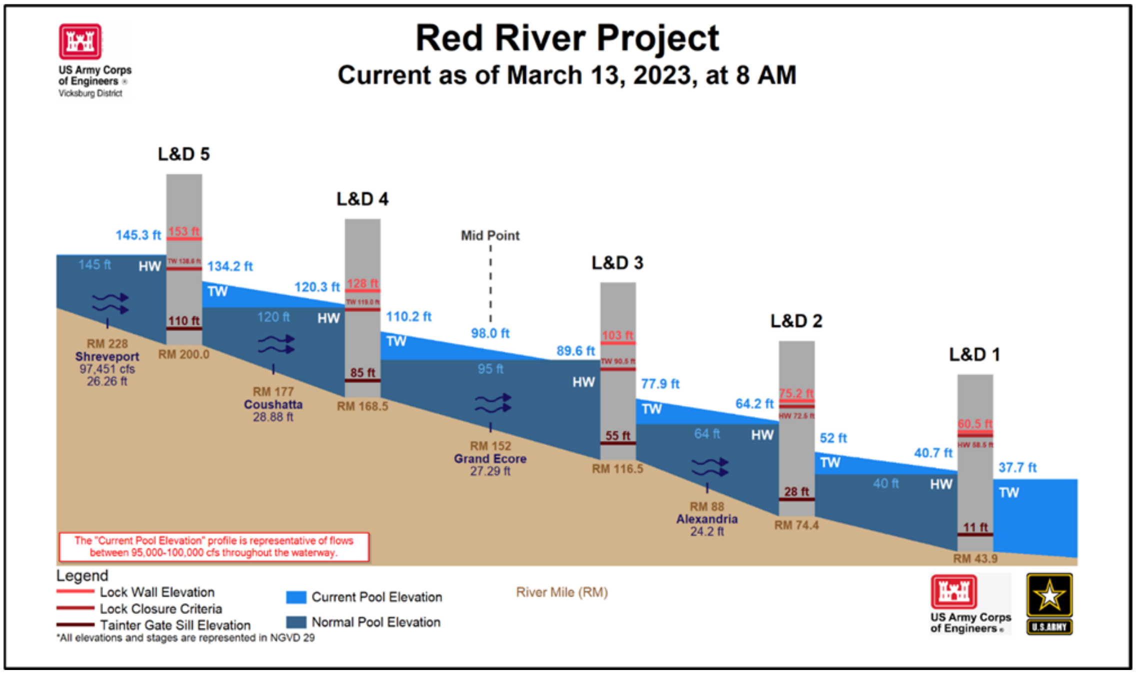
Mississippi Valley Division,
Regional Planning and Environment Division South

# J. Bennett Johnston Waterway 12-FT Channel




# Appendix A – Engineering

## August 2025

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# SECTION 1
# GENERAL OVERVIEW

## 1.1 PROJECT HISTORY

The J. Bennett Johnston (JBJ) Waterway, formerly referred to as the Red River Waterway Project, was authorized in 1968 with the primary purpose of providing a 9-FT deep by 200-FT wide navigation channel from the Mississippi River to Shreveport, Louisiana (Figure A-1). Lock and Dam 1 (L&D 1, also known as the Lindy C. Boggs Lock and Dam) located near Marksville, Louisiana, commenced operation in the fall of 1984. L&D 2 (John H. Overton) located downstream of Alexandria, Louisiana, became operational in the fall of 1987. L&D 3 located at Colfax, Louisiana, became operational in December 1991. L&D 4 (Russell B. Long) located near Coushatta, Louisiana, and L&D 5 (Joe D. Waggoner Jr.) located downstream of Shreveport, Louisiana, were constructed concurrently and became operational in December 1994. The JBJ Waterway extents are from the Mississippi River at the Old River Control Complex (ORCC) up to just north of Shreveport near the I-220 bridge or approximately River Mile (RM) 236. Since the waterway was opened in December 1994, a navigable channel has been maintained as far upstream as the Caddo-Bossier Port near RM 212. In addition to constructing and operating the five locks and dams, the U.S. Army Corps of Engineers (USACE) spent the last three decades of the 20$^{th}$ century developing the lower 280 miles of the Red River channel in Louisiana for commercial navigation. The development included an extensive channel improvement program that included channel realignments, bank stabilization works, and channel contraction.

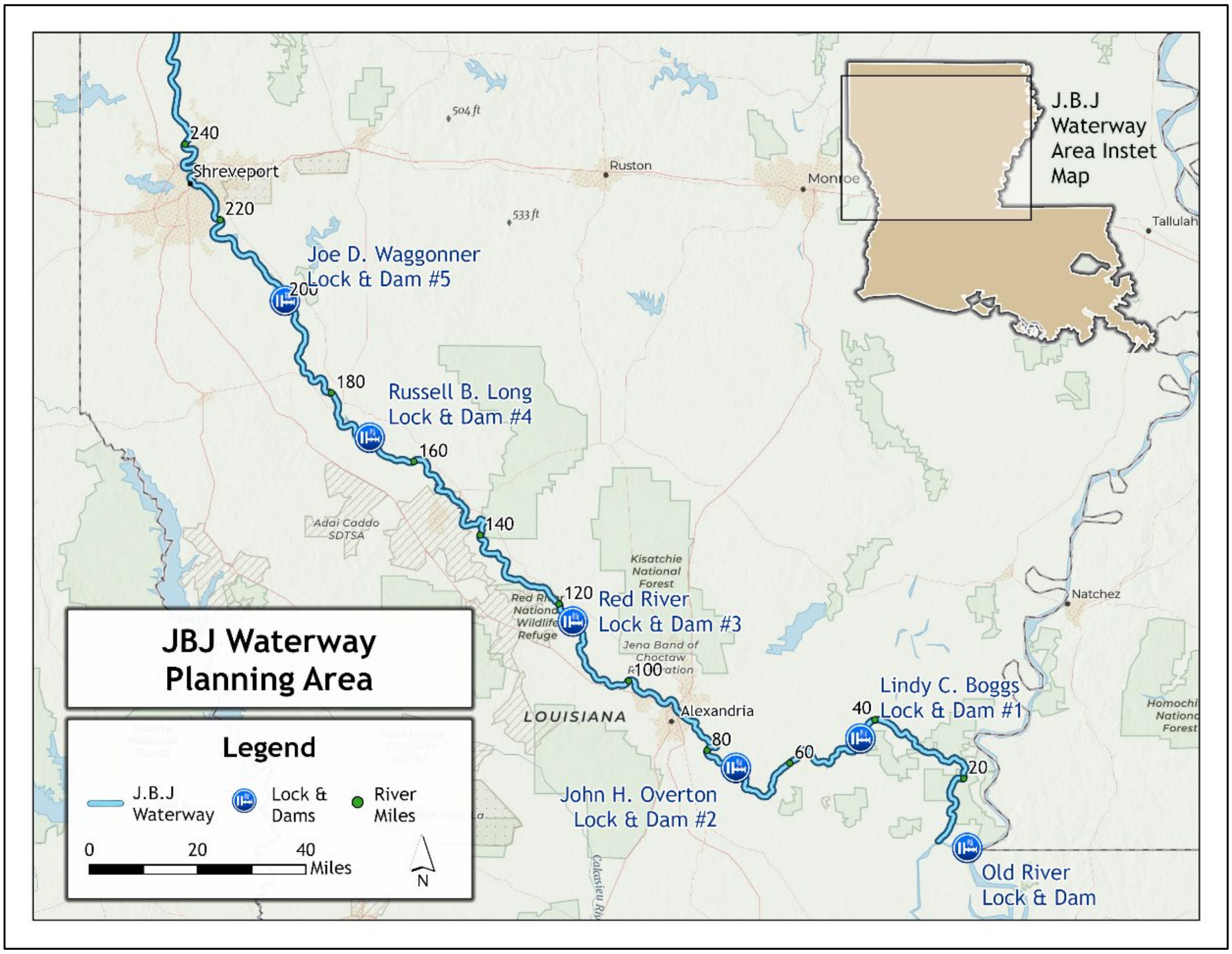


*Figure A-1. JBJ Waterway Map*

## 1.2 BACKGROUND

### 1.2.1 JBJ Waterway Overview

In 1968, the Red River Waterway Mississippi River to Shreveport Project, now referred to as the JBJ Waterway, was authorized to provide navigation from the Mississippi River below Natchez, Mississippi, to Shreveport, Louisiana. The JBJ Waterway navigation project consists of a 9-FT deep by 200-FT wide navigation channel that commences at the confluence of the Old River Outflow Channel and the Red River. The waterway proceeds upstream for approximately 236 miles to the Shreveport area. The project consists of a series of five lock and dams (see Figure A-1 and Figure A-2). L&D 1 near Marksville, Louisiana, began operation in 1984. L&D 2 began operation in 1987 and is located near Alexandria, Louisiana. L&D 3 is located at Colfax, Louisiana, and has been in operation since 1991. L&D 4 is located near Coushatta, Louisiana, and L&D 5 is located downstream of Shreveport. Both L&D 4 and L&D 5 began operation in late 1994.

In addition to construction of the locks and dams, realignment and stabilization of the banks of the Red River have also been vital in maintaining navigation. Dredging, cutoffs, and training works have been used for realigning the banks while revetments, dikes, and other structural methods have been used as stabilization measures. Thus far, 36 channel realignments (bendway cutoffs) have been constructed to shorten the length of the river by approximately 50 miles. Additional channel realignments were constructed between Shreveport, Louisiana, and Index, Arkansas, as part of a separate bank stabilization project. Realignments are used in place of revetments around long meander bends, as they are more economical to construct. Revetments placed on the Red River were constructed to achieve the desired alignment of the bank in comparison to the existing bankline and depth of the river. Dikes are placed at locations where it is necessary to limit channel width and prevent sedimentation in the navigation channel. Primary dike structures used on the Red River have consisted of kicker dikes and lateral contraction structures. Facilities that provide recreation and support fish and wildlife management opportunities are also integral to the waterway project.

Figure A-2 provides a river profile graphic of the JBJ Waterway project that is updated daily by Vicksburg District Water Management. Table A-1 summarizes the lock and dam operations for normal upper and lower pool elevations, open river conditions, and approximate contributing drainage area above each site as documented in the water control manuals.

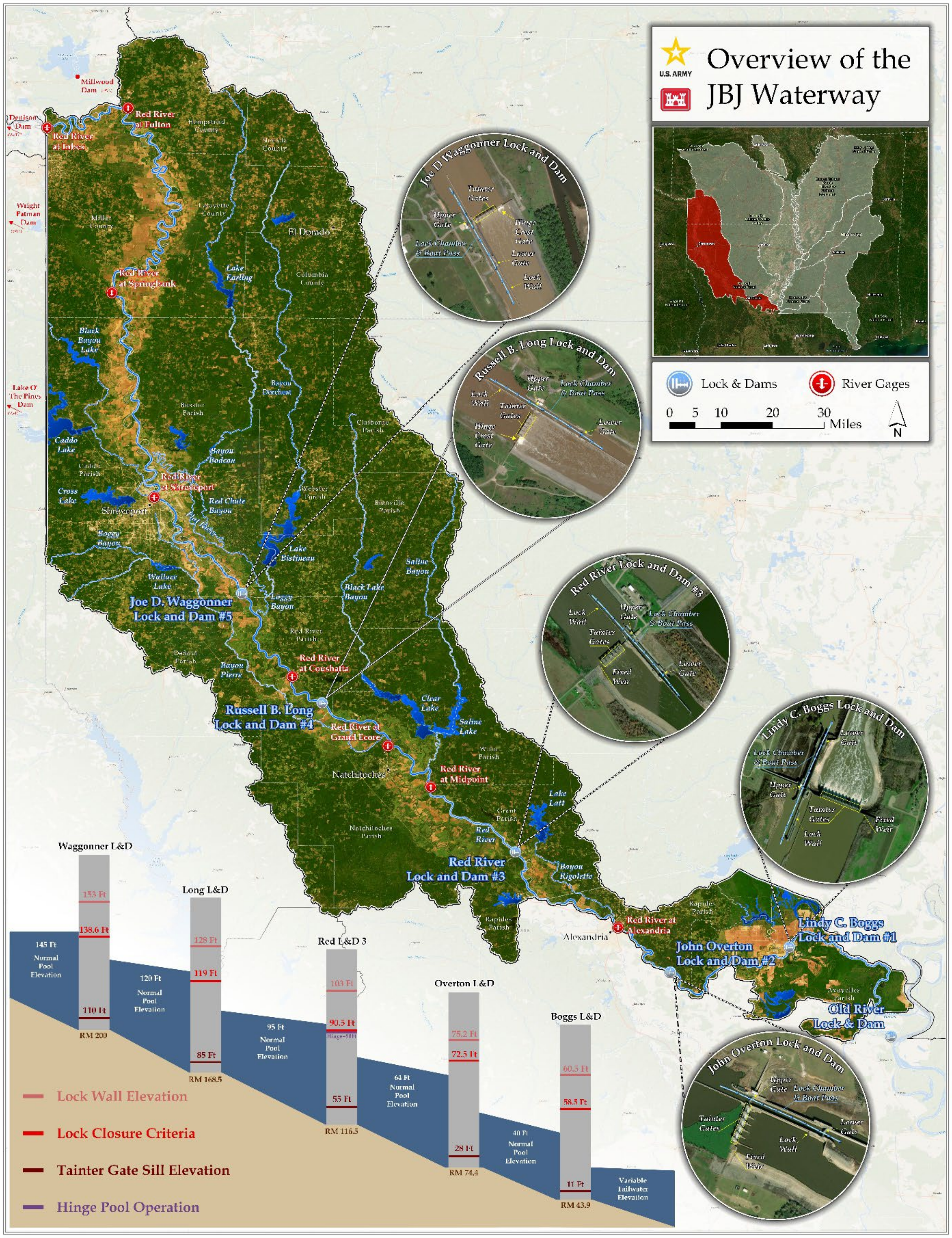


*Figure A-2. JBJ Waterway Project Location Map*

*Table A-1. Navigation Dam Operations Summary*

| Structure | River Mile | Normal Upper Pool (Elevation, Feet NAVD88) | Minimum Lower Pool (Elevation, Feet NAVD88) | Open River Conditions (Flow, cfs) | Approximate Drainage Area Above Lock and Dam (sq mi) |
|---|---|---|---|---|---|
| L&D 5 | 200.0 | 144.8 | 119.8 | 150,000 | 60,650 |
| L&D 4 | 168.5 | 119.8 | 94.8 | 120,000 | 63,650 |
| L&D 3 | 116.5 | 95.0 | 64.0 | 135,000 | 66,860 |
| L&D 2 | 74.4 | 64.1 | 40.1 | 125,000 | 67,458 |
| L&D 1 | 43.9 | 40.1 | 4.1 | 95,000 – 125,000* | 67,530 |

Open river flow conditions are sourced from the most recent Periodic Assessment Hydrologic Hazards (Chapter 4) reports.

- L&D 1 is heavily influenced by tailwater conditions and backwater from releases from the ORCC.

### 1.2.2 General Basin Description

The text in this section primarily comes from the Red River Basin Master Water Control Manual completed in March of 2022.

The Red River originates in the high plains of New Mexico, where it is locally known as Tierra Blanca Creek. In its upper reaches, it is little more than an arroyo. After its confluence with Palo Duro Creek, near Canyon, Texas, it becomes known locally as Prairie Dog Town Fork. From Canyon, Texas, the river flows generally eastward 496 miles across the Texas Panhandle and along the Oklahoma-Texas State boundary to Denison Dam. The river continues to flow eastward a distance of 263 miles along the Oklahoma–Texas and Arkansas–Texas State lines to Fulton, Arkansas. From this point, the river flows 455 miles south and southeast through southwest Arkansas and northwest Louisiana to Barbre Landing. At Barbre Landing, the Red River becomes the Atchafalaya River, which extends 140 miles southward to the Gulf of America. The Atchafalaya River is fed by flow from both the Red River and the Mississippi River. Flow from the Mississippi River is sent to the Atchafalaya River through a series of control structures (ORCC). The amount of flow sent from the Mississippi River to the Atchafalaya River is computed to maintain a desired distribution of flows between the two rivers, based on the total combined flows computed at a point downstream of the confluence of these river systems. The desired relationship is that the Mississippi should carry 70 percent of the combined flow of the two rivers, and the Atchafalaya 30 percent. Old River, through the Old River Lock and Dam, serves as a passage for riverine traffic between the Red and Mississippi Rivers.

The Red River drains an area of 92,600 square miles including parts of New Mexico, Texas, Oklahoma, Arkansas, and Louisiana. Of this drainage, 23,400 square miles are drained by the Ouachita–Black system, which is not part of the Red River system for operational purposes. The remaining 69,200 square miles of the Red River drainage is considered part of a single system for operational purposes. Since the Red River spans multiple States, several USACE districts assume responsibility for water management operations for reaches falling within their district boundary. Those districts include the Tulsa, Little Rock, Fort Worth, and Vicksburg Districts.

The portion of the Red River Basin managed by the Vicksburg District includes the Red River and its tributaries downstream of Index, Arkansas, excluding the tributaries managed by the Fort Worth District. Elevations in this part of the basin range from over 350 feet in the upper basins of tributaries to below 4 feet at the head of the Atchafalaya River. The basin can generally be characterized as having a broad alluvial valley that is approximately 5 to 10 miles wide. It is surrounded by rolling hill lands with intercepting tributaries that pass through narrow, wooded bottoms. Surrounding tributary bottoms and hill areas are largely wooded. Much of the land has been converted into agricultural production.

The major tributaries entering the Red River above Fulton, Arkansas, are the western tributaries (Salt Fork, North Fork, and Pease River), Cache Creek, Wichita River, Beaver Creek, Little Wichita River, Washita River, Blue River, Boggy Creek, Kiamichi River, and Little River. Major tributaries entering the Red River below Fulton are the Sulphur River, Twelve Mile Bayou, Loggy Bayou, Red Chute Bayou, Bayou Pierre, Saline Bayou, and Black River. The Red River Basin situated in the Vicksburg District generally encompasses the northwestern portion of the State of Louisiana. As mentioned, the river enters the floodplain of the Mississippi River below Alexandria, Louisiana.

Over the last 150 years, the Red River has undergone major anthropogenic and morphological changes, including the removal of the great Red River Raft, the construction of basin reservoirs including Denison Dam on the mainstem channel, the construction and operation of the JBJ Waterway, and in some areas, development inside areas protected by levees.

The Red River is often discolored due to the amount of sediment it carries, especially during high-flow periods. In the natural state before dams and other developments, the particulate matter was deposited along the floodplain or carried to the Mississippi River (before the 1830s) or the Atchafalaya River (after the 1830s). This natural process continues but has been altered to some degree by development within the basin. Furthermore, reservoirs tend to slow river flow and accelerate deposition in pools, and irregular releases for flood control, water supply, or power generation often have an erosive effect downstream.

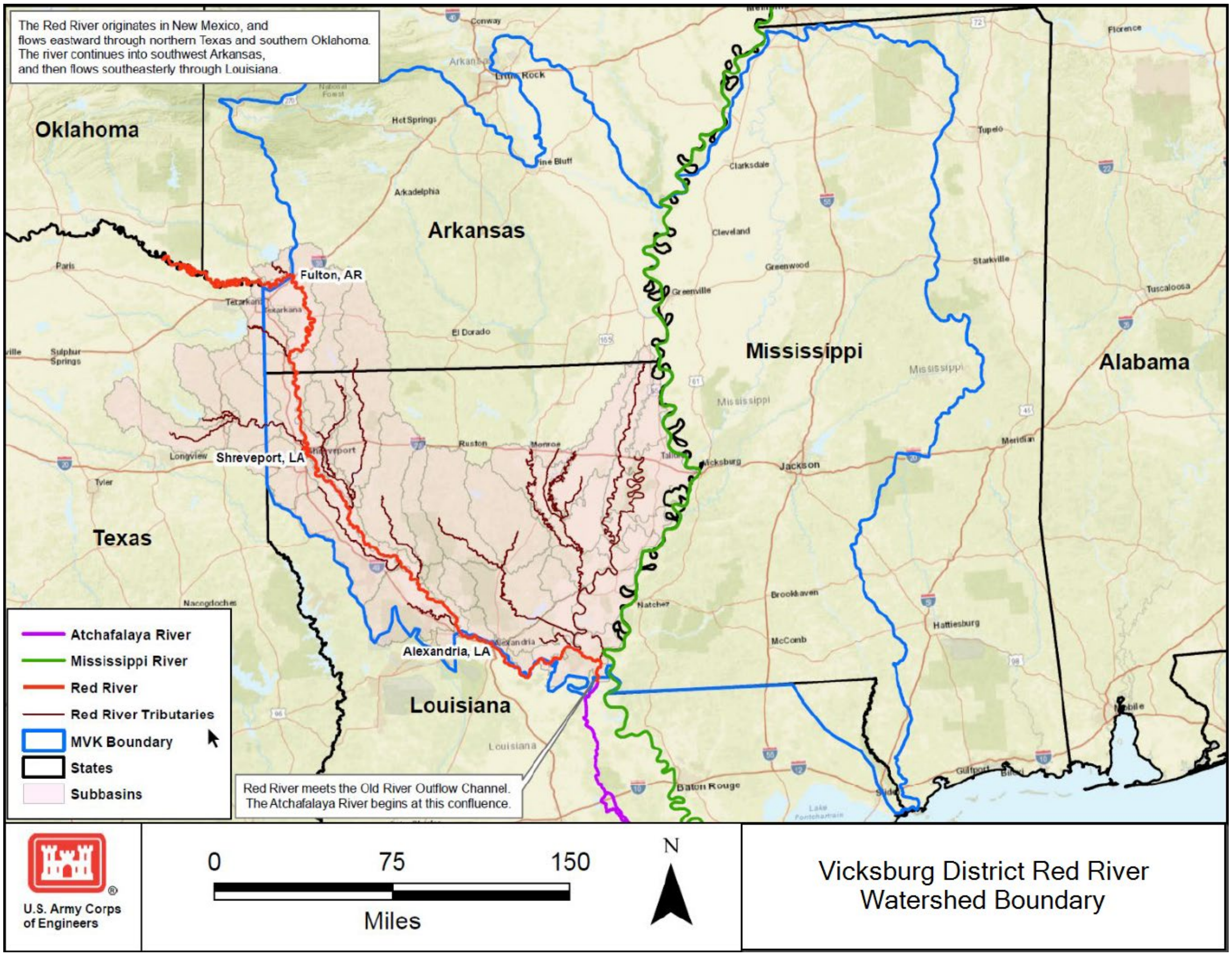


*Figure A-3. Vicksburg District Red River Watershed Boundary*

### 1.2.3 Existing Conditions

#### 1.2.3.1 Channel and Floodway

Near Index, Arkansas, the Red River enters the Vicksburg District. From Index, Arkansas, downstream to Shreveport, Louisiana, the riverbanks generally range from 1,000 to 1,500 feet apart, rising 20 to 40 feet above low-water lines, and have channel-controlling capacities of approximately 90,000 cubic feet per second (cfs) to 100,000 cfs. The riverbanks of the channel from Shreveport, Louisiana, to Alexandria, Louisiana, generally range from 500 to 1,000 feet apart, rising 15 to 20 feet above low-water lines, with channel-controlling capacities ranging from approximately 100,000 cfs to 120,000 cfs. The riverbanks below Alexandria generally range from 500 to 1,000 feet apart between increasingly more stable banks rising 30 to 60 feet above low-water lines and have channel-controlling capacities ranging from 120,000 cfs to 150,000 cfs. Generally, the river reaches near Shreveport have experienced aggradational trends while the river reaches near Alexandria have experienced degradational trends In the lowermost portion of the waterway downstream of Alexandria, the channel traverses the floodplain of the Mississippi River where extreme fluctuations in stages are experienced due to Mississippi River backwater through the ORCC. From Fulton, Arkansas, to the lowermost Red River south of Alexandria, Louisiana, there are a significant number of continuous and discontinuous levee systems that provide various levels of flood protection. The levees typically discontinue at high ground areas or tributary confluences. The major tributaries entering the Red River below Shreveport, Louisiana, are Twelve Mile Bayou, Bayou Bodcau, Cypress Bayou, Red Chute and Loggy Bayou, Bayou Pierre, and Saline Bayou. The Red River is confined by levees or high ground below Fulton, Arkansas, which has removed the river's connection to the natural floodplain. This confinement generally induces significant backwater flooding on intercepting tributaries including Little River (Arkansas), Sulphur River (Arkansas), Twelve Mile Bayou (Louisiana), Red Chute/Loggy Bayou (Louisiana), Bayou Pierre (Louisiana), and Saline and Black Bayous (Louisiana) during high flows.

#### 1.2.3.2 Channel Realignment

During the last three decades of the 20th century, USACE implemented a major channel realignment project on the Lower Red River to improve navigation conditions by eliminating meanders providing for a more desirable channel alignment and navigational channel lengths. The channel cutoff program effectively shortened the waterway portion of the Red River by approximately 50 miles. Table A-2 below provides waterway length measurements from before and after the channel cutoff program was implemented. The channel realignment information is sourced from Technical Report HL-88-15, 1972, and graphically correlated to more recent JBJ Waterway Navigation booklets along with Google Earth River Mile KMZ layers.

*Table A-2. JBJ Waterway Channel Realignment*

| Locations | Existing/Post Project River Mile (approx.) | Pre-Project 1967 River Mile (approx.) | Difference (Existing minus 1967 Pre-Project) | Difference from downstream location | % of total shortening |
|---|---|---|---|---|---|
| **Old River Lock** | 0 | 0 | 0 | 0 | 0% |
| **Lower Old River and Red River Confluence** | 7 | 7 | 0 | 0 | 0% |
| **Acme** | 34 | 34 | 0 | 0 | 0% |
| **Lock 1** | 44 | 50 | -6 | -6 | 12% |
| **Lock 2** | 74 | 88 | -14 | -8 | 15% |
| **Alexandria** | 88 | 105 | -17 | -3 | 6% |
| **Lock 3** | 116 | 153 | -37 | -20 | 38% |
| **Grand Ecore** | 152 | 185 | -33 | 4 | 8% |
| **Lock 4** | 168 | 205 | -37 | -4 | 8% |
| **Coushatta** | 177 | 220 | -43 | -6 | 12% |
| **Lock 5** | 200 | 245 | -45 | -2 | 4% |
| **Caddo Bossier Port** | 212 | 261 | -49 | -4 | 8% |
| **Shreveport** | 228 | 278 | -50 | -1 | 2% |

Notes: This table illustrates the shortening of the river channel between Pre- and Post-Project through the channel cutoff program. 1967 River Miles are sourced from the 1972 Stabilization and Cutoffs Design Memorandum No. 1 Portfolio, and correlated to existing River Miles using Google Earth KMZ's and JBJ Navigation Books. The River Miles are approximated and are not exact. River Miles are labled as miles above the Old River Control Structure Lock and Dam. Within the waterway reach of the Red River, 36 channel realignments were constructed that shortened the river by approximately 50 miles. The table shows that a large majority of the river shortening occured within Lock and Dam No. 1 and 2 pools.

#### 1.2.3.3 Water Management

Per the 2022 Red River Master Water Control Manual, there is not currently a system-wide plan for water control management. Projects in the basin operate individually or as part of smaller subsystems; however, because many projects share downstream control points, coordination does occur between districts. The reservoirs with controlled outlets influencing flows on the Red River are controlled by three other districts upstream of the JBJ Waterway. The reservoirs nearest the JBJ Waterway that are maintained by the Vicksburg District have uncontrolled spillways and uncontrolled outlets and therefore do not require or allow for any operation of flood control gates. The Vicksburg District does operate the series of lock and dam projects on the waterway to maintain navigation pools within +/- 0.5 feet of each project's designated pool levels. This is accomplished by adjusting the Tainter gates such that the pool level is maintained and the excess flow is passed downstream. When the upper pool elevation is higher than the designated pool level, all gates should be elevated sufficiently above the water to allow for the passage of drift. The frequency of gate adjustments is dependent on the rate of change of the flow in the river. Each project has a target normal pool to maintain. There is also a maximum elevation identified where lock operations are suspended.

Notably, the reservoir water management projects influencing flows of the JBJ Waterway do not have operations to augment or supplement low flows in the waterway, and the five locks and dams are not operated to provide flood control during large events but are operated regularly to maintain navigable elevations in the river and to provide open river conditions during large flows (Tainter gates fully open for flow in to equal flow out). L&Ds 4 and 5 do have hinge crest gate operations to provide water quality and fish and wildlife benefits during low-flow periods by increasing dissolved oxygen in the river downstream.

The volume and rate of discharge (flow) of the Red River fluctuate over a wide range. The volume and flows on the mainstem of the Red River have been regulated by Denison Dam (Lake Texoma) since 1944. Flows on the tributaries have been regulated by Wright Patman Dam (Sulphur River) since 1956, Lake O' the Pines Reservoir or Ferrells Bridge Dam (Big Cypress Bayou) since 1959, and Millwood Dam (Little River) since 1965. These projects contain outlet structures with uncontrolled spillways. In addition to the large reservoirs are the Caddo Lake Dam (uncontrolled overflow weir) and Cross Lake Dam (uncontrolled spillway) regulating flow on Twelve Mile Bayou, Wallace Lake Dam (uncontrolled spillway and uncontrolled outlet structure) regulating flow in Cypress Bayou and Bayou Pierre, and Bodcau Dam (uncontrolled outlet structure and uncontrolled spillway) and Lake Bistineau (uncontrolled outlet structures and uncontrol spillway) regulating flows on Red Chute Bayou and Loggy Bayou.

Table A-3 summarizes the major Red River Basin water management projects, and Figure A-4 shows the locations of these works. There are numerous reservoir projects in the Upper Red River Basin, so this is not a comprehensive list of all projects.

*Table A-3. Major Red River Basin Water Management Projects*

| Water Management Project | Operating Agency | Flood Control Type |
|---|---|---|
| Denison Dam (Lake Texoma) | USACE Tulsa District | Uncontrolled spillway with controlled outlet works |
| Hugo Lake Dam (Hugo Lake) | USACE Tulsa District | Uncontrolled spillway with controlled outlet works |
| Millwood Dam (Millwood Lake) | USACE Little Rock District | Uncontrolled spillway with controlled outlet works |
| Wright Patman Dam (Wright Patman Lake) | USACE Fort Worth District | Uncontrolled spillway with controlled outlet works |
| Ferrell's Bridge Dam (Lake O' the Pines Reservoir) | USACE Fort Worth District | Uncontrolled spillway with controlled outlet works |
| Caddo Lake Dam (Caddo Lake) | USACE Vicksburg District | Uncontrolled spillway |
| Bodcau Dam (Bodcau Lake) | USACE Vicksburg District | Uncontrolled spillway with uncontrolled outlet works |
| Lake Bistineau (Lake Bistineau) | State of Louisiana | Uncontrolled spillway with controlled outlet works |
| Wallace Lake Dam (Wallace Lake) | USACE Vicksburg District | Uncontrolled spillway with uncontrolled outlet works |

Additional water management projects located in the basin include the following:

- Black Bayou Lake Dam
- Clear Lake Dam
- Cross Lake Dam
- Cypress Bayou Reservoir Dam
- Lake Iatt Dam
- Larto-Saline Complex
- Nantachie Lake Dam
- Catahoula Lake Control Structure (not a major contributor to Red River flow)

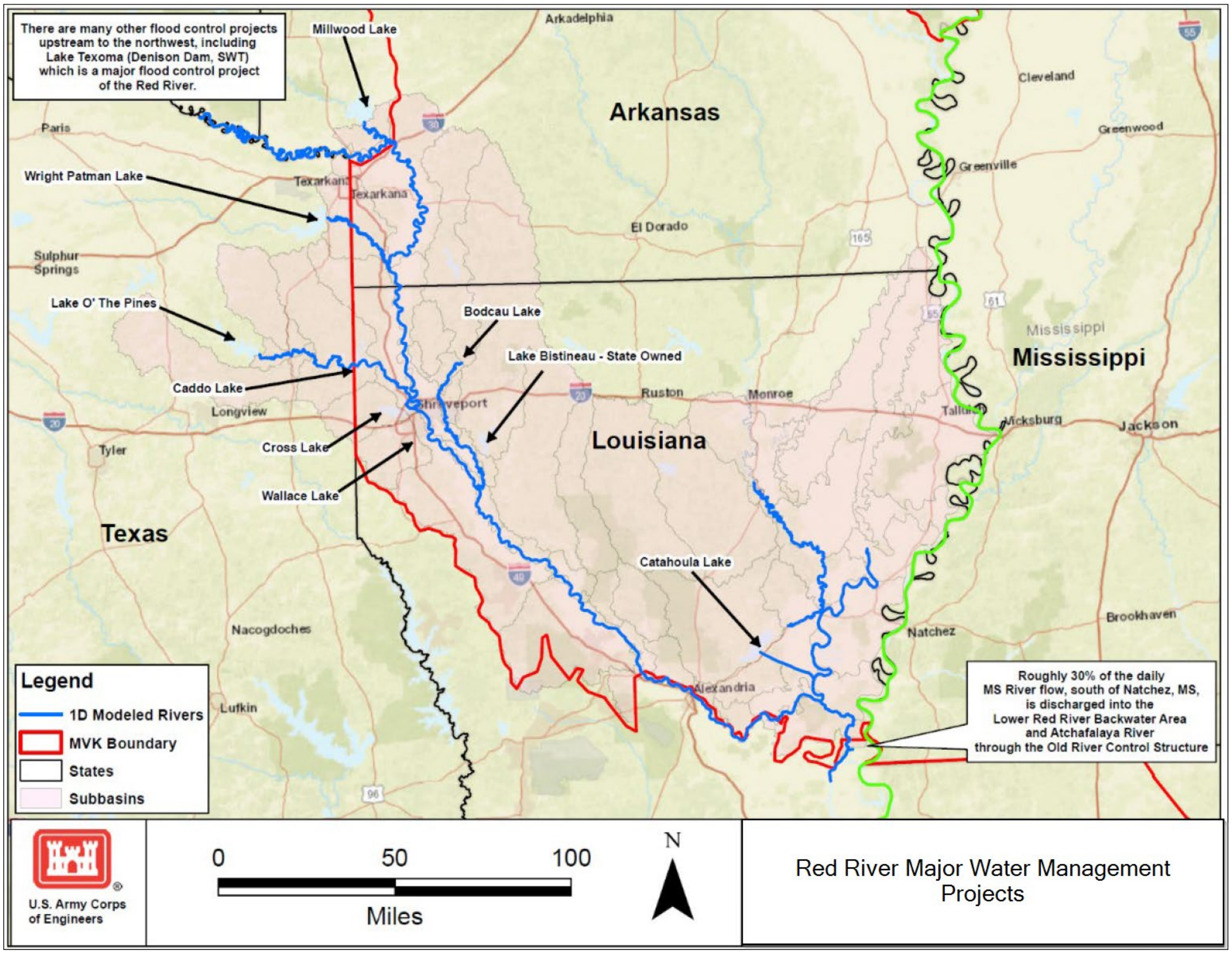


*Figure A-4. Major Water Management Projects along the Red River in the Vicinity of the Vicksburg District*

1.2.3.3.1 Locks and Dams

The following section provides a generalized operation plan along with pertinent information about the locks and dams. Greater detail about the lock and dam operations and other pertinent information can be found in the water control manuals and the most recent Periodic Assessment reports.

<u>L&D 1</u>

The generalized regulation plan for L&D 1 is to maintain a minimum pool level of 40.1 feet, NAVD88 with a +/- 0.5 foot operating range, providing a 9-FT navigation channel throughout the pool. During open river conditions, the dam will maintain sufficient gate openings to pass all inflows with minimum swellhead. Flow through L&D 1 can vary for a given headwater or tailwater stage because of the backwater effect from the Mississippi River flows through the ORCC.

The structure contains eleven 50-foot by 31-foot Tainter gates with a spillway gate sill elevation of 10.7 feet NAVD88 to control pool and pass flows. During the 2015 and 2016 floods, several of the Tainter gates were inoperable causing higher water surface elevations within the pool. USACE Vicksburg District is currently working to replace the gates. Replacement gates are expected to be installed by 2026. The size of the lock chamber is 84 feet by 705 feet, and it provides a maximum lift of 36 feet. This structure does not contain an uncontrolled overflow section. The normal operating lift varies due to the tailwater influence from the Mississippi River. The elevation of the upper miter gate sill is 17.7 feet NAVD88, with a lower miter gate sill elevation of approximately -9.6 feet NAVD88 based on recent 2025 surveys.

L&D 2

The generalized regulation plan for L&D 2 is to maintain a minimum pool level of 64.1 feet NAVD88 with a +/- 0.5 foot operating range to provide a 9-FT navigation channel throughout the pool. During open river conditions, sufficient gate openings will be made to pass all inflows with minimum swellhead.

The structure contains five 60-foot by 38-foot Tainter gates with a spillway crest elevation of 28.8 feet NAVD88 to control pool and pass flows. In addition, there is an overflow monolith from the gated spillway to the right bank with a crest elevation of 65.6 feet NAVD88. The size of the lock chamber is 84 feet by 705 feet providing a maximum lift of 31 feet. The upper miter gate sill elevation is 41.3 feet NAVD88, and the lower miter gate sill is elevation 25.8 feet NAVD88.

Vicksburg District and Engineer Research and Development Center (ERDC) Coastal and Hydraulics Laboratory (CHL) are currently conducting a study to assess the redesign of the lower approach channel to L&D 2. The physical layout of locks and dams can sometimes result in eddies and slack water zones. For decades, the lower approach has proven to be a challenge for pilots to navigate as complex flow patterns occur due to the physical layout of the structural features combined with the river currents and morphology.

L&D 3

The generalized regulation plan for L&D 3 is to maintain a pool between elevations 88.0 and 95.0 feet NAVD88 with a +/- 0.5 foot operating range, utilizing a hinge pool operation to maintain navigability of the Red River through Pool 3 to L&D 4 without adversely affecting adjacent lands. During open river conditions, the bottom of the gates should be raised above the water surface to pass all inflows with minimum swellhead. The hinge pool operation for Pool 3 was established in Design Memorandum No. 3-Revised Hydrology Red River Waterway La., Tex., Ark., and Okla., Mississippi River to Shreveport, La., February 1980 (not digitized for digital reference, hard copy only). The hinge operation is intended to keep the post-project profile below the higher of the ordinary high-water line, the pre-project profile, and elevation 98 feet NAVD88 to reduce real estate requirements.

The structure contains six 60-foot by 42-foot Tainter gates with a spillway crest elevation of 55.0 feet NAVD88 to control pool and pass flows. There is also an overflow weir at elevation 97.0 feet NAVD88. The size of the lock chamber is 84 feet by 705 feet, providing a maximum lift of 31 feet. The upper sill elevation is 70.0 feet NAVD88, and the lower sill elevation is 46.0 feet NAVD88.

L&D 4

The generalized regulation plan for L&D 4 includes maintaining a minimum navigation pool upstream at an elevation of 119.8 feet NAVD88 with a +/- 0.5 foot operating range. During periods of extreme low flow, to enhance dissolved oxygen levels in the downstream channel, the Tainter gates will be closed, and the hinged crest gate will be used to regulate flow and maintain the pool. When the hinged crest gate is in the fully open position, the Tainter gates will be used to pass Inflow and regulate the pool. The hinged crest gate will remain fully open when the Tainter gates are being used to pass flow and during open river conditions.

There are five 60-foot by 37-foot spillway Tainter gates at a crest elevation of 84.8 feet NAVD88 to control pool and pass flows. The hinged crest gated spillway is one 100 feet by 7-foot gate at a crest elevation of 112.8 feet NAVD88. There is also a cutoff wall between the lock chamber and Tainter gated section at an elevation of 127.8 feet NAVD88 and an overflow weir between the hinged crest gated section to the right descending bank at an elevation of 121.8 feet NAVD88. The lock chamber is 84 feet by 705 feet and provides a maximum lift of 25.0 feet NAVD88. The elevation of the upper miter gate sill is 94.8 feet NAVD88, and that for the lower miter gate sill is 76.8 feet NAVD88.

L&D 5

The generalized regulation plan for L&D 5 Includes maintaining a minimum navigation pool upstream at an elevation of 144.8 feet NAVD88 with a +/- 0.5 foot operating range. During periods of extreme low flow, to enhance dissolved oxygen levels in the downstream channel, the Tainter gates will be closed, and the hinged crest gate will be used to regulate flow and maintain the pool. When the hinged crest gate is in the fully open (lowered) position, the Tainter gates will be used to pass Inflow and regulate the pool. The hinged crest gate will remain fully open when the Tainter gates are being used to pass flow and during open river conditions.

There are five 60-foot by 37-foot spillway Tainter gates at a crest elevation of 109.8 feet NAVD88 to control pool and pass flows. The hinged crest gated spillway is 100 feet by 7 feet with one gate. There is an overflow weir section between the hinge crest portion and the left descending bank with a crest of 146.8 feet NAVD88. There is a cutoff wall between the lock and Tainter gate portion with a crest elevation of 152.8 feet NAVD88. The lock chamber dimensions are 84 feet by 705 feet, providing a maximum lift of 25 feet. The upper miter gate sill elevation is 119.8 feet NAVD88, and the lower miter gate sill elevation is 101.8 feet NAVD88.

#### 1.2.3.4 General Sediment Conditions

The Red River is a heavily sediment-laden alluvial river with one of the highest sediment concentrations of all major navigable rivers within the U.S. (Pinkard, 2001). The Red River is often discolored due to the amount of sediment it carries, especially during high-flow periods. In the natural state before dams and other developments, the particulate matter was deposited along the floodplain or carried to the Mississippi River (before the 1830s) or the Atchafalaya River (after the 1830s). This natural process continues but is altered to some degree by development within the basin. Furthermore, reservoirs tend to slow river flow and accelerate deposition in pools and irregular releases for flood control, water supply, or power generation often have an erosive effect downstream. Aside from the knowledge obtained from site-specific dredging for sand mining, maintenance dredging for the 9-FT channel in the river and at the locks and dams, and observing changes in stage–discharge relationships overtime, there has been no gage-specific monitoring of sediment transport on the Red River in decades. It has been observed that sediment frequently accumulates in slack water zones and around structural features. Sediment dynamics were a primary consideration in the design of the navigation structures along the waterway.

Downstream of Shreveport, the Red River basin is heavily agricultural and developed which contributes to erosion and sediment. The basin also contains woody wetlands and low-lying flooded areas, which can intercept sediment before it reaches the main channel. A large amount of sediment is transported in the mainstem from the upper reaches of the basin particularly from the unrevetted portions of the river between Index, Arkansas, and Shreveport, Louisiana (Pinkard 2001). In addition, the physical layout of locks and dams can often result in eddies and slack water areas often acting as sediment deposition zones. This high sediment load and the layout of the lock and dams are the major contributing factors to the dredging that is required to maintain navigation on the JBJ Waterway. Additional mitigation structures have also been constructed at or near the lock and dam structures to attempt to prevent and reduce sedimentation at the lock and dams.

The process of dredging, or removing sediment from the river bed, has influence on riverine sediment conditions. USACE dredging practices for maintaining the navigation channel have been summarized in Section 1.2.3.7. Additional dredging occurs on the Red River by private entities for sand mining. There are multiple permitted sand mining operations on the Red River below Fulton, Arkansas. These activities are expected to continue for the foreseeable future. The impacts to geomorphology and the sediment budget of the river system is unknown.

#### 1.2.3.5 Climate

The following information related to climate is sourced from the Red River Basin Master Water Control Manual completed in 2022. A climate variability assessment characterizing the potential climate variability impacts to inland hydrology (Engineering and Construction Bulletin No. 2018-14) can be found in Section 8.

The climate in the Red River Basin is generally mild, with long hot summers and short moderate winters, except in the western portion of the basin where winters are more severe. The climate varies gradually from semiarid in the extreme western parts of the basin to humid in the eastern portion. In the western portion of the basin, weather patterns are under continental controls characteristic to the Great Plains region, which produces pronounced daily and seasonal temperature changes and considerable variation in seasonal and annual precipitation. Sudden changes in temperature due to frontal systems moving in and out of the area are common throughout most of the year, except during summer months when cold fronts seldom reach far enough south to noticeably affect the regional weather. The area lies close enough to the Gulf of America to be affected by tropical disturbances and is subject to intense local rainfall. During the spring and fall seasons, cool fronts move into the area quickly and mix with the warm moist air from the Gulf of America to form thunderstorms and tornadoes. During the winter months, Arctic cold fronts move into the area causing the temperature to drop as much as 45 to 50 degrees Fahrenheit within a few hours and sometimes affect the weather for several weeks. The western portion of the basin is located in a semiarid region where wind movements are generally extreme, and the evaporation is high. In the central and eastern portion of the watershed, precipitation is usually adequate for agricultural purposes, and wind movements and evaporation are moderate. The zero index line of moisture deficiency–surplus is approximately aligned with the 97th Meridian, which runs north–south through the basin at the approximate upstream limits of Lake Texoma. This line separates areas with moist climates from those with dry climates. Significant amounts of precipitation occur in all seasons in most areas east of the 97$^{th}$ Meridian. Winter rainfall (and sometimes snowfall) is associated with large storms steering from west to east. Most summer rainfall occurs during thunderstorms and an occasional tropical storm or hurricane.

Extreme temperatures vary from over 120 °F to values below zero in the western portion of the basin where most of the extreme temperatures are experienced, though extreme temperatures can be experienced throughout the basin. Severe cold weather rarely lasts longer than a few days. Per the 2022 Red River Master Manual, average monthly maximum temperatures in the Vicksburg District portion of the basin generally range from 85 to 95 °F in the summer months and 55 to 65 °F in the winter months. The average monthly minimum temperatures generally range from 60 to 70 °F in the summer months and 35 to 45 °F in the winter months.

Rainfall distribution in the western portion of the basin is highly erratic. Drought periods of varying lengths interspersed with short violent storm periods are characteristic, particularly during the growing season. Further east, in the Ouachita Mountains portion of the basin, rainfall is normally abundant and usually occurs in the form of high intensity, local thunderstorms usually in the late spring and early fall. These storms are frequently accompanied by high winds, hail, and occasional tornadoes. The winter rains are generally of several days' duration and are more extensive in areal distribution. Periods of intense drought have also occurred in the general area. As the river moves east through the basin, rainfall is more evenly distributed and uniform during the year. In the eastern portion of the basin south of the Ouachita Mountains, the driest season of the year generally occurs from

August to October, with the wettest season occurring from January to May. Snowfall does occur throughout the basin but has not been a contributing factor to major floods. Average annual precipitation for regions of the Red River basin, per the 2022 Red River Master Manual, are 27.62 inches west of the 97$^{th}$ Meridian, 47.19 inches east of the 97$^{th}$ Meridian, 53.95 inches in Little Rock District, 47.78 inches in Fort Worth District, and 54.51 inches in Vicksburg District.

#### 1.2.3.6 River Training Structure Conditions

River training structures are manmade structures designed and constructed in a river reach to modify hydraulic flow and sediment transport. Some examples of this type of work include dikes, bendway weirs, and bank paving. These engineered structures help to mitigate the cost and impact of dredging as well as ensure safe and sustainable alignment of navigation channels. River training structures have a design life of 50 years. Many of the structures along the Red River were constructed during the 1970s and 1980s, and deterioration of revetments has occurred throughout the waterway. Limited maintenance funding has led to a backlog of repairs and improvements. In addition to the expected degradation, excess damages have occurred due to various environmental factors such as prolonged periods of high water, barge impacts, and natural channel migration. Some revetments within the system were also never built to the design grade, as a planned channel cap-out program was never fully implemented.

Many of the degradational issues identified in the river have occurred within the most downstream portion of the river known as the “Gauntlet.” This region runs from RM 42 to RM 33 and is between L&D 1 (RM 43) and the Mississippi River ORCC (RM 0). This reach is within the greater Lower Red River Backwater Area and thus sees variations in flow velocities due to the backwater effects of the Mississippi River. This leads to greater wear on structures and general channel instability. As of 2000, the Red River Channel Improvement Data Report showed that Pool 2 closely followed the Gauntlet in maintenance work. This was followed by Pools 3, 5, and 4; the last of which had repairs only at one location. While the majority of the channel can accommodate a 12-FT draft in its current condition, there are 11 reaches throughout the channel that struggle to consistently provide this depth. High-priority reaches currently struggling to maintain 9-FT of depth are located at RMs 192–191 (Westdale), RMs 34–38, and RMs 38–42, with the latter two reaches being located within the Gauntlet. Reaches that currently have difficulty providing 12-FT are located at RM 194 (Williams), RM 158 (Campti), RM 154 (Socot), RM 64, RM 61, and RM 52. Locations that would likely struggle to consistently provide 12-FT are low priority areas at RM 108 and RMs 163–165. These locations are further discussed within Section 4.4.2, which includes depth maps indicating depositional trends in these reaches.

Vicksburg District’s River Stabilization Section typically conducts annual inspections of the approximately 41 dike fields, 10 realignments, and 132 revetments within the JBJ Waterway. The current condition of these structures is based on the most recent inspection that was conducted in August 2023. The following tables (Table A-4 through Table A-9) include descriptions of conditions or possible repairs based on damages to existing dikes throughout the system. Revetments not listed in the tables are considered undamaged based on these inspections and the existing Red River hydraulic model (further described in Section 4).

Most of the repairs listed in the following tables are located in sections of the river that currently maintain a 9-FT navigable channel depth. Therefore, no additional work is needed in these areas to support a 12-FT channel. The only exceptions are the Joffrion (RM 37.5) and Westdale (RM 191.0) Revetments, which are frequently dredged to maintain the existing authorized depth and require additional repairs for a 12-FT channel. While there is some risk that other structures may experience more rapid deterioration than expected, for the purposes of this project, they were assumed to remain functional in maintaining river depth throughout the 50-year project life.

*Table A-4. River Training Structure Conditions – Pool 5*

| Revetment/Dike Name | River Mile | Construction or Maintenance | Description of Condition/Repair Linear Feet (LF) |
|---|---|---|---|
| Elm Grove | 205.5-L | M | Trail dike eroded (2,000 LF) |
| Morameal | 208.7-L | M | Trail degraded (1,800 LF) |
| Cupples Landing | 211.0-R | M | Kicker degraded (1,200 LF) |
| Wilkerson Point | 213-L | M | Trail dike is missing and land mass behind it washed away in flood |
| Sunny Point | 217.3-R | C | Possible connection of revetments (1,000 LF) |
| Curtis | 219.5-L | M | Trail dike eroded (2,500 LF) |
| Eagle Bend | 222-L | M | Kicker dike eroded (1,500 LF) |
| Douglas Island | 229.3-R | M | Trail dike eroded around timber piling. Encroaching on Clyde Fant Memorial Parkway (7,920 LF) |
| Honore Bend | 231.0-L | M | Revetment repair (1,200 LF) |
| Twelve Mile Bayou | 234.5-R | M | 5 dikes degraded on stream end (200 LF each) |

*Table A-5. River Training Structure Conditions – Pool 4*

| Revetment/Dike Name | River Mile | Construction or Maintenance | Description of Condition/Repair Linear Feet (LF) |
|---|---|---|---|
| Bull | 172.2-L | C | Trail dike gone & 3 dikes degraded (1,000 LF each) |
| Hanna | 174.0-R | M | 800 LF of trail dike is missing |
| Coushatta | 178.8-L | C | Possible connection of Coushatta (700 LF) to boat ramp |
| Gahagan | 181.0-R | M | Repair kicker and tieback (3,200 LF) |
| Carrol | 185.2-L | C | Possible 3,000 LF extension downstream to prevent flanking from bankline erosion |
| Abington | 188.5-R | M | 800 LF of kicker dike tip gone |
| Abington | 189.8-R | M | Bankline erosion threatening structure (1,200 LF) |
| Westdale | 190.5-R | M | Dikes are degraded (3,000 LF) |
| Westdale | 191.5-L | M | Dikes are degraded (5,000 LF) |
| East Point Revetment | 193.2-L | M | 500 LF gap in revetment |
| East Point Revetment | 193.9-L | M | 700 LF gap in revetment |
| East Point Dikes | 194-R | M | 400 LF gap in first dike |
| Williams | 196.3-R | M | Longitudinal dike (7,000 LF) and tiebacks severely degraded (1,200 LF) |

*Table A-6. River Training Structure Conditions – Pool 3*

| Revetment/Dike Name | River Mile | Construction or Maintenance | Description of Condition/Repair Linear Feet (LF) |
|---|---|---|---|
| Grappe Revetment and Dikes | 118.5-R | M | Three transverse dikes low on RDB (3,000 LF per dike) & Upstream RDB trail dike is missing and/or low (1,500 LF) |
| Grappe Revetment | 119.5-L | M | Trail dike low between two islands (2,200 LF) |
| Grappe Revetment | 120.0-R | M | Trail dike low between two islands (2,200 LF) |
| Eureka | 127.0-L | C | Degradation between sections of revetment requiring 1,400 LF connection |
| Dunn Lake | 137.5-L | M | End of trail dike is missing (1,400 LF) |
| Cadoche | 139.5-R | M | Two holes in the revetment (2,100 LF) |
| St. Maurice | 140.2-L | M | Kicker dike is degraded (2,000 LF) |
| St. Maurice | 142.0-L | M | Hole in revetment (700 LF) |
| Cadney | 143.2-R | M | Kicker dike is degraded 1,000 LF |
| Cadney | 143.2-R | M | Revetment is flanked (1,500 LF) |
| Poisson Dikes | 145.0-R | M | Trail dikes are low (600 LF) |
| Clarence | 148.5-R | M | Kicker dike is degraded 1,500 LF |
| Ile Ave Vaches | 151.8-R | M | Trail dike low (500 LF) |
| Campti | 158.0-L | M | Trail dike is degraded |
| Powhatan | 162.5-R | M | Short gap in right descending bank (1,800 LF) |
| Crain | 166.5-L | M | Kicker dike eroded 1,000 LF |
| Crain | 166.6-L | M | Gap in the end of the dike (600 LF) |

*Table A-7. River Training Structure Conditions – Pool 2*

| Revetment/Dike Name | River Mile | Construction or Maintenance | Description of Condition/Repair Linear Feet (LF) |
|---|---|---|---|
| **Colfax Revetment** | 114.5-L | M | Extend upstream 300 LF |
| **Colfax Dike** | 112.4-L | M | Dike degradation requiring 300 LF of stone to repair 7 dikes |
| **Raven Camp** | 112.3-L | M | Kicker dike flanked, requires 600 LF of stone to repair and protect |
| **Deloges Bluff Dikes** | 111.4-L | M | Dike 2 (150 LF) and Dike 3 (250 LF) are low |
| **Deloges Bluff Revetment** | 111.0-R | M | Low trail dike, requires 400 LF of stone to repair |
| **Kateland** | 109.2-L | M | Small blowout and timber piles exposed (300 LF) & tieback flanked requiring 400 LF stone to repair and protect |
| **Darrow Revetment** | 104.7-L | C | Upstream extension 1,600 LF |
| **Darrow Revetment** | 104.5-L | M | 500 LF of revetment is gone |
| **Bertrand Dikes** | 103.2-R | M | Dikes degraded from original alignment requiring 300 LF stone to fix six dikes |
| **Meade Revetment** | 102.4-L | M | 1200 LF of trail dike is gone |
| **Meade Revetment** | 101.6-L | M | Kicker tip is gone |
| **Marteau Revetment** | 99.7-L | M | 200 LF gap in trail dike |
| **Cotton** | 96.0-L | M | Kicker dike eroded 1,200 LF |
| **Philip Bayou** | 90.8-L | M | 1,200 LF of trail dike is low |
| **Maria Cutoff** | 86.0-L | M | Trail dike eroded 1,800 LF |
| **Hudson Cutoff** | 82.7-R | M | 1,500 LF of trail dike degraded |
| **Grand Bend Revetment** | 80.7-L | C | 2,400 LF downstream end extension |
| **Lick Revetment** | 76.2-L | C | 1,700 LF downstream extension |

*Table A-8. River Training Structure Conditions – Pool 1*

| Revetment/Dike Name | River Mile | Construction or Maintenance | Description of Condition/Repair Linear Feet (LF) |
|---|---|---|---|
| St. Agnes Revetment | 46.4-R | C | 1,800 LF upstream extension |
| Saline Revetment | 49.2-L | C | 1,000 LF upstream extension |
| Derussy Revetment | 50.6-L | M | Kicker and tieback low; 1,400 LF of stone required |
| Hadden Fort Revetment and Dikes | 52.5-L | M | Trail Dike eroded 1,800 LF & 3 Dikes eroded 500 LF |
| Barbin | 53.4-R | M | Kicker gone 800 LF |
| Vick Downstream Extension | 55.0-L | M | Kicker Dike eroded 1,500 LF |
| Ben Routh | 58.6-R | M | Small blowout |
| Moncla | 59.5-L | M | Trail Dike eroded 5,500 LF |
| Dupre | 60.2-R | M | Kicker Dike and tieback eroded 2200 LF |
| Choctaw Bayou Bend Cutoff | 62.2-L | M | Repair revetment 3,000 LF |
| Lower Gin Lake | 64.9-L | C | 1,000 LF downstream end extension |
| Lower Gin Lake | 65.2-L | M | 2 scour pockets totaling 500 LF |
| Bijou Revetment | 65.6-R | M | 250 LF scour pocket |
| Once More Realignment | 68.0-L | M | Reconstruct lower 3,000 LF of revetment |
| Hog Lake Dikes | 72.5-R | M | 10 dikes are degraded requiring 200 LF stone each to repair |

*Table A-9. River Training Structure Conditions – The Gauntlet*

| Revetment/Dike Name | River Mile | Construction or Maintenance | Description of Condition/Repair Linear Feet (LF) |
|---|---|---|---|
| **Blakewood Lake Revetment** | 12.1-R | M | Extend downstream 2,000 LF |
| **Pump Bayou Revetment** | 15.4-R | M | 250 LF scour pocket |
| **Pump Bayou Revetment** | 15.6-R | M | 750 LF scour pocket |
| **Bayou Cocodrie Dikes** | 21.1-L | M | Most downstream dike is gone |
| **Grand Lake Revetment** | 25.0-R | M | Bank encroaching on roadway |
| **Bonnie Revetment** | 29.2-R | M | Trenchfill exposed |
| **Six Mile Bayou Dikes** | 29.5-R | C | Dikes are completely gone |
| **Mouliere Revetment** | 31.5-R | M | Scour hole |
| **Joffrion** | 37.5-L | C | Two trail dikes degraded 2800 LF |

#### 1.2.3.7 Existing Conditions Dredging

Dredging is the process of removing sediment from the bottom of the river within the navigation channel and placing it elsewhere within deeper parts of the river channel or outside of the river channel. USACE Vicksburg uses in-channel displacement on the Red River. The contracted dredge team coordinates with the USACE Vicksburg survey team to locate nearby areas of channel that have swift waters, usually coinciding with the deeper areas. The contractor then anchors the dredge–discharge at this location, and the survey team monitors the depths during the dredging operations. There are many different types of dredges as well as many mitigation strategies to lessen the amount of dredging needed or to provide utility of the dredge–disposal material. Neither sedimentation nor the physical act of dredging are exact sciences or procedures. Hydrographic survey data of the river bottom are essential to estimating quantities, but there are a variety of influential factors that impact dredging trends such as flow conditions, funding levels, district priorities, changes in the sediment load, and changes to the riverbed.

<u>Dredge Depths</u>

The navigation depths portion of the study is primarily focused on the existing river training structures (dikes and revetments) and dredging related to channel dredging. The study is not focused on the dredging that occurs at locks and dams. Historic channel dredge records provide insight into the river reaches that require occasional or annual dredging for the 9-FT channel. In some cases, the adjacent river training structures have deteriorated such that the structures are no longer performing as intended. In those cases, the dredge data are used as a starting point to identify historically known problem areas for the 9-FT channel that would therefore remain as problems for the 12-FT channel. This section is not meant to be an exhaustive overview of historical JBJ Waterway O&M dredging practices but rather a supporting tool for analysis of the river training structures while understanding many variables influence dredging trends. Due to the somewhat flashy nature of a relatively large river with a high sediment load, and lock and dam tapering of flows, a problem area could be dredged; however, if a high-water event occurs after that dredge, the area may just shoal again. To a degree, localized challenges may remain even with sound river training practices.

Under current dredge authorizations, the Vicksburg District River Operations branch primarily maintains the JBJ Waterway from RM 0 (Mississippi River ORCC) up to RM 212 near the Caddo-Bossier Port, which is the most upstream port on the waterway. However, the farthest that a dredge will typically travel upstream is to the lower approach at L&D 5 (RM 200). Dredging is broken out into "advanced maintenance" and "allowable overdepth," per Engineer Regulation 1130-2-520. For the purpose of maintaining projects, District Commanders may approve advanced maintenance dredging within the authorized project limits to avoid frequent redredging throughout the year. Such advanced maintenance (dredging to depths or widths in excess of authorized project dimensions) can be performed in critical, fast shoaling areas to the extent it will result in the least overall cost. Allowable overdepth dredging (depth and/or width) outside the required prism is permitted to allow for inaccuracies in the dredging process. Authorization allows for 25 percent of overdepth, which is 3 feet below the 9-FT channel. Regarding funding to maintain the channel, River Operations has faced initial budgetary funding constraints just to maintain navigation under existing conditions. Additionally, commercial sand and gravel mining occurs on the Red River JBJ Waterway and in areas upstream of the waterway limits, but this aspect is not included within this study in any manner.

<u>Historical Dredge Records</u>

Dredge records from 2012 to 2024 were provided by the River Operations Branch Dredging Unit. Notably, the data were provided in September; therefore, the 2024 data recorded do not reflect the entire year but up until 34 September 2024. River Operations noted that the dredge recording system changed in 2020, and is said to be an optimized approach compared to the previous recording system, and that data prior to 2012 are not easily accessible or discernable. River Operations also noted that there was no channel dredging for the years 2013 and 2016; however, the exact reasons are unknown. Upon reviewing dredge records from 2012–2024. it is clear that a large majority of the frequent channel dredging occurs in the reaches downstream of L&D 1 between RMs 33 and 42. There is also a clear trend of recurring dredging near RM 191. In addition to these two locations, there are a few other locations that have been occasionally dredged since 2012. The data also illustrate the considerable efforts taken to maintain depths near the locks and dams each year; however, as previously noted, this is not a focus of the study. The study assumes that there will always be a major need for dredging near the locks. Dredge records per pool and below L&D 1 are also provided from 1989 to 1999 per Pinkard 2001. The dataset is documented in Section 4.1.2.3 of this report and is informative as it provides a look into the dredge activities in the first several years around the time when the locks and dams were being constructed and opening for navigation (L&D 1 opened in 1984) and when the waterway was finished in 1995 with L&D 5. The 1989 to 1999 data for the reach below L&D 1 are plotted in Figure A-12 for comparison with the data from 2012 to 2024. The data reveal that the 2012 to 2024 dredge volumes are dramatically lower than the 1989 to 1999 volumes.

Depositional Areas

Deposition in lock approaches is the routine build-up of bed and suspended material in the upper and lower approach zones: most commonly along and behind guide walls, near the heads and tails of guide walls, and on or just riverward of lower miter gate sills. On the JBJ Waterway, much of the annual channel-maintenance effort occurs at the locks and dams themselves, and monitoring has long documented persistent shoaling below L&D 1 (the Gauntlet, further defined in following paragraphs), consistent with the hydraulics that develop immediately downstream of the structure. The physical layout of locks and dams can and often do result in eddies and slack water areas. These areas are natural sediment deposition zones. The drivers are straightforward hydraulically. The lock monoliths, dam bays, and long guide walls create separation, eddies, and slack water pockets that reduce local velocities and trap sediment. Because the Red River carries a high sediment load, there is ample material available to infill these low-energy cells whenever conditions allow. Flat pool slopes during low flow further limit transport capacity, and below L&D 1 the reach is strongly influenced by Lower Mississippi River backwater through Old River, which can suppress velocities in the tailrace and exacerbate deposition. Because these slack water zones are a by-product of the structures and operating objectives that make safe lockage possible, approach deposition cannot be fully “designed away.” Even with local geometry refinements, some recirculation persists across the operating range; the river’s sediment supply is continuous; and there are no upstream reservoirs on the JBJ Waterway that can be used to generate controlled “flushing” flows to clear approaches. The practical consequence is that shoaling can be shifted or reduced, but not eliminated. The district manages approach deposition with routine surveys and targeted dredging, complemented by localized sediment-reduction/training features where feasible, and by analytical modeling to ensure any modifications for navigation depth do not degrade approach hydraulics or lockage safety. These measures improve conditions and reliability but, by design and by physics, do not remove the underlying depositional tendency.

During the first high-water event after L&D 1 was opened in the fall of 1984, significant sediment deposition occurred at four primary locations (Pinkard-Stewart, 2001). These problems were in the upstream lock approach channel, along the riverside wall, downstream lock approach channel, and against the lock miter gates. As a result of these problems, design features at L&D 2 were implemented such as a cross-section at the structure that more closely represented that of the natural river section, no separation between the lock structure and the dam structure, and fixed guide walls instead of floating guide walls.

However, potential for deposition in the approach channels remained. Stone sediment control dikes were placed to extend downstream from the riverside lock wall narrowing the approach channel along with a high-velocity scour jet system. Based on the experience gained along with physical and numerical model studies, L&Ds 3, 4, and 5 included structural modifications aimed at significantly reducing sediment deposition.

Additionally, USACE Vicksburg District and ERDC CHL completed the 2020 Red River Hydraulic Analysis at Shreveport to assess changes that had occurred in the Red River in the vicinity of Shreveport to L&D 5 between the 1990 and 2015 floods. A few aspects of the analysis were slope trends, stream power, and sediment transport analyses. Technical details can be found in the report, but general conclusions were that river slopes were greatly reduced with the dam in place, with the largest reduction occurring at the lower flows. River slopes were comparable to pre-dam slopes once flow becomes open-river flow, i.e., not as controlled by the dam; however, some degree of swellhead or backwater influence still exists at such flows. Additionally, the lock and dam structures lead to a lesser stream power, which equates to a reduction in sediment transport capacity in the reach above the dam. The sediment model analysis showed that very little suspended coarse load moves through the dam until flows begin to exceed 100,000 cfs, which is in the range of bankfull flows within this reach.

Study Focus

As mentioned, the stretch of the river downstream of L&D 1 between RMs 33 and 42 is often referred to as the Gauntlet. This lowermost stretch of the waterway is situated between L&D 1 (RM 43) and the Mississippi River ORCC structure (RM 0) within the greater Lower Red River Backwater Area. This lowermost area of the Red River is heavily influence by Mississippi River flows through ORCC (Mississippi River Backwater) in addition to the Red River and Ouachita/Black River flows. Due to the significant influence of the Mississippi River, the lower most Red River below L&D 1 can experience significant periods of low water during the seasonal low-water period of the Mississippi River that typically occurs during the late summer to early winter months, which is generally during the lowest-flow period of the Red River Basin. This backwater influence also suppresses velocities driven by Red River headwater flows, which can exacerbate deposition in the channel. In recent times such as 2022 and 2023, the Mississippi River has experienced historically low water levels during the usual fall low-water season. The area below L&D 1 has been analyzed various times, as shown in historical studies and design memorandums. For example, the 1972 Design Memorandum No. 1 concluded that navigation for the 9-FT channel would be restricted 15 percent of the time without channel contraction (river training) through the entire reach (L&D 1 to Acme, Louisiana) and 9 percent of the time with maximum channel contraction. This stretch of the waterway is naturally narrower than upper reaches; therefore, channel contraction structures have limitations regarding lengths of the structures. The memorandum also concluded comparative cost estimates indicated that the then present worth of the reduction in annual maintenance dredging costs over the project life through channel contraction would more than offset the cost of contraction. If, as the result of the lock and dam site selections, L&D 1 were located near the mouth of the Black River, the previously mentioned contraction would no longer be necessary.

The annually dredged channel locations are RM 191 (Westdale), and the stretch of channel below L&D 1, RM 34 to 42. Although all locations below L&D 1 are not annually dredged, from a systematic standpoint some location within this reach is dredged every year. The occasionally dredged locations, or those areas that have been dredged a few times between 2012 and 2024, are RM 194 (Williams/East Point), RM 185 (Campti), RM 154 (Socot), and RM 106 (Boyce). Noted that RM 106 is primarily referring to an oxbow entryway.

Figure A-5 provides a map of the historically dredge locations and frequency of dredging on the JBJ Waterway.

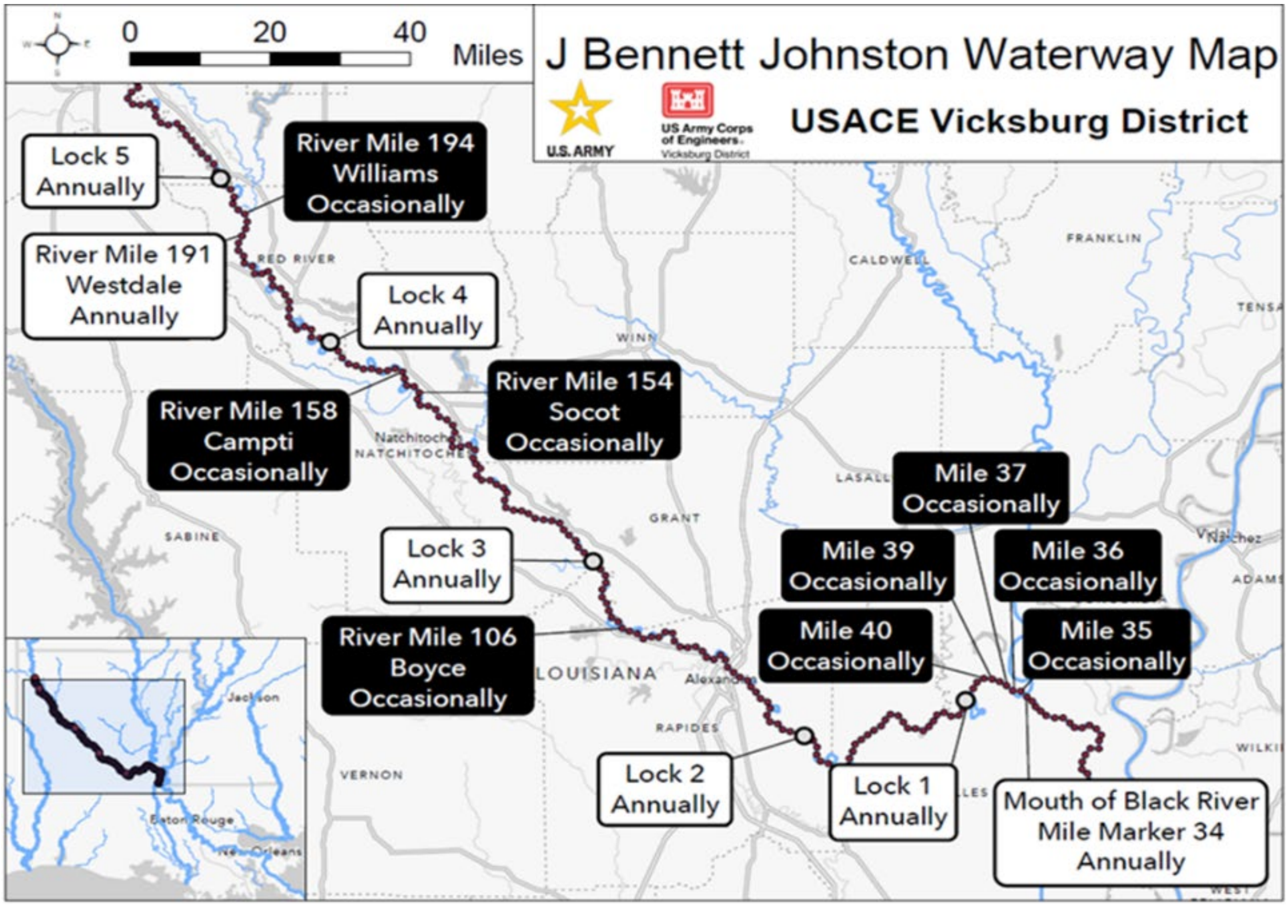


*Figure A-5. JBJ Waterway Existing Conditions: Dredge Locations Map*

Figure A-6 presents historical dredge records from 1989 to 1999, as provided in the referenced 2001 Pinkard report.

2001 Pinkard Report Dredge Volumes (Cubic Yards) Per Each Pool and Lock and Dam - 1989 to 1999

| Lock and Dam Dredging | 1989 | 1990 | 1991 | 1992 | 1993 | 1994 | 1995 | 1996 | 1997 | 1998 | 1999 | | Average |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Lock 5 | NA | NA | NA | | NA | NA | 32540 | 0 | 30033 | 18040 | 21956 | | 20,514 |
| Lock 4 | NA | NA | NA | | NA | NA | 41923 | 23396 | 73134 | 49937 | 58930 | | 49,464 |
| Lock 3 | NA | NA | NA | Data Recorded reported as not complete | 279974 | 65733 | 129246 | 33128 | 113111 | 81114 | 32932 | | 126,902 |
| Lock 2 | 266041 | 464785 | 114180 | | 590500 | 223275 | 245803 | 45794 | 238389 | 195587 | 203637 | | 288,954 |
| Lock 1 | 966297 | 1482097 | 809001 | | 1606385 | 685761 | 640342 | 180275 | 390068 | 812367 | 664384 | | 894,851 |
| | | | | | | | | | | | | | |
| Total | 1,232,338 | 1,946,882 | 923,181 | - | 2,476,859 | 974,769 | 1,089,854 | 282,593 | 844,735 | 1,157,045 | 981,839 | | 1,307,905 |
| | | | | | | | | | | | | | |
| Channel Dredging | | | | | | | | | | | | | |
| Below Lock 1 | 1,237,031 | 1,791,417 | 2,127,066 | | 1,323,493 | 843,404 | 426,283 | 0 | 414,594 | 1,416,730 | 591,782 | | 1,045,027 |
| Pool 1 | 131,215 | 460,053 | 354,000 | | 0 | 0 | 0 | 0 | 20,573 | 136,065 | 37,306 | | 103,565 |
| Pool 2 | 0 | 0 | 0 | Data Recorded as not complete | 0 | 0 | 0 | 77,547 | 0 | 124,418 | 47,240 | | 22,655 |
| Pool 3 | N/A | N/A | N/A | | N/A | N/A | 1,126,915 | 319,489 | 231,762 | 728,944 | 271,122 | | 535,646 |
| Pool 4 | N/A | N/A | N/A | | N/A | N/A | 258,980 | 0 | 0 | 140,381 | 21,363 | | 84,145 |
| Pool 5 | N/A | N/A | N/A | | N/A | N/A | 0 | 0 | 0 | 0 | 0 | | 0 |
| | | | | | | | | | | | | | |
| Total | 1,368,246 | 2,251,470 | 2,481,066 | - | 1,323,493 | 843,404 | 1,812,178 | 397,036 | 666,929 | 2,546,538 | 968,813 | | 1,452,970 |

*Figure A-6. JBJ Waterway Dredge Volumes, Pinkard 2001 (1989–1999)*

Figure A-7 provides a tabulation of the annualized number of days dredged per location from 2018 to 2024.

| Location | 2024 (# of days) (through 24 Sept) | % of Total Days | 2023 (# of days) | % of Total Days | 2022 (# of days) | % of Total Days | 2021 (# of days) | % of Total Days | 2020 (# of days) | % of Total Days | 2019 (# of days) | % of Total Days | 2018 (# of days) | % of Total Days |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Lock and Dam Dredging** | | | | | | | | | | | | | | |
| Lock 5 | 8 | 7.4% | 6 | 6.8% | 3 | 4.8% | 3 | 3.1% | 5 | 17.9% | 2 | 2.6% | 7 | 11.9% |
| Lock 4 | 4 | 3.7% | 9 | 10.2% | 3 | 4.8% | 5 | 5.2% | - | 0.0% | 16 | 20.5% | - | 0.0% |
| Lock 3 | 4 | 3.7% | 3 | 3.4% | 3 | 4.8% | 5 | 5.2% | - | 0.0% | 11 | 14.1% | - | 0.0% |
| Lock 2 | 13 | 12.0% | 13 | 14.8% | 5 | 8.1% | 17 | 17.7% | - | 0.0% | 10 | 12.8% | 8 | 13.6% |
| Lock 1 Upper | 9 | 8.3% | 2 | 2.3% | 3 | 4.8% | 3 | 3.1% | 2 | 7.1% | 18 | 23.1% | - | 0.0% |
| Lock 1 Chamber | - | 0.0% | - | 0.0% | 5 | 8.1% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% |
| Lock 1 Lower | 19 | 17.6% | 18 | 20.5% | 13 | 21.0% | 28 | 29.2% | 9 | 32.1% | 11 | 14.1% | 37 | 62.7% |
| **Channel Dredging** | | | | | | | | | | | | | | |
| Williams RM 194 | 5 | 4.6% | - | 0.0% | - | 0.0% | 5 | 5.2% | - | 0.0% | 7 | 9.0% | - | 0.0% |
| Westdale RM 191 | 12 | 11.1% | 9 | 10.2% | 4 | 6.5% | 13 | 13.5% | - | 0.0% | 3 | 3.8% | - | 0.0% |
| Campti RM 158 | - | 0.0% | - | 0.0% | - | 0.0% | 3 | 3.1% | - | 0.0% | - | 0.0% | - | 0.0% |
| Socot RM 154 | - | 0.0% | - | 0.0% | 5 | 8.1% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% |
| Boyce RM 106 (Oxbow Entryway) | - | 0.0% | 2 | 2.3% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% |
| RM 52 | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% |
| RM 42 | 2 | 1.9% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% | 4 | 6.8% |
| RM 40 | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% | 12 | 42.9% | - | 0.0% | 3 | 5.1% |
| RM 39 | 11 | 10.2% | 15 | 17.0% | 11 | 17.7% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% |
| RM 38 | 2 | 1.9% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% |
| RM 37 | 4 | 3.7% | 3 | 3.4% | 3 | 4.8% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% |
| RM 36 | 3 | 2.8% | 4 | 4.5% | - | 0.0% | 4 | 4.2% | - | 0.0% | - | 0.0% | - | 0.0% |
| RM 35 | 8 | 7.4% | - | 0.0% | - | 0.0% | 7 | 7.3% | - | 0.0% | - | 0.0% | - | 0.0% |
| RM 34 | 4 | 3.7% | 4 | 4.5% | 4 | 6.5% | 3 | 3.1% | - | 0.0% | - | 0.0% | - | 0.0% |
| RM 33 | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% |
| RM 17.5 | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% |
| RM 0 | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% |
| **TOTAL DAYS** | **108** | **100.0%** | **88** | **100.0%** | **62** | **100.0%** | **96** | **100.0%** | **28** | **100.0%** | **78** | **100.0%** | **59** | **100.0%** |
| **TOTAL CY** | **1,108,622** | | **615,677** | | **504,144** | | **1,114,808** | | **295,521** | | **638,744** | | **486,138** | |
| PBUD | **$ 6,000,000** | | $ 5,000,000 | | $ 3,000,000 | | $ 3,000,000 | | $ - | | | | | |
| WorkPlan | - | | $ - | | $ 1,350,000 | | $ - | | $ 3,250,000 | | | | | |
| Supplemental | - | | $ 2,000,000 | | $ 400,000 | | $ - | | $ 3,500,000 | | | | | |
| **TOTAL FUNDS** | **$ 6,000,000** | | **$ 7,000,000** | | **$ 4,750,000** | | **$ 3,000,000** | | **$ 6,750,000** | | | | | |

*Figure A-7. JBJ Waterway Annualized Number of Days Dredged (2018–2024)*

Figure A-8 provides a tabulation of the annualized number of days dredged per location from 2012 to 2017. River Operations breaks out the data into two different categories: lock and dam dredging and channel dredging. These two categories are further separated by location. L&D 1 data are additionally separated into upper approach, lock chamber, and lower approach.

| Location | 2017 (# of days) | % of Total Days | 2016 (# of days) | 2015 (# of days) | % of Total Days | 2014 (# of days) | % of Total Days | 2013 (# of days) | 2012 (# of days) | % of Total Days |
|---|---|---|---|---|---|---|---|---|---|---|
| **Lock and Dam Dredging** | | | | | | | | | | |
| Lock 5 | - | 0% | - | 7 | 9% | - | 0% | - | 5 | 4% |
| Lock 4 | 2 | 3% | - | 4 | 5% | - | 0% | - | 2 | 2% |
| Lock 3 | 6 | 8% | - | 10 | 13% | - | 0% | - | 4 | 4% |
| Lock 2 | - | 0% | - | 8 | 10% | - | 0% | - | 4 | 4% |
| Lock 1 Upper | 6 | 8% | - | 1 | 1% | - | 0% | - | 13 | 11% |
| Lock 1 Chamber | 10 | 13% | - | 7 | 9% | - | 0% | - | 31 | 27% |
| Lock 1 Lower | - | 0% | - | - | 0% | - | 0% | - | - | 0% |
| | | | | | | | | | | |
| **Channel Dredging** | | | | | | | | | | |
| Williams RM 194 | - | 0% | - | 5 | 6% | - | 0% | - | 1 | 1% |
| Westdale RM 191 | - | 0% | - | 2 | 3% | - | 0% | - | 1 | 1% |
| Campti RM 158 | - | 0% | - | 5 | 6% | - | 0% | - | - | 0% |
| Socot RM 154 | 7 | 9% | - | 9 | 12% | - | 0% | - | 2 | 2% |
| Boyce RM 106 (Oxbow Entryway) | - | 0% | - | - | 0% | - | 0% | - | - | 0% |
| RM 52 | 6 | 8% | - | - | 0% | - | 0% | - | - | 0% |
| RM 42 | - | 0% | - | - | 0% | - | 0% | - | 10 | 9% |
| RM 40 | 16 | 21% | - | 12 | 16% | 9 | 41% | - | 9 | 8% |
| RM 39 | - | 0% | - | - | 0% | - | 0% | - | 21 | 18% |
| RM 38 | - | 0% | - | - | 0% | - | 0% | - | - | 0% |
| RM 37 | - | 0% | - | - | 0% | - | 0% | - | - | 0% |
| RM 36 | 10 | 13% | - | - | 0% | 8 | 36% | - | 9 | 8% |
| RM 35 | 7 | 9% | - | 3 | 4% | 5 | 23% | - | - | 0% |
| RM 34 | 4 | 5% | - | - | 0% | - | 0% | - | 1 | 1% |
| RM 33 | - | 0% | - | 2 | 3% | - | 0% | - | - | 0% |
| RM 17.5 | 1 | 1% | - | - | 0% | - | 0% | - | - | 0% |
| RM 0 | - | 0% | - | 2 | 3% | - | 0% | - | 1 | 1% |
| | | | | | | | | | | |
| **TOTAL DAYS** | **75** | **100.0%** | | **77** | **100.0%** | **22** | **100.0%** | | **114** | **100.0%** |
| **TOTAL CY** | **502,144** | | | **958,996** | | **195,849** | | | **1,486,833** | |
| | | | | | | | | | | |
| PBUD | | | **No Dredging Recorded** | | | | | **No Dredging Recorded** | | |
| WorkPlan | | | | | | | | | | |
| Supplemental | | | | | | | | | | |
| **TOTAL FUNDS** | | | | | | | | | | |

*Figure A-8. JBJ Waterway Annualized Number of Days Dredged (2012–2017)*

Figure A-9 provides a tabulation of the annualized dredged volumes per location from 2012 to 2024. The L&D 1 data for the upper approach, lower approach, and lock chamber were combined. The data for the reach below L&D 1, RMs 33 to 42, were combined. Data for RM 106 Boyce Oxbow Entryway were removed, as were those for RMs 17.5 and 0.

| Location | 2024 Volumes (cubic yards) (through 24 Sept) | % of total volume | 2023 Volumes (cubic yards) | % of total volume | 2022 Volumes (cubic yards) | % of total volume | 2021 Volumes (cubic yards) | % of total volume | 2020 Volumes (cubic yards) | % of total volume | 2019 Volumes (cubic yards) | % of total volume | 2018 Volumes (cubic yards) | % of total volume |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Lock and Dam Dredging** | | | | | | | | | | | | | | |
| Lock 5 | 45,846 | 4.1% | 55,705 | 9.0% | 9,028 | 1.8% | 25,380 | 2.3% | 15,144 | 5.1% | - | 0.0% | - | 0.0% |
| Lock 4 | 30,381 | 2.7% | 36,665 | 6.0% | 8,032 | 1.6% | 38,917 | 3.5% | 38,917 | 13.2% | 122,583 | 19.2% | - | 0.0% |
| Lock 3 | 28,311 | 2.6% | 17,808 | 2.9% | 6,793 | 1.3% | 32,983 | 3.0% | - | 0.0% | 61,509 | 9.6% | - | 0.0% |
| Lock 2 | 86,175 | 7.8% | 40,716 | 6.6% | 7,188 | 1.4% | 89,028 | 8.0% | - | 0.0% | 107,981 | 16.9% | - | 0.0% |
| Lock 1 | 287,363 | 25.9% | 220,219 | 35.8% | 136,598 | 27.1% | 609,792 | 54.7% | 136,892 | 46.3% | 197,916 | 31.0% | 366,301 | 75.3% |
| **Channel Dredging** | | | | | | | | | | | | | | |
| Williams RM 194 | 53,473 | 4.8% | - | 0.0% | - | 0.0% | 36,805 | 3.3% | - | 0.0% | 61,327 | 9.6% | - | 0.0% |
| Westdale RM 191 | 179,846 | 16.2% | 71,553 | 11.6% | 35,850 | 7.1% | 120,878 | 10.8% | - | 0.0% | 87,428 | 13.7% | - | 0.0% |
| Campti RM 158 | - | 0.0% | - | 0.0% | - | 0.0% | 7,849 | 0.7% | - | 0.0% | - | 0.0% | - | 0.0% |
| Socot RM 154 | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% |
| RM 52 | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% | - | 0.0% |
| RM 42 to 33 | 397,227 | 35.8% | 173,010 | 28.1% | 300,655 | 59.6% | 153,176 | 13.7% | 104,568 | 35.4% | - | 0.0% | 119,837 | 24.7% |
| **TOTAL CY** | **1,108,622** | **100.0%** | **615,677** | **100.0%** | **504,144** | **100.0%** | **1,114,808** | **100.0%** | **295,521** | **100.0%** | **638,744** | **100.0%** | **486,138** | **100.0%** |

| Location | 2017 Volumes (cubic yards) | % of total volume | 2016 Volumes (cubic yards) | % of total volume | 2015 Volumes (cubic yards) | % of total volume | 2014 Volumes (cubic yards) | % of total volume | 2013 Volumes (cubic yards) | % of total volume | 2012 Volumes (cubic yards) | % of total volume |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Lock and Dam Dredging** | | | **No Dredging Recorded** | | | | | | **No Dredging Recorded** | | | |
| Lock 5 | - | 0% | - | - | 63,540 | 7% | - | 0% | - | - | 17876 | 1% |
| Lock 4 | 14,381 | 3% | - | - | 60,861 | 6% | - | 0% | - | - | 12673 | 1% |
| Lock 3 | - | 0% | - | - | 141,375 | 15% | - | 0% | - | - | 13327 | 1% |
| Lock 2 | 45,508 | 9% | - | - | 80,667 | 8% | - | 0% | - | - | 33152 | 2% |
| Lock 1 | 14274 | 3% | - | - | 80,073 | 8% | - | 0% | - | - | 692007 | 47% |
| **Channel Dredging** | | | | | | | | | | | | |
| Williams RM 194 | - | 0% | - | - | - | 0% | - | 0% | - | - | 8,520 | 1% |
| Westdale RM 191 | - | 0% | - | - | 96,638 | 10% | - | 0% | - | - | 6,307 | 0% |
| Campti RM 158 | - | 0% | - | - | 75,919 | 8% | - | 0% | - | - | - | 0% |
| Socot RM 154 | 54,901 | 11% | - | - | 173,492 | 18% | - | 0% | - | - | 12,827 | 1% |
| RM 52 | 34,340 | 7% | - | - | - | 0% | - | 0% | - | - | - | 0% |
| RM 42 to 33 | 338,740 | 67% | - | - | 186,431 | 19% | 195,849 | 100% | - | - | 690,144 | 46% |
| **TOTAL CY** | **502,144** | **100.0%** | | | **958,996** | **100.0%** | **195,849** | **100.0%** | | | **1,486,833** | **100.0%** |

*Figure A-9. JBJ Waterway Annualized Dredge Volumes (2012–2024)*

Figure A-10 provides a tabulation of the average annual dredge data for number of days dredged and volumes from 2012 to 2024 per location. Also provided is a summation of the total average annualized dredge data for number of days dredged and volumes. Lock and dam dredge data were not recorded for 2013, 2014, and 2016; however, these years were included as zero for the averaged calculations. In-channel dredge data were not recorded for 2013 and 2016; however, the years were included as zero for the averaged calculations.

| Location | Average Annual No. of Days Dredged (2012 to 2024) | % of Overall Total Average | | Average Annual Dredge Volume (cubic yards) (2012 to 2024) | % of Overall Total Average |
|---|---|---|---|---|---|
| **Lock and Dam Dredging** | | | | | |
| Lock 5 | 3.5 | 5.7% | | 19,869 | 3.4% |
| Lock 4 | 3.5 | 5.6% | | 26,997 | 4.7% |
| Lock 3 | 3.5 | 5.7% | | 19,983 | 3.5% |
| Lock 2 | 6.0 | 9.7% | | 34,853 | 6.0% |
| Lock 1 | 18.8 | 30.6% | | 210,880 | 36.4% |
| Total Lock and Dam | 35.4 | 57.4% | | 312580.9 | 54.0% |
| | | | | | |
| **Channel Dredging** | | | | | |
| Williams RM 194 | 1.8 | 2.9% | | 11,584 | 2.0% |
| Westdale RM 191 | 3.4 | 5.5% | | 45,932 | 7.9% |
| Campti RM 158 | 0.6 | 1.0% | | 5,090 | 0.9% |
| Socot RM 154 | 1.8 | 2.9% | | 13,401 | 2.3% |
| RM 52 | 0.5 | 0.7% | | 1,908 | 0.3% |
| RM 42 to 33 | 18.2 | 29.6% | | 188,391 | 32.5% |
| Total Channel | 26.2 | 42.6% | | 266,307 | 46.0% |
| | | | | | |
| Overall Total | 61.6 | 100.0% | | 578,888 | 100.0% |
| Note: Dredge records are not a complete picture of the total dredging needs in any given year. In addition to sediment conditions that warrant dredging, dredging is also influenced by the availibility of funding and resources. Therefore, the total dredging needs in any given year may actually exceed the districts ability to execute that dredging. | | | | | |

*Figure A-10. JBJ Waterway Average Annual Dredge Data – 2012 to 2024*

Figure A-11 plots the total annualized days of dredging from 2012 to 2024.

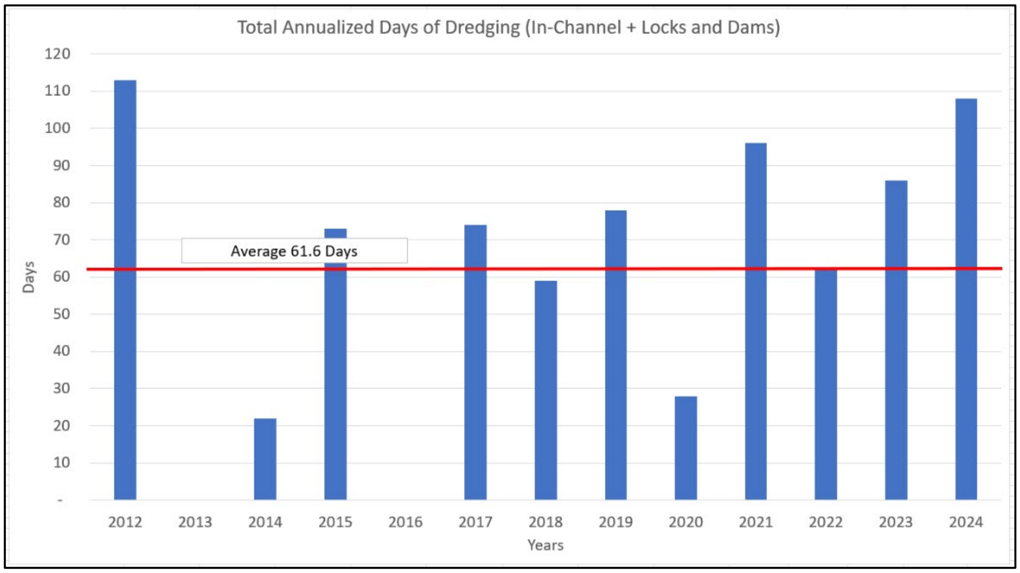


*Figure A-11. Total Annualized Days of Dredging – In-Channel + Locks and Dams*

Figure A-12 plots the total annualized dredge volumes from 2012 to 2024.

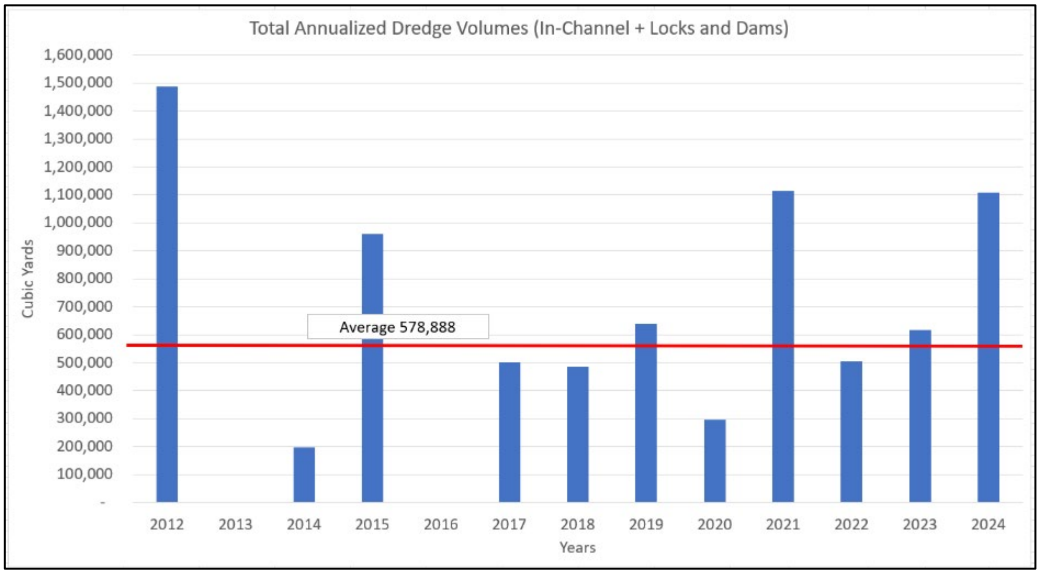


*Figure A-12. Total Annualized Dredge Volumes – In-Channel + Locks and Dams*

Figure A-13 compares the annualized lock and dam dredge volumes from 1989 to 1999 and 2012 to 2024. The dredge volumes have reduced considerably in recent years compared to the volumes recorded from 1989 to 1999. The river was undergoing major changes in the 1980s and 1990s as the waterway features such as the locks and dams were being constructed and completed. Various factors influence dredge volumes other than actual sediment conditions such as availability of funding and resources.

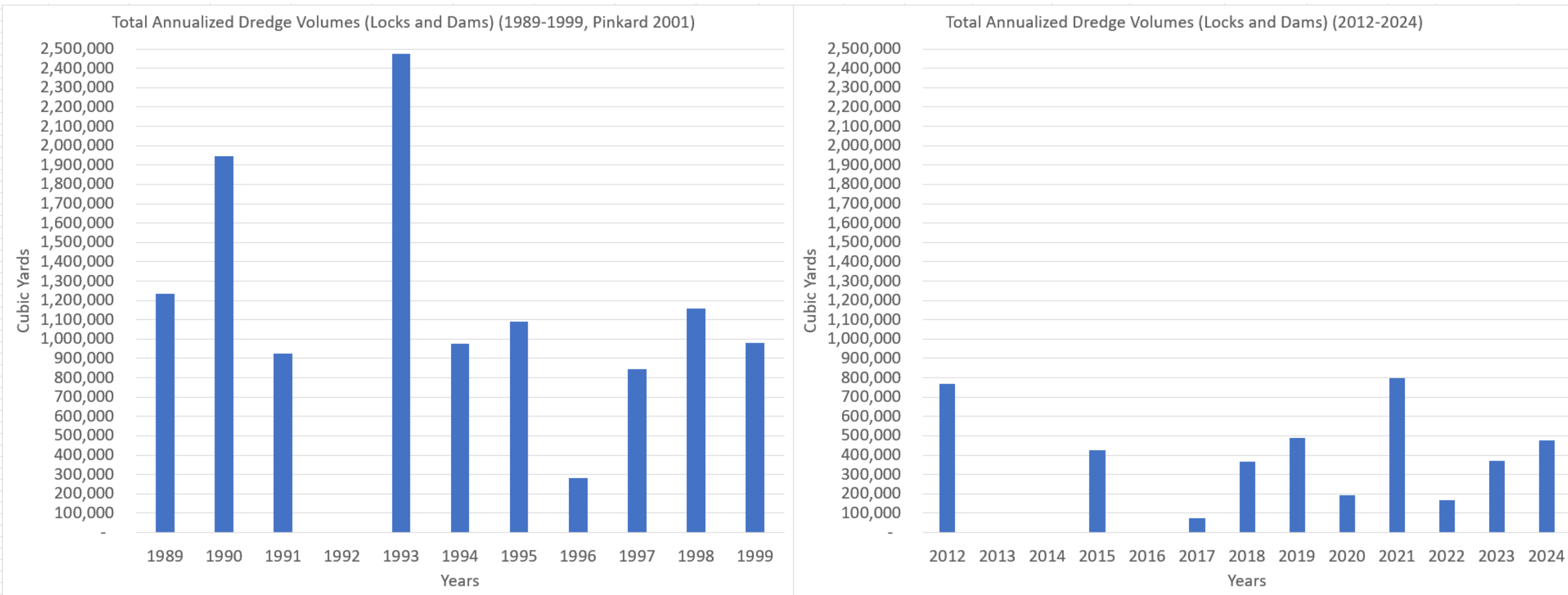


*Figure A-13. Total Annualized Lock and Dam Dredging Comparison – 1989 to 1999 Versus 2012 to 2024*

Figure A-14 illustrates the annualized number of in-channel dredge days per location from 2012 to 2024. Noted that 2013 and 2016 do not have recorded dredge data.

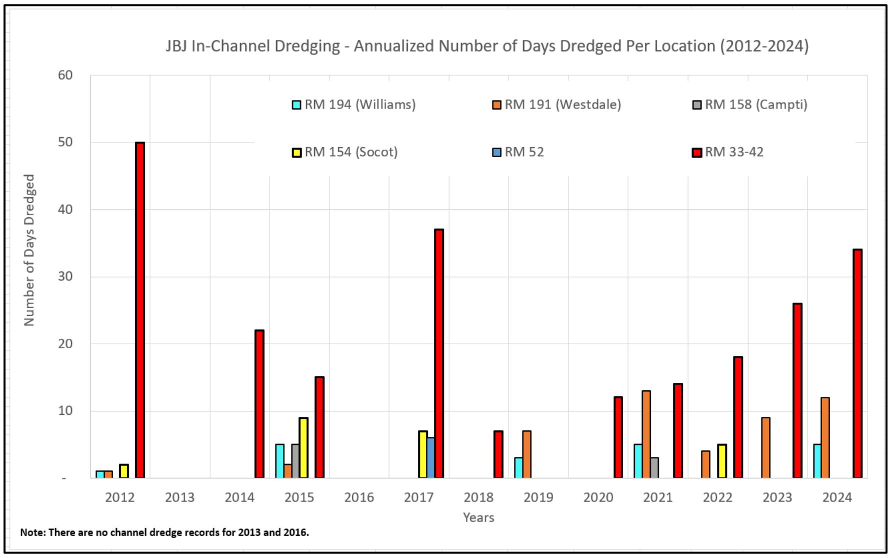


*Figure A-14. JBJ Waterway Annualized Number of In-Channel Dredge Days Per Location (2012–2024)*

Figure A-15 illustrates the annualized in-channel dredge volumes per location from 2012 to 2024. Noted that 2013 and 2016 do not have recorded dredge data.

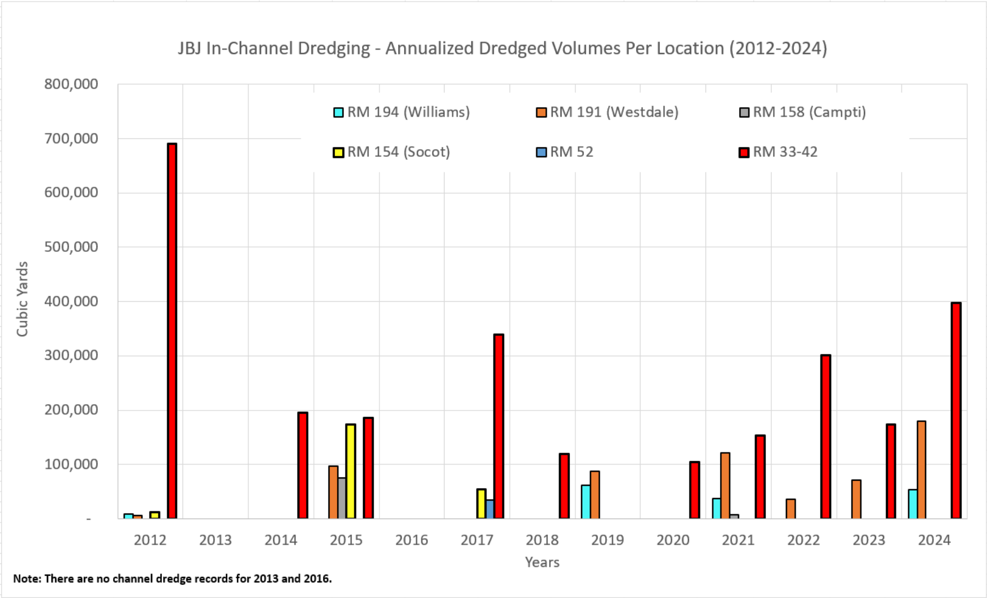


*Figure A-15. JBJ Waterway Annualized In-Channel Dredged Volumes Per Location (2012–2024)*

Figure A-16 plots the annualized number of dredged days for each lock and dam from 2012 to 2024.

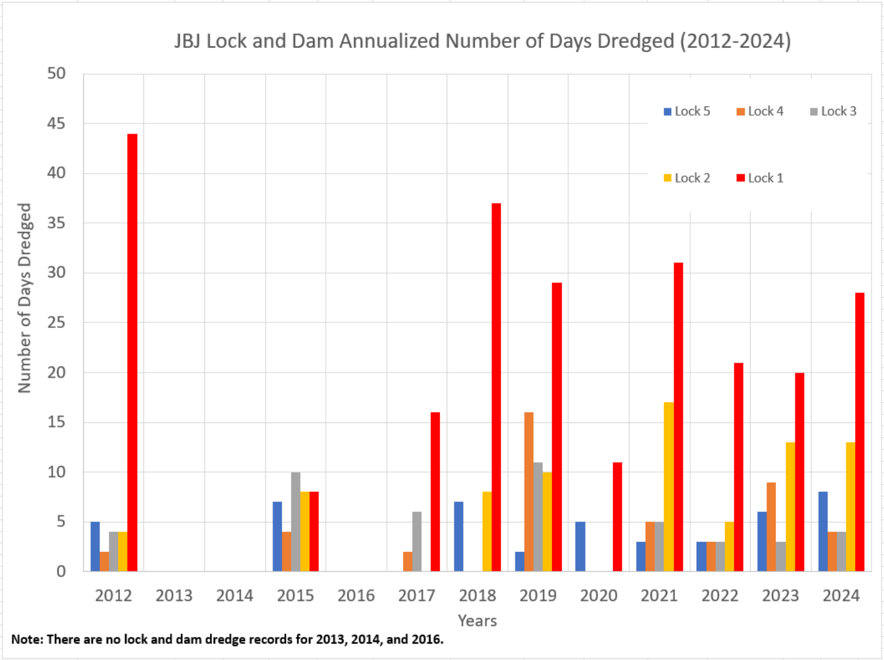


*Figure A-16. JBJ Waterway Annualized L&D of Dredge Days (2012–2024)*

Figure A-17 plots the annualized dredged volumes for each lock and dam from 2012 to 2024.

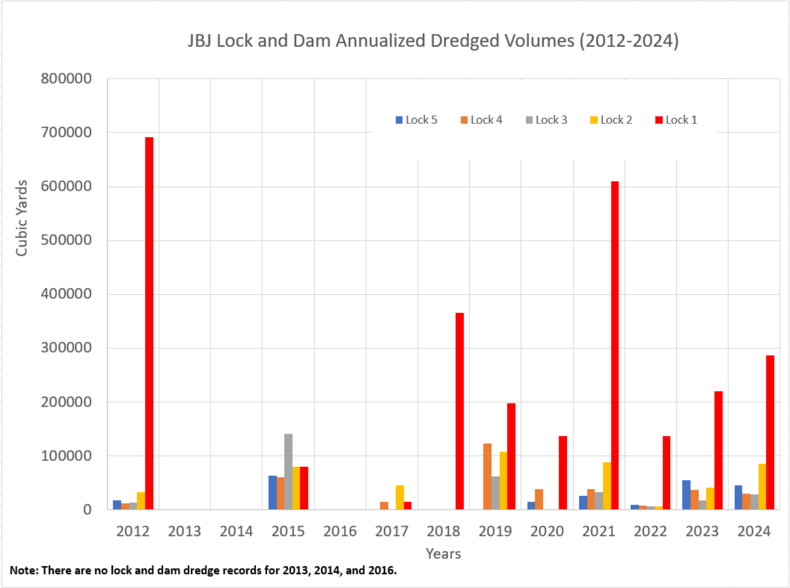


*Figure A-17. JBJ Waterway Annualized Lock and Dam Dredge Volumes (2012–2024)*

Figure A-18 plots the annualized dredge data for the historically documented problem reach at RM 191 from 2012 to 2024. Years that were not dredged at this location were excluded from the plot. The plot shows an increasing trend in dredge volumes.

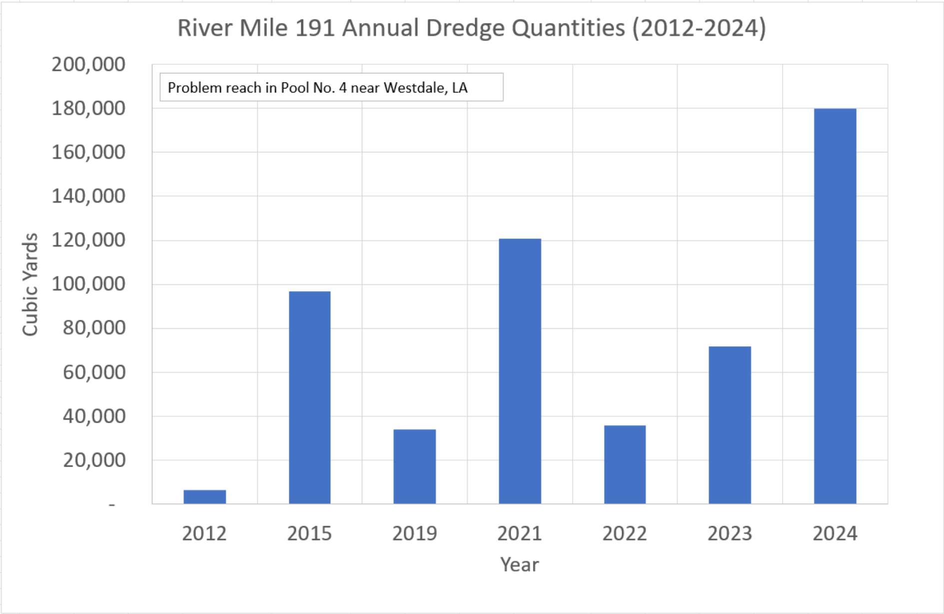


*Figure A-18. JBJ Waterway Annualized Dredge Quantities at RM 191*

Figure A-19 plots the annualized dredge data for the historically documented problem reach below L&D 1 from RM 33 to 42. Data from 1989 to 1999 were sourced from the referenced 2001 Pinkard Report to be plotted in comparison to the 2012 to 2024 dredge data. The plot shows that dredged volumes for the 2012 to 2024 period were dramatically reduced from those for the 1989 to 1999 period; however, present-day dredge efforts remain considerable and constant for this reach.

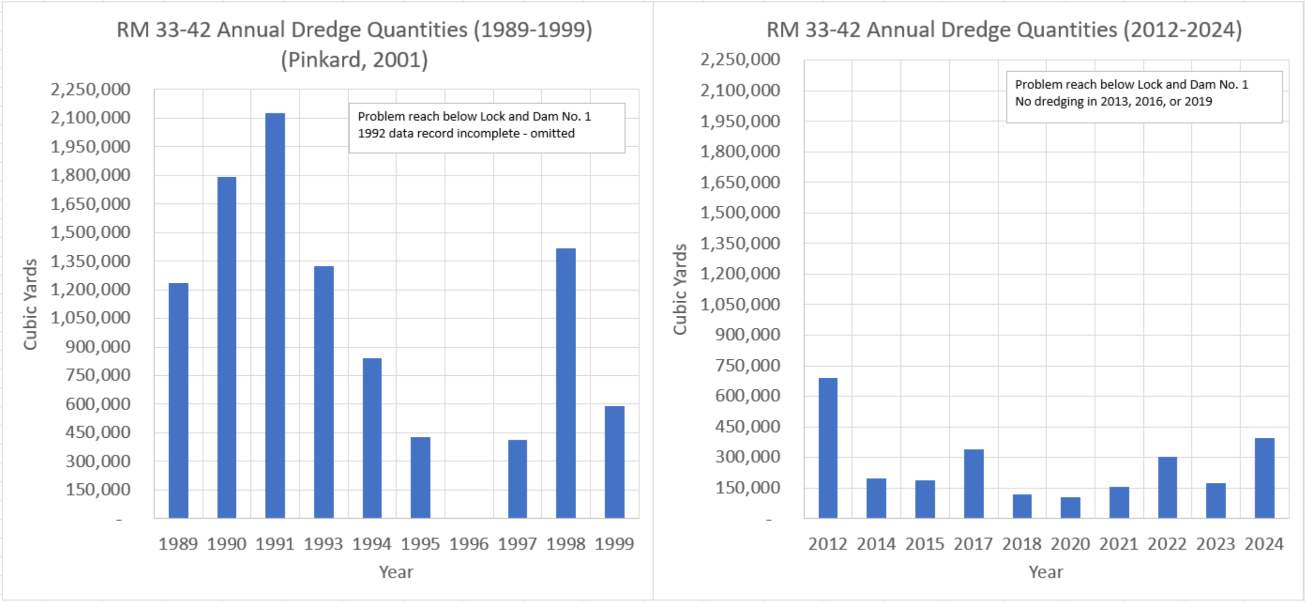


*Figure A-19. JBJ Waterway Annualized Dredge Quantities Below L&D 1*

## 1.3 ENGINEERING SCOPE

Various investigations were performed to assess the feasibility of providing a 12-FT draft for the JBJ Waterway, including structural, geotechnical, and hydraulic and hydrologic investigations. Need for this study was driven by federal interest in the economic advantages that could be achieved by allowing barges from the Mississippi River to travel through the JBJ Waterway without the need to offload cargo.

## 1.4 LIMITATIONS

This study faces several limitations, but the combination of historical dredge records, existing channel surveys (2016 single-beam survey; 2012 multi-beam survey), a calibrated 1D/2D hydraulic model, and design expertise provide a sufficient foundation for 35 percent feasibility-level river training design consistent with Engineering and Construction Bulletin (ECB) 2023-9. It is standard practice to obtain survey data closer to construction to ensure that recent data are informing construction activities. Therefore, additional data collection, such as bathymetric surveys, will be performed during the Preconstruction Engineering and Design phase and have not been used to inform initial estimates presented in this draft report. These surveys will inform the final hydraulic model.

The 212-mile study area restricts new survey collection due to time, cost, and access challenges. While more recent data would improve precision, existing surveys adequately support preliminary design, as the dynamic riverine system will naturally channel change before construction can occur. The existing 1D/2D Hydrologic Engineering Center's River Analysis System (HEC-RAS) model estimates water surface elevations and depths but lacks resolution for detailed velocity analysis around structures. Therefore, this model was not used for dike analysis. However, when checked against problem areas identified by dredge records, the model proved to be accurate at identifying areas that experience depositional areas and was, therefore, accurate enough to identify depth deficiencies for a 12-FT channel and allow for preliminary river engineering solutions.

Sediment transport modeling was not performed and is not expected to be performed in the future due to limited sediment data availability for the Red River. This should not have major impacts on the project, as dike improvements performed as part of the chosen alternative will have minimal effect on the sediment regime compared with new construction. Available historic sediment data and estimated channel-forming discharges guided design assumptions. Accurate geotechnical data will be collected once refined dike locations from updated surveys are identified during later design stages.

Supplemental data used to limit the effects of these limitations include updated multi-beam channel bathymetry within problem reaches and dike fields. Original design documentation, summarized in Section 4.1.2, established typical channel widths to be used for the JBJ Waterway channel design. Those original design dimensions will be considered in this effort. Further information on these limitations can be found in Section 4.1.1.

## 1.5 AVAILABLE DATA

Engineering analysis for this report is based on the most comprehensive data that could be acquired within the defined, relatively compressed schedule and budget. A significant data mining and compilation effort was completed during the initial phases of this project. A comprehensive list of data analyzed is provided below:

- A literature review of pertinent Red River and JBJ Waterway existing studies and original design documentation. The literature review is to be summarized in the Hydraulics section of the report.
- Dredge records from the Vicksburg District River Operations Branch were used to inform the 12-FT channel study about existing problem areas regarding insufficient channel depths for the existing 9-FT channel authorizations.
- The 2023 Red River Priority Repair List documents necessary construction and maintenance of revetments and dikes based on annual inspections and surveys.
- Hydraulic Model Assessments for Navigation Channel Depths:
  - Existing Red River HEC-RAS models were used to simulate a low-flow calibration event for model validity and then for project design conditions.
  - Utilized channel depth maps (RAS depth grids), ArcGIS, and JBJ Waterway project layers such as a navigation track centerline, navigation channel boundary polygon (200-FT channel) and dikes and revetments layers to identify reaches that show to have inadequate depths for the 12-FT deep by 200-FT wide navigation channel.
  - The HEC-RAS channel depth maps were used to validate the hydraulic model's ability to show that locations documented in the dredge record do not have adequate depths.

- 2012 multi-beam channel and thalweg surveys were compared to 2016 single-beam surveys to provide longitudinal profile view of the approximate changes in the navigation channel. Once problem reaches were identified, cross-section were plotted to compare the normal pool elevations (or minimum elevations for the reach below L&D 1) within the area to show channel bottom elevations relative to the normal pool elevation to approximately quantify the channel depths.

## 1.6 QUANTITY CALCULATIONS

Preliminary designs for individual measures are based off engineering judgement in coordination with Vicksburg District's River Stabilization Section to determine preliminary designs needed to achieve the desired results. To keep those portions of the study in budget and within the time constraints, designs were based off existing survey information and aerial imagery that is already available for the entire study region. Locations of new revetments and dredging sites were identified using the channel depth maps. Volumes for stone placement measures were determined with hand calculations, based on linear and area measurements taken from aerial imagery in Google Earth. Quantities for these calculations were based on typical construction values used within the waterway: 25 tons per foot for new construction and 10 tons per foot for improvements. These tonnage rates were derived from typical cross-sections of the expected dike dimensions, converting the calculated area into a tons per foot value. At 25 tons per foot, a dike approximately 15 feet tall with a 5-foot crown width can be constructed, which is consistent with the size of new dikes typically built on the Red River. The 10 tons per foot improvement rate represents the construction of a dike approximately 10 feet tall with a 5-foot crown width. Dike improvements generally consist of raising the existing structure by 2 to 3 feet and extending it further into the channel. These extensions are typically 100 to 200 feet in length. To account for both the average extension and height increase, a rate of 10 tons per foot is used for improvements. Dredging volumes were calculated using the dredge pump flowrates and start/stop times from daily dredge reports. Volume calculations based on aerial imagery introduces a significant source of error in quantities, which is accounted for in contingency calculations. See Appendix B for further information on contingency calculations. During the Preconstruction Engineering and Design phase, topographical surveys will allow increased accuracy to less than +/- 10 percent of actual values.

# SECTION 2
# GEOTECHNICAL ASSESSMENT

## 2.1 GEOTECHNICAL PROJECT OVERVIEW

The five locks and dams within the JBJ Waterway are in the alluvial valley of the Lower Red River between Marksville and Shreveport, Louisiana. These structures are founded on Pleistocene terrace deposits predominantly classified as high-plasticity clay (CH), characterized by very stiff to hard, fat clay in red, gray, and brown hues. The terrace deposits also contain interbedded layers of silt (ML) and lean clay (CL), along with abundant secondary features such as shell fragments, ironstone and claystone nodules, and lenticular beds of silt and silty sand.

Figure A-20 presents an excerpt from the Alexandria Geologic Quadrangle, illustrating typical surface materials between L&Ds 2 and 3. Along the Red River, soils primarily consist of point bar deposits composed mainly of sand, with variable amounts of silt, clay, and occasional gravel.

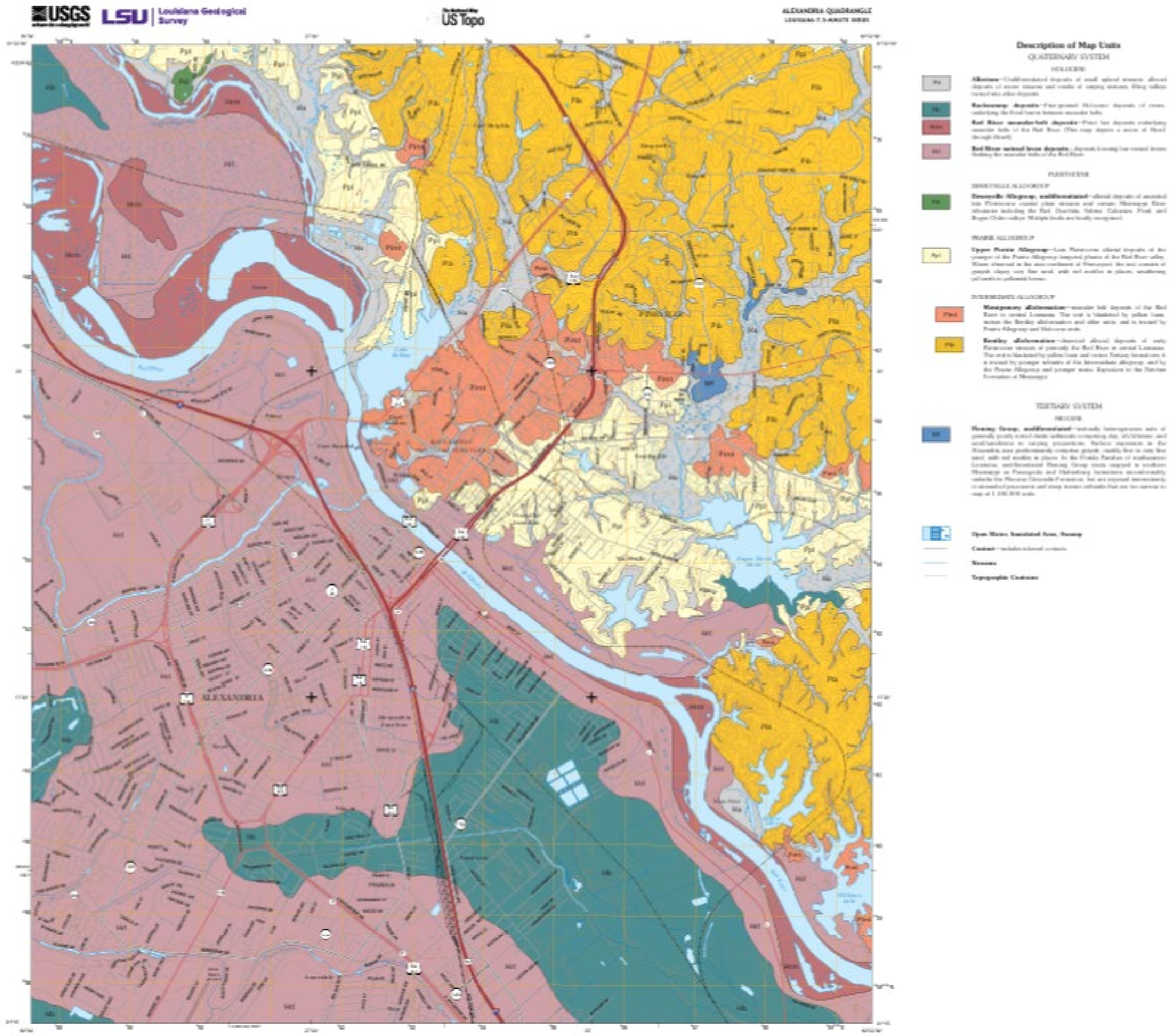


*Figure A-20. Excerpt from the Alexandria, Louisiana, Geologic Quadrangle*

The construction of the lock chambers is largely the same across the five projects. The locks consist of reinforced concrete U-frame sections supported directly on foundation soils. There have been no historical issues of voids or seepage occurring in or around the five lock chambers.

## 2.2 LEVEE SYSTEMS

Within the Vicksburg District's jurisdiction, ten distinct levee segments span approximately 300 miles along the Red River's east and west banks: 140 miles on the east bank and 160 miles on the west. These levees were constructed by local levee districts rather than

USACE, resulting in limited available geotechnical data regarding their foundation and construction.

Due to the significant expense and complexity involved in conducting comprehensive geotechnical investigations on these levees, alternatives that would directly affect levee structures were excluded from further consideration.

Figure A-21 illustrates the locations of the ten levee segments managed by the Vicksburg District, with Table A-10 detailing their respective sponsors and lengths.

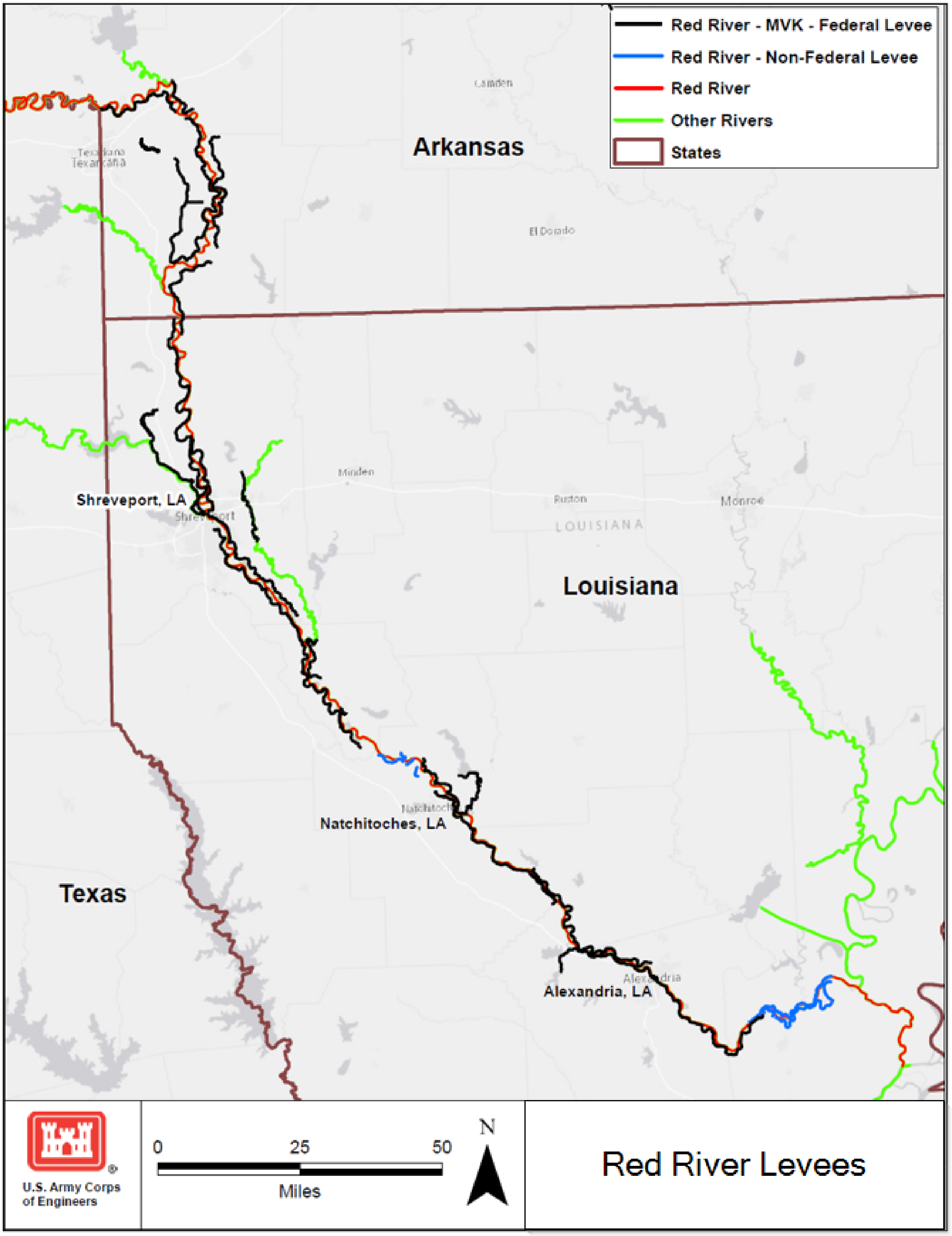


*Figure A-21. Representation of the 10 Levee Segments Within Vicksburg District's Portfolio on the Red River*

*Table A-10. Breakdown of the 10 Levee Segments and Their Sponsors*

| Bank | Levee | Sponsor | Miles |
|---|---|---|---|
| WEST | Lower Red River | Red River Atchafalaya and Bayou Levee District (RRAB) | 58 |
| | Natchitoches Levee | Natchitoches Levee District | 36 |
| | Caddo South/RRLDD | Red River Levee and Drainage District | 36 |
| | Caddo South/Caddo LD | Caddo Levee District | 29 |
| EAST | Pineville Levee | RRAB | 1.5 |
| | Aloha Rigolette | RRAB | 16 |
| | 19th Aloha Rigolette | 19th Louisianna Levee and Drainage District | 26 |
| | Campti Clarence Levee | Natchitoches Levee District | 33 |
| | East Point Levee | Red River Levee and Drainage District | 14 |
| | Bossier Levee | Bossier Levee District | 48 |

Dike modification alternatives do not directly interface with any levee segments and therefore do not require geotechnical analysis unless construction or modifications impact levee stability.

## 2.3 DAM SAFETY

The five locks along the Red River have undergone periodic inspections (PIs) or periodic assessments (PAs), with their current Dam Safety Action Classification (DSAC) ratings summarized in Table A-11.

*Table A-11. Lock and Dam Inspections and DSAC*

| Lock and Dam | Most Recent PI or PA | DSAC |
|---|---|---|
| L&D 1 | 2024 (PI) | 5 |
| L&D 2 | 2018 (PA); 2025 (PI) | 4 |
| L&D 3 | 2021 (PI) | 4 |
| L&D 4 | 2023 (PI) | 4 |
| L&D 5 | 2023 (PI) | 5 |

The primary dam safety concern involves increased loading scenarios, particularly from potential barge impacts. A dedicated barge impact analysis has been conducted (see Section 3). Potential seepage issues around lock chambers related to raising pool levels were considered; however, since alternatives involving pool elevation increases were screened out, seepage analyses were not pursued.

### 2.4 SUMMARY

The locks and dams on the JBJ Waterway are founded on stiff clay and silt terrace deposits within the Lower Red River alluvial valley, while adjacent riverbank soils predominantly comprise sandy point bar deposits. The Vicksburg District manages ten levee segments totaling approximately 300 miles, constructed by local districts with limited geotechnical information available. Due to the cost of detailed investigations, levee-impacting alternatives were excluded. Similarly, no seepage analyses were conducted since pool-raising options were screened out.

# SECTION 3
# STRUCTURAL ASSESSMENT

## 3.1 PROBABILITY BARGE IMPACT ANALYSIS (PBIA)

### 3.1.1 Background

With heavier barge vessels traveling the Red River in the proposed 12-FT channel, there is concern that the existing guide walls will be at a higher risk of damage due to barge impacts. The L&D 1 guide wall differs from the other four lock approaches, as it has a floating guide wall design. The four upper locks have similar guide wall designs that differ slightly in monolith height and length. L&D 2 is used as a representative of the entire Red River system for this analysis as it is the most trafficked of the upper four locks, with approximately 1,000 lockages per year according to Lock Performance Monitoring System (LPMS) data shown in Figure A-22 below.

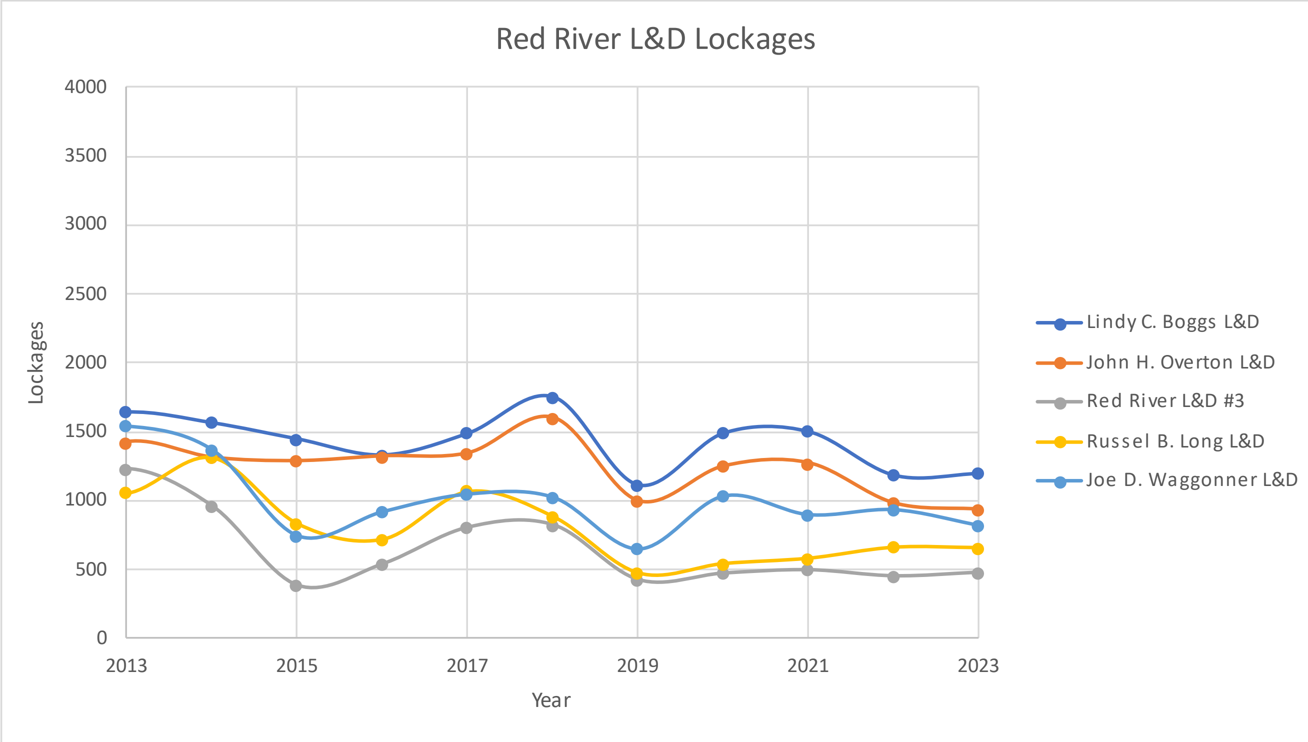


*Figure A-22. Annual Number of Lockages per Lock on the Red River*

L&D 2 has only had one significant barge impact in the history of the project. According to the 2018 Periodic Inspection Report, a set of five barges were tied off to an upstream mooring and they broke loose. One empty barge impacted Tainter Gate #2 and blocked

Gates 2, 3, 4, and part of 5. The four loaded barges floated into the dam and sank immediately upstream of the gated spillway. There was damage to a Tainter gate and concrete on the structure, with loss of navigation pool for approximately 1 month. This event occurred in December 2004, and no significant barge impacts have been recorded since.

L&D 4 has two reported incidents, one on the upstream bull nose and one on the downstream bullnose. The incident on the upstream bull nose was noted in the 2023 Periodic Inspection report showing significant damage to concrete parapet wall on the bull nose. Apart from damage to concrete, no significant effects have been noted at this location. The damage to the downstream bull nose occurred in 2014 due to barge impact resulting in a small spall with no reinforcement exposed, and no significant damage to the area.

L&D 5 has damage to the upstream guide wall that was reported as early as 2009 in the Periodic Inspection report. This caused spalling of the concrete in the walkway and reportedly due to barge impact, but no further documentation on the incident was found.

The Louisiana Department of Transportation and Development was contacted to update them on this study in regard to barge impacts on their bridges. There are approximately 7 bridges crossing the waterway: 3 road crossings in Alexandria (5 bridges total), 1 crossing in Boyce, and 1 crossing in Natchitoches. The Louisiana Department of Transportation and Development contact reported knowledge of some barge impacts on bridge piers in the downstream direction with mostly superficial damage to the piers. When asked about upstream vessel impacts, the contact reported that these impacts are rare. Tugs typically will reverse back downstream and try again if there is concern of not passing under the crossing cleanly.

### 3.1.2 Purpose

The purpose of this design report is to determine barge impact forces and guidance for consideration in the JBJ Waterway 12-FT Channel study. The Red River currently allows for 9-FT draft through the channel and were designed with a 120-kip barge impact force acting perpendicular to the guide wall. A 12-FT channel will allow for heavier tonnage vessels to transit through the five navigational locks on the Red River.

This statistical analysis is performed to evaluate new impacts on the lock guide walls due to heavier vessels and focuses specifically on L&D 2. The following datasets will be estimated using historical data:

a. Flotilla configurations
b. Barge and towboat mass
c. Estimate of flotilla velocities, both upstream and downstream
d. Estimate of flotilla impact angle
e. A deterministic barge impact analysis performed using the probabilistic values of mass, angles, and velocities above

### 3.1.3 Probabilistic Modeling

This design report uses the guidance methods presented in Engineer Manual (EM) 1110-2-3402 (EM 3402) Barge Impact Forces for Hydraulic Structures. EM 3402 discusses probabilistic and deterministic analysis methods to determine barge impacts. This analysis for Overton Lock (L&D 2) utilizes both methods. Probabilistic analysis is used to obtain statistical values of mass, velocity, and impact angle to be used as input into empirical design equations for semi-flexible walls, rigid walls, and pile founded guide walls.

The probabilistic barge impact analysis (PBIA) model was developed in an Excel model using @Risk Version 8.8 using W.D. Mayo L&D Analysis as a template. The model simulates both upstream and downstream events that could cause an impact with the existing guide walls based off collected historical data. The simulation model used 50,000 iterations to estimate the contributing factors for probabilities of barge impacts. Factors include direction of traffic, size of flotillas, size of barges, impact angle, and velocity. Mass is a constant value based off size of barge. Each of the 50,000 iterations changes these factors based off collected input data. The full PBIA is on file with the Vicksburg District’s Structures Section.

The probabilistic modeling is used to develop the probability distributions for mass, angle, and velocities used to calculate impact forces. Historical barge traffic data from the LPMS and AccessAIS website (https://marinecadastre.gov/accessais/) from both approaches of Overton Lock. These data are in spreadsheets titled "LPMS_FlotillaData_2000-23" and "AIS Combined Red River Data". LPMS data were used to determine mass, and AccessAIS was used to determine velocity and angle.

<u>Flotilla Configuration</u>

This LPMS data were captured for lockages at each of the five locks from 2000 to 2023. This data were extracted from LPMS to model both the upstream and downstream movement of flotillas in the vicinity of the guide walls.

Any flotilla comprised of solely empty barges was excluded from the data. The ratio of upstream to downstream traffic is given in Table A-12 below. Notably, upstream traffic typically consists of fully loaded barges delivering cargo upriver, while downstream traffic typically consists of empty barges exiting the Red River. Since empty barges are excluded from this analysis, upstream traffic is far greater than downstream traffic. When empty barges are considered, total number of upstream flotillas is 10,484 and downstream flotillas is 10,507.

*Table A-12. Direction of Loaded Flotillas at L&D 2 on the Red River*

| Direction | Flotillas | Percent |
|---|---|---|
| Upstream | 9,612 | 85% |
| Downstream | 1,663 | 15% |

Based on the LPMS data, probability density functions and cumulative density functions were developed for both the upstream and downstream loaded flotillas. These functions define the number of barges in each flotilla statistically for use in estimating the mass of each flotilla.

Historically, six barges (2x3 barge configuration) are the most common flotilla configuration, shown in Figure A-23 and Figure A-24. Therefore, this analysis focuses specifically on this flotilla configuration as it travels upstream to the lower approach.

**Barge CDF**

**Upstream**

Total Flotillas 9612

| No of Barges | No of Flot | pdf | CDF | 1-CDF |
|---|---|---|---|---|
| 1 | 710 | 0.0739 | 0.0739 | 0.9261 |
| 2 | 1155 | 0.1202 | 0.1940 | 0.8060 |
| 3 | 979 | 0.1019 | 0.2959 | 0.7041 |
| 4 | 841 | 0.0875 | 0.3834 | 0.6166 |
| 5 | 319 | 0.0332 | 0.4166 | 0.5834 |
| 6 | 5529 | 0.5752 | 0.9918 | 0.0082 |
| 7 | 26 | 0.0027 | 0.9945 | 0.0055 |
| 8 | 23 | 0.0024 | 0.9969 | 0.0031 |
| 9 | 9 | 0.0009 | 0.9978 | 0.0022 |
| 10 | 18 | 0.0019 | 0.9997 | 0.0003 |
| 11 | 2 | 0.0002 | 0.9999 | 0.0001 |
| 12 | 1 | 0.0001 | 1.0000 | 0.0000 |
| | | 1.0000 | | |

*Figure A-23. Upstream Barge Cumulative Density Functions*

**Barge CDF**

| | | | | | |
|---|---|---|---|---|---|
| | **Downstream** | | | | |
| Total Flotillas | 1663 | | | | |
| | No of Barges | No of Flotillas | pdf | CDF | 1-CDF |
| | 1 | 389 | 0.2339 | 0.2339 | 0.7661 |
| | 2 | 615 | 0.3698 | 0.6037 | 0.3963 |
| | 3 | 142 | 0.0854 | 0.6891 | 0.3109 |
| | 4 | 125 | 0.0752 | 0.7643 | 0.2357 |
| | 5 | 85 | 0.0511 | 0.8154 | 0.1846 |
| | 6 | 239 | 0.1437 | 0.9591 | 0.0409 |
| | 7 | 14 | 0.0084 | 0.9675 | 0.0325 |
| | 8 | 20 | 0.0120 | 0.9796 | 0.0204 |
| | 9 | 13 | 0.0078 | 0.9874 | 0.0126 |
| | 10 | 19 | 0.0114 | 0.9988 | 0.0012 |
| | 11 | 1 | 0.0006 | 0.9994 | 0.0006 |
| | 12 | 1 | 0.0006 | 1.0000 | 0.0000 |
| | | | 1.0000 | | |

Note: Total is 5,399 for six barges when empty barges are included, approximately 51 percent of all traffic.

*Figure A-24. Cargo Tonnage*

Mass of Flotilla

Only the mass of the barges in the lead row was used for guide wall impact. The Red River locks have a chamber width of 80 feet, restricting flotillas to double-wide 35-foot barges, or a single stack of 54-foot barges. The PBIA uses past traffic data to determine likely flotillas based off barge sizes. A mass of 1,530 short tons was assumed for a single 35-foot wide, 9-FT draft barge, with 2,058 short tons assumed for a single 35-foot wide, 12-FT draft barge. A mass of 3,348 short tons was assumed for a single 54-foot wide, 9-FT draft barge, with 4,584 short tons assumed for a single 54-foot wide, 12-FT draft barge. These masses were assumed based on industry design referencing Jeffboat shipyard barge designs shown in Figure A-25 and Figure A-26.

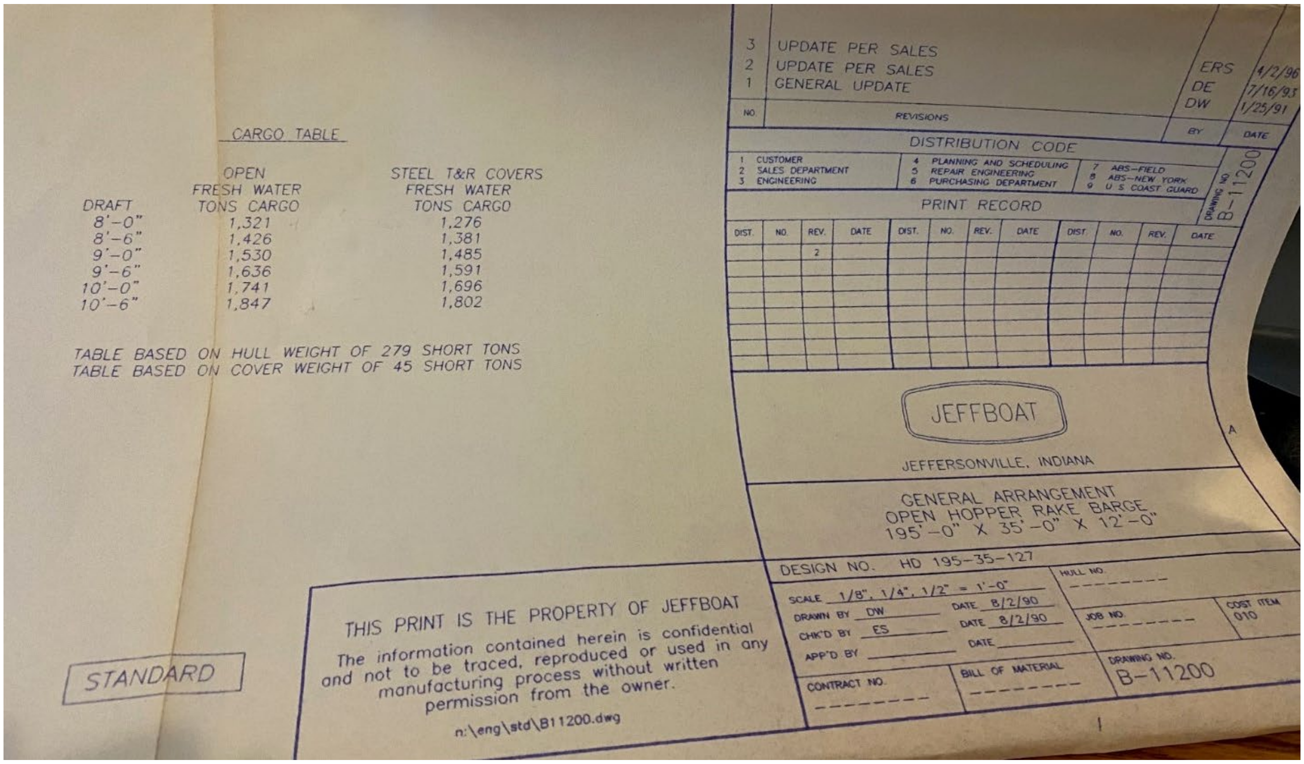


*Figure A-25. Cargo Tonnage Used for Jeffboat Shipyard Design for 135 foot x 35 foot x 12-FT Barge*

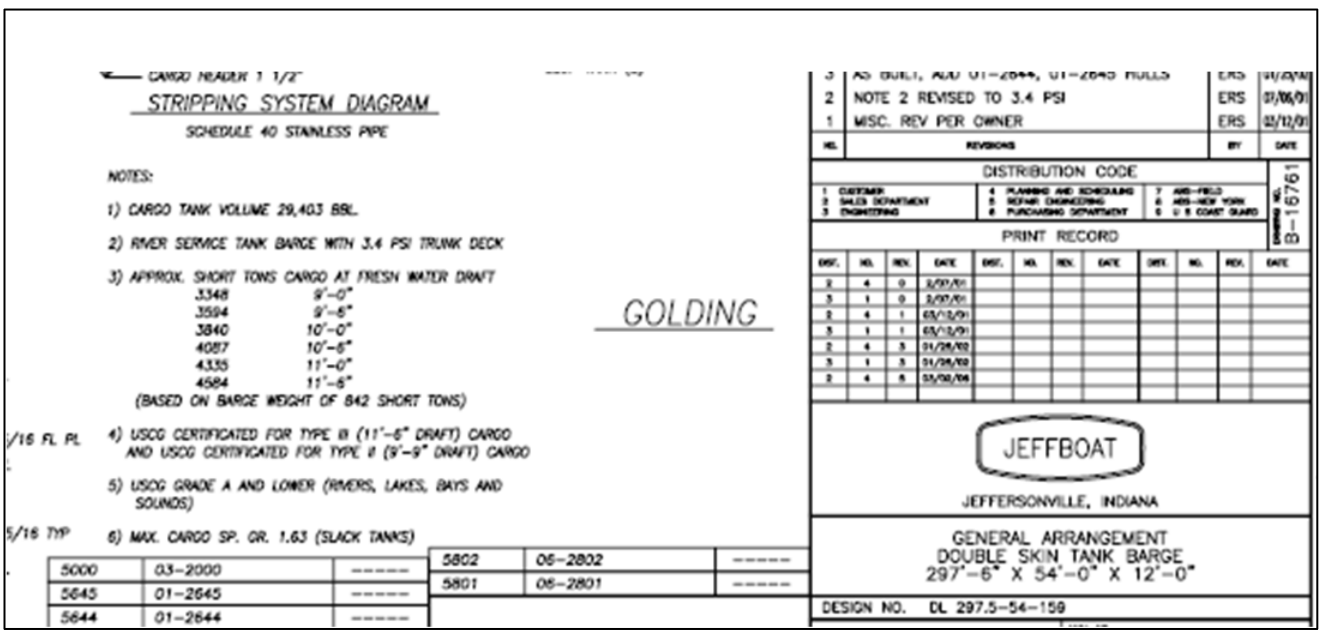


*Figure A-26. Cargo Tonnage Used for Jeffboat Shipyard Design for 297.5 foot x 54 foot x 12-FT Barge*

Flotilla Impact Angle

The angle of the lock was determined using satellite images from Google Maps and Microstation shown in Figure A-27. AIS data were analyzed to produce distribution plots of COG to determine potential angles of impact. The angle of the lock was determined to be 114° for traffic travelling downstream and 294° for traffic travelling upstream.

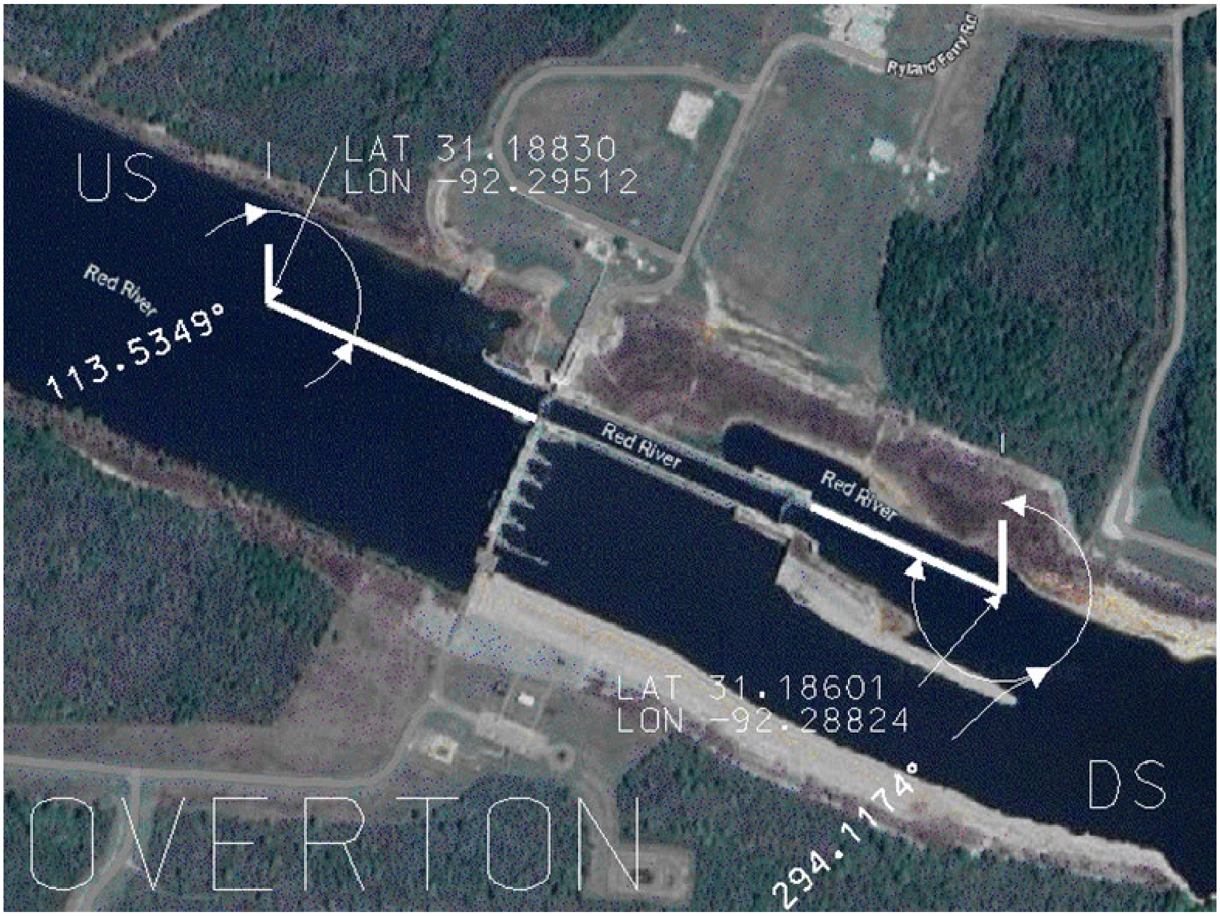


*Figure A-27. Lock Angle of Overton Lock*

Data from AIS were filtered to find vessels traveling in each direction using Course Over Ground (COG). Direction is analyzed assuming each vessel is traveling opposite the side of the dam it is located on (i.e., direction "UP" indicates vessels on the downstream side of the dam traveling upstream). COG was filtered based on the lock angle of +/- 30° toward the guide wall. Figure A-28 shows the statistical characteristics of the collected data. An angle of 3.4° was calculated as highest probability impact angle of barge traffic as they approach the guide wall.

Course Over Ground (COG)
Up

| Mean | Std Dev | Min | Max |
|---|---|---|---|
| 3.7 | 5.539 | 0.01 | 30.0 |

3.4 deg

*Figure A-28. COG Probabilistic Input for Upstream Traffic*

Velocity of Flotilla

The velocity of the flotillas at L&D 2 was determined using AccessAIS data. These data are given in knots and converted to feet per second. Data were filtered by latitude and longitude values to points located inside the channel along the guide walls for both approaches. COG was filtered to isolate only traffic approaching the lock. Speeds were filtered to remove any vessels traveling less than 0.34 feet per second to eliminate vessels exiting and entering the lock chamber and greater than 7.0 feet per second to eliminate any recreational speed boats. Figure A-29 shows the statistical characteristics of the collected data. A velocity of 2.3 feet per second was calculated as the highest probability velocity of barge traffic as they approach the guide wall.

Speed Over Ground (SOG)
Up

| Mean | Std Dev | Min | Max |
|---|---|---|---|
| 2.4 | 1.22 | 0.34 | 7.0 |

2.3 ft/s

*Figure A-29. Velocity Probabilistic Input for Upstream Traffic*

### 3.1.4 Impact Force Results

Highlighted data produced by PBIA was then used to calculate impact force. This input is not constant but is a random variable that varies over the 50,000 iterations in order to produce a distribution shown in Figure A-30 and Figure A-31. The usual, unusual, and extreme events are taken from this distribution based on confidence intervals and return periods as defined in EM 3402 Chapter 2. The primary empirical equation used in this analysis for a pile founded guide wall is defined in Equation 4.3 below. This equation reflects the 84th percentile or one standard deviation above the mean bilinear fit to the finite element model data in EM 3402. The use of one standard deviation reflects the uncertainty of the AIS velocity data used in the probabilistic barge impact model. Impact forces are shown in Table A-13, with 12-FT draft forces slightly higher due to heavier vessels. Figure A-32 and Figure A-33 show the output distribution for the impact forces.

$$F_{84.1\%} = \begin{cases} 3.152 \cdot m_{LR} \cdot v \cdot \sin\theta & \text{if } m_{LR} \cdot v \cdot \sin\theta \leq 116 \text{ kip-sec} \\ 366 + (0.533 + 0.000205 \cdot k) \cdot (m_{LR} \cdot v \cdot \sin\theta - 116) & \text{otherwise} \end{cases} \quad (4.3)$$

Where mlr = mass of lead row barges in kips-s$^2$/ft
V = velocity in ft/s
θ = impact angle
k = lateral wall stiffness in kip/in

(Note: Other values in Equation 4.3 are constants from the bilinear equation in EM 3402.)

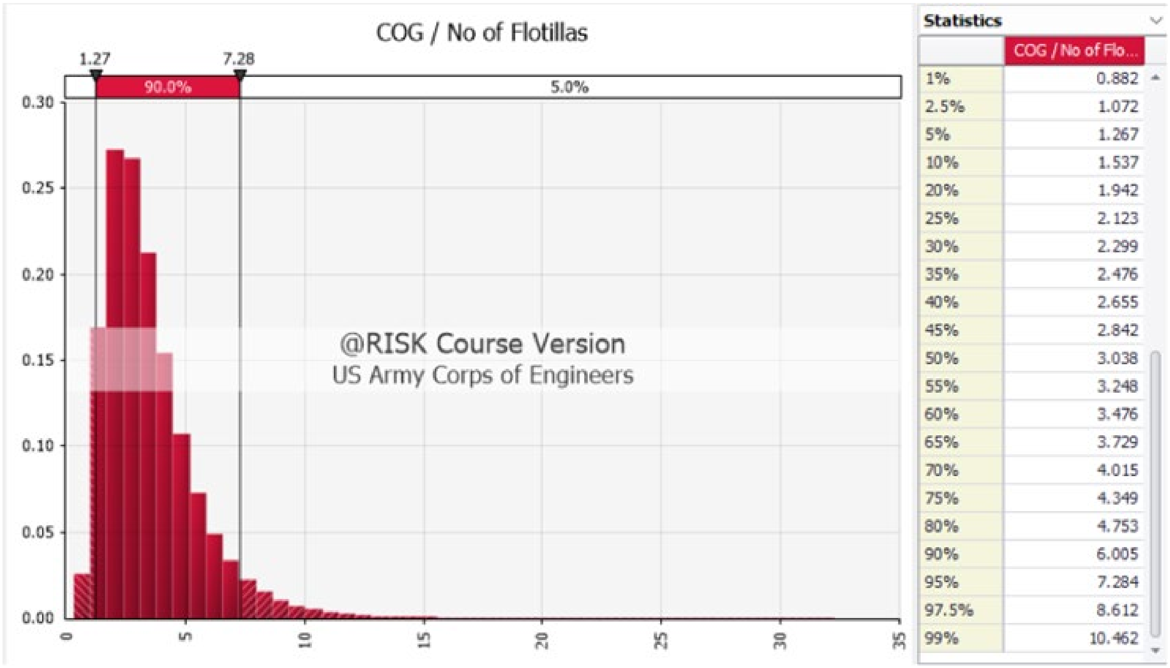


*Figure A-30. @Risk Output Impact Angle Distribution*

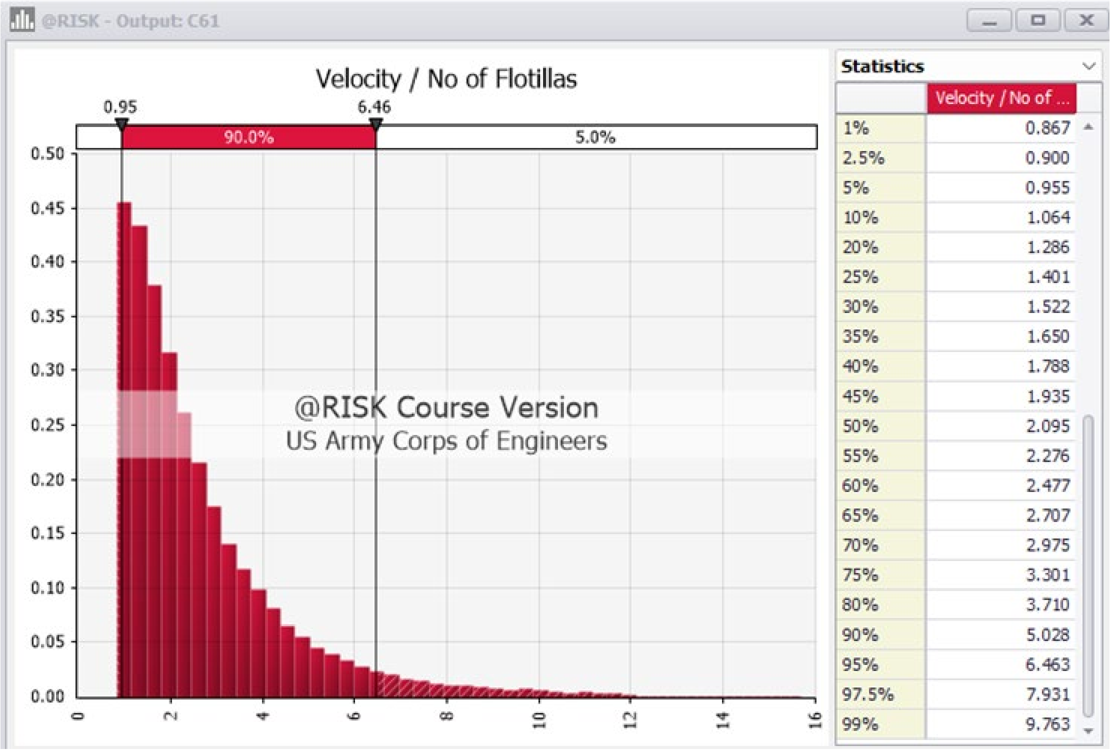


*Figure A-31. @Risk Output Velocity Distribution*

*Table A-13. Return Periods and Barge Impact Forces for 9-FT Draft and 12-FT Draft at Overton Lock Guide Walls*

| EM 3402 Event | Probability of Exceedance | Return Period (Years) | Impact Force (kips): 9-FT Channel | Impact Force (kips): 12-FT Channel |
|---|---|---|---|---|
| Usual | P(E)<0.10 | 5 | 109 | 135 |
| Unusual | 0.10<P(E)<0.0033 | 150 | 301 | 375 |
| Extreme | 0.0033<P(E)<0.00033 | 1500 | 409 | 451 |

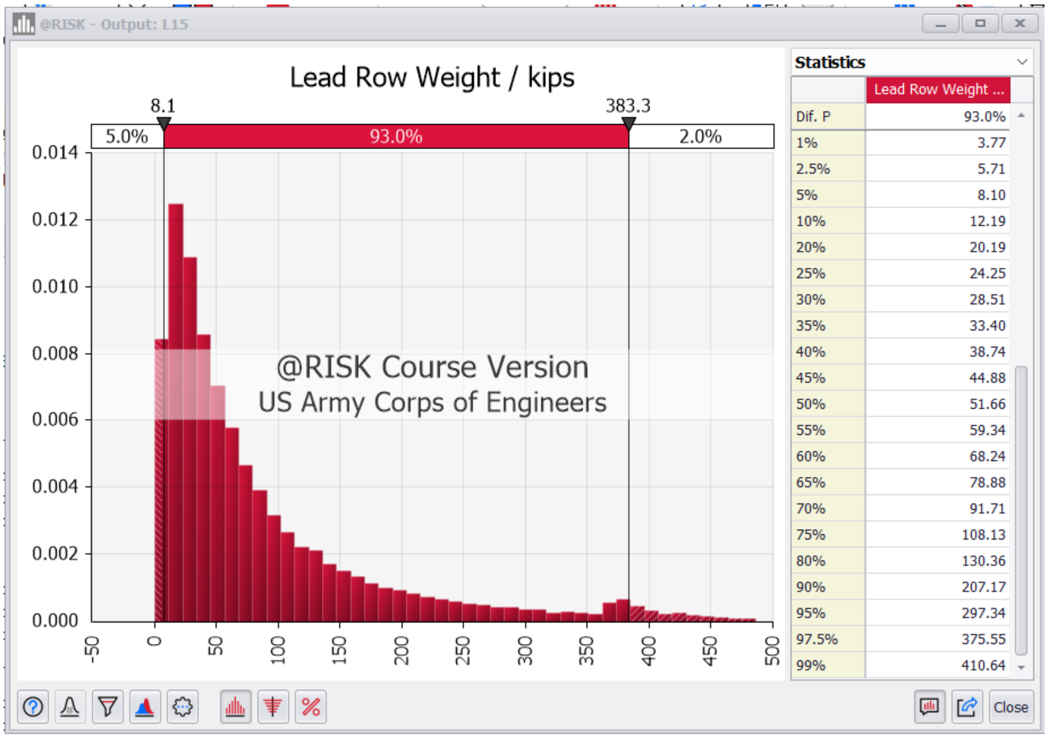


*Figure A-32. @Risk Output Impact Force Distribution for 9-FT Draft*

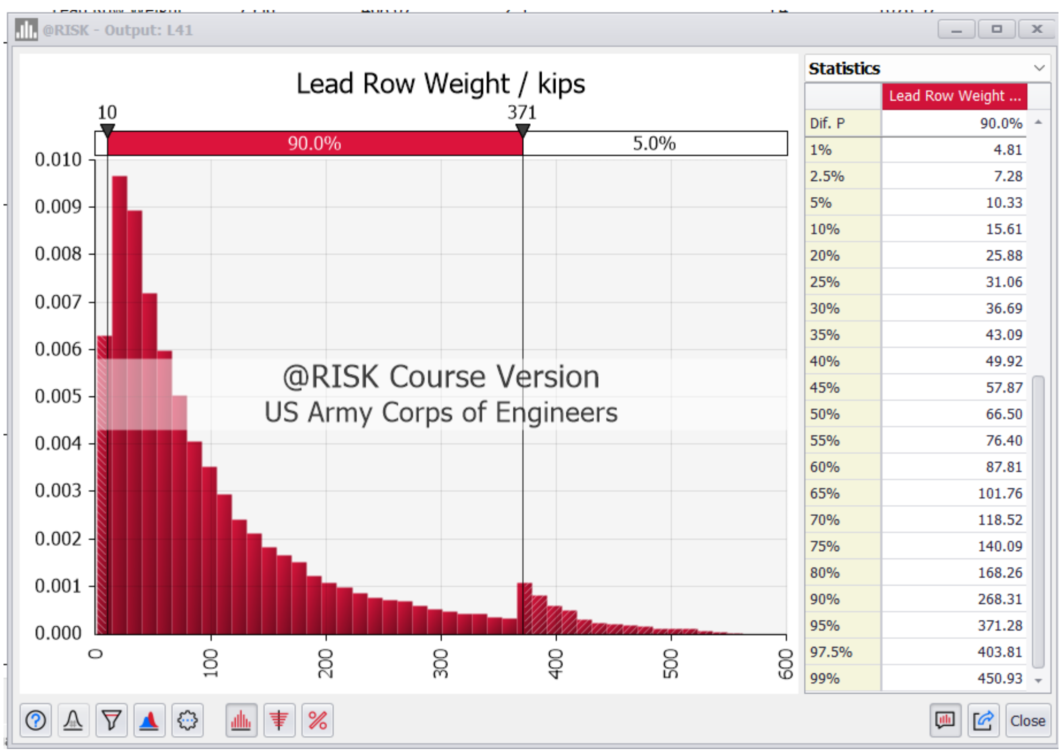


*Figure A-33. @Risk Output Impact Force Distribution for 12-FT Draft*

## 3.2 CONCRETE COMPONENT CAPACITY CHECKS

The PBIA impact results shown above in Table A-13 were used to check the capacity of each concrete component of the guide wall: skin panel, girders, and columns. The forces were input into a STAAD Model as concentrated point loads to obtain forces acting in each member and then checked for flexural and shear capacity. The skin panel was assumed to not carry shear and was only checked for flexural capacity. As-built drawings were used to determine concrete and reinforcement properties for each component.

### 3.2.1 Skin Panel Capacity Check

Figure A-34 through Figure A-36 show the calculations to check the concrete capacity of the skin panel. The flexural design strength was calculated and compared to the ultimate moments resulting from the PBIA forces applied to the STAAD model (Figure A-35). The 9-FT versus 12-FT comparison is shown in Figure A-36 with unity check included to visualize the capacity and how it changed from increased draft.

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**JBJWW 12 FT CHANNEL FEAS. STUDY**
**SKIN PANEL CONCRETE CHECKS**

Designed By: RSJ

**SKIN PANEL - DESIGN DATA**

| | Input |
|---|---|
| Concrete Strength | $f'_c := 3\ ksi$ |
| Reinforcement Yield Strength | $f_y := 40000\ psi$ |
| Design Section Width | $b := 12\ in$ |
| Panel Depth | $h := 36\ in$ |
| Concrete Cover | $cover := 4\ in$ |
| Steel Modulus of Elasticity | $E_s := 29000\ ksi$ |
| B1 Value | $\beta_1 := 0.85$ |

**AREA OF FLEXURAL STEEL**

$d_b := 1.00\ in$ Bar Diameter

$n := 1$ Number Bars

$$d := h - cover - \frac{d_b}{2} = 31.5\ in$$

$$A_b := \frac{\pi \cdot d_b^{\ 2}}{4} = 0.785\ in^2$$

$$A_s := A_b \cdot n = 0.785\ in^2$$

**REDUCTION FACTOR**

$$\varepsilon_{ty} := \frac{f_y}{E_s} = 0.001 \qquad a := \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = 1.027\ in$$

$$c := \frac{a}{\beta_1} = 1.208\ in \qquad \varepsilon_t := 0.003 \cdot \left(\frac{d - c}{c}\right) = 0.075$$

*Figure A-34. Skin Panel Concrete Check Calculations (1/3)*

US Army Corps of Engineers
Vicksburg District

**JBJWW 12 FT CHANNEL FEAS. STUDY**
**SKIN PANEL CONCRETE CHECKS**

Designed By: RSJ

$$\phi_b := \begin{cases} 0.75 & \text{if } \varepsilon_t \le \varepsilon_{ty} \\ 0.9 & \text{else if } \varepsilon_t \ge \varepsilon_{ty} \\ 0.65 + 0.25 \cdot \dfrac{(\varepsilon_t - \varepsilon_{ty})}{0.003} & \text{else} \end{cases} = 0.9$$

## FLEXURAL STRENGTH OF SKIN PANEL

$$M_n := f_y \cdot A_s \cdot \left(d - \frac{a}{2}\right) = 81.12\ kip \cdot ft$$

$$\phi M_n := \phi_b \cdot M_n = 73.01\ kip \cdot ft$$

## ULTIMATE MOMENT - 9FT CHANNEL

Shear and moment stress values are pulled from STAAD model and analyzed per foot basis of panel. Load factors are from EM 3402.

$$M_{u9.usual} := 2.2 \cdot (24\ kip \cdot ft) = 52.8\ kip \cdot ft$$

$$M_{u9.unusual} := 1.6 \cdot (65\ kip \cdot ft) = 104\ kip \cdot ft$$

$$M_{u9.extreme} := 1.3 \cdot (89\ kip \cdot ft) = 115.7\ kip \cdot ft$$

## ULTIMATE MOMENT - 12FT CHANNEL

Shear and moment stress values are pulled from STAAD model and analyzed per foot basis of panel. Load factors are from EM 3402.

$$M_{u12.usual} := 2.2 \cdot (29\ kip \cdot ft) = 63.8\ kip \cdot ft$$

$$M_{u12.unusual} := 1.6 \cdot (81\ kip \cdot ft) = 129.6\ kip \cdot ft$$

$$M_{u12.extreme} := 1.3 \cdot (98\ kip \cdot ft) = 127.4\ kip \cdot ft$$

*Figure A-35. Skin Panel Concrete Check Calculations cont. (2/3)*

US Army Corps of Engineers
Vicksburg District

**JBJWW 12 FT CHANNEL FEAS. STUDY**
**SKIN PANEL CONCRETE CHECKS**

Designed By: RSJ

**DESIGN CHECKS ("Ch") - 9FT CHANNEL (left) vs 12FT CHANNEL (right)**

Utilization ratio shown below each check to compare changes in effects of drafts.

$$Ch_{9.M1} := \text{if } \phi M_n > M_{u9.usual} \;\|\; \text{"OK"} \;\text{else}\; \|\; \text{"NOT OK"} = \text{"OK"}$$

$$Ch_{12.M1} := \text{if } \phi M_n > M_{u12.usual} \;\|\; \text{"OK"} \;\text{else}\; \|\; \text{"NOT OK"} = \text{"OK"}$$

$$\frac{M_{u9.usual}}{\phi M_n} = 0.72 \qquad \frac{M_{u12.usual}}{\phi M_n} = 0.87$$

$$Ch_{9.M2} := \text{if } \phi M_n > M_{u9.unusual} \;\|\; \text{"OK"} \;\text{else}\; \|\; \text{"NOT OK"} = \text{"NOT OK"}$$

$$Ch_{12.M2} := \text{if } \phi M_n > M_{u12.unusual} \;\|\; \text{"OK"} \;\text{else}\; \|\; \text{"NOT OK"} = \text{"NOT OK"}$$

$$\frac{M_{u9.unusual}}{\phi M_n} = 1.42 \qquad \frac{M_{u12.unusual}}{\phi M_n} = 1.78$$

$$Ch_{9.M3} := \text{if } \phi M_n > M_{u9.extreme} \;\|\; \text{"OK"} \;\text{else}\; \|\; \text{"NOT OK"} = \text{"NOT OK"}$$

$$Ch_{12.M3} := \text{if } \phi M_n > M_{u12.extreme} \;\|\; \text{"OK"} \;\text{else}\; \|\; \text{"NOT OK"} = \text{"NOT OK"}$$

$$\frac{M_{u9.extreme}}{\phi M_n} = 1.58 \qquad \frac{M_{u12.extreme}}{\phi M_n} = 1.74$$

*Figure A-36. Skin Panel Concrete Check Calculations cont. (3/3)*

### 3.2.2 Girder Capacity Check

Figure A-37 through Figure A-40 show the calculations to check the concrete capacity of the girders. The shear and flexural design strengths were calculated and compared to the ultimate shear and moments resulting from the PBIA forces applied to the STAAD model (Figure A-39). The 9-FT versus 12-FT comparison is shown in Figure A-39 and Figure A-40 with unity check included to visualize the capacity and how it changed from increased draft.

US Army Corps of Engineers
Vicksburg District

**JBJWW 12 FT CHANNEL FEAS. STUDY**
**GIRDER CONCRETE CHECKS**

Designed By: RSJ

## GIRDER - DESIGN DATA

| | Input |
|---|---|
| Concrete Strength | $f'_c := 3\ ksi$ |
| Reinforcement Yield Strength | $f_y := 40000\ psi$ |
| Design Section Width | $b := 60\ in$ |
| Beam Depth | $h := 102\ in$ |
| Concrete Cover | $cover := 4\ in$ |
| Steel Modulus of Elasticity | $E_s := 29000\ ksi$ |
| B1 Value | $\beta_1 := 0.85$ |

## AREA OF FLEXURAL STEEL

$d_b := 1.128\ in$ Bar Diameter

$n := 5$ Number Bars

$$d := h - cover - \frac{d_b}{2} - 1.128\ in - 1.0\ in = 95.308\ in$$

$$A_b := \frac{\pi \cdot d_b^{\ 2}}{4} = 0.999\ in^2$$

$$A_s := A_b \cdot n = 4.997\ in^2$$

## REDUCTION FACTOR

$$\varepsilon_{ty} := \frac{f_y}{E_s} = 0.001 \qquad a := \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = 1.306\ in$$

$$c := \frac{a}{\beta_1} = 1.537\ in \qquad \varepsilon_t := 0.003 \cdot \left(\frac{d - c}{c}\right) = 0.183$$

*Figure A-37. Girder Concrete Check Calculations (1/4)*

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JBJWW 12 FT CHANNEL FEAS. STUDY
GIRDER CONCRETE CHECKS

Designed By: RSJ

$$\phi_b := \text{if } \varepsilon_t \le \varepsilon_{ty} \;\Vert\; 0.75 \;\; \text{else if } \varepsilon_t \ge \varepsilon_{ty} \;\Vert\; 0.9 \;\; \text{else} \;\Vert\; 0.65 + 0.25 \cdot \frac{(\varepsilon_t - \varepsilon_{ty})}{0.003} \quad = 0.9$$

**FLEXURAL STRENGTH OF GIRDER**

$$M_n := f_y \cdot A_s \cdot \left(d - \frac{a}{2}\right) = 1576.52 \; kip \cdot ft$$

$$\phi M_n := \phi_b \cdot M_n = 1418.87 \; kip \cdot ft$$

**SHEAR STRENGTH OF CONCRETE**

$\phi_v := 0.75$ $\lambda := 1$

$$N_u := 0 \; lbf$$

$$A_g := b \cdot d = 5718.48 \; in^2$$

$$V_c := \left(2 \cdot \sqrt{f'_c \cdot psi} + \frac{N_u}{6 \cdot A_g}\right) \cdot b \cdot d = 626.43 \; kip$$

$$\phi V_c := \phi_v \cdot V_c = 469.82 \; kip$$

**SHEAR STRENGTH OF REINFORCEMENT**

#6 rebar @ 10" spacing

$$d_{bv} := 0.75 \; in$$

$$s_v := 10 \; in$$

$$n_v := 2$$

$$A_{bv} := \frac{\pi \cdot {d_{bv}}^2}{4} \cdot n_v = 0.884 \; in^2$$

*Figure A-38. Girder Concrete Check Calculations cont. (2/4)*

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Vicksburg District

JBJWW 12 FT CHANNEL FEAS. STUDY
GIRDER CONCRETE CHECKS

Designed By: RSJ

$$V_s := \frac{A_{bv} \cdot f_y \cdot d}{s_v} = 336.85\ kip$$

**TOTAL SHEAR STRENGTH OF COLUMN**

$$V_n := V_c + V_s = 963.27\ kip$$

$$\phi V_n := \phi_v \cdot V_n = 722.46\ (kip)$$

**ULTIMATE SHEAR AND MOMENT - 9 FT CHANNEL**

Shear and moment values are pulled from STAAD model. Load factors are from EM 3402.

$V_{u9.usual} := 2.2 \cdot (49\ kip) = 107.8\ kip$ $\quad$ $M_{u9.usual} := 2.2 \cdot (503\ kip \cdot ft) = 1106.6\ kip \cdot ft$

$V_{u9.unusual} := 1.6 \cdot (134\ kip) = 214.4\ kip$ $\quad$ $M_{u9.unusual} := 1.6 \cdot (1390\ kip \cdot ft) = 2224\ kip \cdot ft$

$V_{u9.extreme} := 1.3 \cdot (183\ kip) = 237.9\ kip$ $\quad$ $M_{u9.extreme} := 1.3 \cdot (1890\ kip \cdot ft) = 2457\ kip \cdot ft$

**ULTIMATE SHEAR AND MOMENT - 12 FT CHANNEL**

Shear and moment values are pulled from STAAD model. Load factors are from EM 3402.

$V_{u12.usual} := 2.2 \cdot (60\ kip) = 132\ kip$ $\quad$ $M_{u12.usual} := 2.2 \cdot (623\ kip \cdot ft) = 1370.6\ kip \cdot ft$

$V_{u12.unusual} := 1.6 \cdot (168\ kip) = 268.8\ kip$ $\quad$ $M_{u12.unusual} := 1.6 \cdot (1733\ kip \cdot ft) = 2772.8\ kip \cdot ft$

$V_{u12.extreme} := 1.3 \cdot (202\ kip) = 262.6\ kip$ $\quad$ $M_{u12.extreme} := 1.3 \cdot (2083\ kip \cdot ft) = 2707.9\ kip \cdot ft$

**DESIGN CHECKS ("Ch") - 9FT CHANNEL (left) vs 12FT CHANNEL (right)**

Utilization ratio shown below each check to compare changes in effects of drafts.

$$Ch_{9.M1} := \text{if } \phi M_n > M_{u9.usual} \;\; \| \text{"OK"} \;\; \text{else} \;\; \| \text{"NOT OK"} = \text{"OK"}$$

$$\frac{M_{u9.usual}}{\phi M_n} = 0.78$$

$$Ch_{12.M1} := \text{if } \phi M_n > M_{u12.usual} \;\; \| \text{"OK"} \;\; \text{else} \;\; \| \text{"NOT OK"} = \text{"OK"}$$

$$\frac{M_{u12.usual}}{\phi M_n} = 0.97$$

*Figure A-39. Girder Concrete Check Calculations cont. (3/4)*

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Vicksburg District

**JBJWW 12 FT CHANNEL FEAS. STUDY**
**GIRDER CONCRETE CHECKS**

Designed By: RSJ

$Ch_{9.M2} :=$ if $\phi M_n > M_{u9.unusual}$ ‖ "OK" else ‖ "NOT OK" $=$ "NOT OK"

$Ch_{12.M2} :=$ if $\phi M_n > M_{u12.unusual}$ ‖ "OK" else ‖ "NOT OK" $=$ "NOT OK"

$$\frac{M_{u9.unusual}}{\phi M_n} = 1.57 \qquad \frac{M_{u12.unusual}}{\phi M_n} = 1.95$$

$Ch_{9.M3} :=$ if $\phi M_n > M_{u9.extreme}$ ‖ "OK" else ‖ "NOT OK" $=$ "NOT OK"

$Ch_{12.M3} :=$ if $\phi M_n > M_{u12.extreme}$ ‖ "OK" else ‖ "NOT OK" $=$ "NOT OK"

$$\frac{M_{u9.extreme}}{\phi M_n} = 1.73 \qquad \frac{M_{u12.extreme}}{\phi M_n} = 1.91$$

$Ch_{9.V1} :=$ if $\phi V_n > V_{u9.usual}$ ‖ "OK" else ‖ "NOT OK" $=$ "OK"

$Ch_{12.V1} :=$ if $\phi V_n > V_{u12.usual}$ ‖ "OK" else ‖ "NOT OK" $=$ "OK"

$$\frac{V_{u9.usual}}{\phi V_n} = 0.15 \qquad \frac{V_{u12.usual}}{\phi V_n} = 0.18$$

$Ch_{9.V2} :=$ if $\phi V_n > V_{u9.unusual}$ ‖ "OK" else ‖ "NOT OK" $=$ "OK"

$Ch_{12.V2} :=$ if $\phi V_n > V_{u12.unusual}$ ‖ "OK" else ‖ "NOT OK" $=$ "OK"

$$\frac{V_{u9.unusual}}{\phi V_n} = 0.3 \qquad \frac{V_{u12.unusual}}{\phi V_n} = 0.37$$

$Ch_{9.V3} :=$ if $\phi V_n > V_{u9.extreme}$ ‖ "OK" else ‖ "NOT OK" $=$ "OK"

$Ch_{12.V3} :=$ if $\phi V_n > V_{u12.extreme}$ ‖ "OK" else ‖ "NOT OK" $=$ "OK"

$$\frac{V_{u9.extreme}}{\phi V_n} = 0.33 \qquad \frac{V_{u12.extreme}}{\phi V_n} = 0.36$$

*Figure A-40. Girder Concrete Check Calculations cont. (4/4)*

### 3.2.3 Column Capacity Check

Figure A-41 through Figure A-44 show the calculations to check the concrete capacity of the columns. The shear and flexural design strengths were calculated and compared to the ultimate shear and moments resulting from the PBIA forces applied to the STAAD model (Figure A-43). The 9-FT versus 12-FT comparison is shown in Figure A-43 and Figure A-44 with unity check included to visualize the capacity and how it changed from increased draft.

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**JBJWW 12 FT CHANNEL FEAS. STUDY**
**COLUMN CONCRETE CHECKS**

Designed By: RSJ

**COLUMNS - DESIGN DATA**

| | Input |
|---|---|
| Concrete Strength | $f'_c := 3\ ksi$ |
| Reinforcement Yield Strength | $f_y := 40\ ksi$ |
| Design Section Width | $b := 144\ in$ |
| Column Depth | $h := 220.5\ in$ |
| Concrete Cover | $cover := 4\ in$ |
| Steel Modulus of Elasticity | $E_s := 29000\ ksi$ |
| B1 Value | $\beta_1 := 0.85$ |

**AREA OF FLEXURAL STEEL**

$d_b := 0.75\ in$ Bar Diameter

$n := 12$ Number Bars

$$d := h - cover - \frac{d_b}{2} - 0.75\ in = 215.38\ in$$

$$A_b := \frac{\pi \cdot d_b^{\ 2}}{4} = 0.44\ in^2$$

$$A_s := A_b \cdot n = 5.3\ in^2$$

**REDUCTION FACTOR**

$$\varepsilon_{ty} := \frac{f_y}{E_s} = 0 \qquad a := \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b} = 0.58\ in$$

$$c := \frac{a}{\beta_1} = 0.68\ in \qquad \varepsilon_t := 0.003 \cdot \left(\frac{d - c}{c}\right) = 0.95$$

*Figure A-41. Column Concrete Check Calculations. (1/4)*

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Vicksburg District

**JBJWW 12 FT CHANNEL FEAS. STUDY**
**COLUMN CONCRETE CHECKS**

Designed By: RSJ

$$\phi_b := \text{if } \varepsilon_t \le \varepsilon_{ty} \;\; \| \; 0.75 \;\; \text{else if } \varepsilon_t \ge \varepsilon_{ty} \;\; \| \; 0.9 \;\; \text{else} \;\; \| \; 0.65 + 0.25 \cdot \frac{(\varepsilon_t - \varepsilon_{ty})}{0.003} \quad = 0.9$$

**FLEXURAL STRENGTH OF COLUMN**

$$M_n := f_y \cdot A_s \cdot \left(d - \frac{a}{2}\right) = 3800.89 \; kip \cdot ft$$

$$\phi M_n := \phi_b \cdot M_n = 3420.8 \; kip \cdot ft$$

**SHEAR STRENGTH OF CONCRETE**

$\phi_v := 0.75$ $\lambda := 1$

$N_u := 0 \; lbf$

$A_g := b \cdot d = 31014 \; in^2$

$$V_c := \left(2 \cdot \sqrt{f'_c \cdot psi} + \frac{N_u}{6 \cdot A_g}\right) \cdot b \cdot d = 3397.41 \; kip$$

$$\phi V_c := \phi_v \cdot V_c = 2548.06 \; kip$$

**SHEAR STRENGTH OF REINFORCEMENT**

#6 rebar @ 12" spacing

$d_{bv} := 0.75 \; in$

$s_v := 12 \; in$

$n_v := 2$

$$A_{bv} := \frac{\pi \cdot d_{bv}^{\;2}}{4} \cdot n_v = 0.884 \; in^2$$

*Figure A-42. Column Concrete Check Calculations cont. (2/4)*

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**JBJWW 12 FT CHANNEL FEAS. STUDY**
**COLUMN CONCRETE CHECKS**

Designed By: RSJ

$$V_s := \frac{A_{bv} \cdot f_y \cdot d}{s_v} = 634.33\ kip$$

**TOTAL SHEAR STRENGTH OF COLUMN**

$$V_n := V_c + V_s = 4031.75\ kip$$

$$\phi V_n := \phi_v \cdot V_n = 3023.81\ (kip)$$

**ULTIMATE SHEAR AND MOMENT - 9 FT CHANNEL**

Shear and moment values are pulled from STAAD model. Load factors are from EM 3402.

$V_{u9.usual} := 2.2 \cdot (108\ kip) = 237.6\ kip$ $M_{u9.usual} := 2.2 \cdot (1150\ kip \cdot ft) = 2530\ kip \cdot ft$

$V_{u9.unusual} := 1.6 \cdot (288\ kip) = 460.8\ kip$ $M_{u9.unusual} := 1.6 \cdot (7614\ kip \cdot ft) = 12182.4\ kip \cdot ft$

$V_{u9.extreme} := 1.3 \cdot (377\ kip) = 490.1\ kip$ $M_{u9.extreme} := 1.3 \cdot (16208\ kip \cdot ft) = 21070.4\ kip \cdot ft$

**ULTIMATE SHEAR AND MOMENT - 12 FT CHANNEL**

Shear and moment values are pulled from STAAD model. Load factors are from EM 3402.

$V_{u12.usual} := 2.2 \cdot (133\ kip) = 292.6\ kip$ $M_{u12.usual} := 2.2 \cdot (1424\ kip \cdot ft) = 3132.8\ kip \cdot ft$

$V_{u12.unusual} := 1.6 \cdot (358\ kip) = 572.8\ kip$ $M_{u12.unusual} := 1.6 \cdot (9486\ kip \cdot ft) = 15177.6\ kip \cdot ft$

$V_{u12.extreme} := 1.3 \cdot (415\ kip) = 539.5\ kip$ $M_{u12.extreme} := 1.3 \cdot (17872\ kip \cdot ft) = 23233.6\ kip \cdot ft$

**DESIGN CHECKS ("Ch") - 9FT CHANNEL (left) vs 12FT CHANNEL (right)**

Utilization ratio shown below each check to compare changes in effects of drafts.

$Ch_{9.M1} :=$ if $\phi M_n > M_{u9.usual}$ = "OK"
‖ "OK"
else
‖ "NOT OK"

$$\frac{M_{u9.usual}}{\phi M_n} = 0.74$$

$Ch_{12.M1} :=$ if $\phi M_n > M_{u12.usual}$ = "OK"
‖ "OK"
else
‖ "NOT OK"

$$\frac{M_{u12.usual}}{\phi M_n} = 0.92$$

*Figure A-43. Column Concrete Check Calculations cont. (3/4)*

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**JBJWW 12 FT CHANNEL FEAS. STUDY**
**COLUMN CONCRETE CHECKS**

Designed By: RSJ

$$Ch_{9.M2} := \begin{cases} \text{"OK"} & \text{if } \phi M_n > M_{u9.unusual} \\ \text{"NOT OK"} & \text{else} \end{cases} = \text{"NOT OK"}$$

$$\frac{M_{u9.unusual}}{\phi M_n} = 3.56$$

$$Ch_{12.M2} := \begin{cases} \text{"OK"} & \text{if } \phi M_n > M_{u12.unusual} \\ \text{"NOT OK"} & \text{else} \end{cases} = \text{"NOT OK"}$$

$$\frac{M_{u12.unusual}}{\phi M_n} = 4.44$$

$$Ch_{9.M3} := \begin{cases} \text{"OK"} & \text{if } \phi M_n > M_{u9.extreme} \\ \text{"NOT OK"} & \text{else} \end{cases} = \text{"NOT OK"}$$

$$\frac{M_{u9.extreme}}{\phi M_n} = 6.16$$

$$Ch_{12.M3} := \begin{cases} \text{"OK"} & \text{if } \phi M_n > M_{u12.extreme} \\ \text{"NOT OK"} & \text{else} \end{cases} = \text{"NOT OK"}$$

$$\frac{M_{u12.extreme}}{\phi M_n} = 6.79$$

$$Ch_{9.V1} := \begin{cases} \text{"OK"} & \text{if } \phi V_n > V_{u9.usual} \\ \text{"NOT OK"} & \text{else} \end{cases} = \text{"OK"}$$

$$\frac{V_{u9.usual}}{\phi V_n} = 0.08$$

$$Ch_{12.V1} := \begin{cases} \text{"OK"} & \text{if } \phi V_n > V_{u12.usual} \\ \text{"NOT OK"} & \text{else} \end{cases} = \text{"OK"}$$

$$\frac{V_{u12.usual}}{\phi V_n} = 0.1$$

$$Ch_{9.V2} := \begin{cases} \text{"OK"} & \text{if } \phi V_n > V_{u9.unusual} \\ \text{"NOT OK"} & \text{else} \end{cases} = \text{"OK"}$$

$$\frac{V_{u9.unusual}}{\phi V_n} = 0.15$$

$$Ch_{12.V2} := \begin{cases} \text{"OK"} & \text{if } \phi V_n > V_{u12.unusual} \\ \text{"NOT OK"} & \text{else} \end{cases} = \text{"OK"}$$

$$\frac{V_{u12.unusual}}{\phi V_n} = 0.19$$

$$Ch_{9.V3} := \begin{cases} \text{"OK"} & \text{if } \phi V_n > V_{u9.extreme} \\ \text{"NOT OK"} & \text{else} \end{cases} = \text{"OK"}$$

$$\frac{V_{u9.extreme}}{\phi V_n} = 0.16$$

$$Ch_{12.V3} := \begin{cases} \text{"OK"} & \text{if } \phi V_n > V_{u12.extreme} \\ \text{"NOT OK"} & \text{else} \end{cases} = \text{"OK"}$$

$$\frac{V_{u12.extreme}}{\phi V_n} = 0.18$$

*Figure A-44. Column Concrete Check Calculations cont. (4/4)*

### 3.2.4 Concrete Capacity Summary

The draft impact force results on the concrete guide wall show that the structure does not fully meet current design requirements in either the existing or proposed condition. The utilization ratios show slight increases in the structure's capacity to withstand the forces. Comparing the existing 9-FT traffic to new 12-FT traffic, there is as much risk of barge impact damage in the existing condition as with the proposed 12-FT traffic.

## 3.3 PILE ANALYSIS

The pile analysis was performed using Ensoft GROUP software to perform stress checks and unity checks for the existing 9-FT draft forces versus the new 12-FT draft forces according to EM 1110-2-2906.

The initial PBIA results were used and recorded in Figure A-45 and Figure A-46 under the "2.3 ft/s (Actual)" column. The compression Factor of Safety (FOS) did not meet EM 2906 minimum requirements for the existing 9-FT forces and for the 12-FT forces but the met unity check and tension FOS.

In efforts to find conditions for which the compression FOS passes, the analysis was performed at various speeds in attempts to observe effects of restricting speed to minimize impact forces. However, even at slower speeds the results rarely met minimum FOS for compression.

### 3.3.1 9-FT Channel Pile Analysis

Figure A-45 shows a summary of the pile analysis for a 9-FT channel. Required FOSs are taken from the EM 2906, Chapter 4 Design Criteria, table with pile driving analyzer used. The "Barge Impact Loading" section shows impact forces from PBIA for a specific controlled speed. The compression axial FOS shows that the majority of load cases fail to meet minimum requirements in the existing condition, but never exceed the unity check, meaning the stress is never fully exceeded.

| 9 FT CHANNEL | | | | | | |
|---|---|---|---|---|---|---|
| | | | | | | |
| BARGE IMPACT LOADING (KIPS) | | | | | | |
| | 1.0 ft/s | 1.5 ft/s | 2.0 ft/s | 2.3 ft/s (Actual) | 2.5 ft/s | Confidence Interval |
| Usual | 47.00 | 70.00 | 94.00 | 109.00 | 117.00 | 75% |
| Unusual | 131.00 | 196.00 | 261.00 | 301.00 | 329.00 | 95% |
| Extreme | 235.00 | 364.00 | 391.00 | 409.00 | 422.00 | 99% |
| | | | | | | |
| AXIAL FOS (COMPRESSION) | | | | | | |
| | 1.0 ft/s | 1.5 ft/s | 2.0 ft/s | 2.3 ft/s (Actual) | 2.5 ft/s | Required FOS (C) |
| Usual | 2.38 | 2.35 | 2.30 | 2.28 | 2.25 | 2.50 |
| Unusual | 2.37 | 1.93 | 1.63 | 1.49 | 1.40 | 1.90 |
| Extreme | 1.81 | 1.30 | 1.23 | 1.19 | 1.16 | 1.40 |
| | | | | | | |
| AXIAL FOS (TENSION) | | | | | | |
| | 1.0 ft/s | 1.5 ft/s | 2.0 ft/s | 2.3 ft/s (Actual) | 2.5 ft/s | Required FOS (T) |
| Usual | N/A | N/A | N/A | N/A | N/A | 3.00 |
| Unusual | N/A | N/A | N/A | N/A | N/A | 2.25 |
| Extreme | N/A | 8.63 | 6.65 | 5.77 | 5.26 | 1.70 |
| | | | | | N/A=No tension in piles | |
| | | | | | | |
| PILE STRESS UNITY CHECK | | | | | | |
| | 1.0 ft/s | 1.5 ft/s | 2.0 ft/s | 2.3 ft/s (Actual) | 2.5 ft/s | Required Unity |
| Usual | 0.44 | 0.47 | 0.49 | 0.51 | 0.52 | |
| Unusual | 0.37 | 0.46 | 0.55 | 0.61 | 0.65 | 1.00 |
| Extreme | 0.38 | 0.54 | 0.58 | 0.60 | 0.62 | |

*Figure A-45. 9-FT Channel Pile Analysis Results*

### 3.3.2 12-FT Channel Pile Analysis

Figure A-46 shows a summary of the pile analysis for a 12-FT channel. Required FOSs are taken from the EM 2906, Chapter 4 Design Criteria, table with pile driving analyzer used. The “Barge Impact Loading” section shows impact forces from PBIA for a specific controlled speed. The compression axial FOS shows that the majority of load cases fail to meet minimum requirements in the existing condition, but never exceed the unity check, meaning the stress is never fully exceeded.

| 12 FT CHANNEL | | | | | | |
|---|---|---|---|---|---|---|
| | | | | | | |
| | | | BARGE IMPACT LOADING (KIPS) | | | |
| | 1.0 ft/s | 1.5 ft/s | 2.0 ft/s | 2.3 ft/s (Actual) | 2.5 ft/s | Confidence Interval |
| Usual | 60.00 | 91.00 | 121.00 | 135.00 | 152.00 | 75% |
| Unusual | 171.00 | 255.00 | 342.00 | 375.00 | 381.00 | 95% |
| Extreme | 315.00 | 391.00 | 429.00 | 451.00 | 463.00 | 99% |
| | | | | | | |
| | | | AXIAL FOS (COMPRESSION) | | | |
| | 1.0 ft/s | 1.5 ft/s | 2.0 ft/s | 2.3 ft/s (Actual) | 2.5 ft/s | Required FOS (C) |
| Usual | 2.36 | 2.31 | 2.22 | 2.14 | 2.04 | 2.50 |
| Unusual | 2.08 | 1.65 | 1.37 | 1.28 | 1.27 | 1.90 |
| Extreme | 1.46 | 1.23 | 1.14 | 1.09 | 1.07 | 1.40 |
| | | | | | | |
| | | | AXIAL FOS (TENSION) | | | |
| | 1.0 ft/s | 1.5 ft/s | 2.0 ft/s | 2.3 ft/s (Actual) | 2.5 ft/s | Required FOS (T) |
| Usual | N/A | N/A | N/A | N/A | N/A | 3.00 |
| Unusual | N/A | N/A | N/A | N/A | N/A | 2.25 |
| Extreme | 18.72 | 6.65 | 5.03 | 4.47 | 4.21 | 1.70 |
| | | | | | N/A=No tension in piles | |
| | | | | | | |
| | | | PILE STRESS UNITY CHECK | | | |
| | 1.0 ft/s | 1.5 ft/s | 2.0 ft/s | 2.3 ft/s (Actual) | 2.5 ft/s | Required Unity |
| Usual | 0.46 | 0.49 | 0.52 | 0.55 | 0.57 | |
| Unusual | 0.43 | 0.54 | 0.68 | 0.74 | 0.75 | 1.00 |
| Extreme | 0.48 | 0.58 | 0.63 | 0.66 | 0.68 | |

*Figure A-46. 12-FT Channel Pile Analysis Results*

### 3.3.3 Pile Analysis Summary

*Table A-14. Pile Analysis Results – Axial FOS (Compression)*

| EM 2906 Event | Required FOS | Axial FOS (C) - 9-FT Channel | Axial FOS (C) - 12-FT Channel |
|---|---|---|---|
| Usual | 2.5 | 2.28 | 2.14 |
| Unusual | 1.9 | 1.49 | 1.28 |
| Extreme | 1.4 | 1.19 | 1.09 |

*Table A-15. Pile Analysis Results – Axial FOS (Tension)*

| EM 2906 Event | Required FOS | Axial FOS (T) - 9-FT Channel | Axial FOS (T) - 12-FT Channel |
|---|---|---|---|
| Usual | 2.5 | N/A | N/A |
| Unusual | 1.9 | N/A | N/A |
| Extreme | 1.4 | 5.57 | 4.47 |

Note: N/A means no tension forces were present in loading.

*Table A-16. Pile Analysis Results – Unity Checks*

| EM 2906 Event | Required FOS | Unity - 9-FT Channel | Unity - 12-FT Channel |
|---|---|---|---|
| Usual | 1.0 | 0.51 | 0.55 |
| Unusual | 1.0 | 0.61 | 0.77 |
| Extreme | 1.0 | 0.60 | 0.66 |

The results of the pile analysis show that the existing 9-FT draft and 12-FT draft rarely meet minimum compression FOS requirements per EM 2906 at a controlled speed of only 1.0 feet per second. These results indicate that the stresses are within the piles' capacity but that the FOSs are not reached. However, when looking at the FOS and unity checks, the results show that there is no significant increase in stress acting on the piles. Since they behave similarly, there is no more risk for a 12-FT draft channel than expected for the current operation with a 9-FT draft.

## 3.4 STRUCTURAL ASSESSMENT CONCLUSION

The original structure was designed to a 120-kip impact loading. The 12-FT PBIA shows an increase to 135-kip impact load, and the results of the design checks show that this does not have a significant effect on the stability of the structure. The PBIA produced unusual and extreme impact loads that exceed the original design and that the structure cannot sustain impacts from. However, the guide wall behaves similarly in all checks for a 9-FT versus 12-FT channel. The guide walls are in good condition, with no reported issues. There have also been very few barge impacts along this river system. These factors combined indicate there is no additional risk to the structure with heavier vessels navigating a 12-FT channel, so no modifications to the guide wall structure are recommended.

# SECTION 4
# HYDRAULIC AND HYDROLOGIC ASSESSMENT

## 4.1 GENERAL

### 4.1.1 Scope of Work

Hydrology and hydraulics investigations were performed to assess the feasibility of providing a 12-FT depth of navigational channel in the JBJ Waterway. Much of the background and existing conditions related to Hydraulics and hydrology was previously described in Section 1 of this report.

The primary purpose of this report is to provide an overview and background of the Hydraulic and hydrologic conditions of the Red River and the JBJ Waterway, and to identify potential problem areas within the waterway regarding the availability of a 12-FT channel. Dredge records and existing Red River 1D/2D HEC-RAS models are used in this process. Noted that the main river channel in the model is mostly captured by 1D cross-sections with the overbank and overland flow areas captured by 2D areas, which are generally not a focus of this study. The identified problem areas to be prioritized as high, medium, or low priority to allow for a more systematic approach in creating possible solutions for providing the 12-FT channel throughout the waterway. In the Feasibility phase of the study, it is expected that site-specific 2D HEC-RAS modeling will be performed to provide additional numerical results to illustrate and support the feasibility of the Tentatively Selected Plan (TSP).

The scope of work includes the following:

1. Collecting background documentation.
    - A general Red River Basin and JBJ Waterway documentation summary.
    - A literature review of pertinent Red River and JBJ Waterway existing studies and original design documentation. The literature review is to be summarized in the background section of the report.
    - Discussions amongst a group experienced in river engineering that includes USACE Vicksburg District hydraulics, river stabilization, and Mississippi River channel improvement engineers and ERDC CHL employees.
2. Summarizing dredge records.
    - Gathering and summarizing dredge records from Vicksburg District River Operations branch along with discussions amongst the personnel.

- Using the dredge records to inform the 12-FT channel study about existing problem areas regarding insufficient channel depths for the existing 9-FT channel authorizations.
- This scope of work is primarily focused on assessing or mitigating for the dredging that occurs and/or may occur within the navigation channel, and not at the locks and dams.

3. Determining the conditions of existing river training structures.

   - Summarizing the existing conditions of the river training structures along the JBJ Waterway such as dikes and revetments.
   - Input about the existing conditions of channel improvement is needed from the Design Branch River Stabilization Section. Design Branch conducts annual evaluations of the JBJ Waterway structures.
   - The River Stabilization Section prepared a 2023 Red River Priority Repair List, which documents the construction or maintenance of revetments and dikes. This work is necessary to continue to provide the most reliable navigation project along with providing for flood risk management such as protecting the integrity of levees against the meandering river.
   - Greater detail can be found in the River Training Structure Conditions section of this appendix.

4. Summarizing lock and dam information and the 1.5x draft requirement over miter gate sills.

   - A summary of the physical characteristics and general operation plans of the locks and dams.
   - Per EM 1110-2-1604, navigation waterways are recommended to have locks that provide a depth of water above the miter gate sills of equal to or greater than 1.5x the authorized draft for vessels to safely enter and exit lock chambers. A 12-FT channel would therefore need 18 feet of depth above the miter gate sills for commercial traffic to safely enter and exit the lock chamber. The depth is the difference between the miter gate sill elevation documented in original design documentation and the water surface elevation.
   - A stage frequency analysis to inform the percentage of time certain depths are present at the lock and dam headwater and tailwaters. Conducting the stage frequency analyses at locks and dams informs both the durations of the given depths, or the percentage of time that given depths are available on an annual and quarterly basis.

5. Assessing navigation channel depths via hydraulic modeling.

   - Utilizing existing Red River HEC-RAS models to simulate a low-flow calibration event for model validity and for project design conditions. The project design conditions are considered to be normal pool conditions at low flows (98 percent duration exceedance probability (DEP) inflows) and will be used to generate

existing conditions channel depth maps. This is consistent with project design conditions noted in existing lock and dam water control manuals and similar to Mississippi River practices for determining low water reference planes noting that low water reference planes for a freely flowing river are not necessarily the same as a controlled or pooled river system.
- Utilizing channel depth maps (RAS depth grids), ArcGIS, and JBJ Waterway project layers such as a navigation track centerline, navigation channel boundary polygon (200-FT channel), and dikes and revetments layers to identify reaches that illustrate to have inadequate depths for the 12-FT deep by 200-FT wide navigation channel.
- The HEC-RAS channel depth maps will first be used to assess the locations that are documented within the dredge records to provide validity about the hydraulic model's ability to show that these locations do indeed have inadequate depths based on the inputs and outputs of the model.
- Following the assessment of known dredge locations, the entire waterway will be scanned to identify other potential problem reaches relating to depths across the 200-FT wide navigation channel.
- The channel depths assessments will help to inform the team about prioritizing specific locations to provide for a more systematic approach and effectively develop the TSP.

6. Performing 2012 multi-beam versus 2016 single-beam channel and thalweg comparisons.

- Provide a longitudinal profile view of the approximate changes in navigation channel thalweg between the collection of the 2012 multi-beam surveys and the 2016 single-beam surveys.
- Notably, the Red River experienced average low and high flows between the completion of the locks and dams in 1995 and 2012. However, in 2015 and 2016, the Red River experienced two historical flood flow events that generated an energy through the system that had not been seen since the flood of 1990. Therefore, a comparison of the 2012 and 2016 survey data provides an illustration of how the channel may have responded to the 2015 and 2016 floods by comparing the 2016 survey data to the 2012 data.
- Upon identifying the problem reaches, cross-section comparisons between the 2012 multi-beam and 2016 single-beam surveys will be made and plotted versus the normal pool elevations (or minimum elevations for the reach below L&D 1) within the problem reaches to show channel bottom elevations relative to the normal pool elevation. This will provide some generalized cross-sectional view of channel depths at specific locations.
- Additional thalweg and channel cross-section comparisons between 2016 and 1981 hydrographic survey, which serves as a pre-project and pre-dike condition.

#### 4.1.1.1 General Limitations and Considerations

Although the known limitations have impacts to varying degrees, the combination of historical dredge records, existing channel surveys (2016 single-beam and 2012 multi-beam), existing hydraulic data and hydraulic model, original design documentation, and dike design experience within the District, provide for a level of river training design analysis that is consistent with ECB 2023-9 (Policy Guidelines for Determining the 35% Design for River Training Structures) within this phase of the study.

Project Scope and Data Collection:

- The study area comprises 212.0 RMs. Collecting new surveys would require large amounts of Right of Entry and large expenditures of time and funding. This limits accuracy of calculated quantities and costs; however, existing survey data are considered sufficient for preliminary (35 percent) dike design as there is a very high likelihood that existing conditions would change between early design and construction. This is consistent with ECB 2023-9. New survey data will be obtained for construction-level designs.
- The initial effort includes the utilization of an existing and calibrated 1D/2D HEC-RAS model. The model was originally developed under the Modeling, Mapping, and Consequences Production Center (MMC) Corps Water Management Systems (CWMS) program, and with additional updates completed in 2023 for the Red River Flowline Analysis. The 1D model is not suitable to characterize complex flow patterns such as local velocities around dikes and bends; therefore, it is not used for dike analysis or design. It is suitable for computing water surface elevations and channel depths, subject to its inputs and calibration performance, within the Feasibility level of design. Therefore, the model is used to illustrate available channel depths during a low flow normal pool under existing conditions. The model is comprised of 1D cross-sections in the river channel representing 2016 single-beam surveys and 2D overland flow areas representing the adjacent floodplain utilizing 2018 LiDAR from bank to levee. The lack of more recent channel bathymetry is a considerable limitation; however, the model aligns with historical dredge records regarding the identification of insufficient channel depths at known problem areas. This agreement provides confidence in the models' ability to identify potentially new channel depth issues related to a 12-FT channel under existing conditions while not neglecting the uncertainties in using nearly decade old single-beam bathymetry. The identification of channel depth issues allows for the Project Delivery Team (PDT) to prioritize and conceptualize river engineering solutions. The accuracy of the channel depths varies, as single-beam data are highly reliable at the point of collection (cross-sectional survey) but contain uncertainties between the cross-sections due to interpolation methods between points. To this point, 2012 multi-beam data were utilized to compare to 2016 single-beam primarily from a conceptual point of view at channel thalwegs and graphically visualizing channel bed layouts. There is no obvious, consistent trend in elevation differences throughout the waterway, but the general shape and

picture of the river bottoms are consistent, such as deeper and shallower segments. At some points, the 2012 data have a higher bed, and at some other points, the 2016 data have a higher bed. Notably, 2016 data were collected following two historic flood years in 2015 and 2016. The river is generally scoured out during such floods for some period of time. Following a high flow scouring period, the river typically seems to revert back to its mean channel condition through sediment deposition over time. While the river is highly dynamic, historic dredge records from 2012 to 2024 have shown that most of the annually and occasionally dredged locations have remained the same over that time frame with no new problem areas arising in recent years. The lack of systematic dredge records for the channel prior to 2012 limits the overall historical viewpoint and knowledge about the river channel performance. The lack of historical channel surveys also limits the knowledge about the channel evolution. Existing specific gage analyses at Shreveport and Alexandria (Red River Hydraulic Analysis at Shreveport) provide insight into the stage to flow relationship over time, particularly for in-channel flows within the context of this navigation focused study. This is a fairly simplistic view of river conveyance but can prove to be valuable in understanding long term evolutions of a channels trend whether it be aggradation, degradation, or dynamic equilibrium. On free flowing navigation channels such as the Lower Mississippi River, trends like this may not be as imperative because the depth and water surface generally shift with the riverbed but under a pooled system with fixed normal pool levels, aggradation and degradation can dramatically impact long term channel depths. In general, the observed trends show major changes in the first five to ten years following the completion of the locks and dams after which the channel may have begun to stabilize. While specific to Shreveport and Alexandria, and subject to long term large scale geomorphic trends, recent evidence suggests the channel bed may fluctuate about a mean between high- and low-flow periods, providing further confidence in the current approach to identify potentially insufficient areas relating to the 12-FT channel depth. Additional detail provided in remaining sections of the report.

2D site-specific dike and sediment modeling:

- Performing detailed modeling with the intent to assess the performance of dikes and revetments generally requires 2D models along with recently collected bathymetry in the channel and in the dike (river training structure) fields. This setup would provide a more accurate depiction of flow conveyance and velocity distributions within the channel and within the dike fields. It is expected that more detailed, site-specific modeling will be performed in the Feasibility phase. The current 1D model is effective at computing water surface elevations and depths given it has satisfactory calibration performance; however, it would not be appropriate for assessing velocities within and around dike fields to be used for design level analysis. 1D models provide a single averaged velocity at cross-sections, neglecting the variation in velocities across a channel. 2D models are better suited for modeling complex flow patterns, and river training structure design level analyses.

- In later phases of the study, site-specific 2D modeling is expected to be completed to support TSP Alternative 3a, which includes improvements to dikes within the identified high-priority problem areas. The model will provide insight into the channel velocities, flow patterns, and shear stresses under both existing and post-project conditions. The strategic improvement of dikes is expected to increase channel velocities and inform the effects on sediment transport but the 2D modeling will help to support those assumptions and decisions. 2012 multi-beam data could be appropriate for this modeling approach to show the incremental changes; however, more recent, site-specific multi-beam data would prove more reliable by representing present day channel conditions.
- Sediment transport modeling is not anticipated to be performed primarily due to data scarcity, and associated uncertainties and complexities. Additionally, the selected alternative (Alternative 3a) is not introducing new dikes to the river but improving existing dikes. The improvement of the dikes was not assumed to dramatically change sediment regimes that would warrant numerical sediment analyses. The U.S. Geological Survey (USGS) has not monitored sediment transport on the Red River since the 1980s; therefore, the observed data are quite dated, and the latest multi-beam data are from 2012. More recent bathymetry would provide a more reliable starting point for bed conditions, although bed conditions are generally ever changing even if fluctuating about a mean.
- In lieu of sediment modeling, historic sediment data will be used to inform the 2D dike modeling by estimating a type of channel-forming discharge (i.e., bankfull, specified recurrence interval, or effective discharge), which can serve as a dike design flow when comparing velocities and flow patterns between existing and post-project conditions. This flow would be a calculated or estimated flow (or range of flows) that most effectively defines the dominant long-term shape of the river channel (channel forming) or is most effective at transporting sediment over time (effective discharge).
- Existing river training structures along the waterway have deteriorated over the last few decades due to limited funding. While many of these dikes and revetments require repairs to be restored to their original design, most of these repairs are not currently limiting navigation of the 9-FT channel. The only emergency repairs required for the 9-FT are located at the Westdale and Joffrion Revetments, which are assumed to be completed for future without project conditions.
- Collection of accurate geotechnical data cannot be requested until the specific locations of dikes are refined using survey data obtained for construction-level design.

### 4.1.2 Historic Red River Reports and Original Design Documentation

This section intends to capture historically significant documentation and original design documentation regarding the Red River and the JBJ Waterway Project. This section does not intend to fully capture the considerable design documentation that is available for the river or the waterway but rather to provide a centrally located summary of such documentation.

#### 4.1.2.1 Shreveport Hydraulic and Geomorphic Analysis – 2020

During the 2015 Red River flood, crest stages in the Shreveport and Bossier areas were 2 to 4 feet higher than expected. Concerns over these elevated stages led to a congressional inquiry that called for an updated analysis of flows on the Red River. The resulting Red River Hydraulic Analysis was conducted from 2017 to 2020 and included the following three main phases:

- Updated 1 percent annual exceedance probability (AEP) flow frequency analysis for the Red River at Shreveport gage performed by the USACE Tulsa District.
- Updated 1 percent AEP water surface elevation profile (flowline) performed by USACE Vicksburg District using HEC-RAS.
- A geomorphic assessment and Adaptive Hydraulics (AdH) modeling performed by the ERDC CHL.

The purpose of the flow frequency analysis and detailed hydraulic modeling of the Red River for the Caddo and Bossier Parish, Louisiana, area was to provide a more accurate and updated flowline (water surface profile). This update was not intended to supersede the 1991 published flowline. The model and results were provided to the Federal Emergency Management Agency (FEMA) to facilitate their update of the 1 percent and 0.2 percent AEP floodplains. These model results changed as part of the most current 2023 update. The geomorphic assessment and AdH modeling were focused on identifying some of the major causes of the increase in stages observed between the 1990 and 2015 flood events. The flow-frequency analysis, conducted by the Tulsa District, determined the 1 percent and 0.2 percent  AEP flows to be 224,000 and 264,000 cfs, respectively. Further discussion about the Tulsa District's flow frequency analysis can be found in Section 5.1.4.2 of this report.

The results of the ERDC analysis indicated an abrupt, upward shift in the stage–discharge relationship of the Red River in the reach between Shreveport and L&D 5 following its construction in 1995. This resulted in the 1 percent AEP water surface elevation profile for this reach shifting upward.

As part of the Red River Hydraulic Analysis of 2020, the employees of ERDC CHL performed a geomorphic assessment that included numerical modeling to provide insight into the observed 2- to 4-foot increase in stages on the Red River at the Shreveport, Louisiana, gage between the 1990 and 2015 flood events. The geomorphic assessment revealed that the river system changed dramatically between 1990 and 2015 with changes in-channel bed elevation, channel area, overbank area, river slope, sedimentation in dike fields, and vegetation growth. L&D 5 has reduced the slope and energy in the river system, inducing deposition and aggradation in the reach upstream of the dam. Evidence suggests that most of the dramatic changes in the channel elevation and stages of the Red River-Shreveport, Louisiana, area occurred during the first five to ten years after the commencement of the operation of L&D 5. Details of the geomorphic assessment can be found in the report titled Geomorphic Assessment and Adaptive Hydraulics Modeling of the Red River Hydraulic Analysis (USACE 2019).

One aspect of the geomorphic assessment was the documentation and plotting of observed stage–discharge curves at key gaged locations along the Red River. Stage–discharge rating curves are a simple but extremely valuable relation. In its simplest form, a stage–discharge rating curve is an XY graph plotting water levels verses discharge. In this case, the ability to add a third variable, time, by color contouring each point allows for a visual representation of the possible shifting of the relation through time.

Trends in stage–discharge relationship shifts are not uniform at locations throughout the Red River System. They are also not uniform at a single gage location for the entire range of flows. For example, the 2019 geomorphic assessment revealed a downward trend in stages in the reaches above Shreveport at Index, Arkansas, and Fulton, Arkansas, for low to mid-midrange flows and an upward trend for higher flows. This contrasts with the upward shift of the river at Shreveport and Coushatta following construction of the lock and dams. In addition, a downward trend was observed for stages in the reach near Alexandria, Louisiana.

#### 4.1.2.2 Red River Flowline Update – 2023

Upon the completion of the 2020 Shreveport Hydraulic and Geomorphic Analysis, the Vicksburg District was funded by FEMA to conduct a cursory update to the Red River 1 and 0.2 percent AEP water surface profiles (commonly referred to as the Flowline Update) throughout the JBJ Waterway portion of the Red River. The analysis extents were from just north of Shreveport, Louisiana, to L&D 1. The update was not intended to supersede the 1991 JBJ Waterway project flowline but instead intended to assist FEMA in updating Base Flood Elevations throughout the project extents and considered best available data. In comparison to the 1991 flowline, the 2023 flowline generally concluded that higher water surface elevations are expected in Pools 3, 4, and 5 and lower water surface elevations are expected in Pools 1 and 2 during 1 and 0.2 percent AEP flood magnitudes. This is consistent with the observed comparative trends between the 1990 flood and the 2015 and 2016 floods.

#### 4.1.2.3 Management of the JBJ Waterway – 2001 Vicksburg District White Paper

In March of 2001, Vicksburg District Hydraulic and Civil Engineers, Freddie Pinkard and Jerry Stuart, attended the Federal Interagency Sedimentation Conference in Reno, Nevada. In preparation for the conference, the engineers authored a white paper titled “The Management of Sediment on the J. Bennett Johnston Waterway.” These river engineers authored the white paper providing a concise overview of the general sediment conditions, channel improvement features (channel realignments, bank stabilization, dikes), recent dredge efforts at that time, and sedimentation concerns around the five locks and dams. This paper has chronological significance, as these engineers were observing the river responses and challenges of managing the waterway for navigation in real time as the

Waterway was essentially completed just 6 years prior to this report with the commencements of the final two locks and dams (L&Ds 4 and 5). Below is a brief summary of the white paper.

The authors wrote that during the last three decades of the 20th century, USACE developed the lower 280 miles of the Red River in Louisiana for commercial navigation. The development included five locks and dams in association with an intensive channel improvement program that included channel realignments, bank stabilization works, and channel contraction. At the time of this 2001 white paper, the waterway had been opened for approximately 6 years, and the channel realignment and bank stabilization program were essentially complete, with only some raising of a few revetments and the construction of some channel control dikes at isolated trouble locations remaining.

Regarding sediment, the authors note that the Red River is a heavily sediment-laden alluvial river with one of the highest sediment concentrations of all major navigable rivers within the U.S. The JBJ Waterway engineers and designers were tasked with the responsibility of developing a system that effectively managed the incoming sediment load, which required a delicate balance of keeping velocities high enough to transport the sediment but low enough not to adversely impact navigation. The channel improvement work at the time had reduced potential sediment problems within the navigation channel and revetments had limited the availability of sediment that historically entered the river through bank caving. Kicker dikes on the downstream end of revetments had resulted in maintenance free crossings and dikes constructed within troublesome depositional reaches had provided the contraction required to insure adequate depths for navigation. Some maintenance dredging had been required at a few isolated locations within the navigation pools but the district had continued to raise revetments and add dikes, which proved to lessen or eliminate costly dredging.

The Red River is a high-energy system characterized by high channel velocities. During high-water event, mean channel velocities often approach 7 feet per second with maximum velocities exceeding 10 feet per second. Combining the high channel velocities and easily erodible banks comprised mostly of fine sand and silt, the result is very active bank caving and lateral migration of hundreds of feet of bankline during high-water events. The primary source of sediment transported on the Red River is said to be sourced from the erosion of unrevetted banks, especially those upstream of Shreveport, with minimal contributions from tributaries.

The channel improvement program of the JBJ Waterway included channel realignments, bank stabilization works, and dikes. Many bendways within the Red River were too tight to be navigated by the channel design tow and so channel cutoffs were developed by the engineers across the necks of old bendways using pilot channel concepts. Within the waterway reach of the Red River, 36 channel realignments were constructed that shortened the river by 50 miles. This shortened the river and increased river slopes. 24 of the realignments resulted in 5,900 acres of oxbow bendways. The bendways were preserved by constructing a non-overtopping closure dam across their upstream ends. The downstream end of the oxbow bendways is left open to allow fish migration and recreational access and interchange of water with the river. Undesirable sediment deposition had occurred at the downstream end of the oxbow bendways. The dredging of a small low water channel was executed, and at most of the bendways, a small natural channel has been maintained without any dredging.

The types of revetments used on the Red River were trenchfill, stonefill, and timber pile revetments. At the sites where the desired bankline alignment was landward of the existing bankline, trenchfill revetment was used. This entailed excavation of a trench along the desired channel alignment and filling the trench with stone. As the bankline continued to erode, the trench was undermined, and the stone in the trench launched down the face of the bank, thus stabilizing the bank to maintain the desired channel alignment. The authors note that trenchfill revetments have proved very effective on high-energy rivers like the Red River that primarily traverse easily erodible soils. At the sites where the desired bankline alignment was located riverward of the existing bankline, stonefill revetment, timber pile, or a combination of the two were used. These types of revetments protect the bank by inducing sediment deposition behind the revetment and thus build the bankline out toward the revetment. In shallower sections along the desired bankline, stonefill revetment was used. In the deeper river sections, time pile, with some stoned placed around the toe of the piling was used. Once sediment deposition had occurred behind these revetments, the revetments were raised or capped out by placing additional stone on top of the deposited sediment along the revetment alignments. This construction procedure resulted in less costly revetment than initially constructing the revetment to its ultimate height.

Another aspect of the channel improvement program were dikes. Dikes generally contract the river channel by redirecting or forcing flow into the main channel to promote self-scouring of the channel. Typically, channel crossings and river bends/meanders are natural sediment deposition locations, particularly on the inside of the bends. The 2001 white paper noted that project design studies determined that to maintain navigation depths, channel widths for a channel forming or design flow discharge, must be limited to 450 feet in the crossings in the upper reaches of the pools where depths are critical and 600 feet throughout the remainder of the pool. To provide the limiting channel crossing widths, kicker dikes are provided on the downstream end of revetments. The dikes are effectively an extension of the revetments and reduce sediment deposition in the crossings by contracting the channel thus keeping velocities high enough to prevent excessive deposition. Once raised to their ultimate height, kicker dikes have provided very effective in maintaining an adequate navigation channel in the crossings on the JBJ Waterway. In the very upper ends of the pools where navigation depths are most critical, structures referred to as additional contraction structures (ACS) have been incorporated. Generally, spur dikes have been used that extend from the convex bank to contract the channel. These spur dikes have proved to be very effective in maintaining a developed navigation channel within the upper most reaches of pools.

Figure A-47 provides dredge quantities by pool from 1989 through 1999 except for 1992, which was omitted because the record was incomplete. The data show that the 9-mile reach below L&D 1 to the mouth of the Black River required considerable dredging, averaging over 1,000,000 cubic yards per year.

The authors of the 2001 white paper report that some dredging had been required at locations within all pools except for Pool 5 primarily because the navigation channel is only maintained as far upstream as the Caddo-Bossier Port at RM 212 or approximately 12 miles upstream of L&D 5. In Pools 1 through 4, varying amounts of dredging were required with the most dredging occurring in the upper end of Pool 3. Within each pool, the dredging was limited to a few isolated locations. Since the Red River was (and still is) highly dynamic, problem sites occasionally developed as the hydraulics and geometry of the channel changed and river engineers designed and constructed channel control features to reduce the problems or deposition. During the early 1990s dredging was required in Pool 1 but was limited to one location at the Vick-Barbin crossing near RM 55. As a result, a kicker dike was constructed off the downstream end of the Vick Revetment and dredging was eliminated at this crossing. In Pool 3, substantial dredging was required in the upper end and as a result, revetments were capped out at Kadesh, Socot, and Campti, and dikes were constructed at Powhatan and downstream of the Highway 6 Bridge at Grand Ecore. In Pool 4, sediment deposition hindered navigation at two locations (Eastpoint and Westdale) within the upper reach of the pool near RM 194 to RM 191. At each of the locations, additional channel control work helped to reduce the need for costly maintenance dredging. As of 2024, these two areas have seemed to require at least occasional dredging during low water periods.

| | | Dredging Quantities (cubic yards) | | | | |
|---|---|---|---|---|---|---|
| Year | Below L&D 1 | Pool 1 | Pool 2 | Pool 3 | Pool 4 | Pool 5 |
| 1999 | 591,782 | 37,306 | 47,240 | 271,122 | 21,363 | 0 |
| 1998 | 1,416,730 | 136,065 | 124,418 | 728,944 | 140,381 | 0 |
| 1997 | 414,594 | 20,573 | 0 | 231,762 | 0 | 0 |
| 1996 | 0 | 0 | 77,547 | 319,489 | 0 | 0 |
| 1995 | 426,283 | 0 | 0 | 1,126,915 | 258,980 | 0 |
| 1994 | 843,404 | 0 | 0 | N/A | N/A | N/A |
| 1993 | 1,323,493 | 0 | 0 | N/A | N/A | N/A |
| 1992 | Data Record Not Complete | | | | | |
| 1991 | 2,127,066 | 354,000 | 0 | N/A | N/A | N/A |
| 1990 | 1,791,417 | 460,053 | 0 | N/A | N/A | N/A |
| 1989 | 1,237,031 | 131,215 | 0 | N/A | N/A | N/A |

*Figure A-47. JBJ Waterway In-Channel Dredge Records (1989–1999)*

| | Dredging Quantities (cubic yards) | | | | |
|---|---|---|---|---|---|
| Year | Boggs L&D | Overton L&D | L&D No. 3 | Long L&D | Waggonner L&D |
| 1999 | 664,384 | 203,637 | 32,932 | 58,930 | 21,956 |
| 1998 | 812,367 | 195,587 | 81,114 | 49,937 | 18,040 |
| 1997 | 390,068 | 238,389 | 113,111 | 73,134 | 30,033 |
| 1996 | 180,275 | 45,794 | 33,128 | 23,396 | 0 |
| 1995 | 640,342 | 245,803 | 129,246 | 41,923 | 32,540 |
| 1994 | 685,761 | 223,275 | 65,733 | N/A | N/A |
| 1993 | 1,060,385 | 590,500 | 279,764 | N/A | N/A |
| 1992 | Data Record Not Complete | | | | |
| 1991 | 809,001 | 114,180 | N/A | N/A | N/A |
| 1990 | 1,482,097 | 464,785 | N/A | N/A | N/A |
| 1989 | 966,297 | 266,041 | N/A | N/A | N/A |
| Average | 769,098 | 258,799 | 105,004 | 49,464 | 20,514 |

*Figure A-41. JBJ Waterway Lock and Dam Dredge Records (1989–1999)*

The authors conclude the 2001 white paper with the fact that the Red River is a high-energy system with a high sediment transport capacity. These aspects were both an asset and hindrance in the design, construction, operation, and maintenance of the waterway project. These characteristics were assets in that they reduced the project cost by providing for the development of project features including pilot channel development, trenchfill revetments, and capping out of stonefill and timber pile revetments. The sediment conditions were a hindrance because they resulted in troublesome deposition at the locks and dams that required costly removal by dredging. Additionally, the availability of sediment transported by the river required the costly construction of transverse dikes in depositional reaches and kicker dikes on the downstream end of revetments in crossings.

Since the waterway was opened, dredging was required in the approach channels to the locks and dams and to a lesser extent within the navigation channel within the pools. This is still generally the case as of 2024. With lessons learned for sedimentation issues at L&Ds 1 and 2, 3, and 4, L&D 5 incorporated structural modification aimed at significantly reducing sediment deposition but still some dredging is required. Additional details can be found in the 2001 white paper. As of 2001, the Vicksburg District was continuing to cap out revetments and construct channel control dikes to reduce dredging within the navigation channel. However, given the flow and sediment conditions on the Red River, the required dredging to provide and maintain the navigation channel was of manageable quantities. The existing conditions as of 2024, are discussed in the remaining sections.

#### 4.1.2.4 Sediment Study Below L&D 1 – 1998 ERDC CHL

In 1998, the ERDC CHL, formerly the Waterways Experiment Station (WES), conducted a sedimentation study for the Vicksburg District focused on the Red River downstream from

L&D 1. The investigation report is Technical Report HL-88-15. Below is a summary of the study.

The study was focused on the effects of the recently constructed and proposed channel improvements on sedimentation in the Red River downstream from L&D 1 from approximately RM 44 to the confluence with the Black River (RM 34). A 1D numerical model (HEC-6) was used to evaluate the effect of contraction works on dredging requirements in the navigation channel. A 2D numerical model was used to evaluate proposals to reduce deposition in the downstream lock approach channel at L&D 1. Recommendations were made to reduce sediment problems in the study reach.

A 1D sediment transport model, HEC-6, was used to calculate deposition, scour, and dredging quantities for various trace widths below L&D 1 to the confluence with the Black River. A trace width is a designated river width that is assumed to convey all the flow. When training dikes are present, trace width is taken as the distance between the outer ends of the dikes on opposite banklines. Trace widths of 200, 300, 400, and 500 feet were tested with a 7-year hydrograph. The model calculated dredging requirements necessary to maintain a 200-FT wide navigation channel with a 9-FT draft. The model also calculated average velocities in the contracted channel.

Cross-sections for the 1D numerical model were taken from the 1967–1968 hydrographic survey of the Red River. The primary area of interest in this study extended from L&D 1 (RM 46) to the confluence of the Red and the Black rivers (RM 34). In this reach, cross-sections were located at approximately 0.5-mile intervals. The model was extended to Shreveport (RM 277) to account for possible channel storage and supply downstream from the Shreveport sediment gage, and to make use of sediment measurements at Alexandria (RM 105) to adjust the model. Between L&D 1 and RM 140, cross-sections were located at approximately 2-mile intervals. Upstream from RM 140, cross-section intervals averaged 14 miles. This geometry was used in the adjustment phase of the study in which roughness coefficients and bed material gradation were determined.

The effect of various trace widths downstream from L&D 1 on aggradation and degradation was evaluated by restricting flow and sediment movement to the specified width, ignoring dike overtopping and overbank flows. This channel configuration was simulated in the model with frictionless vertical walls. Trace widths of 200, 300, 400, and 500 feet were tested. A more detailed study would include an accurate definition of the dikes including the sloping crest elevations and the area between dikes, accounting for deposition and increase of roughness due to vegetation. It would also include overbank areas for conveyance of flood flows.

The water-surface elevation at the downstream boundary is controlled by flows in the Atchafalaya River and the ORCC Outflow Channel and is not directly a function of discharge in the Red River. Starting water-surface elevations in the numerical model were therefore determined from the stage hydrograph at Acme, Louisiana (black RM 0.1). In the steady-state numerical simulations, stages at Acme for a specific day were assumed to correspond to the discharge at Alexandria for the same day, ignoring possible attenuation of the hydrograph due to storage and routing in the 71 miles between the gages.

Channel improvements were incorporated into the adjusted numerical model to establish a base condition for the trace width tests. The improvements included L&Ds 1 and 2 and existing and proposed cutoffs upstream from L&D 1. Dredging in the model occurred once a year during the lowest stage at Acme. A cross-section was dredged if the water depth anywhere in the designated 200-foot-wide navigation channel was less than 9-FT. A new dredging routine was incorporated into HEC-6 to meet this specification. Two feet of overdredging was specified. Dredged material was removed from the river. During the 7-year simulation, approximately 4 million cubic yards of material were dredged from the study reach downstream from L&D 1.

Dredging requirements with 200-, 300-, 400- and 500-foot trace widths were compared. Dredging would be relatively insignificant with a 200-foot trace width. With a 300-foot trace width, most of the dredging requirements were met early (during the first 2 years) as existing crossings were removed. After this initial clearing, average annual dredging was estimated at 84,000 cubic yards. Average annual dredging during the last 5 years was calculated to be 318,000 and 393,000 cubic yards or the 400-foot and 500-foot trace widths, respectively. Compared to the base (no dikes) condition, dredging was reduced in all the contracted channels except the 500-foot trace width. The slight increase in dredging with the 500-foot trace width, which is closest to the natural river width, is attributed to a decrease in sediment transport capacity caused by a decrease in channel width, which is not compensated for by an increase in velocity. Total accumulated dredging is shown in Figure A-48.

The effectiveness of the various trace widths in moving sediment through the study reach can be evaluated by comparing the sums of dredging and accumulated deposition. Accumulated deposition within the trace width can occur because only a 200-FT wide navigation channel is dredged and because deposition in the navigation channel can occur below the authorized 9-FT depth. Dredging and accumulated deposition were calculated to be about 6 million cubic yards in 7 years without constrictive works. Results with various trace widths are shown in Figure A-49. With a 200-foot trace width, 3.8 million cubic yards of material were removed from the study reach primarily because of scour. The effect of this scour on thalweg elevations is demonstrated in Figure A-50. With a 300-foot trace width, deposition and dredging are essentially balanced, and the thalweg profile is determined primarily by dredging requirements.

Contracting the river channel will generally result in an increase in velocity and depth. The effect of the trace widths on these hydraulic parameters was determined using the numerical model. Several discharges, ranging from 25,000 cfs to 142,000 cfs (navigation design flow), were tested. In these tests, starting water-surface elevations at Acme were assigned the same percent exceedance value as the discharge (stages and discharges were taken from Plates 22 and 4, USAED, New Orleans, 1980a). Average channel velocity between Acme and L&D 1 was determined from the calculated channel velocities at 13 cross-sections (Figure A-51). At the navigation design flow, the 200-foot trace width increased average velocity over 100 percent to approximately 10 frames per second. The 300-foot trace width increased average velocity 60 percent to 7.6 frames per second. These increases may affect the navigability of the river. Changes in water-surface elevation with the constricted channel were relatively minor.

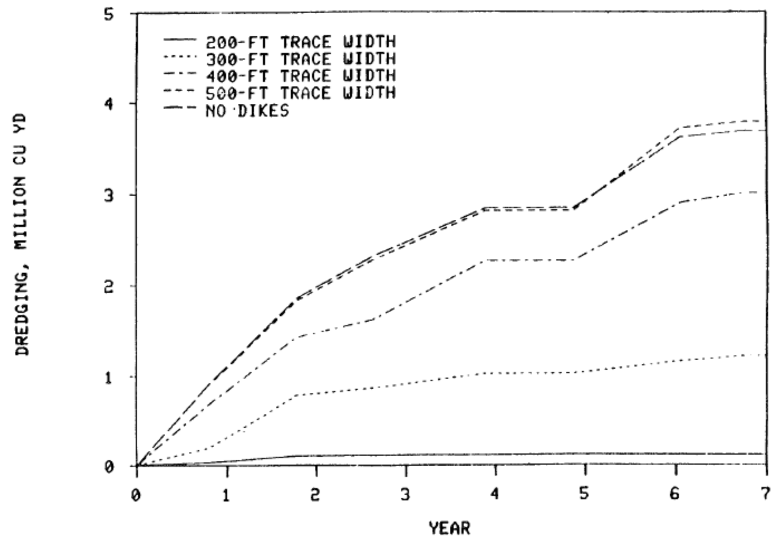



Figure 11. Accumulated dredging with tested trace widths, 1975-1981

*Figure A-48. Accumulated Dredging below L&D 1 (1988 Study)*

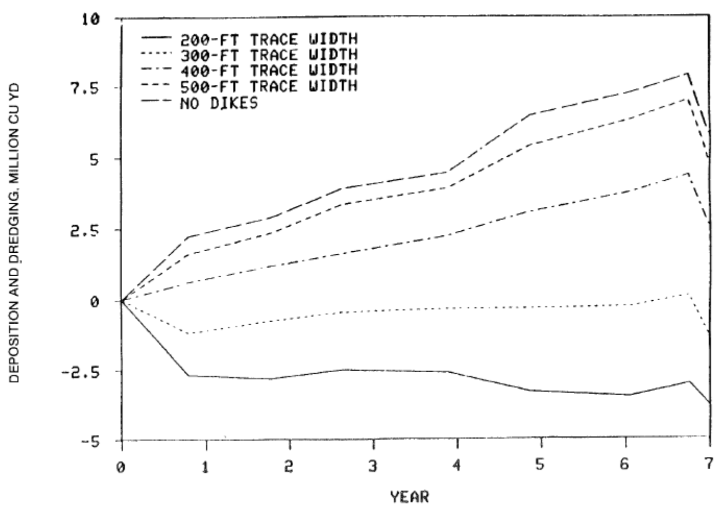



Figure 12. Accumulated dredging and deposition with tested trace widths, 1975-1981

*Figure A-49. Deposition Below L&D 1 (1988 Study)*

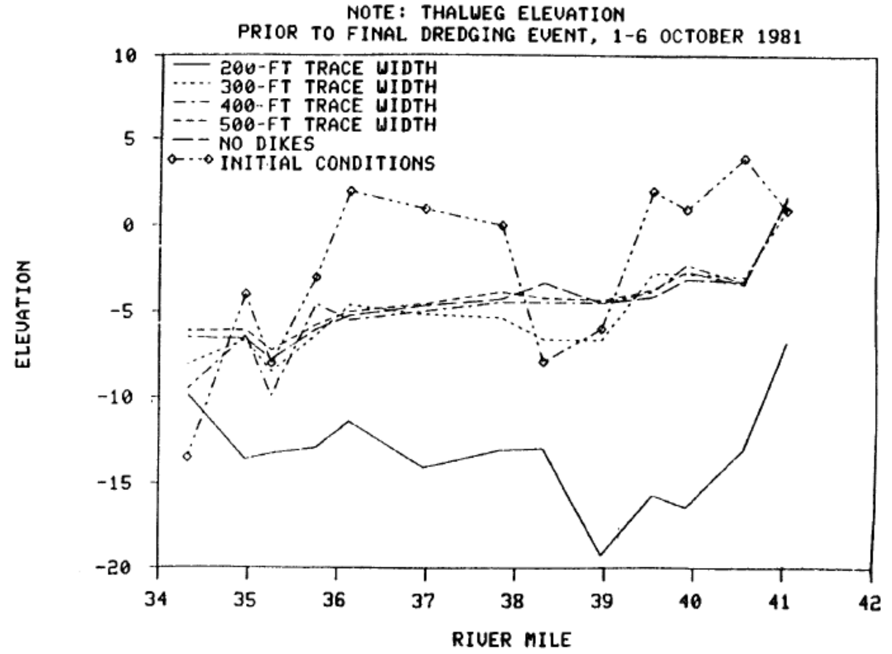



Figure 13. Calculated thalwegs for tested trace widths

*Figure A-50. Calculated thalwegs below L&D 1 (1988 Study)*

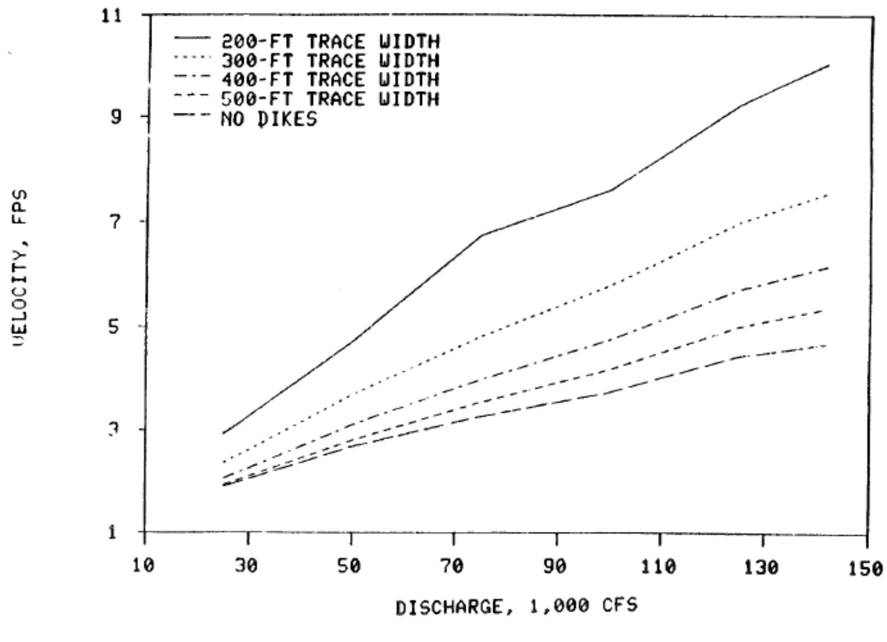



Figure 14. Calculated average channel velocities with tested trace widths, miles 34-41

*Figure A-51. Calculated average channel velocities below L&D 1 (1988 Study)*

4.1.2.5 Typical Navigation Channel Development and Maintenance - 1982 ERDC CHL

The investigation report is Technical Report HL-82-6. In 1982, ERDC CHL (formerly Waterways Experiment Station, WES), conducted a hydraulic physical model investigation for the New Orleans District regarding typical channel development and maintenance for the Red River. The Red River between RMs 68.6 and 79.2 (approximately RMs 60 to 71 as of 2024 mileage) were selected as a typical troublesome reach in which to determine the general channel realignment, training, and stabilization structures necessary to provide a navigation channel of adequate depth and width that would be stable and require minimum dredging. This reach is in Avoyelles Parish just upstream of the Moncla, Louisiana, bridge.

Numerous plans regarding channel realignment were made as plans A, B, and C, and additional subsets of the plans. Detailed information can be found in the referenced technical report.

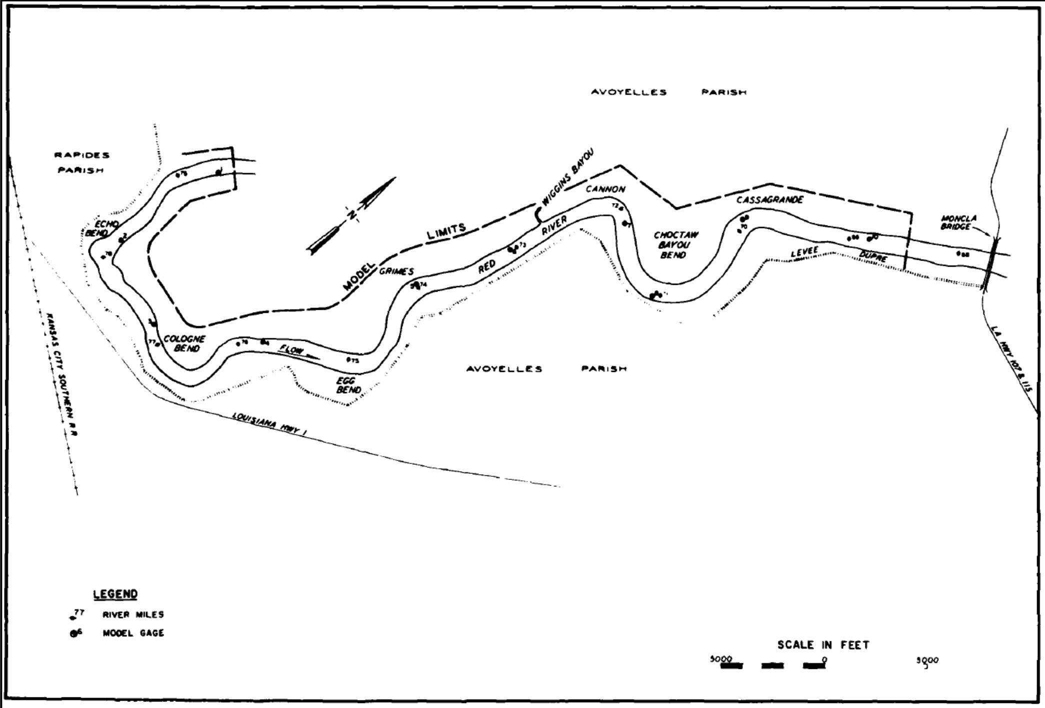



Figure 2. Model layout and gage location

*Figure A-52. 1982 Model Layout and Gage Location*

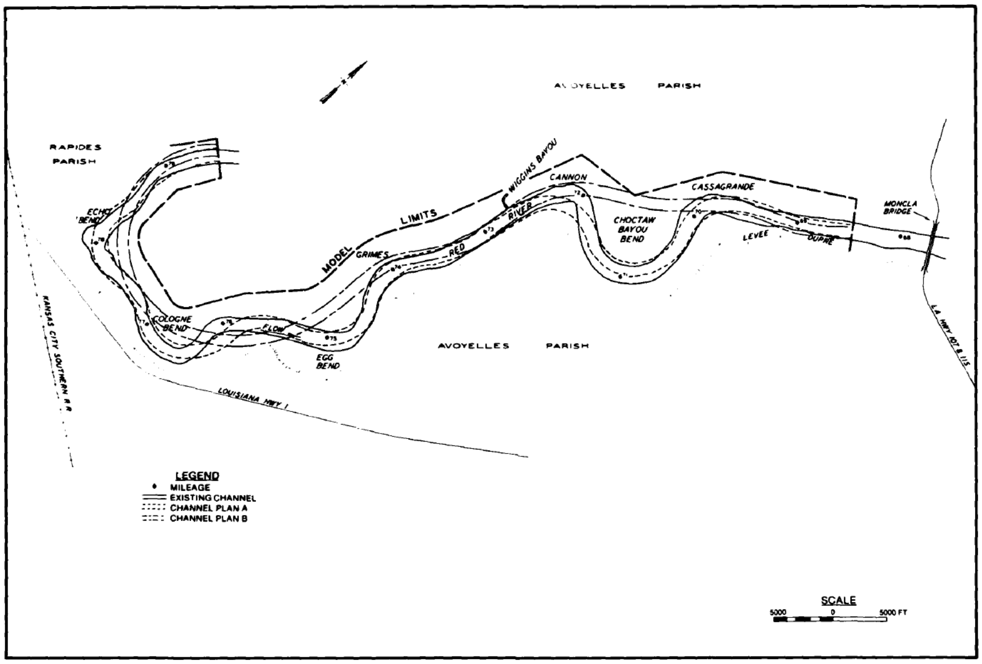



Figure 3. Channel alignment

*Figure A-53. 1982 Channel Alignments – Plan B Best Represents Alignments as of 2024*

General results and conclusions of the model investigation:

- The natural channel of the Red River and typical cross-sections proposed for cutoffs were too wide to provide adequate navigation channel depths without contraction and stabilization structures.
- Considerably more length of dike was required to maintain an adequate navigation channel when the existing river alignment was followed than when an improved channel alignment was used.
- Preservation of old bendways created by channel realignment was substantially improved by a closure of the upper end to top bank elevation and the construction of structures designed to block movement of sediment-carrying bottom currents entering the lower end of the channel.
- The alignment of the channel for Plan A included many rather sharp and irregular bends, some long straight reaches, and some short crossings. Development of a satisfactory channel with this alignment would involve the use of a considerable amount of construction in the form of training structures of various types and revetment.
- With Plan A, alignment structures would be required to contract the channel sufficiently to move the sediment entering the reach from upstream and to provide the additional depth and width of channel required for navigation. Structures would also be required to improve the alignment of most bends and provide adequate depths over crossings during low flows. The alignment of Plan B consisted of one

long bend of more than 180º and a large number of short flat bends and short crossings. Because of the alignment and cutoffs involved, the length of channel of Plan B was considerably shorter than that of Plan A.

- Structures would be required with the Plan B alignment to contract the channel, particularly in the long bend, to provide adequate navigation channel width and depth, reduce the tendency for the channel to meander within the bend, force the channel to cross between the short flat bends, and close off some of the side channels through the old bendways.
- The length of dikes required per mile of channel with the alignment of Plan A would be about 50 percent more than with the Plan B alignment. Considering the shorter length of channel with Plan B and the greater number of dikes on the deeper concave side of the channel with Plan A, the amount of dike construction required would be considerably less with the Plan B alignment. Development of the channel with the Plan B alignment would require considerable excavation. The length of dikes required with Plan B could be further reduced by reducing the width of the excavated channel, particularly in the long bend.
- The reach downstream of the long bend with the Plan B alignment was generally too straight with relatively short flat bends and short crossings to provide a satisfactory channel without the use of training structures.
- The rate of development of a cutoff as tested with Plan C would depend on flow conditions and the amount of flow passing through the old bendway. Other factors that could affect the rate of development would be the erodibility of the material through which the cutoff is made and the relative length of the cutoff channel with respect to the bendway channel.
- The openings in the dikes with the 380-foot permeable pile section were too large to have any appreciable effect on the cutoff during the early stages of development. With the single closure dike at the upper end of the old bendway channel, there was a deeper connection between the main channel and the bendway than with the closure dike farther downstream in the bendway.
- The deeper channel within the cutoff tended to meander and be somewhat unstable during the early stages of development. Based on the results of tests of Plan B, structures would probably be required to maintain the channel along the revetted bank in the lower reach of the cutoff.
- Shoaling would occur in the bendway channel starting at its upper end when there is substantial flow through the bendway channel. Maintaining the old bendway for fish, wildlife, and recreation or port facilities would require that the upper end of the old bendway be closed as soon as conditions permit. Shoaling would also occur in the lower end of the bendway. Maintenance of an entrance at the lower end of the old bendway without dredging would require structures designed to block the movement of sediment-carrying bottom currents from entering the channel.
- In general, results of this investigation indicated that the typical cross-sections furnished and natural channel widths in some reaches were too large to provide adequate channel depths and widths for navigation without changes in flow

conditions and rate of sediment movement. Additionally, development of a satisfactory channel would require the closure of old bendways that are bypassed or any secondary channels that would divert some of the flow from the main channel.

#### 4.1.2.6 Stabilization and Cutoffs – Design Memorandum No. 1 – 1972 New Orleans District

In May of 1972, the USACE New Orleans District drafted the Stabilization and Cutoffs Design Memorandum No. 1 to provide a general plan for establishing a stabilized channel that will, with subsequent establishment of the necessary locks and dams required to sustain navigable depths during low-flow periods, provide a 9-foot deep by 200-foot wide channel for the Mississippi River to Shreveport waterway project. This summary in no means attempts to capture the entire memorandum but rather provide a concise background of the original stabilization, cutoff, and contraction design works. The document provides a plan of development for each navigation pool and for the reach below the lowermost section downstream of L&D 1.

The memorandum illustrates that construction of channel cutoffs and bank stabilization was to start downstream and work in the upstream direction from approximately 1973 to 1983. It was noted that impoundment of the navigation pools prior to the completion of the major elements of the bank stabilization and channel rectification works in any section of the river would retard the desired channel developments. Currents in the slack water pools during low to moderately high flows will be so slow that the desired erosive action induced on the bed of the stream by the training and contraction works will be greatly reduced since erosion would only occur at high river stages (otherwise referred to as channel forming, effective, or bankfull types of discharges). It was therefore desirable to complete the bank stabilization works located in critical reaches in advance of the impoundment of the pools to allow time for the works to become effective in forming the channel, thereby reducing, or possibly eliminating the amount of dredging required to develop the navigation channel. Early development was also highly desirable to permit more accurate determination of the cross-sections of the river channel as a basis for final design requirements for the navigation locks and dams and additional contraction structures, primarily located in the upper reaches of pools where depth is critical. Construction of the additional contraction works was suggested to be deferred until just prior to the completion of the locks and dams to permit the observation of the effectiveness of the bank stabilization structures in deepening the navigation channel providing a more accurate indication of the extent to which the structures would be needed.

At the time of the 1972 memorandum, the existing channel was characterized as having wide fluctuations in stage, caving banks, unpredictable shoaling, and meandering reaches that featured alignments varying from flat, unstable bends to sharp, well-defined bends that migrate rapidly until natural cutoffs occurred. Such characteristics were averse to the interests of navigation and posed a continuing threat to the integrity of the existing flood control works and improvements along the river. As of 1972, the more serious threats to improvements along the river had been diminished by construction of bank stabilization structures and cutoffs under the authorities of the Red River below Fulton, Arkansas, project;

the Red River Levees and Ban Stabilization below Denison Dam, Texas, Arkansas, and Louisiana, project; the Lower Red River-South Bank Red River Levees project; and various emergency authorizations. The only projects noted along the Red River to have included a comprehensive program of bank stabilization were the Red River in the Vicinity of Shreveport, Louisiana, project and the Overton-Red River Waterway, Lower 31 Miles project.

A general overview of the channel improvements per the 1972 design memorandum are summarized. The locks and dams would provide a dependable 9-FT by 200-FT navigation channel. The number and height of dams required would be minimized by contracting the channel in the upstream reaches of each pool to maximize channel depths. As flow increased, the pools would gradually become inundated until, at one-half bankfull stage, open channel flow conditions would prevail. During such periods, an unregulated channel would migrate and cause bank caving. Therefore, even with the locks and dams, a comprehensive channel improvement program is required. The channel must be stabilized to maintain the necessary trace (or contracted width). The stabilization necessary to maintain the navigation channel would also eliminate the bank caving problems with its associated threats of flanking existing structures, destroying levees and other improvements; all of which discourages development along the river.

The design memo determined average values of cross-sectional area and of carrying capacity ($Ad^{2/3}$, where A is area and d is weight average depth) for a range of flows. Relating the values to the same cross-section, but with contraction structures in place, indicated that generally, approximately 2 feet of additional depth could be obtained. Because averages mostly represented average conditions and approximately 50 percent of the sections for any given radius reflected ground surfaces that would be above the average section. For that reason, average sections could not be used for design as navigable depths would be available through approximately 50 percent of the bendways. A critical section was developed for each radius studied, or radius of curvature about a bend. The critical section for each radius represented a natural channel section, but one in which the ground surface was above the preponderance of all sections studied for that radius. The critical sections were then used to develop a composite critical section. The critical section was considered to be self-maintaining through the complete range of expected flows with proper channel alignments and bank stabilization. The stabilized channel trace widths, between the basic stabilization structures would vary from 600 feet in crossings where navigable depths were not critical and in bendways, to 450 feet in crossings where depth in navigation channel were critical. These trace widths were deemed compatible with the present regime of the stream, and slopes and flood heights after stabilization of the trace would be substantially the same as under existing conditions. The major portion of the flow is maintained within the trace widths for the complete range of flows thus maintaining flow essentially parallel to the rectified and stabilized channel so that the angle of attack on the stabilization will be controlled by aligning the stabilization structures so that the crossings will be minimized. The direction of flow into the crossings will be controlled by aligning the stabilization structures so that the crossings will be maintained to the maximum extent practicable by the natural forces of the stream. The Trace width will then be controlled by structures along both banks to provide navigable depths through the crossings. In the bends, the stream generally has more flexibility in developing its own optimum width depending on the degree of curvature but the dikes on the convex banks can be extended to provide greater control of the navigation channel if future conditions warrant.

The document noted that the reach downstream from L&D 1 to the mouth of the Black River is extremely critical considering the need for maintaining navigable depths during low-flow periods. Studies using the critical and improved design sections, and stage–duration curves at Acme (due to the significant backwater influence from the Mississippi River), indicated that navigation would be restricted 15 percent of the time without channel contraction and 9 percent of the time with maximum contraction throughout the entire reach. Comparative cost estimates indicated that the then present worth of the reduction in annual maintenance dredging costs over the project life through channel contraction would more than offset the cost of contraction. If, as the result of the lock and dam site selections, L&D 1 were located near the mouth of the Black River, the previously mentioned contraction would no longer be necessary.

## 4.2 STAGE–DISCHARGE MEASUREMENTS

Stage–discharge rating curves are a simple but valuable relation. These types of concepts are unequivocally valuable for flood risk management purposes characterizing how a rivers ability to convey a flood shift over time when viewing the upper ends of these types of curves. These curves also provide some value in that the lower ends of the curves shed light on the in-channel river processes such as degradation and aggradation.

In the simplest form, a stage–discharge rating curve is an XY graph plotting water levels versus discharge. In this case, the ability to add a third variable, time, by color contouring each point allows for a visual representation of the possible shifting of the relation through time. The following figures (Figure A-54 to Figure A-56) were sourced from the 2023 Red River Flowline Update. In general, the gages well upstream of the waterway at Index and Fulton, Arkansas show a degradational trend over time at various flows while the gage at Shreveport (Pool 5) shows an aggregational trend over time at various flows for which the low flows are primarily subject to the lock and dam operations holding pools while the medium to high flows are subject to lock and dam operations along with other geomorphic shifts in the river over time. The gage at Coushatta (Pool 4) shows similar trends as Shreveport. The gage at Alexandria (Pool 2) illustrates more of a degradational trend over time.

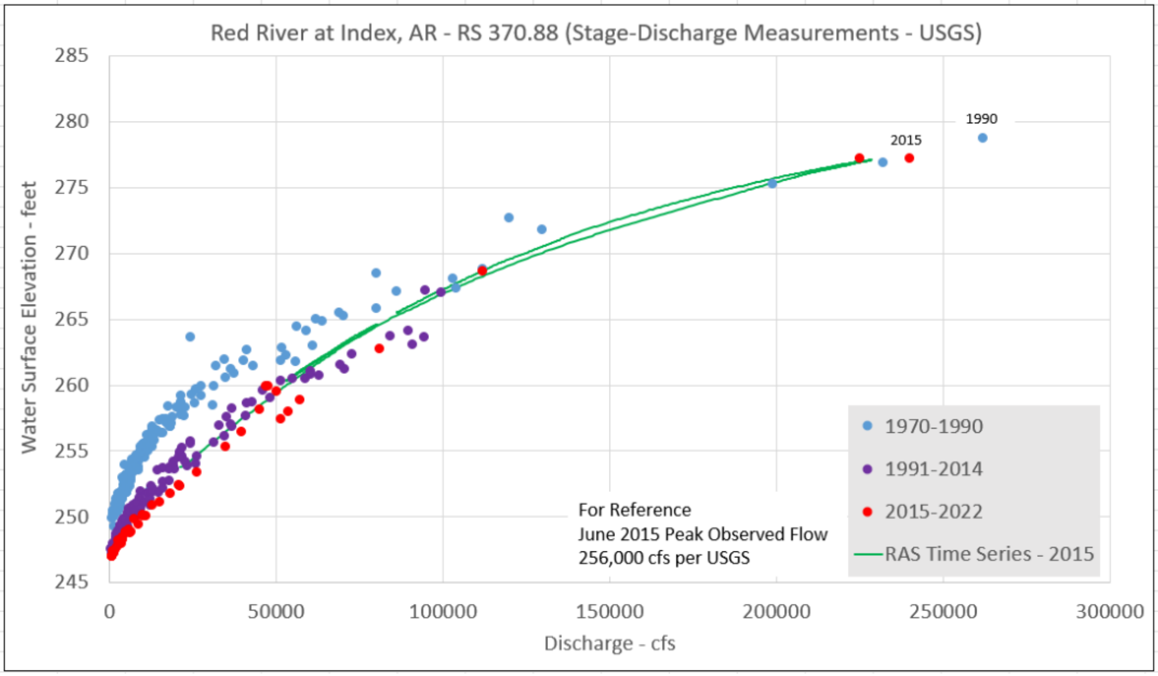



*Figure A-54. Stage–Discharge Relationship at Index, Arkansas*

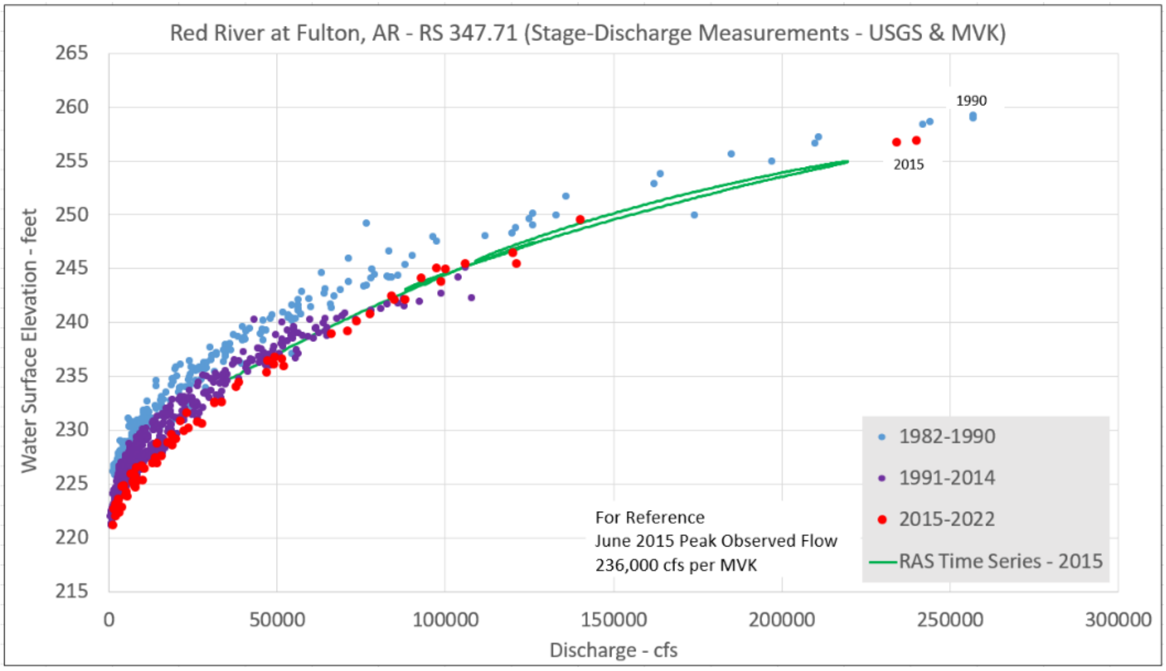



*Figure A-55. Stage–Discharge Relationship at Fulton, Arkansas*

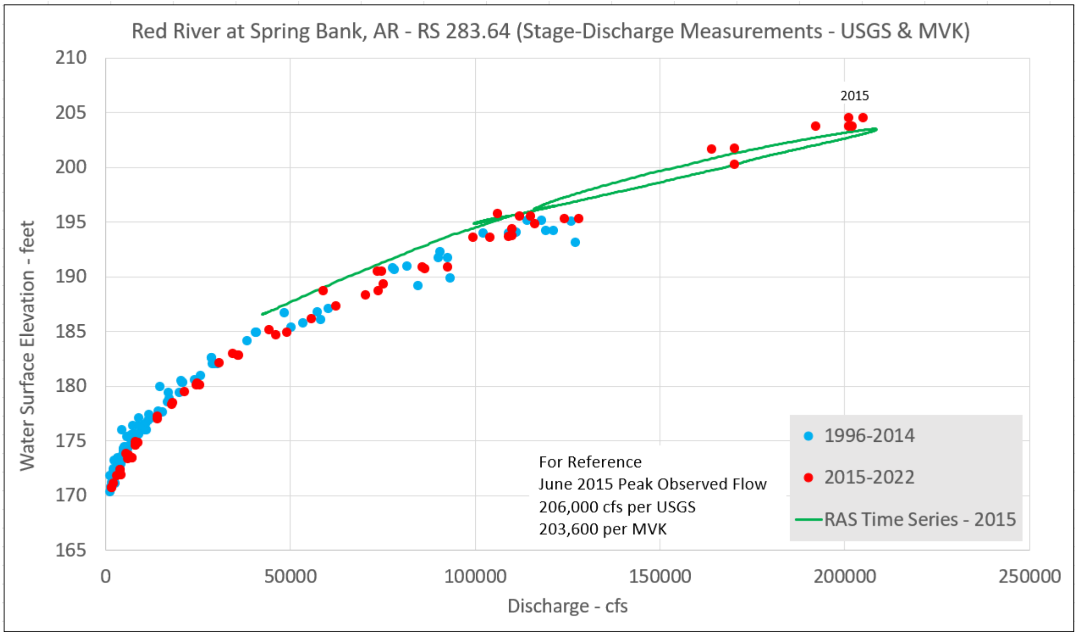



*Figure A-56. Stage–Discharge Relationship at Spring Bank, Arkansas*

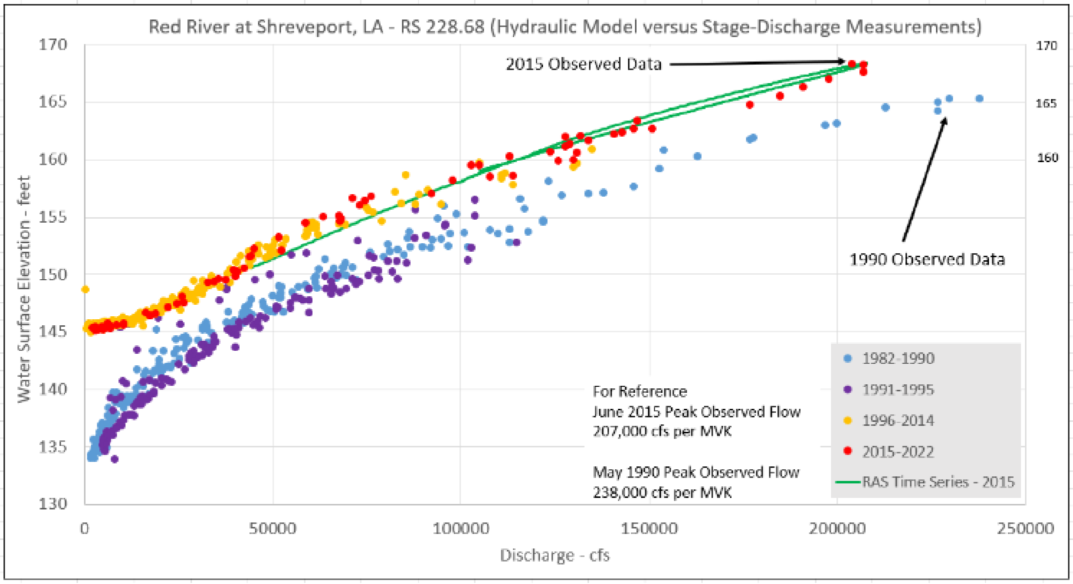



*Figure A-57. Stage–Discharge Relationship at Shreveport, Louisiana*

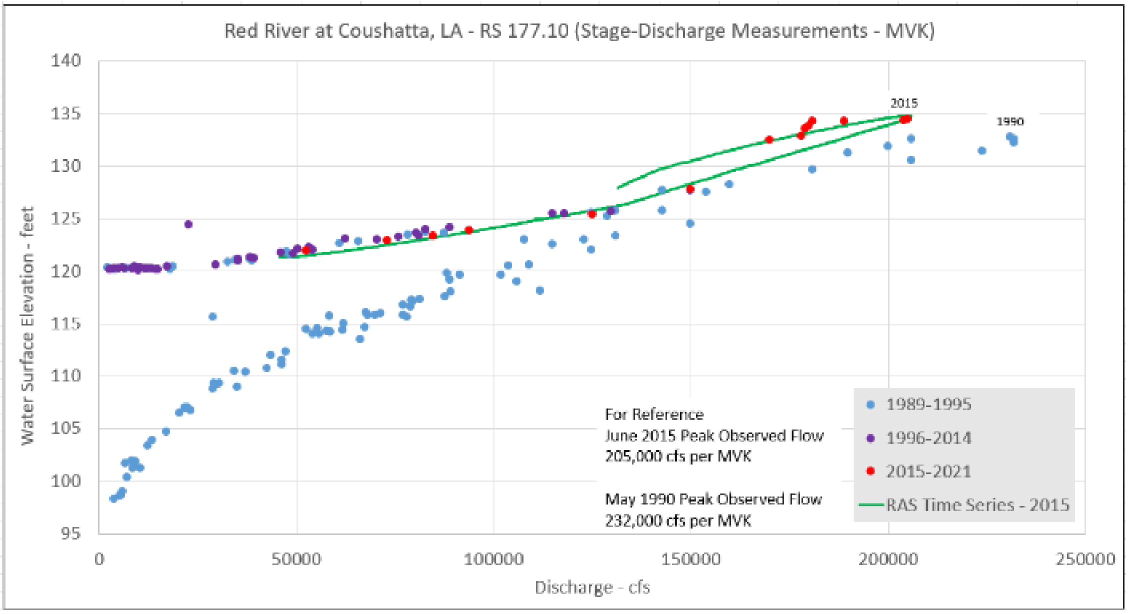



*Figure A-58. Stage–Discharge Relationship at Coushatta, Louisiana*

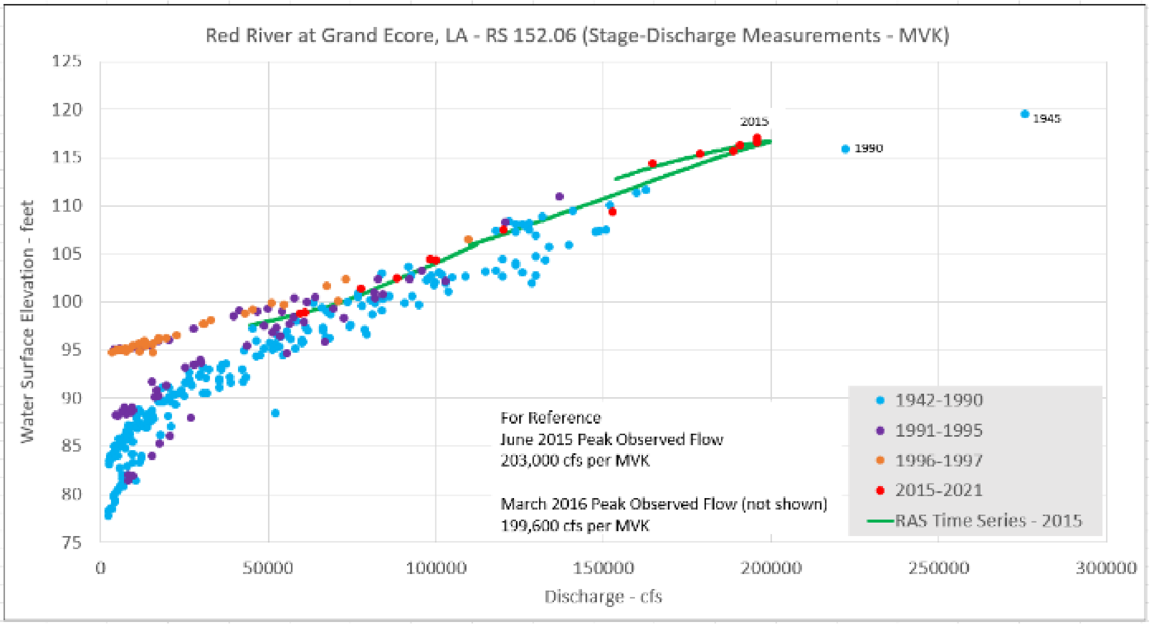



*Figure A-59. Stage–Discharge Relationship at Grand Ecore, Louisiana*

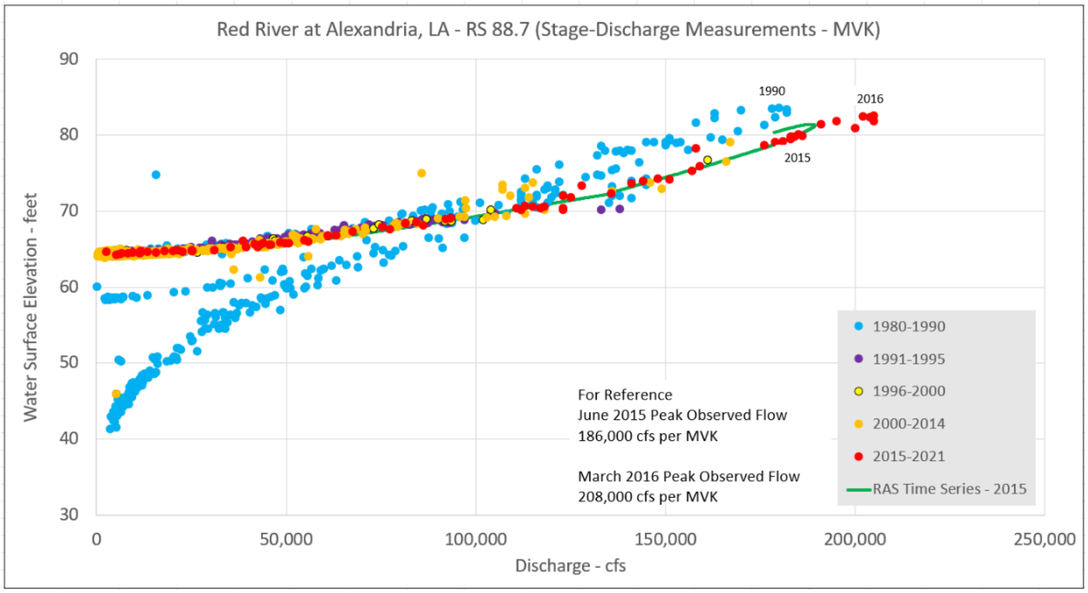



*Figure A-60. Stage–Discharge Relationship at Alexandria, Louisiana*

Discharge measurements, which are used to build stage–discharge rating curves, typically contain a much larger dataset beyond just the stage and flow data that are often reported. Hydrographic surveying typically is performed with acoustic doppler current profilers to measure stream velocity and integrate the measurements across the channel cross-sectional area to compute a flux measurement. USACE Vicksburg District Hydrologic Technicians have collected many measurements over the course of decades at the Shreveport and Alexandria gages and store the data on the rivergage websites. The data typically include stage, maximum depth, area, and width to be reported along with the measured discharge. Using these parameters, estimates of thalweg elevation (or lowest point in the channel along a cross-section) and an average channel elevation can be made. The equation sets and parameters are provided below. The data provide another illustration of how channel conveyance and depths changes with time. Though there are more limitations and uncertainty with this approach than typical bathymetry surveys, there is value in the continuous data through time.

Discharge measurements are available from 1982 to 2024 at Shreveport and 2002 to 2024 at Alexandria; however, the data from 2019 to 2024 report only stage and flow. Therefore, the calculations cannot be made within that time frame. Figures A-61 and A-62 below show the Shreveport and Alexandria thalweg and average channel elevations from 1982 to 2018.

$$(El_T = WSE - D_{max})\ and\ (El_{avg} = WSE - \frac{A}{W})$$

Where

$El_T$ = thalweg elevation, ft

WSE = water surface elevation, ft

$D_{max}$ = maximum depth, ft

$El_{avg}$ = average channel elevation, ft

A = channel cross-sectional area, $ft^2$

W = channel width at water surface, ft

The Shreveport data plot shows similar trends to the specific gage analyses referenced in the 2020 ERDC CHL hydraulic analysis. There is a decreasing thalweg and average channel elevation from 1982 to 1995 when L&D 5 began operations, and then there is an immediate steady increase in 1995 and 1996. There is an unexplained decrease from 1997 to 1999 followed by a gap in the data until 2002. The temporary decrease may be explained by the construction of the nearby Horseshoe Casino within the river that likely included a cofferdam during the period of construction which could have influenced channel geometries. There is a noticeable decrease between 2015 and 2018. This is attributed to the historic Red River headwater flood of 2015 followed by another moderate localized flood in 2016. The 2016 flood was a major flood at areas downstream; however, because it was more localized, much of the flows came into the river downstream of Shreveport. This decreasing trend was also observed following the historic 1990 flood. Trends generally show that floods or even typical high flows pass through causing a scouring of the channel followed by a slow upward trend after the flood as the channel builds back up.

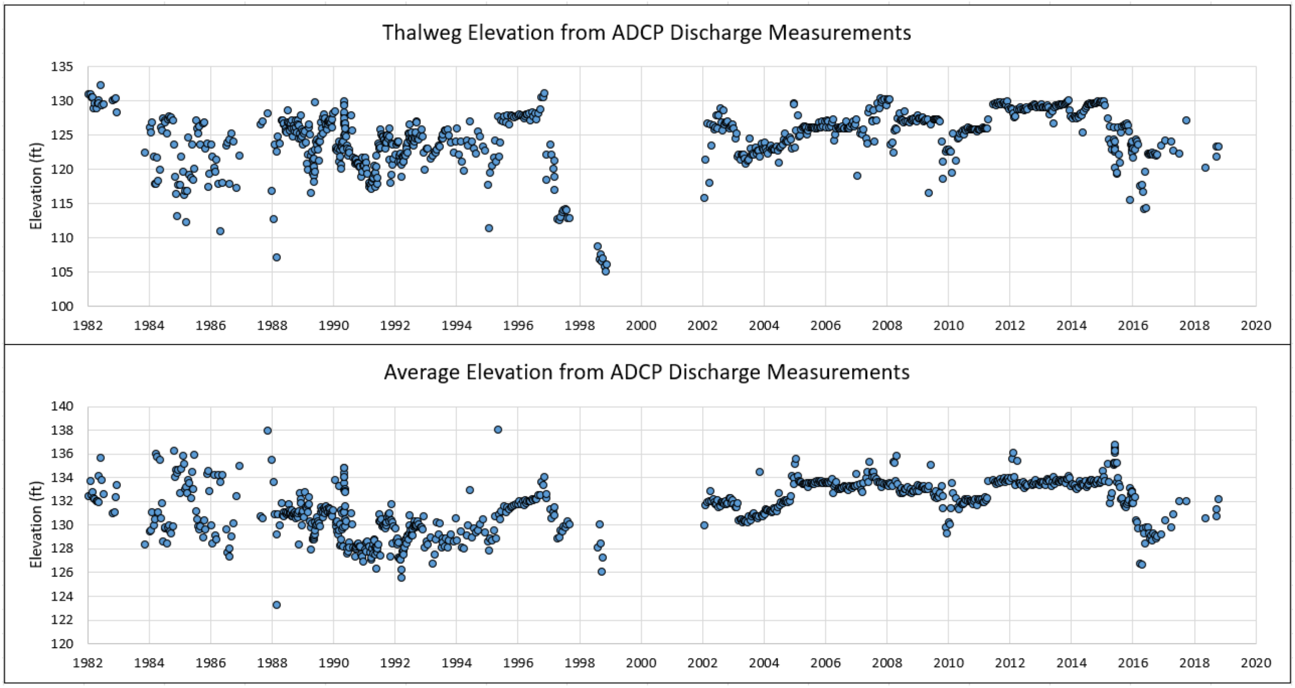



*Figure A-61. Channel Bed Elevations from Shreveport Discharge Measurements*

The Alexandria dataset is somewhat more limited, as the data only go back to 2002, and L&D 2 downstream of Alexandria was completed in 1987. The data show a fairly steady thalweg and average channel elevation between 2002 and 2015, after which a noticeable decrease occurs. This is attributed to the historic Red River headwater flood of 2015 followed by another major flood in 2016 caused primarily by localized Red River runoff downstream of the Shreveport gage combined with considerable headwater flows passing the Shreveport gage. Following the 2016 flood, a slow upward trend can be seen into 2018 as the channel builds back up from being scoured by the 2015 and 2016 floods.

The 2024 USGS measurements were supplemented; however, although the data do not include the maximum depth to calculate the thalweg, but data include channel width and area for calculating average channel bed elevation. The 2024 data appear to show a continued degradation or downward trend in average bed elevation from 2018 to 2024. Notably, USGS also characterizes each measurement with specific ratings such as Fair, Good, or Poor as shown in the figure below. However, there is some degree of additional uncertainty due to the absence of data between 2018 and 2024 and the potential differences in collection processes between USACE and the USGS.

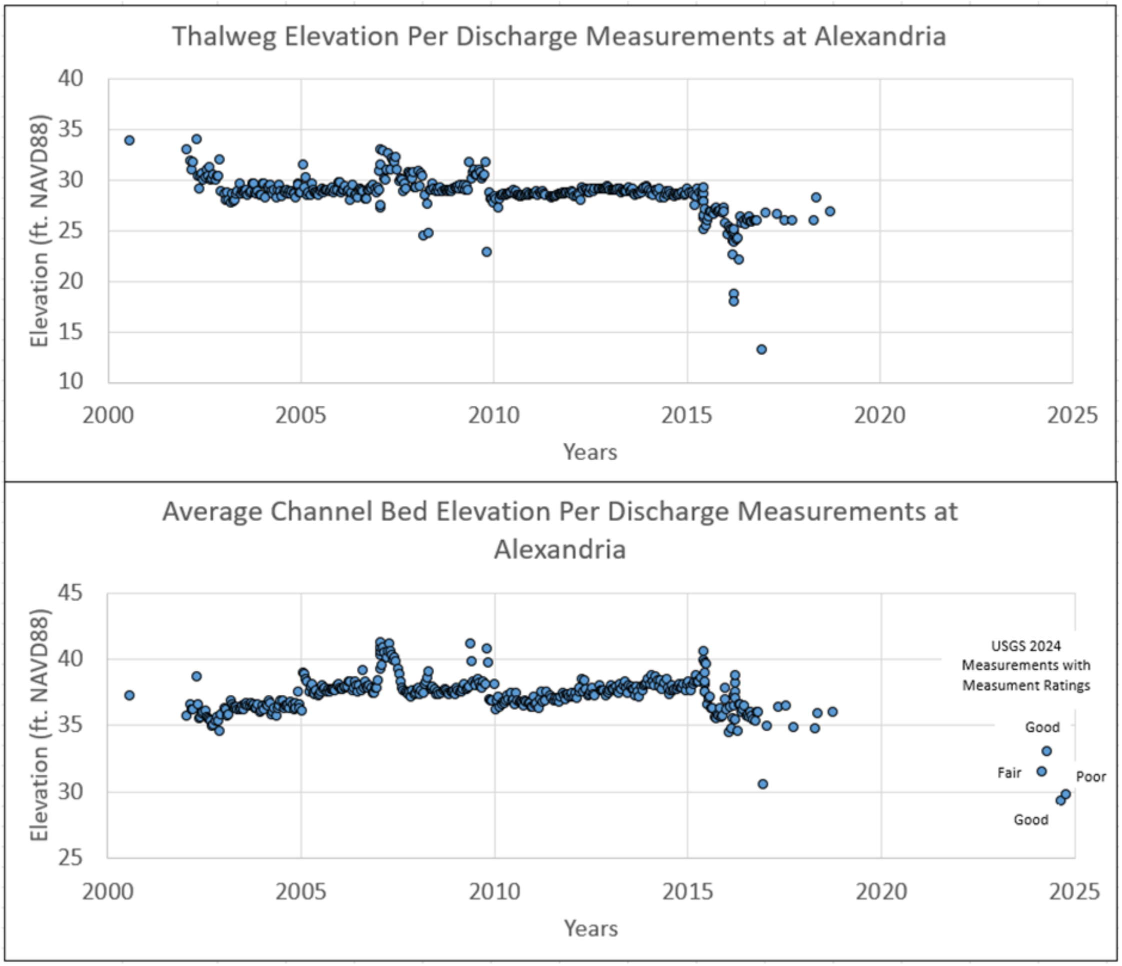



*Figure A-62. Channel Bed Elevations from Alexandria Discharge Measurements*

### 4.3 VERTICAL DATUM ADJUSTMENTS

Vertical datum adjustments are provided in Table A-17 below to convert from NGVD29 (used for river gages) to NAVD88 (used in the hydraulic models).

*Table A-17. Vertical Datum Adjustments from NGVD29 to NAVD88*

| Location | HEC-RAS River – Reach and River Station | Gage Datum (Feet NAVD88) | Vertical Datum Shift NGVD29 to NAVD88 (Feet) |
|---|---|---|---|
| *Red River at Shreveport, LA* | Red River – Below Big Cypress – 228.68 | 131.48 | -0.23 |
| *Red River L&D 5 (Joe D. Waggoner)* | Red River – Below Big Cypress – 200.00 | 0 | -0.16 |
| *Red River at Coushatta, LA* | Red River – Below Red Chute – 177.10 | 95.78 | -0.19 |
| *Red River L&D 4 (Russel B. Long)* | Red River – Below Red Chute – 168.57 | 0 | -0.18 |
| *Red River at Grand Ecore, LA* | Red River – Below Red Chute – 152.06 | 75.09 | -0.15 |
| *Red River at Midpoint Pool 3* | Red River – Below Red Chute – 138.63 | 0 | -0.10 |
| *Red River L&D 3* | Red River – Below Red Chute – 116.16 | 0 | -0.03 |
| *Red River at Alexandria, LA* | Red River – Below Red Chute – 88.7 | 44.26 | 0.04 |
| *Red River L&D 2 (John H. Overton)* | Red River – Below Red Chute – 74.375 | 0 | 0.08 |

| Location | HEC-RAS River – Reach and River Station | Gage Datum (Feet NAVD88) | Vertical Datum Shift NGVD29 to NAVD88 (Feet) |
|---|---|---|---|
| *Red River L&D 1 (Lindy C. Boggs)* | Red River – Below Red Chute – 43.90 | 0 | 0.12 |
| *Red River at Acme, LA, Gage* | Red River – Below Black – 34.29782 | 0.77 | 0.09 |
| *Atchafalaya River at Simmesport, LA* | Atchafalaya River – Below Old River – 13.10020 | 0 | 0.07 |

## 4.4 CHANNEL DEPTHS ASSESSMENT

This section documents information regarding the physical characteristics and operations of the locks and dams, typical draft requirements through the lock chambers for vessels to safely enter and exit the chambers over the miter gate sills using a duration statistics approach. Further, this section documents an analysis completed to characterize the varying channel depths throughout the waterway and identify the potential problem reaches regarding the availability of channel depths sufficient for a 200-foot-wide by 12-foot-deep channel.

### 4.4.1 Hydraulic Modeling

Existing conditions channel depths were assessed using historical dredge records, existing hydraulic models, channel bathymetry, and discussion with Vicksburg District River engineering personnel. The assessment of navigation channel depths is focused on the depths within the river channel and not the depths at locks and dams.

An existing Vicksburg District Red River HEC-RAS model was used to establish an understanding of the existing conditions of the navigation channel depths. The model is sourced from the 2023 Red River 1 and 0.2 percent AEP Water Surface Profile Update (also referred to as the Red River Flowline Update), which was ultimately a model update from the 2016 MMC Red River CWMS model. The major updates in the flowline model included updating cross-sections with 2016 single-beam channel surveys, 2018 bank to levee LiDAR, manning’s roughness calibrations, mainline levee systems with the latest NLD, and modifying the physical setups and operations of the locks and dam structures where needed. The flowline model was primarily focused on calibrating to high-water events and flood events such as the 2015 flood. The current JBJ 12-FT channel study is focused on low water or navigation project flow conditions at normal pool.

The major aspect of the modeling is the utilization of the 2016 channel surveys, and the operations of the locks and dams. The following procedures were completed in the flowline update and carried forward for the JBJ 12-FT channel study. The 2016 surveys were captured on average every 500 feet with some greater spacings upwards of 1,000 feet, and only captured in the river channel, meaning that the channel bathymetry within dike fields is not captured. This is not so significant as the project conditions mentioned in the following sections are considered navigation channel depths at low flows where water levels are well below dike crests. The channel surveys were implemented in the HEC-RAS geometry editor, and RAS Mapper was used to create a Geotiff from the cross-section data. Due to the data being single-beam and the noted workflow, RAS Mapper generates a raster file from the cross-section using an interpolation method. Therefore, the data are of high quality at each captured transect but ultimately only an estimated of channel bathymetry between each transect.

#### 4.4.1.1 Navigation Project Design Conditions

Currently, the 98 percent Duration Exceedance inflows and normal pool operations are used as navigation channel design conditions regarding identifying channel depths at a low water plane. This is consistent with practices for determining low water planes on the Mississippi River along with project conditions noted in water control manuals. The duration exceedance concept characterizes flows exceeded a certain percentage of time on an annual basis. For example, the 1994 L&D 5 Water Control Manual states the project condition as a flow exceeded 98 percent of the time. The project condition flow was documented as 1,200 cfs. These types of conditions create completely flat pools under existing normal pool operations, meaning that moving upstream, the riverbed naturally slopes upwards, but the pool remains flat. Therefore, illustrating that the upper reaches of the pool are generally the critical reaches regarding the sustained availability of depths.

Utilizing Shreveport (Pool 5) and Alexandria (Pool 2) daily flow records from 1935–2024, the 98 percent DEP flows were determined to be approximately 1,700 cfs and 2,200 cfs. These flows are carried forward for a design conditions simulation in HEC-RAS. The Red River became a fully regulated river basin by about 1965 by upstream reservoirs; however, it is not expected that dividing Periods of Record (POR) between unregulated and regulated would have any drastic influence on the low flow duration exceedance calculations. Any small change would ultimately not change the flat pool or flat water surface elevations as it takes more than 4 or 5 times the 98 percent DEP flows to see any slope in the river profiles under current normal conditions. Currently, there are no upstream reservoir operations that augment (provide controlled releases) the waterway during low-flow periods.

It has been noted that below L&D 1, the water levels are uncontrolled, meaning there is no downstream pool controlling structure. This portion of the waterway is situated within the Lower Mississippi River floodplain and is heavily influenced by Mississippi River backwater flows through the ORCC. Due to the backwater influence, a water surface elevation reference for design conditions is more appropriate than flow. The Red River at Acme gage is situated approximately 10 miles downstream of L&D 1 near the confluence with the Black River. Vicksburg District Water Management contains daily stage readings at Acme from 1932–2024. The daily data reveal that a 98 percent DEP stage is approximately 5 feet NAVD88.

The model is setup as unsteady flow with constant inflows along the mainstem and tributaries and simulated long enough to establish stable, steady state flow conditions to achieve the 98 percent DEP design conditions previously mentioned. Upon model execution, HEC-RAS generates water surface and depth grids. The static depth grids and underlying terrain data are imported into ArcGIS. In addition, a river training structure layer, the recommended navigation track centerline, and an approximate 200-FT wide navigation polygon boundary are also imported into GIS. The depth grid color ramps are modified to show channel depths in 2-FT intervals. The maps are used to assess the entire waterway to help identify areas where potential problems may be present or likely to occur regarding inadequate depths across the 200-foot-wide navigation channel. Notably, any area with less than 15-feet of channel depth is flagged for further assessment, and areas with 15 or more feet of depth are screened.

#### 4.4.1.2 Model Development

The Hydrologic Engineer Center River Analysis System (HEC-RAS) software is used for the hydraulic modeling. The basis of this model stems from the 2023 Red River 1 and 0.2 percent AEP, otherwise referred to as the 2023 flowline update. There are no major changes made to the model for this effort. The flowline model was completed in RAS version 6.3.1 and has been carried forward to version 6.5. This effort does add a low-flow calibration event as the flowline update was focused on calibrating to flood events. Detail regarding the development of the model can be found in the referenced report.

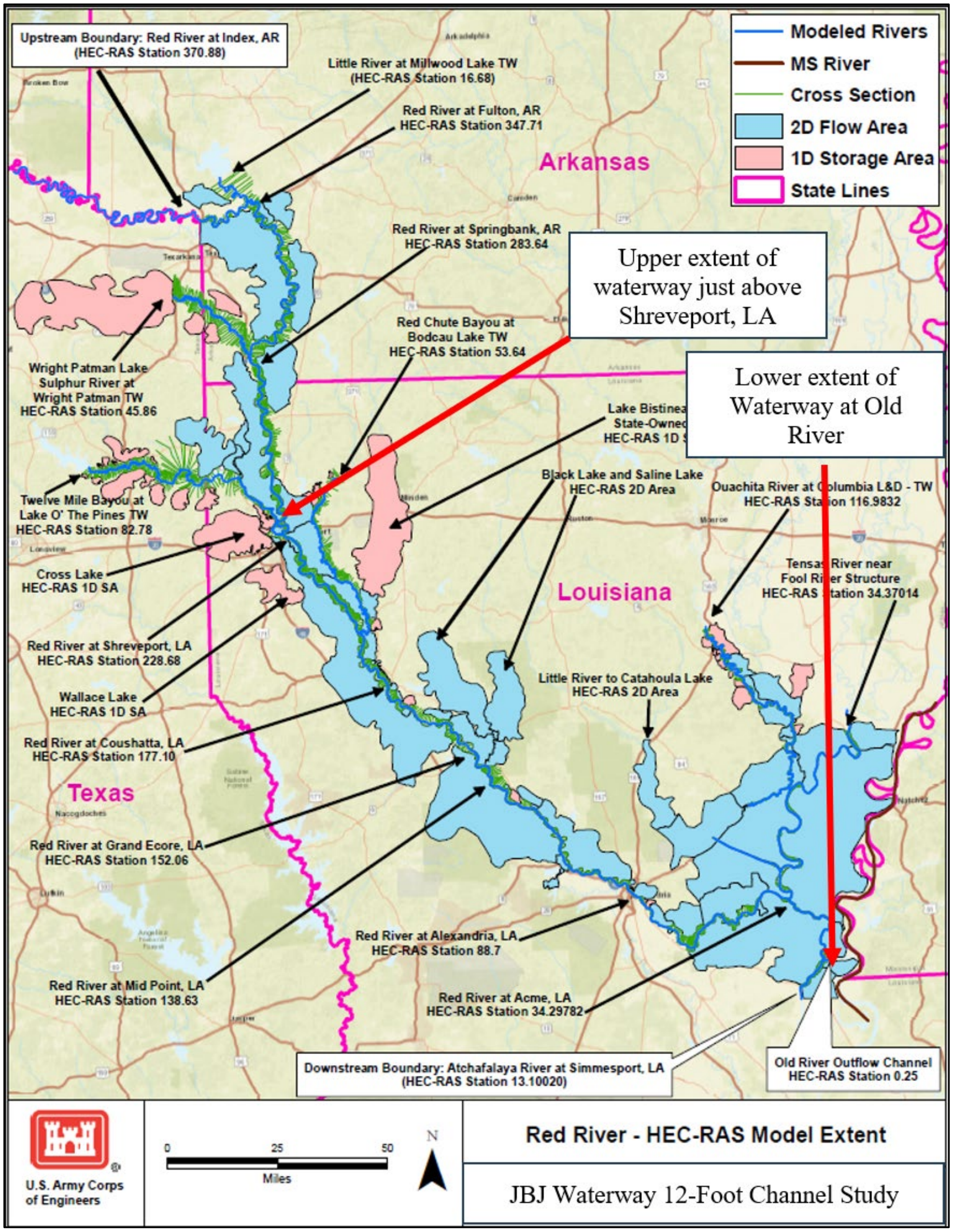



*Figure A-63. HEC-RAS Model Overview*

### 4.4.1.3 Model Calibration

The base model stems from the Lower Red River 1 and 0.2 percent AEP water surface profile update (or Flowline Update) that was completed in April 2023. Extensive calibration efforts and plots were performed and documented within that referenced report. For the sake of this study, a low-flow calibration plan was added into the model to show a calibration simulation at low flow with normal pool conditions that would mimic the project design conditions for this effort which is the normal pool condition with 98 percent DEP low flows.

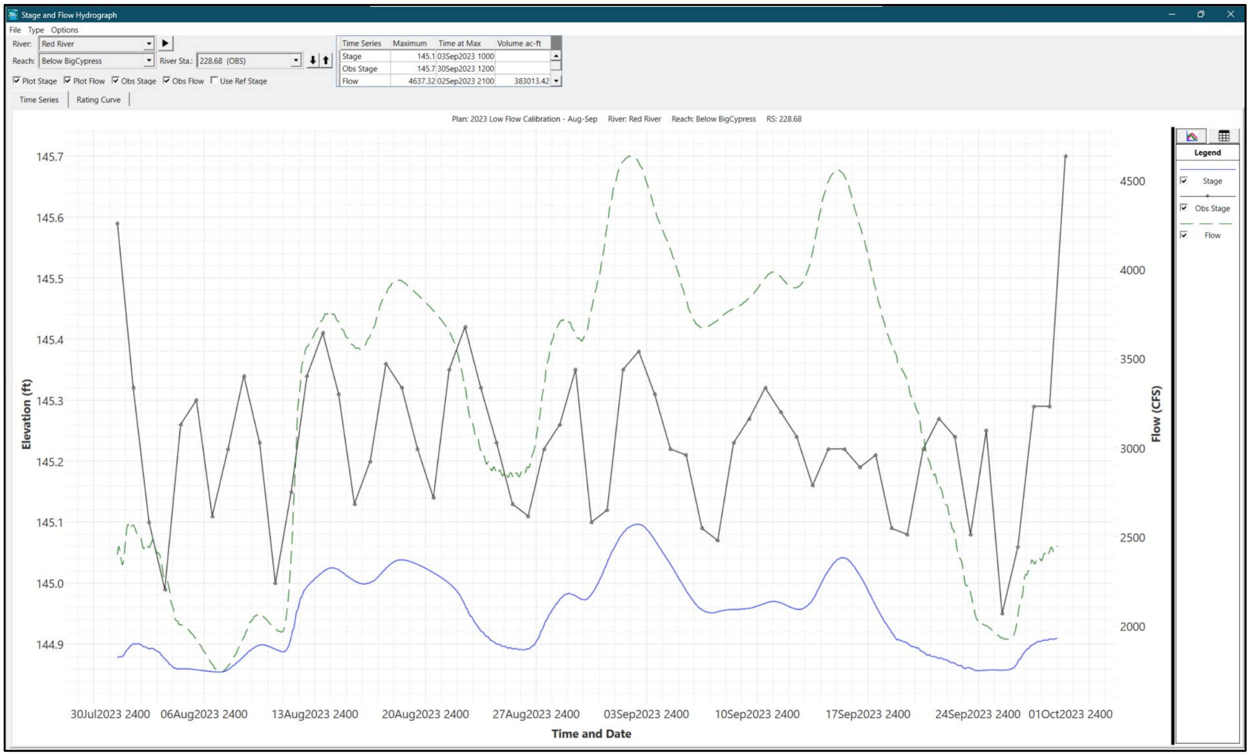



*Figure A-64. 2023 Low Flow Calibration - Shreveport*

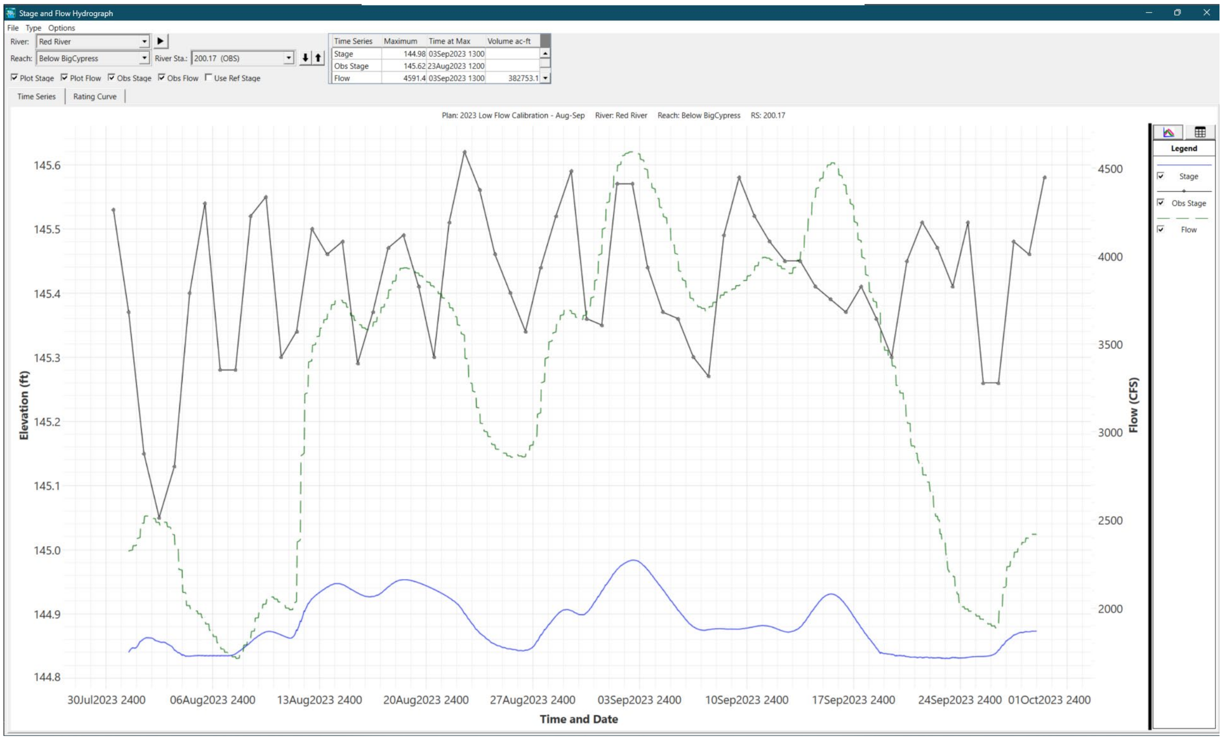



*Figure A-65. 2023 Low Flow Calibration – L&D 5 Headwater*

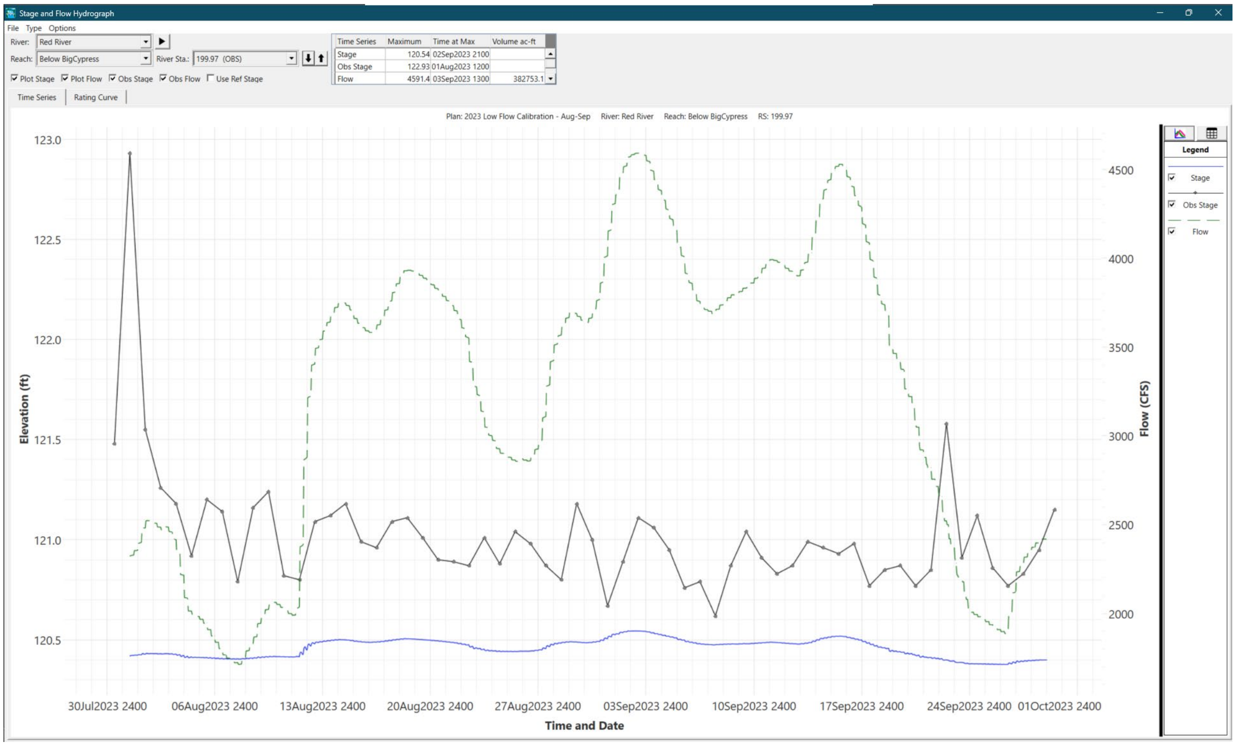



*Figure A-66.2023 Low Flow Calibration – L&D 5 Tailwater*

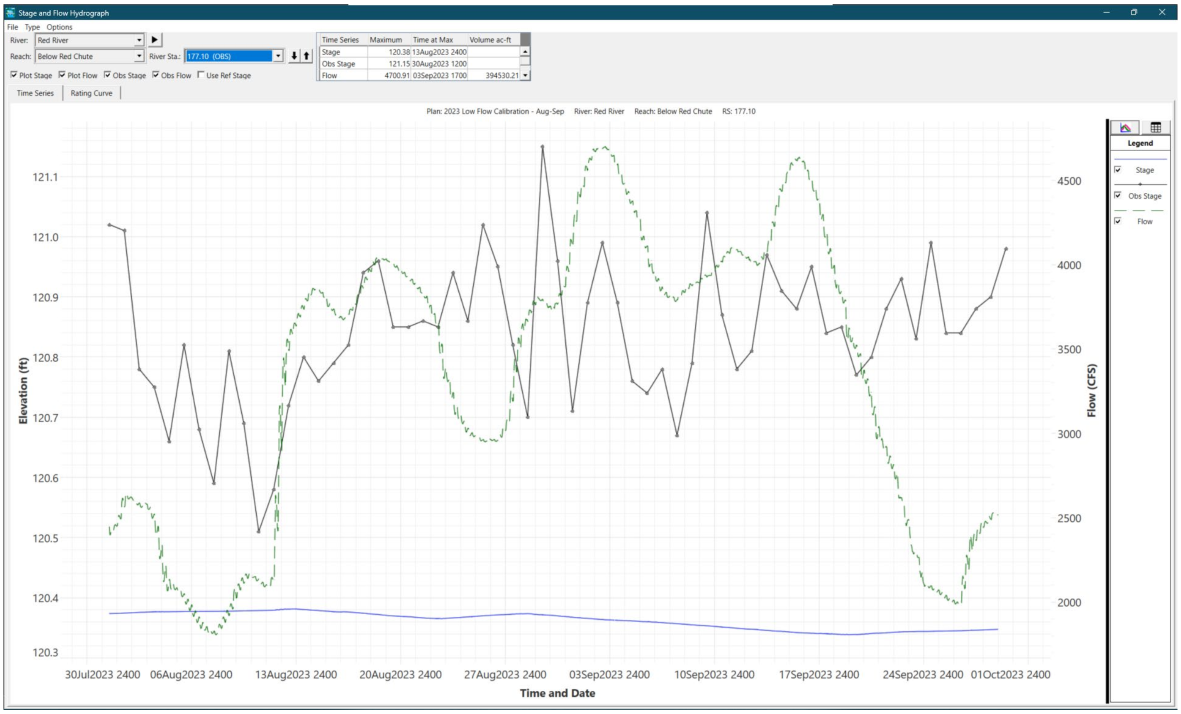



*Figure A-67. 2023 Low Flow Calibration - Coushatta*

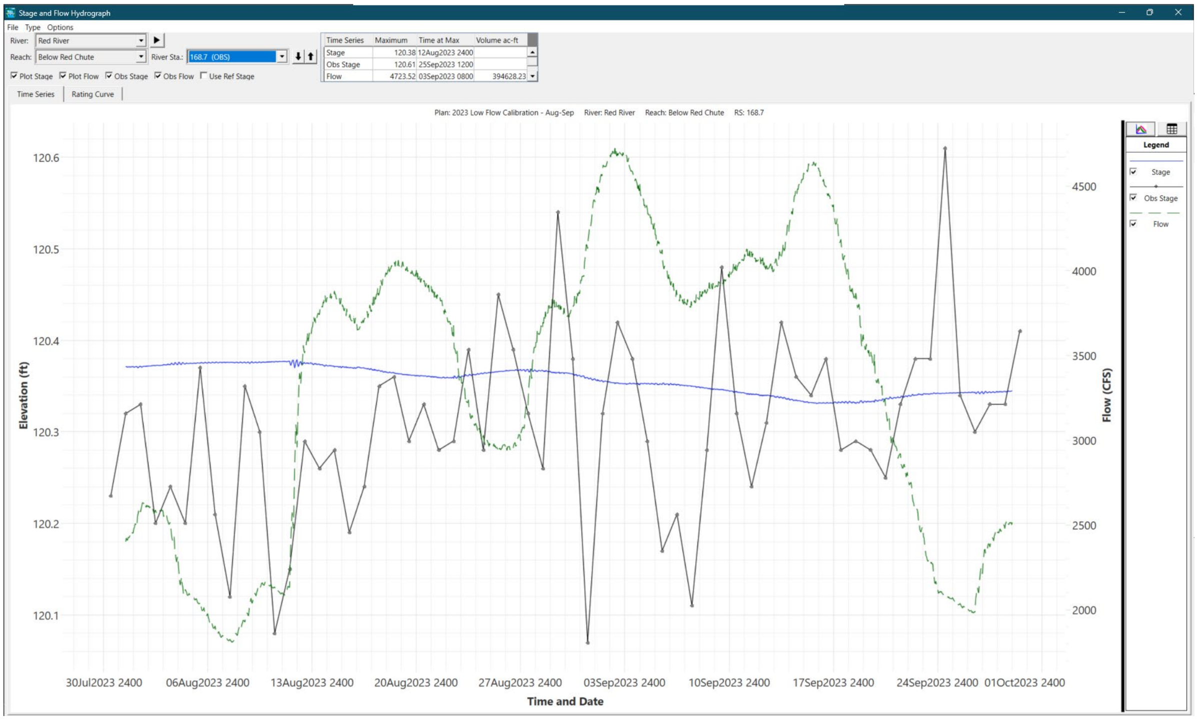



*Figure A-68. 2023 Low Flow Calibration – L&D 4 Headwater*

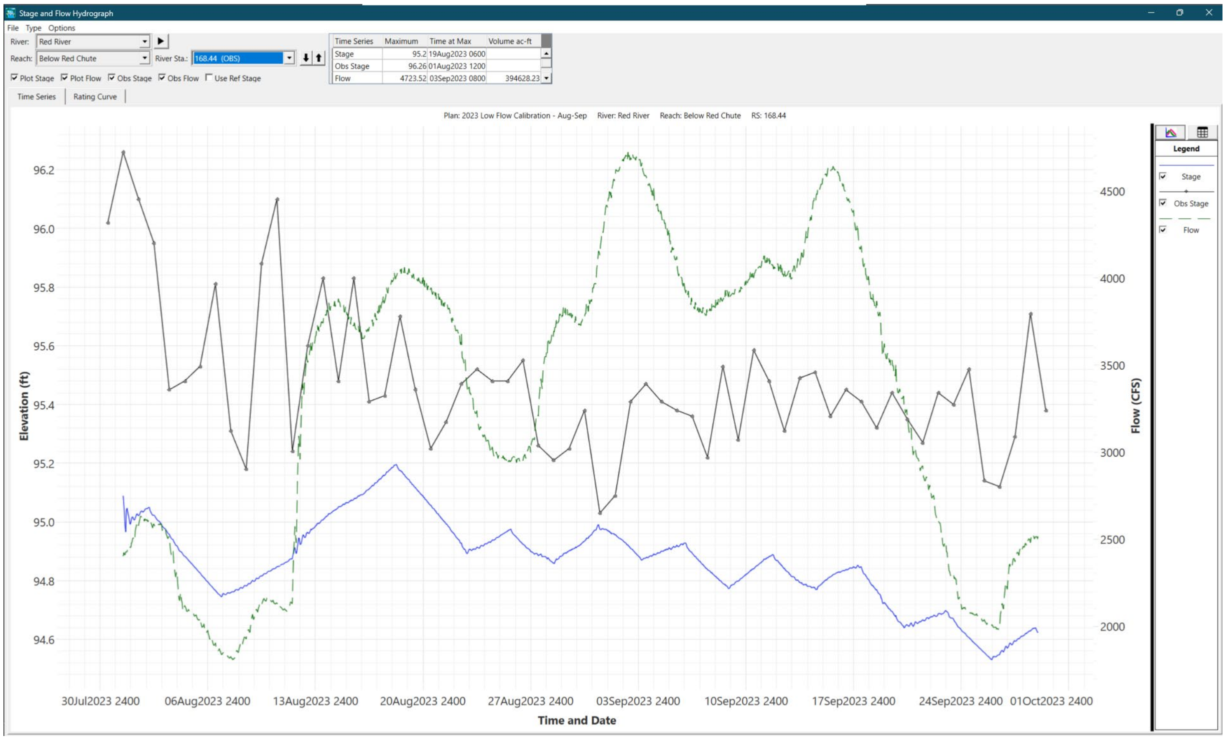



*Figure A-69. 2023 Low Flow Calibration – L&D 4 Tailwater*

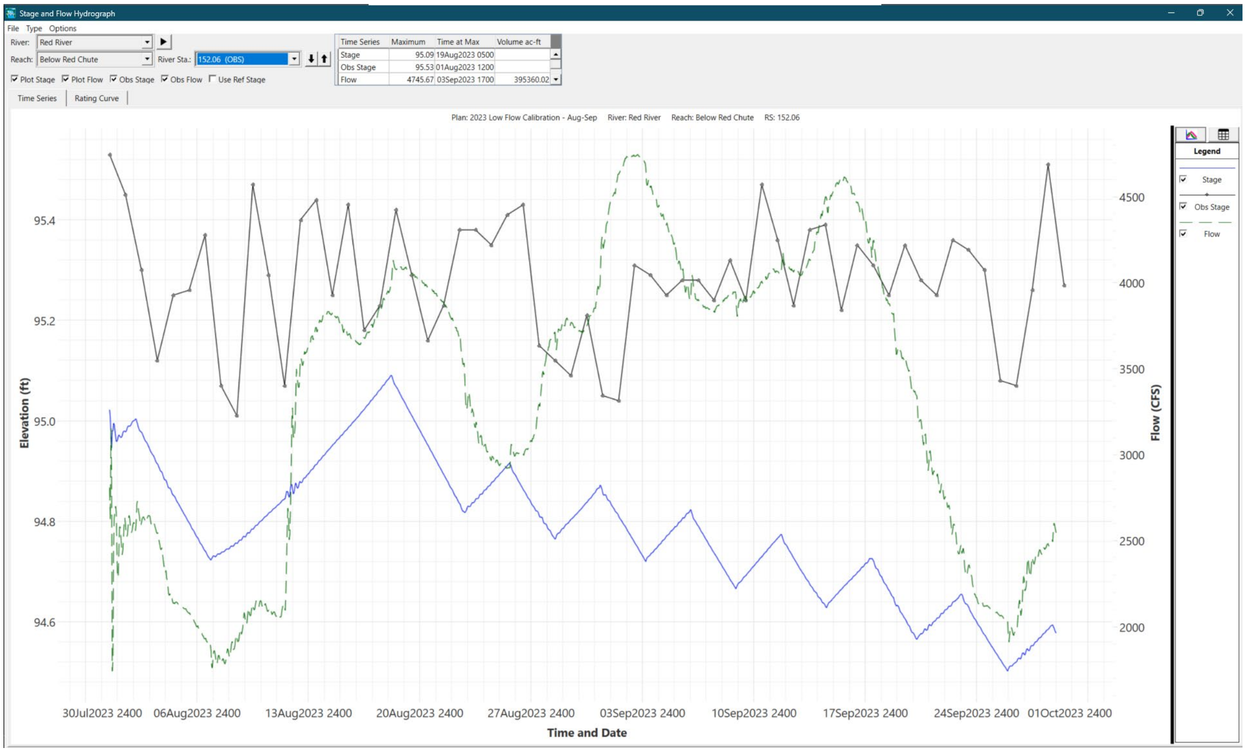



*Figure A-70. 2023 Low Flow Calibration – Grand Ecore*

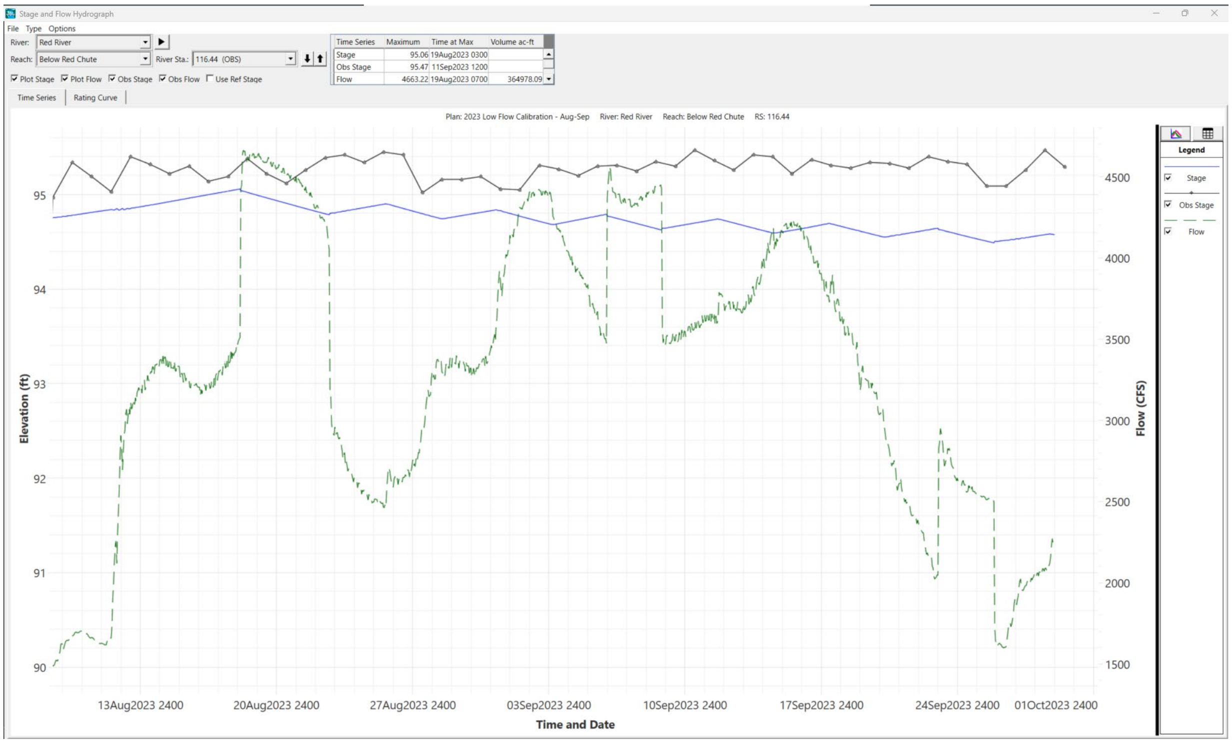



*Figure A-71. 2023 Low Flow Calibration – L&D 3 Headwater*

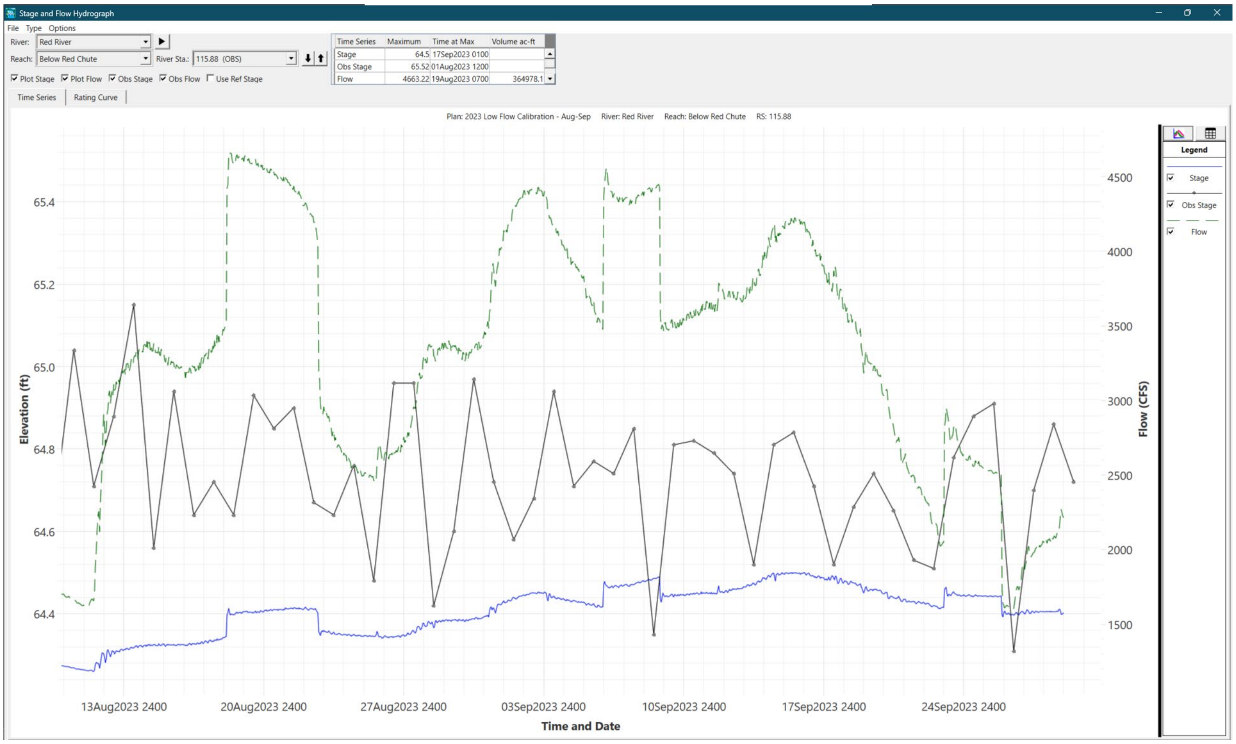



*Figure A-72. 2023 Low Flow Calibration – L&D 3 Tailwater*

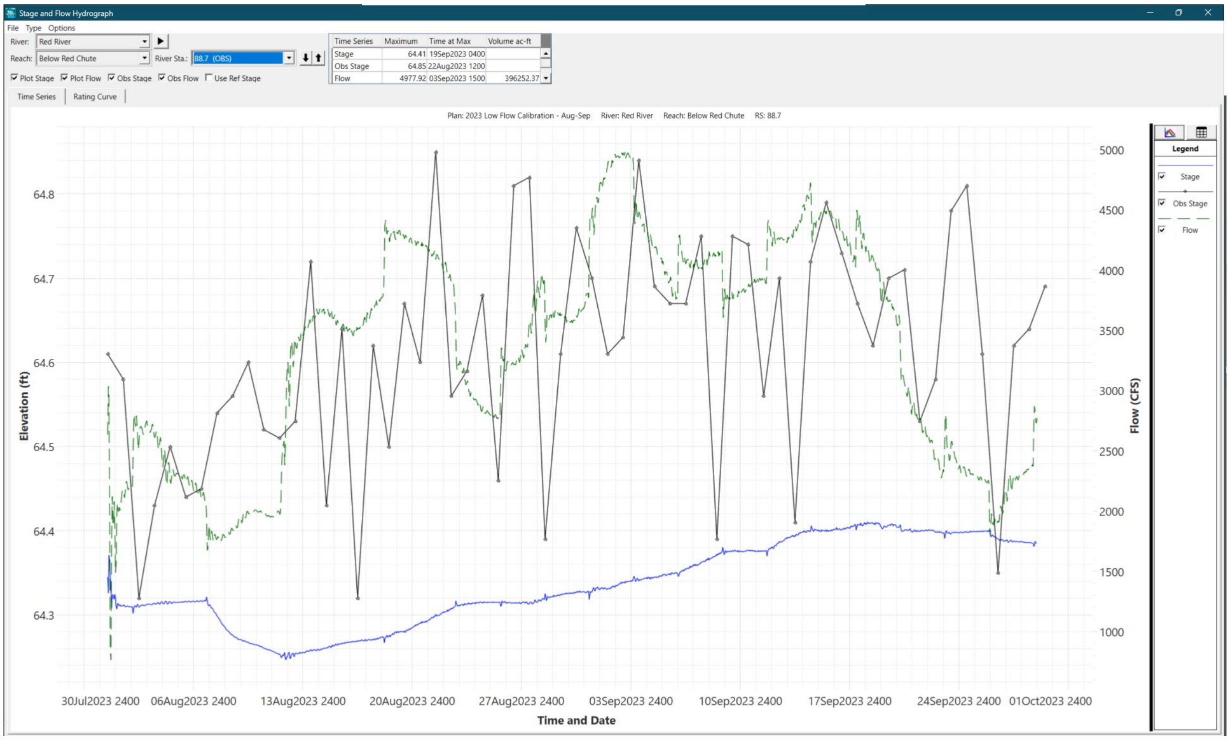



*Figure A-73. 2023 Low Flow Calibration – Alexandria*

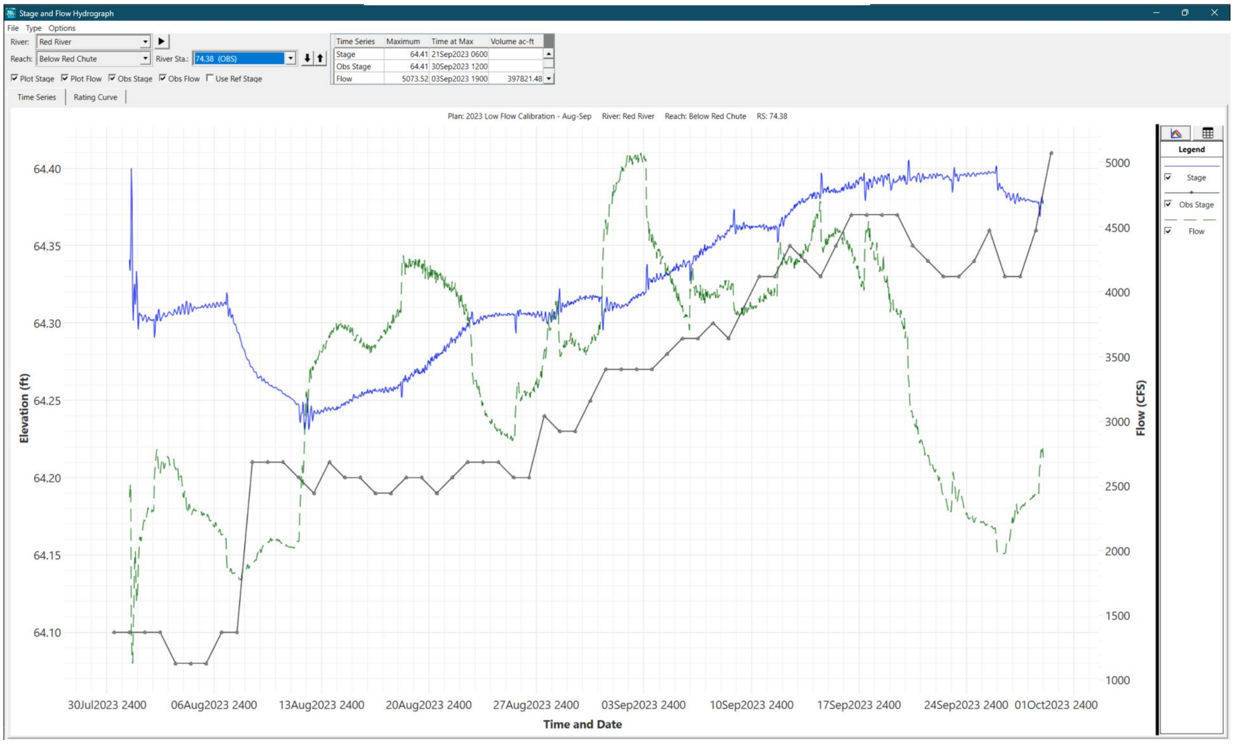



*Figure A-74. 2023 Low Flow Calibration – L&D 2 Headwater*

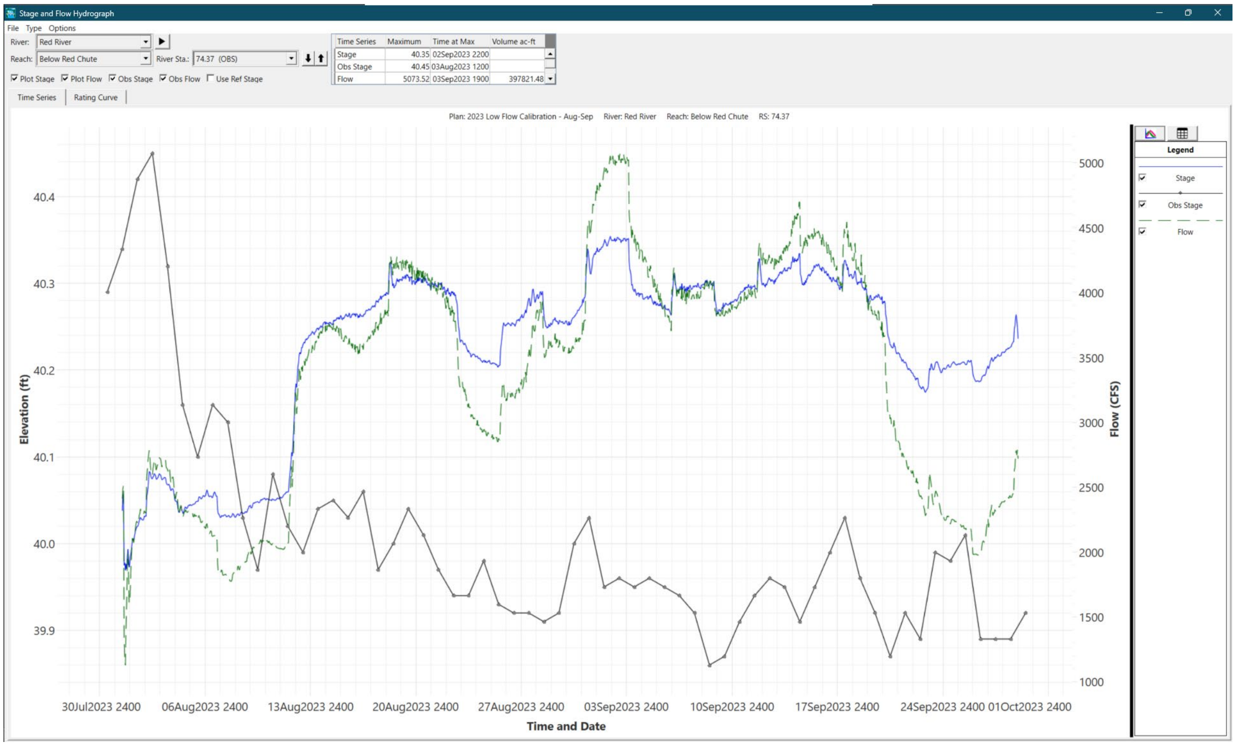



*Figure A-75. 2023 Low Flow Calibration – L&D 2 Tailwater*

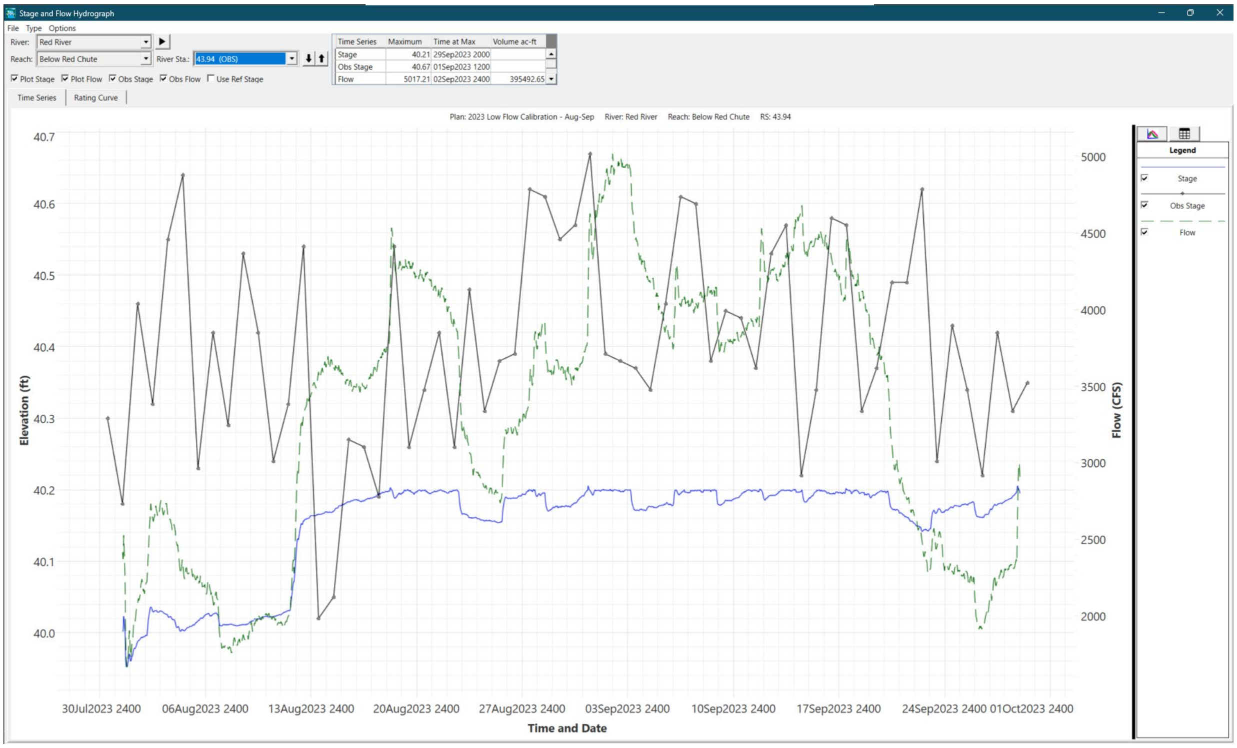


*Figure A-76. 2023 Low Flow Calibration – L&D 1 Headwater*

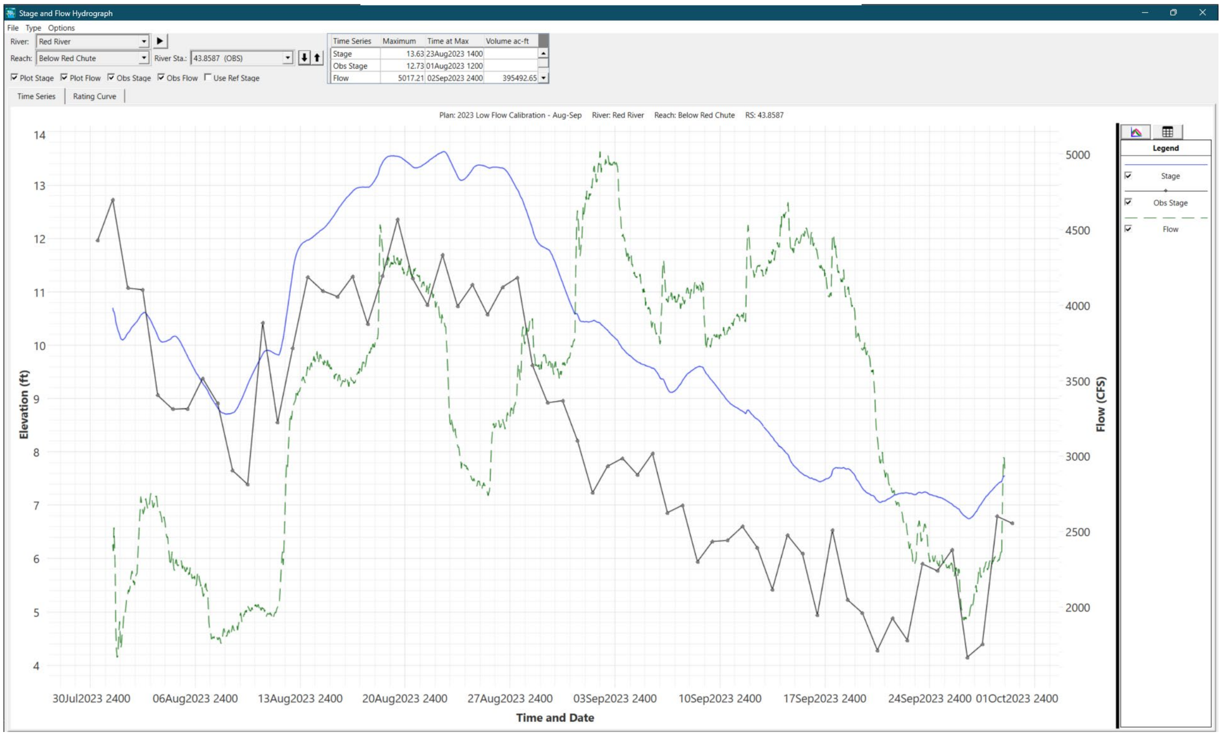


*Figure A-77. 2023 Low Flow Calibration – L&D 1 Tailwater*

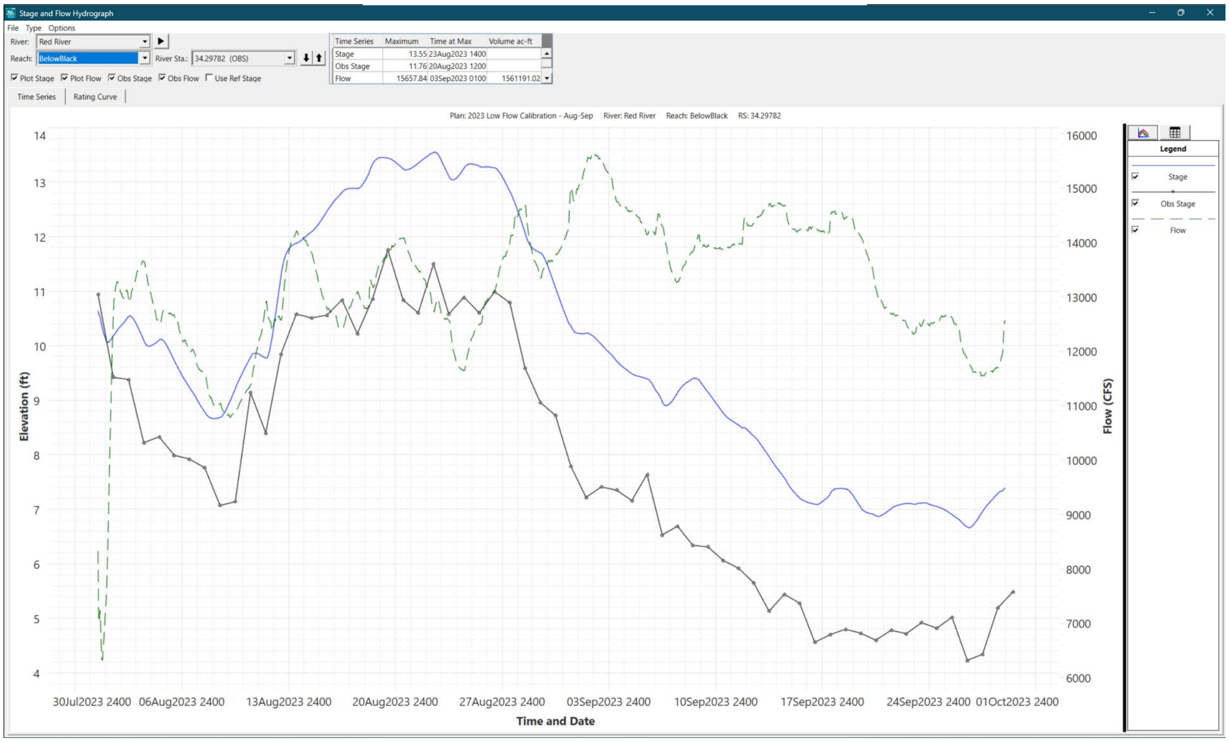



*Figure A-78. 2023 Low Flow Calibration – Acme, Louisiana*

#### 4.4.1.4 Flow Durations and Water Surface Profiles

Table A-18 provides a tabulation of the HEC-RAS simulations used to illustrate the typical water surface profiles and slopes throughout the JBJ Waterway.

*Table A-18. HEC-RAS Flows and Stages Used for HEC-RAS Water Surface Profile Simulations*

| Location | **Normal Pool with 98% DEP Flows** | | **Average Flows** | | **Open River Conditions based on L&D 1** | |
|---|---|---|---|---|---|---|
| | Water Surface Elevation (feet NAVD88) | Flow (cfs) | Water Surface Elevation (feet NAVD88) | Flow (cfs) | Water Surface Elevation (feet NAVD88) | Flow (cfs) |
| Shreveport (RM 228) | 145.0 | 1,600 | 147.6 | 27,000 | 155.3 | 75,000 |
| L&D 5 (RM 200) | 145.0 | 1,600 | 145.0 | 27,000 | 145.0 | 75,000 |
| Coushatta (RM 177) | 120.0 | 1,700 | 120.7 | 27,000 | 122.8 | 76,000 |
| L&D 4 (RM 168) | 120.0 | 1,700 | 120.0 | 27,000 | 120.0 | 76,000 |
| Grand Ecore (RM 152) | 95.0 | 1,700 | 96.6 | 33,000 | 100.8 | 77,000 |
| L&D 3 (RM 116) | 95.0 | 1,700 | 95.0 | 33,000 | 90.0 | 77,000 |
| Alexandria (RM 88) | 64.0 | 1,700 | 65.0 | 32,000 | 67.6 | 77,000 |
| L&D 2 (RM 74) | 64.0 | 1,700 | 64.0 | 32,000 | 64.0 | 77,000 |
| L&D 1 (RM 40) | 40.0 | 1,700 | 40.0 | 32,000 | 40.0 | 77,000 |
| Acme (RM 34) | 4.0 | 4,000* | 11.0 | 35,000 | 18.6 | 79,000 |

The model was simulated using unsteady flow simulations but assuming steady state inflow conditions with a simulation time set long enough to achieve constant stages and flows. The flow targets are based on the Shreveport and Alexandria gage statistics regarding the 98% DEP and Average Flows. The Open River flow simulation is based on the documented open river flow at L&D 1, which is 72,000 cfs. The open river flow conditions vary throughout the waterway so just achieving the open river flow at L&D 1 was completed to provide illustrations of the sloping water surface profile throughout the waterway during high flow conditions. The Open River simulation shows the hinge pool in operation at L&D 3 with the pool being lowered when 40,000 and 50,000 cfs are exceeded.

*Acme has backwater impacts and, therefore, experiences a nuanced stage and flow relationship.

<u>Shreveport and Alexandria Flow Durations</u>

The Vicksburg District Water Management Section provided POR for the Shreveport and Alexandria gages containing daily stage and flow data spanning from 1935 to 2024. Utilizing the flow data and HEC-DSS, the 98 percent DEP flows and the average flows were determined. These flows were used to inform the HEC-RAS model simulations to create comparative water surface profiles along the waterway as visual illustrations for the PDT and readers of this report. The 98 percent DEP flows are considered low flows and used as project design conditions for the JBJ Waterway. The DEP can be described as percentage of time that a given value is exceeded on an annual basis; therefore, the 98 percent flows are flows are the exceeded approximately 98 percent of the time, meaning that flows could be lower the other 2 percent of the time. The Shreveport and Alexandria average flows are approximately 26,000 cfs and 35,000 cfs, respectively. Excel was used to calculate the median flows at Shreveport and Alexandria as approximately 14,000 cfs and 26,000 cfs. HEC-DSS calculates the Shreveport and Alexandria 98 percent DEP flows as approximately 1,700 cfs and 2,100 cfs, respectively.

Figure A-79 provides a daily flow hydrograph of the Shreveport and Alexandria gages, and Figure A-82 provides a daily flow hydrograph of the Acme gage. Figure A-80 and Figure A-81 provide the HEC-DSS DEP plots of the Shreveport and Alexandria gages, and Figure A-83 provides a plot for the Acme gage.

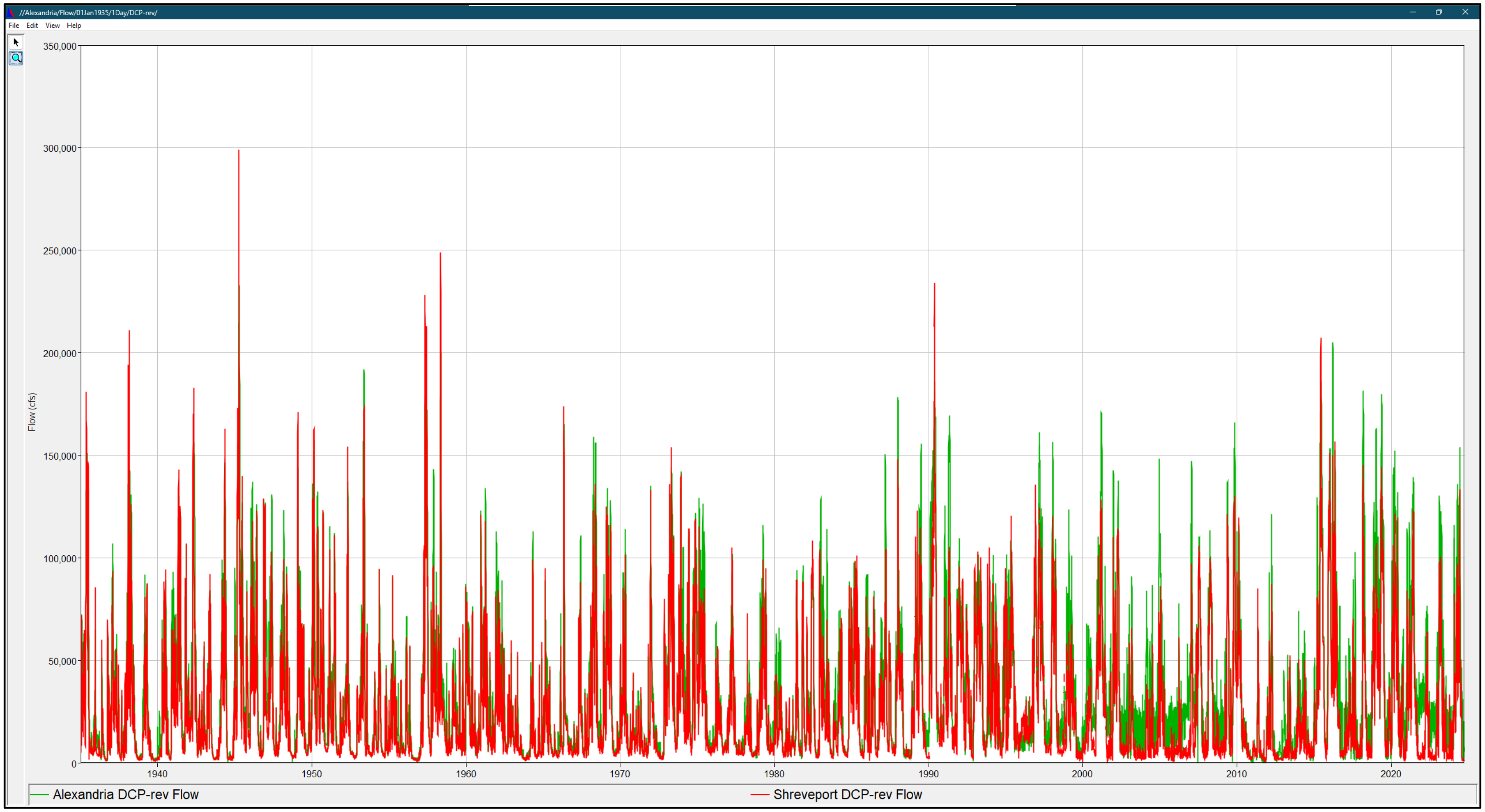
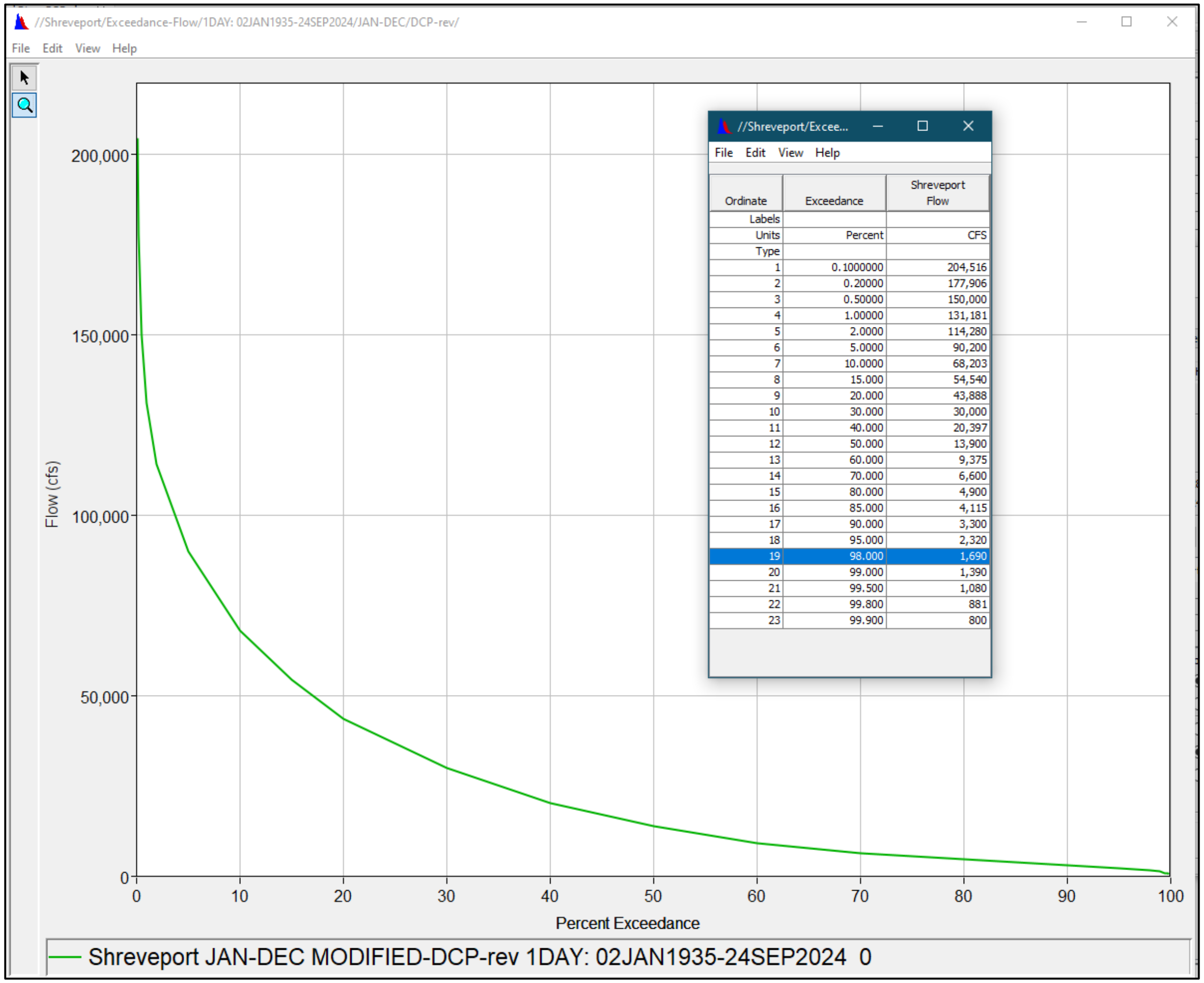



*Figure A-79. Shreveport and Alexandria Daily Flows (1935–2024)*




| Ordinate | Exceedance | Shreveport Flow |
|---|---|---|
| Labels | | |
| Units | Percent | CFS |
| Type | | |
| 1 | 0.1000000 | 204,516 |
| 2 | 0.20000 | 177,906 |
| 3 | 0.50000 | 150,000 |
| 4 | 1.00000 | 131,181 |
| 5 | 2.0000 | 114,280 |
| 6 | 5.0000 | 90,200 |
| 7 | 10.0000 | 68,203 |
| 8 | 15.000 | 54,540 |
| 9 | 20.000 | 43,888 |
| 10 | 30.000 | 30,000 |
| 11 | 40.000 | 20,397 |
| 12 | 50.000 | 13,900 |
| 13 | 60.000 | 9,375 |
| 14 | 70.000 | 6,600 |
| 15 | 80.000 | 4,900 |
| 16 | 85.000 | 4,115 |
| 17 | 90.000 | 3,300 |
| 18 | 95.000 | 2,320 |
| 19 | 98.000 | 1,690 |
| 20 | 99.000 | 1,390 |
| 21 | 99.500 | 1,080 |
| 22 | 99.800 | 881 |
| 23 | 99.900 | 800 |

*Figure A-80. Shreveport Flow DEP (1935–2024 Daily Flows)*

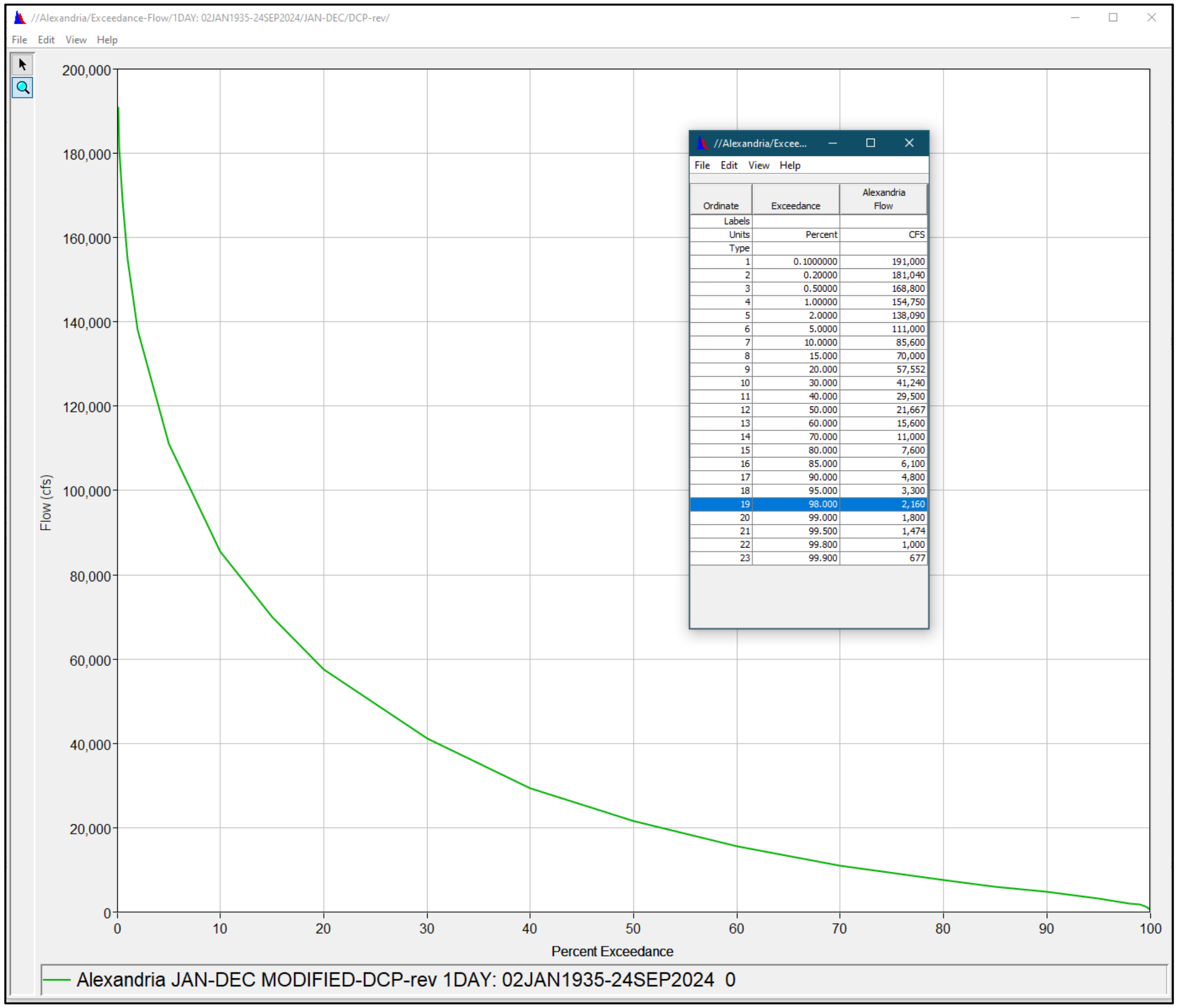



*Figure A-81. Alexandria Flow DEP (1935–2024 Daily Flows)*

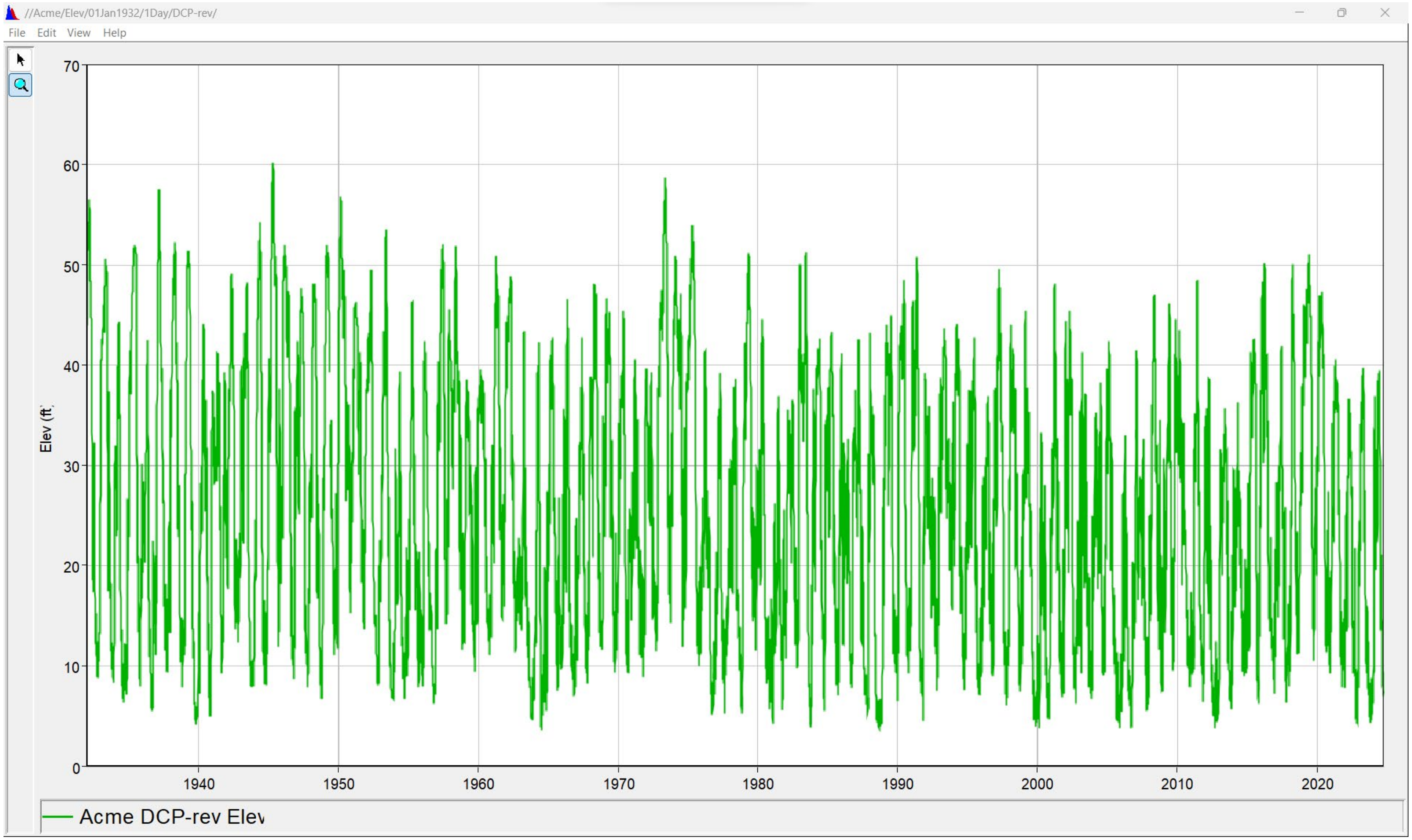



*Figure A-82. Acme Daily Water Surface Elevation (1935–2024)*

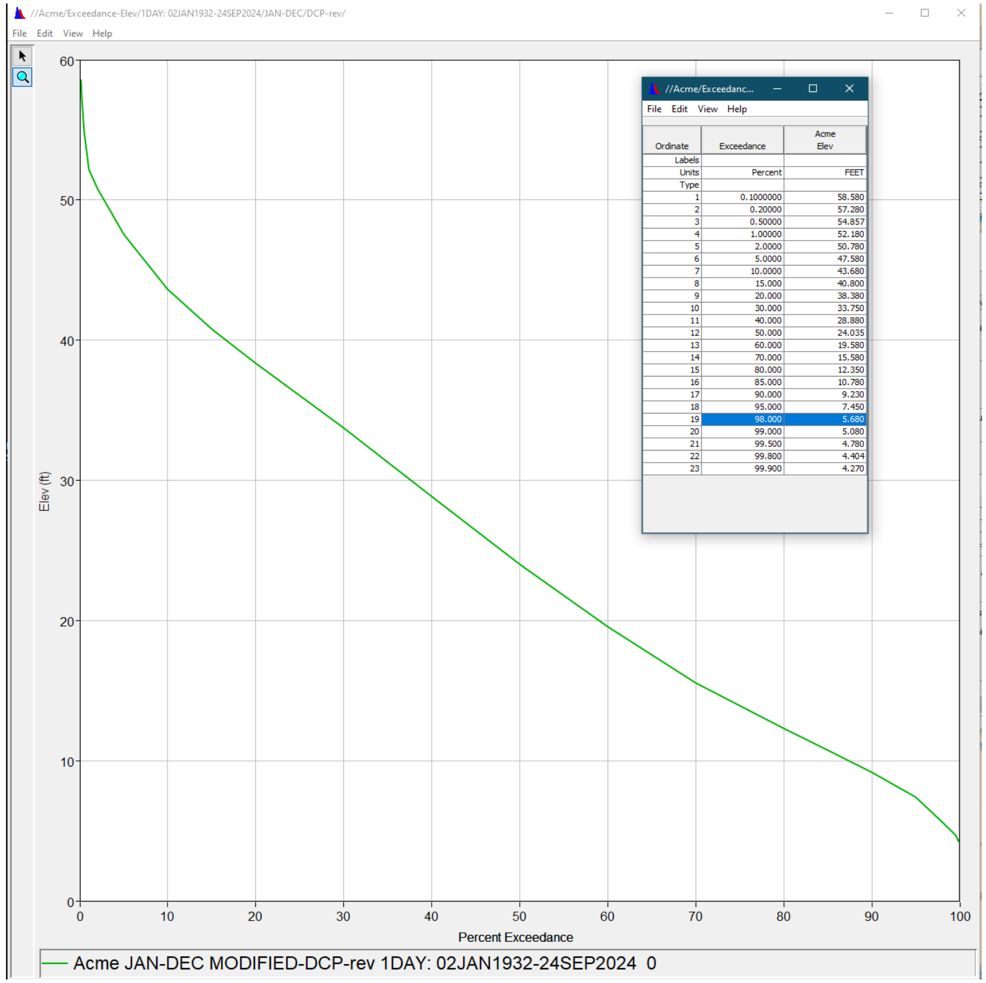



*Figure A-83. Acme Water Surface Elevation DEP (1932–2024 Daily Flows)*

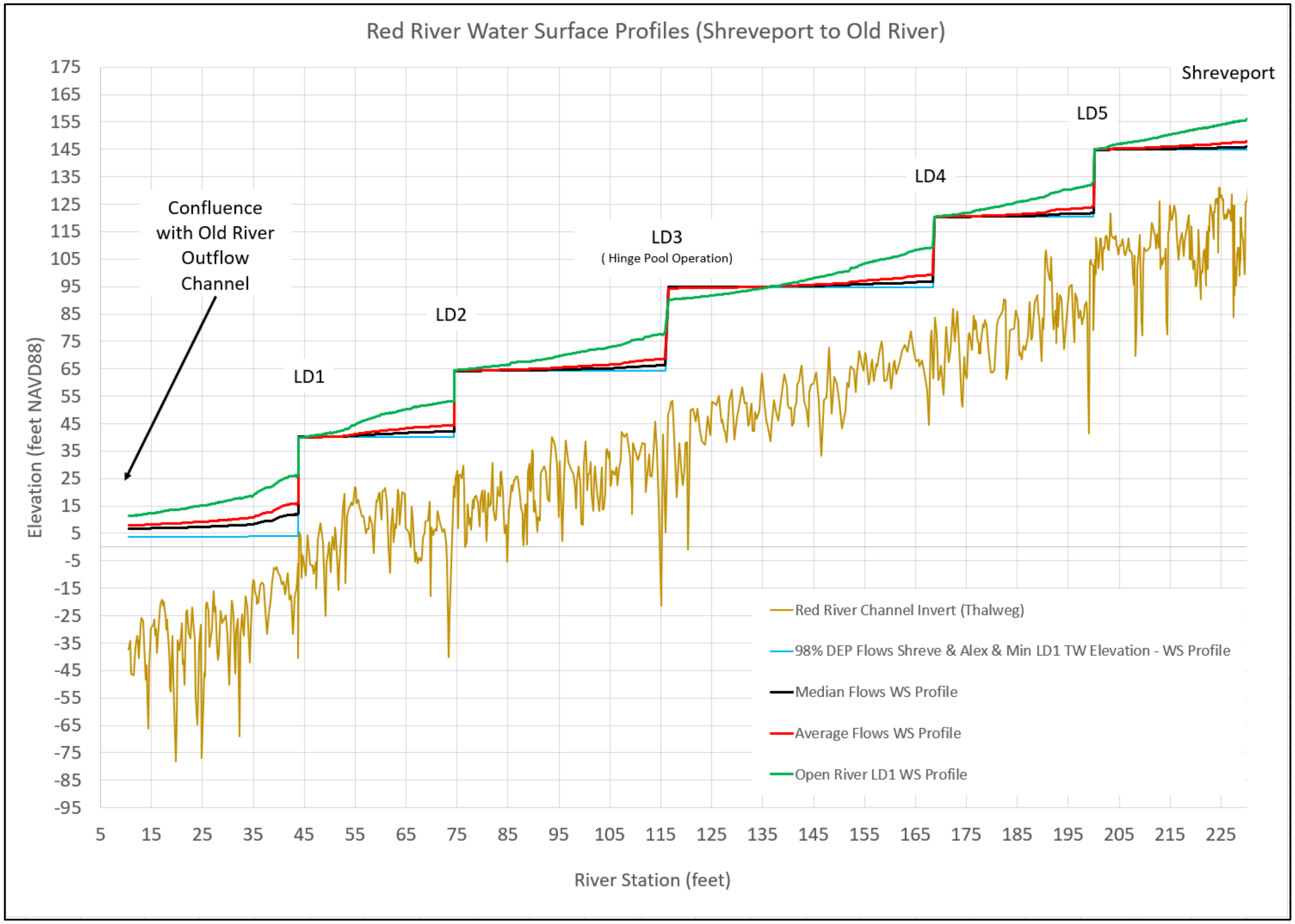



*Figure A-84. JBJ Waterway – Typical Water Surface Profiles*

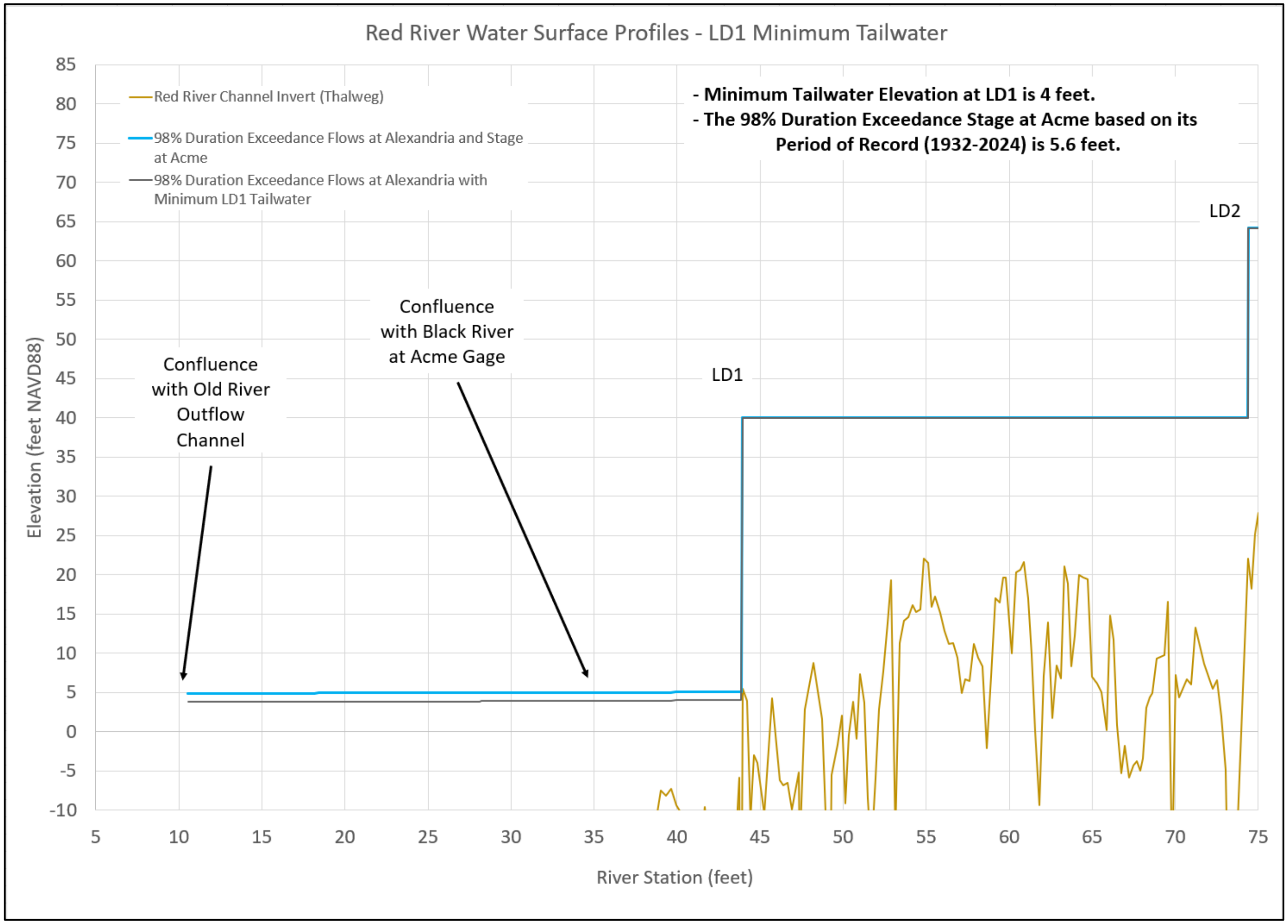



*Figure A-85. Low Water Surface Profiles Below L&D 1*

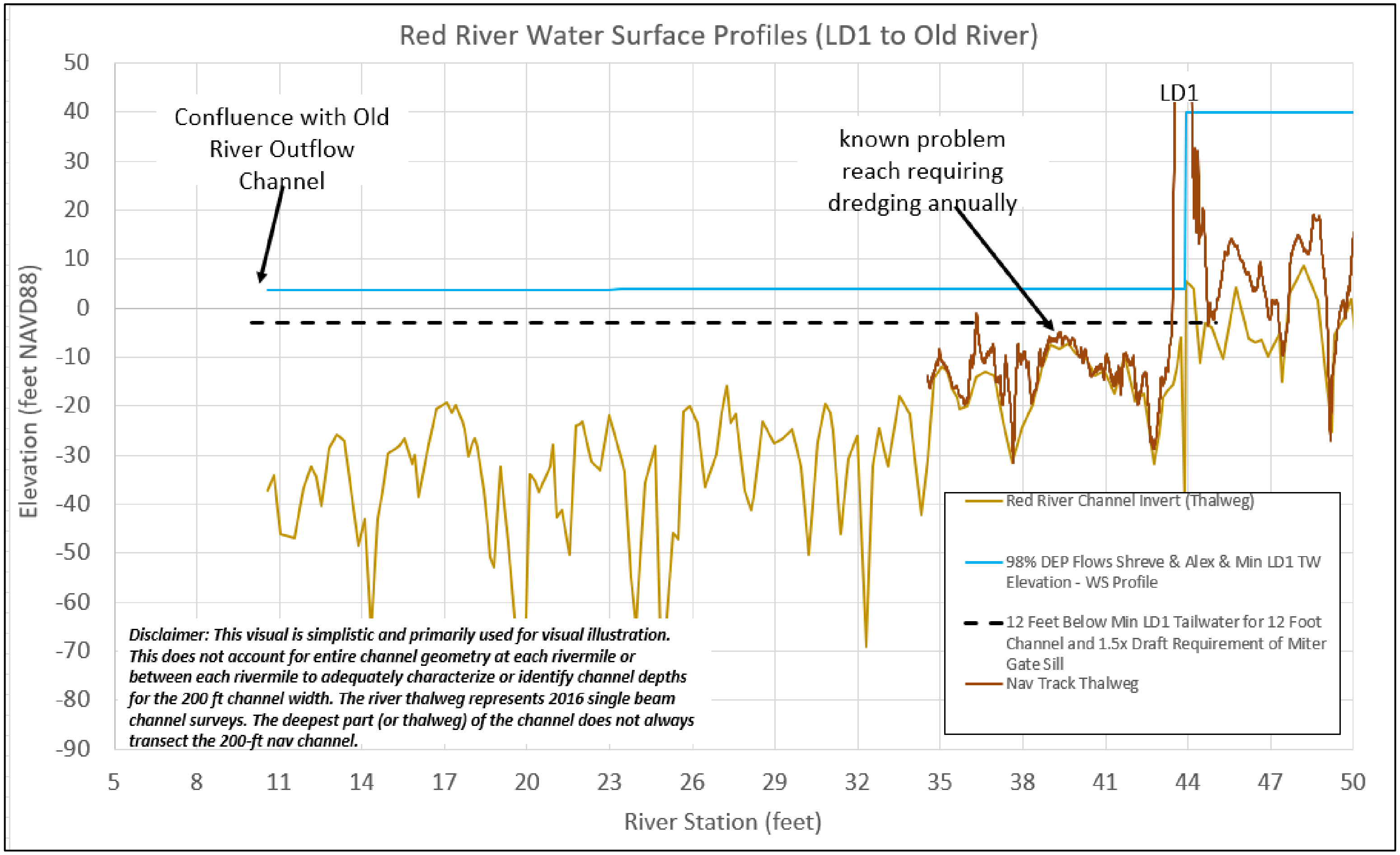



*Figure A-86. Water Surface Profiles Relative to 2016 Single-Beam Survey Thalweg Below L&D 1*

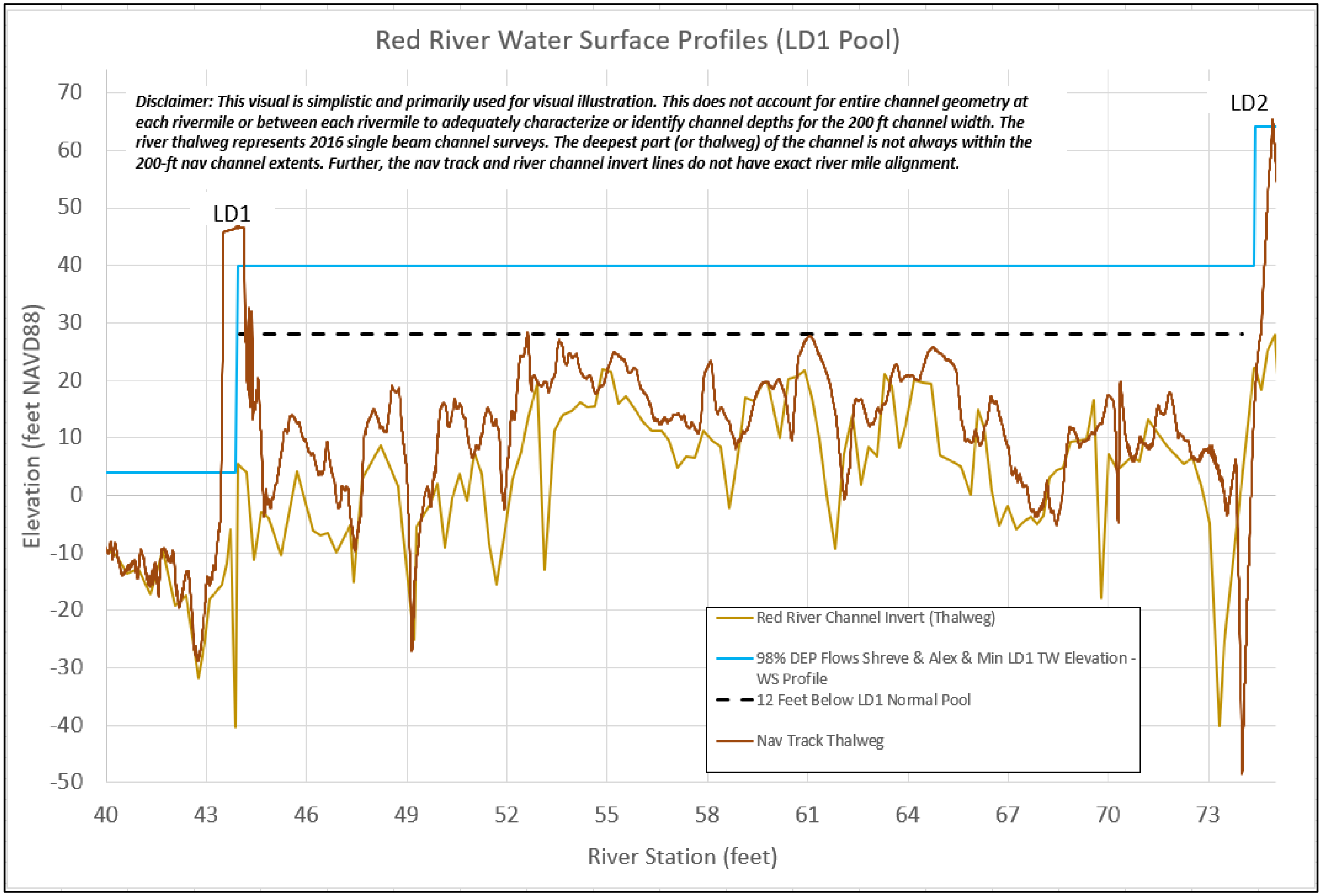



*Figure A-87. Water Surface Profiles Relative to 2016 Single-Beam Survey Thalweg in Pool 1*

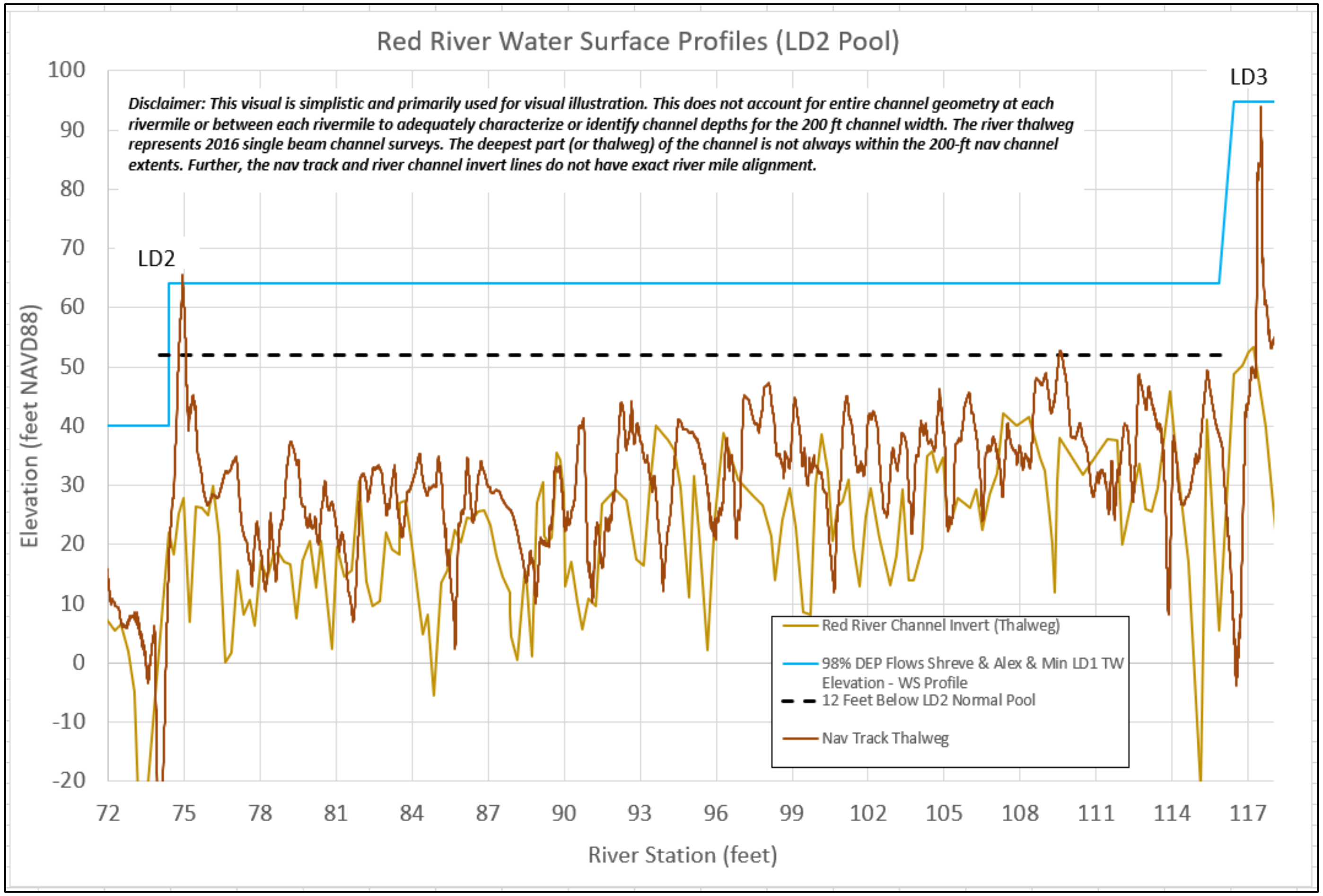



*Figure A-88. Water Surface Profiles Relative to 2016 Single-beam Survey Thalweg in Pool 2*

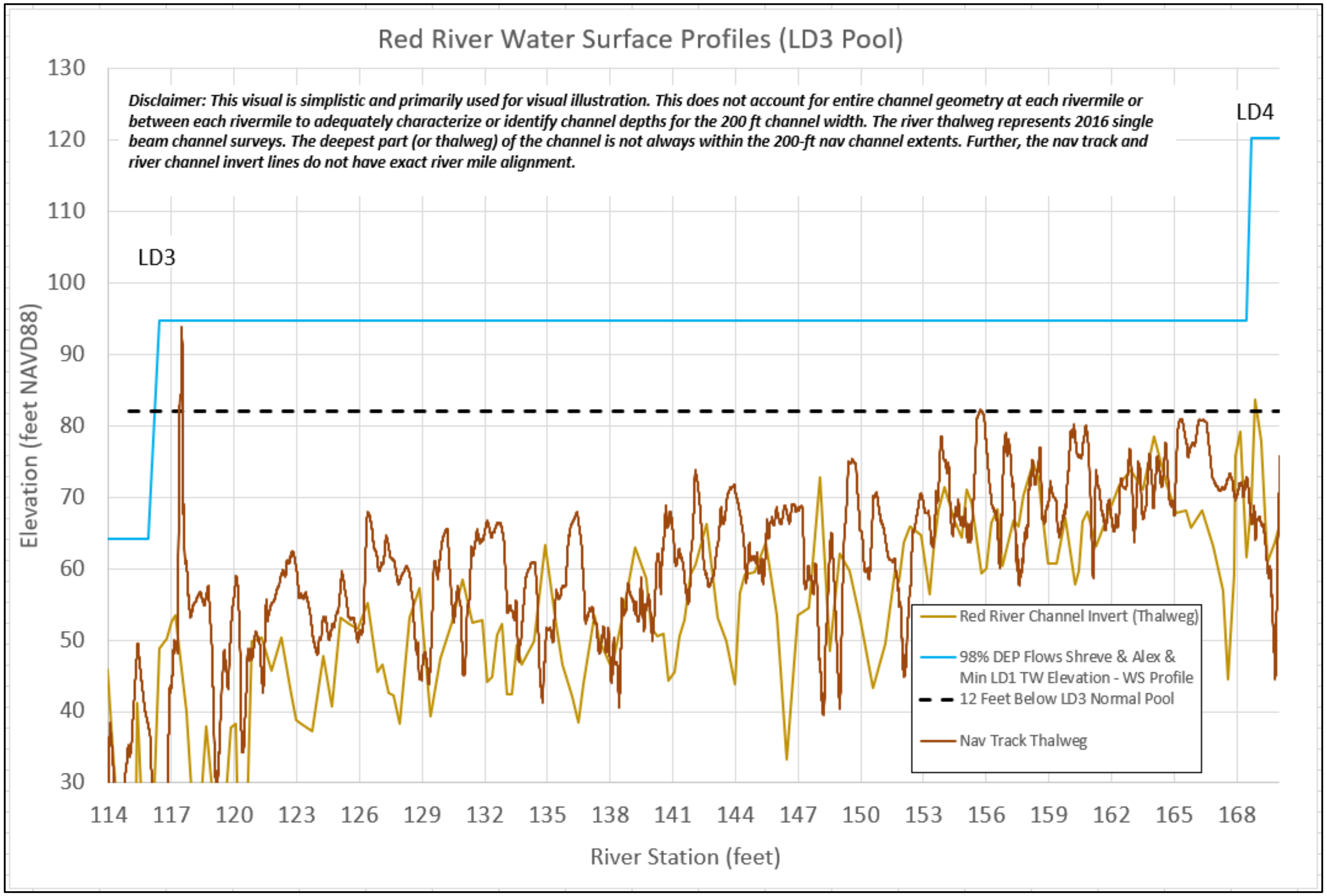



*Figure A-89. Water Surface Profiles Relative to 2016 Single-Beam Survey Thalweg in Pool 3*

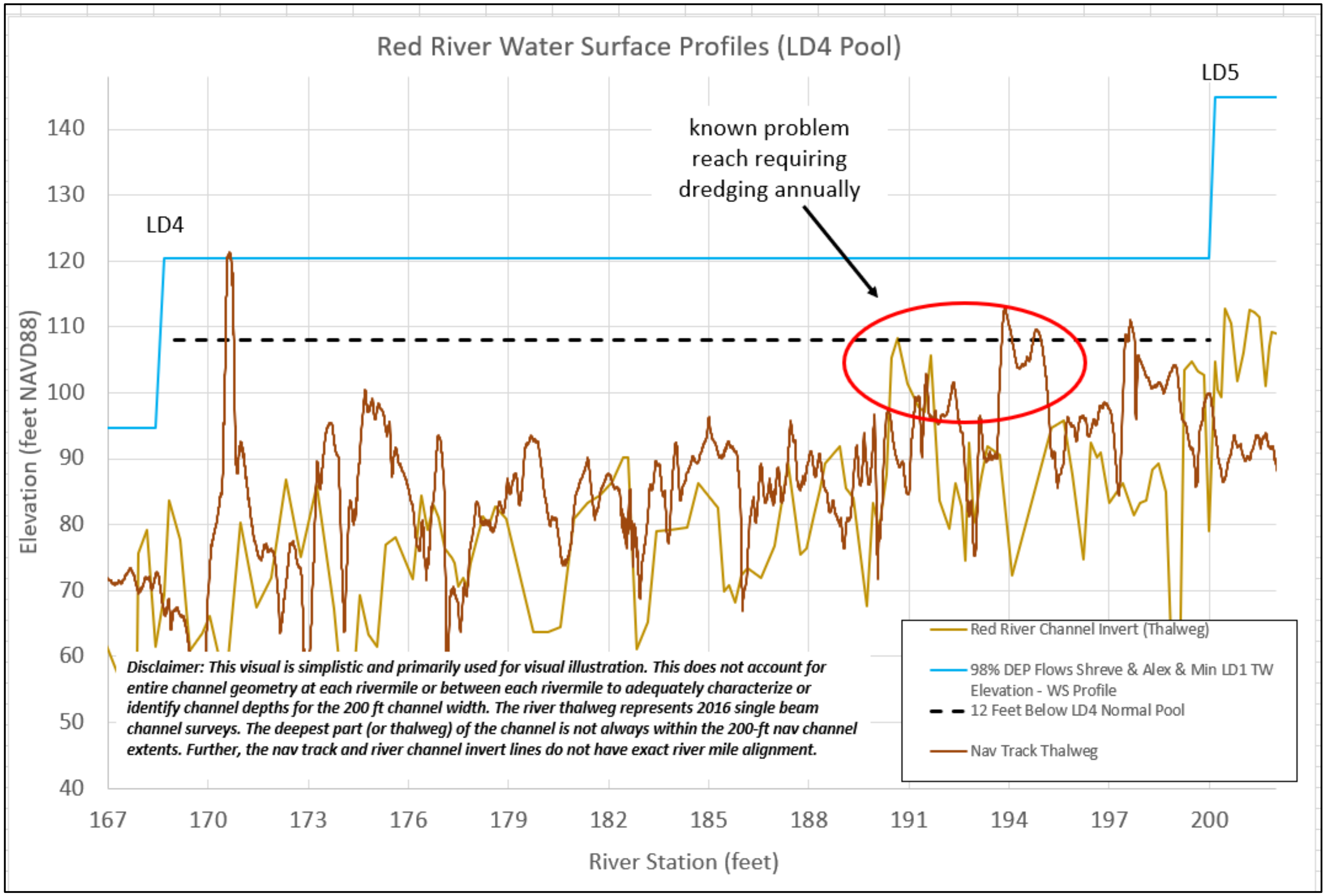



*Figure A-90. Water Surface Profiles Relative to 2016 Single-Beam Survey Thalweg in Pool 4*

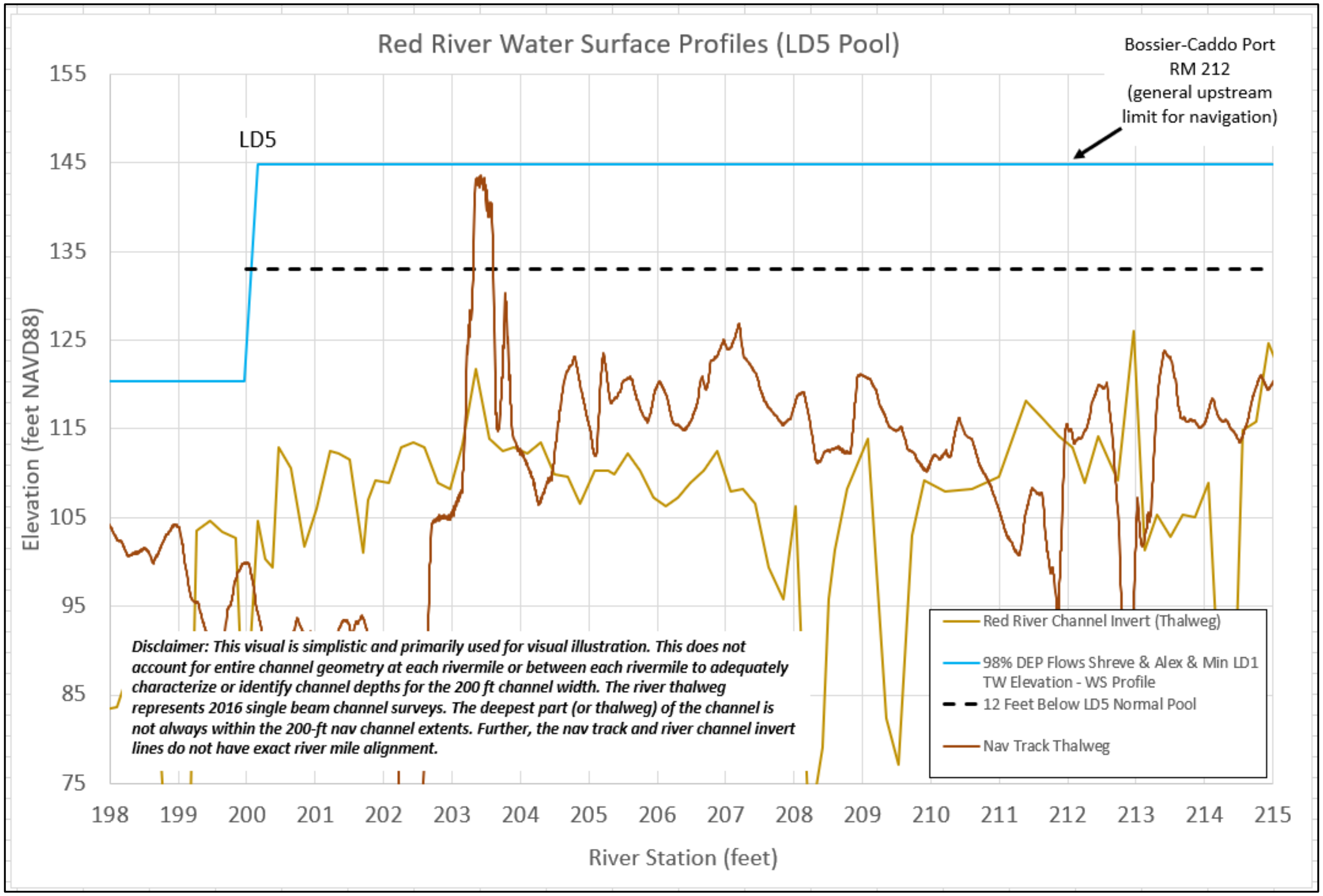



*Figure A-91. Water Surface Profiles Relative to 2016 Single-Beam Survey Thalweg in Pool 5*

#### 4.4.2 Channel Depths Findings

Of the roughly 230 miles of navigable river channel from the Mississippi River at ORCC (RM 0) to the Shreveport area (RMs 228–230) a large majority of the problems and/or challenges of maintaining the 9-FT navigation channel with mechanical dredging occur at the lock and dam approaches with a few areas outside of the lock and dam extents also requiring annual or occasional dredging to maintain the 9-FT channel. Utilizing existing dredge records from 2018–2024, an existing HEC-RAS 1D hydraulic model, discussions among Hydraulics, River Stabilization, and River Operations, and the 2023 Red River Priority Repair list for bank stabilization and dikes, potential problem areas were identified for a 9-FT, 12-FT, and 15-FT channel. The initial problem areas were determined from dredge records and discussions, and the hydraulic model was used to validate these areas do indeed have inadequate depths at normal pool with minimal flow and at low stages below L&D 1. The problem areas were prioritized considering the recurrence of dredging, location within the waterway, and statistical analyses illustrating the percentage of time that a given stage (below L&D 1) or flow (in pools) was available to characterize how often the channel depth problem could exist.

Many of the problem areas exist in reaches where existing bank stabilization and contraction works have deteriorated over time. The initial assessment considers that, if rehabilitating the structures back to original design dimensions, the channel depth problems could lessen from the existing conditions or in some cases likely become satisfactory for the 12-FT channel.

The project design conditions are considered to be the normal pool within each of the five pools with accompanying inflows approximately the 98 percent DEP. These inflows are minimal; therefore, a flat pool is held throughout each pool with minimal flows being passed through the lock structure Tainter gates.

The design conditions downstream of L&D 1 are nuanced in that this reach is not controlled by any downstream lock and dam. Further, there is a requirement for draft over lock and dam miter gate sills of 1.5x the authorized channel draft (See Section 2.3.3) to allow vessels to safely enter and exit the lock chambers as water sloshing occurs during the entering and exiting process and water depths vary. Currently, the L&D 1 lower miter gate sill elevation is -9 feet NAVD88. Therefore, the current 9-FT channel requires a tailwater water surface elevation of 4 feet per the L&D 1 Water Control Manual and to satisfy the 1.5x draft requirement over the miter gate sills. For a 12-FT channel, a tailwater water surface elevation of 9 feet would be required to satisfy the 1.5x draft requirement for a 12-FT channel. Therefore, it may be irrational to maintain a 12-FT channel depth in the reaches below L&D 1 at water surface elevations below 9 feet, if during this period the 1.5x draft requirement for a 12-FT channel is not met and barges would need to light load to even pass through the first lock and dam (L&D 1) upon coming off the Mississippi River. Notably, the JBJ 12-FT Channel PDT is pursuing a waiver of the 1.5x draft to reduce the amount of time that Locks 1 and 2 would be under draft restrictions. This does not change the question of determining the “low water reference plane” downstream of L&D 1 for which to design the channel reach for the 12-FT channel. Overall, the conservative approach would be to optimize this reach with river training structures such that during extreme low water periods such as the current minimum tailwater (4 feet NAVD88) required at L&D 1, a 12-FT channel would be available within the reach. However, the economics may not justify such an approach. During the period of time for which a 12-FT channel barge could pass through L&D 1 because the 1.5x draft is available over the lower miter gate sill, it is likely that there is adequate channel depth within this reach simply due to the higher water surface elevation. Referencing the stage duration statistical analysis at the L&D 1 tailwater documented within Section 2.3.2, under existing conditions a 12-FT channel is likely available within this reach approximately 88–90 percent of the time while the 1.5x 12-FT channel draft requirement is also available approximately 88–90 percent of the time.

| Existing Conditions - Flow Required to Provide Channel Depths at Identified Potential Problem Areas | | | | | | | |
|---|---|---|---|---|---|---|---|
| Location | | 9 ft. | | 12 ft. | | 15 ft. | |
| Pool | River Mile | Flow Required (cfs) | % Time Available (Annual Basis) | Flow Required (cfs) | % Time Available (Annual Basis) | Flow Required (cfs) | % Time Available (Annual Basis) |
| Lock 1 Pool (Dam RM 44) | 52-53 | 800 | 100 | 40000 | 28 | 70000 | 15 |
| | 60-61 | 800 | 100 | 25000 | 45 | 40000 | 32 |
| | 64-65 | 800 | 100 | 800 | 100 | 20000 | 53 |
| | | | | | | | |
| Lock 2 Pool (Dam RM 74) | 108 - 109 | 800 | 100 | 25000 | 45 | 40000 | 32 |
| | | | | | | | |
| Lock 3 Pool (Dam RM 115) | *154 | 800 | 100 | 1600 | 100 | 40000 | 29 |
| | *158 | 800 | 100 | 5000 | 85 | 40000 | 29 |
| | 163-164 | 800 | 100 | 1600 | 100 | 30000 | 35 |
| | 164-165 | 800 | 100 | 1600 | 100 | 15000 | 47 |
| | | | | | | | |
| Lock 4 Pool (Dam RM 168) | *191 | 40000 | 28 | 70000 | 15 | 100000 | 7 |
| | *194 | 14000 | 55 | 25000 | 37 | 50000 | 25 |
| | | | | | | | |
| Notes | An existing HEC-RAS model was used to simulate project low flow conditions (98% DEP) and a resulting depth grid color coded to show areas of potentially inadequate depths. Areas of less than 15ft of depth at the low flow conditions were flagged for further investigation. The primary flow targets in the simulation were to achieve steady flows of approximately 1700 cfs and 2100 cfs at the Shreveport and Alexandria gages, respectively. These period of records were considered as daily flows are available at both gages from 1935-2024. | | | | | | |
| | *Potential problem areas initially identified per historical dredge records, for which existing dredging is completed to provide the 9-foot channel. Noted that the HEC-RAS model simulated with 98% DEP (min flows) and normal pool essentially validated the inadequate depths at the historically dredged locations. | | | | | | |
| | The analysis considers the depths across the 200 foot channel when estimating the "% of time available". I.e, depths across the entire 200 foot channel must be greater than or equal to 12 feet for the 12 foot channel to be available. | | | | | | |
| | There are no apparent channel depth issues between Lock and Dam No. 5 (RM 200) and the Caddo Bossier Port (RM 212), which is currently the most upstream port along the waterway. | | | | | | |
| | Based upon the layout of the Red River headwater and tributary inflows (or lack thereof between gage locations), the flow statistics for pools 1 & 2 utilize the Red River at Alexandria (RM 88) daily flow period of record spanning 1935-2024. The flow statistics for pools 3 & 4 utilize the Red River at Grand Ecore (RM 152) spanning 1995-2024. It is noted that Shreveport has a lengthy period of record similar to Alexandria however, there are additional tributary contributions (localized runoff) that come into the Red River downstream of Shreveport (RM 228) and Lock and Dam No. 5 (RM 200) that would not be captured by the Shreveport gage records. | | | | | | |

*Figure A-92. Flow Required to Provide Given Channel Depths at Potential Problem Areas*

<table>
<tr><th colspan="7">Existing Conditions - Water Surface Elevations Required to Provide Channel Depths at Problem Areas D/S of Lock and Dam No. 1</th></tr>
<tr><th colspan="2">Location</th><th colspan="2">9 ft.</th><th colspan="2">12 ft.</th><th colspan="2">15 ft.</th></tr>
<tr><th></th><th>River Mile</th><th>WSEL Required</th><th>% Time Available</th><th>WSEL Required</th><th>% Time Available</th><th>WSEL Required</th><th>% Time Available</th></tr>
<tr><td></td><td>*34-42</td><td>7</td><td>94</td><td>10</td><td>85</td><td>13</td><td>78</td></tr>
<tr><td colspan="8"></td></tr>
<tr><td rowspan="5">Notes</td><td colspan="7">The JBJ Waterway reach below Lock and Dam No. 1 is effectively uncontrolled. This reach is highly influenced by the Mississippi River (backwater) outflows through the Old River Control Strucutre with subsequent influences from the Red River and Ouachita/Black Rivers. Due to the uncontrolled nature and MS River backwater influence, a statistical analysis of the water surface elevation is more representative to the percentage of time a certain channel depth is available as opposed to a stastical flow analysis was completed on the nav pool reaches. The 2018 Red River JBJ Waterway Action Plan notes that the 9ft channel is lost below LD1 when the gage at Acme reaches a low elevation of 4 feet. They hydraulic analysis shows that you likely need an approximate elevation of 7 feet on the Acme gage to confidently provide a 9 foot channel under existing conditions. Dredge records from 2024 reveal that dredging occured between River Miles 34-42 during the period of August 20 and September 25. During this time, the Acme gage was falling from an elevation of about 10 feet down to its September low of about 8 feet. Dredge records seem to be in agreement with the hydraulic analysis that shows atleast a 7 or more foot water elevation at Acme is likely required to provide a reliable 9 foot channel and 10 or more foot elevation for the 12 foot channel.</td></tr>
<tr><td colspan="7">The Red River at Acme Gage (River Mile 34) was used for the water surface elevation statistical duration exceedance analysis. The gage record spans from 1932-2024. It is noted that the Old River Control Structure, separating the MS River from the Lower Red River, was constructed in the 1960's.</td></tr>
<tr><td colspan="7">The existing conditions minimum tailwater pool for Lockand Dam No. 1 to safely allow navigable draft over the lower miter gate sill is elevation 4 feet NAVD88 which is a function of the miter gate sill elevation of -9 feet and a 1.5x draft recommendation for depth above the lower sill. (1.5x9ft draft = 13 feet, 13 feet + -9 low sill elevation = elevation 4 feet for minimum pool). 1.5x a 12' draft would require 18 feet of depth above the sill.</td></tr>
<tr><td colspan="7">River Miles 36-42 are considered "The Gauntlet" by Vicksburg District personnel. This has been historically the most problematic reach for maintaining navigable channel depths (at 9 feet) and has been dredged annually between 2021 - 2024 during the low water periods. It is noted that the MS River experienced historically low water levels between 2022 and 2024, which equates to historically low flows being diverted through the Old River Control Structure into the Lower Red River.</td></tr>
<tr><td colspan="7">*Potential problem areas initially identified per historical dredge records, for which existing dredging is completed to provide the 9-foot channel. Noted that the HEC-RAS model simulated with 98% DEP (min flows) and normal pool essentially validated the inadequate depths at the historically dredged locations.</td></tr>
</table>

*Figure A-93. Water Surface Elevation Required to Provide Given Channel Depths at Problem Areas Below L&D 1*

| Lock and Dam | River Mile |
|---|---|
| LD5 | 200 |
| LD4 | 168 |
| LD3 | 115 |
| LD2 | 74 |
| LD1 | 40 |

| | Locations (River Miles) | Approximate Length of Channel Near or Below 12 Foot Depth within 200 ft. Nav Channel Boundary (Miles) |
|---|---|---|
| Historically Dredged Locations per MVK River Operations Branch validated with HEC-RAS model | 194 | 0.4 |
| | 191 | 0.6 |
| | 158 - 159 | 0.2 |
| | 154 | 0.4 |
| | 42-41 | 0.2 |
| | 40 - 41 | 0.3 |
| | 39 - 40 | 1 |
| | 36 -37 | 0.2 |
| | 35 | 0.2 |
| Additional Locations identified with HEC-RAS model | 229 - 230 | 0.75 |
| | 226 - 227 | 0.5 |
| | 224 | 0.6 |
| | 215 | 1.1 |
| | 164 - 165 | 0.4 |
| | 163-164 | 0.4 |
| | 108 | 0.2 |
| | 64-65 | 0.4 |
| | 60 - 61 | 4 |
| | 52-53 | 0.2 |
| | Total estimated length of nav channel near or below 12 feet of depth at 98% DEP low flows (miles) | 12.05 |
| | Total Miles of Waterway from Old River to Shreveport Area | 230 |
| | Total Miles of Waterway from Old River to Caddo Bossier Port (miles) | 5% |

*Figure A-94. Approximate Total Length of Potential Problem Reaches*

### 4.4.2.1 Prioritization of Channel Depths Assessment

Upon identifying potential problem reaches within the waterway regarding the availability of 12 feet or more of navigable depth, a workflow for addressing the problem reaches should be considered. There are two concurrent navigation deepening studies in the McClellan-Kerr Arkansas River Navigation System (MKARNS) 12-FT Channel Validation Report (2023) and the Tennessee-Tombigbee Waterway Deepening Study. Both of these systems laid out screening level or tiered approaches regarding the workflow for addressing problem reaches with river engineering practices. The Tennessee-Tombigbee approach assessed two screening levels where level 1 identified shoaling sites that are both repetitive and recent, selecting sites dredged within the previous 5 years and on average every 3 years. Level 2 was stated to be analyzing remaining sites, estimating costs for feasible solutions, and screening out areas with no conceivable cost-benefit or possible design solution. The MKARNS 12-Foot Channel Validation Report (2023) noted a tiered approach where Tier 1 was high priority, characterized as areas with high risks of shoaling, existing depths of 9-12 feet, or requiring downstream protection features, meaning any change to the existing dikes within the area or new construction would divert energy or sediment downstream which requires a subsequent bank stabilization or dike feature. Tier 2 would be similar to Tier 1 except focusing on areas with depths of 12-15 feet with subsequent tiers up to five tiers addressing additional locations.

The aforementioned example studies have many more stretches of river that were identified to be deficient for their respective navigation studies. The JBJ Waterway was well designed and currently sustains 9 feet of depth for a large majority of the waterway with the primary exception being the reach below L&D 1 that has complexities which involve Mississippi River Backwater influence. Further, existing assessment of the channel depths illustrates that a large majority of the river also sustains 12 or more feet with the exception of a few problem reaches.

The JBJ Waterway 12-Foot Channel Study identified problem reaches as High, Medium, or Low priority. The problem reaches were identified using a combined approach. First, the dredge records from 2012–2024 were consulted to identify areas that have been annually or occasionally dredged. Then, an existing hydraulic model (HEC-RAS 1D river channel) with 2016 single-beam surveys was used to simulate normal pool conditions with minimum inflows (98 percent DEP) considered project design conditions. Depth grids were created from the hydraulic model, and the model was used to validate the dredge records by showing that the documented dredged areas do show up within the hydraulic model output as having insufficient depths. Then, the model outputs were used to assess the entire 212-mile waterway from Old River to the Caddo-Bossier Port to identify other potential problem areas that have not shown up in the dredge records for which the dredging is targeting the maintenance of a 9-FT channel. All areas with less than or equal to 15 feet were flagged for assessment. Utilizing the HEC-RAS simulated water surface outputs, depths are also created within GIS using the 2012 multi-beam data. The water surface grid was imported into GIS, where the Raster Calculator is used to determine the difference (depth) between the water surface grid and the underlying 2012 multi-beam data. The resulting depth grid is used to compare to the 2016 single-beam depth grids at the identified problem reaches.

High Priority – Areas that have experienced consistent shoaling impeding navigable depths, dredged annually, existing depths are less than 12 feet, and located within the recommended navigation track.

Medium Priority – Areas that have experienced occasional shoaling impeding navigable depths, dredged occasionally, or existing depths are 12–15 feet, and located within the recommended navigation track.

Low Priority – Additional areas identified by the hydraulic modeling channel depths assessment as potentially having depths near a level of inadequacy for a 12-FT channel during project design or normal pool low flow conditions (98 percent DEP).

Currently, a high majority of the navigation travels and stops at Alexandria, Louisiana. Therefore, reaches between Alexandria and Old River, such as the problem reaches below L&D 1, would be an example of identifying an area as a high-priority problem reach due to its location, and due to known dredging records. I.e., the stretch of river between Old River and Alexandria should be considered a seemingly important stretch due to most of the traffic stopping at Alexandria and not continuing further north.

Upon conversations with the Vicksburg District Design Branch River Stabilization Section, Notably, many of the channel improvement structures along the JBJ Waterway have essentially been neglected for decades causing some structures to significantly deteriorate or complete failure in some cases. Therefore, the possible first step in addressing the problem reaches would be to assess the existing conditions of the dike and revetment systems within the problem reaches. If deteriorated, simply rehabbing these structures to existing design dimensions may prove to be a substantial first step in using the river to induce the scouring necessary to provide sufficient navigable depths or more than 12 feet.

4.4.2.2 Historical Thalweg and Channel Comparisons

Utilizing the 2012 multi-beam and 2016 single-beam data within the HEC-RAS model, the thalweg underlying the recommended navigation track centerline was extracted and plotted using excel. This comparison is meant to provide a visual illustration of the change in thalweg between the two time frames. The multi-beam survey is far more detailed than the single-beam survey as it provides seamless data throughout the river while the single-beam data only provides data at collected cross-sections. Using the single-beam cross-sections, the RAS Mapper model was used to create a seamlessly, interpolated DEM between those cross-sections. This is ultimately estimating the channel bathymetry between each surveyed cross-section. It is also noted that the 2012 data were collected during a time frame in which the river had not experienced any major flood since 1990, although many bank exceedance flow events or annual type high-water events had occurred, likely illustrating long term normal channel conditions. Normal was defined as the channel conditions present following the completion of the fifth and final lock and dam in 1995. However, the 2016 data were collected following the 2016 flood event, which was preceded by the 2015 flood event. Therefore, the 2016 channel was assumed to have scoured out some following these historical flood events. Furthermore, these thalwegs are showing the channel depth beneath the recommended navigation centerline (or the center of the 200-FT navigation channel), whereas the deepest part of the channel does not always coincide with the recommended navigation centerline.

Upon discussions with a retired Vicksburg District Hydraulic Engineer and Channel Improvement Coordinator, it was recommended that a 1981 hydrographic survey be located to compare to more recent surveys. The 1981 survey is a good depiction of the channel conditions prior to the JBJ waterway project that includes channel realignments, bank stabilization (revetments) and river training structures (dikes), and the locks and dams. Prior to 1981, some channel improvement work had been completed so the 1981 survey is not a complete pre-project condition; however, many of the dike and revetments were constructed within this time frame. Additionally, the pre-project 1981 survey includes areas of channels that are no more due to the channel cutoff program so direct channel or thalweg comparisons cannot necessarily be made in those areas as the post project channel is now different. Notably, a large majority of the channel cutoffs occurred between Acme and L&D 3 as discussed in the Channel Realignment Section. The comparison to the 2016 survey will give a general overview of the deepening of the channel with all of the project features in place over the course of decades.

The 1981 cross-sections are likely in NGVD29 while the 2016 cross-sections are in NAVD88. Ultimately, the conversion is relatively insignificant. The 1981 data were extracted from a historical HEC-2 hydraulic model located on Vicksburg District internal servers. The 1981 data were compared to 2016 data using HEC-RAS and excel although the river station correlation may not be exact. Generally, the 2016 data shows a consistently lower channel due to the JBJ Waterway Project (with some exception above L&D 5, which is the most upstream lock and dam) and its river training features that deepened the channel for navigation while also capturing major scour holes that were not present prior to the project. Notably, the located HEC-2 model contained a numerous amount of geometry files making it challenging to fully comprehend the inputs of the model. Therefore, the 1981 data are used with caution primarily for graphical informational purposes as it was extracted from a model and as opposed to extracting from the actual survey data. Notably, the HEC-2 model files contained geometries that referenced pre-project and post-project conditions. It was assumed that the pre-project conditions utilized the 1981 survey as-is while the post-project geometry altered the channel conditions to represent contraction and potentially scouring and/or deposition. For this comparison, the data from the 1981 pre-project were extracted to represent pre-project or pre-contraction conditions to compare to the 2016 single-beam survey. Comparisons are also provided to illustrate the 1981 pre- and post-project conditions as captured and assumed in the HEC-2 model.

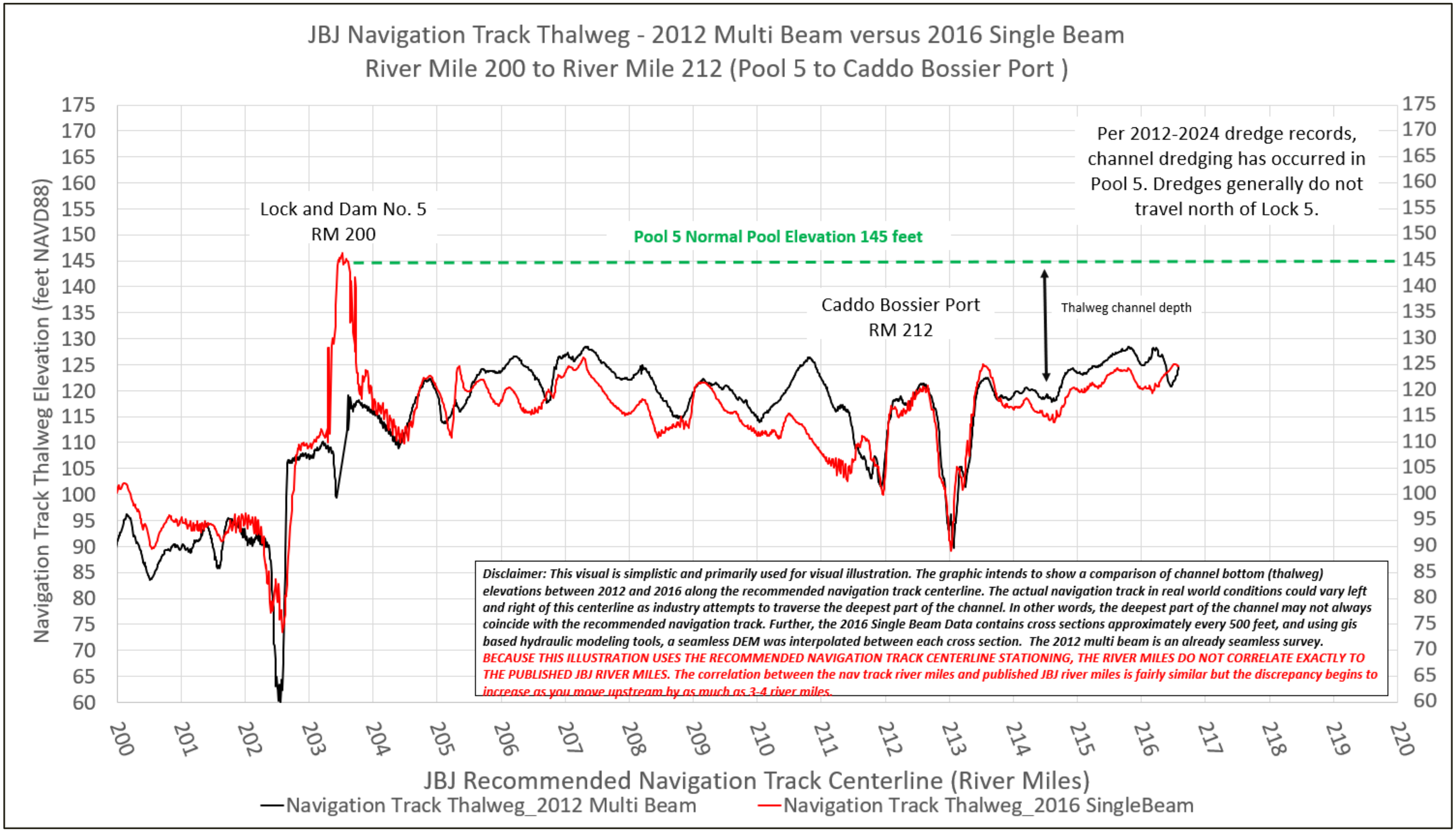



*Figure A-95. Navigation Track 2012 and 2016 Thalweg Comparisons – Pool 5*

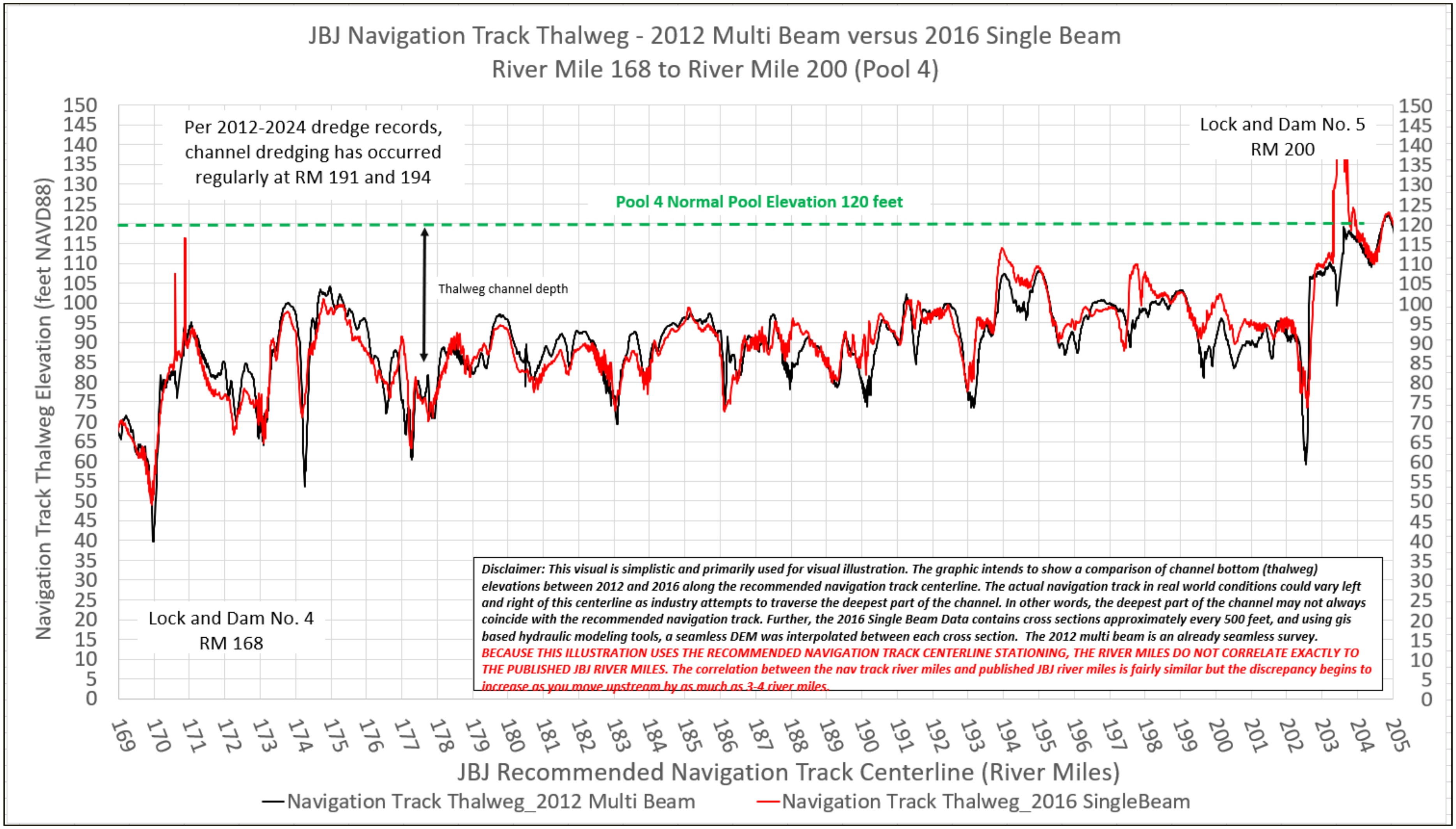



*Figure A-96. Navigation Track 2012 and 2016 Thalweg Comparisons – Pool 4*

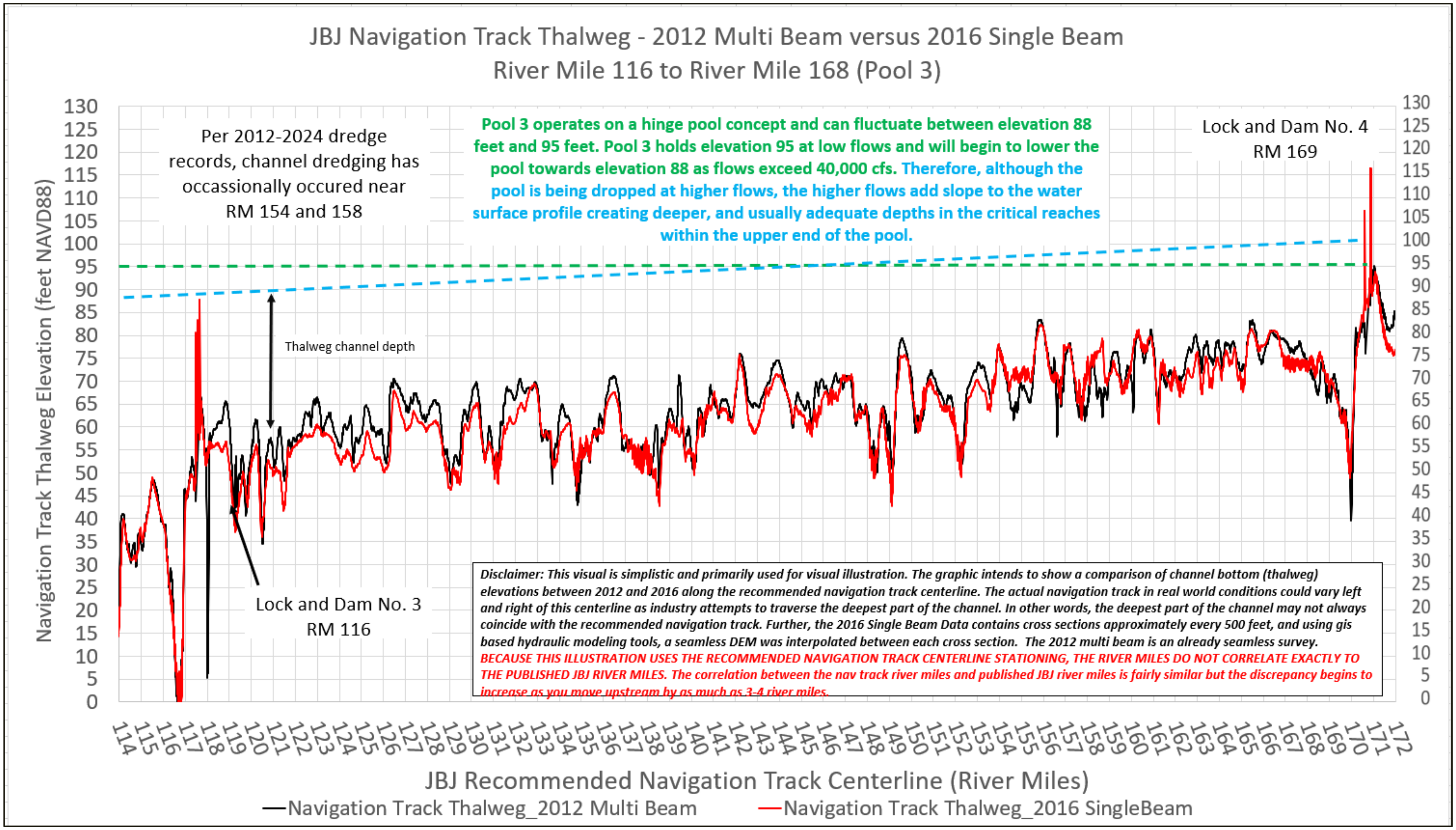



*Figure A-97. Navigation Track 2012 and 2016 Thalweg Comparisons – Pool 3*

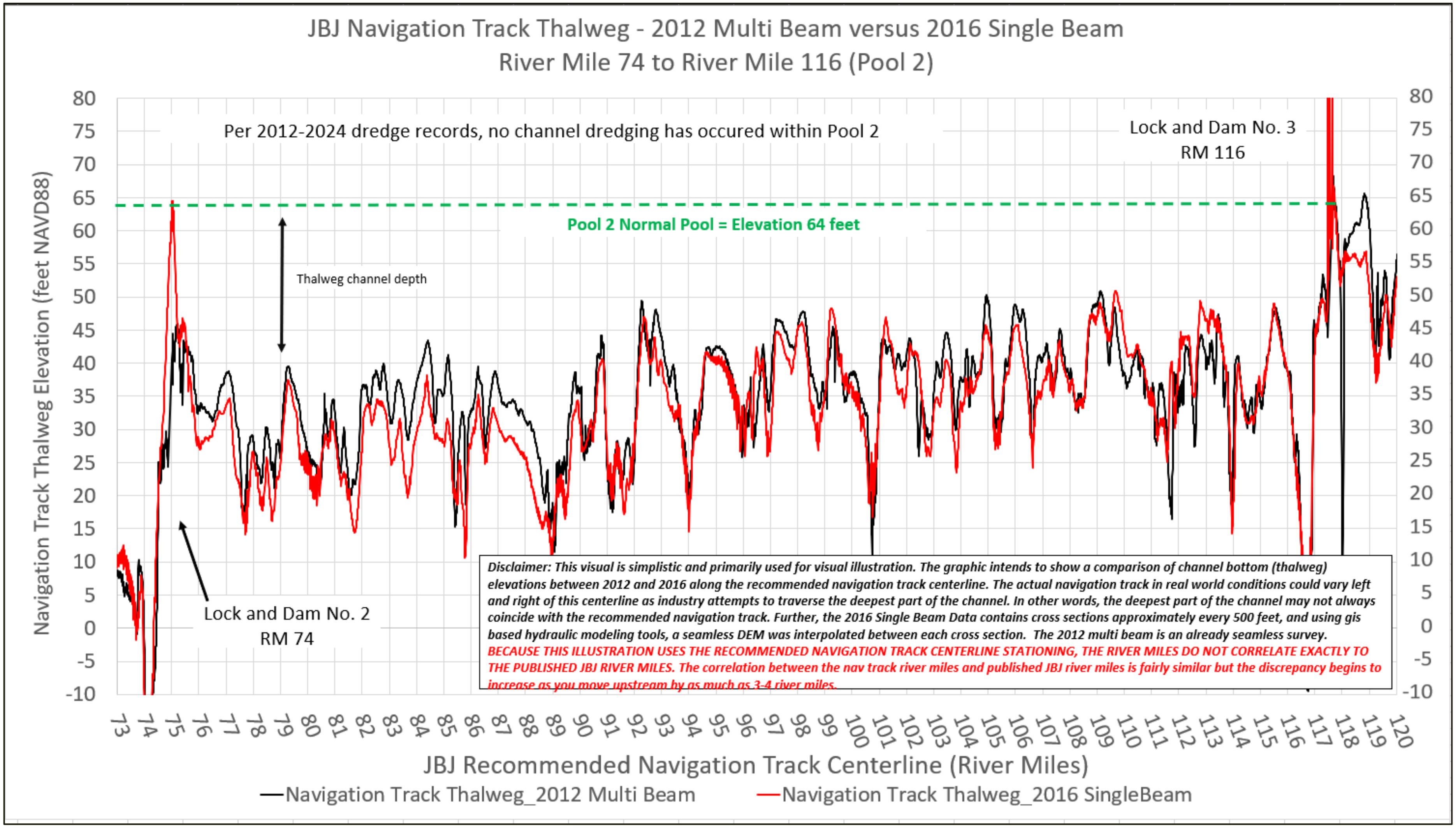



*Figure A-98. Navigation Track 2012 and 2016 Thalweg Comparisons – Pool 2*

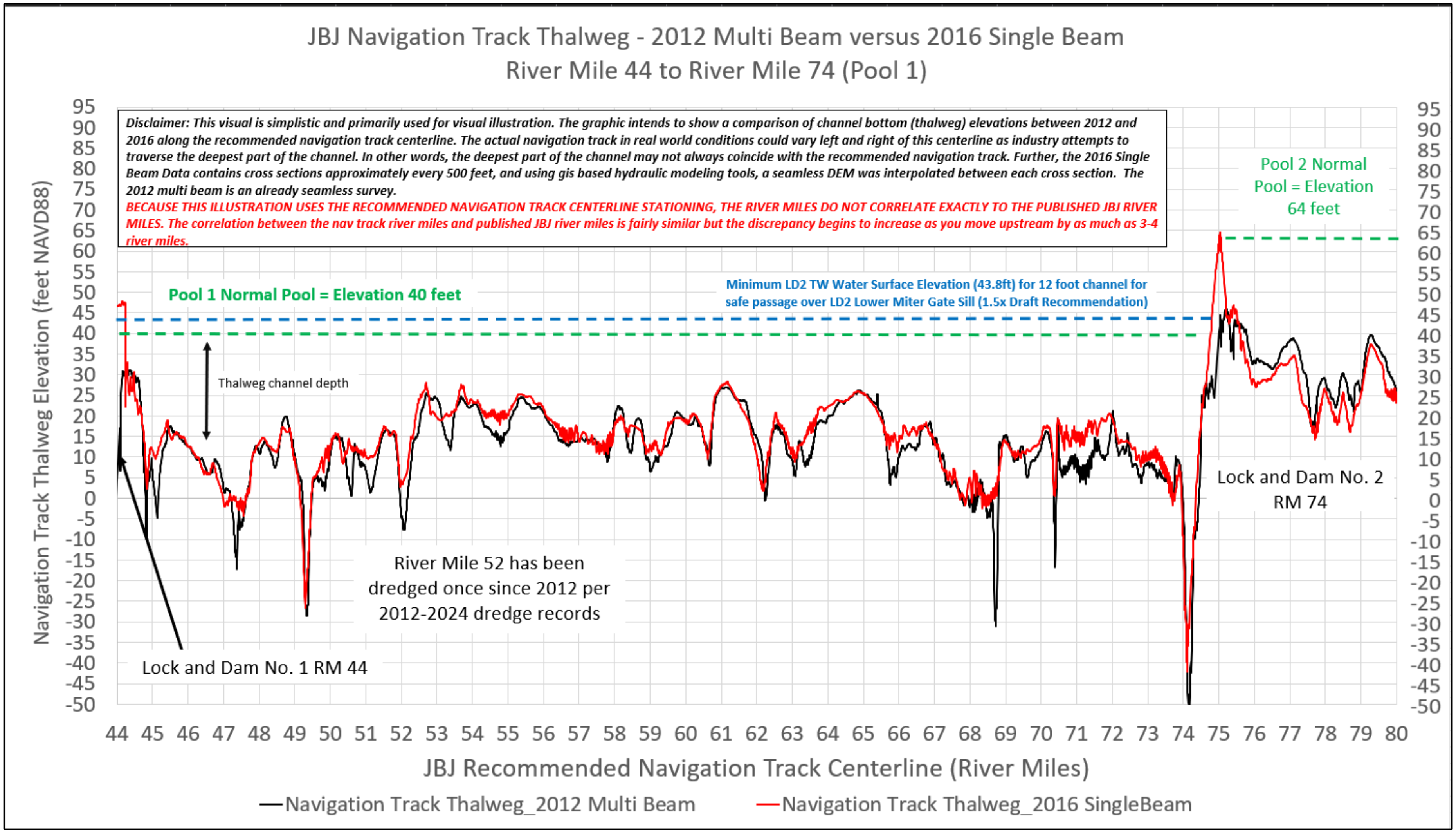



*Figure A-99. Navigation Track 2012 and 2016 Thalweg Comparisons – Pool 1*

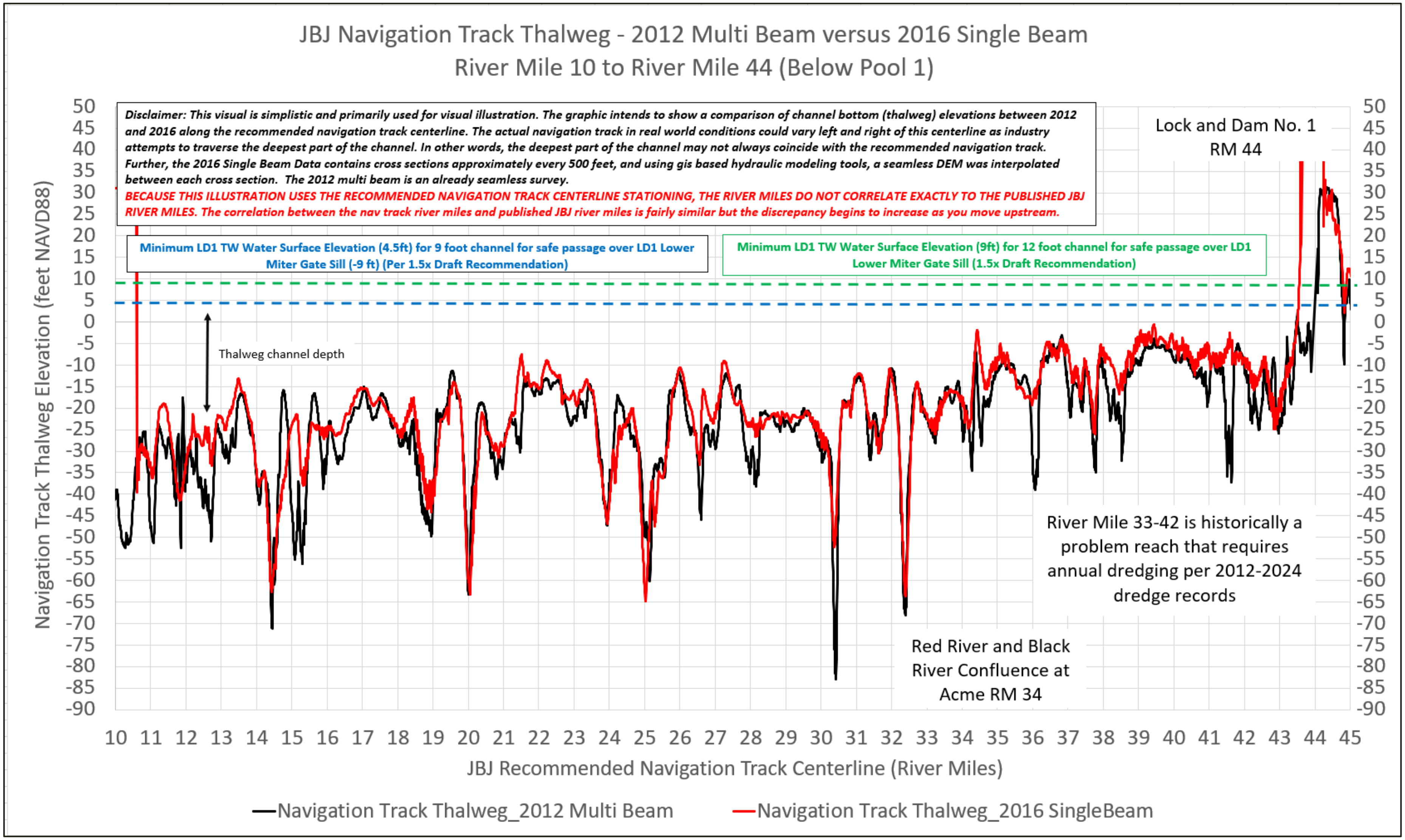



*Figure A-100. Navigation Track 2012 and 2016 Thalweg Comparisons – Below L&D No. 1*

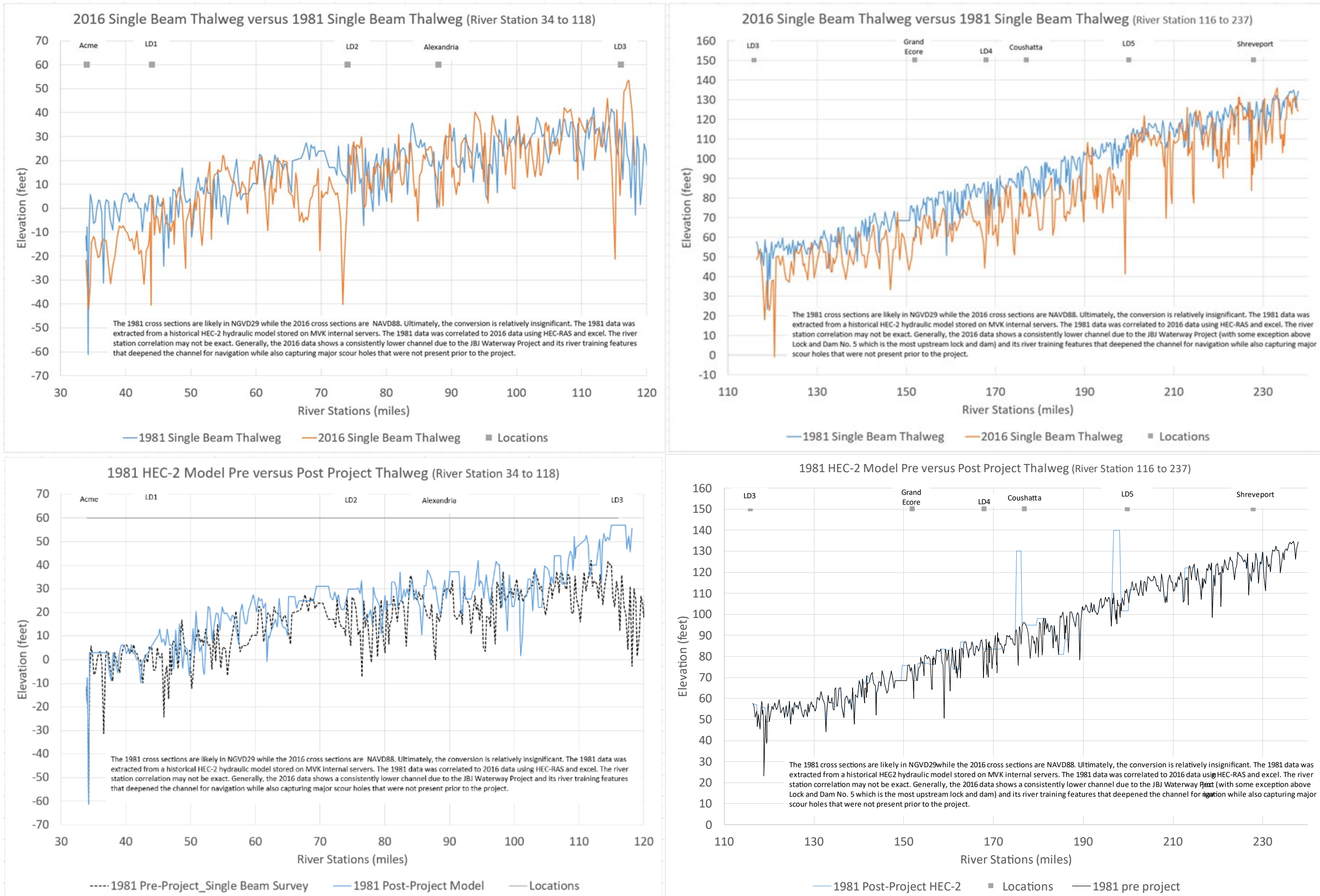



*Figure A-88. Red River Thalweg Comparisons – 1981 Versus 2016 – RMs 34 to 237*

Figure A-89 compares the 1981 pre-project hydrographic survey and 1981 post-project model to the 2012 and 2016 surveys for the problem reach below L&D 1. The 2012 and 2016 surveys show a much deeper channel than the pre-project 1981 survey and 1981 post-project model; however, this reach continues to be a consistently dredged area to maintain navigation. The comparisons between 1981 and 2012/2016 are not exact correlations but provide a generalized view of the channel changes. Notably, there was a relatively short channel cutoff (Lorraine) completed in the 1980s near RMs 35 to 36; therefore, the pre-project survey occurred prior to this channel cutoff, whereas the 1981 post-project model is assumed to consider the cutoff along with the dike contractions and some degree of scouring. The 2012 and 2016 surveys are comparable except near RMs 36 and 38, which show some degree of deposition to have been present causing a higher bed elevation in the 2016 survey.

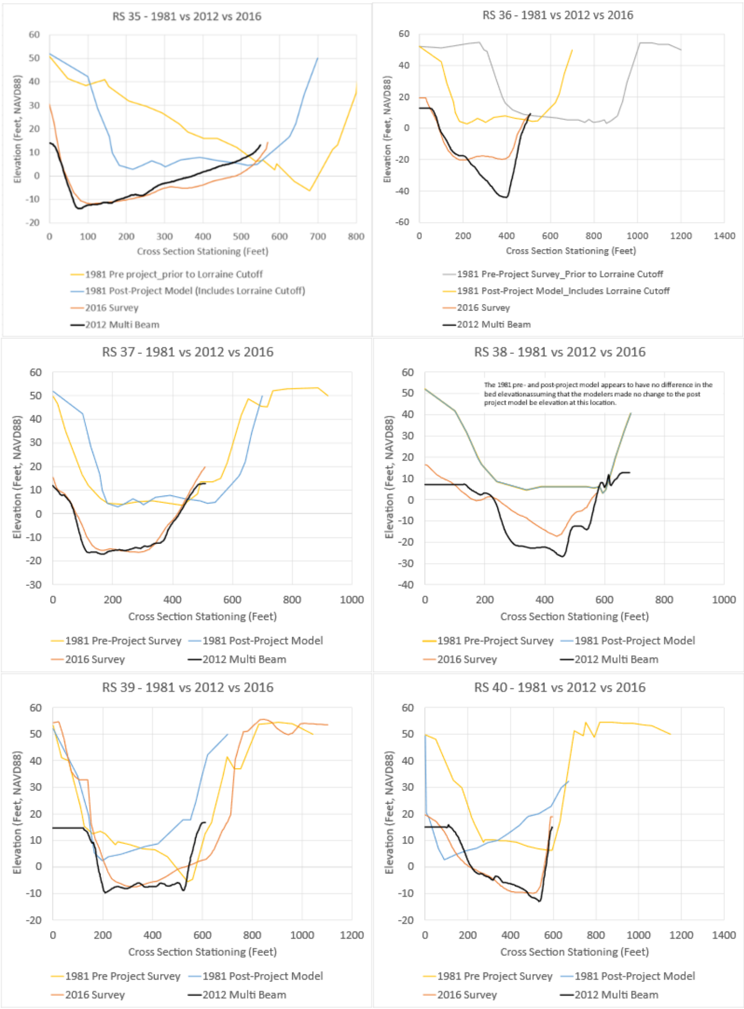



*Figure A-89. Cross-Section Comparisons 1981 Versus 2012 and 2016 – Consistent Problem Reach Below L&D 1*

Figure A-90 shows the 1981 hydrographic survey versus the 2012 and 2016 surveys for the consistent problem reach within Pool 4 between RMs 190 to 192 (Westdale). The comparison shows little difference in bed elevation; however, channel shifts can be seen as the 2016 survey illustrates the channel contraction with dikes. This comparison mostly shows that controlling bed elevations have not changed very much throughout the reach as the area continues to primarily act as a depositional reach.

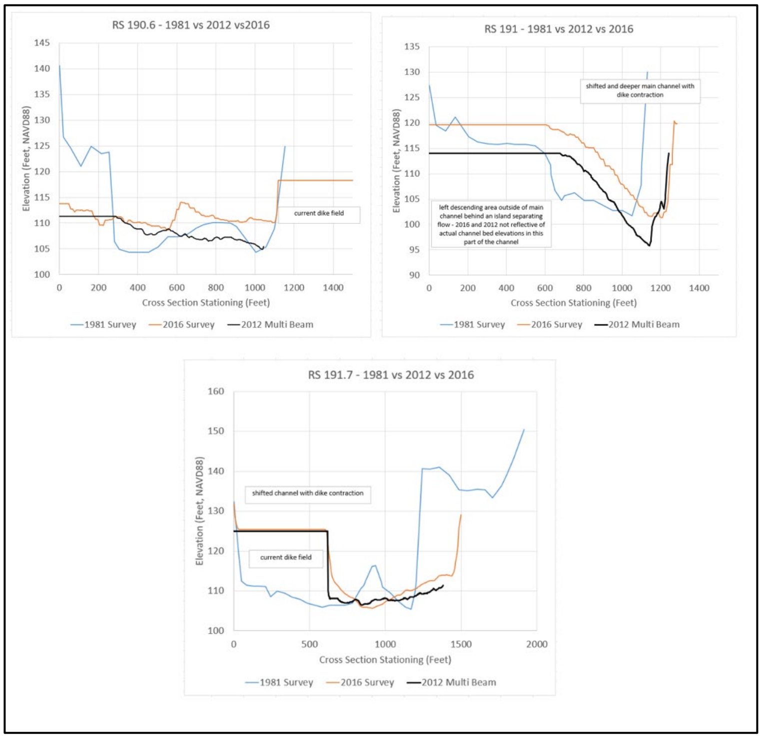



*Figure A-90. Cross-Section Comparisons of 1981 Versus 2012 and 2016 – Consistent Problem Reach Near RMs 190 to 192 (Westdale)*

#### 4.4.2.3 High-Priority Problem Reaches

<u>Pool 4 - RMs 192–191 (Westdale)</u>

Figure A-91 is using HEC-RAS-generated depth grids at normal pool project design conditions (water surface elevation 120 feet NAVD88) to illustrate the potential problems related to navigation channel depths between RMs 192 and 190.

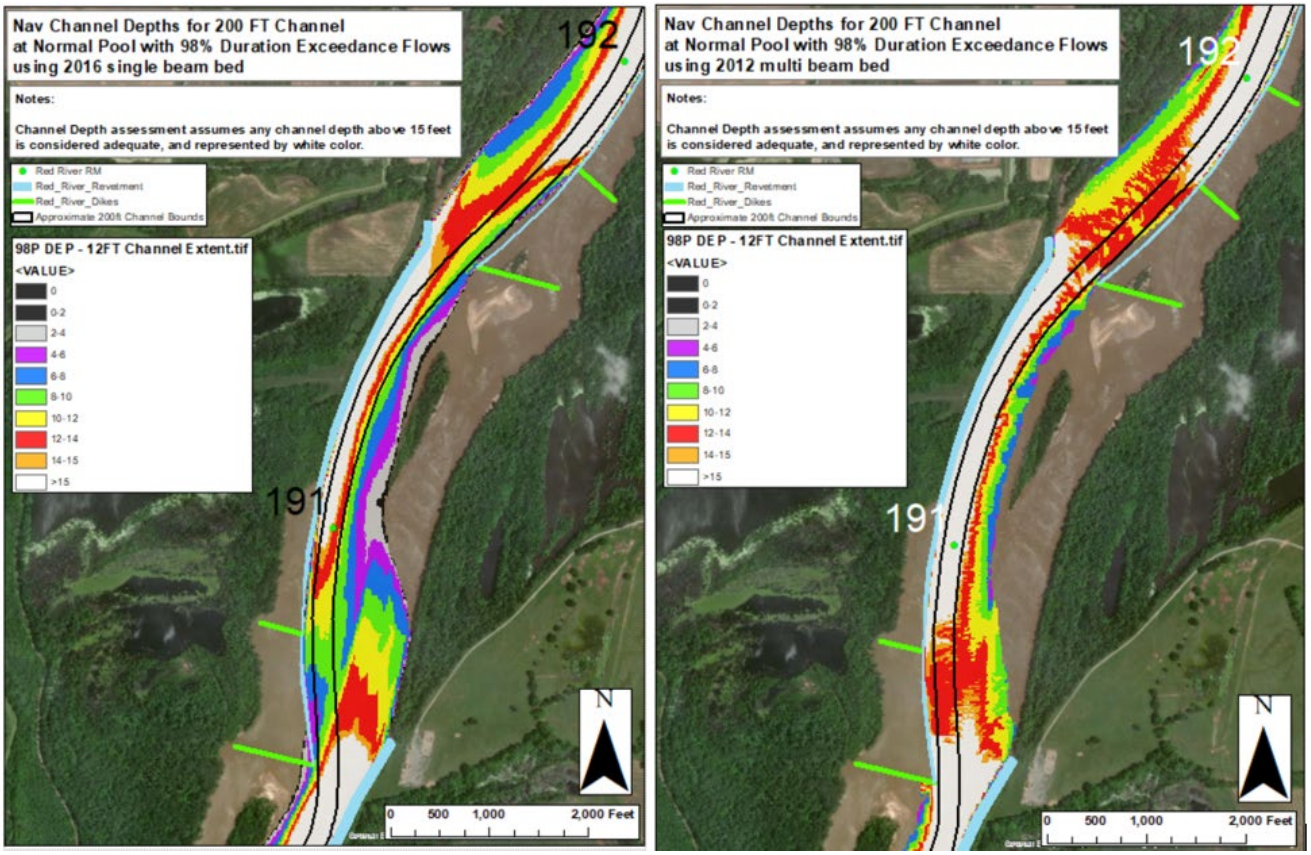



*Figure A-91. Pool 4 Normal Pool (WSEL 120 Feet) Channel Depth Maps Near RMs 191 and 192*

The 2012 multi-beam and 2016 single-beam data provide a visual illustration of the depositional reach just below RM 192 and just below RM 191, as shown in the following two figures. The 2016 single-beam survey is an interpolated DEM between each cross-section using RAS Mapper.

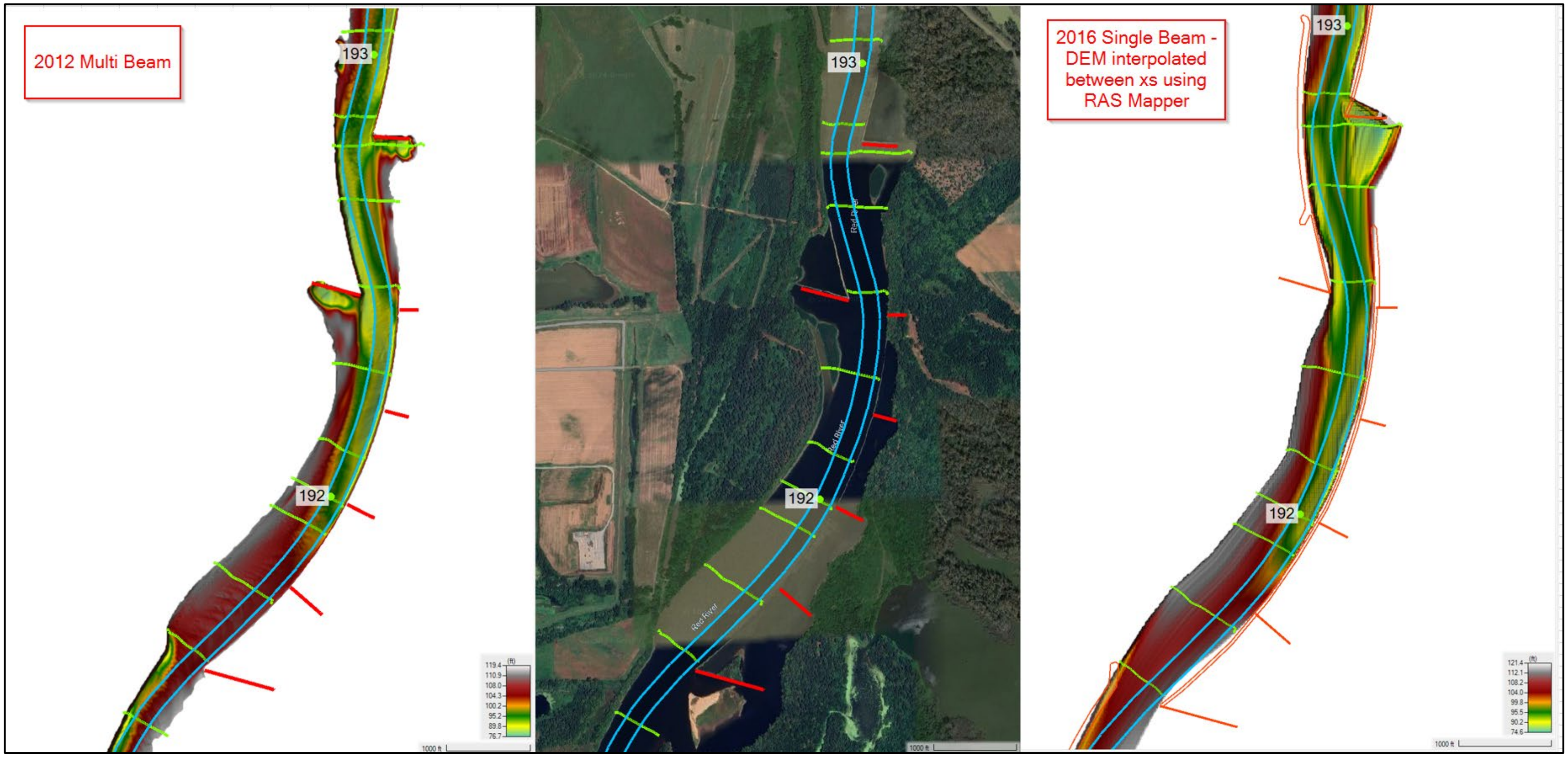



*Figure A-92. 2012 Multi-Beam and 2016 Single-Beam Data Near RM 192*

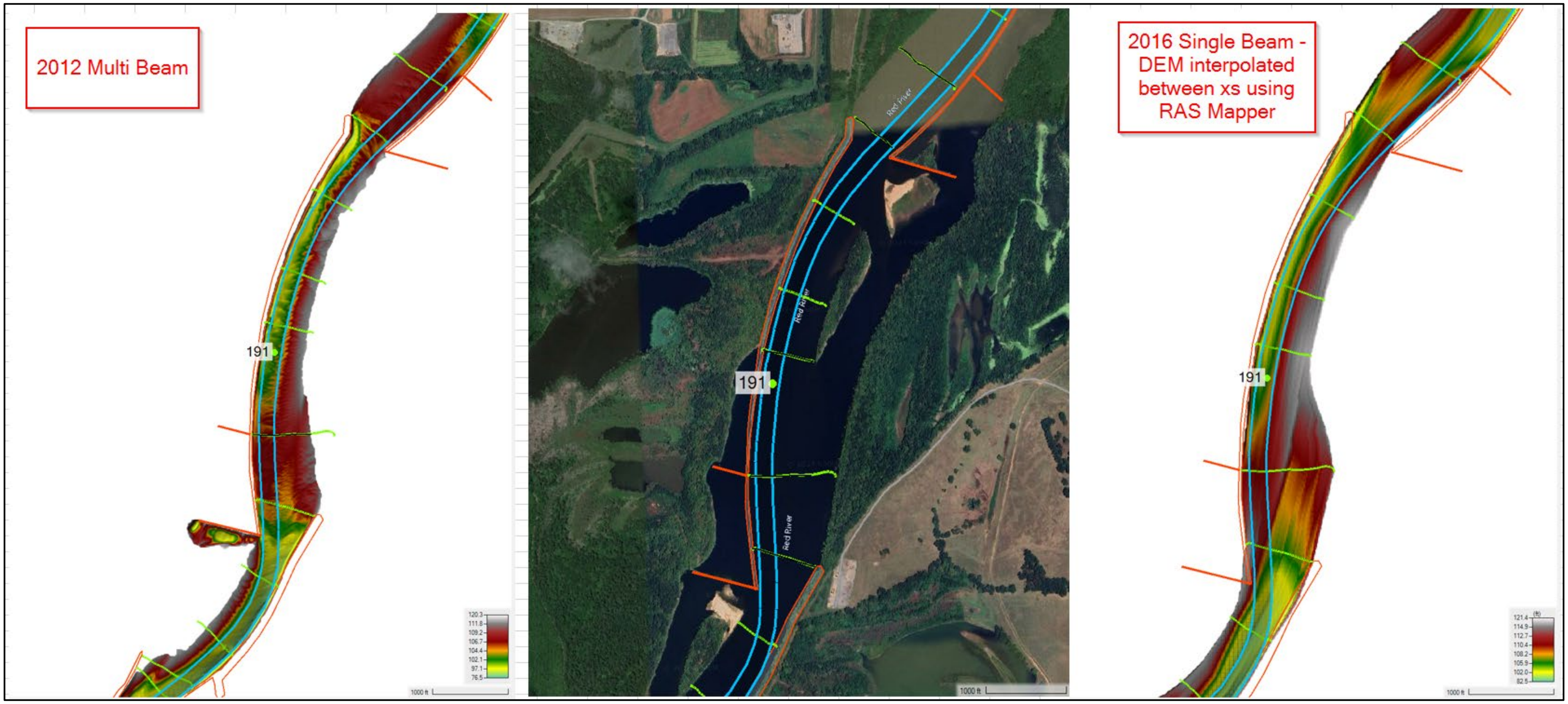



*Figure A-93. 2012 Multi-Beam and 2016 Single-Beam Data Near RM 191*

A cross-sectional comparison of the 2016 single-beam and 2012 multi-beam is provided in Figure A-94. The cross-section comparison is an illustration and does not fully satisfy the representation of the channel between the cross-sections.

The green lines in the aerial imagery represent the 2016 single-beam cross-sections. The blue lines represent an approximate 200-FT wide channel polygon. Additionally, spur dikes and longitudinal revetments are visible within the aerial imagery.

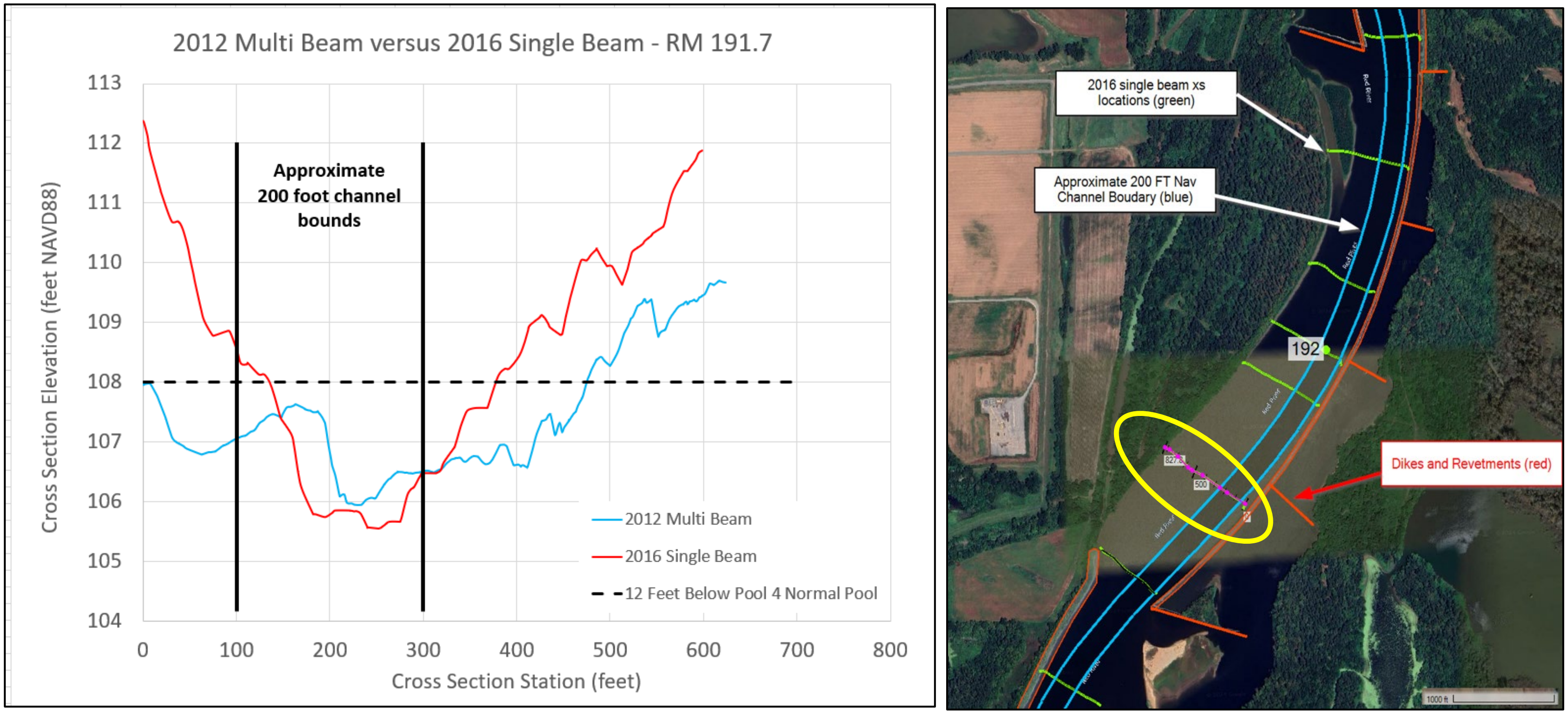



*Figure A-94. 2012 Multi-Beam Versus 2016 Single-Beam Cross-Section Comparison (RM 191.7)*

A cross-sectional comparison of the 2016 single-beam and 2012 multi-beam is provided in Figure A-95. The cross-section comparison is an illustration and does not fully satisfy the representation of the channel between the cross-sections.

The green lines in the aerial imagery represent the 2016 single-beam cross-sections. The blue lines represent an approximate 200-foot wide channel polygon. Additionally, spur dikes and longitudinal revetments are visible within the aerial imagery.

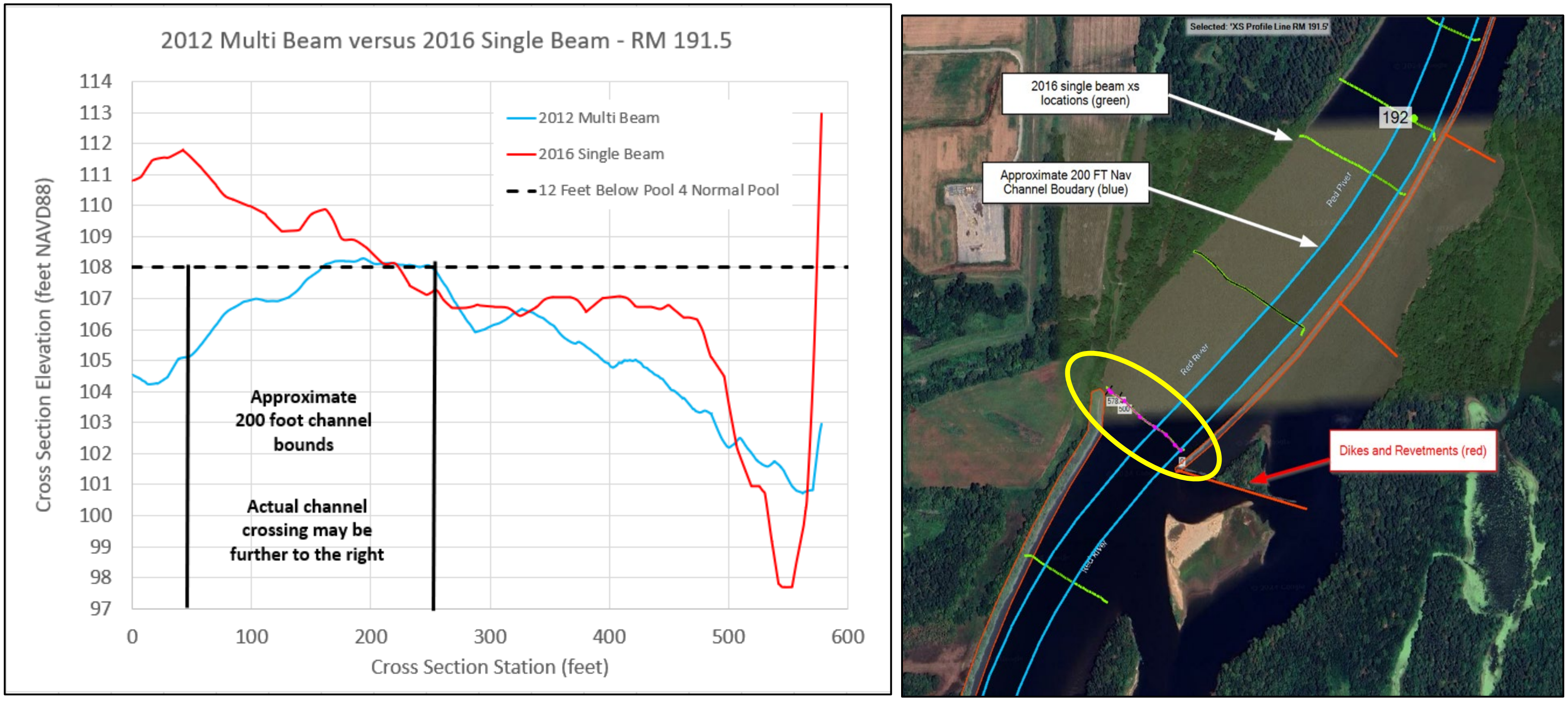
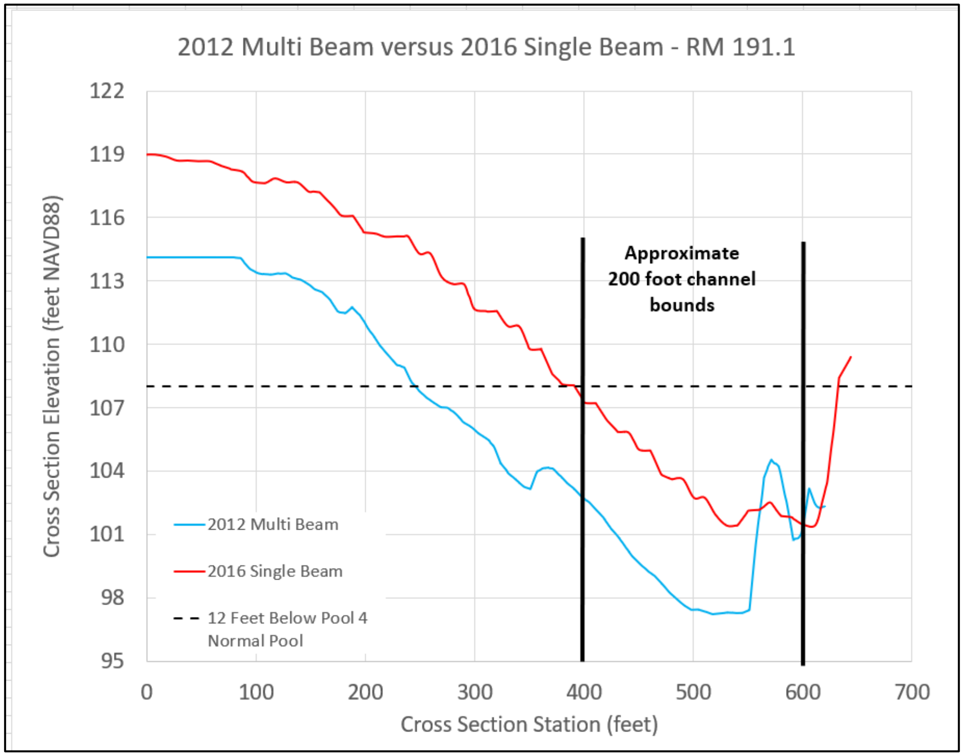



*Figure A-95. 2012 Multi-Beam Versus 2016 Single-Beam Cross-Section Comparison (RM 191.5)*

A cross-sectional comparison of the 2016 single-beam and 2012 multi-beam is provided in Figure A-96. The cross-section comparison is an illustration and does not fully satisfy the representation of the channel between the cross-sections.

The green lines in the aerial imagery represent the 2016 single-beam cross-sections. The blue lines represent an approximate 200-foot wide channel polygon. Additionally, spur dikes and longitudinal revetments are visible within the aerial imagery.



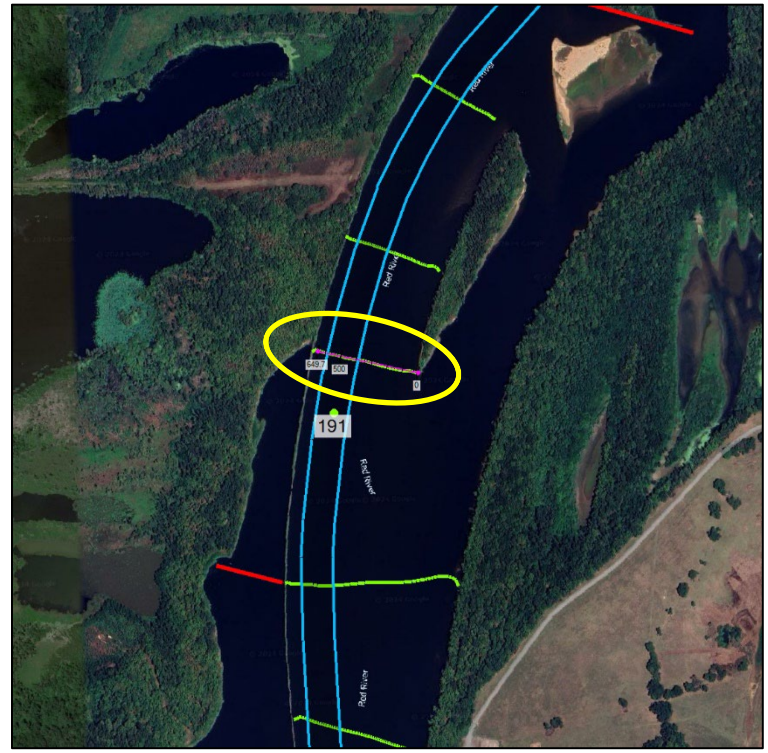
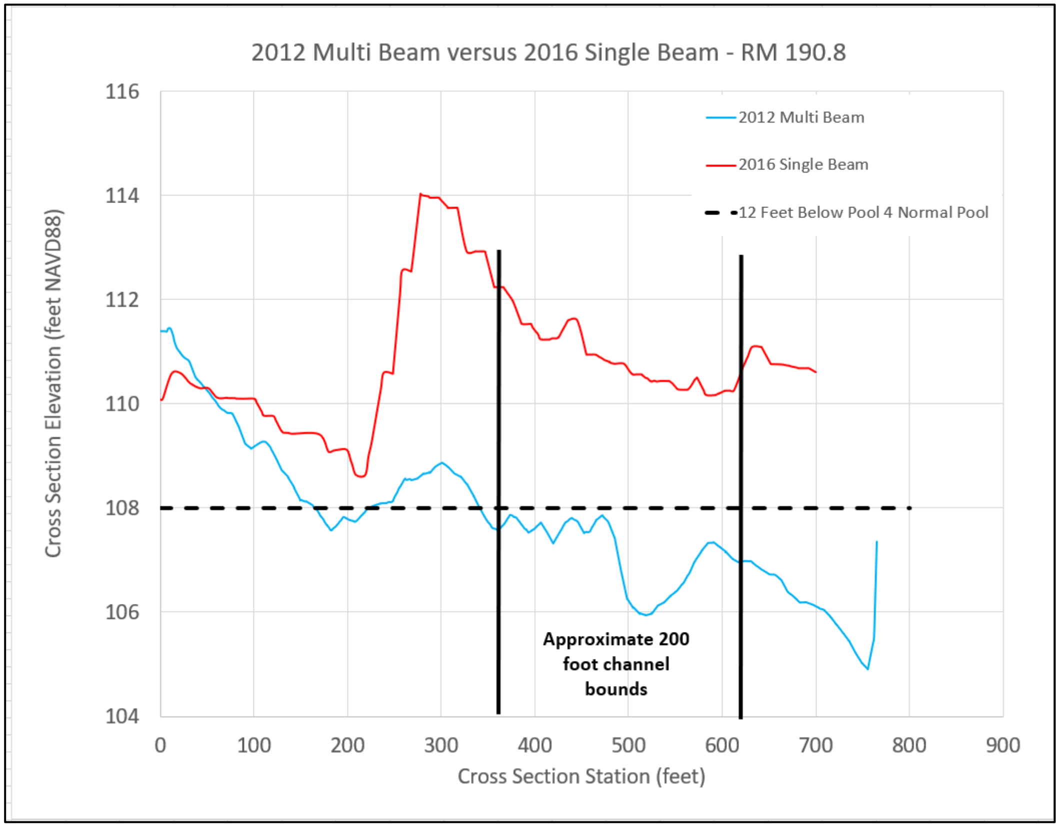




*Figure A-96. 2012 Multi-Beam Versus 2016 Single-Beam Cross-Section Comparison (RM 191.2)*

A cross-sectional comparison of the 2016 single-beam and 2012 multi-beam is provided in Figure A-97. The cross-section comparison is an illustration and does not fully satisfy the representation of the channel between the cross-sections.

The green lines in the aerial imagery represent the 2016 single-beam cross-sections. The blue lines represent an approximate 200-foot wide channel polygon. Additionally, spur dikes and longitudinal revetments are visible within the aerial imagery.



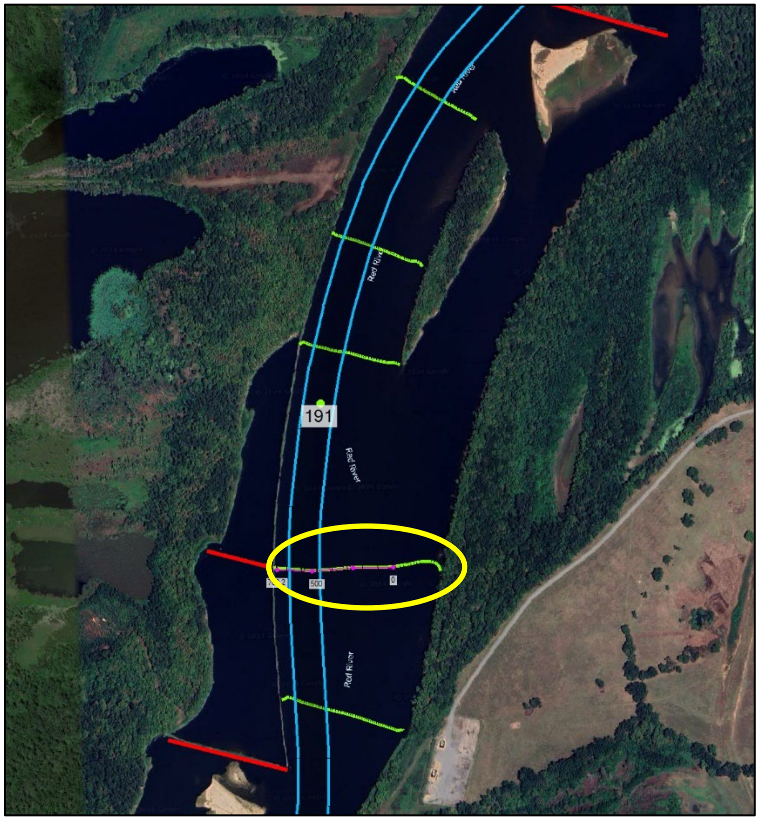




*Figure A-97. 2012 Multi-Beam Versus 2016 Single-Beam Cross-Section Comparison (RM 190.8)*

Below L&D 1 - RMs 34–42 (The Gauntlet)

The channel depths downstream of L&D 1 are nuanced from those within the pools because L&D 1 tailwater levels are not controlled by any downstream structure but heavily influenced by Mississippi River backwater flows through the ORCC. Further, for vessels to safely enter the L&D 1 lock chamber over the lower miter gate sill, a 1.5x channel draft depth is recommended. Currently, the lower miter gate sill elevation is -9 feet NAVD88, and the current water control manual states that the minimum tailwater level is water surface elevation 4 feet, which is 13 feet of depth over the sill or approximately 0.5 feet shy of meeting the 1.5x draft recommendation over the sill. For a 12-FT channel, 18 feet of depth would be required over the sill to satisfy the 1.5x draft recommendation. This would call for a tailwater water surface elevation of 9 feet. Therefore, channel depths are compared utilizing a water surface of 4 feet and 9 feet. Due to the Mississippi River backwater influences, the water surface between L&D 1 and the ORCC generally has a flat slope.

In general, when the water surface elevation downstream of L&D 1 is at or above 9 feet (providing 1.5x draft over miter gate sill), there is typically 12 or more feet of channel depth available based on the modeling. This satisfies both the 12-FT channel within the reach and the 1.5x draft requirement through L&D 1 lock chamber over the lower miter gate sill.

Currently, River Operations Branch states that a 9-FT channel is lost when the L&D 1 tailwater water surface elevation reaches an elevation of 4 feet or below and mechanical dredging efforts begin. Current modeling and depth maps seem to agree that when the river goes to a water surface elevation of 4 feet neither a 12-FT nor a 9-FT channel are adequately available throughout this reach.

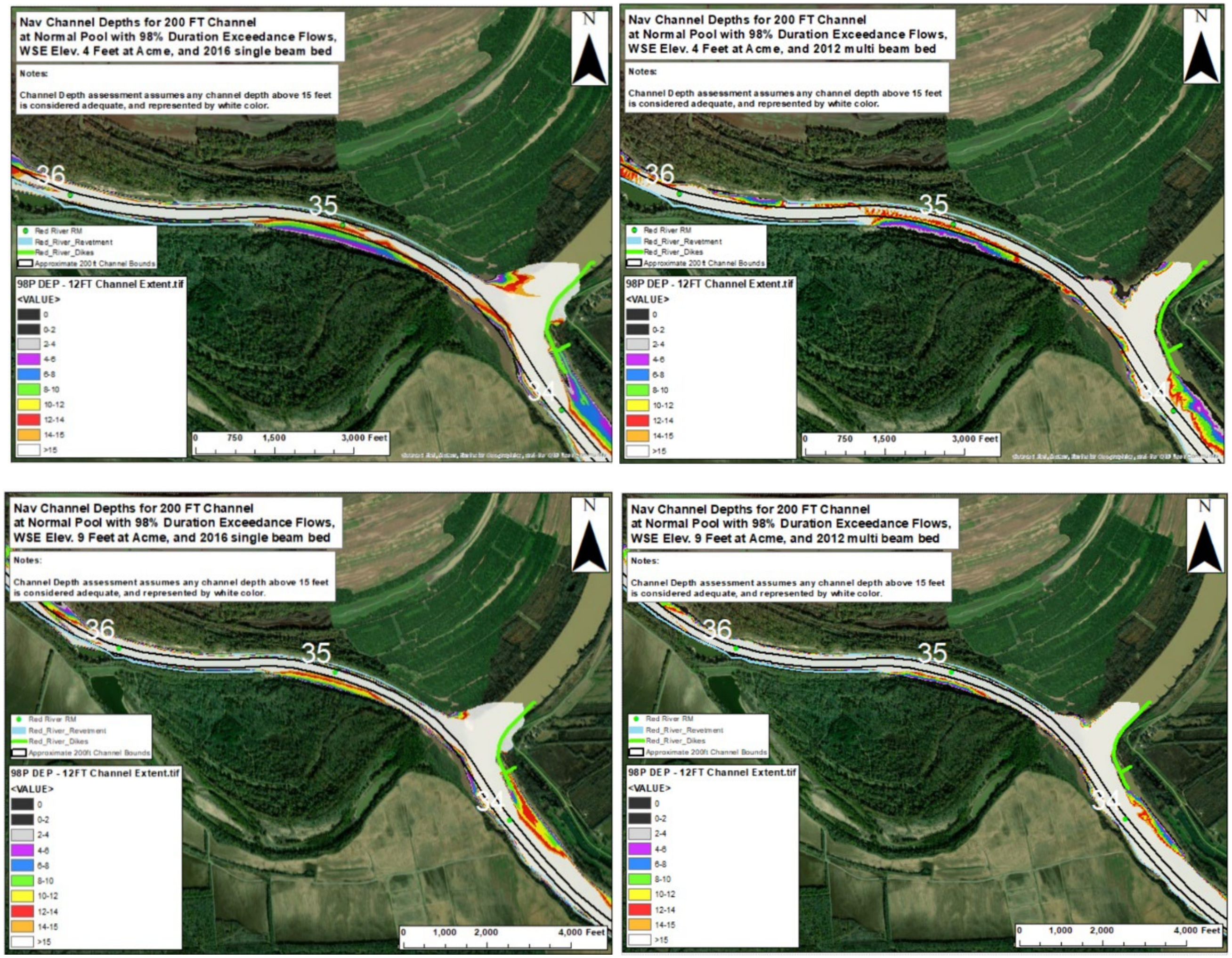



*Figure A-98. Channel Depth Maps Near RMs 34–36 with WSE 4 Feet and 9 Feet at Acme, Louisiana*

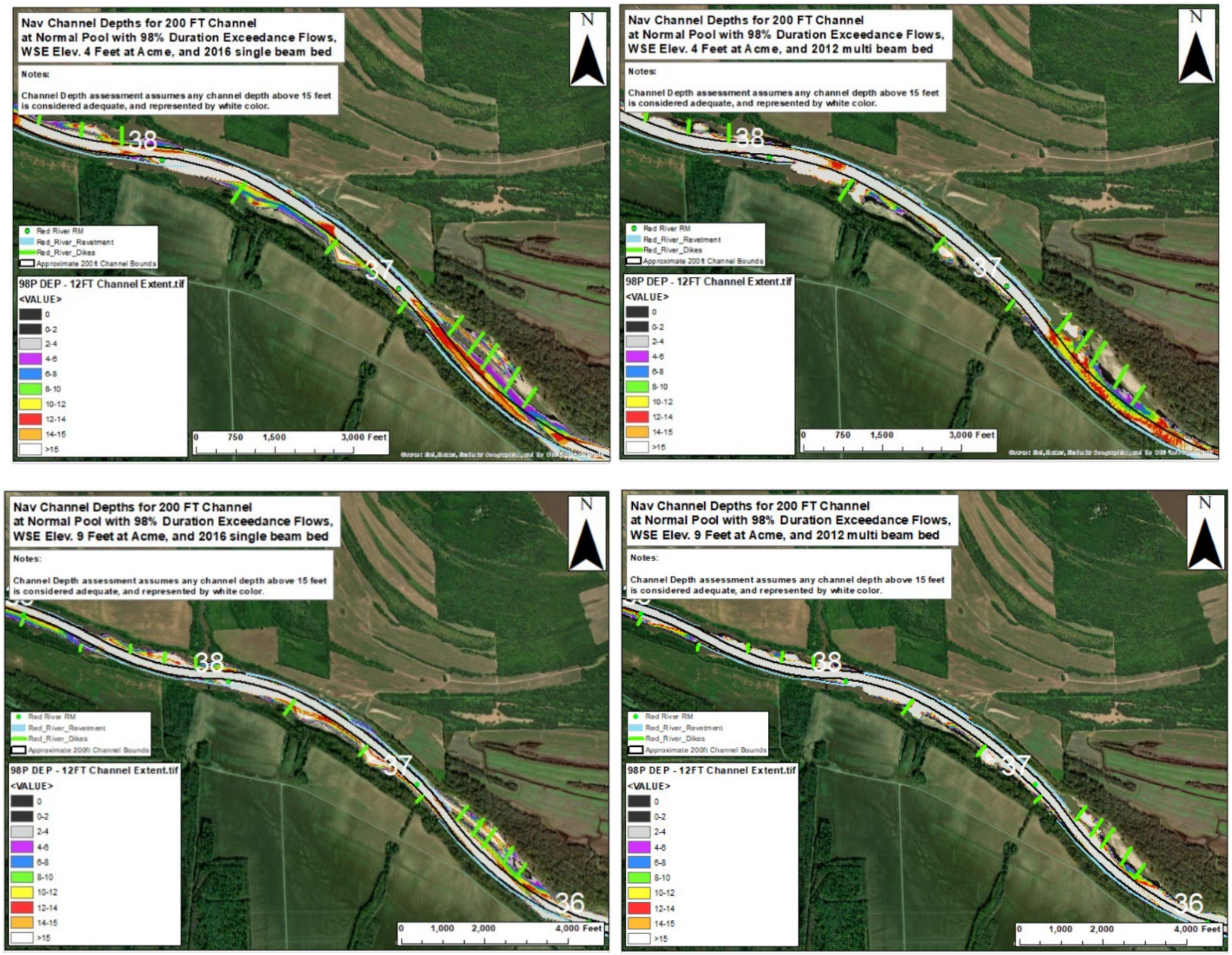



*Figure A-99. Channel Depths Maps Near RMs 36–38 with WSE 4 Feet and 9 Feet at Acme, Louisiana*

The 2012 multi-beam and 2016 single-beam provide a visual illustration of the depositional segments between RMs 34 and 35. The 2016 single-beam survey is an interpolated DEM between each set of adjacent cross-sections using RAS Mapper.


*Figure 100. 2012 Multi-Beam and 2016 Single-Beam Data Near RMs 34–35*

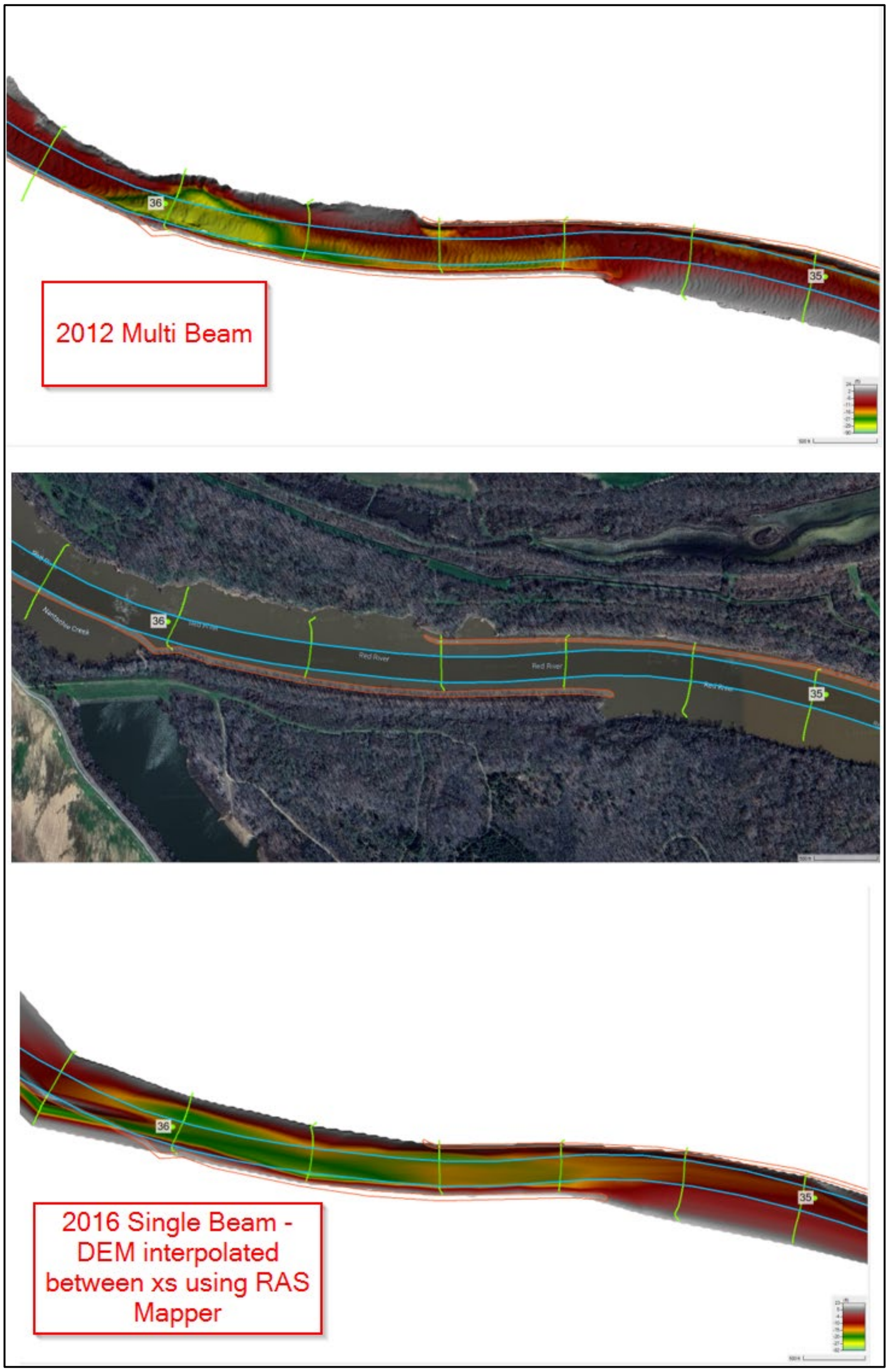



*Figure A-101. 2012 Multi-Beam and 2016 Single-Beam Data Near RMs 35–36*

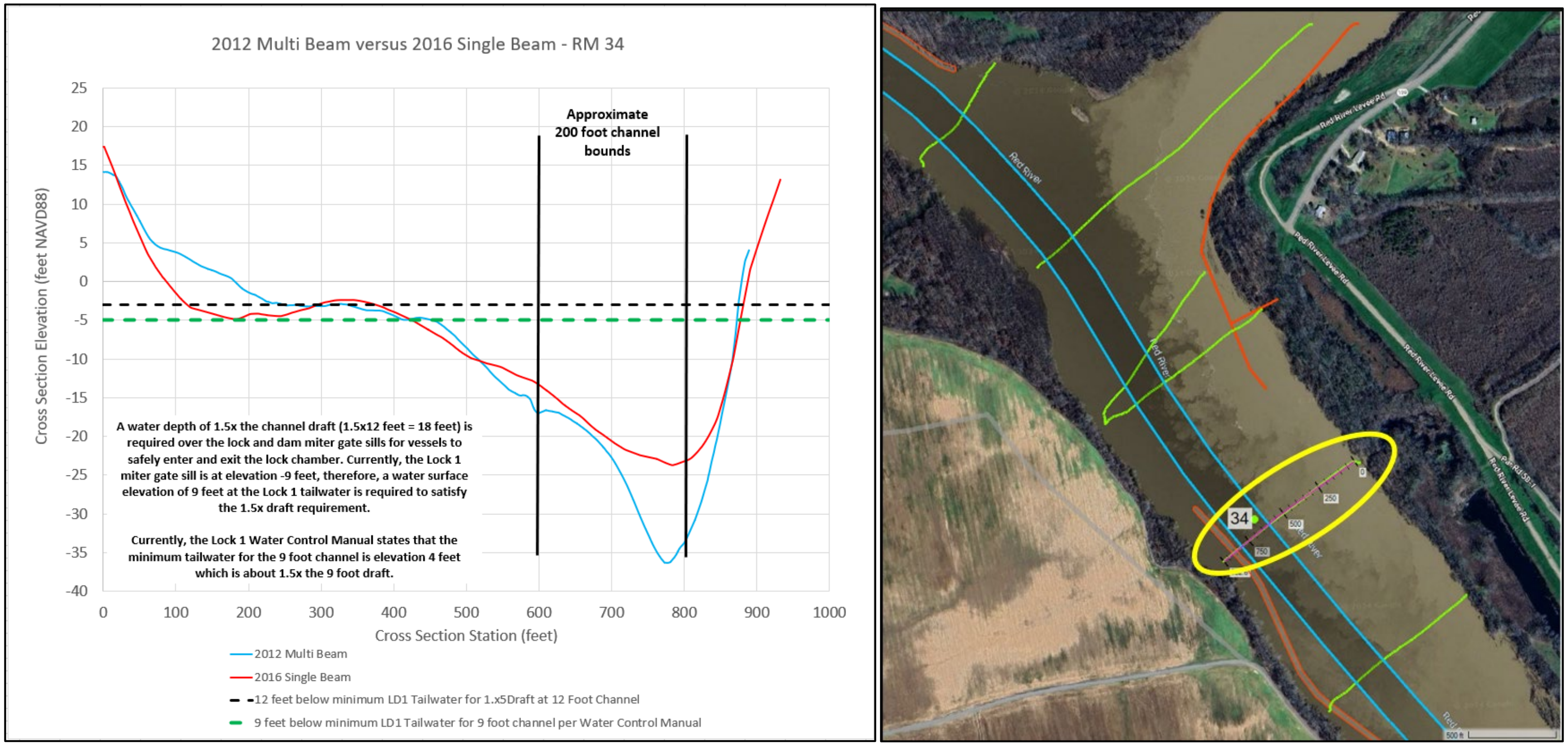
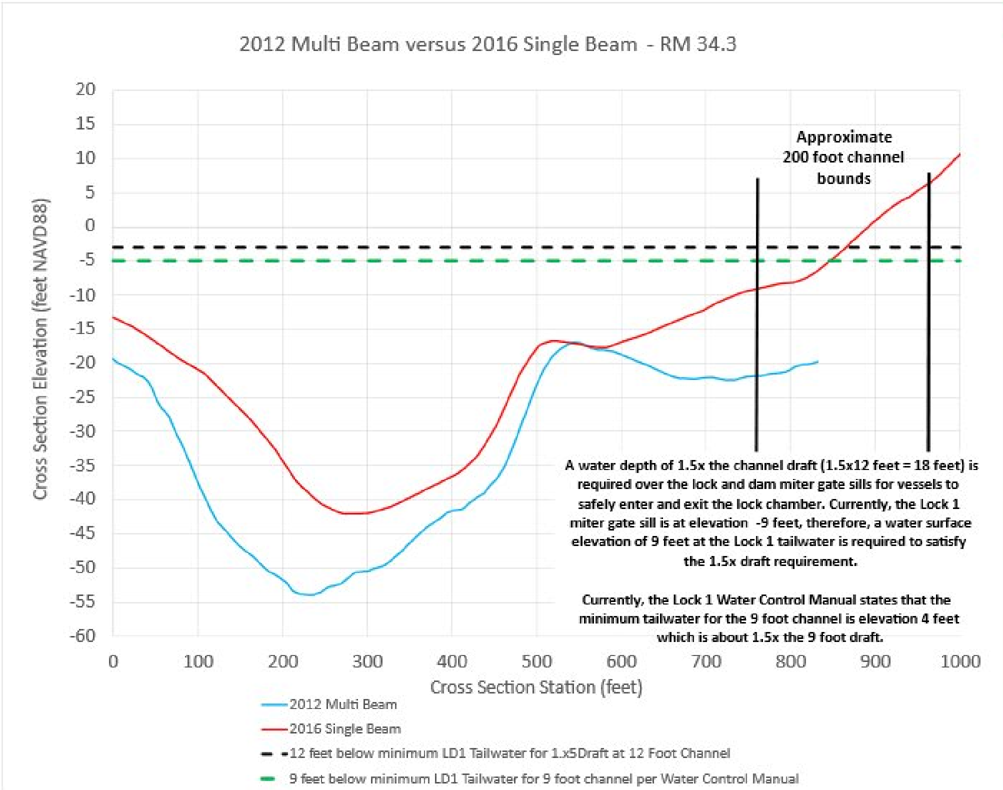



*Figure A-102. 2012 Multi-Beam Versus 2016 Single-Beam Cross-Section Comparison (RM 34)*



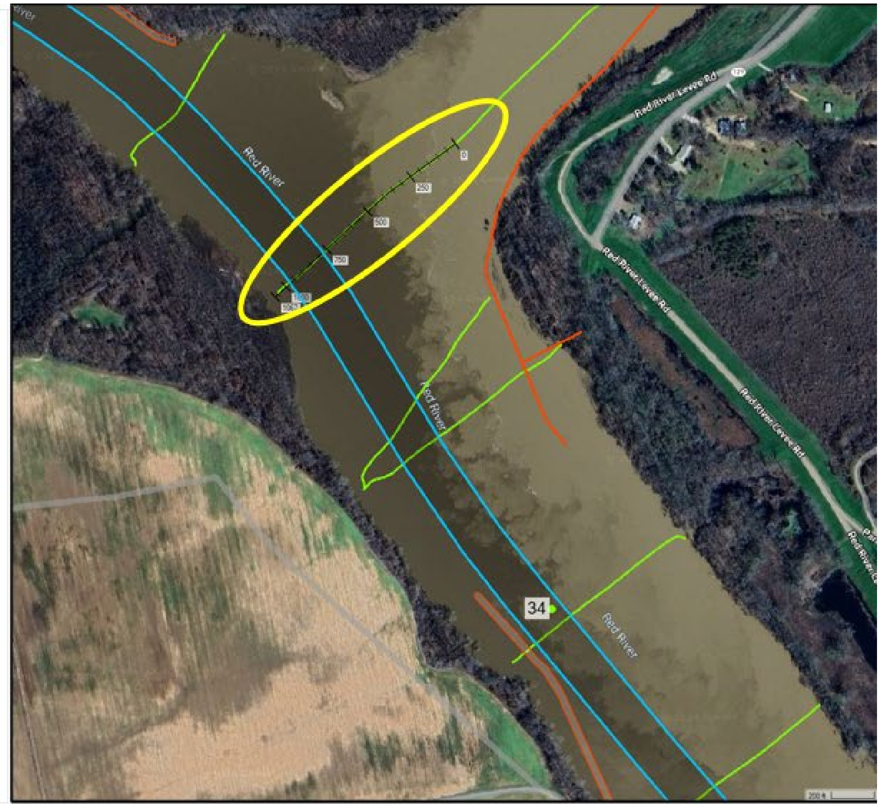




*Figure A-103. 2012 Multi-Beam Versus 2016 Single-Beam Cross-Section Comparison (RM 34.3)*

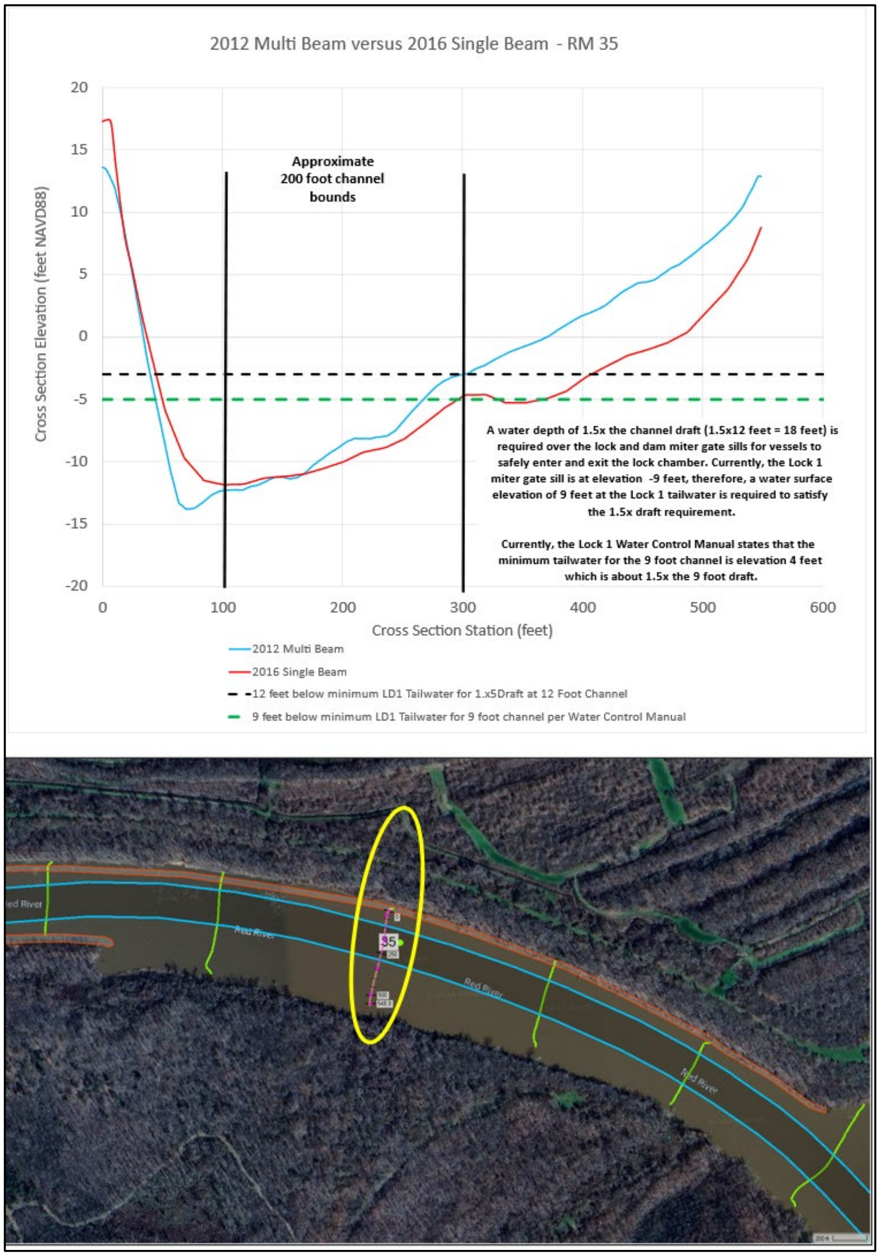



*Figure A-104. 2012 Multi-Beam Versus 2016 Single-Beam Cross-Section Comparison (RM 35)*

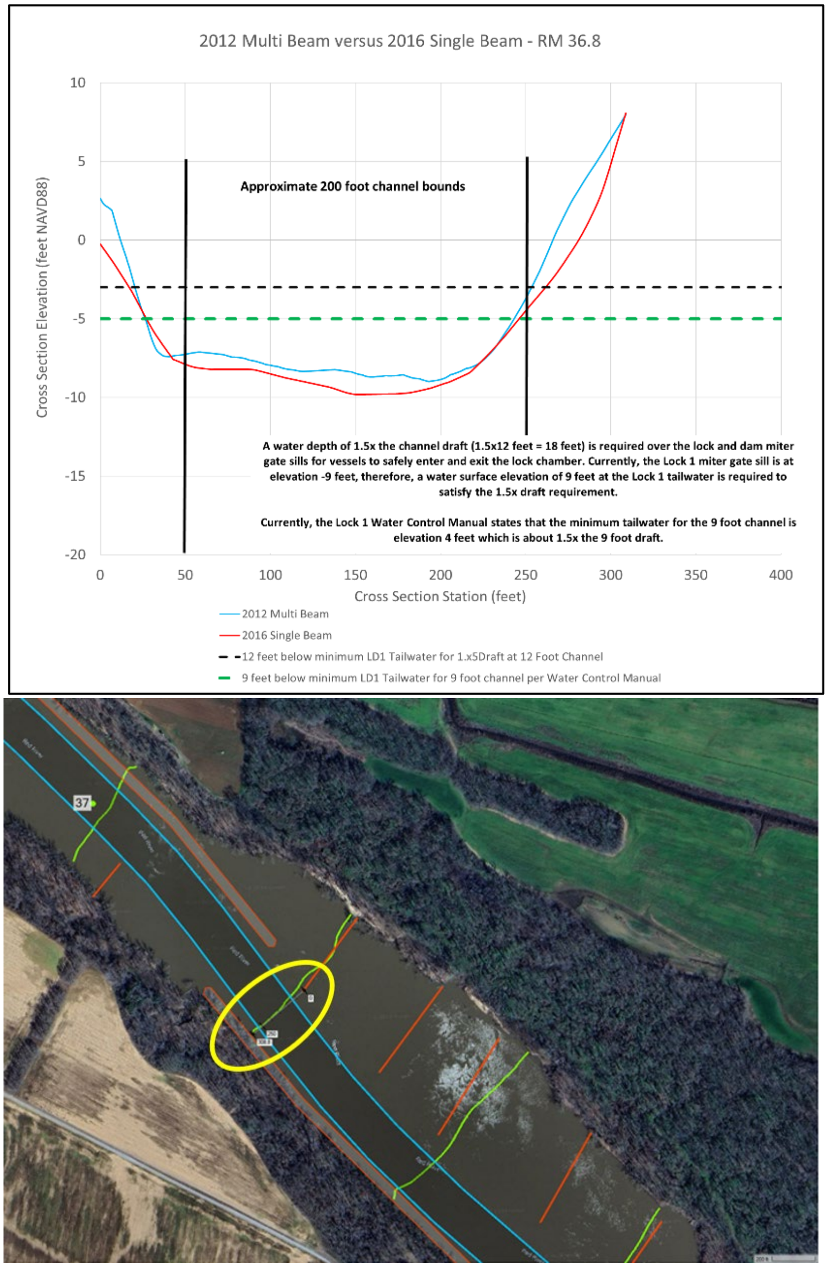



*Figure A-105. 2012 Multi-Beam Versus 2016 Single-Beam Cross-Section Comparison (RM 36.8)*

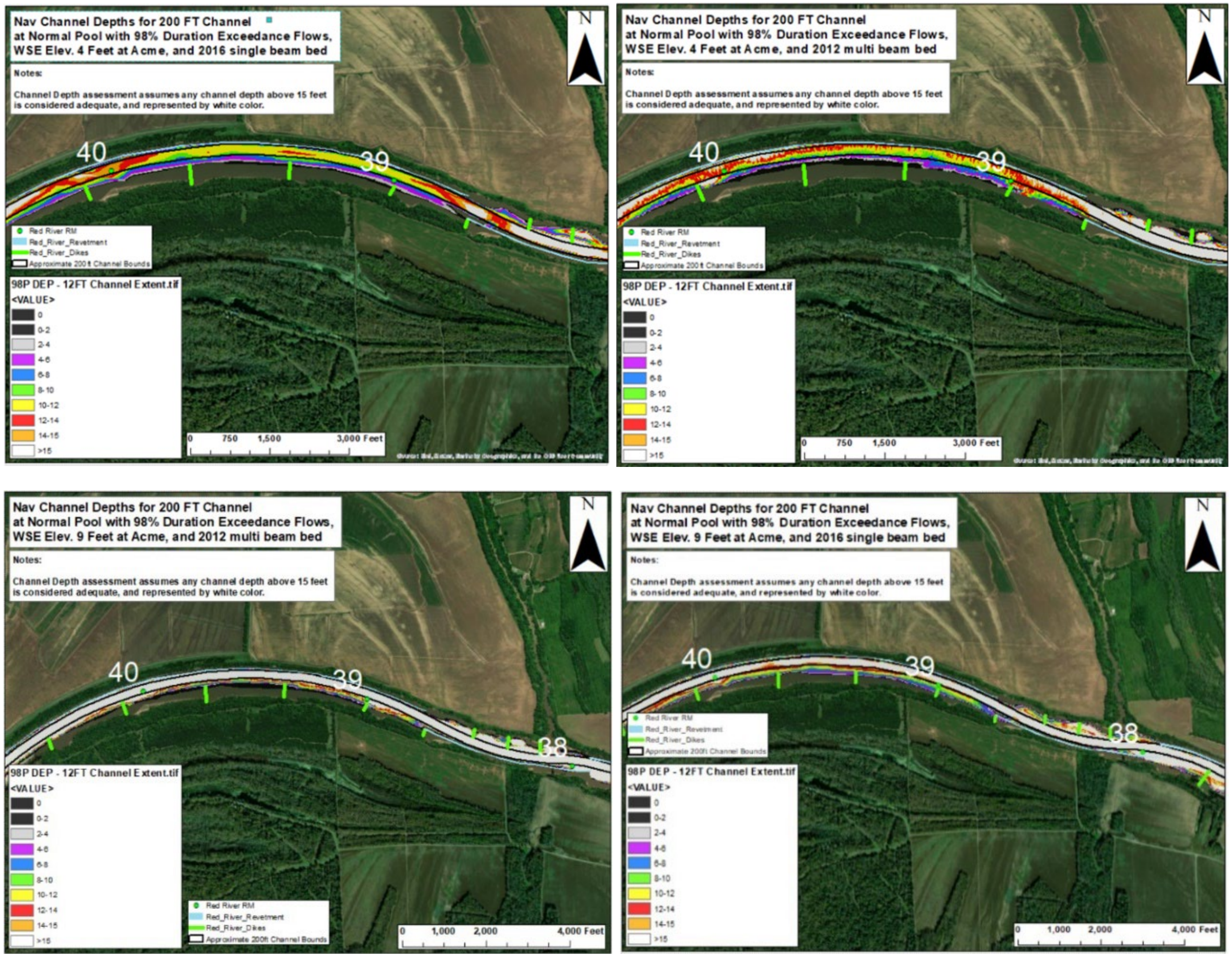



*Figure A-106. Channel Depths Maps Near RMs 38–40 with WSE 4 Feet Versus 9 Feet at Acme*

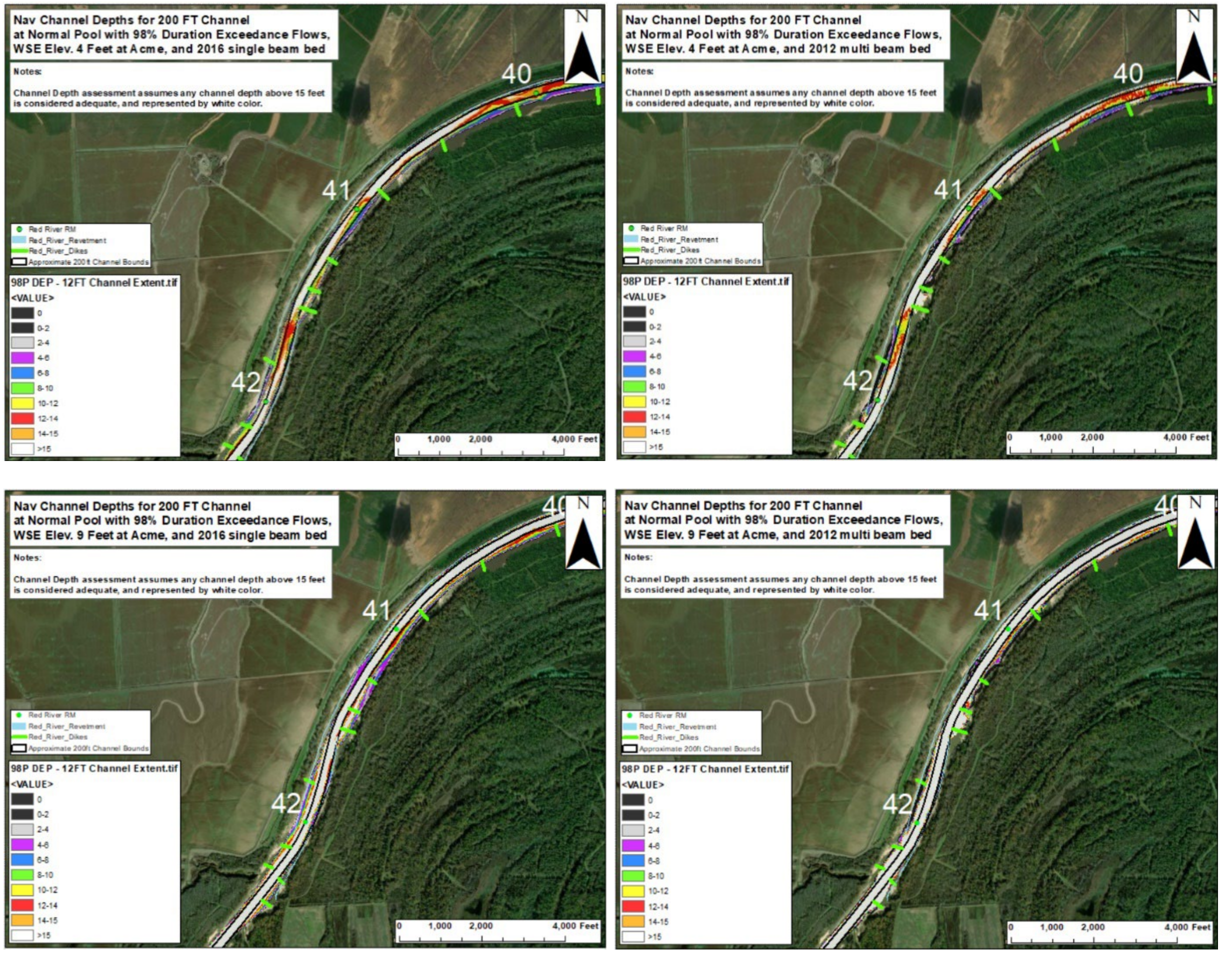



*Figure A-107. Channel Depths Maps Near RMs 40–42 with WSE 4 Feet Versus 9 Feet at Acme, Louisiana*

The 2012 multi-beam and 2016 single-beam provide a visual illustration of the depositional reach just in the vicinity of RM 39. The 2016 single-beam survey is an interpolated DEM between each set of adjacent cross-sections using RAS Mapper.

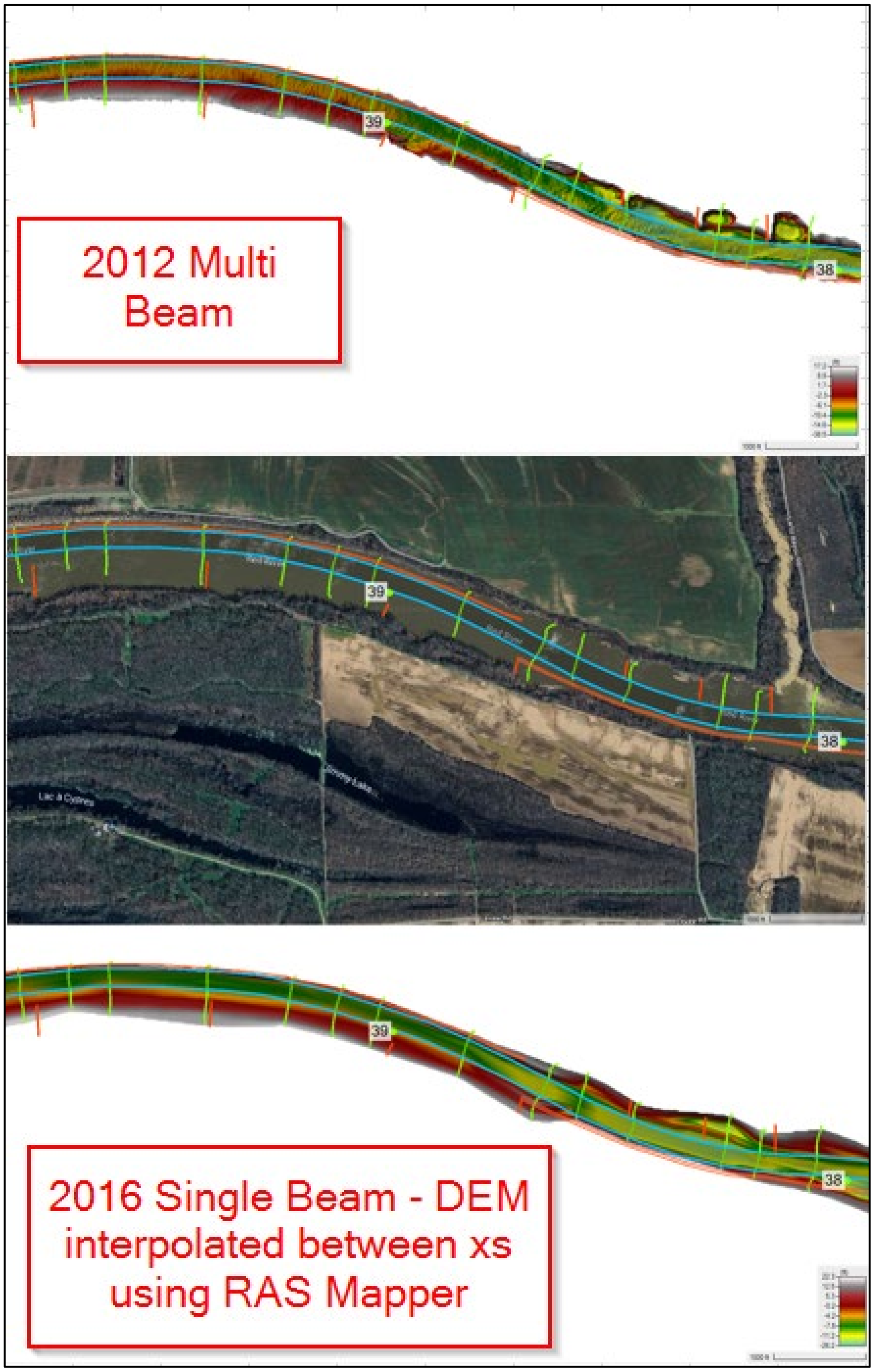



*Figure A-108. 2012 Multi-Beam and 2016 Single-Beam Near RMs 38–39*

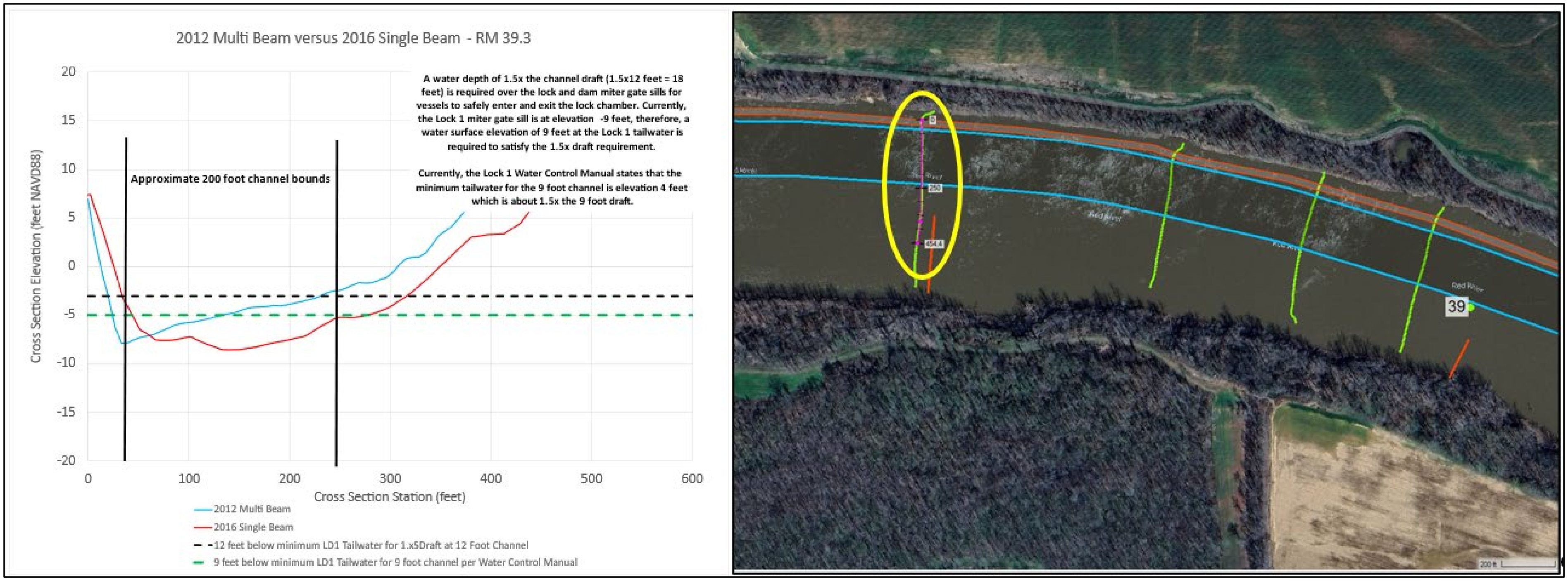



*Figure A-109. 2012 Multi-Beam Versus 2016 Single-Beam Cross-Section Comparison (RM 39.3)*

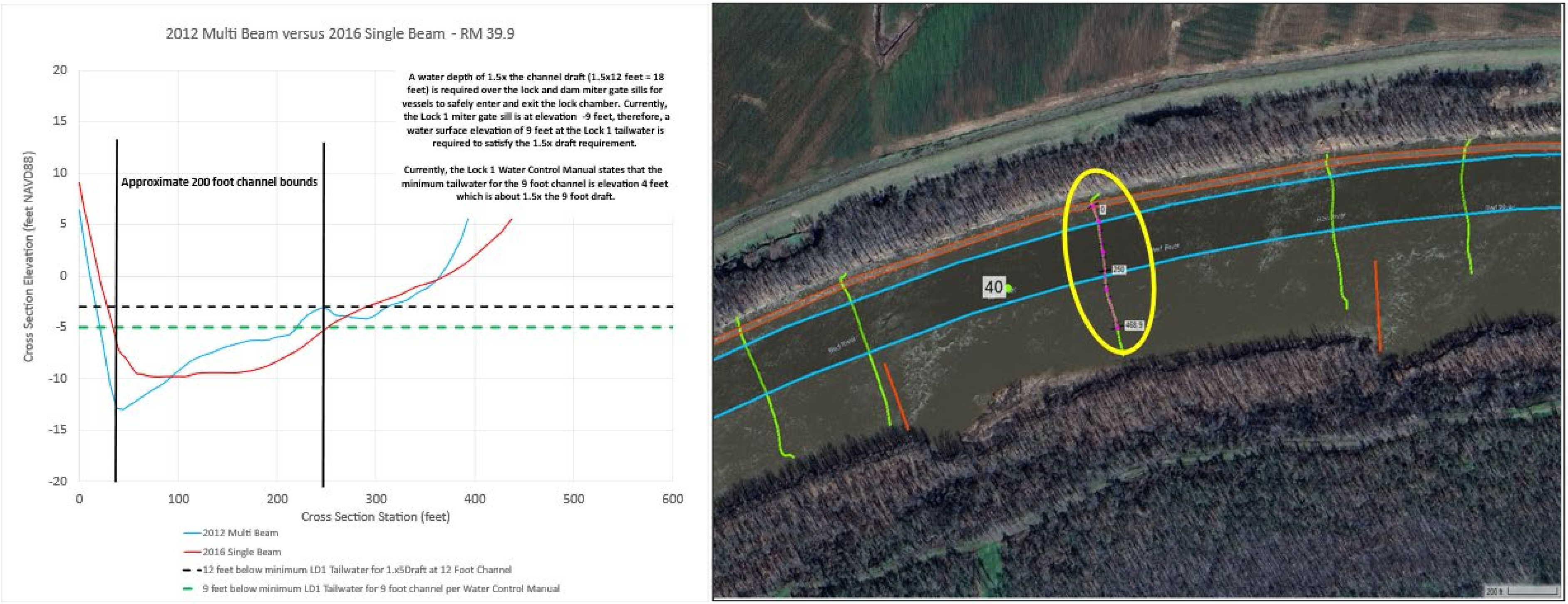



*Figure A-110. 2012 Multi-Beam Versus 2016 Single-Beam Cross-Section Comparison (RM 39.9)*

4.4.2.4 Medium Priority Problem Reaches

Pool 4 - RMs 194 (Williams/East Point)

Figure A-111 uses HEC-RAS generated depth grids at normal pool project design conditions (water surface elevation 120 feet NAVD88) to illustrate the potential problems related to navigation channel depths between RMs 195 and 193.

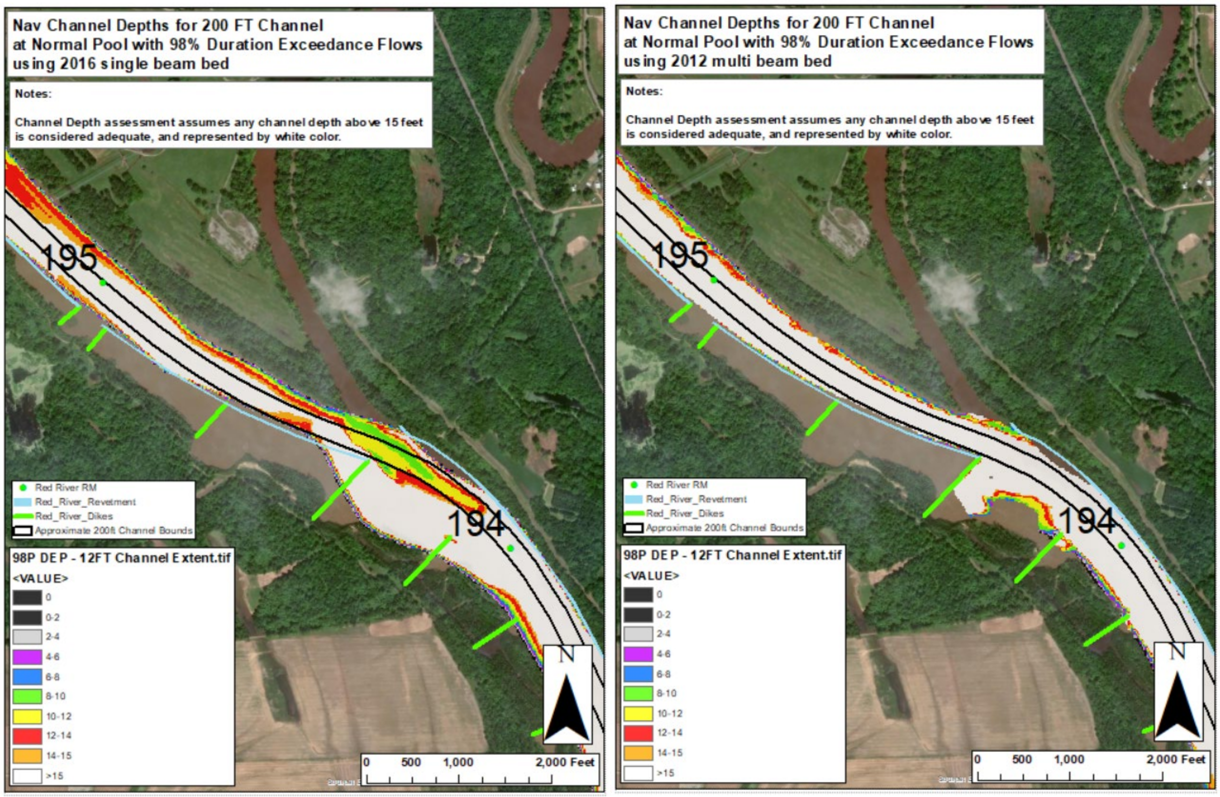



*Figure A-111. Pool 4 Normal Pool (WSEL 120 feet) Channel Depths Maps near RM 194*

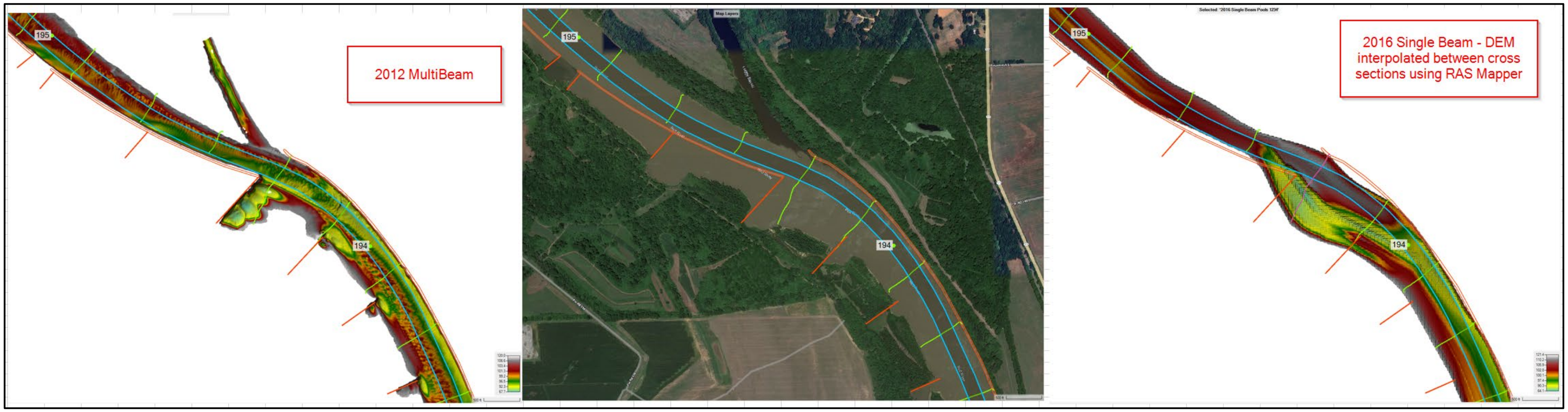



*Figure A-112. 2012 Multi-Beam and 2016 Single-Beam Near RM 194*

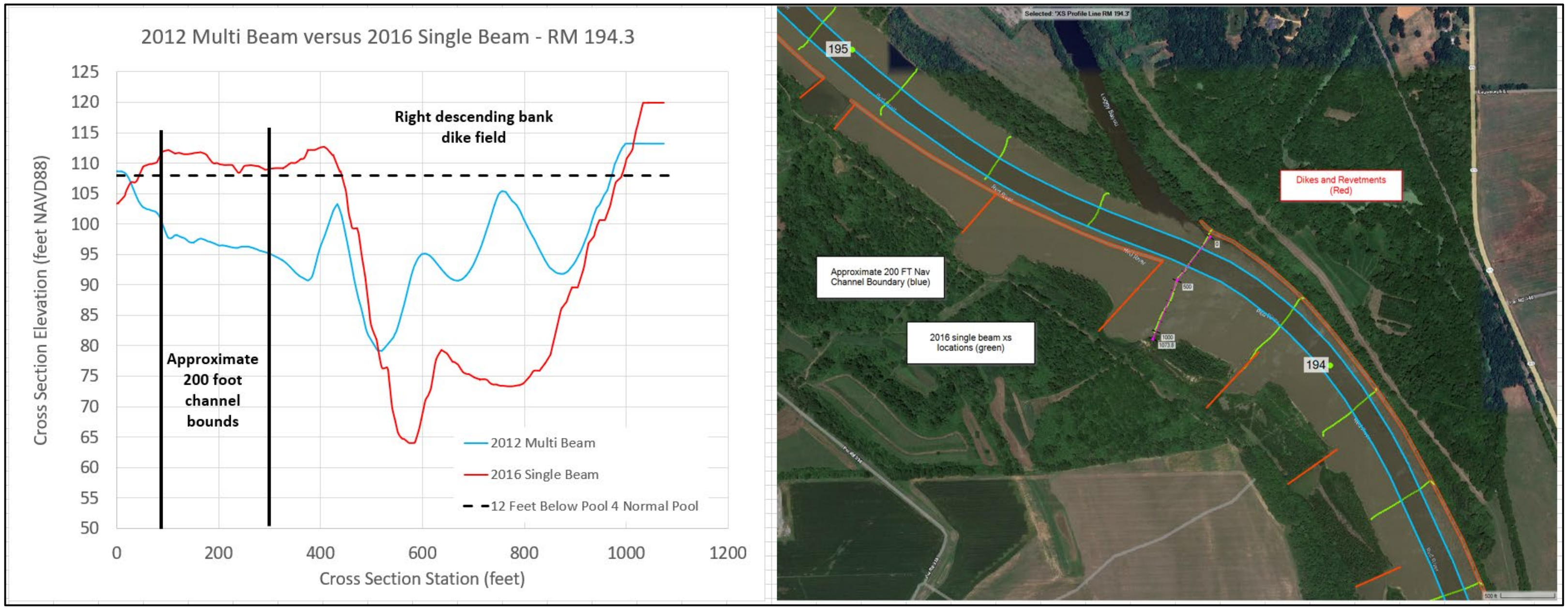



*Figure A-113. 2012 Multi-Beam Versus 2016 Single-Beam Cross-Section Comparison (RM 194)*

Pool 3 - RM 158 (Campti)

Figure A-114 uses HEC-RAS generated depth grids at normal pool project design conditions (water surface elevation 95 feet NAVD88) to illustrate the potential problems related to navigation channel depths near RM 158.

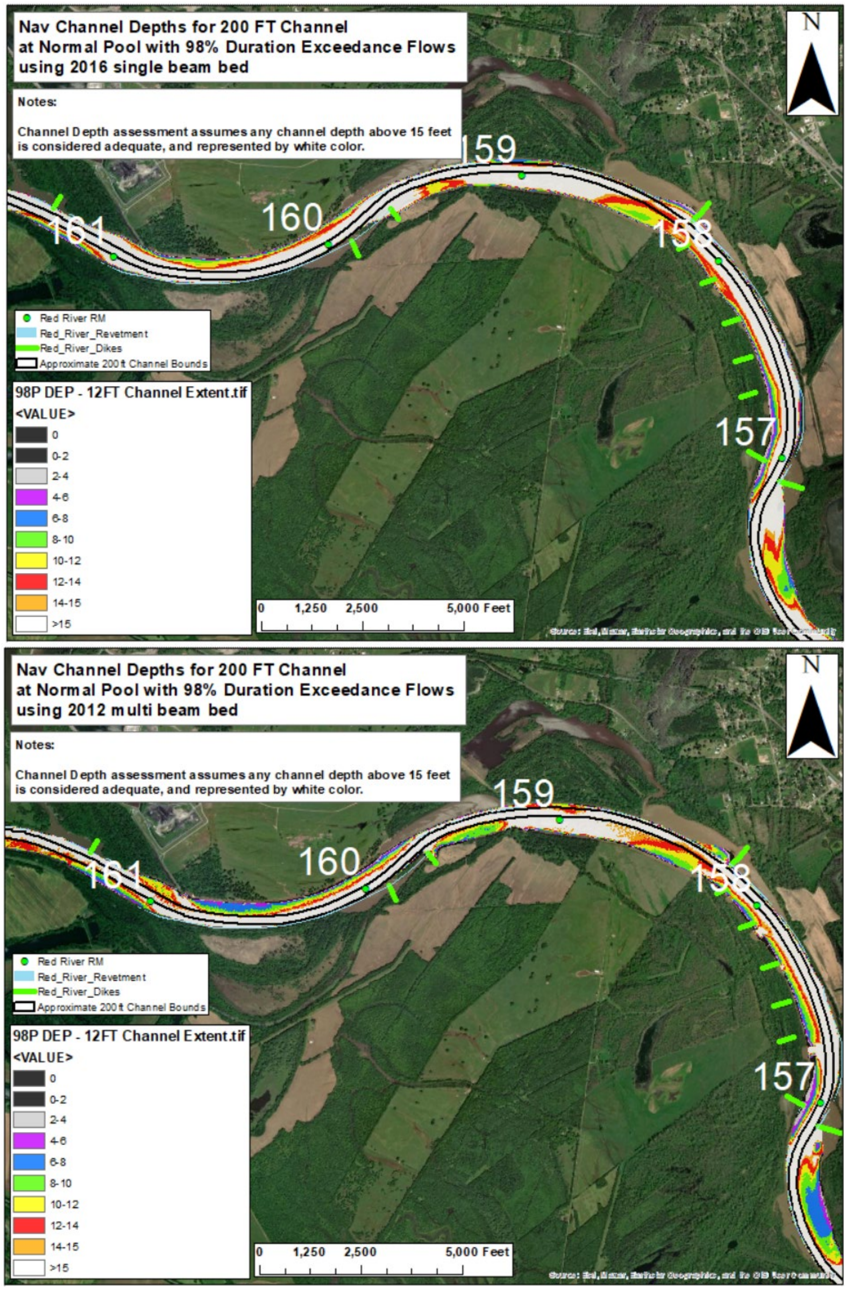



*Figure A-114. Pool 3 Normal Pool (WSEL 95 Feet) Channel Depths Maps Near RM 158*

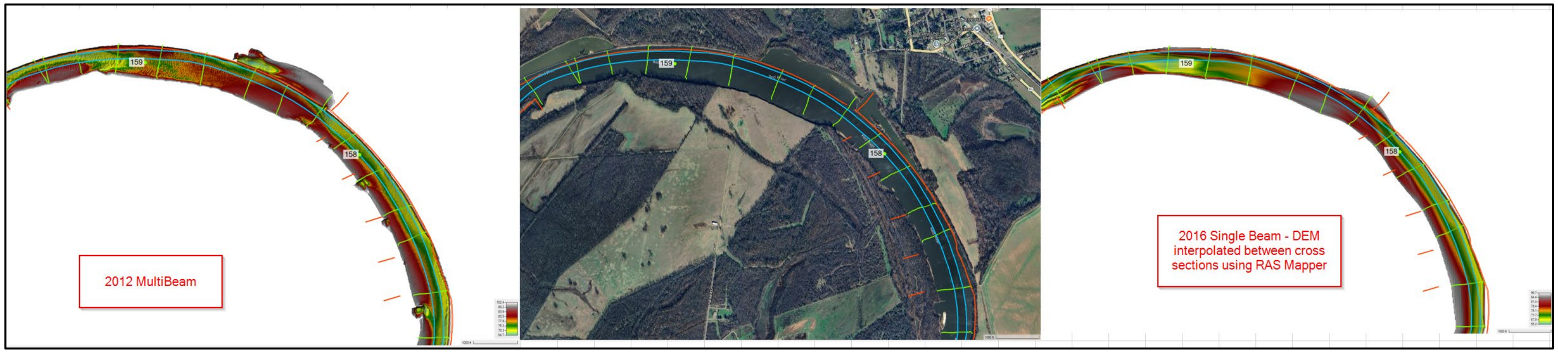



*Figure A-115. 2012 Multi-Beam Versus 2016 Single-Beam Cross-Section Comparison (RM 158)*

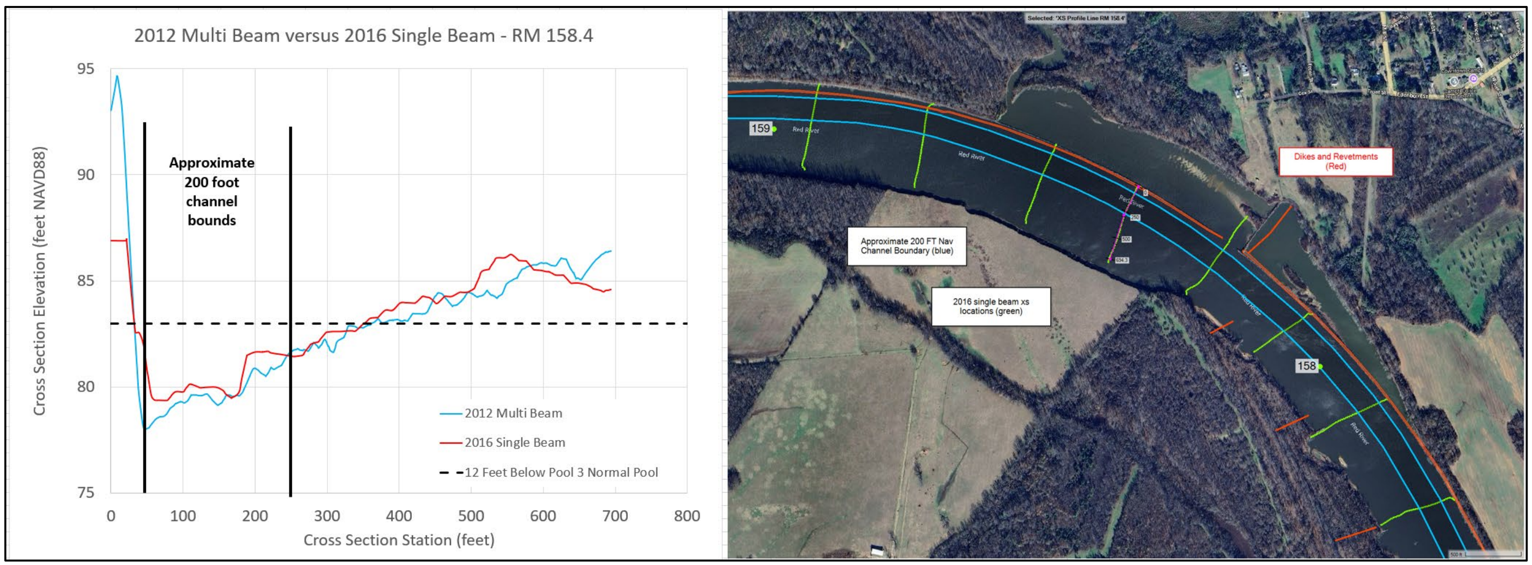



*Figure A-116. 2012 Multi-Beam Versus 2016 Single-Beam Cross-Section Comparison (RM 158)*

<u>Pool 3 - RM 154 (Socot)</u>

Figure A-117 uses HEC-RAS generated depth grids at normal pool project design conditions (water surface elevation 95 feet NAVD88) to illustrate the potential problems related to navigation channel depths near RM 154.

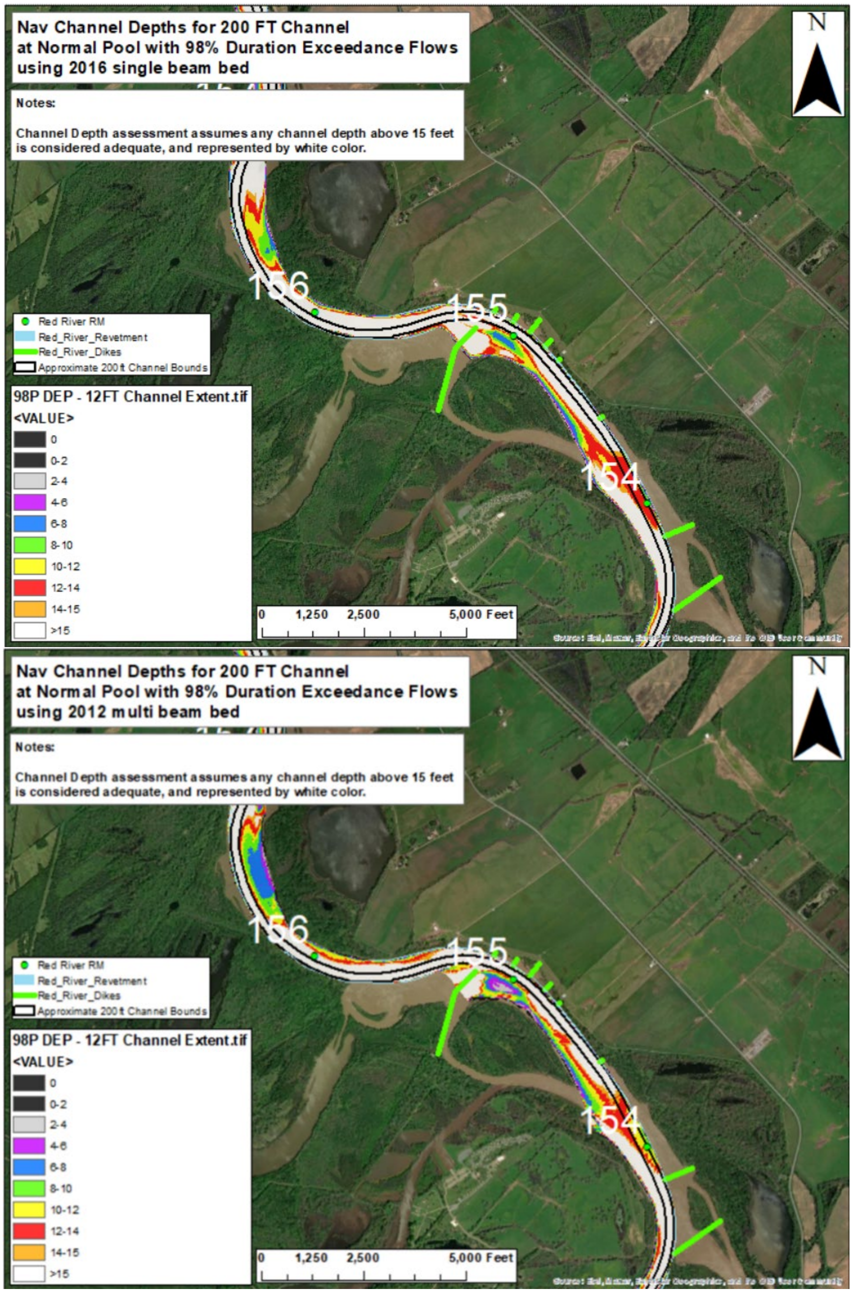



*Figure A-117. Pool 3 Normal Pool (WSEL 95 Feet) Channel Depths Maps near RM 154*

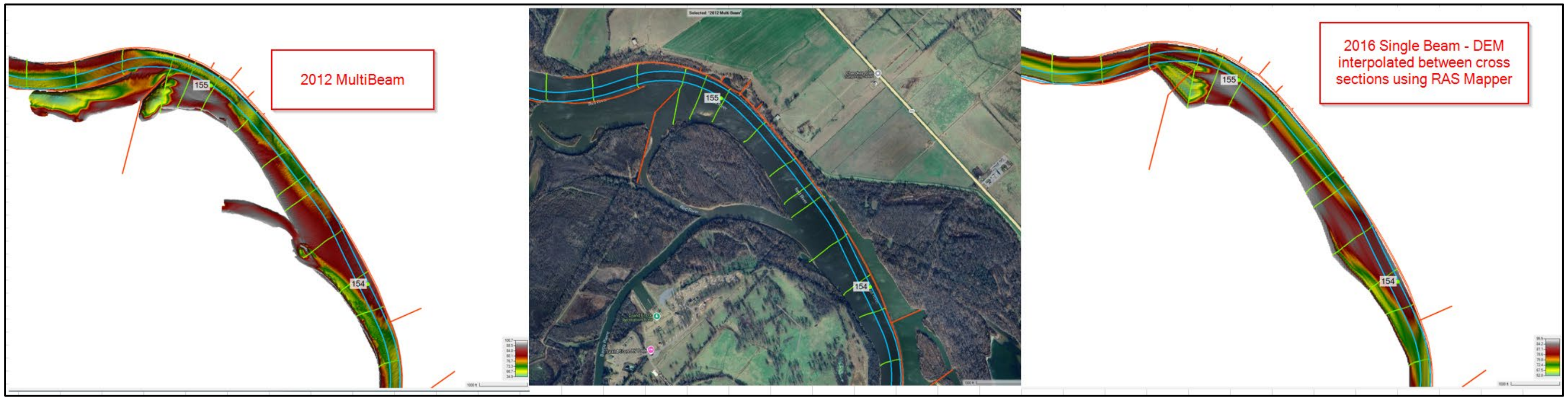



*Figure A-118. 2012 Multi-Beam Versus 2016 Single-Beam Cross-Section Comparison (RM 154)*

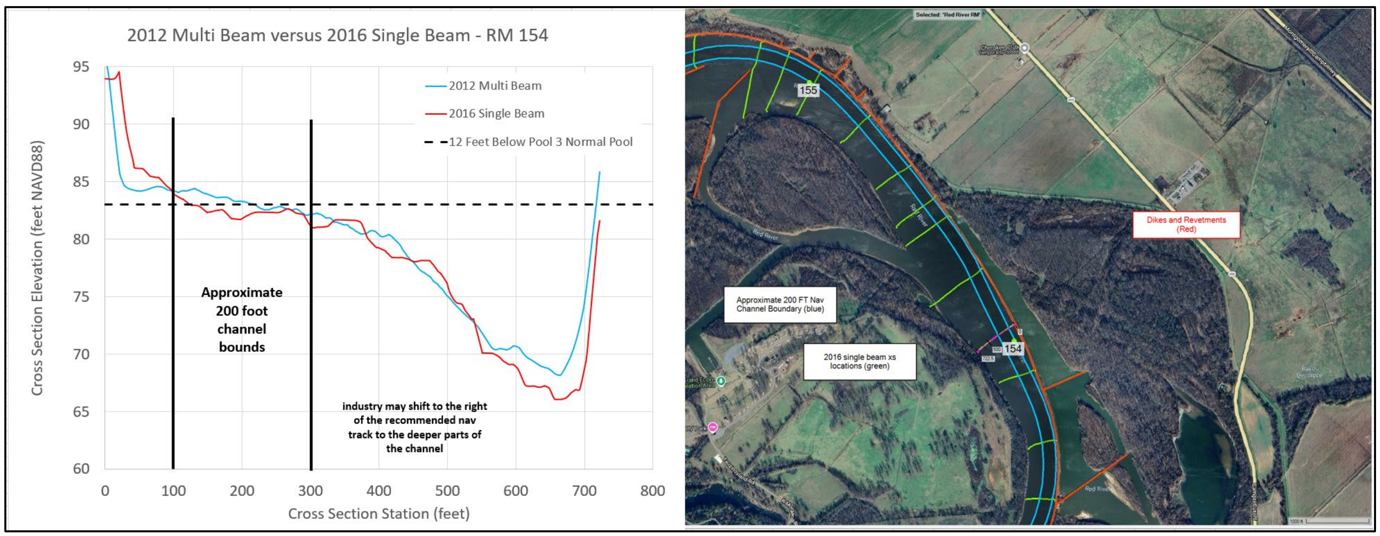



*Figure A-119. 2012 Multi-Beam Versus 2016 Single-Beam Cross-Section Comparison (RM 154)*

Pool 1 - RM 64

Figure A-120 uses HEC-RAS generated depth grids at normal pool project design conditions (water surface elevation 40 feet NAVD88) to illustrate the potential problems related to navigation channel depths near RM 64.

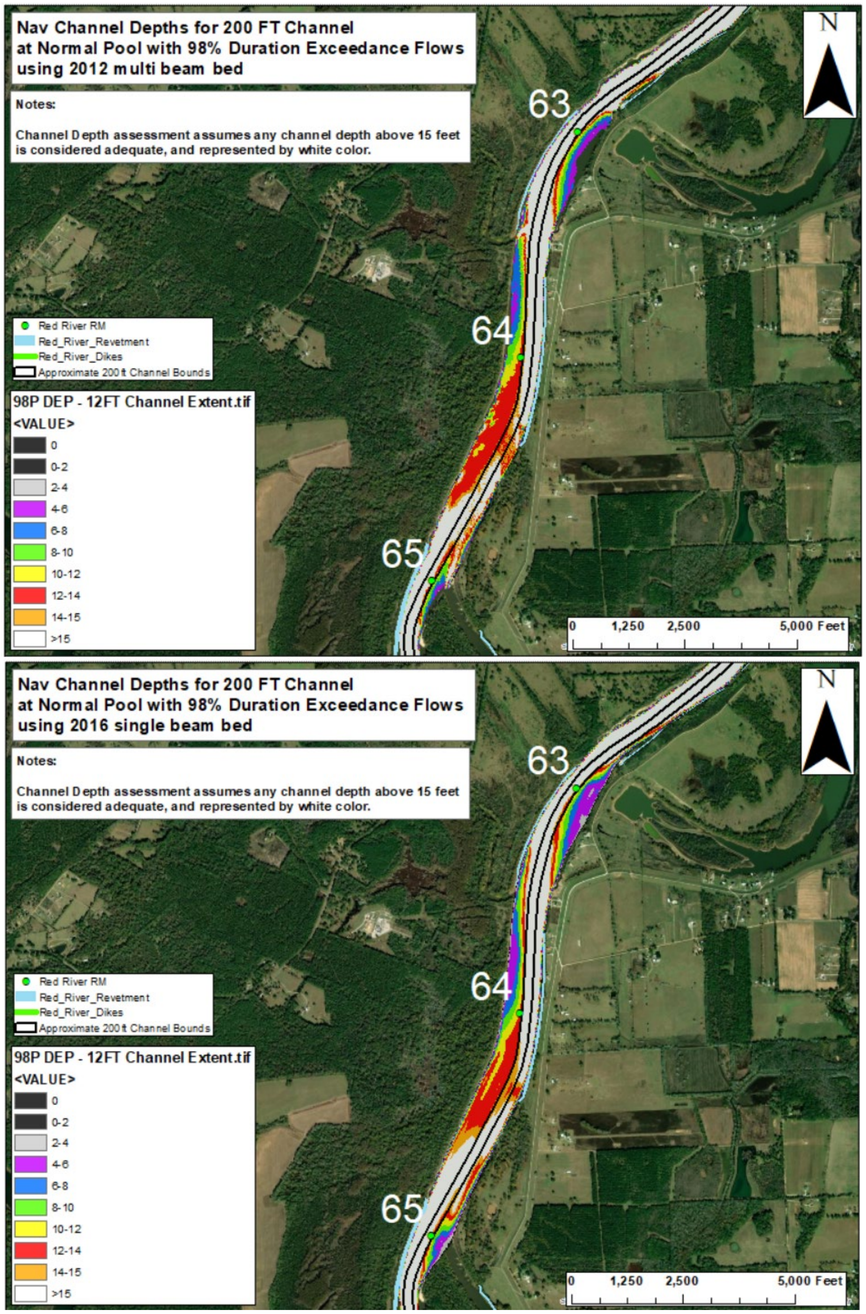



*Figure A-120. Pool 1 Normal Pool (WSEL 40 feet) Channel Depths Maps Near RM 64*

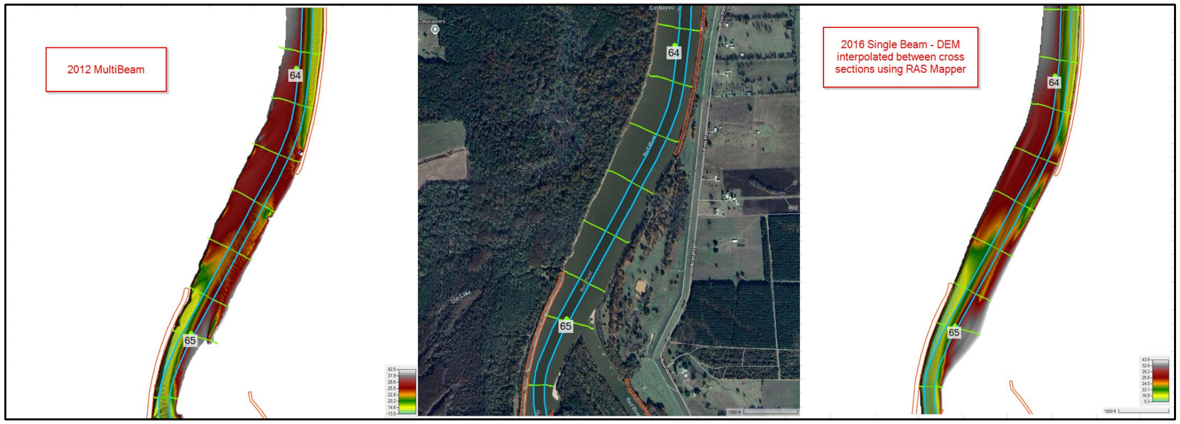



*Figure A-121. 2012 Multi-Beam Versus 2016 Single-Beam Cross-Section Comparison (RM 64)*

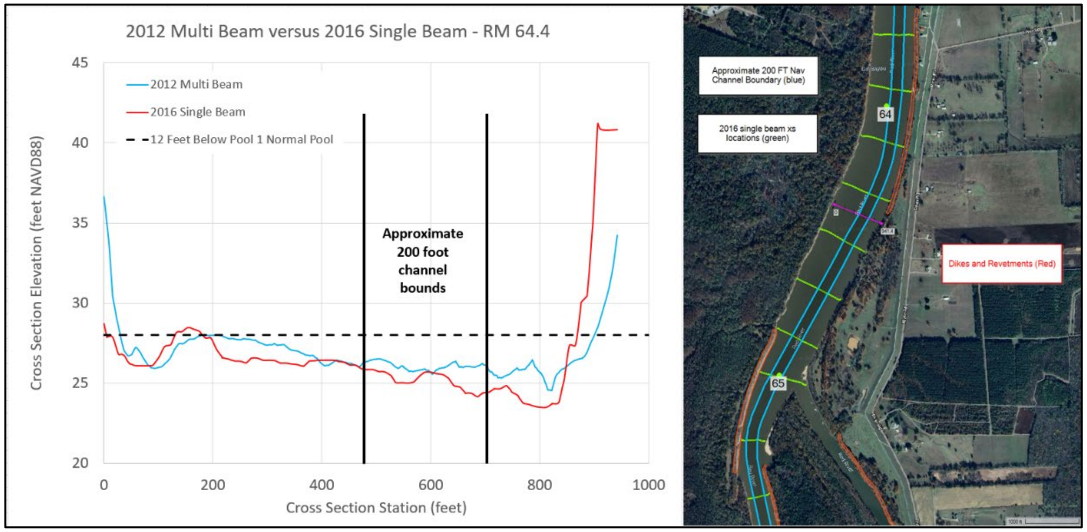



*Figure A-122. 2012 Multi-Beam Versus 2016 Single-Beam Cross-Section Comparison (RM 64)*

Pool 1 - RM 61

Figure A-123 uses HEC-RAS generated depth grids at normal pool project design conditions (water surface elevation 40 feet NAVD88) to illustrate the potential problems related to navigation channel depths near RM 61.

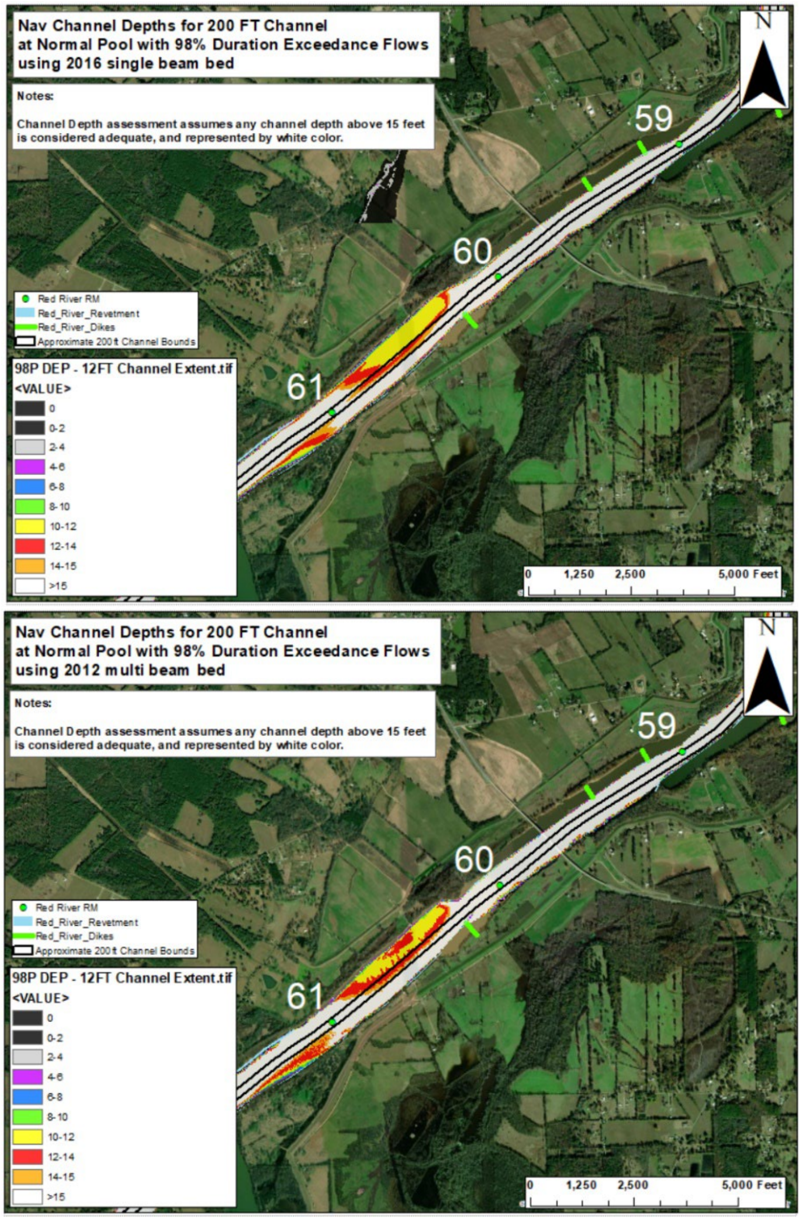



*Figure A-123. Pool 1 Normal Pool (WSEL 40 Feet) Channel Depths Maps near RM 61*

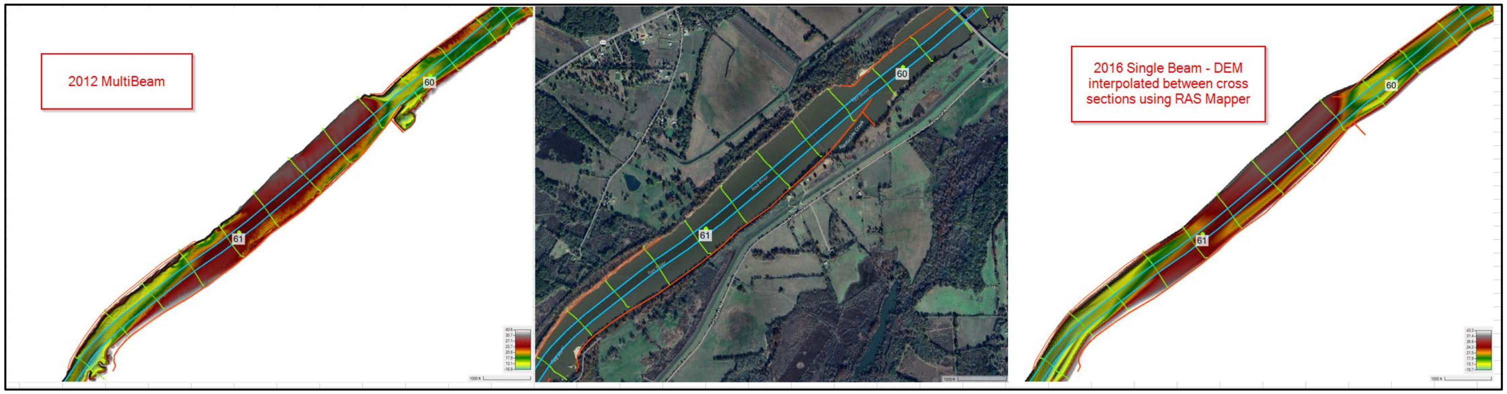



*Figure A-124. 2012 Multi-beam versus 2016 Single-beam Cross-Section Comparison (RM 61)*

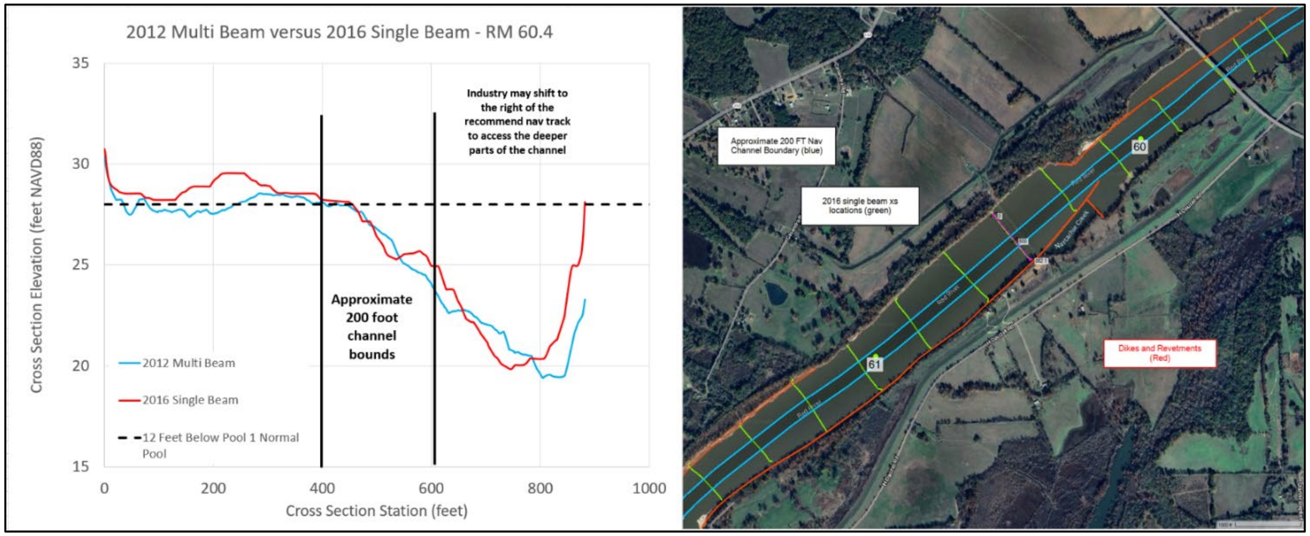



*Figure A-125. 2012 Multi-Beam Versus 2016 Single-Beam Cross-Section Comparison (RM 61)*

Pool 1 - RM 52

Figure A-126 uses HEC-RAS generated depth grids at normal pool project design conditions (water surface elevation 40 feet NAVD88) to illustrate the potential problems related to navigation channel depths near RM 52.

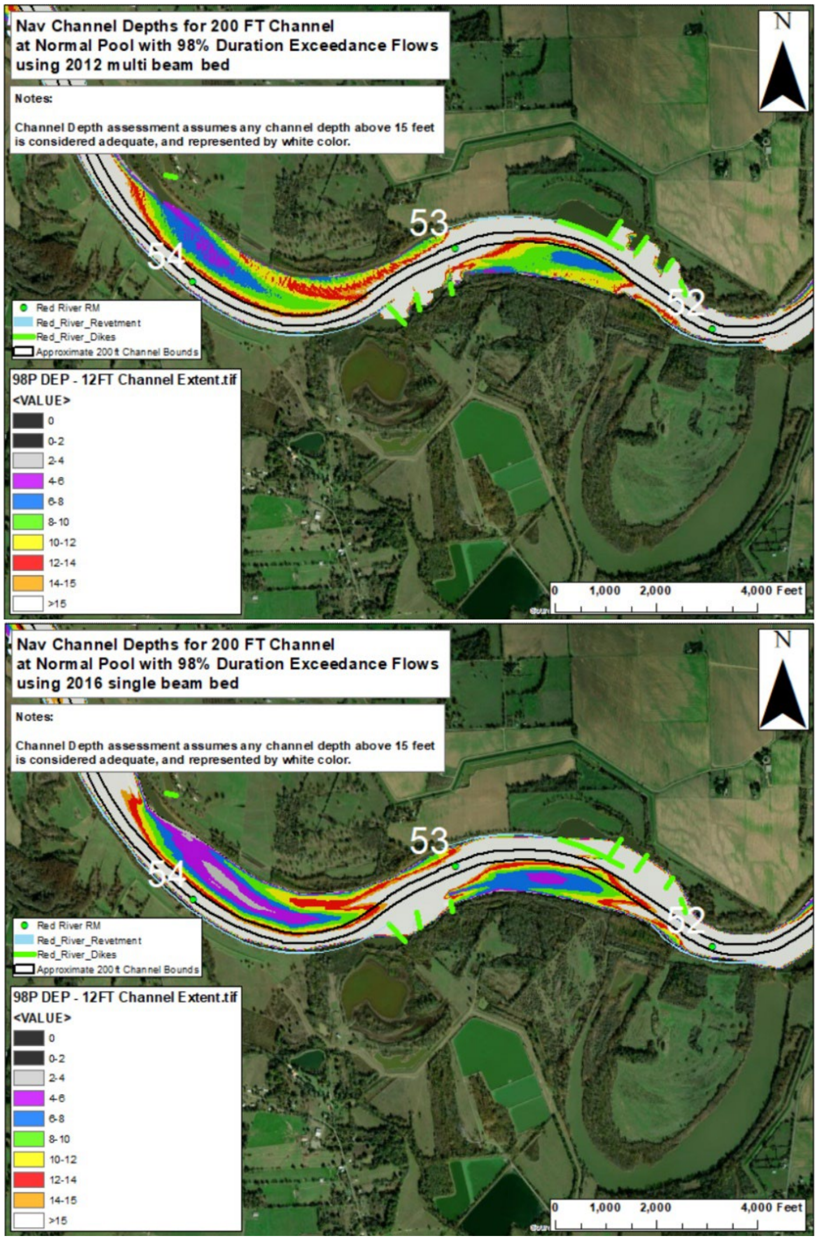



*Figure A-126. Pool 1 Normal Pool (WSEL 40 Feet) Channel Depths Maps Near RM 52*

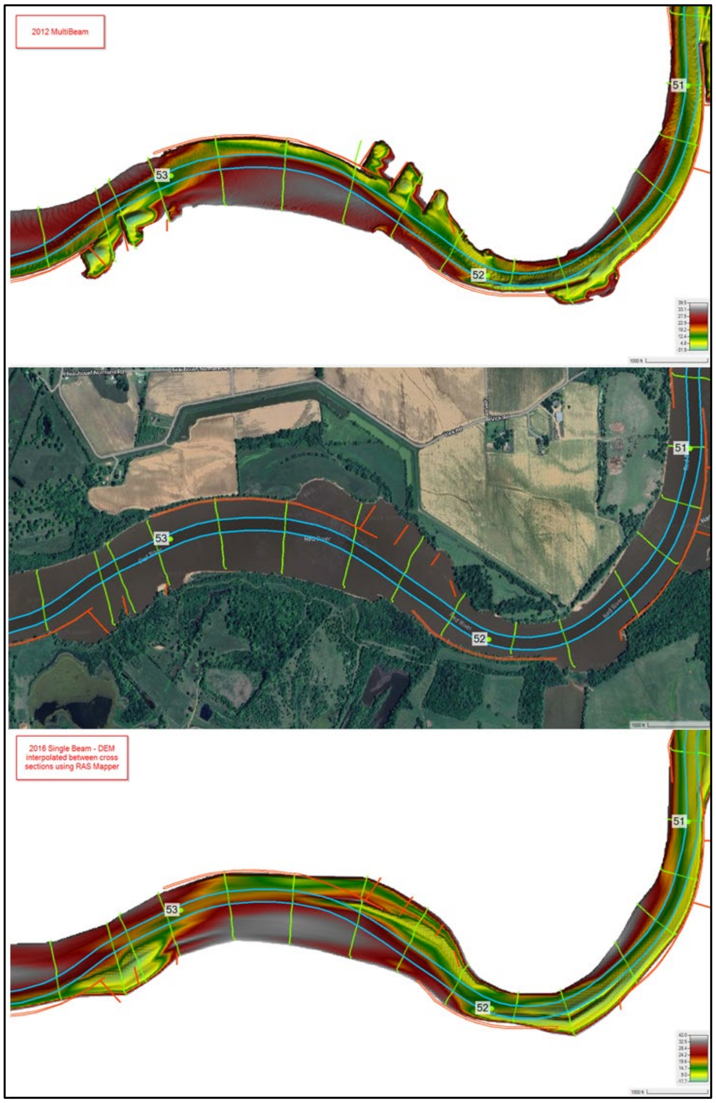



*Figure A-116. 2012 Multi-Beam and 2016 Single-Beam Near RM 52*

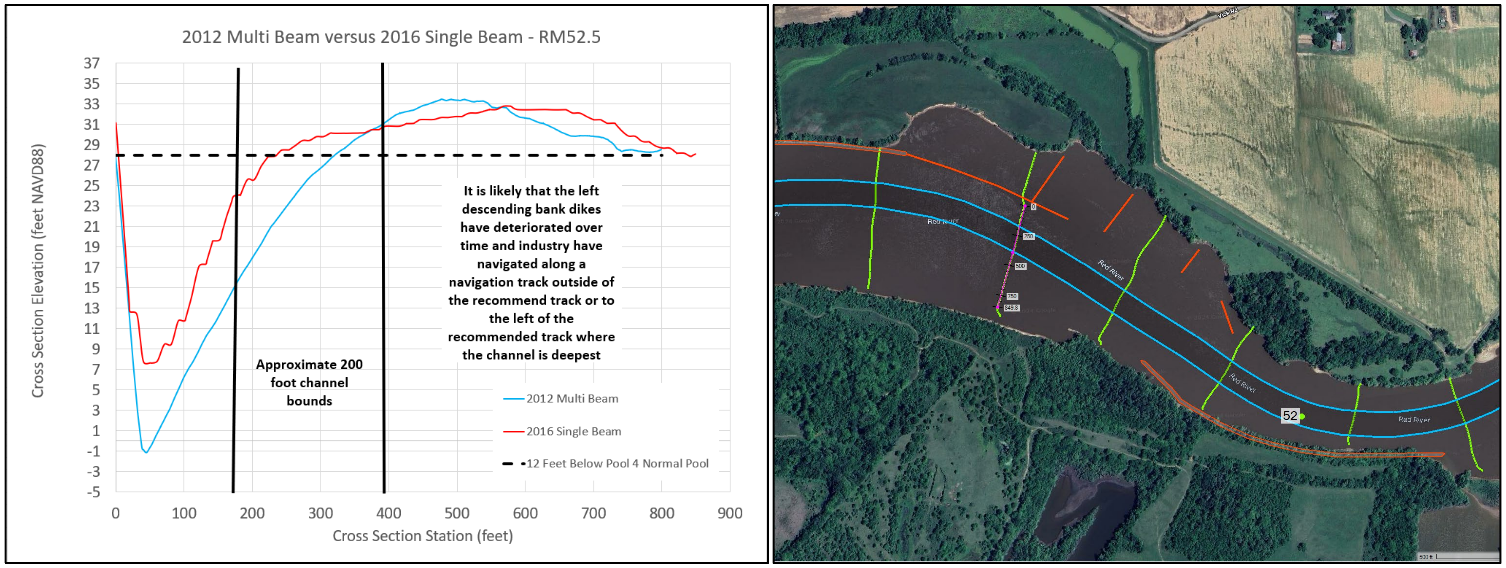



*Figure A-127. 2012 Multi-Beam Versus 2016 Single-Beam Cross-Section Comparison (RM 52)*

#### 4.4.2.5 Lower Priority Problem Reaches

Pool 3 - RMs 163–165

Figure A-128 uses HEC-RAS generated depth grids at normal pool project design conditions (water surface elevation 95 feet NAVD88) to illustrate the potential problems related to navigation channel depths near RM 163 to 165.

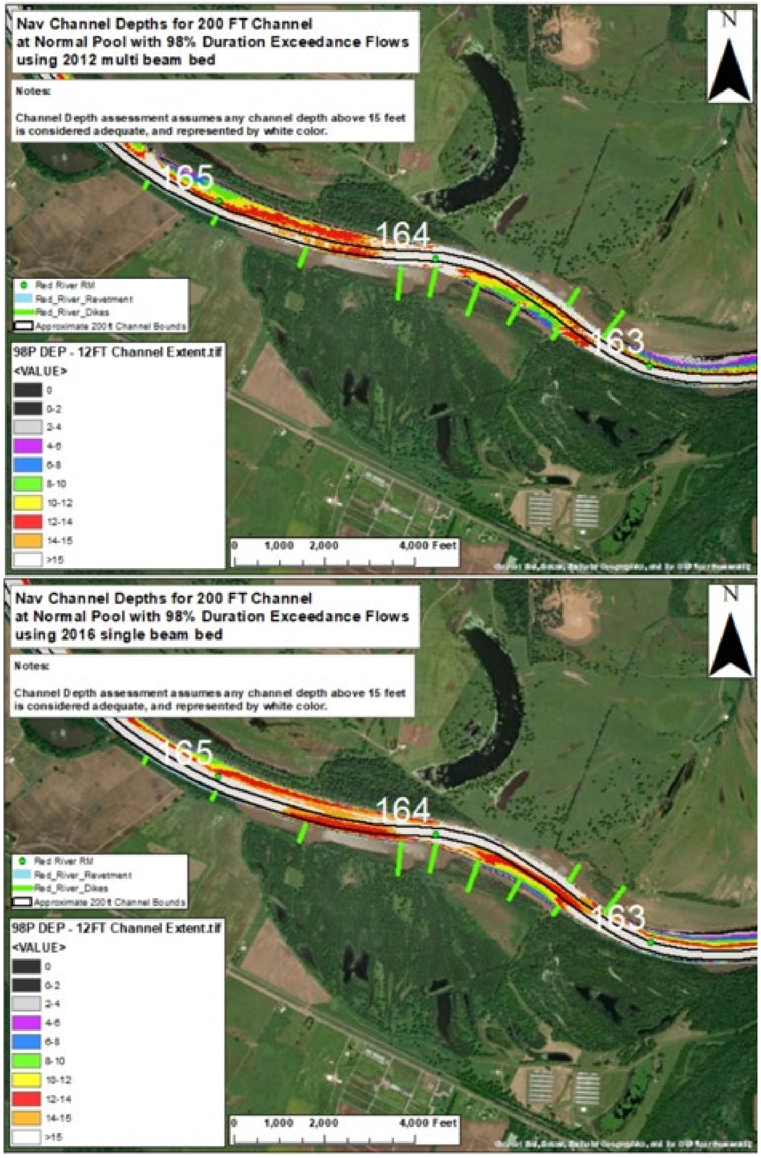



*Figure A-128. Pool 3 Normal Pool (WSEL 95 feet) Channel Depths Maps Near RMs 163-165*

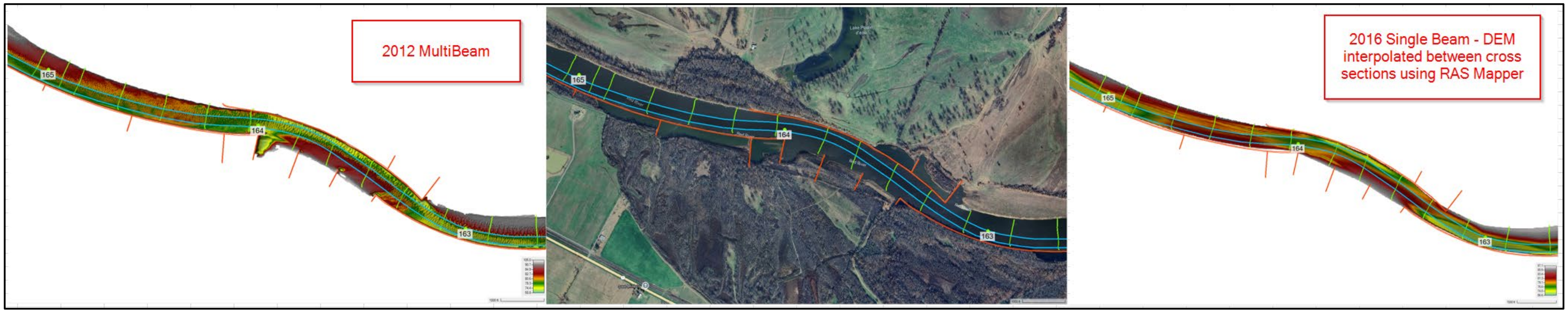



*Figure A-129. 2012 Multi-Beam and 2016 Single-Beam Near RMs 163-165*

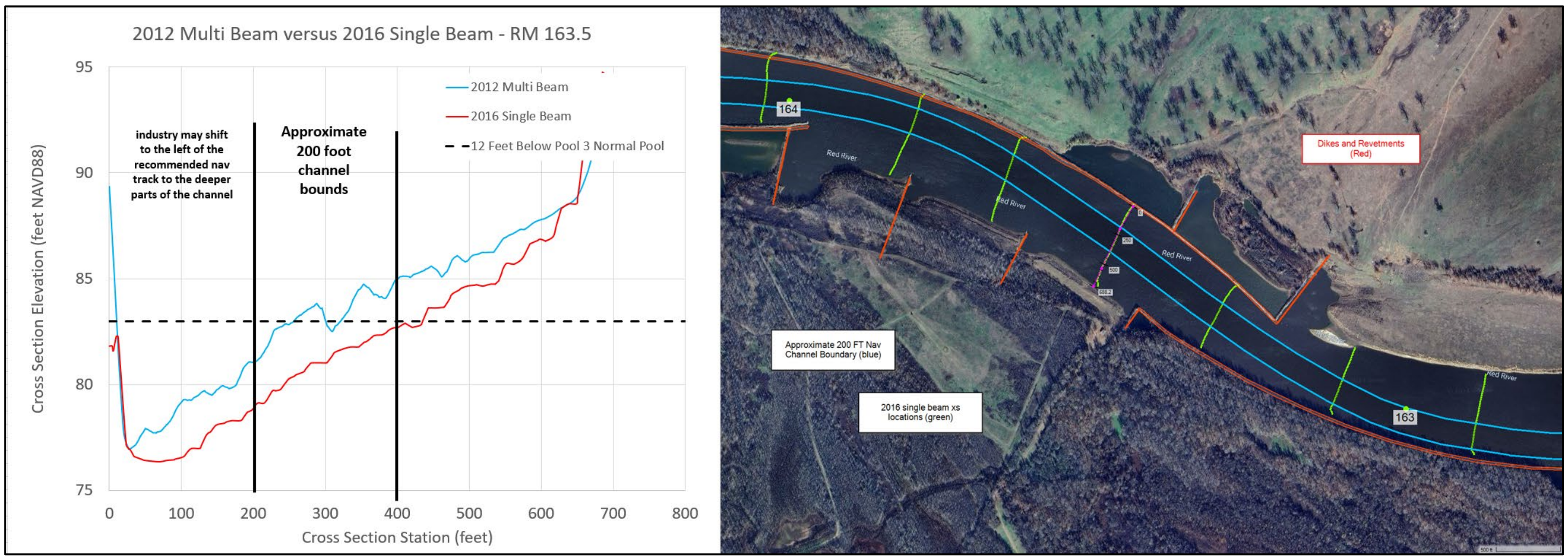
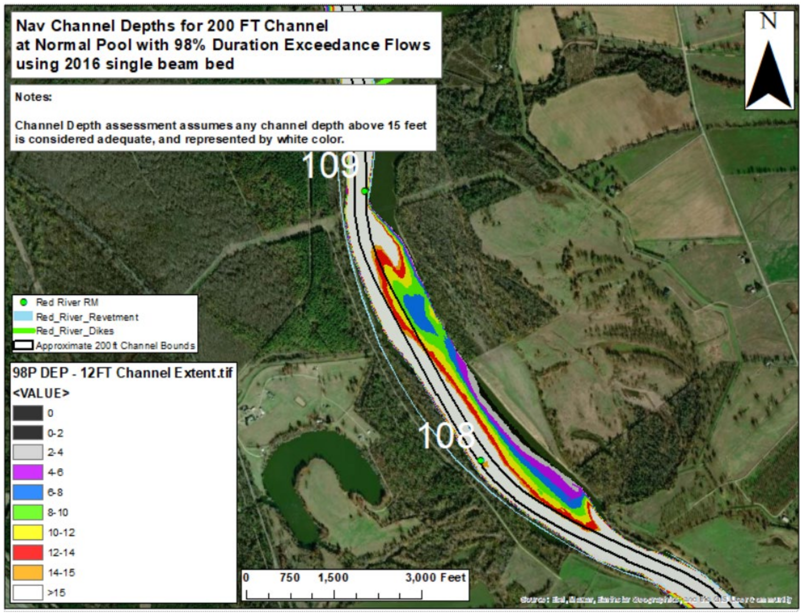



*Figure A-130. 2012 Multi-Beam Versus 2016 Single-Beam Cross-Section Comparison (RM 163.5)*

Pool 2 - RM 108

Figure A-131 uses HEC-RAS generated depth grids at normal pool project design conditions (water surface elevation 64 feet NAVD88) to illustrate the potential problems related to navigation channel depths near RM 108.



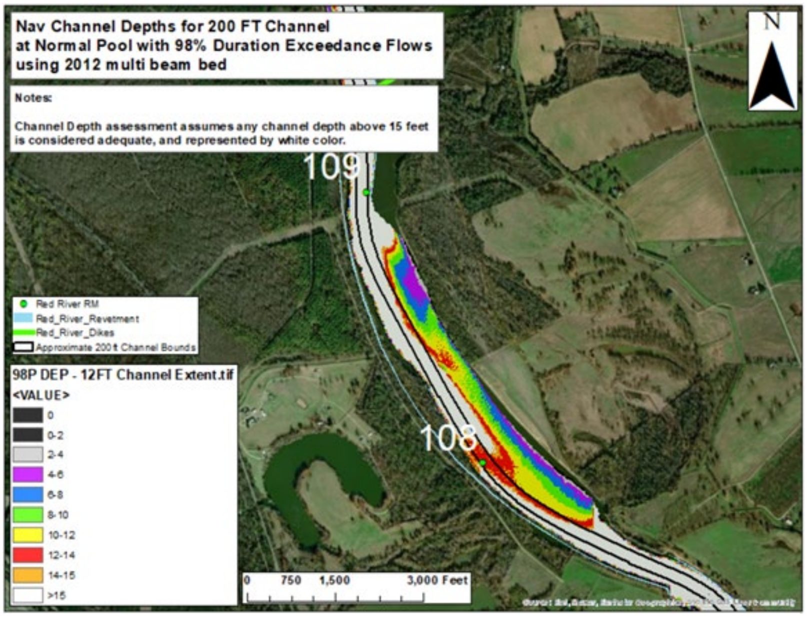




*Figure A-131. Pool 2 Normal Pool (WSEL 64 Feet) Channel Depths Maps near RM 108*

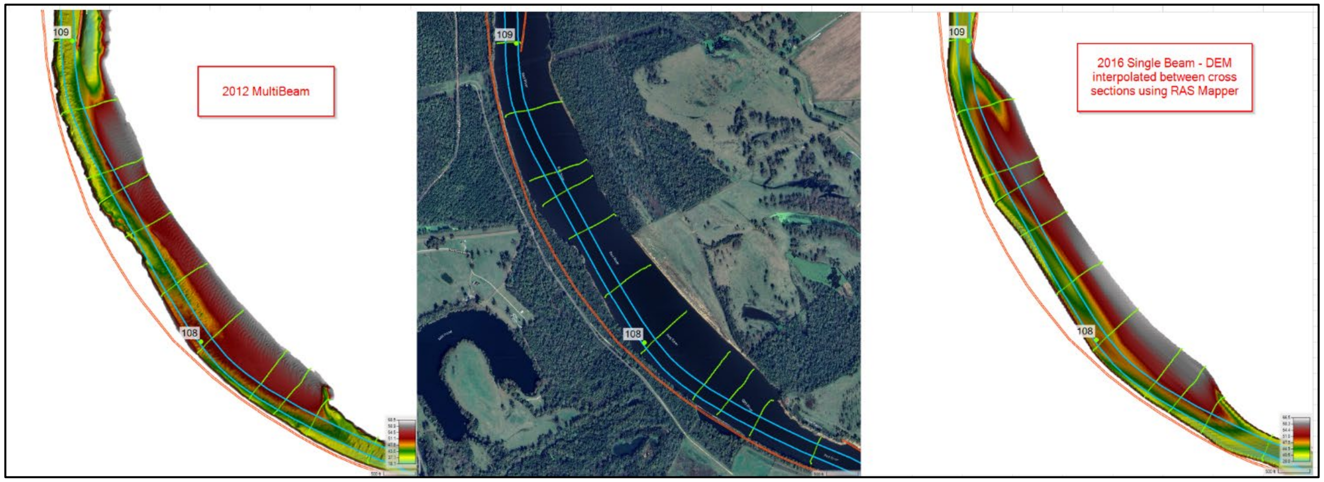



*Figure A-132. 2012 Multi-beam and 2016 Single-beam Near RMs 108-109*

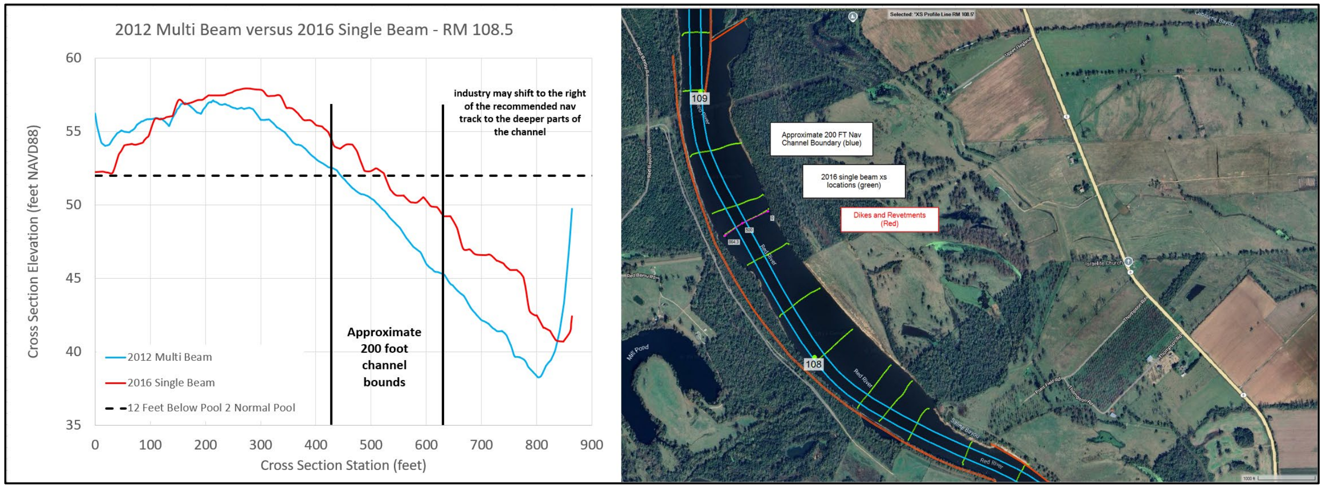



*Figure A-133. 2012 Multi-Beam Versus 2016 Single-Beam Cross-Section Comparison (RM 108)*

### 4.4.3 Additional Findings

This section provides information for additional reaches within the upper most portion of the waterway that have insufficient depths for a 12-FT channel as illustrated by hydraulic model results. However, these locations are upstream of the most upstream port that is the Caddo-Bossier Port at RM 212. The section of waterway upstream of RM 212 does not have a recommended navigation track centerline as does the rest of the waterway; therefore, the depth maps are of slightly different detail than the depth maps previously portrayed in this report. These areas are not of the same focus for this study as are the areas downstream of RM 212, the Caddo-Bossier Port.

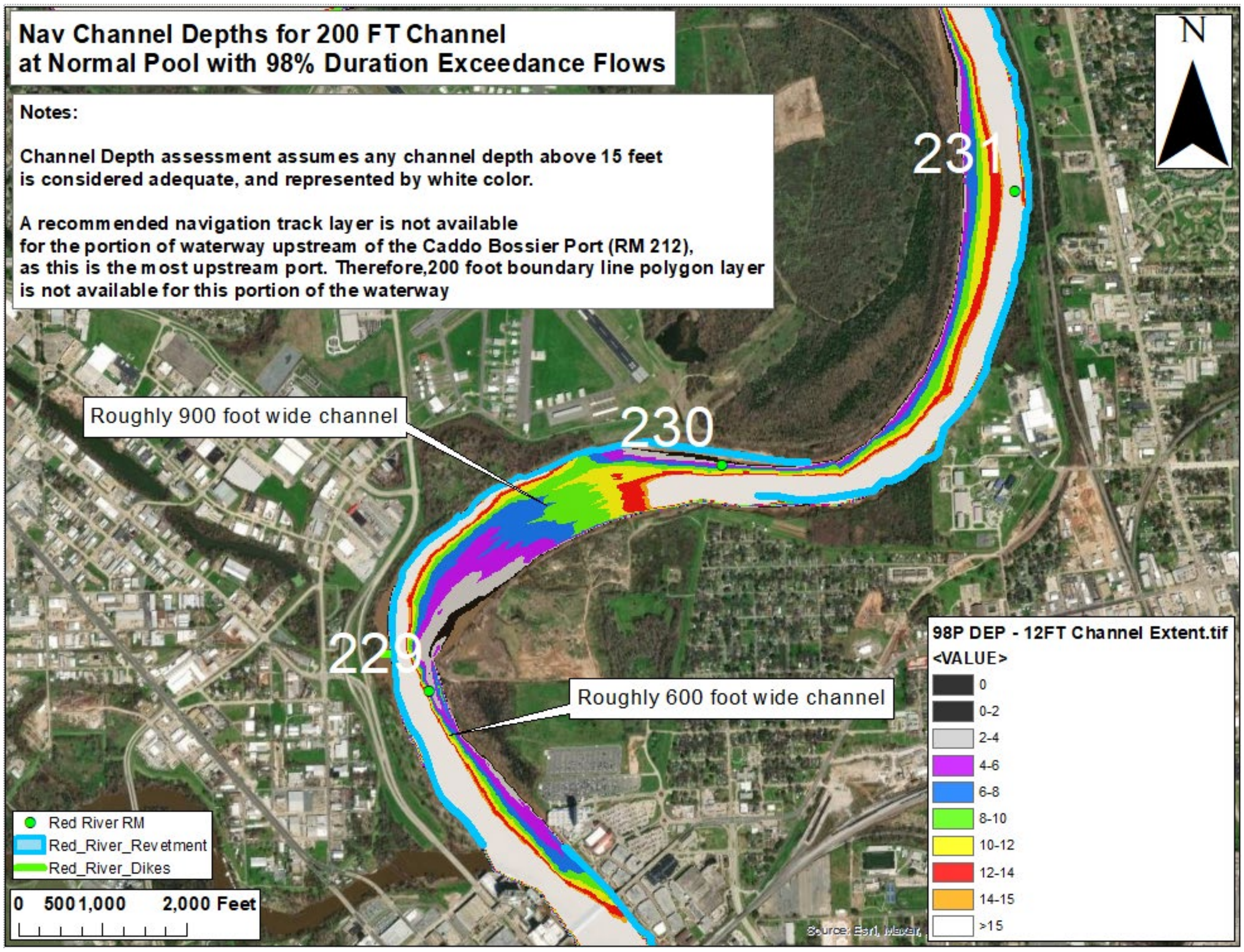



*Figure A-134. Pool 5 Normal Pool (WSEL 120 Feet) Channel Depths Maps Near RM 230*

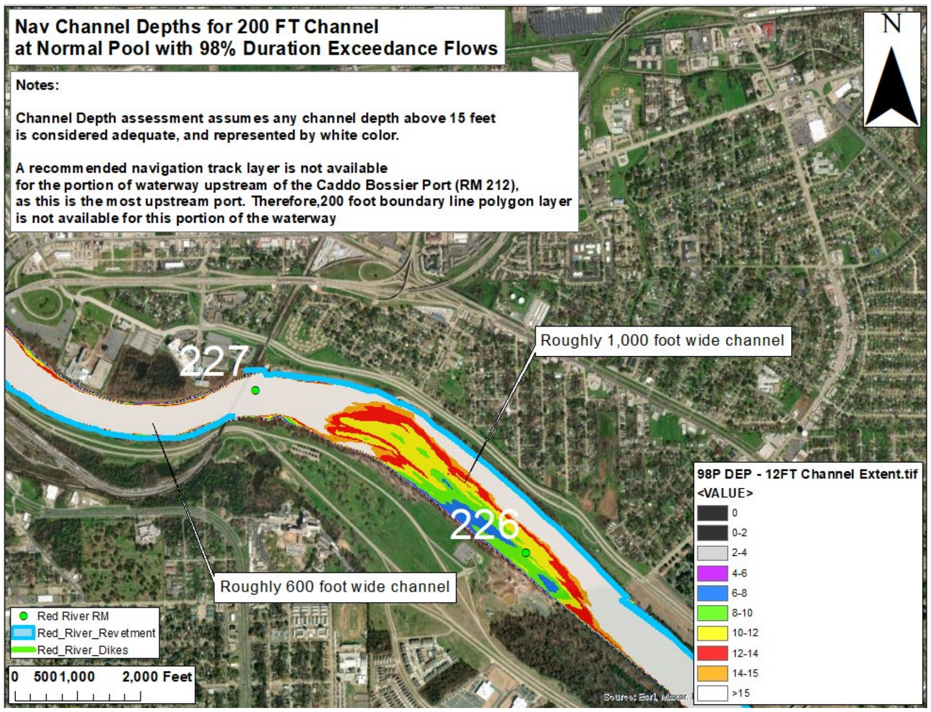



*Figure A-135. Pool 5 Normal Pool (WSEL 120 Feet) Channel Depths Maps near RM 226*

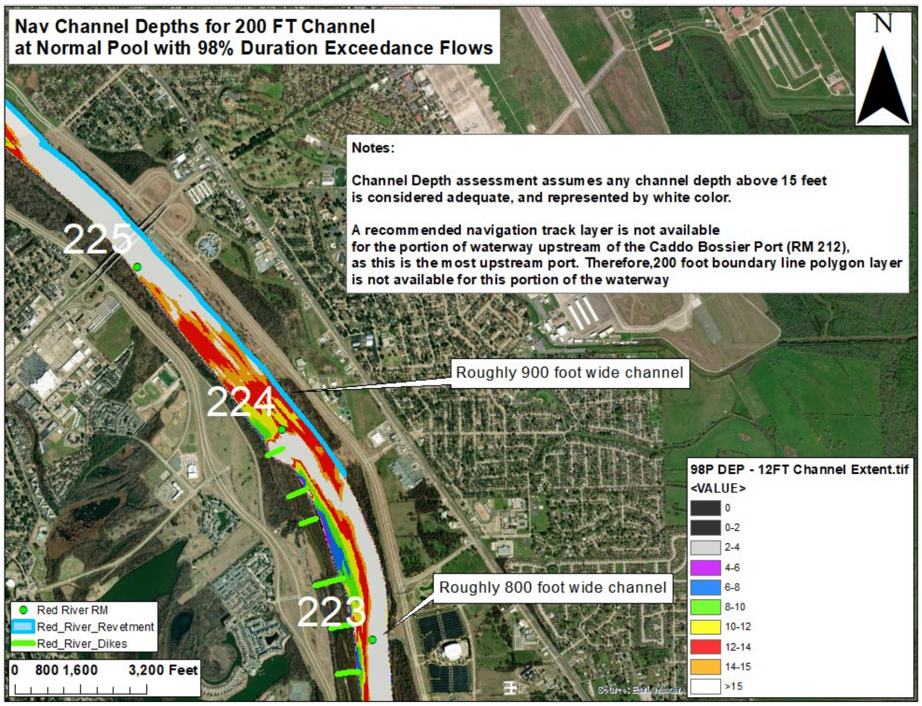



*Figure A-136. Pool 5 Normal Pool (WSEL 120 Feet) Channel Depths Maps near RM 224*

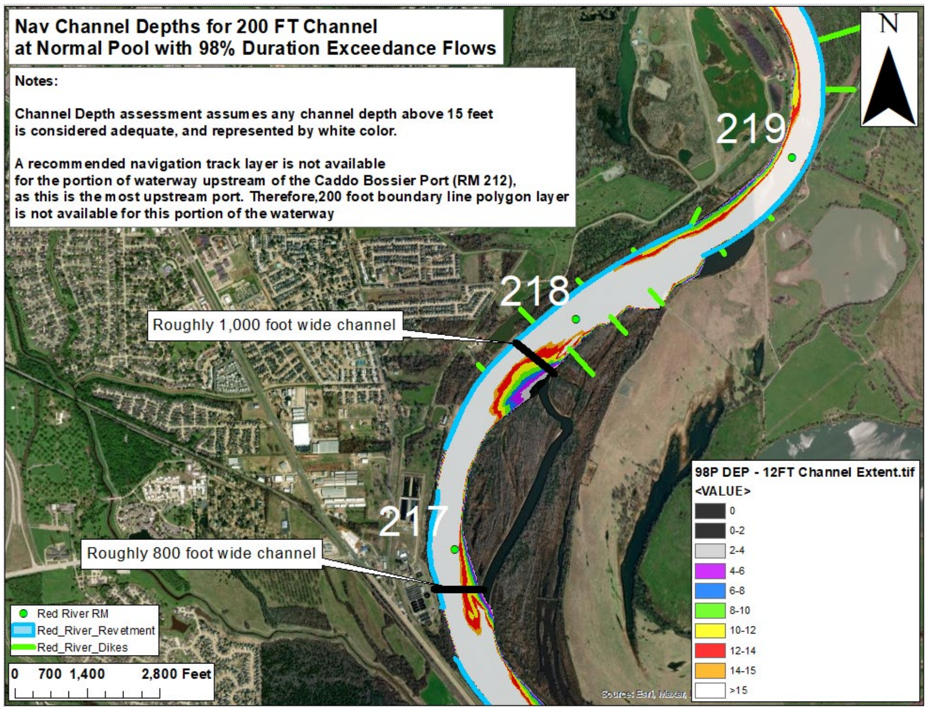



*Figure A-137. Pool 5 Normal Pool (WSEL 120 Feet) Channel Depths Maps near RM 218*

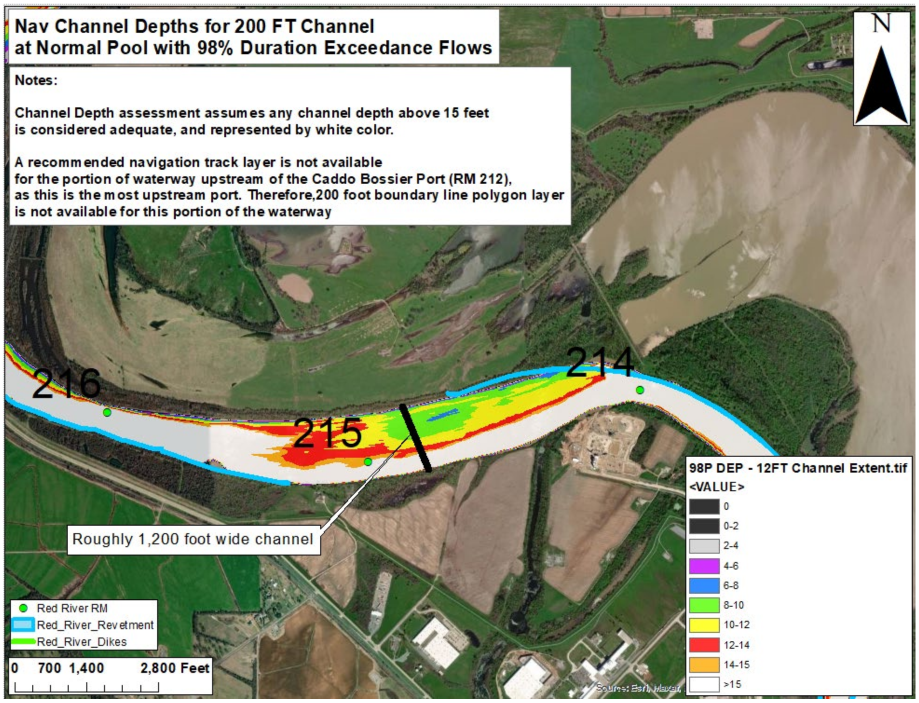



*Figure A-138. Pool 5 Normal Pool (WSEL 120 Feet) Channel Depths Maps near RM 215*

## 4.5 CONCLUSIONS AND CONSIDERATIONS

Although the known limitations have impacts to varying degrees, the combination of historical dredge records, existing channel surveys (2016 single-beam and 2012 multi-beam), existing hydraulic data and hydraulic models, original design documentation, and dike design experience within the district provide for a level of river training design analysis that is consistent with ECB 2023-9 (Policy Guidelines for Determining the 35% Design for River Training Structures) for this phase of the study.

Of the approximately 230 miles of navigable waterway from Old River to the Shreveport area, approximately 10 to 12 cumulative miles (approximately 5 percent) of waterway segments show to have potential problems providing a 12-FT channel under existing conditions assuming a low-flow project design condition as the 98 percent DEP, or a low flow that is exceeded 98 percent of the time. This statistical guideline is consistent with past Red River design documentation and consistent with Mississippi River practices for determine low water reference planes. The TSP, Alternative 3a, focuses on modifying or improving the existing river training structures to better induce self-scouring within the problem reaches of the river, given the assumption that some level of maintenance will be required. Overall, significant improvements to the existing river training structures is not assumed for the TSP, Alternative 3a, and no new dikes are intended to be added for this alternative; therefore, a meaningful impact to the current sediment regime is not expected as a result of the TSP. A lack of funding over the course of many years has played a role in the inability to most effectively maintain existing river training structures; therefore, a number of these structures have deteriorated well below their original design dimensions such that bringing them back to original design grade may prove to be a pivotal first step in providing satisfactory self-scouring and channel depths.. Many of the problem areas denoted in this report are located in the upper ends of navigation pools where water levels tend to be most critical during normal low-flow periods, and in the uncontrolled portion of the waterway downstream of L&D 1. With the exception of the denoted problem areas, much of the waterway varies in depth to 20 feet or greater.

The segment of river downstream from L&D 1 (RMs 34 to 40) is highly complex due to its uncontrollable nature while being a crucial portion of river as it is the entryway to the JBJ Waterway for Mississippi River traffic. This segment of river is often referred to as the Gauntlet because of the challenges experienced with operating, maintaining, and navigating this particular stretch of river. This area is situated within the Mississippi River floodplain and is therefore significantly influenced by Mississippi River backwater flows from the ORCC, in addition to the influence from Red River flows. Furthermore, the Ouachita River Basin flows have some influence on stages within this section of river as well. Due to the Mississippi River backwater influence and the fact that there is no downstream structure holding a pool, major fluctuations in water levels are experienced and velocities from Red River headwater flows may become suppressed exacerbating sediment deposition in this reach. The segment can experience prolonged periods of low water during the typical Mississippi River low water period from late summer to early winter. During high Mississippi River flows and normal or low Red River flows, there is a generally a flat water slope from Acme (RM 34) to L&D 1 (RM 44). During normal to low Mississippi River flows and typical high Red River flows, there would be some slope (0.3–0.5 feet/mile) between Acme (RM 34) and L&D 1 (RM 44). These different variables have proved to be a challenge for the existing waterway project such that considerable annual dredging efforts are required here each year. Additionally, the dikes and revetments systems in this segment of river have deteriorated over time such that portions of the system are functioning less efficiently than originally designed to function and, in some cases, completely deteriorated and unfunctional. Rehabbing these river training structures back to original design grades may prove to be a major step in achieving a more desirable balance in self-scouring and mechanical dredging efforts. The area below L&D 1 has been analyzed various times, as shown in historical studies and design memorandums. For example, the 1972 Design Memorandum No. 1 concluded that navigation for the 9-FT channel would be restricted 15 percent of the time without channel contraction (river training) through the entire reach (L&D 1 to Acme) and 9 percent of the time with maximum channel contraction. This stretch of the waterway is naturally narrower than upper reaches; therefore, channel contraction structures have limitations regarding lengths of the structures. The memorandum also concluded the comparative cost estimates indicated that the then present worth of the reduction in annual maintenance dredging costs over the project life through channel contraction would more than offset the cost of contraction. If, as the result of the lock and dam site selections, L&D 1 were located near the mouth of the Black River, the previously mentioned contraction would no longer be necessary.

The dredge records from 1989 to 1999 compared to 2012 to 2024 below L&D 1 show that 2012 to 2024 have considerably less dredging than 1989 to 1999. It was assumed that the channel was going through major changes during the earlier periods as the waterway features such as dikes and revetments, channel cutoffs, and the locks and dams had been built or were being built. It is believed that the dikes did function effectively by deepening the channel over time at which the channel began to stabilize about some equilibrium as shown by the 2012 and 2016 channel comparisons being similar. This comparative reduction in dredging has occurred even with the knowledge that the adjacent dikes and revetments in the reach have deteriorated below original design grades or become completely unfunctional in some areas. The time frame in which the deterioration has occurred is not precisely known. As noted in the report, maintenance dollars have not been available to adequately maintain the structures over the course of years. Although the dikes have deteriorated, it is possible that the resulting dike fields have somewhat remained in place and thus still allowing for some degree of channel contraction. Historical design documentation and studies along with observed data and experience show that the reach below L&D 1 will likely always require some degree of dredging (for either the 9-FT or 12-FT channel) as contraction of the channel is limited due to a naturally narrower river than upstream reaches, and most importantly the influencing variable of the Mississippi River backwater often suppressing headwater driven velocities and often causing wide fluctuations in stages, along with prolonged periods of low flows.

Notably, while dikes and revetments along with dredging downstream of L&D 1 may achieve some satisfactory navigable depth during low water periods while helping to reduce dredging efforts, the draft requirements over the L&D 1 lower miter gate sill may still prove to be a constraint. In essence, traffic may be able to navigate a 12-FT channel in the river but if the water level at the L&D 1 lower miter gate sill is not providing the 1.5x draft requirement, then commercial traffic would not be able to pass the sill with a 12-FT channel type of cargo. The lower miter gate sill elevation is documented as -9 feet NAVD88; therefore, a water surface elevation of 9 feet NAVD88 would be required at L&D 1 tailwater to achieve the 1.5x draft requirement for a 12-FT channel (1.5 x 12 feet = 18 feet of depth over the sill). For example, at a water surface elevation of 7 feet downstream of L&D 1, the river channel may achieve 12-FT with the improved or rehabbed dike systems; however, the lock sill would only be providing 16-FT of draft over the sill, 2 feet shy of the 1.5x draft requirement.

### 4.5.1 TSP Hydraulic Modeling

Hydraulic modeling analysis of the TSP, Alternative 3a, is ongoing. The analysis is focused on the problem reaches below L&D 1 (the gauntlet) and near RM 191 Westdale. Discussion and results are to be added once complete. Utilizing existing channel bathymetry (2012 multi-beam), the River Stabilization section will develop a terrain dataset that includes the TSP Alternative 3a dike improvements representing with-project conditions. In addition, the existing as-built dike conditions (assuming deteriorated dikes have been built back to original design dimensions) will also be developed representing without-project conditions. These terrain datasets will be incorporated into a 2D HEC-RAS model to simulate a range of flows to assess the incremental changes in flow patterns and velocities to support the selection of Alternative 3a.

When performing dike modeling, a dike design flow is often necessary to assess the performance of the dikes. In channel design and restoration practices, there is a concept called channel-forming discharge. ERDC/CHL CHETN-VII-5 defines channel-forming discharge as a theoretical discharge that if maintained indefinitely would produce the same channel geometry as the natural long-term hydrographs. Three deterministic discharges are often used to characterize the channel-forming discharge such as 1) bankfull discharge, which is the maximum discharge that the channel can convey without flowing onto its floodplain, 2) a specified recurrence interval typically between the 99 percent AEP (1 year) and 50 percent AEP (2 year), or 3) the effective discharge defined as the discharge that transports the largest fraction of the average annual bed-material load. Utilizing hydrologic POR, existing flow frequency analyses, HEC-RAS, and historical USGS sediment data, each of the three types of channel-forming discharges will be estimated at the Shreveport and Alexandria gages, which provide the most robust records of data and allow for appropriate characterization of the waterway as Shreveport is situated at the upper end and Alexandria is situated toward the lower end. Additionally, estimated bankfull calculations can be made using the hydraulic model within the problem reaches previously identified within this report.

# 1.5x Depth Draft Requirement

Per EM 1110-2-1604, the ideal depth of water at lock and dam miter gate sills is 1.5 times (1.5x) the authorized navigable draft for vessels to safely enter and exit the lock chambers. For a 12-FT channel, 18 feet of water depth over the sills is ideal. For the existing 9-FT channel, 13 feet of depth over the sills is ideal. Currently, L&Ds 1 and 2 do not have 18 feet of depth over the lower approach miter gate sills based upon normal pool operations. L&Ds 3, 4, and 5 have an approximate minimum of 18 feet over the lower approach miter gate sills at all times based upon existing normal pool operations. All five locks and dams have well over 18 feet of depth at their respective upper approach miter gate sills based on existing normal pool operations.

Based on lower miter gate sill elevations and daily water levels (period of records), stage duration exceedance plots were generated using HEC-DSS to quantify the percentage of time that water levels are exceeded. Results are presented and illustrate that L&D 1 lower miter gate sill achieves 18 feet of depth (or 1.5x the 12-FT draft) approximately 89 percent of the time and L&D 2 lower miter gate sill has 18 feet of depth approximately 42 percent of the time. Based on normal pool operations and miter gate sill elevations, L&Ds 3, 4, and 5 have a minimum of 18 feet of depth over their respective sills, meaning the 1.5x depth draft requirement is achieved 100 percent of the time. The PDT have discussed the possibility of a waiver to allow for year-round navigation at 12-FT when 1.5x depth is not achieved at L&Ds 1 and 2. Otherwise, draft restrictions will be required during periods of insufficient depths over the miter gate sills.

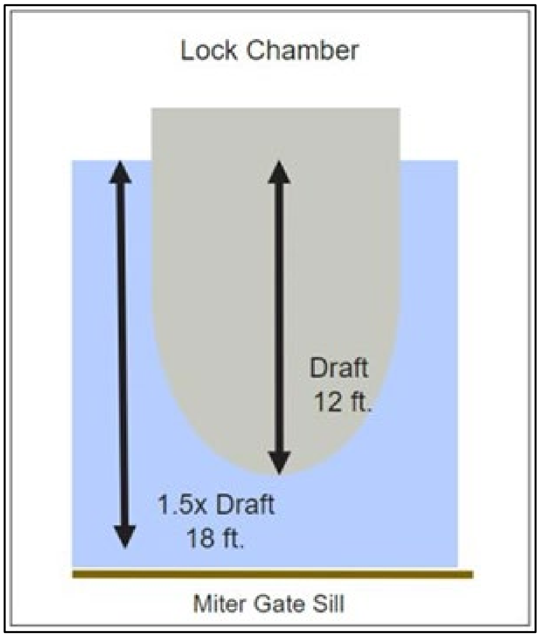



*Figure A-139. Lock Chamber Draft Schematic*

| L&D | SILL | NORMAL POOL EL. (feet, NAVD88) | SILL EL. (feet, NAVD88) | DEPTH OF WATER AT SILL (feet) | Percentage (%) of time 1.5x draft requirement is satisfied on an annual basis** |
|---|---|---|---|---|---|
| 1 | LOWER | 4.1* | -9.6 | 13.7 | 89 |
| | UPPER | 40.1 | 18.1 | 22.0 | 100 |
| 2 | LOWER | 40.1 | 25.9 | 14.2 | 42 |
| | UPPER | 64.1 | 40.6 | 23.5 | 100 |
| 3 | LOWER | 64.0 | 46.1 | 17.9 | 100 |
| | UPPER | 95.0 | 70.1 | 24.9 | 100 |
| 4 | LOWER | 94.8 | 77.0 | 17.9 | 100 |
| | UPPER | 119.8 | 95.0 | 24.9 | 100 |
| 5 | LOWER | 119.8 | 101.8 | 18.0 | 100 |
| | UPPER | 144.8 | 119.8 | 25.0 | 100 |
| *Lower pool cannot be controlled at Boggs L&D. | | | | | |
| **Depth of Water at Sill must be 18 feet or greater to satisfy the 1.5x Draft Requirement for 12 Foot Channel Depths. | | | | | |

*Figure A-140. Normal Pool Depths Over the Miter Gate Sills*

## 5.1 STAGE DURATION EXCEEDANCE

### 5.1.1 L&D 1

Original design documentation and water control manuals state that the L&D 1 lower approach miter sill elevation is -9 feet NAVD88. Recent survey shows a sill elevation of -9.6 feet NAVD88, so -9.6 feet is used for calculations. The Red River is uncontrolled below L&D 1, meaning there is no downstream structure to control pool levels. The lower end of the waterway is heavily influenced by Mississippi River flows through the ORCC. The L&D 1 water control manual notes that the minimum tailwater elevation is 4 feet, therefore providing 13 feet of depth over the sill and satisfying the existing conditions 1.5x ideal draft recommendation. Assuming that the lower sill elevation is -9 feet, then a water surface elevation of 9 feet would be required to provide 18 feet of depth over the sill for a 12-FT vessel. Based on the lock and dam daily tailwater records from 1987–2024, a water surface elevation of 9 feet at the tailwater is available approximately 89 percent of the time on an annual basis using the HEC-DSS duration exceedance analysis tool. On a quarterly basis, 18 feet is available 95–97 percent of the time between January and March and April and June, 78 percent of the time between July and September, and 70 percent of the time between October and December.

River Operations personnel have suggested that the JBJ Waterway does not experience the same type of seasonality in waterway traffic as the Mississippi River. Rather, the JBJ Waterway has fairly steady traffic throughout the year relative to its annual cumulative traffic. Therefore, an annual statistical analysis may be sufficient.

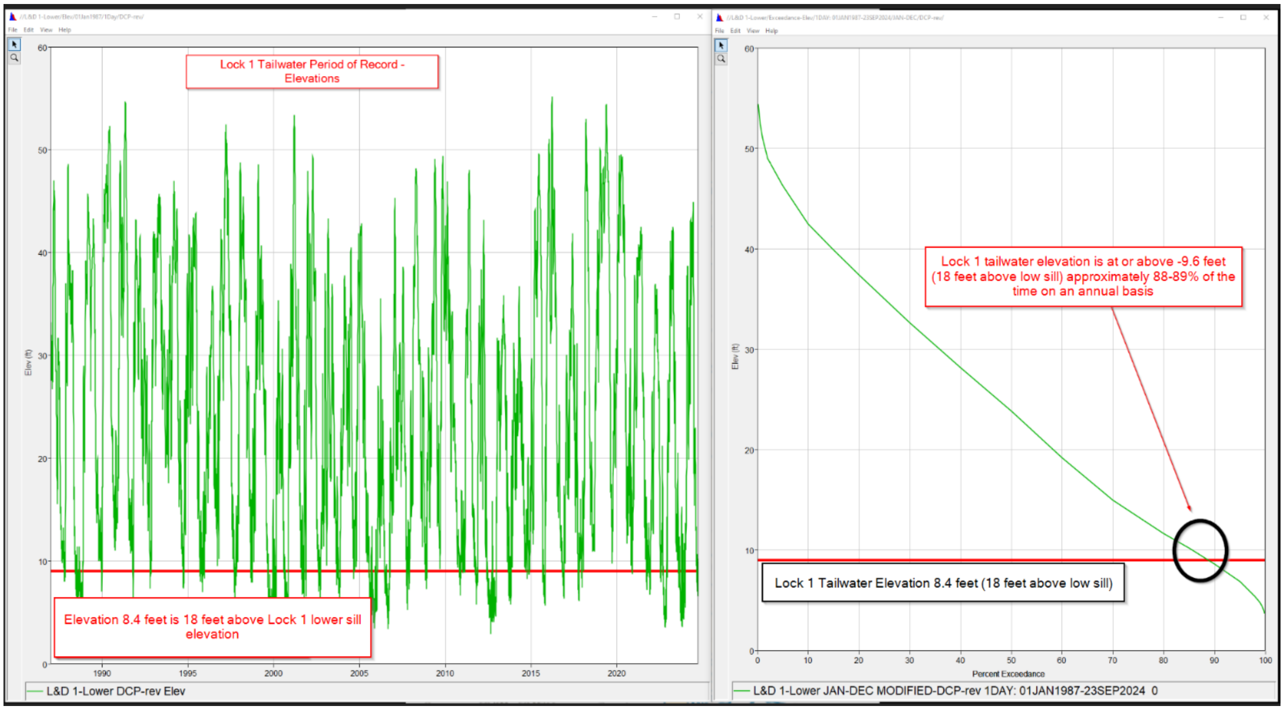



*Figure A-141. L&D 1 Tailwater Duration Exceedance Analysis on an Annual Basis*

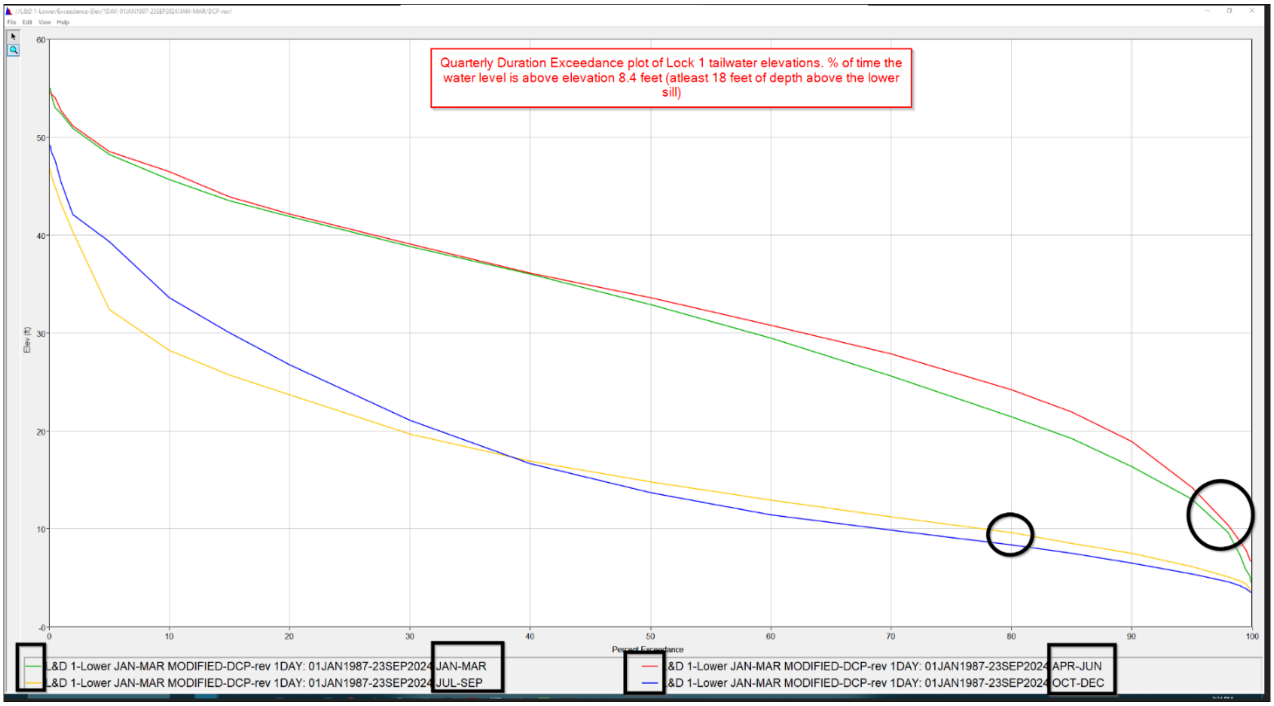



*Figure A-142. L&D 1 Tailwater Duration Exceedance Analysis on a Quarterly Basis*

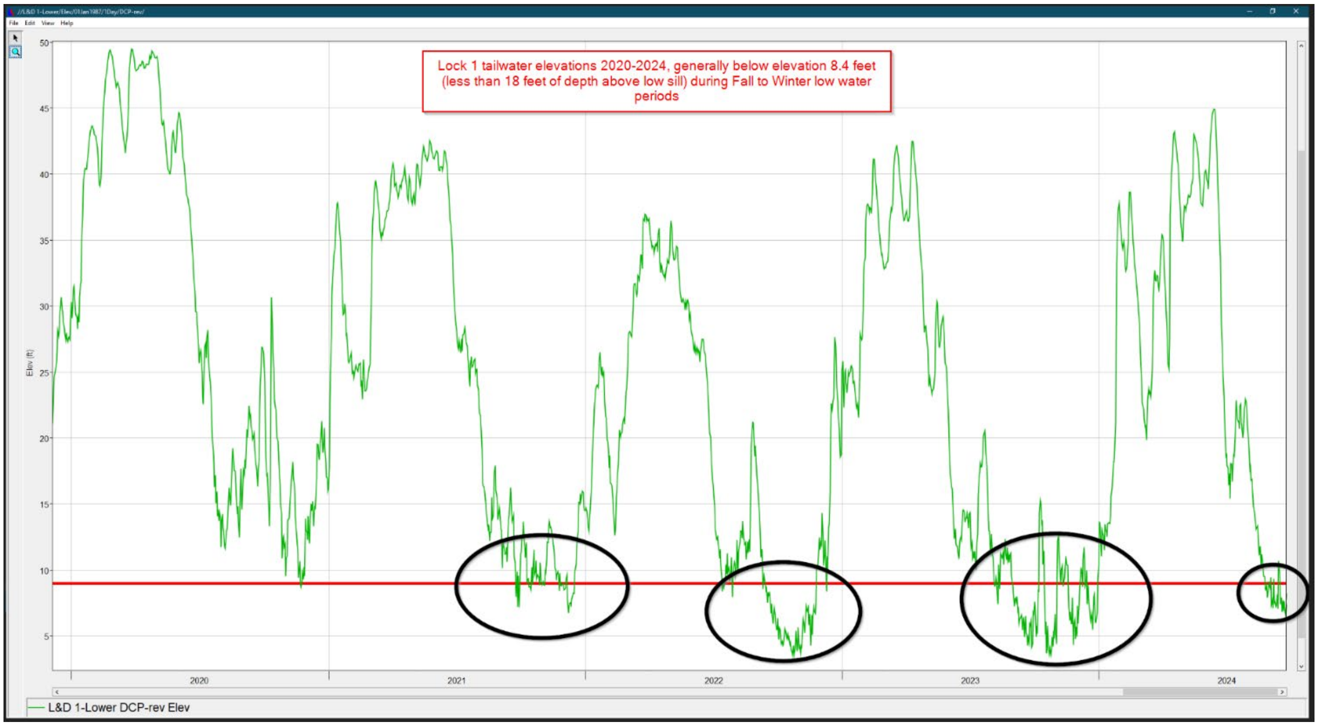



*Figure A-143. L&D 1 Tailwater Hydrograph (2020–2024)*

### 5.1.2 L&D 2

Original design documentation and water control manuals state that the L&D 2 lower approach miter sill elevation is 25.8 feet NAVD88. A survey may reveal slightly different elevations at the sill. The lower pool is controlled by L&D 1 which currently holds a normal pool elevation of 40 feet NAVD88. At average- to low-flow periods, Pool 1 is completely flat up to L&D 2. A lower sill elevation of 25.8 feet requires a water surface elevation of 43.8 feet to achieve 18 feet of water depth over the sill for a 12-FT authorized channel. Based on the lock and dam daily tailwater records from 1987–2024, a water surface elevation of 43.8 feet at the tailwater is available approximately 42 percent of the time on an annual basis using the HEC-DSS duration exceedance analysis tool. On a quarterly basis, 18 feet is available 44 percent of the time between January and March, 68 percent of the time between April and June, 41 percent of the time between July and September, and 13 percent of the time between October and December.

River Operations personnel have suggested that the JBJ Waterway does not experience the same type of seasonality in waterway traffic as the Mississippi River. Rather, the JBJ Waterway has fairly steady traffic throughout the year relative to its annual cumulative traffic. Therefore, an annual statistical analysis may be sufficient.

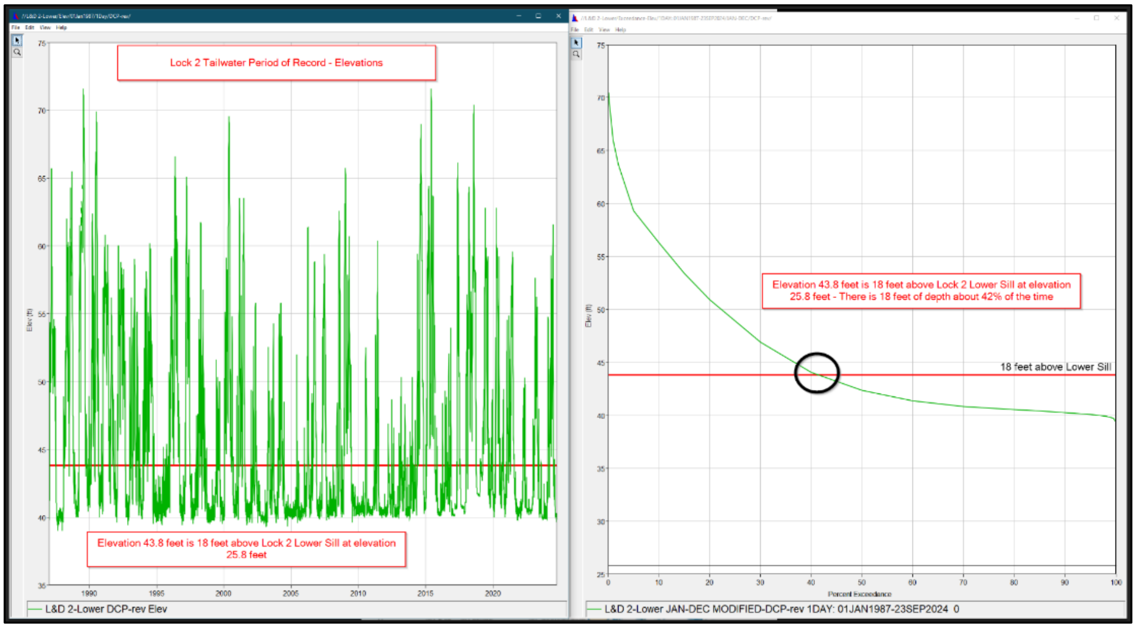



*Figure A-144. L&D 2 Tailwater Duration Exceedance Analysis on an Annual Basis*

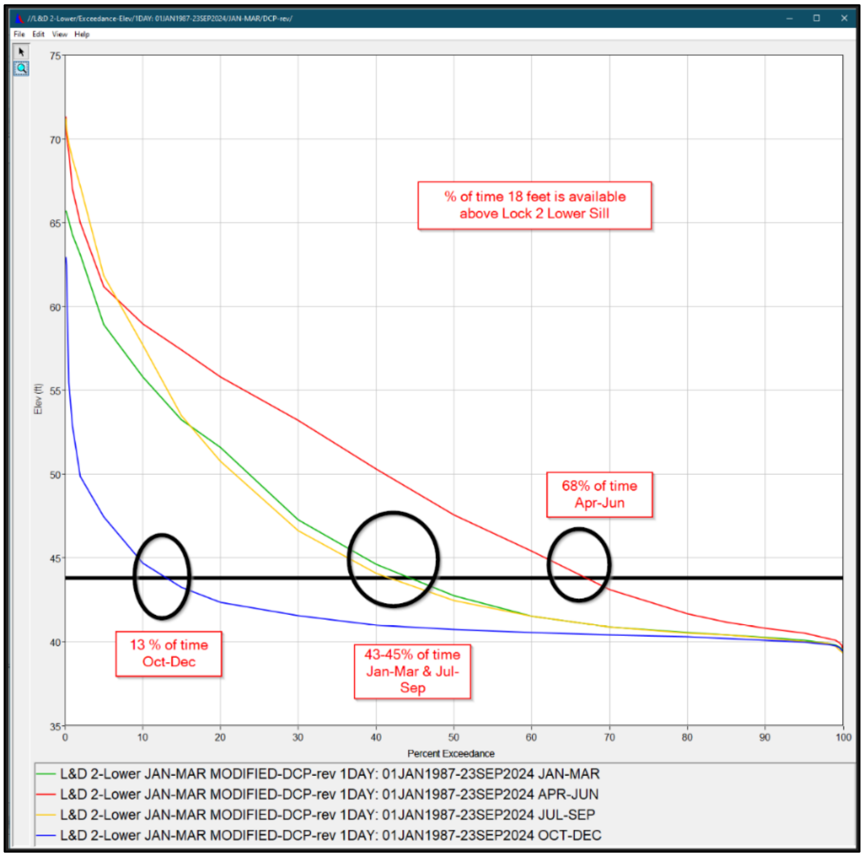



*Figure A-145. L&D 2 Tailwater Duration Exceedance Analysis on a Quarterly Basis*


*Figure A-146. L&D 2 Tailwater Hydrograph (2020–2024)*

### 5.1.3 L&Ds 3, 4, and 5

Based upon existing designs and normal pool operations, L&D 3, 4, and 5 always have at least 18 feet at the lower miter gate sills, and well above 18 feet at the upper miter gate sills.

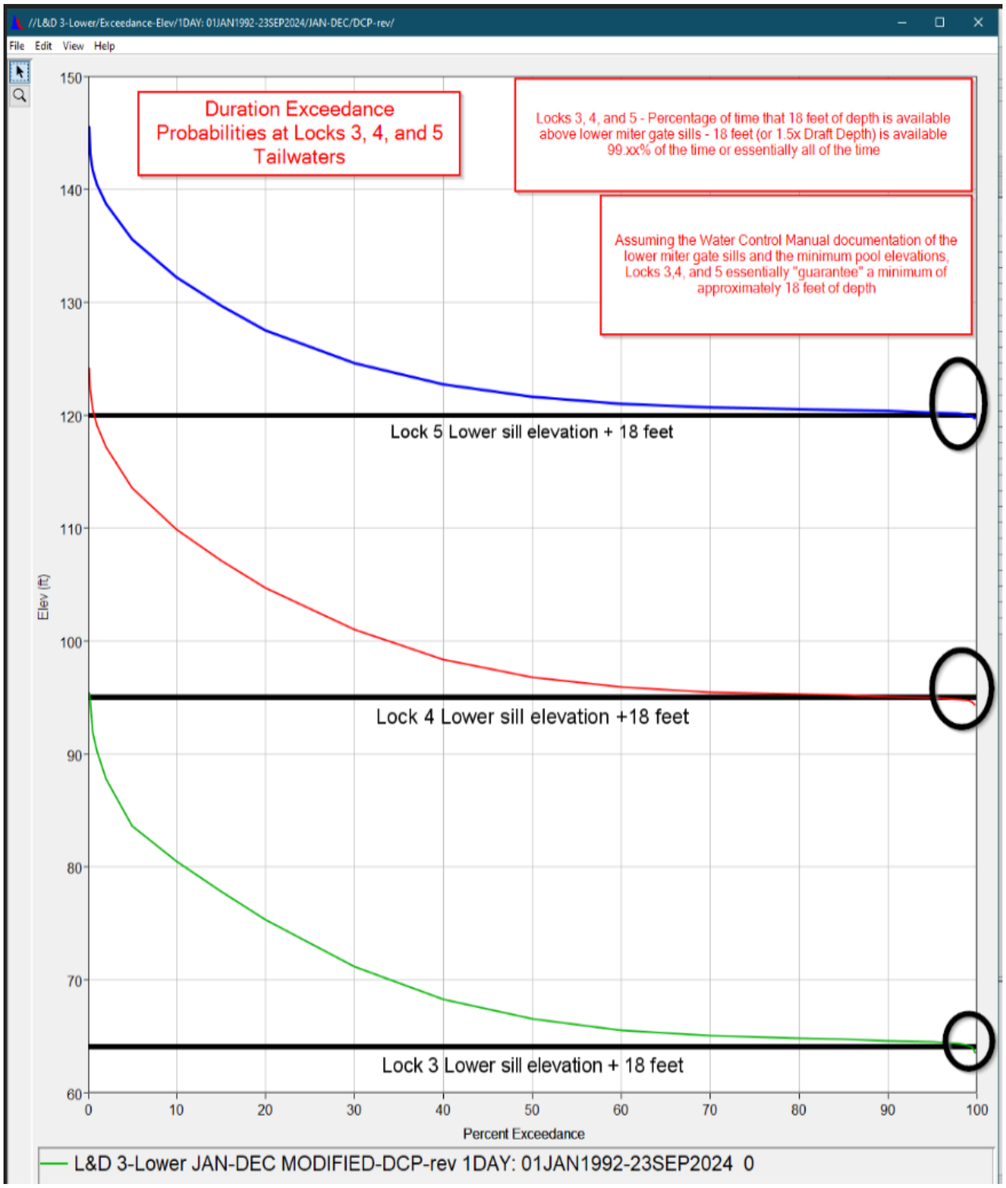



*Figure A-147. L&Ds 3, 4, and 5 Tailwater Duration Exceedance Analysis on an Annual Basis*

## 5.2 ERDC PHYSICAL MODELING

The ERDC CHL will assist the USACE Vicksburg District in evaluating the operational performance of L&D 2 under modified channel and sill depths. This study will assess the hydraulic performance related sill clearance and filling and emptying (F/E) system's performance for barges drafted to 12-FT—an increase from the current 9-FT draft. This increased depth allows barges to carry greater tonnage, improving overall transportation efficiency. Deepening the channel necessitates corresponding modifications to navigation locks. As part of this effort, the lock gate's sill depth must be evaluated to ensure adequate under keel clearance for barges drafting up to 12 feet. Additionally, Hawser forces—

hydraulic forces acting on barges within the lock chamber during F/E operation—must be measured for these deeper-draft barges.

CHL will construct a 1:25 scale physical model of the Overton Lock F/E system and conduct testing on the sill depth clearance for barges with a 12-FT draft navigating into and out of the lock chamber. Testing will also be conducted with the barges inside the lock chamber during F/E operations to ensure the lock can be operated safely with the deeper drafted barges. Validation of the physical model will be conducted by comparing the F/E curves of the prototype with the scale model. The modeling approach for the Overton Lock study follows established USACE protocols for lock F/E system evaluations, which have been in practice since the 1960s. Model construction utilizes standard materials that have been reliably employed in hydraulic model studies for decades. The model is scaled using Froude similitude, the internationally accepted standard for modeling open channel flows in large hydraulic structures. A geometric scale of 1:25 has been selected, consistent with the standard practice at ERDC. This scale has been successfully applied in the design and evaluation of numerous USACE navigation locks and is appropriate for producing the data required for the Overton Lock model investigation.

The estimated start date for this study is 1 February 2026. The physical model study from beginning of model design to the completion of the draft technical report is 49 weeks. For additional details regarding tasks, schedule, and cost, please refer to the Statement of Work. Results and discussion will be included in future versions of this report.

# SECTION 6
# MEASURES AND ALTERNATIVES

## 6.1 OVERVIEW

### 6.1.1 Geographic Regions

The study area was divided into two geographic regions based on the benefit-to-cost ratio calculated from the work required to achieve a 12-FT channel. The first region extends through Pool 2, from L&D 3 (RM 116) down to the ORCC. The second region begins in the Shreveport, Louisiana, area (RM 236) and extends down to the ORCC (RM 0). These pools and regions can be seen in Figure A-148 below. This second region encompasses the entirety of the maintained navigation channel, while the more limited first region includes only Pools 1 and, 2 and the Gauntlet. This division was selected to focus on locations where the greatest benefits could be realized at the lowest cost. Most of these benefits are expected to be achieved from improving navigation up to Alexandria, Louisiana, which led to the cutoff at L&D 3 for the first region.

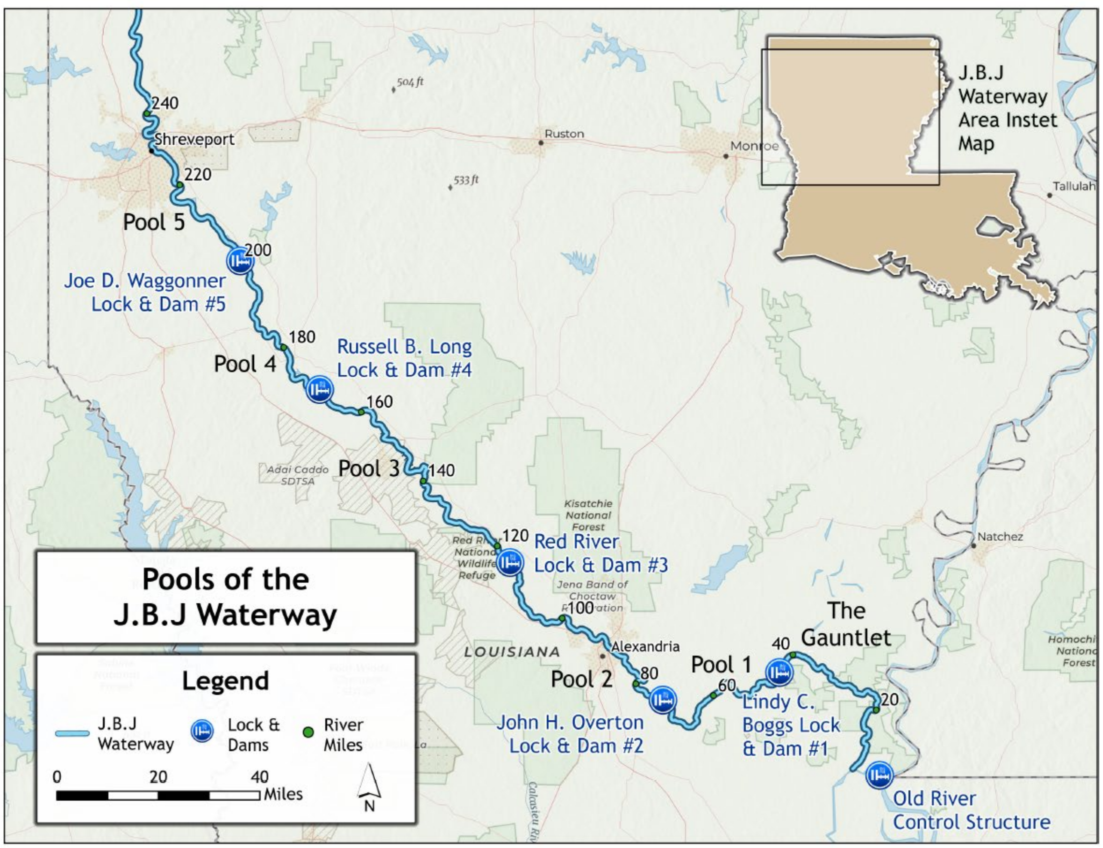



*Figure A-148. Map of JBJ Waterway Regions*

## 6.2 ASSUMPTIONS

Major assumptions made for this report include the following:

- Existing surveys and aerial imagery are acceptable for reconnaissance-level (35 percent) dike design as the dynamic nature of the river environment would change site conditions before construction-level design is completed.
- It is acceptable to base estimated quantities on trends, past projects, or engineering judgement informed by available data.
- Existing utilities, if any, should be avoided. Detailed survey data were not available to determine the location of utilities. Relocations of sewer and gas lines are prohibitively expensive and shall be minimized or avoided.
- Demolition of existing structures is typically not justified for the anticipated benefits and should be avoided.
- For dike maintenance projects, the hydraulic model using older or existing survey data that were completed as part of the original design is considered sufficient for dike layout and design.
- Geotechnical data are typically not required when local standard dike designs are used (standard dig ins and geometry). When bank stabilization is part of the dike project, geotechnical data may be required but are not required for the 35 percent level of design.
- 100 percent of benefits will be achieved from the Draft Deviation that is necessary for all measures considered (more information on the Draft Deviation is available in Main Report Sections 2.6.3 and 3.6.2 and in EM 1110-2-1604.
- Annual dredging will still be required at the approaches of all locks as the shoaling rate at these locations will remain the same regardless of implementation, rehabilitation, and improvement of dikes.
    - Future maintenance dredging volumes and frequencies were based on the original dredge volumes computed from the 2003 hydrographic survey. Based on previous shoaling locations and frequencies, the shoaling rate is 50 percent at areas where no additional river training structures are being placed and 10 percent at areas with new or modified structures.
- All stone placements will be achieved via barge except in shallow locations where land-based equipment is required.
- All necessary work to bring the channel to 100 percent of the 9-FT depth has been completed. This includes the Joffrion and Westdale Revetments, which, as of fiscal year 2025, have not received necessary O&M funding.
    - Costs and analysis have been performed under the assumption that repairs and construction necessary for ensuring a 9-FT channel will be completed prior to this project. Costs for rehabilitation of these sites have been

mentioned within this report to ensure adequate funding is captured; however, they were not included in the total cost for the 12-FT channel.

- If work is not completed, the following impacts would occur:
  - Significant increase in O&M dredging and dredging costs as revetments further degrade.
  - Inability to provide for and maintain a 12-FT channel year-round.

## 6.3 MEASURES CONSIDERED

During the initial planning phase, potential alternatives were identified with input from the project team and industry stakeholders through a charrette. The team evaluated and refined the proposed measures, resulting in a shortlist of preliminary alternatives. These alternatives were evaluated based on cost, impacts to navigation, cultural considerations, and environmental impacts. Measures that were deemed infeasible based on this preliminary analysis were excluded from further evaluation. A summary table of the proposed alternatives is shown in Table A-19.

*Table A-19. Array of Selected Alternatives*

| Region | Alternative | Description | Construction or Maintenance | Number of Dikes Improved or Constructed | O&M Channel Dredging Return Period (Years) | O&M Channel Dredging Per Dredging Period (Days) | L&D Dredging Return Period (Years) |
|---|---|---|---|---|---|---|---|
| - | 1 | No action | - | - | - | - | - |
| Through Pool 2 (RM 116 to RM 0) | 2 | Draft restrictions; deviation; dredging to 12-FT | - | - | 1 | 20 | 1 |
| | 2a | Draft restrictions; deviation; improvement of dikes to 12-FT | M | 6 | 2 | 20 | 1 |
| | 2b | Draft restrictions; deviation; construction of new dikes to 12-FT | C | 12 | 25 | 20 | 1 |
| | 2c | Draft restrictions; deviation; construction of high-priority dikes and improvement of dikes to 12-FT | C & M | 11 | 25 | 20 | 1 |
| | 2ab | Draft restrictions; deviation; construction of new dikes and improvement of dikes to 12-FT | C & M | 18 | 0 | 0 | 1 |

| Region | Alternative | Description | Construction or Maintenance | Number of Dikes Improved or Constructed | O&M Channel Dredging Return Period (Years) | O&M Channel Dredging Per Dredging Period (Days) | L&D Dredging Return Period (Years) |
|---|---|---|---|---|---|---|---|
| **Through Shreveport, LA (RM 236 to RM 0)** | 3 | Draft restrictions; deviation; dredging to 12-FT | - | - | 1 | 28 | 1 |
| | 3a | Draft restrictions; deviation; improvement of dikes to 12-FT | M | 8 | 2 | 28 | 1 |
| | 3b | Draft restrictions; deviation; construction of dikes to 12-FT | C | 20 | 25 | 28 | 1 |
| | 3c | Draft restrictions; deviation; construction of high-priority dikes and improvement of dikes to 12-FT | C & M | 15 | 25 | 28 | 1 |
| | 3ab | Draft restrictions; deviation; construction of new dikes and improvement of dikes to 12-FT | C & M | 28 | 0 | 0 | 1 |

### 6.3.1 Measures Not Carried Forward

Several additional alternatives were considered that were eliminated from further analysis for various reasons. Alternatives involving the modification or reconstruction of locks and dams were screened out due to concerns regarding construction costs and extended construction-related closures. The rough order-of-magnitude (ROM) cost for lock reconstruction in 2025 was estimated to be between $340 million to $740 million. With only one lock chamber at each of the five structures, any modification or reconstruction would necessitate a complete river shutdown with no temporary bypass options. Historically, extended lock closures have negatively impacted industry, and similar disruptions in the future are expected to result in the loss of industry users and negative economic consequences.

Another possible alternative that was rejected was the raising of the pool at L&D 1. This alternative was rejected due to significant cost and safety concerns. To achieve the necessary pool elevation for a 12-FT channel, extensive structural and hydraulic analyses would have been required. Structural analysis would have been necessary to determine if the locks and dams could support the elevated pool levels. Additionally, a new water control plan would have been required that dictates the requisite amount of flow to be released to maintain the new pool. This alternative would also have necessitated the raising of all levees and dikes along the Red River, resulting in substantial construction and material costs. Additionally, construction dredging down to 12-FT throughout the entire length of the JBJ Waterway was considered. However, this alternative was eliminated due the high cost of dredging such a long stretch and the cost of annual dredging that would be required to maintain the depth.

### 6.3.2 Explanation of Measures Selected

The eleven alternatives chosen for further analysis are composed of combinations of five distinct measures. Further information on each measure is provided in the subsequent sections. Each of these measures also require the implementation of a Draft Deviation that would limit barge capacity when there is less than 12-FT of draft through the river. Notification of channel restrictions will be sent out by the Coast Guard through an existing system (more information on the Draft Deviation is available in Main Report Sections 2.6.3 and 3.6.2, and in EM 1110-2-1604). Additionally, yearly maintenance dredging at the lower lock approaches that is already being conducted will still be required.

#### 6.3.2.1 Dredging to 12-FT

This measure involves dredging at specific locations that do not currently support a 12-FT channel. This includes Alternatives 2 and 3. For Alternative 3 in Region 1, dredging locations are listed in Table A-20 above the blue line. Alternative 3 includes all locations listed in the table. The 11 sites for Alternative 2 will require approximately twenty days of dredging annually to maintain 12-FT of channel depth. The additional seven sites from Alternative 3 bring the total days of dredging per year to 28. When used as the sole alternative, dredging must be repeated annually to remove accumulations of sediment at areas with shoaling.

*Table A-20. Dredging Locations to Achieve 12-FT*

| Name | River Mile | Pool | Priority | Est. Area (sy) | Est. Depth (FT) | Excavated Material (cy) (w/ 20% contingency) |
|---|---|---|---|---|---|---|
| Lower Lorran Dikes | 35.0 | G | H | 12,773 | 3.0 | 15,400 |
| Lorran Dikes | 36.5 | G | H | 63,146 | 4.0 | 101,100 |
| Joffrion Dikes | 37.2 | G | H | 10,314 | 4.0 | 16,600 |
| Larto Revetment (A) | 40.0 | G | H | 247,665 | 4.0 | 396,300 |
| Larto Revetment (B) | 41.0 | G | H | 37,309 | 5.0 | 74,700 |
| Lac Amelia Revetment | 41.5 | G | H | 47,060 | 4.0 | 75,300 |
| Hadden Fort Revetment | 52.3 | 1 | M | 20,699 | 2.0 | 16,600 |
| Barbin Dikes | 53.2 | 1 | M | 17,051 | 2.0 | 13,700 |
| Dupre Revetment | 60.7 | 1 | M | 19,491 | 2.0 | 15,600 |
| Bringol Revetment | 64.5 | 1 | M | 22,438 | 2.0 | 18,000 |
| Pointfield Revetment | 108.7 | 2 | L | 25,202 | 2.0 | 20,200 |
| Socot Revetment | 154.1 | 3 | M | 67,577 | 3.0 | 81,100 |
| Campti Revetment (B) | 158.4 | 3 | M | 77,500 | 3.0 | 93,000 |
| Campti Revetment (A) | 159.2 | 3 | M | 25,450 | 3.0 | 30,600 |
| Powhatan Revetment | 163.5 | 3 | L | 34,168 | 3.0 | 41,100 |
| Lumbra Revetment | 164.1 | 3 | L | 45,030 | 3.0 | 54,100 |
| Westdale Revetment Dredging | 191.0 | 4 | H | 205,155 | 5.0 | 410,400 |
| East Point Revetment Dredging | 194.2 | 4 | M | 42,787 | 3.0 | 51,400 |

#### 6.3.2.2 Improvement of Dikes

Dike improvement would be required for any dikes with existing placement that works well within the system but require realignments, extensions, raises, or reinforcement to adequately provide for a 12-FT channel. Dikes are designed to maintain a particular channel depth without the requirement of continued intervention—such as dredging—by directing the flow of water and pushing the thalweg to a more advantageous location. Dikes can also be built parallel to the flow to either maintain channel alignment or to realign the channel for desired flow conditions. Dike improvements would apply to various types of revetments throughout the river, including trail dikes, kicker dikes, and tiebacks.

This measure is included in Alternatives 2a, 2c, 2ab, 3a, 3c, and 3ab. For the first region of analysis through Pool 2, the recommended plan proposes improvements to six existing dikes listed in Table A-21 above the blue break line. Two additional dikes require improvements for the second region. Operation and maintenance (O&M) will only be required on improved structures every 50 years. As a standalone alternative, dike improvements would still require some dredging—frequency of two years—to account for locations with no depth maintaining structures. This includes Alternatives 2a and 3a. However, when used in combination with new dike construction, the frequency of in-channel dredging decreases to every 25 years as the channel's ability to self-scour increases.

*Table A-21. Improvements of Existing Dikes*

| Name | River Mile | L/R | Pool | Priority | B Stone (tons) (w/ 20% contingency) |
|---|---|---|---|---|---|
| Lorran Lake Realignment | 35.0 | L | G | H | **116,100** |
| Lorran Dikes | 36.5 | L | G | H | **28,800** |
| Joffrion Dikes (M1) | 37.3 | R | G | H | **10,700** |
| Joffrion Revetment | 38.2 | R | G | H | **51,500** |
| Joffrion Dikes (M2) | 38.2 | L | G | H | **8,800** |
| Bringol Revetment (M1) | 64.0 | R | 1 | M | **45,800** |
| Westdale Revetment | 192.0 | L | 4 | H | **39,500** |
| East Point Revetment (M1) | 194.0 | R | 4 | M | **59,900** |

### 6.3.2.3 New Dikes

A number of locations throughout the Red River would require the construction of new dikes to achieve a 12-FT draft at locations that do not currently maintain that depth. New dike locations were divided by priority with high-priority dikes discussed in the following subsection. Priority was determined by a location's current ability to maintain a 9-FT or 12-FT draft. High-priority dikes were placed at locations that currently struggle to maintain 9-FT. Medium priority dikes were placed at locations that would have difficulty maintaining 12-FT. Finally, low priority dikes were placed at locations that would occasionally have difficulty maintaining 12-FT. All new dikes, including the high-priority dikes listed in the next subsection, would be constructed as part of those measures described via “new dikes.” This includes Alternatives 2b, 2ab, 3b, and 3ab. All new construction dikes can be seen in Table A-22 and Table A-23 (Section 6.3.2.3.1), with the latter table showing only high-priority dikes. Structures that are only included within Region 1 (through Pool 2) are above the blue break lines in each table.

The construction of all new dikes would limit O&M channel dredging to once every 25 years (Alternatives 2b and 3b). When combined with dike improvements, the channel becomes fully self-scouring, and dredging is no longer needed (Alternatives 2ab and 3ab).

*Table A-22. New Construction Dikes*

| Name | River Mile | L/R | Pool | Priority | B Stone (tons) (w/ 20% contingency) |
|---|---|---|---|---|---|
| Hadden Fort Revetment | 52.5 | L | 1 | M | **116,300** |
| Barbin Dikes | 52.8 | R | 1 | M | **52,500** |
| Vick Downstream Extension Dikes | 54.0 | L | 1 | M | **37,700** |
| Dupre Dikes | 60.5 | L | 1 | M | **31,200** |
| Bringol Revetment (C1) | 64.5 | R | 1 | M | **108,700** |
| Bringol Dikes | 64.5 | L | 1 | M | **43,000** |
| Pointfield Dikes | 108.3 | L | 2 | L | **68,700** |
| Socot Dikes (C2) | 154.5 | R | 3 | M | **71,100** |
| Socot Dikes (C1) | 156.5 | L | 3 | M | **44,000** |
| Campti Dikes | 159.0 | R | 3 | M | **65,800** |
| Powhatan Dike Extensions | 163.6 | R | 3 | L | **18,300** |
| Lumbra Dikes | 164.8 | L | 3 | L | **26,400** |
| East Point Revetment (C1) | 194.2 | L | 4 | M | **26,600** |

### 6.3.2.3.1 High-Priority New Dikes

High-priority new dikes are those dikes whose construction is necessary for a 12-FT draft when paired with dike improvements. This combination of alternatives allows for the same benefits derived from only constructing new dikes but at a reduced cost. This includes Alternatives 2c and 3c. These alternatives would have dredging requirements of just once every 25 years.

*Table A-23. High-Priority New Dikes*

| Dike Name | River Mile | L/R | Pool | B Stone (tons) (w/ 20% contingency) |
|---|---|---|---|---|
| Lower Lorran Dikes | 35.0 | R | G | **27,100** |
| Joffrion Dikes (C1) | 37.3 | R | G | **26,300** |
| Joffrion Dikes (C2) | 37.5 | L | G | **88,700** |
| Joffrion Dike 4 | 38.5 | L | G | **5,500** |
| Larto Revetment | 40.5 | L | G | **213,300** |
| Westdale Dikes (C1) | 191.0 | L | 4 | **84,000** |
| Westdale Dikes (C2) | 191.7 | R | 4 | **28,600** |

### 6.3.3 Construction Considerations

#### 6.3.3.1 Dredging

Two different types of dredging are included in the proposed alternatives for this project: construction dredging and O&M dredging. USACE must perform annual O&M dredging to sustain sufficient channel width and depth for navigation at locations with sedimentation issues, most notably at the approaches to the locks and dams. Dredging maintenance records from the last 12 years for the Red River presented in Section 1 show that approximately 57 percent of O&M dredging days occur at the locks and 43 percent occur in the channel with an average of 26 days per year of in-channel dredging occurring annually. In channel O&M dredging will be reduced based on the rehabilitation and improvement of existing river training structures to the system. To achieve a 12-FT channel solely through dredging, approximately 1.5 million cubic yards of dredged material would be required. Dredged material is disposed of in locations of deeper, swift moving water that can transport sediment away from problem areas.

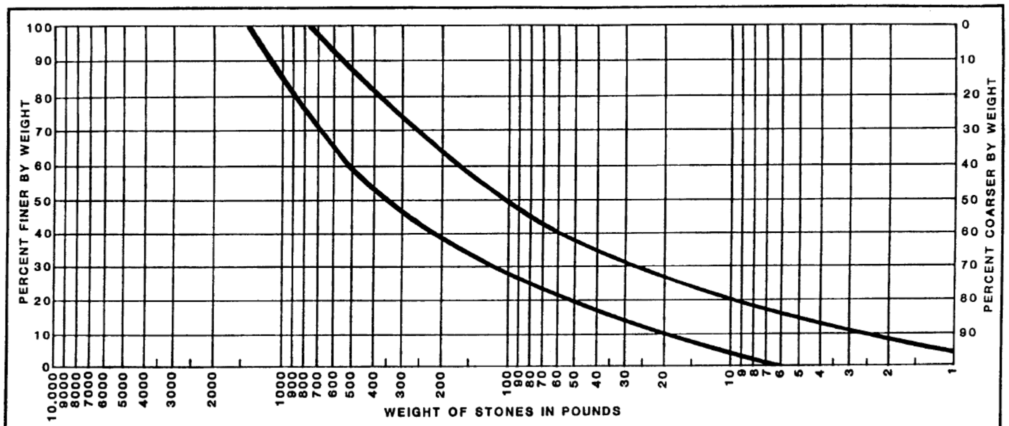
Dredging operations can typically only be performed during the low-water season due to the high currents of the Red River during high-water events. A cutterhead dredge is the best option for virgin material, due to the teeth of the cutterhead being able to effectively breakup compact river bottoms. The cutterhead dredge has spud anchors in the back and swings from side to side moving the anchors one at a time to walk up the dredge cut. Material is pumped through the cutterhead and out a discharge pipe. The discharge pipe can be assembled to various lengths to ensure proper placement of the material into swift currents.

Approximately 10 contracted cutterheads work within the lower half of the Mississippi Valley Division on an average low water year. The Regional Shallow Draft Team, through the Mississippi Valley Division, can work with other USACE groups and industry to ensure seasonal work is performed on time, given the number of available dredges each year. A phased approach allows for a balance with the dredging needs of the Mississippi Valley Division.

#### 6.3.3.2 River Training Structures

Dike construction work on the Red River is typically performed using Graded B-Stone or C-Stone, which are both uniformly graded and large enough to remain in place at flows typically seen within the Red River. Gradation curves for each of these stone types are in Figure A-149 and Figure A-150 below. Ancillary benefits of stone structures include habitat for fish and microinvertebrates and the ability to self-adjust. Revetment construction is completed from the channel by either a barge-mounted excavator or a dragline. In shallow areas of the river or when tiebacks are necessary, excavation equipment requires top bank access.

Local rock sources listed in the 2025 Mississippi Valley Division Master List of Stone Protection Sources can provide an adequate quantity of rock for all construction alternatives; however, based on contractor availability, rock placement is limited to approximately 200,000 tons per construction season. Based on these tonnages and funding awarded, the construction period is estimated to last 2 years for all dike alternatives. A phased approach allows both large marine contractors and small businesses to compete for this work.




| STONE WEIGHT POUNDS | CUMULATIVE % FINER BY WEIGHT |
|---|---|
| 1200 | 100 |
| 750 | 72-100 |
| 200 | 40-65 |
| 50 | 20-38 |
| 10 | 5-22 |
| 5 | 0-15 |
| 1 | 0-5 |

GRADATION

GRADED STONE B

MAY 1975

PLATE I

*Figure A-149. Gradation Curve for Graded B-Stone*

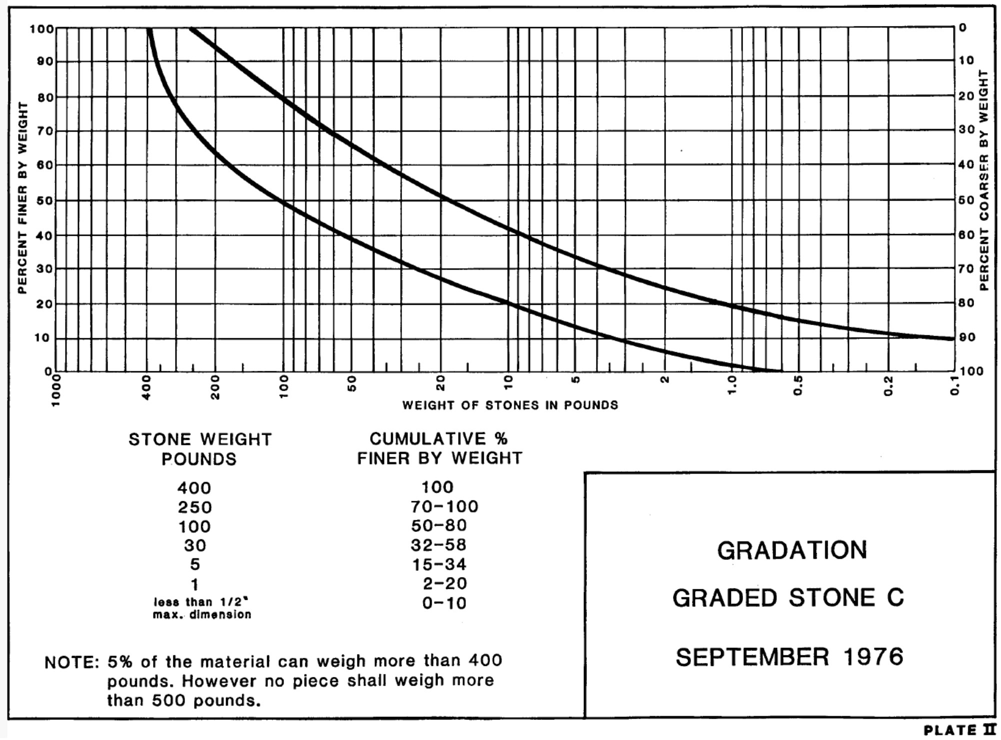



*Figure A-150. Gradation Curve for Graded C-Stone*

### 6.3.3.2.1 Typical Structures

Most alternatives for this project propose the improvement or construction of river training structures where a 12-FT depth is not currently maintained. These structures are strategically placed parallel or perpendicular to the navigation channel to concentrate flow into the main part of the channel and alter geomorphology; therefore, limited construction and maintenance dredging is required. Type and layout of structures are designed such that an acceptable channel alignment and dimensions are maintained. The typical designs for structures used as part of this project follow design standards that have been utilized throughout this system and have been proven to withstand the natural flow regimes of the Red River. Typical structure design parameters will be discussed in the following sections.

#### 6.3.3.2.1.1 Trail Dikes

Trail dikes are constructed within the channel parallel to the existing bank either against the erosional bankline or just riverward to protect the toe from further degradation. This type of revetment is used to either maintain the existing alignment or to realign streambanks. It was assumed that upper bank scour will continue until a stable slope is reached. These dikes must be placed beginning and ending at stable portions of the bank.

Trail dikes are typically constructed to a specified top elevation and crown width. Along the Red, the standard crown width is 5 to 10 feet. Crown elevations typically match adjacent existing structures but can also be selected based on providing a particular tonnage rate over a constant elevation. Typical side slopes for these structures are 1V:1.5H. End slopes are constructed to the natural slope of repose, which is between 1V:2.5H and 1V:5H. The ends of the structures can also be angled into the bank at between 20° and 30° from the bank to protect against flanking. Several alternatives for this project also require capouts or raising of existing stone toes. These are constructed conservatively by placing new stone such that the existing landward side slope is maintained. These structures are usually constructed in lifts from barge-mounted equipment. Geotechnical considerations are not required for the construction of these structures.

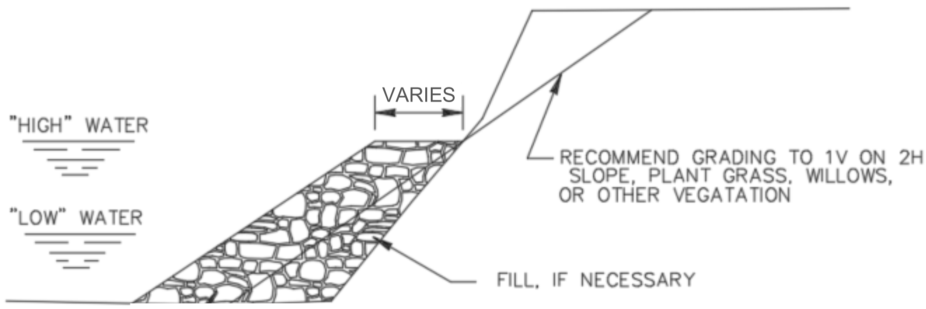



*Figure A-151. Typical Trail Dike Section*

6.3.3.2.1.2 Dikes

Dikes are constructed perpendicular to the riverbank and help maintain channel depth by concentrating flows in the deepest part of the river. These structures are constructed from the river by barge-mounted equipment. Total length of the structures is based on the desired channel width at the location. These structures have typical side slopes of 1V:1.5H. The river-end slope is usually constructed at the natural slope of repose which is between 1V:2.5H and 1V:5H; however, a flatter end slope, such as 1V:10H, can be used where more scour is anticipated. Existing dikes along the Red River also have a typical crown width of 5- to 10-FT. These dikes are also commonly constructed as part of a dike field with adequate spacing between structures equivalent to the spacing between existing structures.

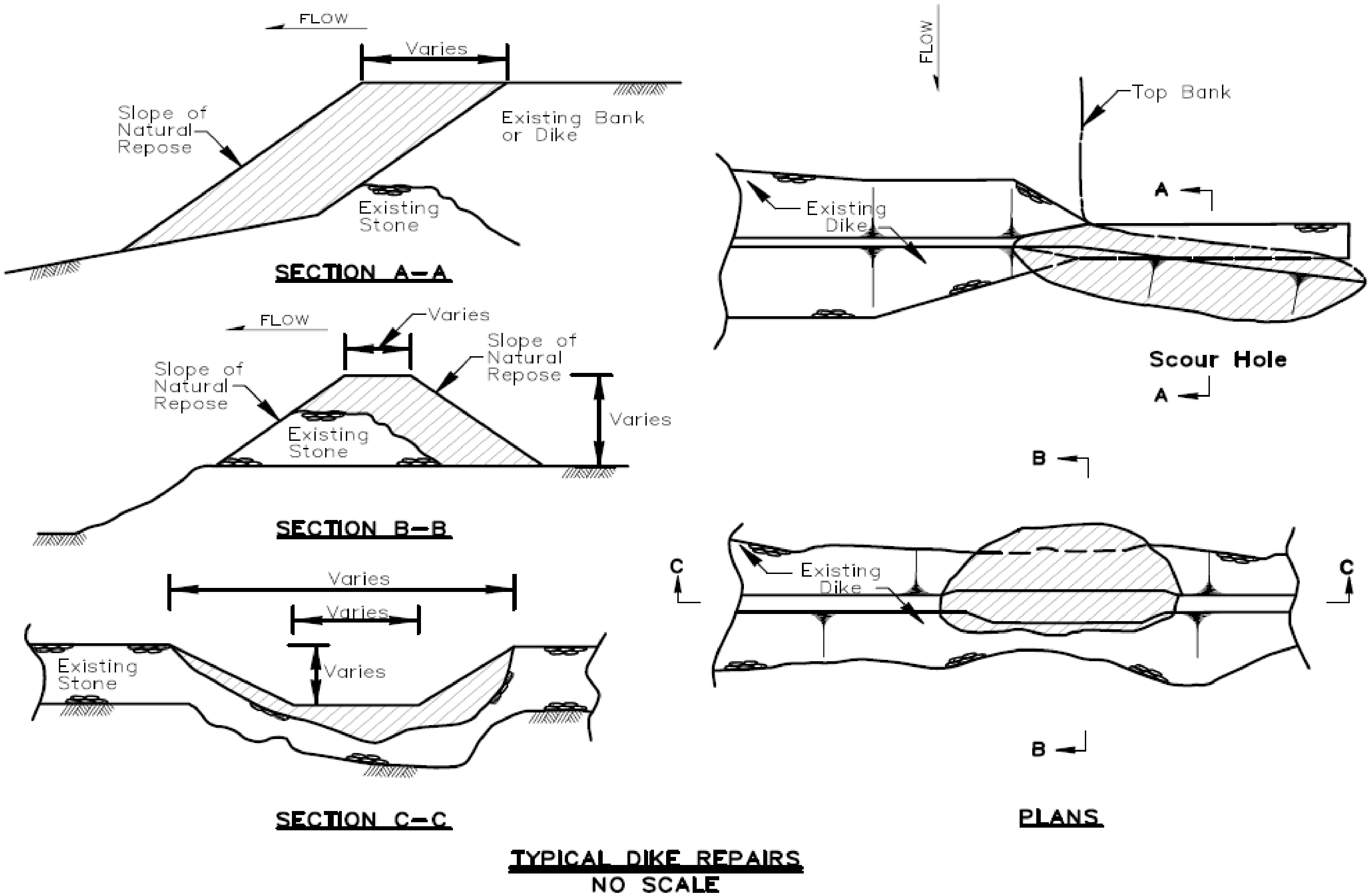



*Figure A-152. Repairs to Existing Dikes Typical Details*

A dig-in is also constructed on the bank end of these structures to prevent flanking. Dig-ins typically extend into the bank about 200 feet and slope down to the main portion of the dike at a 1V:5H slope. Dig-ins have a typical crown width of 34.5 feet with side slopes at the natural slope of repose. Typical sections for dig-in construction can be seen in Figure A-153 to Figure A-156.

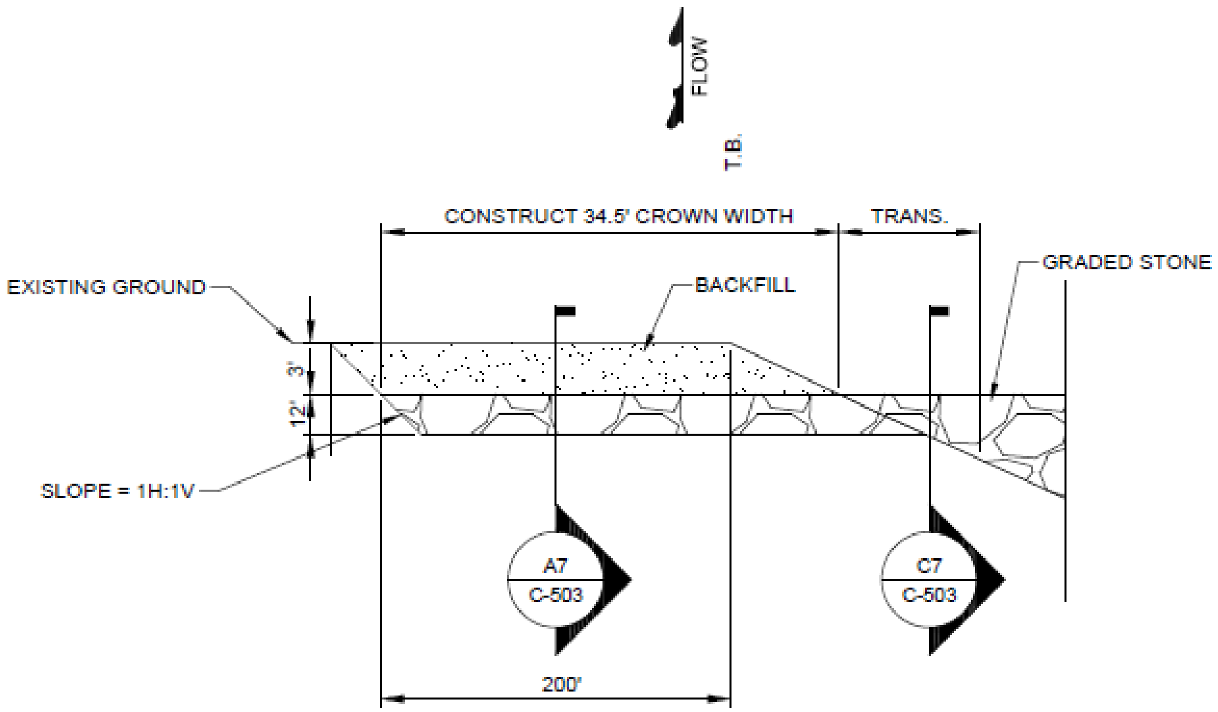



*Figure A-153. Typical Section for 35.4-Foot Crown Width Dig-In (Looking Upstream)*

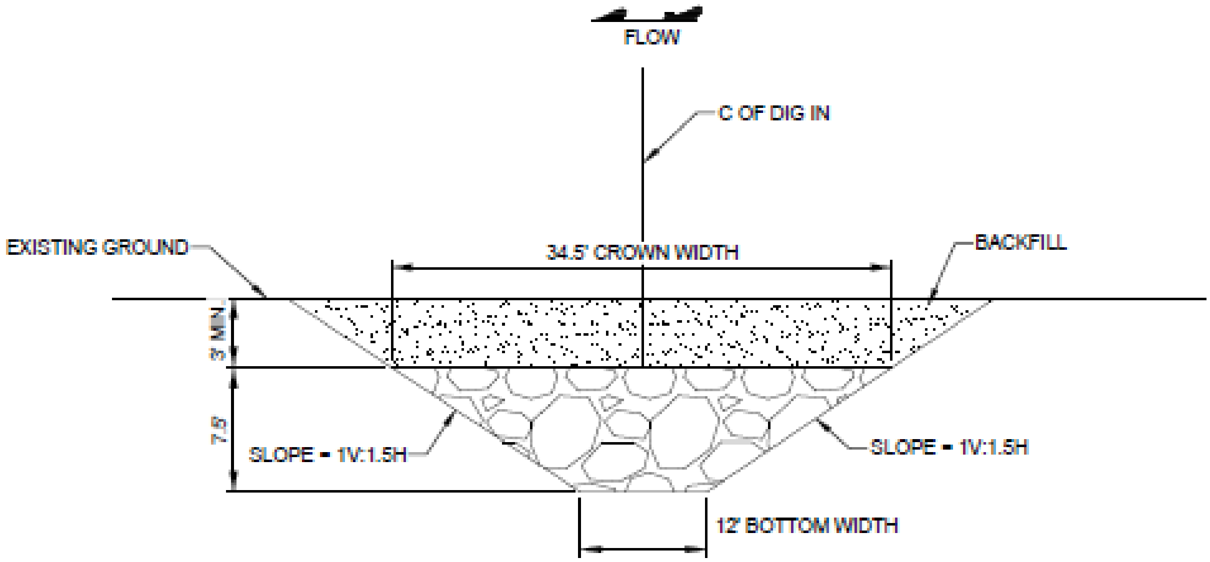



*Figure A-154. Typical Section for 34.5-FT Crown Width Dig-In (A7)*

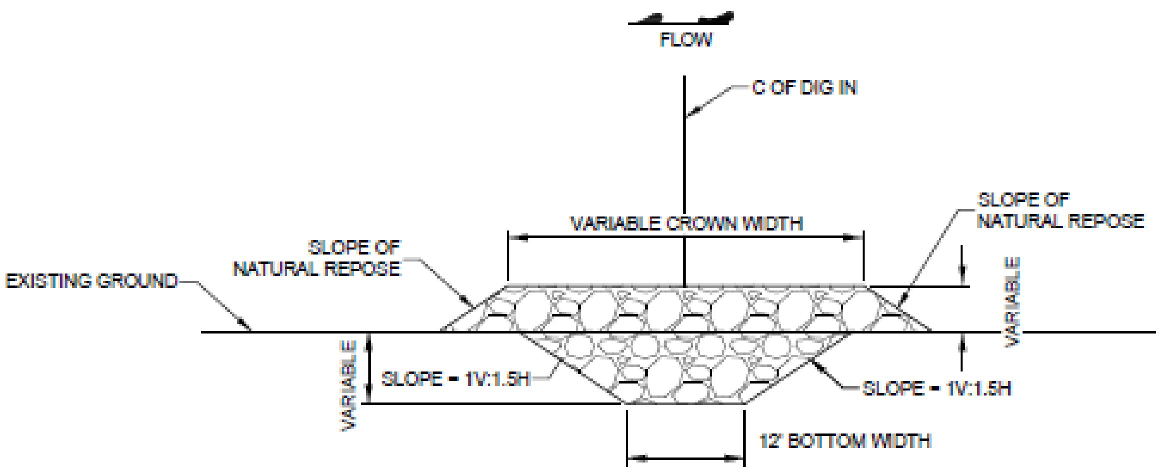



*Figure A-155. Typical Section for Transition Between Dike and Dig-In (C7)*

*Figure A-156. Typical Stone Dike Construction with Dig-In*

6.3.3.2.1.3L-Head Dikes (Kicker Dikes)

Kicker dikes are L-shaped extensions constructed off the downstream end of a typical dike structure. These extensions follow the curve of the river alignment and extend until the downstream end of the kicker is approximately 400 feet from the revetment on the opposite side of the river. These structures help to reduce scour at the stream end of a dike, pushing it farther downstream. These dikes are typically constructed with the same design parameters (i.e., crown width) as typical dikes discussed in the previous section.

6.3.3.2.1.4 Tiebacks

Stone tiebacks are constructed from the crest of a stone toe into the riverbank to prevent flanking or erosion of the structure by breaking up currents caused by overtopping. Tiebacks can only be used when there is adequate batture, this prevents any negative impacts to the structural integrity of the adjacent levee. If there is inadequate batture, stone bank paving could be used to further protect the bank.

Tiebacks are usually designed to the same height as the stone toe or slightly elevated. They are keyed into the bank. They are spaced every 100 to 200 feet depending on the length of the connected stone toe and the channel width. These structures do not require geotechnical analysis because there are no compaction requirements for the backfill over the structure. Construction for tiebacks does require some top bank access for excavation equipment.

### 6.4 IMPACTS OF RECOMMENDED PLAN

The Vicksburg District's recommended plan is Alternative 3a (improvement of dikes). This alternative proposes deepening the navigation channel along the JBJ Waterway from 9-FT to 12-FT by improving existing river training structures. This will be accomplished through targeted realignments, extensions, raises, and reinforcement of existing dikes at selected locations along the Red River. These structural modifications are designed to constrict and redirect the river's flow within the navigation channel, increasing flow velocity and promoting natural bed scour to achieve and maintain the desired channel depth.

The anticipated hydraulic response includes increased conveyance and more efficient sediment transport within the main channel, reducing the need for maintenance dredging. By narrowing the main flow path, the improved dikes will concentrate energy toward the centerline of the channel, resulting in a deeper, self-maintaining navigation corridor. These changes are expected to improve navigability, reduce shoaling in key areas, and provide more consistent channel geometry throughout the waterway. Overall, channel capacity will not be significantly impacted by this project as dike improvements require only minor alterations to the existing river training structures.

From a geomorphic perspective, impacts to local topography and geology are expected to be negligible. Since most work occurs within the river, no borrow material will be required, and any soil excavated to construct tiebacks will be replaced after rock placement. Soil composition is unlikely to change, as dredged material will be disposed of in areas with sufficient flow to transport and naturally disperse sediments downstream. Existing access points will be used when possible to avoid unnecessary soil disturbance. While dike improvements to increase from a 9-FT to 12-FT channel will affect sediment deposition patterns, these impacts are expected to be minor to the overall sediment budget.

Overall, the plan is expected to improve navigation reliability and reduce maintenance requirements in the channel. The hydraulic efficiency gained through structural modifications will be balanced with ongoing monitoring efforts, including yearly inspections and studies conducted by ERDC across the system. Continued coordination among engineering, environmental, and navigation stakeholders will be essential to ensure the system functions as intended and to optimize benefits.

Regarding the hydraulic effects of dikes on water surface elevations, dike dimensions and their associated hydraulic roughness have varying degrees of impacts based on the overall change of channel conveyance imposed by the dikes. For example, a larger dike in the channel relative to the overall channel area would be expected to have greater effect than a smaller dike in the channel. The recommended plan is incrementally improving existing dikes in select areas and not adding new dikes to the system. Therefore, the degree of influence is expected to be much less than the influence when introducing a new dike. This is an important consideration because existing dikes on the Mississippi River and Red River have been shown to have somewhat negligible influence on water levels at high flows due to the physical presence of the dikes when designed appropriately such that they are conducive to scouring the channel while mitigating the increase to water surface elevations.

However, dikes do have potential for secondary impacts to water surface elevations due to the sediment deposition and vegetation that occurs over time within the dike fields; that decreases channel conveyance while adding increased resistance to flow (hydraulic roughness). The 2020 Red River at Shreveport Hydraulic Analysis determined that the existing system of dikes impose inch-level variability on water surface elevations that occurred during the 2015 flood through model sensitivities, further described in the report. The analysis determined that raising all of the dikes in Pool 5 by 3 feet would have increased peak water surface elevations for the 2015 flood by a maximum of 3 inches within the pool, and that raising all of the dikes by 8 feet, the peak water surface elevations could have been almost 1 foot higher. Notably, this analysis was looking at modifying an entire system of dikes within an entire navigation pool, and the current plan is not recommending any changes within Pool 5. The analysis assessed the secondary impacts of dikes by removing the sediment deposition and vegetation that has developed in the Pool 5 dike fields, and found that 2015 peak water surface elevations could have been as much as 1 foot lower if deposition and vegetation had not occurred in the dike fields; however, this is generally a natural response for dike systems and the response can be exacerbated if high flow events do not occur for prolonged periods of time. The absence of high flows, to potentially flush out some portion of the vegetation, creates favorable conditions for the vegetation to grow and mature. Mississippi River Geomorphology and Potamology (MRG&P) studies, particularly MRG&P reports 37 and 44, have shown that dikes on the Mississippi River impose inch-level variability on water surface elevations at flood flows due to the presence of the dikes and the sediment deposition and vegetation that occurs within the dike fields. While the responses from the dike systems on the Mississippi River cannot be directly transposed to the Red River, the results of the aforementioned studies are informative. The recommended plan for this study is expected to incrementally improve existing dikes, in which dike fields are already well established. Further incrementally increasing the dimensions of the dikes to promote scouring for the 12-FT channel would be expected to have some localized influence to water surface elevations, but evidence suggest the influence would be at an inch-level of variability. Further, one of the four reaches selected for improvement under the TSP is downstream of L&D 1 that is situated in the uncontrolled lower Red River Backwater Area and significantly subjected to the conditions of the Mississippi River through ORCC. At flood flows from the Mississippi River and also from the Red River, the dikes in this stretch of river are substantially submerged essentially eliminating any influence on water surface elevations at flood flows. The other three areas selected for improvement in Pool 1 and Pool 4 are expected to have localized, inch-level variability on water surface elevations at flood flows. At these locations, levees are situated on both sides of the river confining flows within the river and within existing flowage easements.

# SECTION 7
# References and Resources

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U.S. Army Corps of Engineers (USACE). 2025. Vicksburg District Water Management: https://www.mvk-wc.usace.army.mil/watercontrol.html

**Software:**

Hydrologic Engineering Center's River Analysis System (HEC-RAS) version 6.5.

Hydrologic Engineering Center's Data Storage System (HEC-DSS) version 7.

AutoCAD Civil 3D 2023

Ensoft GROUP software

## SECTION 8 – ANNEX A

# ECB 2018-14 Analysis of Potential Climate Variability Vulnerabilities

This assessment is performed to highlight existing and future challenges facing the study area due to evolving hydrology and is conducted in accordance with U.S. Army Corps of Engineers' (USACE) Engineering Construction Bulletin (ECB) 2018-14, revised 19 August 2024. In accordance with ECB 2018-14, this evaluation identifies potential vulnerabilities to navigation projects in the Red River basin (Hydrologic Unit Codes (HUCs) 1114 and 0804). This assessment highlights existing and projected hydrology change driven risks for the study area.

<u>Literature Review</u>

Crimmins et al. (2023), the USACE *Civil Works Technical Report CWTS-2015-01,* as well as state and watershed specific resources published by the National Oceanic and Atmospheric Administration (NOAA) are the basis for this literature review. The focus of these references is on summarizing trends in historic temperature, precipitation, and streamflow records, as well providing an indication of future, projected hydrology based on the outputs from Global Circulation Models (GCMs)/Earth Systems Models (ESMs). For this assessment, background on observed and projected temperature and precipitation is provided as context for the impact that they have on observed and projected streamflow.

The NCA5 considers hydrological change research at both a national and regional scale (USGCRP, 2023), with a chapter on the Southeast region of the U.S. (Hoffman et al 2023). *Civil Works Technical Report CWTS-2015-01* was published as part of a series of regional summary reports covering peer-reviewed hydrology literature. The 2015 USACE reports cover two-digit USGS HUC watersheds in the U.S. The Red River is located in two-digit HUC 08, the Lower Mississippi River Region (USACE, 2015) and in the NCA5 Southeast region.

In many areas, temperature, precipitation, and streamflow have been measured since the late 1800s and these records provide insight into how the hydrology in the study area has changed over the past century. GCMs/ESMs are used in combination with different representative concentration pathways (RCPs)/ shared socioeconomic pathways (SSPs) reflecting projected radiative forcings up to the year 2100. Radiative forcings encompass the change in net radiative flux due to external drivers of changed hydrology, such as changes in carbon dioxide or land use/land cover. Projected temperature and precipitation results can be transformed to regional and local scales (a process called downscaling) for use as inputs in precipitation-runoff models (Graham, Andreasson, and Carlsson, 2007).

Uncertainty is inherent to projections of temperature and precipitation due to the GCMs/ESMs, RCPs/SSPs, downscaling methods, and many assumptions needed to create projections (USGCRP, 2017). There is less confidence in GCM/ESM simulations of mean precipitation than there is in their simulations of mean temperature. The coarse spatial resolution of GCMs/ESMs mean that they are not always able to include those processes and physical features of the earth system which operate at smaller spatial scales and are important for the formation of precipitation (Kotamarthi et al., 2016). When applied, precipitation-runoff models introduce an additional layer of uncertainty.

**Observed Temperature Trends**. Multiple studies indicate that annual average temperatures have increased in most of the U.S. For the Lower Mississippi River Region, no such trend was detected. In fact, a mild cooling for most of the region, particularly for summer and fall months, is presented (although not seemingly statistically significant) (Wang et al. 2009, Westby et al. 2013). However, another study noted that the cooling trend for their study region (which includes Water Resources Region 08) appears to end in the mid-1970s and is followed by a warming trend from about 1976 onward (Liu et al. 2012). There has also been an apparent shift in seasonality in the region, with spring warming occurring a few days later than in the past. Based on analysis which relied on observations collected at stations in the Lower Mississippi River Region, a statistically significant increasing trend in the number of one day extreme minimum temperatures was found, but no significant trend for the number of one day extreme maximum temperatures was found (Grundstein and Dowd 2011).

**Projected Temperature Trends**. Annual average temperatures are projected to rise throughout this century for the contiguous U.S. and Canada. Results of studies inclusive of the Lower Mississippi River Region typically fall in line with this generalization. Strong consensus exists in the literature that projected temperature in the study region show a sharp increasing trend over the next century. Many studies (Liu et al. 2013, Sherer and Diffenbaugh 2014, Elguindi and Grundstein 2013) indicate steadily increasing air temperatures throughout the 21$^{st}$ century. More recently, Hoffman et al (2023) noted a possible increase by 2050 of 30 days per year of extreme heat days (maximum temperatures above 95 °F) for a wide stretch that along the Mississippi Valley in Alabama and Louisiana, including Shreveport, Louisiana.

**Observed Precipitation Trends.** A mild upward trend in precipitation in the study region has been identified by multiple authors but a clear consensus is lacking. Palecki et al. (2005) found increasing trends in winter and fall storm intensities and decreasing trends in spring and summer storm intensities for the study region from 1972 to 2002. Grundstein (2009) identified significant positive linear trends (period 1895–2006) in both annual precipitation and the soil moisture index for multiple sites within the Lower Mississippi River Region. Wang et al. (2009) identified a significant increasing trend in precipitation for the southern half of the region, particularly in the fall and winter. For the northern half, a mild decreasing trend for all seasons was identified, except for the fall which shows an increasing trend. McRoberts and Nielsen-Gammon (2011) identified linear positive trends in annual precipitation for most of the U.S., including Water Resources Region 08. For this region the trend in annual precipitation indicates an increase on the order of 2–15 percent per century.

Examining trends in more extreme precipitation events, Wang and Zhang (2008) identified statistically significant increases in the frequency of the 20-year storm event. An increase in frequency of approximately 25–50 percent was quantified from the period 1949 to 1976 to the period 1977 to 1999. Pryor et al. (2009) found no trend or even a potentially decreasing trend for precipitation intensity associated with extreme events (e.g., 90$^{th}$ percentile precipitation days). Brommer et al. (2007) found no significant changes in long-duration precipitation events for the Lower Mississippi River Region during the 20$^{th}$ century, despite such changes quantified for many other areas in the U.S. Small et al. (2006) identified statistically significant increasing trends for the region in annual and fall precipitation for multiple locations in the region. There were also multiple stations within the region where no statistically significant trends in precipitation were identified. Li et al. (2011) identified statistically significant increasing trends in the occurrence of heavy rainfall in a region inclusive of Water Resources Region 08 for multiple meteorological stations with at least 50 years of historical record. While significant trends were identified for a number of stations in the region, an even greater number of stations in Water Resources Region 08 exhibited no significant trends. Wang and Killick (2013) found nonstationarity in monthly precipitation totals for the 8 of 56 study watersheds but not in Water Resources Region 08, suggesting potential changes in low, or base, precipitation, but not in high flow storm events in the region. Two studies (Chen et al. 2012 and Cook et al. 2014) identified a slightly decreasing trend in the occurrence of drought in the study region, though not statistically significant.

**Projected Precipitation Trends.** In line with projections for the rest of the country, projections of future changes in precipitation in the Lower Mississippi River Region are variable and generally lacking in consensus among studies or across models. Liu et al. (2013) quantified significant increases in spring precipitation associated with a 2055 planning horizon, relative to a recent historical baseline (1971–2000, centered around 1985) for the Lower Mississippi River Region. Smaller increases, or even slight decreases, are projected for the other seasons. However, increases in the severity of future droughts for the region are projected, as projected temperature and evapotranspiration (ET) impacts outweigh the increases in precipitation. Projections presented by Zhang et al. (2010) display differences within the Water Resources Region 08, with increased precipitation projected for parts of the region (particularly the coast) and decreased precipitation for others. The Gao et al. (2012) study generally projects increases in the magnitude of annual and daily extreme (95$^{th}$ percentile) storm events and in the frequency of precipitation, for their 2058 planning horizon. Liu et al. (2012) generally projects an overall small increase in annual precipitation for the Water Resources Region 08 by the end of the 21$^{st}$ century, as well as increased year to year variability in rainfall totals. Studies by Tebaldi et al. (2006) and Wang and Zhang (2008) project small increases in the occurrence and intensity of storm events by the end of the 21$^{st}$ century for the general study region.

**Observed Streamflow Trends**. A mild upward trend in mean streamflow in the study region has been identified by multiple authors but a clear consensus is lacking. Studies of trends and nonstationarity in streamflow data collected over the past century have been performed throughout the continental U.S., some of which include Water Resources Region 08. There appears to be a reasonable consensus among these studies that trends, if any, show a general increase in river flow in the region (Mauget 2004, Kalra et al. 2008, Small et al. 2006). It is noted, however, that Hoffman et al (2023) observed that “in recent years, low-flow conditions due to regional droughts on Southeast waterways such as the Mississippi have halted or delayed the movement of barges carrying bulk goods, with regional and national implications.”

**Projected Streamflow Trends.** Although consensus is lacking, a small number of reviewed studies indicate a mild decreasing trend in streamflow for the study region through the next century. Thomson et al. (2005) generated contradictory results: for the same set of input assumptions, one model predicts significant decreases in water yield, and the other projects significant increases in water yield. This study highlights the significant uncertainties associated with global projected hydrology, particularly with respect to parameter selection. Doll and Zhang (2010) projected regional impacts to include small (10–20 percent) decreases to both low and average annual flows for their 2055 planning horizon compared to the historical baseline.

**Summary.** Within the literature reviewed, there is evidence that streamflow and precipitation have slightly increased over the observed period of record within the Lower Mississippi River Basin. No significant trend in temperature has been observed in the region. Temperature is projected to increase. Little consensus exists in projected trends in future precipitation in the study region. Streamflow is projected to mildly decrease in the study region through the next century. Figure AA-1 from the 2015 USACE *Civil Works Technical Report CWTS-2015-08* provides a visual summary of the trends in observed and projected hydrometeorological variables for 2-digit HUC 08, the Lower Mississippi River Region.

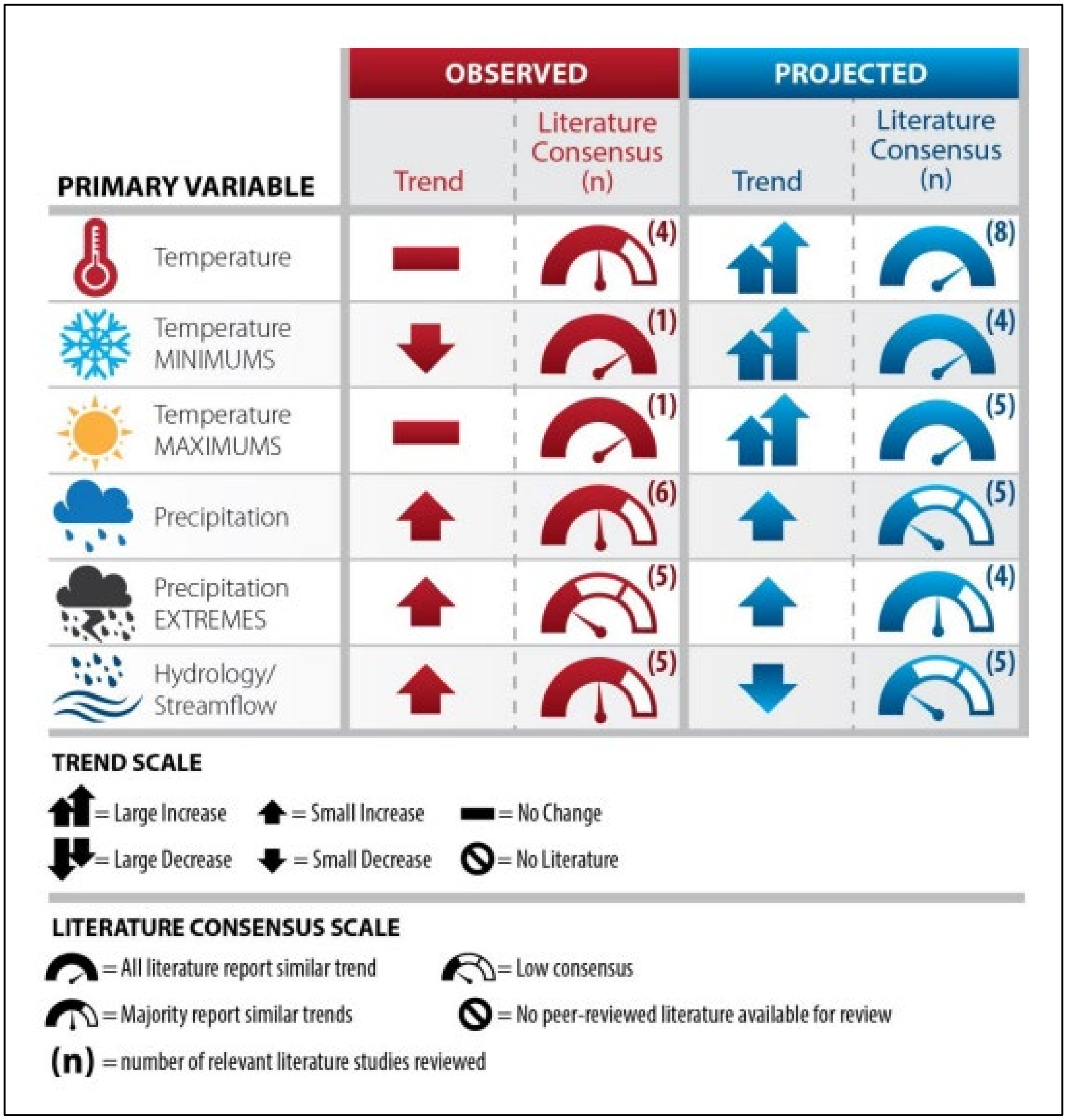
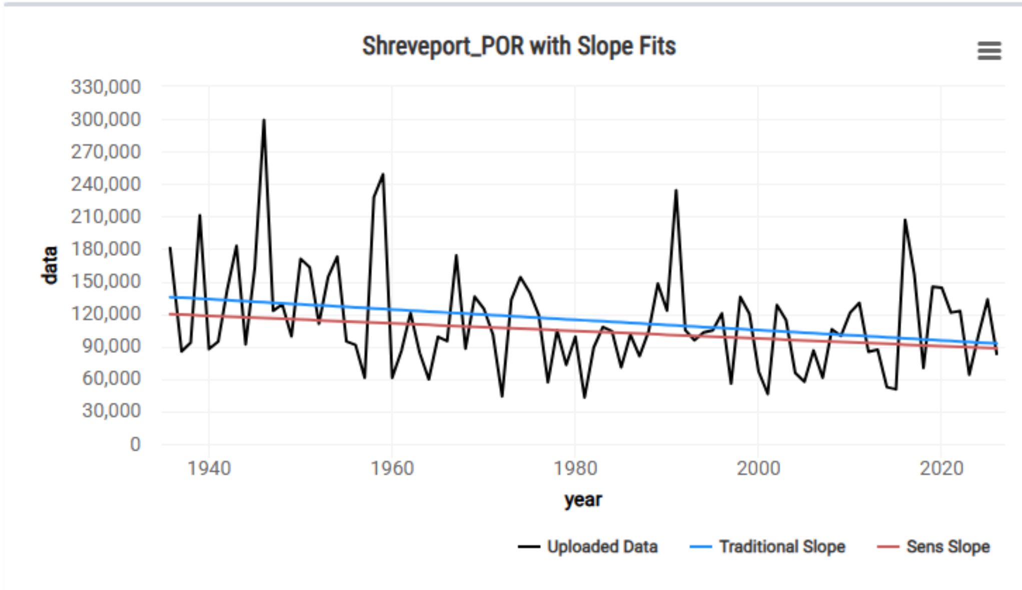



*Figure AA-1. Summary Matrix of Lower Mississippi River Region (HUC 08) Observed and Projected Hydrology Trends (USACE, 2015)*

### Nonstationarity Detection and Trend Analysis

The assumption that hydrologic timeseries are stationary (their statistical characteristics are unchanging) in time underlies many traditional hydrologic analyses. Statistical tests can be used to test this assumption using the techniques outlined in USACE Engineering Technical Letter (ETL) 1100-2-3, Guidance for Detection of Nonstationarities (2017). The USACE Time Series Toolbox (TST) tool is a web-based tool that performs the statistical tests described in the guidance. The hydrologic timeseries examined for this application is the annual instantaneous peak streamflow as recorded by USGS gage 07355500, Red River near Alexandria; 07348500, Red River at Shreveport, Louisiana; 07350500, Red River at Coushatta; and 07351930, Red River at Grand Ecore, Louisiana. The Shreveport and Alexandria gages have a period of record from 1935 to 2025, and the Coushatta and Grand Ecore gages have a period of record from 1960 to 2025. The data were analyzed with the TST tool for both the period of record and for the period from 1995 to 2025 (after construction was completed on the J. Bennett Johnston (JBJ) Waterway).

Monotonic trends are evaluated using the t-test, Mann-Kendall and Spearman rank order tests. A p-value threshold of 0.05 (<0.05 is considered statistically significant) is applied to evaluate whether trends are statistically significant. Analysis indicates a statistically significant (t-test), negative trend in the 1935–2025 period of record for the Shreveport gage (see trendline in Figure AA-2). Trends for the other gages, for both POR and the 1995–2025 period, as well as the 1995–2025 analysis for the Shreveport gage, indicate no statistically significant trends.




**Trend Line Coefficients**

| Method | Directionality | Slope | Intercept |
|---|---|---|---|
| Traditional Slope | Negative | -476 | 1057110 |
| Sen's Slope | Negative | -350 | 797639 |

**Trend Hypothesis Test**

| Test | P-Value |
|---|---|
| t-Test | 0.0115 |
| Mann-Kendall | 0.0631 |
| Spearman Rank-Order | 0.0509 |

- **A statistically significant trend (at the alpha = .05 level) was detected by the t-Test.**
- **A statistically significant trend (at the alpha = .05 level) was NOT detected by the Mann-Kendall Test.**
- **A statistically significant trend (at the alpha = .05 level) was NOT detected by the Spearman Rank-Order Test.**

*Figure AA-2. Trend Analysis for Annual Peak Streamflow (cfs) at Shreveport, Louisiana, with Trendline Coefficients and Significance*

Nonstationarities were detected for the Shreveport gage in 1976 using the period of record data, but there was not a consensus between the different statistical tests; therefore, the nonstationarities were not considered robust. A strong nonstationarity is one that demonstrates a degree of consensus, robustness, and a significant increase or decrease in the sample mean and/or variance. The nonstationarity is identified by only one test, that targeted change in the overall statistical distribution, and no other types of tests identified a nonstationarity. No other gages, for either period of data, had nonstationarities detected.

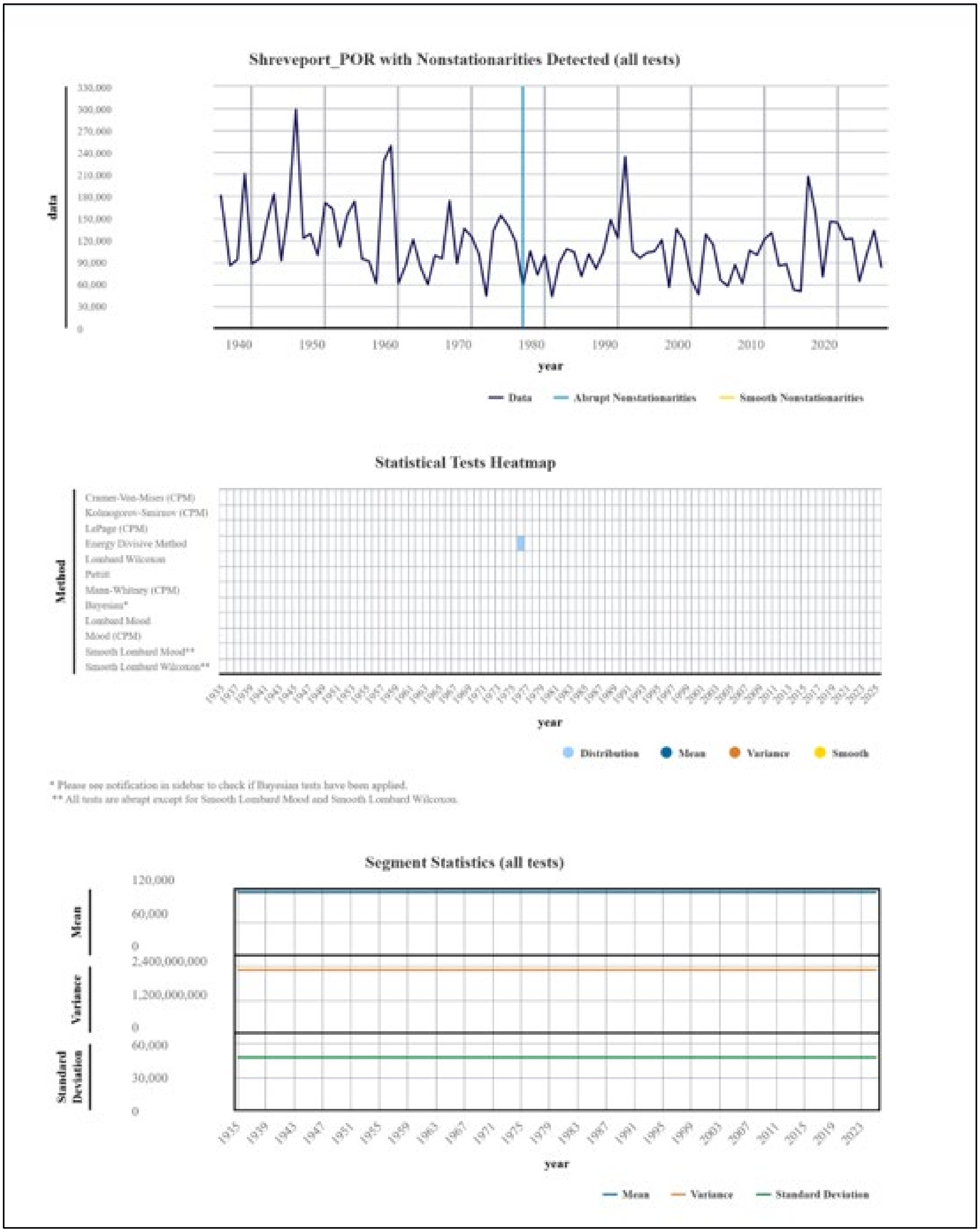



*Figure AA-3. Time Series Toolbox Output for Annual Peak Streamflow Red River Near Shreveport, Louisiana (1935–2025)*

### Comprehensive Hydrology Assessment Tool (CHAT)

The USACE Comprehensive Hydrology Assessment Tool (CHAT) displays hindcasts and projections of streamflow, temperature, and precipitation outputs, derived from 32 GCMs. The CHAT uses Coupled Model Intercomparison Project Phase 5 (CMIP5) GCM meteorological data outputs that have been statistically downscaled using the Localized Constructed Analogs (LOCA) method. Projected results in the CHAT for 2006 to 2099 are produced using two future scenarios: RCP 4.5 (where carbon dioxide ($CO_2$) emissions stabilize by the end of the century) and RCP 8.5 (where $CO_2$ emissions continue to increase throughout the century). Simulated output representing the historic period of 1951 to 2005 are generated using a reconstitution of historic greenhouse gas emissions.

To analyze runoff, LOCA-downscaled GCM outputs are used to force an unregulated, Variable Infiltration Capacity (VIC) hydrologic model. Areal runoff from VIC is then routed through a stream network using MizuRoute. Outputs represent the daily in-channel, routed streamflow for each stream segment—valid at the stream segment endpoint. Since the runoff is routed, the streamflow value associated with each stream segment is a representation of the cumulative flow, including all upstream runoff, as well as the local runoff contributions to that specific segment. Within the CHAT, streamflow output can be selected by stream segment and precipitation/temperature output can be selected for a given 8-digit HUC watershed.

The Red River near Shreveport gage is in the 4-digit HUC 1114 (Red-Sulphur Basin). The 8-digit HUC of interest specific to the study area is the Middle Red-Coushatta watershed (HUC 11140202). The stream segment used for CHAT analysis was stream segment 11002807. In CHAT, the annual maximum of mean monthly streamflow and the annual maximum 3-day precipitation are analyzed to investigate if and how potential, future peak streamflow conditions will change. Figure AA-4 and Figure AA-5 show the range of the modeled, annual maximum of mean monthly streamflow and annual maximum 3-day precipitation output presented for the historic period (1951–2005) and the future period (2006–2099). The range of output is indicative of the uncertainty associated with projected streamflow and precipitation.

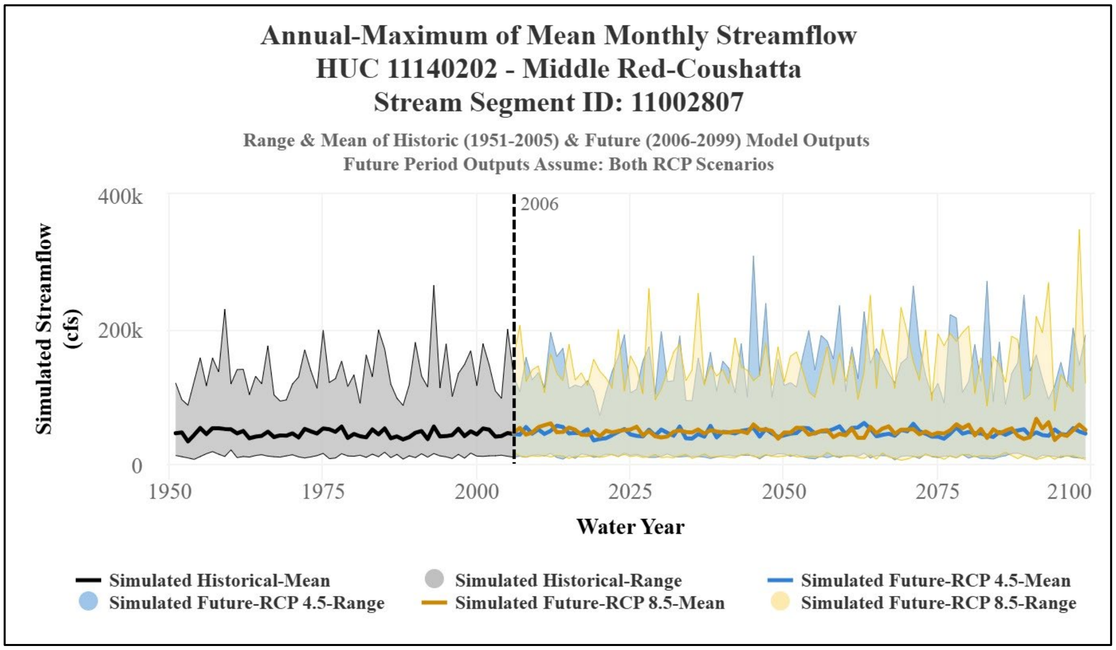



*Figure AA-4. Range of Annual Maximum of Mean Monthly Streamflow Model Output for the Middle Red-Coushatta Watershed (HUC 11140202) Stream Segment: 11002807*

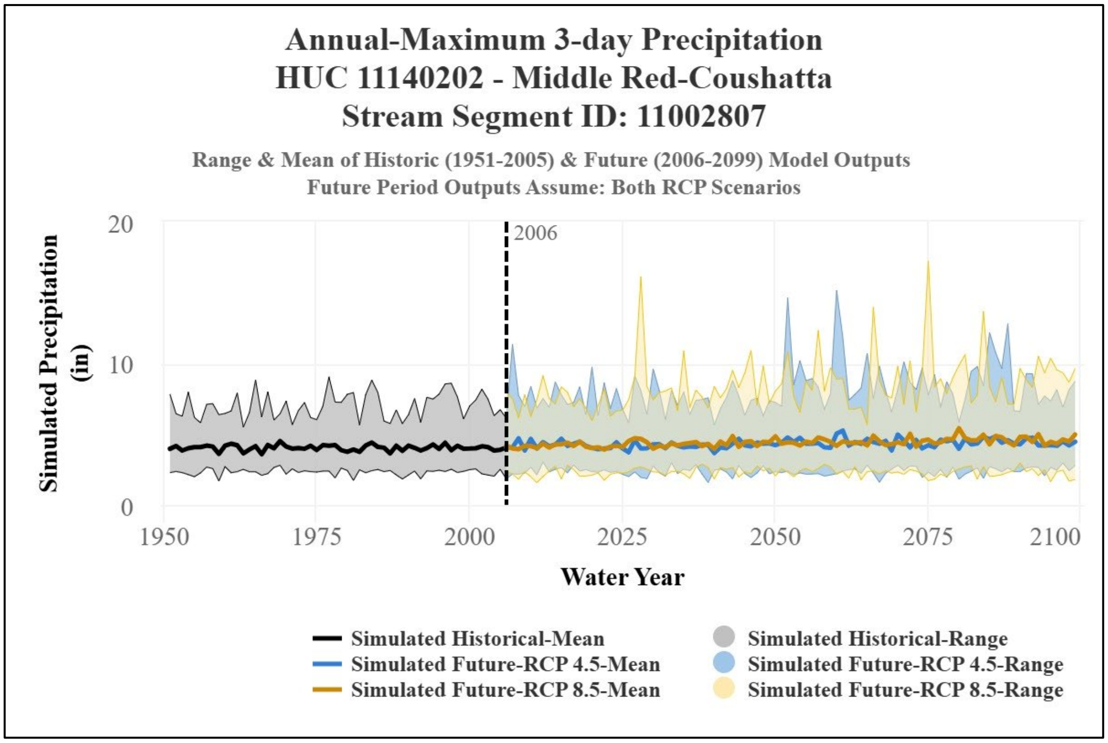



*Figure AA-5. Range of Annual Maximum 3-Day Precipitation Model Output for the Middle Red-Coushatta Watershed (HUC 11140202)*

For the Middle Red-Coushatta watershed (HUC11140202) trends in mean model output are evaluated using the t-Test, Mann-Kendall and Spearman rank order tests. All three statistical tests are applied using a 0.05 level of significance (p-values<0.05 are considered statistically significant). The results of the three statistical tests and the slopes associated with identified, statistically significant trends are presented in Figure AA-6 and in Table AA-1. The mean of the 32 projections of simulated, annual maximum of mean monthly streamflow for the future period (2006–2099) shows no statistically significant trends. The RCP 8.5 trendline has a slope of 2.1 cfs a year, which equates to a 105 cfs change in the average of the 32 projections of annual maximum of mean monthly streamflow over a 50-year period (between 0.2 and 0.25  percent of the current value).

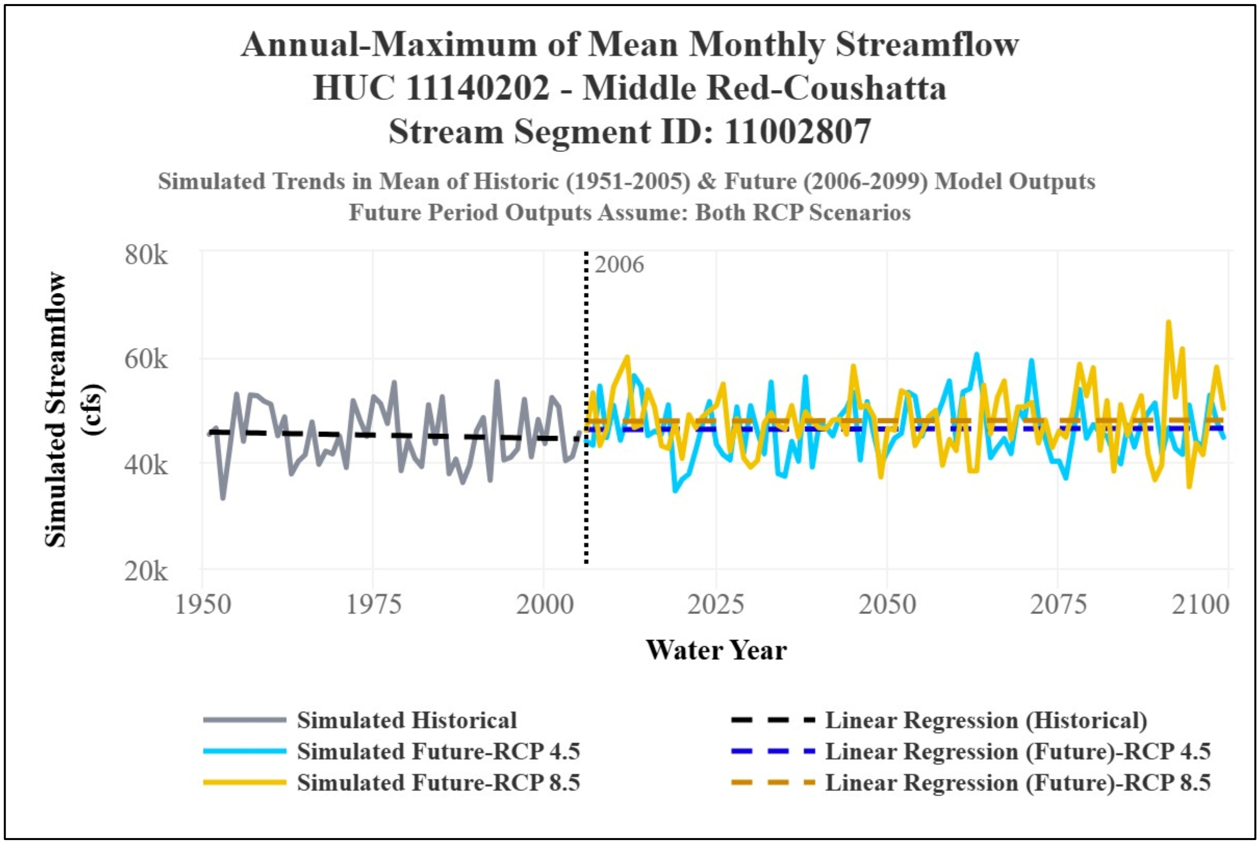



*Figure AA-6. Trend Analysis of Average Model Output: Annual Maximum of Mean Monthly Streamflow Middle Red-Coushatta Watershed (HUC111400202) Stream Segment: 11002807*

*Table AA-1. Trend Analysis of Average Model Output: Annual Maximum of Mean Monthly Streamflow Middle Red-Coushatta watershed (HUC11140202) Stream Segment 11002807*

| Trend Analysis | Historic (1951–2005) | Future (2006–2099) | | Historic (1951–2005) | | | Future (2006–2099) | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | RCP 4.5 | RCP 8.5 | | | | RCP 4.5 | | | RCP 8.5 | | |
| | p-values | | | Statistically Significant? (<0.05) | Slope (cfs/year) | Direction | Statistically Significant? (<0.05) | Slope (cfs/year) | Direction | Statistically Significant? (<0.05) | Slope (cfs/year) | Direction |
| t-Test | 0.616 | 0.904 | 0.926 | No | -23.87 | ↓ | No | 2.49 | ↑ | No | 2.08 | ↑ |
| Mann-Kendall | 0.45 | 0.958 | 0.974 | No | | | No | | | No | | |
| Spearman Rank Order | 0.447 | 0.974 | 0.988 | No | | | No | | | No | | |

For the mean of the 32 projections (per RCP) of annual maximum 3-day precipitation, the results of the three statistical tests and the slopes associated with statistically significant trends are presented in Figure AA-7 and Table AA-2. The mean of the simulated, annual maximum precipitation projections (future period: 2006–2099) shows a statistically significant, positive trend for the Red River watershed under both the moderate (RCP 4.5) and higher (RCP 8.5) emission scenarios. The CHAT computes a trendline slope of 0.0059 inches per year for the higher emission scenario, which would be a 0.295 inch or approximately 8 percent increase in maximum 3-day precipitation over a 50-year period. There are no statistically significant trends in simulated, historic precipitation between 1951 and 2005.

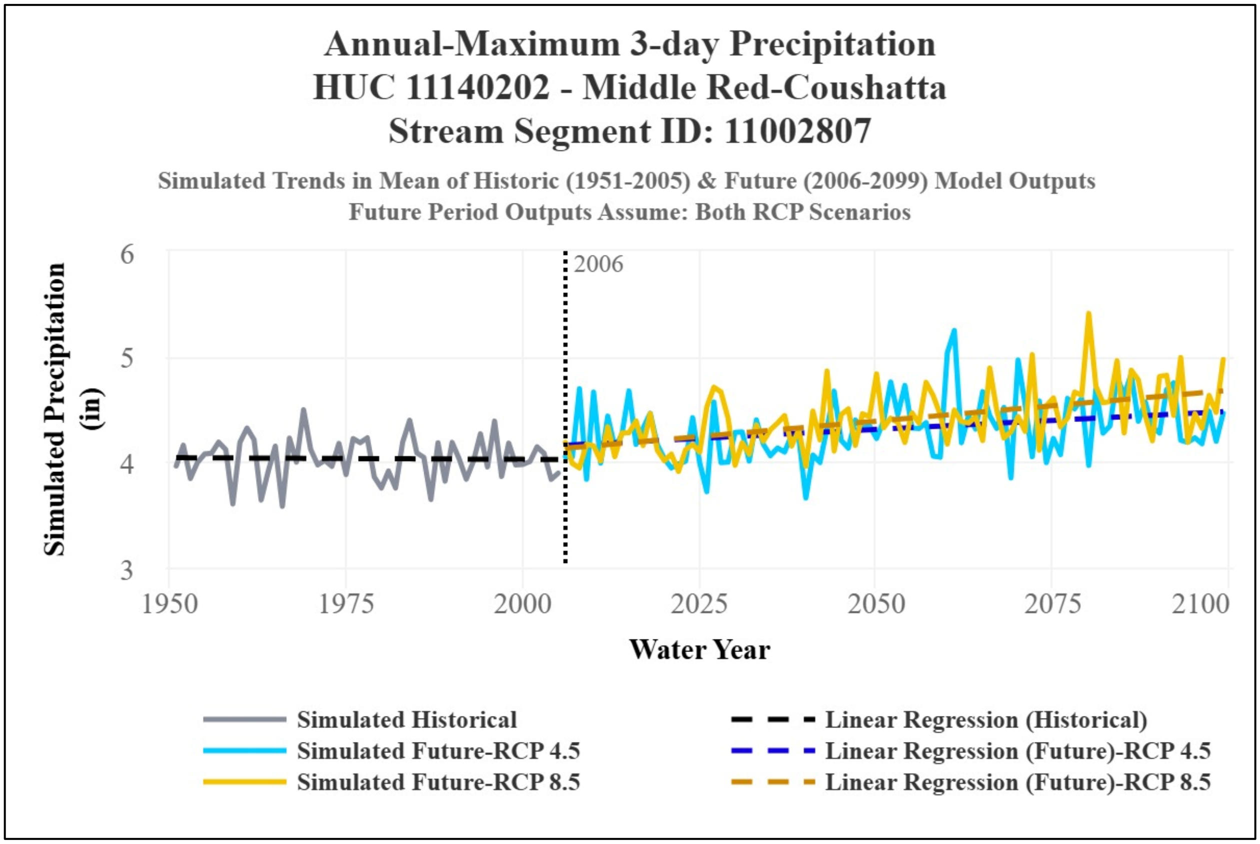



*Figure AA-7. Historic and Projected Trends in Historic and Projected Annual Maximum 3-day Precipitation for the Middle Red-Coushatta Watershed (HUC 11140202)*

*Table AA-2. Trend Analysis of Average Model Output: Annual Maximum 3-Day Precipitation for Middle Red-Coushatta Watershed (HUC09010003)*

| Trend Analysis | Historic (1951–2005) | Future (2006–2099) | | Historic (1951–2005) | | | Future (2006–2099) | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | RCP 4.5 | RCP 8.5 | | | | RCP 4.5 | | | RCP 8.5 | | |
| | p-values | | | Statistically Significant? (<0.05) | Slope (in/year) | Direction | Statistically Significant? (<0.05) | Slope (in/year) | Direction | Statistically Significant? (<0.05) | Slope (in/year) | Direction |
| t-Test | 0.858 | 0.00173 | <0.001 | No | $-3e^{-4}$ | ↓ | Yes | 0.0034 | ↑ | Yes | 0.0059 | ↑ |
| Mann-Kendall | 0.674 | <0.001 | <0.001 | No | | | Yes | | | Yes | | |
| Spearman Rank Order | 0.601 | <0.001 | <0.001 | No | | | Yes | | | Yes | | |

The CHAT provides streamflow and precipitation outputs analyzed comparatively by describing simulated changes in monthly and annual streamflow and precipitation between a baseline epoch (1976–2005) and two future epochs: 2035–2064 (mid-century) and 2075–2099 (end of century). Epoch-based monthly and annual change in streamflow and precipitation is presented using boxplot visualizations. The monthly boxplots provide insight into the seasonality of changes in streamflow and precipitation overtime.

For stream segment 11002807 in the Middle Red-Coushatta watershed (HUC11140202), changes in epoch-mean of simulated monthly mean streamflow are presented in Figure AA-8. Change in Epoch-Mean of Simulated Monthly Mean Streamflow - HUC 11140202 – Middle Red-Coushatta- Stream segment ID: 11002807. For the stream segment of the Red River analyzed, it appears that for both emission scenarios for end-century epochs flows in most of the year, except for late summer to autumn, are slightly decreasing. For the mid-century epoch, flows do not appear to be significantly changing. When the CHAT is used to evaluate the change in the epoch-mean of simulated annual-mean streamflow it is found that the median change from the base Epoch (1976–2005) to the mid-century epoch (2035–2064) is -1 percent under the RCP 8.5 scenario. By the end-century epoch (2070–2099) the change relative to the base period is -7 percent under the RCP 8.5 scenario.

For the Middle Red-Coushatta watershed (HUC11140202), changes in epoch-mean of simulated monthly maximum 3-day precipitation are presented in Figure AA-9. Change in Epoch-Mean of Simulated Monthly Maximum 3-Day Precipitation - HUC 11140202 – Middle Red-Coushatta. Results for both the mid-century epoch (2035–2064) and the end-century epoch (2070–2099) indicate a slight increase in winter and springtime precipitation. Changes to summer precipitation do not appear to be as substantial. When the CHAT is used to evaluate the change in epoch-mean of simulated annual maximum 3-day precipitation it is found that the median change from the base epoch (1976–2005) to the mid-century epoch (2035–2064) is 0.12 inches for RCP 4.5 and 0.20 inches for RCP 8.5. By the end-century epoch (2070–2099) the change relative to the base period is 0.20 inches for RCP 4.5 and 0.33 inches for RCP 8.5.

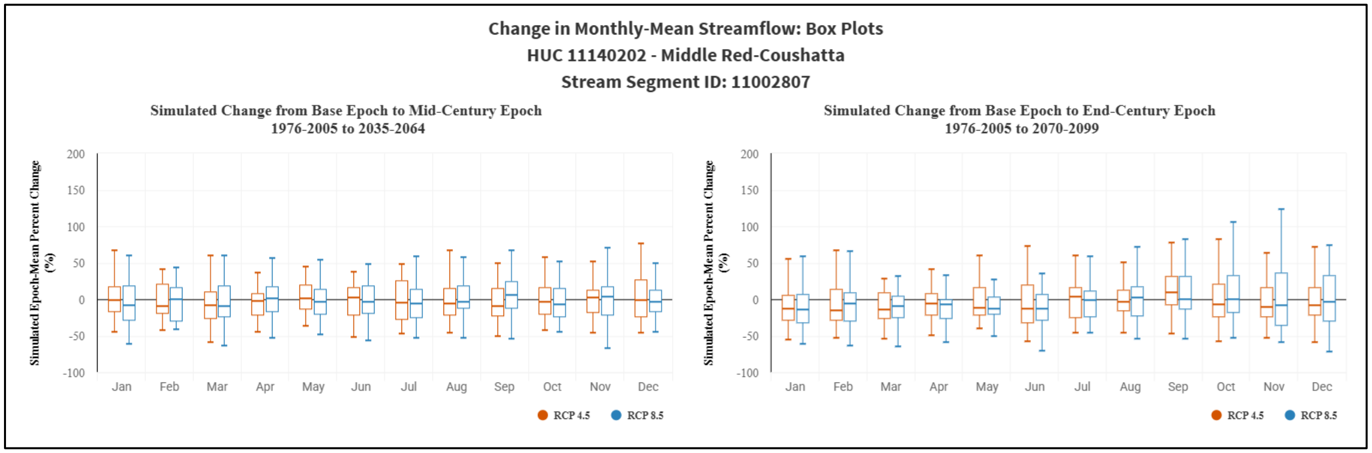



*Figure AA-8. Change in Epoch-Mean of Simulated Monthly Mean Streamflow - HUC 11140202 – Middle Red-Coushatta-Stream Segment ID: 11002807*

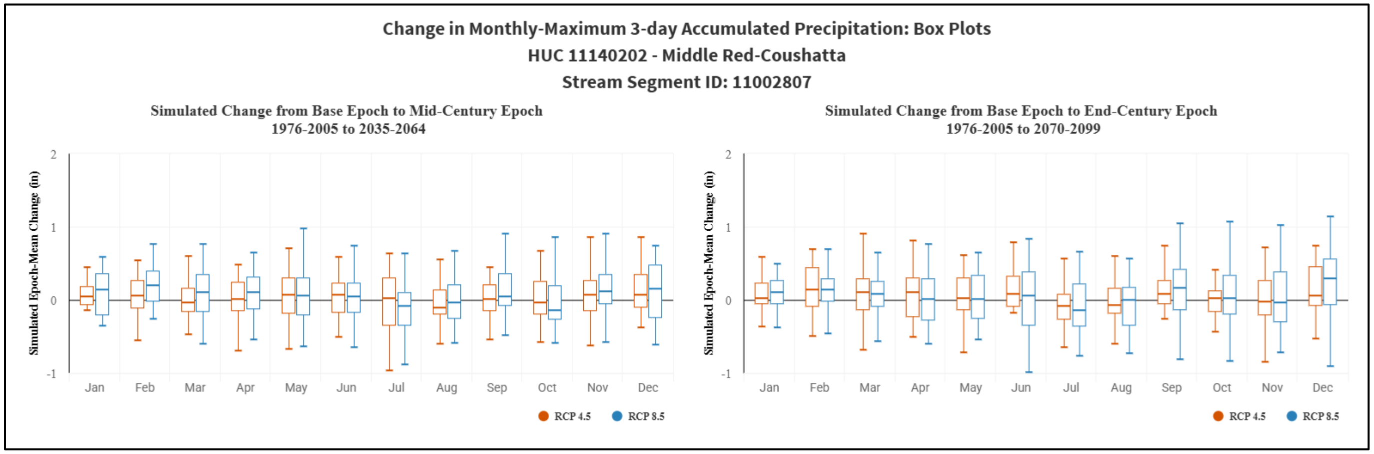



*Figure AA-9. Change in Epoch-Mean of Simulated Monthly Maximum 3-Day Precipitation - HUC 11140202 – Middle Red-Coushatta*

**Screening Level Relative Vulnerability Assessment: Flood Risk Reduction**

The U.S. Army Corps of Engineers (USACE) Screening-Level Civil Works Vulnerability Assessment Tool (CWVAT) analyzes vulnerability for the entire U.S. at the watershed level. The tool contains maps, visualizations, and tables designed to evaluate the ways in which a USACE project, or portfolio of projects, might be impacted by natural hazards, both currently and in the future, as part of an overall risk assessment. The tool helps to identify the hazard to which sites and regions are most exposed, and how that exposure is anticipated to change over time. This is a critical first step in addressing the potential physical harm, security impacts, and degradation in readiness to the Civil Works mission.

Conceptually, vulnerability is the degree to which infrastructure, systems, people, organizations, missions, operations, or activities are exposed, sensitive, and able to adapt to adverse impacts of natural hazards. Exposure is the geographic proximity of infrastructure, systems, people, organizations, missions, operations, or activities to a hazard. Sensitivity is the degree to which a hazard beneficially or adversely affects the intended function of infrastructure, systems, people, organizations, missions, operations, or activities. Adaptive Capacity is the ability of infrastructure, systems, people, organizations, missions, operations, or activities to adjust to adverse impacts caused by a hazard.

Currently, the CWVAT is focused on exposure; however, the sensitivity of each hazard to the primary USACE business is also contextually described throughout the tool. Sensitivity and overall vulnerability will be further incorporated into the CWVAT during future updates.

Two primary use cases within the tool are a comparative analysis, intended for a portfolio of locations, and a project area analysis, for a single location. The purpose of a comparative analysis using the CWVAT is to compare exposure and assess sensitivity at different sites or locations. An example use case for the comparative analysis is evaluating vulnerability to the water supply mission at a portfolio of reservoirs. The purpose of a project area analysis using the CWVAT is to assess exposure and sensitivity for a specific project area. An example use case for the project area analysis is evaluating vulnerability to an aquatic ecosystem restoration project with standard measures.

The tool features two future epochs, defined as 30-year periods centered on either 2050 or 2085. The 2050 epoch is consistent with medium-range planning, while the 2085 epoch is for long-range planning. The tool also contains a historical Base epoch that represents historical conditions. The tool also features two scenarios to account for uncertainty in modeled future conditions and to capture the range of potential future conditions that projects might be subjected to. The Base epoch is not split into different scenarios since it is not based on projections. The CWVAT measures exposure for eight hazards for each epoch (timeline) and each scenario (low or high projections), also referred to as epoch scenarios, based on the exposure to various indicators associated with each hazard.

The tool sources indicators from raw data inputs that are spatially varied. In calculating indicator and exposure scores, data are aggregated to a standardized U.S. Geological Survey (USGS) Hydrologic Unit Code 8-digit (HUC-8) watershed resolution. The spatial extents covered in the tool include the contiguous U.S., Alaska, and Hawaii, enabling comprehensive analysis across these regions.

The CWVAT measures the exposure for each watershed (HUC-8) by using a "z-score." This score shows how much the exposure to a hazard (like extreme precipitation) in one watershed differs from the median of all watersheds across the U.S. A z-score can be positive (above the median), negative (below the median), or zero (the median). For example, if a watershed has a z-score of +2.0 for exposure to extreme precipitation, it means that this watershed's exposure is two standard deviations above the exposure of the median watershed in the U.S., signifying increased exposure. The exposure score (z-score) for each hazard and epoch scenario is based on the z-score of each indicator associated with a specific hazard.

Each hazard in the CWVAT was assessed by a series of indicators, which are measurable data points that capture aspects of each hazard that are salient for decision-making. Authoritative datasets spanning each domain were not available for some potential indicators (e.g., projected lightning strikes for wildfire). Indicators used to assess each hazard are described in Table AA-3 below. In the CWVAT Guides & Fact Sheets section, fact sheets are available to describe the way each indicator was calculated, the datasets used, and guidelines for interpreting indicator values.

*Table AA-3. Hazard Indicators*

| Hazard | Supporting Indicators |
|---|---|
| Drought | Flash drought frequency, drought year frequency, aridity, consecutive dry days, mean annual runoff |
| Coastal Change | Coastal flood extent, coastal erosion |
| Riverine Flooding | Riverine flood extent, flood magnification factor, maximum 1-day precipitation, maximum 5-day precipitation, extreme precipitation days |
| Extreme Temperature | Days above 95 °F, 5-day maximum temperature, high heat days, frost days, high heat index days |
| Energy Demand | Heating degree days, cooling degree days, 5-day minimum temperature, 5-day maximum temperature |
| Wildfire | Fuel abundance, ignition rate, fire season length, flash drought frequency |
| Land Degradation | Fire season length, aridity, soil loss, coastal erosion, permafrost hazard potential |
| Historical Extreme Conditions | Tornado frequency, tropical cyclone destructive winds, tropical cyclone frequency, tropical cyclone maximum average precipitation, historical drought frequency, ice jam occurrence, wildland urban interface, ice storms occurrence |

For this project, the comparative analysis and project area analysis in the CWVAT were used to analyze 3 HUC basins covering the project area: Middle Red-Coushatta (11140202), Lower Red-Lake Latt (11140207), and Lower Red (08040301).

The basins were analyzed in the comparative analysis for All Hazards. The dominant hazard for all 3 HUC basins is Historical Extreme Conditions. The Exposure Score metric shows the spread of exposure scores for the selected watersheds and epoch scenario. Figure AA-10 shows a box plot with the minimum and maximum values across all watersheds. The overlaid points represent where the selected watersheds fall within the entire population of exposure score values.

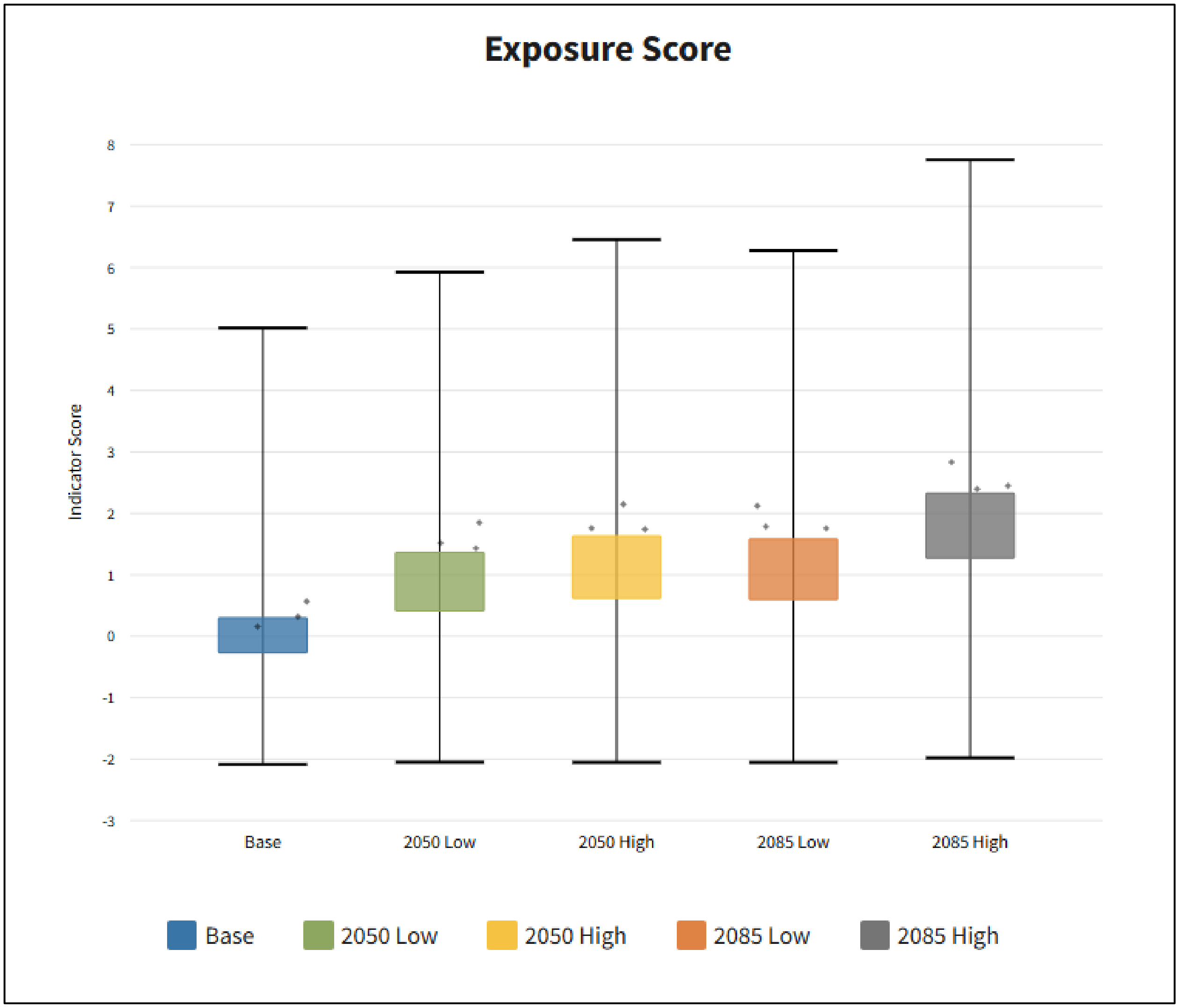



*Figure AA-10. Exposure Score Box Plot*

The project area analysis section of the tool was used to analyze how the basins are impacted by different factors. The exposure scores, hazard scores, and normalized indicator values presented in the project area analysis are calculated using a z-score methodology. z-score values are calculated using the median of all HUC-8 watershed values in the base epoch, with the standard deviation calculated across all epoch scenarios. The sign of the normalized value indicates whether a site is either more (positive values) or less (negative values) exposed than the median (or "typical") watershed and the magnitude indicates by how much. The summary of the exposure is shown in Figure AA-11.

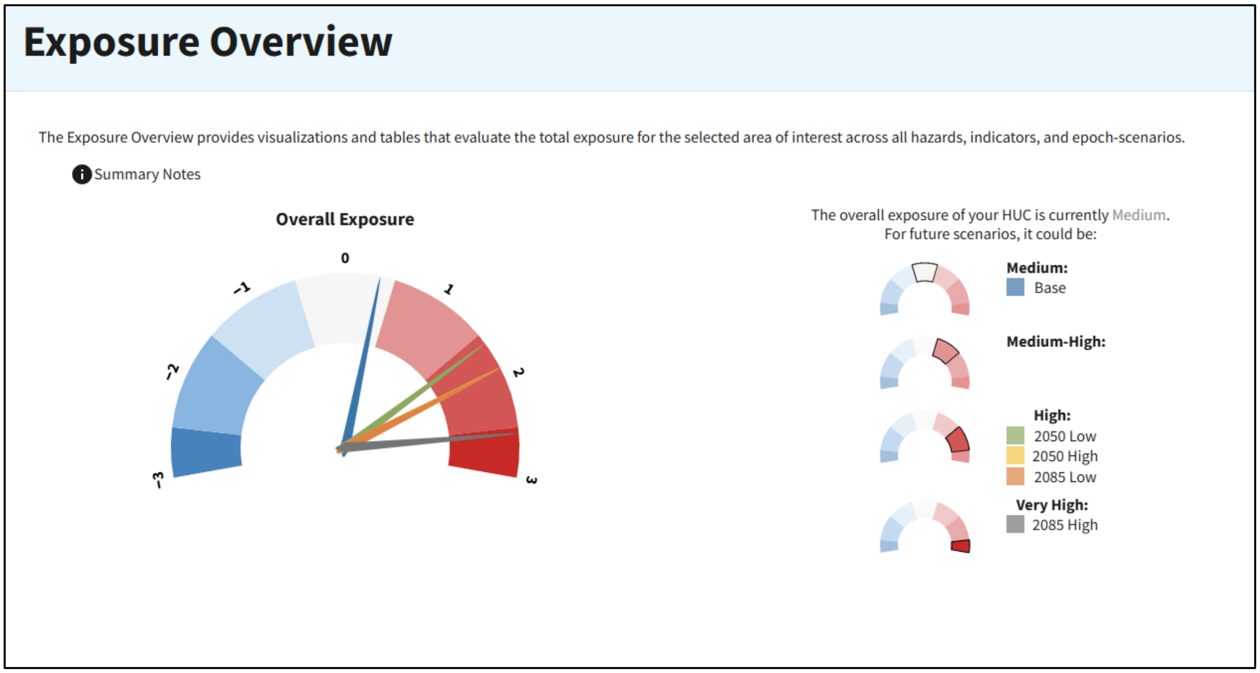



*Figure AA-11. Exposure Overview Summary from Project Area Analysis*

The overall exposure scores range from 0.35 to 2.56 across all epoch scenarios. These exposure scores place the project area in the 70th to 90th percentiles across all epoch scenarios.

*Table AA-4. Exposure Overview Table*

| Epoch Scenario | Z-Score | Percentile | Change from Base | Change from 2050 |
|---|---|---|---|---|
| Base | 0.35 | 78% | - | - |
| 2050 Low | 1.60 | 83% | 1.25 | - |
| 2050 High | 1.88 | 84% | 1.53 | - |
| 2085 Low | 1.89 | 85% | 1.54 | 0.29 |
| 2085 High | 2.56 | 84% | 2.21 | 0.68 |

The CWVAT evaluates a project area's total exposure to eight hazards. The hazard exposure scores across the five evaluated epoch scenarios for all eight hazards are shown in Figure AA-12.

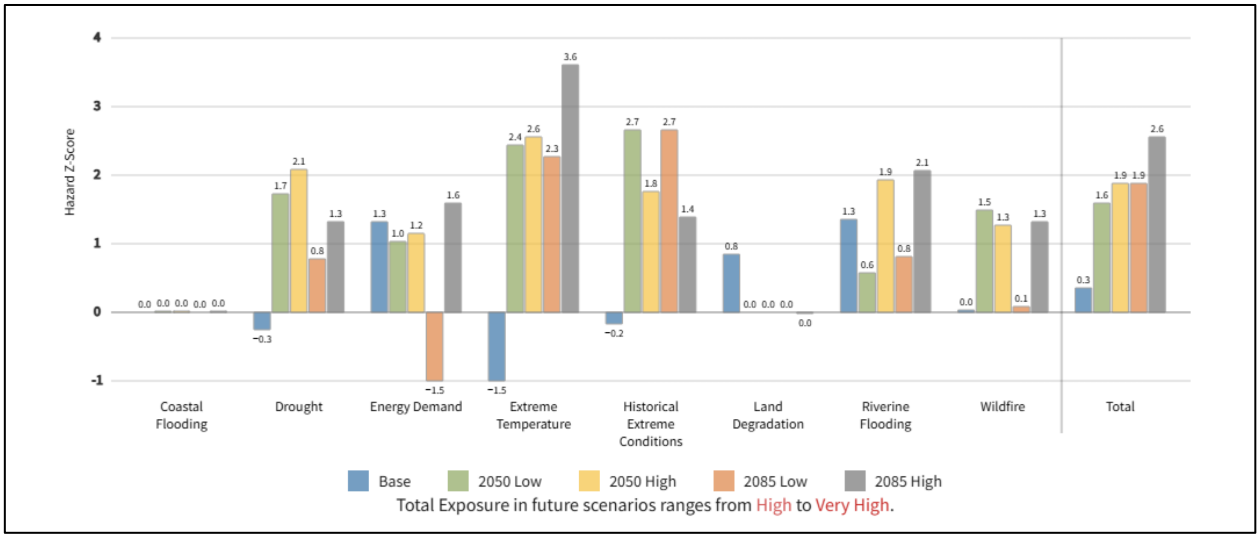



*Figure AA-12. Exposure Overview Hazards*

The two hazards with Base levels that are higher than average are Historical Extreme Conditions and Riverine Flooding. The Historical Extreme Conditions of the basins are currently High, and the Riverine Flooding exposure is currently Medium-High.

The historic extreme weather conditions hazard generally highlights the historic threat of extreme weather events such as tropical cyclones, tornadoes, convective storms, hail, ice storms, and ice jams. These events have historically been destructive to communities, infrastructure, and ecosystems.

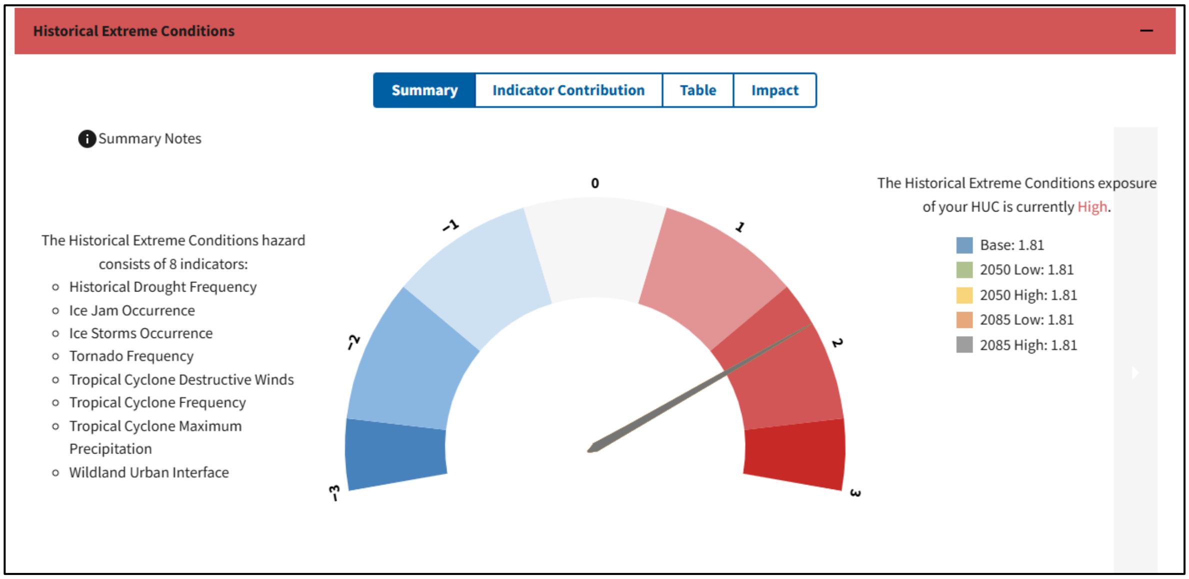



*Figure AA-13. Historical Extreme Conditions Summary*

The Historical Extreme Conditions Hazard consists of 8 indicators: Historical Drought Frequency, Ice Jam Occurrence, Ice Storms Occurrence, Tornado Frequency, Tropical Cyclone Destructive Winds, Tropical Cyclone Frequency, Tropical Cyclone Maximum Precipitation and Wildland Urban Interface. Figure AA-14 below shows the extent to which each indicator contributes to the hazard exposure score for all epoch scenarios.

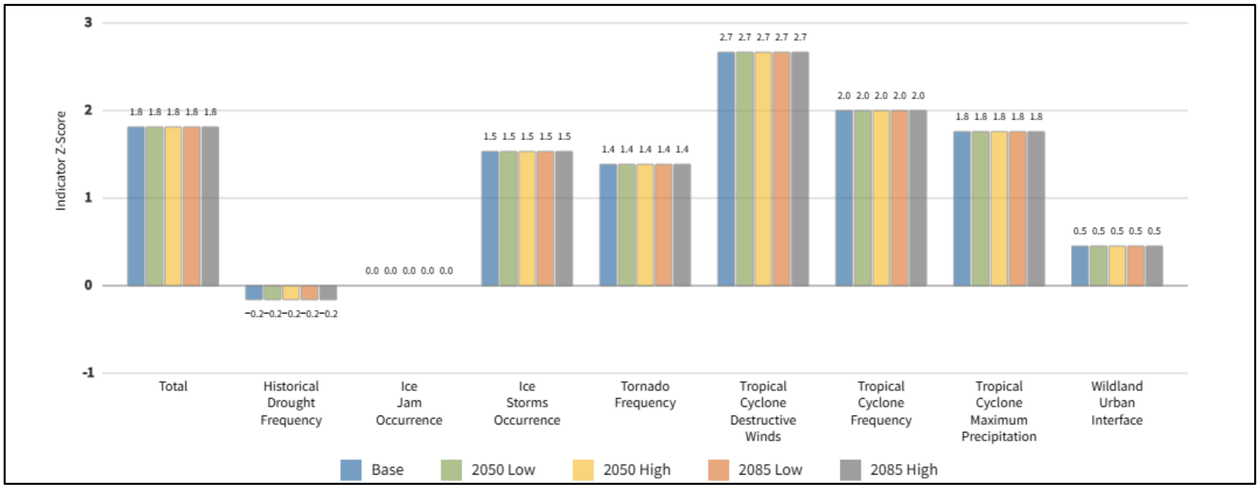



*Figure AA-14. Historical Extreme Conditions Indicator Contribution to Hazard Exposure Across All Epoch Scenarios*

* The exposure scores for Historical Extreme Conditions are 1.81 for all epoch scenarios, which places the project area in the 90th percentile across all epoch scenarios.

*Table AA-5. Historical Extreme Conditions: Hazard Exposure Across All Epoch Scenarios*

| Epoch Scenario | Z-Score | Percentile | Change from Base | Change from 2050 |
|---|---|---|---|---|
| Base | 1.81 | 90% | - | - |
| 2050 Low | 1.81 | 90% | 0.00 | - |
| 2050 High | 1.81 | 90% | 0.00 | - |
| 2085 Low | 1.81 | 90% | 0.00 | 0.00 |
| 2085 High | 1.81 | 90% | 0.00 | 0.00 |

The riverine flood hazard generally highlights the increased risk of flooding due to an increase in precipitation and flood magnitude. Increase in flood risk can result in more frequent flooding and an increase in areas impacted by flooding. Extreme flood events can have devastating impacts to communities, threaten life safety, and damage critical infrastructure. Increased flooding also increases the need for emergency response. Additionally, floods increase destruction to riverine navigation due to unsafe water conditions and can threaten ecosystem restoration efforts.

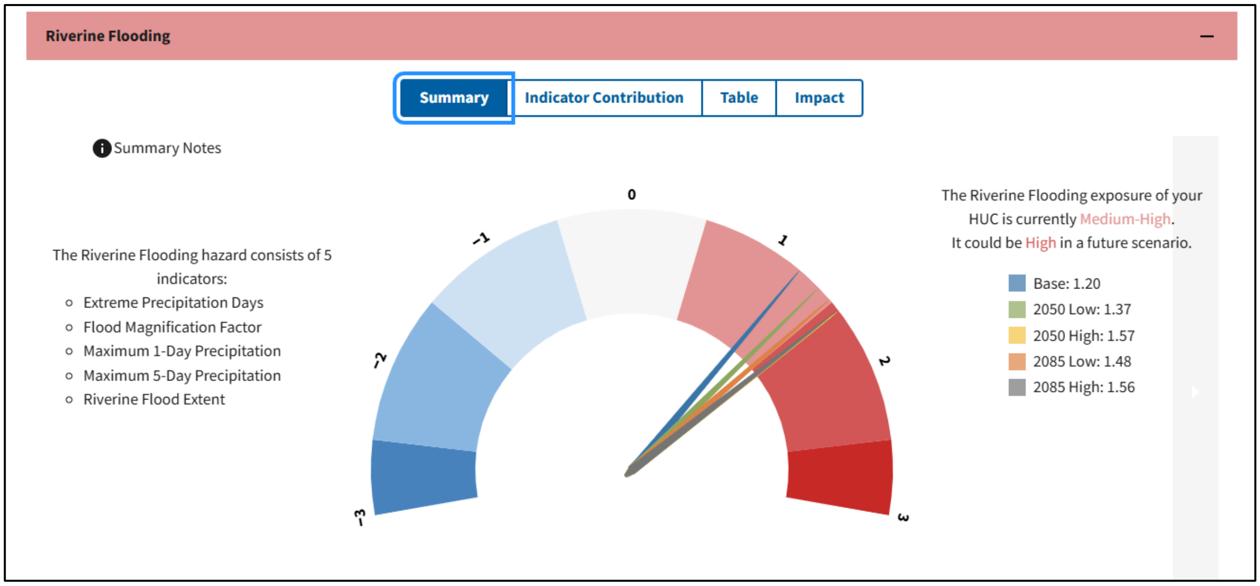



*Figure AA-15. Riverine Flooding Hazard Exposure Across All Epoch Scenarios*

Riverine Flooding hazard consists of 5 indicators: Extreme Precipitation Days, Flood Magnification Factor, Maximum 1-Day Precipitation, Maximum 5-Day Precipitation and Riverine Flood Extent. Figure AA-16 below shows the extent to which each indicator contributes to the hazard exposure score for all epoch scenarios.

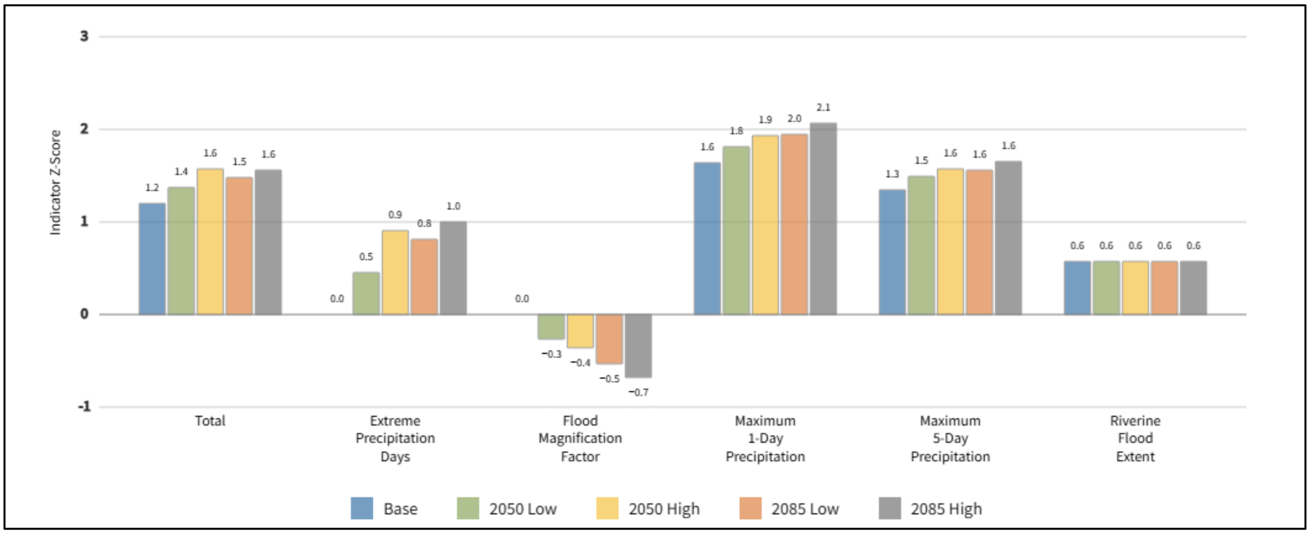



*Figure AA-16. Indicator Contribution to Riverine Flooding Hazard Exposure Score Across All Epoch Scenarios*

The exposure scores for Riverine Flooding range from 1.20 to 1.57 across all epoch scenarios. These exposure scores place the project area in the 60$^{th}$ to 80$^{th}$ percentiles across all epoch scenarios.

*Table AA-6. Riverine Flooding Hazard Exposure Across All Epoch Scenarios*

| Epoch Scenario | Z-Score | Percentile | Change from Base | Change from 2050 |
|---|---|---|---|---|
| Base | 1.20 | 89% | - | - |
| 2050 Low | 1.37 | 85% | 0.17 | - |
| 2050 High | 1.57 | 84% | 0.37 | - |
| 2085 Low | 1.48 | 82% | 0.28 | 0.10 |
| 2085 High | 1.56 | 62% | 0.36 | -0.01 |

**Conclusion**

The purpose of the JBJ Waterway study is to develop and evaluate alternatives to improve the navigational transportation within the JBJ Waterway and the five lock and dams. To gain a sense of how conditions might change in the future, historic and projected data were analyzed using USACE tools to investigate how projected meteorological inputs may impact future streamflows in the JBJ Waterway.

According to the literature reviewed, warmer weather is expected in the future. There was a consensus that the temperature is expected to increase in the study region. Little consensus exists in projected trends of future precipitation in the study region, and streamflow is projected to mildly decrease in the study region.

Most of the gages in the study region do not have evidence of nonstationarity or statistically significant trends in the peak annual flow through the period of record. At the Shreveport gage, the data show a statistically significant decrease in peak annual flow for the period of record (1935–2025), and one test shows evidence of a nonstationarity in 1976. However, this is not considered a strong nonstationarity since it is not supported by the results of the other nonstationarity tests. Evaluations of future extreme precipitation and streamflow generated using the CHAT indicate future increases in both 3-day maximum precipitation and annual maximum mean monthly precipitation when RCP 4.5 or 8.5 is assumed. The CWVAT indicated that Historical Extreme Conditions and Riverine Flooding are the main drivers of the vulnerability score for the project area. Table AA-7 indicates potential residual risks for navigation project features due to the projections made, along with a qualitative rating of how likely those residual risks are to materialize and undermine project features resulting in harm to the study area.

*Table AA-7. Residual Risk Due to Projected Hydrology*

| Project Feature | Trigger | Hazard | Harm | Qualitative Likelihood[1] | Justification of Likelihood Rating |
|---|---|---|---|---|---|
| 12-FT JBJ Waterway channel | Decreased streamflows, higher temperatures | Lower water levels in channel | Low water levels would prevent barges from traveling through the channel, unless they have lighter loads | Unlikely | Streamflow is projected to mildly decrease, strong consensus that temperature will increase |

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