelihood. This likelihood rating is justified, as increases in precipitation were observed in every future projection trend test conducted, with variable robustness. As the Lower Missouri River basin is not regulated downstream of Gavin Point dam, located 666 miles upstream of the study area, increased precipitation is the main driver of flood events in this location. Increase in precipitation volumes due to severe or prolonged storm events would directly impact peak streamflow.

Attachment A – Hydraulic Modeling FWOP Inundation Maps

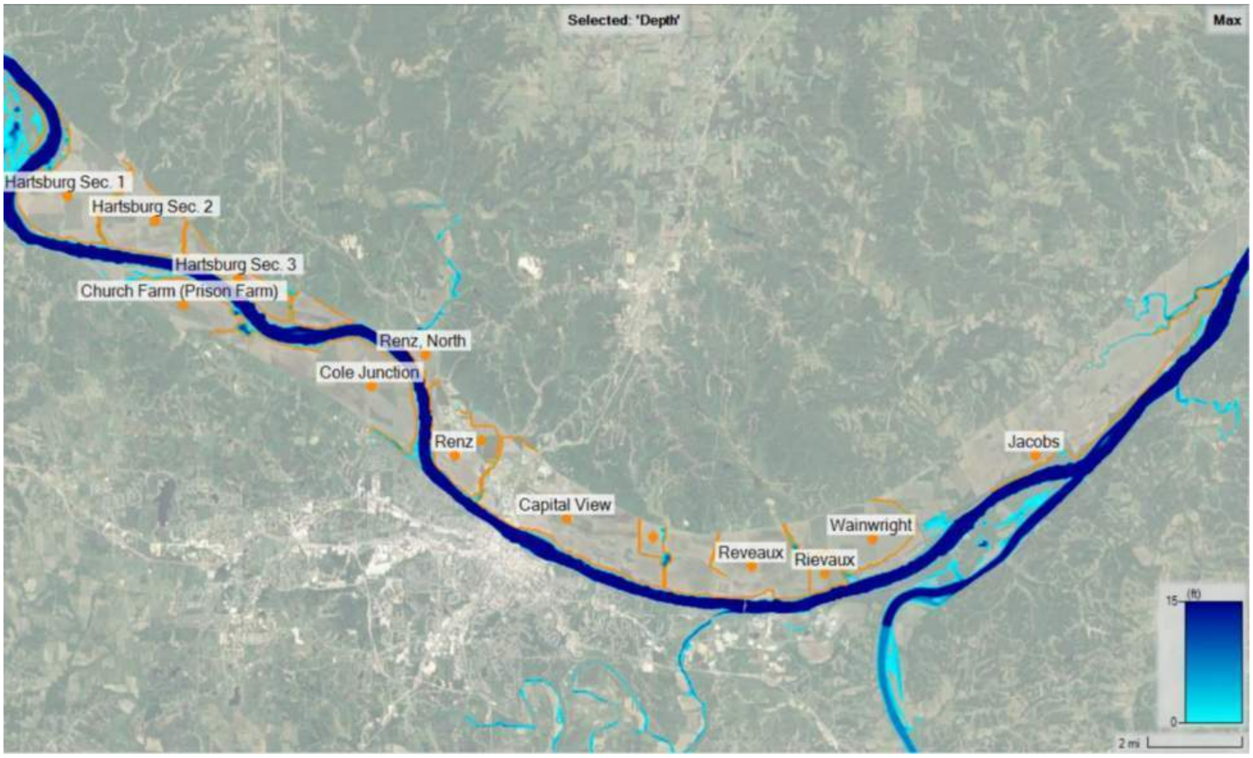

Figure 42. Inundation - CA0 - FWOP - 50% AEP

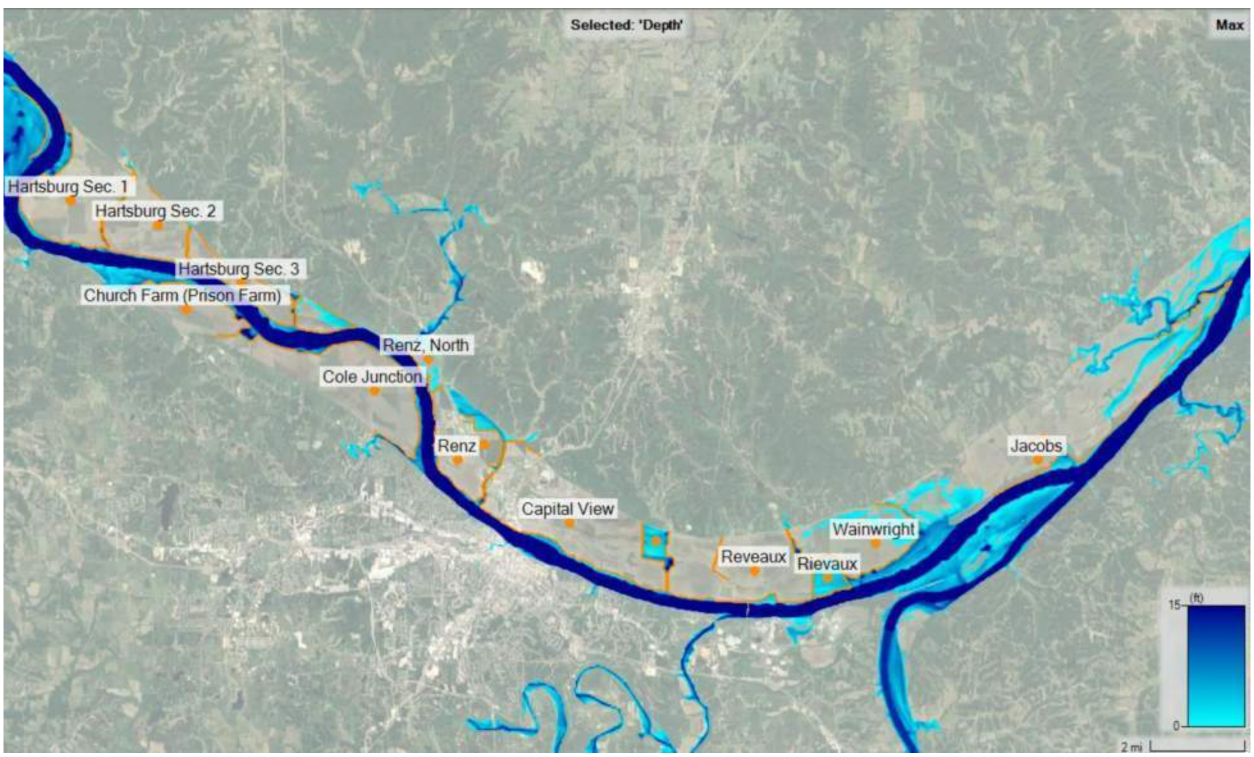

Figure 43. Inundation - CA0 - FWOP - 20% AEP

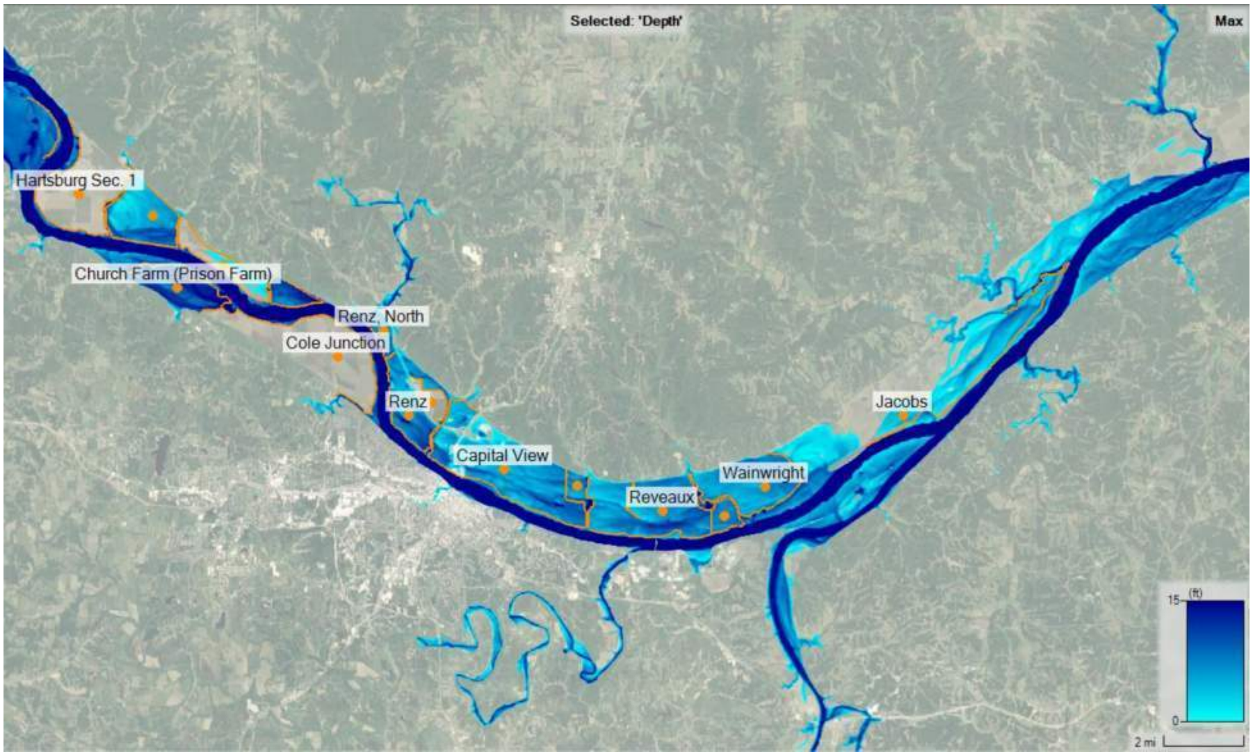

Figure 44. Inundation - CA0 - FWOP - 10% AEP

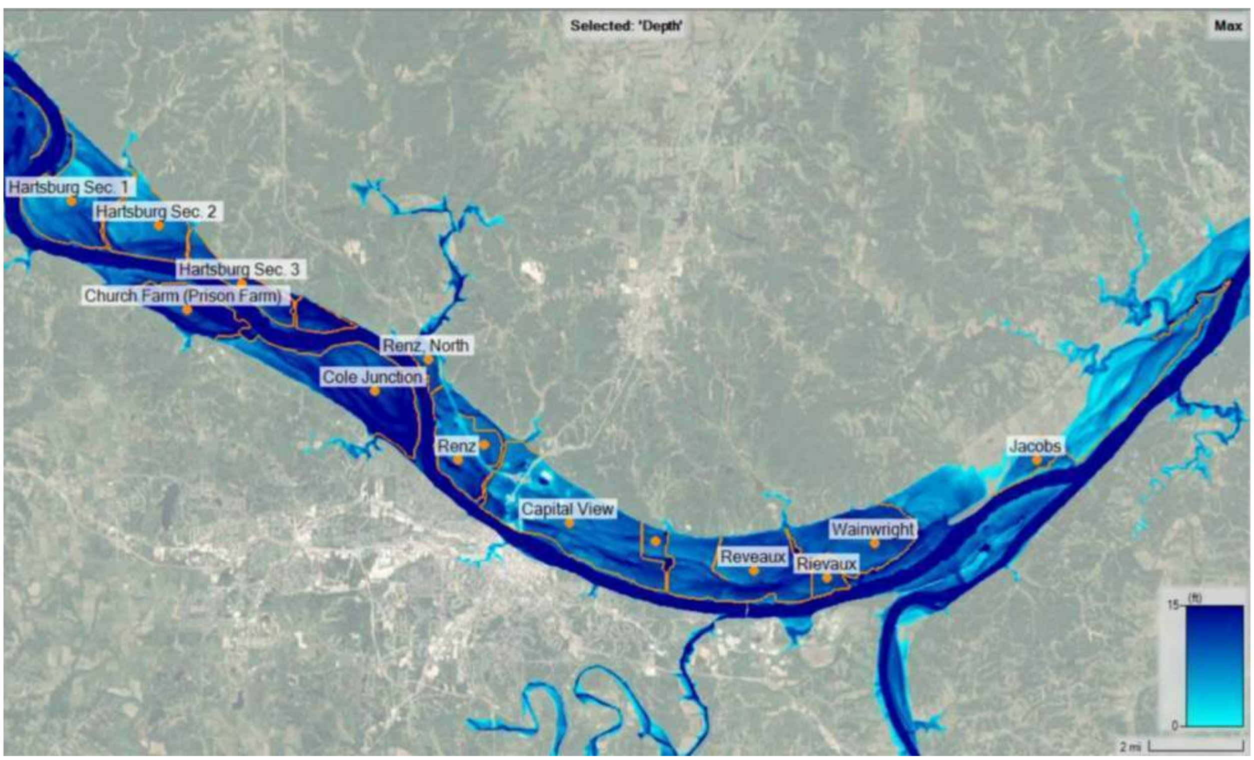

Figure 45. Inundation - CA0 - FWOP - 4% AEP

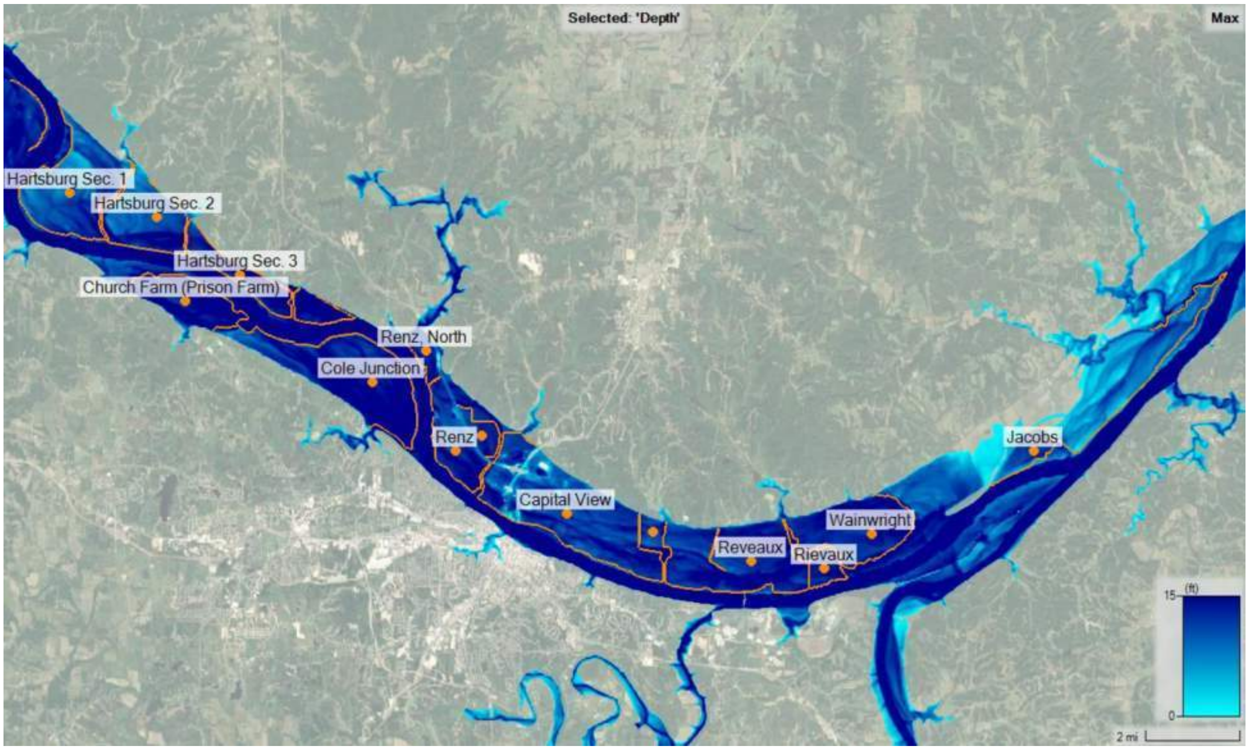

Figure 46. Inundation - CA0 - FWOP - 2% AEP

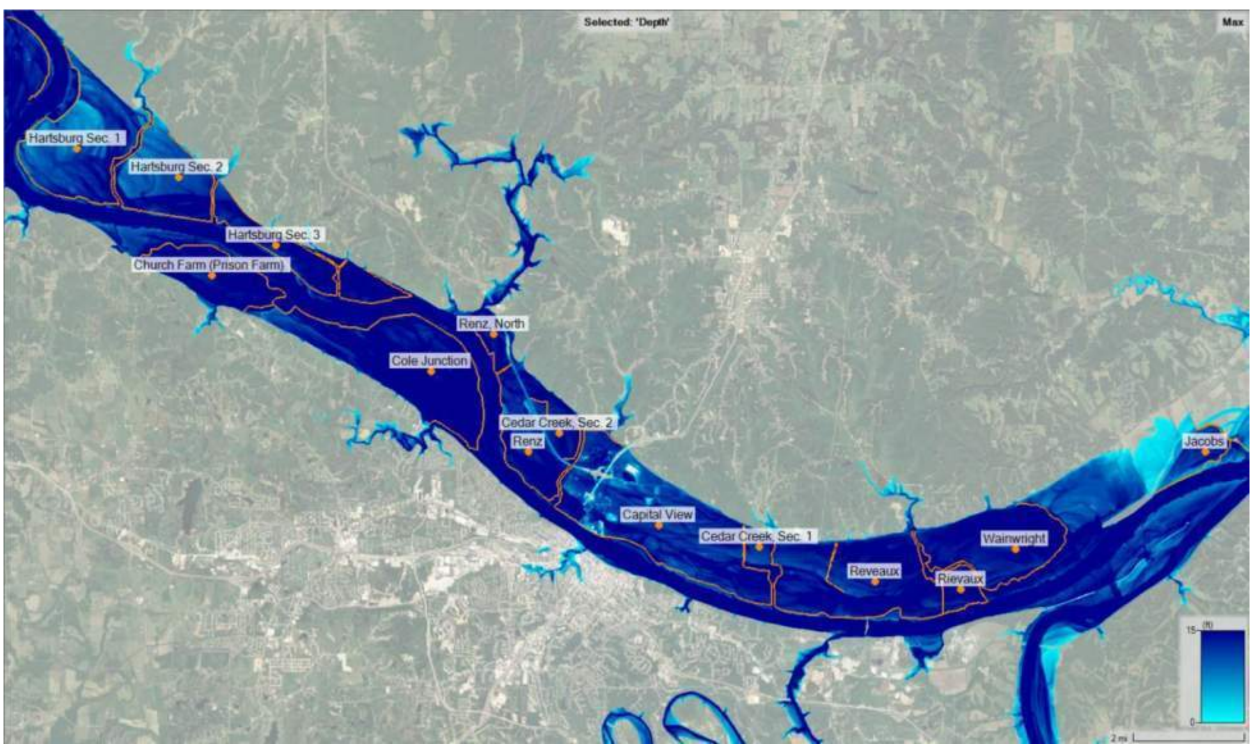

Figure 47. Inundation - CA0 - FWOP - 1% AEP

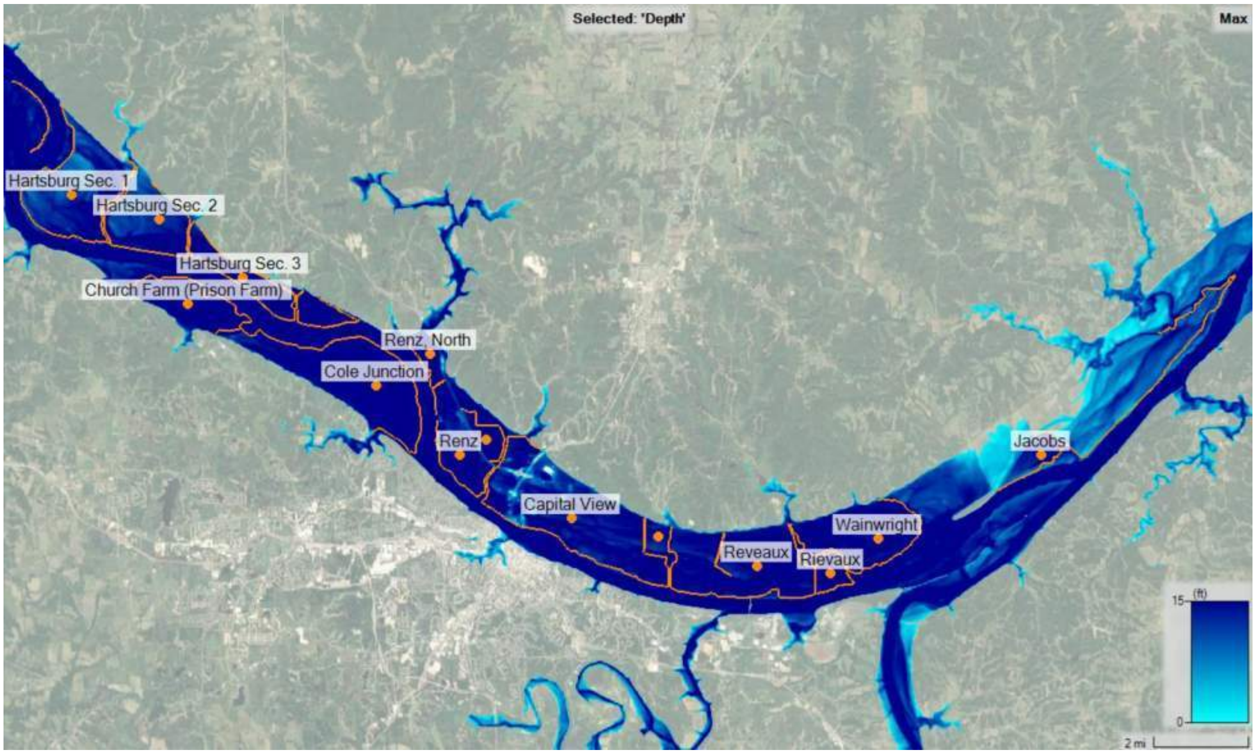

Figure 48. Inundation – CA0 - FWOP - 0.5% AEP

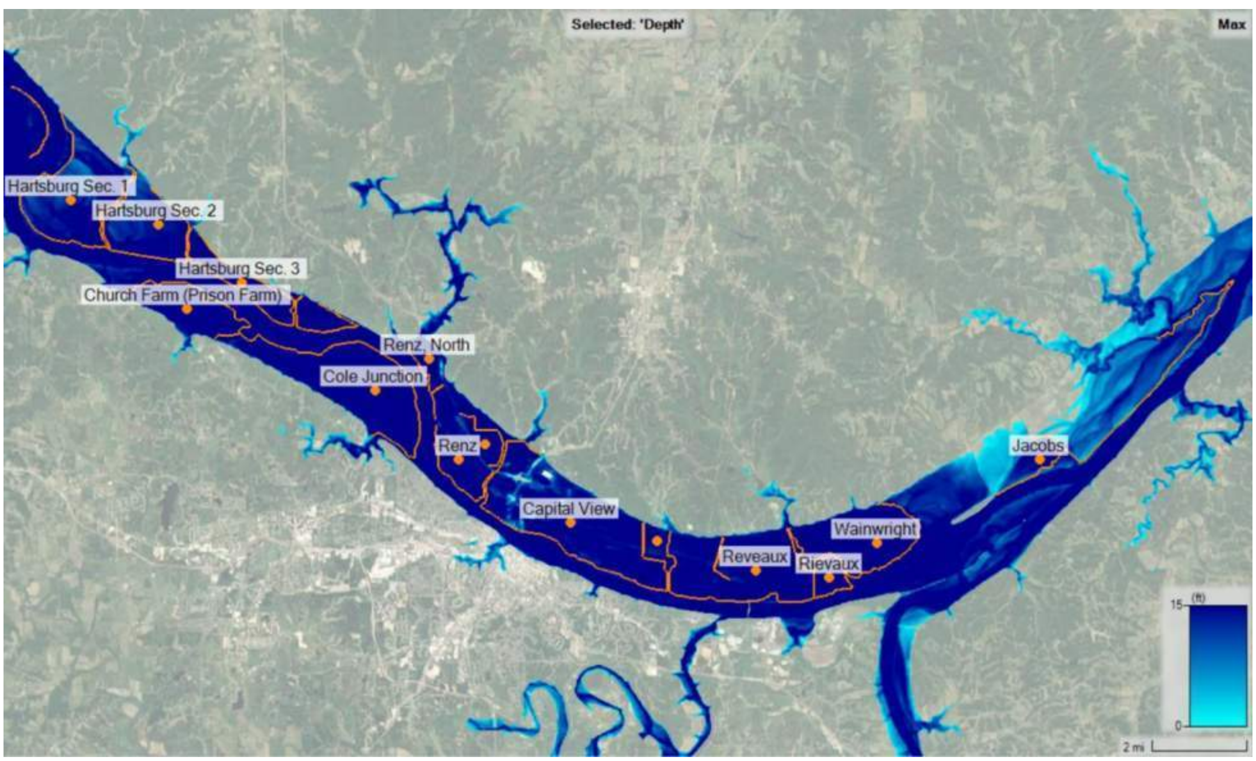

Figure 49. Inundation – CA0 - FWOP - 0.2% AEP

ATTACHMENT B – HYDRAULIC MODELING MISSOURI RIVER PROFILE OUTPUTS

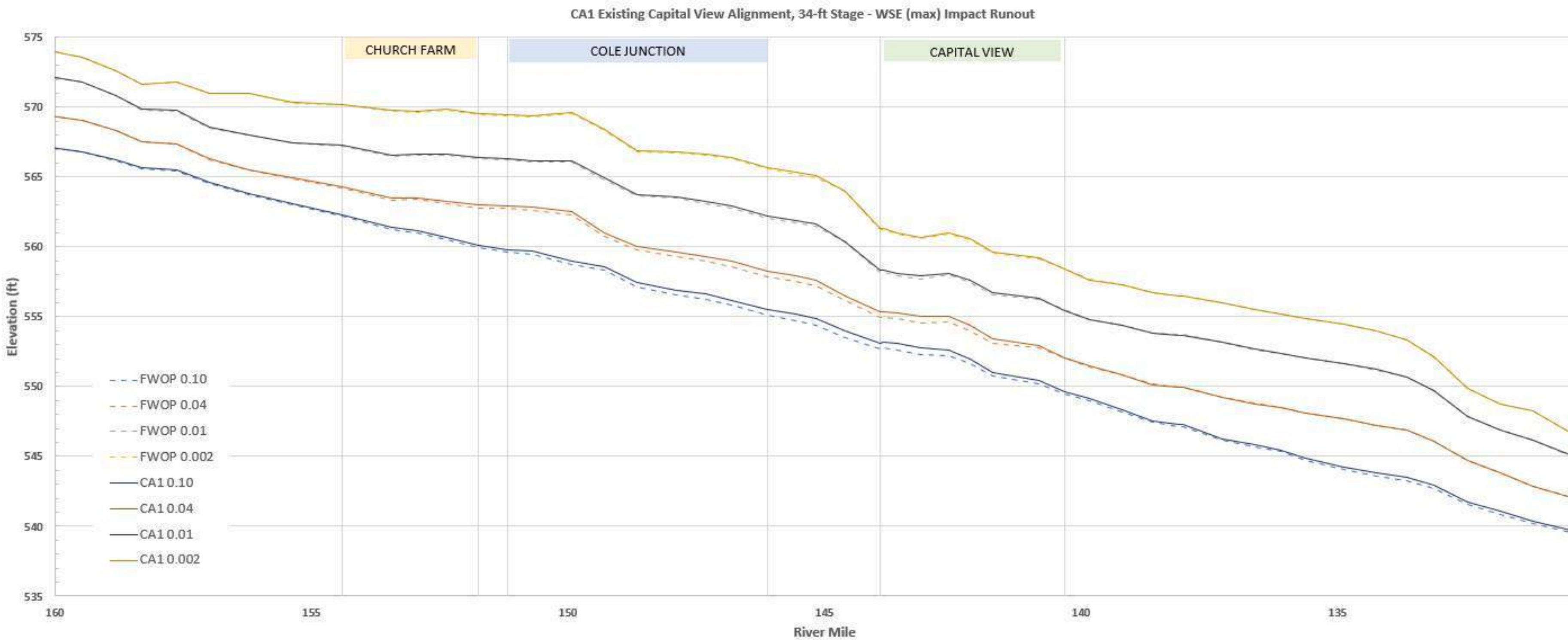

Figure 50. CA1 Existing Capital View Alignment, 34-ft Stage - Missouri River Maximum WSE Profile Impacts

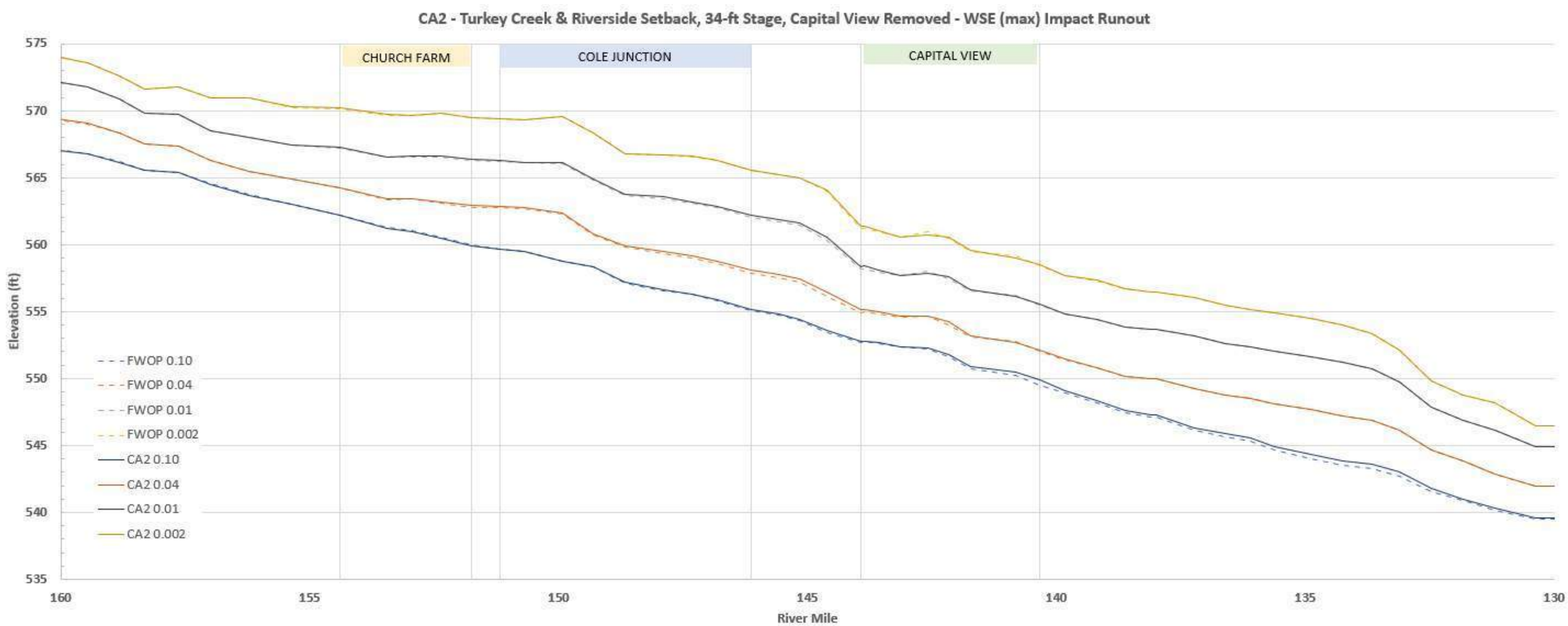

Figure 51. CA2 Turkey Creek & Riverside Setback, 34-ft Stage, Capital View Removed - Missouri River Maximum WSE Profile Impacts

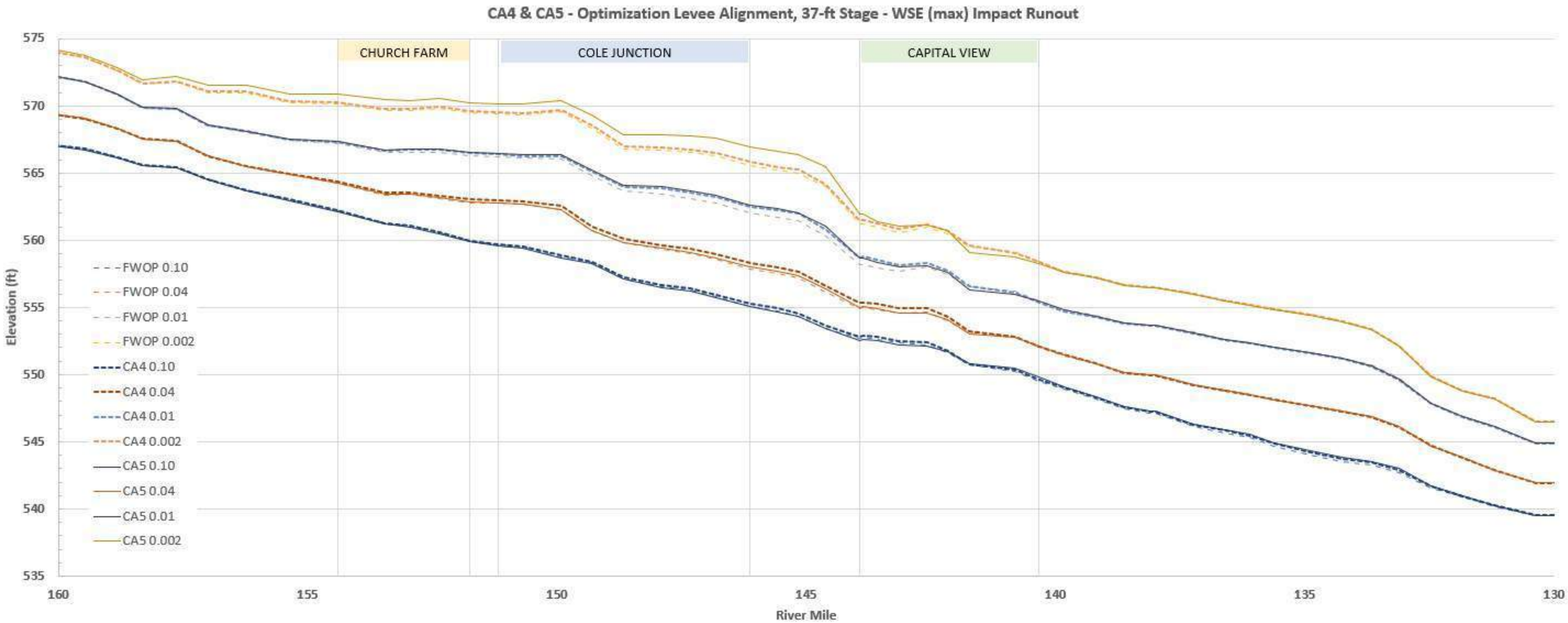

Figure 52. CA4 & CA5 Optimization Alignment - Missouri River Maximum WSE Profile Impacts

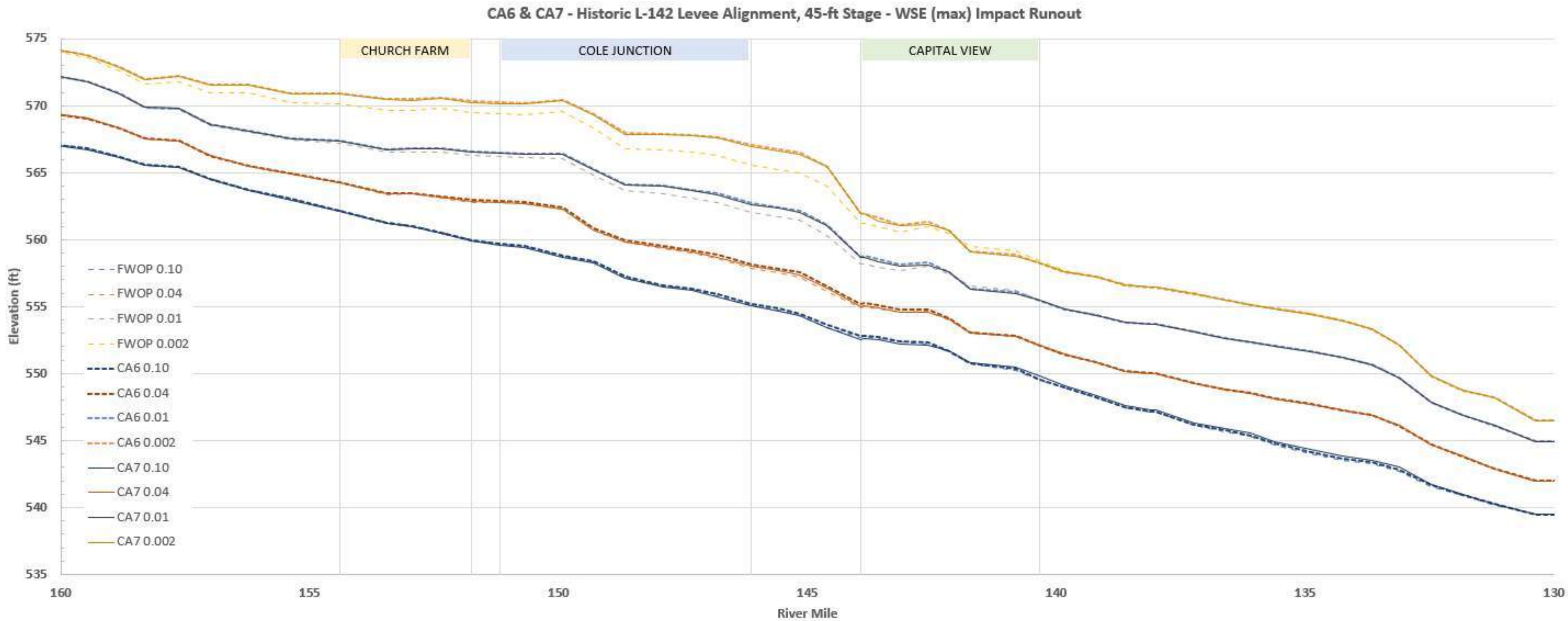

Figure 53. CA6 & CA7 Historic L-142 Alignment - Missouri River Maximum WSE Profile Impacts

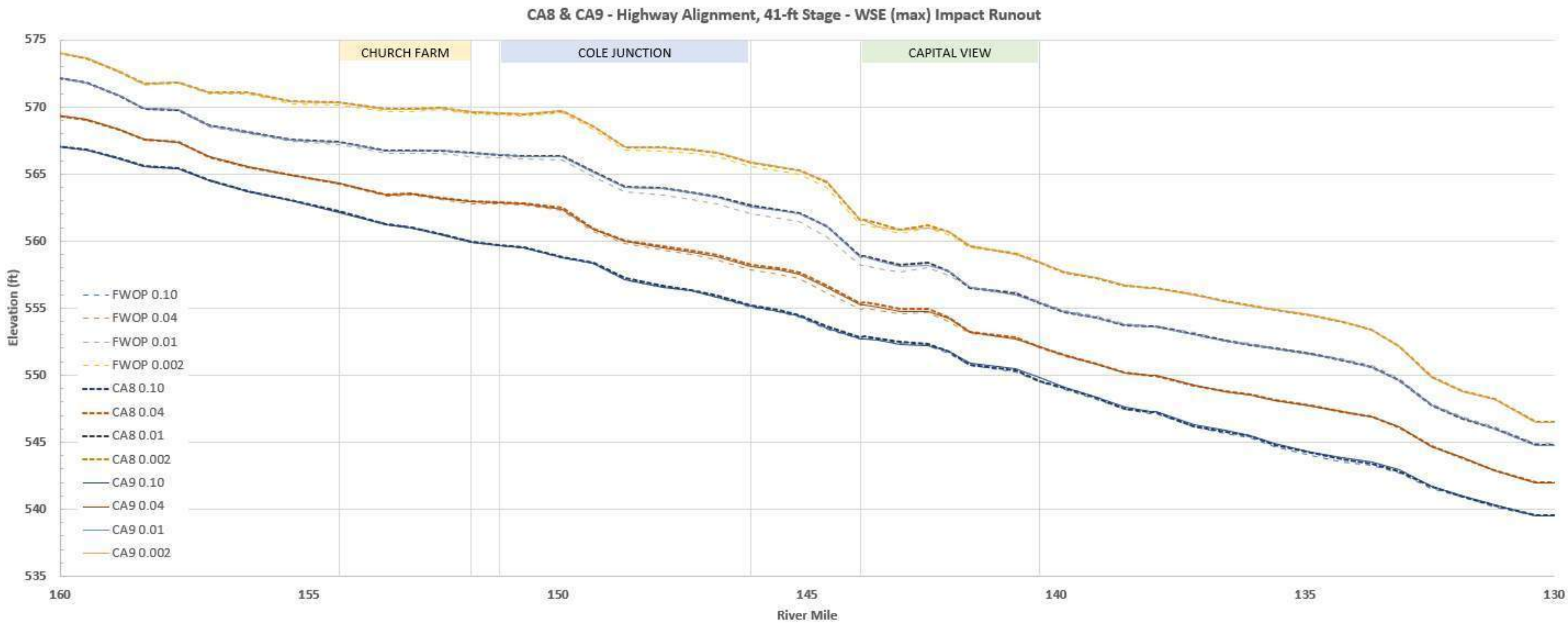

Figure 54. CA8 & CA9 Highway Alignment - Missouri River Maximum WSE Profile Impacts

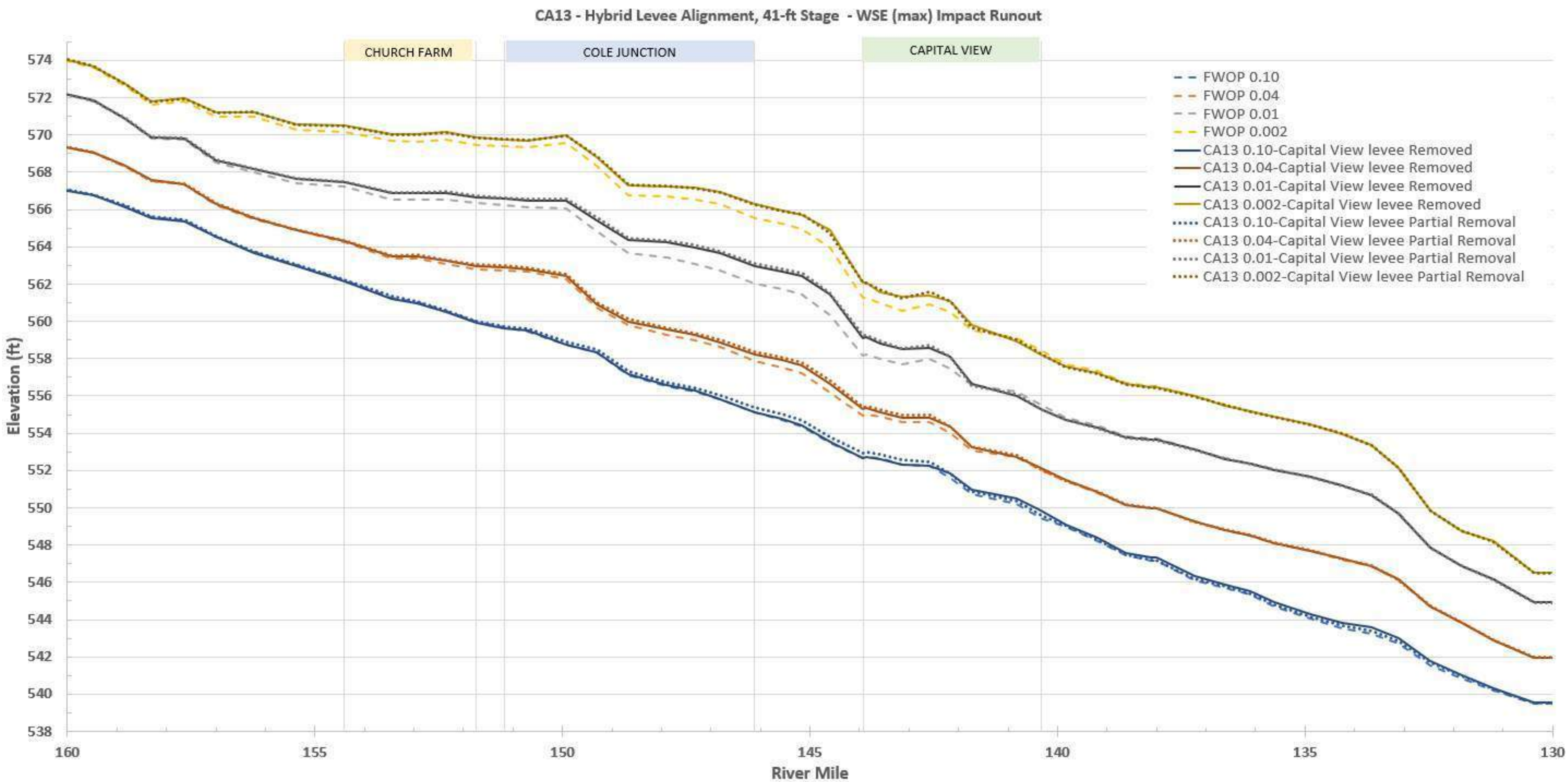

Figure 55. CA13 Hybrid Alignment - Missouri River Maximum WSE Profile Impacts

ATTACHMENT C – HEC-FDA INDEX POINT INPUTS

Jefferson City - FWOP - Summary of Maximum Water Surface Elevations (ft NAVD88)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93	561.24	560.01	558.17	557.30	554.94	552.74	550.08	544.17
Missouri River, River Station 143.14	560.56	559.41	557.71	556.88	554.57	552.37	549.69	543.76
Missouri River, River Station 141.73	559.53	558.33	556.55	555.64	553.08	550.72	548.11	542.40

1. WSE reported as the maximum water surface within the channel. The overbank WSE will vary depending on breach.

Jefferson City - FWOP - Summary of Peak Flows (cfs)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93	748,542	685,619	588,816	548,601	434,975	345,996	289,888	212,259
Missouri River, River Station 143.14	744,981	681,158	584,195	544,335	432,361	345,234	289,890	212,262
Missouri River, River Station 141.73	740,448	679,054	584,808	545,429	433,984	345,005	289,904	212,289

1. Flow calculations include both the 1D channel and 2D overbanks (determined in RAS Mapper by drawing a profile cross-section across the channel and overbanks) .
2. Levee were assumed to breach when WSE over levee exceeds 1-ft at lowest point

Jefferson City - FWOP - Summary of Maximum Depths (ft NAVD88)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93 Min. Channel Elev = 503.938	57.3	56.1	54.2	53.4	51.0	48.8	46.1	40.2
Missouri River, River Station 143.14 Min. Channel Elev = 506.688	53.9	52.7	51.0	50.2	47.9	45.7	43.0	37.1
Missouri River, River Station 141.73 Min. Channel Elev = 503.079	56.4	55.3	53.5	52.6	50.0	47.6	45.0	39.3

1. Depths differ dramatically between river channel and overbanks. These depths are measured from the minimum channel elevation. Channel elevations are listed in table.

Jefferson City - CA1 - Summary of Maximum Water Surface Elevations (ft NAVD88)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93	561.32	560.09	558.30	557.45	555.27	553.11	550.08	544.17
Missouri River, River Station 143.14	560.66	559.52	557.87	557.07	554.93	552.75	549.69	543.76
Missouri River, River Station 141.73	559.63	558.45	556.71	555.83	553.42	551.02	548.11	542.40

1. WSE reported as the maximum water surface within the channel. The overbank WSE will vary depending on breach.

Jefferson City - CA1 - Summary of Peak Flows (cfs)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93	748,868	685,914	589,128	549,224	436,897	345,023	289,890	212,259
Missouri River, River Station 143.14	745,104	679,936	583,155	543,591	433,056	344,987	289,891	212,262
Missouri River, River Station 141.73	741,114	679,693	585,310	546,134	435,154	344,923	289,904	212,289

1. Flow calculations include both the 1D channel and 2D overbanks (determined in RAS Mapper by drawing a profile cross-section across the channel and overbanks) .
2. Levee were assumed to breach when WSE over levee exceeds 1-ft at lowest point

Jefferson City - CA1 - Summary of Maximum Depths (ft NAVD88)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93 Min. Channel Elev = 503.938	57.4	56.2	54.4	53.5	51.3	49.2	46.1	40.2
Missouri River, River Station 143.14 Min. Channel Elev = 506.688	54.0	52.8	51.2	50.4	48.2	46.1	43.0	37.1
Missouri River, River Station 141.73 Min. Channel Elev = 503.079	56.6	55.4	53.6	52.8	50.3	47.9	45.0	39.3

1. Depths differ dramatically between river channel and overbanks. These depths are measured from the minimum channel elevation. Channel elevations are listed in table.

Jefferson City - CA2 - Summary of Maximum Water Surface Elevations (ft NAVD88)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93	561.37	560.17	558.37	557.51	555.13	552.74	549.85	544.16
Missouri River, River Station 143.14	560.57	559.41	557.72	556.91	554.69	552.37	549.46	543.75
Missouri River, River Station 141.73	559.57	558.37	556.59	555.68	553.16	550.87	548.01	542.39

1. WSE reported as the maximum water surface within the channel. The overbank WSE will vary depending on breach.

Jefferson City - CA2 - Summary of Peak Flows (cfs)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93	754,260	690,294	591,664	550,748	436,172	346,779	289,817	212,250
Missouri River, River Station 143.14	741,007	679,521	585,015	545,956	434,897	347,002	289,993	212,252
Missouri River, River Station 141.73	747,858	685,198	588,972	548,904	435,873	347,127	289,718	212,278

1. Flow calculations include both the 1D channel and 2D overbanks (determined in RAS Mapper by drawing a profile cross-section across the channel and overbanks) .
2. Levee were assumed to breach when WSE over levee exceeds 1-ft at lowest point

Jefferson City - CA2 - Summary of Maximum Depths (ft NAVD88)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93 Min. Channel Elev = 503.938	57.4	56.2	54.4	53.6	51.2	48.8	45.9	40.2
Missouri River, River Station 143.14 Min. Channel Elev = 506.688	53.9	52.7	51.0	50.2	48.0	45.7	42.8	37.1
Missouri River, River Station 141.73 Min. Channel Elev = 503.079	56.5	55.3	53.5	52.6	50.1	47.8	44.9	39.3

1. Depths differ dramatically between river channel and overbanks. These depths are measured from the minimum channel elevation. Channel elevations are listed in table.

Jefferson City - CA4 - Summary of Maximum Water Surface Elevations (ft NAVD88)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93	561.53	560.40	558.73	557.93	555.33	552.83	550.08	544.17
Missouri River, River Station 143.14	560.86	559.76	558.15	557.38	554.94	552.47	549.69	543.76
Missouri River, River Station 141.73	559.65	558.43	556.61	555.69	553.24	550.81	548.11	542.40

1. WSE reported as the maximum water surface within the channel. The overbank WSE will vary depending on breach.

Jefferson City - CA4 - Summary of Peak Flows (cfs)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93	802,730	730,228	619,871	573,892	443,644	347,671	289,886	212,260
Missouri River, River Station 143.14	732,613	668,412	571,765	534,120	433,028	346,279	289,886	212,262
Missouri River, River Station 141.73	739,934	678,478	583,717	543,930	433,278	346,390	289,898	212,289

1. Flow calculations include both the 1D channel and 2D overbanks (determined in RAS Mapper by drawing a profile cross-section across the channel and overbanks) .
2. Levee were assumed to breach when WSE over levee exceeds 1-ft at lowest point

Jefferson City - CA4 - Summary of Maximum Depths (ft NAVD88)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93 Min. Channel Elev = 503.938	57.6	56.5	54.8	54.0	51.4	48.9	46.1	40.2
Missouri River, River Station 143.14 Min. Channel Elev = 506.688	54.2	53.1	51.5	50.7	48.3	45.8	43.0	37.1
Missouri River, River Station 141.73 Min. Channel Elev = 503.079	56.6	55.3	53.5	52.6	50.2	47.7	45.0	39.3

1. Depths differ dramatically between river channel and overbanks. These depths are measured from the minimum channel elevation. Channel elevations are listed in table.

Jefferson City - CA5 - Summary of Maximum Water Surface Elevations (ft NAVD88)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93	561.45	560.32	558.64	557.84	555.24	552.67	549.82	544.16
Missouri River, River Station 143.14	560.80	559.67	558.03	557.24	554.79	552.29	549.42	543.74
Missouri River, River Station 141.73	559.67	558.45	556.60	555.66	553.20	550.91	548.04	542.38

1. WSE reported as the maximum water surface within the channel. The overbank WSE will vary depending on breach.

Jefferson City - CA5 - Summary of Peak Flows (cfs)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93	823,818	749,653	636,405	589,114	459,615	357,193	293,409	212,262
Missouri River, River Station 143.14	737,568	672,869	575,495	537,368	435,396	346,176	289,454	212,219
Missouri River, River Station 141.73	739,293	677,648	582,717	542,765	433,146	345,900	289,747	212,222

1. Flow calculations include both the 1D channel and 2D overbanks (determined in RAS Mapper by drawing a profile cross-section across the channel and overbanks) .
2. Levee were assumed to breach when WSE over levee exceeds 1-ft at lowest point

Jefferson City - CA5 - Summary of Maximum Depths (ft NAVD88)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93 Min. Channel Elev = 503.938	57.5	56.4	54.7	53.9	51.3	48.7	45.9	40.2
Missouri River, River Station 143.14 Min. Channel Elev = 506.688	54.1	53.0	51.3	50.6	48.1	45.6	42.7	37.1
Missouri River, River Station 141.73 Min. Channel Elev = 503.079	56.6	55.4	53.5	52.6	50.1	47.8	45.0	39.3

1. Depths differ dramatically between river channel and overbanks. These depths are measured from the minimum channel elevation. Channel elevations are listed in table.

Jefferson City - CA6 - Summary of Maximum Water Surface Elevations (ft NAVD88)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93	561.97	560.69	558.75	557.82	555.18	552.79	550.08	544.17
Missouri River, River Station 143.14	561.12	559.94	558.16	557.29	554.79	552.42	549.69	543.76
Missouri River, River Station 141.73	559.10	557.96	556.32	555.48	553.10	550.77	548.11	542.40

1. WSE reported as the maximum water surface within the channel. The overbank WSE will vary depending on breach.

Jefferson City - CA6 - Summary of Peak Flows (cfs)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93	755,946	691,655	592,863	552,371	437,972	347,565	289,888	212,259
Missouri River, River Station 143.14	745,827	682,871	586,191	546,133	434,114	346,054	289,890	212,262
Missouri River, River Station 141.73	739,017	677,982	583,796	545,007	435,158	346,155	289,904	212,289

1. Flow calculations include both the 1D channel and 2D overbanks (determined in RAS Mapper by drawing a profile cross-section across the channel and overbanks) .
2. Levee were assumed to breach when WSE over levee exceeds 1-ft at lowest point

Jefferson City - CA6 - Summary of Maximum Depths (ft NAVD88)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93 Min. Channel Elev = 503.938	58.0	56.8	54.8	53.9	51.2	48.9	46.1	40.2
Missouri River, River Station 143.14 Min. Channel Elev = 506.688	54.4	53.3	51.5	50.6	48.1	45.7	43.0	37.1
Missouri River, River Station 141.73 Min. Channel Elev = 503.079	56.0	54.9	53.2	52.4	50.0	47.7	45.0	39.3

1. Depths differ dramatically between river channel and overbanks. These depths are measured from the minimum channel elevation. Channel elevations are listed in table.

Jefferson City - CA7 - Summary of Maximum Water Surface Elevations (ft NAVD88)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93	561.91	560.62	558.67	557.74	555.01	552.56	549.77	544.16
Missouri River, River Station 143.14	561.07	559.86	558.04	557.14	554.56	552.17	549.36	543.75
Missouri River, River Station 141.73	559.06	557.92	556.26	555.40	553.02	550.85	547.99	542.39

1. WSE reported as the maximum water surface within the channel. The overbank WSE will vary depending on breach.

Jefferson City - CA7 - Summary of Peak Flows (cfs)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93	856,572	773,448	649,095	598,935	460,231	356,758	292,833	212,256
Missouri River, River Station 143.14	748,076	685,166	588,284	548,388	434,336	346,134	289,183	212,258
Missouri River, River Station 141.73	737,725	676,729	582,578	543,717	432,453	345,962	289,487	212,271

1. Flow calculations include both the 1D channel and 2D overbanks (determined in RAS Mapper by drawing a profile cross-section across the channel and overbanks) .
2. Levee were assumed to breach when WSE over levee exceeds 1-ft at lowest point

Jefferson City - CA7 - Summary of Maximum Depths (ft NAVD88)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93 Min. Channel Elev = 503.938	58.0	56.7	54.7	53.8	51.1	48.6	45.8	40.2
Missouri River, River Station 143.14 Min. Channel Elev = 506.688	54.3	53.2	51.4	50.5	47.9	45.5	42.7	37.1
Missouri River, River Station 141.73 Min. Channel Elev = 503.079	55.9	54.8	53.2	52.3	49.9	47.8	44.9	39.3

1. Depths differ dramatically between river channel and overbanks. These depths are measured from the minimum channel elevation. Channel elevations are listed in table.

Jefferson City - CA8 - Summary of Maximum Water Surface Elevations (ft NAVD88)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93	561.61	560.50	558.85	558.05	555.37	552.81	550.08	544.17
Missouri River, River Station 143.14	560.85	559.76	558.18	557.42	554.96	552.45	549.69	543.76
Missouri River, River Station 141.73	559.61	558.39	556.56	555.64	553.25	550.79	548.11	542.40

1. WSE reported as the maximum water surface within the channel. The overbank WSE will vary depending on breach.

Jefferson City - CA8 - Summary of Peak Flows (cfs)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93	815,722	743,003	631,853	586,596	449,497	347,297	289,888	212,259
Missouri River, River Station 143.14	730,721	666,576	571,081	534,490	433,777	345,571	289,890	212,262
Missouri River, River Station 141.73	740,022	678,504	583,569	543,106	433,905	345,667	289,904	212,289

1. Flow calculations include both the 1D channel and 2D overbanks (determined in RAS Mapper by drawing a profile cross-section across the channel and overbanks) .
2. Levee were assumed to breach when WSE over levee exceeds 1-ft at lowest point

Jefferson City - CA8 - Summary of Maximum Depths (ft NAVD88)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93 Min. Channel Elev = 503.938	57.7	56.6	54.9	54.1	51.4	48.9	46.1	40.2
Missouri River, River Station 143.14 Min. Channel Elev = 506.688	54.2	53.1	51.5	50.7	48.3	45.8	43.0	37.1
Missouri River, River Station 141.73 Min. Channel Elev = 503.079	56.5	55.3	53.5	52.6	50.2	47.7	45.0	39.3

1. Depths differ dramatically between river channel and overbanks. These depths are measured from the minimum channel elevation. Channel elevations are listed in table.

Jefferson City - CA9 - Summary of Maximum Water Surface Elevations (ft NAVD88)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93	561.52	560.41	558.76	557.95	555.22	552.66	549.79	544.16
Missouri River, River Station 143.14	560.79	559.67	558.06	557.28	554.76	552.28	549.39	543.74
Missouri River, River Station 141.73	559.64	558.41	556.55	555.59	553.17	550.90	548.01	542.38

1. WSE reported as the maximum water surface within the channel. The overbank WSE will vary depending on breach.

Jefferson City - CA9 - Summary of Peak Flows (cfs)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93	845,720	770,513	655,153	607,711	462,489	357,399	292,606	212,261
Missouri River, River Station 143.14	735,611	670,933	574,482	537,503	433,864	346,051	288,990	212,219
Missouri River, River Station 141.73	739,352	677,722	582,528	541,934	431,629	345,764	289,300	212,220

1. Flow calculations include both the 1D channel and 2D overbanks (determined in RAS Mapper by drawing a profile cross-section across the channel and overbanks) .
2. Levee were assumed to breach when WSE over levee exceeds 1-ft at lowest point

Jefferson City - CA9 - Summary of Maximum Depths (ft NAVD88)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93 Min. Channel Elev = 503.938	57.6	56.5	54.8	54.0	51.3	48.7	45.9	40.2
Missouri River, River Station 143.14 Min. Channel Elev = 506.688	54.1	53.0	51.4	50.6	48.1	45.6	42.7	37.1
Missouri River, River Station 141.73 Min. Channel Elev = 503.079	56.6	55.3	53.5	52.5	50.1	47.8	44.9	39.3

1. Depths differ dramatically between river channel and overbanks. These depths are measured from the minimum channel elevation. Channel elevations are listed in table.

Jefferson City - CA13 - Summary of Maximum Water Surface Elevations (ft NAVD88)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93	561.92	560.90	559.11	558.13	555.28	552.69	549.81	544.16
Missouri River, River Station 143.14	561.26	560.25	558.50	557.56	554.83	552.30	549.41	543.75
Missouri River, River Station 141.73	559.69	558.35	556.52	555.64	553.18	550.90	548.03	542.39

1. WSE reported as the maximum water surface within the channel. The overbank WSE will vary depending on breach.

Jefferson City - CA13 - Summary of Peak Flows (cfs)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93	745,514	682,887	586,342	546,809	434,348	346,327	289,676	212,255
Missouri River, River Station 143.14	732,981	680,113	588,495	548,668	435,264	346,126	289,352	212,222
Missouri River, River Station 141.73	737,427	675,972	581,539	542,942	432,793	345,801	289,685	212,226

1. Flow calculations include both the 1D channel and 2D overbanks (determined in RAS Mapper by drawing a profile cross-section across the channel and overbanks) .
2. Levee were assumed to breach when WSE over levee exceeds 1-ft at lowest point

Jefferson City - CA13 - Summary of Maximum Depths (ft NAVD88)

Index Point	AEP							
	500YR	200YR	100YR	50YR	25YR	10YR	5YR	2YR
	0.002	**0.005**	**0.01**	**0.02**	**0.04**	**0.1**	**0.2**	**0.5**
Missouri River, River Station 143.93 Min. Channel Elev = 503.938	58.0	57.0	55.2	54.2	51.3	48.7	45.9	40.2
Missouri River, River Station 143.14 Min. Channel Elev = 506.688	54.6	53.6	51.8	50.9	48.1	45.6	42.7	37.1
Missouri River, River Station 141.73 Min. Channel Elev = 503.079	56.6	55.3	53.4	52.6	50.1	47.8	44.9	39.3

1. Depths differ dramatically between river channel and overbanks. These depths are measured from the minimum channel elevation. Channel elevations are listed in table.

ATTACHMENT D – CHANGES IN INUNDATION PATTERNS

1

JEFFERSON CITY FEASIBILITY IMPACTS SUMMARY

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- One alternative (CA13) with hydraulic changes / impacts is carried forward to the TSP final array. The H&H appendix will show this alternative with, without, and with partial removal of the existing Capital View non-federal levee. Alternatives seeking public input include CA13 without the existing Capital View levee and with partial removal of the existing Capital View levee. As such, results are presented here for both conditions.

- Hydraulic impacts are minor in scale relative to the overall Missouri River system due to:
  - bluff-to-bluff flow limits change in the inundation footprint,
  - in the existing conditions, the vertical nature of the floodplain and low non-federal levee systems in the area result in frequent inundation of the full floodplain (10% AEP+),
  - the proposed alternative results in only local changes in water surface elevation,
  - the proposed alternative with removal of the existing Capital View levee results in positive impacts for many properties and a lower water surface elevation at the mouth of Wears Creek for frequent storms,
  - and in most cases the proposed alternative results in upstream and downstream increases in depth at locations that would already be significantly inundated in the FWOP.

1

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FINAL ARRAY OF ALTERNATIVES

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Combined Alternative	Description	Hydraulic Impacts
CA0	**No Action Alternative**	No adjacent hydraulic impacts
~~CA1~~	~~Existing Capital View Alignment, 34 ft Stage~~	
~~CA2~~	~~Existing Capital View Alignment Setback, 34 ft Stage, Turkey Creek & Riverside Setback ~1000 ft~~	
~~CA3~~	~~Existing Capital View Alignment, No raise, Reinforce existing Levee~~	
~~CA4~~	~~Optimization Alignment: 37 ft Stage, Existing Capital View Levee to Remain~~	
~~CA5~~	~~Optimization Alignment: 37 ft Stage, Remove Existing Capital View Levee~~	
~~CA6~~	~~Historic L-142 Alignment & Profile, Existing Capital View Levee to Remain~~	
~~CA7~~	~~Historic L-142 Alignment & Profile, Remove Existing Capital View Levee~~	
~~CA8~~	~~Highway Alignment, Existing Capital View Levee to Remain~~	
~~CA9~~	~~Highway Alignment, Remove Existing Capital View Levee~~	
~~CA10~~	~~Limited resiliency measures outside of Capital View Footprint~~	
CA11	**Non-Structural Measures**	No adjacent hydraulic impacts
~~CA12~~	~~Remove Church Farm Levee~~	
CA13	**Hybrid Alignment, Profile at 1% AEP, Partial Capital View Removal**	Included with Partial Capital View levee removal and full Capital View levee removal

2

ASSUMPTIONS AND LIMITATIONS

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Modeling impacts using an unsteady flow analysis is dependent upon assumptions of flow hydrograph shape and levee performance. Hydrographs are based on the shape of the 2018 flood and multiplying flows up or down to meet the peak flow rates of various frequency floods based on methodology from EM 1110-2-1415. Hydrograph shapes from other events could result in longer or shorter duration of high flows than as assumed and could potentially alter results by filling more or less of the leveed areas in smaller events that do not fully inundate the floodplain. Levee breach was assumed with 1 feet of overtopping, whereas the previous 2003 UMRSFFS assumed breach occurs at or below top of levee which would tend to estimate lower stages than this study. However, breach could occur earlier, later, or not occur during a given flood event.

For duration analysis, areas landward of levees are complex to model. After overtopping, as the flood recedes, water would flow out through the levee breaches (if formed) or be trapped within the leveed area at the elevation of the lowest point on the top of levee and would only able to flow out through small gravity drains with their sluice gates left open. Many levee districts intentionally breach levees to drain in this scenario after the river recedes below top of levee (see 2013 Tebbets Levee example). As the time to drain the levees after a flood is based on the volume of water below top of levee inside of each levee, which is not impacted by the project, coupled with actions and levee performance outside of USACE control, the best assumption is that there is no difference in ponding duration for levee protected areas. For riverside areas, assumptions applied for hydrograph shape previously discussed will be used to assess duration impacts.

Aside from the Osage, Moreau, and Gasconade Rivers, all other tributaries shown are modeled for Missouri River backwater impacts only. Flows from these streams will result in more frequent flooding at the tributaries than shown on the maps but would usually occur at different timing than Missouri River floods.

Analysis conducted here is not using the FEMA effective HEC-RAS model, which is a steady-state model of only the 1% AEP event and would not be capable of computing downstream impacts from changes to levee overtopping volumes. For a new levee system that does not overtop in a 1% AEP event, it should be assumed that separate modeling coordination with FEMA and remapping will be required during design and post-project, and that it cannot be built with a “no-rise” certification. New levee alignments are on the edge of the mapped floodway to the extent feasible.

SUMMARY OF ADJACENT LEVEE OVERTOPPING AEP VALUES IN FWOP AND CA13

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UPSTREAM LEVEE OVERTOPPING FLOWS AT JEFF CITY GAGE AND ASSOCIATED AEP

	FWOP			Alt 13 Capital View Removed				Alt 13 Capital View Partial Removal			
	% AEP	YR AEP	cfs	% AEP	YR AEP	cfs	% Δ from FWOP	% AEP	YR AEP	cfs	% Δ from FWOP
RENZ	17.3	5.8	299,000	16.4	6.1	302,000	-0.9	17.3	5.8	299,000	0
CEDAR CREEK SEC. 2	5.3	18.9	397,000	5.6	17.9	391,000	0.3	5.8	17.2	387,000	0.5
RENZ NORTH	21.6	4.6	281,000	21.1	4.7	283,000	-0.5	21.6	4.6	281,000	0
COLE JUNCTION	6.1	16.4	383,000	6.2	16.1	381,000	0.1	6.3	15.9	379,000	0.2
BOONE CO, MO 1	21.4	4.7	281,000	21	4.8	284,000	-0.4	21.4	4.7	281,000	0
CHURCH FARM (PRISON FARM)	14.4	6.9	312,000	14.6	6.8	311,000	0.2	15.0	6.7	309,000	0.6
HARTSBURG	17.6	5.7	298,000	18.8	5.3	298,000	1.2	17.7	5.6	297,000	0.1

DOWNSTREAM LEVEE OVERTOPPING FLOWS AT JEFF CITY GAGE AND ASSOCIATED AEP

	FWOP			Alt 13 Capital View Removed				Alt 13 Capital View Partial Removal			
	% AEP	YR AEP	cfs	% AEP	YR AEP	cfs	% Δ from FWOP	% AEP	YR AEP	cfs	% Δ from FWOP
CEDAR CREEK SEC. 1	21.8	4.6	279,000	24.7	4.0	265,000	2.9	22.5	4.4	275,000	0.7
REVEAUX	14.4	6.9	312,000	15.6	6.4	306,000	1.2	15.3	6.5	307,000	0.9
RIEVAUX	20.7	4.8	286,000	20.4	4.9	287,000	-0.3	20.7	4.8	286,000	0
WAINWRIGHT	21.4	4.7	281,000	21.2	4.7	283,000	-0.2	21.5	4.7	281,000	0.1

** green highlighted cells are an improvement (less frequent) in overtopping frequency with Alternative construction vs FWOP condition.*

MAX DEPTH AND WATER SURFACE ELEVATION COMPARISON
FWOP CONDITION
&
CA13 REMOVING EXISTING CAPITAL VIEW LEVEE

Draft Report

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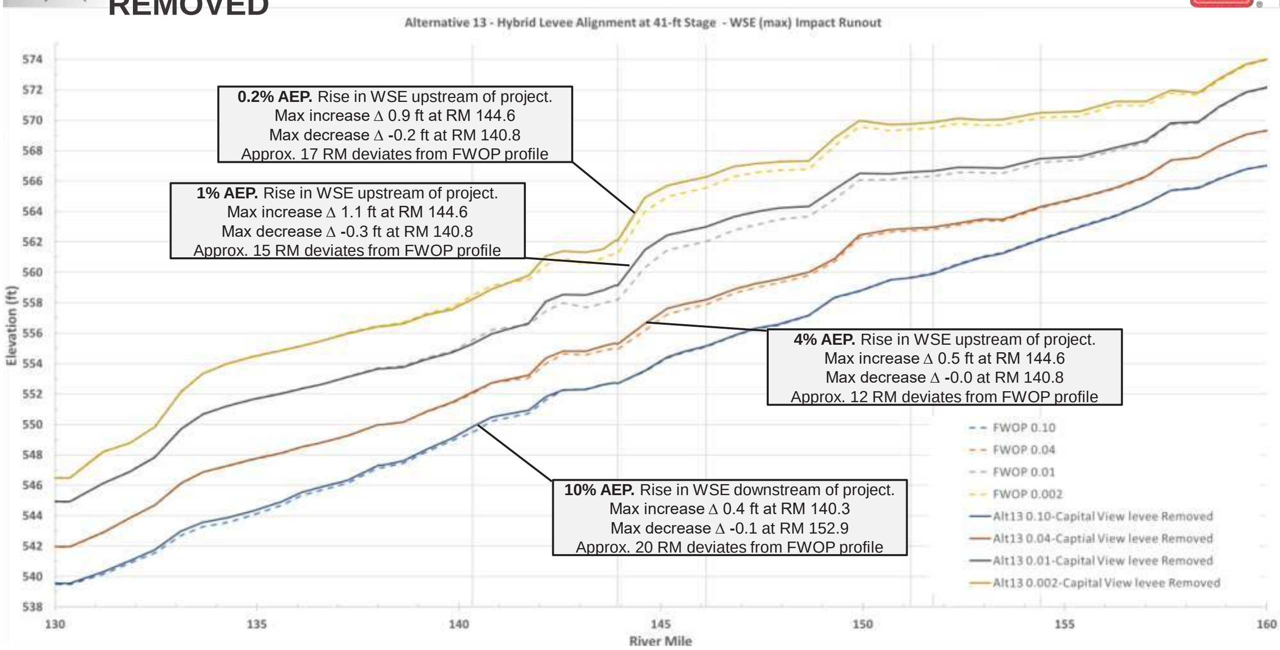
MISSOURI RIVER CHANNEL MAX. WSE PROFILE-CAPITAL VIEW REMOVED



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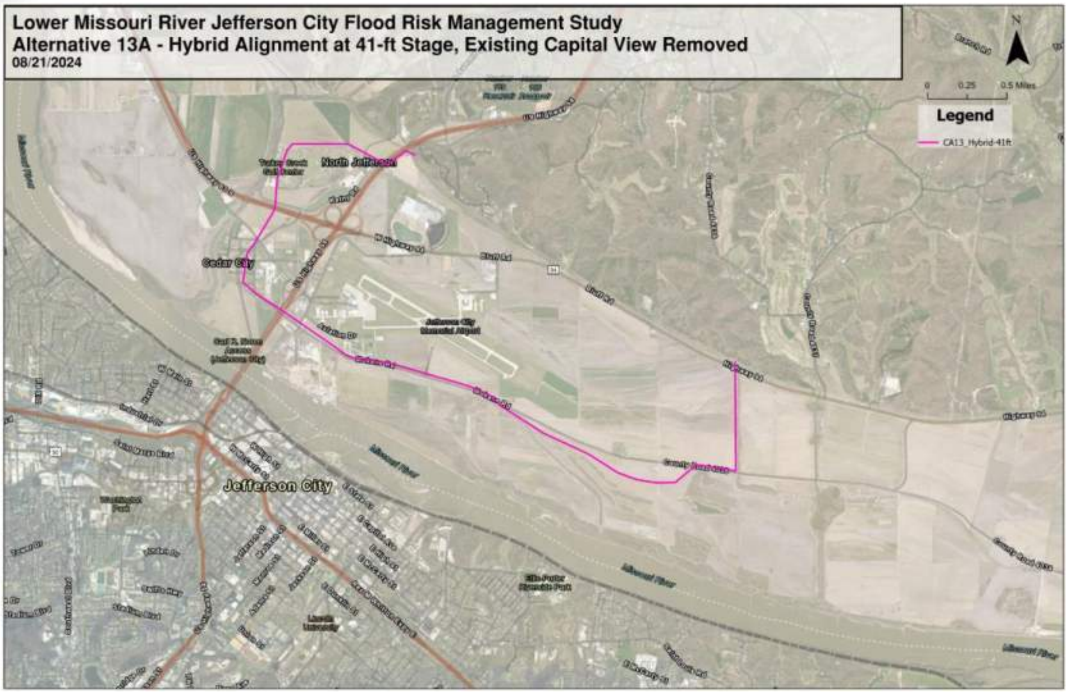

50% AEP (1/2 YEAR)

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FWOP 50% AEP INUNDATION

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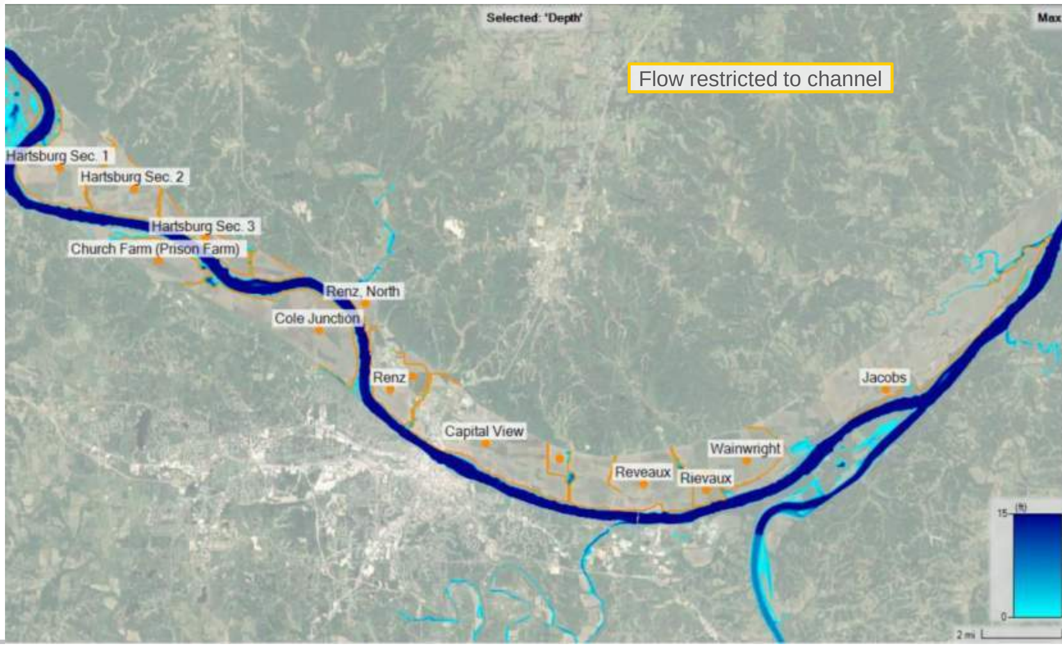

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CA13 50% AEP INUNDATION – CAPITAL VIEW REMOVED

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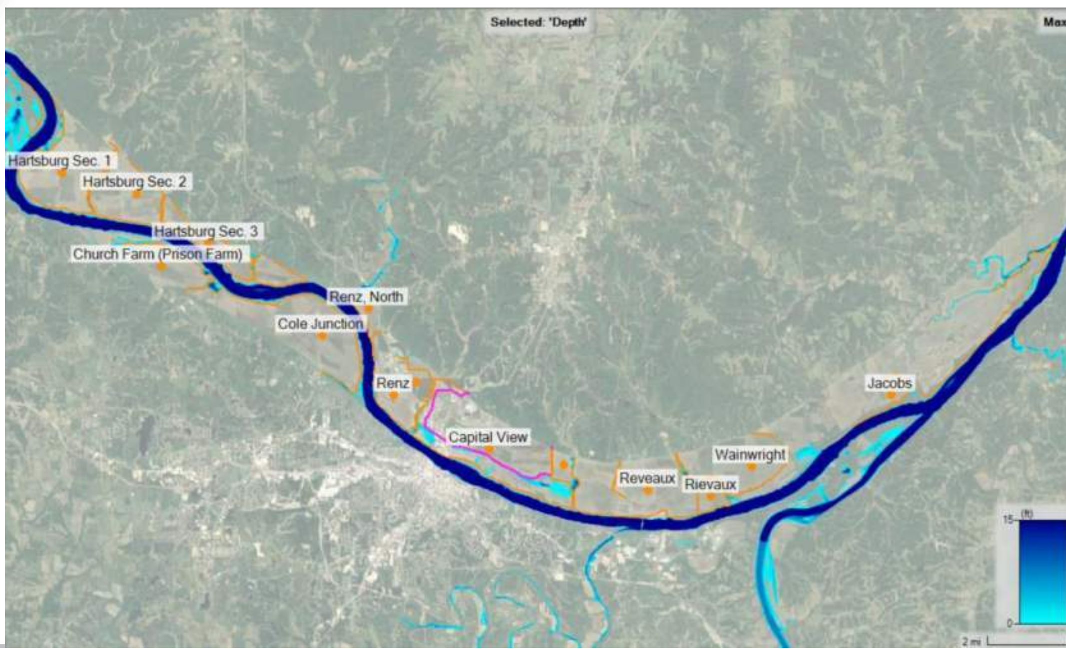
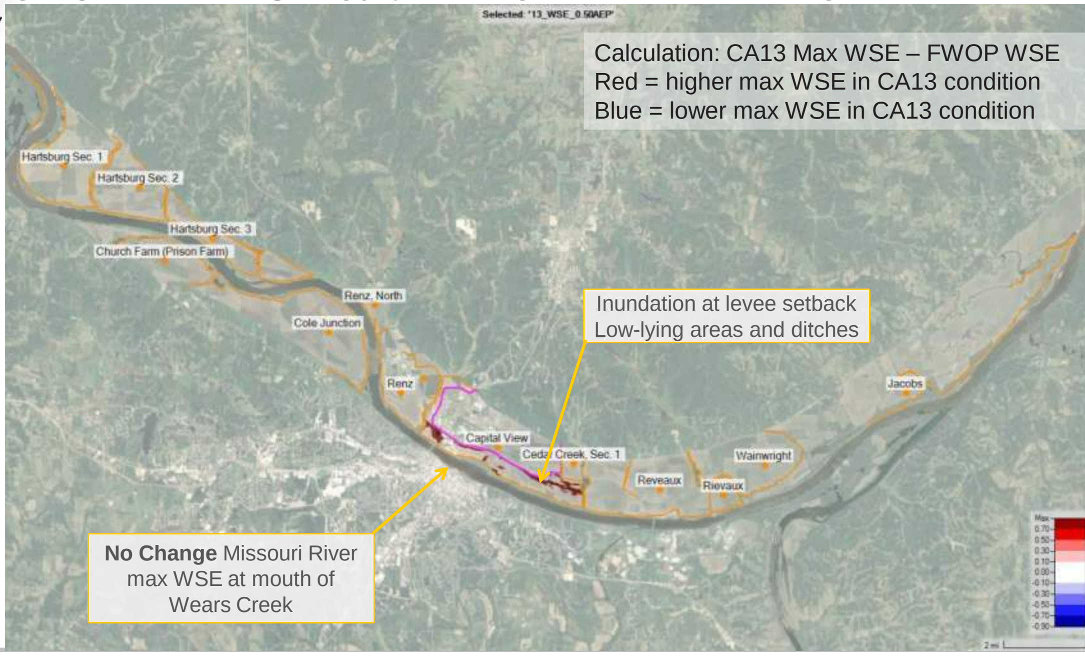


CA13 – ∆ MAX WSE - 50% AEP – CAPITAL VIEW REMOVED



20% AEP (1/5 YEAR)

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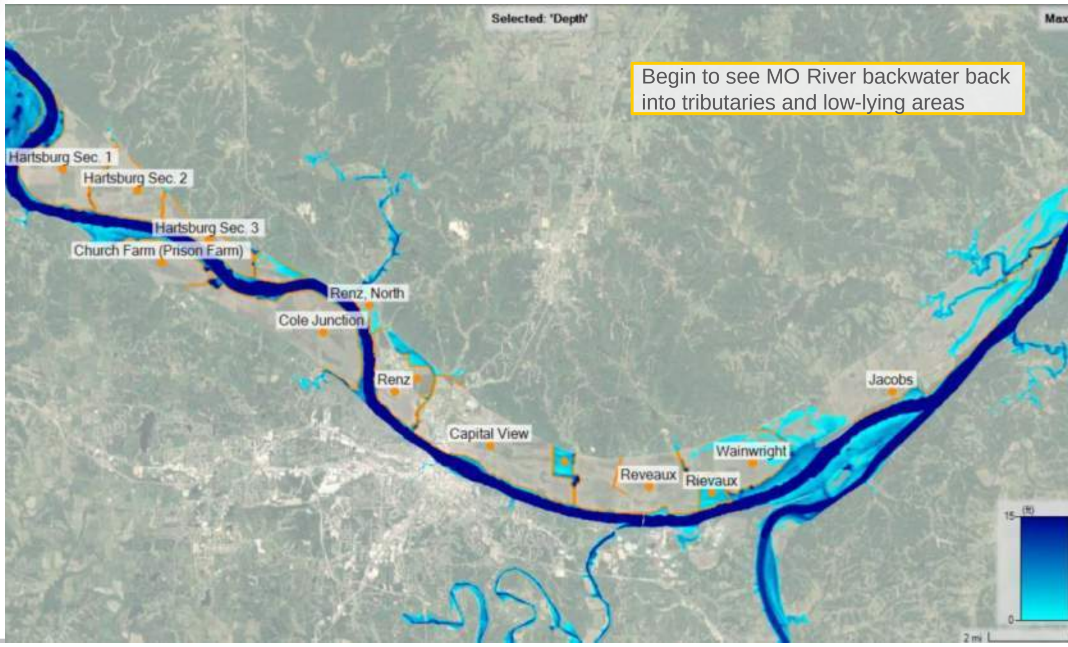
FWOP 20% AEP INUNDATION



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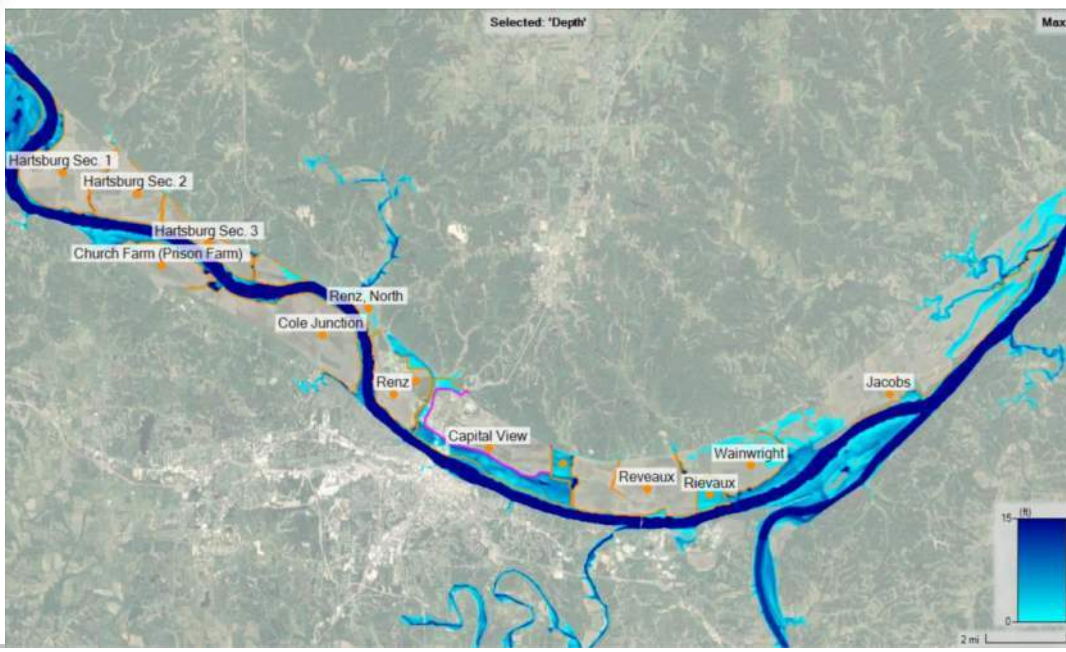
CA13 20% AEP INUNDATION – CAPITAL VIEW REMOVED



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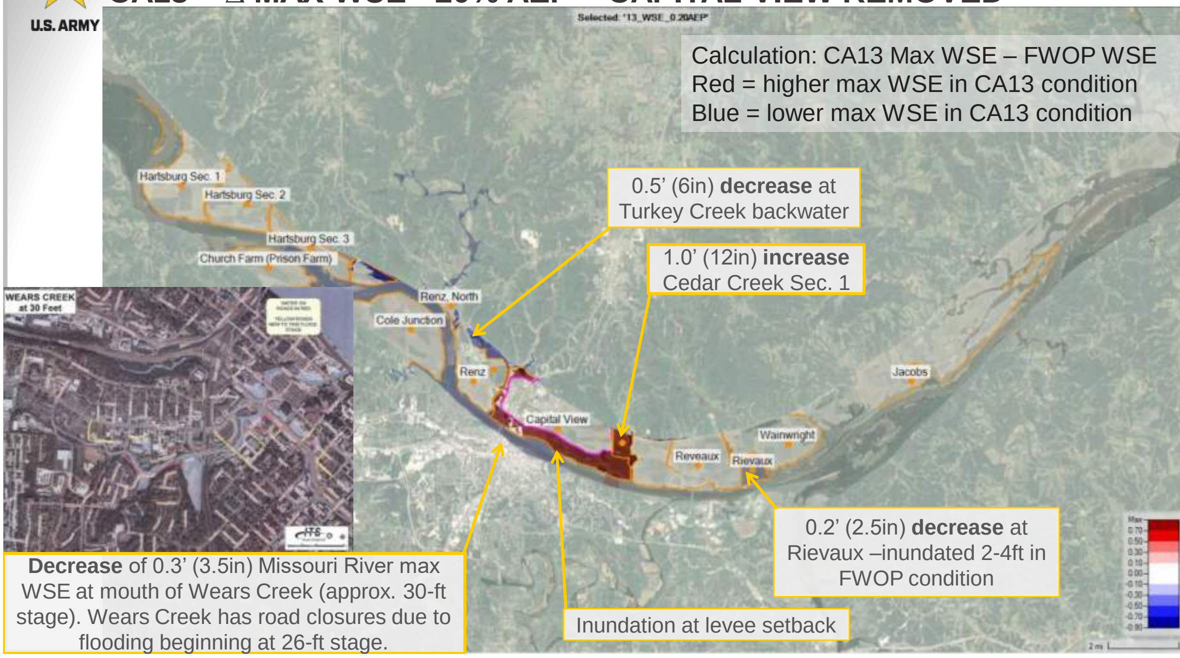
CA13 – Δ MAX WSE - 20% AEP – CAPITAL VIEW REMOVED



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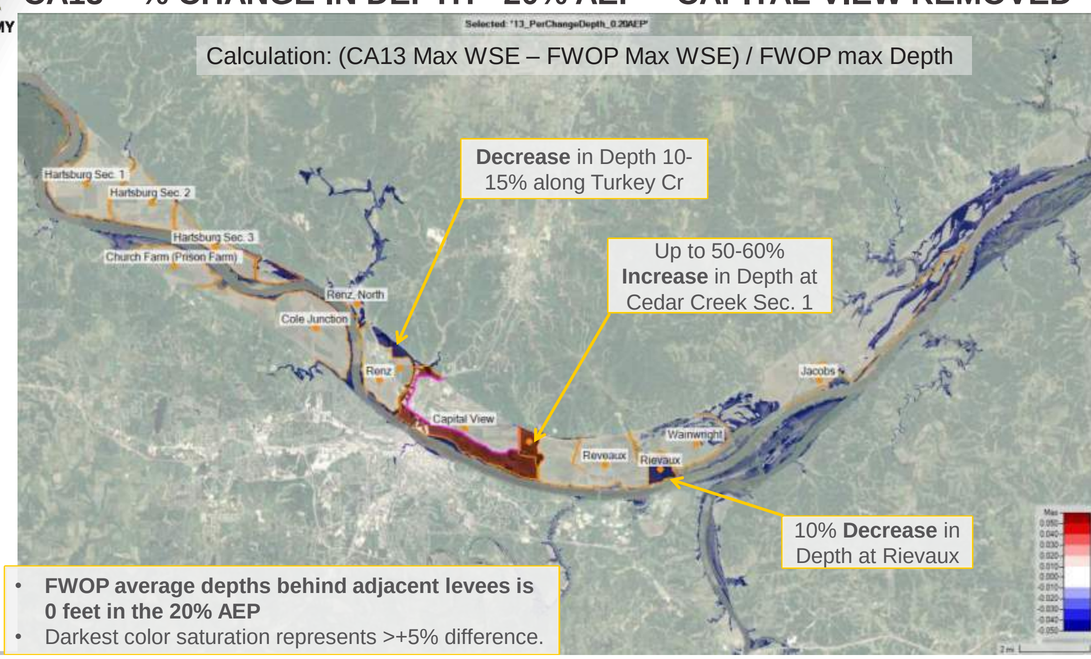
CA13 – % CHANGE IN DEPTH - 20% AEP – CAPITAL VIEW REMOVED



10% AEP (1/10 YEAR)

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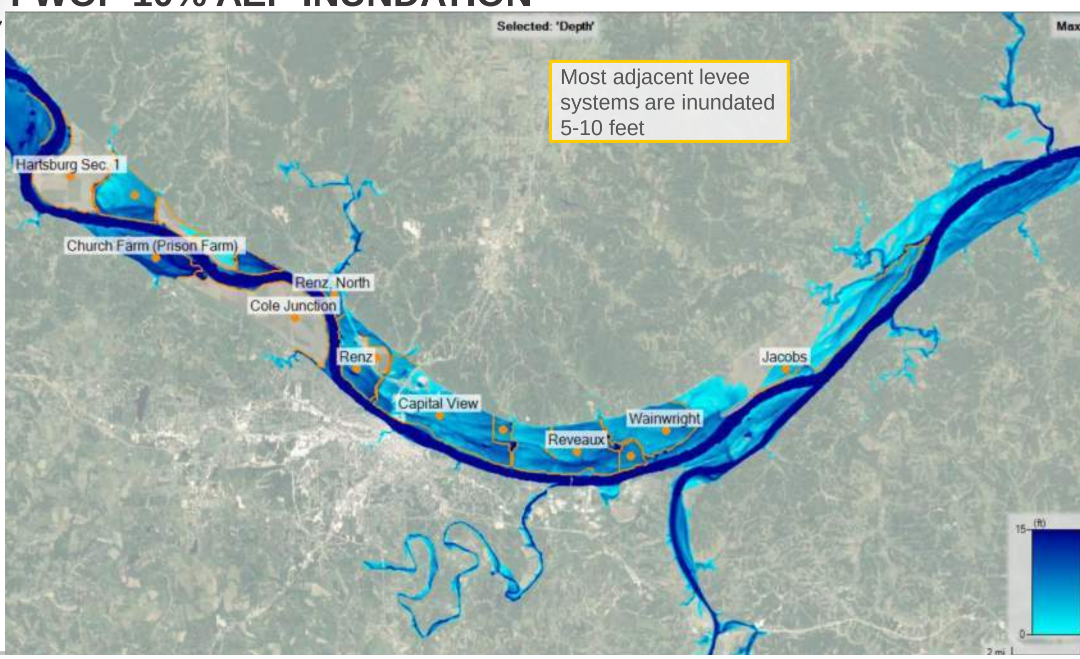
FWOP 10% AEP INUNDATION



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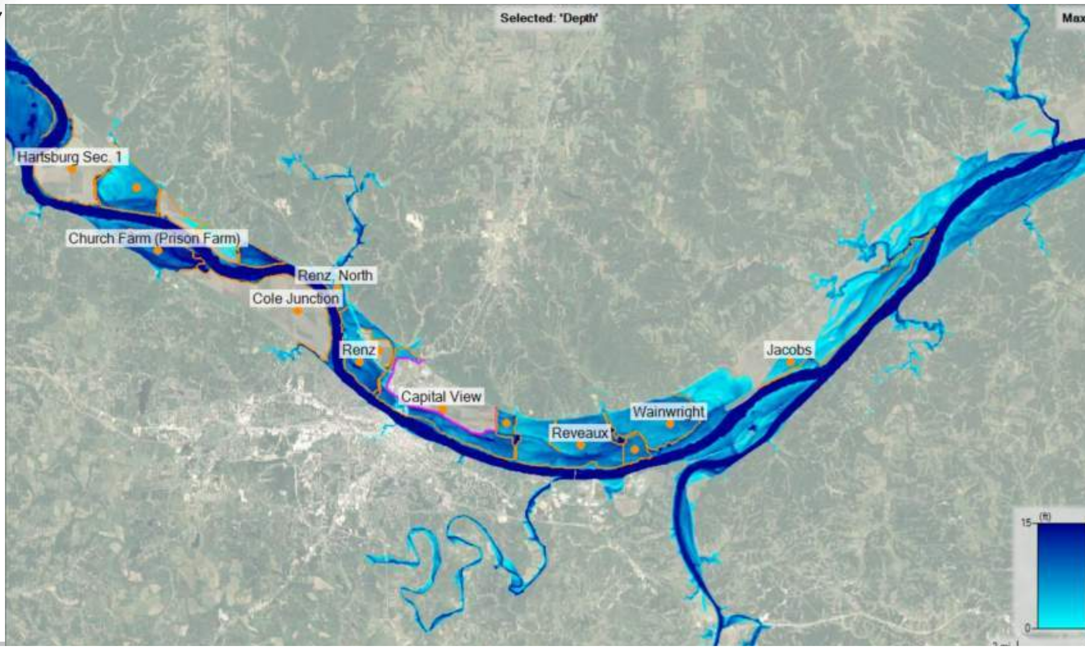
CA13 10% AEP INUNDATION – CAPITAL VIEW REMOVED



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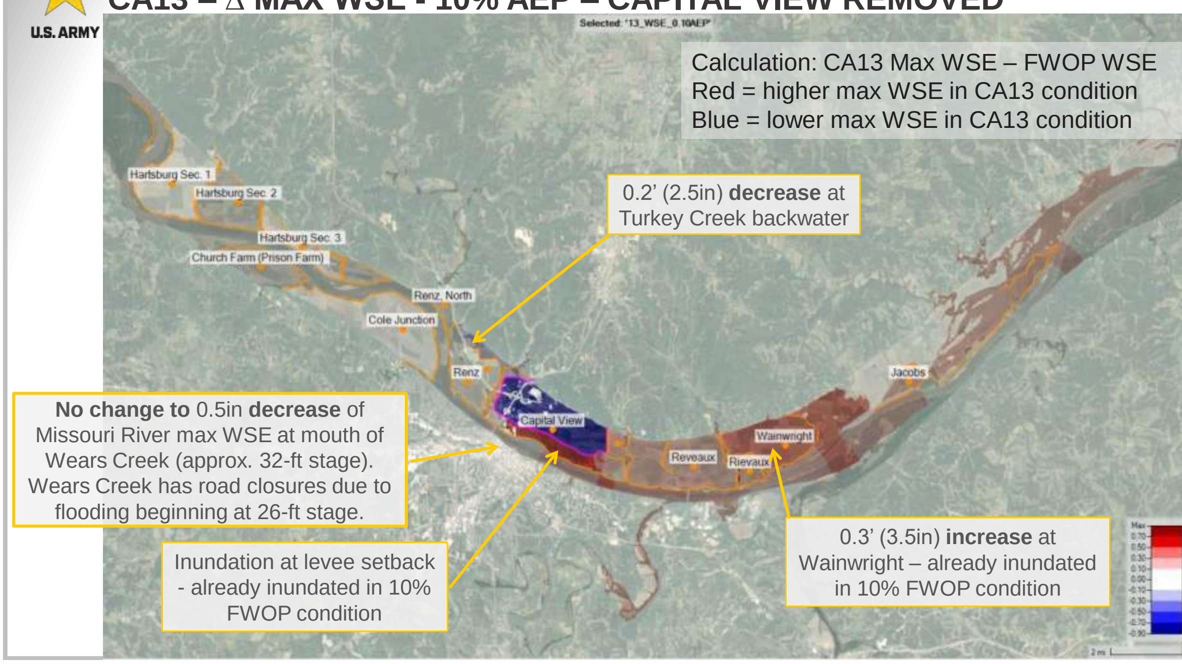
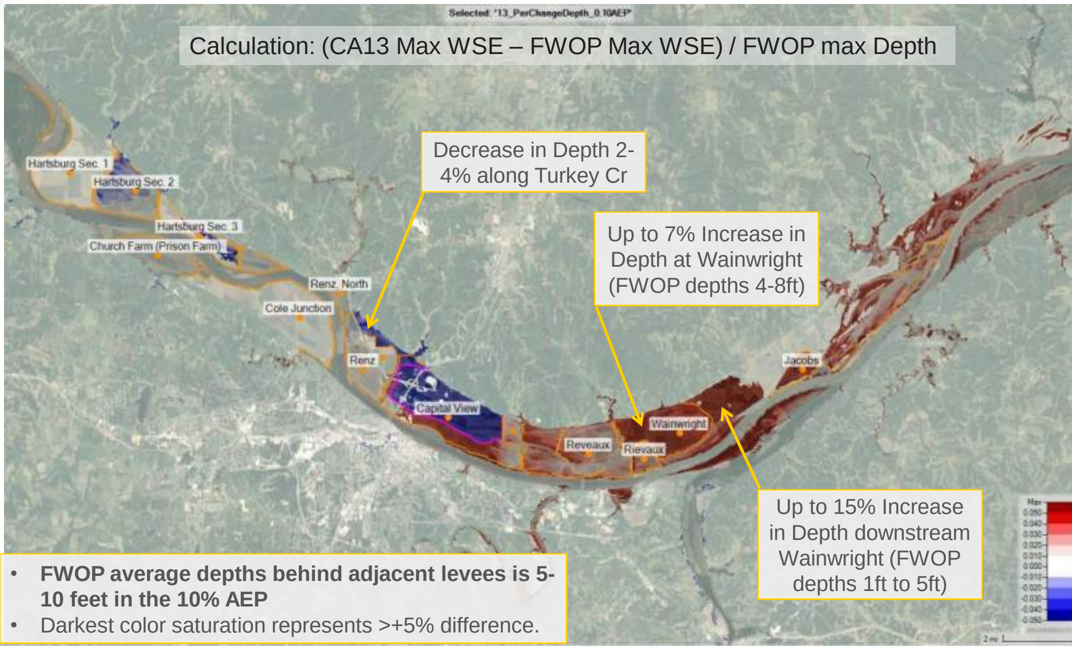
CA13 – ∆ MAX WSE - 10% AEP – CAPITAL VIEW REMOVED



20

U.S. ARMY

CA13 – % CHANGE IN DEPTH - 10% AEP – CAPITAL VIEW REMOVED



- **FWOP average depths behind adjacent levees is 5-10 feet in the 10% AEP**
- Darkest color saturation represents >+5% difference.

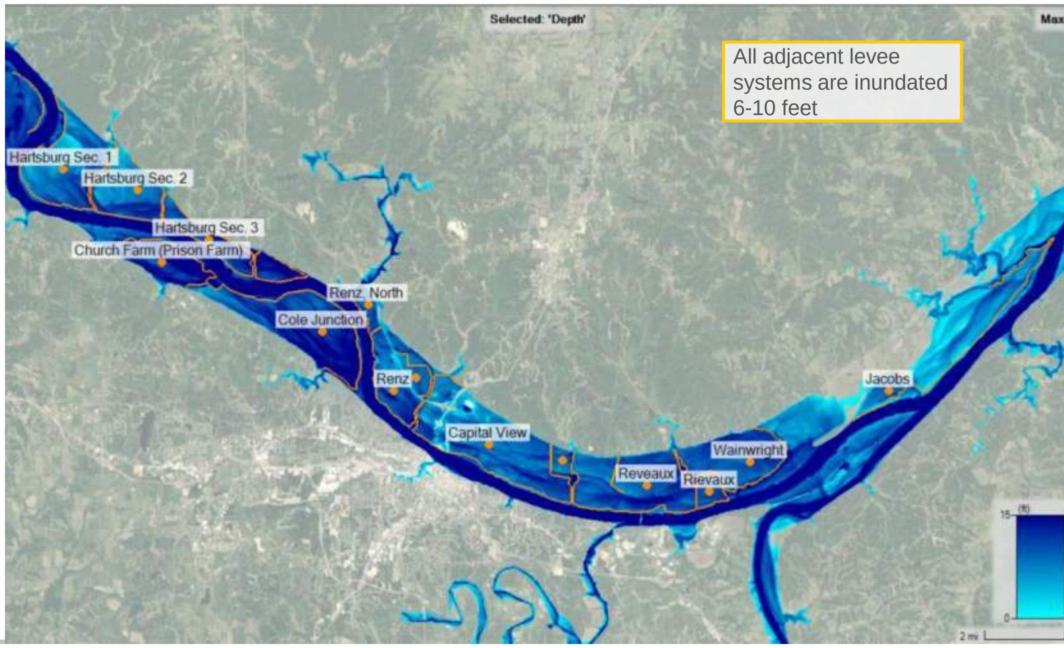
4% AEP (1/25 YEAR)

U.S. ARMY

US Army Corps of Engineers

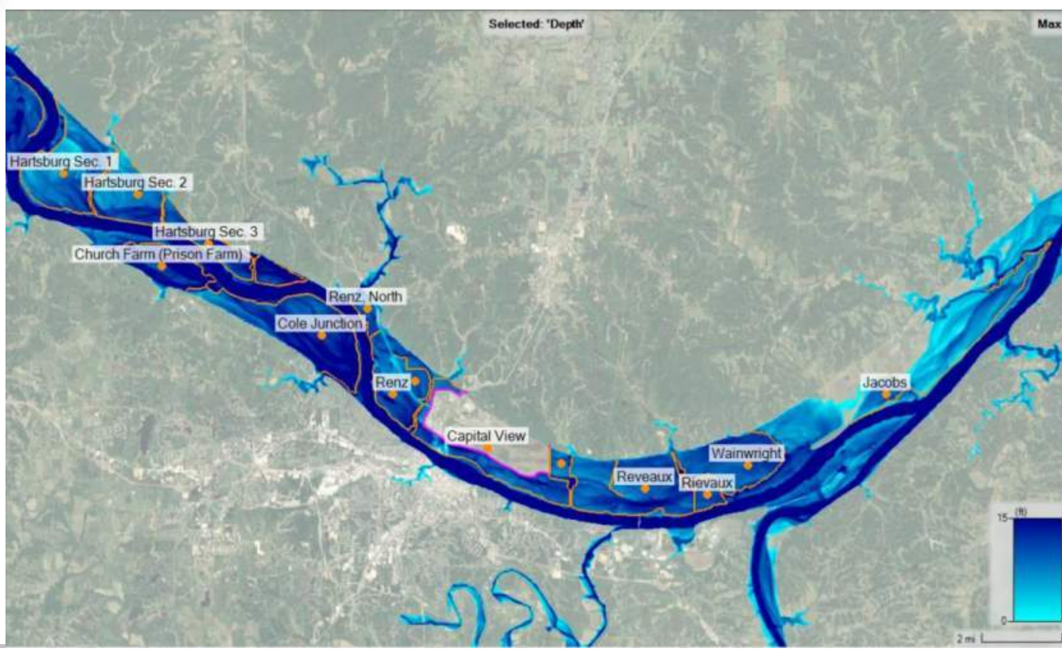
U.S. ARMY

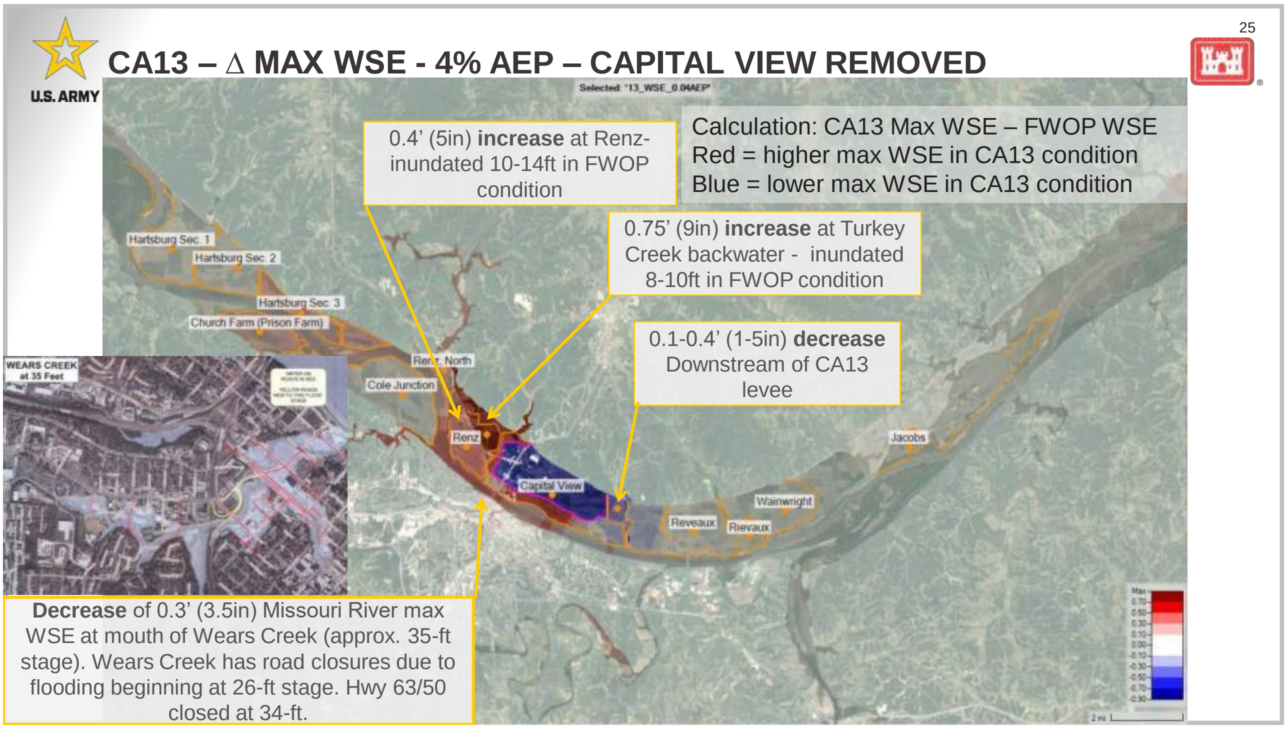
FWOP 4% AEP INUNDATION



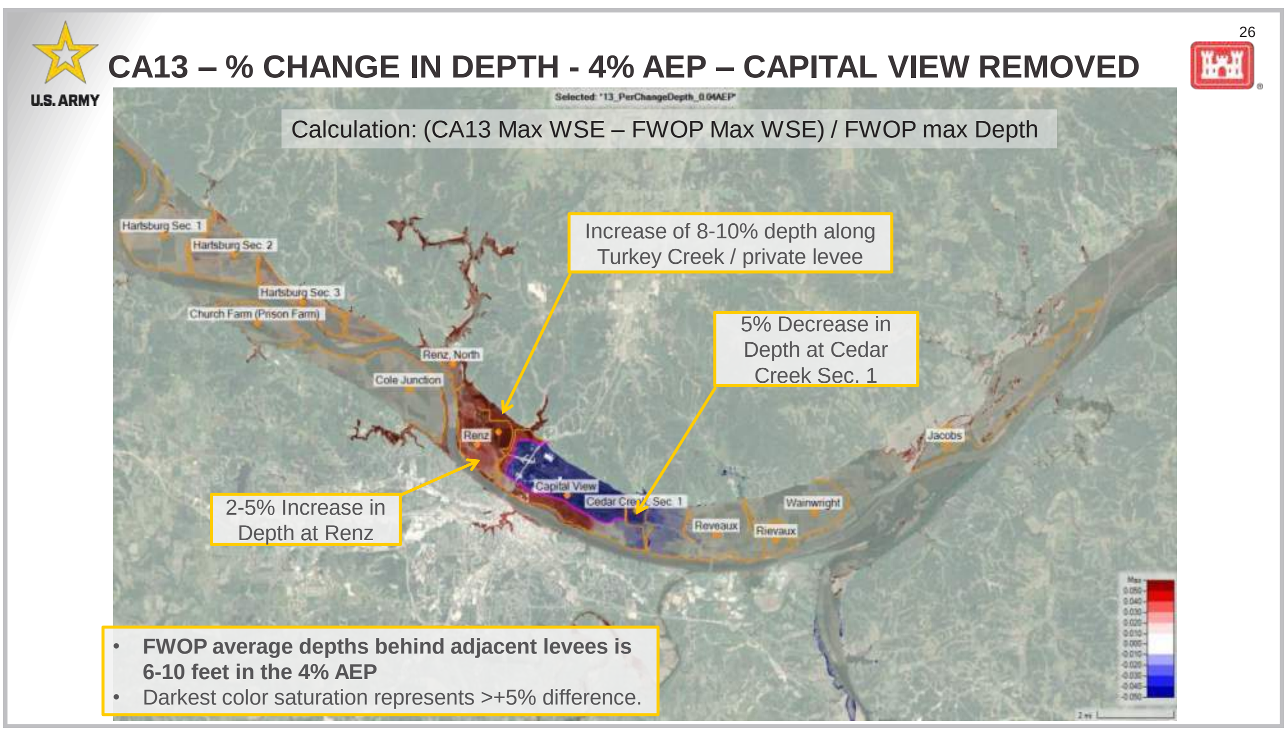
U.S. ARMY

CA13 4% AEP INUNDATION – CAPITAL VIEW REMOVED




25
CA13 – ∆ MAX WSE - 4% AEP – CAPITAL VIEW REMOVED
U.S. ARMY
Selected: "13_WSE_0.04AEP"
0.4' (5in) **increase** at Renz- inundated 10-14ft in FWOP condition
Calculation: CA13 Max WSE – FWOP WSE
Red = higher max WSE in CA13 condition
Blue = lower max WSE in CA13 condition
0.75' (9in) **increase** at Turkey Creek backwater - inundated 8-10ft in FWOP condition
0.1-0.4' (1-5in) **decrease** Downstream of CA13 levee
Hartsburg Sec. 1
Hartsburg Sec. 2
Hartsburg Sec. 3
Church Farm (Prison Farm)
Renz, North
Cole Junction
Renz
Capital View
Jacobs
Wainwright
Reveaux
Rievaux
WEARS CREEK at 35 Feet
Decrease of 0.3' (3.5in) Missouri River max WSE at mouth of Wears Creek (approx. 35-ft stage). Wears Creek has road closures due to flooding beginning at 26-ft stage. Hwy 63/50 closed at 34-ft.

25


26
CA13 – % CHANGE IN DEPTH - 4% AEP – CAPITAL VIEW REMOVED
U.S. ARMY
Selected: "13_PerChangeDepth_0.04AEP"
Calculation: (CA13 Max WSE – FWOP Max WSE) / FWOP max Depth
Increase of 8-10% depth along Turkey Creek / private levee
5% Decrease in Depth at Cedar Creek Sec. 1
2-5% Increase in Depth at Renz
Hartsburg Sec. 1
Hartsburg Sec. 2
Hartsburg Sec. 3
Church Farm (Prison Farm)
Renz, North
Cole Junction
Renz
Capital View
Jacobs
Wainwright
Reveaux
Rievaux
• **FWOP average depths behind adjacent levees is 6-10 feet in the 4% AEP**
• Darkest color saturation represents >+5% difference.

26

2% AEP (1/50 YEAR)

U.S. ARMY

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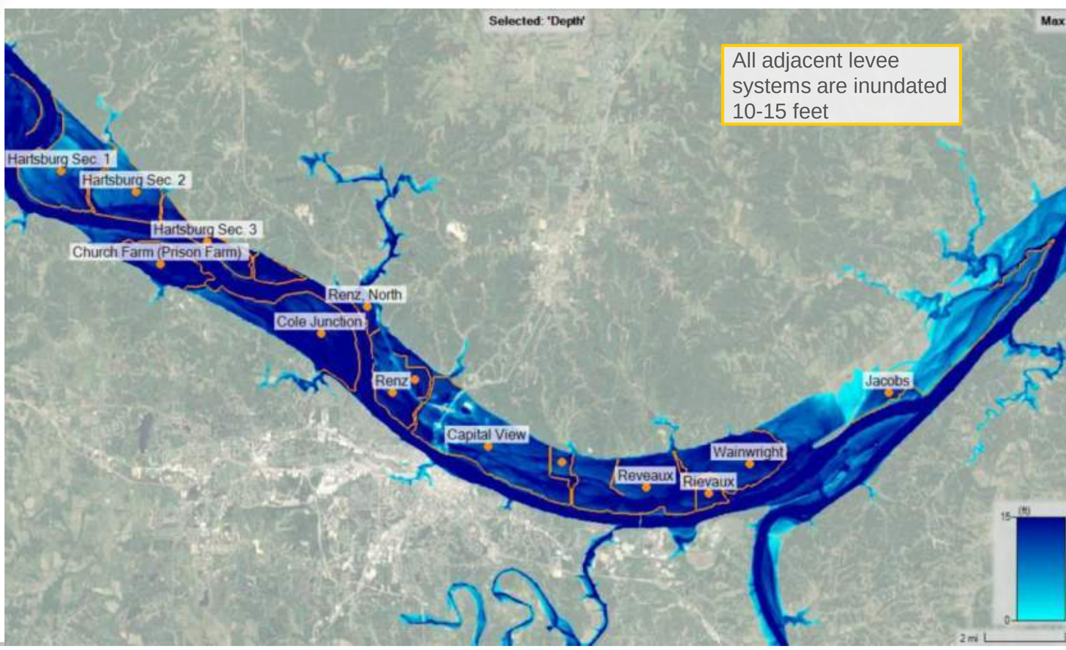
U.S. ARMY

FWOP 2% AEP INUNDATION

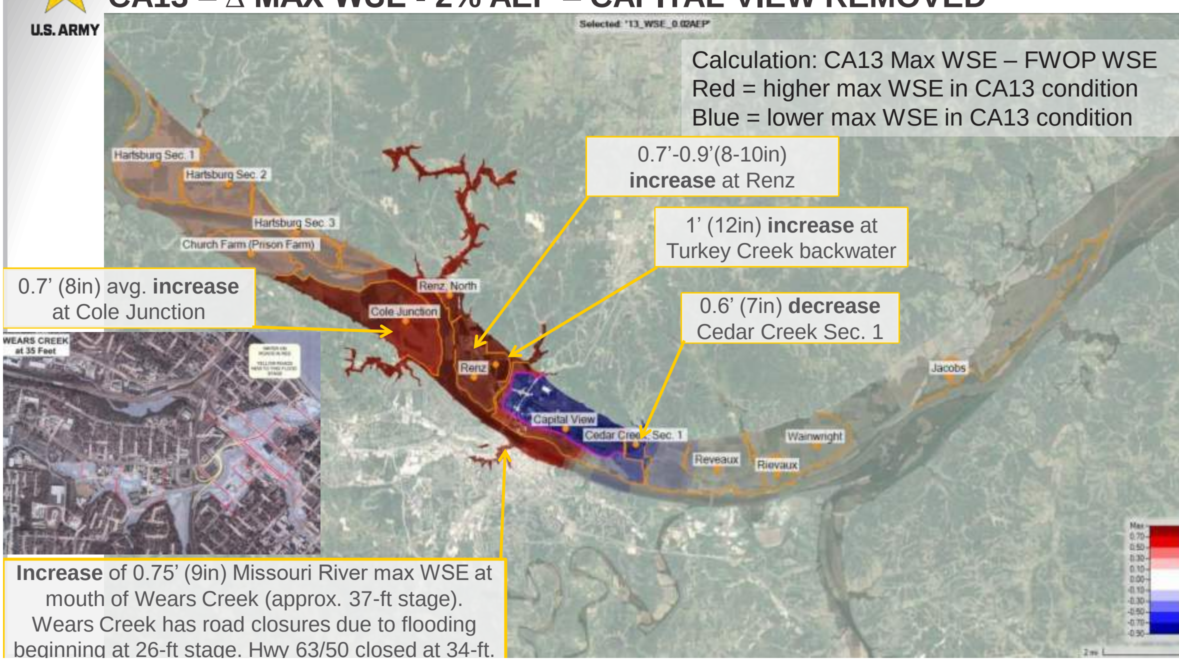


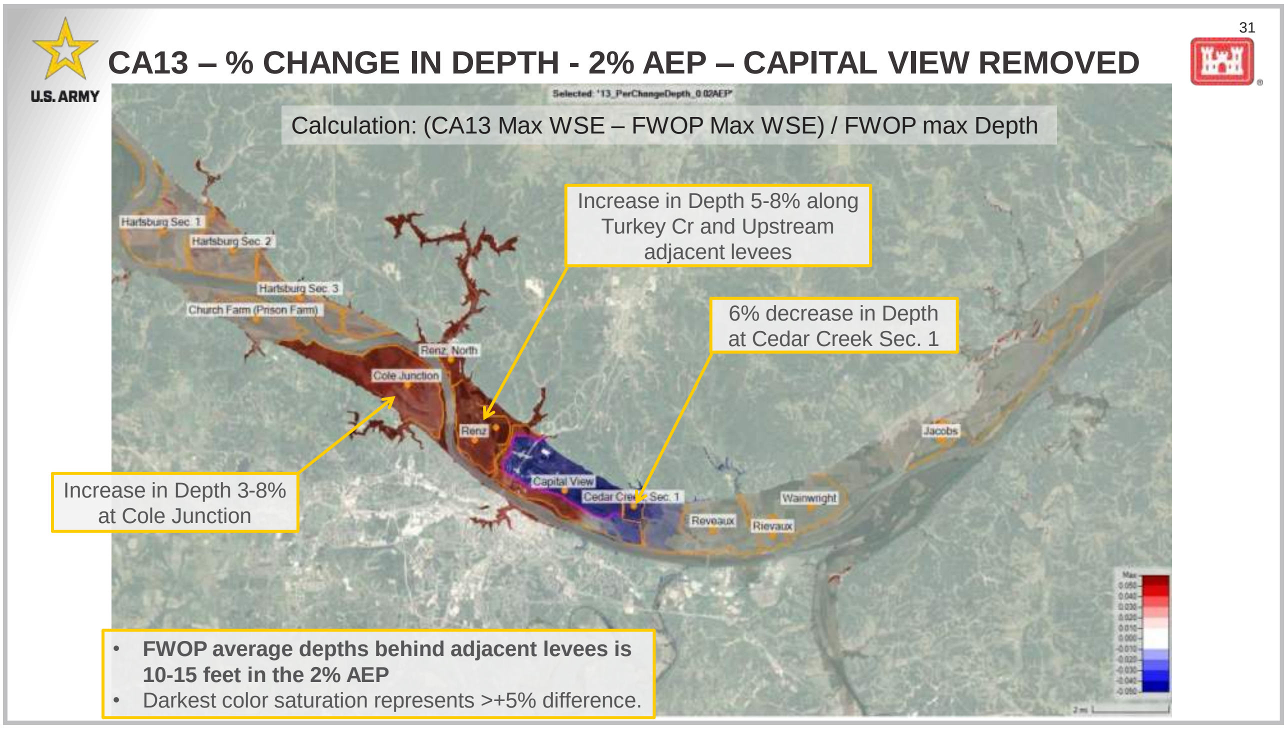
U.S. ARMY

CA13 2% AEP INUNDATION – CAPITAL VIEW REMOVED

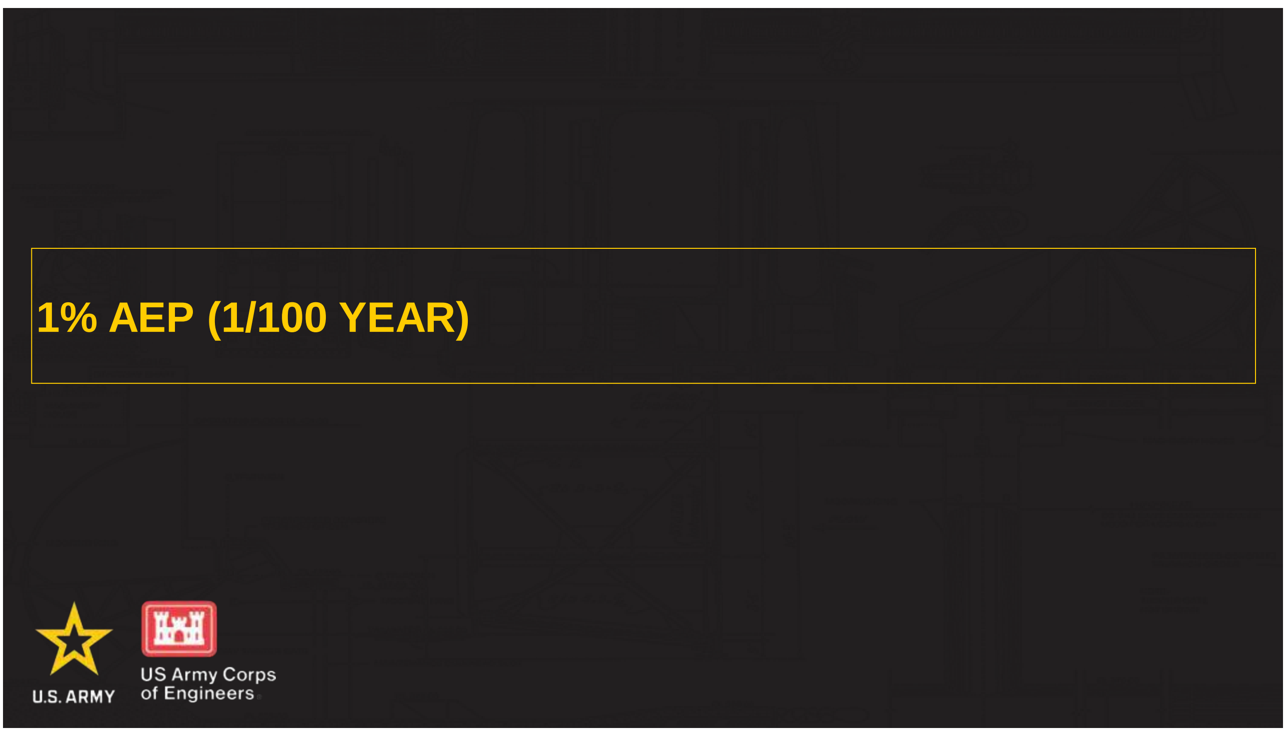
U.S. ARMY

CA13 – ∆ MAX WSE - 2% AEP – CAPITAL VIEW REMOVED




31
CA13 – % CHANGE IN DEPTH - 2% AEP – CAPITAL VIEW REMOVED
U.S. ARMY
Calculation: (CA13 Max WSE – FWOP Max WSE) / FWOP max Depth
Increase in Depth 5-8% along Turkey Cr and Upstream adjacent levees
6% decrease in Depth at Cedar Creek Sec. 1
Increase in Depth 3-8% at Cole Junction
Hartsburg Sec. 1
Hartsburg Sec. 2
Hartsburg Sec. 3
Church Farm (Prison Farm)
Renz North
Cole Junction
Renz
Capital View
Reveaux
Rievaux
Wainwright
Jacobs
• FWOP average depths behind adjacent levees is 10-15 feet in the 2% AEP
• Darkest color saturation represents >+5% difference.

31


1% AEP (1/100 YEAR)
U.S. ARMY
US Army Corps of Engineers

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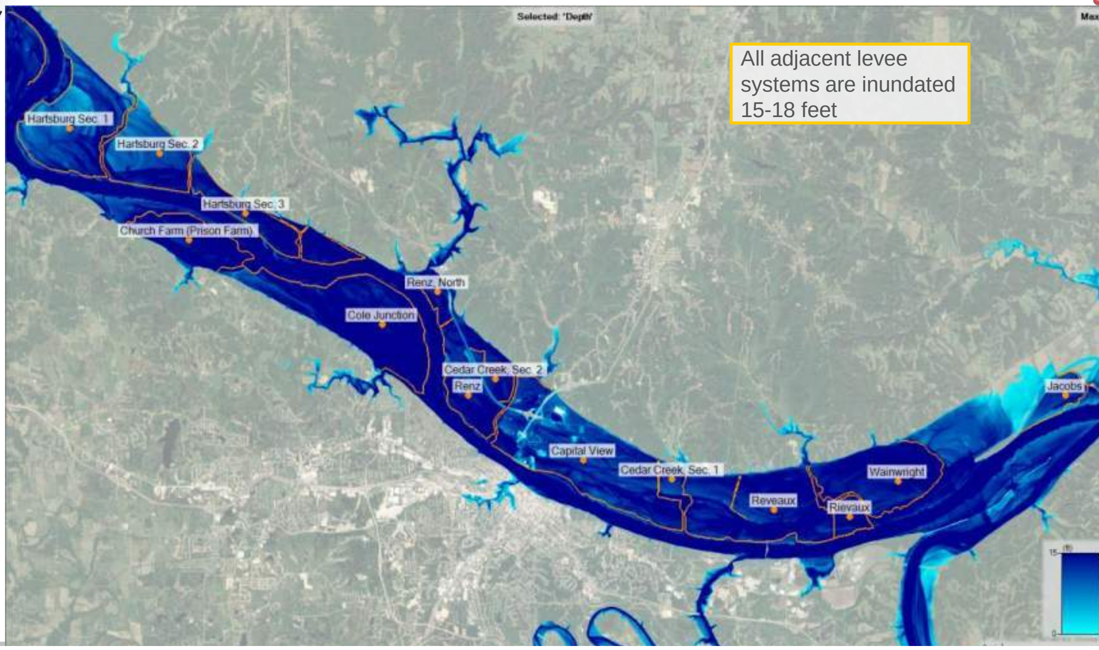
33

U.S. ARMY

FWOP 1% AEP INUNDATION

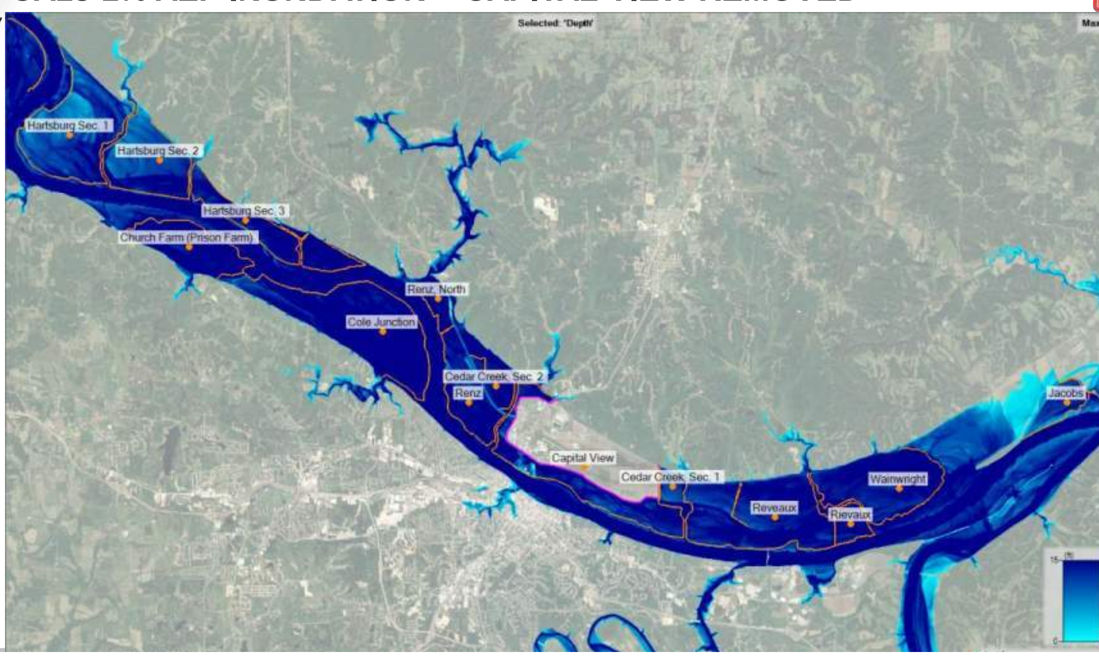


33

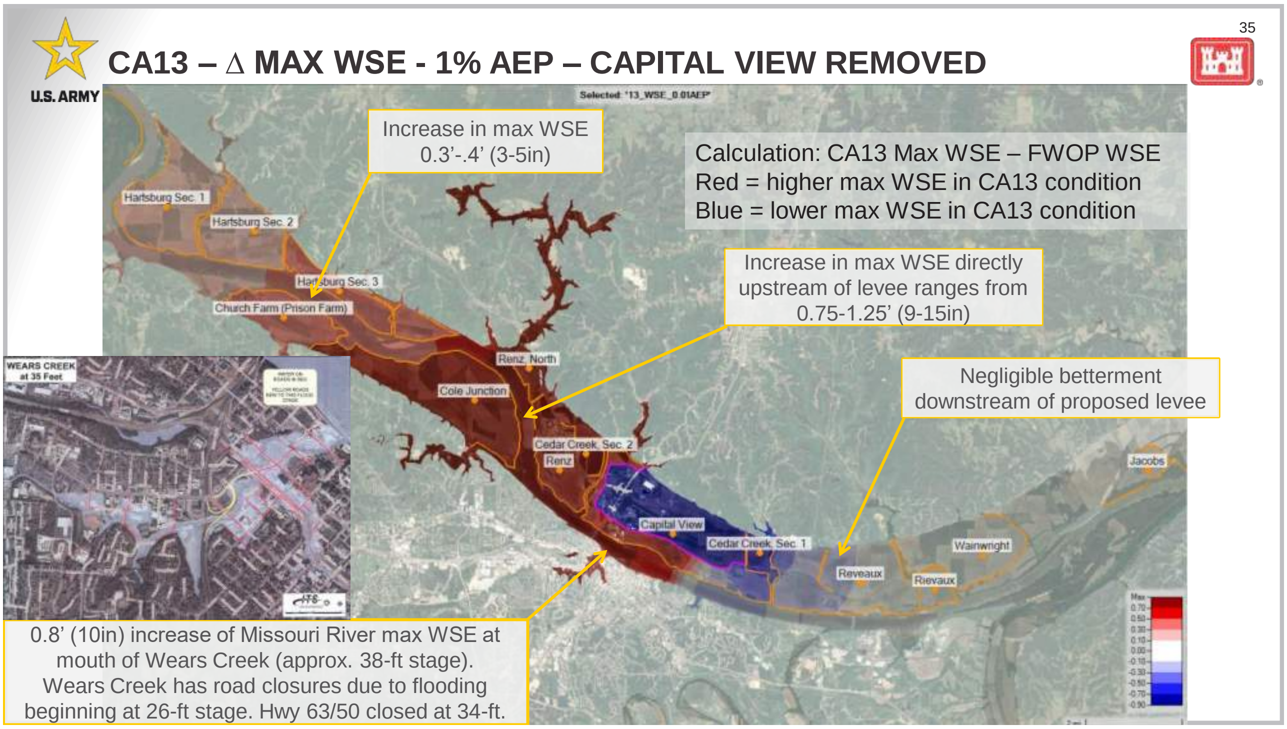
34

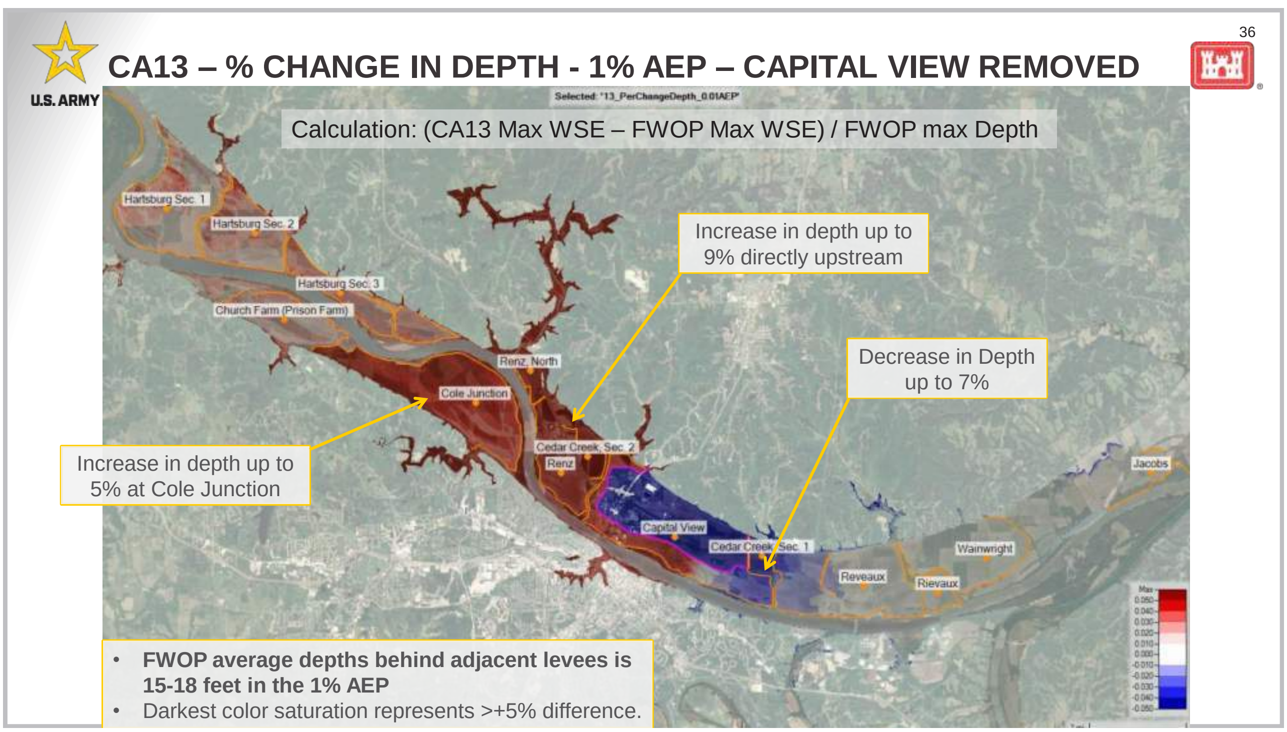
U.S. ARMY

CA13 1% AEP INUNDATION – CAPITAL VIEW REMOVED



34


35
CA13 – Δ MAX WSE - 1% AEP – CAPITAL VIEW REMOVED
U.S. ARMY
Increase in max WSE
0.3'-.4' (3-5in)
Calculation: CA13 Max WSE – FWOP WSE
Red = higher max WSE in CA13 condition
Blue = lower max WSE in CA13 condition
Increase in max WSE directly upstream of levee ranges from 0.75-1.25' (9-15in)
Negligible betterment downstream of proposed levee
Hartsburg Sec. 1
Hartsburg Sec. 2
Hartsburg Sec. 3
Church Farm (Prison Farm)
Renz North
Cole Junction
Cedar Creek Sec. 2
Renz
Capital View
Cedar Creek Sec. 1
Reveaux
Rievaux
Wainwright
Jacobs
WEARS CREEK at 35 Feet
0.8' (10in) increase of Missouri River max WSE at mouth of Wears Creek (approx. 38-ft stage).
Wears Creek has road closures due to flooding beginning at 26-ft stage. Hwy 63/50 closed at 34-ft.


36
CA13 – % CHANGE IN DEPTH - 1% AEP – CAPITAL VIEW REMOVED
U.S. ARMY
Calculation: (CA13 Max WSE – FWOP Max WSE) / FWOP max Depth
Increase in depth up to 9% directly upstream
Decrease in Depth up to 7%
Increase in depth up to 5% at Cole Junction
Hartsburg Sec. 1
Hartsburg Sec. 2
Hartsburg Sec. 3
Church Farm (Prison Farm)
Renz North
Cole Junction
Cedar Creek Sec. 2
Renz
Capital View
Cedar Creek Sec. 1
Reveaux
Rievaux
Wainwright
Jacobs
FWOP average depths behind adjacent levees is 15-18 feet in the 1% AEP
Darkest color saturation represents >+5% difference.

0.5% AEP (1/200 YEAR)

U.S. ARMY

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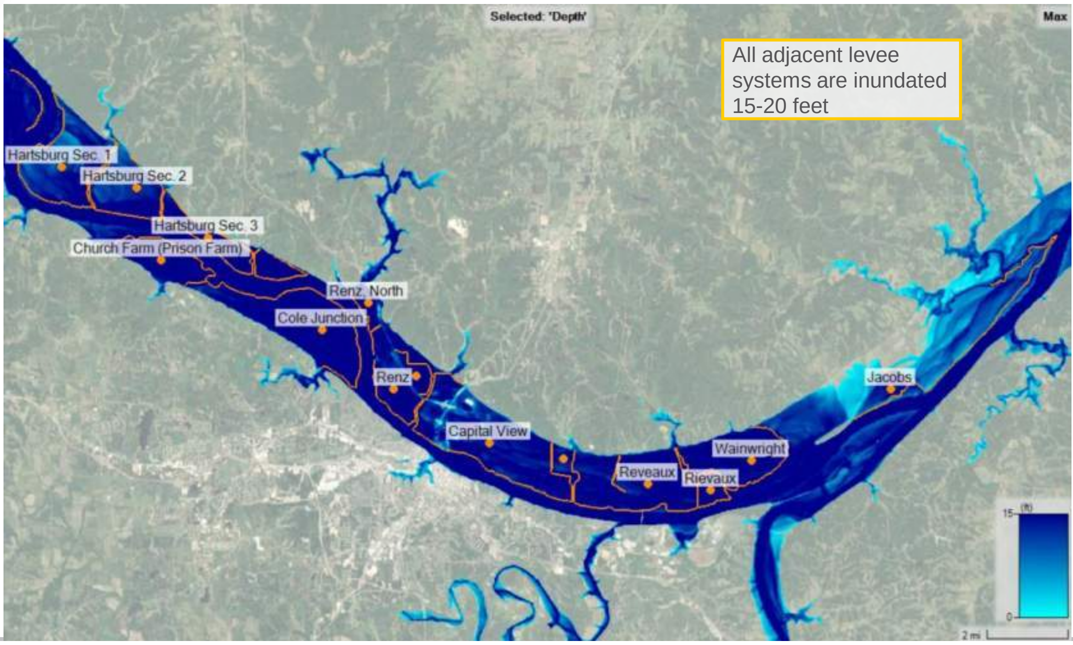
U.S. ARMY

FWOP 0.5% AEP INUNDATION



39

CA13 0.5% AEP INUNDATION – CAPITAL VIEW REMOVED

U.S. ARMY

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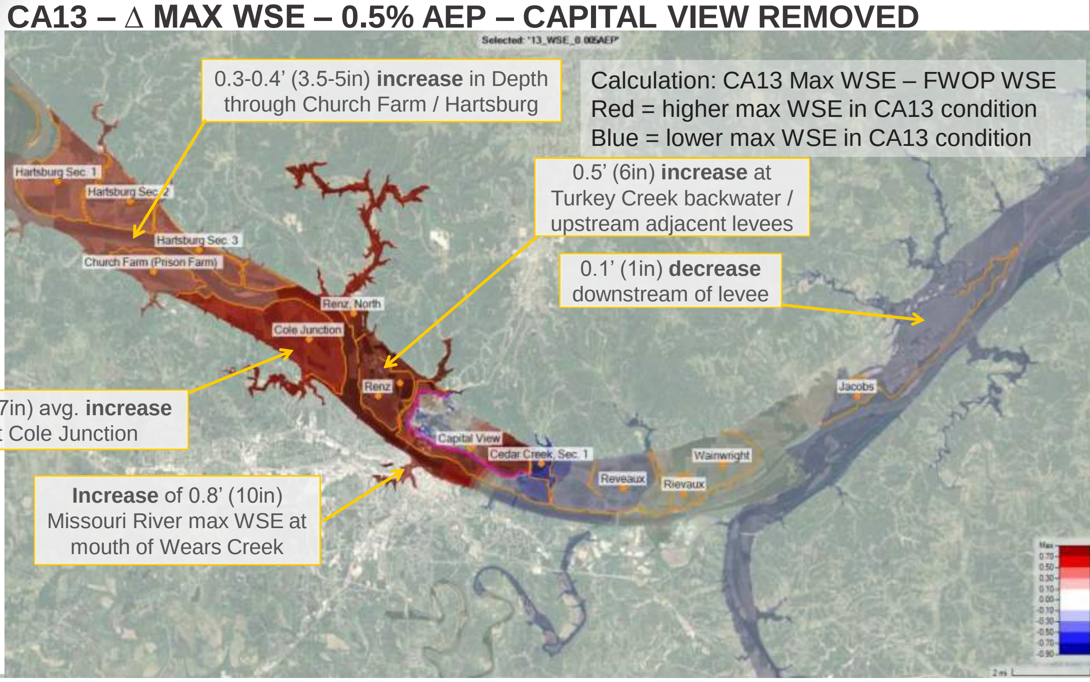
40

CA13 – ∆ MAX WSE – 0.5% AEP – CAPITAL VIEW REMOVED

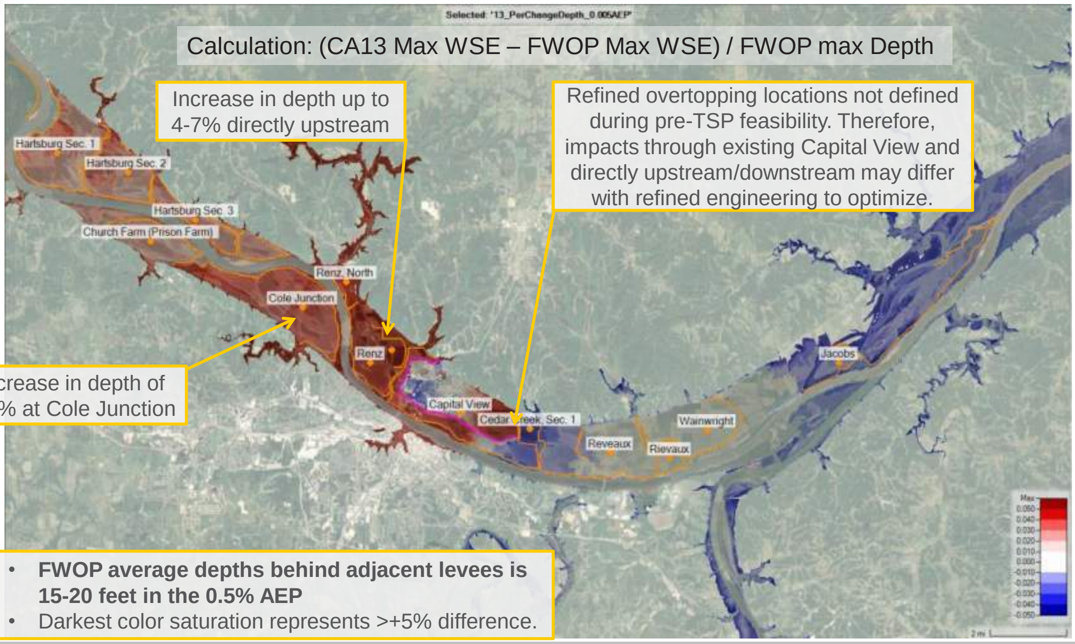
U.S. ARMY



40

U.S. ARMY

CA13 – % CHANGE IN DEPTH – 0.5% AEP – CAPITAL VIEW REMOVED



- **FWOP average depths behind adjacent levees is 15-20 feet in the 0.5% AEP**
- Darkest color saturation represents >+5% difference.

0.2% AEP (1/500 YEAR)

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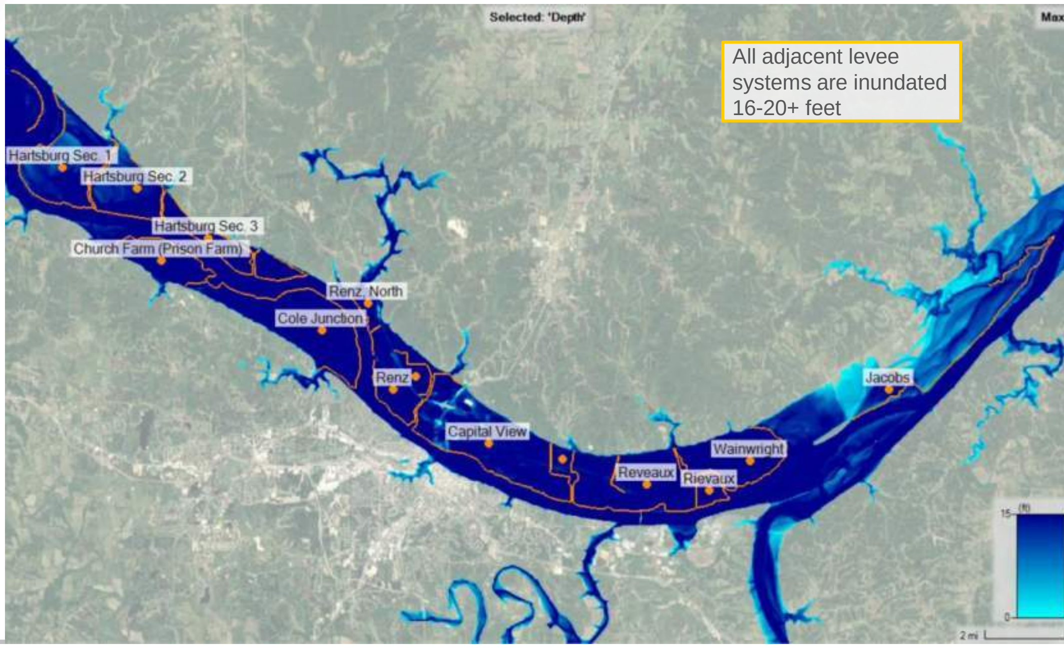
U.S. ARMY

FWOP 0.2% AEP INUNDATION

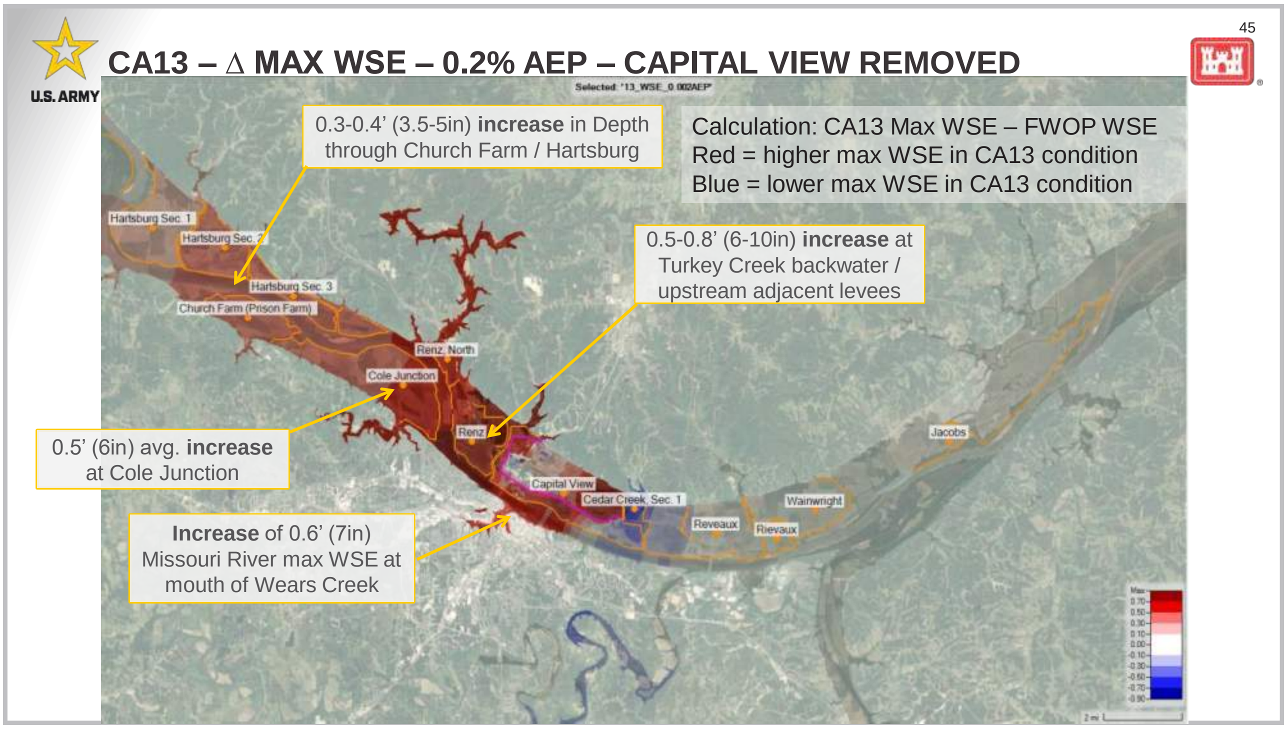


43

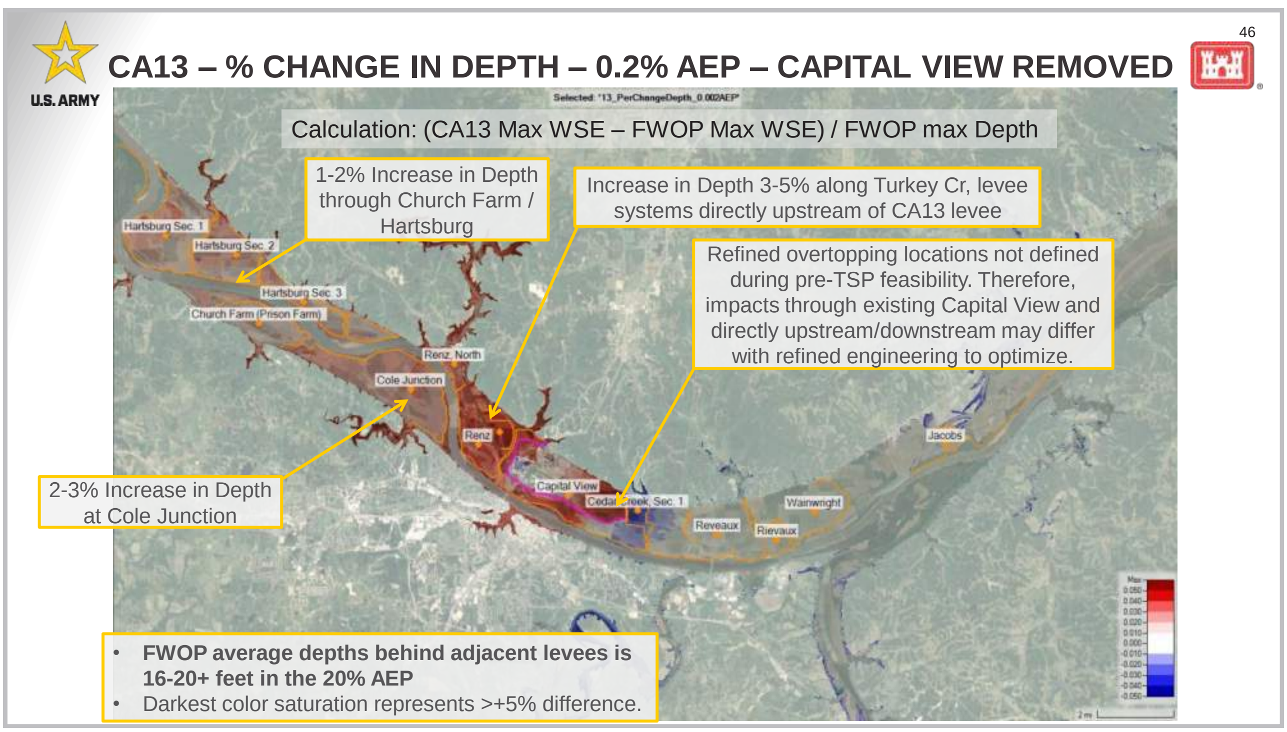
44

U.S. ARMY

CA13 0.2% AEP INUNDATION – CAPITAL VIEW REMOVED


45
U.S. ARMY
CA13 – ∆ MAX WSE – 0.2% AEP – CAPITAL VIEW REMOVED
0.3-0.4' (3.5-5in) **increase** in Depth through Church Farm / Hartsburg
Calculation: CA13 Max WSE – FWOP WSE
Red = higher max WSE in CA13 condition
Blue = lower max WSE in CA13 condition
0.5-0.8' (6-10in) **increase** at Turkey Creek backwater / upstream adjacent levees
0.5' (6in) avg. **increase** at Cole Junction
Increase of 0.6' (7in) Missouri River max WSE at mouth of Wears Creek
Hartsburg Sec. 1
Hartsburg Sec. 2
Hartsburg Sec. 3
Church Farm (Prison Farm)
Renz, North
Cole Junction
Renz
Capital View
Cedar Creek Sec. 1
Reveaux
Rievaux
Wainwright
Jacobs

45


46
U.S. ARMY
CA13 – % CHANGE IN DEPTH – 0.2% AEP – CAPITAL VIEW REMOVED
Calculation: (CA13 Max WSE – FWOP Max WSE) / FWOP max Depth
1-2% Increase in Depth through Church Farm / Hartsburg
Increase in Depth 3-5% along Turkey Cr, levee systems directly upstream of CA13 levee
Refined overtopping locations not defined during pre-TSP feasibility. Therefore, impacts through existing Capital View and directly upstream/downstream may differ with refined engineering to optimize.
2-3% Increase in Depth at Cole Junction
FWOP average depths behind adjacent levees is 16-20+ feet in the 20% AEP
Darkest color saturation represents >+5% difference.
Hartsburg Sec. 1
Hartsburg Sec. 2
Hartsburg Sec. 3
Church Farm (Prison Farm)
Renz, North
Cole Junction
Renz
Capital View
Cedar Creek Sec. 1
Reveaux
Rievaux
Wainwright
Jacobs

46

MAX DEPTH AND WATER SURFACE ELEVATION COMPARISON FWOP CONDITION & CA13 PARTIAL REMOVAL OF EXISTING CAPITAL VIEW LEVEE

Draft Report

U.S. ARMY

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MISSOURI RIVER CHANNEL MAX. WSE PROFILE-CAPITAL VIEW PARTIAL

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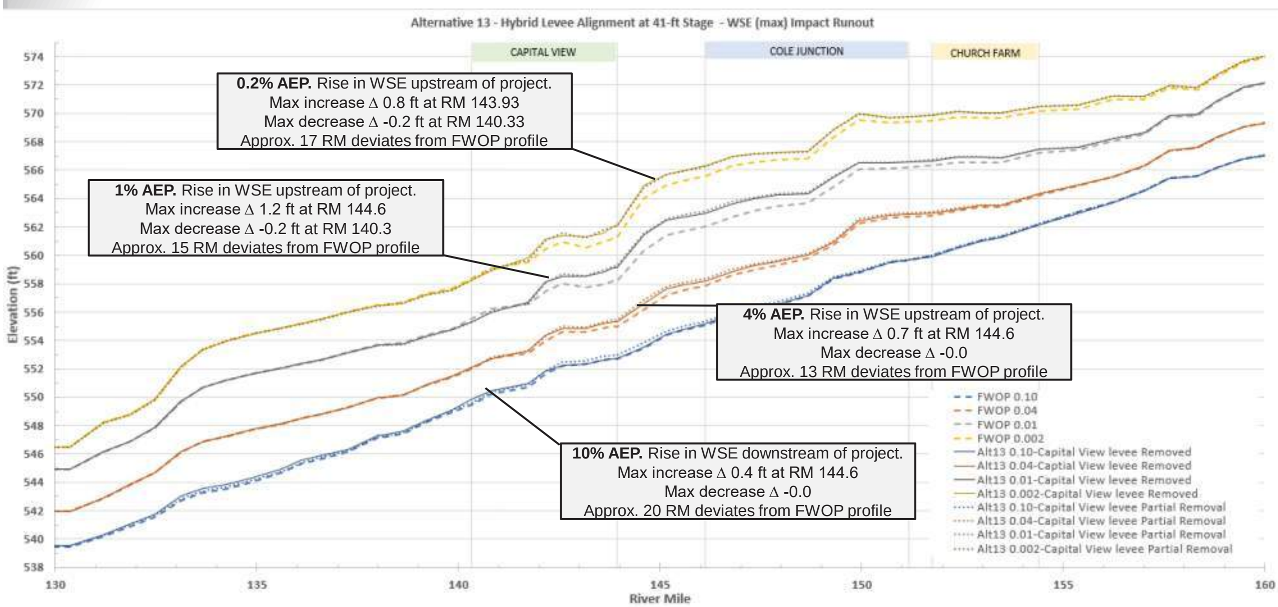

48

U.S. ARMY

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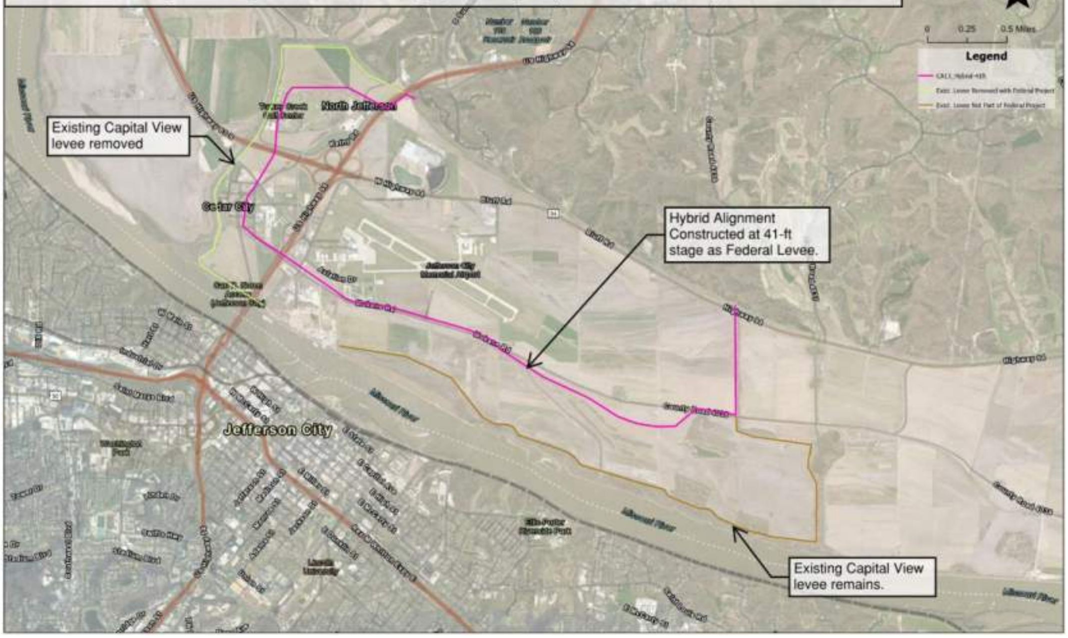
Lower Missouri River Jefferson City Flood Risk Management Study
Alternative 13 - Hybrid Alignment at 41-ft Stage, Existing Capital View Partially Removed



49

50% AEP (1/2 YEAR)

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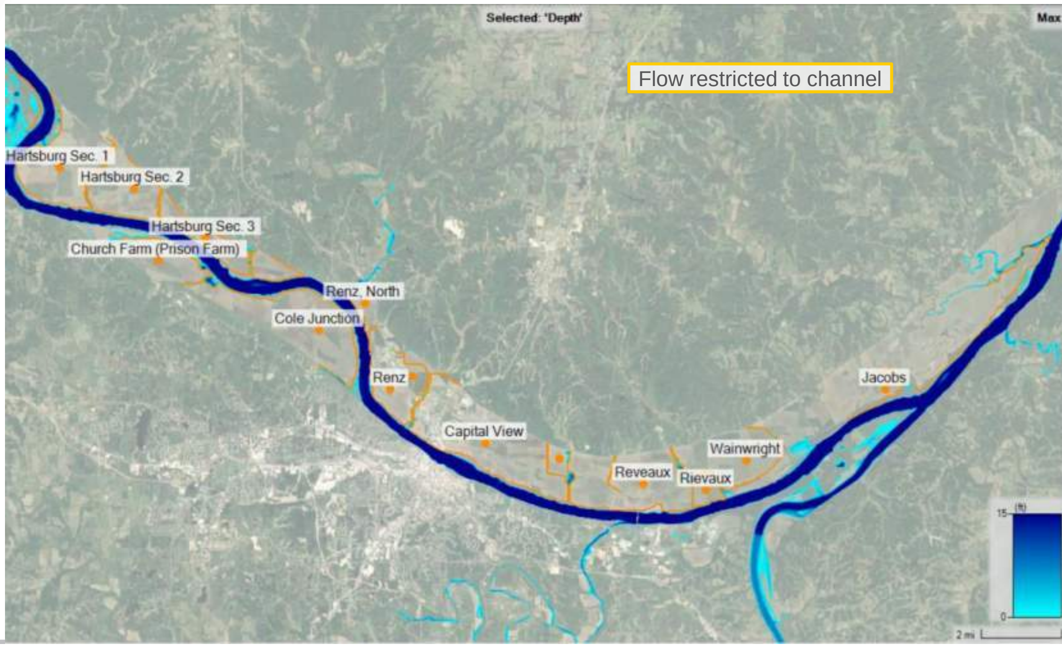
U.S. ARMY

FWOP 50% AEP INUNDATION



51

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U.S. ARMY

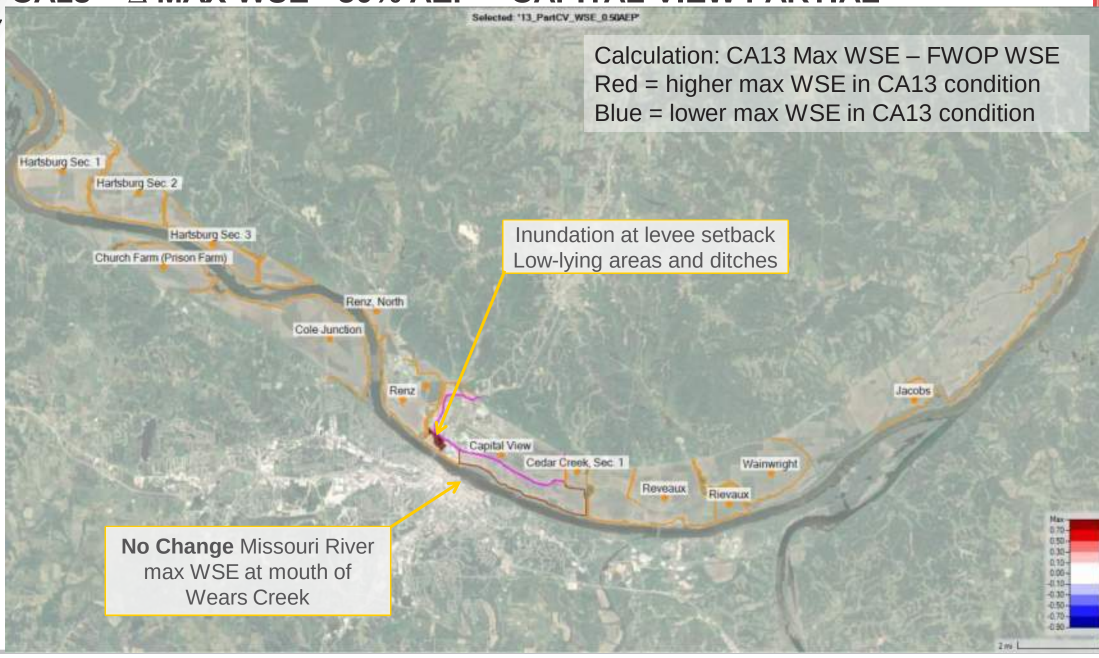
CA13 50% AEP INUNDATION – CAPITAL VIEW PARTIAL

52

53

U.S. ARMY

CA13 – ∆ MAX WSE - 50% AEP – CAPITAL VIEW PARTIAL



53

20% AEP (1/5 YEAR)

U.S. ARMY

US Army Corps of Engineers

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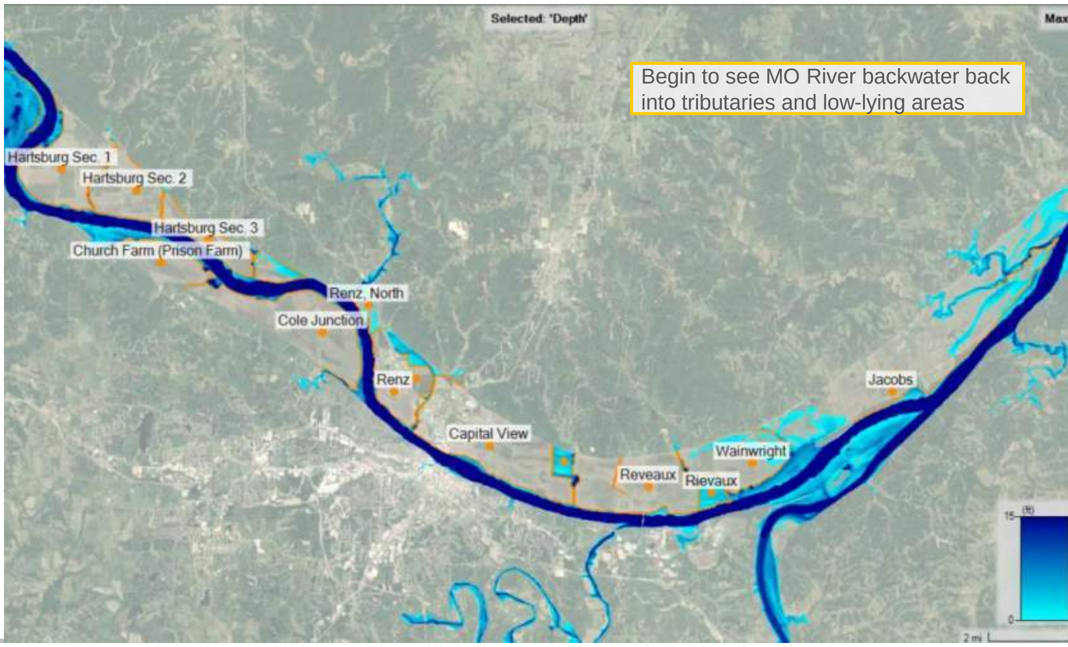
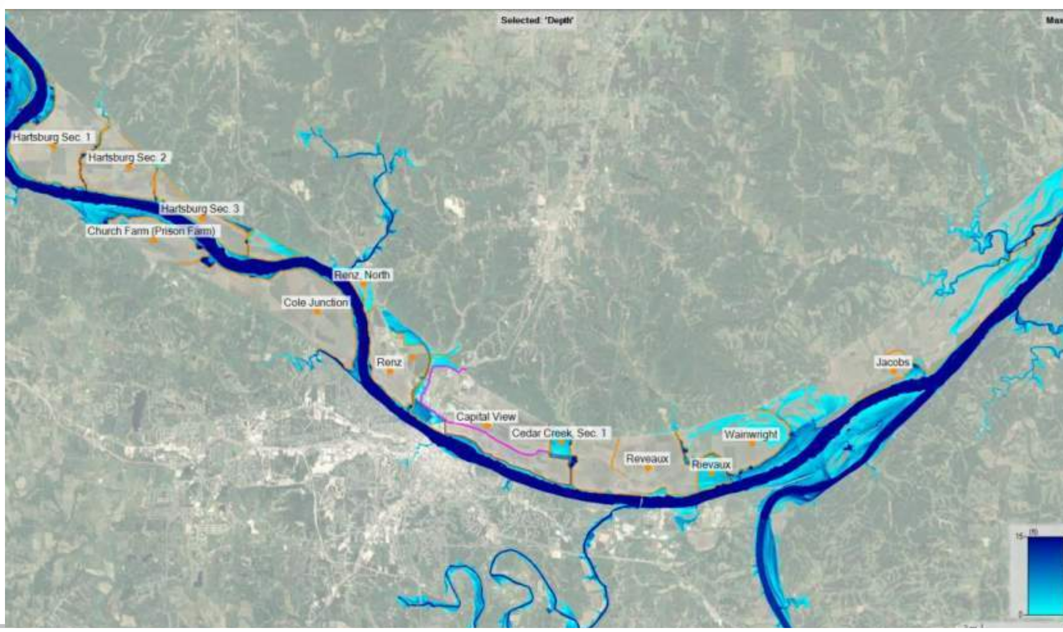
FWOP 20% AEP INUNDATION



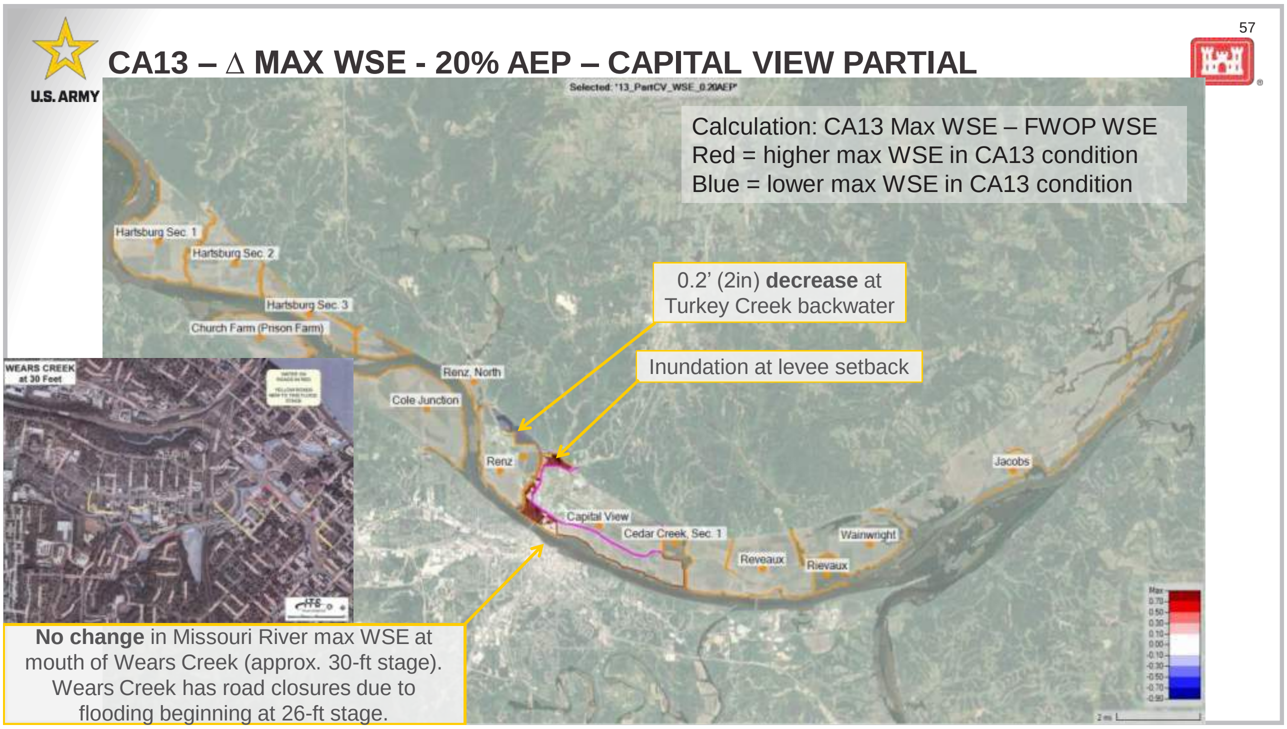
55

56

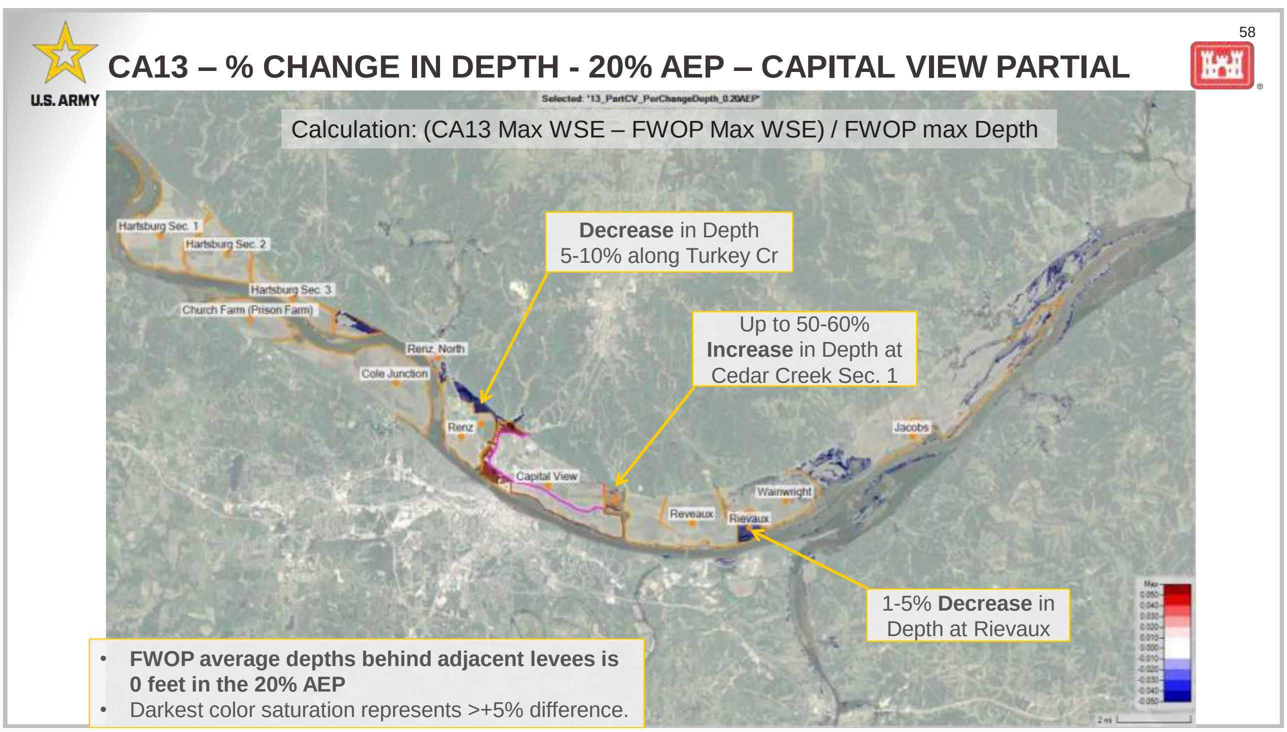
CA13 20% AEP INUNDATION – CAPITAL VIEW PARTIAL



56


57
CA13 – ∆ MAX WSE - 20% AEP – CAPITAL VIEW PARTIAL
U.S. ARMY
Calculation: CA13 Max WSE – FWOP WSE
Red = higher max WSE in CA13 condition
Blue = lower max WSE in CA13 condition
0.2' (2in) **decrease** at Turkey Creek backwater
Inundation at levee setback
No change in Missouri River max WSE at mouth of Wears Creek (approx. 30-ft stage). Wears Creek has road closures due to flooding beginning at 26-ft stage.
WEARS CREEK at 30 Feet
Hartsburg Sec. 1
Hartsburg Sec. 2
Hartsburg Sec. 3
Church Farm (Prison Farm)
Renz North
Cole Junction
Renz
Capital View
Cedar Creek Sec. 1
Reveaux
Rievaux
Wainwright
Jacobs

57


58
CA13 – % CHANGE IN DEPTH - 20% AEP – CAPITAL VIEW PARTIAL
U.S. ARMY
Calculation: (CA13 Max WSE – FWOP Max WSE) / FWOP max Depth
Decrease in Depth 5-10% along Turkey Cr
Up to 50-60% **Increase** in Depth at Cedar Creek Sec. 1
1-5% **Decrease** in Depth at Rievaux
• **FWOP average depths behind adjacent levees is 0 feet in the 20% AEP**
• Darkest color saturation represents >+5% difference.
Hartsburg Sec. 1
Hartsburg Sec. 2
Hartsburg Sec. 3
Church Farm (Prison Farm)
Renz North
Cole Junction
Renz
Capital View
Reveaux
Rievaux
Wainwright
Jacobs

58

10% AEP (1/10 YEAR)

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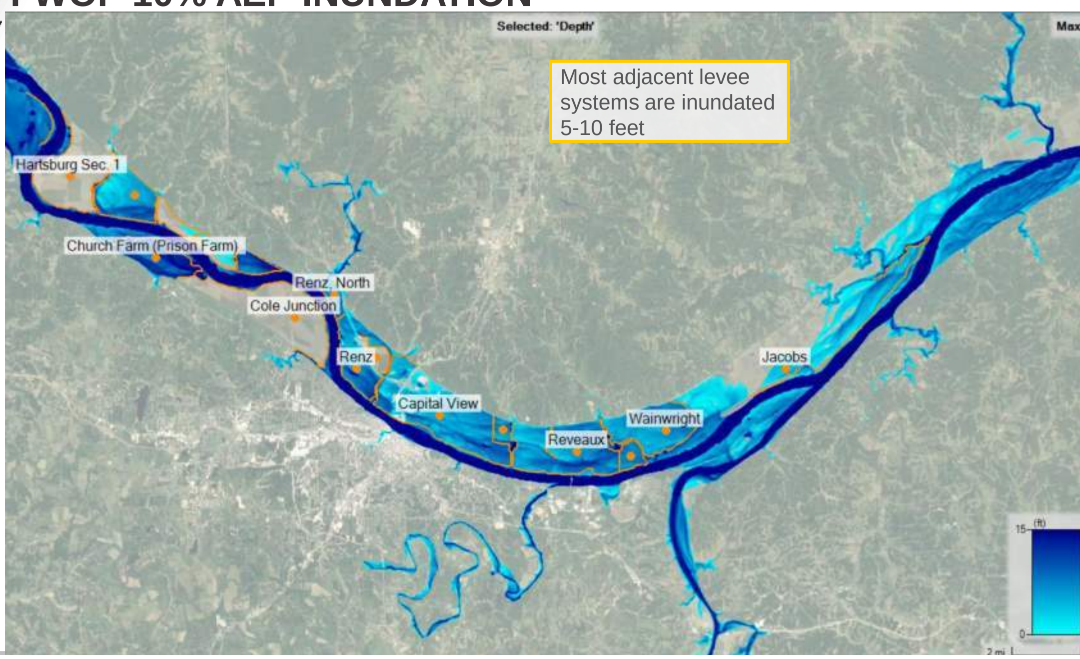
U.S. ARMY

FWOP 10% AEP INUNDATION



61

U.S. ARMY

CA13 10% AEP INUNDATION – CAPITAL VIEW PARTIAL

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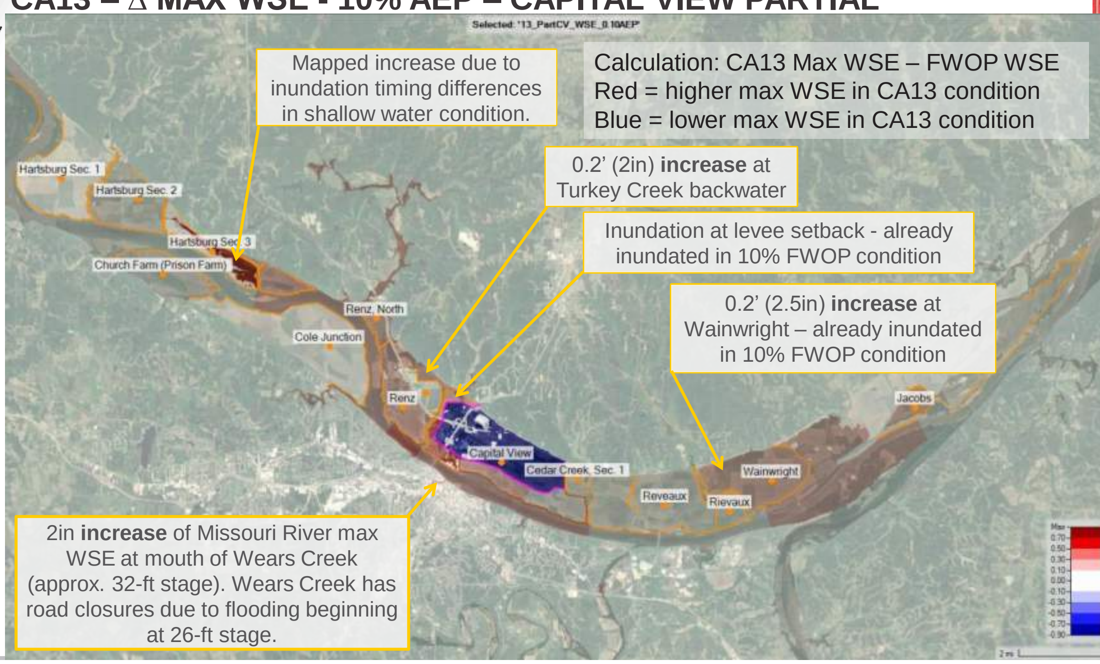
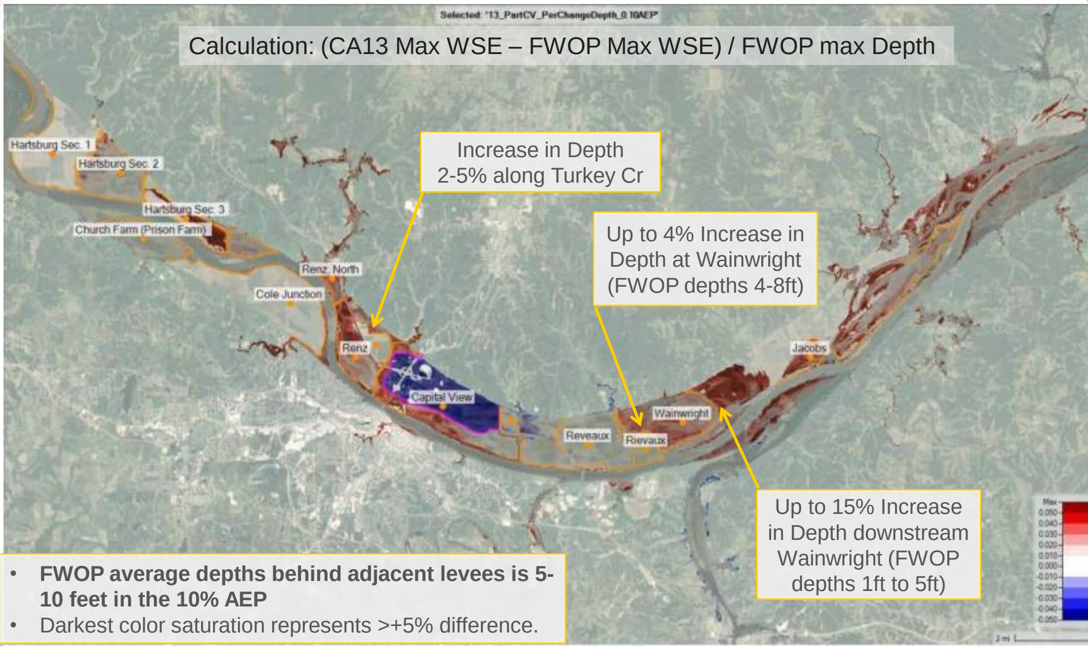
U.S. ARMY

CA13 – Δ MAX WSE - 10% AEP – CAPITAL VIEW PARTIAL



63

CA13 – % CHANGE IN DEPTH - 10% AEP - – CAPITAL VIEW PARTIAL



- **FWOP average depths behind adjacent levees is 5-10 feet in the 10% AEP**
- Darkest color saturation represents >+5% difference.

63

4% AEP (1/25 YEAR)

U.S. ARMY

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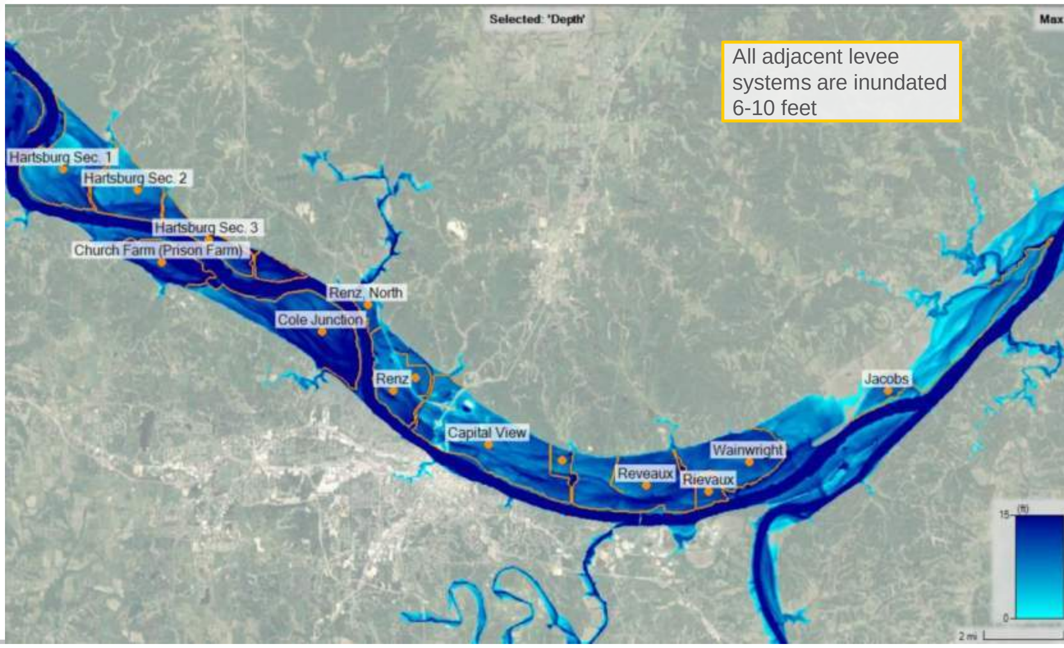
65

U.S. ARMY

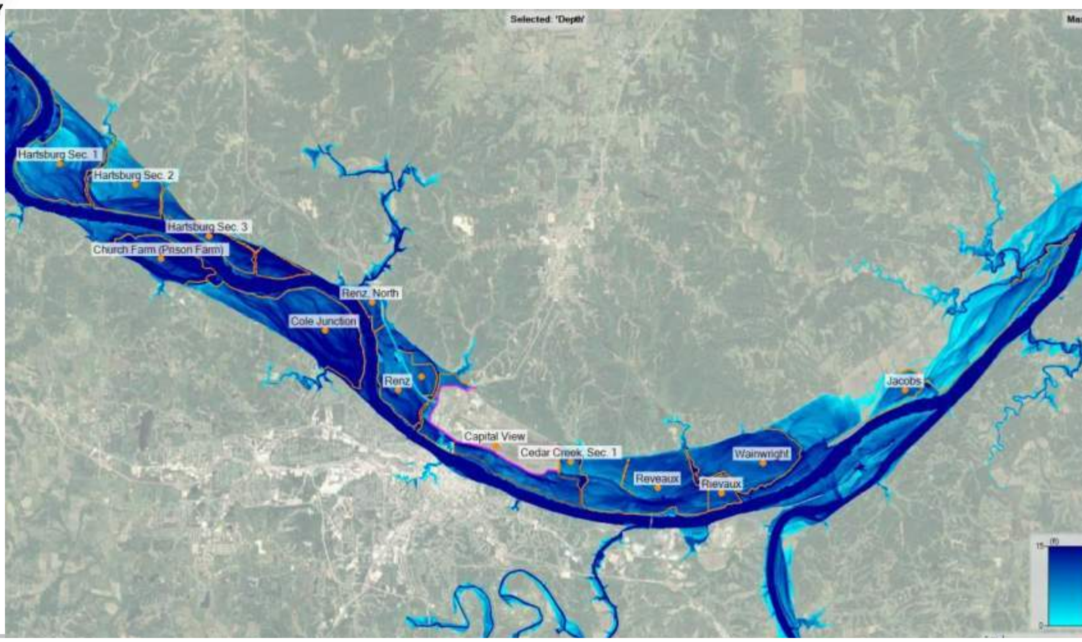
FWOP 4% AEP INUNDATION

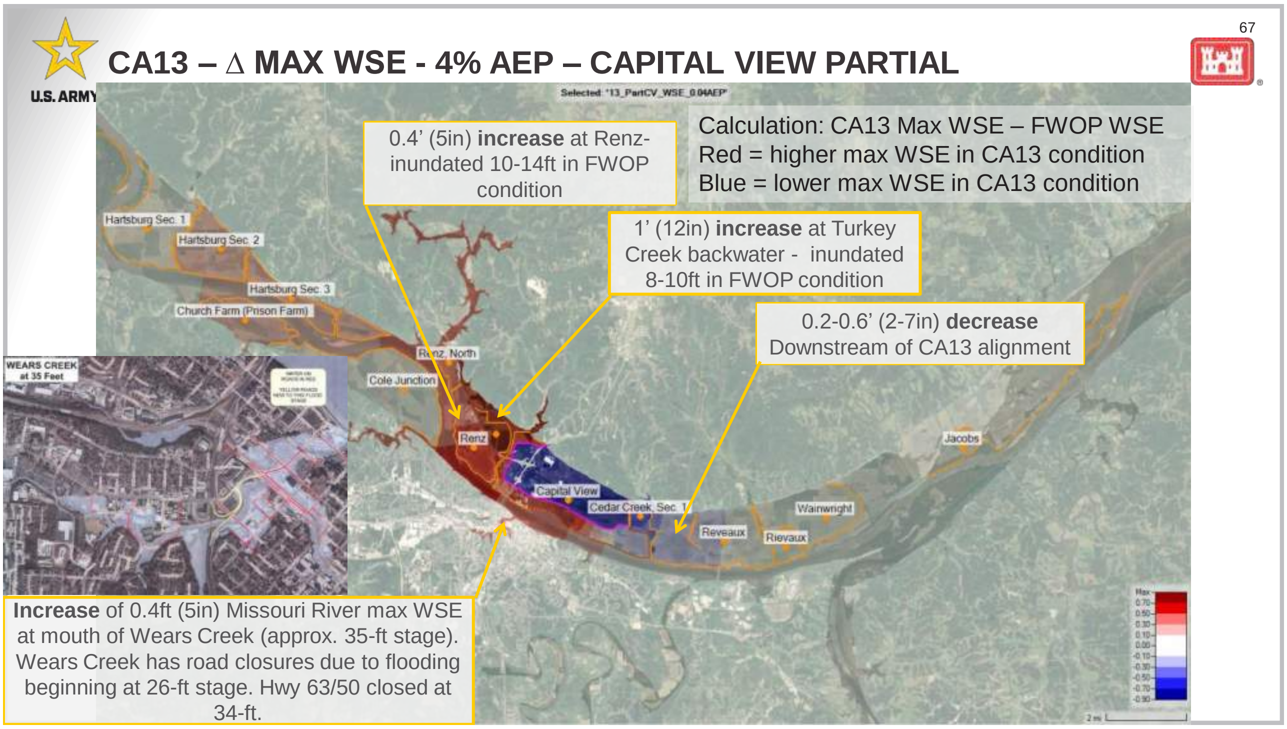


66

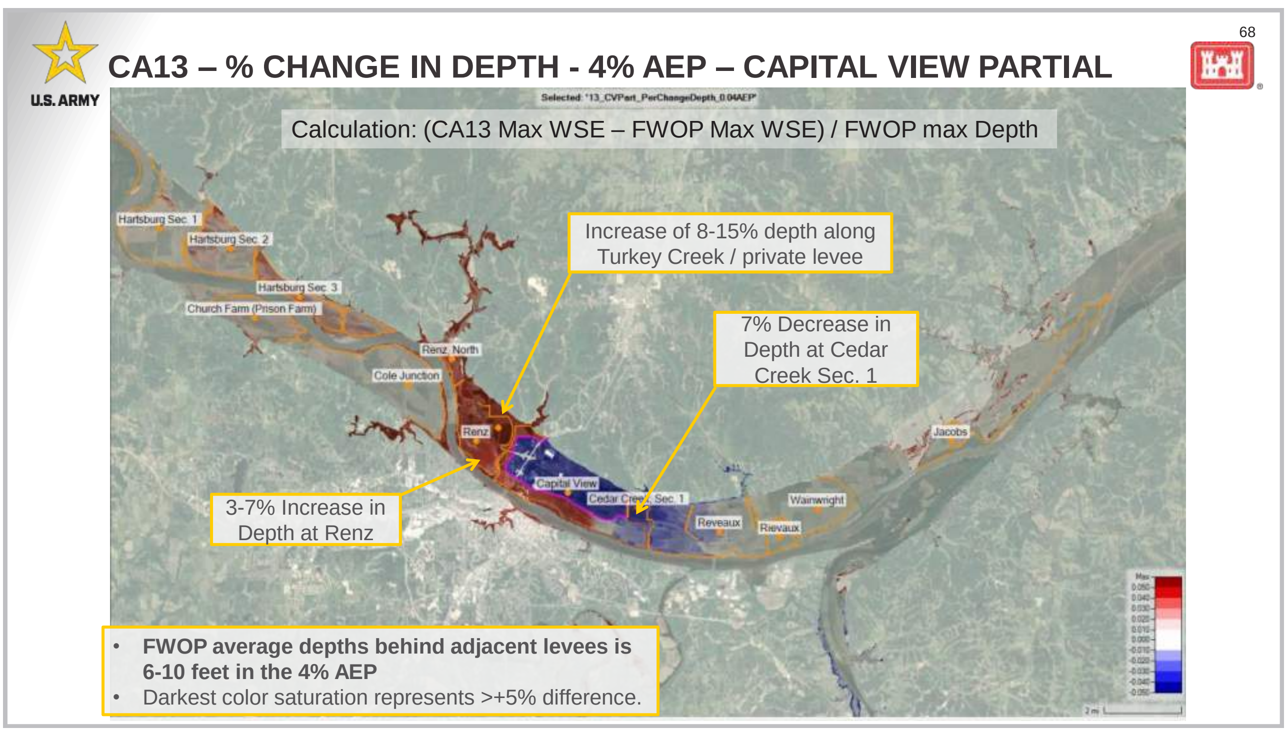
U.S. ARMY

CA13 4% AEP INUNDATION – CAPITAL VIEW PARTIAL




67
CA13 – ∆ MAX WSE - 4% AEP – CAPITAL VIEW PARTIAL
U.S. ARMY
Selected: '13_PartCV_WSE_0.04AEP'
0.4' (5in) **increase** at Renz- inundated 10-14ft in FWOP condition
Calculation: CA13 Max WSE – FWOP WSE
Red = higher max WSE in CA13 condition
Blue = lower max WSE in CA13 condition
1' (12in) **increase** at Turkey Creek backwater - inundated 8-10ft in FWOP condition
0.2-0.6' (2-7in) **decrease** Downstream of CA13 alignment
Hartsburg Sec. 1
Hartsburg Sec. 2
Hartsburg Sec. 3
Church Farm (Prison Farm)
Renz North
Cole Junction
Renz
Capital View
Cedar Creek Sec. 1
Reveaux
Rievaux
Wainwright
Jacobs
WEARS CREEK at 35 Feet
Increase of 0.4ft (5in) Missouri River max WSE at mouth of Wears Creek (approx. 35-ft stage). Wears Creek has road closures due to flooding beginning at 26-ft stage. Hwy 63/50 closed at 34-ft.

67


68
CA13 – % CHANGE IN DEPTH - 4% AEP – CAPITAL VIEW PARTIAL
U.S. ARMY
Selected: '13_CVPart_PerChangeDepth_0.04AEP'
Calculation: (CA13 Max WSE – FWOP Max WSE) / FWOP max Depth
Increase of 8-15% depth along Turkey Creek / private levee
7% Decrease in Depth at Cedar Creek Sec. 1
3-7% Increase in Depth at Renz
Hartsburg Sec. 1
Hartsburg Sec. 2
Hartsburg Sec. 3
Church Farm (Prison Farm)
Renz North
Cole Junction
Renz
Capital View
Cedar Creek Sec. 1
Reveaux
Rievaux
Wainwright
Jacobs
• **FWOP average depths behind adjacent levees is 6-10 feet in the 4% AEP**
• Darkest color saturation represents >+5% difference.

68

2% AEP (1/50 YEAR)

U.S. ARMY

US Army Corps of Engineers

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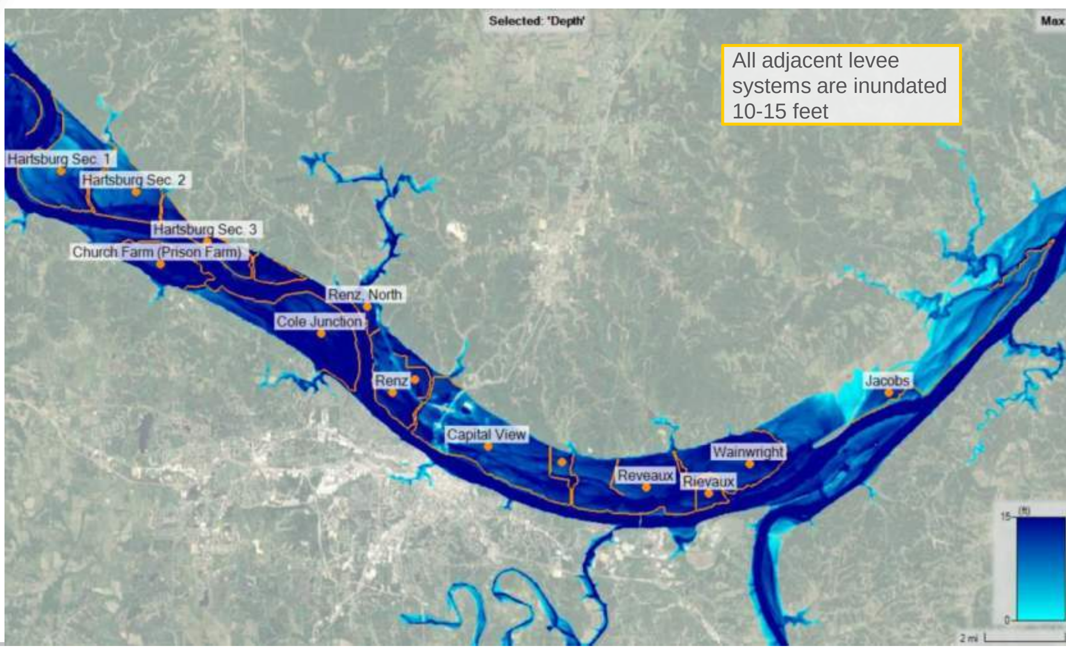
U.S. ARMY

FWOP 2% AEP INUNDATION



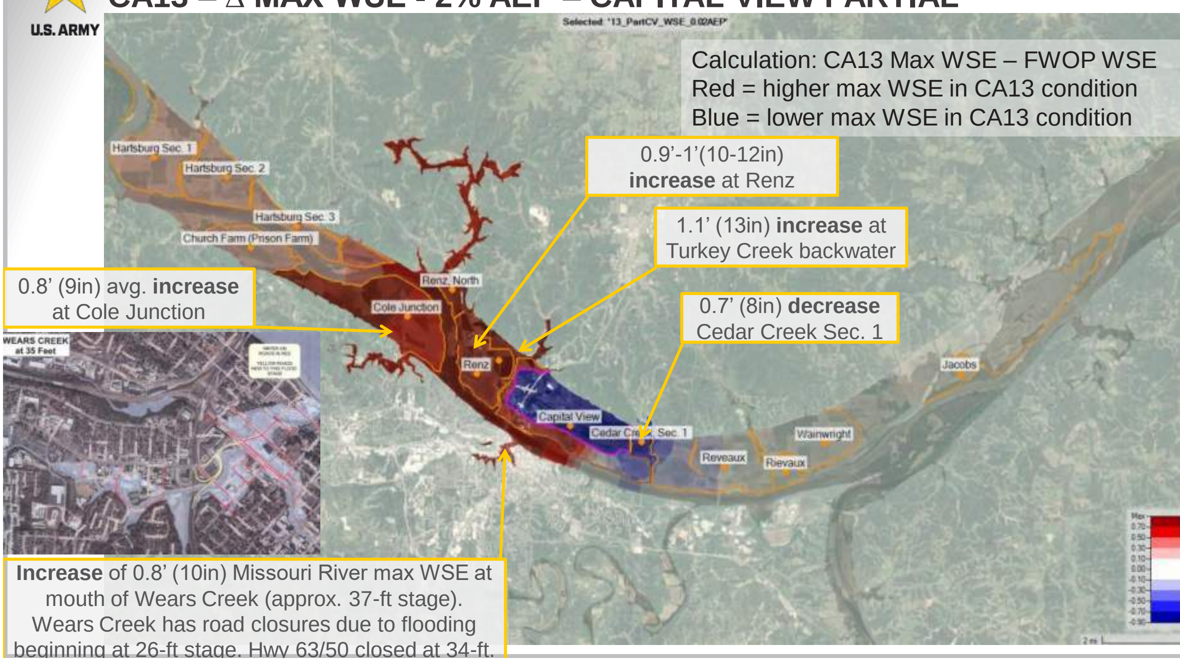
70

CA13 2% AEP INUNDATION – CAPITAL VIEW PARTIAL

U.S. ARMY

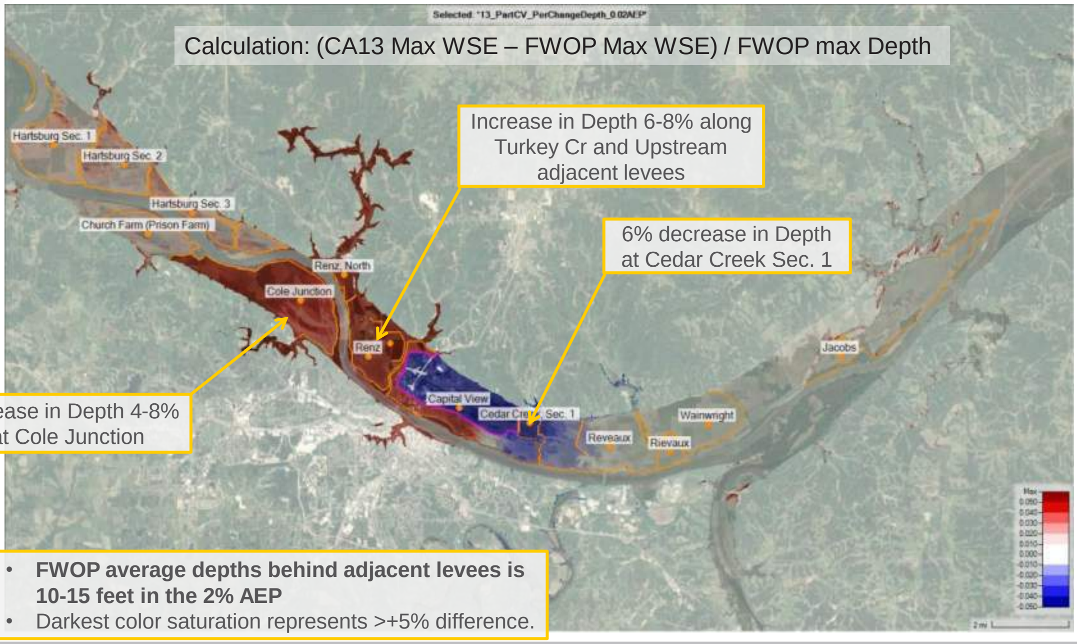
CA13 – ∆ MAX WSE - 2% AEP – CAPITAL VIEW PARTIAL

U.S. ARMY



U.S. ARMY

CA13 – % CHANGE IN DEPTH - 2% AEP – CAPITAL VIEW PARTIAL



Calculation: (CA13 Max WSE – FWOP Max WSE) / FWOP max Depth

Increase in Depth 6-8% along Turkey Cr and Upstream adjacent levees

6% decrease in Depth at Cedar Creek Sec. 1

Increase in Depth 4-8% at Cole Junction

- **FWOP average depths behind adjacent levees is 10-15 feet in the 2% AEP**
- Darkest color saturation represents >+5% difference.

1% AEP (1/100 YEAR)

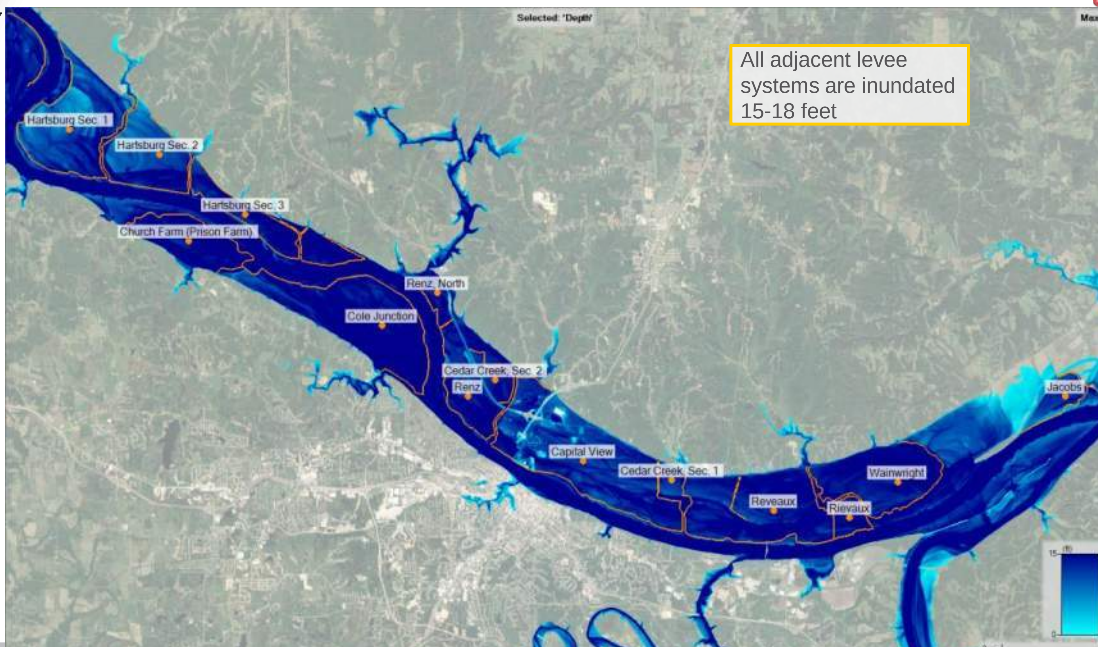
U.S. ARMY

US Army Corps of Engineers

75

FWOP 1% AEP INUNDATION

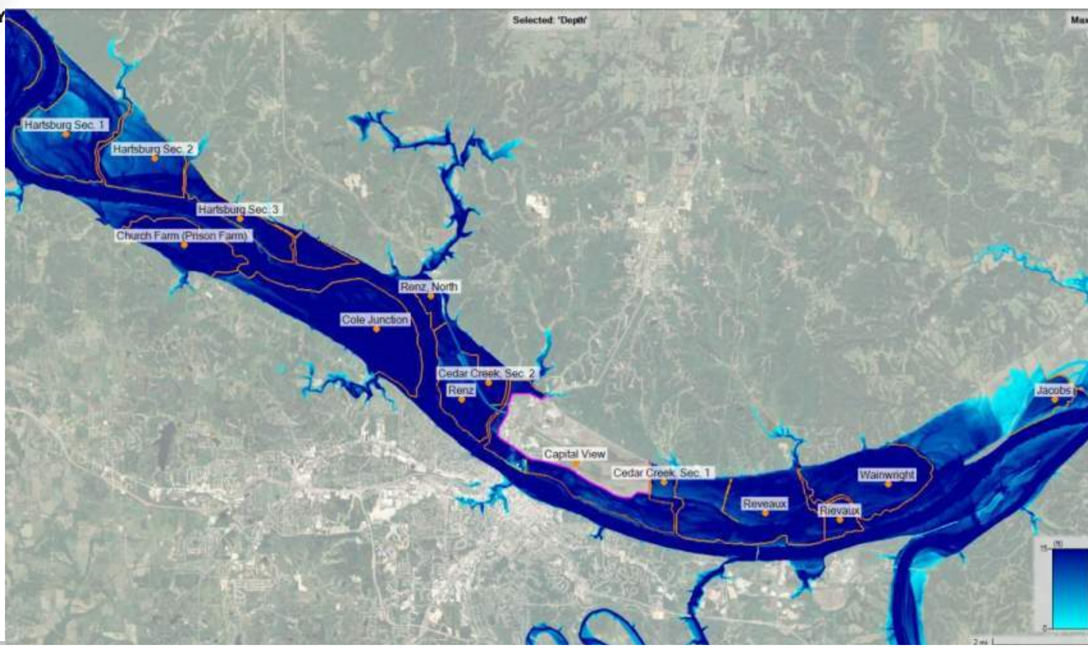
U.S. ARMY

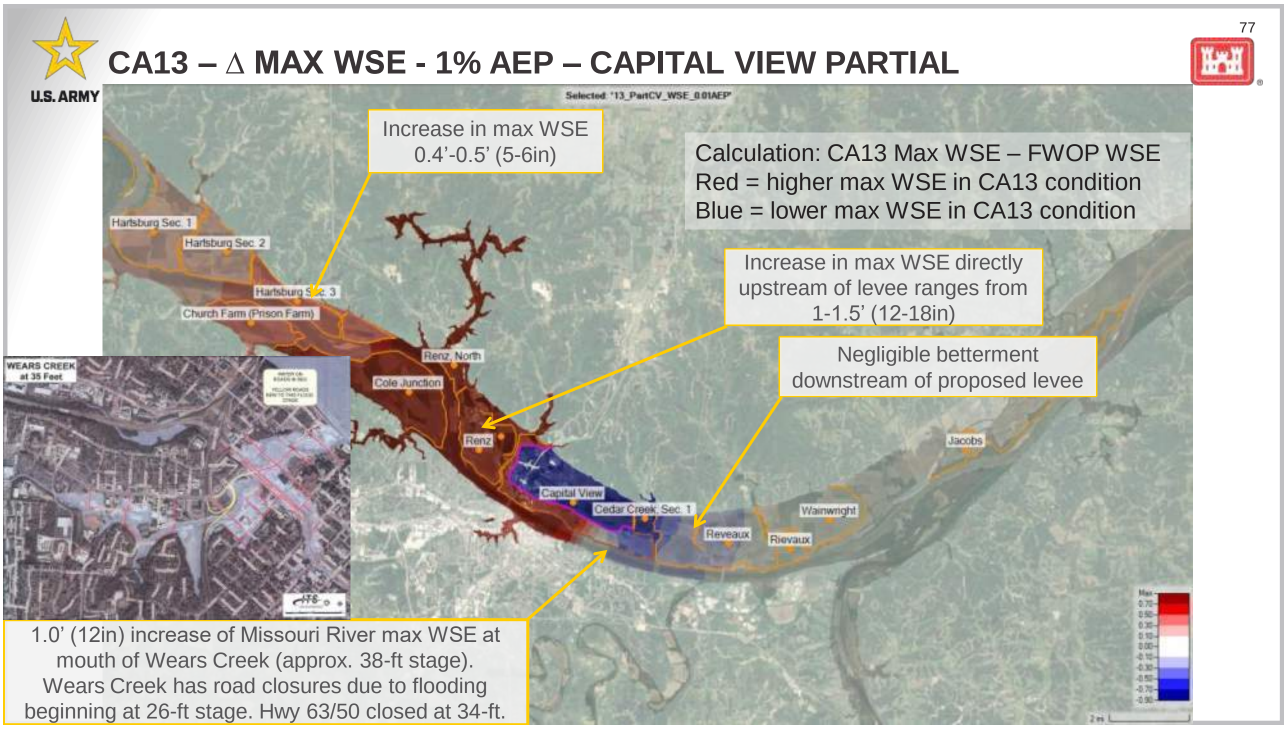


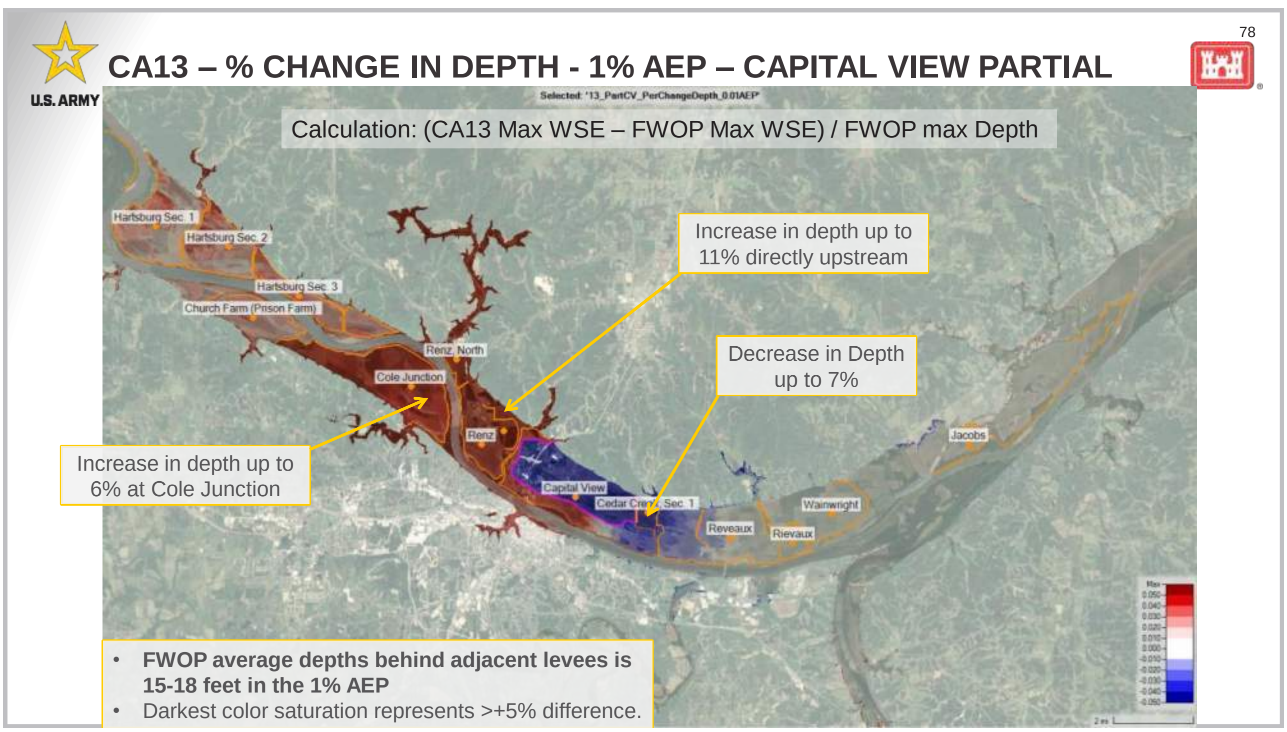
76

CA13 1% AEP INUNDATION – CAPITAL VIEW PARTIAL

U.S. ARMY




77
CA13 – ∆ MAX WSE - 1% AEP – CAPITAL VIEW PARTIAL
U.S. ARMY
Selected: '13_PartCV_WSE_0.01AEP'
Increase in max WSE
0.4'-0.5' (5-6in)
Calculation: CA13 Max WSE – FWOP WSE
Red = higher max WSE in CA13 condition
Blue = lower max WSE in CA13 condition
Increase in max WSE directly upstream of levee ranges from 1-1.5' (12-18in)
Negligible betterment downstream of proposed levee
Hartsburg Sec. 1
Hartsburg Sec. 2
Hartsburg Sec. 3
Church Farm (Prison Farm)
Renz North
Cole Junction
Renz
Capital View
Cedar Creek Sec. 1
Reveaux
Rievaux
Wainwright
Jacobs
WEARS CREEK at 35 Feet
1.0' (12in) increase of Missouri River max WSE at mouth of Wears Creek (approx. 38-ft stage).
Wears Creek has road closures due to flooding beginning at 26-ft stage. Hwy 63/50 closed at 34-ft.


78
CA13 – % CHANGE IN DEPTH - 1% AEP – CAPITAL VIEW PARTIAL
U.S. ARMY
Selected: '13_PartCV_PerChangeDepth_0.01AEP'
Calculation: (CA13 Max WSE – FWOP Max WSE) / FWOP max Depth
Increase in depth up to 11% directly upstream
Decrease in Depth up to 7%
Increase in depth up to 6% at Cole Junction
Hartsburg Sec. 1
Hartsburg Sec. 2
Hartsburg Sec. 3
Church Farm (Prison Farm)
Renz North
Cole Junction
Renz
Capital View
Cedar Creek Sec. 1
Reveaux
Rievaux
Wainwright
Jacobs
• FWOP average depths behind adjacent levees is 15-18 feet in the 1% AEP
• Darkest color saturation represents >+5% difference.

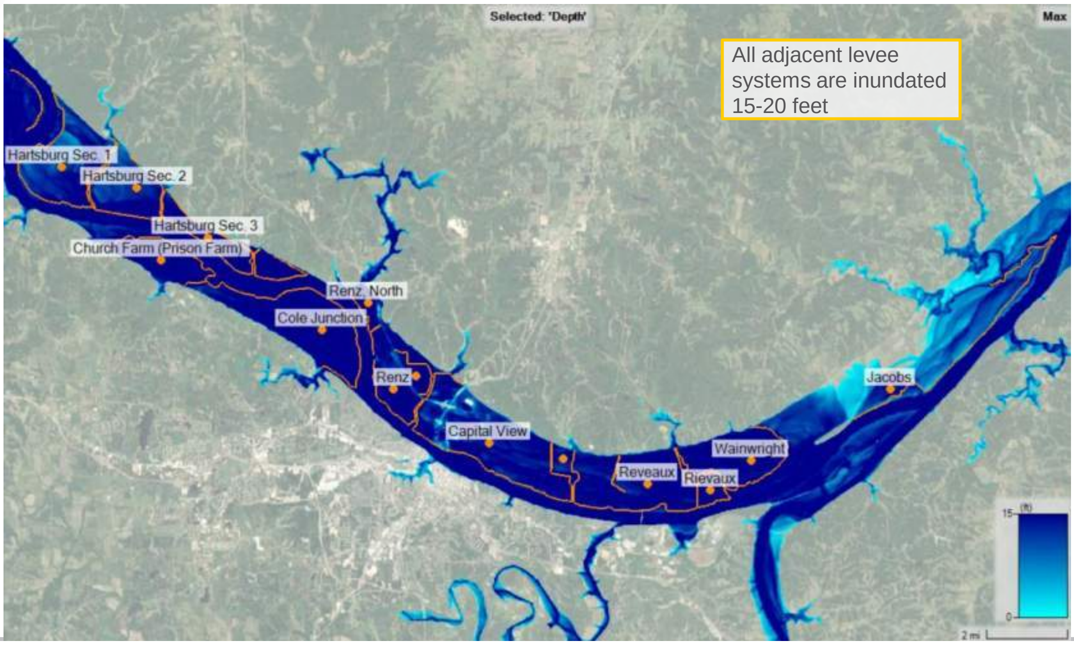
0.5% AEP (1/200 YEAR)

U.S. ARMY

US Army Corps of Engineers

U.S. ARMY

FWOP 0.5% AEP INUNDATION

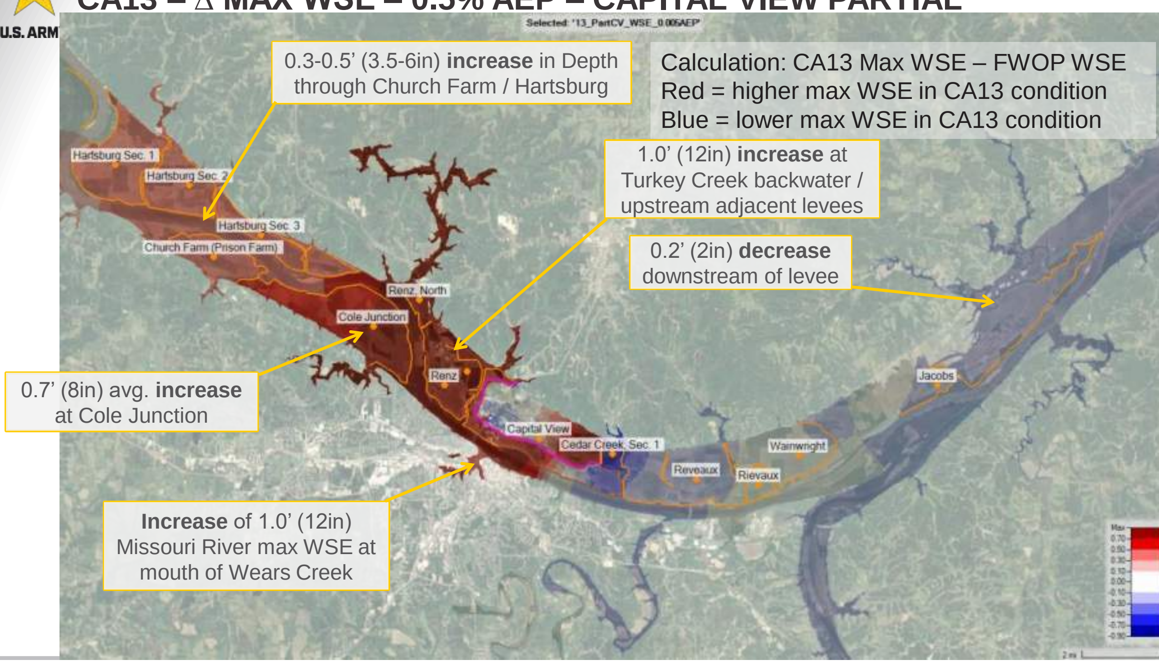


81

CA13 0.5% AEP INUNDATION – CAPITAL VIEW PARTIAL

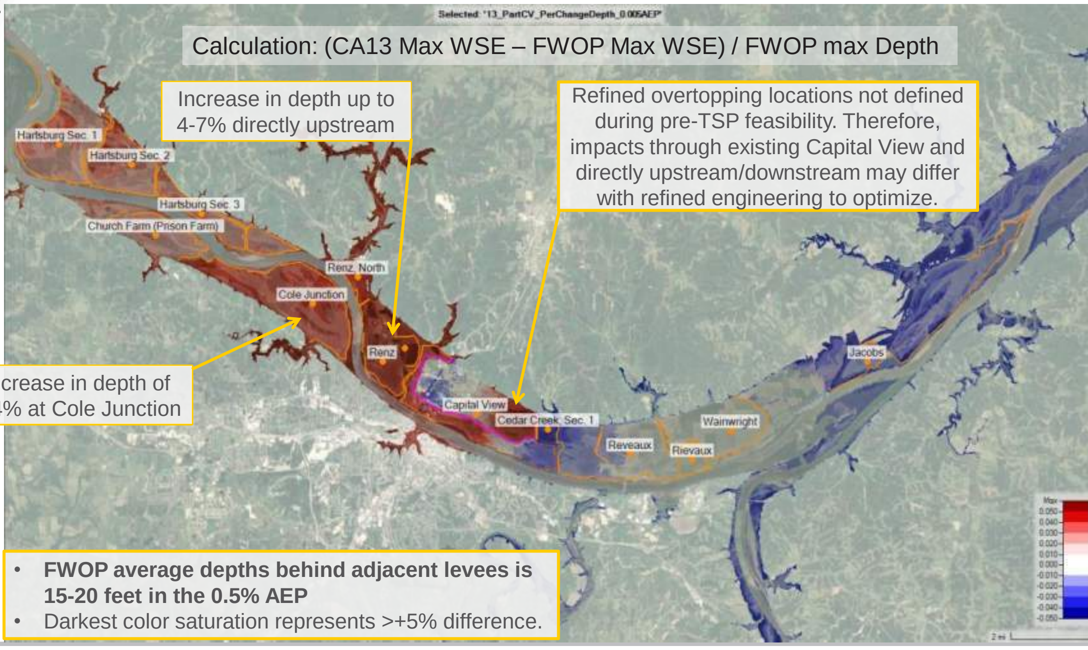
82

CA13 – ∆ MAX WSE – 0.5% AEP – CAPITAL VIEW PARTIAL



U.S. ARMY

CA13 – % CHANGE IN DEPTH – 0.5% AEP – CAPITAL VIEW PARTIAL



- **FWOP average depths behind adjacent levees is 15-20 feet in the 0.5% AEP**
- Darkest color saturation represents >+5% difference.

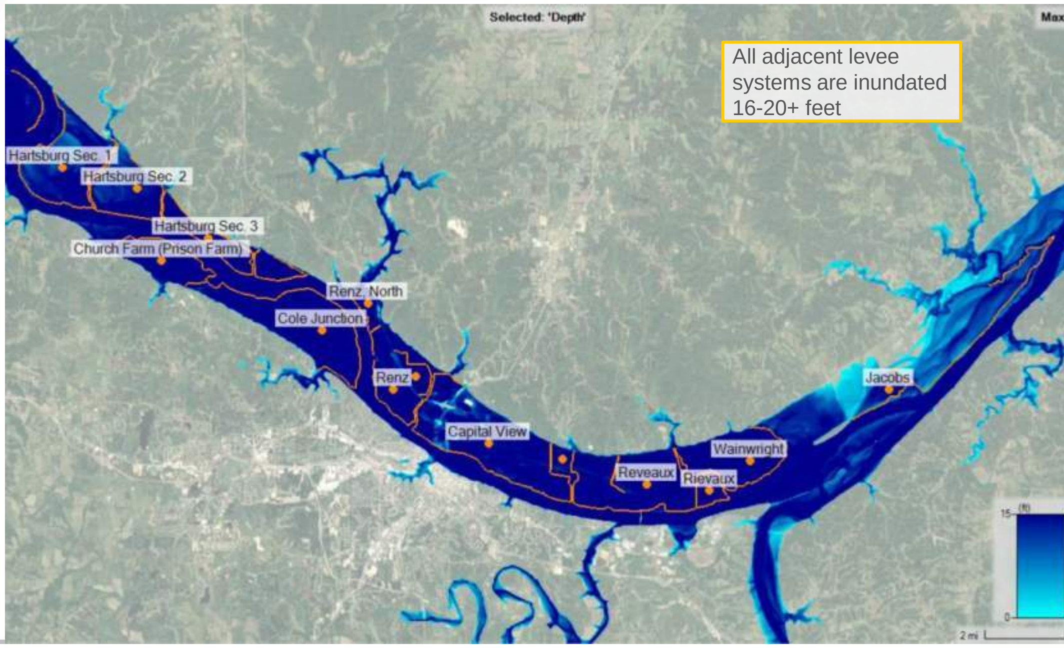
0.2% AEP (1/500 YEAR)

U.S. ARMY

US Army Corps of Engineers

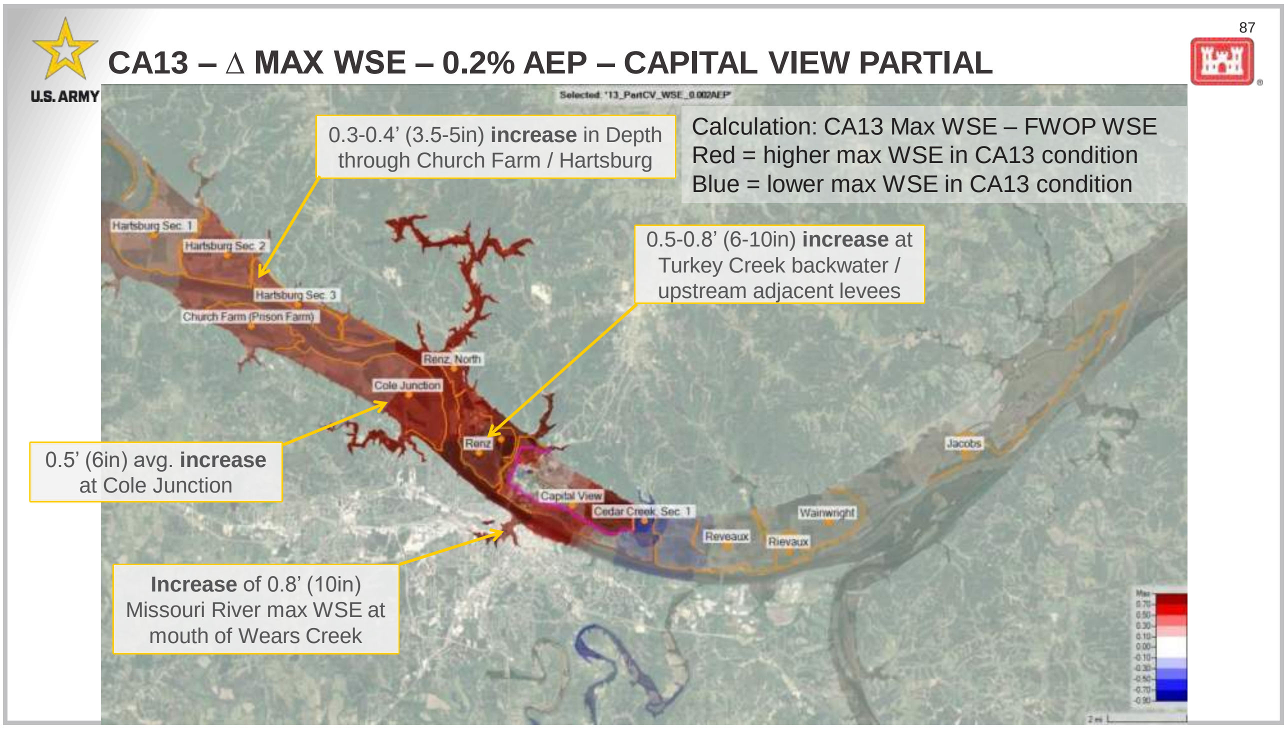
FWOP 0.2% AEP INUNDATION

U.S. ARMY

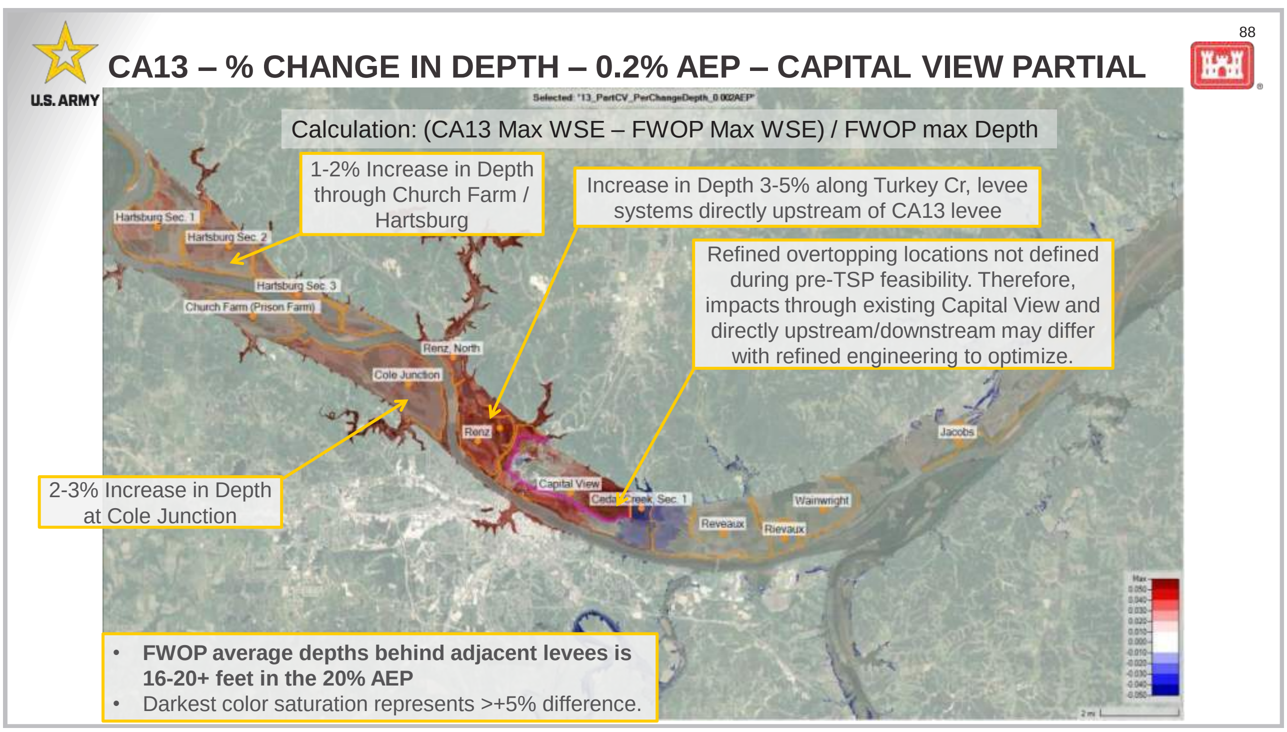


CA13 0.2% AEP INUNDATION – CAPITAL VIEW PARTIAL

U.S. ARMY


87
U.S. ARMY
CA13 – ∆ MAX WSE – 0.2% AEP – CAPITAL VIEW PARTIAL
Selected: '13_PartCV_WSE_0.002AEP'
0.3-0.4' (3.5-5in) **increase** in Depth through Church Farm / Hartsburg
Calculation: CA13 Max WSE – FWOP WSE
Red = higher max WSE in CA13 condition
Blue = lower max WSE in CA13 condition
0.5-0.8' (6-10in) **increase** at Turkey Creek backwater / upstream adjacent levees
0.5' (6in) avg. **increase** at Cole Junction
Increase of 0.8' (10in) Missouri River max WSE at mouth of Wears Creek
Hartsburg Sec. 1
Hartsburg Sec. 2
Hartsburg Sec. 3
Church Farm (Prison Farm)
Renz, North
Cole Junction
Renz
Capital View
Cedar Creek Sec. 1
Reveaux
Rievaux
Wainwright
Jacobs

87


88
U.S. ARMY
CA13 – % CHANGE IN DEPTH – 0.2% AEP – CAPITAL VIEW PARTIAL
Selected: '13_PartCV_PerChangeDepth_0.002AEP'
Calculation: (CA13 Max WSE – FWOP Max WSE) / FWOP max Depth
1-2% Increase in Depth through Church Farm / Hartsburg
Increase in Depth 3-5% along Turkey Cr, levee systems directly upstream of CA13 levee
Refined overtopping locations not defined during pre-TSP feasibility. Therefore, impacts through existing Capital View and directly upstream/downstream may differ with refined engineering to optimize.
2-3% Increase in Depth at Cole Junction
• **FWOP average depths behind adjacent levees is 16-20+ feet in the 20% AEP**
• Darkest color saturation represents >+5% difference.
Hartsburg Sec. 1
Hartsburg Sec. 2
Hartsburg Sec. 3
Church Farm (Prison Farm)
Renz, North
Cole Junction
Renz
Capital View
Cedar Creek Sec. 1
Reveaux
Rievaux
Wainwright
Jacobs

88

CA13 COMPARED TO MAPPED FEMA BOUNDARIES

U.S. ARMY

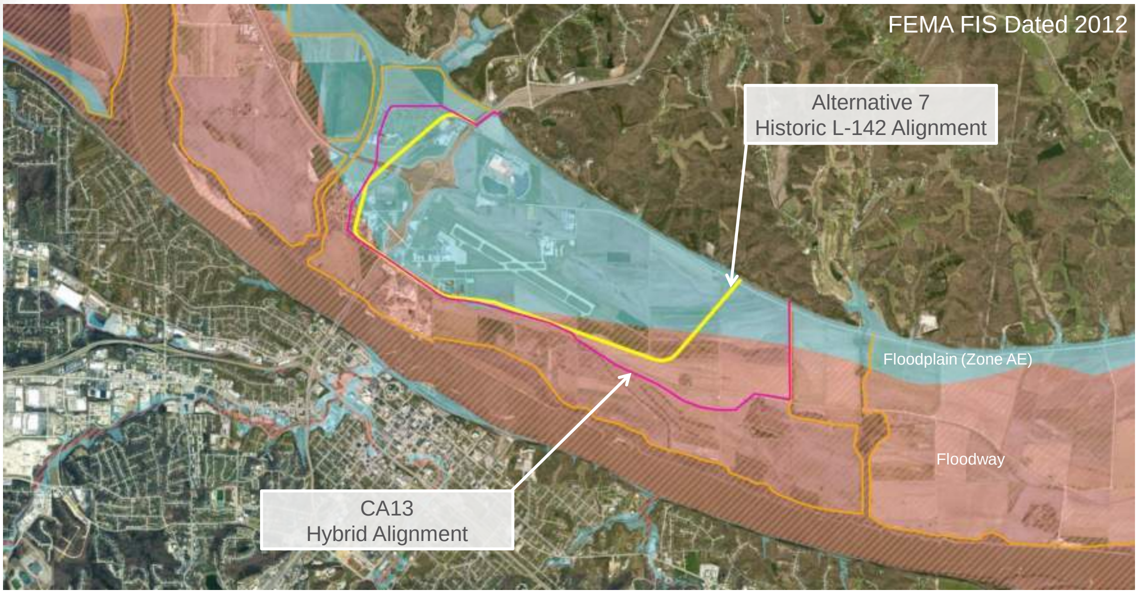
US Army Corps of Engineers

89

90

CA13 COMPARED TO MAPPED FEMA BOUNDARIES

U.S. ARMY



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Lower Missouri Jefferson City L-142 FRM Study

Appendix A1
Part 4: Federal Interest Plan Refinement

Capital View Drainage District, looking southeast toward inundated airport May 28, 2019

U.S. Army Corps of Engineers
Kansas City District

Final Report, July 2026

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LIST OF TABLES

1 INTRODUCTION

This appendix documents refinements to hydrologic calculations and hydraulic modeling conducted after the selection of a Federal Interest Plan for the Lower Missouri River Jefferson City L-142 Flood Risk Management Study. Development of measures and alternatives in accordance with USACE Planning Policies and Procedures is discussed in the main-body feasibility report. For documentation regarding hydrology development refer to Appendix A1, Part 1. For documentation regarding development and constraints of the base hydraulic model utilized for this study refer to Appendix A1, Part 2. For documentation regarding the selection of the Federal Interest Plan, refer to Appendix A1, Part 3.

Federal Interest Plan refinements are required to aid in plan development to support a certified Class 3 cost estimate. Refinements utilized the tentatively selected plan hydraulic model used for comparison of alternatives to further design. Special attention was given to the refinement of major cost items or identified project risks including estimating earthwork, defining utility impacts, development of closure structures, identifying interior drainage requirements, and further understanding site constraints.

2 IDENTIFICATION OF FEDERAL INTEREST PLAN ALIGNMENT

The final alignment of the Federal Interest Plan was developed by incorporating minor shifts from the CA13 alignment to accommodate road alignments, minimize impact to utilities, optimize closure structure locations, and efficiently incorporate the proposed levee into the existing site terrain. **Figure 1** compares the final Federal Interest Plan alignment to the CA13 alignment. No wholistic alignment changes were identified post plan selection. Key revisions and refinements for hydrologic or hydraulic analysis to support the Federal Interest Plan design are summarized in this report. Revisions due to site considerations are included in Appendix A3, Engineering.

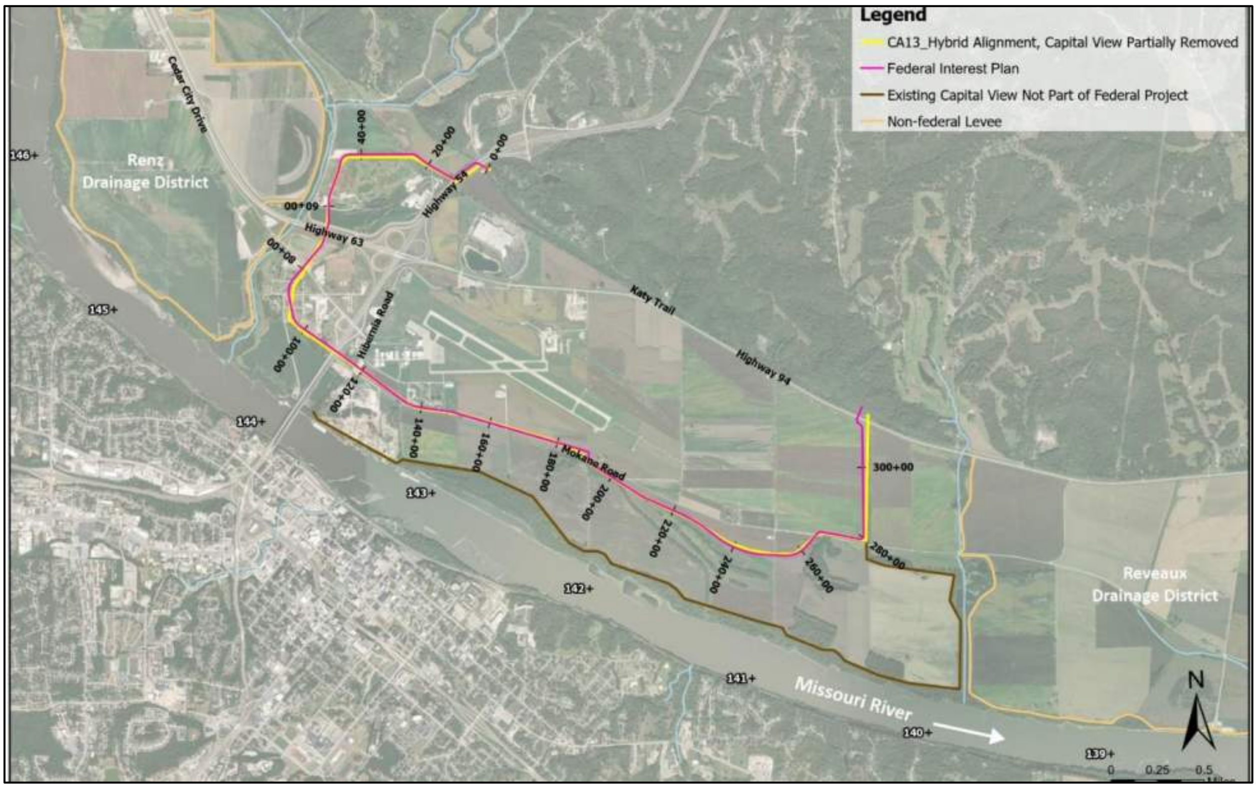

Figure 1. Comparison of CA13 and Federal Interest Plan Alignments

The Federal Interest Plan includes removal of the existing Capital View levee west of the U.S. Highway 54/63 overpass to allow additional conveyance and storage volume along Turkey Creek. The existing, non-federal Capital View levee east of the U.S. Highway 54/63 overpass is assumed to remain and is not part of the federal project footprint.

The minimum top of levee elevation is set at the profile produced by the 590,000 cfs (based on the 1% AEP event as identified at the time of this study), plus one foot of additional height for uncertainty. Upstream of the Missouri River bridge abutment, an extra foot of levee height was added to ensure initial overtopping downstream, consistent with ECB 2019-8 Managed Overtopping of Levee Systems. This design is expected to flood at less than 1% AEP with a 54% assurance of passing the 1% AEP event.

3 HYDRAULIC MODEL REFINEMENT

Hydraulic model refinement was driven by civil design modifications to the levee alignment and finalization of the top of levee height. Incorporation of the final Federal Interest Plan alignment and profile into the hydraulic model was conducted to confirm adjustments to the final alignment meet the original intent and produce comparable results to the CA13 modeling conducted for alternative selection, see Appendix A1, Part 3. Planimetric revisions of the levee alignment are discussed in Appendix A3, Engineering. The top of levee was refined as documented in Section 4 of this report. The levee refinements were incorporated into the hydraulic model by adjusting the 2D Structure named “L142”.

The hydraulic model was continued forward in HEC-RAS Version 6.4.1 software for consistency and efficiency. The HEC-RAS software updates since adoption of 6.4.1 for this study do not warrant a full update to the newest version. Attempts to run the model in HEC-RAS 6.6 resulted in errors at the Missouri River bridge, which is modeled as both a 1D bridge and 2D connection. The 6.6 version does not accept a 2D connection adjacent to a cell originating flow. In this case, the 2D connection is adjacent to the lateral structure representing the Missouri River top of bank which is required to move water from the 1D modeled main channel to the 2D overbank. Resolution of this error in HEC-RAS 6.6 would require geometry modifications to the FWOP model condition, which would require revalidation of the calibration and is not desired. The HEC-RAS Version 6.4.1 model is adequate for representing the site conditions and was utilized for model refinements.

4 Top of Levee Elevation Refinement

The top of levee elevation approach is guided by and in accordance with EM 1110-2-1913 *Evaluation, Design, and Construction of Levees* and ECB 2019-8 *Managed Overtopping*. The levee overtopping frequency was developed during the plan formulation process prior to the TSP milestone utilizing iterative hydraulic and economic models. Determination of the Federal Interest Plan and related design considerations are documented in Appendix A1 Part 3.

Design levee heights were further refined from the calculated water surface elevation (WSE) and energy grade line (EGL) elevations from the HEC-RAS model. To ensure the levee would first overtop at the downstream end, the design levee top elevation at the transition of the mainstem levee to the upstream tieback was provided a 2-ft superiority over the Missouri River 1% AEP maximum WSE. The top of levee elevation then transitions to a 1-ft superiority at the Missouri River bridge abutment which is carried through to a location downstream of the airport runway. The addition of superiority did not impact water surface elevation along Turkey Creek until an overtopping event, resulting in a water surface elevation increase of less than 0.1 feet. The top of levee then transitions to a 0.5-ft elevation above the maximum Missouri River WSE profile at the downstream tieback.

High water mark documentation from previous floods document increases in WSE near the bluff as compared to the main channel. This change in WSE is due to changes in velocity head and the transition from the faster moving channel to slower moving water near the bluff. The Missouri River through Jefferson City is straight, with the study area located on the inside of a very gradual curve. Per EM 1110-2-1601, the Missouri River is a tranquil flow system with gradual invert slopes, influential overbank storage, and a vast floodplain for overbank flow. Therefore, the amount of WSE superelevation anticipated through curves in the system is small and does not need special treatment for design except for consideration for the outside of the curve. In this case, the proposed Federal Interest Plan levee is on the inside of the river curve. To accommodate the potential for higher WSE and maintain the superiority of the levee profile, levee tieback elevations along Turkey Creek and Niemans Creek were determined utilizing the Missouri River profile EGL at the tieback locations. Coincident flows for the tiebacks were checked in Appendix 1.3 Section 8.7 and showed the Missouri River controls the design elevations. The upstream levee tieback utilized a consistent elevation of 0.5-ft greater than the Missouri River channel EGL elevation north of U.S. Highway 63, before it ties into the mainstem levee elevation. The downstream levee tieback was designed to increase to the Missouri River channel EGL elevation at the bluff line.

A summary of refined top of levee elevations at project landmarks are included in **Table 1**. The elevations provided in this appendix represent the project (final) levee grade, which does not include settlement overbuild. Settlement overbuild assumptions utilized for earthwork quantities are provided in Appendix A3.

Table 1. Refined Top of Levee Elevations – Federal Interest Plan

Levee Station (ft)	Elevation (ft NAVD88)	Elevation Determination	Location / Landmark
00+00	563.4	Missouri River EGL +0.5ft	Upstream Bluff Tieback
44+00	563.4	Missouri River EGL +0.5ft	Northwest Levee Tieback Inflection
67+00	563.4	Missouri River EGL +0.5ft	Highway 63 Embankment
94+00	562.2	Missouri River Max. WSE +2ft	Riverward Upstream Tieback
110+00	561.2	Missouri River Max. WSE +1ft	Missouri River Bridge Abutment
180+00	559.7	Missouri River Max. WSE +1ft	WSE Profile High Point
280+00	556.3	Missouri River Max. WSE +0.5ft	Riverward Downstream Tieback
316+60	557.4	Missouri River EGL	Downstream Bluff Tieback

Figure 2 compares the top of levee elevations of CA-13 utilized for determination of the TSP and the further refined recommened plan top of levee elevations utilized for the development of the Class 3 cost estimate. The figure also compares the existing ground surface, existing Capital View top of levee and Missouri River WSE and EGL.

The refined top of levee elevations maintain the intent of the tentatively selected plan. Results of the Missouri River WSE profile of the routed flow through the hydraulic model are identical to the tentatively selected plan results. Therefore, an average 82% assurance to pass a flow of 590,000 cfs (1% AEP as identified at the time of this study) developed for the TSP analysis remains applicable to the final plan. The 82% assurance applies for the levee reach upstream of the designated overtopping location. At the feasibility designed managed overtopping location, the calculated assurance level is 46%. Documentation of the assurance level is located in the economic appendix, Appendix E. The assurance level for this system is not an engineering performance target, as the Sponsor does not desire the option to accredit the levee through FEMA.

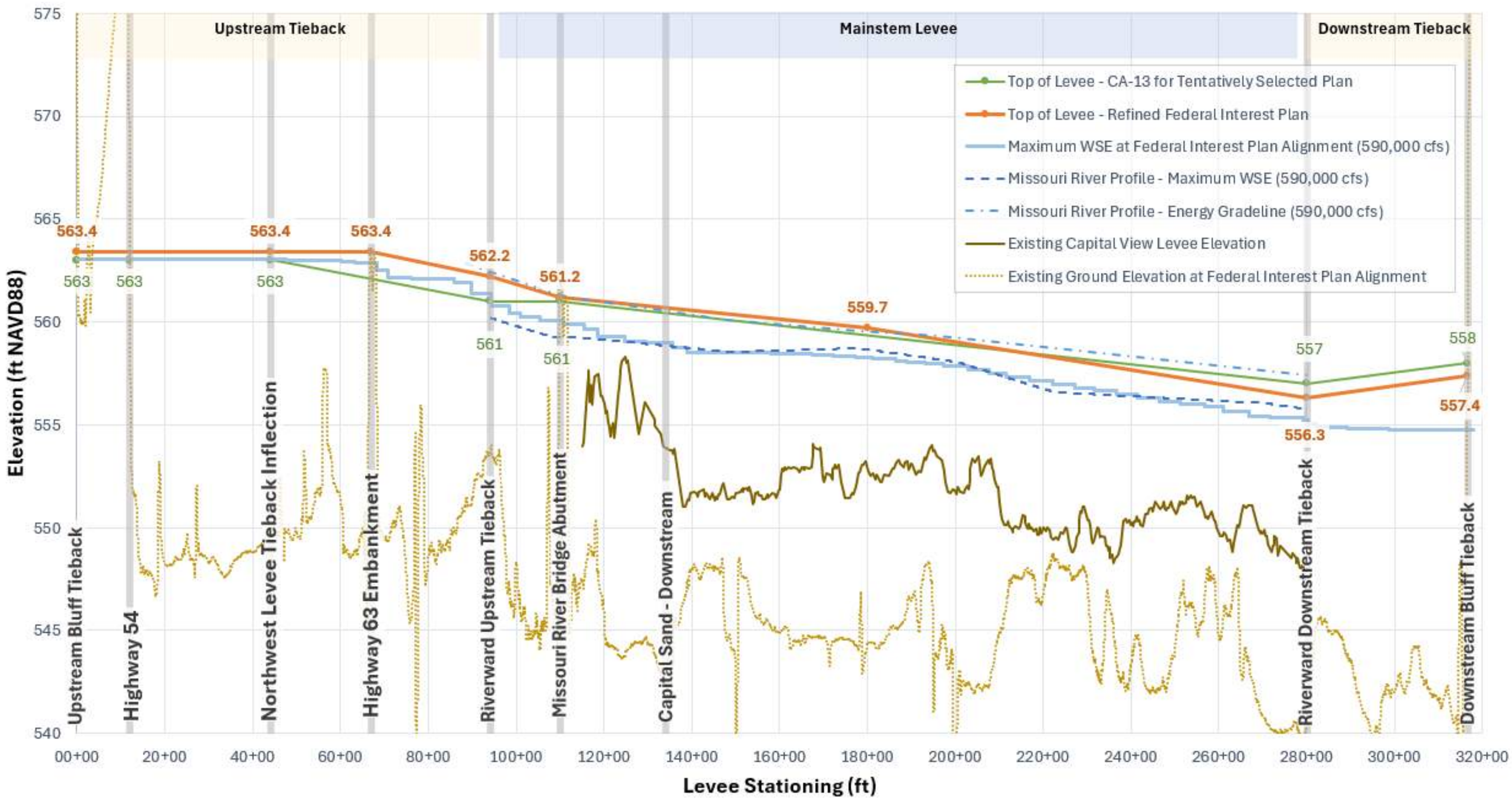

Figure 2. Refined Top of Levee Elevation Profile

5 INTERIOR DRAINAGE DESIGN

Interior drainage design was completed per EM-1110-2-1413 to finalize the quantity of outlets through the levee embankment and determine the coincident nature of flooding of the Missouri River and watershed of the interior area of the Federal Interest Plan levee alignment. Per ER 1165-2-21 in urban or urbanizing areas, provision of a basic drainage system to collect and convey the local runoff to a stream is a non-federal responsibility.

The level of design developed for the feasibility study interior drainage design effort was limited to defining the number of gates required, their general location, and size. Additionally, conservative interior ponding was analyzed for the 10% and 1% AEP events to ensure interior inundation during a gate closed scenario would not inundate critical facilities or roadways. The intent of the feasibility level design is to provide a confident analysis for development of a class 3 cost estimate. Conservative values were adopted to account for unknown conditions at the time of the study that may change due to the typical service life of enclosed storm systems and cross road culverts. During the PED phase, additional information will need to be gathered of the existing conditions to refine the conservative design.

The Federal Interest Plan includes only interior drainage facilities considered integral to the functionality of the recommended levee. At this site, an existing levee is present and interior drainage is present. Therefore, the scope of interior drainage facilities is limited to placement and sizing of drainage structures through the new levee. No extensive grading of the interior area will be included in the Federal Interest Plan. The minimum facility for interior drainage will provide interior flood relief such that during low exterior stages the local storm drainage system functions as it does in existing conditions.

Assessment of coincidence determined medium to high likelihood of interior and exterior events occurring simultaneously. Dependence is generally low for coincident events due to the different driving factors of Missouri River flooding versus interior peak flows. Missouri River events are driven by events over the entire watershed upstream of Jefferson City including regional wet seasons and timing of snowmelt. Depending on the origination of the flooding, Missouri River peak stages can take days to weeks to migrate downstream. Contrastingly, interior drainage peak flows are driven by localized rain events with relatively small time of concentrations due to the limited upstream area.

5.1 Future Without Project Condition

The existing Capital View levee has a total of 18 drainage structures through the levee. The existing culverts are corrugated metal with flap gates and range in size from 15 to 36-inches in diameter. Ten of the culverts exit the upstream tieback to Turkey Creek, seven exit the downstream tieback to Niemans Creek, and one 36-inch culvert discharges to the Missouri River. **Figure 3** maps the existing Capital View levee structures and related drainage areas.

The interior area of the Capital View levee is located within the natural Missouri River floodplain and is very flat. The area does not drain well, resulting in scattered low-points and nearly flat ditches that can hold water for long durations. Within the interior area, drainage is generally dictated by roadway and airport ditches that route water away from the pavement. Interior drainage northwest of the Highway 63/54 cloverleaf interchange does not appear to follow a channelized route and is intercepted by a series of golf course lakes. Historic aerial images show a depression southeast of the golf course that appears to hold water and act as a detention pond in wet conditions. Southwest of the Highway 63/54 interchange, a road network from the previous Cedar City remains in place, including the crossroad culverts generally routing drainage west to Turkey Creek. East of Highway 54, drainage ditches are very flat and route 2.5 miles east to Niemans Creek by means of Mokane Road ditches or through the open field east of the airport. Roadways and the airport facilities are built up in elevation, aiding drainage away from these structures, however much of the agricultural land is low-lying and does not have distinct drainage patterns. From east to west, the maximum elevation difference over two miles is approximately 10-feet, resulting in a maximum straight-line slope to drain of 0.09% slope (approximately 1-inch vertical per every 100 horizontal feet).

The area south of Mokane Road was heavily impacted by scour during the Flood of 1993, requiring realignment and setback of the existing Capital View levee downstream of the Missouri River bridge. A channelized ditch is present in this location that directly outlets to the Missouri River through a 36-inch culvert. West of the ditch, near the Capital Sand plant, the land does not efficiently drain and often results in saturated soils (confirmed by aerials, site visits, and discussions with stakeholders).

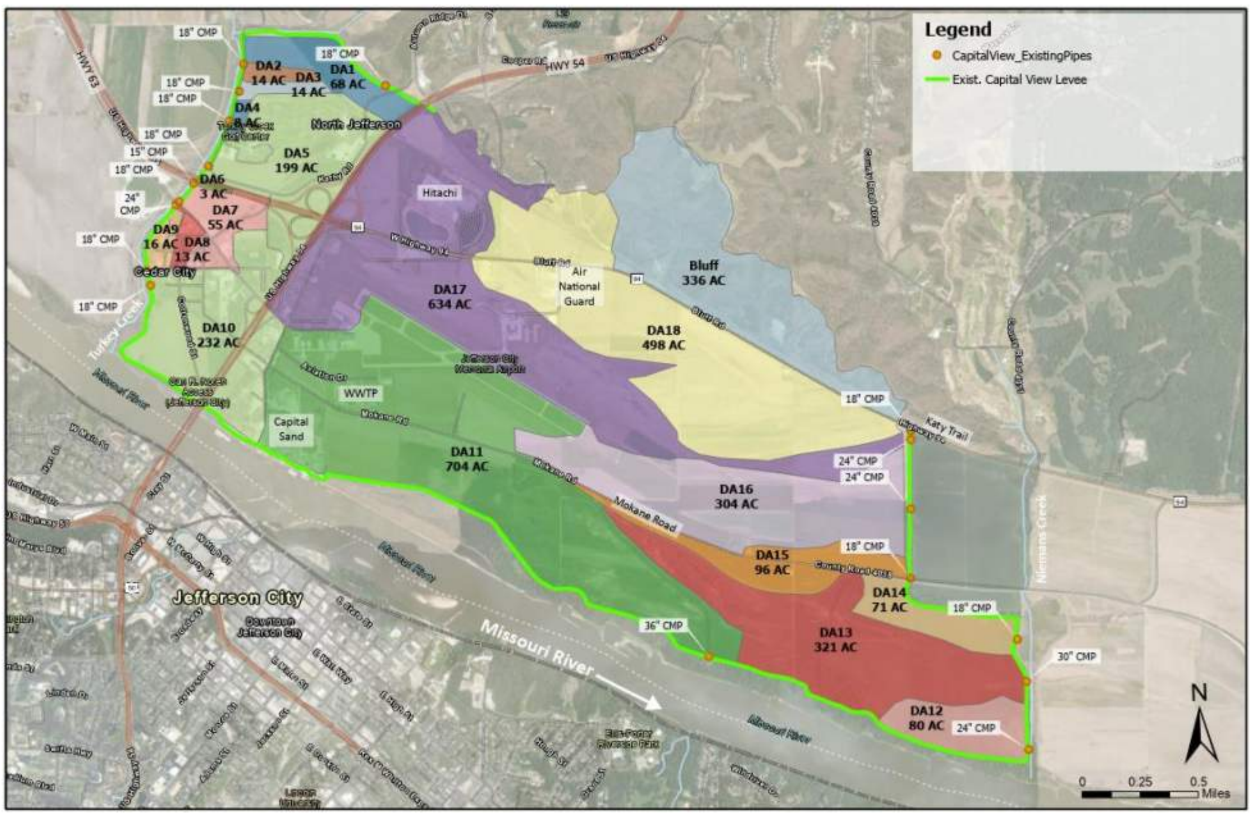

Figure 3. Existing Capital View Levee Drainage Areas and Outlet Structures

In existing conditions with the presence of the Capital View levee, the interior area is fully inundated with the exception of isolated high ground south of Highway 94 and the Hitachi property north of Highway 94 in the 10% AEP Missouri River flood. The existing Capital View levee is overtopped more frequently than a 10% AEP - at a 17% AEP Missouri River event.

For peak flow events originating from the interior watershed when the Missouri River is not in flood stage, the interior drainage condition is assumed to be sized to pass a minimum of 10% AEP peak flow as dictated by Jefferson City's adoption of the KC-APWA 5600 (2011) design criteria (Code of Ordinances 31.II. Sec. 31-200.A..1). Highways within the project area are assumed to be designed for the Missouri Department of Transportation (MODOT) design standard of passing a 2% AEP peak flow based on roadway classification. Similarly, buildings are assumed to be at an elevation that would provide a minimum of 1-ft freeboard during a 1% AEP event deriving from the upstream interior watershed per adopted KC-APWA 5600 design criteria and City floodplain development policy.

5.2 Federal Interest Plan Interior Drainage Considerations

Proposed culvert and gatewell locations align with existing conditions to minimize grading needed to connect the existing interior area drainage channels, shown in **Figure 4**. Where the levee construction footprint is allowed, existing ditches are assumed to be intercepted by final grading and combined to reduce the total number of drainage structures. Consolidating structures from 18 gravity outlets to 8 will result in less long-term maintenance needs and result in larger culvert sizes which are easier to inspect and clear of sediment. All outlets through the levee embankment are designed as gravity outlets.

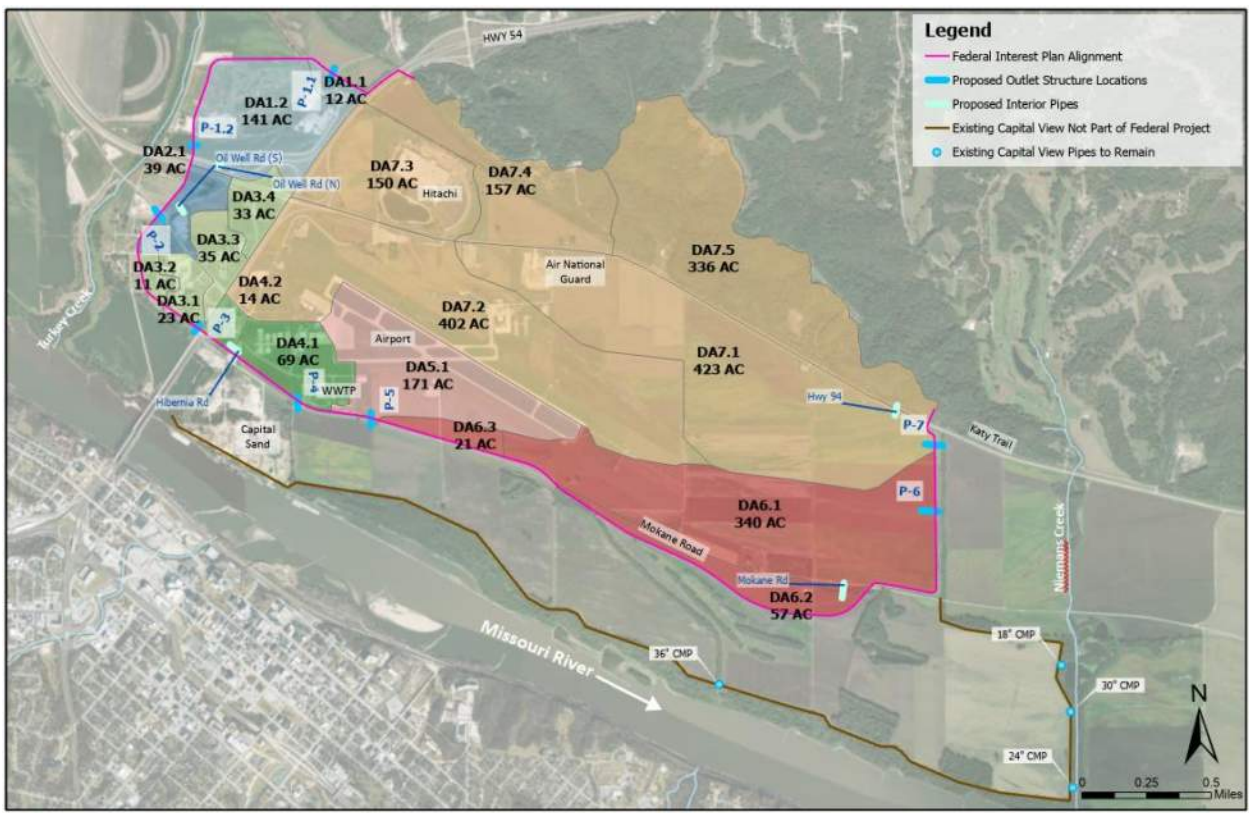

Figure 4. Federal Interest Plan Drainage Areas and Levee System Outlet Structures

Pumping stations were determined to be ineffective for this location due to the small interior drainage footprint and the flatness of the topography behind the levee. Limited fall in grade at the site would require a large grading effort to consolidate stormwater volume to a central location. In lieu of a centralized pumping station, mobile pumps are included in the Federal

Interest Plan cost estimate to assist with floodfight efforts and mitigate ponding at the interior levee toe. Likewise, interior detention storage basins were not considered for this site due to limited elevation change within the internal drainage area. There is also not enough elevation difference to drain from the interior elevation to the Missouri River elevation. Inclusion of a detention basin would also require further real estate analysis of the change in internal drainage patterns. Additionally, the Federal Aviation Administration maintains restrictions regarding ponding of water near an airport which restricts the creation of depressions that could hold water in this location. The project will be equipped with gravel-surfaced pads to serve as sites for the temporary pumping facilities. Because these internal surface waters will tend to follow the existing internal swales and ditches and accumulate at the several gatewell inlets, these pads will be sited at or near the proposed gatewells.

Geotechnical assumptions for the Federal Interest Plan are summarized in Appendix A3, Chapter 3.11. The Federal Interest Plan will include underseepage berms for remediation but does not require relief wells. Underseepage contributions for existing conditions were assumed to have a negligible impact and would be partially mitigated with the proposed portable pumps.

Development of peak flows for sizing of gravity outlets was conducted utilizing the Rational Method for all drainage areas totaling less than 200 acres. The USGS Regional Rural Regression Equation was utilized to develop the peak flow for drainage area DA-6. HEC-HMS was utilized as a tool to organize the DA-7 sub-drainage areas and output hydrographs that could be input into HEC-RAS.

Surface runoff from precipitation is the predominate source of interior drainage to the levee. No agricultural irrigation field runoff was assumed. The wastewater treatment facility discharge is at the Missouri Riverbank, riverside of the recommended alignment. The golf course located at the upstream levee tieback is assumed to match flows that would be typical of undeveloped park land. Irrigation at the golf course is likely, however the interior ponds are assumed to offset both irrigation and precipitation runoff.

A HEC-RAS model was created to determine flood inundation extents behind the proposed levee in the event of a 1% AEP internal drainage peak flow event in conjunction with levee gate closure (high stage on the Missouri River). This model was created by utilizing the 2D area from the Jefferson City study Federal Interest Plan model including the 2D connection used to represent the Federal Interest Plan alignment and profile. The base 2D area mesh (500-ft mesh) from the base model was refined landward of the Federal Interest Plan alignment to have a 200-ft mesh. Breaklines were maintained to enforce paved roadways in the mesh. The denser mesh better represents routing of the lower flow events associated with internal drainage. Boundary conditions defined as a hypothetical SCS hydrograph with a Type II rainfall distribution for the 1% AEP peak flow were added to the 2D flow area for each defined internal drainage area as shown in **Figure 5**. Where an obstruction is present in the drainage area, such as a roadway culvert of an unknown size, the boundary condition was placed downstream of the obstruction to provide a conservative estimate of inundation at the Federal Interest Plan alignment. A summary of boundary condition peak flow development is listed in **Table 2**, **Table 3**, and **Table 4**.

Table 2. Drainage Area Peak Flow Development – Rational Method

DA Name	Area (Acres)	% Impervious	Flowpath (ft)	C	Tc Time of Conc. (min)	Peak Flow (cfs) 10%	Peak Flow (cfs) 4%	Peak Flow (cfs) 2%	Peak Flow (cfs) 1%
DA1.1	11.8	0%	710	0.3	16.8	17	22	27	31
DA1.2	141.0	8%	2,900	0.3	23.2	210	272	330	382
DA2	38.7	5%	1,900	0.3	20.2	59	76	92	106
DA3	101.3	12%	3,300	0.4	24.1	158	204	248	287
DA4	82.9	40%	2,100	0.5	17.1	220	282	342	395
DA5	170.9	53%	4,000	0.6	22.1	462	596	724	836

Table 3. Drainage Area Peak Flow Development – Rural Regression Equation

DA Name	Area (Squ. Miles)	Flowpath (ft)	Basin Shape ($Flowpath^2$/ DA)	Peak Flow (cfs) 10%	Peak Flow (cfs) 4%	Peak Flow (cfs) 2%	Peak Flow (cfs) 1%
DA6	0.7	9400	4.86	507	699	849	1,003

Table 4. Drainage Area Peak Flow Development – TR-55 SCS Method (HEC-HMS)

DA Name	Area (Acres)	% Impervious	Area (Squ. Miles)	Flowpath (ft)	Tc Time of Conc. (min)	Lag Time (min)	CN	Peak Flow (cfs) 10%	Peak Flow (cfs) 2%	Peak Flow (cfs) 1%
DA7.1	422.9	1%	0.66	10,200	48.3	29.0	74	589	1,019	1,236
DA7.2	401.7	8%	0.63	10,300	47.8	28.7	75	591	1,008	1,218
DA7.3 *	150.4	10%	0.24	2,150	20.5	12.3	75	379	639	769
DA7.4	157.1	3%	0.25	5,875	33.7	20.2	74	284	489	593
DA7.5	336.0	3%	0.52	5,250	31.6	19.0	74	615	1,059	1,282

* Impervious area edited from true 28.8% to 10% to account for detention basin onsite.

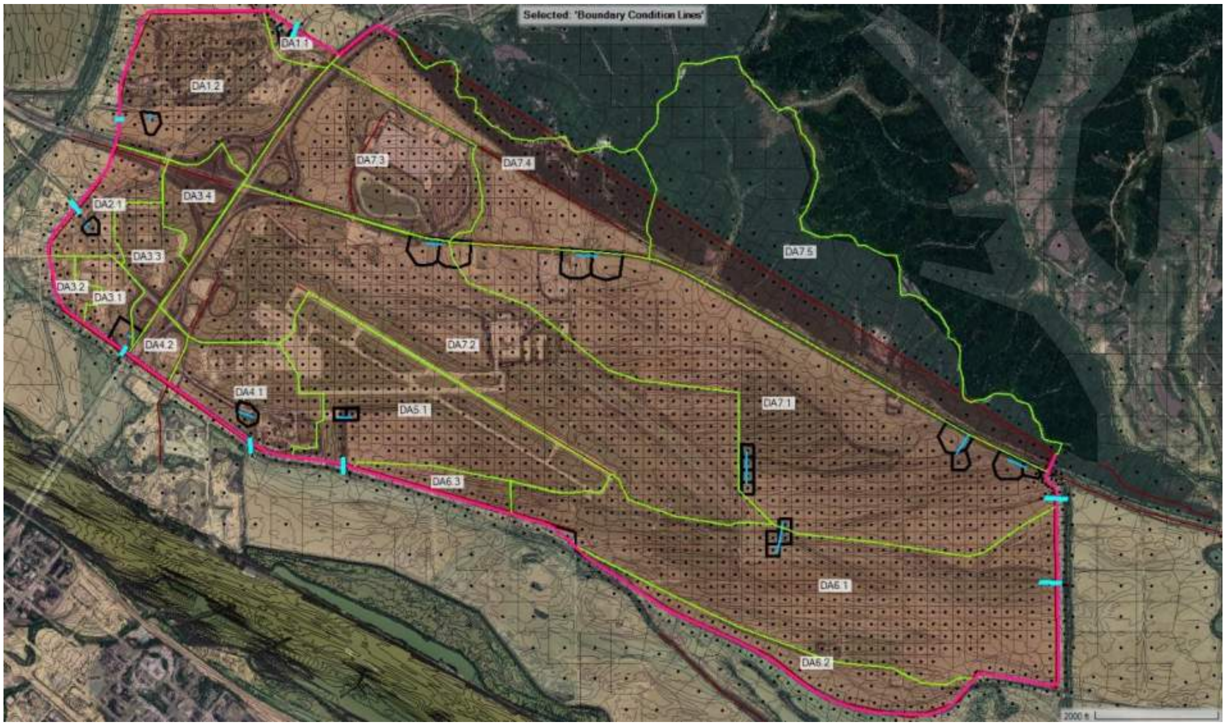

Figure 5. Boundary Condition Locations for Internal Drainage Modeling

To determine the maximum inundation extents, all proposed gates were set to have hypothetical flap gates to not allow positive flow. The interior drainage area provides adequate storage for flood volumes without impact to critical facilities and roadways. The maximum inundation extent for the 1% AEP internal drainage flow event during a gate closed condition is shown in **Figure 6**. Boundary conditions are labeled at the locations where flow is inserted into the model. Adjacent facilities utilized for determining the maximum headwater elevations are depicted by yellow markers and labeled.

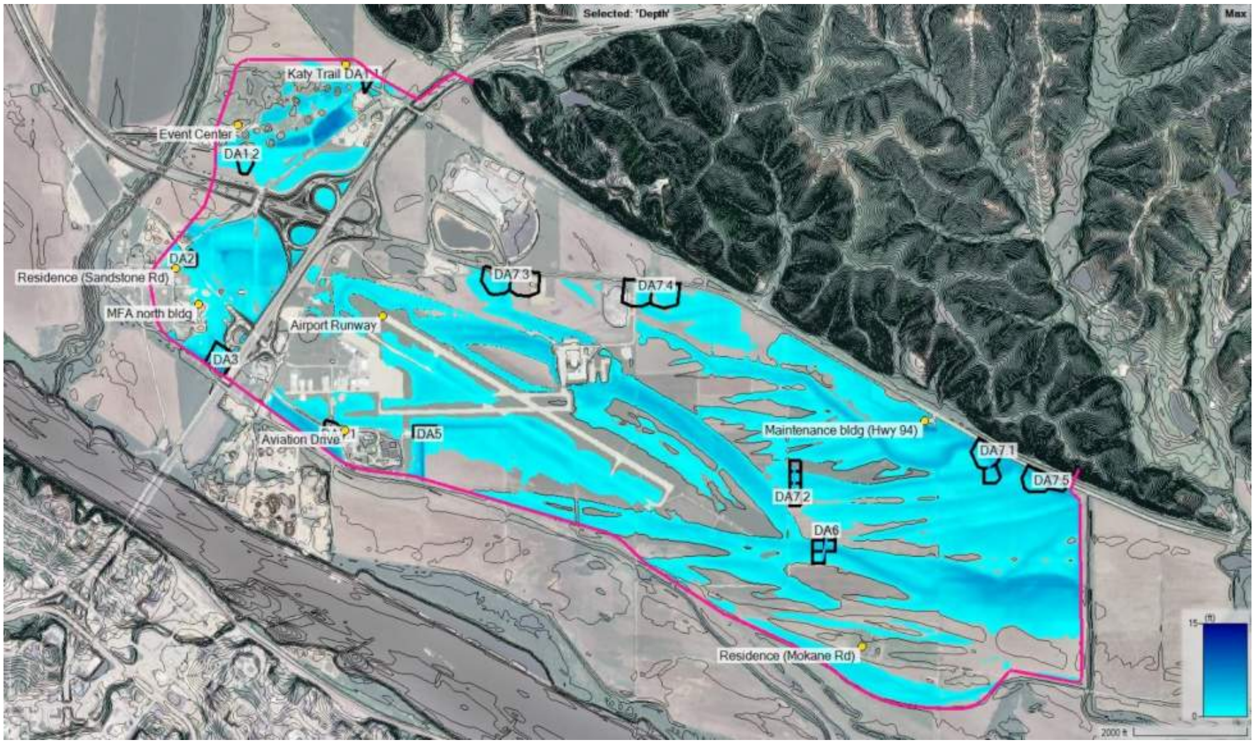

Figure 6. Maximum Inundation Extents for the Interior 1% AEP Flow Event

Impacts due to the closure of levee outlet gates, damming internal drainage, is unlikely to cause effects greater than the existing condition where overtopping of the Capital View levee leads to nearly complete inundation of the leveed area in the 10% AEP Missouri River event.

5.3 Levee Outlet Structures

The system of gatewells for this project is designed to accommodate runoff from rainfall that occurs during periods of low river stages. The gatewell sluice gates will be in the open position for these periods. However, during periods of high river stages these sluice gates will be closed and all internal surface water will accumulate along the interior faces of the levee or its underseepage berms. Levee outlet structures were identified as shown in **Figure 4**.

Sizing of the gatewell structure pipes was iterative. First, the pipe size needed to pass the 2% AEP peak flow was calculated using the Manning's equation. In an open gatewell condition, this is equivalent to the maximum passing size of MODOT structures upstream. Peak flows were then placed into the internal drainage HEC-RAS model by scaling a hypothetical SCS hydrograph with a Type II rainfall distribution to match the targeted peak flow. Gatewell pipe sizes were refined with the interior drainage HEC-RAS model to ensure routing of peak flows and storage of water volume was accounted for. In most cases, routing of the hypothetical peak flow across the flat terrain results in detention of water upstream of the gatewell. Therefore, resulting peak flow experienced at most gatewell structures is less than the peak hypothetical value and the size of the gatewells could be reduced. At locations where the drainage patterns are generally contained to roadway ditches or otherwise prevented from inundating areas that would detain water, a conservative approach to sizing was maintained and pipes were sized for the unrouted hypothetical peak flow developed. A maximum storage headwater limitation was set for each gatewell location by evaluating the lowest structure or roadway elevation (adjacent facilities) within the drainage area to limit impact. The headwater limitation was verified in the HEC-RAS model to ensure the maximum inundation extents of the hypothetical 1% AEP event would be at or below adjacent facility elevations in a Missouri River flood stage condition where the gatewells were in a closed position and all internal drainage was being stored.

For maintenance considerations, a minimum pipe diameter of 48-inches was assumed. The only location affected by this minimum assumption is P-1.1, where sizing the pipe outlet utilizing the Manning's Equation resulted in a calculated pipe size of 30-inches.

Table 6 summarizes all levee outlet structure assumptions for the development of the Federal Interest Plan estimate.

5.4 Internal Drainage – Interior Pipes

In select cases, interior drainage patterns are affected by the proposed levee and will require interior pipes to allow for unrestricted flow. These pipes are light blue in color in **Figure 4**. All pipes were sized to pass the 2% AEP peak flow for their respective drainage area using the Manning's equation. The HEC-RAS modeling does not reflect these pipes because runoff was routed via terrain modifications or the location was bypassed with a downstream boundary condition. Interior pipes are summarized in **Table 5**.

Table 5. Summary of Interior Pipes

Pipe Name	Drainage Area (ac)	Design Flow (cfs)	Pipe Type	Pipe Size (in)
Oilwell Road (North)	3.3	12	RCP	24
Oilwell Road (South)	13.5	50	RCP	36
Hibernia Road	14.2	79	RCP	42
Mokane Road	56.6	201	RCP	60
Highway 94	336.0	575	Concrete Box	6' x 6' (equivalent pipe size of 90-in)

5.5 Interior Drainage Interaction with Capital View levee

Existing drainage patterns were maintained to route the same amount of water to the four remaining Capital View levee pipes that will not be touched with the federal project. These four remaining pipes are within the Capital View levee segment at the southeastern quadrant of the project site, riverward of the L142 tieback. Drainage area delineation for the existing condition is provided in **Figure 3**. Drainage area delineation for the recommended condition is provided in **Figure 4**.

Table 6. Federal Interest Plan Summary of Drainage Structures

Pipe Name	Station	Drainage Area (ac)	Peak Flow Calculation Methodology	Pipe Size Determination Methodology	Top of Levee Elevation	Approx. Invert Elevation	Pipe Size (in)	Pipe Type	Min. Pipe Slope	HW Elev (10% AEP)	HW Elev (2% AEP)	HW Elev (1% AEP)
P-1.1	**20+30**	12	rational	Manning's Equ.	563.4	548.3	48	RCP	0.002	547.6	547.8	547.9
P-1.2	**62+80**	141	rational	HEC RAS storage	563.4	547.8	60	RCP	0.005	548.4	548.6	548.7
P-2	**79+20**	39	rational	Manning's Equ.	562.8	547.3	60	RCP	0.005	547.1	547.3	547.4
P-3	**109+00**	101	rational	HEC RAS storage	561.2	541.8	60	RCP	0.005	545.2	545.9	546.2
P-4	**133+00**	83	rational	HEC RAS storage	560.7	538.3	60	RCP	0.005	543.2	543.9	544.2
P-5	**150+00**	171	rational	HEC RAS storage	560.3	538.3	60	RCP	0.005	543.6	544.3	544.5
P-6	**296+20**	418	USGS rural regression	HEC RAS storage	556.8	537.6	60	RCP	0.005	542.6	543.6	543.8
P-7	**309+70**	1468	TR 55 SCS (HEC-HMS) Routed through HEC-RAS	HEC RAS storage	557.2	539.0	60	RCP	0.005	543.6	544.0	544.2

6 Levee Features Refinement

6.1 Embankment Armoring

The largest erosive threat for the proposed earthen levee embankment is river-flow velocity-induced shear during high Missouri River events. However, the toe of the proposed levee is in-land from the riverbank on average 1,500 feet and will only be loaded during high water conditions. The Missouri River is stabilized in-place for navigation and is not likely to incur a shift in channel. Wind-wave is not a driving consideration for the river system.

Velocities increase to the highest levels of interaction with the levee for the 2% and 1% AEP events prior to levee overtopping. Maximum velocities at the levee face are 3.5 ft/s. The area of highest velocity is downstream of the Missouri River bridge and Capital Sand plant. This area has historically experienced breaches of the existing non-federal levee. Like in existing conditions, the river velocity regime for a typical Missouri River floodplain cross section experiences the maximum velocity within the channel. Overbank flow routes are interrupted by variation in surface elevations, roadway embankments, and changes in land cover.

Due to the considerable overbank providing a buffer from the Missouri River channel and because velocities are less than 5 ft/s adjacent to any proposed levee face, embankment armoring is not needed to protect from shear stresses originating from river flow.

Past experience with the overall Missouri River system suggests riprap on the upstream tieback would be warranted to protect the embankment in areas of overbank contraction. In existing conditions, the U.S. 54 Highway embankment and Missouri River bridge abutment in the left overbank create a restriction of the floodplain. When fully inundated during a Missouri River flood event with a flow greater than 10% AEP, inundation expands bluff-to-bluff to a width of 9,000 feet. The active conveyance area, corresponding to the mapped FEMA floodway, totals 7,000 feet upstream and downstream of Jefferson City. Through the study area, the Missouri River bridge abutment on the left bank restricts flow to a 2,950 foot cross section, see **Figure 7**. The Federal Interest Plan maintains this restriction, tying into the abutment at an elevation approximately 3-feet below the existing abutment height. Historically, locations within the river system that cause constrictions can experience scour during high flows. This change in the otherwise uniform floodplain is a natural place for erosion to develop due to flow dynamics around the transition.

For feasibility level analysis, riprap armoring is included in the cost estimate from the Old 63 Road closure structure (Station 79+40) to the Missouri River bridge abutment (Station 110+00), **Figure 8**. Due to the overbank location resulting in mild velocities at the levee face, sizing of the riprap was not driven by scour velocities and instead a gradation controlled by the minimum size to prevent vandalism (W_{50} of 80 lbs) was adopted per EM 1110-2-1601.

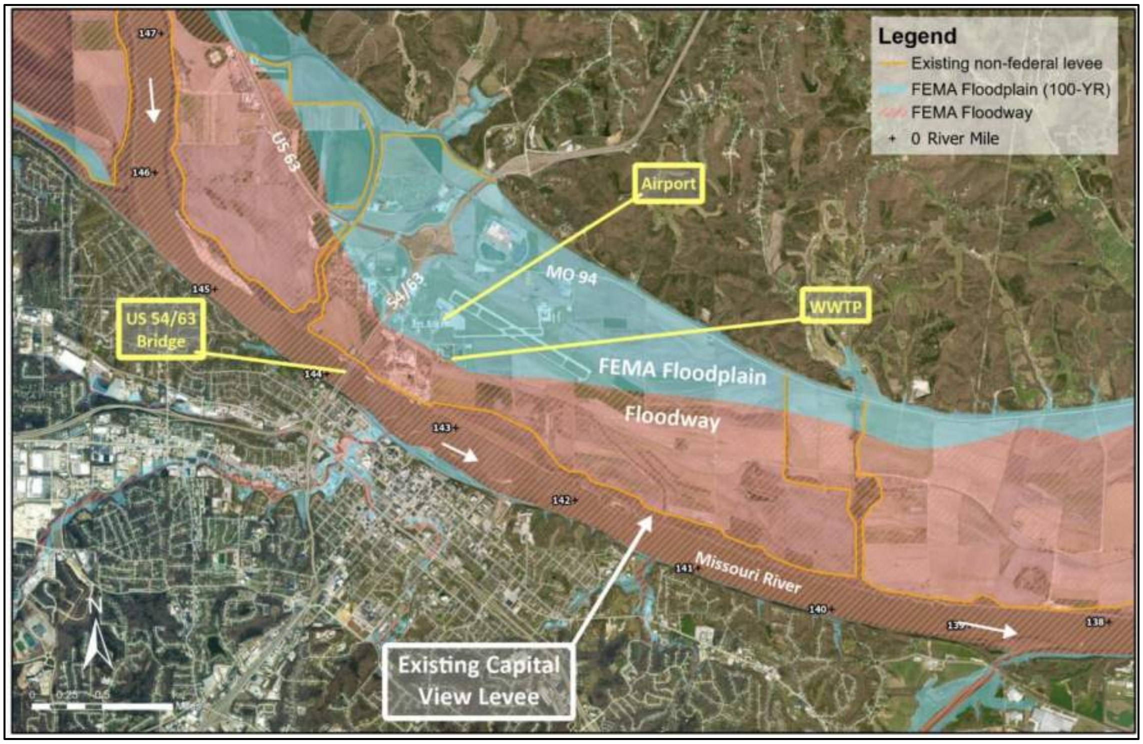

Figure 7. FEMA Mapped Floodplain through Study Area

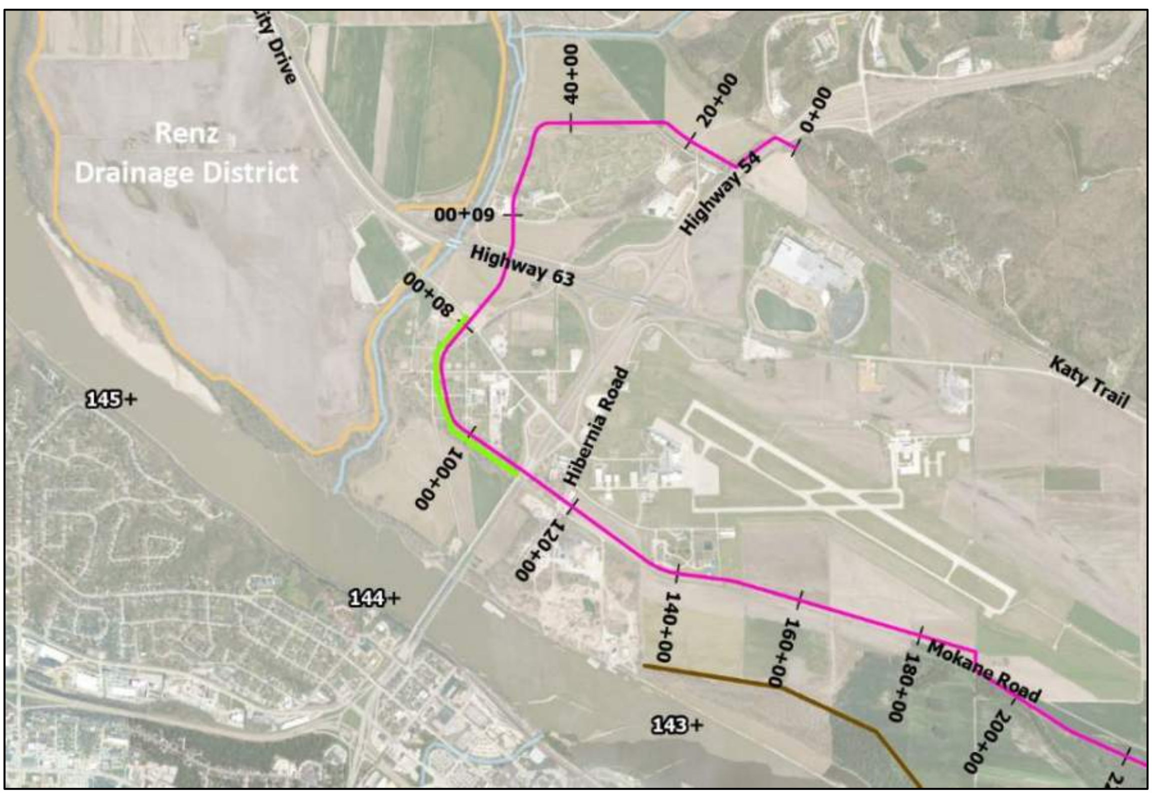

Figure 8. Federal Interest Plan Limits of Riprap Embankment Armor (green line)

6.2 Freeboard Gages

Freeboard gages are utilized during flood events to obtain accurate localized water levels and implement the flood action plan. For the feasibility estimate, it was assumed a freeboard gage will be installed at each gatewell, for a total of eight. The spacing of the gatewells allows for full coverage along the full top of levee profile, WSE profile differences due to river geometry, and tiebacks. The overtopping location of the levee is within 1,000 feet to the proposed gatewell outletting to Niemans Creek.

7 RESIDUAL RISK

As with all risk reduction projects, occupants of the floodplain will always have risk associated with the land use. This risk will remain after completion of the Federal Interest Plan. Residual flood risk from infrequent, large flood events that overtop the proposed top of levee. Many factors influence the nature of flood events and the performance of flood risk management systems, such that an event of historical magnitude is not necessarily required to overwhelm the project and cause catastrophic damage. However, the implementation of a project may lead to public perception that the area interior to the flood risk management system provides an unjustified level of security against flooding. Therefore, it is important to emphasize and communicate the level of flood risk that remains after project implementation such that floodplain occupants are aware of the nature of the flood threats and are able to make informed decisions about acceptable levels of risk.

The goal of the Federal Interest Plan design assessed in this feasibility report is to manage the risk of flooding by implementing a cost effective and efficient flood risk management plan that reasonably maximizes benefits.

Lands within leveed areas are inherently at risk of damage from flooding. These risks will remain regardless of modification to levee design or maintenance. Regular practices for public information and communication coordination between USACE and Jefferson City will be established with the Federal Interest Plan to ensure an Emergency Action Plan is in place and Operation and Maintenance needs are coordinated.

The Federal Interest Plan provides a significant increase in reliability against flooding. Flooding of the leveed area will be less frequent; however, the residual risk of flooding remains. Although the Federal Interest Plan passes a much greater flow than the existing levee, a historic flood event would overtop the Federal Interest Plan. Additionally, a levee exposed to riverward loading for any length of time has a chance of breach. The Federal Interest Plan will reduce the chance of levee exceedance from the Missouri River. It will also provide more reliable protection from high Missouri River flows and allow for additional evacuation time in the event of a rising flood event. However, significant risk remains to the community after the construction of the Federal Interest Plan.

Hydrologic and hydraulic considerations for residual risk associated with the Federal Interest Plan include:

1. Warning Time of Impending Inundation: The lower Missouri River corridor has a robust gage network with National Weather Service flood prediction. Missouri River floods in this location typically have days of warning time for preparation and evacuations as defined in the Emergency Action Plan and Operation & Maintenance Manual. Warning time was taken into account for the design of closure structures for this levee, to ensure closures could occur with limited staff or in a timely manner for evacuation routes. For example, the access road to Capital Sand is defined as a rolling gate closure structure to allow for a quick closure which will allow for flood fighting materials to be accessible as long as possible.

2. Rate of rise, duration, depth, and velocity of inundation are summarized in Appendix A1.3 Section 8.4, 8.5, and 8.6. Federal Interest Plan implementation is not anticipated to

change the overall Missouri River regime or timing as evaluated with the FWOP including rate of rise, duration, or velocity. Depths with the Federal Interest Plan will experience a localized increase adjacent to the project alignment for events that result in nearly full inundation of the floodplain in the FWOP condition. The City's existing flood action plan will still be relevant with the implementation of the Federal Interest Plan, with the addition of closure structure and gatewell closures for the Federal Interest Plan as specified in the Operation & Maintenance Manual. Closure structures and gatewell structures will be provided closure and warning times during PED.

3. Mapping of inundation for all modeled frequency flows are provided in Appendix A1.3 – Attachment D including discussion on differences between the FWOP and Federal Interest Plan conditions.

4. With the Federal Interest Plan, access and egress is generally improved in duration for access to jobs, transportation, and emergency care during flood events. See the Economics appendix for additional information.

5. Potential risk for loss of life and physical damages as evaluated with the Federal Interest Plan is provided in the Economics Appendix.

8 COINCIDENT FLOW SENSITIVITY TESTS

Sensitivity of the Missouri River water surface profile to the flow of adjacent major tributaries was analyzed to confirm design conditions for defining the top of levee elevation. This sensitivity test confirms that improbable, coincident backwater conditions from Moreau and Osage River events are unlikely to cause impactful water surface elevation changes related to the Federal Interest Plan design overtopping location or overall levee performance. The base alternatives model (discussed in Appendix A1 – Part 1, 2, and 3) used peak flows on modeled major tributaries to achieve targeted mainstem flows on the Missouri River. This analysis investigates the sensitivity of the project to potential backwater impacts in the event of major flows on tributaries directly adjacent to the project. Sensitivity was tested by routing the 1% AEP Missouri River flow but increasing the tributary hydrograph in question to match the peak 1% AEP major tributary event flow as defined by the FEMA FIS. The hydrograph timing of the tributary was also adjusted to match the tributary peak to the Missouri River peak flow at the Jefferson City gage. **Figure 9** summarizes the findings of Missouri River maximum water surface sensitivity adjacent to the recommended alignment for coincident major tributary flow events.

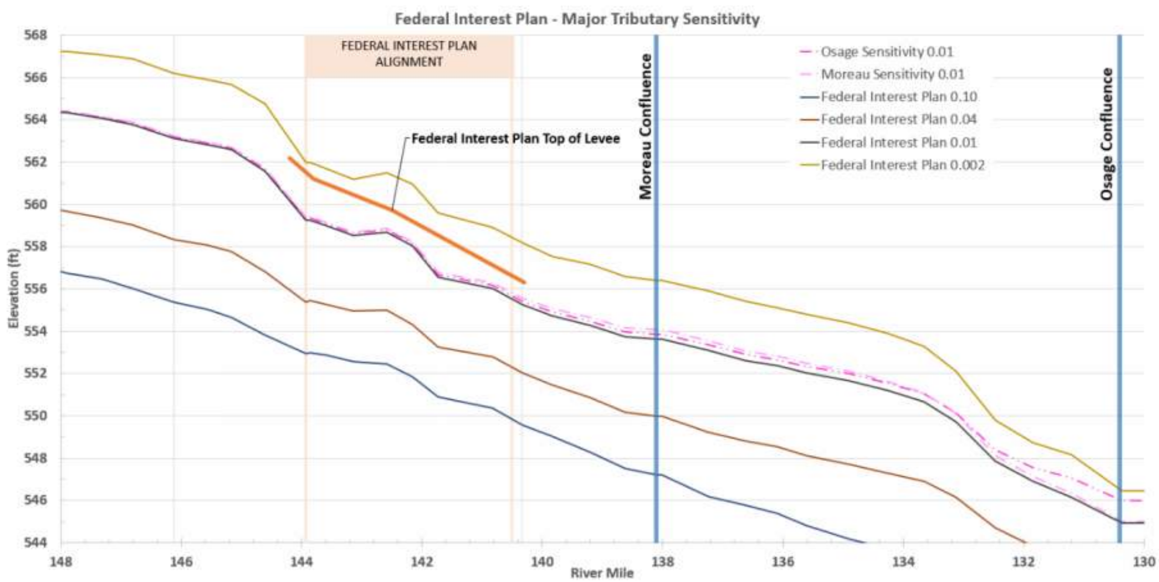

Figure 9. Missouri River Maximum Water Surface Elevation Profile Sensitivity to Major Tributaries

Coincident flows at the upstream and downstream bounding minor tributaries of Turkey Creek and Niemans Creek were evaluated during alternative analysis and are summarized in Appendix A1 Part 3 Section 8.7. Sensitivity was tested by adding lateral inflow hydrographs at the tributary tiebacks with 1% AEP peak tributary flows timed to coincide with the Missouri River peak 1% AEP flow. Impacts to WSE at the levee tiebacks were negligible.

8.1.1 Osage River Coincident Flows

The Osage River confluence with the Missouri River is located fourteen miles downstream of the Jefferson City gage, approximately nine miles downstream of the Federal Interest Plan alignment downstream tieback. The Osage River basin has multiple mainstem dams. The Harry

S. Truman Dam is operated by USACE for multiple uses including flood control, with regulated flow targets on the Osage River at St. Thomas, MO and the Missouri River at Hermann, MO.

For feasibility alternative comparison, modeled Osage River gage targets ranged from 28,600 cfs to 78,650 cfs paired with the 50% to 0.2% AEP Missouri River target flows respectively. For the 1% AEP Missouri River event corresponding to the Federal Interest Plan passing flow, the Osage gage flow was 68,640 cfs to meet the Missouri River 2023 Flow Frequency flow targets at the Hermann gage. However, in the past 10 years, this flow has been exceeded four times on the Osage River, making it necessary to understand the impact of a high flow on the Osage River on the Federal Interest Plan levee.

The 2018 scaled hydrographs utilized for the development of the FWOP and alternatives model for the Jefferson City study reflect upstream water management on the Osage River. The Missouri River peak correlates with a regulated decrease in flow releases on the Osage River upstream of Harry S. Truman dam (**Figure 10**). Therefore, to provide a coincident high flow for both the Missouri River and Osage River systems, the flows between model time of January 7, 2099 to January 12, 2099 were raised to interpolate between the bounding high points (**Figure 11**) for the sensitivity test.

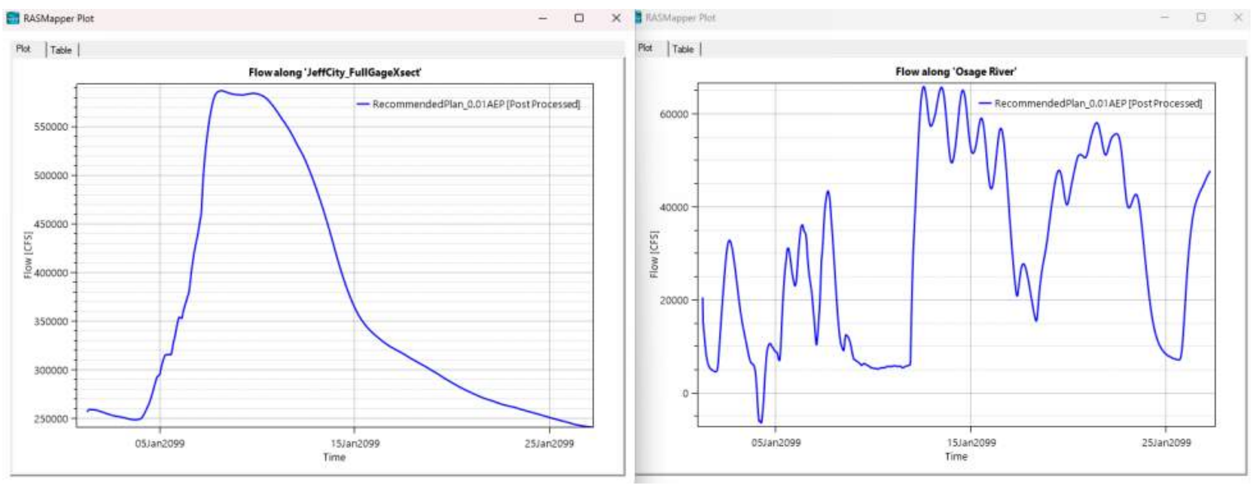

Figure 10. Scaled 2018 Hydrographs for development of the 1% Missouri River AEP event

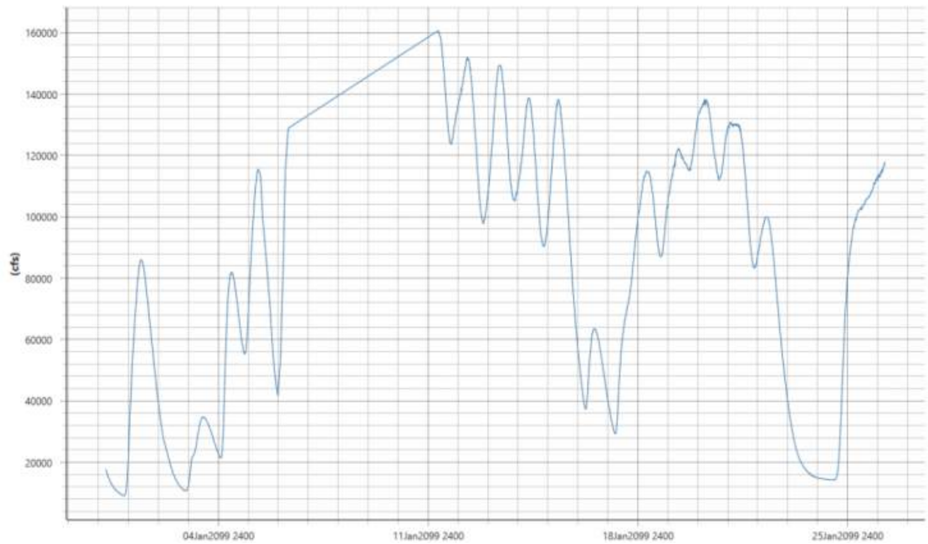

Figure 11. Modified Osage River Hydrograph to time Coincident Flow

To test sensitivity of the WSE for the Missouri River design peak flow, the Osage River boundary condition flow at the St. Thomas gage in the hydraulic model was increased to 160,000 cfs. This value was developed by utilizing the Osage River period of record and FEMA FIS peak discharge value for the 1% AEP Osage River flow to determine the realistic upper limit of Osage River flow when the Missouri River peak flow at the Jefferson City gage is 590,000 cfs (1% AEP as identified at the time of this study).

The maximum increase in the Missouri River water surface profile through the study area adjacent to the Federal Interest Plan alignment is 0.3-ft (3.6 inches) during the 1% AEP Missouri River event coinciding with the 1% AEP Osage River flow of 160,000 cfs. This difference does not justify a change in the top of levee elevation approach.

8.1.2 Moreau River Coincident Flows

The Moreau River confluence with the Missouri River is located six miles downstream of the Jefferson City gage, approximately two miles downstream of the Federal Interest Plan alignment downstream tieback. Flow to the confluence of the Moreau River and the Missouri River is unregulated.

Modeled Moreau River gage targets for peak flow for feasibility alternatives modeling ranged from 4,700 cfs to 28,700 cfs for the 50% to 1% AEP Missouri River target flows respectively.

To test sensitivity of the WSE for the Missouri River design peak flow, the Moreau River boundary condition was increased to 40,000 cfs to represent a 1% AEP flow for the Moreau River as defined by the FEMA FIS. The 2018 peak flow hydrograph utilized for development of the FWOP and alternatives analysis was adjusted to ensure the peak flow of the Moreau River was coincident with the Missouri River peak through Jefferson City.

The maximum increase in Missouri River water surface profile through the study area adjacent to the Federal Interest Plan alignment was 0.3-feet (3.6 inches) in the 1% AEP Missouri River event coinciding with a 1% AEP Moreau River flow event. This difference does not justify a change in the top of levee elevation approach.

9 ADDITIONAL CONSIDERATIONS

9.1 Additional Berming at Riverside – Capital Sand

The feasibility of additional berming was incorporated in the model riverside to provide access to Capital Sand in high water events. Consideration of low points adjacent to Capital Sand access roads suggest a levee tieback could be a potential solution to maintain access to critical facilities during times of flood. There are two locations where shallow water flows are resulting in flooding that could be remedied with short berm lengths or filling of existing low points.

Initial evaluation was conducted for both a 10% AEP and 4% AEP Missouri River flow event. In both cases, the model suggests levee tiebacks to existing Capital Sand high ground at an elevation equivalent to the Missouri River WSE is feasible without major impacts to the Missouri River maximum WSE. However, some volume of excavation offset may be necessary to mitigate local changes in water surface elevation due to the placement of fill. The levee tiebacks are shown in green on **Figure 12**. Preliminary analysis did not include management of internal drainage for the Capital Sand property or needs for any auxiliary drainage structures through the levee embankment.

Figure 12. Capital Sand Potential Tieback Locations

The federal project does not include tying in high ground at the Capital Sand property to the proposed federal levee. An embankment modification would encroach on the federal project footprint and require a Section 408 permit through the USACE Kansas City office for review and approval of all requests to modify, alter, or occupy any existing U.S. Army Corps of Engineers civil works projects. A Section 408 review includes an engineering, environmental, real estate, and legal review to ensure the alteration does not impair the usefulness of the federal project and is not injurious to the public's interest. Engineer Circular 1165-2-216 provides the policies and procedural guidance that the Kansas City District follows in processing requests to alter or modify civil works projects that were constructed by the Corps. A separate, standalone evaluation for a Section 408 permit is required to confirm the feasibility-level assessment conducted herein and will be required to represent final plans to be constructed. The federal project does not guarantee Section 408 permit approval. All local and state permitting requirements also apply.

9.2 Existing Capital View Levee

The existing Capital View levee east of U.S. Highway 54/63 is not part of the federal project footprint. The project analysis assumes the levee will remain in a condition equivalent to the existing condition regarding top of levee elevation, location, standard of maintenance, and drainage structures.

The federal project does not include tying in the existing private levee to the proposed federal levee. A levee tie in would encroach on the federal project footprint and require a Section 408 permit through the USACE Kansas City office for review and approval of all requests to modify, alter, or occupy any existing U.S. Army Corps of Engineers civil works projects. A Section 408 review includes an engineering, environmental, real estate, and legal review to ensure the alteration does not impair the usefulness of the federal project and is not injurious to the public's interest. Engineer Circular 1165-2-216 provides the policies and procedural guidance that the Kansas City District follows in processing requests to alter or modify civil works projects that were constructed by the Corps. A separate, standalone evaluation for a Section 408 permit is required to confirm the feasibility-level assessment conducted herein and will be required to represent final plans to be constructed. The federal project does not guarantee Section 408 permit approval. All local and state permitting requirements also apply.

9.3 Comparison to Preliminary 2D FEMA Model through the Area

Although separate studies and use cases, the Jefferson City feasibility study model was compared to the draft, fully 2D FEMA Missouri River model through the study area to confirm consistency in river profile routing and validate assumptions utilized to determine the Federal Interest Plan top of levee elevations utilized for the cost estimate. Upon comparison of the FEMA model the Jefferson City study FWOP Missouri River 1% AEP results appeared consistent for the Missouri River profile and may be a reasonable resource for future modeling efforts. At the time of reporting, the model results are not directly comparable due to differences in project timelines.

10 CONSIDERATIONS FOR PRE-CONSTRUCTION ENGINEERING AND DESIGN (PED)

10.1 Hydrology

The hydrologic analysis for this study adopted flows from the most recently published Missouri River Flow Frequency Study (2023). Flows at the Jefferson City gage were interpreted based on this study and previous frequency studies on the river. Historic hydrographs were scaled to create a peak flow scenario representative of the target flow frequency.

Targeted technical ATR reviews, in addition to the overall project review schedule, were conducted to ensure the existing condition models correctly applied the flow frequency data.

For final design, validation against the published Missouri River Stage-Frequency values at the Jefferson City gage is anticipated. The stage frequency results are not yet available. Final published flows will be within the realm of those used for the feasibility study flow frequency.

Final design should include analysis of top of levee sensitivity to adopted hydrograph shapes to vary hydrograph volume. The top of levee elevation at this location is not tied to FEMA accreditation and is economically determined rather than flow frequency determined. Levee overtopping frequency changes do not affect the final design.

10.2 Hydraulics

The model is calibrated to the 2019 flood and validated using the 2018 in-bank highwater event. Approximately 100 miles are modeled to see the full resolution of any impacts between the FWOP and Federal Interest Plan. The left-bank (study area) is modeled as a 2D flow area and data was gathered to ensure existing conditions were modeled correctly, with most recent data. The remainder of the 1D model utilized upstream and downstream of the project site was adopted from the fully calibrated, stress tested, and reviewed CWMS model of the Missouri River.

The Federal Interest Plan was assigned a top elevation needed to pass a 590,000 cfs (1% AEP) flood event with managed overtopping enforced at the downstream end and superiority applied to the levee tiebacks, per USACE levee design guidance. This basis of design was determined by economic analysis and through coordination with the study sponsor through analysis of project goals.

The Flood Risk Management – PCX was consulted during development of existing conditions and definition of project alternatives. Targeted technical ATR reviews in addition to the overall product review schedule were conducted.

The final design will incorporate changes that have occurred since feasibility-level design including updated bathymetry, top of levee elevations, survey data, incorporation of proposed levee grading into the model, and any changes to the model geometry due to public works or private development projects within the study area. It is anticipated that the adopted Missouri River stage-frequency model will be available for final design for hydraulic model validation and comparison.

10.3 Interior Drainage

The 2D flow area internal to the Federal Interest Plan alignment was utilized to analyze internal drainage needs with the project and identify needed gate wells. Interior ponding was assessed by assuming unrestricted flow through the levee area. However, roadway culverts and other graded features would inhibit flow and detain water upstream of the gate wells in this developed location. Gatewell sizes and locations were determined with approximate elevations and confirmed by Civil designers to ensure existing ditches could be graded to connect.

Further data collection internal to the levee including survey of culverts and storm systems, would provide a more accurate depiction of internal drainage ponding and timing. However, these structures are likely to change prior to PED due to typical material service life of pipe culverts. A difference in these structures is unlikely to meaningfully impact the basis of design.

10.4 Additional Considerations

Separate modeling coordination with FEMA and remapping via the Conditional Letter of Map Revision (CLOMR) process is anticipated to be required during design and post-project. Refer to Appendix A1.3 sections 8.4 through 8.6 for additional details.

Due to the length of time between the Feasibility milestone and PED, updates are anticipated due to new data, site changes, or design manual updates. Further definition of minor project feature details will be needed for final design, beyond the level-of-detail required to reasonably form an estimate for feasibility-level design.

Permitting and federal agency coordination will need to occur again to ensure plan meets current applicable standards and laws.

ATTACHMENT A – HYDRAULIC MODELING MISSOURI RIVER PROFILE OUTPUTS

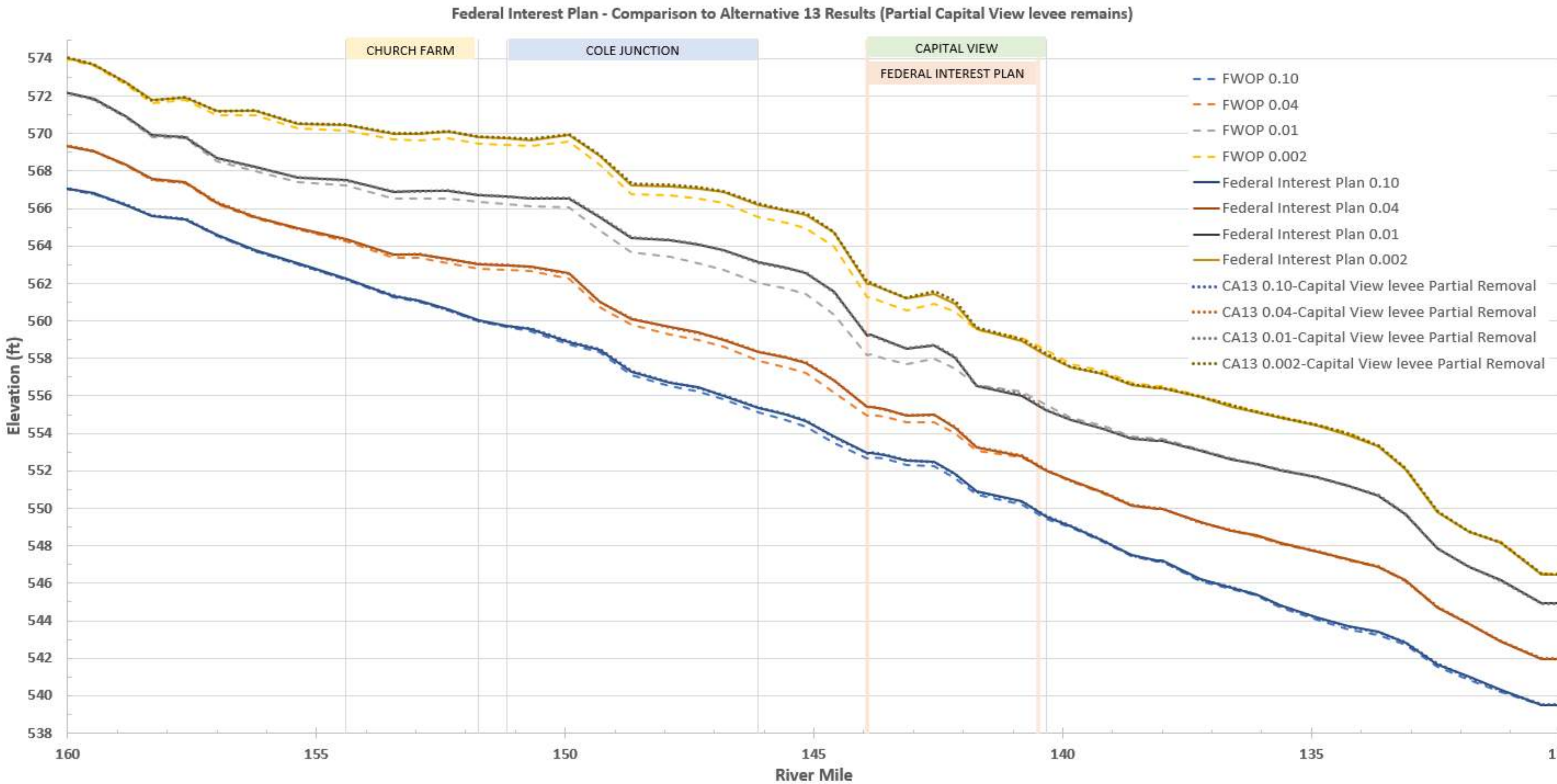

Figure 13. Federal Interest Plan Comparison to Combined Alternative 13 - Missouri River Maximum WSE Profile