Three Rivers Southeast Arkansas Study

Appendix B: Hydraulics and Hydrology

THREE RIVERS SOUTHEAST ARKANSAS

Introduction

The Three Rivers Southeast Arkansas Feasibility Study (Three Rivers Study) is being conducted by the U. S. Army Corps of Engineers (USACE) to recommend modifications to the McClellan-Kerr Arkansas River Navigation System (MKARNS) that would provide long-term sustainable navigation and promote the continued safe and reliable economic use of the MKARNS.

Study Authority

Section 216, Flood Control Act of 1970 (Public Law 91-611) authorizes a feasibility study due to examine significantly changed physical and economic conditions in the Three Rivers study area. The study will evaluate and recommend modifications for long-term sustainable navigation on the MKARNS.

Study Purpose

There is a risk of a breach of the existing containment structures near the entrance channel to the MKARNS on the White River. During high water events, Mississippi backwater can create significant head differentials between the Arkansas and White rivers. The existing containment structures in the isthmus are subject to damaging overtopping, flanking and seepage flows that could result in a catastrophic breach and failure of the system. The uninhibited development of a breach, or cutoff, has the potential to create navigation hazards, increase the need for dredging, and adversely impact an estimated 200 acres of bottomland hardwood forest in the isthmus.

Based on the Section 216 authority, the study is investigating alternatives that would minimize the risk of cut off development, including reducing the cost of maintence associated with preventing cutoff development, while minimizing impacts to the surrounding ecosystem.

Non-Federal Sponsor

The Arkansas Waterways Commission is the non-federal sponsor for the Three Rivers Southeast Arkansas Study. An amended feasibility cost-sharing agreement was executed in June 2015.

Recommended Plan

The recommended plan consists of a newly constructed 2.5-mile long containment structure at an elevation of 157 feet above mean sea level (ft msl) that would begin on natural high ground just south and west of the existing Melinda Structure located on the south side of Owens Lake. It would continue east and cross the Melinda Headcut south of the existing Melinda Structure. From there, it would head northeast and connect to the existing containment structure north of Jim Smith Lake. It continues to follow the existing soil cement containment structure alignment terminating at the existing Historic Cutoff Structure. The recommended plan also includes a relief opening at the Historic Cutoff to an elevation 145 ft msl regardless of the width. In addition, the existing Melinda Structure would be demolished in place and the debris would be pushed into the deep scour hole at the top of the head cut. Finally, adding an opening in the existing Owens Lake structure between Owens Lake and the White River would prevent water from backing up into Owens Lake, which would impact the bottomland hardwood forest. The opening would be designed to allow fish passage into Owens Lake.

Three Rivers: Mississippi River, Arkansas River, and White River
Two-Dimensional Hydraulic Modeling

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1 Study Background and Introduction

1.0 Problem statement

A natural cutoff (or interconnecting, uncontrolled channel between two water courses) historically existed between the lower White River and the Arkansas River. The “natural cutoff” resulted from hydrologic interactions near the confluence of three river systems; the Arkansas, Mississippi and White rivers. Over time this interaction promoted overland erosion creating a free flowing channel connecting the Arkansas and White rivers. During development of the McClellan-Kerr Arkansas River Navigation System (MKARNS) in the mid-1960s, the cutoff was identified as an impediment to the reliability of navigation, and USACE closed the cutoff by constructing a non-overtopping dike named the Historic Closure Structure.

The Historic Closure Structure has increased head differentials between the White River and the Arkansas River during overtopping events across the isthmus, the narrow strip of land that separates the Arkansas River from the White River, resulting in higher energy differences and increased erosion. Subsequently, additional cutoffs have developed throughout the isthmus in effort to restore the natural hydrologic relationship between the systems. This geomorphic process continues to threaten the MKARNS with increasing and more frequent maintenance costs. The uninhibited development of these cutoffs has the potential to create navigation hazards, increase the need for dredging, and adversely impact an estimated 200 acres of bottomland hardwood forest in the isthmus between the Arkansas and White rivers.

The Dale Bumpers White River National Wildlife Refuge (Refuge) contains approximately 160,000 acres of prime bottomland hardwood habitat in the floodplain of the lower White River and is adjacent to the navigation channel. Bottomland hardwood forests flood frequently and are highly sensitive to variations in land and water elevation. An uncontrolled breach through the isthmus would create a head cut (an abrupt degradation of the channel bed) that would proceed up the White River. This in turn would cause bank caving along the main channel and subsequent head cutting up tributaries and cause oxbow lakes to lose both form and function. If continued unchecked, water tables could decline, which would disconnect bottomland hardwoods from the groundwater table. Repairing the breach through the isthmus would stop further head cutting up the White River and would eventually cause aggradation of the channel bed, but the rate of aggradation might be insufficient to catch up with the head cutting nick point before bank caving and loss of some oxbows occurs.

1.1 Site Description

The study focuses on providing environmental benefits to the bottomland hardwoods, wetlands, and oxbow functions in the isthmus and on Refuge while preserving the integrity and long-term dependability of the entrance channel to the MKARNS (Figure 1-1 and 1-2). Tows traveling to the Arkansas River from the Mississippi River enter the MKARNS at the White River's mouth at Montgomery Point Lock and Dam, travel 10 miles up the White River to Lock 01 (Norrell), lock into the Arkansas Post Canal, and navigate the canal to the Arkansas River.

Figure 1-1: General Map of MKARNS Entrance Channel Location

The immediate study area is loosely bounded on the north by Phillips and Desha counties, on the east by levees and the Mississippi River, on the west by levees, and on the south just downstream of the Arkansas River confluence with the Mississippi River (Figure 1-2).

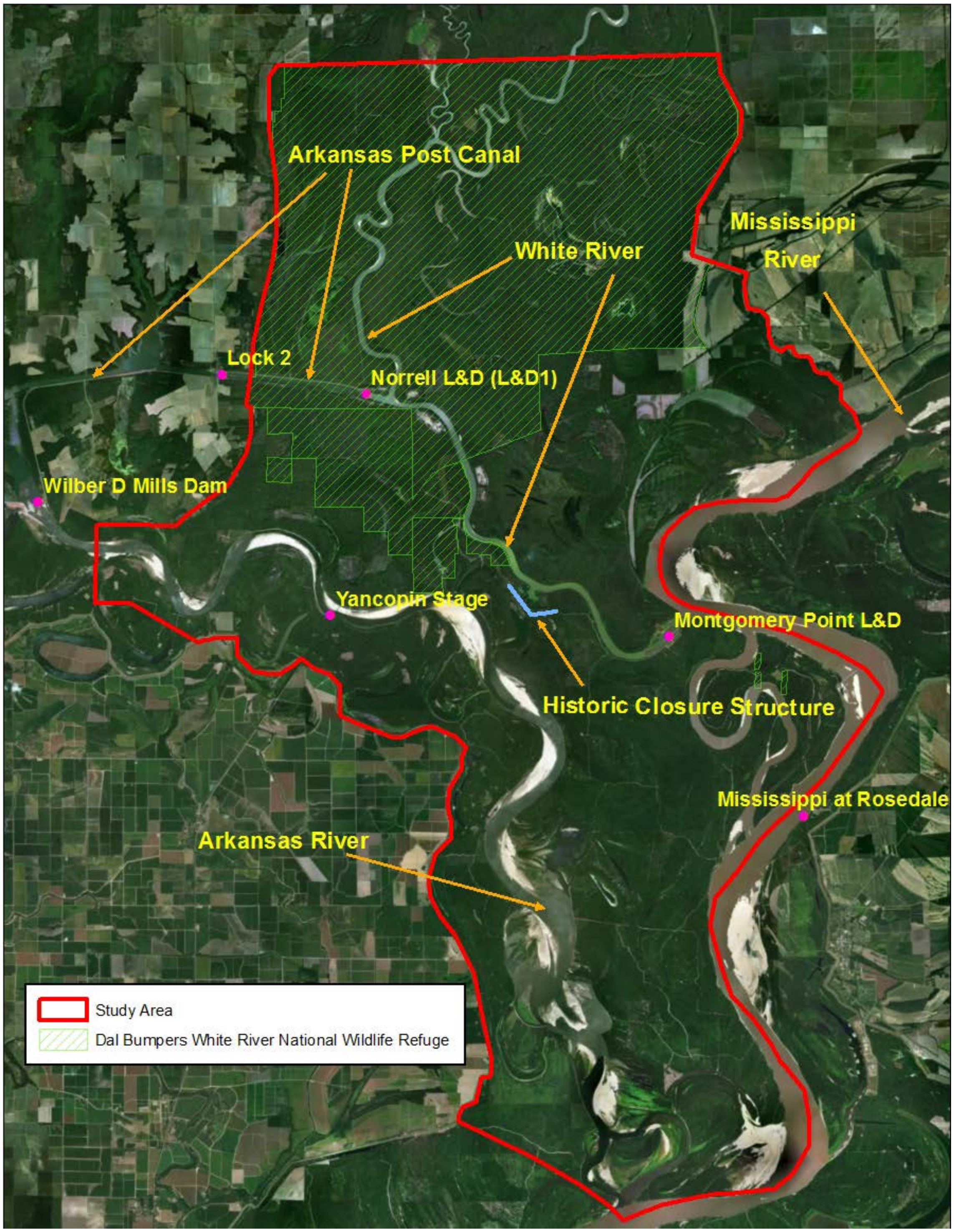

Figure 1-2: Study Area

1.3 History

The confluence of the Arkansas and White rivers with the Mississippi is an area of complex and evolving flow patterns. Early European explorers and cartographers[1] noted that there was a channel connecting the lower White and Arkansas rivers. The channel was either unnamed or simply called a cutoff because vessels could use it as a shortcut depending on their destination. Today, people refer to it as the Historic Cutoff. The Historic Cutoff was deep enough so that it connected the lower Arkansas and White rivers, and the land between the cutoff and the Mississippi came to be known as Big Island (360 square miles in size).

When building the MKARNS, designers were concerned about the high cost of stabilizing the lower Arkansas River. To maintain a stable navigation channel, many stone structures were placed along the Lower Arkansas with varying degrees of success. To avoid the tricky and expensive proposition of challenging the inherently unstable deltaic channel (Saucier, 1994), engineers constructed a canal connecting the Arkansas to the more stable lower White River.

The Historic Cutoff, however, presented two possible problems. For one, dangerous cross currents in the White River were reported when flow passed through the cutoff between the two rivers. Additionally, the Arkansas carries more sediment than the White, and MKARNS designers concluded that the Historic Cutoff contributed sediment to the White River entrance channel at a relatively high rate. Furthermore, they believed that the Historic Cutoff was a geologic relic. So, in 1964 the USACE closed the Historic Cutoff to eliminate navigation hazards and lower dredging costs (Franco, 1967).

This arrangement worked well until 1973, the first year of unusually high water on the Mississippi following construction of the MKARNS. The Arkansas City Gage reached 47.6 feet, Annual Exceedance Probability, AEP, of 38%, on May 13, 1973 and the Helena Gage reached 50.2 feet, AEP of 25%, three days earlier. See Figure 2-1 for gage locations relative to study area. Afterwards, a new head cut appeared on the Arkansas, running up through the isthmus. Over the next two decades, the head cut grew whenever Mississippi River stages at the mouth of the White produced backwater in volumes high enough to push flow across the isthmus into the Arkansas. Little Rock District personnel became alarmed at the head cut's rate of progression in the 1980s.

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[1] Hutchins, Thomas, *The Western Parts of Virginia,* (1778); Collot, George Henri Victor, *A Journey in North America* (1796); Cramer, Zadok, *The Navigator,* (1817). This span of maps brackets any possible impacts of the 1811-1812 New Madrid earthquakes.

The head cut channel became known as the Melinda Corridor. In 1989, USACE tried to stop the head cut by constructing the Melinda Weir (Melinda Structure)[2], which was the first phase of a three structure plan that also included the Owens Lake Weir (Owens Lake Structure[3]) (completed in 1992) and the Containment Structure. Before the project's construction finished, Melinda suffered damage, and the District had to rebuild the structure twice. By 2000, Melinda had been severely damaged and had to be repaired three more times. All the while, the Melinda Corridor continued to widen and deepen.

By 2002, the Arkansas River (House Bend) migrated north to capture Jim Smith Lake. After this new flow path opened, the Containment Structure near Jim Smith Lake's north end had to be reinforced and repaired. In 2004, USACE responded by installing an experimental Geotube™ structure to protect the Containment Structure. In the winter of 2005, the Geotubes breached, and Melinda was damaged again but the structure survived. Melinda soil cement repair completed in 2012 from damages from at least two prior floods, 2008 and 2011. Melinda and Jim Smith were repaired again due to flanking erosion that threatened to bypass the structures. Maintenance costs have risen as new failure paths have developed leading many observers to suspect the Historic Cutoff was not a geologic relic, but rather an important element in governing water surface dynamics at the confluence of the three rivers.

1.4 Mechanism of Cutoff

Isthmus erosion happens in two ways: lateral migration of rivers, and overtopping of a land mass by a river flooding into another river's channel. Locations where overland sheet flows converge receive the greatest damage. When sheet flows converge, they channelize and can result in system failure, which is defined in this study as an uncontrolled channel (cutoff) through which flows exchange between the White and Arkansas rivers, and significantly impede navigation to and from the Mississippi River.

1.4.1 Meandering

The first erosion mechanism, meandering, relates mostly to the Arkansas since the White maintains a stable plan form and Big Island separates the Mississippi from the isthmus. A limited geomorphic study of the lower Arkansas focused on the reach adjacent to the isthmus, identified the bank migration pattern as, primarily, downstream movement of the bends. Migration has eroded natural high ground alongside the river, making new cutoff paths possible primarily through ox-bow lakes on the isthmus (Pinkard, 2003). The Mississippi separation from

[2] Melinda Weir is also referred to as the Melinda Structure.
[3] Owens Lake Weir is also referred to as the Owens Lake Structure or Owens Weir.

the isthmus means that its migration does not directly impact the land mass; having said that, the westward movement of the Mississippi during the channel shortening period (in response to 1927 and 1937 floods) resulted in the Mississippi being able to contribute more flow into White for the cutoff formation process.

Overland Erosion

The second forming mechanism, overland erosion, is direct erosion from overtopping. The Mississippi is the primary source of overtopping flows, and the river will rise creating a backwater condition in the White and this backwater takes relief over the isthmus into the Arkansas. The westward movement of the Mississippi during the shortening period has increased the impact of the backwater condition by shortening the White by several miles (CESWL, Arkansas River - White River Cutoff Study (Ark-White Study), 2009) and relocating the mouth of the White about nine miles further upstream on the Mississippi. These changes allow the White to carry more flow between the Mississippi and the Cutoff, and allow the Mississippi to deliver high backwater conditions to the White River's mouth. Secondly, the Arkansas, more rarely, will overtop the isthmus into the White.

For simplicity, the conditions of overtopping can be described in two ways, White to Arkansas and Arkansas to White. The area begins overtopping when one river's water surface exceeds the Owens Lake Weir crest at elevation 145 feet. For the period of record 1970-1998, Figure 1-3 shows that for White and Arkansas River water surfaces below 137 feet, the Arkansas is higher than the White, but for conditions above 137 feet the White is more likely to overtop into the Arkansas (Mississippi controlled condition). Flow volume contributed to the system by the White is comparatively small. A major flood on the White might contribute enough flow to raise the Mississippi by one foot in the vicinity of the mouth of the White River.

The difference between water surface elevation on the White and the Arkansas is referred to as head differential, which is analogous to the energy input to the land surface as frictional and turbulence loses. Higher head differentials make water passing between the rivers more turbulent and violent. More turbulent water results in greater erosion and increases the risk of a cutoff forming.

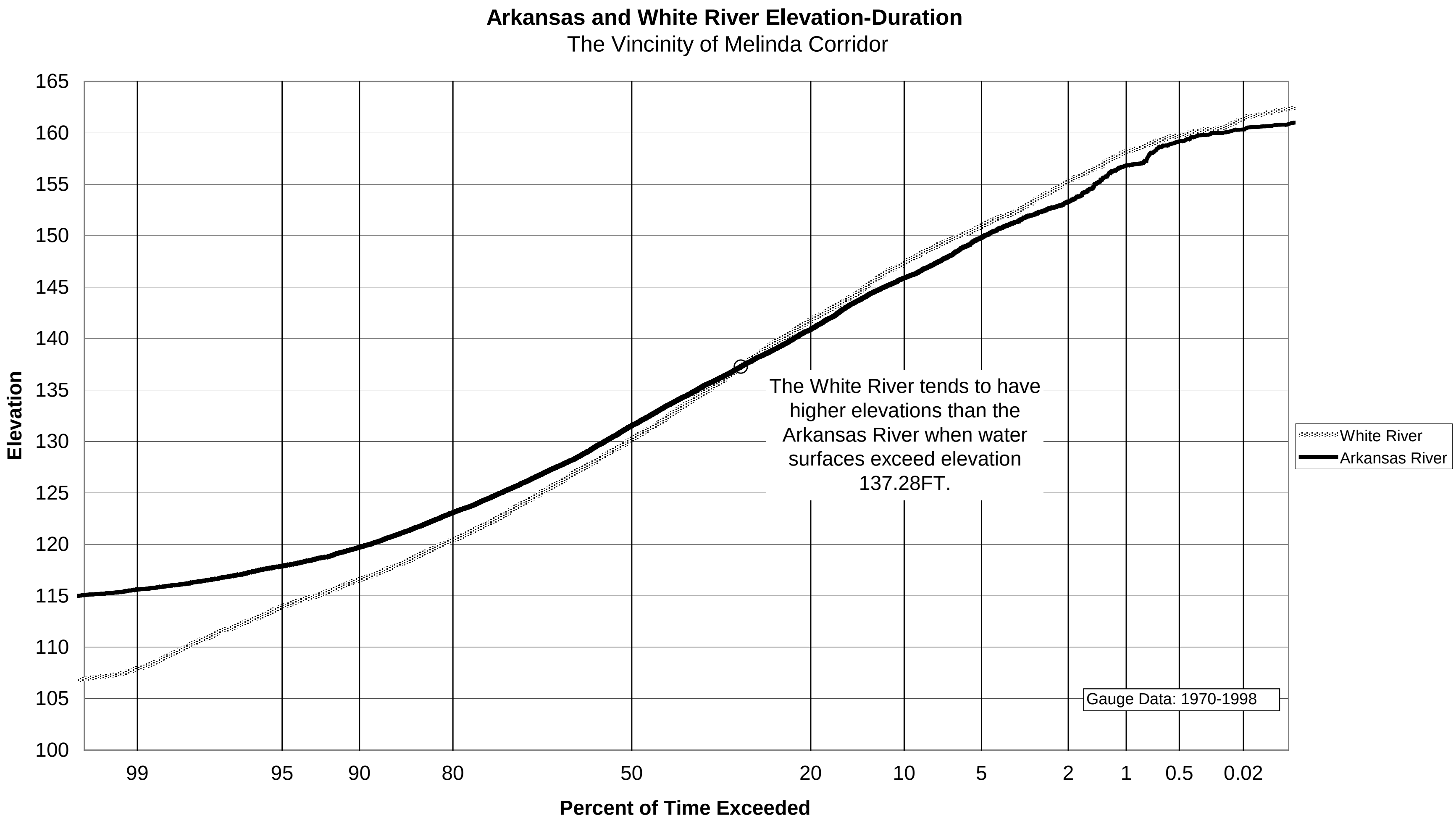

Figure 1-3: Elevation Duration in Vicinity of Melinda Corridor (CESWL, Arkansas River - White River Cutoff Study (Ark-White Study), 2009)

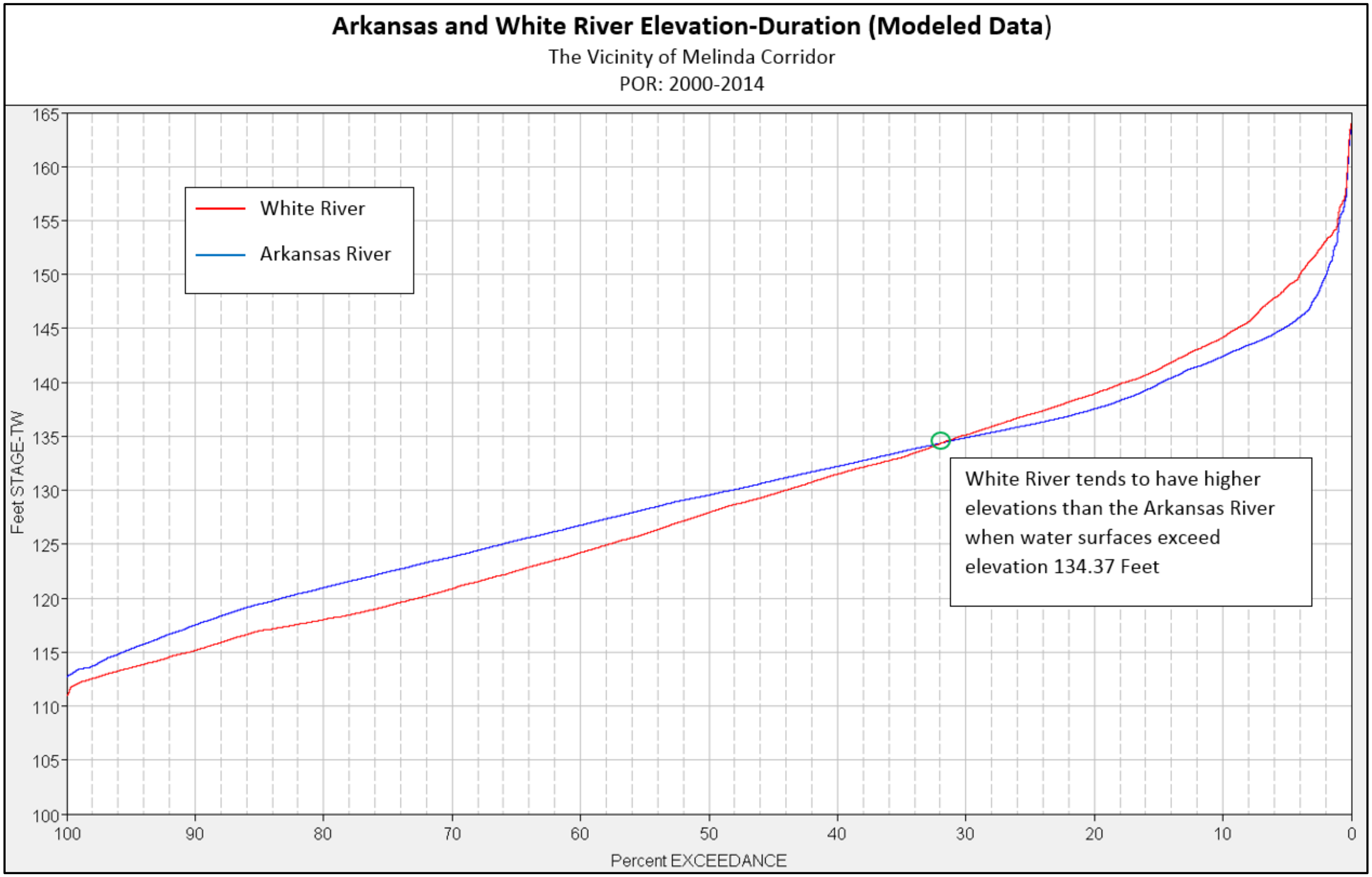

Figure 1-4: Modeled Elevation Duration in vicinity of Melinda Corridor (POR 2000-2014)

Severe damage has not been observed for events with head differentials less than four feet (CESWL, Arkansas River - White River Cutoff Study (Ark-White Study), 2009). Table 1-1 lists events where differentials exceeded the four foot threshold above elevation 145 feet across the Historic Closure Structure (modeled for period of record 2000 through 2014).

Table 1-1: White River Events Exceeding Elevation 145 Feet with 4 Feet Head Differential across Historic Closure Structure

Event	Start Date	End Date	Number of Days Exceeding
Apr-02	3/27/02 6:00	4/10/02 18:00	14.5
May-02	5/12/02 6:00	6/9/02 12:00	28.3
Mar-03	2/27/03 12:00	3/12/03 12:00	13.0
May-03	5/16/03 0:00	6/4/03 12:00	19.5
Dec-04	12/11/04 0:00	12/26/04 6:00	15.3
Jan-05	1/24/05 12:00	2/7/05 6:00	13.8
Apr-05	4/16/05 0:00	4/18/05 6:00	2.3
Jan-07	1/20/07 18:00	1/30/07 0:00	9.3
Feb-08	2/24/08 12:00	2/27/08 12:00	3
Mar-08	3/13/08 6:00	3/20/08 12:00	7.3
May-08	5/22/08 18:00	5/27/08 0:00	4.3
Jun-09	6/4/09 12:00	6/5/09 12:00	1
Nov-09	11/7/09 12:00	11/14/09 18:00	7.3
Feb-10	1/31/10 12:00	2/20/10 0:00	19.5
Apr-10	3/25/10 6:00	4/15/10 0:00	20.8
May-10	5/7/10 6:00	6/4/10 6:00	28.0
Mar-11	3/7/11 12:00	4/8/11 12:00	32.0
Apr-11	4/21/11 18:00	4/27/11 12:00	5.8
Jun-11	6/4/11 12:00	6/14/11 18:00	10.3
Dec-11	12/8/11 12:00	12/26/11 6:00	17.8
Mar-13	3/25/13 12:00	3/28/13 12:00	3
May-13	4/23/13 18:00	5/30/13 12:00	36.8
Jun-13	6/10/13 0:00	6/26/13 6:00	16.3
Jul-13	7/15/13 18:00	7/22/13 0:00	6.3

1.4.3 Failure Paths

A study completed by FTN Associates Ltd., published in 2000, ranks four major failure paths through the isthmus:

1) Melinda Channel Owens Lake Corridor by flanking or breaching of the Owens Lake control structure;

2) Melinda Channel Owens Lake slough via a breach through the containment structure;

3) LaGrues Lake Corridor with elements of the Owens Lake and or Melinda outflow channel being used in the failure path; and,

4) Jim Smith Lake Corridor.

Based on the study, the Melinda Channel Owens Lake Corridor is the most likely location for a cutoff because: 1) it is the current primary flow pathway between the Arkansas and White, 2) it presents the pathway of least hydraulic resistance, 3) it is the pathway with the most damage from existing flows between the two rivers, and 4) it presents the area with the potential to experience the greatest head differential coupled with high flow rates. Failure paths have since been reworked to account for new nick points, sinkhole-depressions, and the meandering of the Arkansas. As of 2016, the risk of failure list is ordered in the following manner:

1) Melinda Channel Owens Lake Corridor, by flanking or rupturing of the Owens Lake Weir, and the Melinda Weir;

2) Jim Smith Lake Corridor, the Arkansas River's House Bend's east by east-west movement captured the lake effectively making the Jim Smith Lake corridor the shortest, most damaged, and least hydraulically resistant flow path between the two rivers;

3) Historic Cutoff: Two sink holes have appeared in the Historic Closure Structure, one in September of 2014 and one in October of 2016. The cause of the sinkholes may be decaying organic material, settlement of fill used as road base, or removal of material by under seepage. Current remedial action consists of filling the sinkholes with loose gravel to road grade. Potential solutions to address the problem include: continue to fill sinkholes with gravel, perform geophysical survey to determine extent of void formation, and rehabilitation the structure. Geophysical surveys and geotechnical subsurface investigations will be completed during to the Preconstruction, Engineering and Design (PED) phase of this study;

4) Jim Smith Lake Historic Cutoff Corridor: A lengthy head cut and nick point has been identified moving through the woods from the Historic Cutoff toward Jim Smith Lake;

5) LaGrues Lake Corridor with elements of the Owens Lake and or Melinda outflow channel being used in the failure path: A nick point has developed here moving along a swale toward LaGrues Lake;

6) Melinda Channel Owens Lake slough via a breach through the containment structure; and,

7) Webfoot Lake: Nick points have developed along the east side of Webfoot Lake. Subsequent head cutting will move across Big Island and connect to the White River about 2 miles upstream of its confluence with the Mississippi.

1.5 Design Criteria

Six major design criteria were used in alternative formulation: 1) isthmus flow velocities, 2) hydraulic head differentials, 3) duration of head differentials, 4) location of overtopping, 5) duration of flooding, and 6) ensuring reliable navigation. Formulation of alternatives was based on a combination of design criteria.

The magnitude and duration head differentials during flow exchanges are measured in two primary corridors: along the Historic Cutoff and in the Melinda Corridor (Figure 1-5). Differentials are measured as variations in modeled water surface elevation at the confluence of the White and Arkansas for each respective corridor, and the duration of absolute head differentials was measured above elevation 145 feet when overtopping occurs under existing conditions.

The Hydrologic Engineering Center River Analysis System (HEC-RAS) program can produce georeferenced gridded hydrologic velocity maps of an entire area. Velocity maps are extremely useful in pinpointing locations on the isthmus where scour is most likely to occur, and identifying potential scour locations increases the effectiveness of alternative formulation.

Controlling the locations of overtopping events would involve armoring the relief channel(s) against erosion and could consist of multiple step-down structures that minimize head differentials across each structure. As noted previously, severe damage has not been observed for events with head differentials less than four feet so reducing the head differential below this threshold or minimizing the duration of damaging head differentials would reduce head cutting erosion.

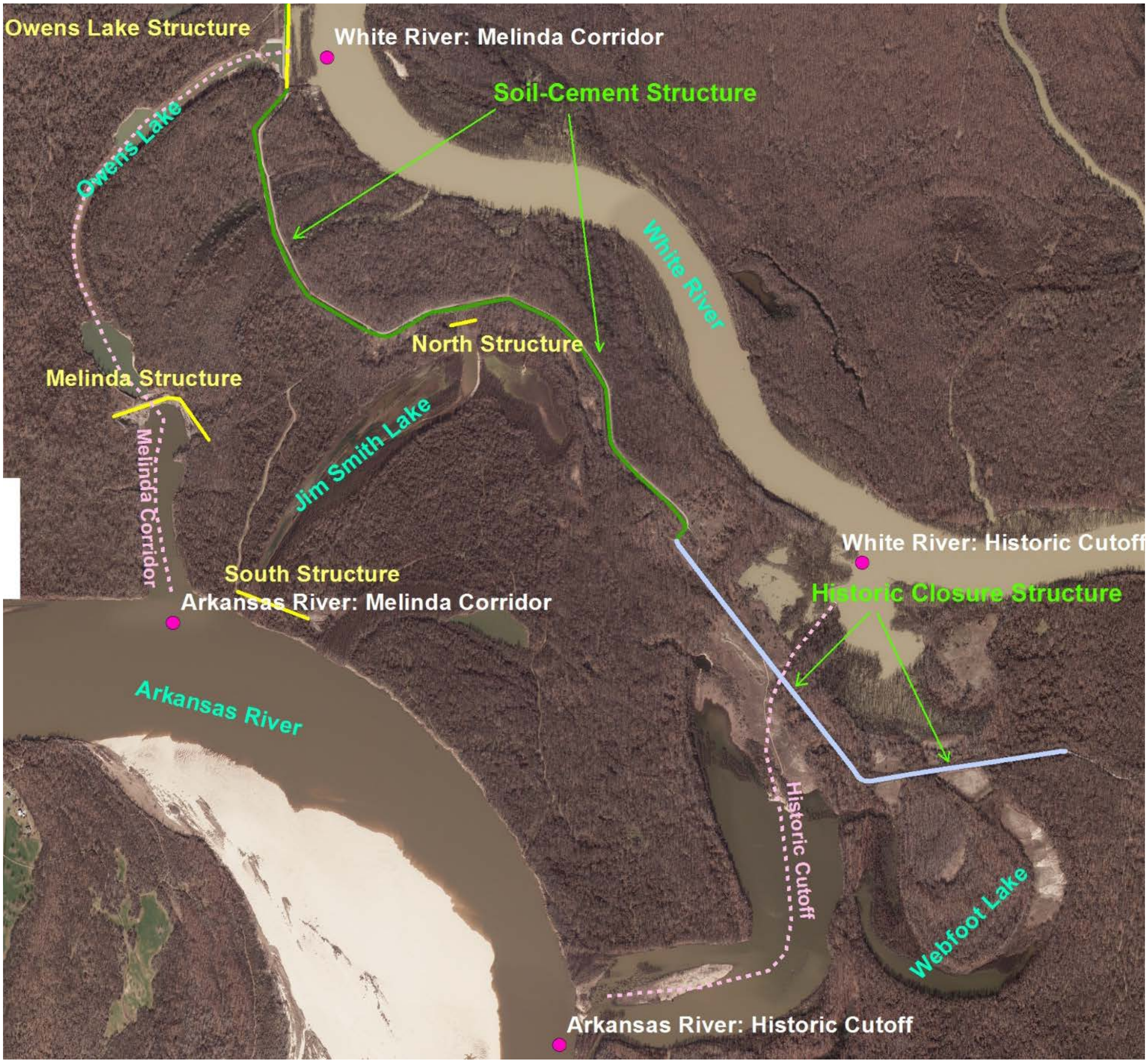

Figure 1-5: Measured Head Differentials

Environmental benefits for terrestrial and aquatic habitat health, form, and function are directly related to the timing and location of flooding. For aquatic habitat, several stage duration analyses were performed at selected locations to determine potential changes in oxbow recharge, fish passage, and in-channel changes across the alternatives. Terrestrial habitat and bottomland hardwood health depends on overland flood duration and location. In addition to elevation duration analysis, team hydrologists used HEC-RAS 5.0.1 to develop “percent time inundated” grids, based on the growing season starting on 15 March ending on 15 November for each alternative. Grids were used to compare how each alternative would affect flood duration on the Refuge when compared to existing conditions. This was very useful in pinpointing locations where hydrology could change the most for each alternative.

The final design consideration was the impact of cross-currents on navigation. Specific configurations of alternatives could affect the navigability of the shipping channel. Two-dimensional mathematical models can estimate in channel velocities, but they do not capture variables such as tow boat capabilities, barge number and configuration, and ship captain experience. Since the selected plan includes a relief channel, a ship tow simulator, which models factors such as tow capabilities and captain experience, may need to be completed to ensure that a relief channel would not create dangerous cross-currents.

1.6 Description of Alternatives

Existing and Future without Project Conditions: With respect to hydrologic analyses, these two scenarios are identical. The future without project condition defines the most likely state of the study area if action is not taken as a result of this study. Traditionally, future conditions relate to a problem where Federal action has not taken place; however, this study relates to a continuation of piecemeal repairs and rehabilitation of existing containment structures to contain head cutting and maintain navigation. Appendix A (Economics) contains detailed discussion of the future without project condition.

Alternative 1 Containment Structure at Elevation 157 feet: The Containment Structure (Figure 1-6), would be approximately 2.5 miles long and begin on natural high ground just south and west of the existing Melinda Weir located on the south side of Owens Lake. It would continue east and cross south of the existing Melinda Weir and then head northeast and connect to the existing soil cement containment structure north of Jim Smith Lake. From there, it would follow the existing soil cement alignment and terminate at the Historic Closure Structure. Alternative 1 would incorporate existing natural high ground, which would decrease construction costs and minimize disturbance to the terrain and natural hydrology of the Refuge. It would restore form and function to oxbow lakes in the isthmus while providing a long-term solution for reducing the risk of a cutoff between the Arkansas and White rivers by reducing the frequency, duration, location, and damaging head differentials of overtopping events. Alternative 1 would include a relief channel ranging from 500 feet to 1,000 feet wide at elevation 145 feet through the Historic Closure Structure. This is the current elevation that the White and Arkansas exchange flow through the Melinda Corridor. The relief channel would further reduce head differentials across the isthmus, but may create strong cross-currents in the shipping lane at widths greater than 500 feet.

Currently, Owens Lake connects to the White River at elevation 145 FT over Owens structure and connects to the Arkansas River at elevation 140 FT over the Melinda Structure. These current elevations give the Arkansas River greater influence on Owens Lake hydrology then the White River. Alternative 1 includes the elimination of the

Melinda Structure and the addition of a containment structure at a higher elevation of 157 FT just south of the current Melinda Structure location. This will reduce fish passage into Owens Lake and this will shift the hydrological influence to the White River and, assuming no leakage through Owens Structure or the new containment structure, will raise Owens Lake maximum elevation from 140 FT to 145 FT. The additional 5 feet in lake elevation has the potential to flood over an additional 100 acres of bottomland hardwood forest. Several free-span openings ranging from 20 feet to 30 feet wide at 4 feet to 6 feet tall were modeled through Owens Weir. See ATTACHMENT F for modeling effort and results.

During the 2009 Ark-White Study, engineers optimized the elevation of the structure at 157 feet and the Three River PDT concurred that this is an optimal height. Currently, there is an average observed difference of 0.6 feet between Lock and Dam 1 (LD01) and Montgomery Point Lock and Dam (MPLD) when MPLD is above elevation 157 feet. The maximum difference is 1.7 feet and the minimum difference is 0.7 feet. Because of this small drop in water elevation between the two locks, elevation records at MPLD (1961 through 2017) were used to calculate the frequency and duration of water elevation for Alt.1 containment structure. Exceedance duration is 1.1 percent with an annual exceedance probability (AEP) of 22 percent at elevation 157 feet.

Observed gage records at MPLD and Yancopin were used to determine head differentials during an event exceeding or overtopping elevation 157 feet. During the overtopping event, AEP of 22 percent, head differentials are two feet and less 85 percent of the time with an average of 1.4 feet and a maximum of three feet. This is because the Mississippi has had time to back up on both sides, Arkansas and White, of the containment structure prior to overtopping. To eliminate structural damage due to toe erosion, the Alt. 1 containment structure would be entrenched in locations where the height of the structure is less than three feet.

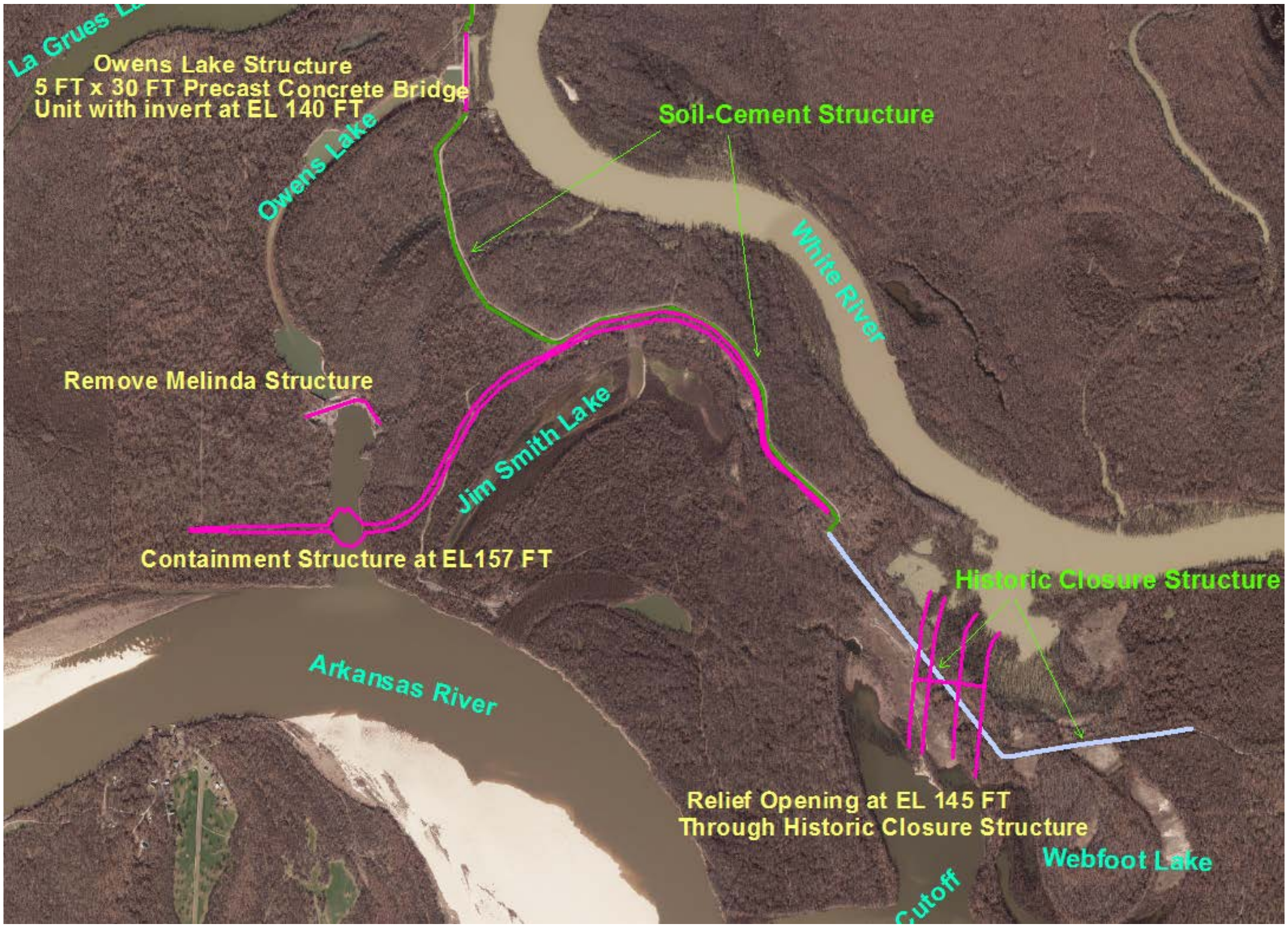

Figure 1-6: Alternative 1 Containment Structure at Elevation 157 Feet with Relief Channel

Alternative 2: Multiple Openings: Alternative 2 would use existing footprints of oxbow lakes and the Historic Cutoff as multiple relief openings (Figure 1-7). Several step-down structures would be placed in Owens Lake and possibly the Historic Cutoff and Jim Smith Lake that would facilitate water exchanges at an environmentally optimized elevation between 115 feet and 135 feet. Alternative 2 would also restore some hydrologic and thus ecological conditions between the Arkansas and White rivers that existed before USACE built the Historic Closure Structure. As noted previously, the Arkansas carries more sediment than the White; and therefore, the PED team should use a sediment transport model to identify changes in deposition and scour in both rivers if Alternative 2 is the preferred plan. Overall, Alternative 2 would provide a long-term solution for reducing the risk of a cutoff forming between the Arkansas and White by minimizing the duration and controlling the location of damaging head differentials during overtopping events. As is the case with Alternative 1, a ship tow simulator would be needed to determine the effects of cross-currents on navigation.

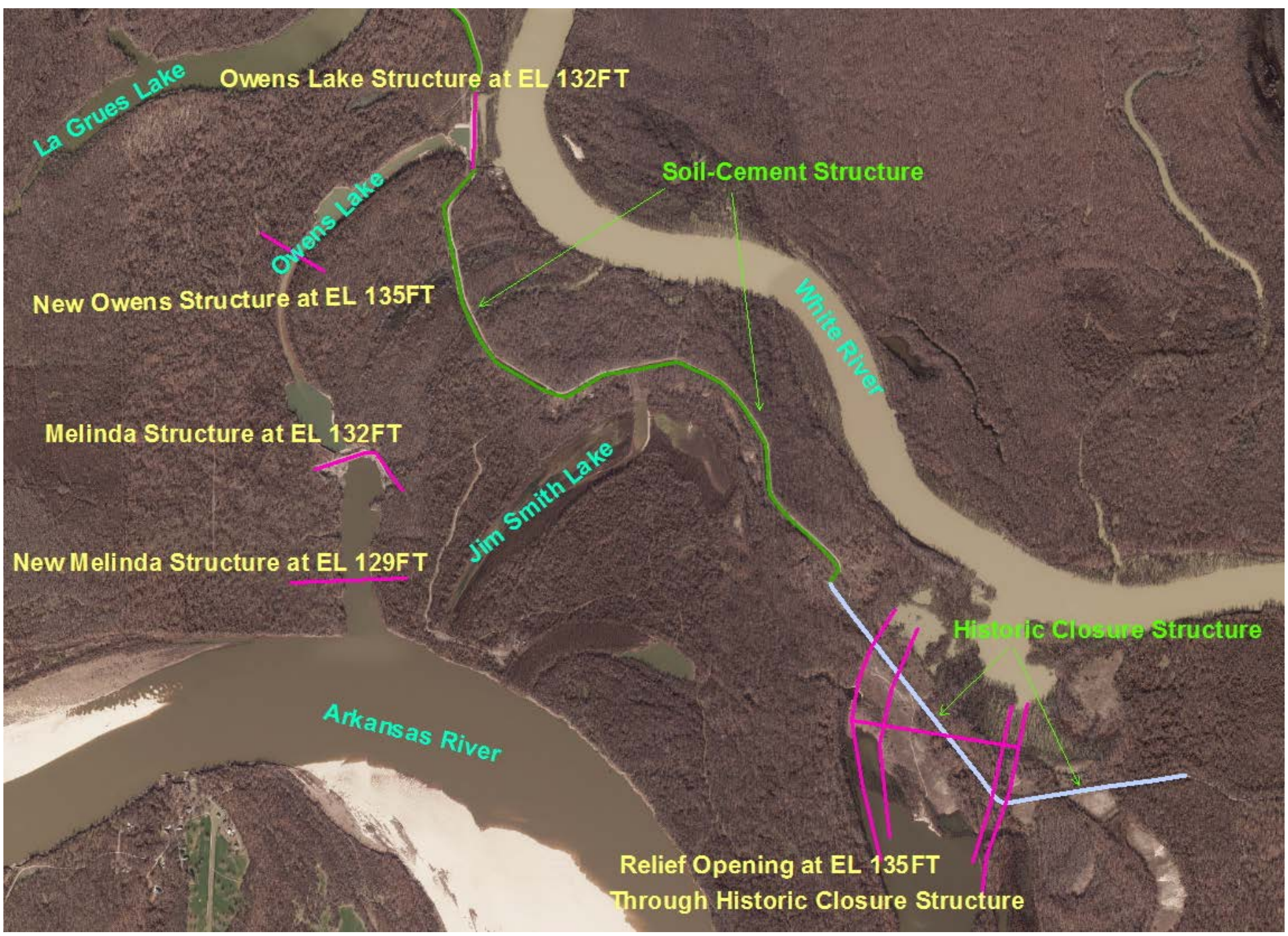

Figure 1-7: Alternative 2 Multiple Openings at Elevation 135 Feet

2 HEC RAS Model Development

Section 2 discusses development of the HEC-RAS model used to evaluate alternatives. Table 2-1 lists model names sometimes referenced in the text for each alternatives and their variation.

Table 2-1: Alternative Short Names

Model name	Corresponding Alternative
EXIST	Existing Condition
C157	Containment Structure at Elevation 157 feet
C157HC145_500ft	Containment Structure at Elevation 157 feet with Relief Channel
C157HC145_1000ft	Containment Structure at Elevation 157 feet with Relief Channel
M115	Multiple Openings at Elevation 115 feet
M125	Multiple Openings at Elevation 125 feet
M135	Multiple Openings at Elevation 135 feet

2.1 HEC-RAS Model Limits

HEC-RAS model limits (Figure 2-1) used in the study are at the following gages:

- Upstream limits, discharge hydrograph:
  - Mississippi River at Helena, Mississippi
  - White River at St. Charles, Arkansas
  - White River Entrance Channel at Lock 2, Arkansas
  - Arkansas River at Wilbur D Mills (Dam 2), Arkansas
- Downstream limit, rating curve:
  - Mississippi River at Greenville, Mississippi

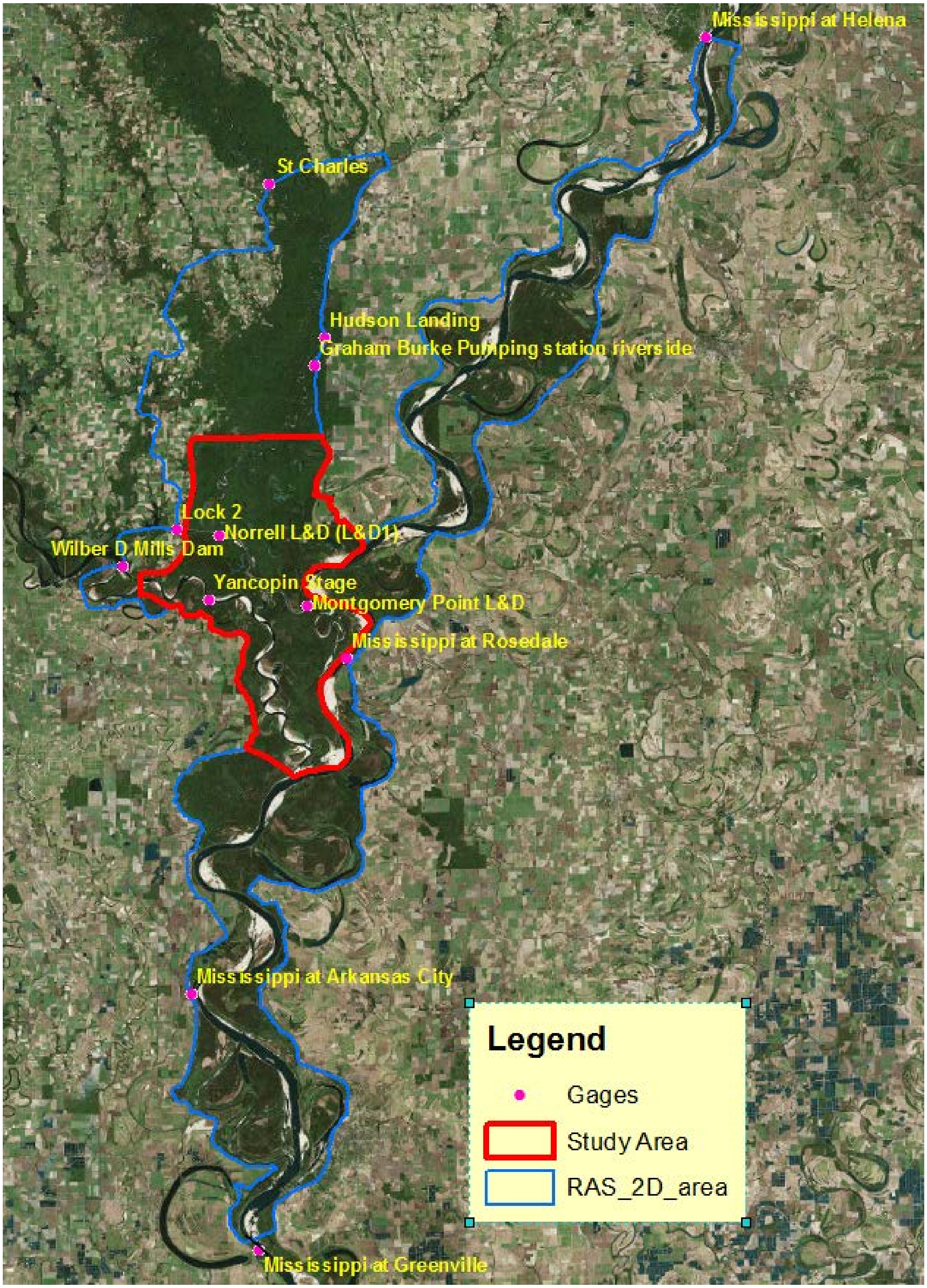

Figure 2-1: HEC RAS Model Limits and Gages

2.2 Flow and Stage Gage Data

The period of record simulated was January 1, 2000 through December 31, 2014. Observed discharge hydrographs, stage hydrographs, and rating curves were obtained from USACE Little Rock, Vicksburg, and Memphis District Water Management sections. Two and five-year frequency flow data were obtained from the 2009 Ark-White Study. Elevation data in rating curves and stage hydrographs were converted to NAVD88 elevations for calibration; and in the study area, conversion to NAVD88 is approximately equal to NGVD29 minus 2.5 inches. Upstream boundary conditions are discharge hydrographs from Mississippi at Helena gage, White River at St. Charles gage, Lock 2 tail water leakage, and Dam 2 releases. The downstream boundary condition is a single rating curve at the Greenville, Mississippi gage. Elevation hydrographs at St Charles, Hudson Landing, Graham Burke Pumping station, Lock and Dam 1, Montgomery Point, Wilber D. Mills (Dam 2), Yancopin, Helena, Rosedale, Arkansas City, and Greenville were used for calibration. More emphasis was placed on gages closer to or in the study area. Figure 2-1 shows gage locations.

2.3 Terrain

2.3.1 LiDAR and Bathymetry

The spatial coordinate projection file is NAD83 UTM Zone 15, U.S Feet and vertical projection is NAVD88 in feet. Mississippi River bathymetry data from the mouth of the White River up to Helena were obtained from the Memphis District. Pool 1 (2015), Arkansas River (2002) and White River bathymetry from Norrell Lock and Dam down to the White River (2015) was obtained from the Little Rock District, and the Vicksburg District provided 2015 Mississippi River bathymetry from the mouth of the White down to Arkansas City.

The Arkansas River channel has changed significantly between 2002 and 2016. The 2002 survey was adjusted horizontally to match the 2016 channel alignment, and changes in vertical elevations were determined using an iterative process of comparing modeled data to the Yancopin elevation hydrograph until a reasonable match was obtained. Final vertical elevations were adjusted 15 feet lower for cross-sections closer to the confluence with Mississippi and adjusted less further upstream, until Yancopin where no further vertical adjustments were applied. Each time, HEC-RAS converted adjusted cross-sections into a bathymetry incorporated into 2-Dimensional HEC-RAS terrain. Overbank, floodplain, and bathymetric data were merged into a single raster of one meter grid cell size. The raster was then resampled to a 10 foot cell size when importing into HEC-RAS 5.0.1 due to large computation run times required for a one meter cell (Figure 2-3).

Possible errors in water surface elevations produced by combining twelve different topographic and bathymetric elevation datasets are minimized by calibrating to observed gage data during modeling (see Section 3). Bottomland and fishery biologists were more interested in relative differences in flood duration and frequency rather than absolute values. Modeled water elevation errors produced by stitching together different terrain data would be reduced or eliminated during the subtraction process. The magnitude and duration of isthmus velocities and cross-currents in the navigation channel will be subjected to a sensitivity analysis after calibration to ensure the most reasonable maximum velocity and cross-currents are used in the final stone gradation and width opening determination in the selected alternative.

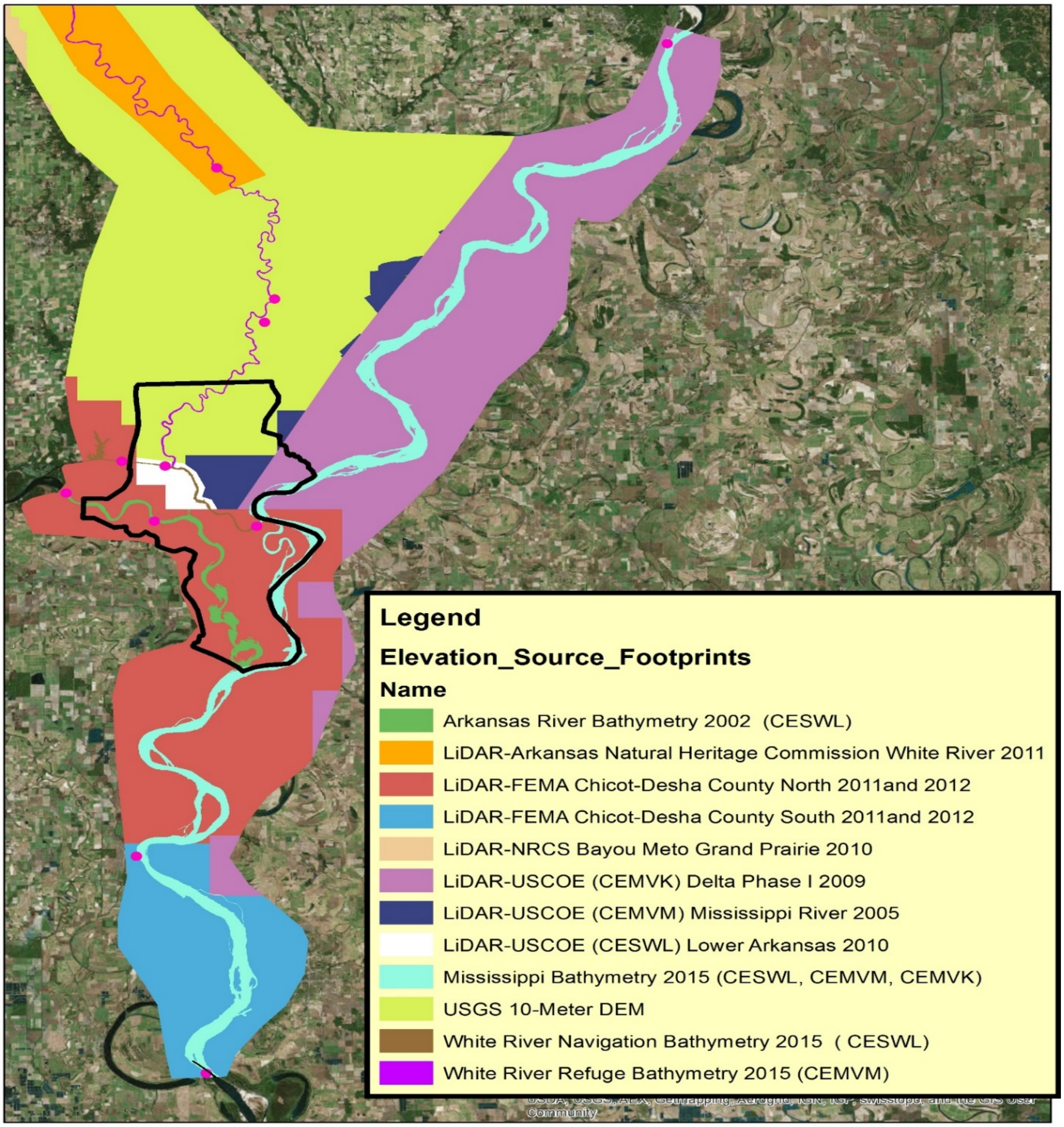

Figure 2-2: Elevation Sources

2.4 Geometry

2.4.1 Manning's n-values

Spatially varying land use classification, NLCD2011, obtained from the USGS website, was used to create a spatially varying Manning's roughness layer. Suggested n-values (Gary W. Brunner, CEIWR-HEC, 2016) and the NLCD2011 land use cover were used for initial model runs, except for the section of White River downstream of the St. Charles, Arkansas gage and north of Lock and Dam 1. This section of the river had meandered and the river channel was now spatially different than land cover. In this case, n-values were overwritten by user specified polygons that covered the footprint of the river channel (Table 2-2). Final n-values were determined through calibration.

Table 2-2: Initial and Calibrated N-values

NLCD Land Cover Classification Code	NLCD Land Cover Descriptions	Associated n-value	Calibrated n-value
0	NoData	0.06	0.06
31	Barren Land Rock/Sand/Clay	0.04	0.04
82	Cultivated Crops	0.06	0.05
41	Deciduous Forest	0.1	0.1
24	Developed, High Intensity	0.15	0.15
22	Developed, Low Intensity	0.08	0.08
23	Developed, Medium Intensity	0.1	0.1
21	Developed, Open Space	0.035	0.035
95	Emergent Herbaceous Wetlands	0.08	0.085
42	Evergreen Forest	0.12	0.12
71	Grassland/Herbaceous	0.04	0.04
43	Mixed Forest	0.08	0.08
11	Open Water	0.03	0.03
81	Pasture/Hay	0.06	0.06
52	Shrub/Scrub	0.08	0.08
90	Woody Wetlands	0.08	0.085

2.4.2 Existing Conditions

The entire area was modeled as a 2-dimensional (2D) area using HEC-RAS 5.0.1. 2-D model mesh limits were contained within levees and bounded upstream and downstream by stage or flow gages. A 500 by 500 foot cell size was used to build the computational mesh and then refined by break lines and manually subdivided where necessary. Break lines were used along oxbow and river banks, levees, railroad embankments, high ground, and at locations requiring finer delineation. River sections at gages, and other cross sections of interest, were modeled as 2-D area connections and were reinforced by using break lines with cell spacing ranging from 100 feet to 300 feet (Figure 2-3). Stage and flow data at the 2-D area connections were written to Hydrologic Engineering Center Data Storage System (HEC-DSS) 2.0.1 files for frequency and duration analysis.

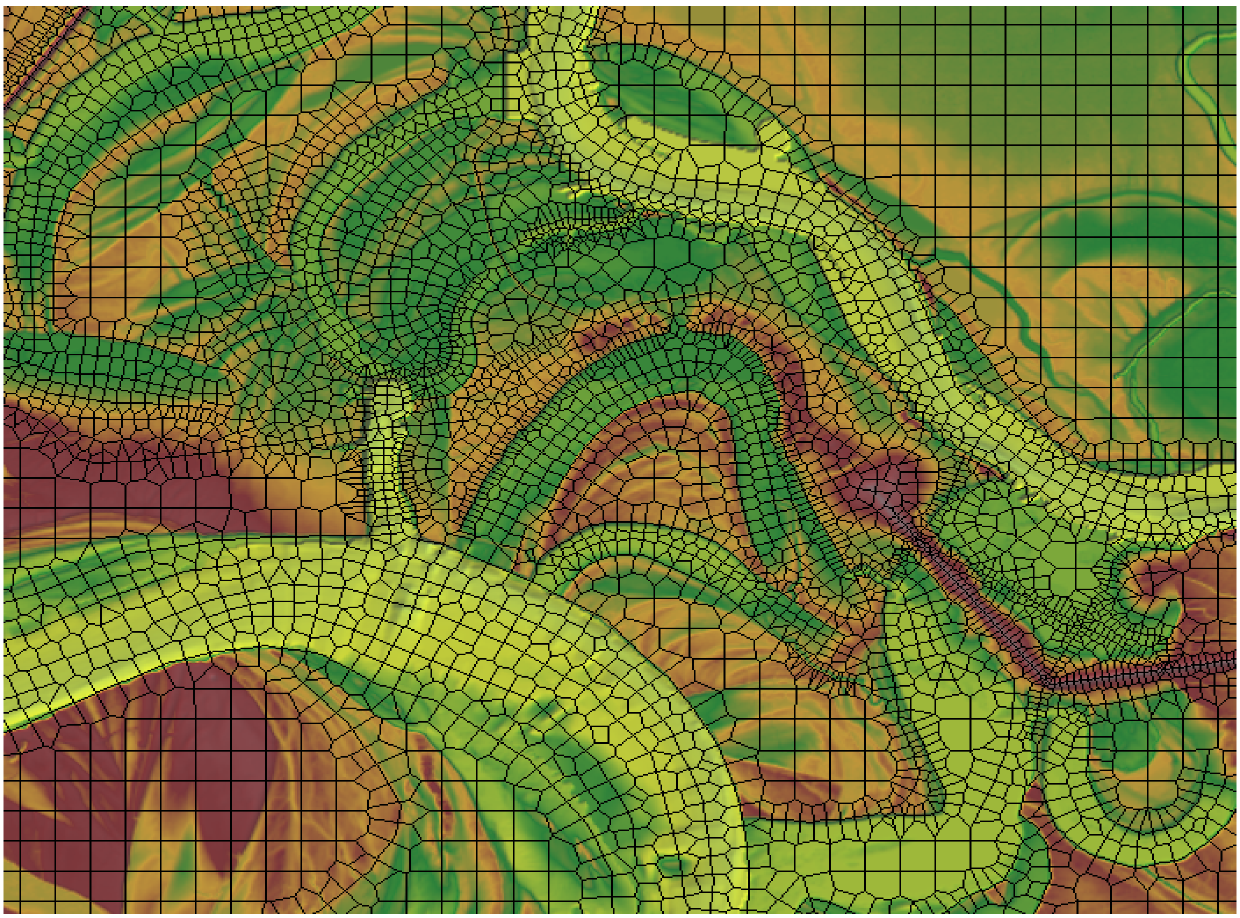

Figure 2-3: Computational HEC-RAS 2-Dimensional Mesh

Lock and dams, locations of interest, and cross sections at gage locations were modeled as 2-D area connections to automate the process of retrieving and writing hydrographs into output files for further analysis and calibration. Montgomery Point was modeled as fully open for the entire simulation period for several reasons. For one, it was not operational until 2004, and tows only lock at the dam when the Mississippi at the mouth of the White River falls below elevation 115 feet, which is well below elevations of the top banks of the White in this location. Operation of Montgomery Point does not affect overland frequency and duration of flooding, overland flow velocity, and scour potential across the floodplain during high flow events.

The study area is a very dynamic biological, seasonal and hydrological system that continually changes. However, to isolate hydrologic effects each alternative, it is assumed that the system is static with the exception of adding alternatives and associated structures to HEC-RAS geometry and terrain. Below is a partial list of dynamic variables not taken into account in HEC-RAS geometry.

- Seasonally changing n-values;
- Decreasing n-values with increasing discharge;
- Migrating channel and active head cutting up the White River, Mississippi River, and Arkansas River;
- Active dredging (this changes both the river cross-section and the dredge pile volume on the land);
- Bank caving and channel widening along the White and the Arkansas rivers;
- Beaver dams;
- Shift in land cover;
- Active head cutting and widening of the Melinda Corridor;
- Levee overtopping or failure; and,
- Regulation changes in upstream projects.

Potential sources of modeling error include:

- Backwater effects on rating curves;
- Rating curve shifts not developed or not updated and applied in a timely matter;
- Discharge measurement errors; and,
- Aggregation of twelve different LiDAR and bathymetric terrain data at different resolutions and collected at different times.

2.4.3 Alternatives: Modifications to Existing Geometry

Containment Structure at Elevation 157 feet (C157): The containment structure with a crest elevation of 157 feet was added to the terrain and geometry as a 2-D area connection. The Melinda Weir was removed to eliminate the risk of erosion at the toe of the new containment structure and the land adjacent to Melinda Weir. (Figure 1-6).

Containment Structure at Elevation 157 feet with Relief Channel (C157HC145): The containment structure at elevation 157 feet decreases duration and frequency of overtopping, but it increases head differentials and therefore increased scour potential across the isthmus. To decrease head differentials, this alternative includes an opening through the Historic Closure Structure at an elevation of 145 feet. A 2-D area connection was added to the 2-D mesh with a top weir elevation of 145 feet and 8 percent side grades from elevation 145 feet up to existing ground level. Two different weir widths were modeled: 500 feet and 1,000 feet (Figure 1-6). Several different CON/SPAN, or precast bridge, structures ranging from 20 feet to 30 feet wide and 4 feet to 6 feet tall were modeled at elevation 140 feet through Owens Weir to reduce or eliminate negative change in hydrology to the surrounding bottomland hardwoods. See ATTACHMENT F for sizing effort and results.

Distributed Flow or Multiple openings at elevation: 115,125,135 (M115, M125, M135): Owens Lake, also referred to as the Melinda Corridor, and the Historic Cutoff were used to model the multiple opening alternative (Figure 1-7). Owens Weir and the Melinda Weir were removed from the Melinda Corridor and the new channel thalweg lowered to elevation 105 feet to allow water to pass uninhibited between the two rivers. The Historic Closure Structure was removed and the Historic Cutoff thalweg was lowered to elevation of 90 feet. The Historic Cutoff was widened to about 0.5 miles on the White River side and 0.25 miles closer to the Arkansas following the existing footprint. Manning's n values were changed in the multiple open channels to reflect open water instead of heavily wooded areas. Three different weir elevations were modeled for this alternative: 115, 125 and 135 feet. Results for each weir elevation were evaluated to determine their effectiveness at shifting the Refuge toward drier hydrology and for reducing the duration of damaging head differentials across the isthmus. The final design will have a minimum of three step-down structures in the Melinda Corridor and if needed, through Jim Smith Lake and the Historic Cutoff to minimize head differentials across the structures to less than four feet.

2.5 HEC RAS Plans

The same flow file was used for each plan. Due to study time constraints, a 15-year period of record starting on January 1st, 2000 ending on December 31st, 2014 was completed for each alternative. Instead of running the entire period in one plan, plans were broken down into 15 one-year plans for each alternative. Each one-year plan took four to six days of continuous computation to complete. Breaking the 15-year period into smaller manageable segments allowed runs to complete by minimizing potential simulation interruptions due to network connection problems, power failures, equipment failures, and software updates requiring restarts.

3 HEC RAS Calibration

3.0 Observed and Calibrated Elevation Hydrographs

ATTACHMENT A: contains plots of observed and calibrated elevation hydrographs, and Table 2-2 displays initial and calibrated N-values.

3.1 Hydraulic Model Statistical Performance Evaluation

Calibration used eleven gages (Figure 2-1). Of the eleven gages, nine elevation hydrographs and one discharge hydrograph were used for goodness of fit statistical performance analysis. The eleventh gage, the Graham Burke Pumping Station, was not rated because its records are incomplete and missing most data below an elevation of 137 feet. The Lock 2 gage was not used due to its proximity to Lock and Dam 1. Prior to evaluation, missing observed data were interpolated using HEC-DSSVue. If too many records were missing and interpolation was not possible, these intervals were removed from the goodness-of-fit evaluation.

Backwater from the Mississippi and resulting head differentials across the isthmus is the driving force in the study area. Most gages in the area are stage instruments with no rating curves, or they have looped and complicated rating curves that can quickly change over time. Any discharge hydrographs developed from elevation hydrographs and corresponding rating curves for the period of record would not be as reliable as performance evaluation parameters as the observed elevation hydrographs. Locations with rating curves include upstream and downstream boundary conditions, and since upstream boundary conditions are observed discharge hydrographs feeding the hydraulic model, only the most downstream modeled discharge hydrograph at the Mississippi at Greenville gage was evaluated for goodness-of-fit.

Performance or goodness of fit tests are based in five statistical parameters (Marinoé Gonzaga da Silva*, 2015) (Dao Nguyen Khoi, 2015), (Golmar Golmohammadi, 2014), and (D. N. Moriasi, 2007):

1) The Nash-Sutcliffe efficiency coefficient (NSE), which can range from minus infinity to one with one being a perfect match between modeled and observed data. Smaller NSE values indicate a poorer fit between modeled and observed data.

2) The percent bias (PBIAS) that has an optimum value of 0.0 with low values indicating a satisfactory model, while positive values show a model's tendency to underestimate while negative values show the model's tendency to overestimate.

3) The root mean square error (RMSE) where values close to zero indicate excellent model fit, and values less than one-half the standard deviation of observed data indicate a model with predictive capability (Table 3-2 has further guidelines for RMSE).

4) Ratio of RMSE to the standard deviation of observations (RSR) has an optimal value of 0 (lower RSR values indicate a better performing model).

5) The coefficient of determination (R^2) ranges from 0 to 1 with R^2 greater than 0.5 considered acceptable and 1.0 a perfect fit. This is a statistical measurement of how well a regression line approximates observed data.

Statistical parameters used to evaluate a range of hydraulic model performance can be judged according to guidelines in Table 3-1 and Table 3-2. Table 3-3 and Table 3-4 summarize test results.

Table 3-1: Statistical Parameter Guidelines for Model Performance 1 (D. N. Moriasi, 2007)

Performance Rating	Ratio of Root Mean Square Error	Nash-Sutcliffe Model Efficiency	Percent Bias
Very Good	$0.0 < RSR \leq 0.50$	$0.75 < NSE \leq 1.00$	$PBIAS < \pm 10$
Good	$0.50 < RSR \leq 0.60$	$0.65 < NSE \leq 0.75$	$\pm 10 \leq PBIAS < \pm 15$
Satisfactory	$0.60 < RSR \leq 0.70$	$0.50 < NSE \leq 0.65$	$\pm 15 \leq PBIAS < \pm 25$
Unsatisfactory	$RSR > 0.70$	$NSE \leq 0.50$	$PBIAS \geq \pm 25$

Table 3-2: Statistical Parameter Guidelines for Model Performance 2 (Axel Ritter, 2012)

Performance Rating	Model efficiency interpretation	$n_t^a = (SD_{obs}/RMSE)-1$	NSE
Very Good	SD $\geq$ 3.2 RMSE	$\geq$ 2.2	> 0.90
Good	SD = 2.2 RMSE -3.2 RMSE	1.2 - 2.2	0.80 - 0.90
Acceptable	SD = 1.2 RMSE - 2.2 RMSE	0.7 - 1.2	0.65 - 0.80
Unsatisfactory	SD < 1.7 RMSE	< 0.7	< 0.65

Based on stricter guidelines in Table 3-2, the model performed at a rating of "very good" at six of the ten gage locations and "good" at four gages. Methods in Table 3-1 resulted in the highest goodness-of-fit ratings available for statistical parameters for all gages with the exception of the Mississippi, Greenville discharge hydrograph. The Greenville hydrograph rated the same for all statistical parameters except for the PBIAS, and it was only a slight decrease in rating from “very good” to “good.” This is due to the fact that the discharge hydrograph was not available; and therefore, it was calculated for the modeled period of record based on a single-valued rating curve instead of multiple looped and shifted rating curves. Errors introduced at this downstream boundary condition significantly fall during backwater calculations in the upstream Mississippi River at the Arkansas City gage and are eliminated in the upstream Mississippi River at Rosedale gage close to the project area.

Several methods and guidelines were used to quantify the goodness-of-fit of modeled hydrographs in relation to observed hydrographs. In general, results of the analysis demonstrated that the hydraulic model performs well and provides a good fit between observed and modeled data with most of the statistical performance ratings in the “very good” category and some in the “good” category.

Table 3-3: Statistical Parameter Model Performance Results based on Table 3-2

Gage	Observed Standard Deviation	Root Mean Square Error (RMSE)		n_t^a = $(SD_{obs}/RMSE)$-1		Nash-Sutcliffe Model Efficiency Coefficient	
		Coefficient	Rating	Coefficient	Rating	Coefficient	Rating
Montgomery Point L&D	11.60	1.48	Very Good	6.9	Very Good	0.98	Very Good
Mississippi at Rosedale	10.66	1.45	Very Good	6.4	Very Good	0.98	Very Good
Arkansas at Yancopin	10.24	4.05	Good	1.5	Good	0.84	Good
Arkansas at Dam 2 TW	9.81	3.53	Good	1.8	Good	0.87	Good
White at L&D01 TW	8.27	1.24	Very Good	5.7	Very Good	0.98	Very Good
White at St. Charles	6.67	0.79	Very Good	7.4	Very Good	0.99	Very Good
White at Hudson Landing	8.27	1.24	Very Good	5.7	Very Good	0.98	Very Good
Mississippi at Helena	11.69	3.43	Very Good	2.4	Very Good	0.91	Very Good
Mississippi at Ark. City	10.43	3.42	Good	2.0	Good	0.89	Good
Mississippi at Greenville	301143	106241	Good	1.8	Good	0.88	Good

Table 3-4: Statistical Parameter Model Performance Results

Gage	Observed Standard Deviation	Root Mean Square Error		Nash-Sutcliffe Model Efficiency Coefficient		Coefficient of Determination		Percentage of Bias		RMSE-Observation Standard Deviation Ratio	
		RMSE	Rating	NSE	Rating	R^2	Rating	PBIAS	Rating	RSR	Rating
Montgomery Point	11.60	1.48	Good	0.98	Very Good	0.99	Acceptable	0.40	Very Good	0.13	Very Good
Mississippi at Rosedale	10.66	1.45	Good	0.98	Very Good	0.99	Acceptable	0.28	Very Good	0.14	Very Good
Arkansas at Yancopin	10.24	4.05	Good	0.84	Very Good	0.96	Acceptable	-2.42	Very Good	0.40	Very Good
Arkansas at Dam 2 TW	9.81	3.53	Good	0.87	Very Good	0.96	Acceptable	-1.95	Very Good	0.36	Very Good
White at L&D01 TW	8.273	1.24	Good	0.98	Very Good	0.99	Acceptable	0.276	Very Good	0.15	Very Good
White at St. Charles	6.671	0.79	Good	0.99	Very Good	0.99	Acceptable	-0.17	Very Good	0.12	Very Good
White at Hudson Landing	8.272	1.24	Good	0.98	Very Good	0.99	Acceptable	0.277	Very Good	0.15	Very Good
Mississippi at Helena	11.69	3.43	Good	0.91	Very Good	0.99	Acceptable	1.85	Very Good	0.29	Very Good
Mississippi at Ark. City	10.43	3.42	Good	0.89	Very Good	0.99	Acceptable	2.73	Very Good	0.33	Very Good
Mississippi at Greenville	301143	106241	Good	0.88	Very Good	0.97	Acceptable	-13.20	Good	0.35	Very Good

4 Hydraulic Model Sensitivity and Uncertainty

As discussed in the previous section, results of the goodness of fit analysis demonstrated that the hydraulic model performs very well and provides a good fit between observed and modeled data with most of the statistical performance ratings in the “very good” category and some in the “good” category for the POR of analysis.

Cross currents in the navigation channel and velocities through the proposed opening in the Historic Closure Structure are the primary concerns since these will determine the final design for rock gradation, structure opening size, and orientation.

A range of n-values for the proposed opening through the Historic Closure Structure for the 2011 event was used to determine the highest reasonable velocities that could exist for both the 1000 foot and the 500 foot opening. The maximum velocity was used to determine the gradation of rock with a factor of safety of 1.2 for both a 10 percent and 20 percent outflow slope. The rock gradation can be found in the Engineering Appendix C. Final gradation will be determined during PED phase.

Currently another 2-D Hydraulic model, Adaptive Hydraulics Model, AdH, is being developed as input into a Ship Tow Simulator. The HEC-RAS model results will be compared to the AdH model results to confirm magnitude and direction of velocity of cross currents in the navigation channel for a range of openings from 500 feet to 1000 feet. The AdH model will feed into a Ship Tow Simulator that industry licensed pilots and others can test interactively to evaluate proposed modifications and opening sizes through the Historic Closure Structure to reduce or eliminate dangerous cross currents affecting navigation. The Ship Tow Simulator and final width opening will be completed during PED phase.

5 Hydraulic Model Outputs

At the onset of this study, five alternatives were modeled: Existing conditions, C157, M115, M125, and M135. Based on model outputs, the study team determined that although M115, M125, and M135 would decrease head cut probability across the isthmus, the alternatives offered no environmental benefits to the Refuge. They would negatively affect bottomland hardwoods and oxbow lakes, and they would likely introduce strong cross currents into the navigation channel and were dropped from further hydraulic analysis. The team refined alternatives C157, C157HC145_500ft and C157HC145_1000ft, and these became the focus of hydraulic modeling.

5.0 Head differentials Plots

Maximum head differentials and duration is a convenient way to determine the effectiveness of each alternative’s ability to reduce scour and head cutting potential across the isthmus. Figure 1-5 shows locations where head differentials were calculated. Differentials between the two rivers exceed 10 feet for extended periods, but scour only occurs as water flows across the Isthmus when either the White or the Arkansas rises above elevation 145 feet at Owens Weir (Owens Lake Structure) or through the Historic Cutoff (Figure 5-1 and Figure 5-2). Differentials of four feet or less do not appear to cause significant damage. Table 5-1 and Table 5-2 show absolute head differentials and corresponding exceedance durations in days for each alternative.

Table 5-1: Absolute Head Differential Annual Exceedance Duration at the Melinda Corridor

Absolute Head Difference Annual Exceedance Duration: Melinda Corridor above Elevation 145 feet				
Head Differential	Alternative Annual Exceedance Duration in Days			
	Existing	C157_HC145 500FT	C157_HC145 1000FT	C157
Feet	Days	Days	Days	Days
4	20.4	21.9	20.8	24.8
5	14.9	17.9	15.3	20.5
6	9.3	11.6	8.9	16.2
7	4.7	6.2	3.9	10.9
8	1.5	1.8	1.5	5.5

Table 5-2: Absolute Head Differential Annual Exceedance Duration at the Historic Cutoff

Absolute Overtopping Head Difference Annual Exceedance Duration: Historic Cutoff Corridor above Elevation 145 feet				
Head Differential	Alternative Annual Exceedance Duration in Days			
	Existing	C157_HC145 500FT	C157_HC145 1000FT	C157
Feet	Days	Days	Days	Days
4	22.9	23.8	22.3	27.2
5	19.8	20.8	19.1	23.6
6	14.1	15.8	11.7	19.7
7	6.9	7.5	5.2	14.3
8	2.2	2.0	1.4	6.2

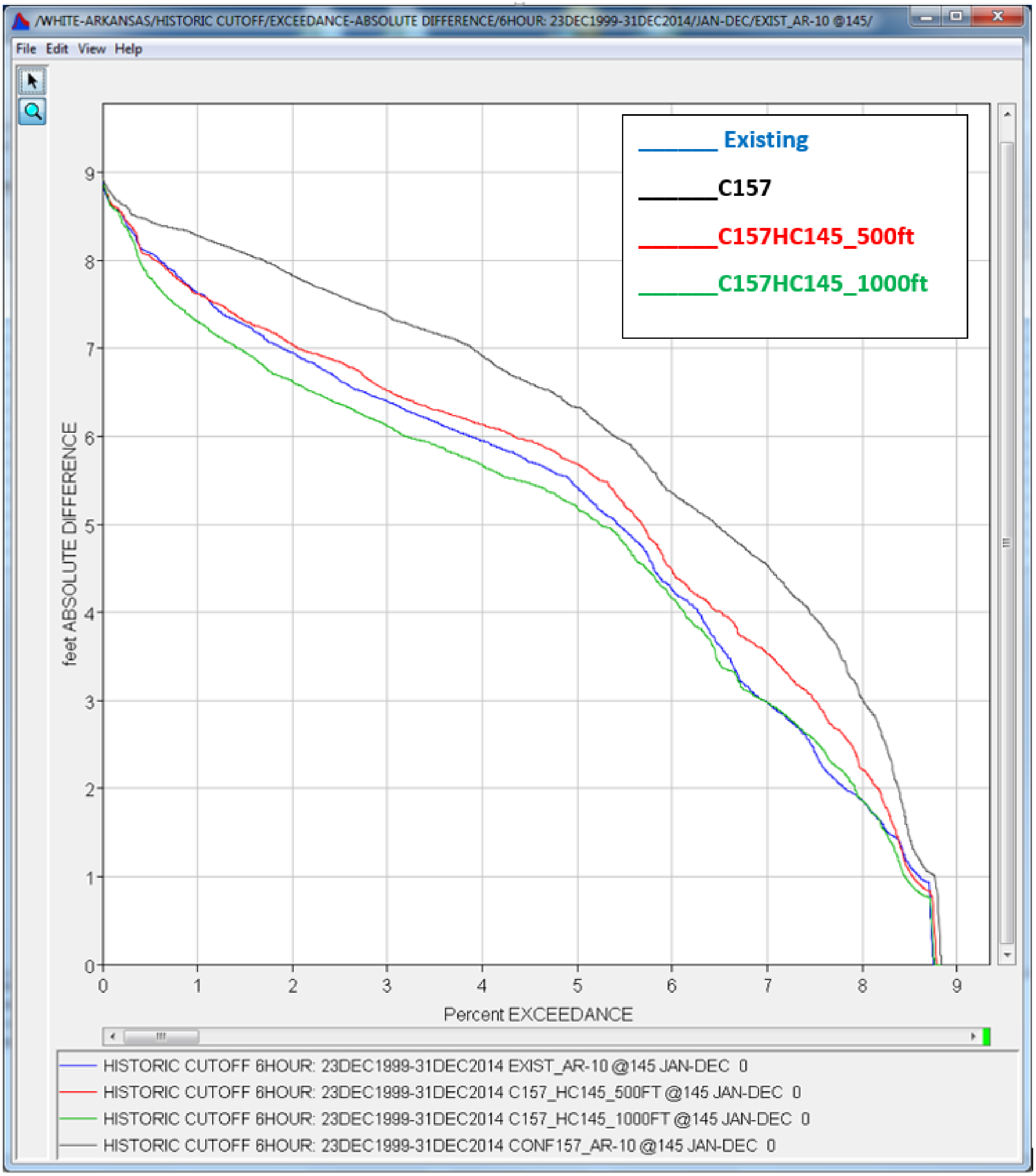

Figure 5-1: Annual Overtopping Absolute Head Differential Exceedance Duration for the Historic Cutoff

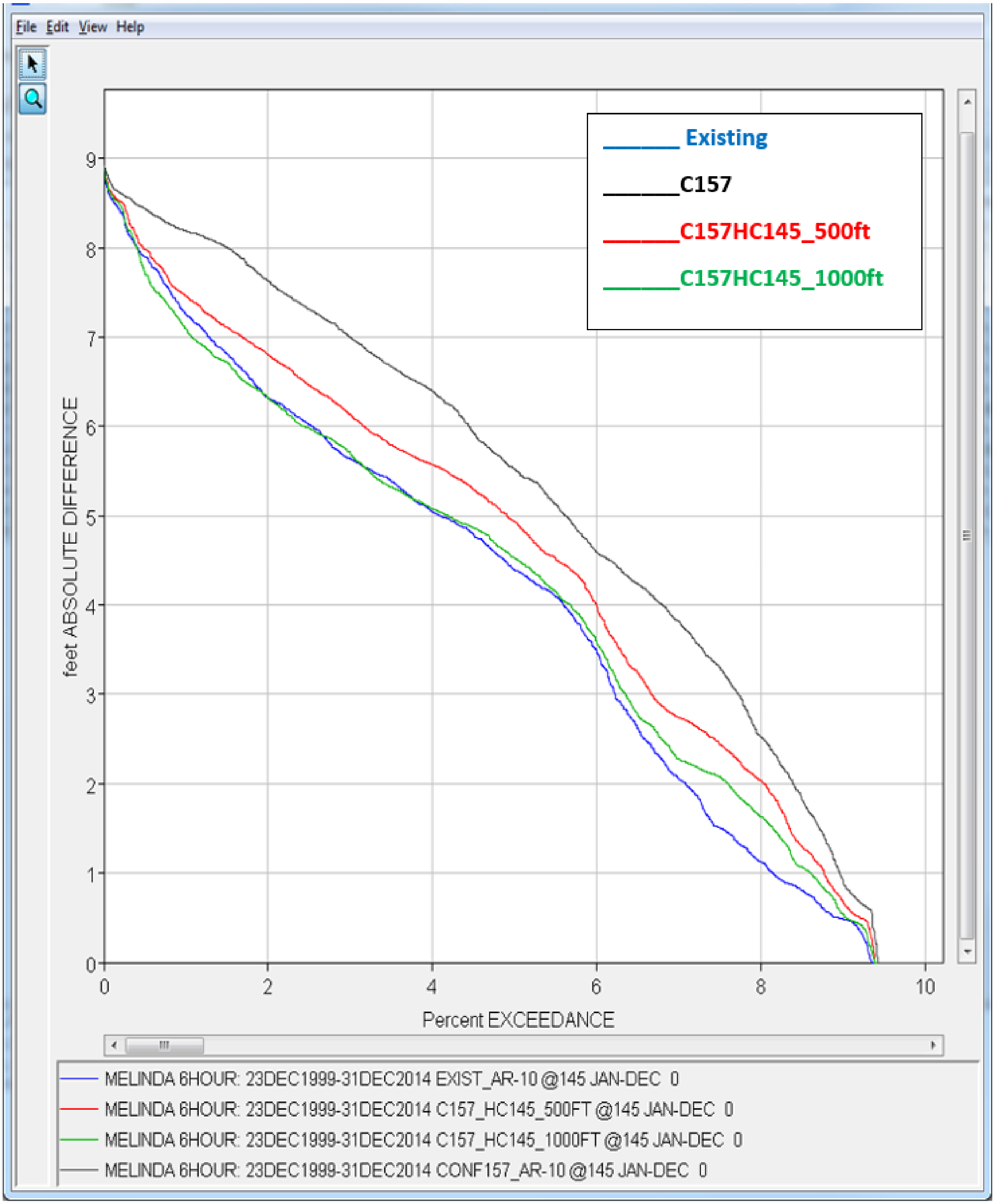

Figure 5-2: Annual Overtopping Absolute Head Differential Exceedance Duration for the Melinda Corridor

5.1 Velocity Maps

NRCS soil survey maps and published permissible mean velocity data were combined to determine a threshold scour velocity. Locations prone to scour and head cutting were easily identified due to increased flow velocities. Permissible or allowable velocity is the greatest mean velocity that will not scour and erode the channel boundary. As shown in Table 5-3, Fortier and Scobey (1926) developed maximum permissible velocities for earthen irrigation canals without vegetation or structural protection (Natural Resources Conservation Service , 2007).

Table 5-3: Permissible Mean Velocity (feet per second) for Straight Canals of Small Slope after Aging with Flow Depths Less than 3 Feet (Fortier and Scobey, 1926)

Original Material excavated for canals	Clear water, no detritus	Water transporting colloidal silts	Water transporting noncolloidal silts, sands, gravels, or rock fragments
Fine sand (noncolloidal)	1.5	2.5	1.5
Sandy loam (noncolloidal)	1.75	2.5	2
Silt loam (noncolloidal)	2	3	2
Alluvial silt (noncolloidal)	2	3.5	2
Ordinary firm loam	2.5	3.5	2.25
Stiff clay (very colloidal)	3.75	5	3
Alluvial silt (colloidal)	3.75	5	3
Shales and hardpans	6	6	5
Volcanic ash	2.5	3.5	2
Fine gravel	2.5	5	3.75
Graded, loam to cobbles (noncolloidal)	3.75	5	5
Graded silt to cobbles (when colloidal)	4	5.5	5
Coarse gravel (noncolloidal)	4	6	6.5
Cobbles and shingles	5	5.5	6.5

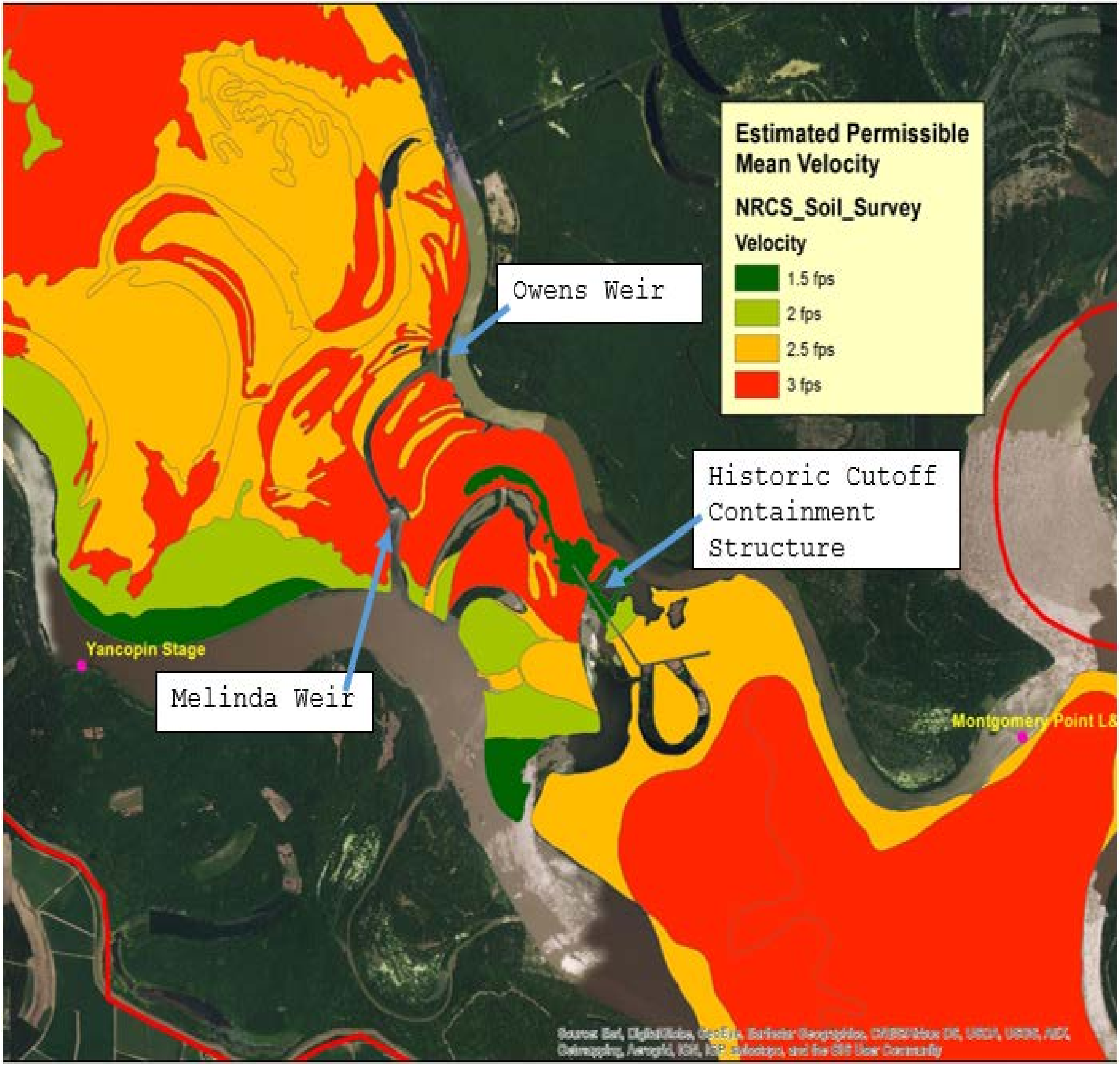

Figure 5-3: Estimated Permissible Mean Velocity (feet per second)

Table 5-4: Estimated Permissible Velocity based on NRCS Soil Surveys for Arkansas and Desha Counties, Arkansas

Soil Types	Estimated Permissible Velocity (feet per second)
Commerce silt loam	2
Crevasse loamy fine sand	1.5
Desha clay	3
Desha silty clay	2.5
Keo loam	2
Perry clay	3
Portland clay	3
Rilla silt loam	2
Riverwash, sandy	1.5
Sharkey-Commerce-Coushatta association	2.5
Sharkey clay	3
Udipsamments	1.5
Yancopin silty clay loam	2.5
Yorktown silty clay	2.5

Based on isthmus soil types and permissible mean velocities, average velocities of 1.5 feet per second (fps) up to 3.0 fps can erode the isthmus. HEC-RAS gridded velocity maps are calculated over an averaged 10 square foot area. Because of the nature of averaging, maximum velocities tend to decline. Therefore, a minimum velocity of 2 fps serves as the threshold for identifying areas susceptible to erosion for each alternative (Figure 5-5 through Figure 5-9). Of particular interest, Figure 5-4 is a close up of a nick point in Webfoot Lake. Aerial photographs show signs of multiple nick points and scour even when velocities are less than 2 fps. Based on soil type and corresponding permissible velocity, this area should withstand 2.5 fps, which supports using the general assumption of 2 fps as a velocity threshold for identifying potential scour locations.

Figure 5-4: Nick Point on East Bank of Webfoot Lake

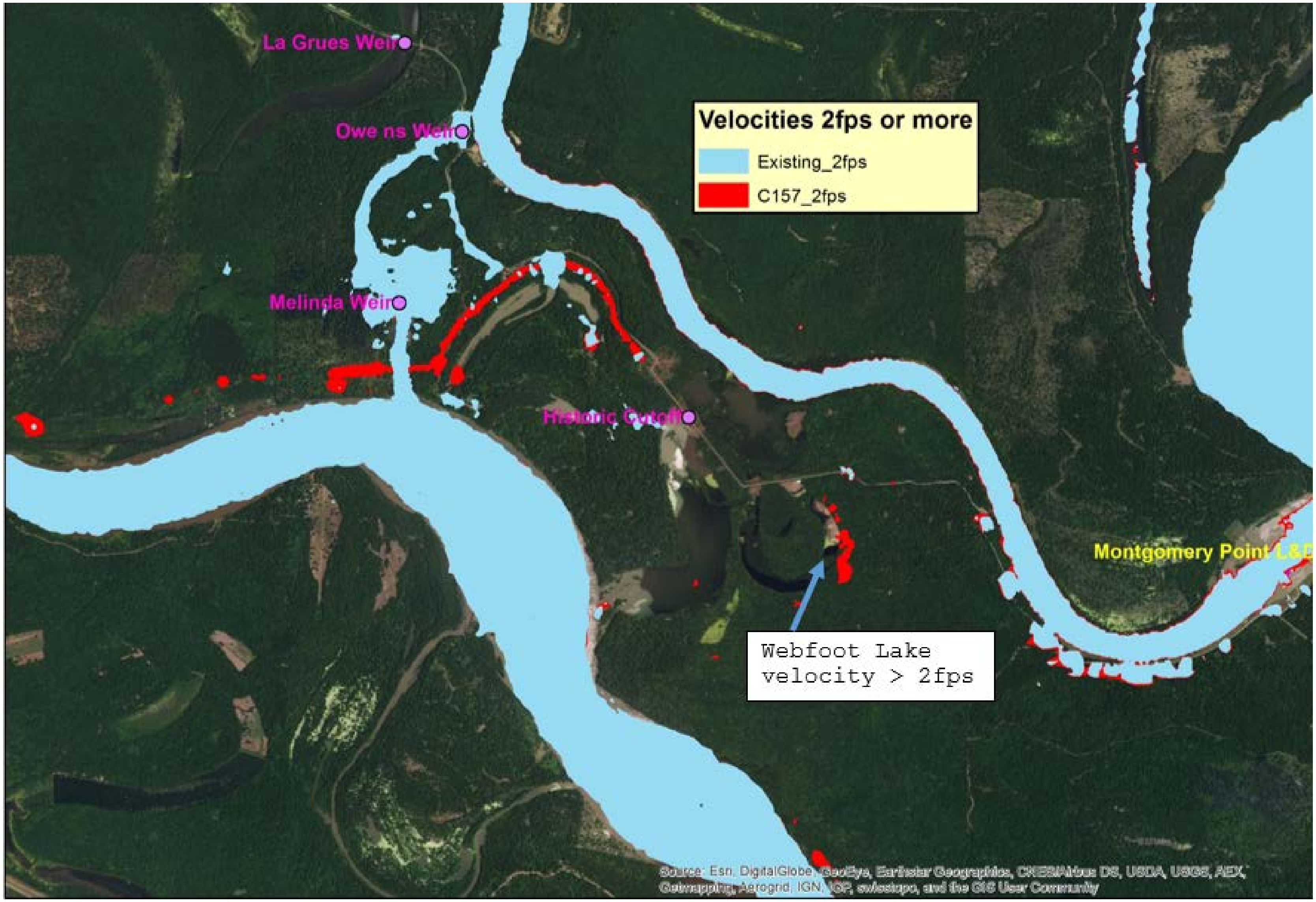

Figure 5-5: Velocities 2 Feet per Second or More: Existing Conditions and C157

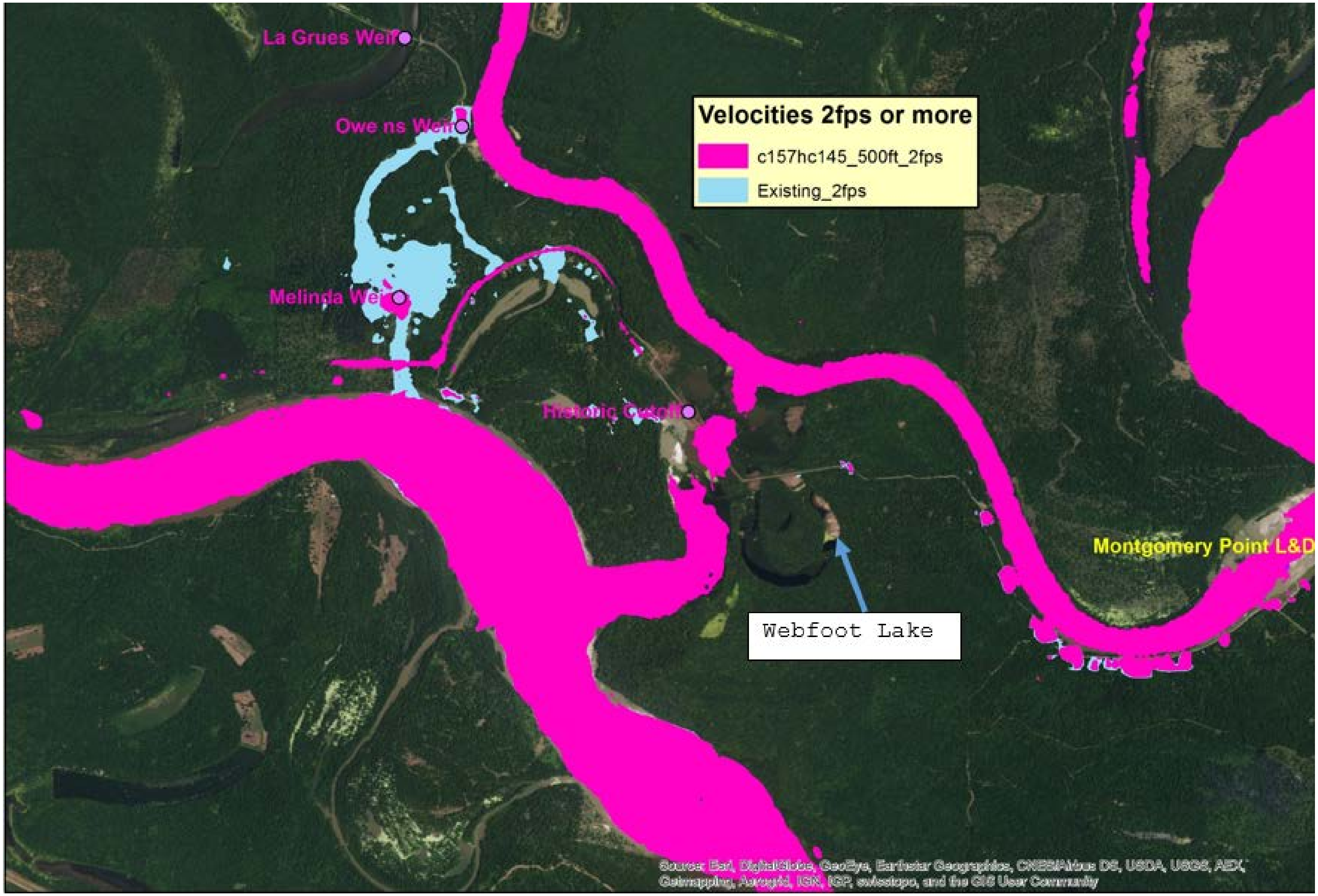

Figure 5-6: Velocities 2 Feet per Second or More: Existing Conditions and C157HC145_500ft

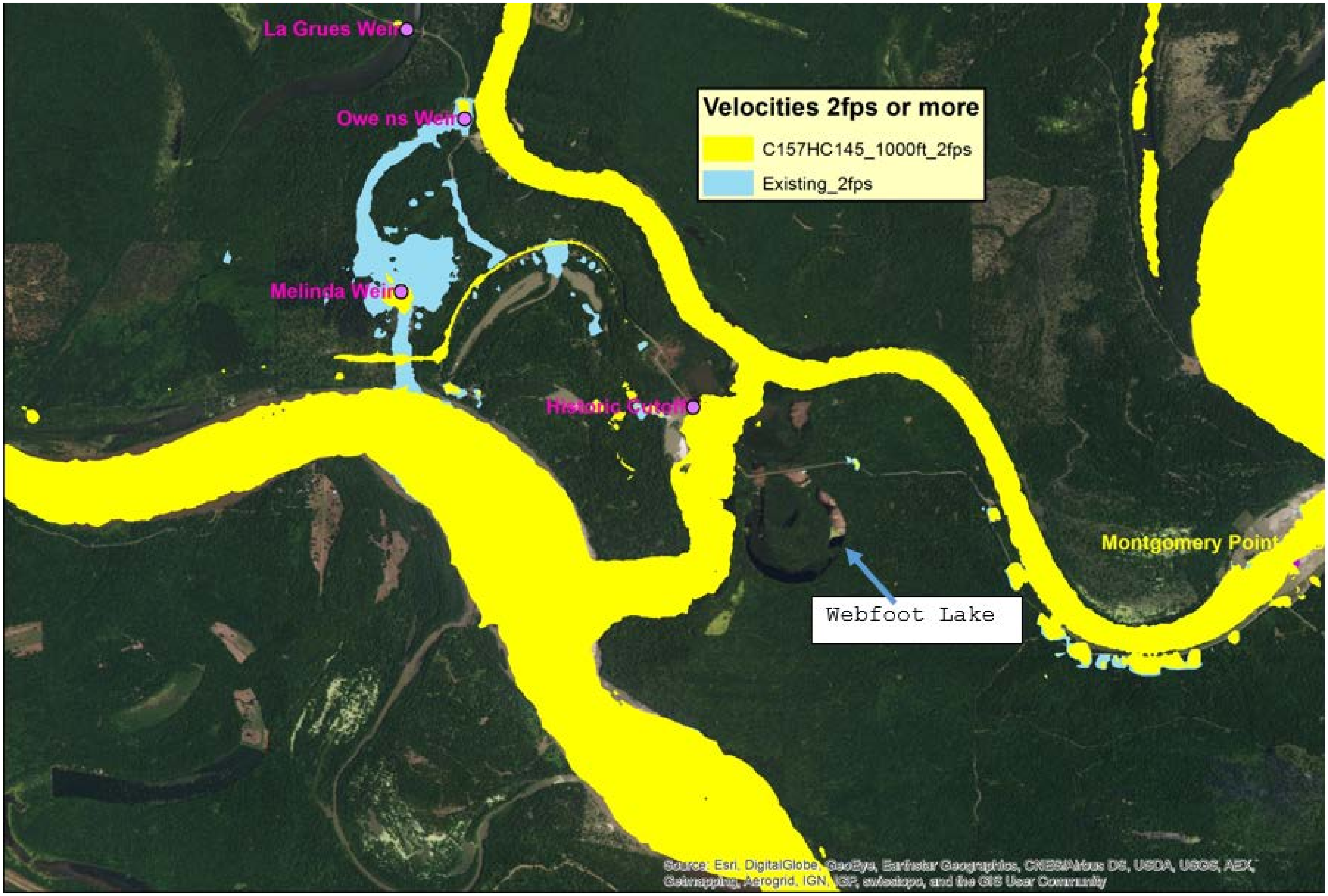

Figure 5-7: Velocities 2 Feet per Second or More: Existing Conditions and C157HC145_1000ft

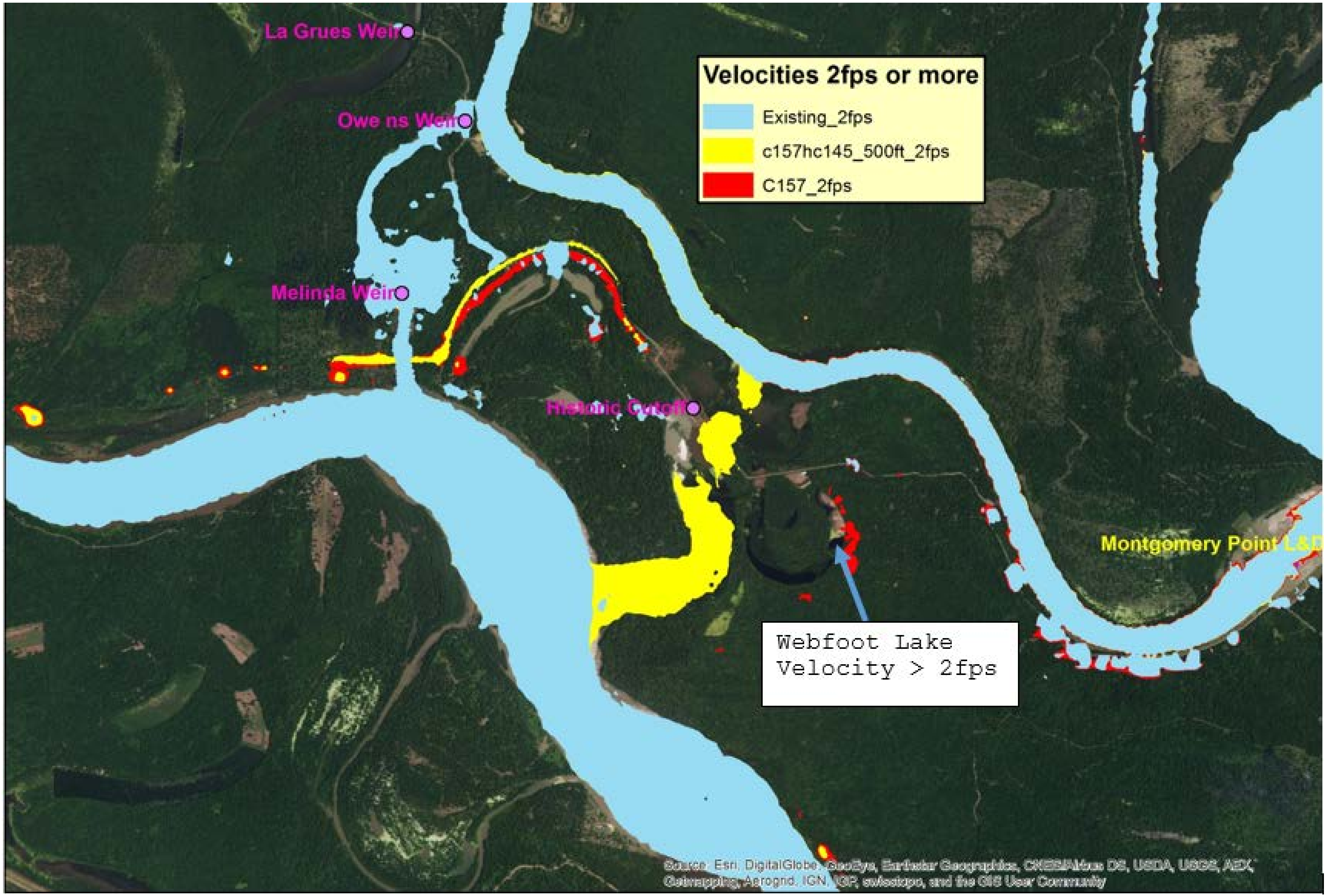

Figure 5-8: Velocities 2 Feet per Second or More: Existing Conditions, C157HC145_500ft, and C157

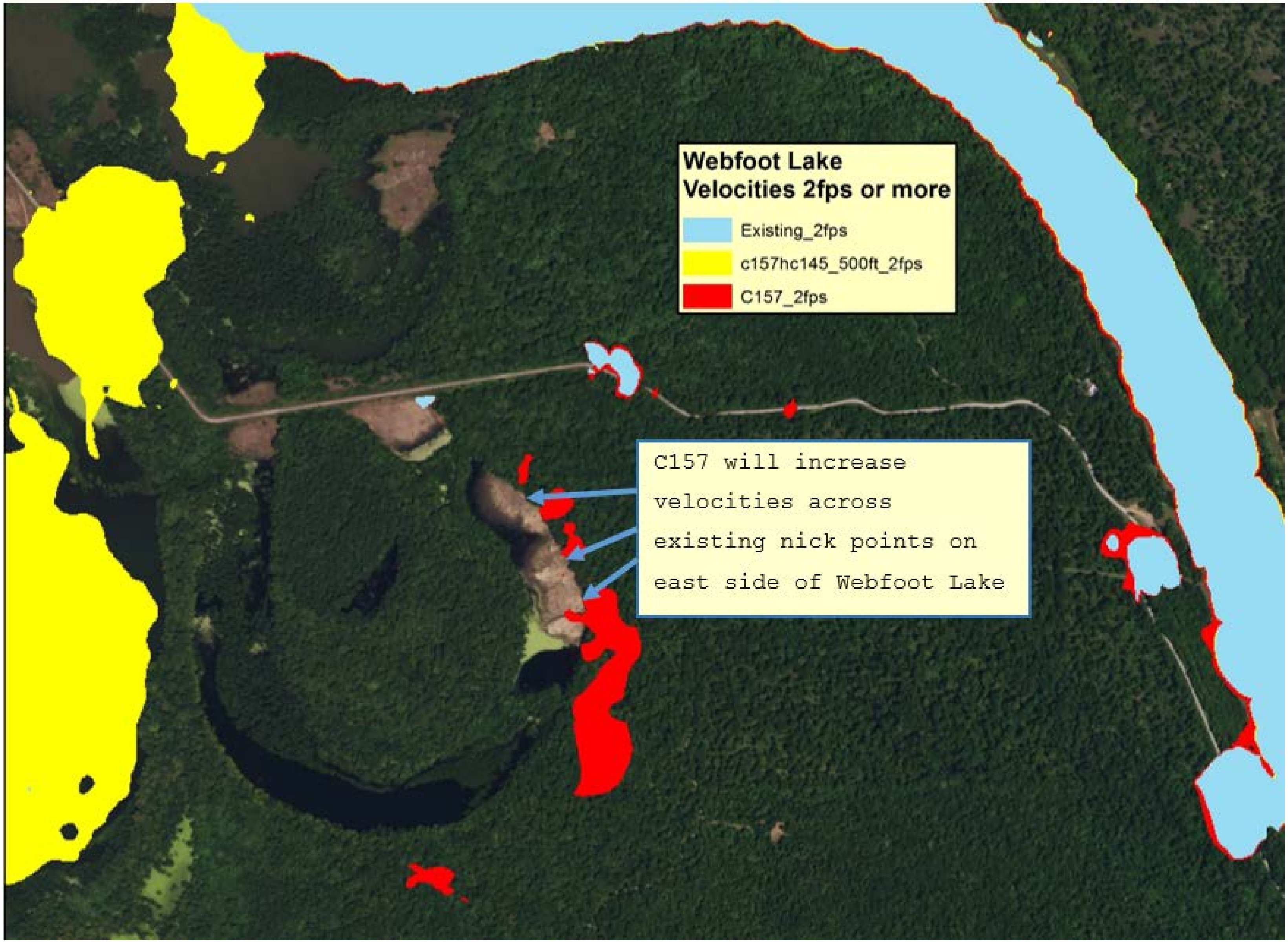
Webfoot Lake
Velocities 2fps or more
Existing_2fps
c157hc145_500ft_2fps
C157_2fps
C157 will increase velocities across existing nick points on east side of Webfoot Lake

Figure 5-9: Velocities 2 Feet per Second or More: Webfoot Lake: Existing Conditions, C157HC145_500ft, and C157

5.2 Flood Duration Maps

Percent time inundated grids for the growing season (15 March through15 November) for the period of record (2000 through 2014) were produced for each alternative and compared to existing conditions. To identify areas most affected by each alternative, the U.S. Fish and Wildlife, Arkansas Game and Fish Commission, and the Arkansas Natural Heritage Commission, requested that percent time inundated grids be changed into grids that identify areas that would experience an average of seven days or more inundation and seven days or less of inundation during the growing season (ATTACHMENT B contains inundation maps).

5.3 Refuge Landform, Microsite, Elevation: Seasonal Inundation Duration

U.S. Fish and Wildlife provided a polygon shapefile containing Landform, Microsite topography delimited by elevations that the study team used to categorize changes in inundation days for the Refuge assuming a growing season of 15 March through 15 November (Figure 5-10 and Table 5-5).

Table 5-5: Change in the Number of Average Annual Days Inundated based on Refuge Landform, Microsite, and Elevation for Period of Record 2000 through 2014

Landform, Microsite based on Elevation	Average No. Days Existing	Change in Average Number of Days from Existing Conditions (-) Drier (+) Wetter: Alternative 1 C157			Alternative 2 M115		
		C157	C157H C145 500ft	C157HC145 1000ft	M115	M125	M135
PVL2 Flats below 147.5 feet	50	0	0	0	(4)	(4)	(4)
PVL2 Flats above 147.5 feet	13	1	0	0	(8)	(8)	(8)
HPS Ridges below 145 feet	42	0	0	0	(2)	(2)	(2)
HPS Ridges above 145 feet	20	1	0	0	(4)	(4)	(4)
HPS Natural Levees below 145 feet	55	0	0	0	0	0	0
HPS Natural Levees above 145 feet	13	1	0	0	(7)	(7)	(7)
HPS Flats below 142 feet	66	0	0	0	0	0	0
HPS Flats above 142 feet	43	0	0	0	(3)	(3)	(3)
Three Rivers back swamp final	73	0	0	0	0	0	(1)

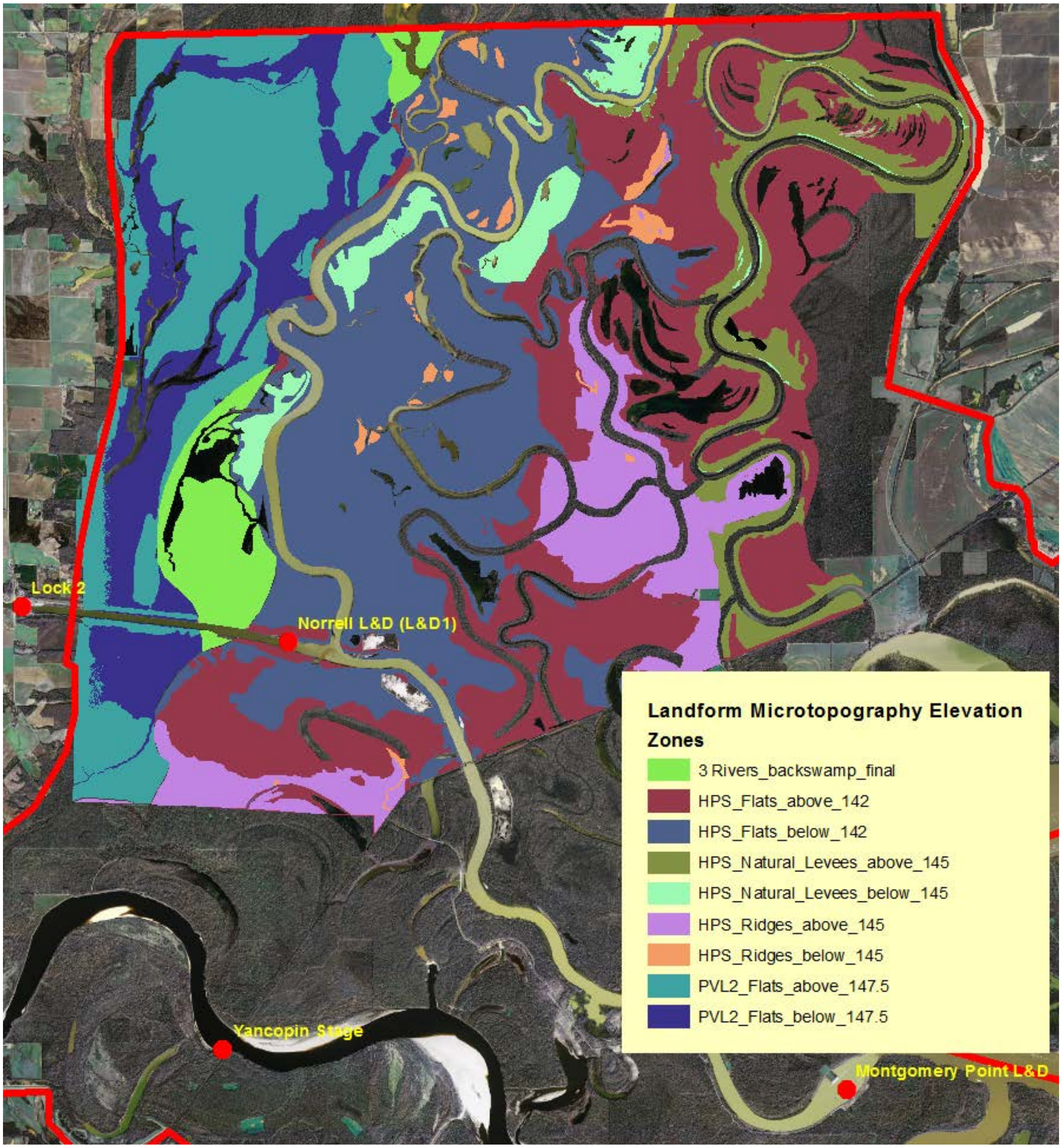

Figure 5-10: Landform Microsite Elevation Zones

5.4 Exceedance Duration: Oxbow Existing Outlets

Exceedance duration for existing conditions, C157HC145_500ft and C157HC145_1000ft are the same. Existing fish passage into LaGrues Lake takes place through three corrugated metal culverts around an elevation of 138 feet with an annual exceedance duration of 22.7 percent and through two corrugated metal culverts around elevation 129 feet with an annual exceedance duration of 47.4 percent (Figure 5-11). Owens Weir, elevation 145 feet, with an annual exceedance duration of 9.7 percent, must overtop before fish can migrate in or out of the lake (Figure 5-12 and Table 5-6).

Owens Lake connects to the White River at elevation 145 feet over Owens Weir and connects to the Arkansas River at elevation 140 feet over the Melinda Weir (Melinda Structure). Current weir, or structure, elevations give the Arkansas River greater influence on Owens Lake hydrology than the White River. The Three Rivers selected plan includes removing the Melinda Weir and adding a containment structure at a higher elevation (157 feet) just south of the Melinda Weir. This would shift hydrological influence to the White River, and assuming no leakage through Owens Weir or the new containment structure, would raise the maximum elevation of Owens Lake from 140 to 145 feet. The additional 5 feet in elevation has the potential to flood an additional 100 acres of bottomland hardwoods. ATTACHMENT F displays results for sizing of the Owens Weir and outlet structure.

Table 5-6: Lake Recharge Elevation Duration Exceedance

Lake Recharge Percent Time Elevation Duration Exceedance Existing Conditions, C157HC145_500ft, and C157HC145_1000ft are statistically Identical		
Oxbow	**Recharge Elevation (White River) (feet)**	**Annual Exceedance**
LaGrues Lake (3 culverts)	138	22.7%
LaGrues Lake (2 Culverts	129	47.4%
Owens Lake (Weir)	145	9.7%

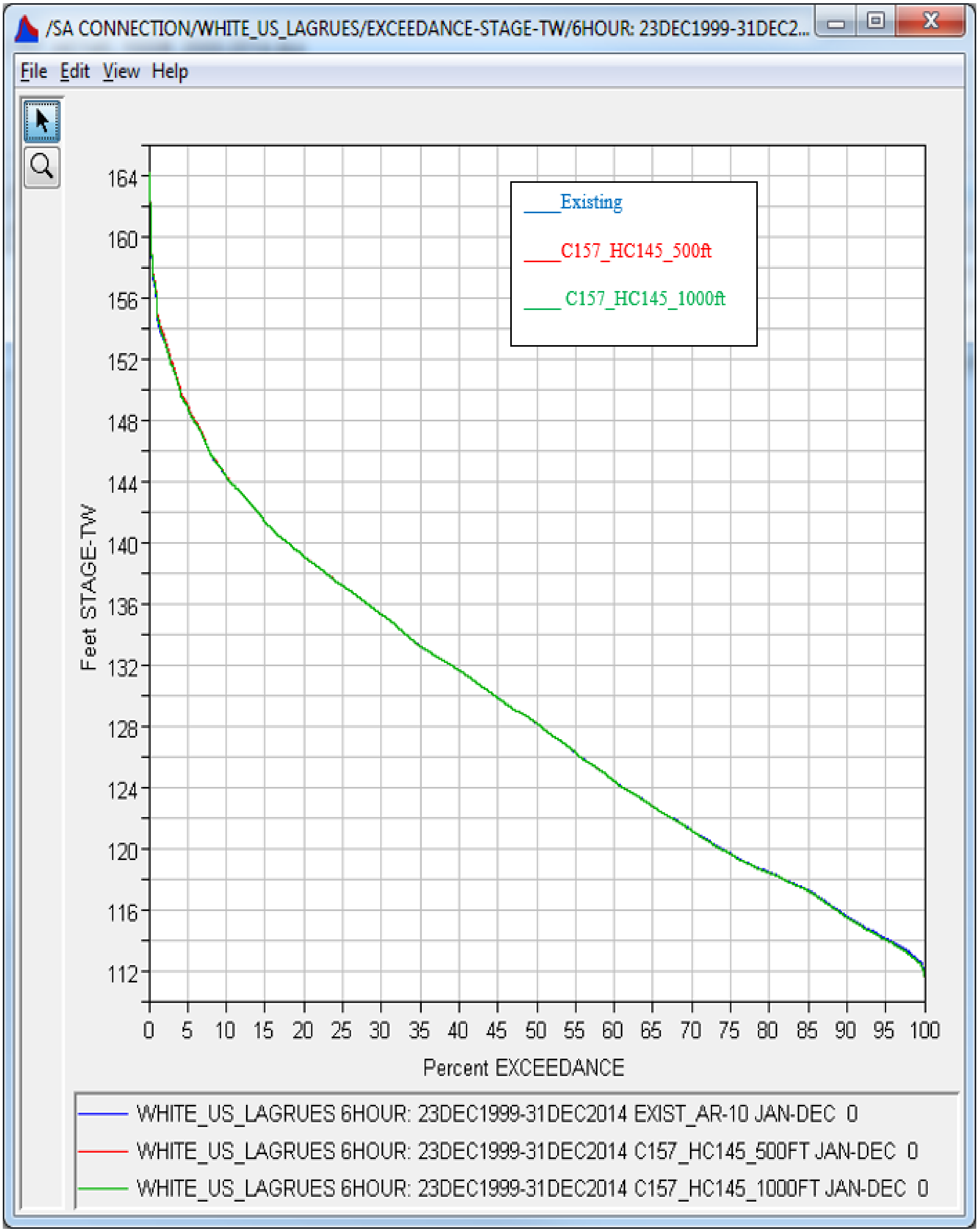

Figure 5-11: Elevation Exceedance Duration: White River La Grues Lake Outlet

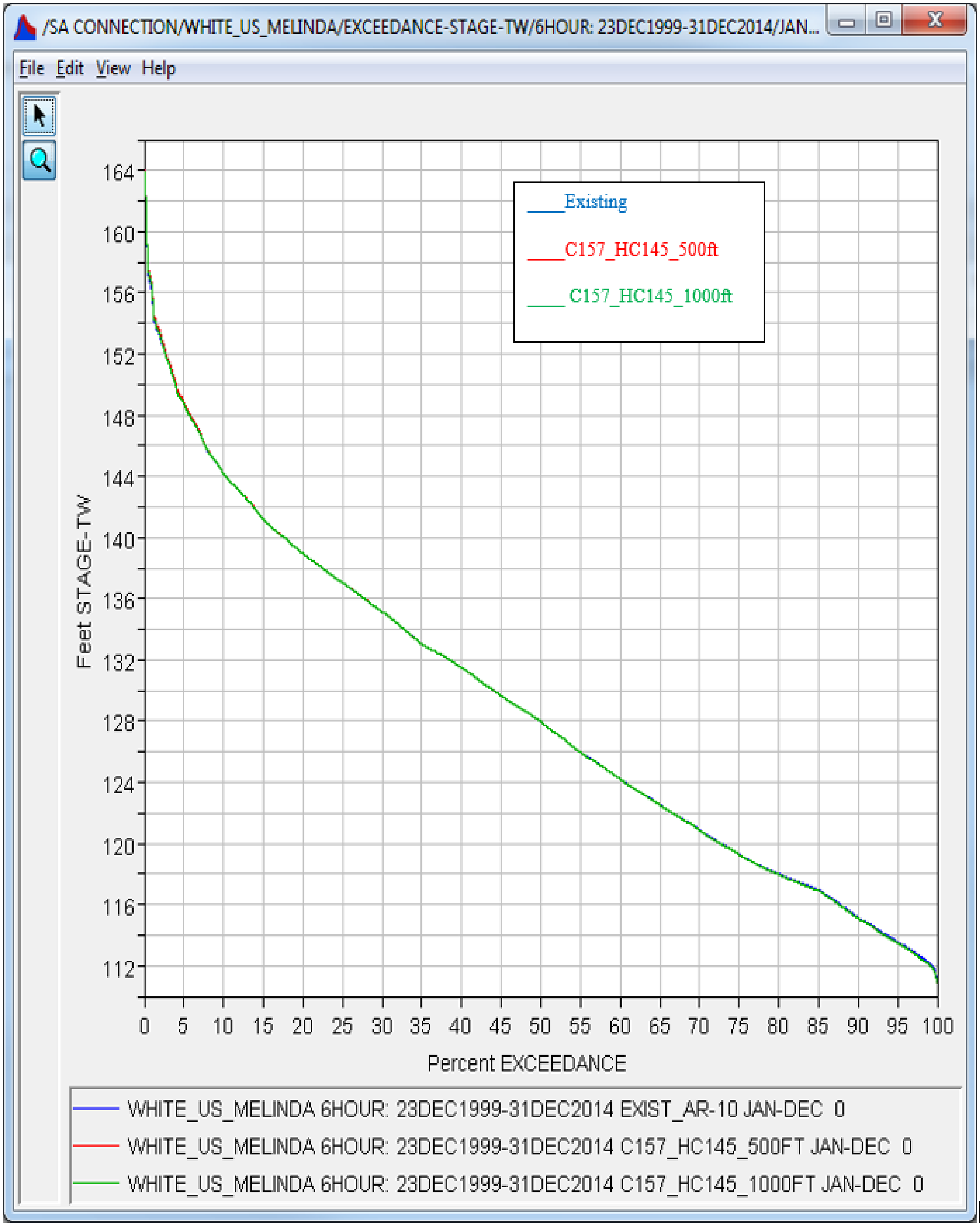

Figure 5-12: Elevation Exceedance Duration: White River Owens Lake Weir

5.5 Exceedance Duration: Areas of Interest

ATTACHMENT C contains exceedance duration analysis for locations identified in Figure 5-13.

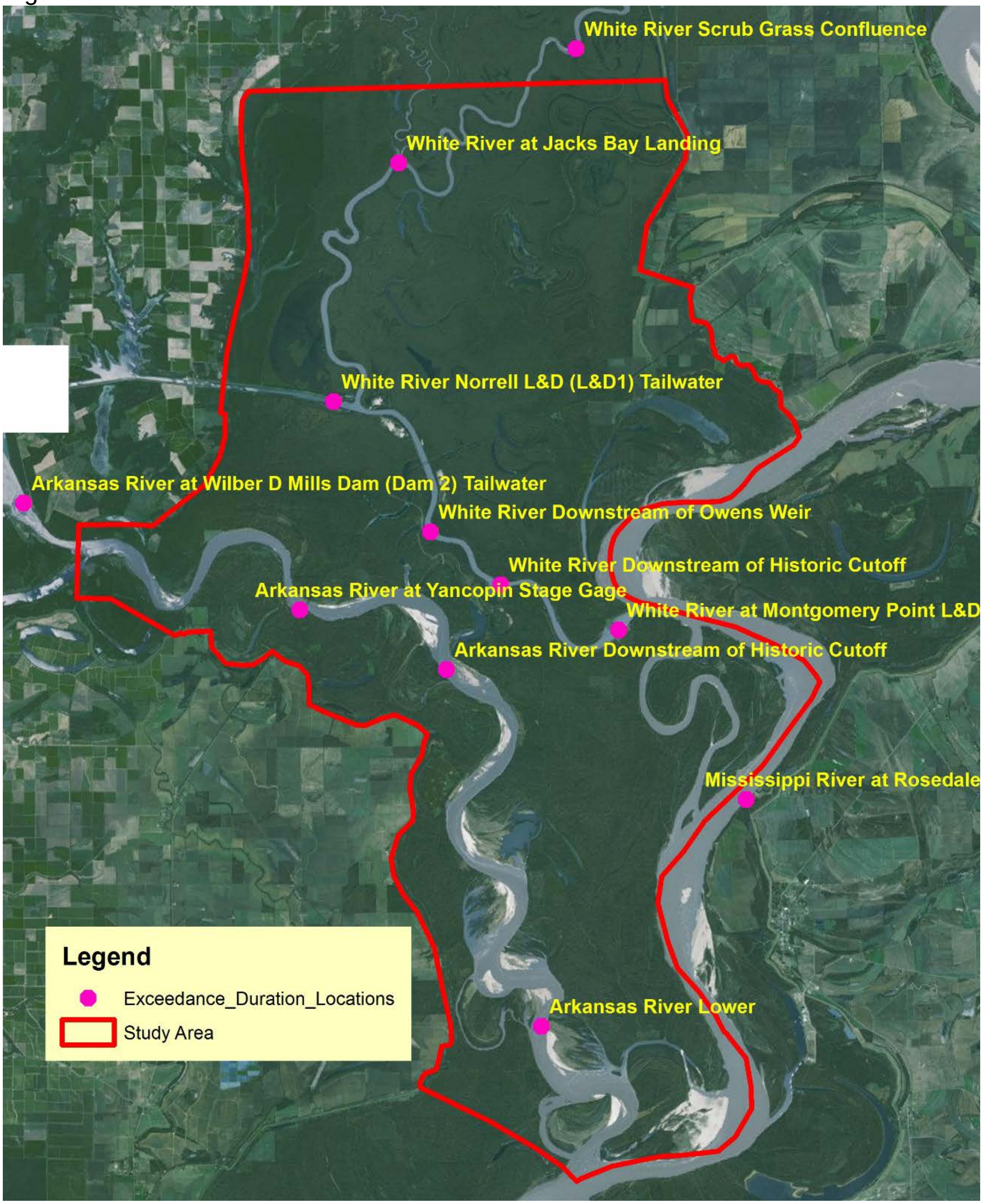

Figure 5-13: Elevation Exceedance Duration: Areas of Interest

5.6 Floodplains

5.6.1 2 year and 5 year Floodplains: Environmental Effects

Figure 5-14 and Figure 5-15 display floodplain inundations maps for the existing condition in the study area, C157, and M135. C157HC145_500ft and C157HC145_1000ft maps are almost identical to existing conditions. Floodplain inundation is essentially the same across alternatives and therefore is not a significant factor in plan selection. However, results do confirm that alternatives would have minimal impacts to Refuges hydrology.

5.6.2 100-year Floodplain FEMA

The project area is FEMA Zone A, which means an alternative may not have a cumulative rise in the Base Flood Elevation (BFE, 1 percent exceedance frequency) of more than 1.00 foot. Floodplains for C157HC145_500ft and C157HC145_1000ft do not exceed the allowable 1.00 foot cumulative rise (Table 5-7 shows change in 100 year elevations and locations of gages). The 100-year floodplain inundation map for C157HC145_500ft and C157HC145_1000ft and existing conditions were the same with less than 0.05 feet difference in water surface elevations.

Table 5-7: Change in 100 year Elevations

Location	Maximum 100 Year Water Surface Elevation Difference from Existing in feet	
	C157HC145_500ft	C157HC145_1000ft
Arkansas: Wilber D Mills (Dam2) Gage	(0.01)	(0.03)
Arkansas: Yancopin Gage	(0.04)	(0.01)
Arkansas River at Melinda Confluence	0.00	0.00
Arkansas River at Historic Cutoff Confluence	0.05	0.05
Arkansas River 11 miles downstream of confluence with Historic Cutoff	0.00	0.00
Owens Lake downstream of Melinda Weir	0.18	0.18
Owens Lake upstream Melinda Weir	0.03	0.03
Mississippi: Rosedale Gage	(0.01)	(0.02)
White River at Historic Cutoff Confluence	0.06	0.06
White River at Melinda Confluence	(0.01)	(0.01)
White: Hudson Landing Gage	0.01	(0.01)
White: Norrell Lock and Dam (LD01) Gage	0.02	0.00
White: Montgomery Point Lock and Dam Gage	(0.01)	(0.04)

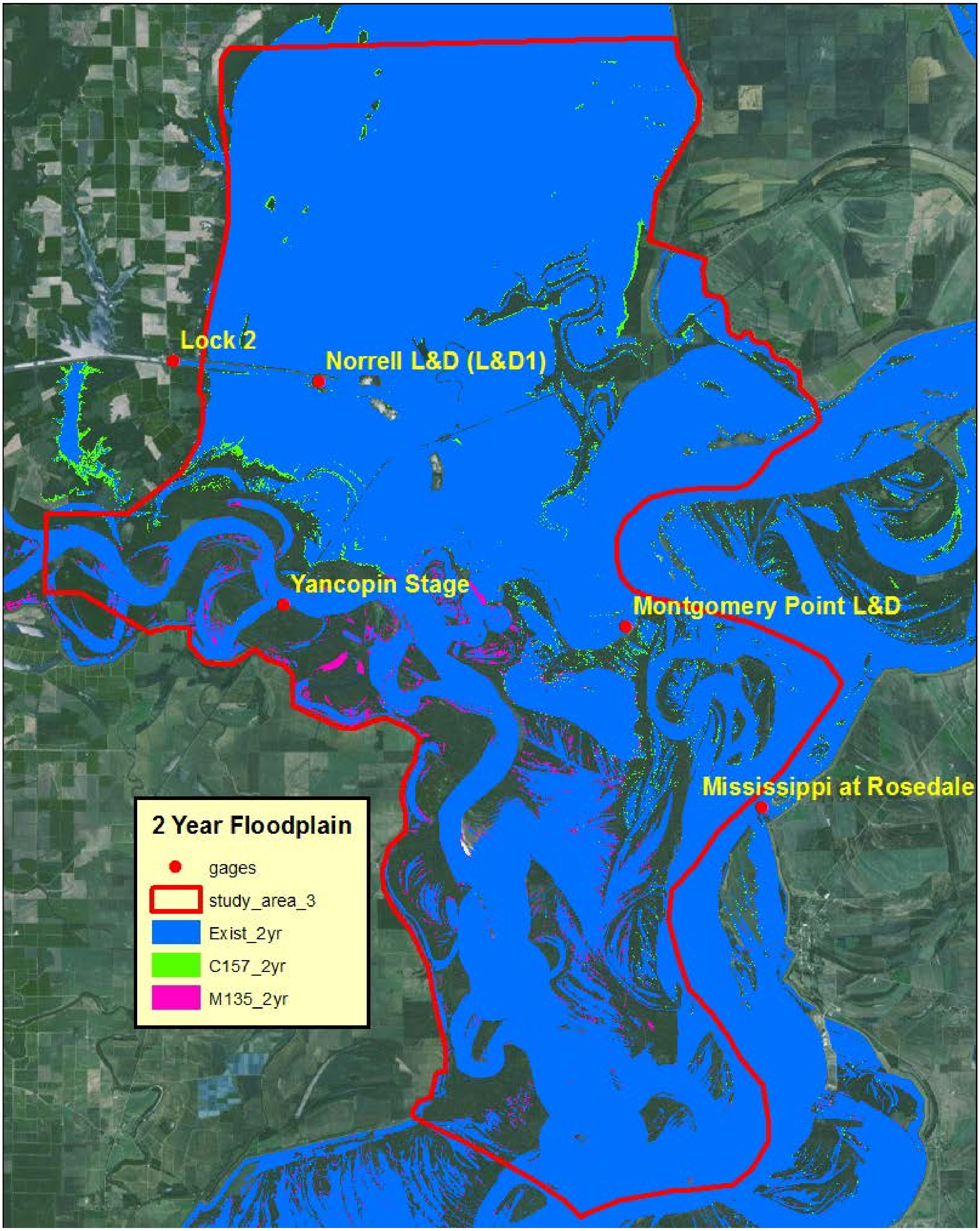

Figure 5-14: 2 Year Floodplain

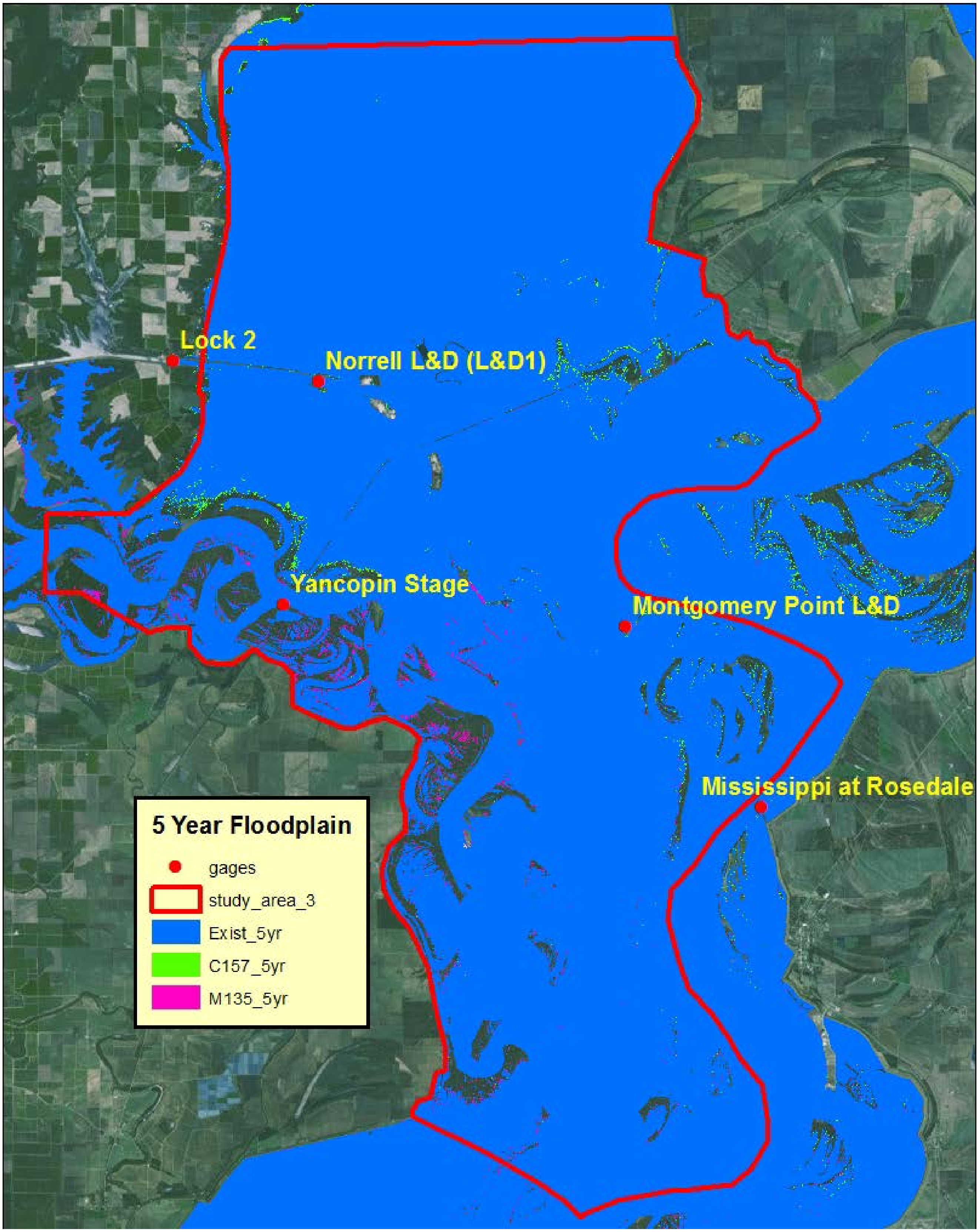

Figure 5-15: 5 Year Floodplain

6 Climate Change

Team hydrologists relied on the Climate Preparedness and Resilience COP Applications Portal (USACE-1, n.d.) to analyze potential impacts of climate change as directed in ECB No. 2016-25 (ECB No. 2016-25: USACE, 2016). Two tools are available for this purpose:

1) The Non-stationarity Detection Tool (NDT) that enables users to apply a series of statistical tests to assess the stationarity of annual instantaneous peak streamflow data series at any USGS streamflow gage site with more than 30 years of annual instantaneous peak streamflow records; and,

2) The Climate Hydrology Assessment Tool (CHAT) that allows users to access both existing and projected climate data to develop repeatable analytical results using consistent information. CHAT guides users through the process of developing information and supplies graphics suitable for use in a report including: trend detection in observed annual maximum daily flow, and trend detection in annual maximum monthly flow models (USACE-4, n.d.).

Both NDT and CHAT indicated that there are no statistically significant trends in annual peak instantaneous streamflow or projected annual maximum monthly flows in the selected gages upstream and downstream of the study area.

6.0 The Climate Hydrology Assessment Tool

P-values measure statistical significance for a fitted regression line; smaller values indicate a greater degree of statistical significance. Although, there is no formal threshold for statistical significance, 0.05 is common as this is associated with a 5 percent risk of a Type I error or false positive (USACE-2, n.d.). Neither annual peak instantaneous streamflows or projected annual maximum monthly flows for four gages upstream and downstream of the study area demonstrated statistical significance. Annual peak instantaneous stream flow at all four sites have a slight upward trend and all but one had a slight upward trend in projected annual maximum monthly flows - 1111 Lower Arkansas River, but again, none were statistically significant. (Figure 6-1 and Figure 6-2 and Table 6-1 and Table 6-2).

Table 6-1: CHAT: Annual Peak Instantaneous Streamflow P-Value for Selected Gages

HUC Basin Name	Site Number and Gage Name	Annual Peak Instantaneous Streamflow P-Value
1111 Lower Arkansas River	7249455 ARKANSAS RIVER AT FT. SMITH, AR	0.81+
1101 Upper White River	7074850 WHITE RIVER NEAR AUGUSTA, AR	0.43+
0802 Lower Miss. St Francis	7076750 WHITE RIVER AT GEORGETOWN, AR	0.77+
0809 Lower Miss. River	7289000 MISSISSIPPI RIVER AT VICKSBURG, MS	0.63+

Notes:
- = downward trend
+ = upward trend
The p-value is for the linear regression fit drawn; a smaller p-value would indicate greater statistical significance. There is no recommended threshold for statistical significance, but typically 0.05 is used as this is associated with a 5 percent risk of a Type I error or false positive.

Table 6-2: CHAT: Projected Annual Maximum Monthly Flow P-Value HUC-4

HUC Basin Name	Projected Annual Maximum Monthly Flow (2000 – 2099) P-Value
1111 Lower Arkansas River	0.055-
1101 Upper White River	0.365+
0802 Lower Miss. St Francis	0.311+
0809 Lower Miss. River	0.872+

Notes:
- = downward trend
+ = upward trend
The p-value is for the linear regression fit drawn; a smaller p-value would indicate greater statistical significance. There is no recommended threshold for statistical significance, but typically 0.05 is used as this is associated with a 5 percent risk of a Type I error or false positive.

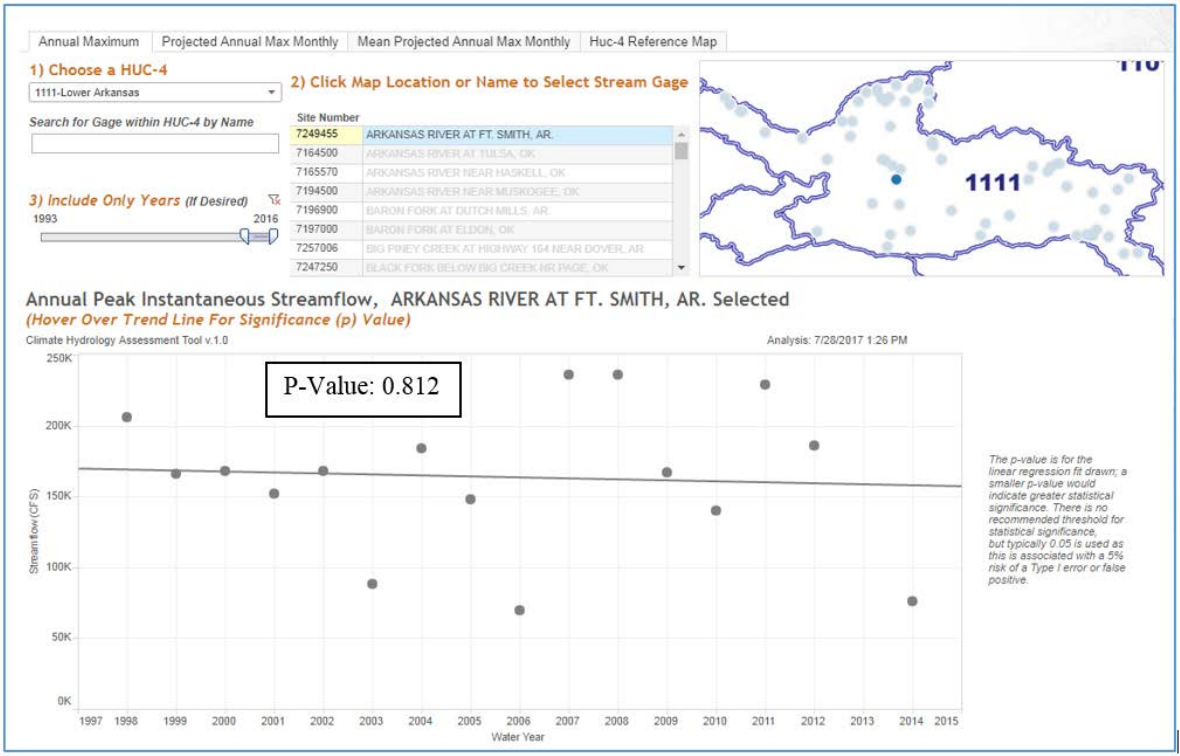

Figure 6-1: CHAT Arkansas River at Ft. Smith, Arkansas, Annual Peak Instantaneous Streamflow P-Value

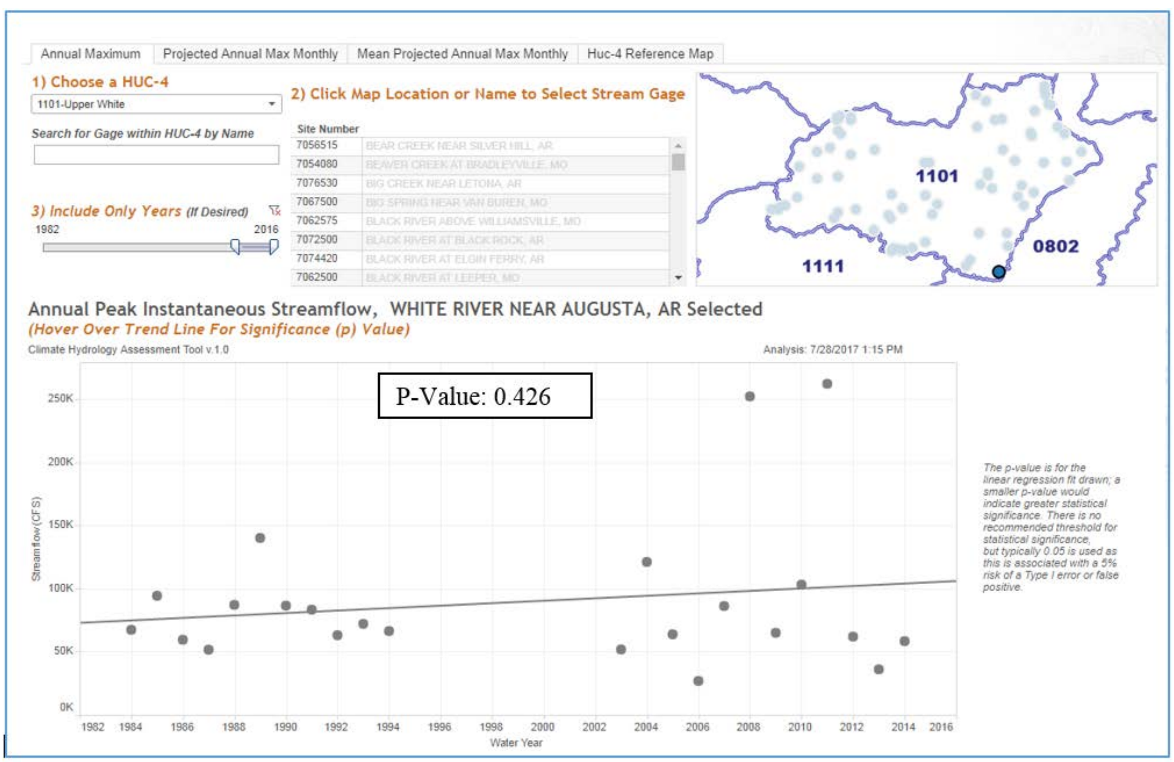

Figure 6-2: CHAT White River near Augusta, Arkansas Annual Peak Instantaneous Streamflow P-Value

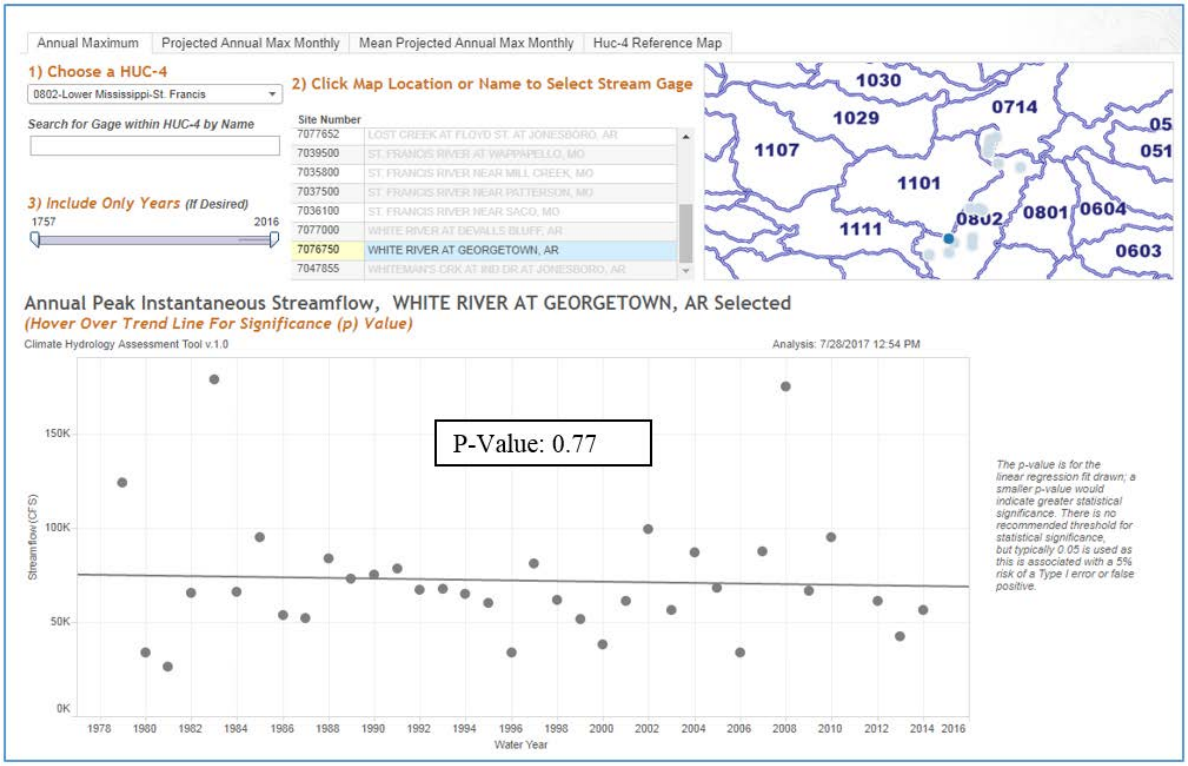

Figure 6-3: CHAT White River at Georgetown, Arkansas Annual Peak Instantaneous Streamflow P-Value

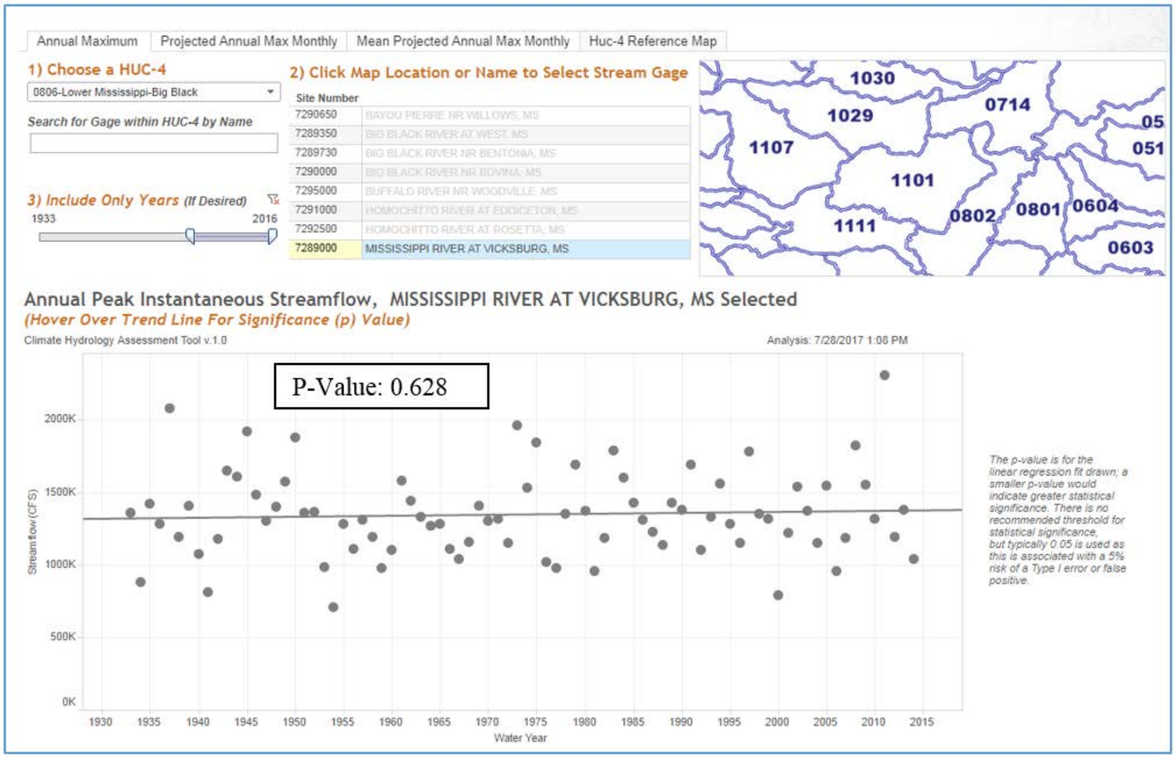

Figure 6-4: CHAT Mississippi River at Vicksburg, Mississippi Annual Peak Instantaneous Streamflow P-Value

6.1 The Non-stationarity Detection Tool

The NDT indicated a lack of trends along the main reach of concern for the Three Rivers Project. There were no significant nonstationarities or significant trends at the nearest USGS gages on the White, Arkansas, and Mississippi rivers (Figure 6-5 through Figure 6-7).

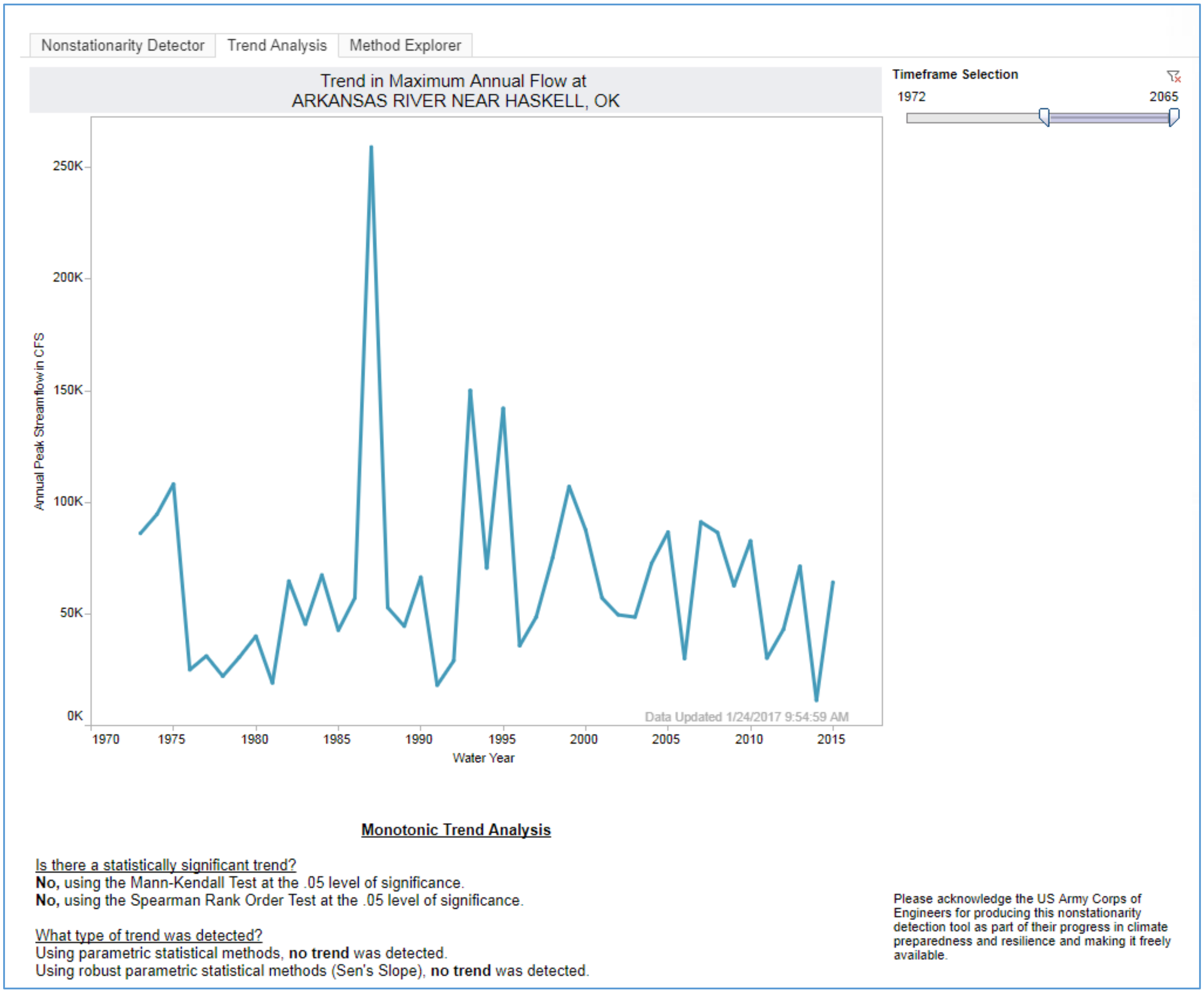

Figure 6-5: NDT Trend in Maximum Annual Flow at Arkansas River near Haskell, Oklahoma

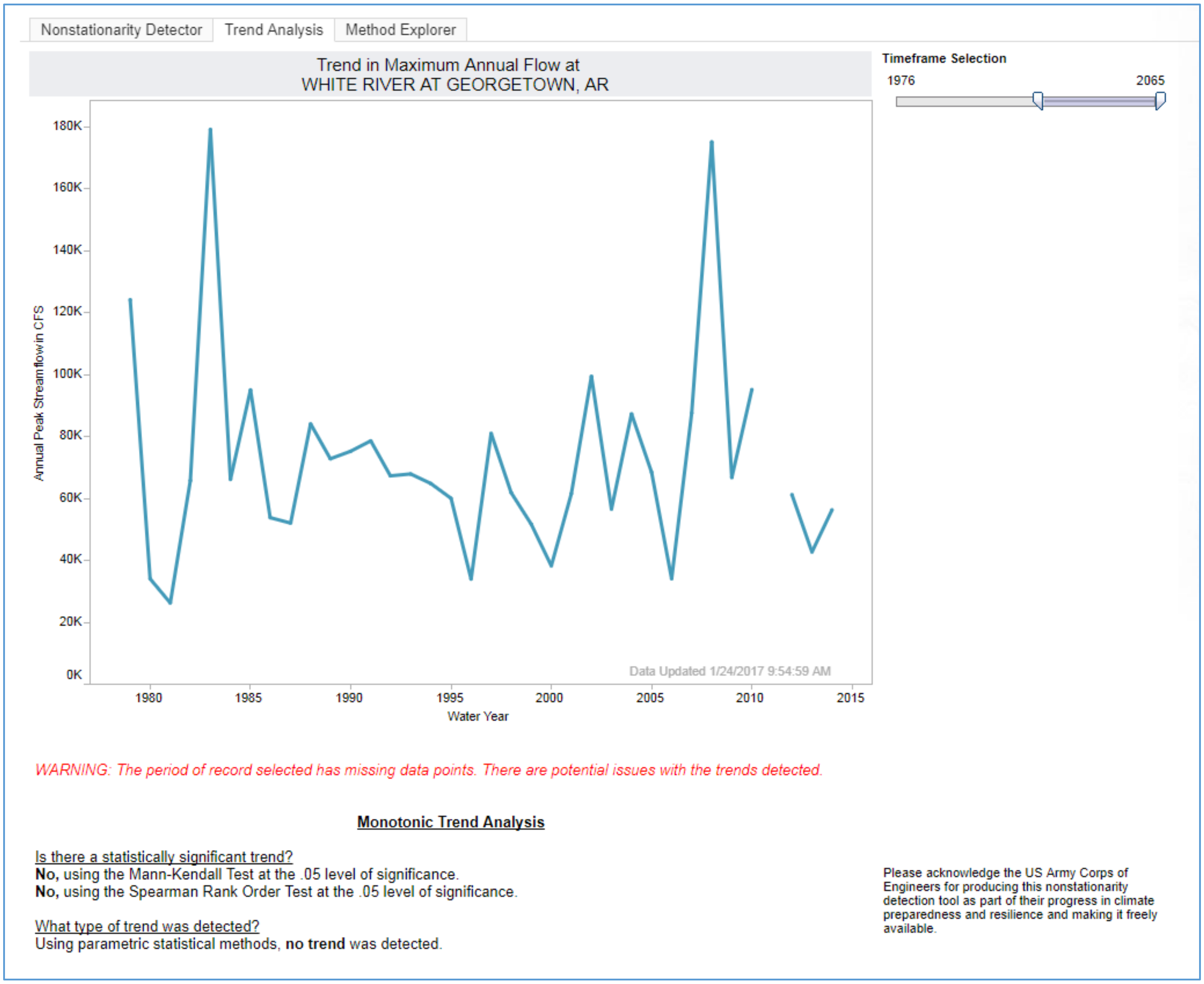

Figure 6-6: NDT Trend in Maximum Annual Flow at White River at Georgetown, Arkansas

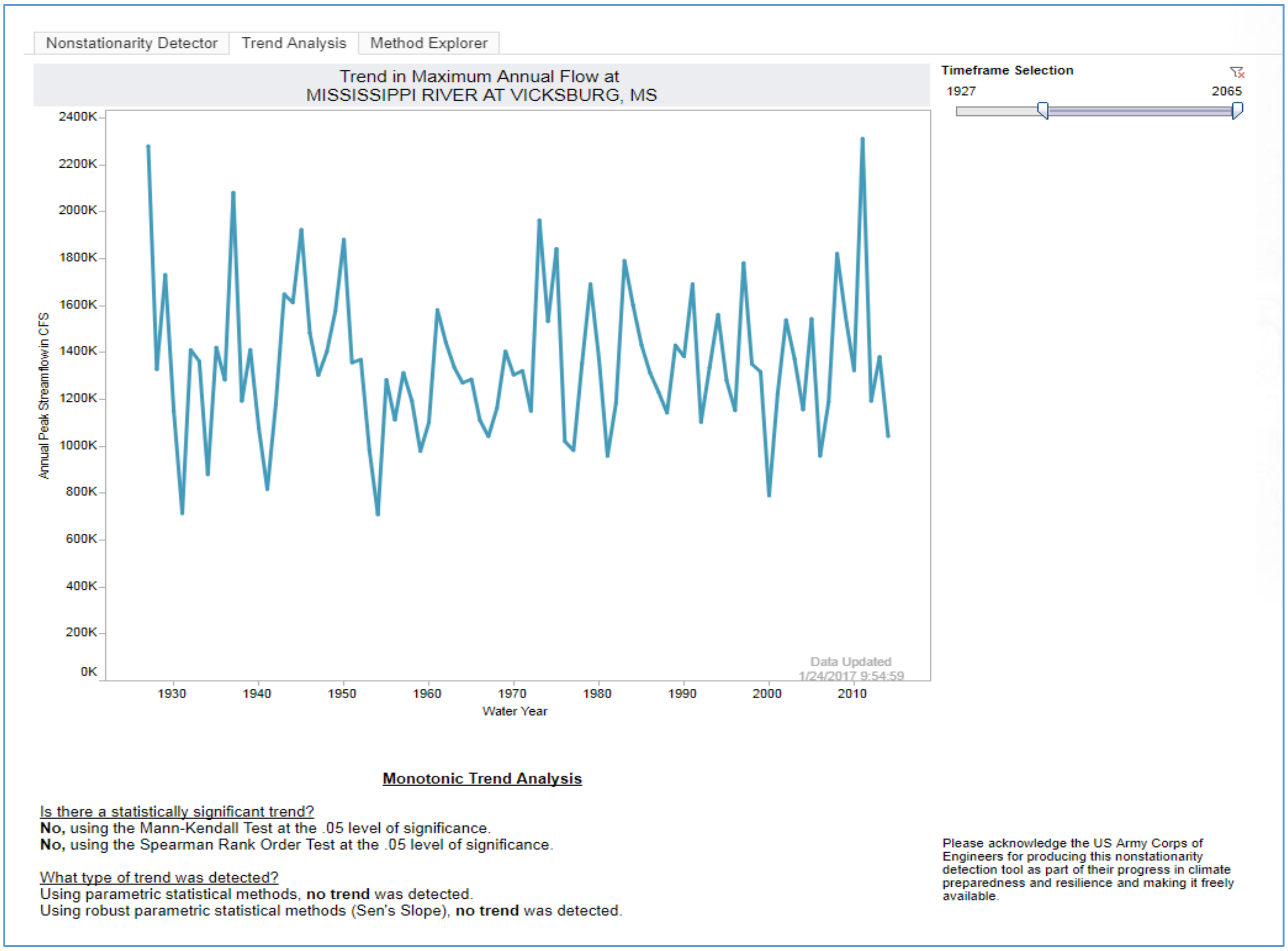

Figure 6-7: NDT Trend in Maximum Annual Flow at Mississippi River at Vicksburg, Mississippi

7 Future Modeling and Studies during PED

7.0 Ship Tow Simulator for Cross Current

Alternatives C157HC145_500ft and C157HC145_1000ft would open the Historic Closure Structure with a relief structure down to an elevation of 145 feet. The final width of the opening would rely on the maximum width of the opening, which would minimize scour and erosion in the isthmus without introducing strong cross currents into the navigation channel. Today, the only location where cross currents occur in the project area is at the Owens Lake Weir with a crest at elevation of 145 feet and a width of about 950 feet. Minimal to no negative impacts occur from cross currents at this location and configuration. The largest proposed opening in the selected plan is also at elevation 145 feet with a width of 1,000 feet, which is only 50 feet wider than Owens Lake Weir. But, the Historic Cutoff has a higher discharge capacity due to the absence of trees and wider channel corridor. A ship tow simulator will be used to maximize the weir width while minimizing negative effects cross currents may have on the shipping industry.

Two-dimensional mathematical hydraulic models can estimate in-channel velocities, but they do not capture variables such as tow boat capabilities, barge number and configuration, and ship captain experience. As discussed previously, results of a ship tow simulator will provide the upper limits on the width of the proposed relief structure. The Little Rock District is currently working with ERDC on the required inputs for the ship tow simulator. The shipping industry has been contacted and is providing feedback on barge/tow combinations, two licensed pilots, and other factors for the ship tow simulation exercise that will be completed during PED.

Although, unlikely, If the ship tow simulator indicates a significant increase in cross-currents at the 500 foot width, then a duration analysis of these cross-currents will be necessary to quantify impacts to navigation. If cross-currents only occur for a few days, or occur during industry self-regulated closures, then impacts to navigation would be minimal.

7.1 Velocity and Shear Stress in White River

The proposed opening through the Historic Closure Structure may increase the velocity and shear stress near the confluence of the Historic Cutoff and the White River. Scour protection in this location may be necessary if the current protection is not adequate. Additional scour protection or stone dikes may be necessary on the upstream and downstream side of the proposed opening to reduce erosion and scour. Scour protection will be designed during the PED phase.

7.2 Geomorphological

Additional work will include final structure orientation and to predict changes that will occur in the Historic Cutoff in response to reopening the Historic Closure Structure as described in the Selected Plan and to provide feedback and suggestions on minimal channel stabilization measures to maintain a healthy channel alignment of the Historic Cutoff. Leaving out bank stabilization and river training structures in the Historic Cutoff will allow the cutoff to develop a natural and balanced stream morphology that will reduce construction costs and eliminate the need for future OMRRR.

The Historic Cutoff will be allowed some lateral migration which is normal and necessary for healthy geomorphological processes of alluvial rivers, but will be constrained at the northern end by the proposed structure. This migration will lead to erosion of the outer meander bend, and the build-up of point bars on the inside meander bends. To allow for the natural migration of the Historic Cutoff channel, up to 300 acres of land may need to be purchased. Although, depending on the final structure orientation, a large portion of the land most affected by possible Historic Cutoff channel migration is already in the Arkansas River channel migration path. See Figure 7-2.

A geomorphic assessment was completed in April 2003 for the previous Ark White Study with primary attention given to the Arkansas River and how its migration path might affect the Melinda Weir and the Historic Closure Structure. See ATTACHMENT G: for report and study results. See Figure 7-1 for the Arkansas River bank lines projected in the 2003 geomorphic report. The bank retreat and migration has been lateral and down-valley, but has generally moved more in the down-valley direction rather than laterally eastward toward the Historic Closure Structure as predicted in the 2003 report. See Figure 7-2 for bank line migration estimated from 2001 to 2017 aerials.

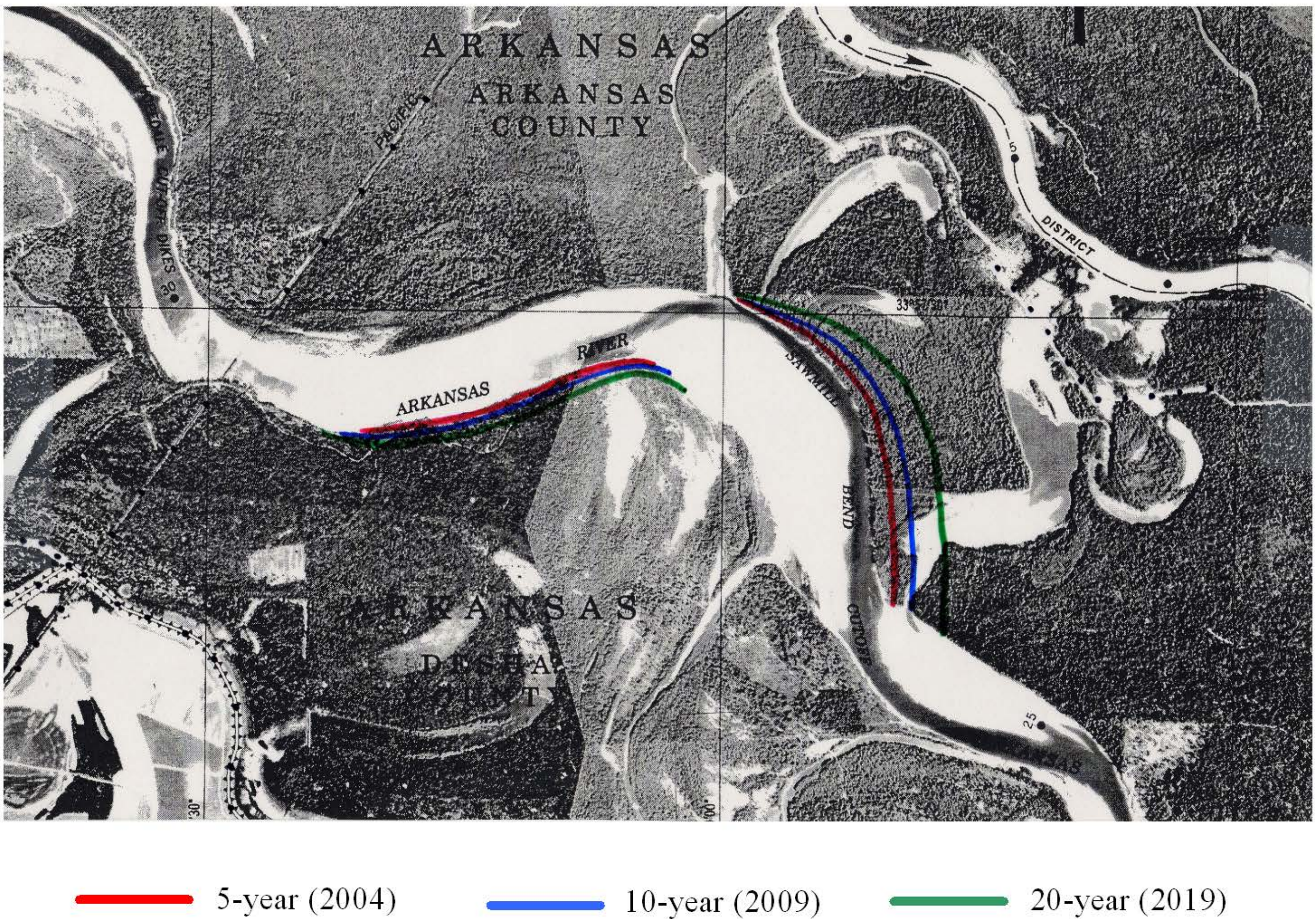

Figure 7-1: Arkansas River Projected Bankline Locations

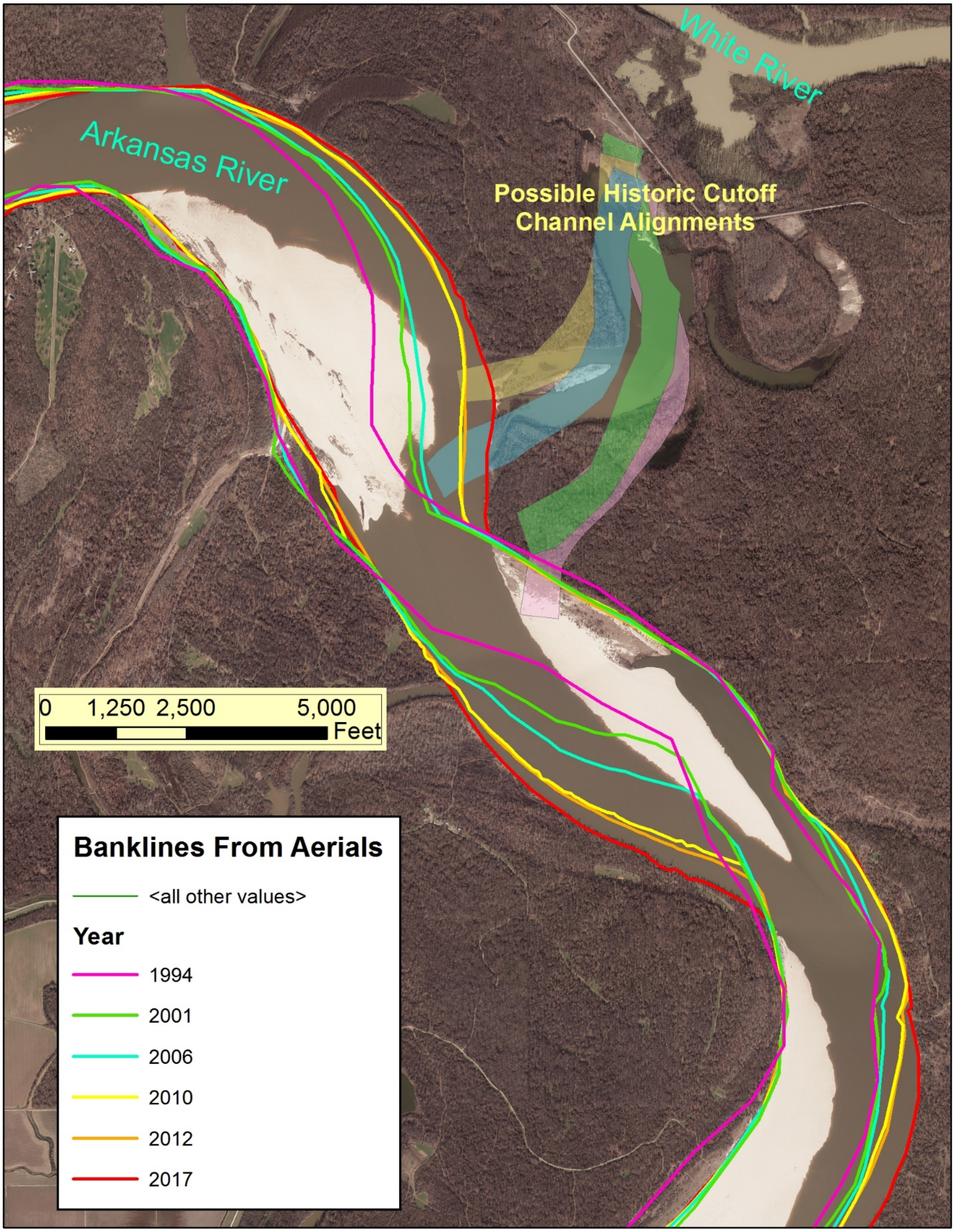

Figure 7-2: Arkansas River Banklines from Aerials

Updated geomorphological studies will be completed prior to the PED phase. The updated study will provide a better estimation of the migration rate and future location of the Arkansas River. The project has been submitted to the Regional Sediment Management (RSM) Program, Mississippi River Geomorphology and Potamology (MRG&P) Program and to the Dredging Operations Technical Support (DOTS) Program for FY18. Another avenue being investigated is ERDC's Technology Transfer and Outreach Division program

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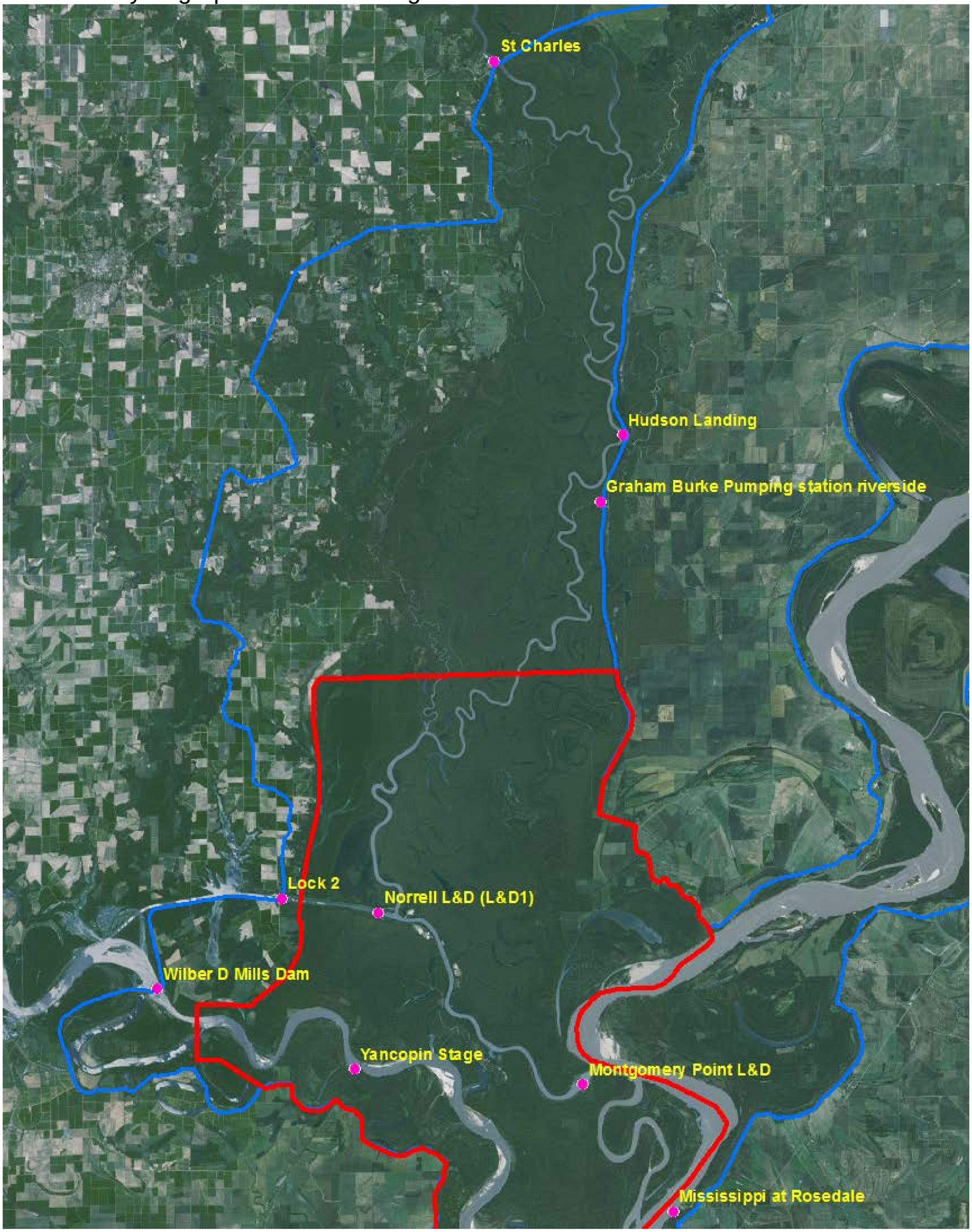
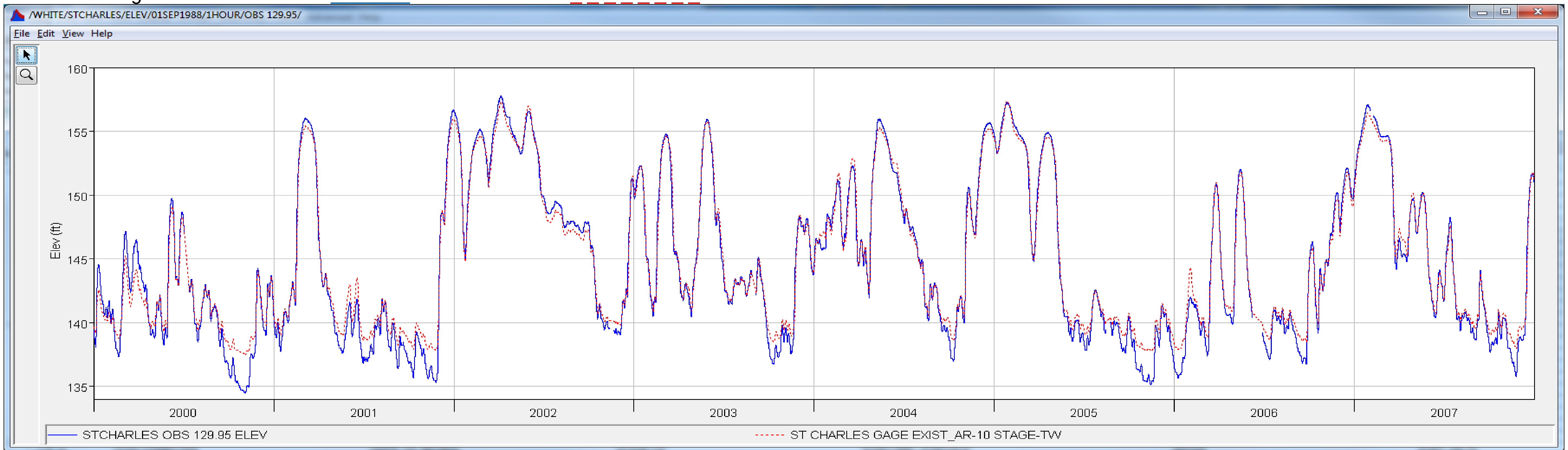
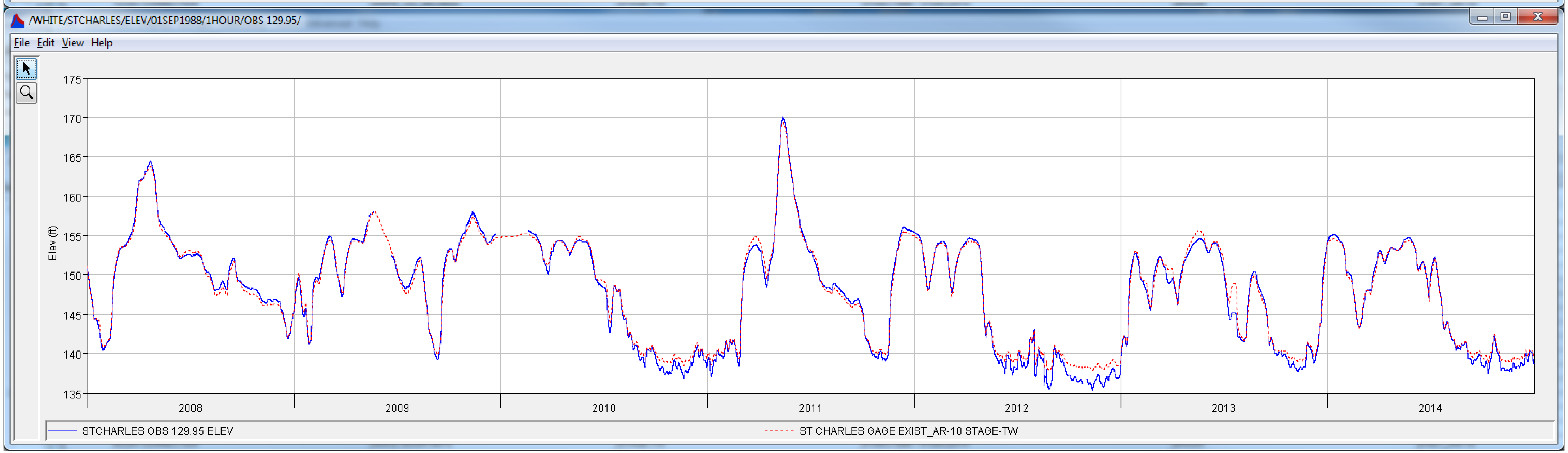
ATTACHMENT A **Calibration Hydrographs**

Elevation Hydrograph Calibration Gage Locations

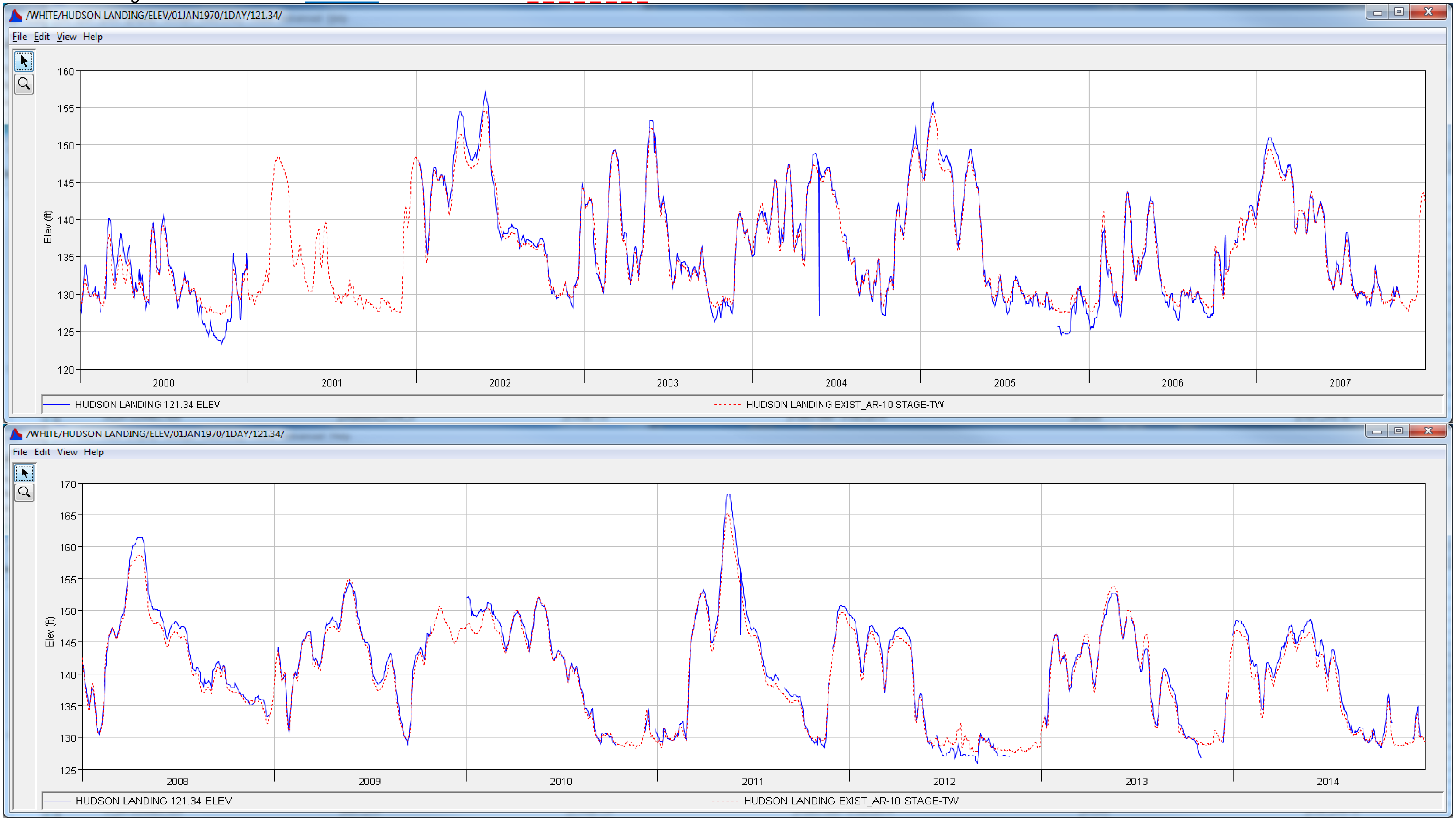


St Charles Gage: White River Observed Modeled

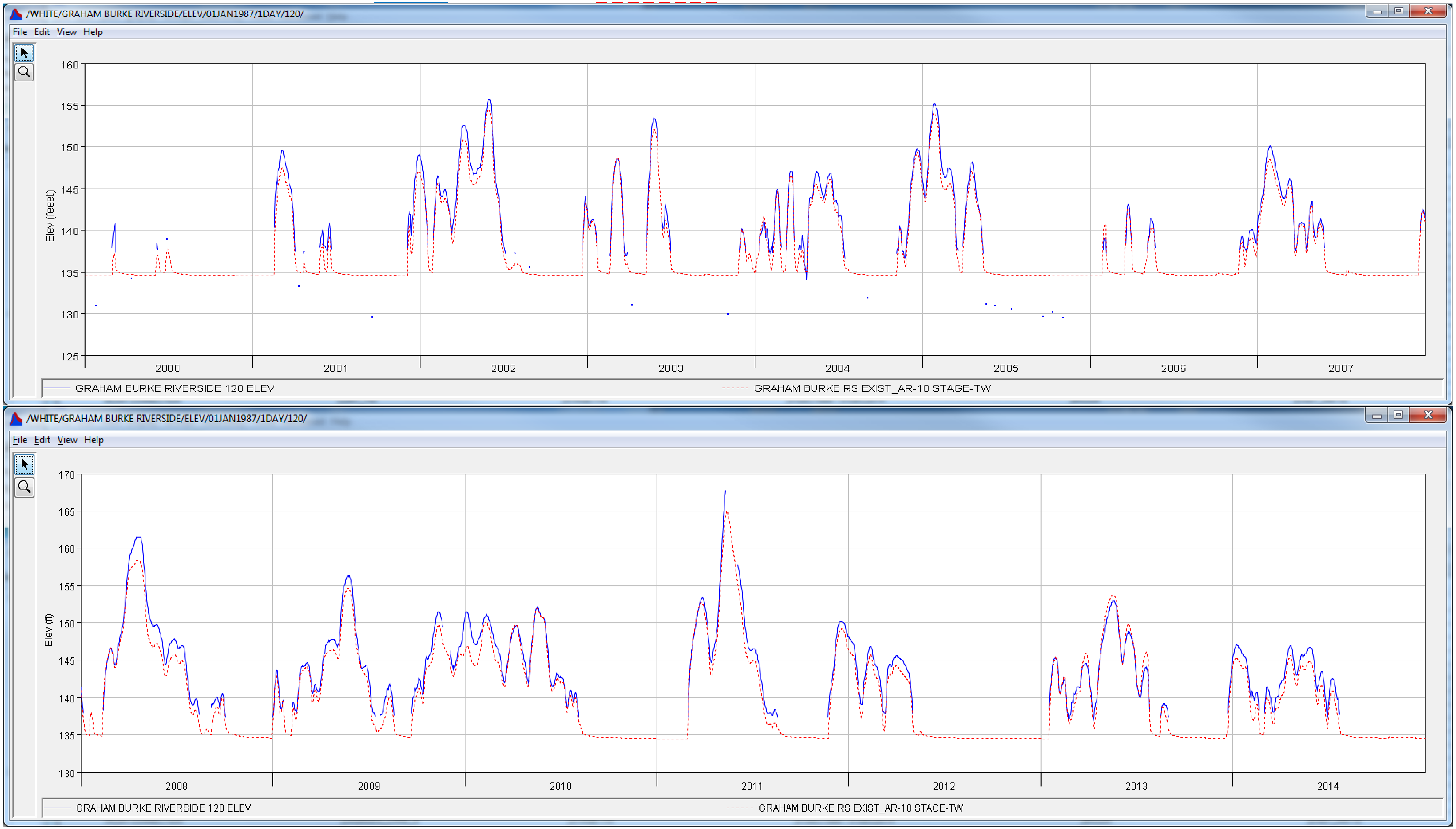




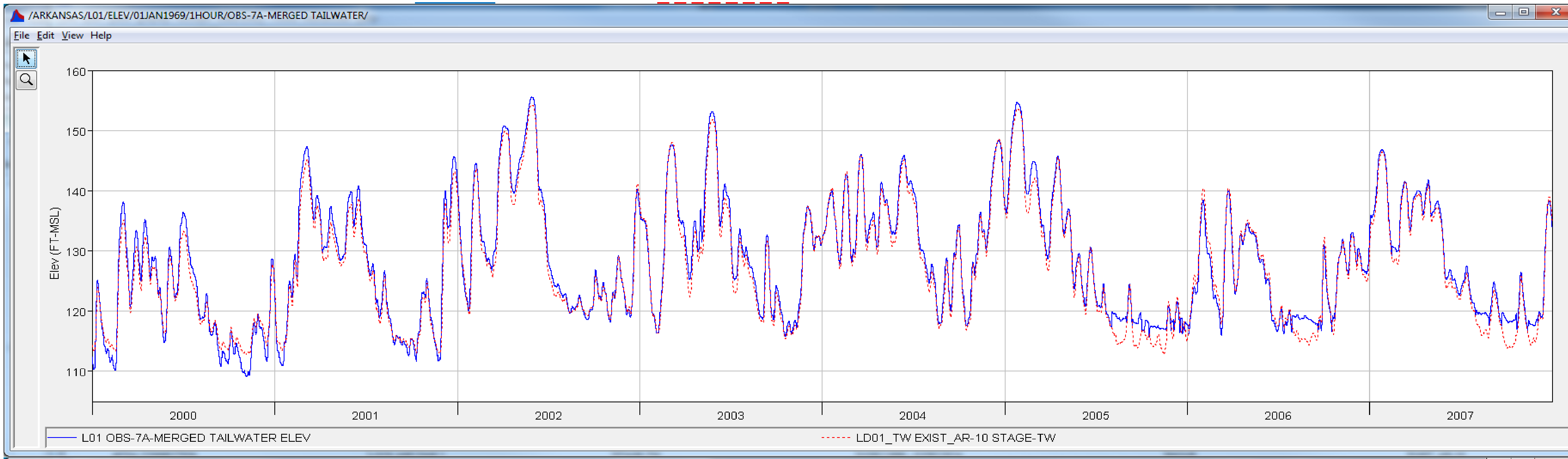
Hudson Landing: White River Observed Modeled

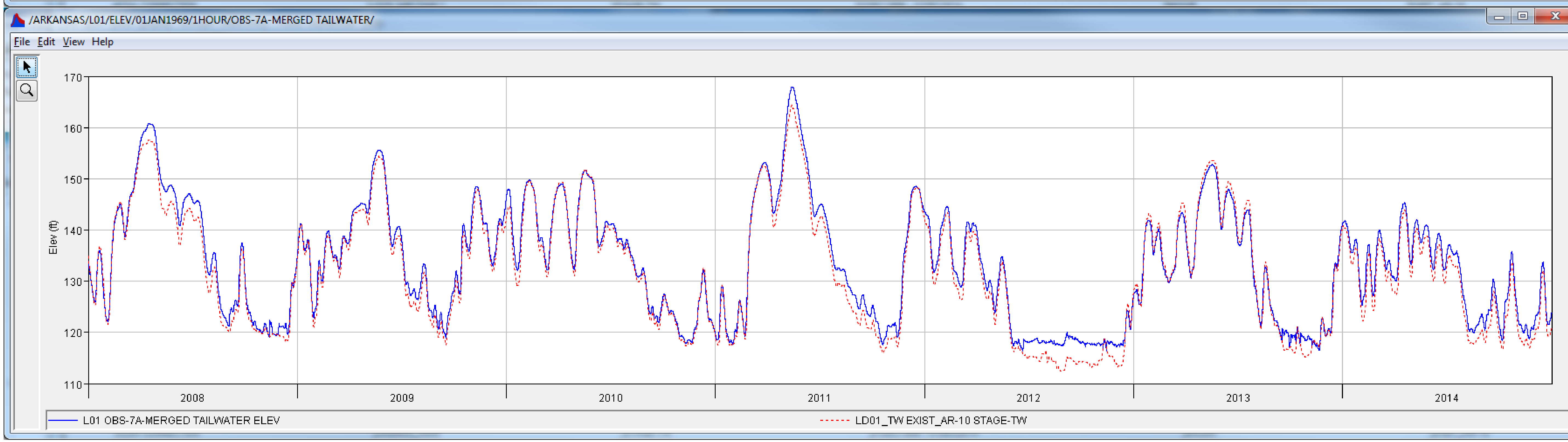


Graham Burke Riverside: White River Observed Modeled

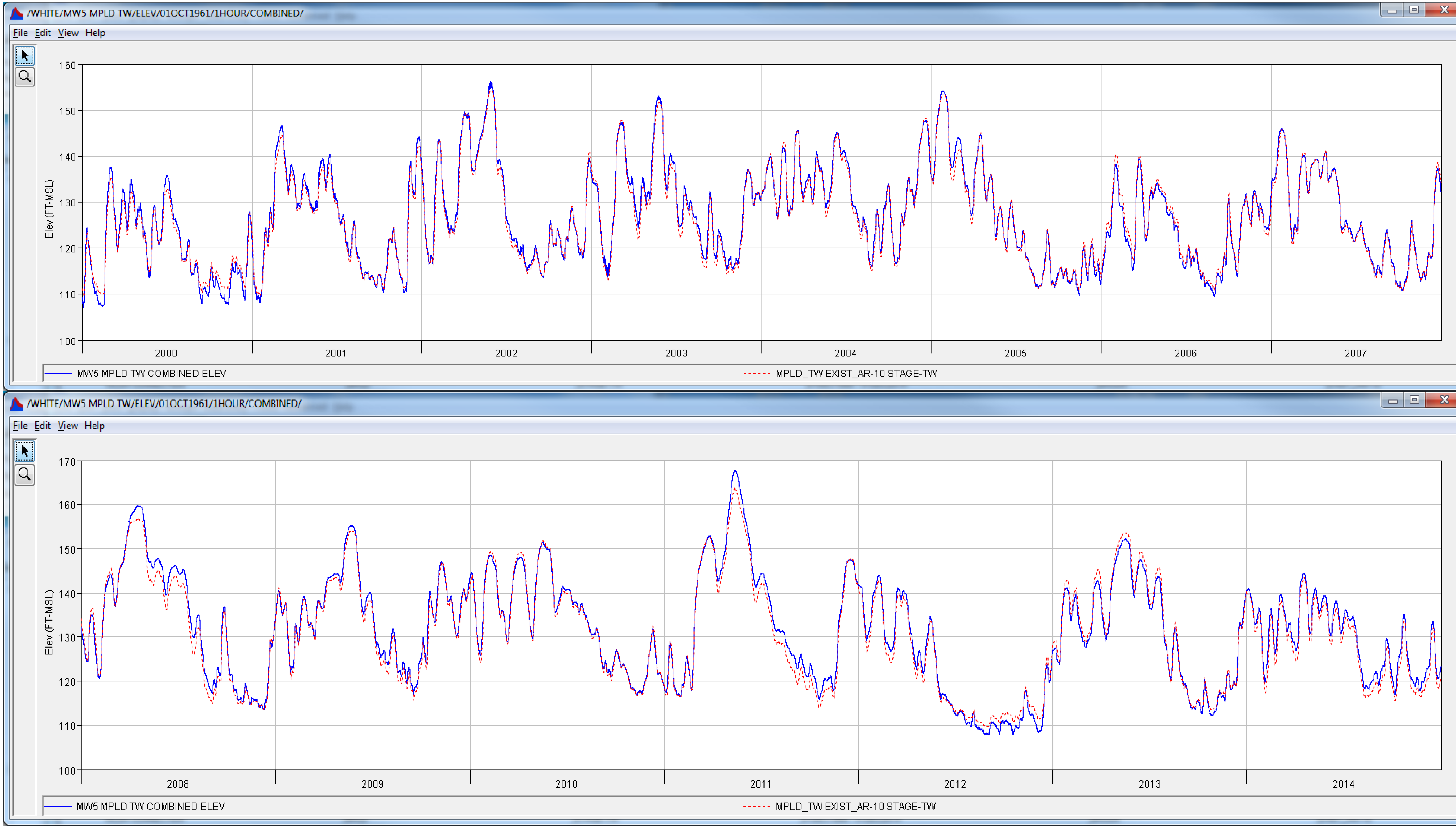


Lock and Dam 1 Tailwater: White River ——— Observed - - - - - - - Modeled

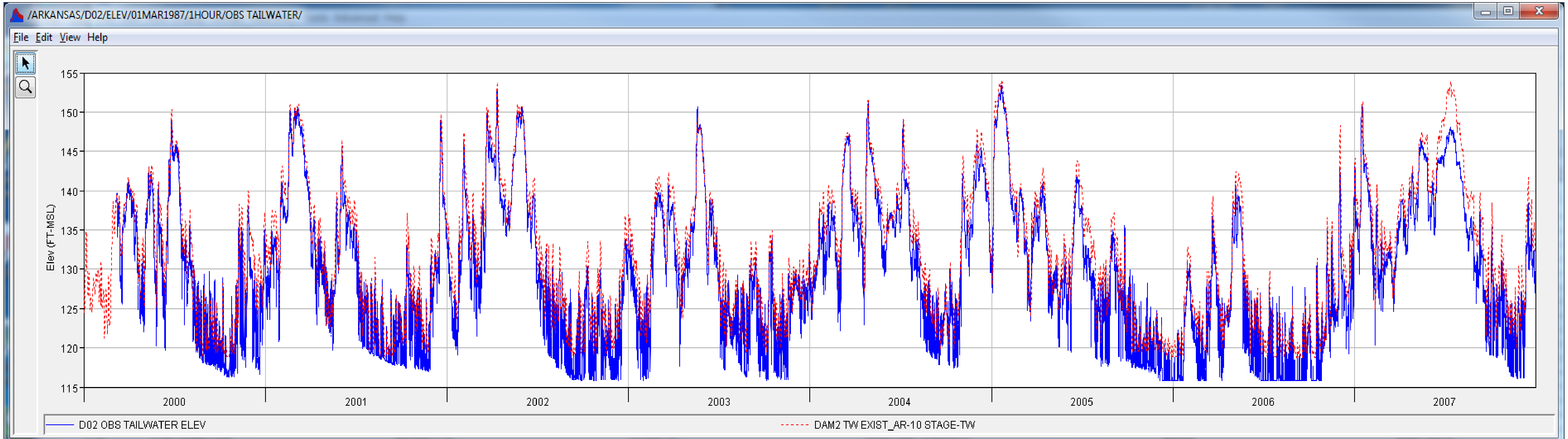




Montgomery Point Lock and Dam Tailwater: White River ——— Observed - - - - - - - Modeled



Lock and Dam 2 Tailwater: Arkansas River ________ Observed _ _ _ _ _ _ _ _Modeled


/ARKANSAS/D02/ELEV/01MAR1987/1HOUR/OBS TAILWATER/
File Edit View Help
Elev (FT-MSL)
155
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130
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D02 OBS TAILWATER ELEV
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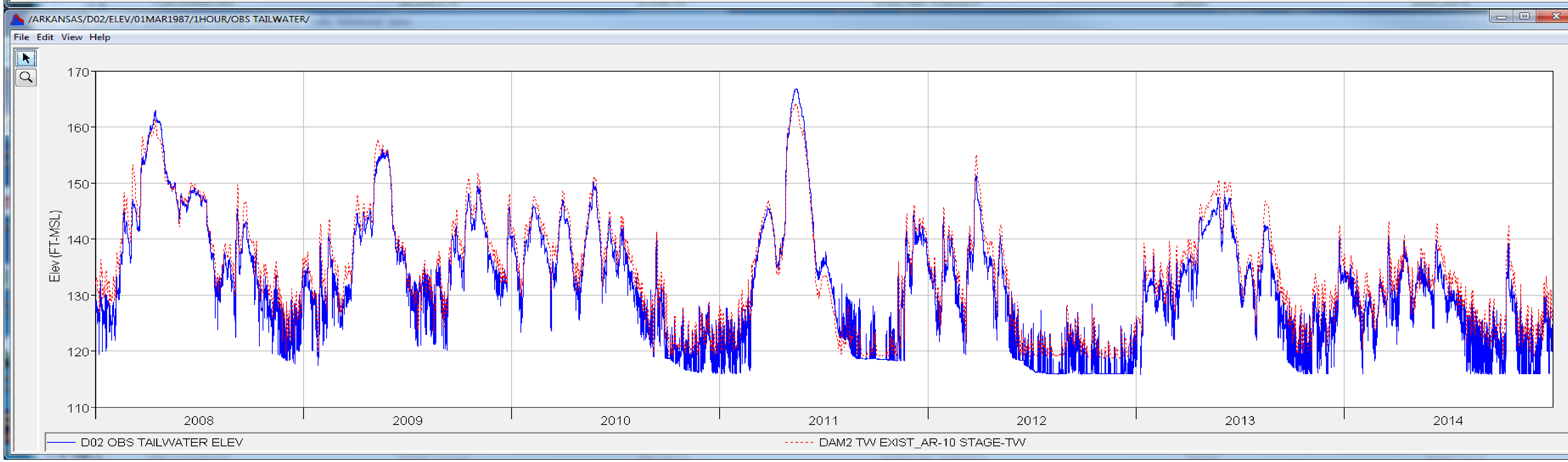
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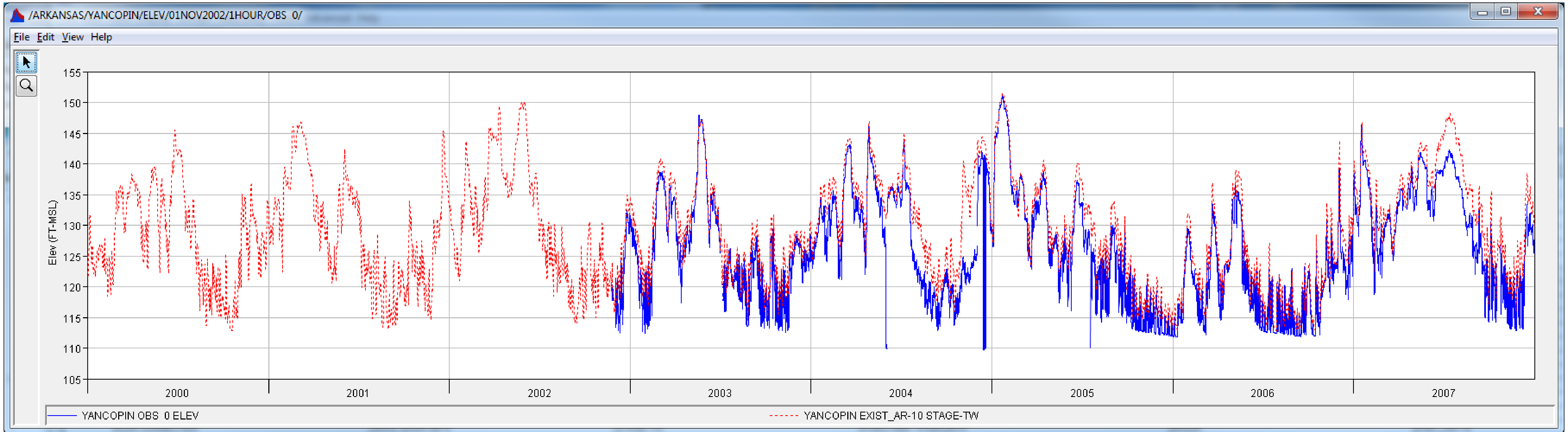
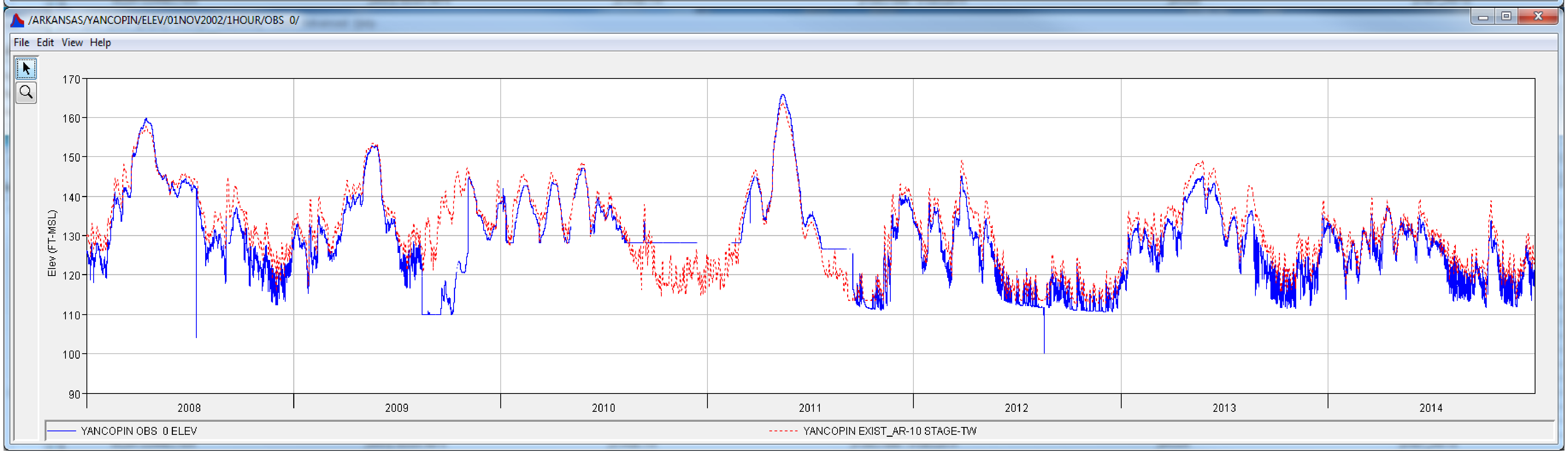
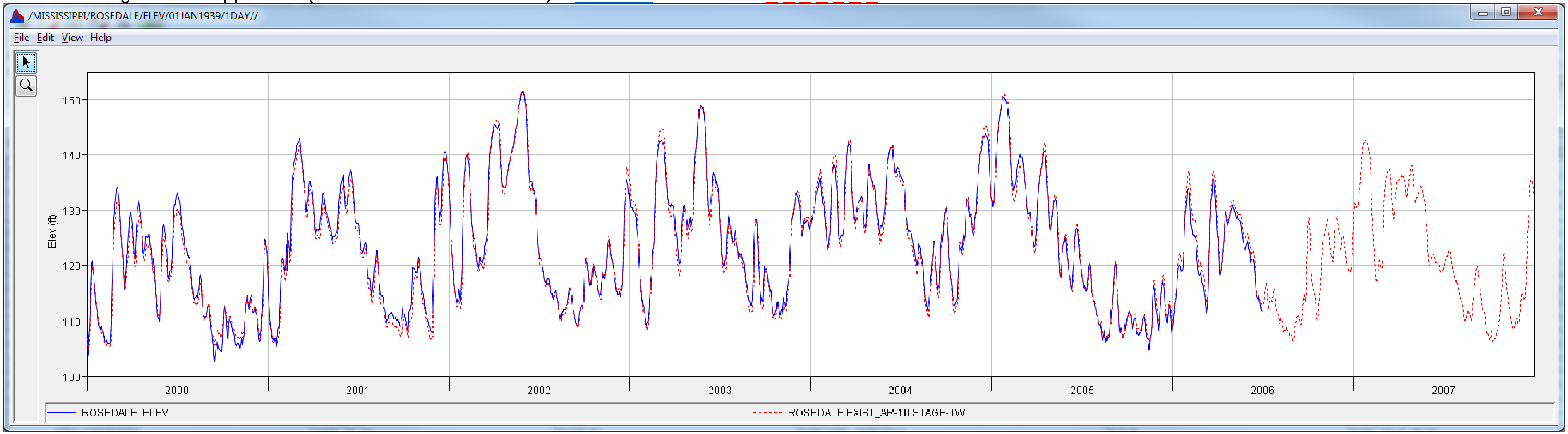
File Edit View Help
Elev (FT-MSL)
170
160
150
140
130
120
110
2008
2009
2010
2011
2012
2013
2014
D02 OBS TAILWATER ELEV
DAM2 TW EXIST_AR-10 STAGE-TW

Yancopin Gage: Arkansas River ________ Observed _ _ _ _ _ _ _ _ _Modeled



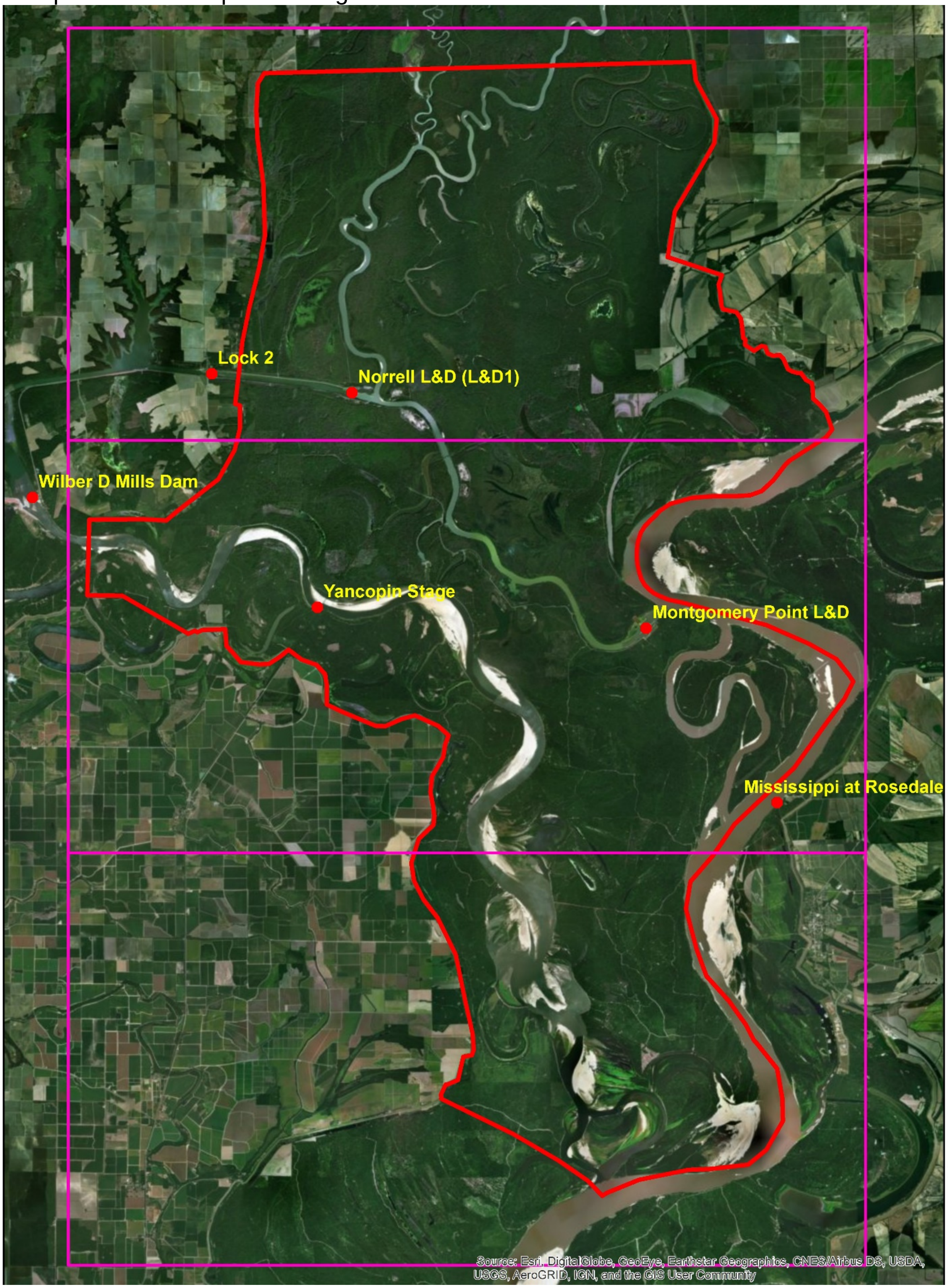


Rosedale Gage: Mississippi River (take out of service in 2006) Observed Modeled

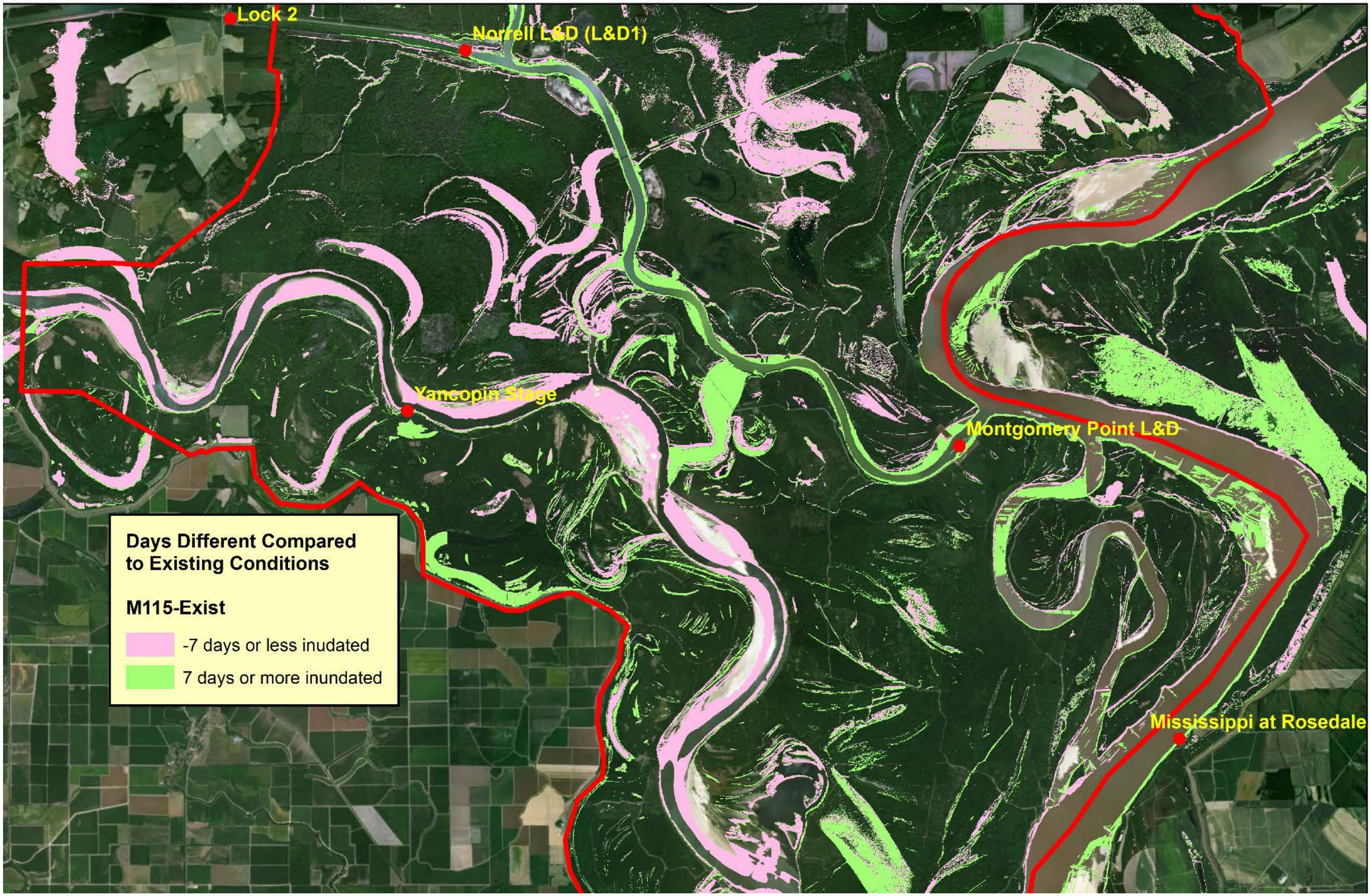


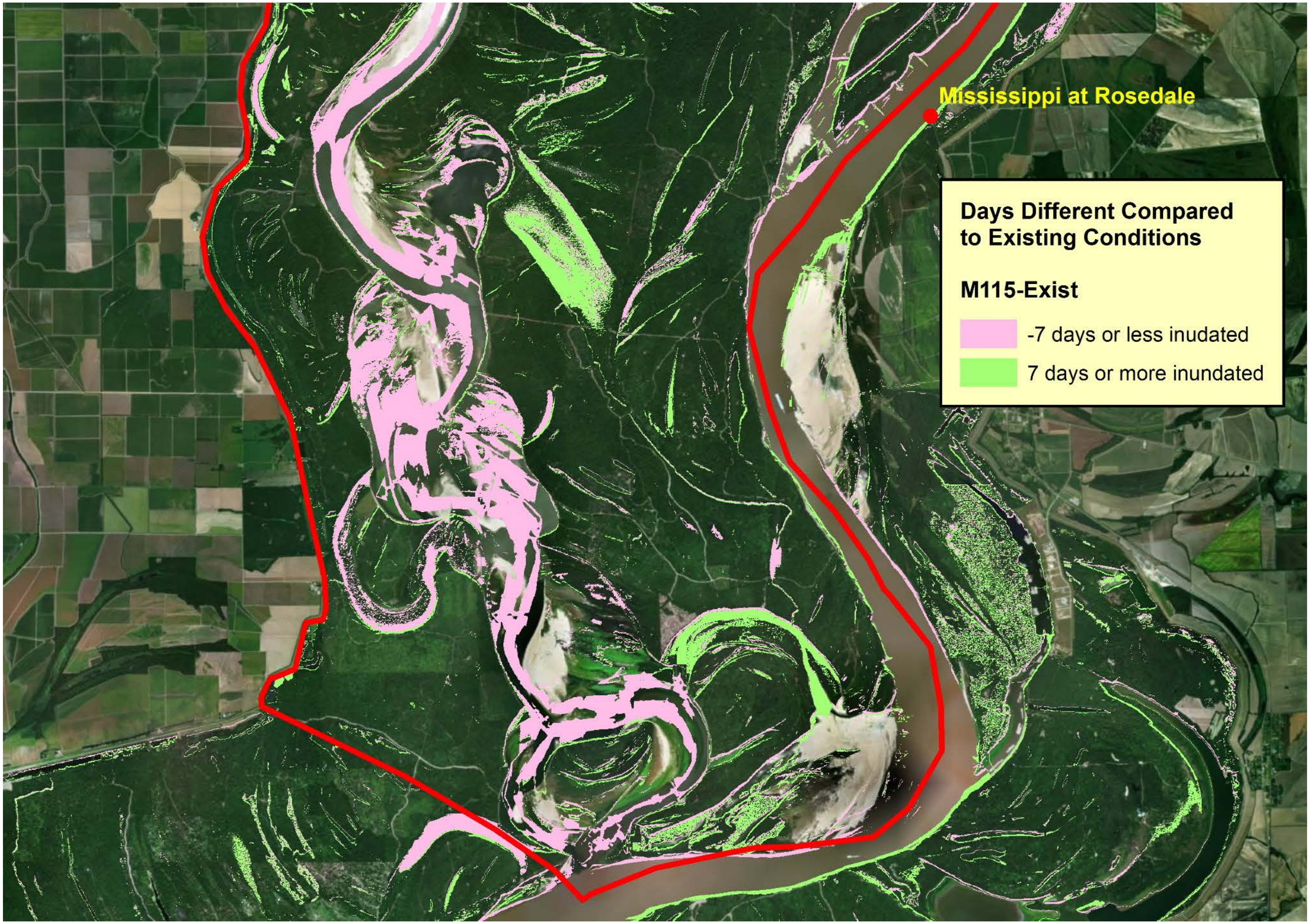
ATTACHMENT B Seven Days Wetter And Drier Inundation Maps

Each plot is broken up into 3 segments as indicated in below.

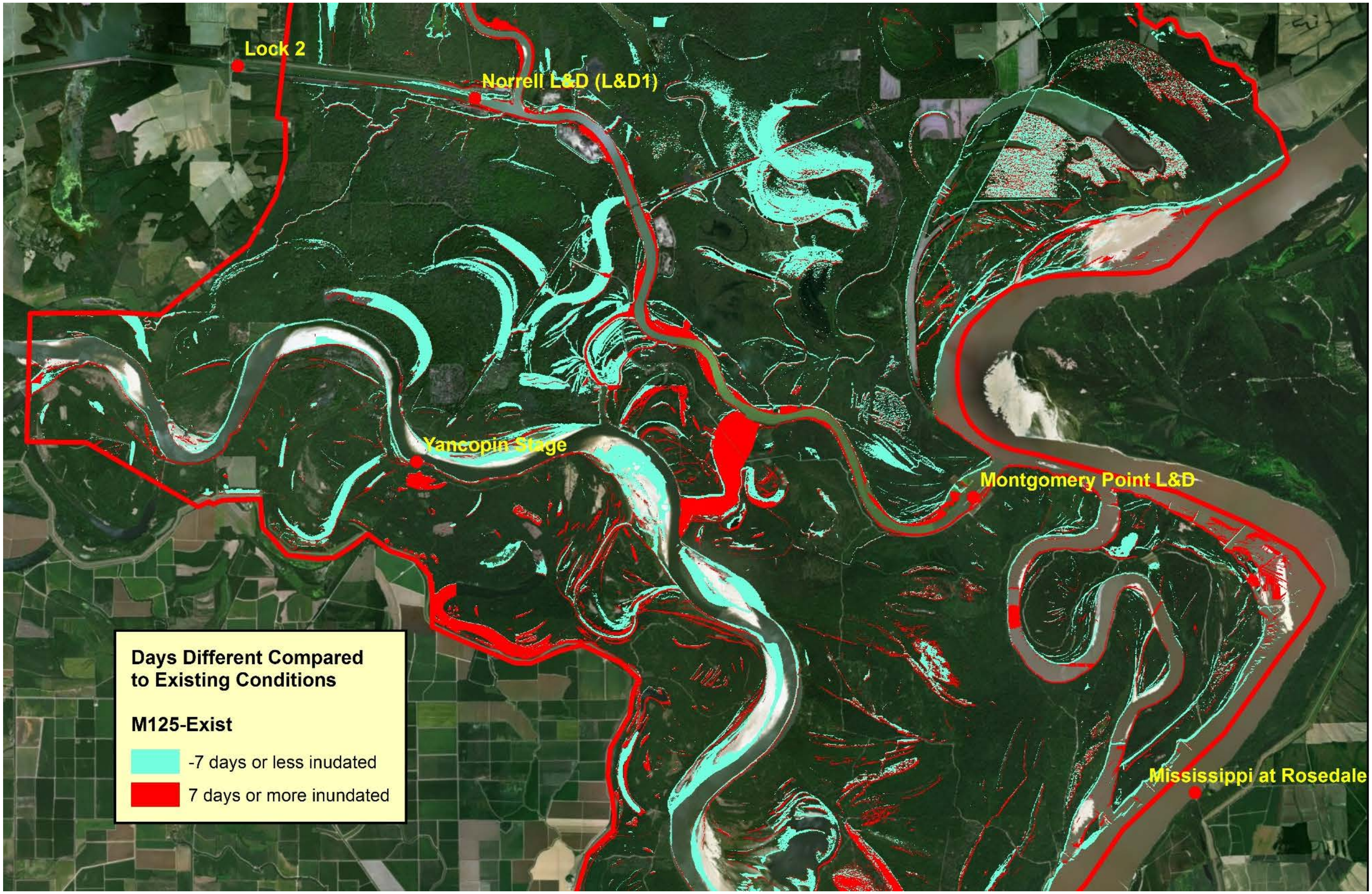


Days Different Compared to Existing Conditions
M115-Exist
-7 days or less inudated
7 days or more inundated
Lock 2
Norrell L&D (L&D1)

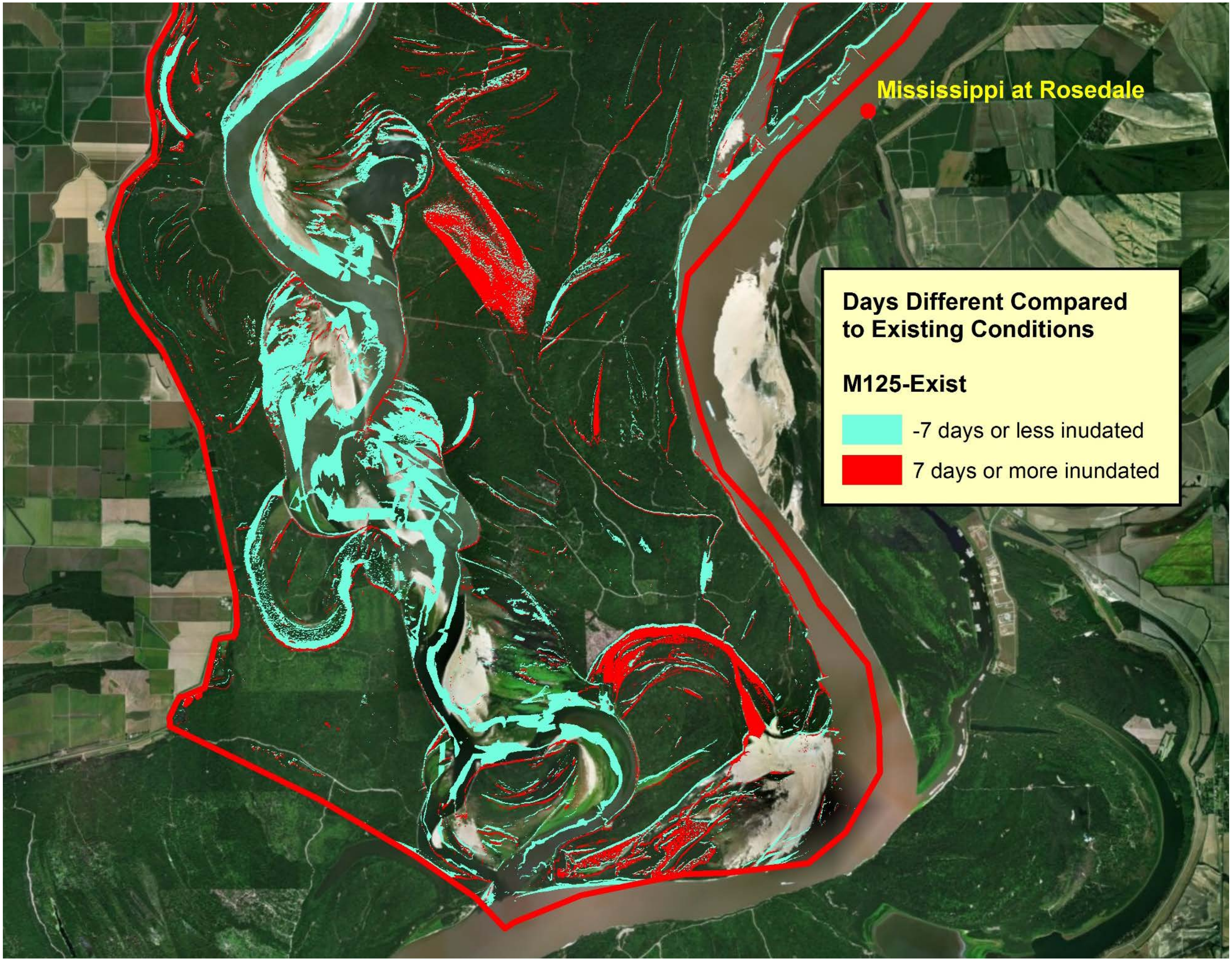

Lock 2
Norrell L&D (L&D1)
Yancopin Stage
Montgomery Point L&D
Mississippi at Rosedale
Days Different Compared
to Existing Conditions
M115-Exist
-7 days or less inundated
7 days or more inundated


Mississippi at Rosedale
Days Different Compared
to Existing Conditions
M115-Exist
-7 days or less inudated
7 days or more inundated

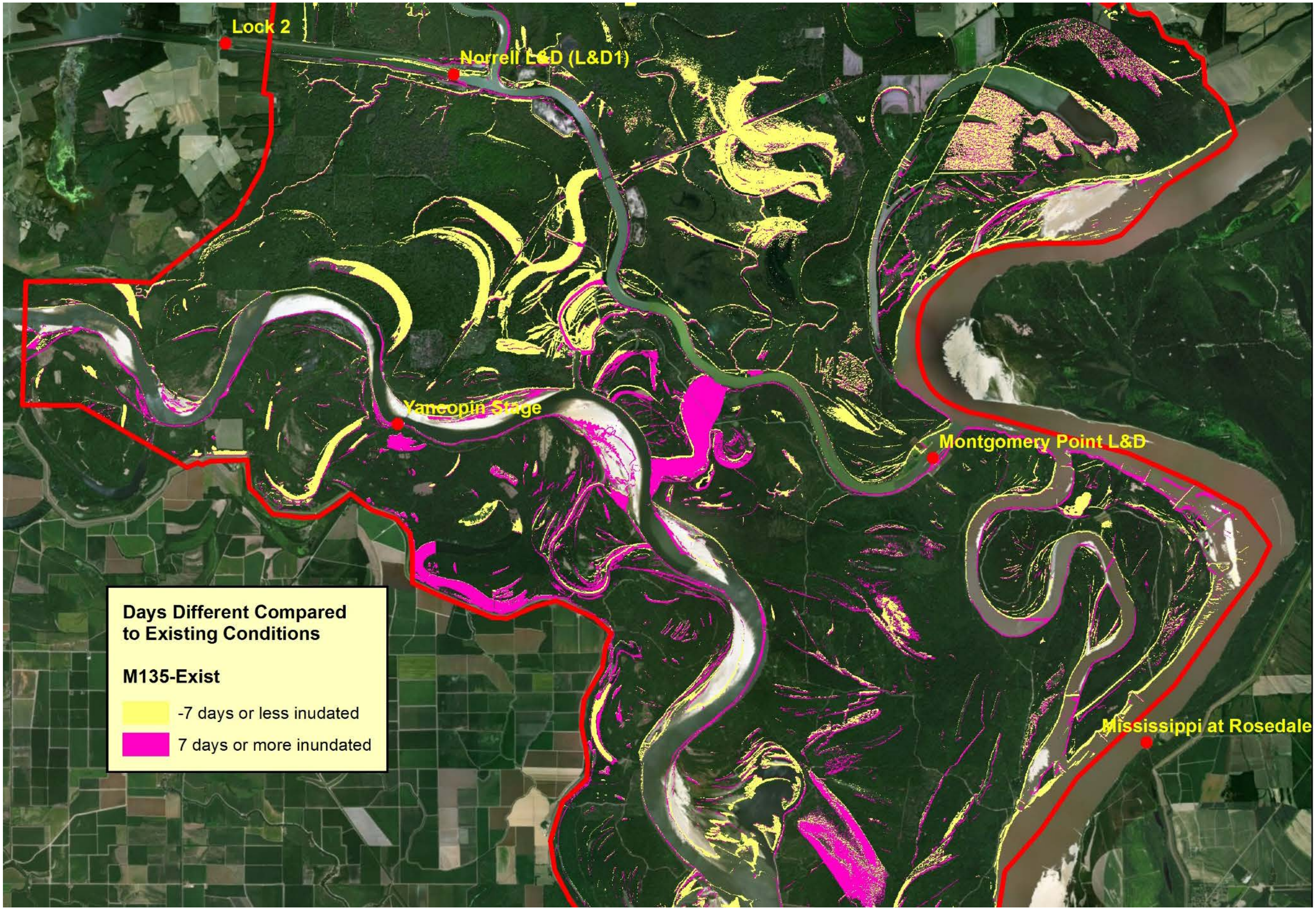
Days Different Compared to Existing Conditions
M125-Exist
-7 days or less inudated
7 days or more inundated
Lock 2
Norrell L&D (L&D1)

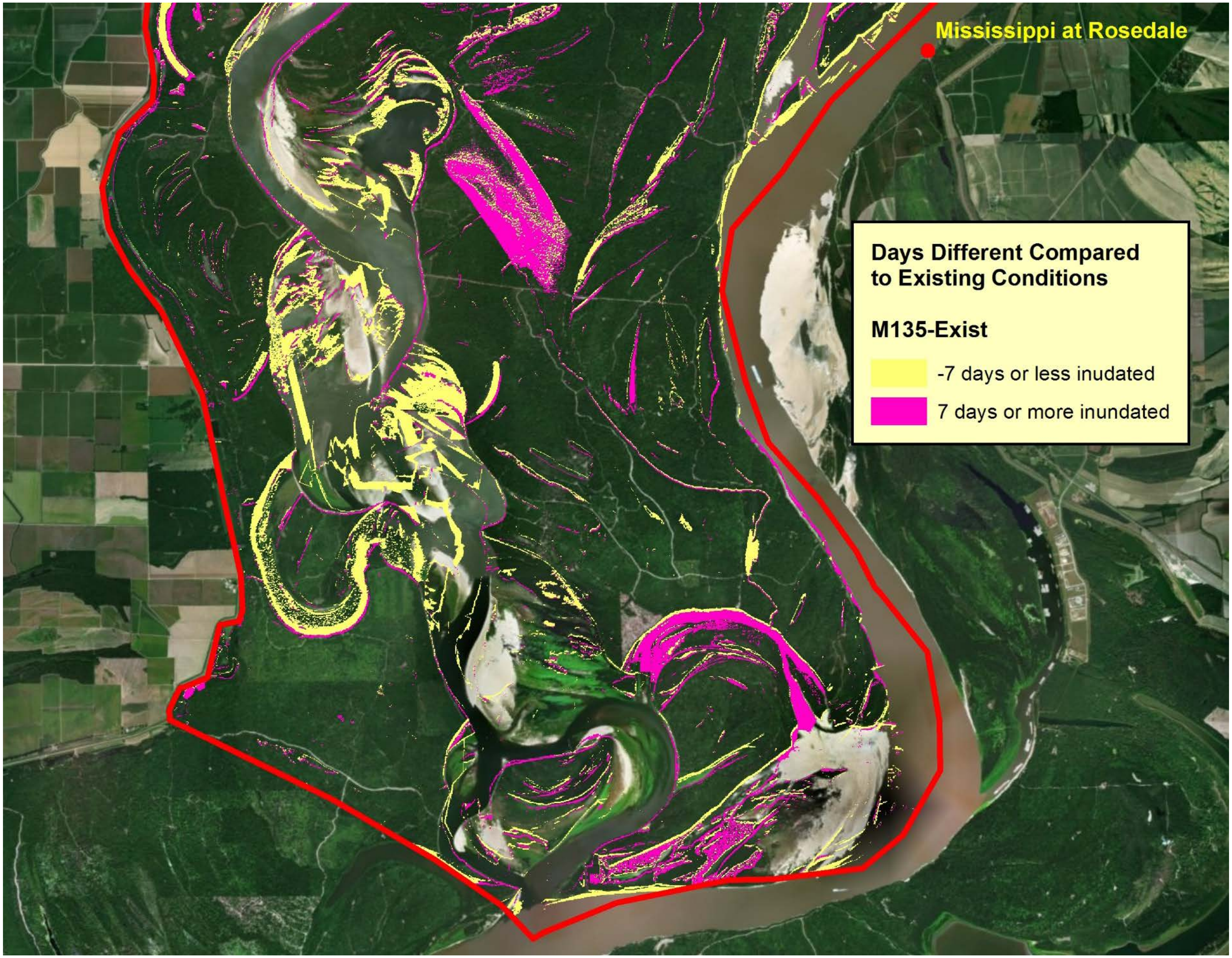


Lock 2
Norrell L&D (L&D1)
Yancopin Stage
Montgomery Point L&D
Mississippi at Rosedale
Days Different Compared
to Existing Conditions
M125-Exist
-7 days or less inudated
7 days or more inundated


Mississippi at Rosedale
Days Different Compared to Existing Conditions
M125-Exist
-7 days or less inudated
7 days or more inundated

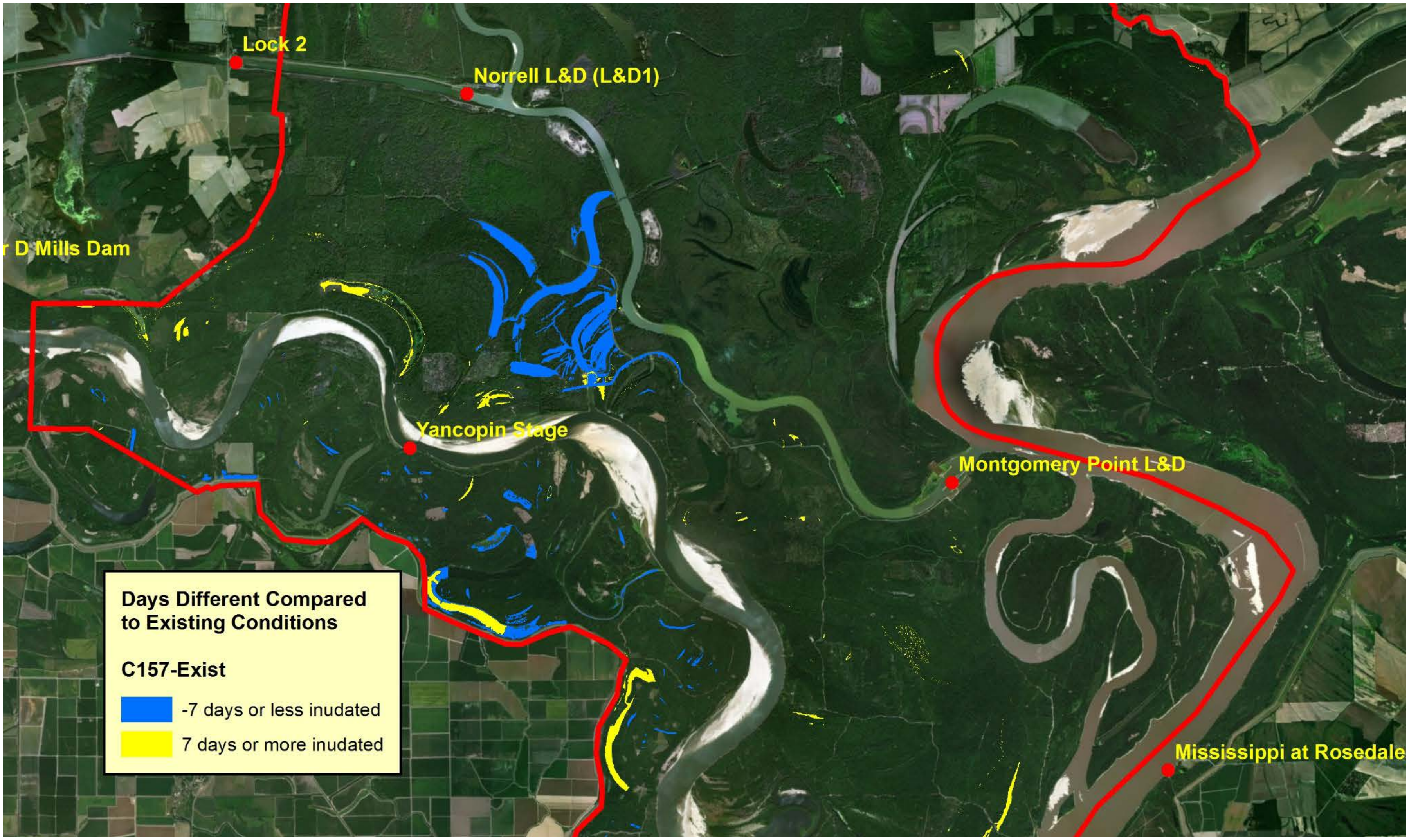
Days Different Compared
to Existing Conditions
M135-Exist
-7 days or less inudated
7 days or more inundated
Lock 2
Norrell L&D (L&D1)


Lock 2
Norrell L&D (L&D1)
Yancopin Stage
Montgomery Point L&D
Mississippi at Rosedale
Days Different Compared
to Existing Conditions
M135-Exist
-7 days or less inudated
7 days or more inundated

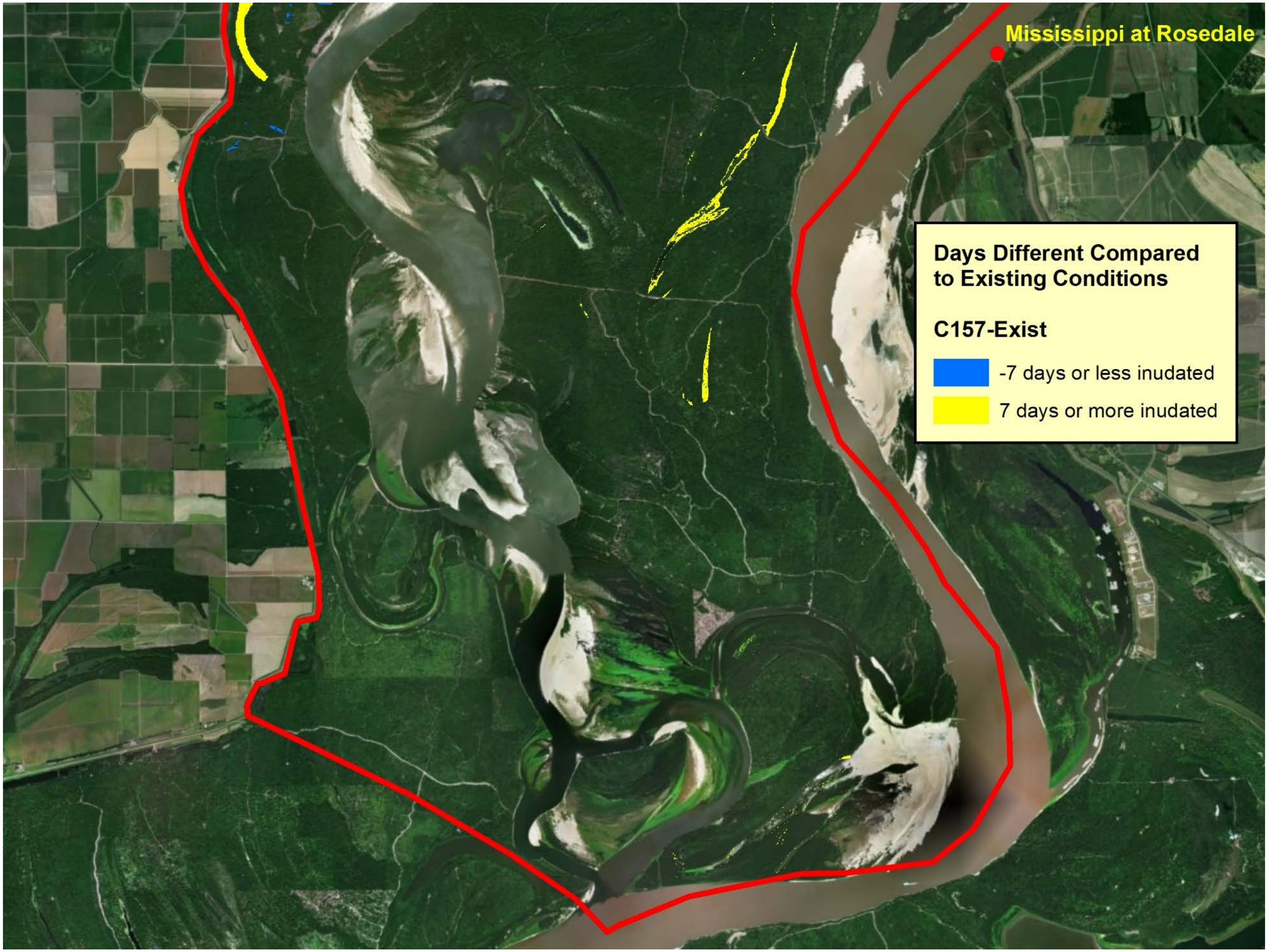


Mississippi at Rosedale
Days Different Compared to Existing Conditions
M135-Exist
-7 days or less inudated
7 days or more inundated

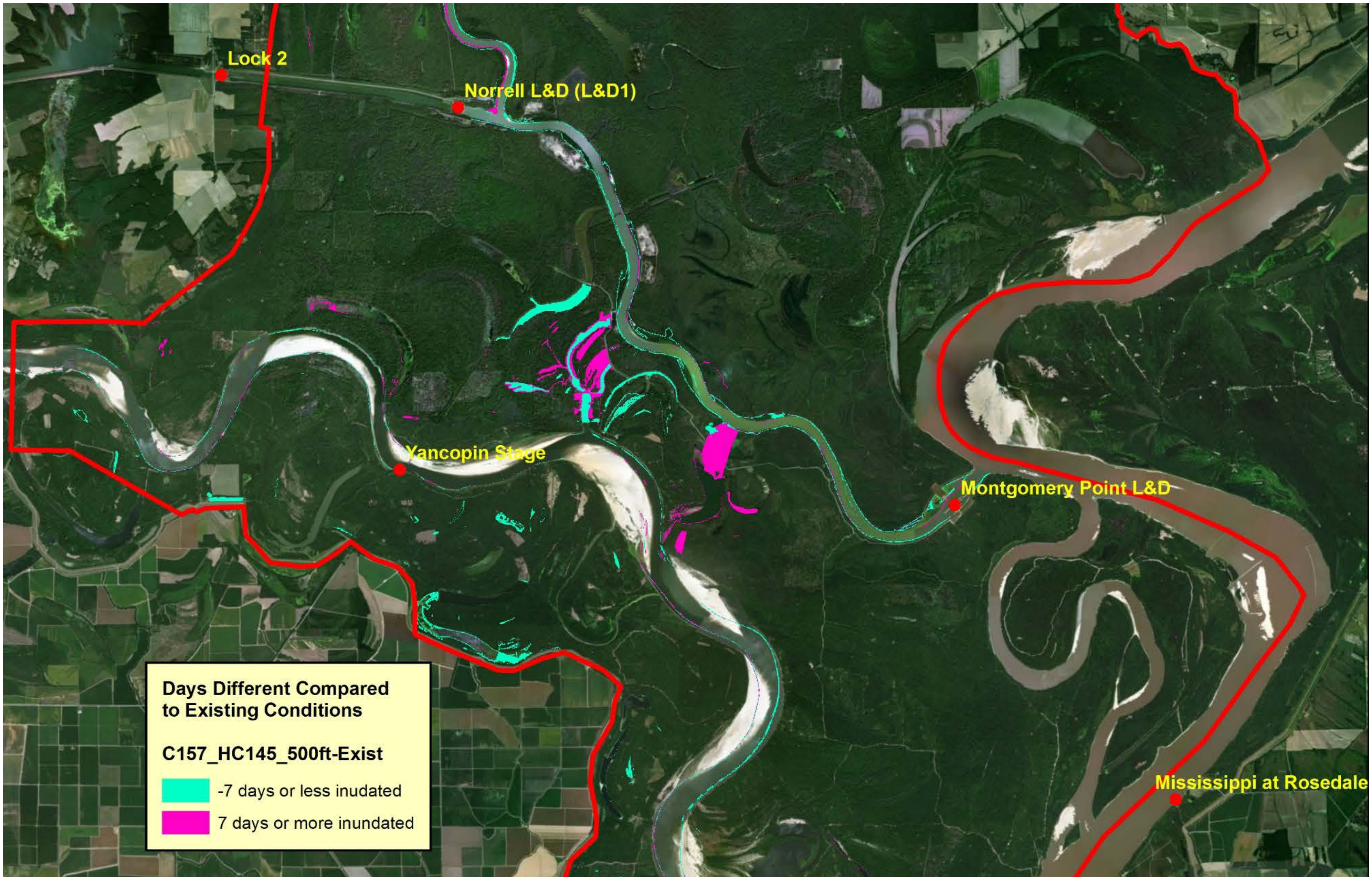
Days Different Compared to Existing Conditions
C157-Exist
-7 days or less inudated
7 days or more inudated
Lock 2
Norrell L&D (L&D1)

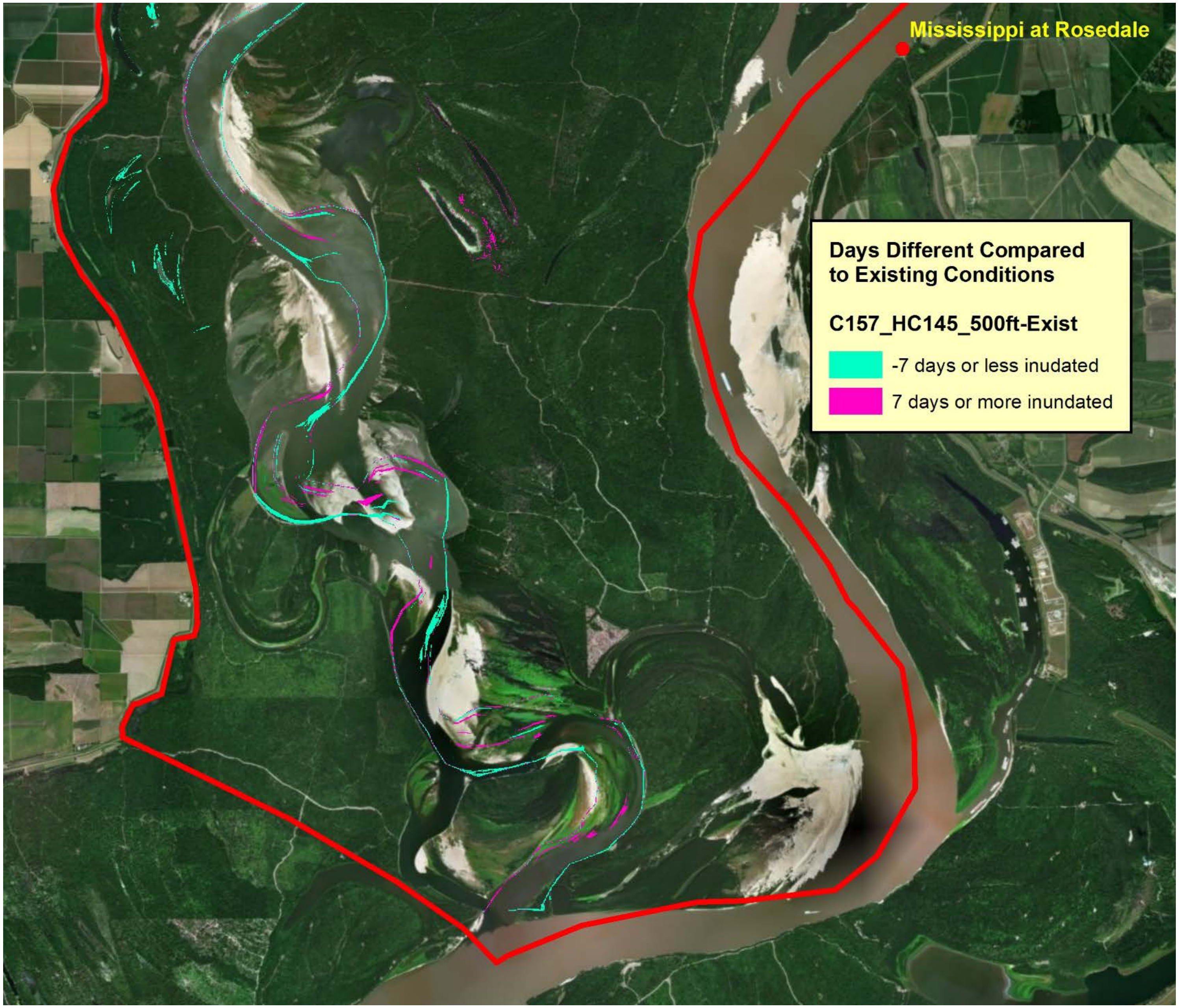


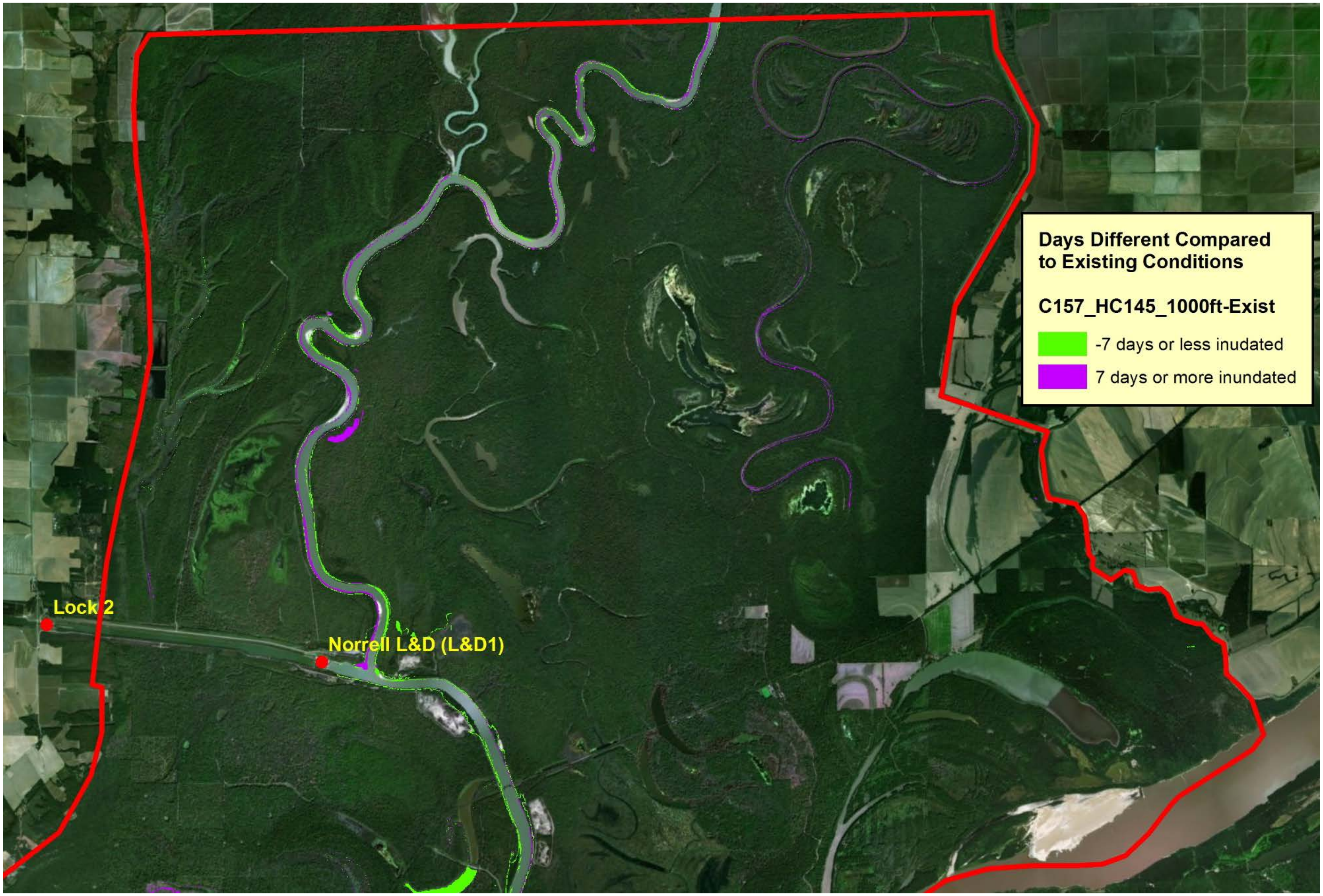
Lock 2
Norrell L&D (L&D1)
r D Mills Dam
Yancopin Stage
Montgomery Point L&D
Mississippi at Rosedale
Days Different Compared
to Existing Conditions
C157-Exist
-7 days or less inudated
7 days or more inudated

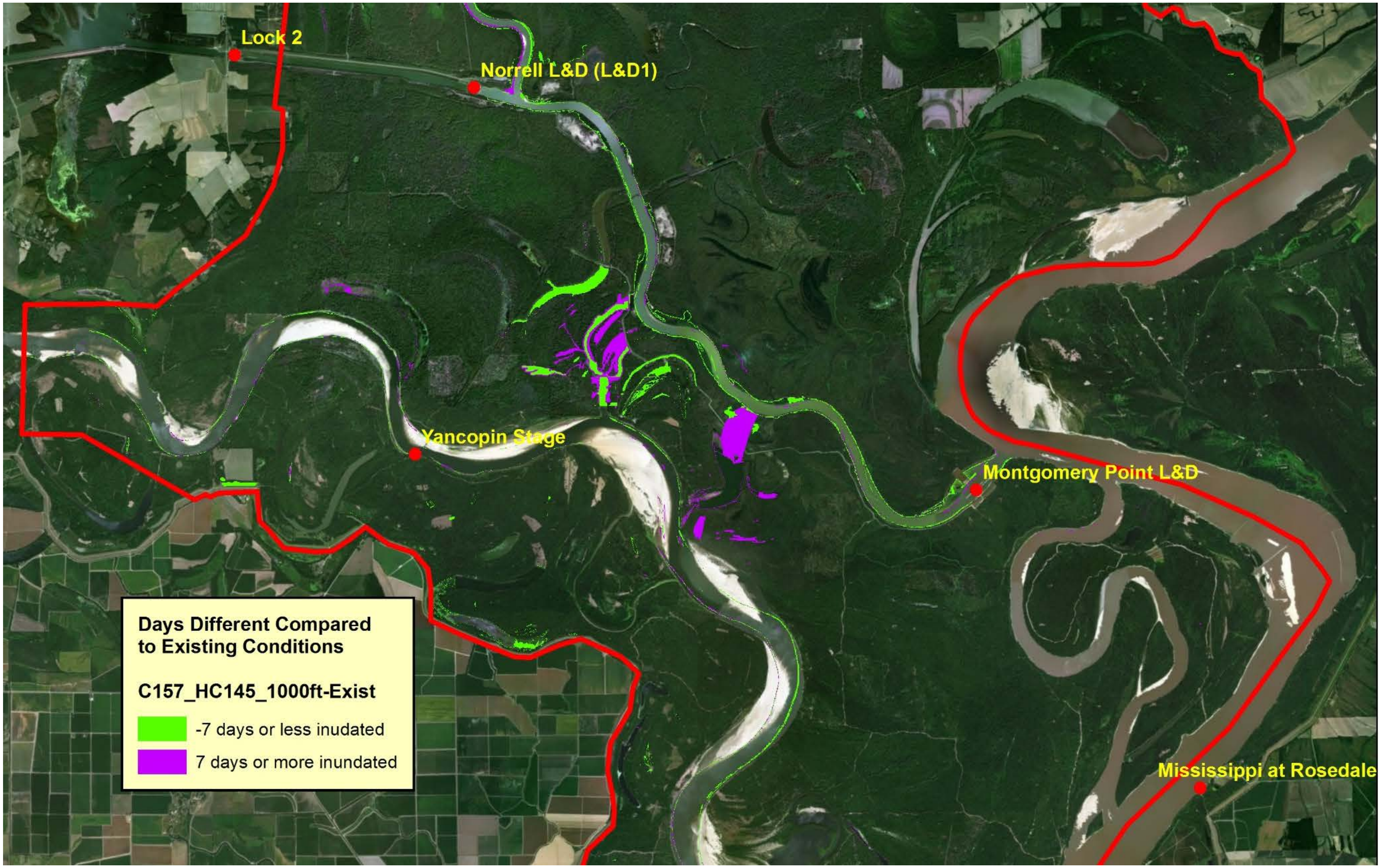

Mississippi at Rosedale
Days Different Compared to Existing Conditions
C157-Exist
-7 days or less inudated
7 days or more inudated


Lock 2
Norrell L&D (L&D1)
Days Different Compared
to Existing Conditions
C157_HC145_500ft-Exist
-7 days or less inudated
7 days or more inundated

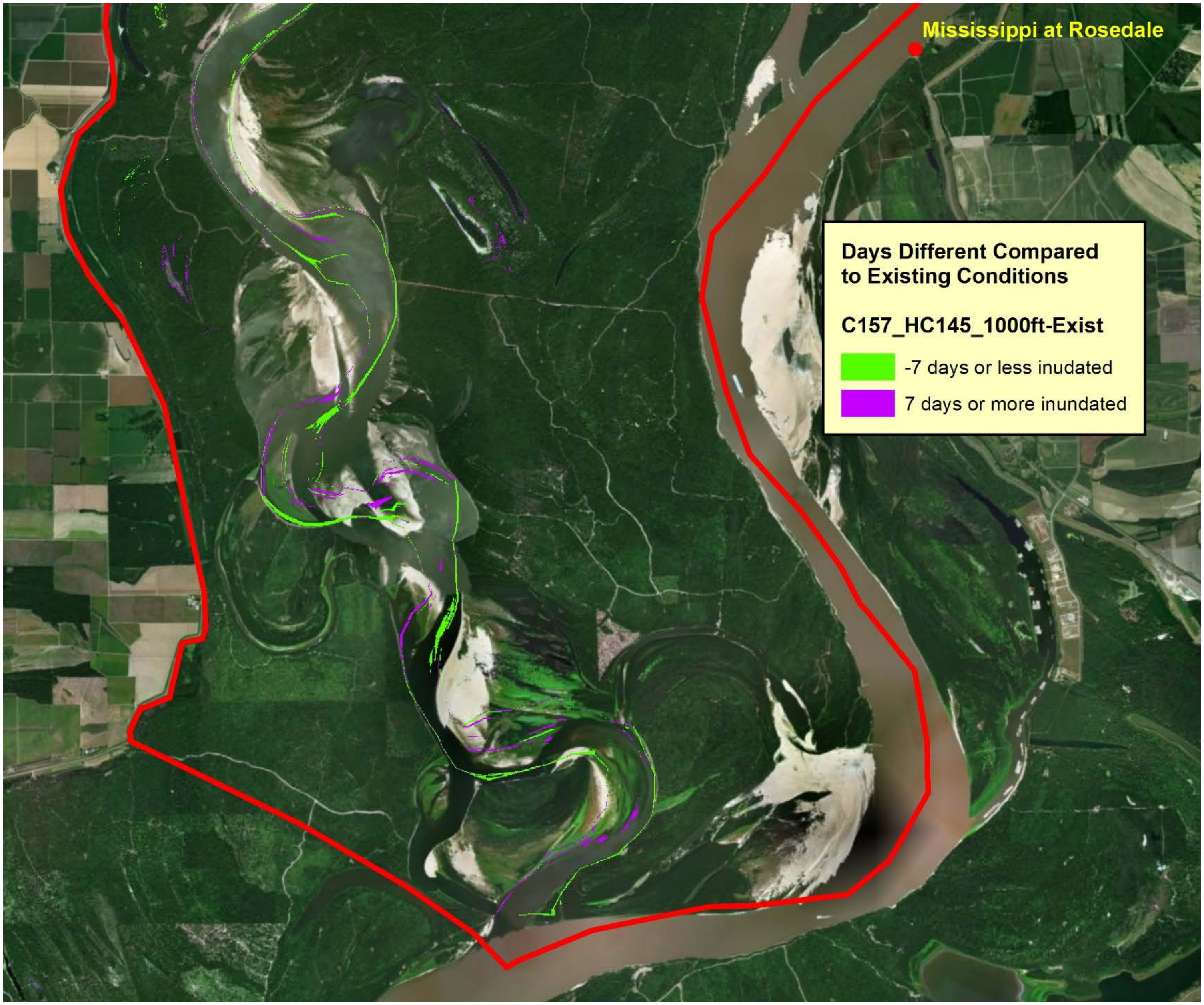

Lock 2
Norrell L&D (L&D1)
Yancopin Stage
Montgomery Point L&D
Mississippi at Rosedale
Days Different Compared
to Existing Conditions
C157_HC145_500ft-Exist
-7 days or less inudated
7 days or more inundated


Mississippi at Rosedale
Days Different Compared
to Existing Conditions
C157_HC145_500ft-Exist
-7 days or less inudated
7 days or more inundated


Days Different Compared
to Existing Conditions
C157_HC145_1000ft-Exist
-7 days or less inudated
7 days or more inundated
Lock 2
Norrell L&D (L&D1)

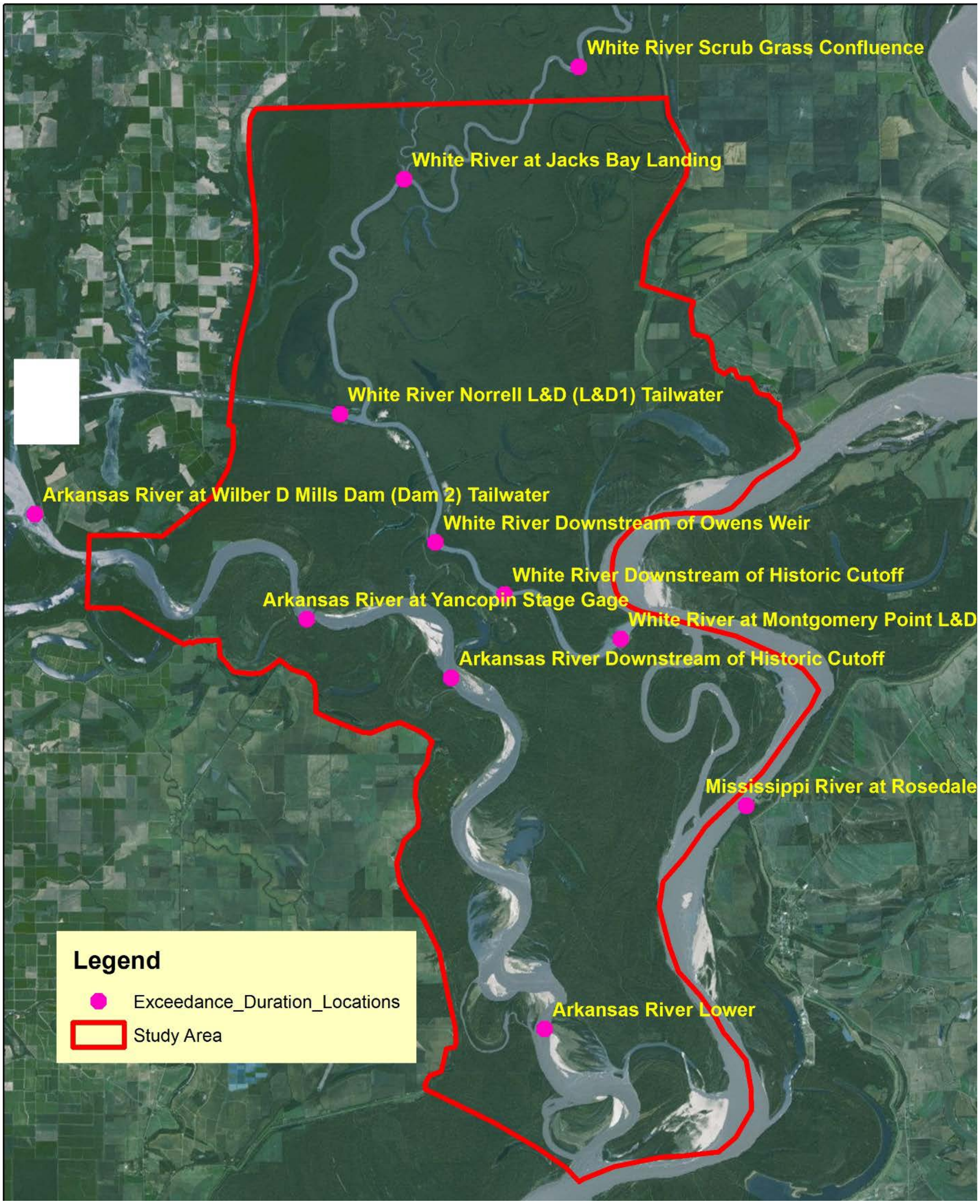


Lock 2
Norrell L&D (L&D1)
Yancopin Stage
Montgomery Point L&D
Mississippi at Rosedale
Days Different Compared to Existing Conditions
C157_HC145_1000ft-Exist
-7 days or less inudated
7 days or more inundated

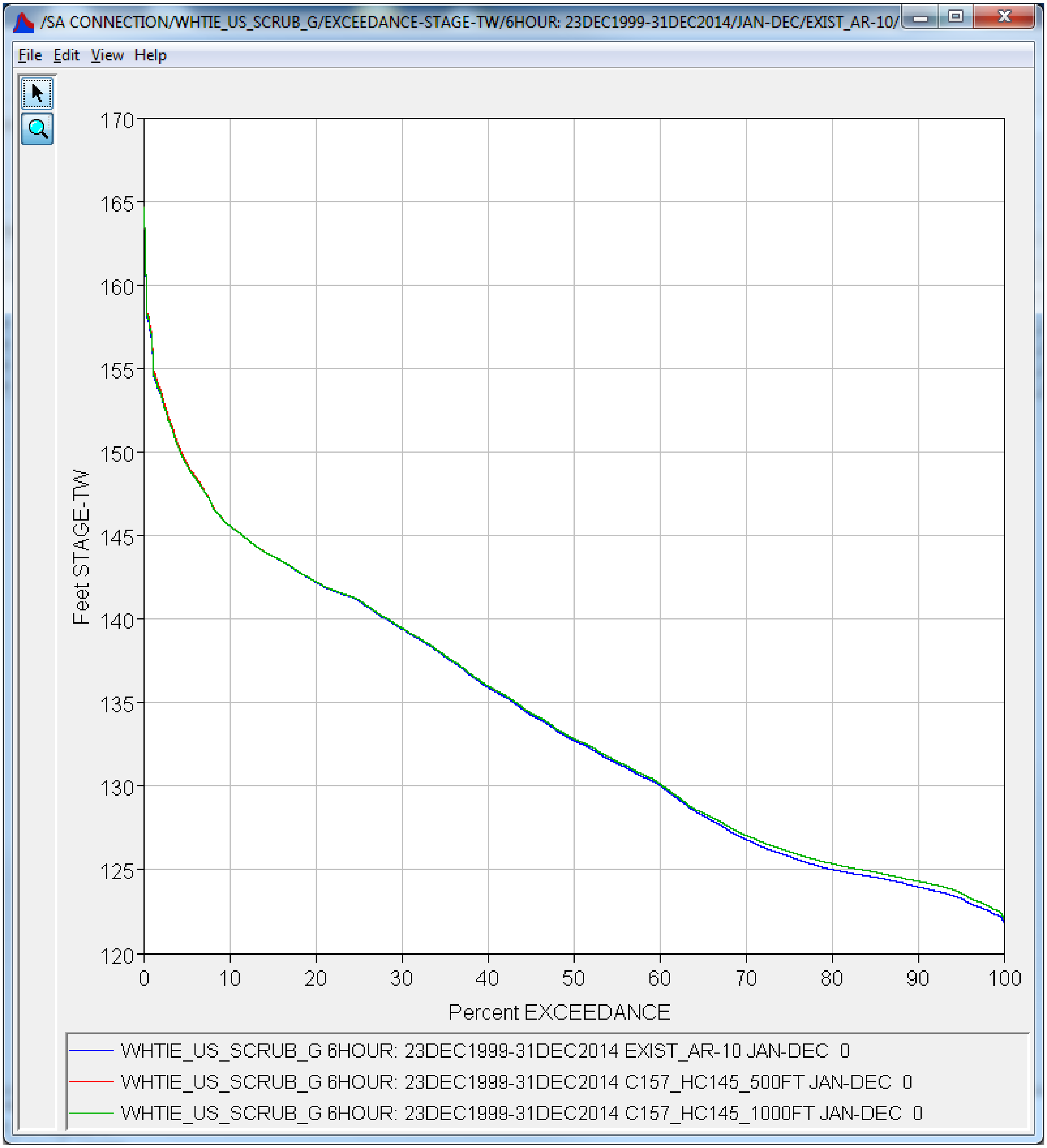

Mississippi at Rosedale
Days Different Compared to Existing Conditions
C157_HC145_1000ft-Exist
-7 days or less inudated
7 days or more inundated

ATTACHMENT C Exceedance Duration Analysis

Exceedance Duration Locations

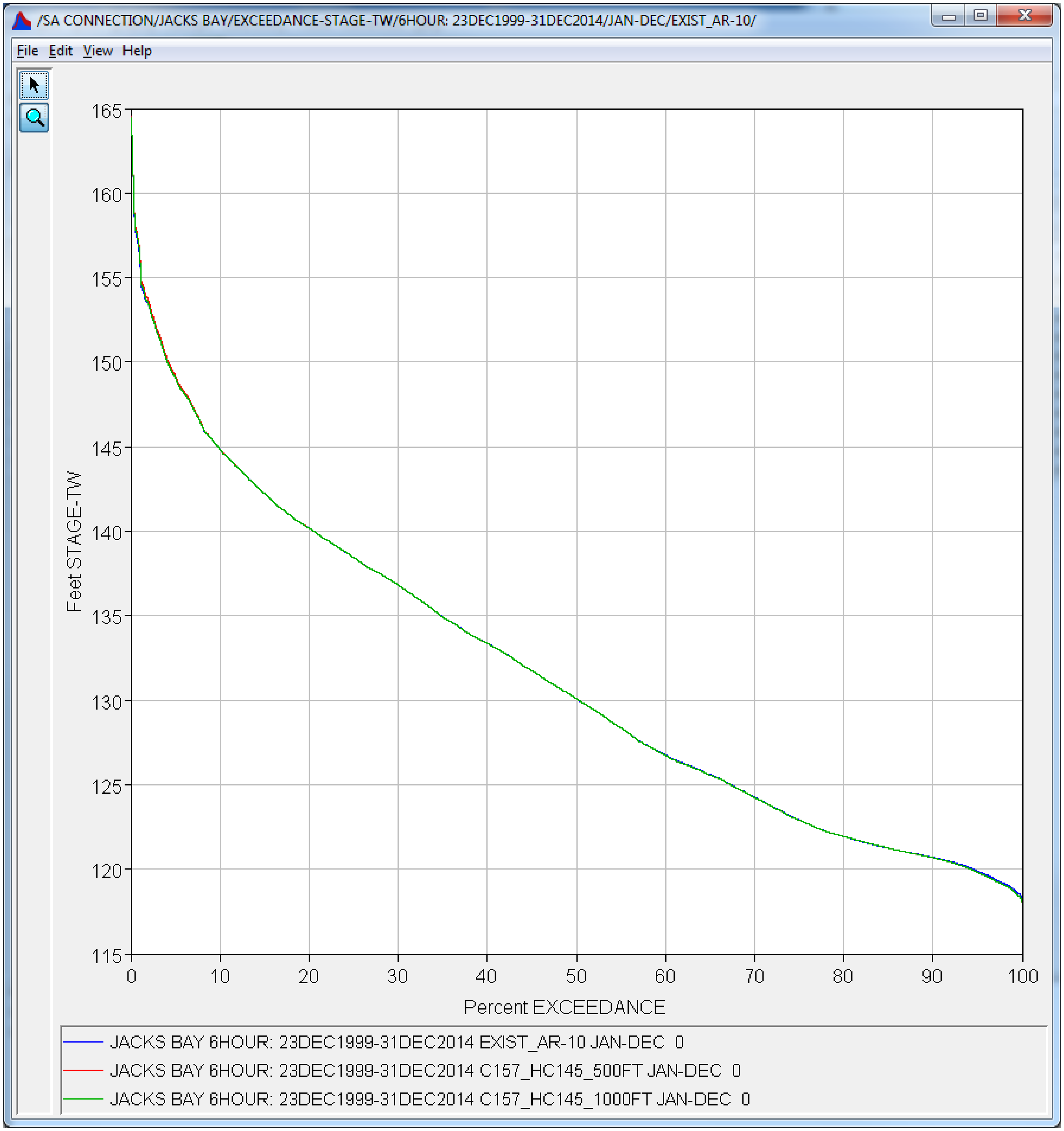


White River Confluence with Scrub Grass
______ Existing ______C157 HC145 500ft Wide ____C157 HC145 1000ft wide



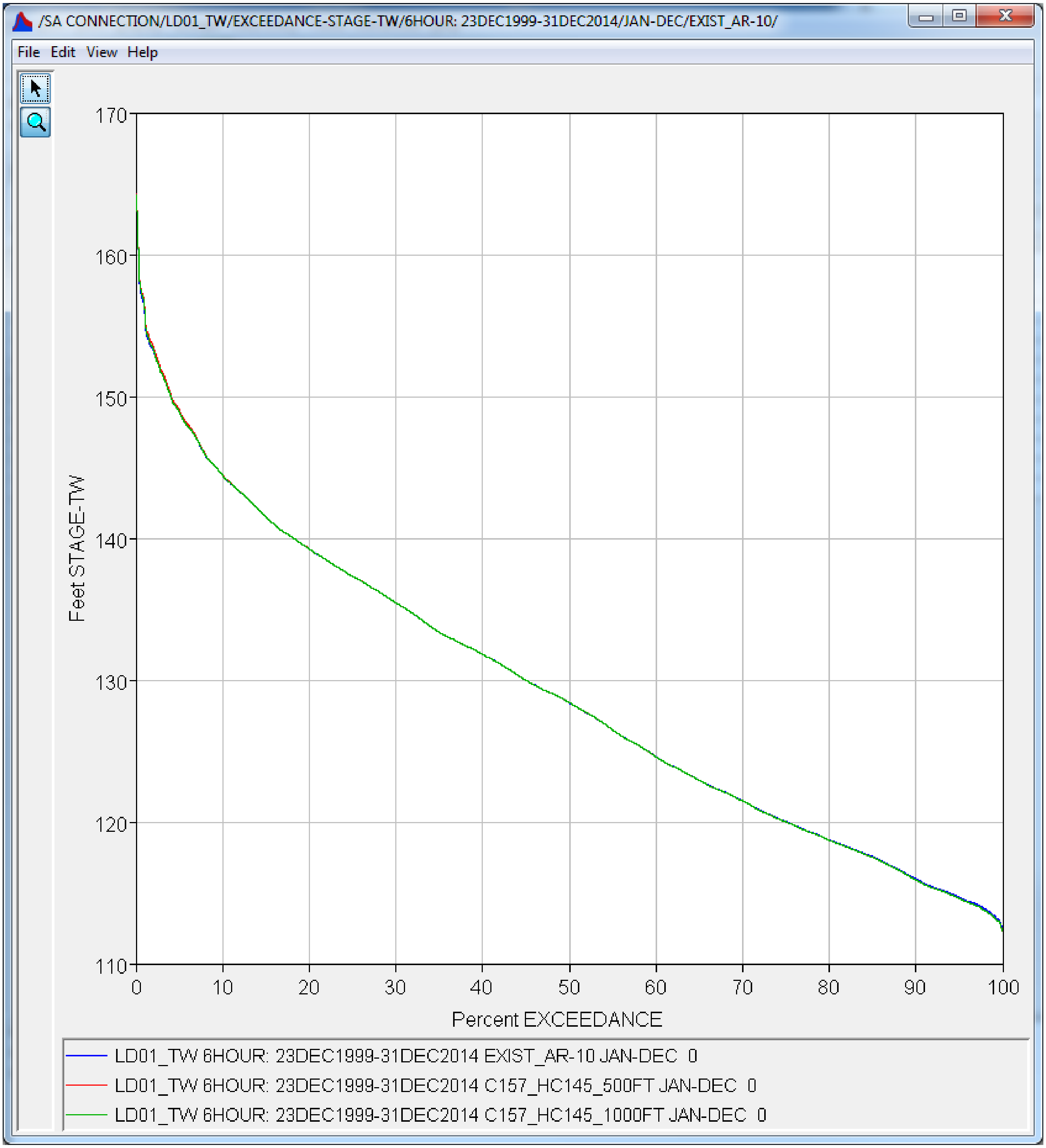
White River at Jacks Bay Landing

______ Existing ______C157 HC145 500ft Wide ____C157 HC145 1000ft wide



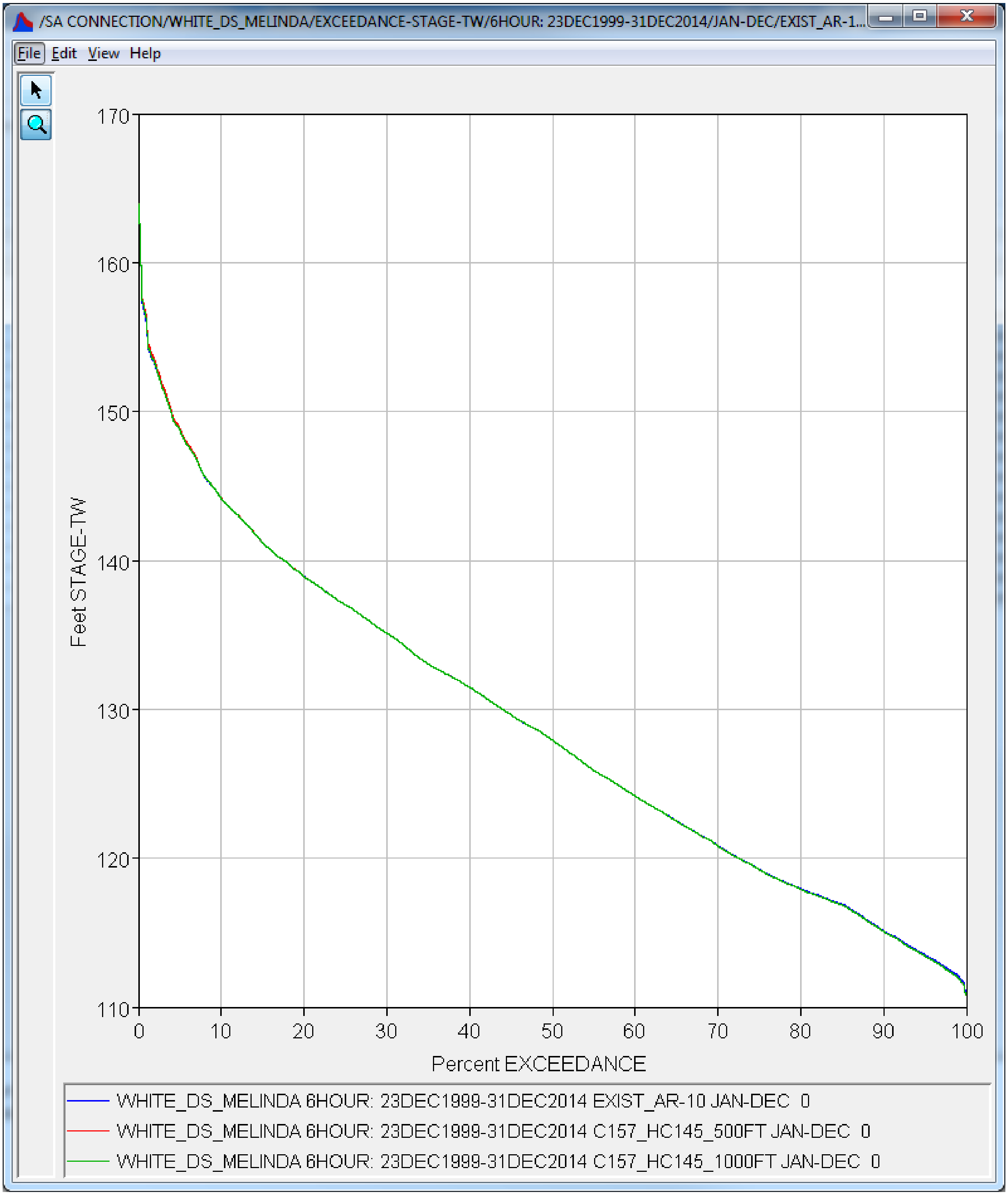
White River Norrell L&D (Lock and Dam 1) Tailwater

______ Existing ______C157 HC145 500ft Wide ____C157 HC145 1000ft wide



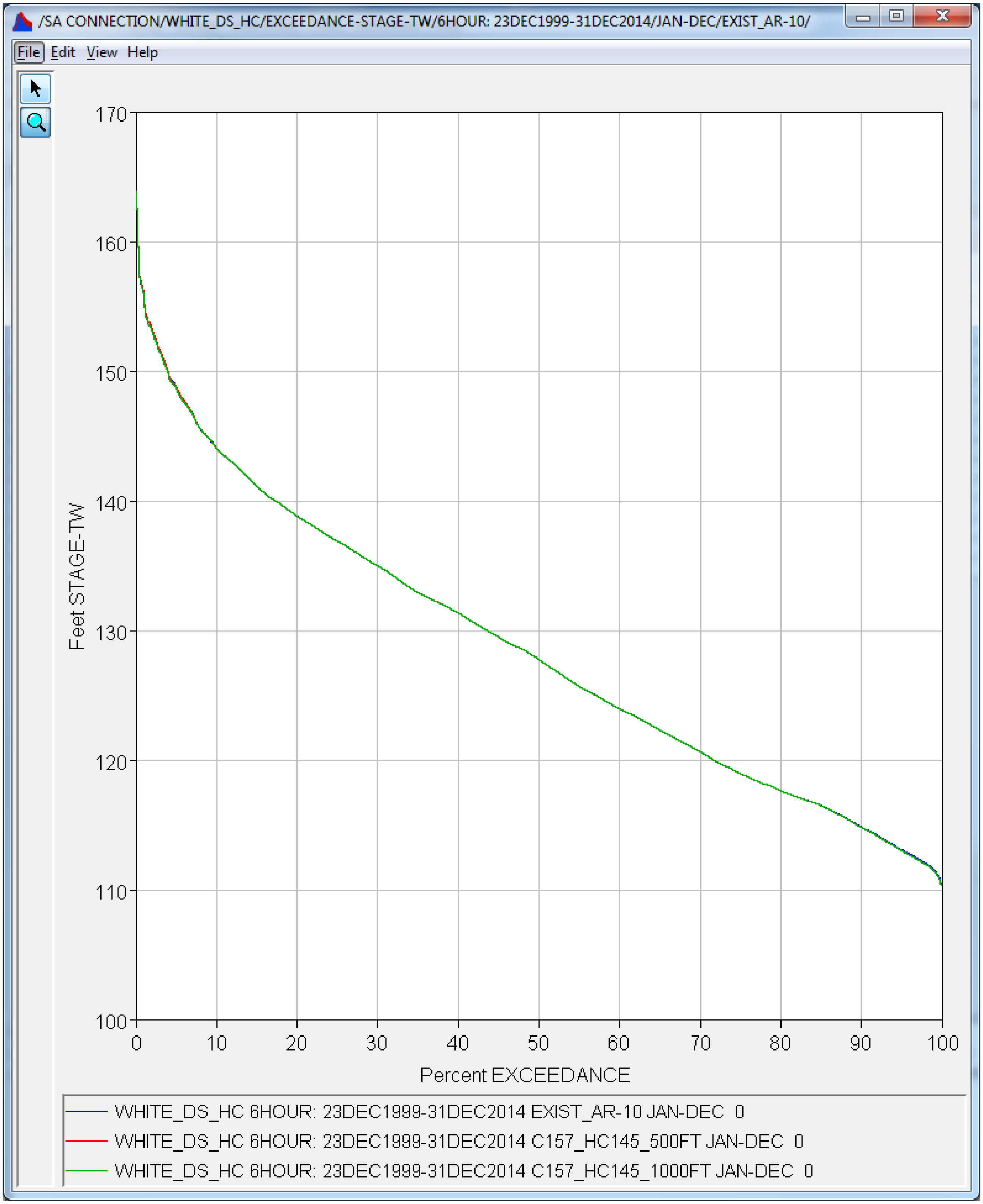
White River Downstream of Owens Weir

______ Existing ______C157 HC145 500ft Wide _____C157 HC145 1000ft wide



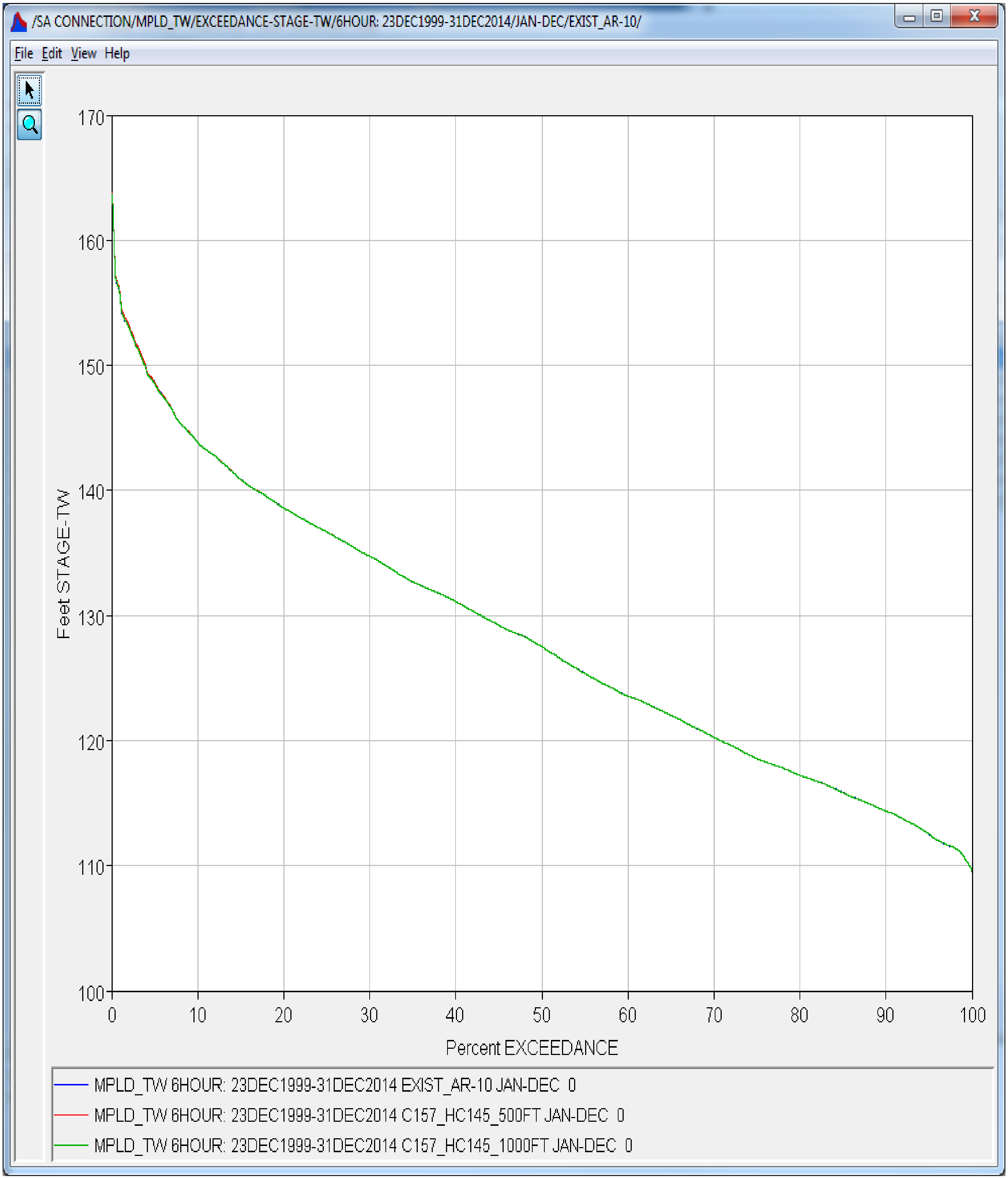
White River Downstream of Historic Cutoff

______ Existing ______C157 HC145 500ft Wide ____C157 HC145 1000ft wide



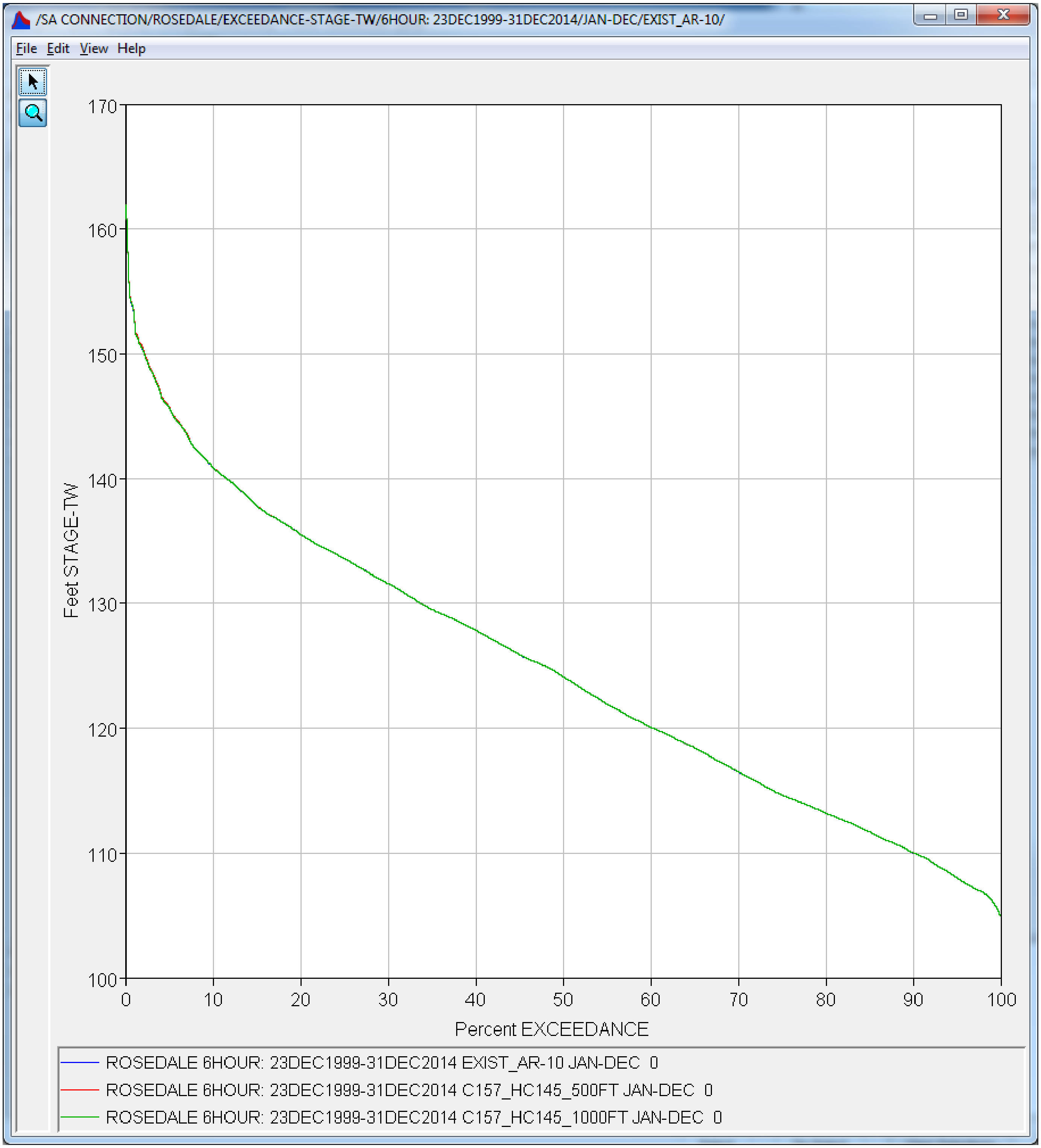
White River at Montgomery Point Lock and Dam Tailwater

______ Existing ______C157 HC145 500ft Wide ____C157 HC145 1000ft wide



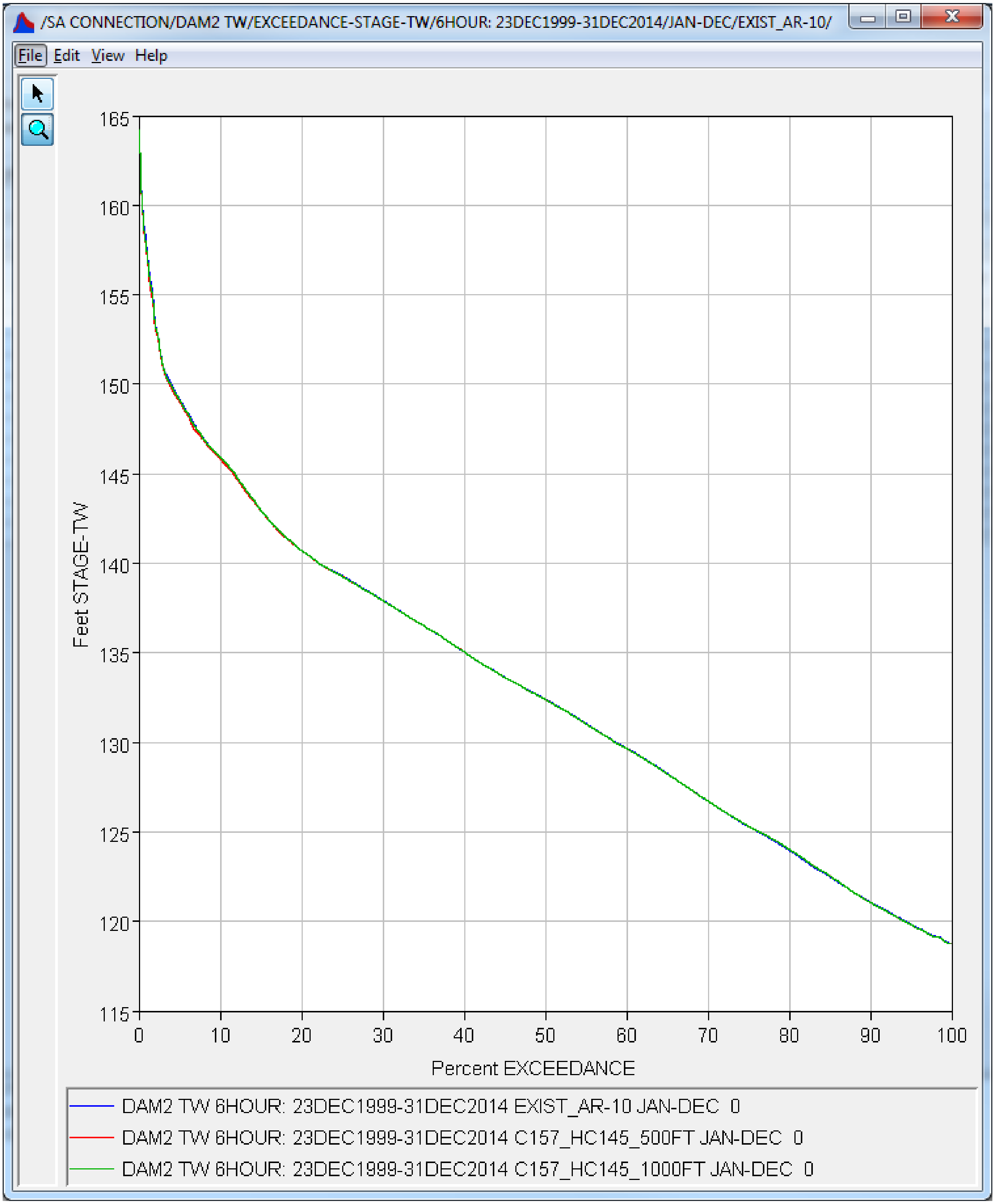
Mississippi River at Rosedale

______ Existing _____C157 HC145 500ft Wide ____C157 HC145 1000ft wide



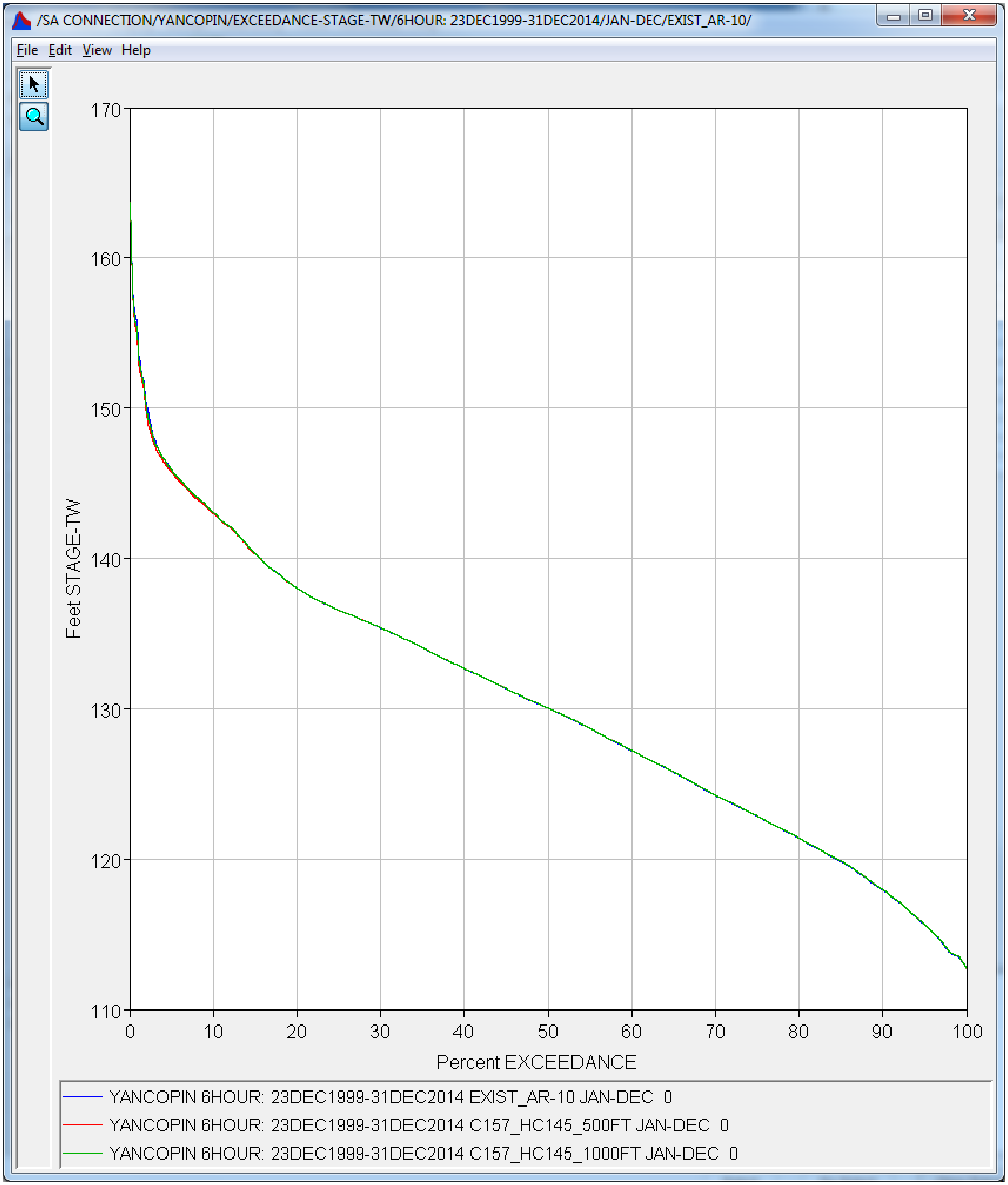
Arkansas River at Wilbur D. Mills (Dam 2) Tailwater

______ Existing ______C157 HC145 500ft Wide _____C157 HC145 1000ft wide



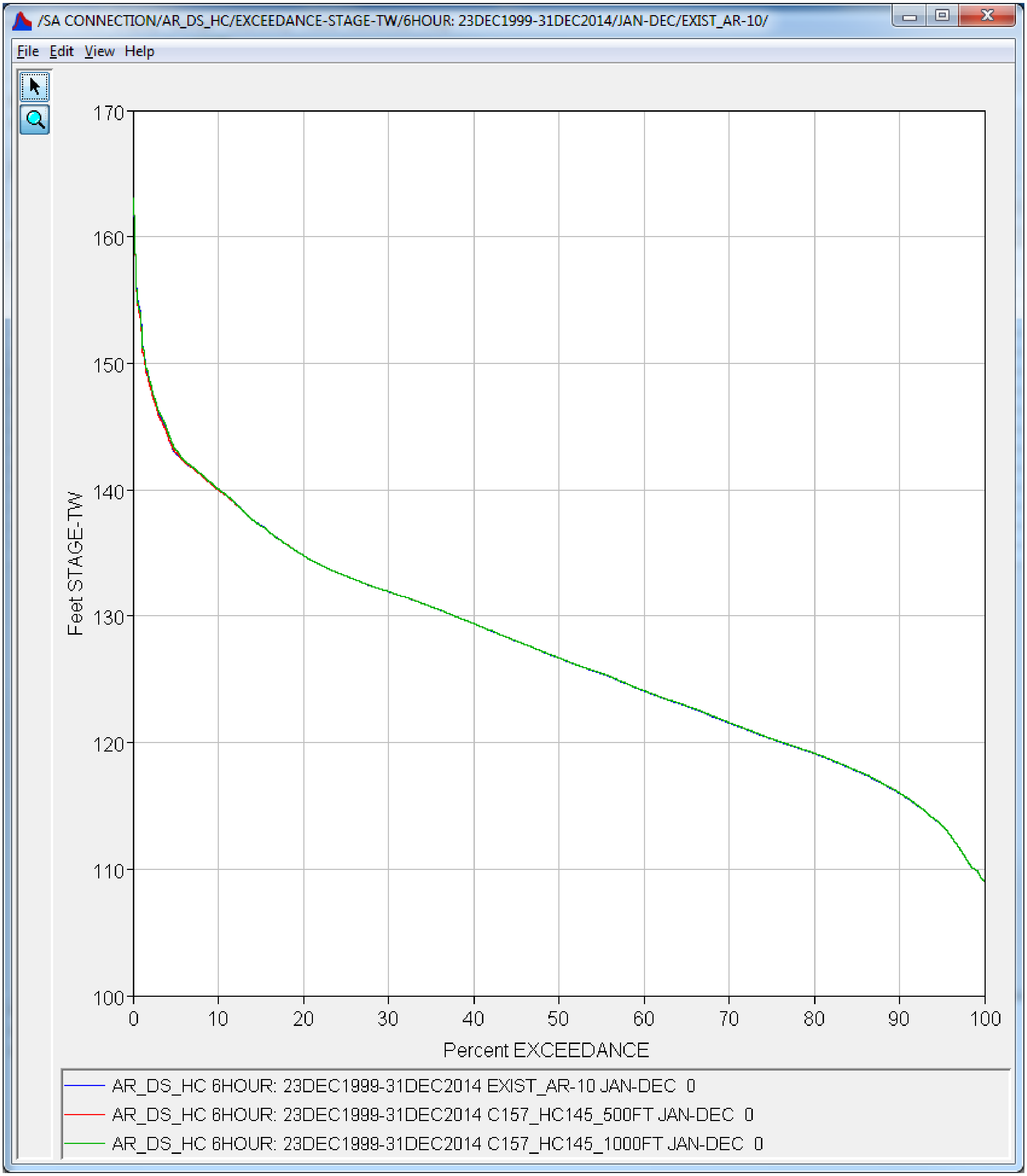
Arkansas River at Yancopin Stage Gage

______ Existing ______C157 HC145 500ft Wide _____C157 HC145 1000ft wide



Arkansas River Downstream of Historic Cutoff

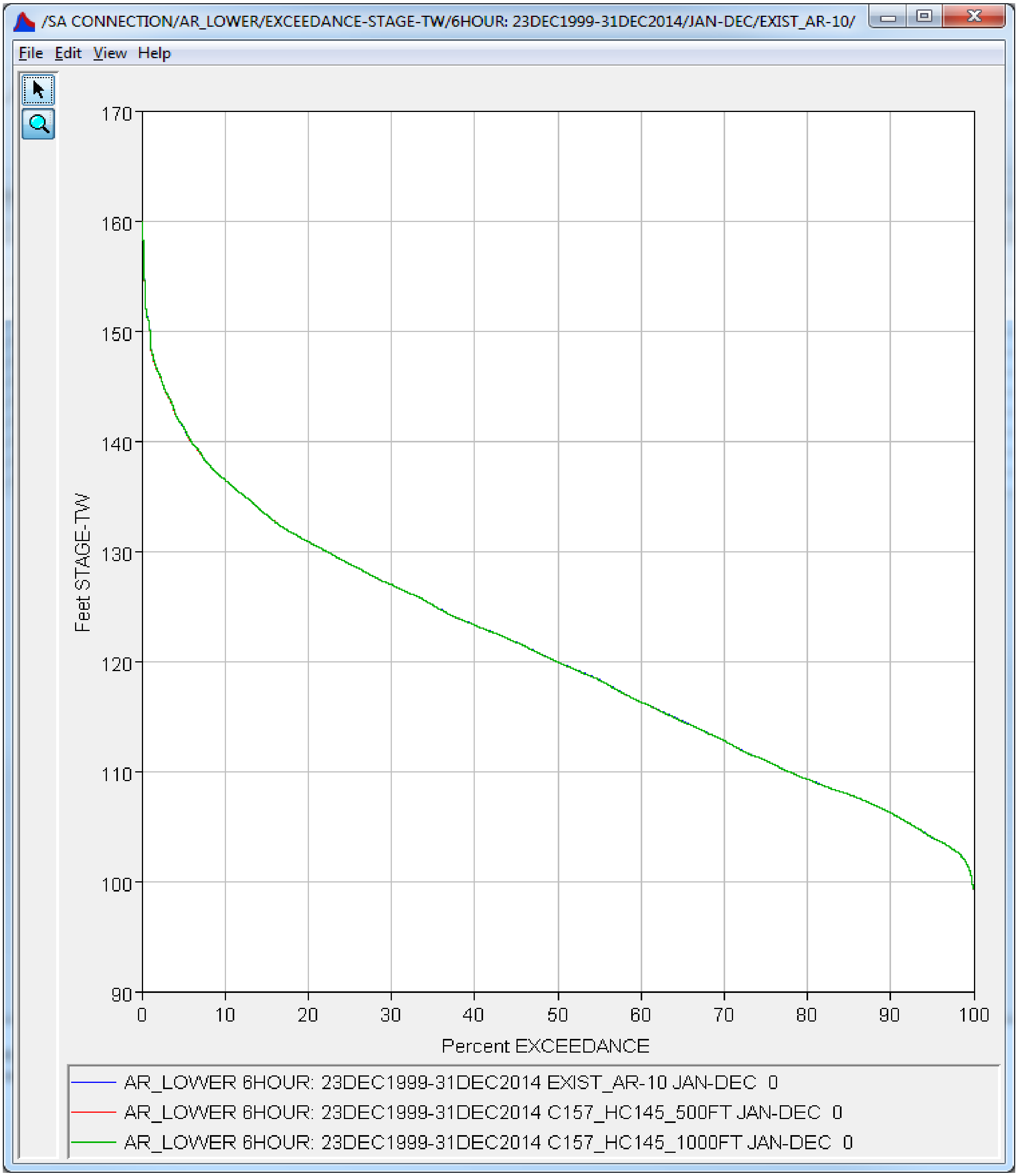
______ Existing ______C157 HC145 500ft Wide _____C157 HC145 1000ft wide



Arkansas River Lower

Arkansas River approximately 5 miles above confluence with Mississippi river

______ Existing ______C157 HC145 500ft Wide ____C157 HC145 1000ft wide



ATTACHMENT D **Lower Arkansas River SIAM Model Study**

Lower Arkansas River SIAM Model Study

By

Charles D. Little, Jr., P.E.

Research Hydraulic Engineer

Engineering Research and Development Center
Waterways Experiment Station
Coastal and Hydraulics Laboratory
River Engineering Branch

November 2005

1. INTRODUCTION

Background. The U.S. Army Corps of Engineers Little Rock District is conducting a study of the historic cut-off area between the White River and the Arkansas River. An alternative being considered in the study is a diversion structure located in the cut-off channel. The purpose of the structure will be to control the development of large hydraulic heads between the two river systems during periods of high water primarily due to Mississippi River flooding, and to reduce erosion and head cutting in the corridor between the rivers. As part of the study, the potential impacts of the alternative structures on the morphology of the lower Arkansas River must be evaluated. Of particular interest is the potential response of the lower Arkansas River to structure induced changes in the sediment regime. In order to evaluate these potential impacts, the U.S. Army Engineer Research and Development Center (ERDC) Coastal and Hydraulics Laboratory (CHL) conducted numerical model investigations of the study area. The first part of the investigation was a two-dimensional hydrodynamic and sediment transport model study of the White River and the diversion structure channel. The results of this portion of the study have been reported under separate cover. The second part of the investigation was an assessment of the sediment impacts of the alternative structures on the lower Arkansas River. This was accomplished through application of the sediment impact assessment model SIAM on the lower Arkansas River from Dam 2 to the mouth. This report describes the application and results of the SIAM model and the potential changes in sedimentation trends in the lower Arkansas River from each alternative. Geomorphic responses of the lower Arkansas River to the potential changes in sedimentation trends are discussed.

Study Objectives. The objectives of this phase of the study were to develop and apply the SIAM model to the cut-off area and the lower Arkansas River, to identify potential sedimentation trends in the lower Arkansas River due to the alternative diversion structures, and to evaluate the impacts of any potential changes and trends on the morphology of the lower Arkansas River.

Alternative Diversion Structures. The two diversion structure alternatives that were investigated in this study are a passive weir structure and an active gated structure. The passive structure consists of a wide, multi-stage uncontrolled weir located in the historic cut-off channel adjacent to the White River. The active structure consists of a low sill structure with sluice gates and overflow weir section located in the same vicinity as the passive structure alternative.

2. STUDY PROCEDURE

The general study procedure was to create the SIAM base model from the HEC-RAS model that was provided, and develop the required SIAM input data for existing conditions, the passive structure alternative and the active structure alternative. SIAM simulations were conducted for the existing condition, the results were checked for validity, and necessary adjustments to the model were made. The SIAM simulations for the passive and active structure scenarios were conducted, and the results were analyzed to determine any trends in sedimentation relative to the existing conditions results. Potential geomorphic responses to the sediment impacts of the alternative structures were assessed.

SIAM Model Description. SIAM is a reach based sediment accounting model that has been embedded in the Hydraulic Design module of HEC-RAS, and provides an expedient means of determining average annual sediment impacts for stream networks. It provides a framework to combine sediment sources and computed sediment transport capacities in order to evaluate sediment imbalances and downstream sediment yields for different alternatives. It should be noted that the current version of SIAM is in a testing and evaluation mode. It is scheduled for release as a beta version in the near future.

SIAM Base Model. The SIAM base model was developed from a HEC-RAS model of the study area developed and supplied by the Little Rock District. The original HEC-RAS model covered an extensive area and contained reaches of the White, Arkansas, and Mississippi Rivers, as well as the Melinda channel and the historic cut-off channel. The portion of the original model for the lower Arkansas River from Dam 2 to the Mississippi River was extracted for the SIAM model. The original HEC-RAS model was assumed to be accurate and calibrated, therefore no further calibration to the SIAM model was attempted. The only changes made to the SIAM model were minor adjustments to the cross section top bank stations.

The SIAM base model was then subdivided into sediment reaches as described in Table 1 and shown in Figure 1. The extents of the sediment reaches were selected primarily by the location of major channel confluences within the study reach. Reach 5 was established as a short reach near the mouth where backwater conditions from the Mississippi River could have significant impact on the computed sediment transport capacities. Since SIAM utilizes steady state hydraulics from a HEC-RAS profile computed with an assumed starting water surface elevation, it is difficult to capture the full range of backwater effects. Therefore, sediment reach 5 computations are highly subjective to the chosen starting conditions, and the results should be interpreted accordingly. SIAM computations are based on average hydraulic conditions within each sediment reach, and results are reported by reach average as well.

Table 1. SIAM model sediment reach descriptions

Reach	Description
1	Dam 2 to above Yancopin Bridge
2	Above Yancopin Bridge to Melinda channel
3	Melinda channel to Historic cut-off
4	Historic cut-off to near Callie Lake
5	Near Callie Lake to mouth

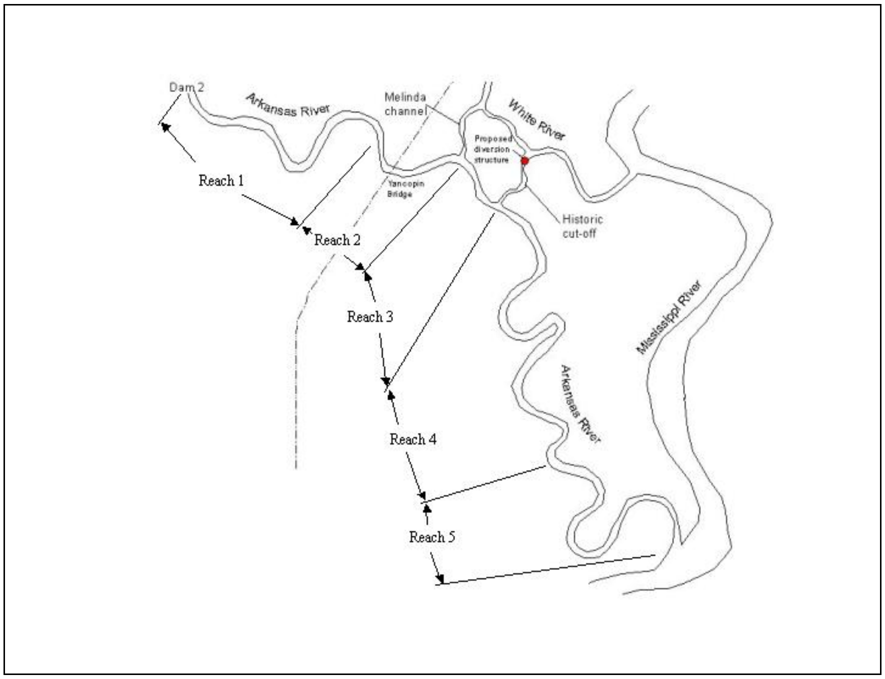


Figure 1. SIAM sediment reaches locations

SIAM Input Data. SIAM input data for each sediment reach consists of bed material composition, hydrology/flow duration, sediment properties, sediment loading from local sources and reach average hydraulic parameters.

Bed Material Data. Gradations from bed material samples on the lower Arkansas River collected by ERDC in 2002 and 2003 were used to determine an average gradation for the SIAM input data. A single representative gradation was used for all SIAM sediment reaches, and is shown in Figure 2.

Hydrology/Flow Duration. Flow duration data were developed from a database of hydraulic routing results furnished by the Little Rock District. The period of record for the routings was approximately 20 years, and the routings were made using HEC-RAS unsteady simulation. Daily routed flows for the period of record were provided for existing conditions and each alternative at river mile 20, 17.2, 15.5 and 4.5. The flow duration curve at river mile 4.5, representative of changes on the lower river due to each structure alternative, is shown in Figure 3.

Discharge rates of 5,000, 10,000, 25,000, 50,000, 100,000, 150,000, 200,000 and 300,000 cfs were selected for the HEC-RAS steady state analysis. The duration curves were used to determine the average annual number of days associated with each discharge rate. A summary of the hydrology input data for existing conditions, the passive structure alternative, and the active structure alternative are listed in Tables 2 through 4, respectively.

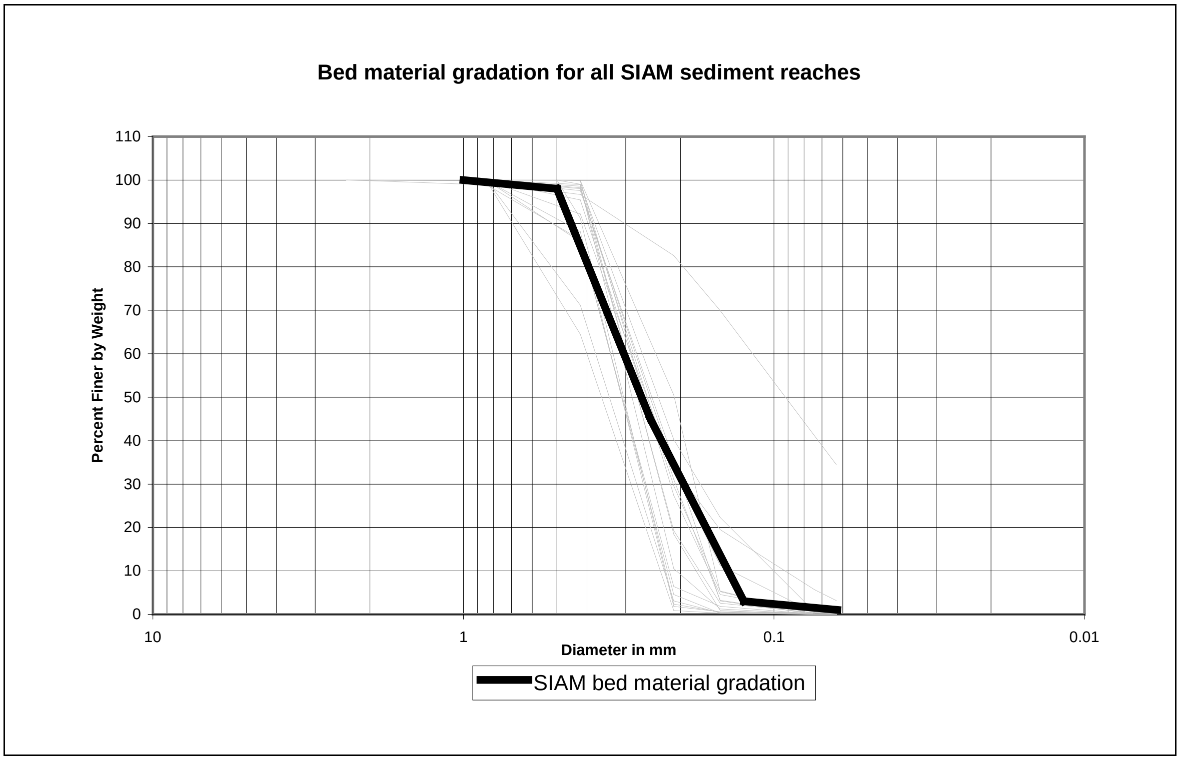


Figure 2. Bed material gradation for all SIAM sediment reaches

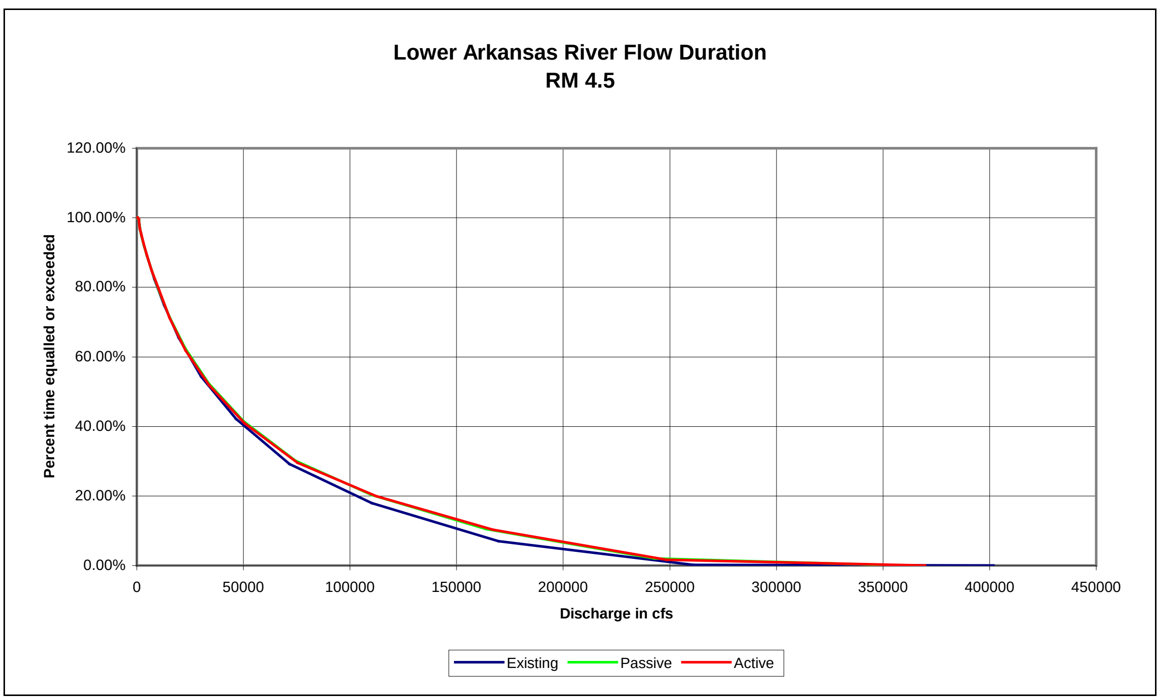


Figure 3. Flow duration for lower Arkansas River developed from HEC-RAS routings

DRAFT

Table 2. Hydrology input data for existing conditions

Reach	Average number of days each discharge occurs annually							
	5,000 cfs	10,000 cfs	25,000 cfs	50,000 cfs	100,000 cfs	150,000 cfs	200,000 cfs	300,000 cfs
1	37.6	55.4	73.5	75.4	49.2	31.0	17.3	2.2
2	37.6	55.4	73.5	75.4	49.2	31.0	17.3	2.2
3	37.4	56.2	71.1	74.5	47.3	31.2	20.0	4.6
4	37.6	56.4	71.2	75.6	47.3	31.6	20.2	3.8
5	37.6	56.4	71.2	75.6	47.3	31.6	20.2	3.8

Table 3. Hydrology input for passive structure alternative

Reach	Average number of days each discharge occurs annually							
	5,000 cfs	10,000 cfs	25,000 cfs	50,000 cfs	100,000 cfs	150,000 cfs	200,000 cfs	300,000 cfs
1	37.6	55.4	73.5	75.4	49.2	31.0	17.3	2.2
2	37.6	55.4	73.5	75.4	49.2	31.0	17.3	2.2
3	37.2	56.3	72.1	75.0	47.5	31.6	19.3	3.0
4	37.2	55.0	69.3	72.6	45.3	30.4	26.9	6.9
5	37.2	55.0	69.3	72.6	45.3	30.4	26.9	6.9

Table 4. Hydrology input for active structure alternative

Reach	Average number of days each discharge occurs annually							
	5,000 cfs	10,000 cfs	25,000 cfs	50,000 cfs	100,000 cfs	150,000 cfs	200,000 cfs	300,000 cfs
1	37.6	55.4	73.5	75.4	49.2	31.0	17.3	2.2
2	37.6	55.4	73.5	75.4	49.2	31.0	17.3	2.2
3	37.4	56.1	71.9	74.5	47.7	32.0	19.3	3.2
4	37.5	56.1	69.5	72.3	43.8	30.0	28.6	6.0
5	37.5	56.1	69.5	72.3	43.8	30.0	28.6	6.0

Sediment Properties. The Yang bed material transport function (1973) was selected for all sediment reaches in SIAM. The Yang transport function is a total bed material load predictor that has had successful application in sand bed channels with a particle size range of 0.15 to 1.7 mm. A comparison of sediment discharge computed with the Yang function and estimates from ERDC measured data for the Arkansas River near Yancopin Bridge is shown in Figure 4.

The wash load threshold diameter for all sediment reaches was set at 0.125 mm, which is the upper limit for very fine sand. Based on the average bed material gradation, approximately 3 percent of bed material in the lower Arkansas River is finer than this threshold diameter.

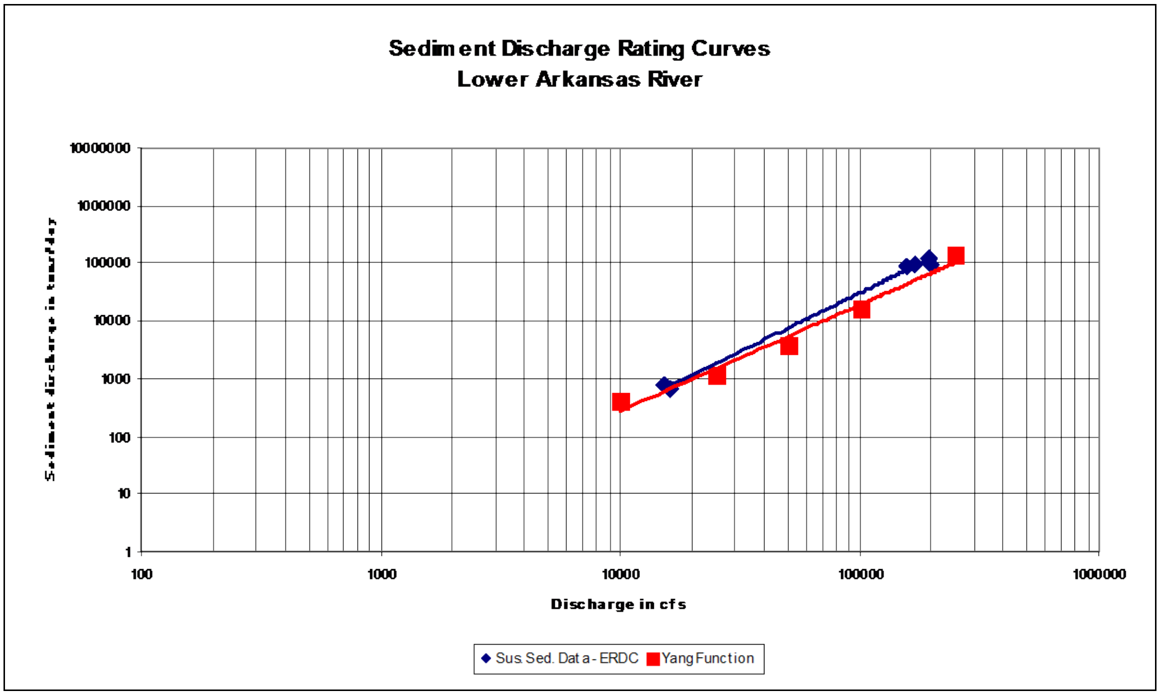


Figure 4. Comparison of sediment discharge computed with Yang (1973) transport function and ERDC measured data

Sediment loadings from local sources. The three local sediment source loads that were used as SIAM inputs were the inflowing load on the Arkansas River, and the loads at the historical cut-off channel for the passive and active structure alternatives. Caving channel banks within the study area are also legitimate sources of sediment, but quantification of the source was beyond the scope of this study. Exclusion of the bank erosion source will not alter the relative impacts of the alternative structures.

The inflowing load on the Arkansas River was used as the upstream sediment boundary condition for all three SIAM scenarios. The inflowing load represents the average annual sediment discharge for the lower Arkansas River. It was estimated by combining the sediment discharge rating curve based on the ERDC measured data with the flow duration curve for existing conditions determined from the HEC-RAS routings. The load was distributed based on an average grain size distribution curve representative of gradations from ERDC suspended sediment samples obtained in 2003. The computed average annual sediment load is approximately 7,339,000 tons/year. The average grain size distribution used to compute the average inflowing load is shown in Figure 5, and the distributed load is shown in Table 5. There is significant variability in the observed suspended sediment gradations; however, a sensitivity analysis was not conducted since any changes in inflowing load would only affect the sediment balance of the most upstream sediment reach. The sediment balance of all other sediment reaches would remain the same.

The sediment loads for the passive and active structure alternatives were estimated by a similar method as the inflowing load. Sediment rating curves for each alternative were developed from the results of the two-dimensional model, and were combined with flow duration curves developed from the HEC-RAS routings for each alternative. These sediment loads represent an average annual sediment discharge for each alternative. The computed average annual sediment loads for the passive and active structure alternatives are 2875 tons/year and 13,368 tons/year, respectively. In comparison, these loads are just a fraction of a percent of the computed annual inflowing load. The sediment loads were distributed based a grain size distribution determined from the two-dimensional model results as shown in Table 6. The D_{50} of the diversion structure sediment loads from the two-dimensional model is 0.2 mm. The distributed sediment load for each alternative is listed in Table 7. No range of grain size distribution was provided from the two-dimensional model results; therefore no sensitivity analysis was conducted.

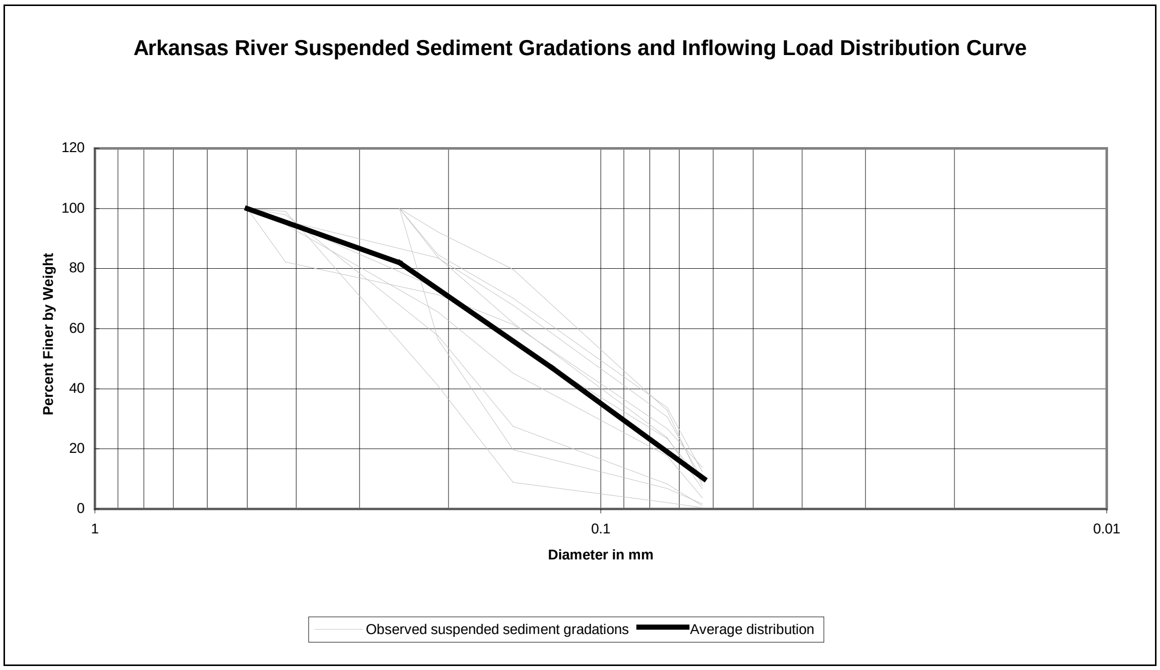


Figure 5. Average grain size distribution of computed inflowing sediment load for the lower Arkansas River

Table 5. Distributed inflowing sediment load for lower Arkansas River

Grain Size (mm)	Average Annual Sediment Load (tons/year)
0.25-0.5 mm (medium sand)	1,321,004
0.125-0.25 mm (fine sand)	2,568,620
0.0625-0.125 mm (very fine sand)	2,715,398
0.032-0.0625 mm (coarse silt)	733,891
Total	7,338,913

Table 6. Grain size distribution for passive and active alternatives (from two-dimensional model results)

Grain Size (mm)	Distribution Fraction (%)
0.25-0.5 mm (medium sand)	0.2
0.125-0.25 mm (fine sand)	0.7
0.0625-0.125 mm (very fine sand)	0.1

Table 7. Distributed sediment loads for passive and active structure alternatives

Grain Size (mm)	Average Annual Sediment Load (tons/year)	
	Passive Structure	Active Structure
0.25-0.5 mm (medium sand)	575	2,674
0.125-0.25 mm (fine sand)	2,012	9,358
0.0625-0.125 mm (very fine sand)	288	1,336
Total	2,875	13,368

Reach average hydraulic parameters. The reach average hydraulic parameters for each flow in the Hydrology/Flow Duration Table are calculated from the HEC-RAS steady state results. The hydraulic parameters for each sediment reach are averaged using a reach length weighted method within HEC-RAS. The hydraulic data input tables for SIAM are automatically populated with the results. An example of a hydraulic input data table is shown in Figure 6.

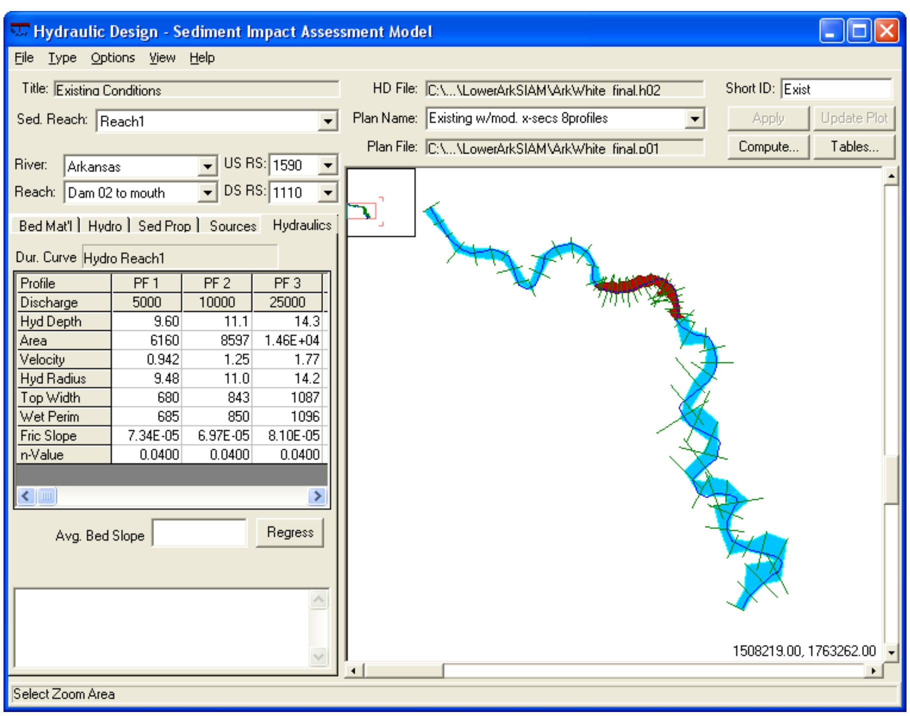


Figure 6. Example of SIAM hydraulic input data table

3. RESULTS AND DISCUSSION

The results from the SIAM assessment are presented according to sediment reach. Emphasis is placed on sediment reach 3 (Melinda channel to historic cut-off) and sediment reach 4 (immediately downstream of the historic cut-off). Results for sediment reach 4 are considered representative of the potential response of the entire lower Arkansas River downstream of the cut-off.

Bed Material Local Balance. The primary output from the SIAM results is the bed material local balance by reach. Bed material local balance is defined as the difference between the annual bed material supply and the annual transport capacity for a given reach in tons/year. Local balance typically indicates whether a reach has a tendency to scour bed sediments (excess transport capacity, negative local balance) or to deposit bed sediments (excess supply, positive local balance).

The bed material local balance for existing conditions and for the passive and active structure alternatives is shown in Table 8 and in Figure 7.

Table 8. Bed material local balance for existing conditions, passive structure and active structure alternatives

Reach	Bed material local balance (tons/year)		
	Existing	Passive	Active
1	840,000	840,000	840,000
2	-592,000	-592,000	-592,000
3	-727,000	-511,000	-553,000
4	897,000	74,900	163,000
5	1,010,000	1,330,000	1,320,000

The local balances for sediment reaches 1 and 2 are the same for all scenarios, and indicate a tendency for bed sediment deposition and scour, respectively. The local balance of reach 1 is directly impacted by the inflowing sediment load boundary condition, and inaccuracies in the inflowing load estimate could affect the result. The local balance for reach 2, the reach between Yancopin Bridge and the Melinda channel, indicates a trend toward scour. The local balance for sediment reach 3, the reach from Melinda channel to the historic channel, also indicates a tendency to degrade, but the degradation potential for the passive and active structure alternatives is less than for existing conditions. The bed material supply for this reach is equal to the transport capacity of reach 2, and is the same for all scenarios. The difference in the local balance is caused by decreases in the transport capacity of reach 3. For the passive and active structure alternatives, moderate to high flows that come through the Melinda channel (and hence through sediment reach 3) for existing conditions are diverted through the structures and enter the Arkansas River at sediment reach 4. This results in a change in the annual flow duration that, in turn, reduces the annual transport capacity for reach 3.

Less transport capacity in the reach results in a lower degradation tendency indicated by the local balance results.

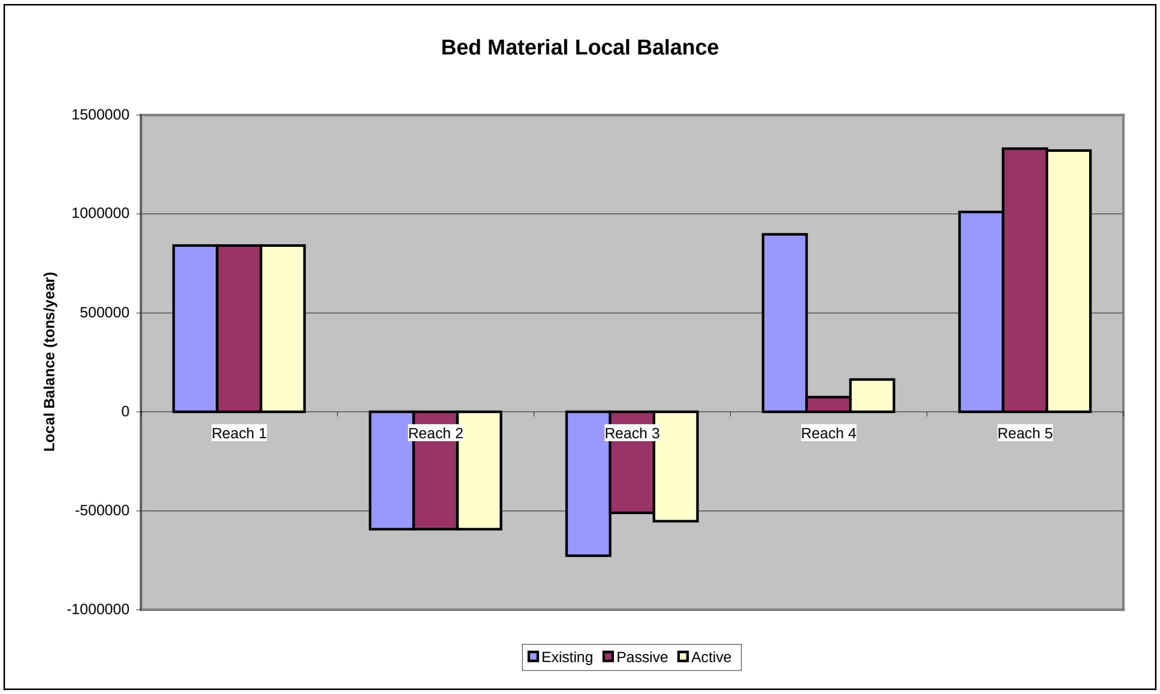


Figure 7. Bed material local balance for lower Arkansas River

The change in local balance for sediment reach 4, the reach downstream of the historic cut-off channel, is the most significant of all reaches. The positive local balance for existing conditions indicates a tendency for deposition, but the local balance for the alternatives indicates that this tendency is significantly less for the passive and active structures. This change can best be explained by considering the bed material supply and transport capacity of reach 4. The total bed material supply for reach 4 consists of channel supply from reach 3 and supply from the diversion structure. The bed material channel supply from reach 3 to reach 4 is equivalent to the transport capacity of reach 3. As discussed in the previous paragraph, the transport capacity for reach 3 is less for the alternatives than for existing conditions, therefore the supply to reach 4 is correspondingly less. As was also discussed earlier, the diverted flow through the alternative structures also increases the average annual duration of moderate to high flows for reach 4 (see Figure 3), which increases the annual transport capacity of the reach. The lower bed material supply and the higher transport capacity results in the lower local balance for the reach.

The local balance for sediment reach 5 indicates an increased tendency for bed material deposition for the alternatives. Since the transport capacity of reach 4 is greater for the alternatives, the corresponding bed material supply for reach 5 is greater, resulting in an increase in deposition potential. However, the range of backwater effects of the Mississippi River that would directly impact reach 5 cannot be adequately captured using steady state simulations in HEC-RAS, so SIAM results from this reach should be viewed with caution.

The local bed material balance results from SIAM give a general indication of the potential changes in sedimentation trends that may occur within each sediment reach as a result of the alternative structures. However, the local balances may not clearly illustrate the scale of potential changes. In order to give some physical representation to the results, the local balances for each sediment reach were converted to average annual bed elevation changes. The local balances were converted to an average annual volume using an assumed unit weight of sediment, and the average annual bed elevation change was determined by dividing the computed volume by the product of the reach length and an average channel width. The computed bed elevation changes are shown in Table 9.

Table 9. Average annual bed elevation changes computed from bed material local balances

Reach	Average annual bed elevation changes (ft)		
	Existing	Passive	Active
1	0.39	0.39	0.39
2	-0.98	-0.98	-0.98
3	-2.10	-1.48	-1.60
4	0.39	0.03	0.07
5	0.79	1.04	1.03

These data indicate reasonable potential changes in bed elevation for average annual conditions. The bed changes in sediment reach 3 appear the most suspect, although bed changes of this order are not unreasonable. One possible explanation is the fairly short length of reach 3. The reach average velocities in this reach are the highest of all sediment reaches in the SIAM model. Averaging of hydraulic parameters across such a short reach may result in a higher reach average velocity and consequently greater transport capacity that may not be representative of the reach as a whole. Regardless of the validity of the magnitude of bed elevation change, the relative impacts of the diversion structures can still be seen. The bed change estimates for sediment reach 4 due to the passive and active structure alternatives indicates less bed sediment deposition than for existing conditions. This indicates a potential to move more bed material with the diversion alternatives, but the potential change is considered minor and should not adversely impact the long-term morphology of the lower Arkansas River to any significant degree.

Average Annual Transport Capacity. The potential impacts of the diversion structure alternatives on the average annual flow duration of the lower Arkansas River can be seen in the SIAM computations for average annual transport capacity. The transport capacity computed by SIAM is for total bed material load. The average annual transport capacity for all three scenarios is shown in Table 10 and Figure 8. The transport capacity for sediment reach 3 is less for the diversion structure alternatives than for existing conditions, and for reaches 4 and 5 the transport capacity is greater for the alternatives. These changes are due to impacts on the annual flow duration between the Melinda channel and the historic cut-off channel as a result of the diversion structures. Without the diversion structures, flow exchange between the White River and the Arkansas River only occurs via the Melinda channel, except when the containment structure is overtopped during floods. With the diversion structures in place, portions of the moderate to high flows bypass the Melinda channel and enter the river via the historic cut-off channel. The diversion structures apparently alter the frequency at which these moderate to high flows occur on an average annual basis, effectively decreasing the number of days of occurrence for reach 3 and increasing the number of days for reach 4. This change in duration can be seen in Figure 3 in the duration curves developed from the period of record HEC-RAS routings. This change in flow duration is sufficient to decrease and increase the transport capacity of reaches 3 and 4, respectively.

Table 10. Average annual transport capacities

Reach	Average annual transport capacities (tons/year)		
	Existing	Passive	Active
1	3,050,000	3,050,000	3,050,000
2	3,640,000	3,640,000	3,640,000
3	4,370,000	4,150,000	4,200,000
4	3,470,000	4,080,000	4,040,000
5	2,460,000	2,750,000	2,720,000

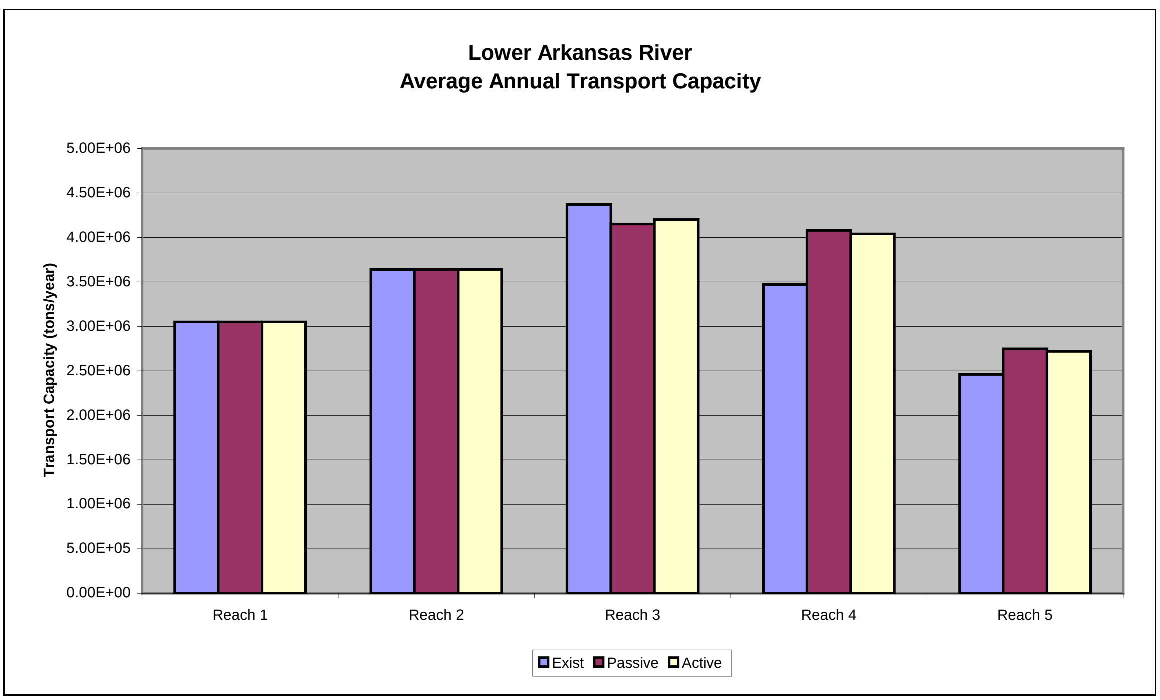


Figure 8. Average annual transport capacity computed in SIAM

Average Annual Bed Material Supply. In general, the bed material supply for a sediment reach in SIAM consists of three parts: 1) channel supply from adjacent upstream reaches (equal to the transport capacity of the upstream reach), 2) supply from local sediment sources within the current reach, and 3) wash load from upstream reaches that transitions into bed material in the current reach. The wash load threshold diameter for all sediment reaches in this study was the same, therefore no wash load will become bed material supply for any reach. The only local sediment sources used in the SIAM study were the inflowing boundary load at sediment reach 1 and the load for each diversion structure alternative (from the two-dimensional model results) at sediment reach 4. The only influences to the bed material supply for reach 4 are the supply from the diversion structures and the channel supply from the sediment reach 3. The average annual bed material supply for all three scenarios is shown in Table 11 and Figure 9.

Table 11. Average annual bed material supply

Reach	Average annual bed material supply (tons/year)		
	Existing	Passive	Active
1	3,890,000	3,890,000	3,890,000
2	3,050,000	3,050,000	3,050,000
3	3,640,000	3,640,000	3,640,000
4	4,370,000	4,160,000	4,210,000
5	3,470,000	4,080,000	4,040,000

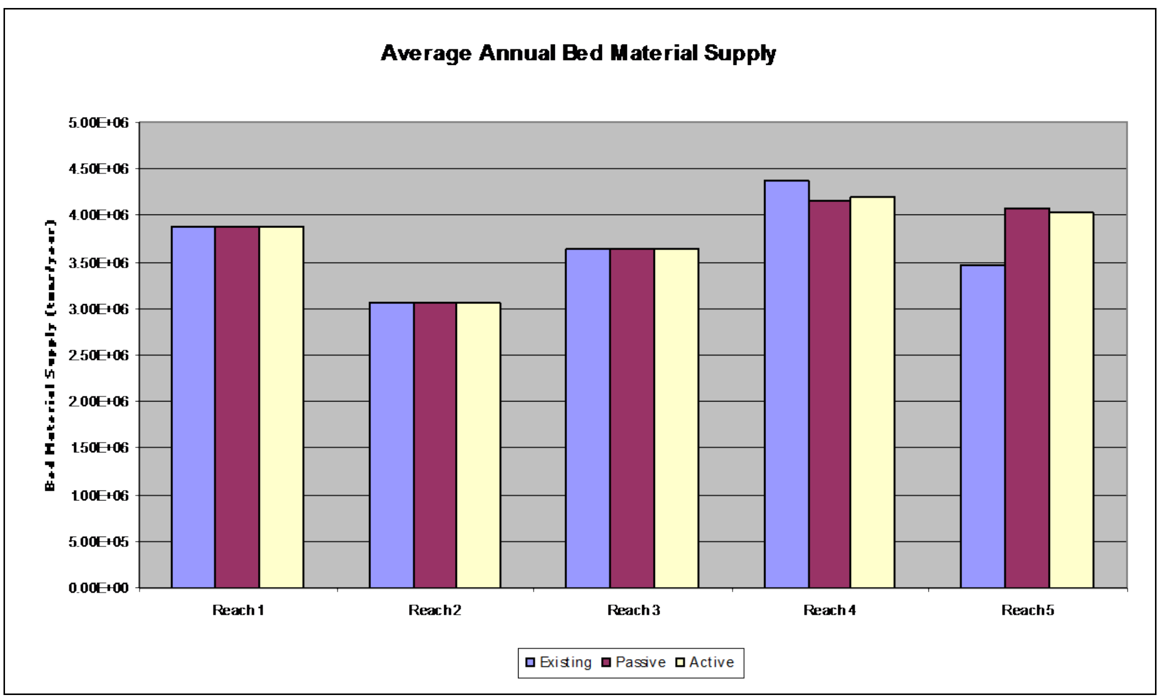


Figure 9. Average annual bed material supply computed in SIAM

The only changes in bed material supply due to the diversion structures are in sediment reaches 4 and 5. The bed material supply for reach 4 is slightly less for the structure alternatives than for existing conditions. This decrease in bed material supply may seem unexpected at first, since bed material is being added to the reach from the diversion structures. Since the transport capacity of reach 3 is lower for the alternatives than for existing conditions, it follows that the channel supply of bed material from reach 3 to reach 4 is also lower. According to Table 10, the transport capacity of reach 3 is approximately 150,000 to 200,000 tons/year lower for the alternatives than for existing conditions, and the channel supply of bed material to reach 4 is proportionately less as well (see Table 11). The bed material supply from the diversion structure alternatives is 2,875 tons/year for the passive structure and 13,368 tons/year for the active structure. These are small percentage increases in bed material supply compared to the large reduction in channel supply from reach 3, and the result is an overall net decrease in bed material supply for reach 4.

4. CONCLUSIONS

Based on the results of the SIAM assessment for the alternative diversion structures, the following conclusions are provided.

1. *The potential impact of the bed material diverted by both alternative structures on the lower Arkansas River is minimal*. In comparison to the average annual bed material transport capacity for the lower river, the average annual sediment load delivered by the diversion structures is less than 0.5 percent. The potential influx of bed material sediment through the structures should have very little long-term influence on the overall sediment balance and stability of the lower river channel.

2. *Changes in annual flow duration due to the alternative structures results in impacts to annual sediment transport capacity*. The diversion structures alter the volume and timing of moderate to high flows through the Melinda channel and the historic cut-off channel, resulting in a change in the annual flow duration relationship. The general effect of the change on the annual sediment transport capacity is a decrease in capacity from Melinda channel to the historic cut-off, and an increase in capacity downstream of the historic cut-off.

3. *There is very little relative difference between the potential impacts of the two diversion alternatives*. Both structures result in very similar potential sediment impacts to the lower river. In general, the active gated structure tends to be more similar to existing conditions.

4. *In general, impacts from the alternative diversion structures on the morphology of the lower Arkansas River are expected to be minimal*. Since the impact on sediment supply and transport due to the structures is relatively small in comparison to existing conditions, no significant impacts to the long-term morphology of the river are expected. Minor local impacts to channel stability immediately after significant flood events would not be

surprising or unexpected. Historically, since the lower Arkansas River has been a very active channel morphologically (Pinkard, et al., 2003), the degree of any potential channel changes due to the structures would be almost indistinguishable from changes that typically occur within the lower reach.

5. REFERENCES

Pinkard, C.F., Jr., D.S. Biedenharn, C.D. Little, Jr., P.H. Hoffman (2003), Arkansas-White Rivers Preliminary Geomorphic Assessment, Final Report, Engineering Research and Development Center, Coastal & Hydraulics Laboratory, Vicksburg, MS.

Yang, C.T. (1973), "Incipient Motion and Sediment Transport", Journal of the Hydraulics Division, ASCE, Vol. 99, No. HY10, Proc. Paper 10067, pp. 1679-1704.

ATTACHMENT E **Sediment Diversion Simulations for the Arkansas / White River Systems**

Sediment Diversion Simulations for the Arkansas / White River System

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Background and Study Goals

This report describes the results of a series of two dimensional (2D) sediment transport model simulations conducted to evaluate the sediment diversion potential of two diversion structure alternatives. These alternatives are being considered to alleviate extreme head differentials between the Arkansas and White Rivers during Mississippi River flood events. The two proposed diversion alternatives consist of a 2000 ft weir and an approximately 1000 ft gated structure in combination with a weir. Five flood events were simulated to evaluate the impacts of the diversions.

The simulations had two main goals: 1) to estimate the sediment discharge through the proposed White River diversion structures and 2) estimate the impact to the White River channel due to the diversion process.

The model simulations are reported in metric units. Table 1 provides a conversion from metric to English.

2D Model Design

The 2D model consists of a six-mile segment of the Mississippi River along with a twelve-mile segment of the lower White River. Figures 1- 3 describe the model mesh bathymetry along with the diversion designs. The mesh does not contain over bank elevations. The White River top bank elevation represented in the mesh is about 145 ft msl. Because no recent channel survey data were available, Montgomery Point lock and dam was not included in the mesh.

Both the Melinda and Historic Channel diversions are represented in the mesh. However, the diversion weirs / structures are not represented in the mesh. The discharge through the weirs / structures was taken directly from the HECRAS simulation for each of the selected flood events and used as an outflow boundary condition at the proposed diversion location.

Bed sediment size gradation data was obtained from Historic records on the Mississippi River and from more recent field data collection efforts on the White River (Waller 2002). The bed sediment size fractions used in the simulations are found in Figure 4.

Simulated Flood Events

Five flood events were simulated with the 2D model. A summary of these events is found in Table 2. The events were chosen to evaluate the impacts of the diversion flows when peak flows originate from the Mississippi River, the White River, or a combination of both. Both passive weir and active structures were simulated for the 1997 and 1984 events, which represent the highest sediment diversion and White River channel impact. Additionally, for the 1997 event, an existing condition simulation was conducted for comparison (no diversion structure at the Historic Channel).

Figures 5-9 show the relative stage (Mississippi River, Historic Channel, and White River) and discharge (Historic Channel and White River) to offer some perspective on the magnitude and direction of flow from the respective Mississippi River and White River channels. The stage at the Historic Channel, White River, and Mississippi River was taken from the HECRAS model at locations 3145, 11, and 599 respectively. Of the five flood events, the 1997, 1989, and 1984 produced the most significant sediment discharge through the diversion and impacts to the lower White River channel. The 1989 event was characterized by a high White River discharge and a low Mississippi River stage, resulting in a predominantly White River Diversion. The 1997 event produced significant diversion of both the White and Mississippi River floodwaters, with the peak diversion predominantly from the White River. The 1984 event was characterized by a high Mississippi River stage and a low White River discharge, with the peak diversion from the Mississippi River.

Suspended Sediment Validation

Before the plan simulations were conducted, the model was validated to field data collected by EDRC-WES in April 2002 (Waller 2002). Suspended sediment profiles were measured by ERDC-WES in the White River at about RM 8. These measurements were compared to depth averaged suspended sediment concentrations computed by the model. The validation boundary conditions are found in Table 3, with the comparison between computed and measure suspended sediment found in Table 4. The measured data was for a depth averaged vertical profile, whereas the computed average was for depth-averaged concentrations averaged over the width of the channel. Only sand sized sediments (bed material fractions) were included in the comparison.

The verification flow boundary conditions indicate a bank full event (~145 ft msl) in the White River. The measured bed material suspended sediment concentration is low, with an averaged value of about 10 mg/l. One important aspect of the measured data is the fact that the top 12 feet of the White River water column did not contain any sand sized sediment for this bank full event, indicating that sand sized sediments would potentially not be available for diversion over a weir in any appreciable quantities. Validation hydrodynamics and sediment concentration computations are shown in Figures 10 – 12.

Results of Simulations

The simulation results are found in Figures 13 – 40. Diversion hydrodynamics (velocity and shear stress distribution), sediment diversion concentration, and White River bed change are depicted for the 1997, 1989, and 1984 events. For the 1997 event an existing condition simulation was conducted without a Historic Channel diversion for comparison. Figures 37 – 39 show the sediment discharge through the diversion structure for the various events, with Figure 40 comparing the estimated sediment transport capacity of the Arkansas River to the sediment discharge through the diversions.

Impacts on Lower White River

For the 1997 and 1989 events that had a predominant discharge from the White River, the highest potential for bed change (erosion and deposition) to occur was in the upper model reach (RM 8.5 – 12, Figures 16 and 28). However, when compared to the bed change due to existing condition (Figure 17), it is apparent that the bed change potential is approximately the same, indicating no impact to this area due to the diversion plan. The highest potential for significant bed change is for both the 1997 and 1984 events that have significant flow from the Mississippi River through the diversion (Figures 19, 20, 24, 32, and 36). These figures depict the sedimentation potential at the mouth of the White River.

Model results indicate that two significant events occur when Mississippi River water enters the White River. A large scour hole appears on the right descending bank of the White River at the mouth due to the downstream momentum of the fast flowing Mississippi River water as it enters the White. Additionally, the Mississippi River is at flood flow during these simulations, thus its transport capacity is very high. This sediment laden water enters the White River which has a much lower transport capacity, thus sediment falls out of suspension in the vicinity of the White River mouth. These phenomena occurred for all simulations that had any appreciable Mississippi River flow into the White River. For the most severe cases (1997 and 1984 events) a large bar formed adjacent to and downstream of the scour hole. Model results indicate that the bed change in this area is approximately 100,000 cubic yards. The amount of sediment deposited will be dependent on the Mississippi River transport capacity, the amount of Mississippi River flow entering the White, and the duration of this flow.

Sediment Diversion Through the Proposed Structures

The sediment discharge through the diversion plans is shown in Figures 37 – 39. The model results indicate that the maximum sediment discharge potential is for the active gated structure events, with a peak discharge of about 2500 tons per day. The only difference between the passive weir and the gated structure simulations was the width of the entrance channel, therefore the results indicate that the narrower channel associated with the gated structure has a higher transport capacity, thus higher potential to transport sediment. Sensitivity simulations using a finer bed size gradation increased the peak sediment discharge by about 50 percent. The average grain size of the diverted sediment was about 0.20 mm.

To put these results in perspective, the active gated structure sediment discharge is compared to the estimated lower Arkansas River transport capacity (Figure 40). This comparison indicates that for the same total discharge, the sediment discharge through the diversion is less that the sediment transport capacity of the Arkansas by at least a factor of 10. The implications of this is that the flow in the diversion channel connecting the White and Arkansas Rivers will be sediment starved, thus having a high potential to erode the channel depending on the stage of the Arkansas River. Although the model did not include over bank flow, it can be reasonably assumed that over bank flow from the White River will contain very low concentrations of bed material sediment. Table 5 shows a comparison of model hydrodynamics and sediment transport for the validation and the 1997 and 1989 simulated events. As described earlier in this report, the model validation was for a bank full flow event. The worst case White River flow simulations (1997 and 1989) indicate that the hydrodynamic conditions were not significantly more severe than the validation conditions for which no bed material sediment was found in the top 12 feet of the water column.

Conclusions

The model study revealed two important findings concerning the proposed sediment diversion plans:

1) Field data collection activities and the modeling simulations indicate very low bed material sediment concentrations will be diverted to the Arkansas River. The diverted sediment will primarily consist of wash load (silts and clays). Model results indicated that the transport capacity of the White River was relatively low for all flow scenarios. The model simulations can be assumed to be a worst-case scenario since the weirs were not actually in the channel. With the weirs in place, only the top 10-15 feet of water will be diverted, which most likely will contain very little sand sized sediments.

2) The most significant impact from the proposed diversion plans is sedimentation at the mouth of the White River. All of the simulations were for a Mississippi River at flood stage. Peak flows in the Mississippi River were approximately 1,400,000 - 1,800,000 cfs. At these flows the sediment transport capacity is very high. As this sediment-laden water enters the White River during diversion, the transport capacity drops appreciably, thus sedimentation occurs. Additionally, model results indicate that the momentum of the fast flowing Mississippi River (~10 ft/s) will tend to significantly scour the right descending bank as it enters the White River. These processes result in a large sediment bar that extends from the mouth to about 1 mile upstream. For the 1997 event, approximately 100,000 cubic yards were deposited in the lower White River Channel (RM 0-1).

References

Waller, Terry, "*Revised Arkansas / White River Discharge and Sediment Sampling Field Effort Trip Report*", ERDC-WES Memorandum For Record to the Little Rock District, USACE, April 23, 2002.

TABLES

Table 1. Conversion from metric to English

To Convert From	Multiply By	To Obtain
meters	3.28	feet
meters per second	3.28	feet per second
cubic meters per second	35.287	cubic feet per second
pascals	0.02089	pounds per square foot
kilograms per cubic meter	1000	milligrams per liter

Table 2. Flow events simulated

Year	Event Duration	Passive	Active	Existing
2002	5/6/02 – 6/7/02 (33 days)	X		
1997	3/3/97 – 4/13/97 (42 days)	X	X	X
1991	12/25/90 – 1/31/91 (37 days)	X		
1989	2/20/89 – 3/23/89 (32 days)	X		
1984	3/26/84 – 6/6/84 (72 days)	X	X	

Table 3. Model validation boundary conditions

Mississippi Q - cms	White Q - cms	Mississippi Stage - m	Bed Roughness
23,868	2,418	44.5	0.03

Table 4. Comparison of measured and computed depth averaged suspended sediment concentration – April 11, 2002 flow event at White River Station 1C

Depth - ft	Measured Conc – mg/l	Measured Depth Averaged Conc – mg/l	* Computed Depth Averaged Conc – mg/l
1	0.0	Average = 9.5	Average = 10.7
12.2	0.0	Max = 20.0	Max = 18.7
24.5	10.0		
36.7	20.0		
48.6	17.5		

Notes: * - The average concentration represents the depth averaged concentration averaged over the width of the crossection at station 1C. The maximum value represents the highest depth averaged concentration in the channel at station 1C

Table 5. Comparison of the computed peak flow and suspended sediment characteristics of selected flow events to the computed model validation data at station WR1

Variable	Validation Event	1997 Event	1989 Event
Velocity$_{avg}$ - m/s	0.68	0.84	0.76
Velocity$_{max}$ - m/s	1.09	1.29	1.24
Conc$_{avg}$ - mg/l	10.7	18.5	16.9
Conc$_{max}$ - mg/l	18.7	33.0	28.4
Bed Shear$_{avg}$ - Pa	2.1	3.0	2.7
Bed Shear$_{max}$ - Pa	4.2	5.0	5.4

FIGURES

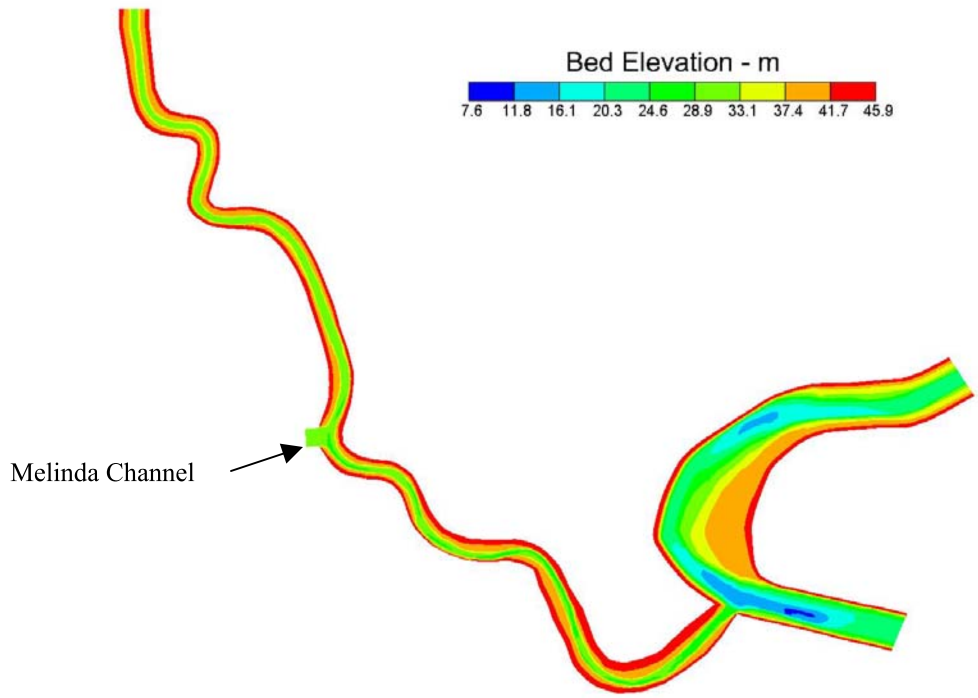


Figure 1. Exiting condition mesh

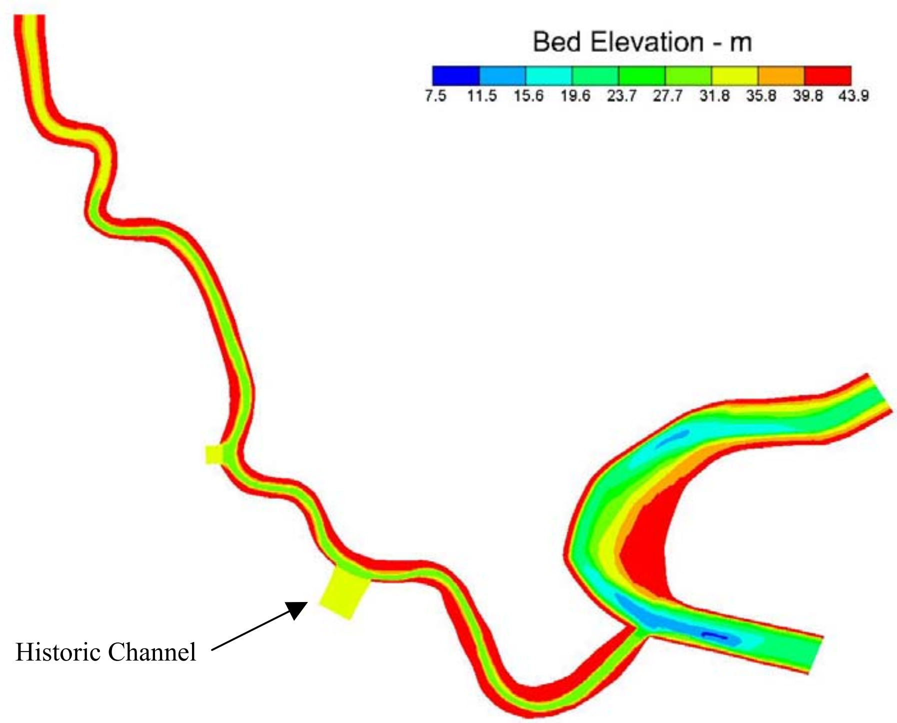


Figure 2. Passive weir mesh

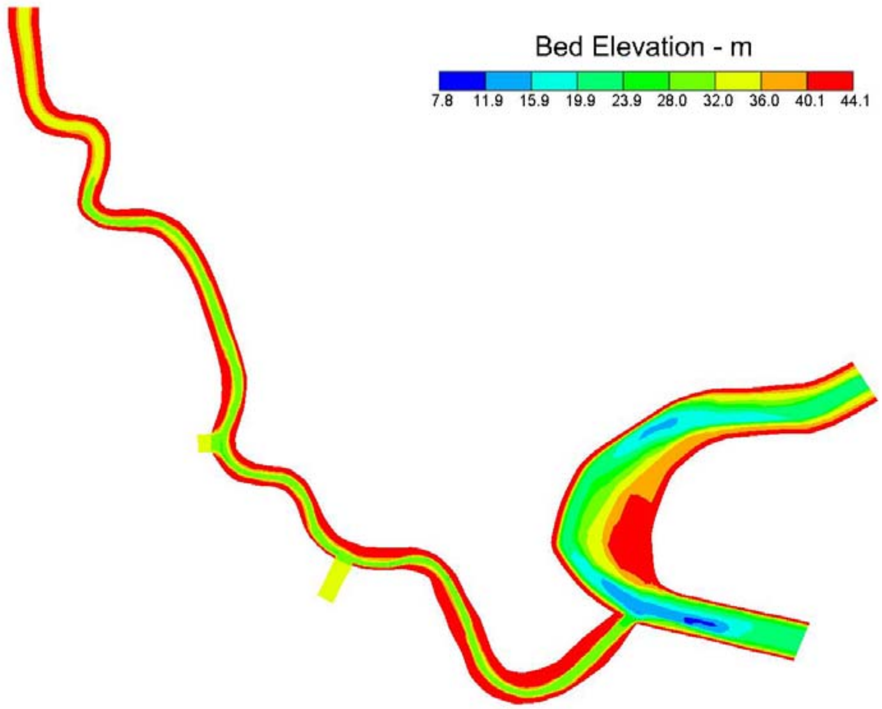


Figure 3. Active gated structure mesh

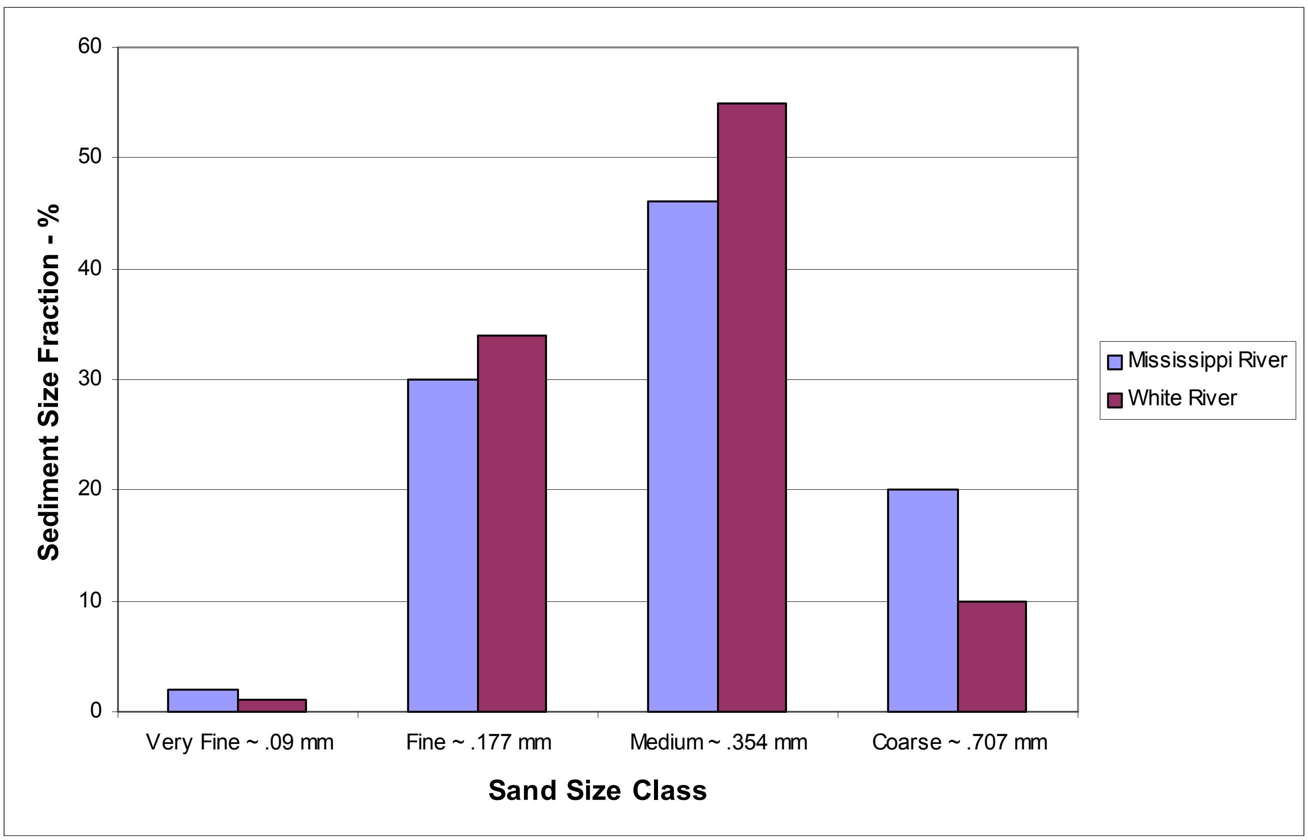


Figure 4. Bed sediment particle size fractions for the Mississippi and White Rivers

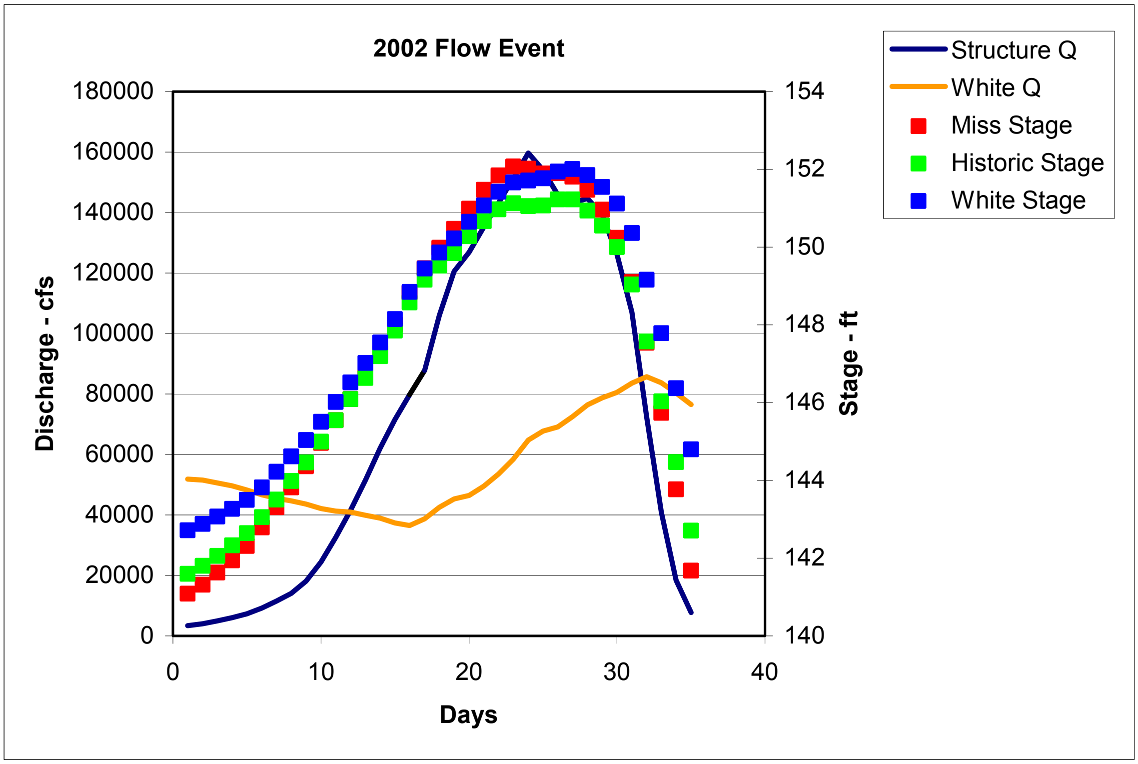


Figure 5. System hydraulics for the 2002 passive weir simulation

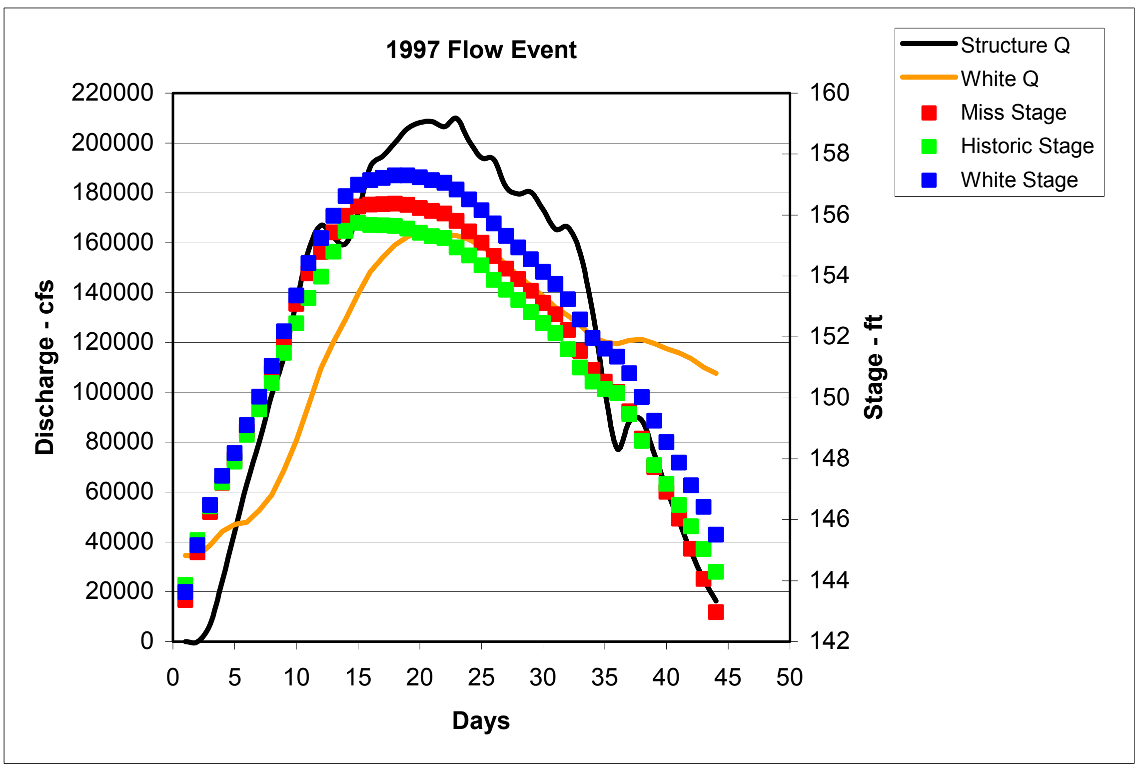


Figure 6. System hydraulics for the 1997 passive weir simulation

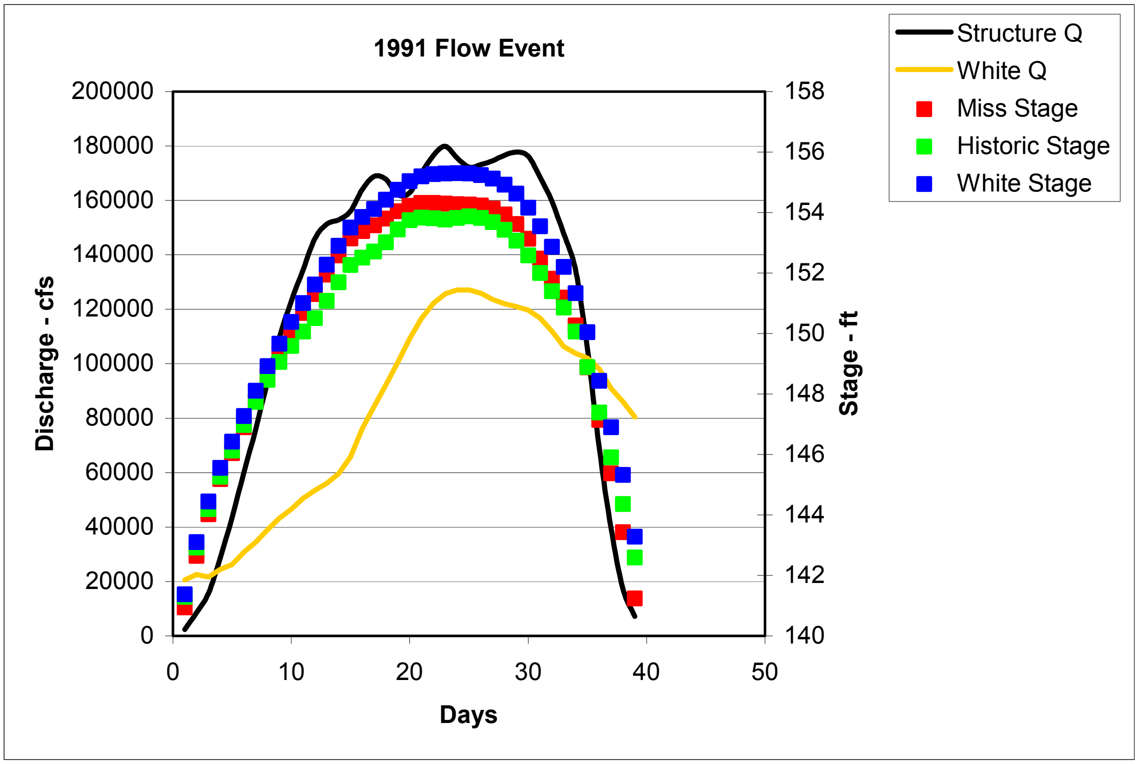


Figure 7. System hydraulics for the 1991 passive weir simulation

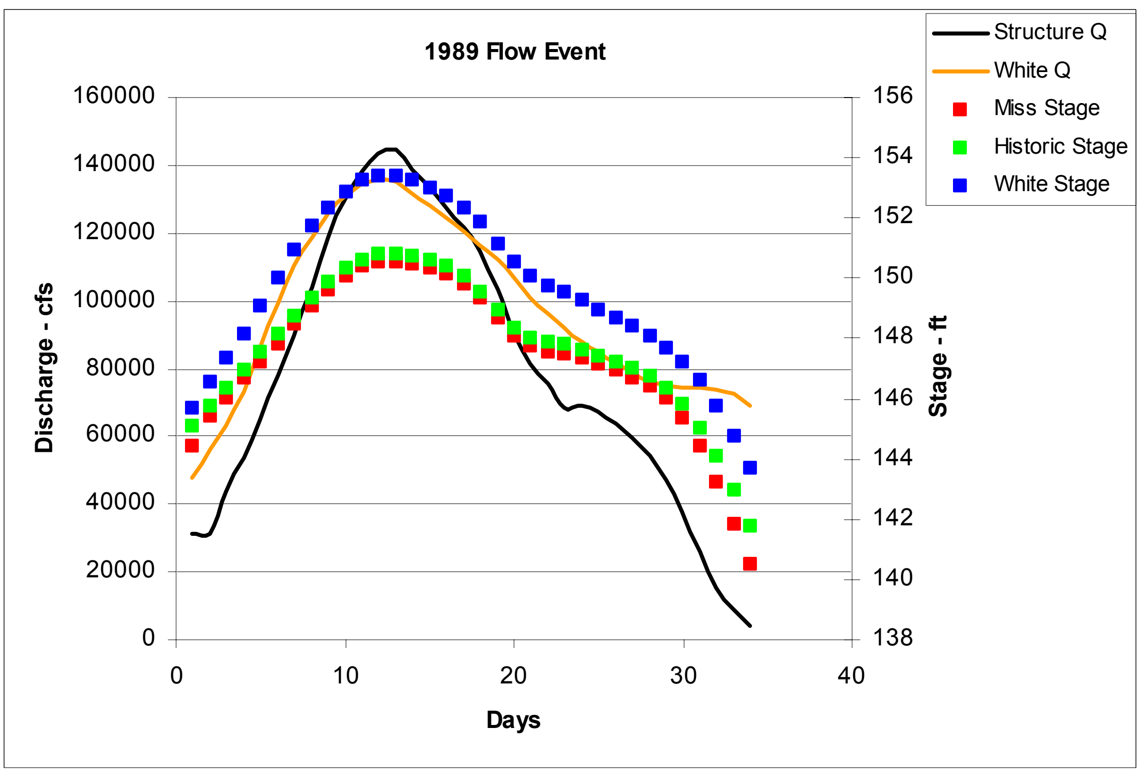


Figure 8. System hydraulics for the 1989 passive weir simulation

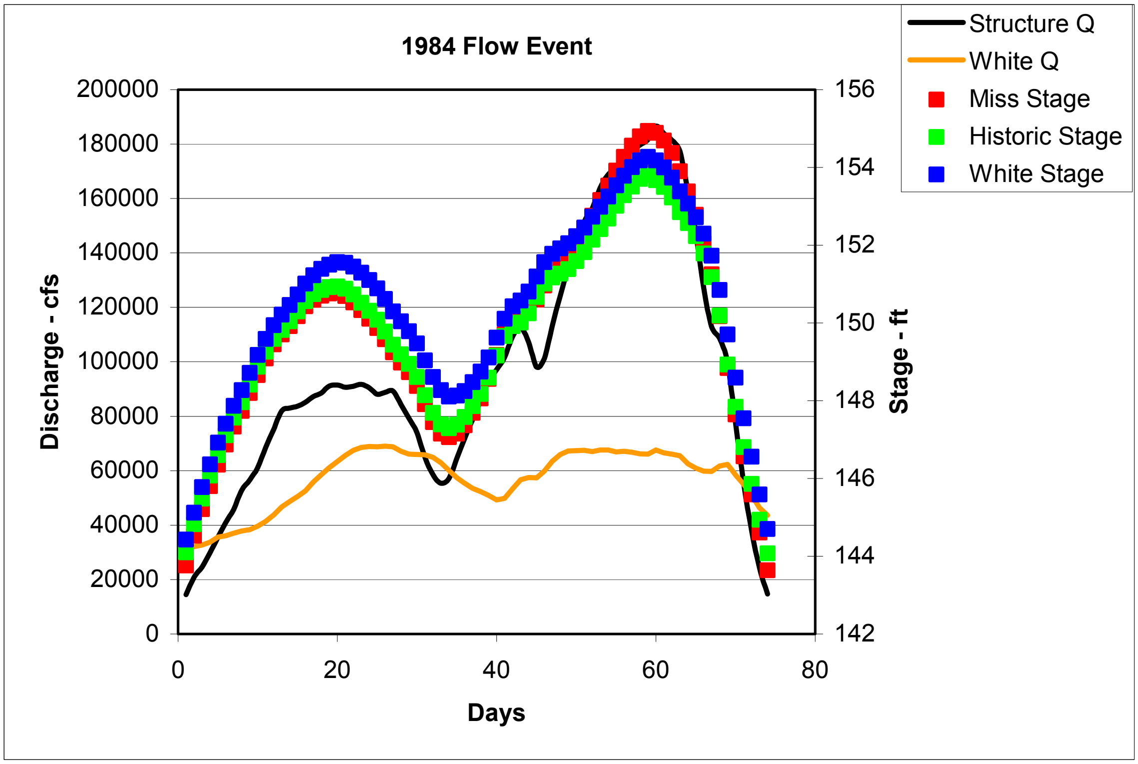


Figure 9. System hydraulics for the 1984 passive weir simulation

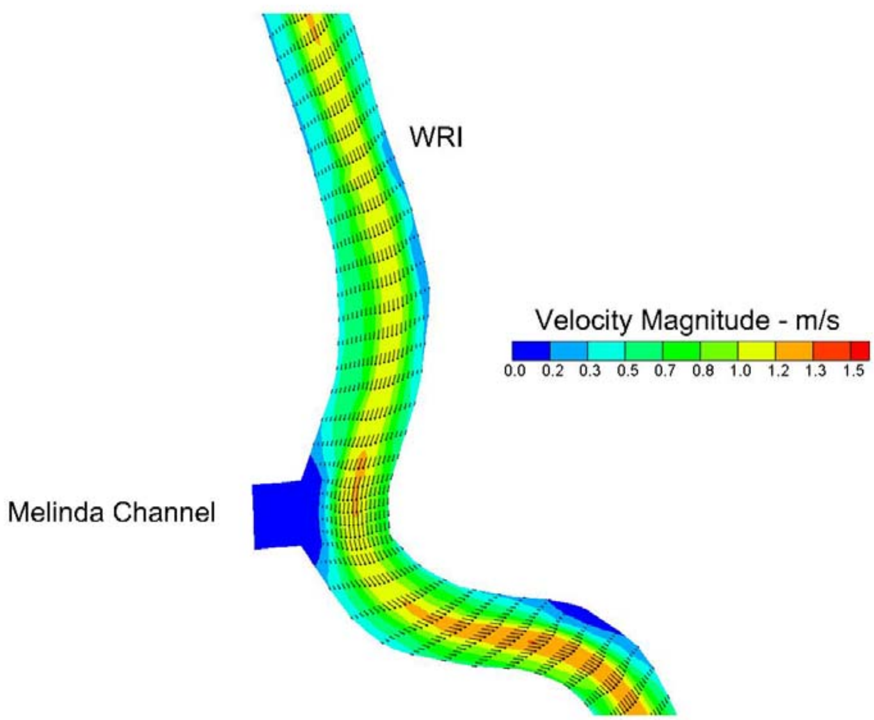


Figure 10. Velocity magnitude and direction for model validation simulation

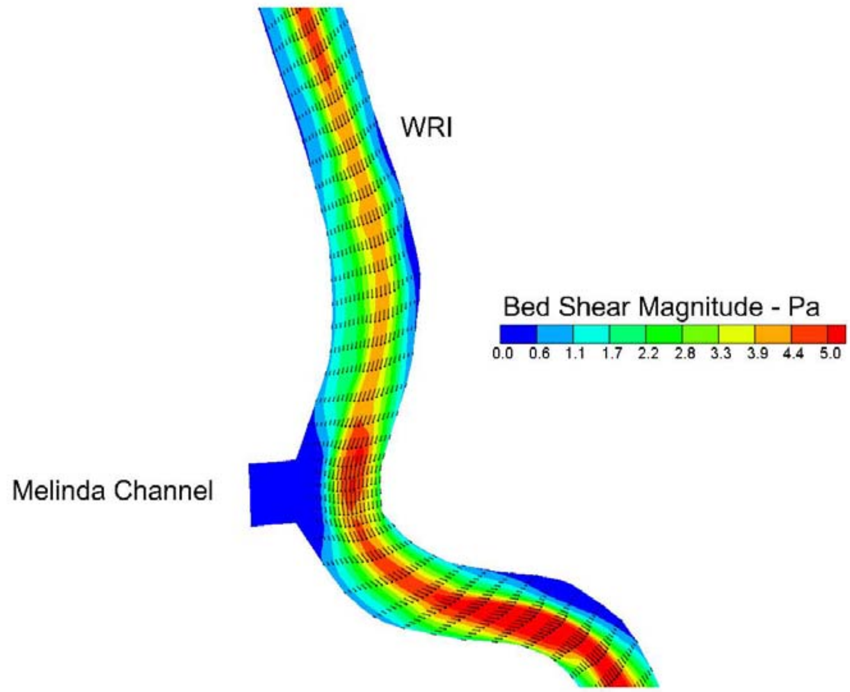


Figure 11. Bed shear stress for the model validation simulation

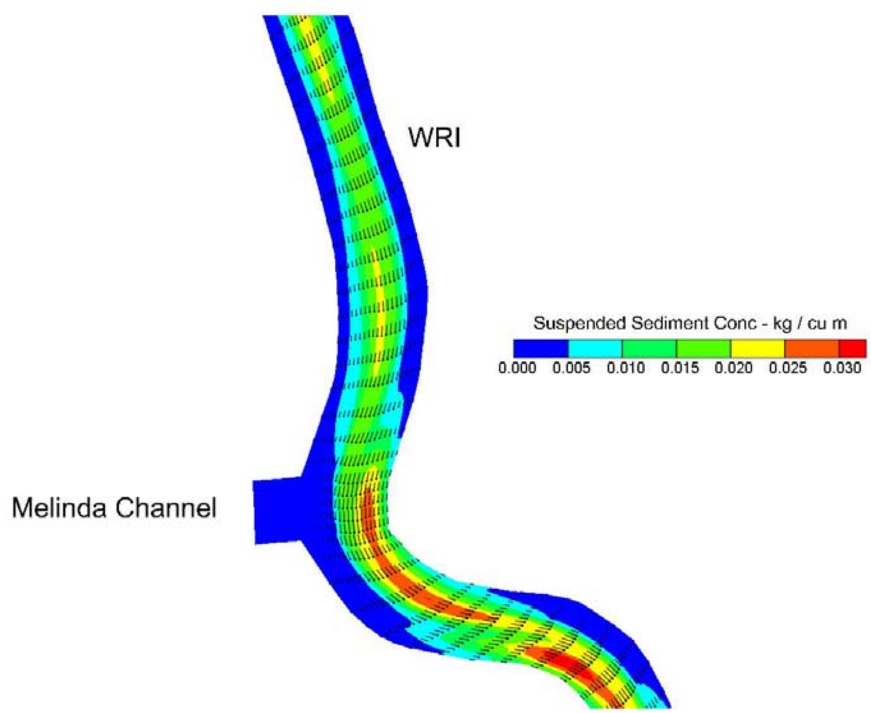


Figure 12. Suspended sediment concentration for the model validation simulation

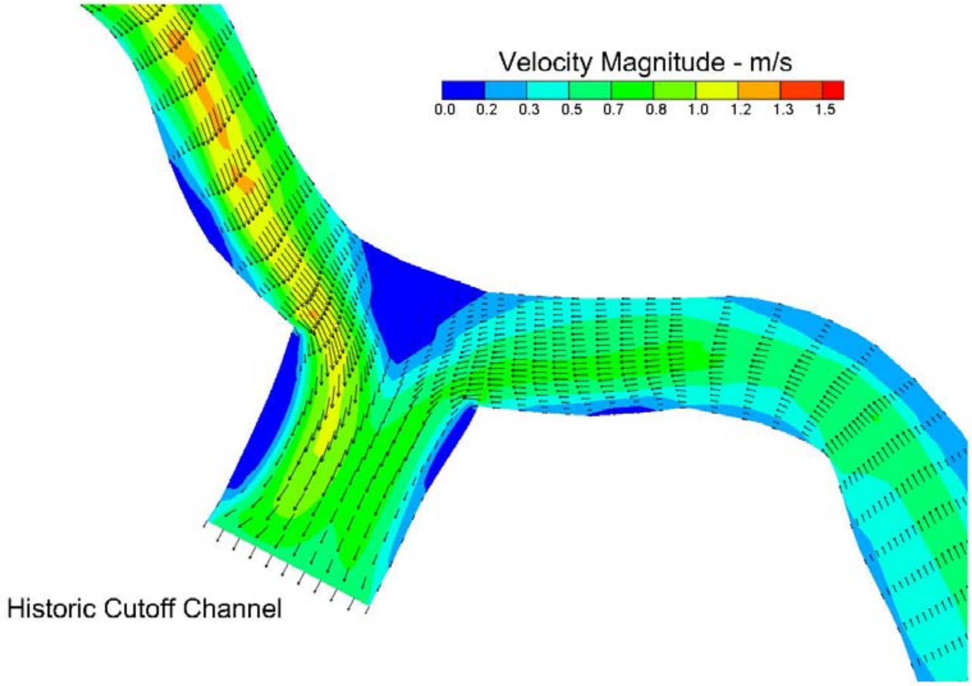


Figure 13. Peak velocity magnitude in the Historic cutoff channel for the 1997 passive weir simulation

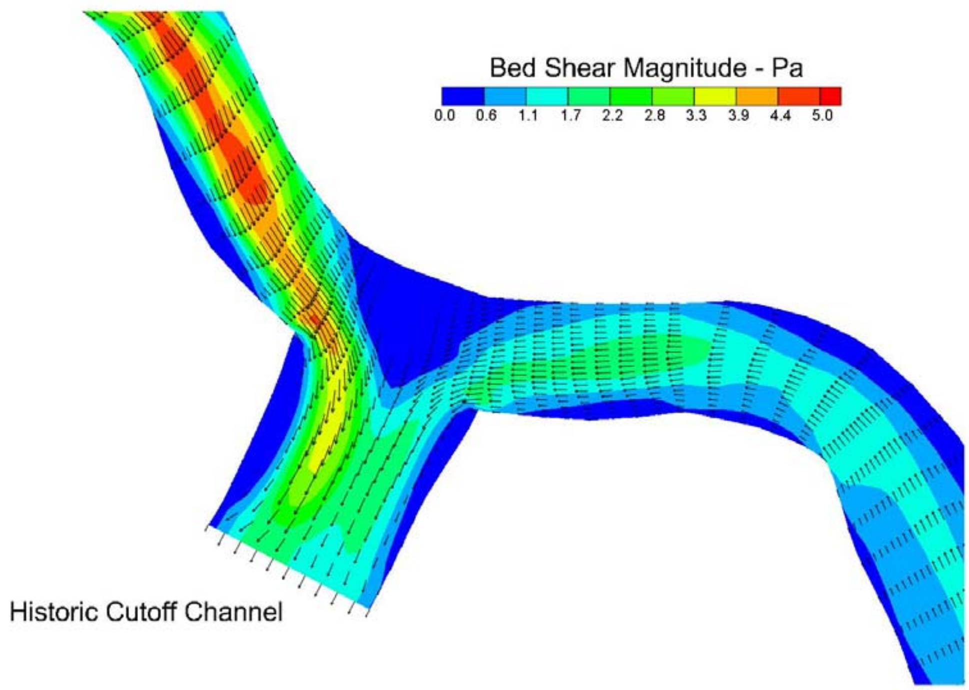


Figure 14. Peak bed shear stress in the Historic cutoff channel for the 1997 passive weir simulation

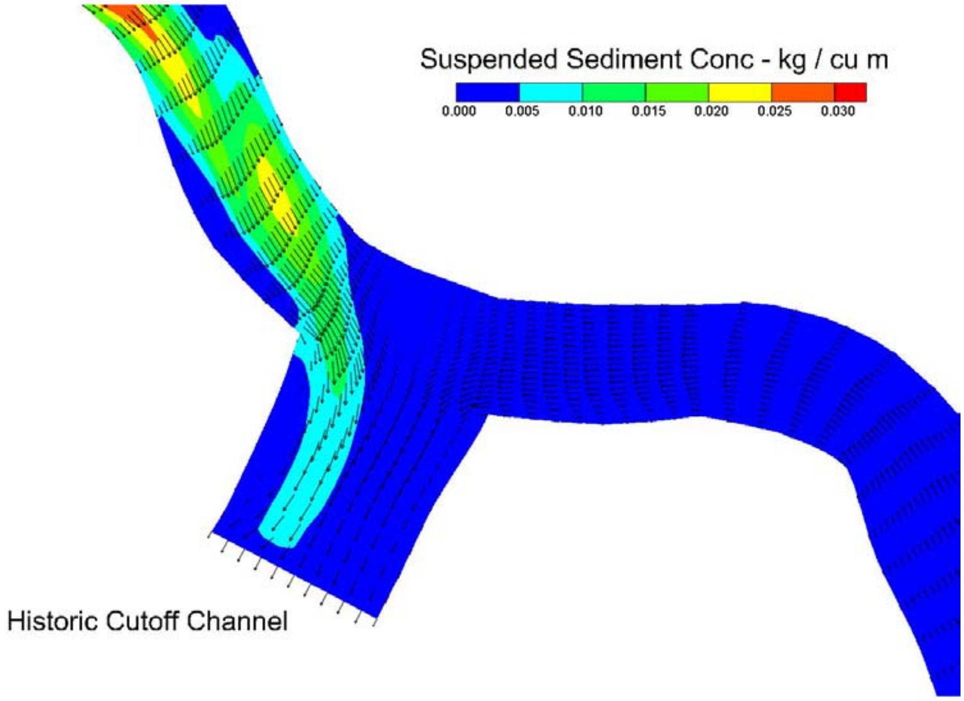


Figure 15. Peak suspended sediment concentration in the Historic cutoff channel for the 1997 passive weir simulation

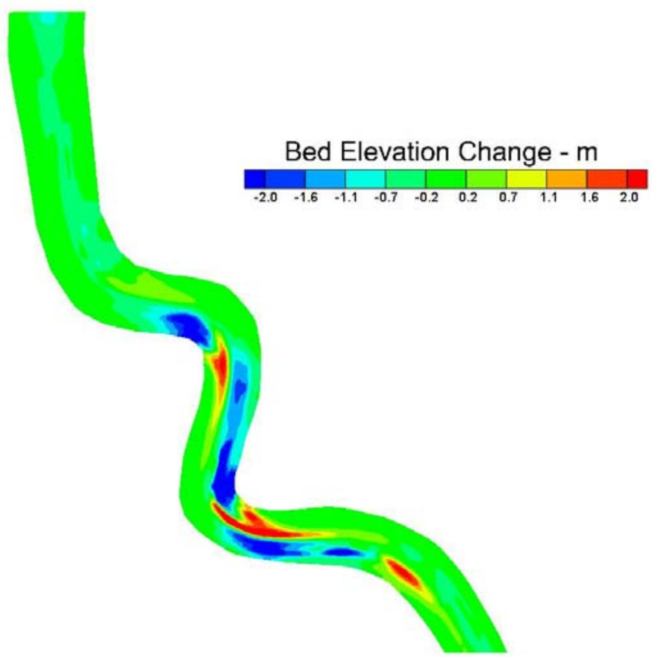


Figure 16. Bed change for RM 8.5 – 12 for the 1997 passive weir simulation

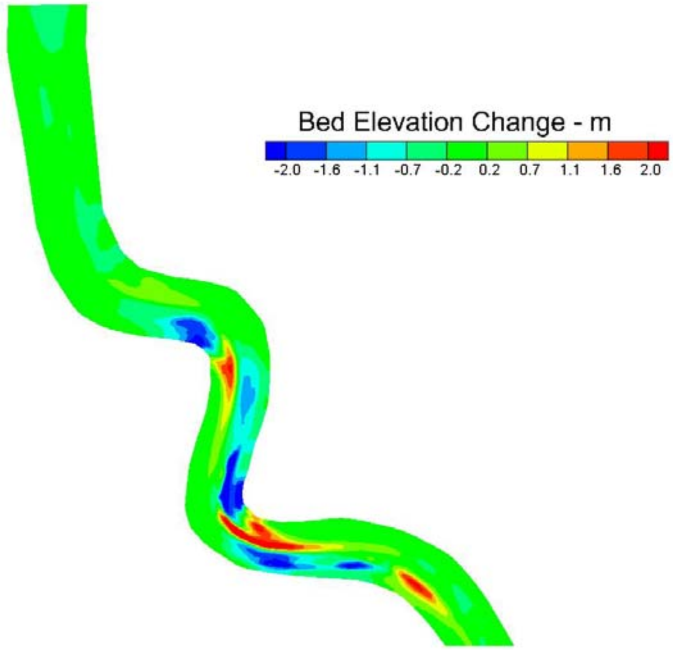


Figure 17. Bed change for RM 8.5-12 for the 1997 existing condition simulation

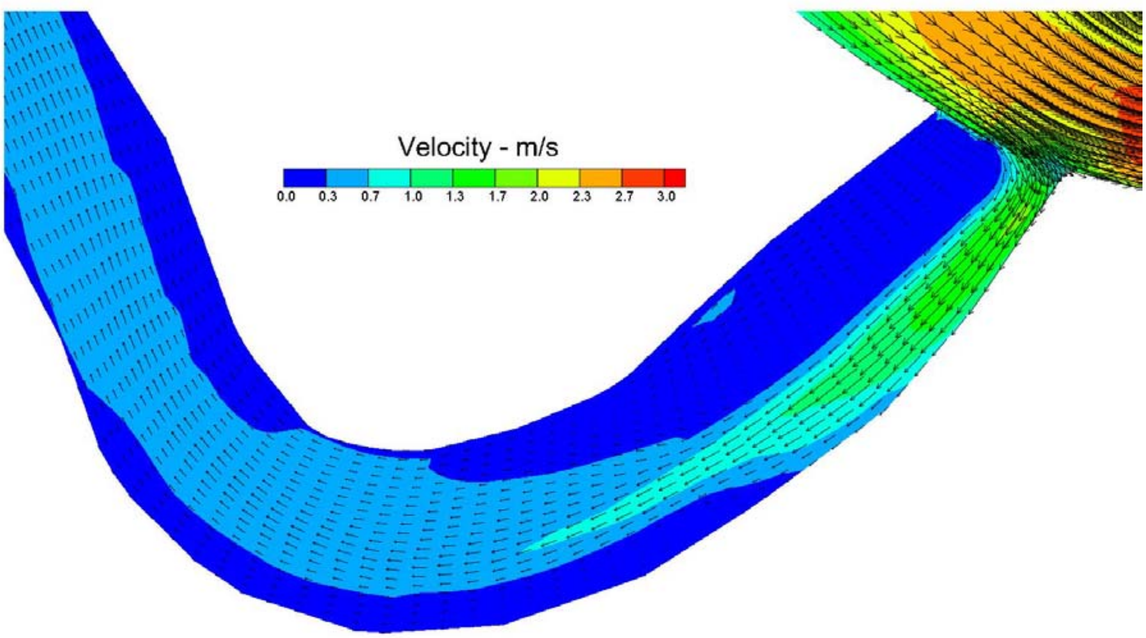


Figure 18. Velocity contour and direction for the 1997 simulation – mouth of the White River

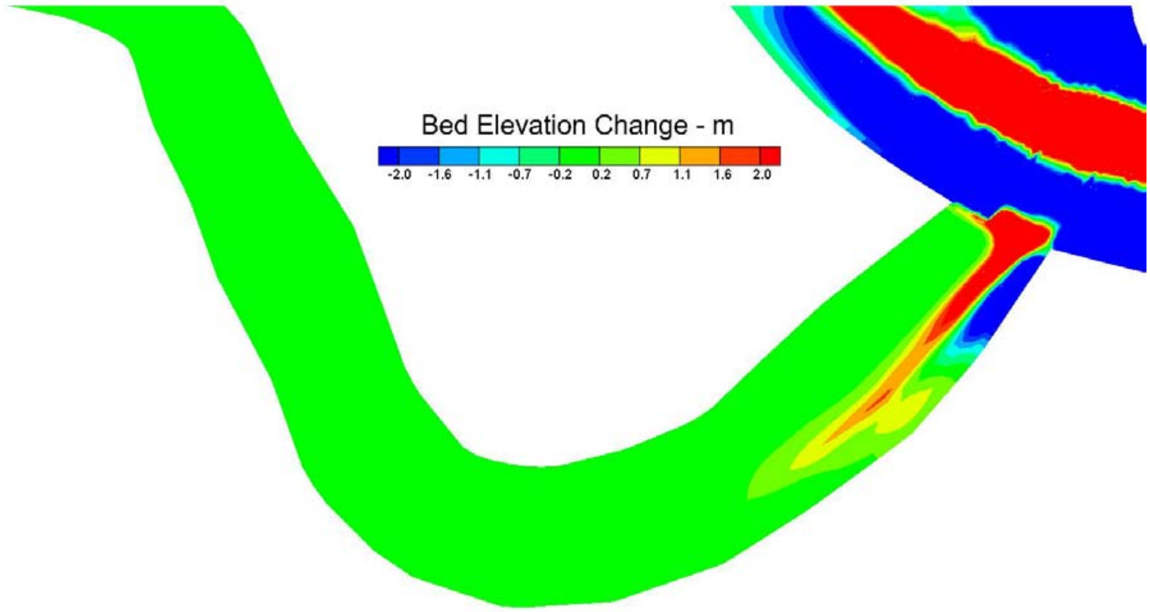


Figure 19. Bed change for RM 0-3 for the 1997 passive weir simulation

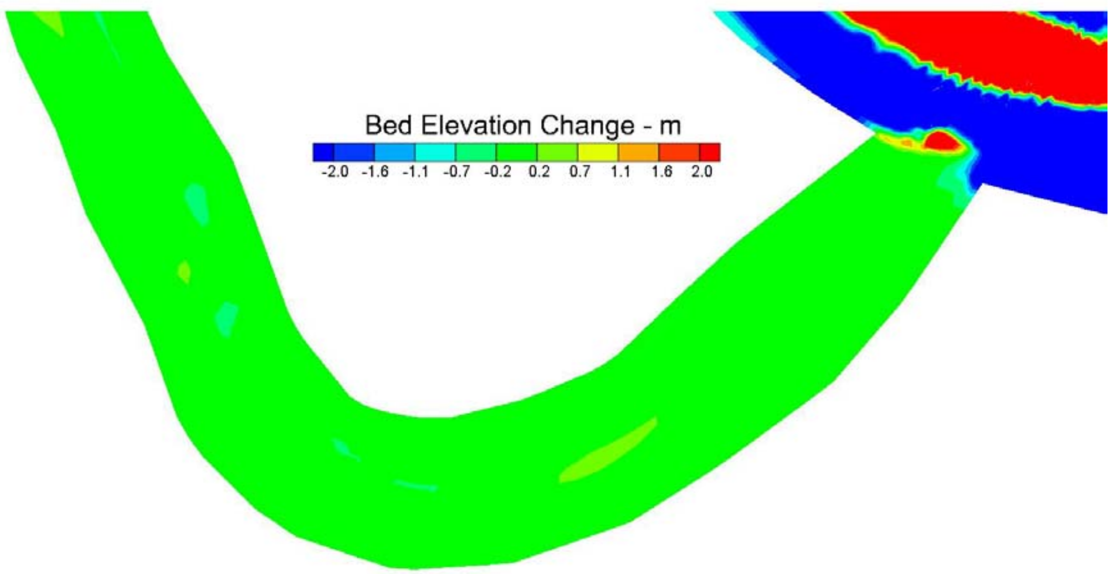


Figure 20. Bed change for RM 0-3 for the 1997 existing condition simulation

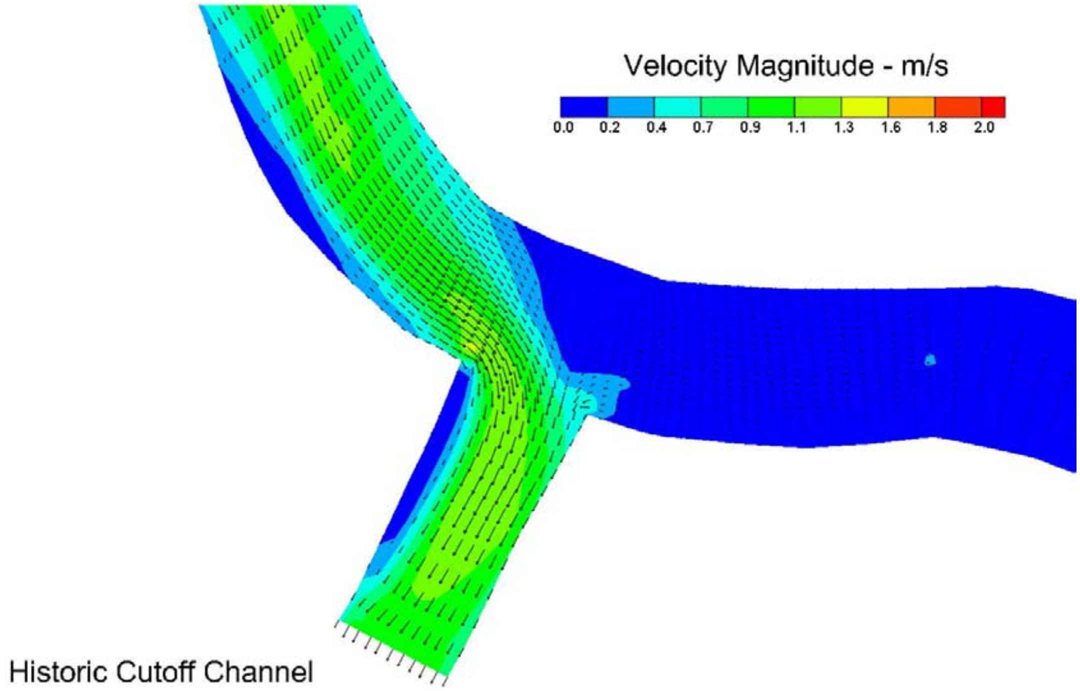


Figure 21. Peak velocity magnitude in the Historic cutoff channel for the 1997 active gated structure simulation

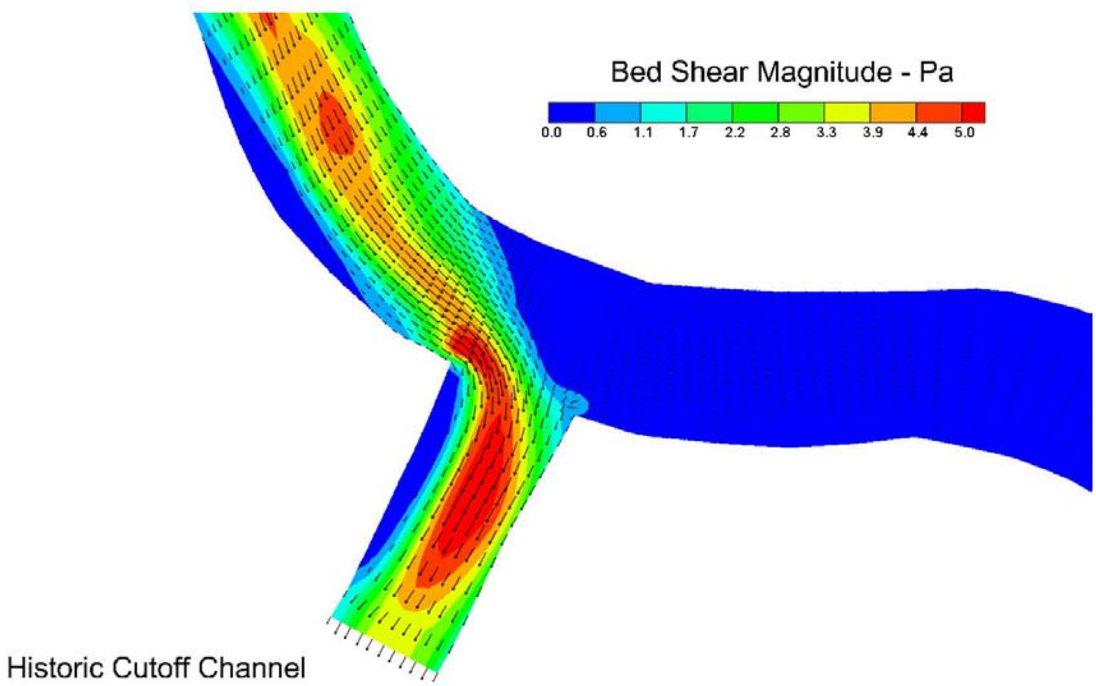


Figure 22. Peak bed shear stress in the Historic cutoff channel for the 1997 active gated structure simulation

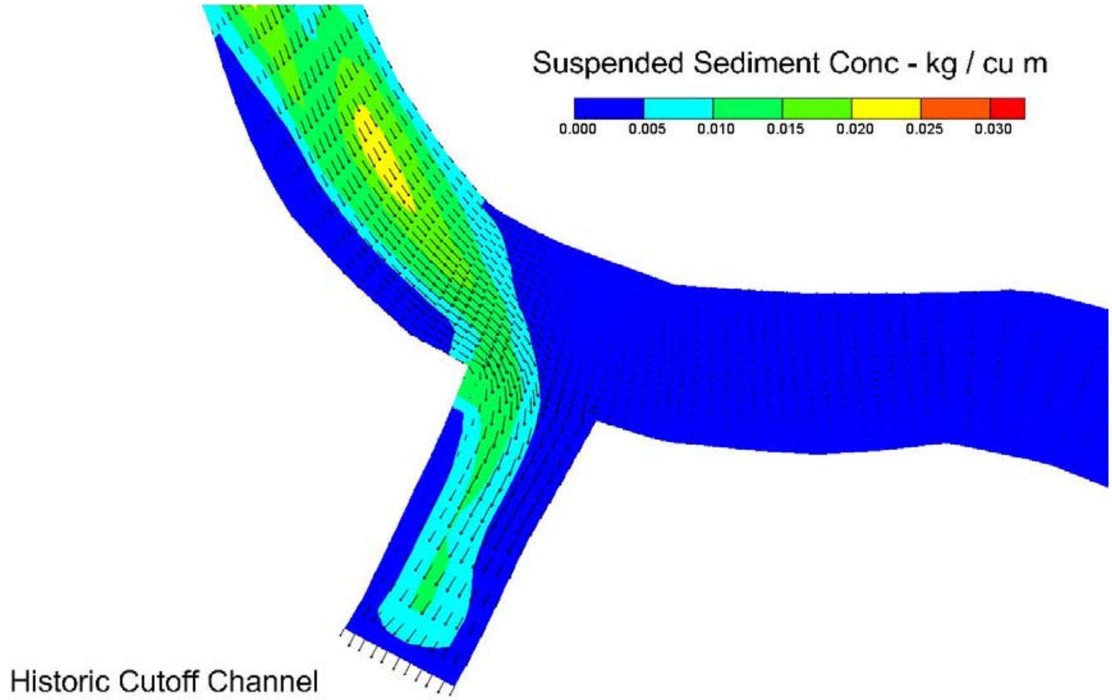


Figure 23. Peak suspended sediment concentration in the Historic cutoff channel for the 1997 active gated structure simulation

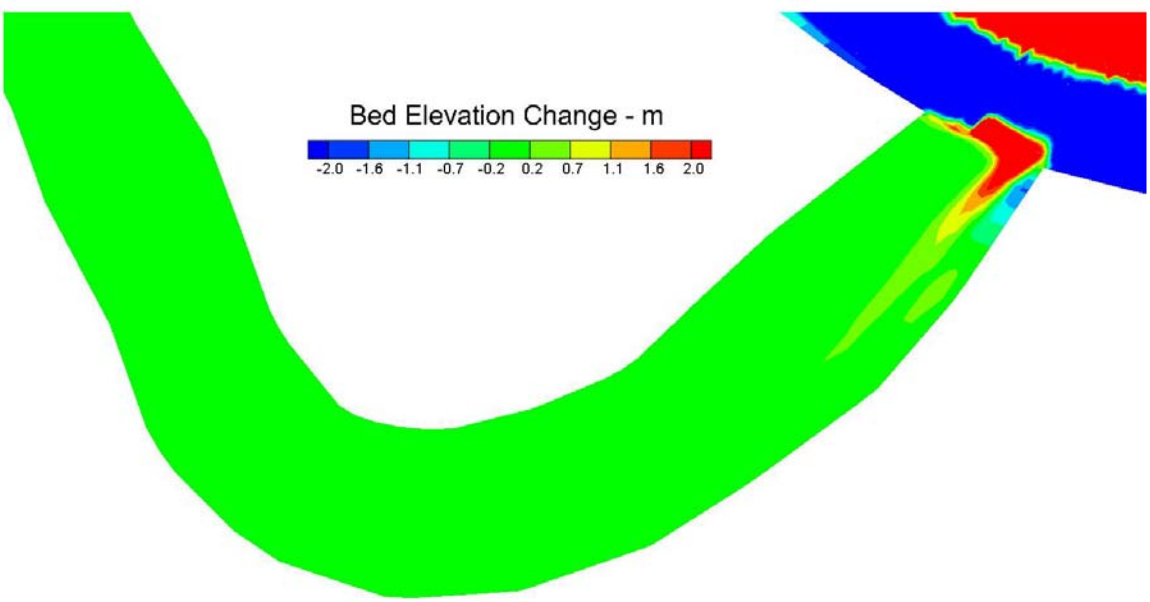


Figure 24. Bed elevation change from RM 0 - 3 for the 1997 active gated structure simulation

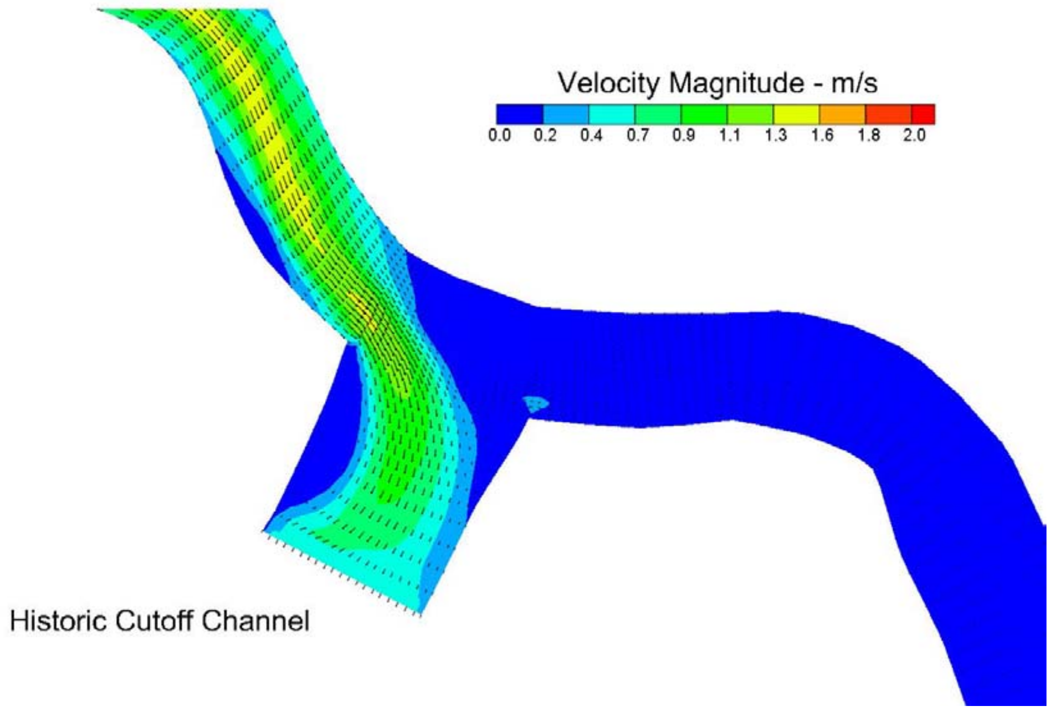


Figure 25. Peak velocity magnitude in the Historic cutoff channel for the 1989 passive weir simulation

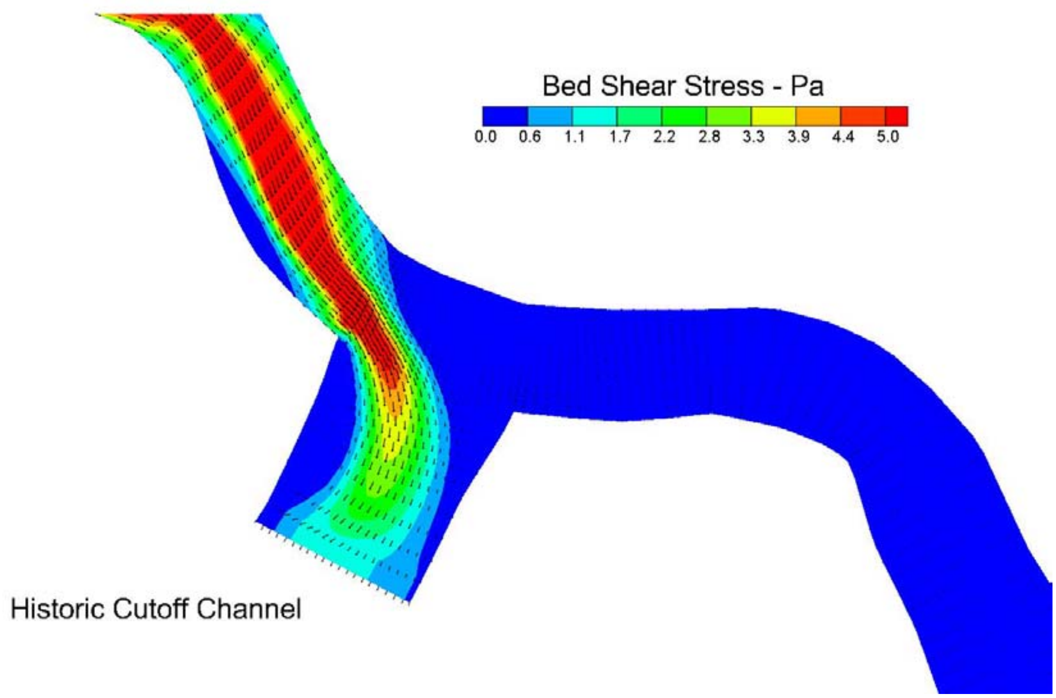


Figure 26. Peak bed shear stress in the Historic cutoff channel for the 1989 passive weir simulation

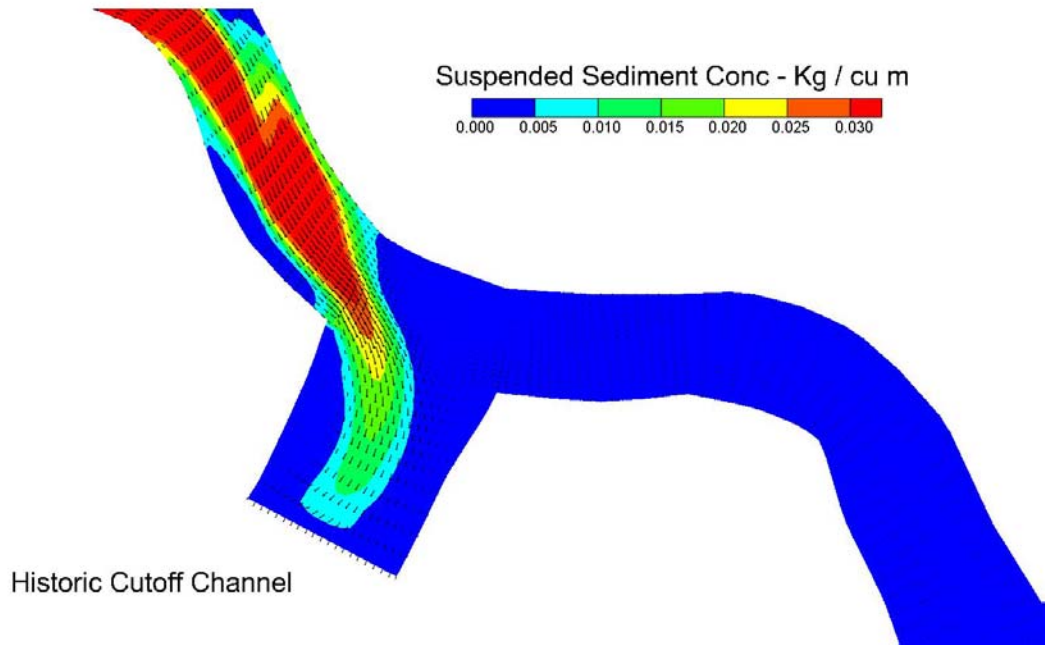


Figure 27. Peak suspended sediment concentration in the Historic cutoff channel for the 1989 passive weir simulation

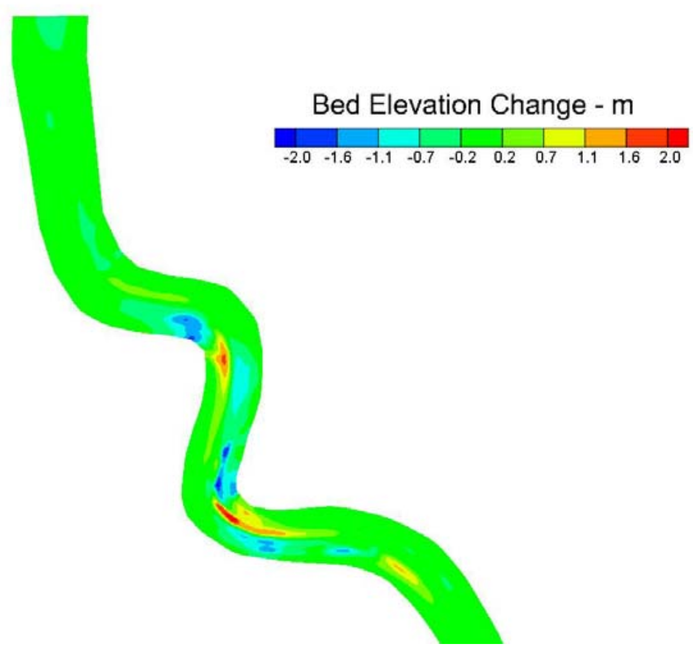


Figure 28. Bed elevation change from RM 8.5-12 for the 1989 passive weir simulation

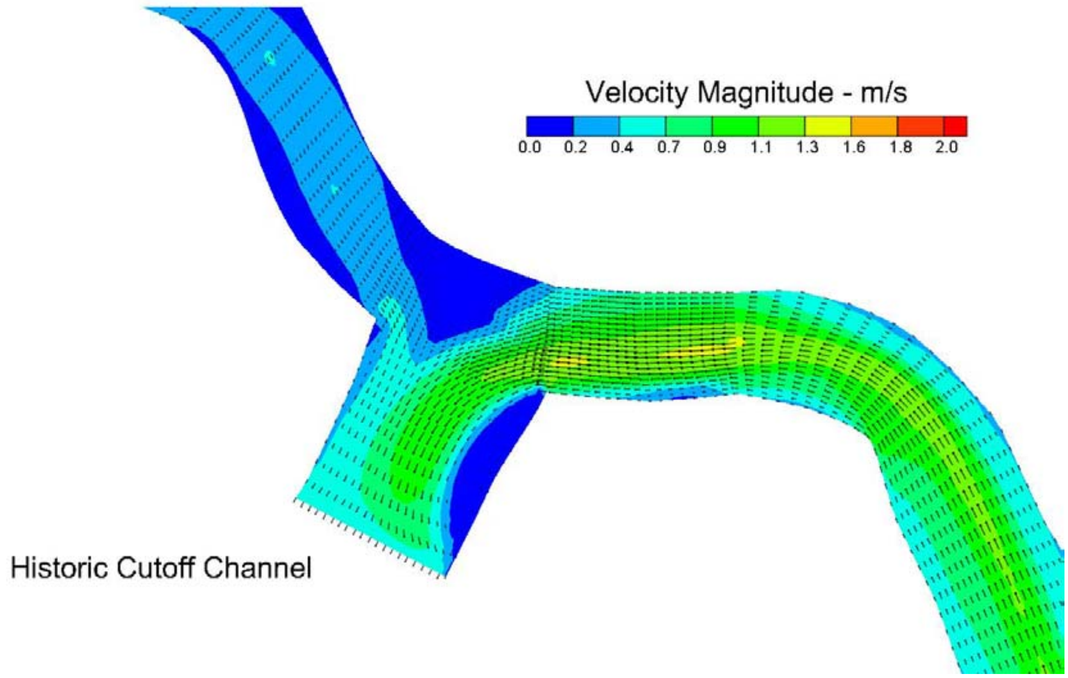


Figure 29. Peak velocity magnitude in the Historic cutoff channel for the 1984 passive weir simulation

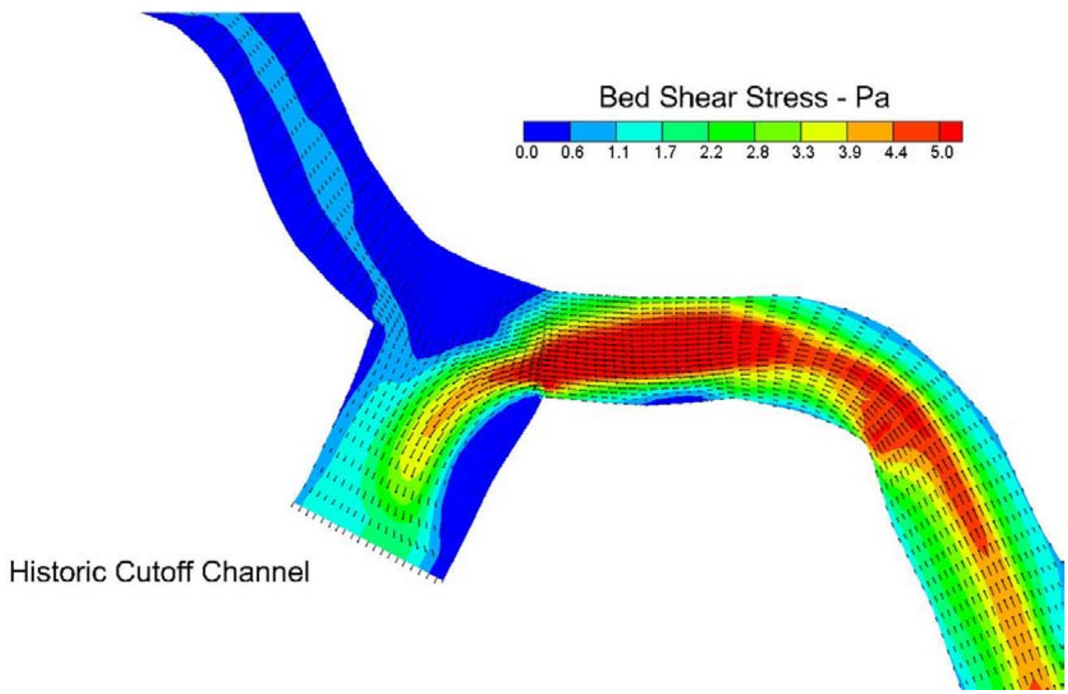


Figure 30. Peak bed shear stress in the Historic cutoff channel for the 1984 passive weir simulation

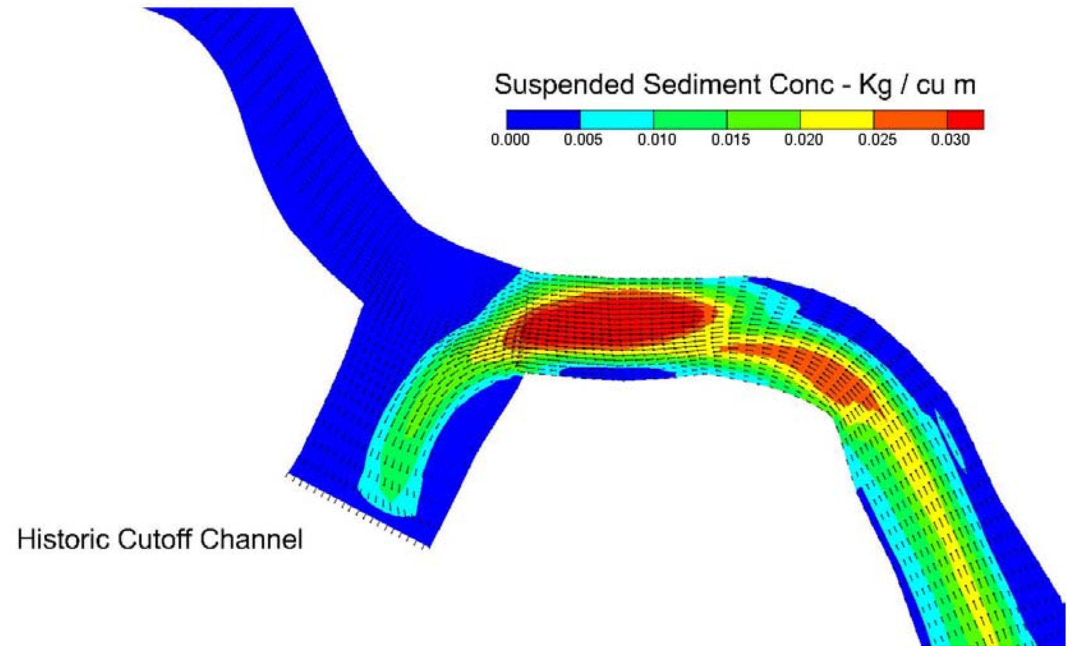


Figure 31. Peak suspended sediment concentration in the Historic cutoff channel for the 1984 passive weir simulation

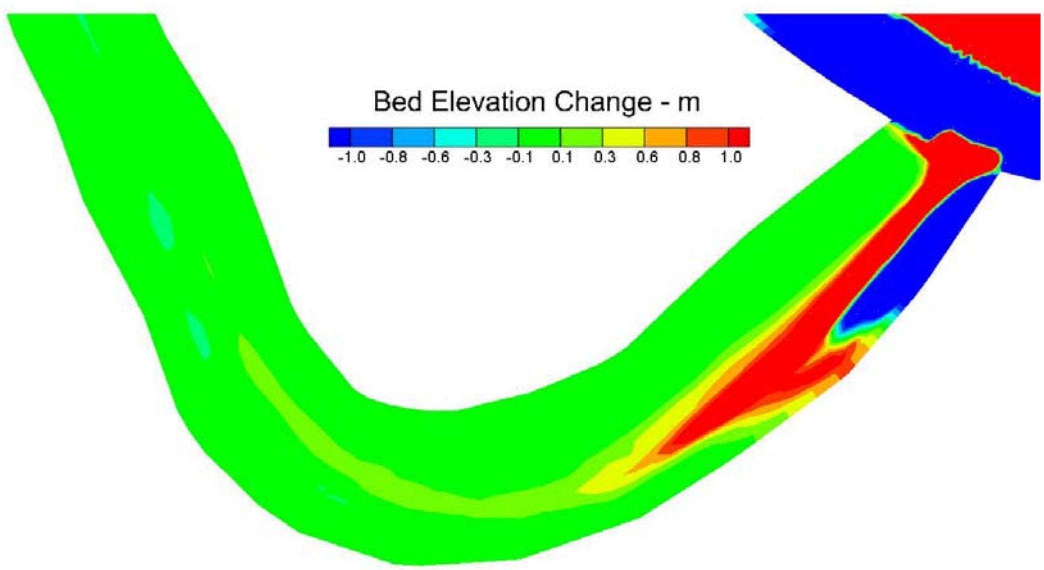


Figure 32. Bed elevation change from RM 0 – 3 for the 1984 passive weir simulation

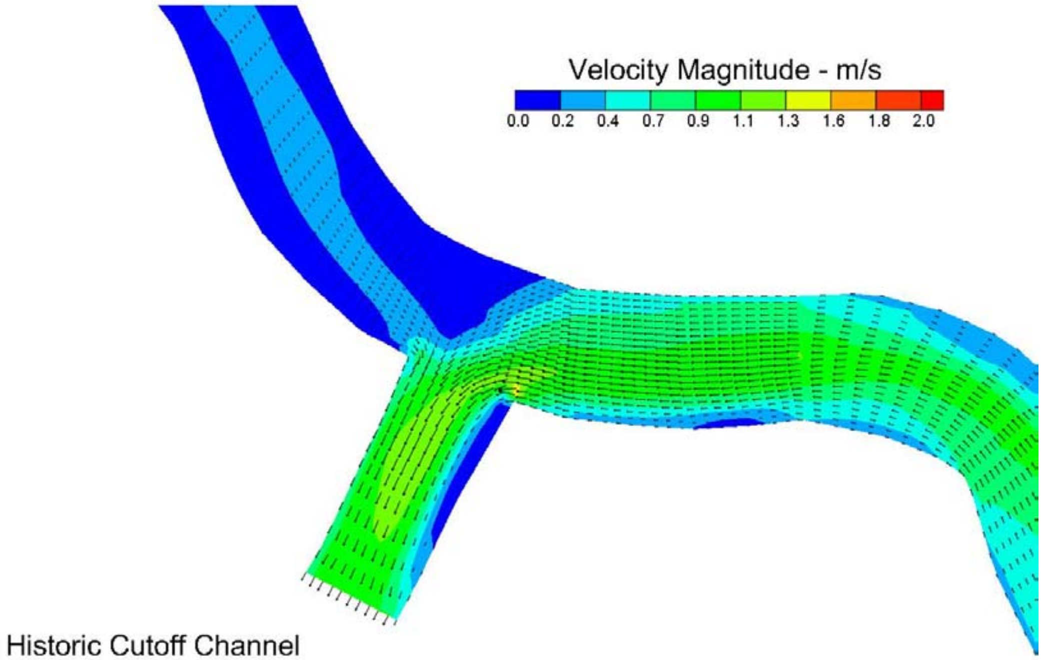


Figure 33. Peak velocity magnitude in the Historic cutoff channel for the 1984 active gated structure simulation

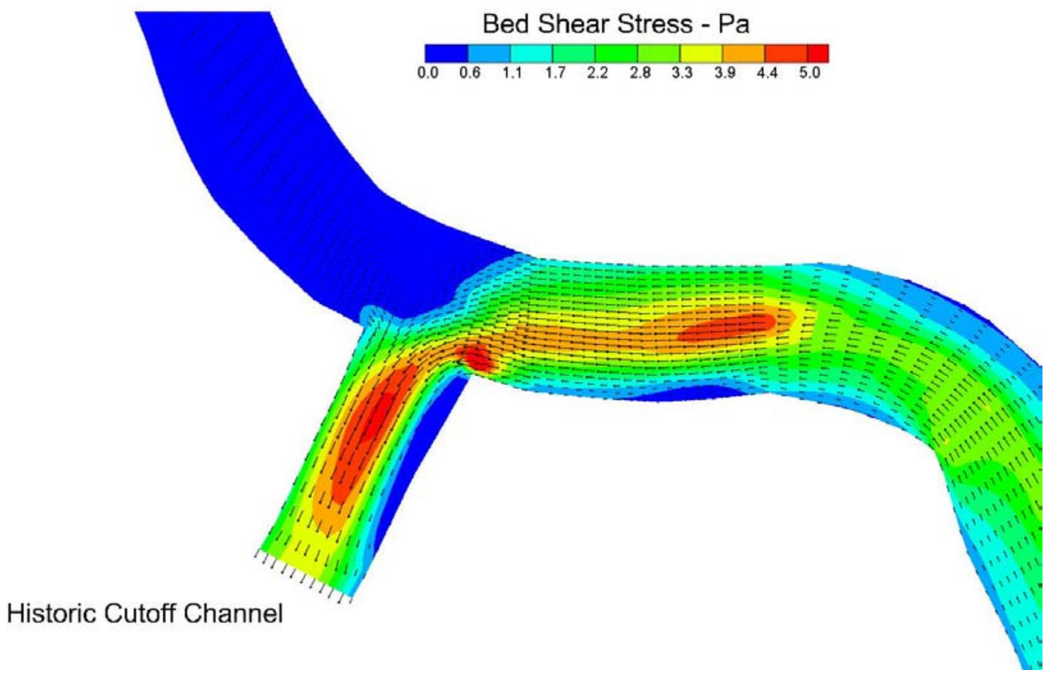


Figure 34. Peak bed shear stress in the Historic cutoff channel for the 1984 active gated structure simulation

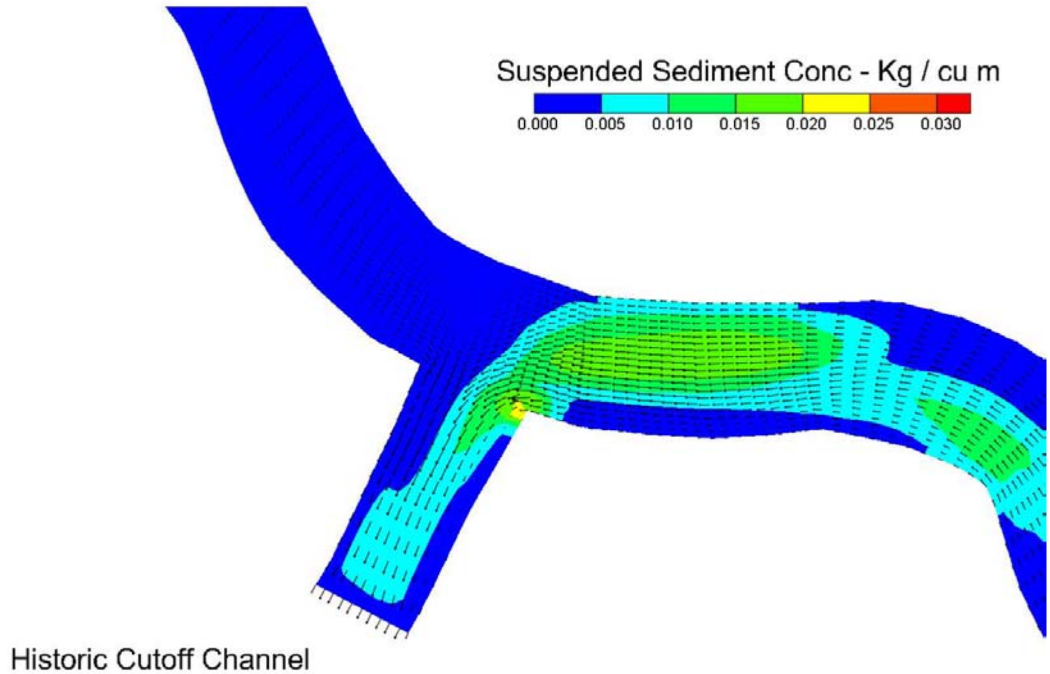


Figure 35. Peak suspended sediment concentration in the Historic cutoff channel for the 1984 active gated structure simulation

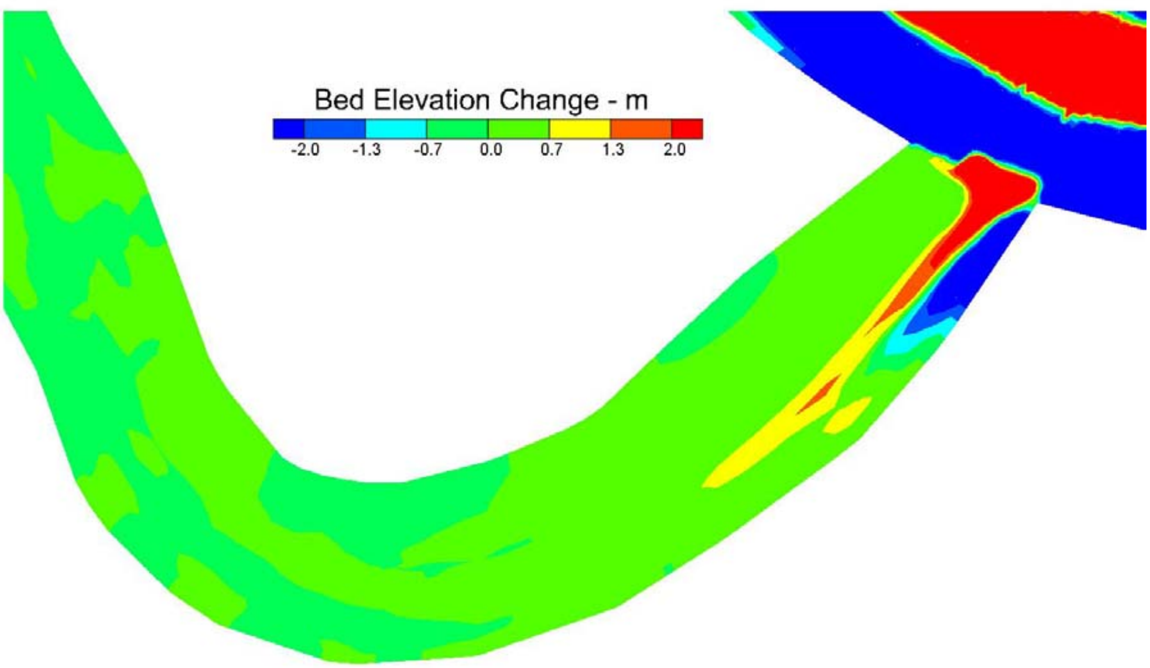


Figure 36. Bed elevation change from RM 0 - 3 for the 1984 active gated structure simulation

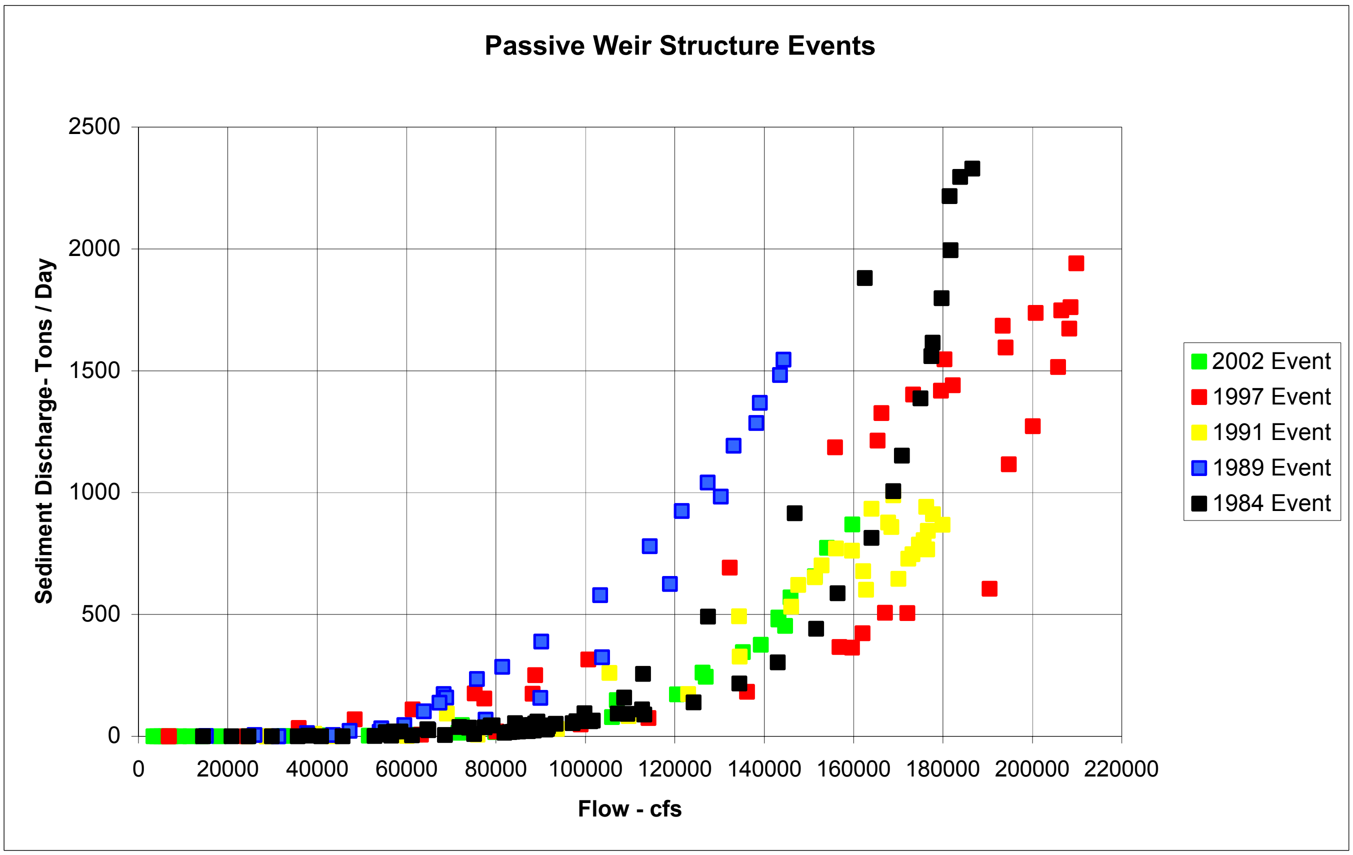


Figure 37. Sediment discharge for the passive weir flow simulations

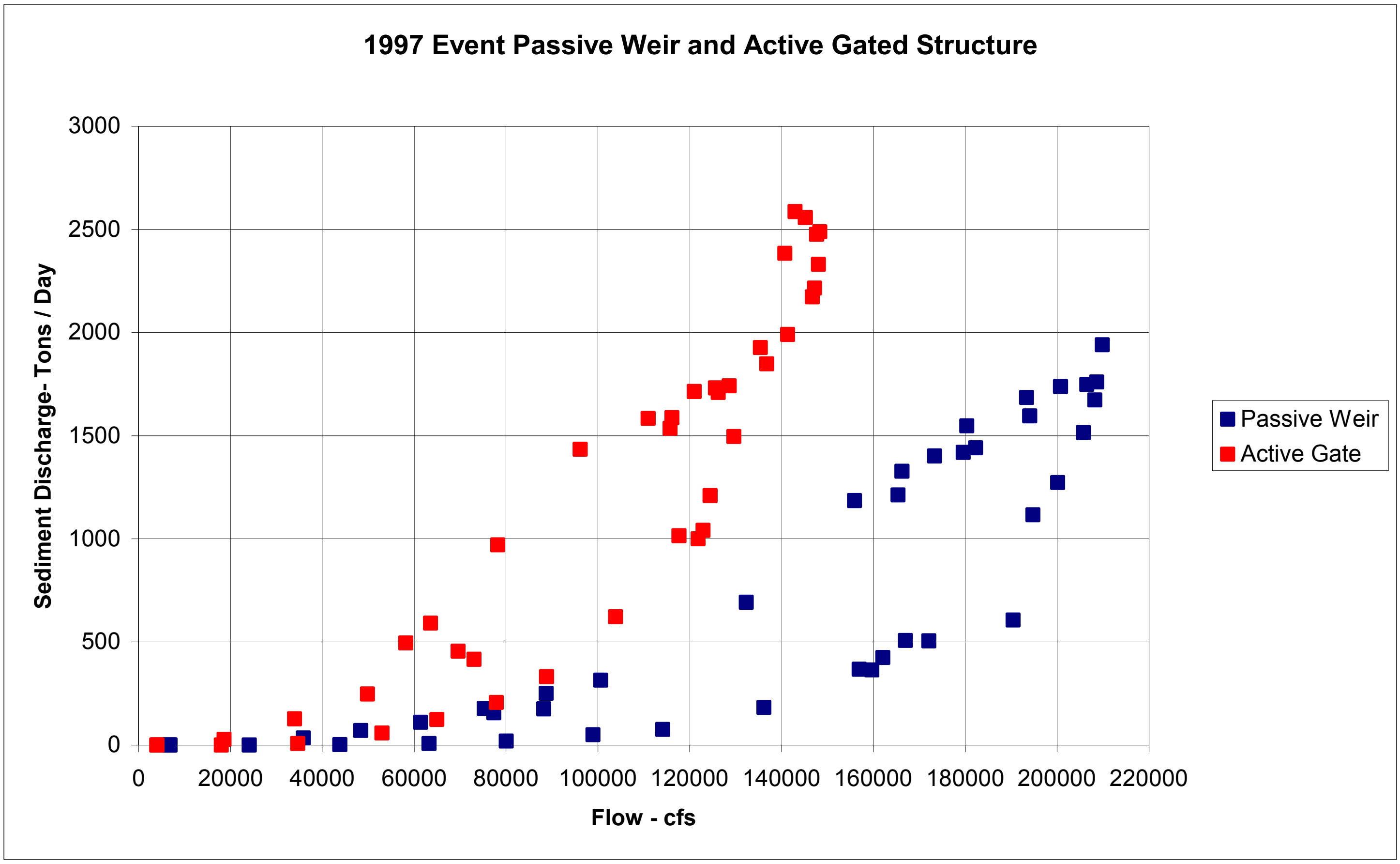


Figure 38. Sediment discharge comparison – 1997 active and passive simulations

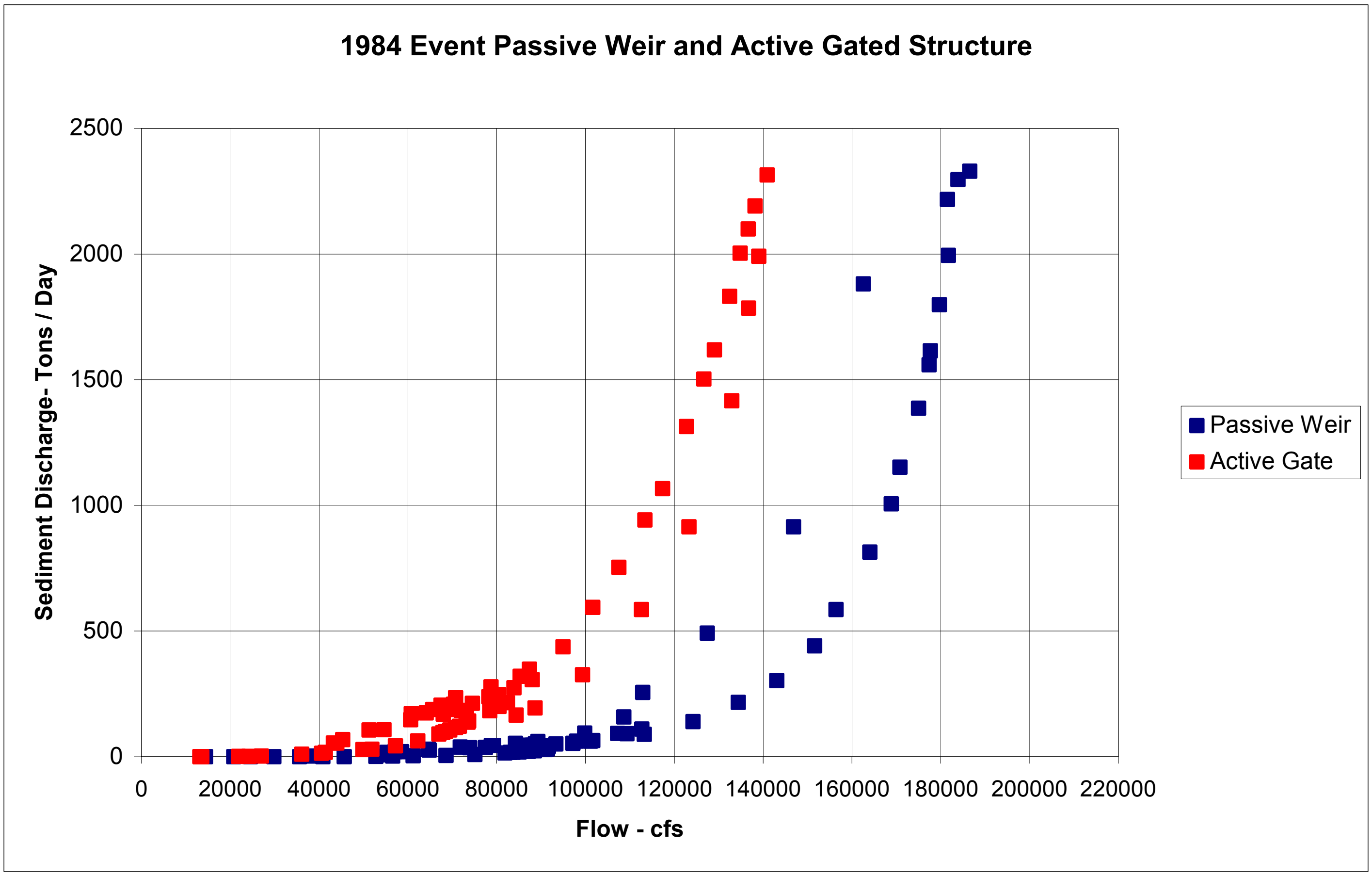


Figure 39. Sediment discharge comparison – 1984 active and passive simulations

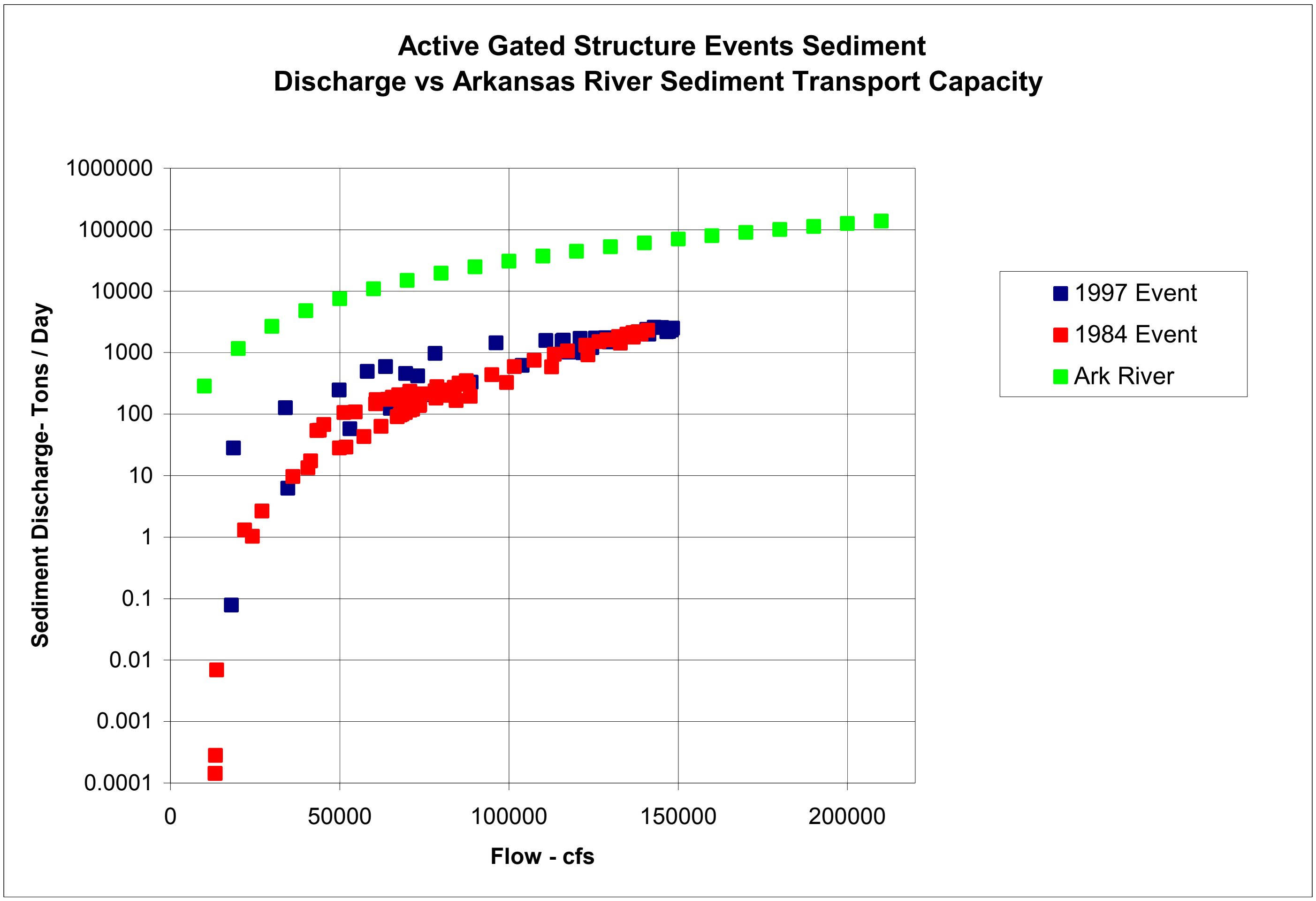


Figure 40. Comparison of active gated structure simulation results with estimated Arkansas River transport capacity

ATTACHMENT F Owens Lake Outlet Structure Sizing

Sizing Outlet Structure for Owens Lake connection to the White River

Conclusion

The final structure will either be a 6X30 CON/SPAN or a 5X30 box culvert. Hydraulically, all alternatives performed similarly, but the 30 foot wide opening would require less maintenance in the form of debris removal. Also, in the event that the opening is partially blocked, the wider structures will still drain Owens Lake at the same rate insuring no flood damage to the surrounding bottomland hardwoods. The 30 foot span is the recommended alternative, taking cost of maintenance of debris removal and the risk reduction of damaging the bottomland hardwoods into consideration. See Figure 1 for structures.

Background

Currently, Owens Lake connects to the White River at elevation 145 FT over Owens Weir and connects to the Arkansas River at elevation 140 FT over the Melinda Weir. These current weir elevations give the Arkansas River greater influence on Owens Lake hydrology then the White River. The Three Rivers selected plan includes the elimination of the Melinda Weir and the addition of a containment structure at elevation of 157 FT, CS157, just south of the current Melinda Weir location. This will shift the hydrological influence to the White River and, assuming no leakage through Owens Weir or the new CS157, will raise Owens Lake maximum elevation from 140 FT to 145 FT. The additional 5 feet in lake elevation has the potential to flood over an additional 100 acres of bottomland hardwood forest.

Method

Annual exceedance duration, growing season (15 March to 15 November) exceedance duration, and eleven flood event specific exceedance duration analyses were completed to adequately size an outlet structure through Owens Weir.

To identify areas that will be affected the most by each alternative, U.S. Fish and Wildlife, in cooperation with Arkansas Game and Fish, and Arkansas Natural Heritage Commission, requested the percent time inundated grids be changed into grids that identify areas that will experience an average of seven days or more inundation. Five days or more inundation was also identified. Figure 4 identifies areas surrounding Owens Lake that could potentially experience an excess of five to seven or more days of inundation if an outlet structure at elevation 140ft is not incorporated into Owens Weir. These locations were further analyzed using an event specific exceedance duration for several Owens Weir outlet configurations.

Event specific exceedance duration, for flood events exceeding elevation 145 feet, instead of period of record exceedance duration was used to size the outlet structure for Owens Lake. A bottomland hardwood ecosystem is damaged as a result of consecutive days of inundation rather than intermittent flooding with periods of drying. Due to its probabilistic nature, a period of record exceedance duration would not provide sufficient information required to determine if the outlet structure was adequately sized to maximize floodwater drainage and subsequently prevent damaging periods of inundation.

Elevation hydrographs of eleven flood events recorded at six locations in and around Owens Lake were used to compare existing conditions to the conditions that would exist with the proposed outlet structures in place. See Figure 2 and Figure 3 for locations. These locations were chosen because they would be the most affected with the addition of CS157. See Figure 4 for areas that will become five to seven or more days wetter. The lowest ground elevation recorded in the LiDAR data at those points are listed next to the locations. See Table 1 for location name and corresponding ground elevations.

Alternatives include existing conditions, four Con/span structures (see Figure 1) and one weir with inverts at elevation 140 feet and then again at 139.5 feet. See Table 2 for description of alternatives.

Event specific elevation hydrographs, annual duration exceedance and growing season duration exceedance Owens Lake, EL 137.22 feet, are presented in this report since this is the lowest elevation in the area of interest and therefore would drain last. Annual duration exceedance and growing season duration exceedance at Owens BLH, Owens BLH2, and Melinda AR are also provided.

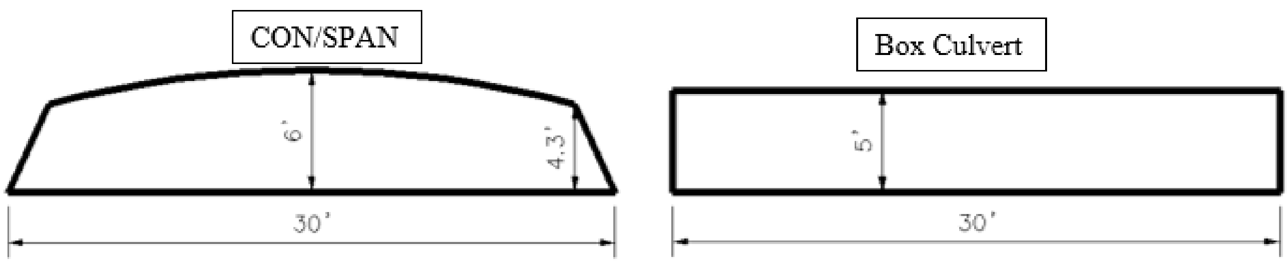


Figure 1: CON/SPAN and Box Culvert

Table 1: Elevation Hydrograph Locations

Elevation Hydrograph Location	LiDAR ground Elevation (feet)
****Owens Lake**	137.22
Owens SE Leg	138.83
Owens BLH SE	141.1
Owens BLH 1	141.35
*Owens BLH	141.46
*Owens BLH2	142.12
*Melinda AR	88.2
*Computed annual and growing season exceedance duration locations	
** Computed event specific, annual and growing season exceedance duration location	

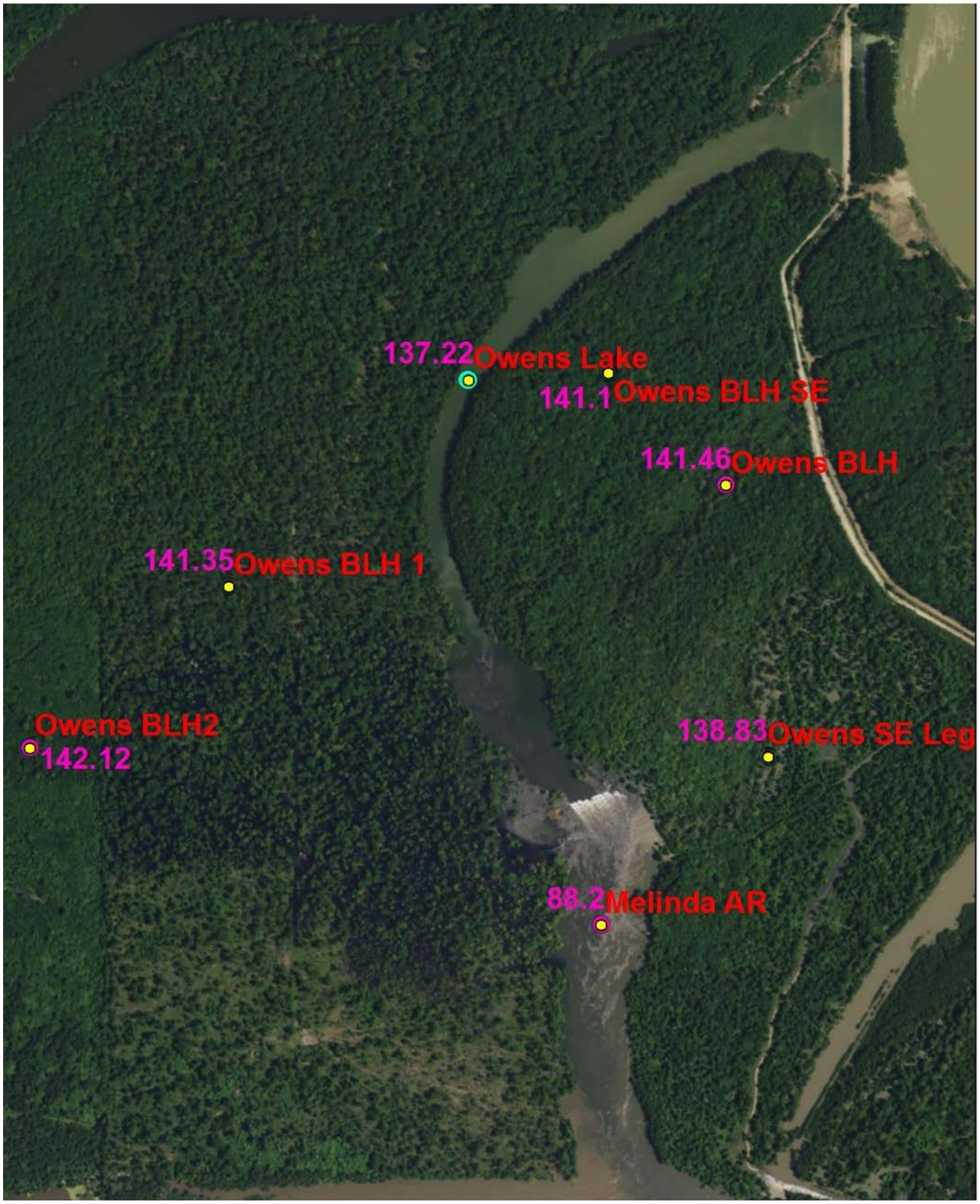


Figure 2: Elevation Hydrograph Locations: Aerial

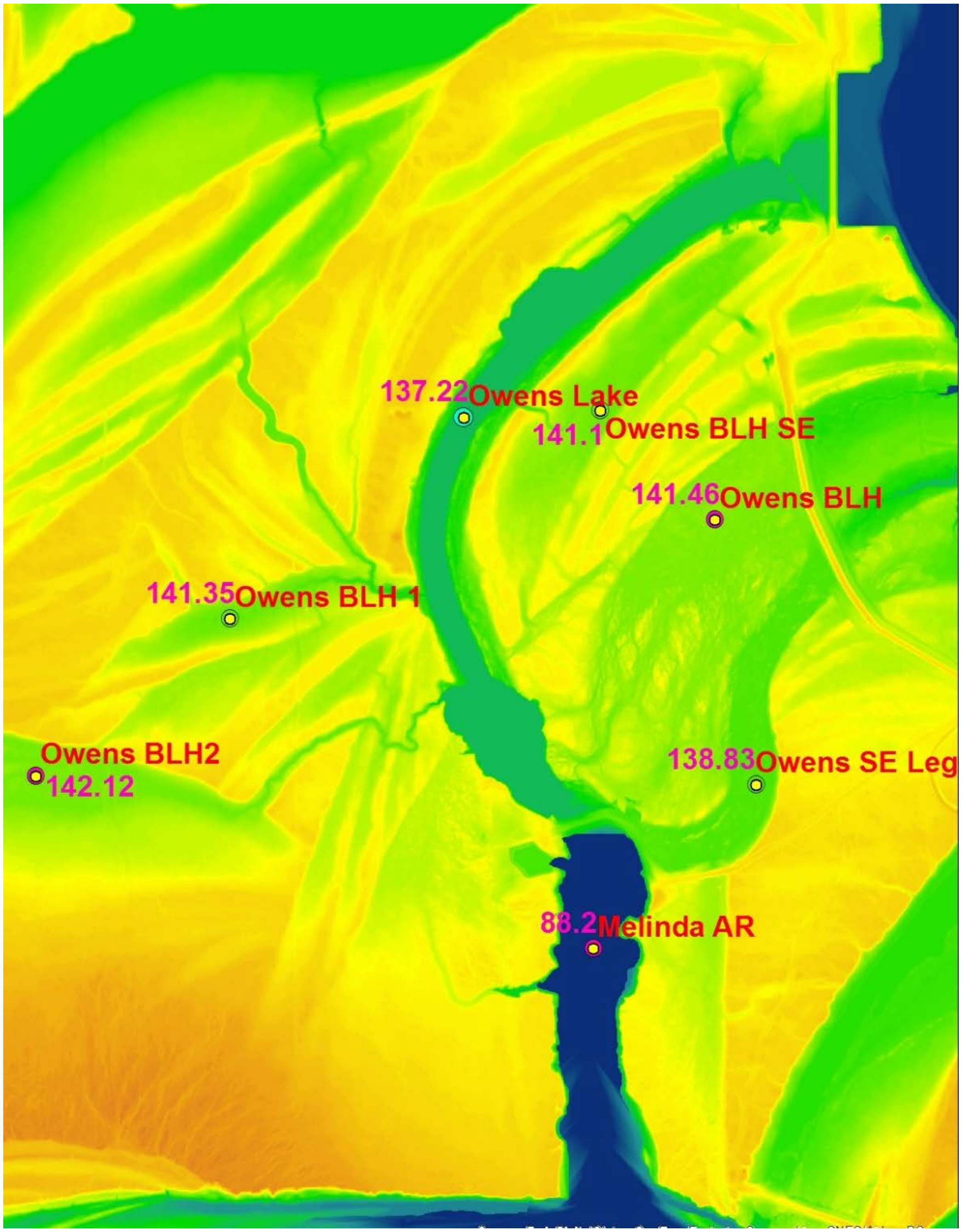


Figure 3: Elevation Hydrograph Locations: Arc Grid

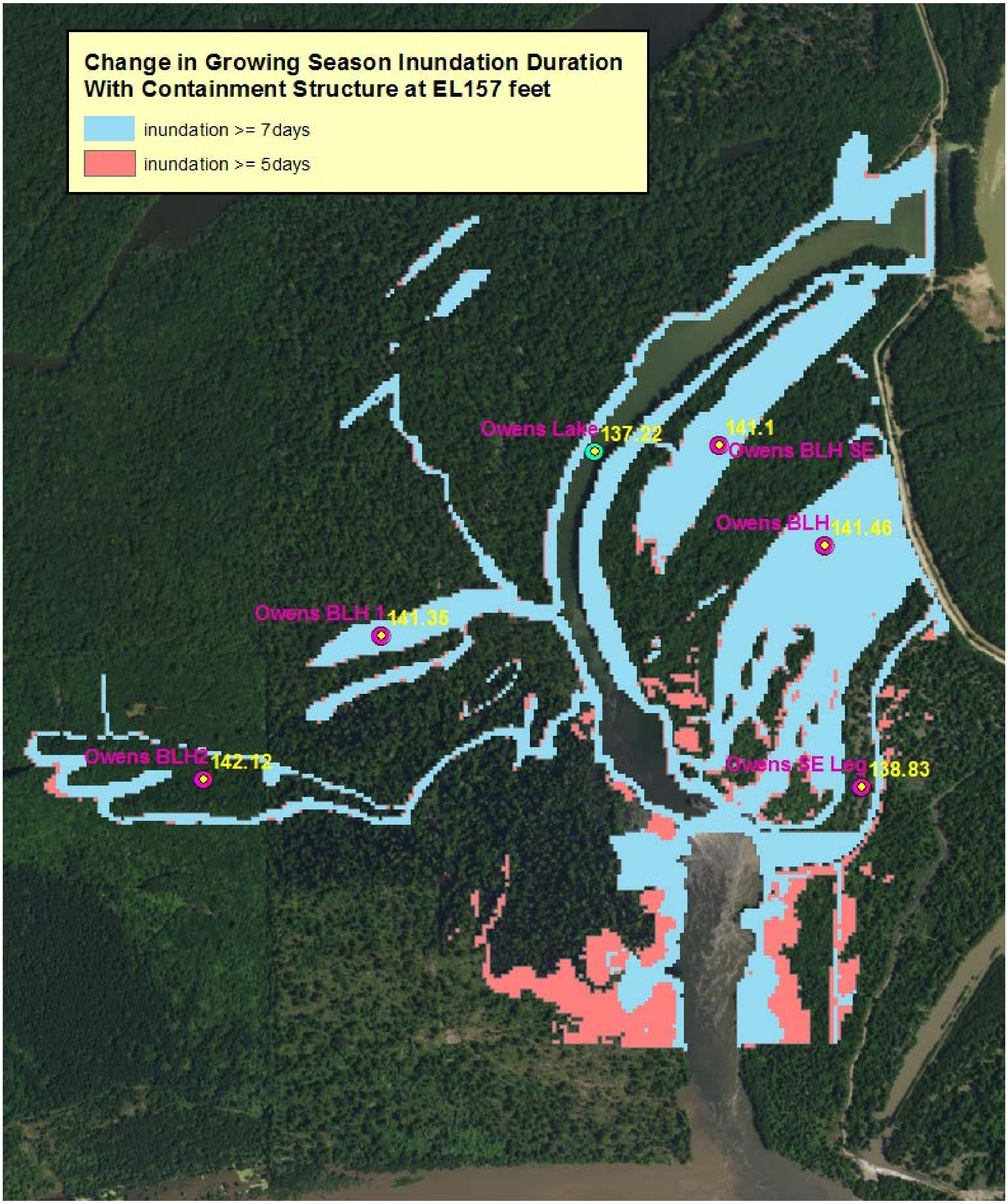


Figure 4: Growing Season Hydrologic Changes with CS157 and Owens Weir at Elevation 145 feet

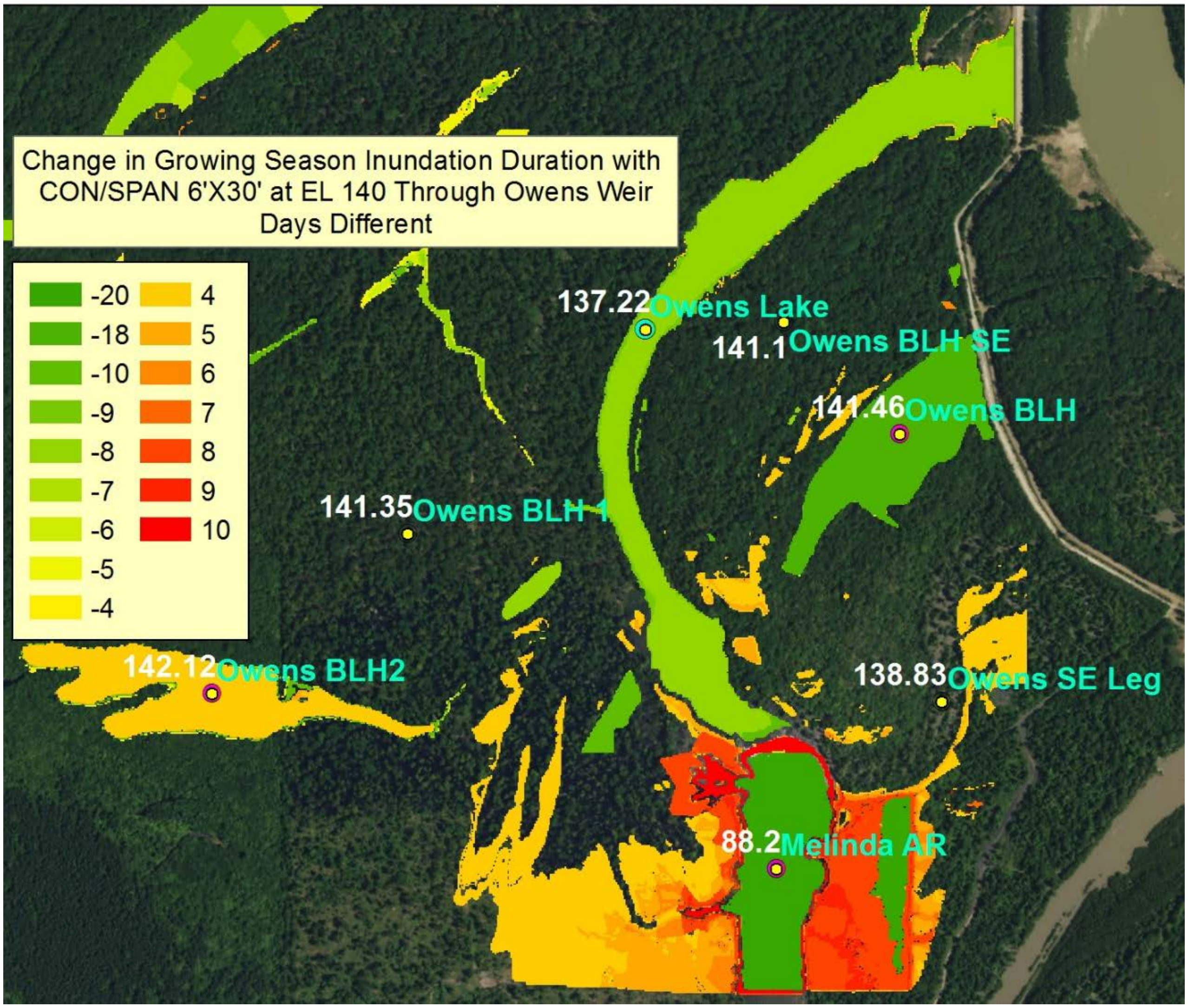


Figure 5: Growing Season Hydrologic Changes with CS157 and Owens Weir at Elevation 145 feet with CON/SPAN 6'X30' at Elevation 140 feet

Table 2: Owens Alternatives

Alternative	Invert Elevation (feet)	Span (feet)	Height (feet)	Description
Existing	Melinda 140 Owens 145	Melinda ~530 Owens ~960	N/A	Owens Weir crest elevation at 145 feet and Melinda Weir crest elevation at 140 feet
Weir EL 140ft	140	~720 *	N/A	Owens Weir crest elevation 140 feet
C6X30 140ft	140	30	6	Con/span 6 feet high and 30 feet wide with invert at EL 140 feet
C6X25 140ft	140	25	6	Con/span 6 feet high and 25 feet wide with invert at EL 140 feet
C6X20 140ft	140	20	6	Con/span 6 feet high and 20 feet wide with invert at EL 140 feet
C4X20 140ft	140	20	4	Con/span 4 feet high and 20 feet wide with invert at EL 140 feet
Weir EL 139.5ft	139.5	~720 *	N/A	Owens Weir crest elevation 139.5 feet
C6X30 139.5ft	139.5	30	6	Con/span 6 feet high and 30 feet wide with invert at EL 139.5 feet
C6X25 139.5ft	139.5	25	6	Con/span 6 feet high and 25 feet wide with invert at EL 139.5 feet
C6X20 139.5ft	139.5	20	6	Con/span or arch 6 feet high and 20 feet wide with invert at EL 139.5 feet

*Weir only 720 feet wide due to adjacent natural ground elevation limitations

Owens Lake elevation hydrographs of the alternatives were compared to existing conditions to see what changes can be expected in and around the lake due to the hydrological influence shifting from the Arkansas River to the White River. Next, each alternative was compared to *Weir EL 140ft* since this represents the most efficient drainage of Owens Lake and the surrounding bottomland hardwoods after the hydrological shift has occurred.

The number of days exceeding elevation 141 feet was chosen to represent the change in hydrology since the surrounding bottomland hardwoods are higher than elevation 141 feet. Number of days exceeding elevation 140.5 feet was also used to show the tapering effects of the receding limb of the hydrograph as the water surface elevation gets closer to the proposed Owens outlet structure invert elevation of 140 feet.

Recommended Structure Based on the Eleven Flood Event Specific Exceedance Duration

Filling and draining of Owens Lake is more dependent on the slow rise and fall of the White River rather than the structure sizes and weirs that were modeled. Subsequently, the days different between the four con/span structures compared to Owens Weir *EL 140ft* ranged from 0.6 days of more flooding and 2.2 days of less flooding at elevation 141 feet. See Table 3: Change in Days Exceedance at Elevation 141 (Alternative – Weir EL 140ft)

When all alternatives were compared to existing conditions, the days different at elevation 141 feet ranged from 11.1 days more flooding down to 6.8 days of less flooding. See Table 4 for results. See Table 5-8 and Figures 4-15 for event specific results.

Hydraulically, all alternatives performed similarly, but the 30 foot wide opening would require less maintenance in the form of debris removal. Also, in the event that the opening is partially blocked, the wider structures will still drain Owens Lake at the same rate insuring no flood damage to the surrounding bottomland hardwoods. The 30 foot span is the recommended alternative, taking cost of maintenance of debris removal and the risk reduction of damaging the bottomland hardwoods into consideration.

Average Exceedance Duration for Owens Lake

Table 3: Change in Days Exceedance at Elevation 141 (Alternative – Weir EL 140ft)

Alternative	Invert Elevation (feet)	Span (feet)	Height (feet)	Maximum Change in Days	Minimum Change in Days
C6X30 140ft	140	30	6	0.3	-1.9
C6X25 140ft	140	25	6	0.5	-2.2
C6X20 140ft	140	20	6	0.6	-1.9
C4X20 140ft	140	20	4	0.6	-1.9
Weir EL 139.5ft	139.5	~720	N/A	0.0	-0.4
C6X30 139.5ft	139.5	30	6	0.3	-0.5
C6X25 139.5ft	139.5	25	6	0.3	-0.7
C6X20 139.5ft	139.5	20	6	0.5	-1.2

Table 4: Change in Days Exceedance at Elevation 141 (Alternative – Existing)

Alternative	Invert Elevation (feet)	Span (feet)	Height (feet)	Maximum Change in Days	Minimum Change in Days
Weir EL 140ft	140	~720	N/A	10.6	-6.8
C6X30 140ft	140	30	6	10.8	-6.8
C6X25 140ft	140	25	6	11.1	-6.8
C6X20 140ft	140	20	6	9.1	-6.8
C4X20 140ft	140	20	4	9.1	-6.8
Weir EL 139.5ft	139.5	~720	N/A	10.6	-1.7
C6X30 139.5ft	139.5	30	6	10.6	-1.7
C6X25 139.5ft	139.5	25	6	10.8	-2.0
C6X20 139.5ft	139.5	20	6	11.1	-1.7

Table 5: Days Exceeding EL 141 feet

Number of Days Exceeding Elevation 141 FT for each flood hydrograph event											
Alternative	4-Feb-02	5-Apr-02	29-May-02	22-May-09	18-Oct-09	9-Nov-09	9-Feb-10	6-Apr-10	18-May-10	26-Mar-11	13-May-11
Existing	2.4	25.3	30.1	33.3	6.85	8.4	20.4	21.5	28.6	32.4	54.8
Weir EL 140ft	7.5	23.6	40.7	34.0	0.02	8.2	24.4	27.7	32.5	38.4	63.4
C6X30 140ft	7.3	23.6	40.9	34.2	0.02	8.1	24.4	27.1	32.5	36.5	63.4
C6X25 140ft	7.3	23.8	41.1	34.4	0.02	8.2	24.8	27.3	32.9	36.2	63.7
C6X20 140ft				34.6	0.02	8.2	24.9	27.4	33.1	36.5	63.9
C4X20 140ft				34.6	0.02	8.2	24.6	27.2	32.4	36.5	63.9
Weir EL 139.5ft	7.5	23.6	40.7				24.3	27.2	32.4	38.4	63.4
C6X30 139.5ft	7.3	23.6	40.7				24.3	27.2	32.4	38.0	63.7
C6X25 139.5ft	7.3	23.4	40.9				24.3	27.2	32.6	37.7	63.7
C6X20 139.5ft	7.3	23.6	41.1				24.3	27.0	32.4	37.2	63.7

Table 6: Difference in Days Exceeding EL 141 feet (Alternative - Existing)

Difference in Days Exceedance at Elevation 141 FT (Alternative - Existing)											
Alternative	4-Feb-02	5-Apr-02	29-May-02	22-May-09	18-Oct-09	9-Nov-09	9-Feb-10	6-Apr-10	18-May-10	26-Mar-11	13-May-11
Existing											
Weir EL 140ft	5.1	-1.7	10.6	0.7	-6.8	-0.2	4.0	6.2	3.8	6.1	8.6
C6X30 140ft	4.9	-1.7	10.8	0.9	-6.8	-0.2	4.0	5.6	3.9	4.1	8.6
C6X25 140ft	4.9	-1.5	11.1	1.1	-6.8	-0.2	4.4	5.8	4.3	3.9	8.8
C6X20 140ft				1.3	-6.8	-0.2	4.5	5.9	4.4	4.1	9.1
C4X20 140ft				1.3	-6.8	-0.2	4.2	5.7	3.7	4.1	9.1
Weir EL 139.5ft	5.1	-1.7	10.6				3.9	5.7	3.7	6.1	8.6
C6X30 139.5ft	4.9	-1.7	10.6				3.9	5.7	3.7	5.6	8.8
C6X25 139.5ft	4.9	-2.0	10.8				3.9	5.7	4.0	5.4	8.8
C6X20 139.5ft	4.9	-1.7	11.1				3.9	5.5	3.7	4.9	8.8
Alt Average	4.9	-1.7	10.8	1.1	-6.8	-0.2	4.1	5.8	3.9	4.9	8.8

Table 7: Difference in Days Exceeding EL 141 feet (Alternative - Owens Weir EL at 140 feet)

Difference in Days Exceedance Elevation 141 FT (Alternative - Weir El 140 ft)											
Alternative	4-Feb-02	5-Apr-02	29-May-02	22-May-09	18-Oct-09	9-Nov-09	9-Feb-10	6-Apr-10	18-May-10	26-Mar-11	13-May-11
Existing											
Weir EL 140ft											
C6X30 140ft	-0.2	0.0	0.3	0.2	0.0	-0.1	0.0	-0.6	0.1	-1.9	0.0
C6X25 140ft	-0.2	0.3	0.5	0.4	0.0	-0.1	0.4	-0.4	0.5	-2.2	0.3
C6X20 140ft				0.6	0.0	0.0	0.5	-0.3	0.6	-1.9	0.5
C4X20 140ft				0.6	0.0	0.0	0.2	-0.4	-0.1	-1.9	0.5
Weir EL 139.5ft	0.0	0.0	0.0				-0.1	-0.4	-0.1	0.0	0.0
C6X30 139.5ft	-0.2	0.0	0.0				-0.1	-0.4	-0.1	-0.5	0.3
C6X25 139.5ft	-0.2	-0.2	0.3				-0.1	-0.4	0.1	-0.7	0.3
C6X20 139.5ft	-0.2	0.0	0.5				-0.1	-0.7	-0.1	-1.2	0.3
Alt Average	-0.2	0.0	0.2	0.5	0.0	0.0	0.1	-0.4	0.1	-1.3	0.3

Table 8: Days Exceeding EL 140.5 feet

Number of Days Exceeding Elevation 140.5 FT for each flood hydrograph event											
Alternative	4-Feb-02	5-Apr-02	29-May-02	22-May-09	18-Oct-09	9-Nov-09	9-Feb-10	6-Apr-10	18-May-10	26-Mar-11	13-May-11
Existing	2.9	26.0	30.6	34.0	7.7	8.9	21.0	22.0	29.2	32.8	55.3
Weir EL 140ft	8.3	24.1	41.6	35.8	0.0	8.6	24.9	28.6	32.9	39.4	65.1
C6X30 140ft	7.3	23.6	40.9	36.9	0.0	8.8	25.8	28.8	33.9	38.0	65.1
C6X25 140ft	7.3	23.8	41.1	37.2	0.0	9.0	26.2	29.0	34.3	37.7	66.6
C6X20 140ft				37.9	0.0	9.2	26.8	29.5	35.0	37.5	68.3
C4X20 140ft				37.9	0.0	9.2	26.9	29.6	35.1	37.5	68.3
Weir EL 139.5ft	7.5	23.6	40.7				25.1	28.2	32.8	39.4	65.1
C6X30 139.5ft	7.3	23.6	40.7				24.8	28.0	33.1	38.9	65.1
C6X25 139.5ft	7.3	23.4	40.9				25.1	28.0	33.1	38.4	65.1
C6X20 139.5ft	7.3	23.6	41.1				24.8	28.0	33.3	38.2	65.6

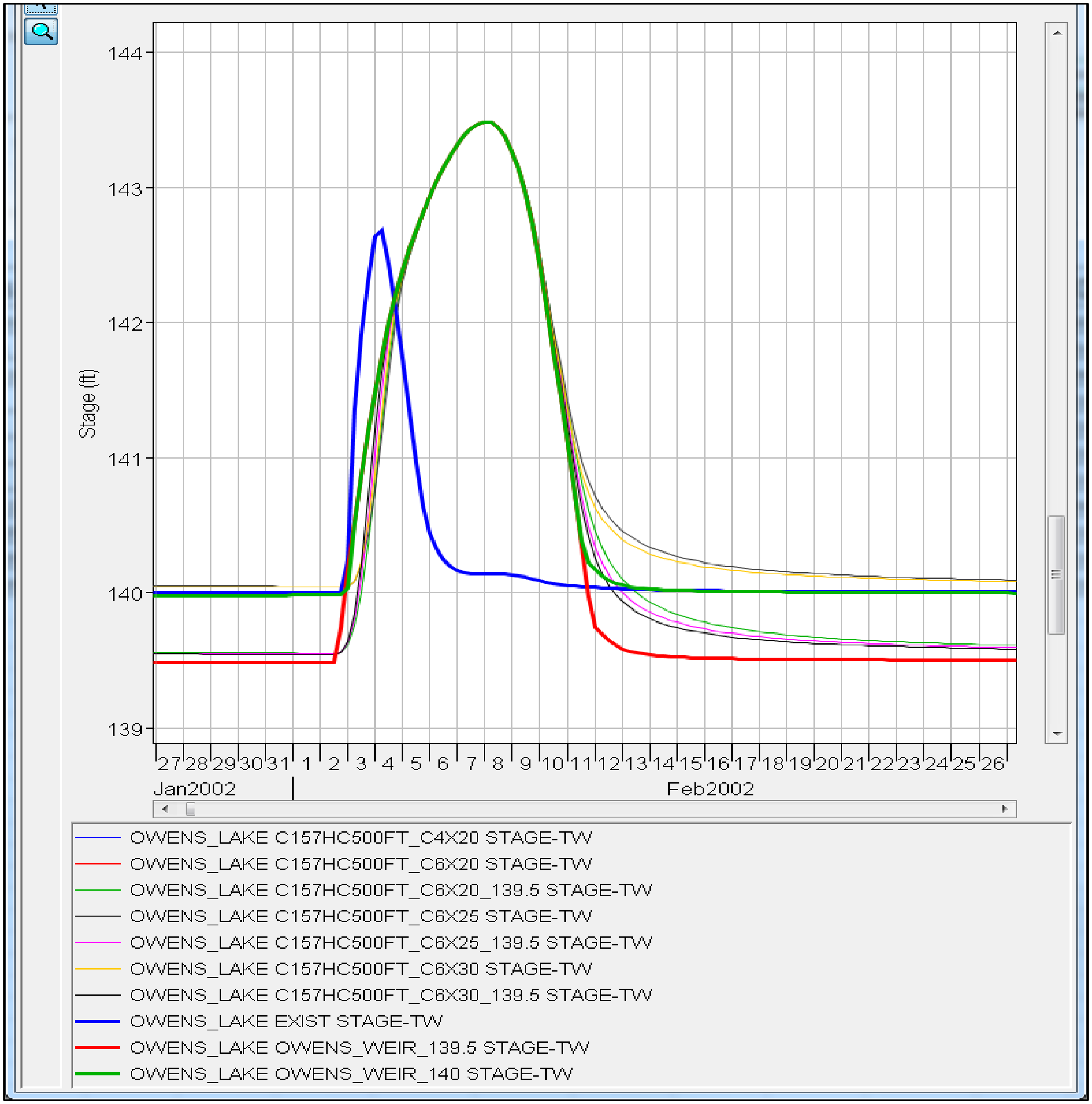
Table 9: Difference in Days Exceeding EL 140.5 feet (Alternative - Existing)

Difference in Days Exceedance at Elevation 140.5 FT (Alternative - Existing)											
Alternative	4-Feb-02	5-Apr-02	29-May-02	22-May-09	18-Oct-09	9-Nov-09	9-Feb-10	6-Apr-10	18-May-10	26-Mar-11	13-May-11
Existing											
Weir EL 140ft	5.8	-2.0	11.1	1.8	-7.7	-0.3	4.0	6.7	3.7	6.6	9.8
C6X30 140ft	4.9	-2.4	10.4	2.8	-7.7	-0.1	4.8	6.8	4.7	5.1	9.8
C6X25 140ft	4.9	-2.2	10.6	3.2	-7.7	0.0	5.2	7.1	5.1	4.9	11.3
C6X20 140ft				3.8	-7.7	0.2	5.8	7.6	5.8	4.6	13.0
C4X20 140ft				3.8	-7.7	0.2	5.9	7.7	5.9	4.6	13.0
Weir EL 139.5ft	5.1	-2.4	10.1				4.1	6.3	3.7	6.6	9.8
C6X30 139.5ft	4.9	-2.4	10.1				3.8	6.0	3.9	6.1	9.8
C6X25 139.5ft	4.9	-2.7	10.4				4.1	6.0	3.9	5.6	9.8
C6X20 139.5ft	4.9	-2.4	10.6				3.8	6.0	4.2	5.3	10.3
Alt Average	5.0	-2.4	10.5	3.1	-7.7	0.0	4.6	6.7	4.6	5.5	10.8

Table 10: Difference in Days Exceeding EL 140.5 feet (Alternative - Owens Weir EL at 140 feet)

Difference in Days Exceedance Elevation 140.5 FT (Alternative - Weir El 140 ft)											
Alternative	4-Feb-02	5-Apr-02	29-May-02	22-May-09	18-Oct-09	9-Nov-09	9-Feb-10	6-Apr-10	18-May-10	26-Mar-11	13-May-11
Existing											
Weir EL 140ft											
C6X30 140ft	-1.0	-0.5	-0.7	1.1	0.0	0.2	0.9	0.1	1.0	-1.5	0.0
C6X25 140ft	-1.0	-0.2	-0.5	1.4	0.0	0.4	1.3	0.4	1.4	-1.7	1.5
C6X20 140ft				2.1	0.0	0.6	1.9	0.9	2.1	-2.0	3.2
C4X20 140ft				2.1	0.0	0.6	2.0	1.0	2.2	-2.0	3.2
Weir EL 139.5ft	-0.7	-0.5	-1.0				0.1	-0.4	-0.1	0.0	0.0
C6X30 139.5ft	-1.0	-0.5	-1.0				-0.1	-0.7	0.2	-0.5	0.0
C6X25 139.5ft	-1.0	-0.7	-0.7				0.1	-0.7	0.2	-1.0	0.0
C6X20 139.5ft	-1.0	-0.5	-0.5				-0.1	-0.7	0.4	-1.2	0.5
Alt Average	-0.9	-0.5	-0.7	1.7	0.0	0.4	0.7	0.0	0.9	-1.2	1.0

Figure 6: 4 February 2002

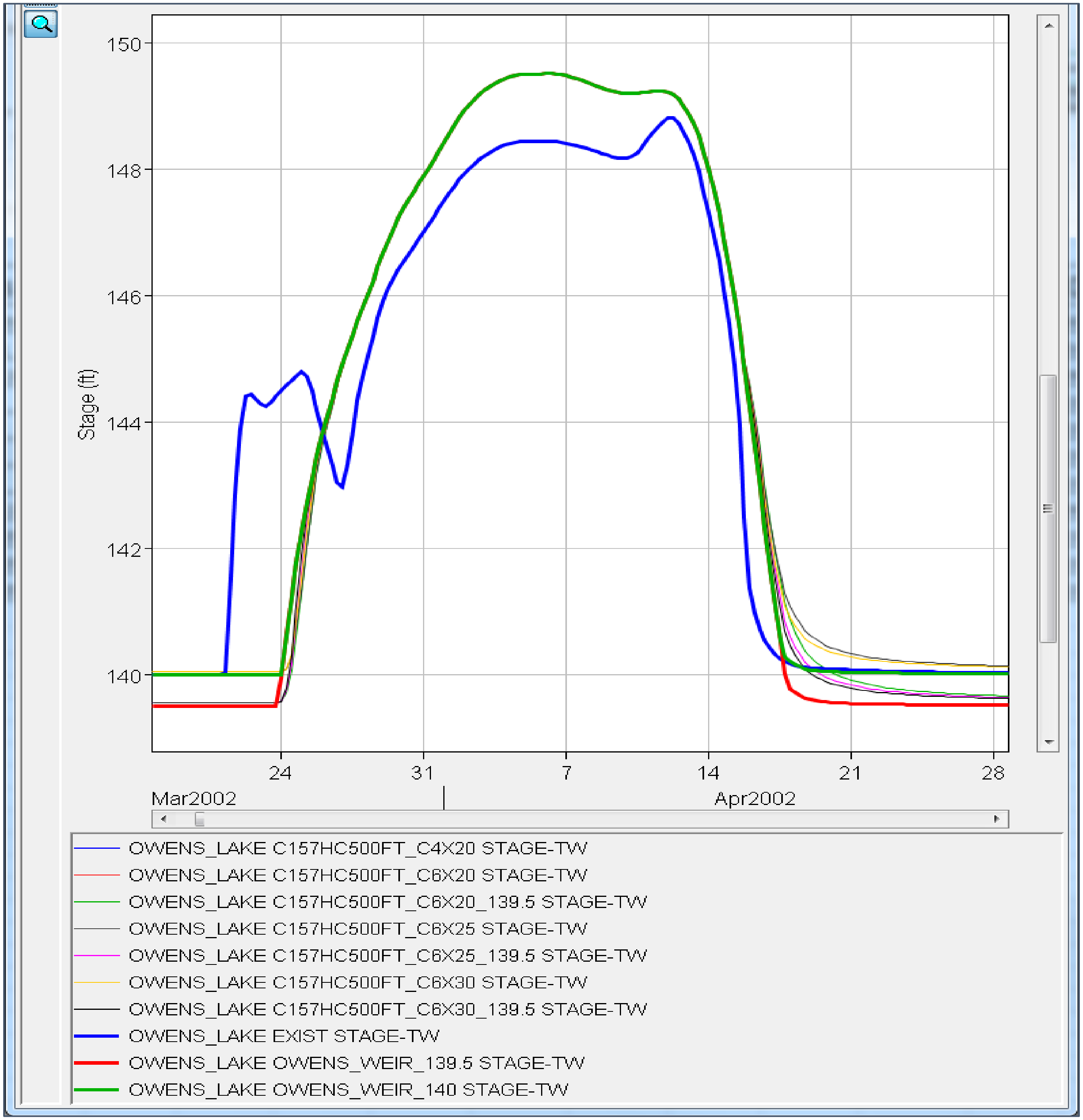


Total Days exceeding 140FT	
Alternative	4-Feb-02
Existing	2.4
Weir EL 140ft	7.5
C6X30 140ft	7.3
C6X25 140ft	7.3
C6X20 140ft	
C4X20 140ft	
Weir EL 139.5ft	7.5
C6X30 139.5ft	7.3
C6X25 139.5ft	7.3
C6X20 139.5ft	7.3

Alternative - Existing	
Alternative	4-Feb-02
Existing	
Weir EL 140ft	5.1
C6X30 140ft	4.9
C6X25 140ft	4.9
C6X20 140ft	
C4X20 140ft	
Weir EL 139.5ft	5.1
C6X30 139.5ft	4.9
C6X25 139.5ft	4.9
C6X20 139.5ft	4.9

Alernative - Weir EL 140 FT	
Alternative	4-Feb-02
Existing	
Weir EL 140ft	
C6X30 140ft	-0.2
C6X25 140ft	-0.2
C6X20 140ft	
C4X20 140ft	
Weir EL 139.5ft	0.0
C6X30 139.5ft	-0.2
C6X25 139.5ft	-0.2
C6X20 139.5ft	-0.2

Figure 7: 5 April 2002

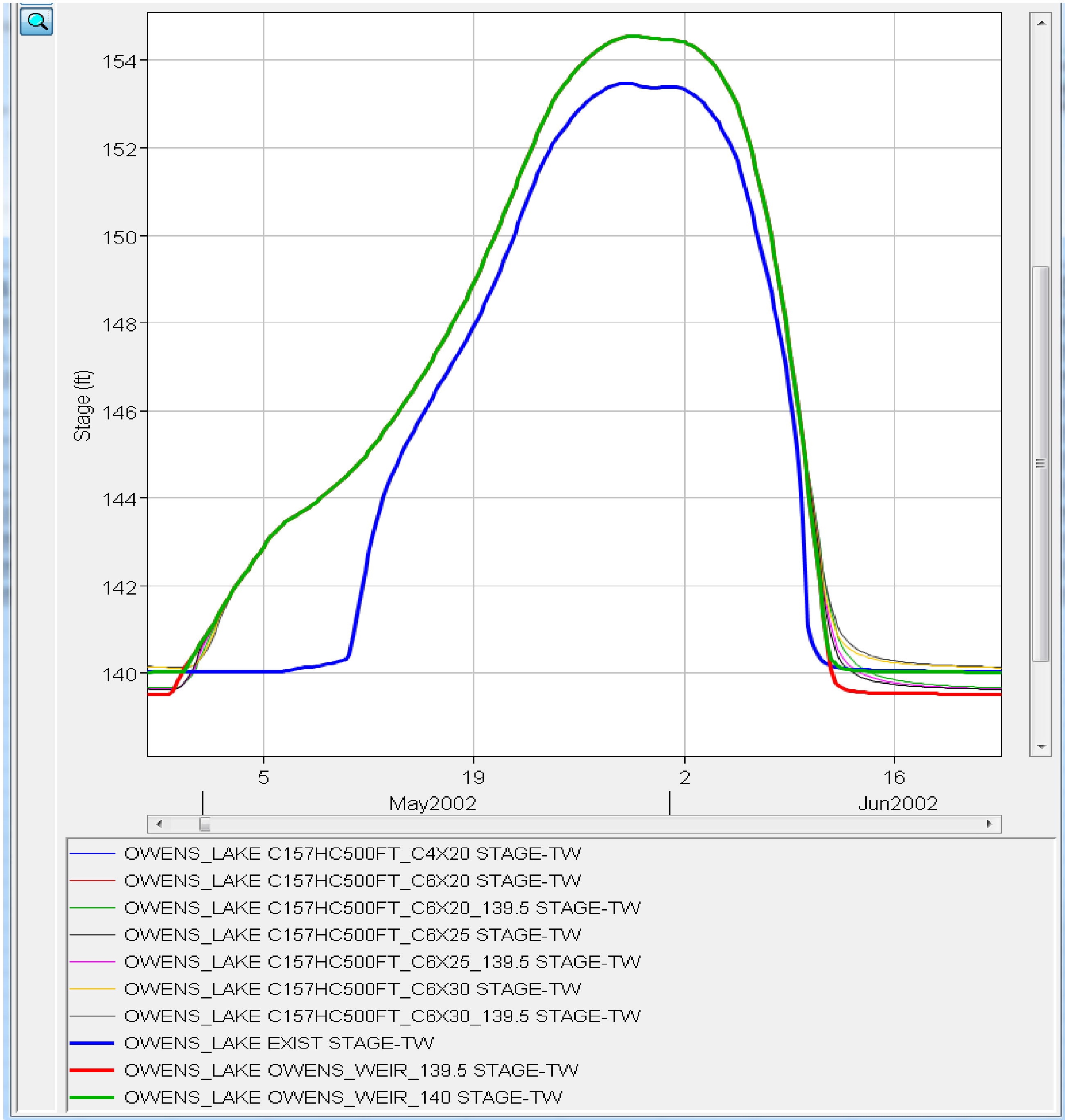


Total Days exceeding 140FT	
Alternative	5-Apr-02
Existing	25.3
Weir EL 140ft	23.6
C6X30 140ft	23.6
C6X25 140ft	23.8
C6X20 140ft	
C4X20 140ft	
Weir EL 139.5ft	23.6
C6X30 139.5ft	23.6
C6X25 139.5ft	23.4
C6X20 139.5ft	23.6

Alternative - Existing	
Alternative	5-Apr-02
Existing	
Weir EL 140ft	-1.7
C6X30 140ft	-1.7
C6X25 140ft	-1.5
C6X20 140ft	
C4X20 140ft	
Weir EL 139.5ft	-1.7
C6X30 139.5ft	-1.7
C6X25 139.5ft	-2.0
C6X20 139.5ft	-1.7

Alernative - Weir EL 140 FT	
Alternative	5-Apr-02
Existing	
Weir EL 140ft	
C6X30 140ft	0.0
C6X25 140ft	0.3
C6X20 140ft	
C4X20 140ft	
Weir EL 139.5ft	0.0
C6X30 139.5ft	0.0
C6X25 139.5ft	-0.2
C6X20 139.5ft	0.0

Figure 8: 29 May 2002

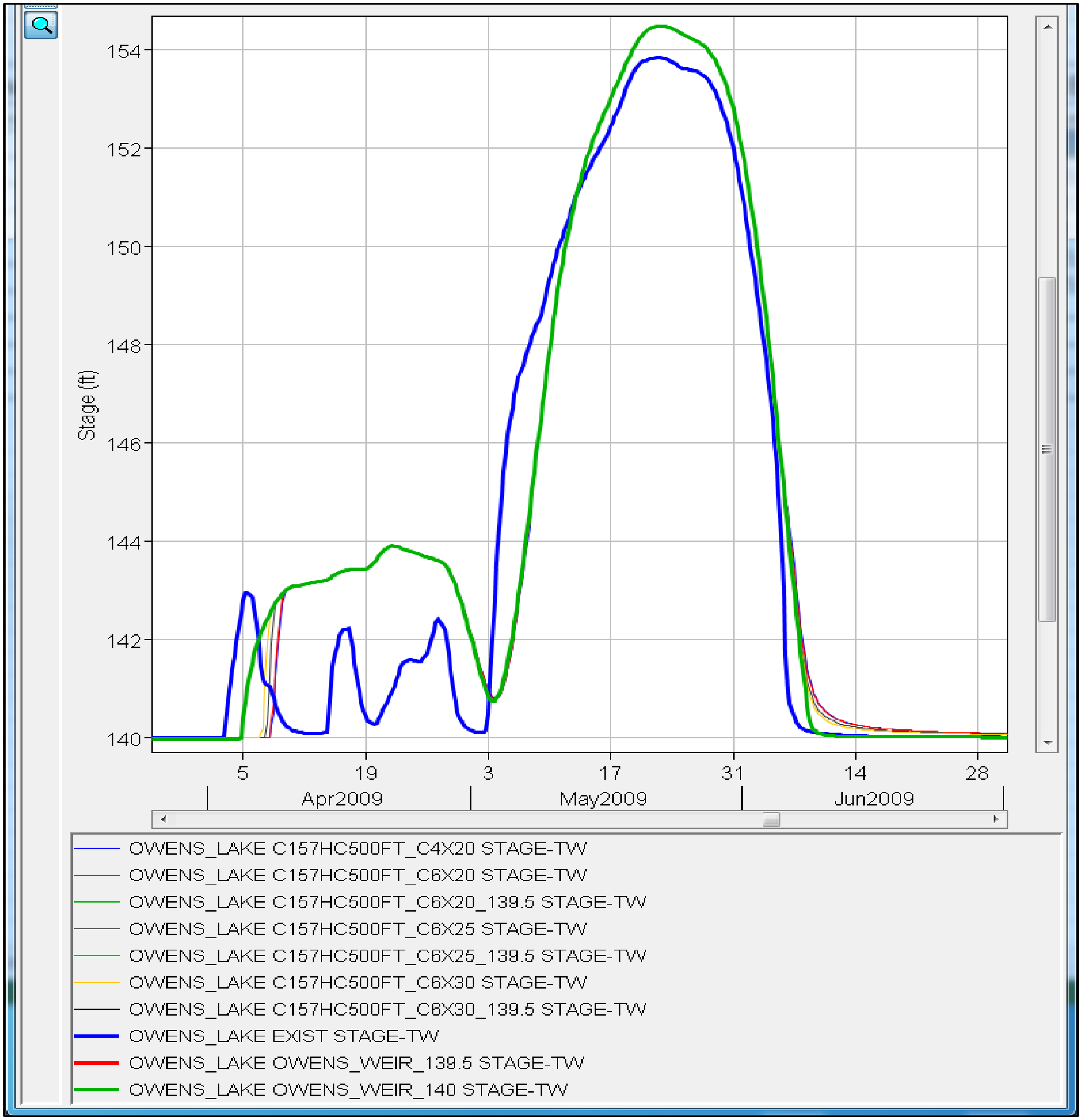


Total Days exceeding 140FT	
Alternative	29-May-02
Existing	30.1
Weir EL 140ft	40.7
C6X30 140ft	40.9
C6X25 140ft	41.1
C6X20 140ft	
C4X20 140ft	
Weir EL 139.5ft	40.7
C6X30 139.5ft	40.7
C6X25 139.5ft	40.9
C6X20 139.5ft	41.1

Alternative - Existing	
Alternative	29-May-02
Existing	
Weir EL 140ft	10.6
C6X30 140ft	10.8
C6X25 140ft	11.1
C6X20 140ft	
C4X20 140ft	
Weir EL 139.5ft	10.6
C6X30 139.5ft	10.6
C6X25 139.5ft	10.8
C6X20 139.5ft	11.1

Alernative - Weir EL 140 FT	
Alternative	29-May-02
Existing	
Weir EL 140ft	
C6X30 140ft	0.3
C6X25 140ft	0.5
C6X20 140ft	
C4X20 140ft	
Weir EL 139.5ft	0.0
C6X30 139.5ft	0.0
C6X25 139.5ft	0.3
C6X20 139.5ft	0.5

Figure 9: 22 May 2009

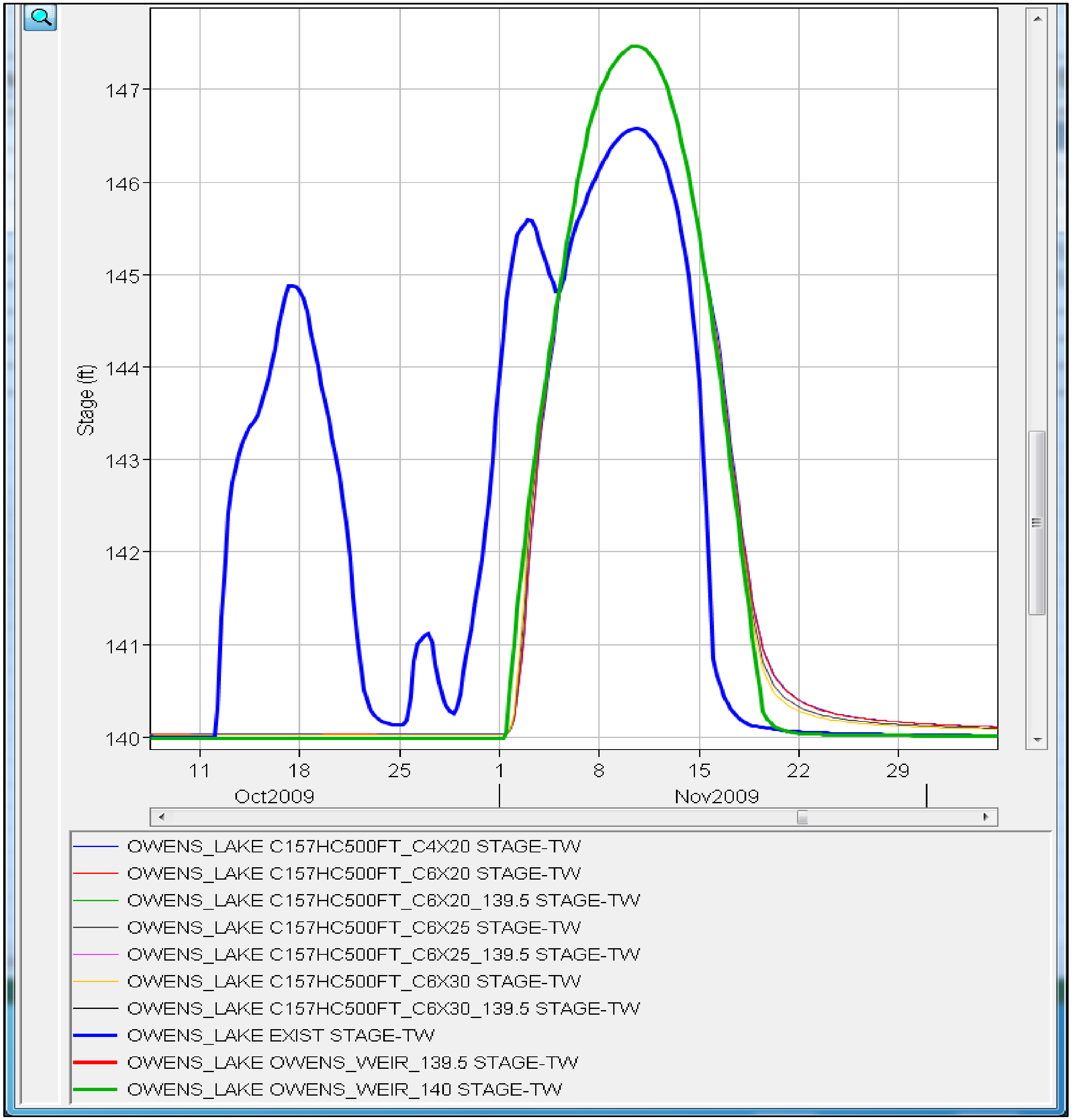


Total Days exceeding 140FT	
Alternative	22-May-09
Existing	33.3
Weir EL 140ft	34.0
C6X30 140ft	34.2
C6X25 140ft	34.4
C6X20 140ft	34.6
C4X20 140ft	34.6
Weir EL 139.5ft	
C6X30 139.5ft	
C6X25 139.5ft	
C6X20 139.5ft	

Alternative - Existing	
Alternative	22-May-09
Existing	
Weir EL 140ft	0.7
C6X30 140ft	0.9
C6X25 140ft	1.1
C6X20 140ft	1.3
C4X20 140ft	1.3
Weir EL 139.5ft	
C6X30 139.5ft	
C6X25 139.5ft	
C6X20 139.5ft	

Alernative - Weir EL 140 FT	
Alternative	22-May-09
Existing	
Weir EL 140ft	
C6X30 140ft	0.2
C6X25 140ft	0.4
C6X20 140ft	0.6
C4X20 140ft	0.6
Weir EL 139.5ft	
C6X30 139.5ft	
C6X25 139.5ft	
C6X20 139.5ft	

Figure 10: 18 Oct 2009 and 9 Nov 2009

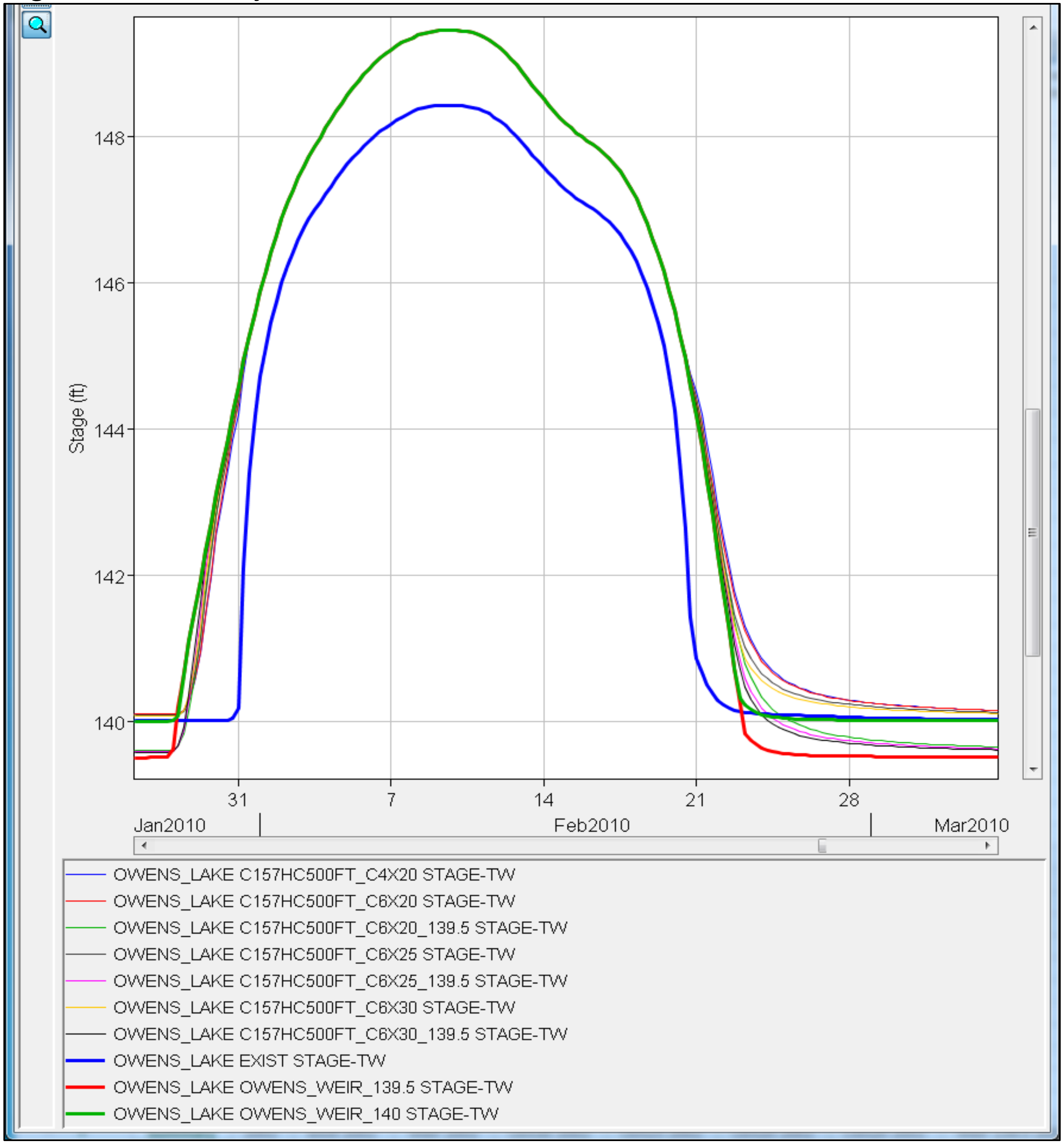


Total Days exceeding 140FT		ys exceedir
Alternative	18-Oct-09	9-Nov-09
Existing	6.8	8.4
Weir EL 140ft	0.02	8.2
C6X30 140ft	0.02	8.1
C6X25 140ft	0.02	8.2
C6X20 140ft	0.02	8.2
C4X20 140ft	0.02	8.2
Weir EL 139.5ft		
C6X30 139.5ft		
C6X25 139.5ft		
C6X20 139.5ft		

Alternative - Existing		rnative - Exi
Alternative	18-Oct-09	9-Nov-09
Existing		
Weir EL 140ft	-6.8	-0.2
C6X30 140ft	-6.8	-0.2
C6X25 140ft	-6.8	-0.2
C6X20 140ft	-6.8	-0.2
C4X20 140ft	-6.8	-0.2
Weir EL 139.5ft		
C6X30 139.5ft		
C6X25 139.5ft		
C6X20 139.5ft		

Alernative - Weir EL 140 FT		ive - Weir E
Alternative	18-Oct-09	9-Nov-09
Existing		
Weir EL 140ft		
C6X30 140ft	0.0	-0.1
C6X25 140ft	0.0	-0.1
C6X20 140ft	0.0	0.0
C4X20 140ft	0.0	0.0
Weir EL 139.5ft		
C6X30 139.5ft		
C6X25 139.5ft		
C6X20 139.5ft		

Figure 11: 9 February 2010

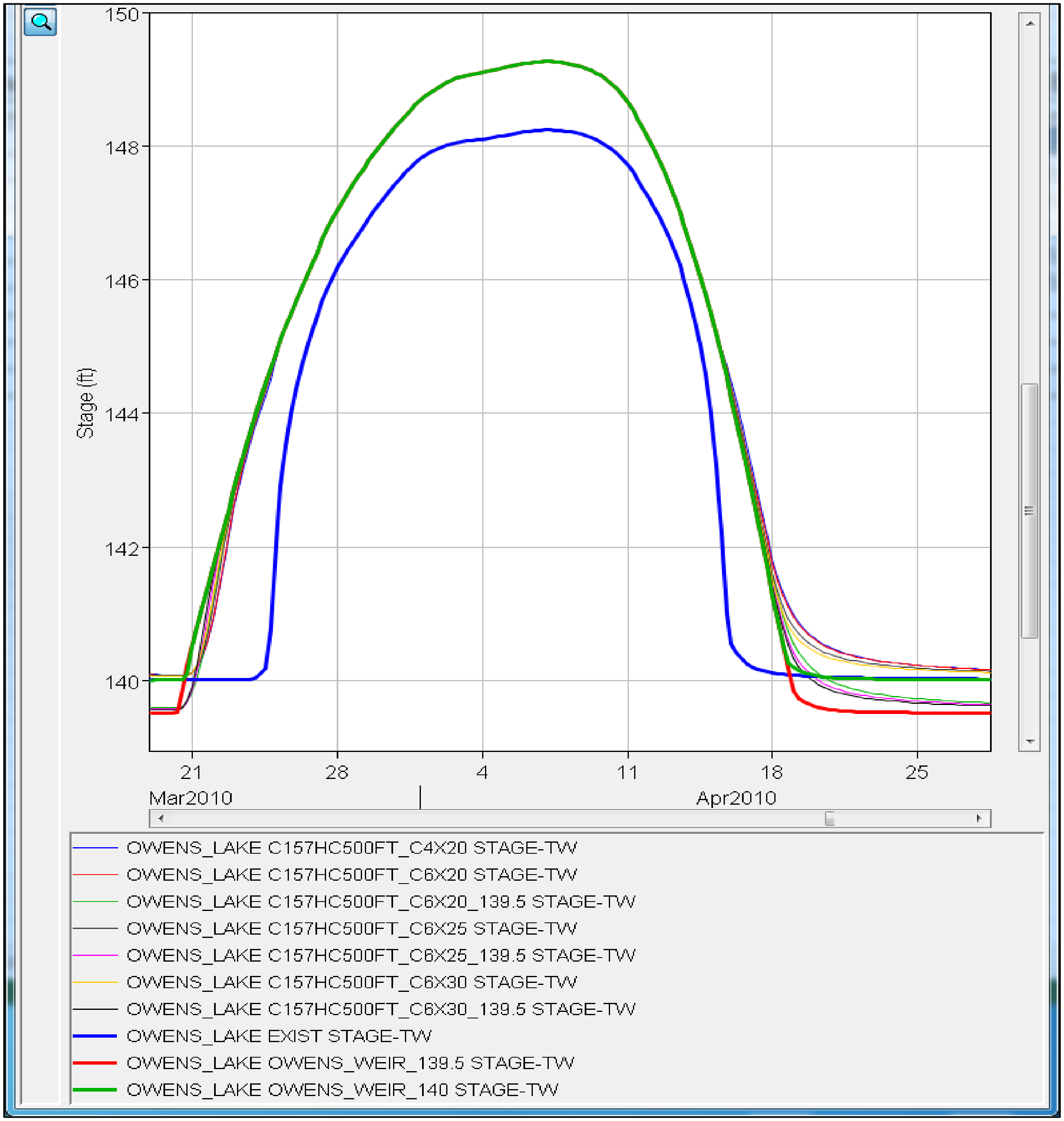


Total Days exceeding 140FT	
Alternative	9-Feb-10
Existing	20.4
Weir EL 140ft	24.4
C6X30 140ft	24.4
C6X25 140ft	24.8
C6X20 140ft	24.9
C4X20 140ft	24.6
Weir EL 139.5ft	24.3
C6X30 139.5ft	24.3
C6X25 139.5ft	24.3
C6X20 139.5ft	24.3

Alternative - Existing	
Alternative	9-Feb-10
Existing	
Weir EL 140ft	4.0
C6X30 140ft	4.0
C6X25 140ft	4.4
C6X20 140ft	4.5
C4X20 140ft	4.2
Weir EL 139.5ft	3.9
C6X30 139.5ft	3.9
C6X25 139.5ft	3.9
C6X20 139.5ft	3.9

Alernative - Weir EL 140 FT	
Alternative	9-Feb-10
Existing	
Weir EL 140ft	
C6X30 140ft	0.0
C6X25 140ft	0.4
C6X20 140ft	0.5
C4X20 140ft	0.2
Weir EL 139.5ft	-0.1
C6X30 139.5ft	-0.1
C6X25 139.5ft	-0.1
C6X20 139.5ft	-0.1

Figure 12: 6 Apr 2010

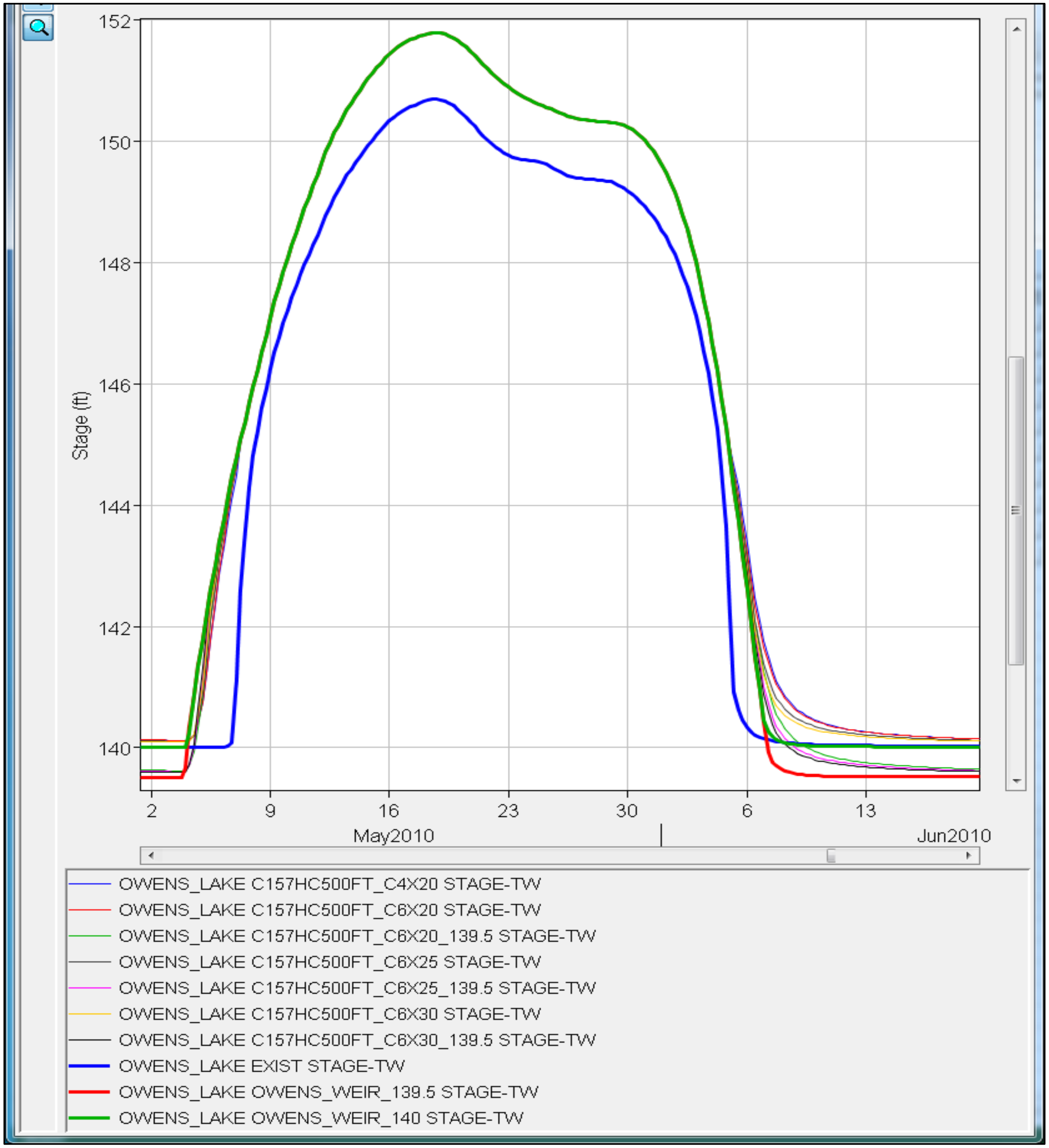


Total Days exceeding 140FT	
Alternative	6-Apr-10
Existing	21.5
Weir EL 140ft	27.7
C6X30 140ft	27.1
C6X25 140ft	27.3
C6X20 140ft	27.4
C4X20 140ft	27.2
Weir EL 139.5ft	27.2
C6X30 139.5ft	27.2
C6X25 139.5ft	27.2
C6X20 139.5ft	27.0

Alternative - Existing	
Alternative	6-Apr-10
Existing	
Weir EL 140ft	6.2
C6X30 140ft	5.6
C6X25 140ft	5.8
C6X20 140ft	5.9
C4X20 140ft	5.7
Weir EL 139.5ft	5.7
C6X30 139.5ft	5.7
C6X25 139.5ft	5.7
C6X20 139.5ft	5.5

Alernative - Weir EL 140 FT	
Alternative	6-Apr-10
Existing	
Weir EL 140ft	
C6X30 140ft	-0.6
C6X25 140ft	-0.4
C6X20 140ft	-0.3
C4X20 140ft	-0.4
Weir EL 139.5ft	-0.4
C6X30 139.5ft	-0.4
C6X25 139.5ft	-0.4
C6X20 139.5ft	-0.7

Figure 13: 18 May 2010

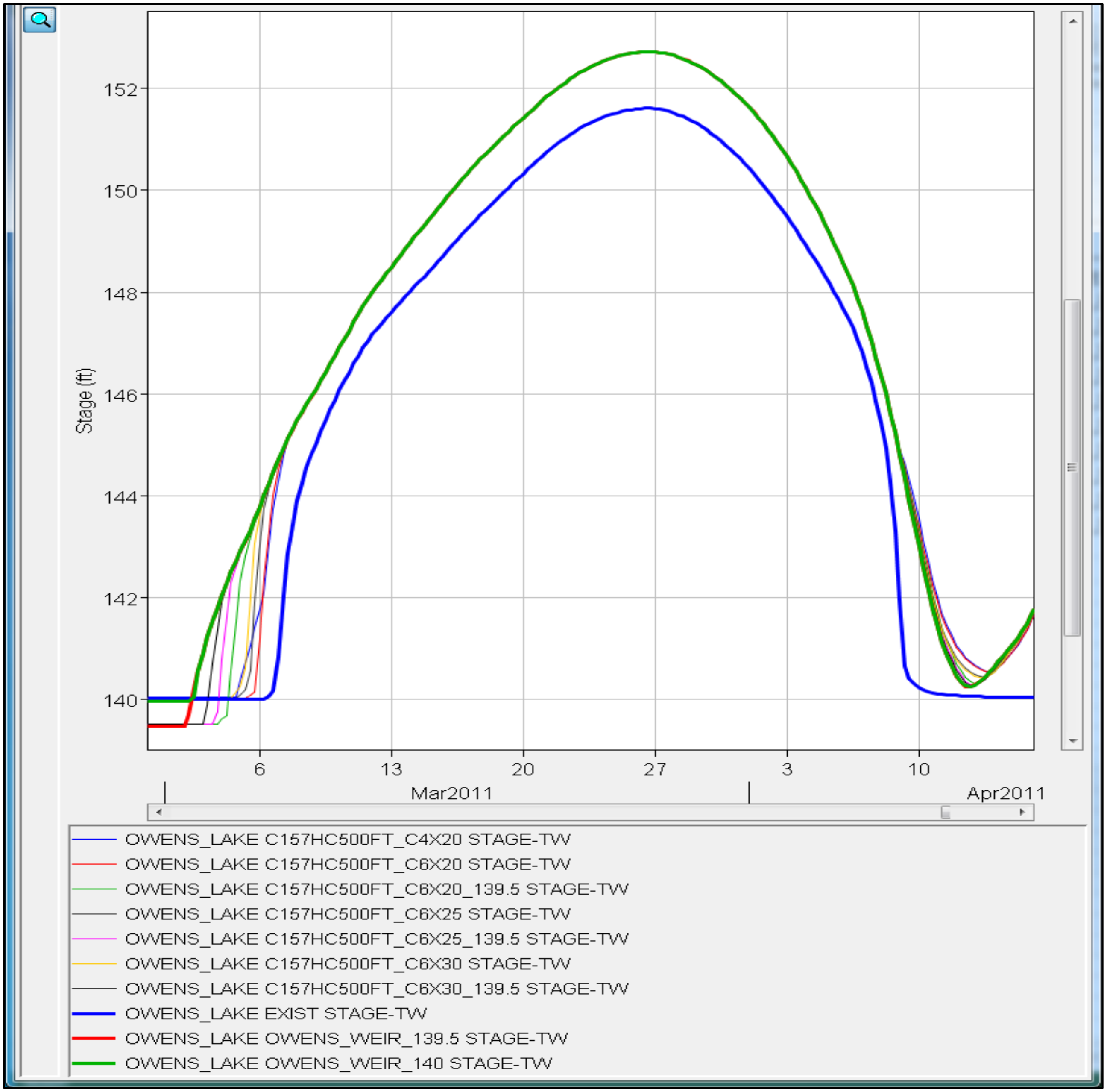


Total Days exceeding 140FT	
Alternative	18-May-10
Existing	28.6
Weir EL 140ft	32.5
C6X30 140ft	32.5
C6X25 140ft	32.9
C6X20 140ft	33.1
C4X20 140ft	32.4
Weir EL 139.5ft	32.4
C6X30 139.5ft	32.4
C6X25 139.5ft	32.6
C6X20 139.5ft	32.4

Alternative - Existing	
Alternative	18-May-10
Existing	
Weir EL 140ft	3.8
C6X30 140ft	3.9
C6X25 140ft	4.3
C6X20 140ft	4.4
C4X20 140ft	3.7
Weir EL 139.5ft	3.7
C6X30 139.5ft	3.7
C6X25 139.5ft	4.0
C6X20 139.5ft	3.7

Alernative - Weir EL 140 FT	
Alternative	18-May-10
Existing	
Weir EL 140ft	
C6X30 140ft	0.1
C6X25 140ft	0.5
C6X20 140ft	0.6
C4X20 140ft	-0.1
Weir EL 139.5ft	-0.1
C6X30 139.5ft	-0.1
C6X25 139.5ft	0.1
C6X20 139.5ft	-0.1

Figure 14: 26 March 2011

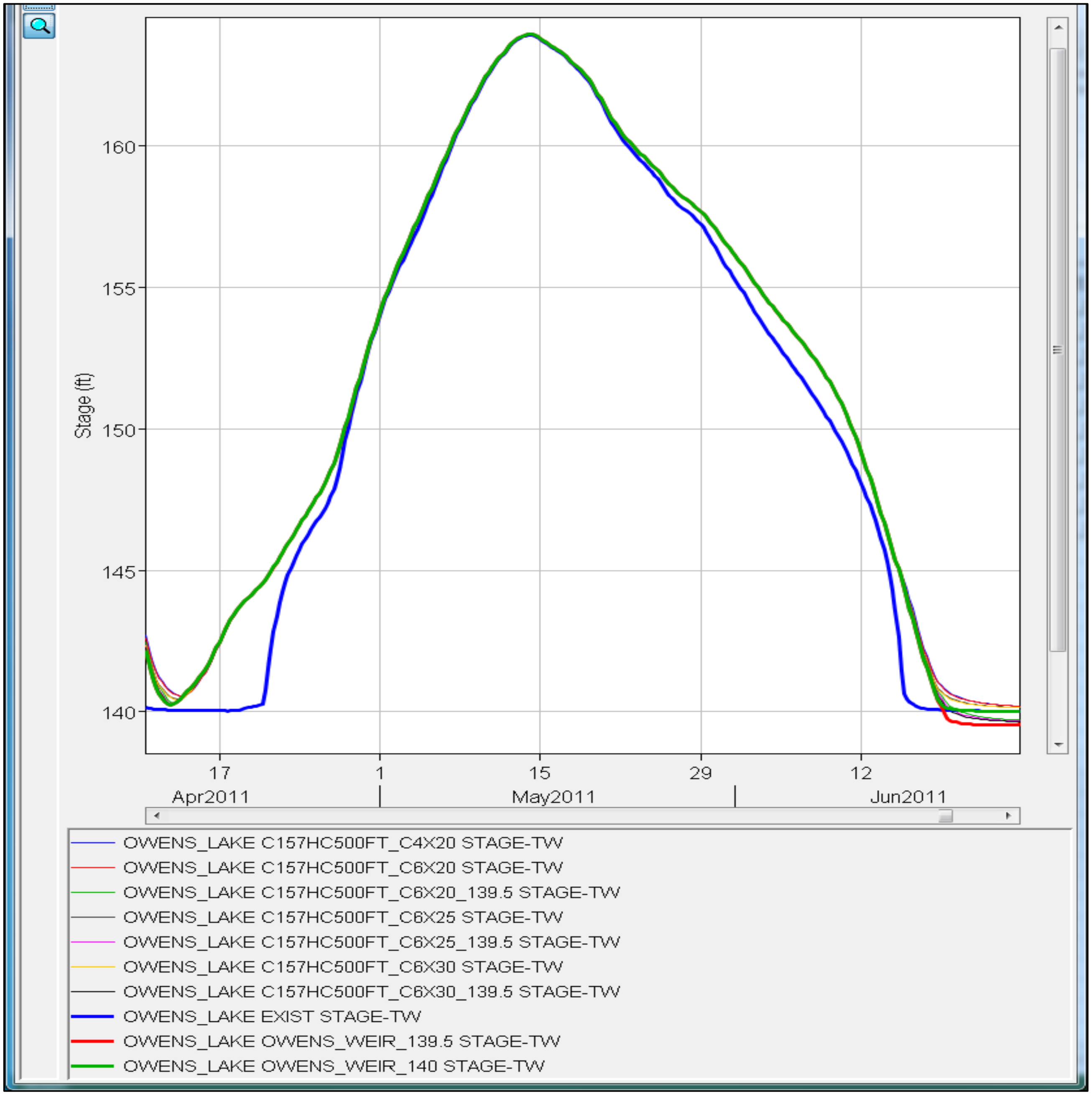


Total Days exceeding 140FT	
Alternative	26-Mar-11
Existing	32.4
Weir EL 140ft	38.4
C6X30 140ft	36.5
C6X25 140ft	36.2
C6X20 140ft	36.5
C4X20 140ft	36.5
Weir EL 139.5ft	38.4
C6X30 139.5ft	38.0
C6X25 139.5ft	37.7
C6X20 139.5ft	37.2

Alternative - Existing	
Alternative	26-Mar-11
Existing	
Weir EL 140ft	6.1
C6X30 140ft	4.1
C6X25 140ft	3.9
C6X20 140ft	4.1
C4X20 140ft	4.1
Weir EL 139.5ft	6.1
C6X30 139.5ft	5.6
C6X25 139.5ft	5.4
C6X20 139.5ft	4.9

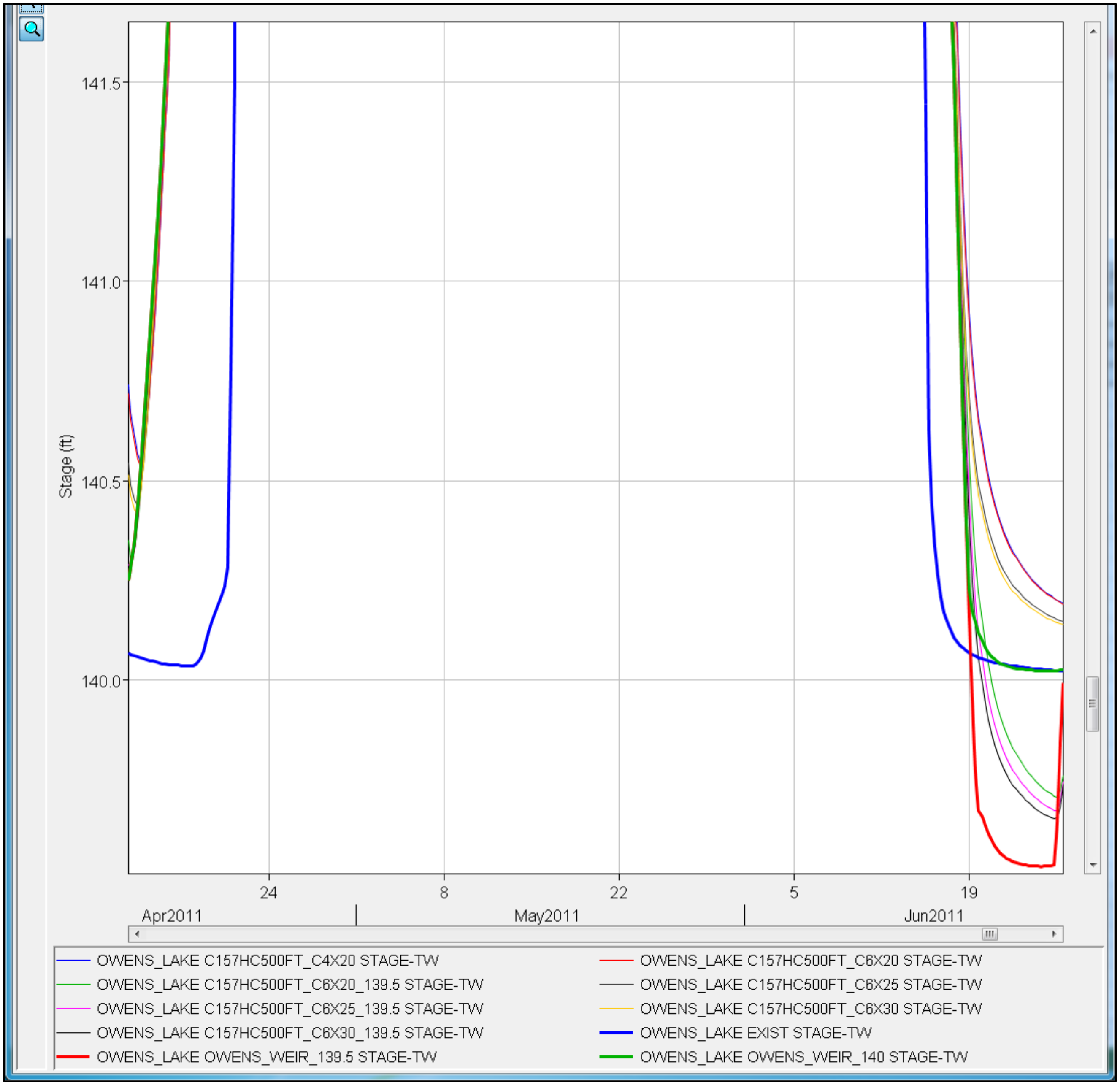
Alernative - Weir EL 140 FT	
Alternative	26-Mar-11
Existing	
Weir EL 140ft	
C6X30 140ft	-1.9
C6X25 140ft	-2.2
C6X20 140ft	-1.9
C4X20 140ft	-1.9
Weir EL 139.5ft	0.0
C6X30 139.5ft	-0.5
C6X25 139.5ft	-0.7
C6X20 139.5ft	-1.2

Figure 15: 13 May 2011



Total Days exceeding 140FT	
Alternative	13-May-11
Existing	54.8
Weir EL 140ft	63.4
C6X30 140ft	63.4
C6X25 140ft	63.7
C6X20 140ft	63.9
C4X20 140ft	63.9
Weir EL 139.5ft	63.4
C6X30 139.5ft	63.7
C6X25 139.5ft	63.7
C6X20 139.5ft	63.7
Alternative - Existing	
Alternative	13-May-11
Existing	
Weir EL 140ft	8.6
C6X30 140ft	8.6
C6X25 140ft	8.8
C6X20 140ft	9.1
C4X20 140ft	9.1
Weir EL 139.5ft	8.6
C6X30 139.5ft	8.8
C6X25 139.5ft	8.8
C6X20 139.5ft	8.8
Alernative - Weir EL 140 FT	
Alternative	13-May-11
Existing	
Weir EL 140ft	
C6X30 140ft	0.0
C6X25 140ft	0.3
C6X20 140ft	0.5
C4X20 140ft	0.5
Weir EL 139.5ft	0.0
C6X30 139.5ft	0.3
C6X25 139.5ft	0.3
C6X20 139.5ft	0.3

Figure 16: 13 May 2011: zoomed in

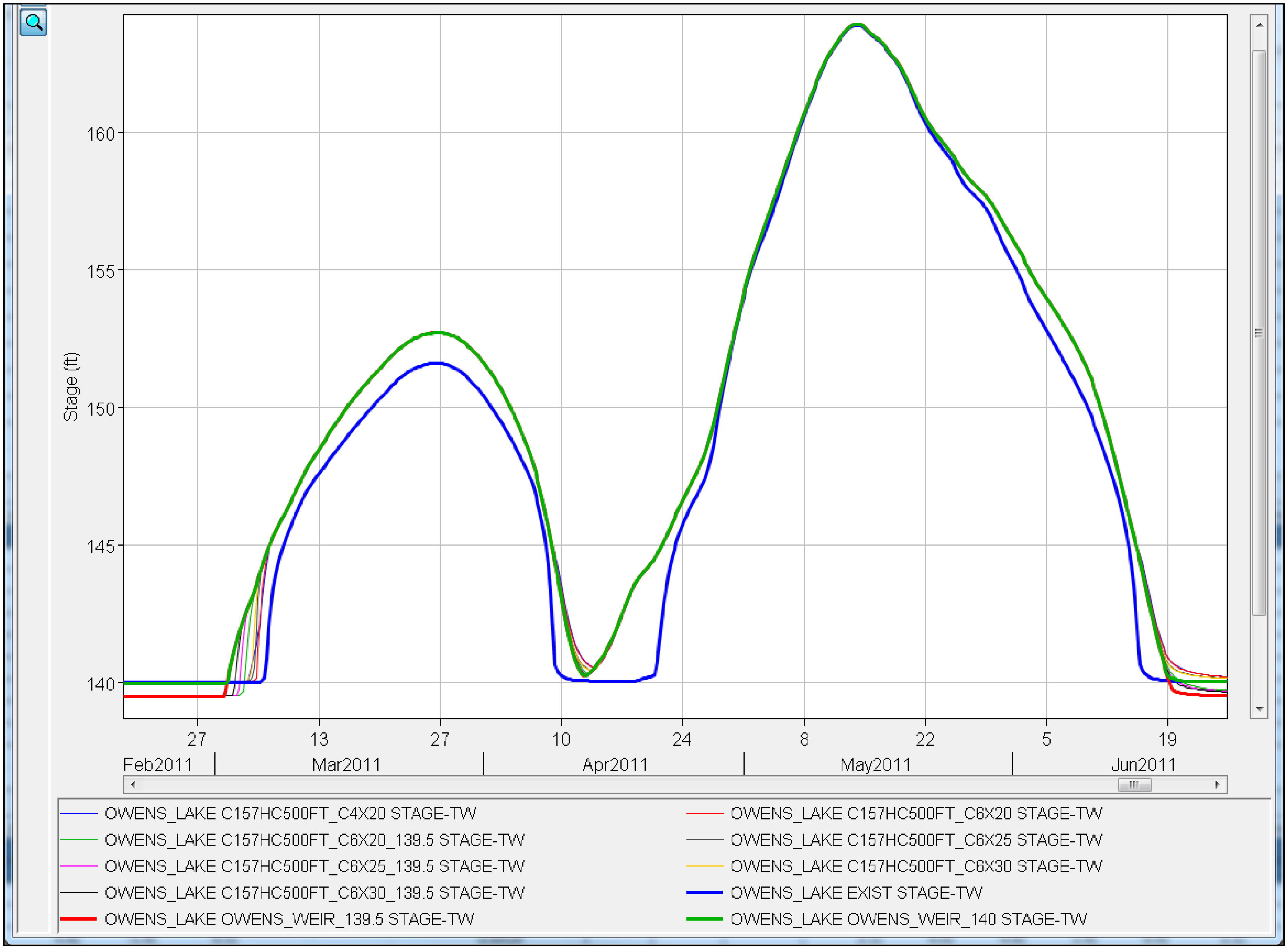


Total Days exceeding 140FT	
Alternative	13-May-11
Existing	54.8
Weir EL 140ft	63.4
C6X30 140ft	63.4
C6X25 140ft	63.7
C6X20 140ft	63.9
C4X20 140ft	63.9
Weir EL 139.5ft	63.4
C6X30 139.5ft	63.7
C6X25 139.5ft	63.7
C6X20 139.5ft	63.7

Alternative - Existing	
Alternative	13-May-11
Existing	
Weir EL 140ft	8.6
C6X30 140ft	8.6
C6X25 140ft	8.8
C6X20 140ft	9.1
C4X20 140ft	9.1
Weir EL 139.5ft	8.6
C6X30 139.5ft	8.8
C6X25 139.5ft	8.8
C6X20 139.5ft	8.8

Alernative - Weir EL 140 FT	
Alternative	13-May-11
Existing	
Weir EL 140ft	
C6X30 140ft	0.0
C6X25 140ft	0.3
C6X20 140ft	0.5
C4X20 140ft	0.5
Weir EL 139.5ft	0.0
C6X30 139.5ft	0.3
C6X25 139.5ft	0.3
C6X20 139.5ft	0.3

Figure 17: 2011 Hydrographs: Both Peaks



Event Specific Elevation Range Exceedance Duration in Owens Lake for Con/Span C6X30

Previous sections of this report focused on determining the event specific (11 flood events) exceedance duration at elevation 140.5 feet and 141 feet to adequately size an outlet structure. Table 11 through Table 13 provides information on the event specific elevation exceedance duration for an elevation range, 140.5ft – 155ft, for the Con/Span C6X30, Existing Conditions, and with Owens Weir at elevation 140 feet and 720 feet wide to ensure that Con/Span C6X30 will adequately drain Owens Lake through the entire elevation range.

The difference in days exceedance between Con/Span C6X30 and both Existing Conditions and Owens Weir at elevation 140 feet and 720 feet wide is provided in Table 14 and Table 15. When compared to Existing Conditions, Con/Span C6X30 had a maximum increase in inundation duration of 12.1 days and a maximum decrease in duration of 7.7 days. When compared to Owens Weir at elevation 140 feet and 720 feet wide, Con/Span C6X30 had a maximum increase in inundation duration of 1.1 days and a maximum decrease in duration of 1.5 days.

Table 11: Existing: Event Specific Incremental Elevation Exceedance

Existing											
Elevation	Elevation Exceedance in days per flood event										
Feet	4-Feb-02	Feet	4-Feb-02	Feet	4-Feb-02	Feet	4-Feb-02	Feet	4-Feb-02	Feet	4-Feb-02
140.5	2.9	26.0	30.6	34.0	7.7	8.9	21.0	22.0	29.2	32.8	55.3
141	2.4	25.3	30.1	33.3	6.8	8.4	20.4	21.5	28.6	32.4	54.8
142	1.5	24.8	29.3	32.7	4.2	7.7	20.0	21.1	28.3	32.1	54.3
143	0.0	24.1	28.6	32.3	3.4	7.3	19.5	20.6	27.9	31.6	53.6
144	0.0	22.1	27.8	31.7	1.6	6.8	19.0	19.8	27.4	30.9	52.9
145	0.0	17.8	26.4	31.0	0.0	5.6	18.0	18.6	26.7	29.7	51.6
146	0.0	16.5	24.1	30.0	0.0	2.6	16.3	16.7	25.5	28.0	49.9
146	0.0	16.5	24.1	30.0	0.0	2.6	16.3	16.7	25.5	28.0	49.9
148	0.0	11.2	20.0	26.2	0.0	0.0	6.4	7.7	21.5	22.4	45.5
149	0.0	0.0	18.2	23.7	0.0	0.0	0.0	0.0	17.8	18.2	43.8
150	0.0	0.0	16.3	21.4	0.0	0.0	0.0	0.0	6.8	13.6	42.0
151	0.0	0.0	14.3	18.7	0.0	0.0	0.0	0.0	0.0	8.3	39.8
152	0.0	0.0	12.1	14.9	0.0	0.0	0.0	0.0	0.0	0.0	37.6
153	0.0	0.0	8.1	10.7	0.0	0.0	0.0	0.0	0.0	0.0	35.2
154	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	32.7
155	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	30.2
156	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	27.5
157	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	25.1
158	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	21.4
159	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	18.4
160	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	15.2

Table 12: Owens Weir: Elevation at 140 feet and 720 feet wide: Event Specific Incremental Elevation Exceedance

Owens Weir at Elevation 140 feet and 720 feet wide											
Elevation	Elevation Exceedance in days per flood event										
Feet	4-Feb-02	Feet	4-Feb-02	Feet	4-Feb-02	Feet	4-Feb-02	Feet	4-Feb-02	Feet	4-Feb-02
140.5	8.3	24.1	41.6	35.8	0.0	8.6	24.9	28.6	32.9	39.4	65.1
141	7.5	23.6	40.7	34.0	0.0	8.2	24.4	27.7	32.5	38.4	63.4
142	5.8	22.6	38.7	32.2	0.0	7.5	23.3	25.7	31.4	37.0	61.2
143	3.4	21.7	36.0	30.7	0.0	6.7	22.2	24.2	30.4	35.3	59.2
144	0.0	20.7	32.3	29.4	0.0	5.9	21.1	22.6	29.3	33.6	56.8
145	0.0	19.2	28.8	28.3	0.0	4.9	19.8	20.8	28.1	31.9	54.1
146	0.0	17.8	26.4	27.0	0.0	3.9	18.2	18.7	26.7	29.9	51.9
146	0.0	17.8	26.4	27.0	0.0	3.9	18.2	18.7	26.7	29.9	51.9
148	0.0	13.6	22.2	24.5	0.0	0.0	11.7	12.9	23.6	25.1	47.2
149	0.0	10.7	20.2	23.1	0.0	0.0	6.3	7.2	21.4	21.9	45.5
150	0.0	0.0	18.2	21.5	0.0	0.0	0.0	0.0	18.2	18.5	43.8
151	0.0	0.0	16.5	19.5	0.0	0.0	0.0	0.0	7.7	14.4	42.0
152	0.0	0.0	14.5	16.9	0.0	0.0	0.0	0.0	0.0	9.0	39.8
153	0.0	0.0	12.3	13.4	0.0	0.0	0.0	0.0	0.0	0.0	37.4
154	0.0	0.0	8.4	8.3	0.0	0.0	0.0	0.0	0.0	0.0	34.7
155	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	31.7
156	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	29.0
157	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	26.3
158	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	22.6
159	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	19.2
160	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	16.0

Table 13: Con/Span C6X30: Event Specific Incremental Elevation Exceedance

Con/Span C6X30											
Elevation	Elevation Exceedance in days per flood event										
Feet	4-Feb-02	Feet	4-Feb-02	Feet	4-Feb-02	Feet	4-Feb-02	Feet	4-Feb-02	Feet	4-Feb-02
140.5	8.5	25.1	42.6	36.9	0.0	8.8	25.8	28.8	33.9	38.0	65.1
141	7.3	23.6	40.9	34.2	0.0	8.1	24.4	27.1	32.5	36.5	63.4
142	5.8	22.6	38.7	32.2	0.0	7.5	23.3	25.7	31.4	35.5	61.2
143	3.4	21.7	36.2	30.7	0.0	6.7	22.3	24.2	30.5	34.8	59.2
144	0.0	20.4	32.3	29.5	0.0	5.9	21.1	22.6	29.4	33.6	56.8
145	0.0	19.2	29.1	28.3	0.0	5.0	19.8	20.9	28.1	31.9	54.1
146	0.0	17.8	26.4	27.0	0.0	3.9	18.2	18.7	26.8	29.9	51.9
146	0.0	17.8	26.4	27.0	0.0	3.9	18.2	18.7	26.8	29.9	51.9
148	0.0	13.6	22.2	24.5	0.0	0.0	11.7	12.9	23.6	25.1	47.2
149	0.0	10.7	20.2	23.1	0.0	0.0	6.3	7.2	21.4	21.9	45.5
150	0.0	0.0	18.5	21.5	0.0	0.0	0.0	0.0	18.2	18.5	43.8
151	0.0	0.0	16.5	19.5	0.0	0.0	0.0	0.0	7.7	14.4	42.0
152	0.0	0.0	14.5	16.9	0.0	0.0	0.0	0.0	0.0	9.0	39.8
153	0.0	0.0	12.3	13.4	0.0	0.0	0.0	0.0	0.0	0.0	37.4
154	0.0	0.0	8.4	8.3	0.0	0.0	0.0	0.0	0.0	0.0	34.7
155	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	31.7
156	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	29.0
157	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	26.3
158	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	22.6
159	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	19.2
160	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	16.0

Table 14: Con/Span C6X30 – Existing: Change in Event Specific Incremental Elevation Exceedance

Con/Span C6X30 - Existing													
Elevation	Difference in Days Exceedance												
Feet	4-Feb-02	5-Apr-02	29-May-02	22-May-09	18-Oct-09	9-Nov-09	9-Feb-10	6-Apr-10	18-May-10	26-Mar-11	13-May-11	Max	Min
140.5	5.6	-1.0	12.1	2.8	-7.7	-0.1	4.8	6.8	4.7	5.1	9.8	12.1	-7.7
141	4.9	-1.7	10.8	0.9	-6.8	-0.2	4.0	5.6	3.9	4.1	8.6	10.8	-6.8
142	4.4	-2.2	9.4	-0.5	-4.2	-0.3	3.3	4.6	3.1	3.4	6.9	9.4	-4.2
143	3.4	-2.4	7.6	-1.5	-3.4	-0.5	2.7	3.7	2.6	3.2	5.7	7.6	-3.4
144	0.0	-1.7	4.4	-2.2	-1.6	-0.9	2.0	2.9	2.0	2.7	3.9	4.4	-2.2
145	0.0	1.5	2.7	-2.7	0.0	-0.7	1.7	2.2	1.4	2.2	2.5	2.7	-2.7
146	0.0	1.2	2.2	-2.9	0.0	1.2	1.9	2.1	1.3	2.0	2.0	2.2	-2.9
146	0.0	1.2	2.2	-2.9	0.0	1.2	1.9	2.1	1.3	2.0	2.0	2.2	-2.9
148	0.0	2.4	2.2	-1.8	0.0	0.0	5.3	5.2	2.1	2.7	1.7	5.3	-1.8
149	0.0	10.7	2.0	-0.6	0.0	0.0	6.3	7.2	3.6	3.7	1.7	10.7	-0.6
150	0.0	0.0	2.2	0.1	0.0	0.0	0.0	0.0	11.4	4.9	1.7	11.4	0.0
151	0.0	0.0	2.2	0.8	0.0	0.0	0.0	0.0	7.7	6.1	2.2	7.7	0.0
152	0.0	0.0	2.5	2.1	0.0	0.0	0.0	0.0	0.0	9.0	2.2	9.0	0.0
153	0.0	0.0	4.2	2.7	0.0	0.0	0.0	0.0	0.0	0.0	2.2	4.2	0.0
154	0.0	0.0	8.4	8.3	0.0	0.0	0.0	0.0	0.0	0.0	2.0	8.4	0.0
155	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.5	1.5	0.0
156	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.5	1.5	0.0
157	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.2	1.2	0.0
158	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.2	1.2	0.0
159	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.7	0.7	0.0
160	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.7	0.7	0.0

Table 15: Con/Span C6X30 – Owens Weir at Elevation 140 feet: Change in Event Specific Incremental Elevation Exceedance

Con/Span C6X30 - Weir at Elevation 140ft and 720 feet wide													
Elevation	Difference in Days Exceedance												
feet	4-Feb-02	5-Apr-02	29-May-02	22-May-09	18-Oct-09	9-Nov-09	9-Feb-10	6-Apr-10	18-May-10	26-Mar-11	13-May-11	Max	Min
140.5	0.2	1.0	1.0	1.1	0.0	0.2	0.9	0.1	1.0	-1.5	0.0	1.1	-1.5
141	-0.2	0.0	0.3	0.2	0.0	-0.1	0.0	-0.6	0.1	-1.9	0.0	0.3	-1.9
142	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-1.5	0.0	0.0	-1.5
143	0.0	0.0	0.3	0.0	0.0	0.0	0.0	0.0	0.1	-0.5	0.0	0.3	-0.5
144	0.0	-0.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	-0.3
145	0.0	0.0	0.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.2	0.0
146	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
146	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
148	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
149	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
150	0.0	0.0	0.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.3	0.0
151	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
152	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
153	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
154	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
155	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
156	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
157	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
158	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
159	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
160	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Annual and Growing Season Elevation Exceedance Duration

Annual and growing season elevation exceedance duration was determined at four locations for the period of record 2000-2014: Owens Lake, Owens BLH, Owens BLH2, and Melinda AR for existing conditions and with a Con/Span C6X30 at invert 140 feet through Owens Weir. See Figure 2, Figure 3, or Figure 5 for locations. The four locations were chosen based on areas that would see the most change in growing season exceedance duration at LiDAR ground elevation. See Figure 5.

Elevation exceedance duration in days and the days different from existing were calculated at one foot increments at these 4 locations. See Table 16 through Table 19 for results. The greatest increase was 4.1 days and the greatest decrease was 14.3 days in growing season duration exceedance above elevation 138 feet at Owens Lake, Owens BLH, and Owens BLH2.

The area south of the existing Melinda Weir and north of the proposed Containment Structure at EL 157 feet showed the greatest increase in inundation duration of 18 days. The Melinda AR location is directly connected to the Arkansas River in existing conditions. This area will be disconnected from the Arkansas River after the construction of the Containment Structure at EL 157 feet. This shifts the greater hydrological influence from the Arkansas River to the White River causing this increase in inundation duration.

Table 16: Annual and Growing Season Exceedance in Days at Owens Lake

Elevation	Annual Exceedance in Days at Owens Lake (based on 365 days)			Growing Season Exceedance in Days at Owens Lake (based on 246 days: 15March to 15Nov)		
feet	Existing	C157_HC145 with Owens C6X30	**Days Increase: C157_H145_ C6X30 - Exist**	Existing	C157_HC145 with Owens C6X30	**Days Increase: C157_H145_ C6X30 - Exist**
137	300.2	293.4	**-6.7**	220.2	211.6	**-8.6**
138	282.8	283.4	**0.6**	205.9	209.9	**4.0**
139	282.7	282.8	**0.1**	205.9	209.8	**3.9**
140	58.4	63.5	**5.1**	44.5	43.4	**-1.1**
141	53.6	54.5	**0.9**	40.9	38.5	**-2.4**
142	47.7	49.2	**1.5**	35.9	35.6	**-0.3**
143	42.4	44.1	**1.7**	32.0	32.5	**0.5**
144	36.8	37.4	**0.6**	27.8	27.8	**0.0**
145	30.8	32.5	**1.7**	23.7	24.6	**0.9**
146	26.0	28.0	**1.9**	20.7	21.7	**0.9**
147	22.0	25.2	**3.1**	18.4	20.0	**1.6**
148	18.3	20.9	**2.6**	16.4	17.9	**1.5**
149	14.8	17.9	**3.1**	13.5	16.0	**2.5**
150	12.6	14.7	**2.1**	11.4	13.4	**2.0**
151	10.2	12.5	**2.4**	9.2	11.4	**2.2**
152	8.1	10.0	**1.9**	7.5	9.1	**1.6**
153	5.8	8.0	**2.3**	5.8	7.4	**1.6**
154	4.2	5.6	**1.4**	4.2	5.6	**1.4**
155	3.8	4.1	**0.3**	3.8	4.1	**0.3**
156	3.3	3.6	**0.4**	3.3	3.7	**0.4**
157	1.9	2.6	**0.7**	1.9	2.6	**0.7**
158	1.4	1.5	**0.1**	1.4	1.5	**0.1**
159	1.2	1.3	**0.0**	1.2	1.3	**0.0**
160	1.0	1.1	**0.1**	1.0	1.1	**0.0**

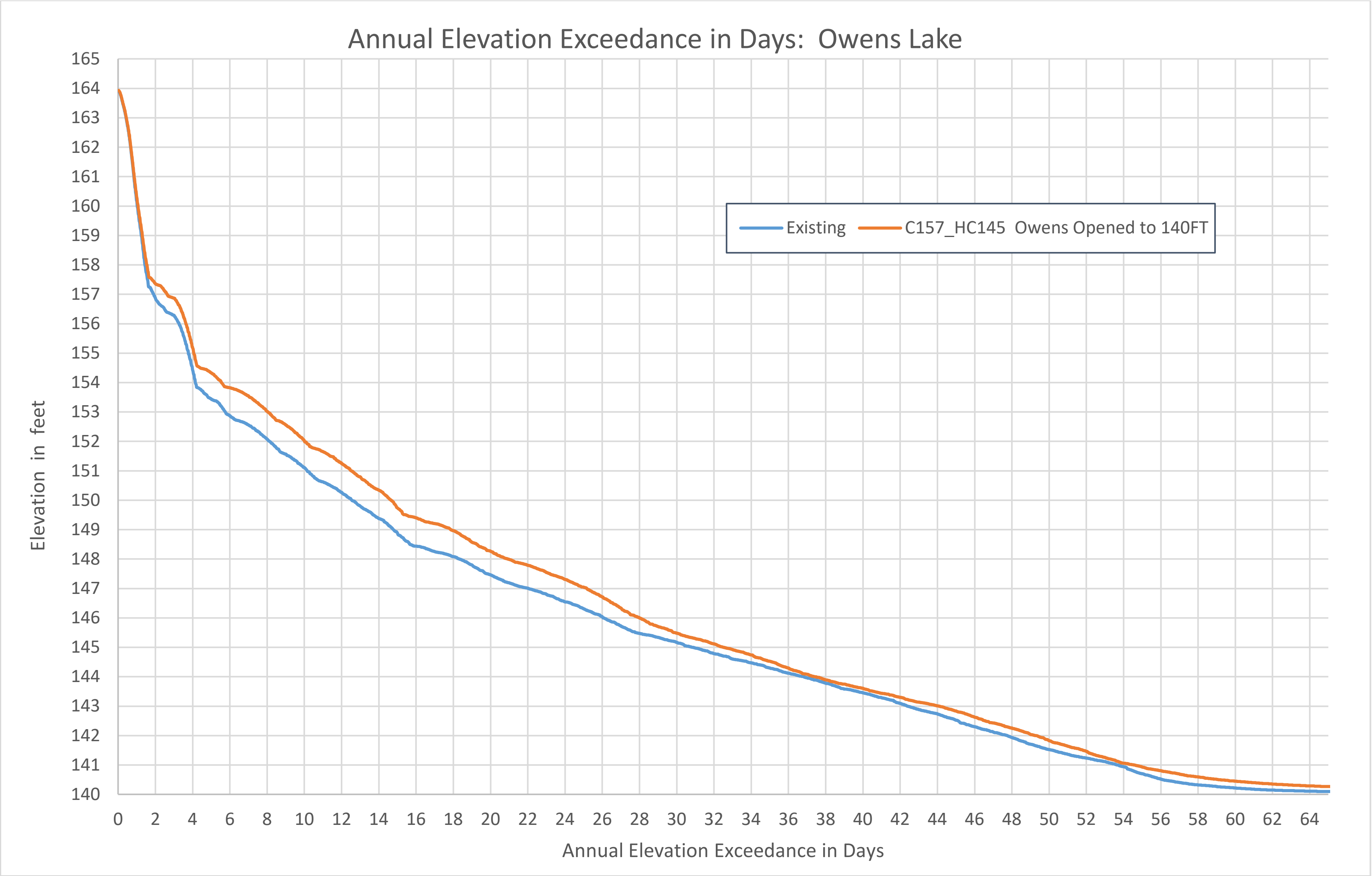


Figure 18: Annual Elevation Exceedance in Days: Owens Lake

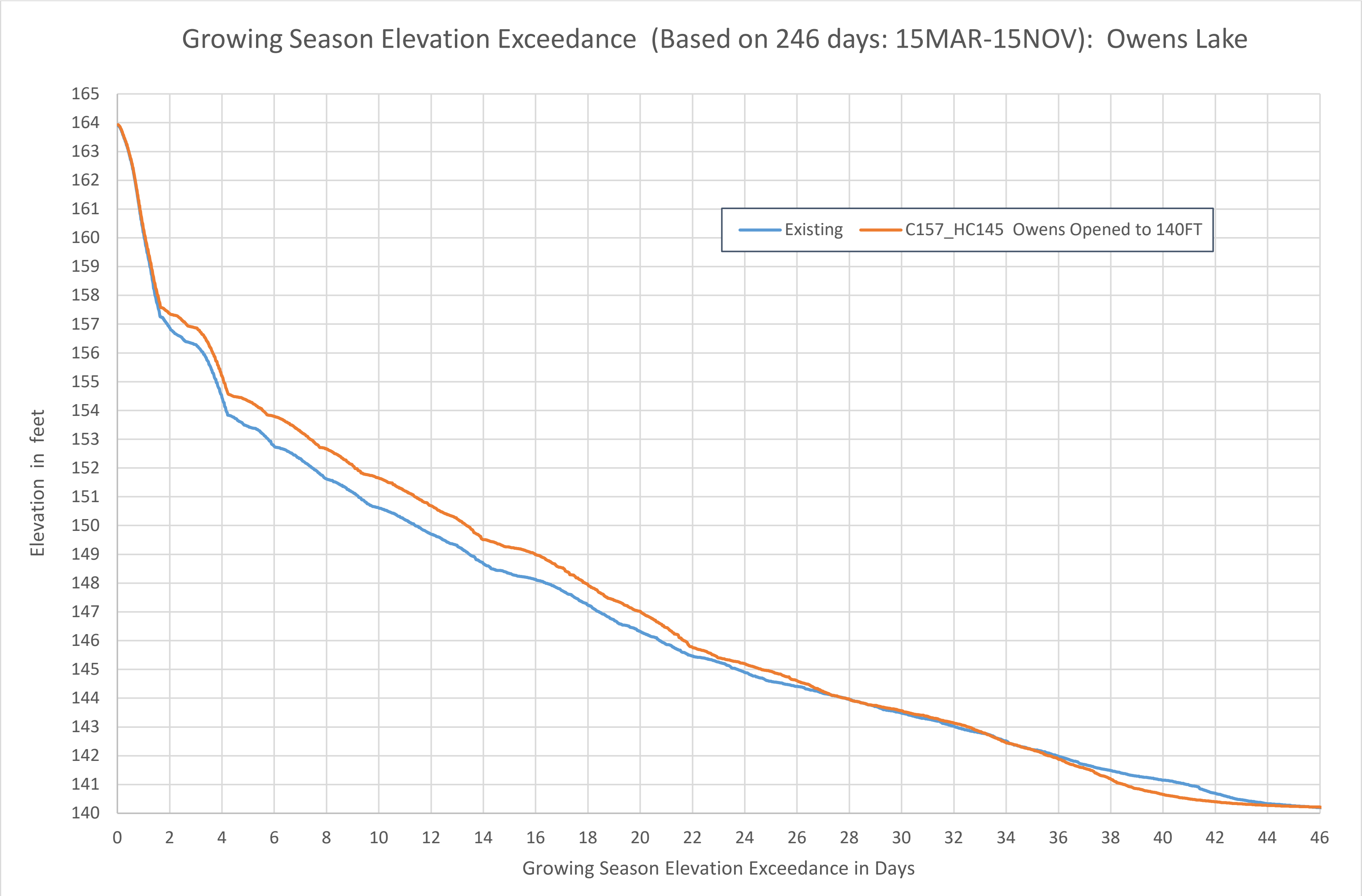


Figure 19: Growing Season Elevation Exceedance in Days: Owens Lake

Table 17: Annual and Growing Season Exceedance in Days at Owens BLH

Elevation	Annual Exceedance in Days at Owens BLH (based on 365 days)			Growing Season Exceedance in Days at Owens BLH (based on 246 days: 15March to 15Nov)		
feet	Existing	C157_HC145	**Days Increase: C157_H145_ C6X30 - Exist**	Existing	C157_HC145	**Days Increase: C157_H145_ C6X30 - Exist**
141	109.3	98.4	**-10.9**	81.3	66.9	**-14.3**
142	58.9	55.8	**-3.1**	43.5	40.1	**-3.4**
143	39.7	44.1	**4.4**	30.2	32.6	**2.3**
144	33.5	37.4	**3.9**	26.0	27.8	**1.8**
145	27.1	32.5	**5.4**	21.4	24.6	**3.3**
146	21.0	28.0	**7.0**	18.3	21.7	**3.3**
147	17.7	25.2	**7.5**	15.9	20.0	**4.1**
148	15.3	20.9	**5.6**	14.0	17.9	**3.9**
149	13.7	17.9	**4.2**	12.4	16.0	**3.6**
150	11.8	14.7	**2.9**	10.7	13.4	**2.7**
151	9.8	12.5	**2.7**	8.9	11.4	**2.5**
152	7.9	10.0	**2.1**	7.2	9.1	**1.8**
153	5.6	8.0	**2.4**	5.7	7.4	**1.7**
154	4.2	5.6	**1.4**	4.2	5.6	**1.4**
155	3.8	4.1	**0.3**	3.8	4.1	**0.3**
156	3.2	3.6	**0.4**	3.2	3.7	**0.4**
157	1.9	2.6	**0.7**	1.9	2.6	**0.8**
158	1.4	1.5	**0.1**	1.4	1.5	**0.1**
159	1.2	1.3	**0.0**	1.2	1.3	**0.0**
160	1.0	1.1	**0.0**	1.0	1.1	**0.0**

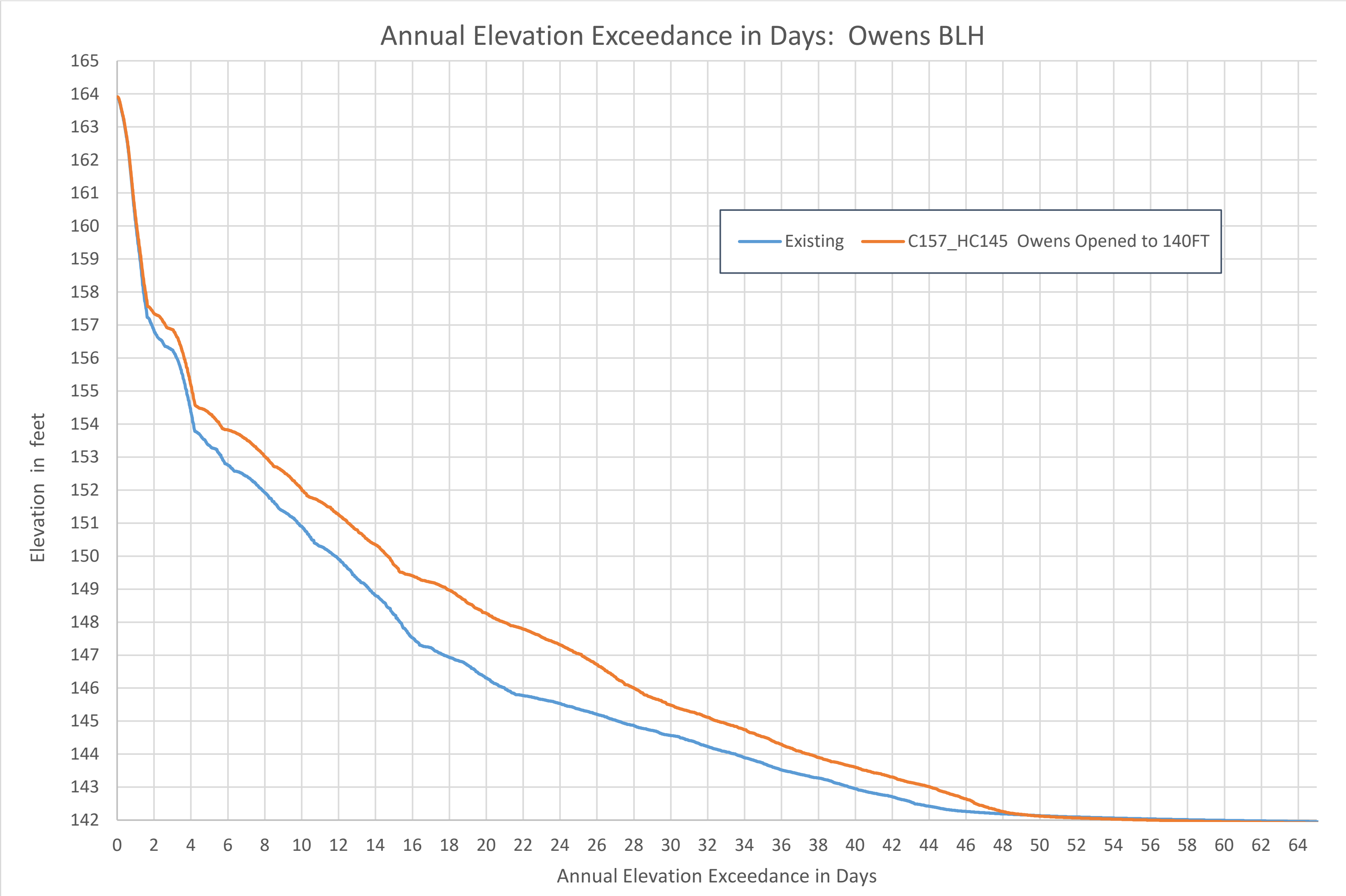


Figure 20: Annual Elevation Exceedance in Days: Owens BLH

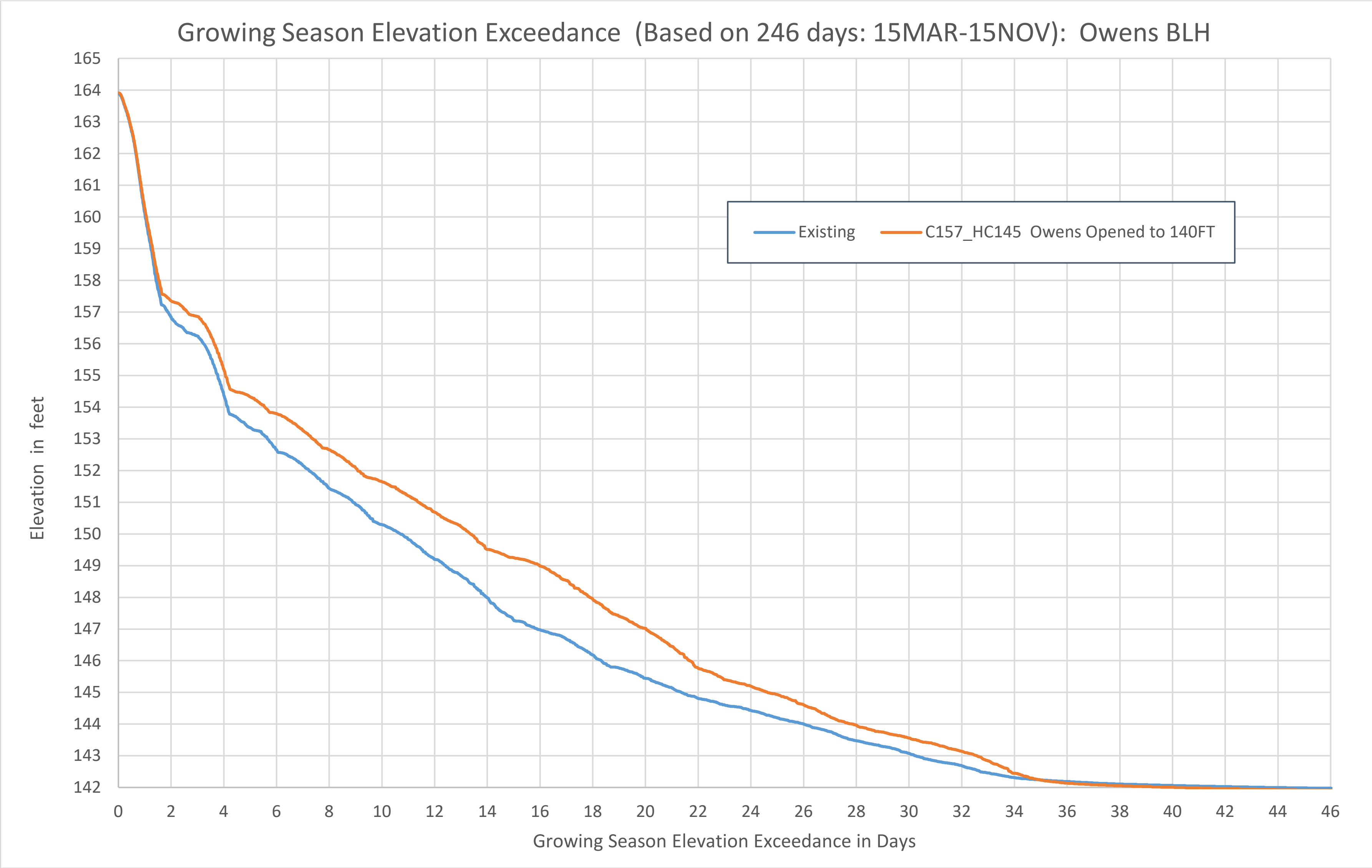


Figure 21: Growing Season Elevation Exceedance in Days: Owens BLH

Table 18: Annual and Growing Season Exceedance in Days at Owens BLH2

Elevation	Annual Exceedance in Days at BLH2 (based on 365 days)			Growing Season Exceedance in Days at BLH2 (based on 246 days: 15March to 15Nov)		
feet	Existing	C157_HC145	**Days Increase: C157_H145_ C6X30 - Exist**	Existing	C157_HC145	**Days Increase: C157_H145_ C6X30 - Exist**
142	272.0	274.7	**2.6**	205.2	209.2	**4.0**
143	53.7	48.9	**-4.8**	40.0	36.2	**-3.8**
144	40.2	37.2	**-2.9**	30.3	27.7	**-2.6**
145	31.9	32.8	**0.9**	24.3	24.9	**0.6**
146	26.5	28.2	**1.7**	21.0	21.8	**0.7**
147	21.3	25.3	**3.9**	18.3	20.1	**1.8**
148	17.9	21.0	**3.1**	16.1	17.9	**1.8**
149	14.7	17.9	**3.2**	13.4	16.0	**2.6**
150	12.2	14.7	**2.5**	11.0	13.4	**2.3**
151	9.7	12.5	**2.8**	8.8	11.4	**2.6**
152	7.7	10.0	**2.4**	7.1	9.1	**2.1**
153	5.6	8.1	**2.5**	5.6	7.4	**1.8**
154	4.1	5.6	**1.4**	4.1	5.6	**1.4**
155	3.8	4.1	**0.3**	3.8	4.1	**0.3**
156	3.2	3.7	**0.4**	3.2	3.7	**0.4**
157	1.9	2.7	**0.8**	1.9	2.7	**0.8**
158	1.4	1.6	**0.1**	1.4	1.6	**0.1**
159	1.2	1.3	**0.1**	1.2	1.3	**0.1**
160	1.0	1.1	**0.1**	1.0	1.1	**0.1**
161	0.8	0.9	**0.0**	0.9	0.9	**0.0**
162	0.7	0.7	**0.0**	0.7	0.7	**0.0**

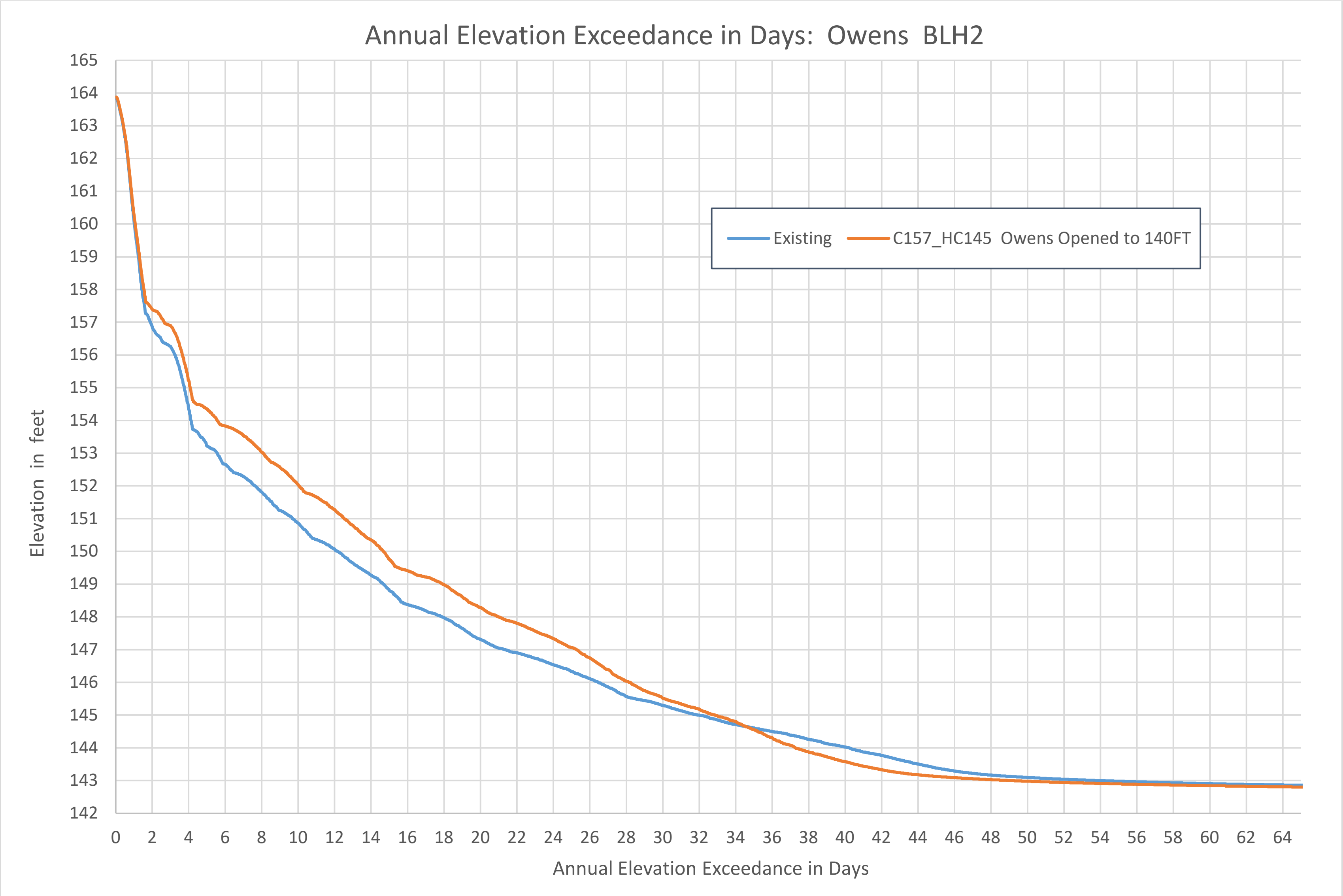


Figure 22: Annual Elevation Exceedance in Days: Owens BLH2

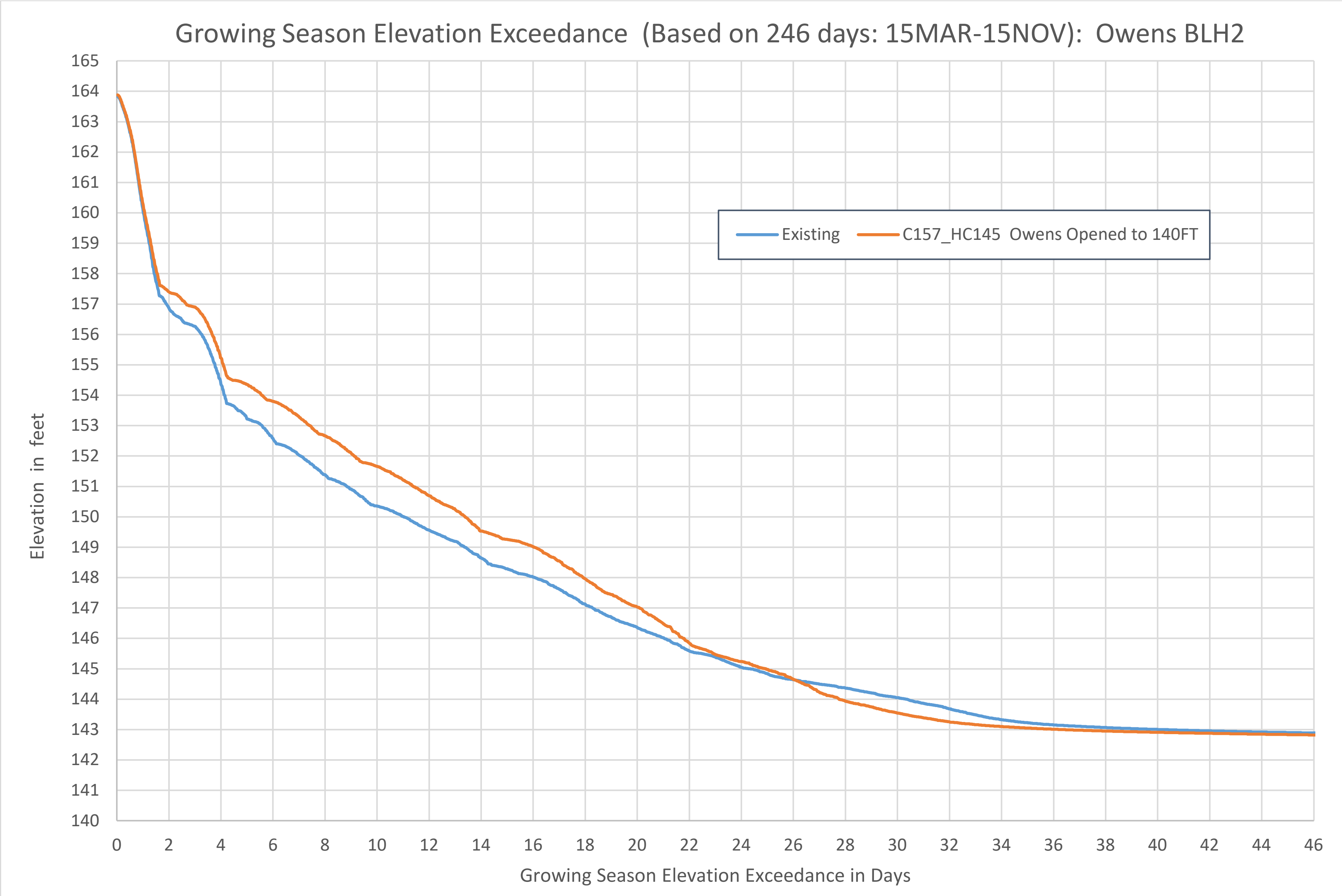


Figure 23: Growing Season Elevation Exceedance in Days: Owens BLH2

Table 19: Annual and Growing Season Exceedance in Days Melinda AR

Elevation	Annual Exceedance in Days South of Melinda Weir (based on 365 days)			Growing Season Exceedance in Days South of Melinda Weir (based on 246 days: 15March to 15Nov)		
feet	Existing	C157_HC145	**Days Increase: C157_H145_ C6X30 - Exist**	Existing	C157_HC145	**Days Increase: C157_H145_ C6X30 - Exist**
115	344.7	288.2	**-56.5**	230.2	211.2	**-19.0**
120	302.6	288.0	**-14.7**	199.5	211.2	**11.7**
140	52.8	88.5	**35.7**	40.8	59.0	**18.2**
141	46.8	54.6	**7.8**	37.2	38.5	**1.4**
142	38.5	49.3	**10.7**	31.2	35.6	**4.4**
143	31.6	44.1	**12.6**	26.2	32.6	**6.4**
144	24.3	37.5	**13.2**	20.5	27.9	**7.4**
145	18.8	32.5	**13.8**	16.1	24.6	**8.6**
146	14.3	28.0	**13.7**	12.8	21.7	**8.8**
147	11.7	25.2	**13.5**	10.4	20.0	**9.6**
148	10.1	21.0	**10.8**	9.0	17.9	**8.9**
149	8.6	17.9	**9.3**	7.7	16.0	**8.3**
150	7.3	14.7	**7.4**	6.9	13.4	**6.5**
151	6.1	12.5	**6.5**	6.0	11.4	**5.4**
152	5.1	10.0	**4.9**	5.1	9.1	**3.9**
153	4.1	8.0	**3.9**	4.1	7.4	**3.2**
154	3.8	5.6	**1.8**	3.8	5.6	**1.8**
155	3.4	4.1	**0.6**	3.4	4.1	**0.6**
156	2.2	3.6	**1.4**	2.2	3.6	**1.4**
157	1.6	2.6	**1.0**	1.6	2.6	**1.0**
158	1.3	1.5	**0.2**	1.3	1.5	**0.2**

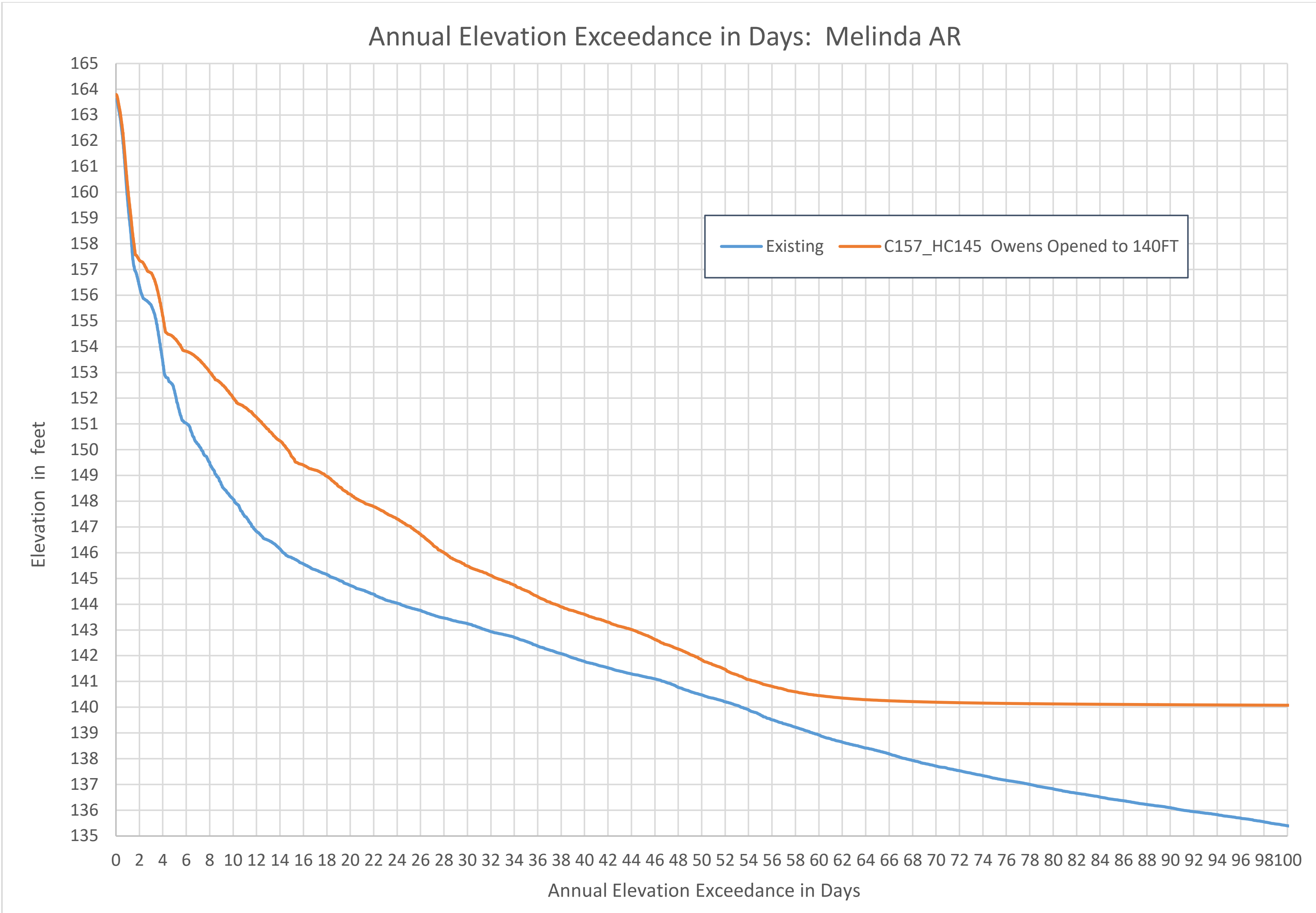


Figure 24: Annual Elevation Exceedance in Days: Melinda AR

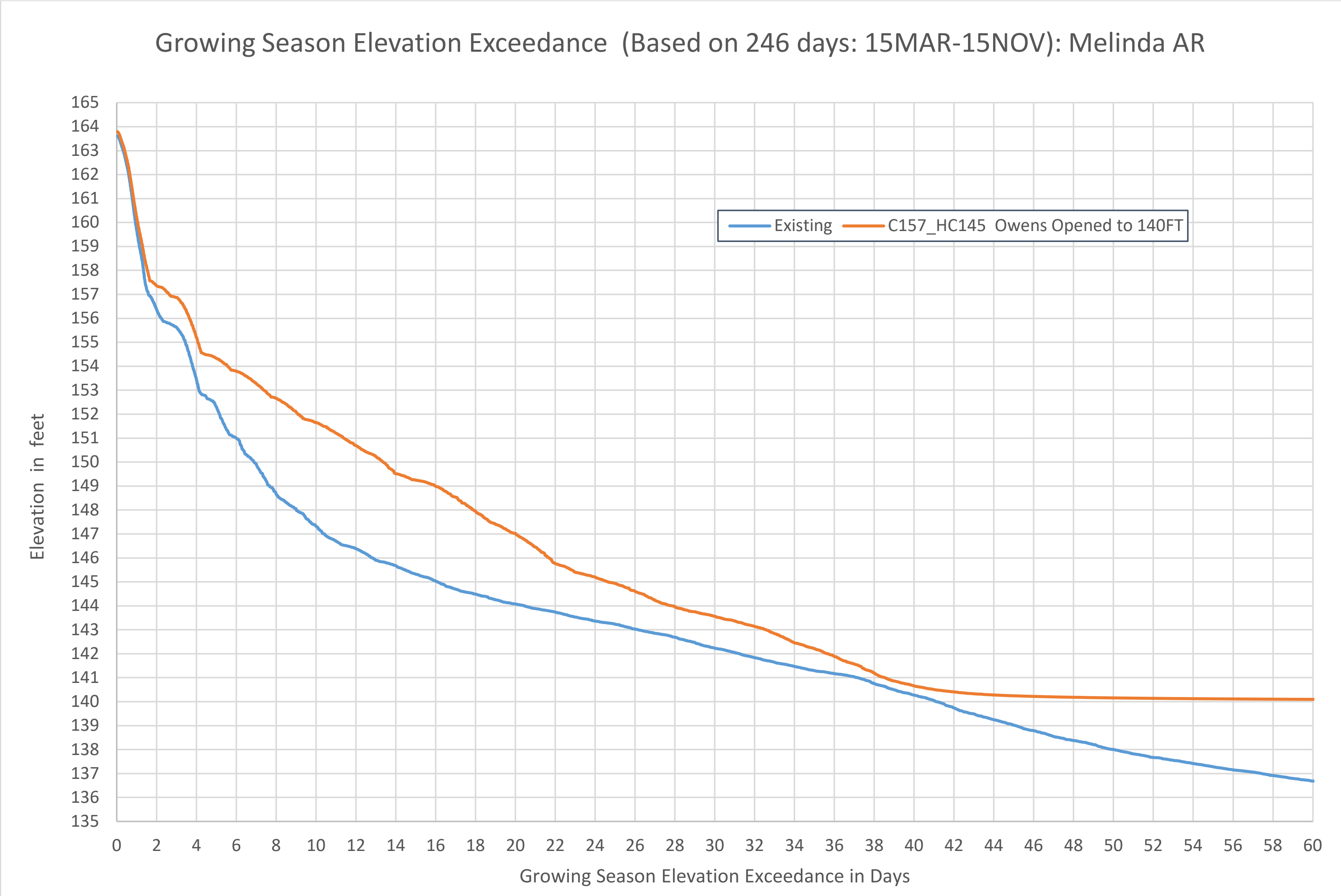


Figure 25: Growing Season Elevation Exceedance in Days: Melinda AR

Table 20: Summary Exceedance Duration: **Owens Lake**

Difference in Elevation Exceedance Duration At Owens Lake: C6X30 - Existing				
Elevation Feet	11 Flood Events		Annual POR 2000-2014	Growing Season POR 2000-2014
	Maximum Days	Minimum Days	Days	Days
140	-	-	5.1	-1.1
140.5	12.1	-7.7	-	-
141	10.8	-6.8	0.9	-2.4
142	9.4	-4.2	1.5	-0.3
143	7.6	-3.4	1.7	0.5
144	4.4	-2.2	0.6	0.0
145	2.7	-2.7	1.7	0.9
146	2.2	-2.9	1.9	0.9
147	3.5	-2.8	3.1	1.6
148	5.3	-1.8	2.6	1.5
149	10.7	-0.6	3.1	2.5
150	11.4	0.0	2.1	2.0
151	7.7	0.0	2.4	2.2
152	9.0	0.0	1.9	1.6
153	4.2	0.0	2.3	1.6
154	8.4	0.0	1.4	1.4
155	1.5	0.0	0.3	0.3
156	1.5	0.0	0.4	0.4
157	1.2	0.0	0.7	0.7
158	1.2	0.0	0.1	0.1
159	0.7	0.0	0.0	0.0
160	0.7	0.0	0.1	0.0

ATTACHMENT G Arkansas – White Rivers Preliminary Geomorphic Assessment Final Report: August 2003

Arkansas – White Rivers Preliminary Geomorphic Assessment
Final Report

By

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August 2003

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1. Study Objective

The objective of this study was to conduct a detailed preliminary geomorphic assessment of the Arkansas-White Rivers Study area. This area includes the lower Arkansas River from Dam 2 to its mouth and to a lesser extent, the lower White River from Lock 1 to its mouth. The geomorphic assessment provides the process-based framework to define past and present channel and watershed dynamics, develop integrated solutions, and assess the consequences of remedial actions such as bank stabilization or other structural modifications within the system.

2. Introduction

The lower Arkansas and White Rivers is a complex hydrologic and hydraulic system that has been greatly influenced by man. The stages in this area are highly dependent upon backwater from the Mississippi River. This area experiences periods during low stages when flow does not interchange between the Arkansas and White Rivers. However, during periods of higher stages, flow can be from the White River to the Arkansas River or from the Arkansas River to the White River.

To develop an understanding of the geomorphic conditions for the area, a detailed study plan was required. This plan included data assembly (stage, discharge, geology, land use changes, aerial photography, mapping, surveys), field reconnaissance, hydrologic analysis, channel morphology (planform, profile, geometry) analysis, sediment analysis, and determination of the impacts of man (construction timeline). The final step in the assessment was integrating the information developed from the various analyses to accurately understand the past and present geomorphic conditions of the study area and to serve as the framework for estimating future geomorphic changes.

Data assembly. The first step in the geomorphic assessment was the gathering and compilation of existing data. The use of historical data enables the identification of trends and provides useful information on rates of change in the study area. The type of data gathered included hydrologic records, sediment data, hydraulic data, construction records, aerial photography, satellite imagery, and other mapping, channel surveys, and geologic data. During the data assembly, the investigators develop familiarity with the system, which is helpful for the development of the field investigation effort.

Field Reconnaissance. A detailed field reconnaissance of the watershed is always extremely important in assessing channel stability because the physical characteristics of the stream are indicators of the dominant geomorphic processes occurring within the basin. Two detailed field investigations of the entire study area were conducted by both land and water. These field investigations involved documenting the status of existing structures, location of problem areas, sediment source and sink areas, vegetative patterns, and significant morphologic features. Sediment samples from the bed, banks, and floodplain were collected and sent to the laboratory for gradation analyses.

Analysis. The analysis portion of the geomorphic assessment involves analyzing the data collected during the data assembly and field investigations. The various analyses included:

Hydrologic analysis. Gage records were analyzed to determine trends, major changes, flow duration, and relative stages between the two systems.

Sediment analysis. Limited sediment data was available for the study area. A grain size analysis of the bed and bank material collected in the field was conducted.

Historic planform analysis. Historical aerial photographs, satellite imagery, and mapping was analyzed to document planform changes through time, and to project future conditions.

Profile analysis. A spatial and temporal analysis of the profile data was conducted. This included the available 1917, 1940, and 1960's hydrographic survey data.

Land use changes. An analysis of historical mapping was conducted to identify any significant changes in land use that would affect the channel system.

Historic timeline. A timeline of all major construction features and natural developments was developed and correlated with observed channel changes to determine any cause and effect relationships.

Integration. The final part of any geomorphic assessment of a channel system includes the integration of the information from all the available analyses. Integration is important because often, individual analyses yield conflicting results. For instance, profile comparisons may indicate a trend of degradation that is not reflected in the gage data. In cases like this, a level of confidence must be assigned to the various components based on the reliability and availability of the data, and the investigators own experience with each tool in order to reconcile any contradictory results.

3. Description of Study Area

The study reach generally included the lower 30 miles of the Arkansas River from Dam 2 to its mouth and the lower ten miles of the White River from Lock 1 to its mouth. Figure 3-1 is a map of the study area. Primary attention was given to the Arkansas River including Camp Bend and House Bend. In our research, no name was found for the right descending bank of the Arkansas River located immediately downstream of the Yancopin Bridge. Therefore for this assessment, this bank has been designated as Camp Bend since a hunting camp is located on top bank.

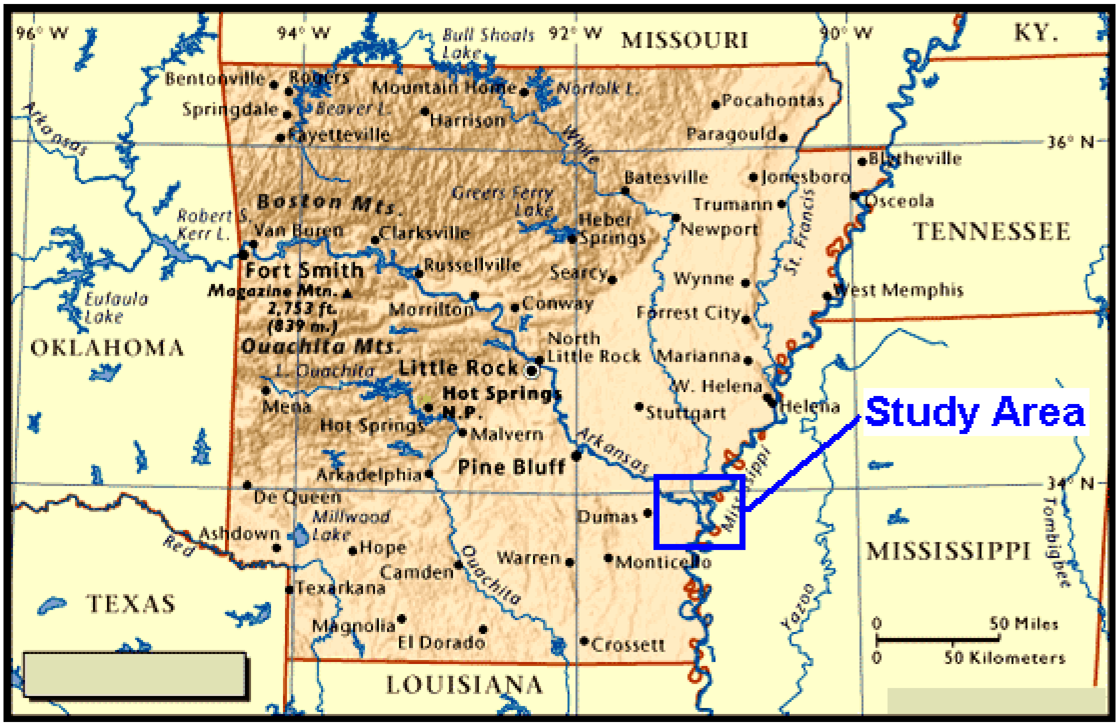

Figure 3-1 Study Area Location Map

4. Data

4.1 Stage and Discharge.

For this assessment, both measured and computed stage and discharge data were used. Stage data was collected for gaging stations on the lower Arkansas River, lower White River, and the Mississippi River. Figure 4-1 is a map that shows the gage locations. Table 4-1 identifies the gages used for the assessment and provides some pertinent data for each gage. As Table 4-1 shows, stage data for the Arkansas River at the Melinda outlet and at its mouth and the White River at Owens Lake were computed by interpolation between recording gages. The Arkansas River at its mouth (River Mile 581) data was interpolated between the Helena gage (River Mile 663) and Arkansas City gage (River Mile 554) on the Mississippi River and between the MW5 gage (River Mile 599) and the Arkansas City gage. The MW5 gage is located at the mouth of the White River. The White River at Owens Lake was interpolated between the Lock 1 tailwater and MW5 stages. The stage data for the Arkansas River at Melinda outlet were interpolated between the Yancopin stages and the stages computed for the Arkansas River at its mouth. The determination of river stages by straight line interpolation between gages can result in error. The farther the gages are apart, the greater the potential for error. As a check, the mouth of the Arkansas River stages generated by

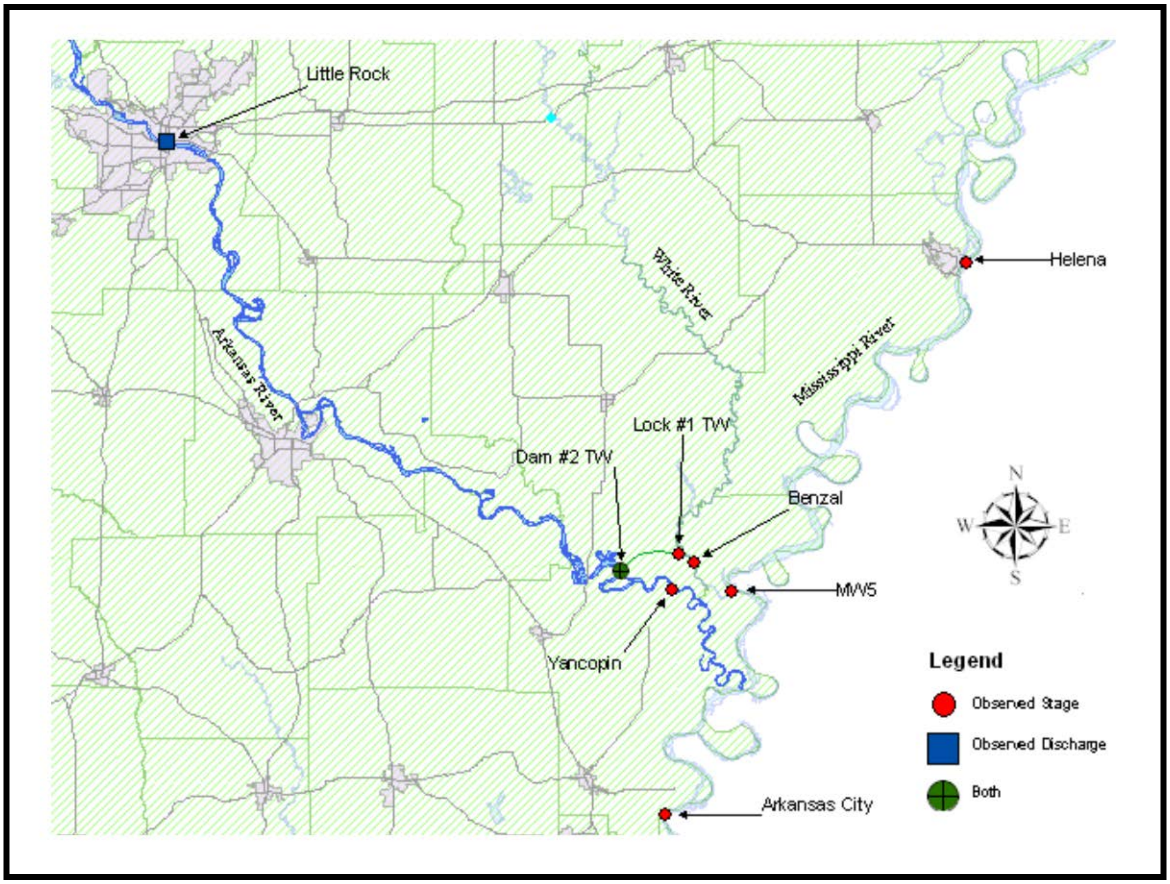

Figure 4-1 Gage Location Map

interpolating between Arkansas City and Rosedale (Mississippi River Mile 592) were compared to those computed by interpolating between Arkansas City and MW5 and to those interpolated between Arkansas City and Helena for the period when all 4 gages were in operation (1969 – 2001). The results showed that the stages computed at the mouth of the Arkansas River by interpolating between Arkansas City and Rosedale, MW5, and Helena compared very well, with the difference being usually less than 1 foot. In only 2 of those 33 years was the comparison less than extremely good. In 1995, the maximum differences were from 2 to 3 feet. In 1999, the maximum differences were from 3 to 3.5 feet. This comparison shows that interpolating between Arkansas City and the other 3 gages produced very consistent results. The distance between the gages in this instance had little impact on the interpolated stages. This does not say that error is not introduced by interpolation, only that the interpolated stages were consistent.

In reviewing the stage data at Yancopin, an interesting discovery was made. Prior to Dam 2 going into operation, the daily low flow stage fluctuations at Yancopin were reasonably small as would be expected on a large, alluvial river like the Arkansas River. However, subsequent to Dam 2, the low flow stages showed much greater daily variability, sometimes rising or falling as much as 8 to 10 feet. The tailwater stage records at Dam 2 also show these large fluctuations. Therefore, the operation of the

Table 4-1
Pertinent Stage Data

Location	Period of Record	Comments
Mississippi River		
Arkansas City	January 1932 – December 2001	
Helena	January 1930 – December 2001	
Arkansas River		
Dam 2 Tailwater	January 1969 – December 2002	
Yancopin	January 1932 – December 2001	Missing Data Estimated
Melinda Outlet	January 1960 – December 1998	Computed by interpolation from Yancopin and Mouth of Arkansas River stages
Mouth	January 1932 – December 1999 January 1970 – December 2001	Computed by interpolation from Arkansas City and Helena stages Computed by interpolation from Arkansas City and MW5 stages
White River		
MW5	January 1962 – December 2000	Mouth of White River
Benzal	January 1932 – December 1998	Missing Data Estimated
Lock 1 Tailwater	January 1969 – December 2002	
Owens Lake	January 1969 – December 2000	Computed by interpolation between Lock 1 Tailwater and MW5 stages

McClellan-Kerr Arkansas River Navigation System (MCKARNS) is having an obvious impact on the low flow stages downstream of Dam 2. Figure 4-2 is a plot of the daily stages at Yancopin and at the mouth of the Arkansas River for September and October 1966. This plot shows the small daily stage fluctuations that were typical during low flow periods prior to Dam 2. Figure 4-3 is a plot of the daily stages at Yancopin and at the mouth of the Arkansas River from the middle of September 1997 through the middle of November 1997. This is a typical low flow period subsequent to putting Dam 2 into operation. The plot shows the large daily fluctuations in stage on the Arkansas River at Yancopin. However, these large fluctuations are not seen at the mouth of the Arkansas River. Therefore, the large fluctuations at Yancopin have little if any impact during these low water periods on the Mississippi River stages. Fluctuations of this magnitude can and often do contribute to bank instability.

The discharge data used for the assessment was limited to historic releases from Dam 2 and computed discharges at the Dam 2 and Yancopin sites based on lagged observed Little Rock discharges. Dam 2 discharges were available for the January 1969 through October 2002 period. The discharge data at Little Rock was obtained from the USGS.

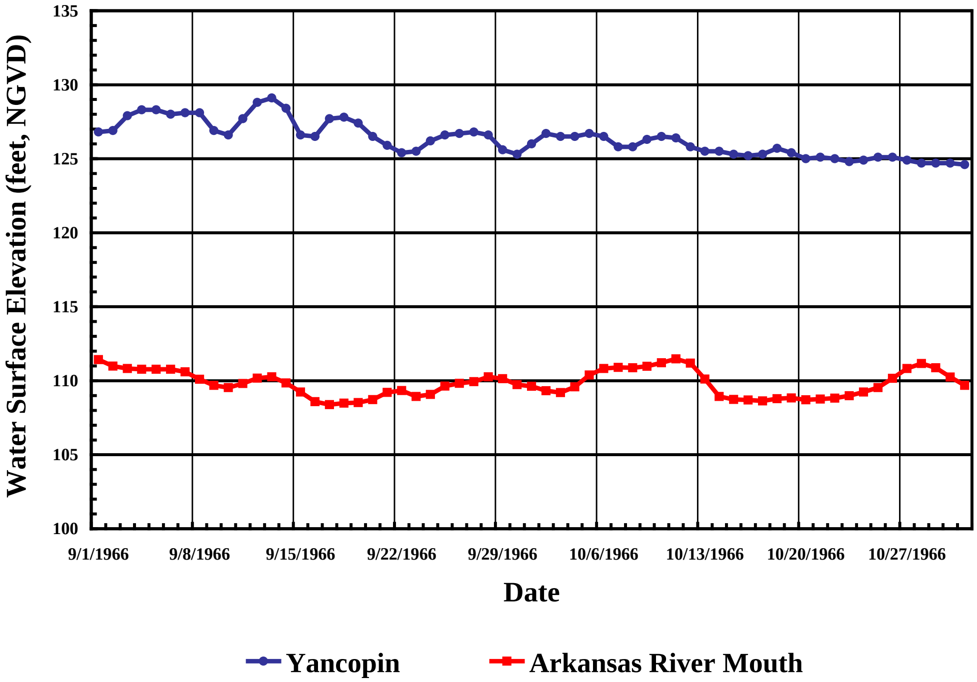

Figure 4-2. Comparison of Observed Stage at Yancopin and Computed Stage at the Mouth of Arkansas River Prior to Dam 2 Operation

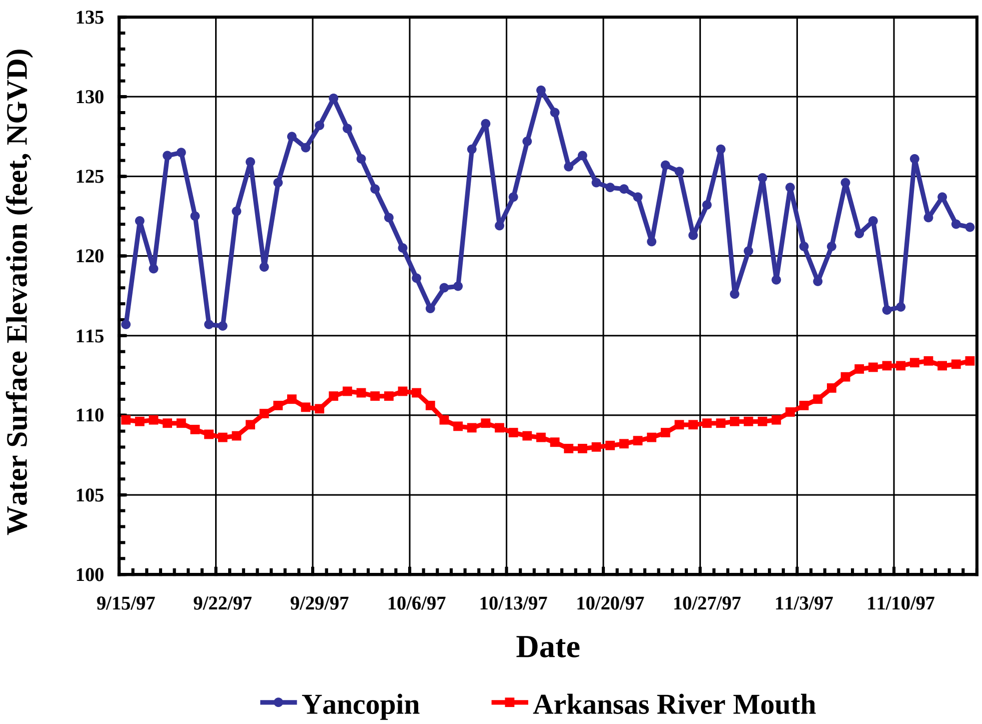

Figure 4-3. Comparison of Observed Stage at Yancopin and Computed Stage at the Mouth of Arkansas River Subsequent to Dam 2 Operation

These discharges cover the January 1932 through December 1970 period. The Little Rock discharges were lagged 1 day to account for the travel time between Little Rock and both Dam 2 and Yancopin. Figure 4-4 is a plot that compares the Dam 2 releases and the lagged Little Rock discharges for the July 1969 through October 1970 time period. The plot is limited to this period because this was the only time when both Little Rock and Dam 2 discharge data were available. The plot shows a good comparison between the discharges during low flow periods. The lagging of the Little Rock flows cannot account for the tributary flow that enters the Arkansas River between Little Rock and Dam 2. The higher the tributary flow, the greater is the difference between the Dam 2 releases and the lagged Little Rock discharges. However, especially for the lower flow periods, the lagged Little Rock discharges present a reasonable approximation of the discharges for the Dam 2 site and for Yancopin.

4.2 Channel Geometry.

Part of the data collection for the geomorphic assessment included a search of the USACE, Vicksburg District and the USACE, Mississippi Valley Division files for historic channel geometry data. This search resulted in the locating of two historic comprehensive hydrographic surveys on the lower Arkansas River. These surveys were conducted in 1917 and 1940. Also, the USACE, Little Rock District furnished a HEC-RAS (River Analysis System) model. The channel geometry contained in this model was derived from an existing UNET (unsteady flow) model. The date of the survey used to develop the UNET model channel geometry is unknown but Little Rock District believes the survey was most likely conducted during the 1960's. Both the 1940 and 1960's surveys provided channel bed elevations. The 1917 survey provided water depths instead of bed elevations. Therefore, these depths were converted to elevations by subtracting the depth shown on the surveys from the known water surface elevation on the day that the surveys were conducted.

4.3 Aerial Photography, Satellite Imagery, and Mapping.

The files at the Vicksburg District and the Mississippi Valley Division were searched for available aerial photography, satellite imagery, and mapping on both the lower Arkansas River and White River. As a result, the photography, imagery, and mapping identified in Table 4-2 were acquired. Most of the available data were collected during low stage periods. The stage at Yancopin on the date that the aerial mosaics and satellite imagery was taken is provided. If the exact water surface elevation was not known, an approximate elevation is given.

4.4 Sediment.

Very little sediment data is available for the study reach. Therefore, as part of the field investigation, both bed and bank samples were collected on the lower Arkansas and

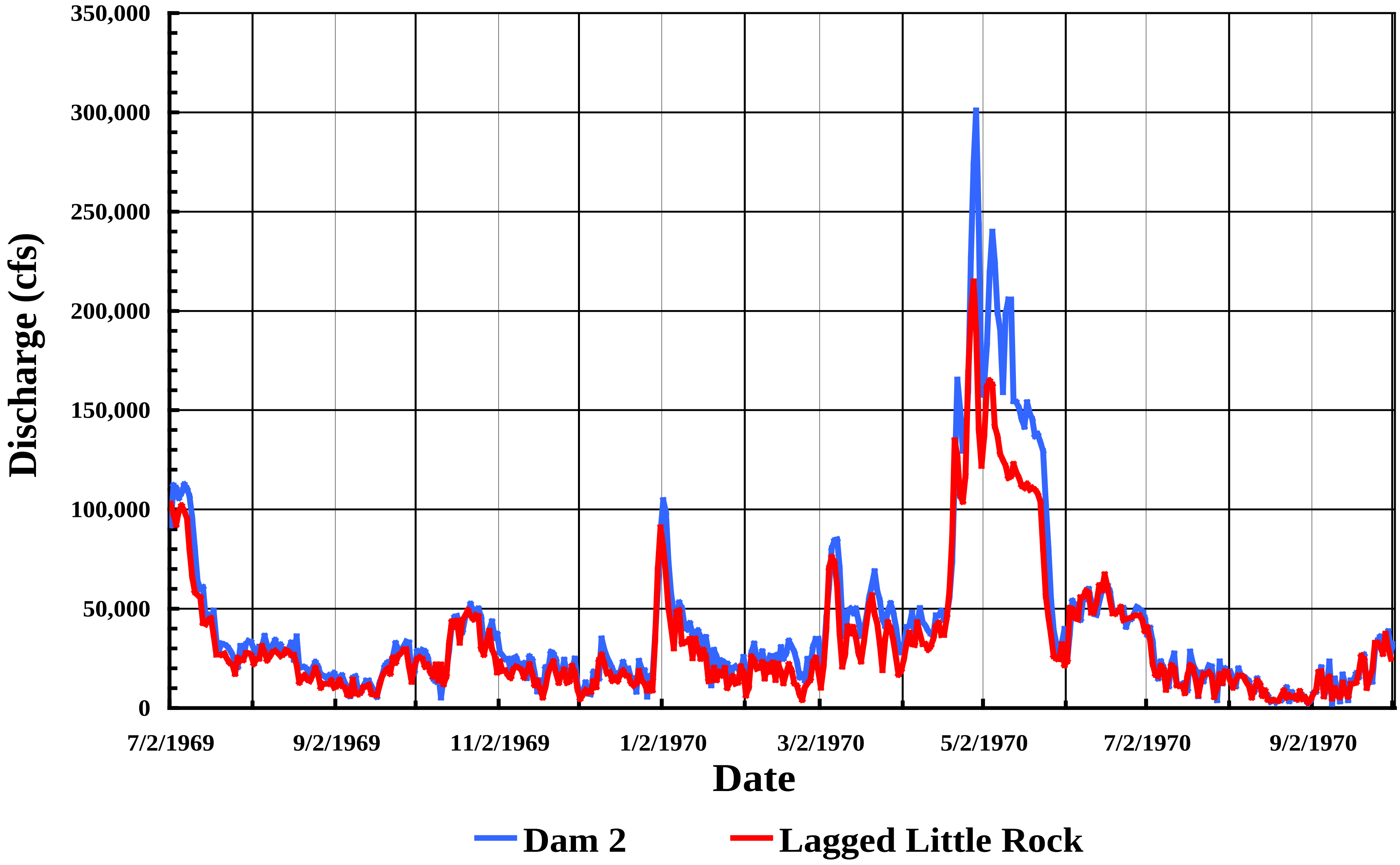

Figure 4-4 Comparison of Dam 2 Releases and Little Rock Discharges

Table 4-2
Aerial Photography, Satellite Imagery, and Mapping

Item	Date	Comments
Aerial Mosaics	September 1999	Yancopin WSE ≈ 121 feet, NGVD
Satellite Imagery	August 1999	Yancopin WSE = 127.7 feet, NGVD
Satellite Imagery	May 1997	Yancopin WSE = 138.1 feet, NGVD
Aerial Mosaics	September 1994	Yancopin WSE ≈ 118 feet, NGVD
Aerial Mosaics	October 1991	Yancopin WSE ≈ 116.7 feet, NGVD
Satellite Imagery	February 1991	Yancopin WSE = 138.6 feet, NGVD
Aerial Mosaics	March/April 1988	Yancopin WSE ≈ 143 feet, NGVD
Satellite Imagery	December 1987	Yancopin WSE = 132.3 feet, NGVD No coverage at mouth of Arkansas River
Aerial Mosaics	September 1985	Yancopin WSE = 123.5 feet, NGVD
Satellite Imagery	January 1983	Yancopin WSE = 142.2 feet, NGVD
Aerial Mosaics	October 1980	Yancopin WSE = 120.0 feet, NGVD
Aerial Mosaics	October 1976	Yancopin WSE = 119.5 feet, NGVD
Aerial Mosaics	November 1949	
Mapping	1940	
Mapping	1930-1932	Arkansas River from historic cutoff to mouth White River from historic cutoff to mouth
Mapping	October 1917	
Mapping	1820-1830	Arkansas River from historic cutoff to mouth White River from historic cutoff to mouth

White Rivers. On the Arkansas River, samples were collected from 3 sites. These sites are identified as Emerson Bend, Melinda (House Bend), and Camp Bend. Emerson Bend is located between the historic cutoff and the mouth. The samples at Melinda, except for the overbank sample were collected at House Bend adjacent to Jim Smith Lake. The overbank sample was collected near the Melinda Headcut Structure. Camp Bend is located along the right descending bank between the Yancopin Bridge and House Bend Of the samples collected on the Arkansas River, two were bed samples, one was a point bar sample, four were bank samples, and one was an overbank sample. Of the four bank samples, two were collected at Melinda (House Bend) and two at Camp Bend. At both sites, an upper and lower bank sample was collected. For the grain size distribution analyses, the upper and lower samples at each site were combined to form a single grain size distribution. On the White River, only one site was sampled. This site is located immediately downstream of the Owens Revetment. At this site, one bed sample, one point bar sample, and one bank sample were collected. Figure 4-5 identifies the sediment sampling sites. The samples were sent to the lab where a grain size distribution was determined. Table 4-3 provides the pertinent data for the samples. Figure 4-6 is the grain size distribution plot.

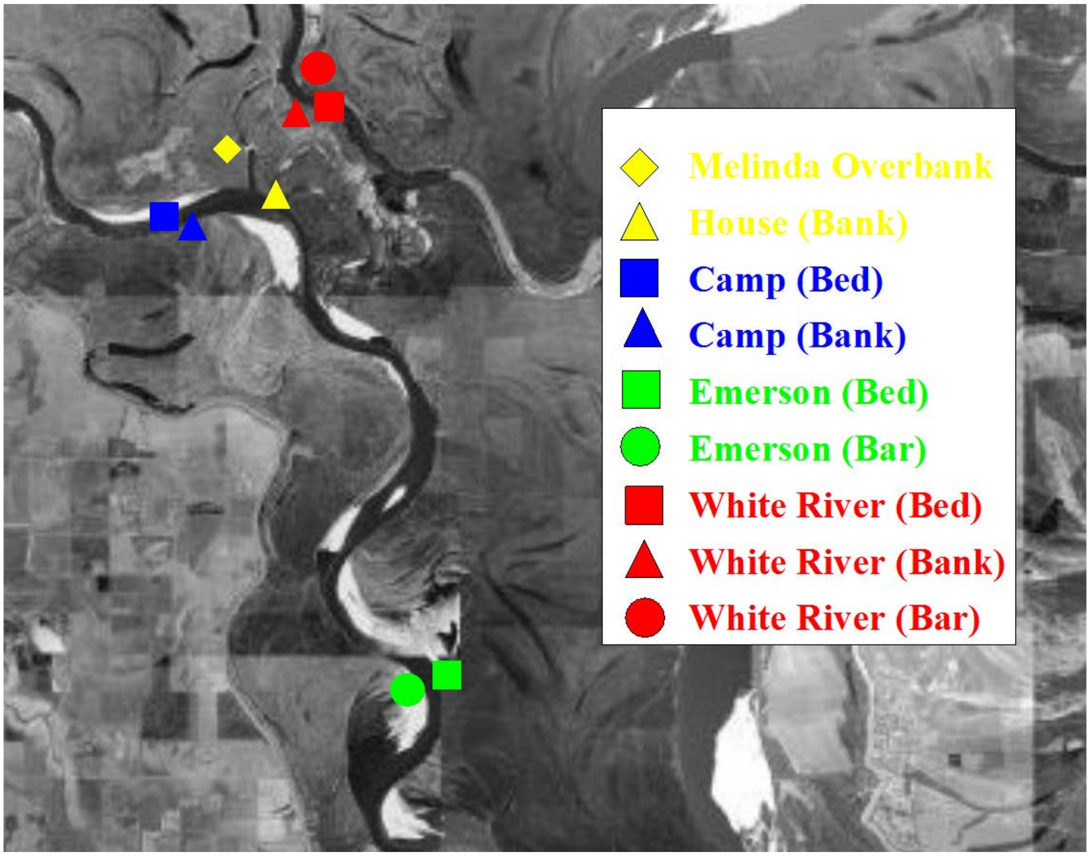

Figure 4-5 Sediment Sampling Sites

4.5 Historic Timeline

Both natural and man induced factors impact channel morphology. The more that is known about these factors, the better the chance of understanding past and present morphology as well as of predicting future morphology. Therefore, a detailed timeline provides the basis for determine the impacts of natural and man induced factors. A historic timeline for the study area is provided in Table 4-4.

5. Methodology

5.1 Specific Gage Analysis

One of the more useful tools used to indicate channel change is the specific gage analysis (Blench, 1966). The specific gage analysis is developed from stage-discharge data and is a plot of river stage or water surface elevation for a specified discharge versus time. A specific gage record with an increasing trend over time indicates that channel aggradation has occurred, while a decreasing trend would indicate degradation.

Table 4-3
Pertinent Sediment Data

Sample	Location	D_{90} (mm)	D_{50} (mm)	D_{10} (mm)	Comments
Arkansas River					
Emerson Bend	Bed	0.48	0.35	0.24	D_{50} = medium sand
	Point Bar (RDB)	0.47	0.34	0.23	D_{50} = medium sand
Melinda	Combined Upper and Lower Bank Samples (LDB)	0.14	0.075	Finer Than Sand	D_{50} = very fine sand At House Bend (John Smith Lake)
	Overbank	0.47	0.33	0.16	D_{50} = medium sand Near Melinda Structure
Camp Bend	Bed	0.88	0.41	0.16	D_{50} = medium sand
	Combined Upper and Lower Bank Samples (RDB)	0.16	Finer Than Sand	Finer Than Sand	D_{50} = finer than sand
White River					
RM4	Bed	0.39	0.18	Finer Than Sand	D_{50} = fine sand Near River Mile 4
	RDB	Finer Than Sand	Finer Than Sand	Finer Than Sand	D_{50} = finer than sand
	Point Bar	0.25	0.17	Finer Than Sand	D_{50} = fine sand

Within the study reach, a specific gage record was developed for the Arkansas River at Yancopin. No specific gage record was developed for the White River due to the lack of discharge data for the study reach. The specific gage record at Yancopin was developed for low flow conditions when there was no backwater influence from the Mississippi River. The first step in developing the specific gage record was to plot Yancopin stage, Arkansas City stage, and Dam 2 discharge for each year of record (1932-2001), and to visually determine the period of time during each year when Mississippi River stages were low enough to cause no backwater influence. This period was typically during the fall of the year. Due to the short distance between Dam 2 to Yancopin, Dam 2 discharge was assumed to be representative of discharge at Yancopin. For the years prior to Dam 2 operation, USGS discharge at Little Rock was lagged to create a flow record at Yancopin For each low flow time period determined, the daily Yancopin stage and Dam 2 discharge was plotted to create a yearly "low flow" discharge rating curve. The plotted data was inspected and obvious outliers were omitted. The plotted data were then curve

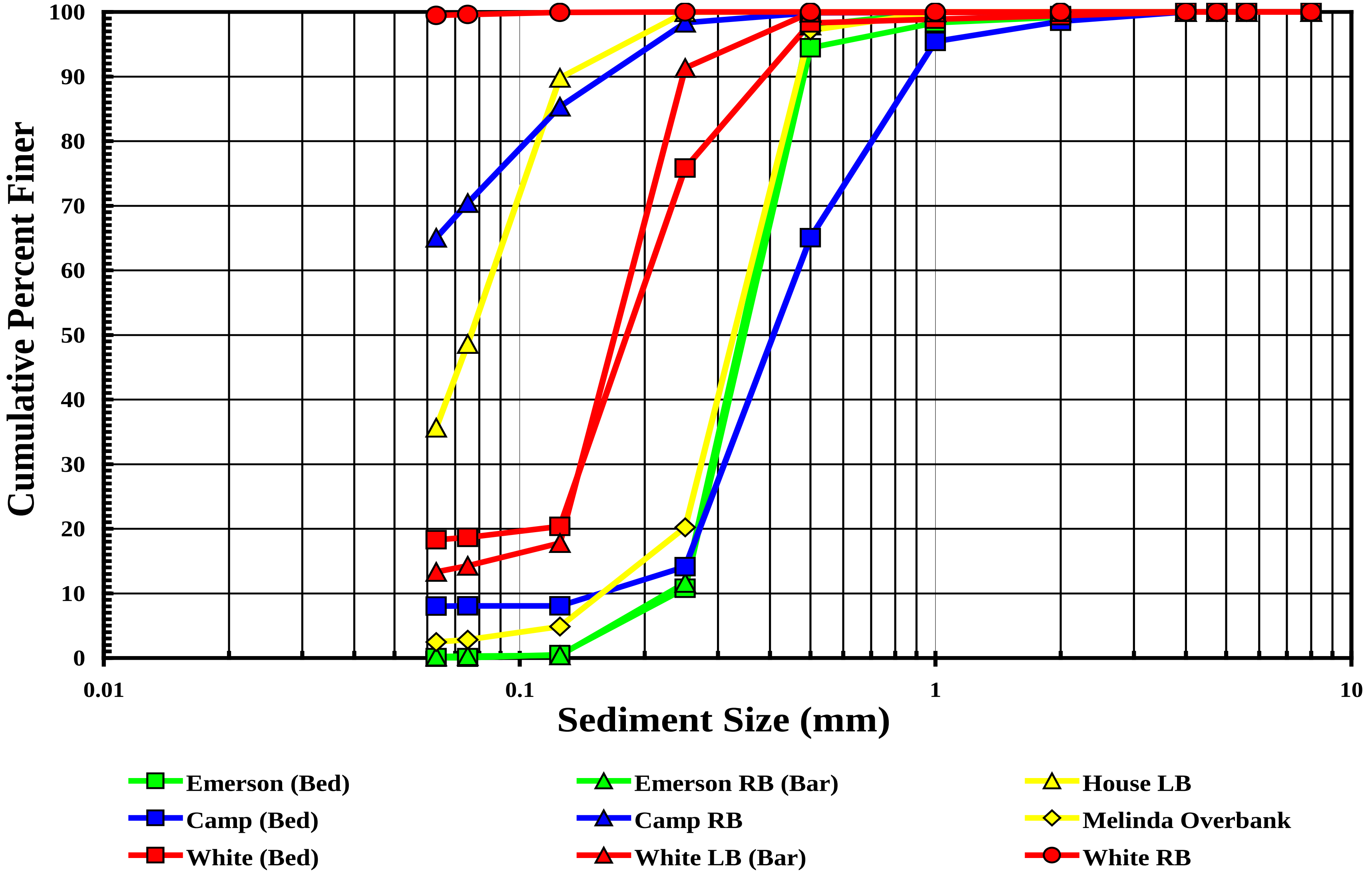

Figure 4-6 **Sediment Samples Grain Size Distribution**

Table 4-4
Study Area Timeline

Feature	Date	Comments
Mississippi River Cutoffs	1930’s – 1940’s	Result in channel degradation
Cutoffs on Lower Arkansas River		
Man-Made Cutoffs		
Red Fork Cutoff	1945	Between Dam 2 and Yancopin near 1949 RM 35
Hopedale Cutoff	1946	Immediately upstream of Yancopin near 1949 RM 30
Sawmill Bend Cutoff	1960	At historic cutoff near 1949 RM 25
Avenue Landing Cutoff	1962-1963	Between historic cutoff and mouth near 1949 RM 11
Morgan Point Cutoff	1966	Immediately downstream of Dam 2 near 1949 RM 41.5
Natural Cutoffs		
Emerson Bend	Early 1980’s	Between historic cutoff and mouth near 1949 RM 19
O’Neal Landing	Early 1990’s	At mouth near 1949 RM 2
Cat Island	Mid 1990’s	At mouth near 1949 RM 5
McClellan-Kerr Arkansas River Navigation Project Features		
Historic Cutoff Closure Dam	1963	
Norrell Lock and Dam (L&D 1)	1967	Put into operation
Lock 2	1968	Put into operation
Wilbur D. Mills Dam (Dam 2)	1967	Put into operation
Dam 2 Hydropower	1999	Put into operation
Lock and Dam No. 3	1968	Put into operation
Emmett Sanders Lock and Dam	1968	Put into operation
Lock and Dam No. 5	1968	Put into operation
David D. Terry Lock and Dam	1968	Put into operation
Murray Lock and Dam	1969	Put into operation
Multiple Purpose Plan Reservoirs		
Keystone Lake	1964	Arkansas River
Oologah Lake	1963	Verdigris River
	1974	Ultimate Development
Eufaula Lake	1964	Canadian River
Tenkiller Ferry Lake	1953	Illinois River
Grand Lake O’ The Cherokees	1940	Grand (Neosho) River

Table 4-4 (Continued)
Study Area Timeline

Feature	Date	Comments
Hudson Lake	1964	Grand (Neosho) River
Fort Gibson	1949	Grand (Neosho) River
	1953	Fully operational (last 4 generators on line)
Non Multiple Purpose Plan Reservoirs		
Wister	1949	Poteau River
Heyburn	1950	Polecat Creek
Blue Mountain	1947	Petit Jean River
Nimrod	1942	Fourche LaFave River
Containment Structure		
Melinda Headcut Structure	1989	initial construction
Soil Cement Containment Structure	1991	
Owens Lake Structure	1992	
Mouth of Arkansas River		
Ozark Island Dike	1995	Constructed to maintain channel east of Cat Island (near 1949 River Mile 7)
Above Ozark Revetment and Dikes	1995	Constructed to protect Ozark Lake and prevent flanking of Ozark Revetment on Mississippi River

fit with a “best fit” equation. This equation was used for discharges of 5000, 10000, 15000 and 20000 cubic feet per second (cfs) to calculate the corresponding stage for each year. The calculated stages for a given discharge were then plotted for the entire period of record to form the specific gage record. Figure 5-1 is the plotted specific gage record for the Arkansas River at Yancopin. For any given specific gage record, the yearly stages can fluctuate up or down without representing a trend. So is the case with the specific gage record for Yancopin for the 1932 though the early 1960’s period. The yearly stages fluctuated both up and down but no significant trend is indicated. However, by the early 1960’s the specific gage record started a downward trend that lasted until the mid 1970’s. The record indicates an approximate 4 to 6-foot drop during that 15- year period. Between the mid 1970’s and the early 1990’s, the specific gage record is fairly stable. Beginning in the mid 1990’s, the specific gage record takes a drastic downward trend.

The record shows a 4 to 5-foot drop from the mid 1990’s to the end of the period of record in 2001.

When data is not available to construct specific gage records, minimum stage plots can be helpful in determining channel aggradation or degradation trends. To verify the Yancopin specific gage record trends, an annual minimum stage plot was developed for the tailwater at Dam 2 for the 1969 through 2001 period of record. This plot is contained in Figure 5-2. As is the case with the Yancopin specific gage record, the annual minimum stage plot at Dam 2 exhibits a definite downward trend. The plot shows a lowering of the annual minimum stage of over 10 feet between 1969 and 2001. Also included on Figure 5-2 is the flow released from Dam 2. These discharges are not necessarily the minimum flow for each year but are the flows that correspond to the days that the annual minimum stages occurred. As can be seen on this plot, no annual minimum stage or corresponding discharge is provided for 1981. During this year, a significant amount of data is missing plus the accuracy of some of the available data that year appears questionable.

As stated earlier, no adequate discharge data was available on the lower White River to construct a specific gage plot. Therefore, an annual minimum stage plot was developed for the Lock 1 tailwater for the 1969 through 2001 period. This plot is contained in Figure 5-3. As can be seen on the plot, the White River experienced greater annual minimum stage fluctuations than the Arkansas River. The plot shows an apparent downward trend. However, due to extreme variability in White River data, the magnitude of the trend is difficult to quantify. This variability may be attributed to the closeness of this gage to the Mississippi River (extreme backwater influence), required dredging to maintain navigation channel, etc. Corresponding discharge data is not provided for the White River. The closest available discharge data is for the Clarendon gage. This gage is located at River Mile 99.1, some 89 miles upstream of Lock 1.

Another method for determining channel aggradation and degradation is by comparing bed elevations from historic hydrographic surveys. However, in many cases, detailed hydrographic survey data are not available. As a part of this assessment, a search of the files at the USACE, Mississippi Valley Division Office was conducted. This search resulted in locating two detailed hydrographic surveys for the lower Arkansas River. These surveys were conducted during 1917 and 1940. In addition, a HEC-RAS model of the study area was provided by USACE, Little Rock District. The channel geometry in that model was obtained from an existing UNET (unsteady flow) model. The date of the survey used for the UNET channel geometry is not known. However, Little Rock District personnel believe that the survey was most likely conducted during the 1960’s. From these surveys, thalweg profiles were developed for the lower Arkansas River from its mouth to the vicinity of the existing Dam 2 site. Figure 5-4 is the thalweg profiles comparison plot. The higher points on the profiles represent the thalweg elevations in the crossings. Channel bed elevation changes in the crossings provide a more reliable indication of system change than do changes in the bends. As can be seen on the plot, the crossing thalweg elevation profile for 1940 is generally lower than the profile developed for 1917. Also, the 1960’s profile is lower than the 1940 profile. Therefore, these

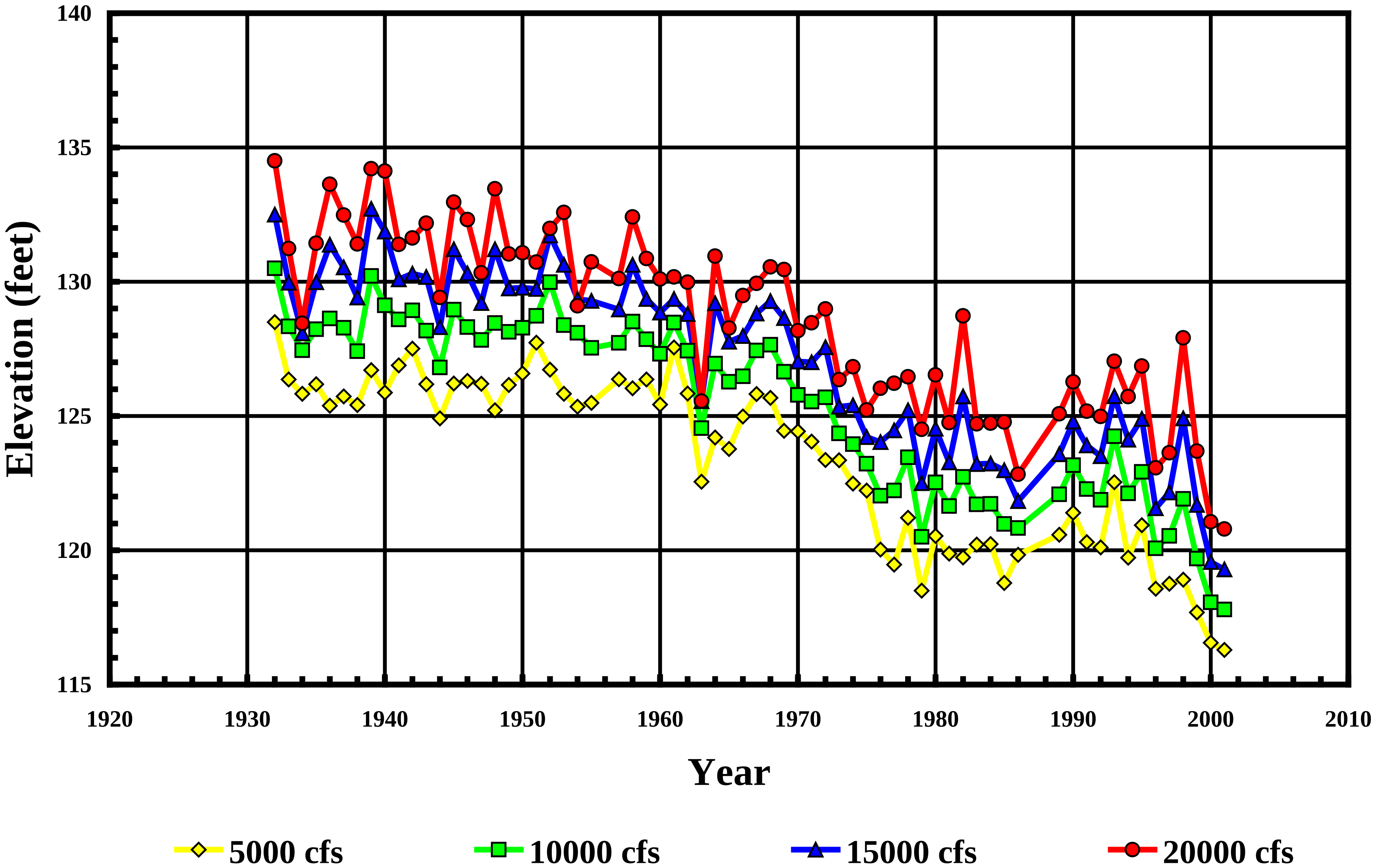

Figure 5-1 Specific Gage Record for Arkansas River at Yancopin

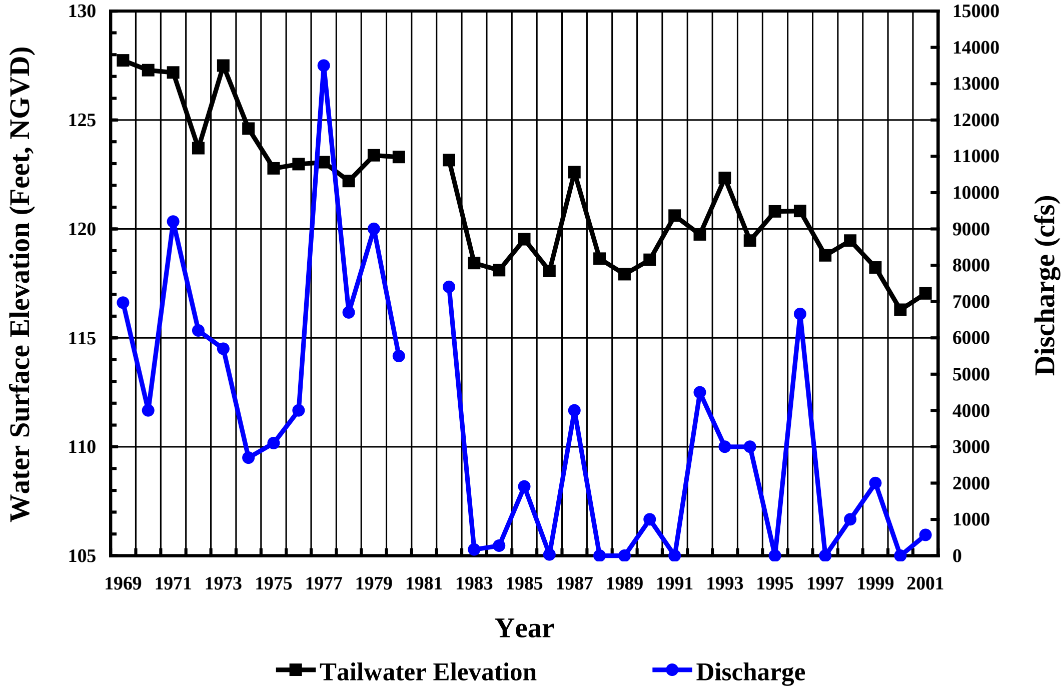

Figure 5-2 Annual Minimum Water Surface Elevations and Corresponding Discharges for Arkansas River at Dam 2 Tailwater

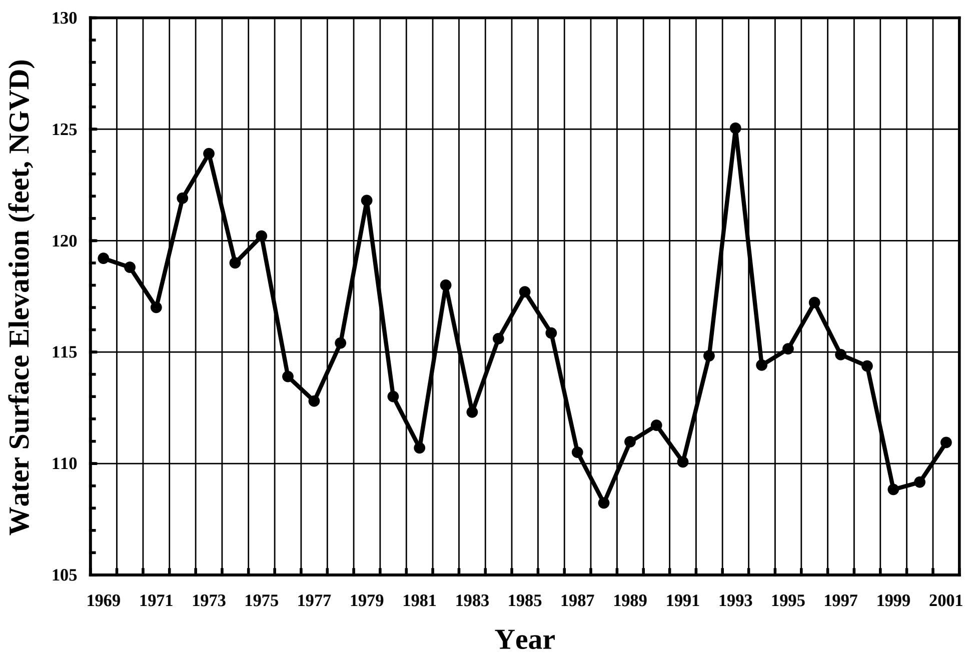

Figure 5-3 Annual Minimum Water Surface Elevations for White River at Lock 1 Tailwater

profiles show that the bed between the mouth and the Dam 2 site on the lower Arkansas River degraded between 1917 and 1940 and then continued to degrade between 1940 and the 1960's. The extent of this degradation is difficult to quantify since the channel distance changes from each survey. A better indication of degradation quantities could be gained if the profiles were plotted against a common river mileage. However, since the date of the 1960's survey is not known, it is not possible to convert the channel distance for this survey to a common river mileage. One fixed point within the mouth to Dam 2 site reach on all 3 surveys is the railroad bridge at Yancopin. Figure 5-4 shows that the thalweg profile through the Yancopin reach was approximately 2 feet lower in 1940 than it was in 1917 and approximately 10 feet lower in the 1960's than in 1940. Therefore, at Yancopin the surveys show that the channel degraded about 12 feet between 1917 and the 1960's.

While specific gage records, minimum stage plots, and channel bed profiles are indicators of channel bed changes, field investigations can provide additional evidence. Such evidence exists within the study reach. As part of this geomorphic assessment, channel reconnaissance field investigations were conducted by boat during October and November 2002. During these trips, the existence of several terraces on the Arkansas River downstream of Dam 2 and on the White River downstream of Lock 1 was documented. Terraces are formed due to channel degradation and results in the building of a new, lower elevation floodplain. Figure 5-5 is a photo of the terrace on the right descending bank of lower Arkansas River at Jimmie Bend. On the lower Arkansas River, typical elevation differences between the terrace and the newly developing floodplain ranged from 10 to 15 feet. The terraces on the lower White River were of similar height. The difference in elevation between the terraces (historic top bank) and the newly developing floodplain correlates reasonably well with the channel degradation indicated by the Yancopin specific gage record, the Dam 2 tailwater annual minimum stage plot, and the channel thalweg profile plots.

As previously mentioned, the study area on the lower Arkansas and White Rivers is impacted by backwater on the Mississippi River. Therefore, historic channel changes on the Mississippi River would have significant impacts on the study area. Channel changes on the Mississippi River have been well documented. The morphologic response of the lower Mississippi River is the product of both natural events and man-made works. The assessment of the individual impacts of each influence is extremely difficult since the response of the river to one specific influence is generally very small or not discernable. However, as a result of the great flood of 1927, the U.S. Army Corps of Engineers was assigned the task of studying, designing, and constructing flood control features on the lower Mississippi River. The first items of work included the construction of bendway cutoffs. The cutoff program on the Lower Mississippi River consisted of allowing 2 natural cutoffs to develop and the construction of 14 neck cutoffs between 1929 and 1942 (Winkley, 1977). The cutoffs shortened the river by approximately 150 miles between Memphis, Tennessee and Old River, Louisiana. The tendency of the river to regain the pre-cutoff length by meandering was restricted by the construction of bank stabilization measures. In the case of the cutoffs, impacts were immediate and obvious. The cutoffs resulted in an immediate flowline lowering at higher flows of about 16 feet at Arkansas

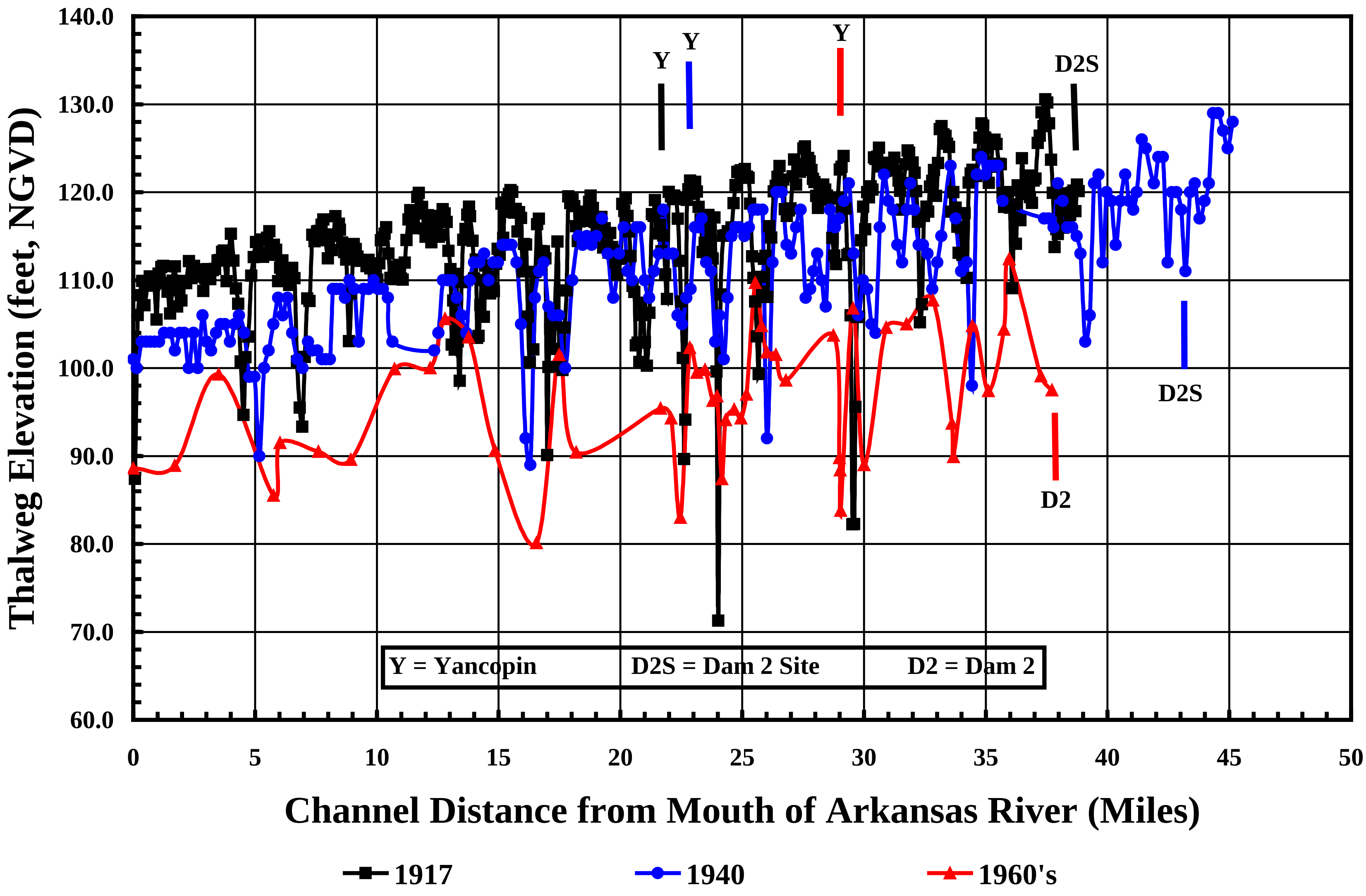

Figure 5-4 Arkansas River Thalweg Profiles

Figure 5-5 Terrace on the Arkansas River at Jimmie Bend

City and 12 feet at Vicksburg (Madden, 1974). Thus, the cutoffs represent the single element that has had the greatest effect of any natural or man-made feature on the recent morphology of the Mississippi River (U.S. Army Corps of Engineers, 1982).

To gain a better understanding of channel changes resulting from the cutoffs, specific gage records for the Mississippi River were developed (Biedenharn and Hubbard, 1998). The specific gage records were developed at New Madrid, Missouri; Memphis, Tennessee; Helena, Arkansas; Arkansas City, Arkansas; Lake Providence, Louisiana; Vicksburg, Mississippi; Natchez, Mississippi; Red River Landing, Louisiana; Bayou Sara, Louisiana; and Simmesport, Louisiana. Both the Arkansas River and the White River empty into the Mississippi River between Helena and Arkansas, City, Arkansas. Therefore, this assessment will concentrate on the specific gage records for those two gages. The specific gage record for Helena is presented in Figure 5-6 and the record for Arkansas City is presented in Figure 5-7. At Helena, discharges of 200,000 cubic feet per second (cfs), 400,000 cfs, 600,000 cfs, 800,000 cfs, 1,000,000 cfs, and 1,200,000 cfs were evaluated. The specific gage record at Helena was developed for the 1879 through 1997 period. At Arkansas City, discharges of 200,000 cfs, 400,000 cfs, 600,000 cfs, 800,000 cfs, and 1,100,000 cfs were evaluated. The specific gage record for Arkansas City was developed for the 1884 through 1997 period. For this assessment, the specific gage records for Helena was updated through 2000 and Arkansas City was updated through 2001. The updated records provided sufficient time to see trends prior to the

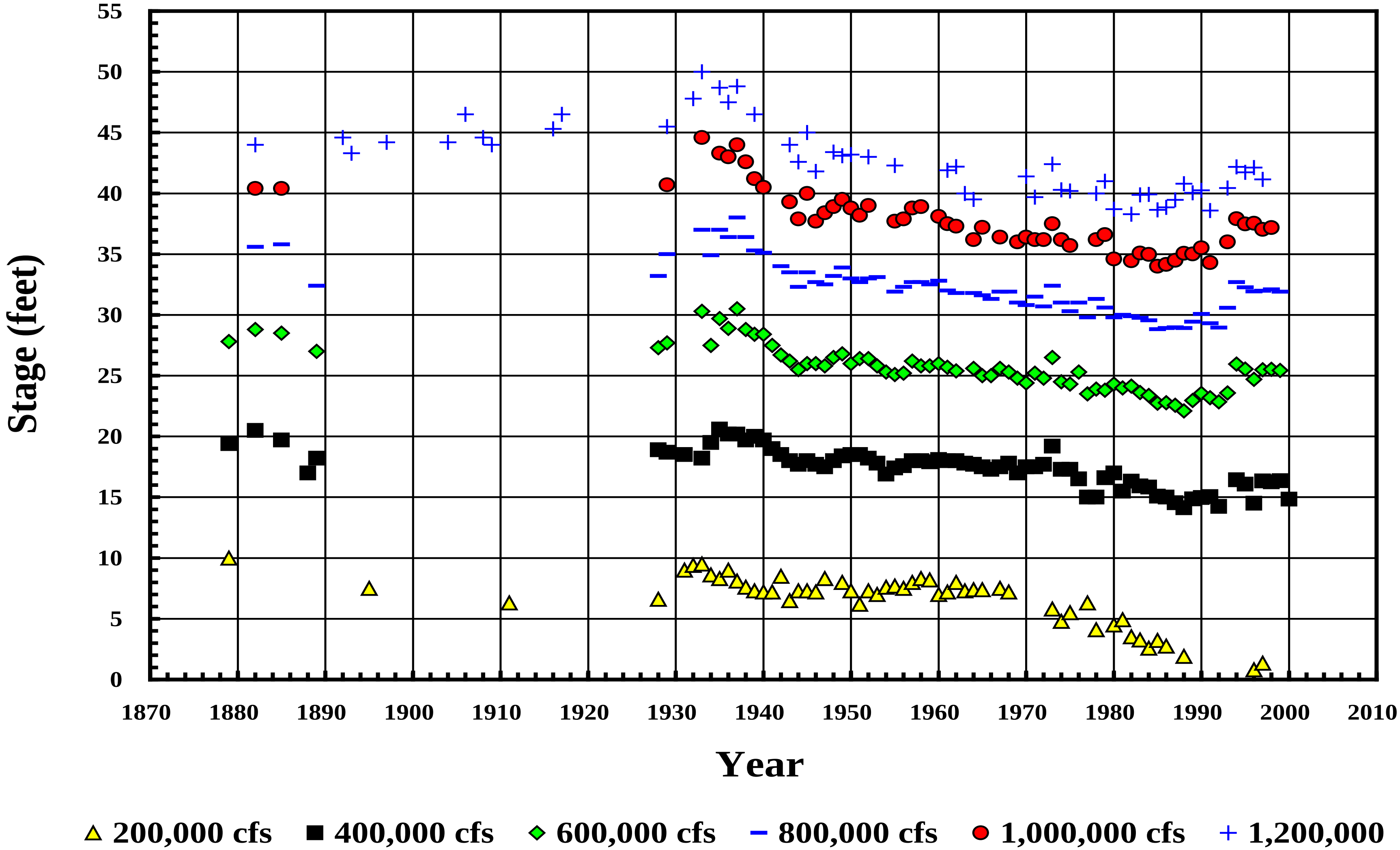

Figure 5-6 Specific Gage Record for Mississippi River at Helena, Arkansas

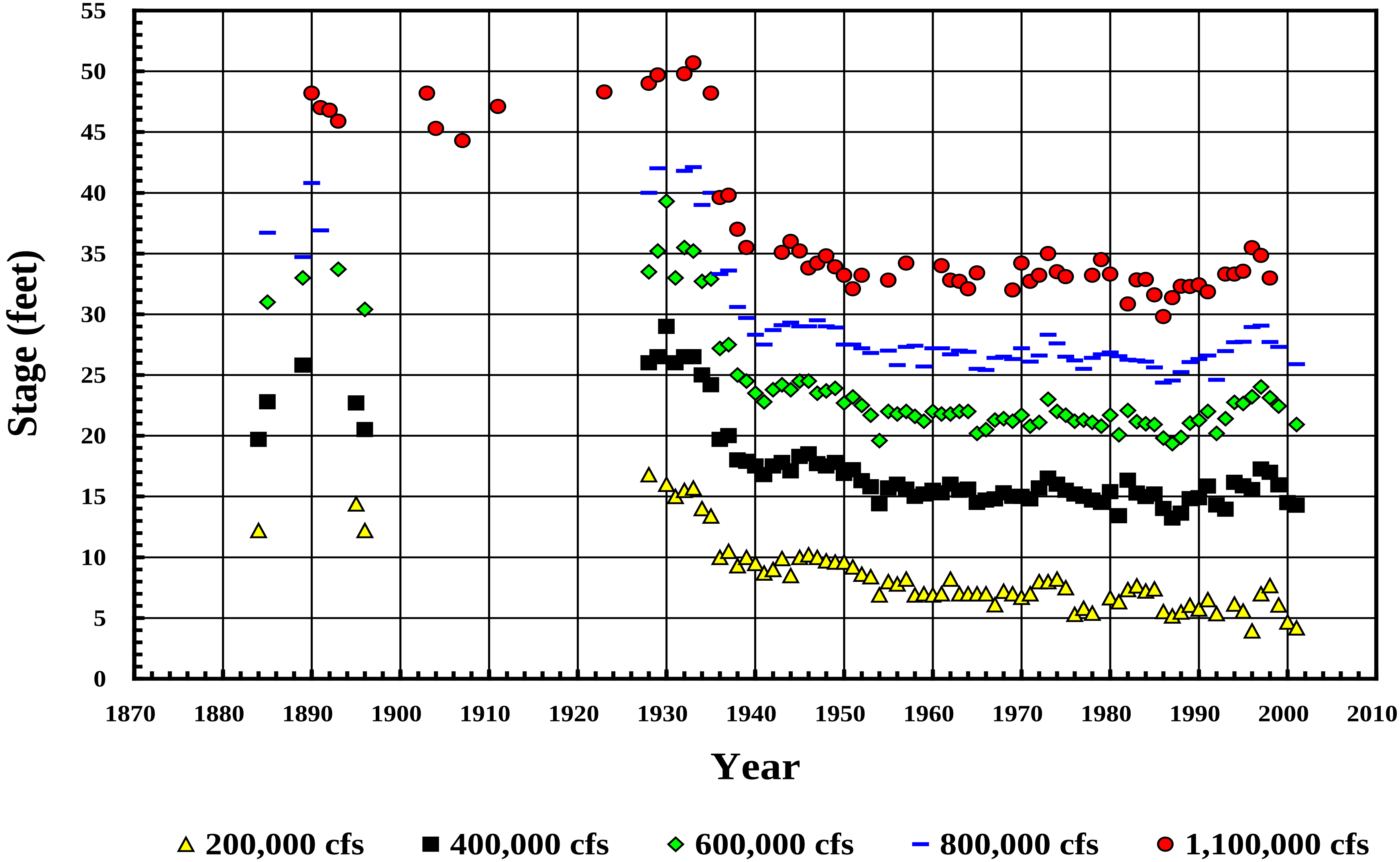

Figure 5-7 Specific Gage Record for Mississippi River at Arkansas City, Arkansas

cutoff program, the initial response to the cutoffs, and current trends. The specific gage records at both Helena and Arkansas City show a dramatic response (lowering) to the cutoff program on the Mississippi River. The response at both locations is not as great for low flow as it is for higher flows. Also, the response at Helena is not as great as the response at Arkansas City. At Helena, the specific gage record shows a lowering of 3 to 5 feet for lower flows and 7 to 8 feet for higher flows between 1930 and 1945. During this same period, the specific gage record at Arkansas City shows a lowering of approximately 5 feet for the lower flows and over 15 feet for the higher flows. Throughout the 1950's, 1960's, 1970's, and 1980's, the specific gage record at Arkansas City shows little change. During the same period at Helena, the specific gage record shows a lowering of 3 to 5 feet. During the 1990's both specific gage records show a slight upward trend. Perhaps this trend can be attributed to unusual hydrologic events that occurred during the 1990's on the Mississippi River.

5.2 Planform Analysis.

One way to classify rivers is by their planform. The three basic types of planform are straight, meandering, and braided. The type pattern that exists along any reach of river is dependent upon channel slope, water discharge, sediment load, and boundary conditions. The most common channel planform is meandering. A meandering channel is characterized by a series of alternating changes in direction or bends. The braided river is defined as having multiple channels within its bed. Relatively straight reaches of alluvial rivers rarely occur naturally. However, some reaches of rivers may maintain relatively straight alignments for considerable periods of time. Even in these reaches, the thalweg may still meander. Although channels are often classified as straight, meandering, or braided, it should be recognized that the transition between these patterns exist as a continuum, which often makes a unique classification difficult.

Channel length is a feature that is easy to determine since it is measured directly from available aerial mosaics, satellite imagery, and maps. For this assessment, the lower Arkansas River channel length was measured from aerial mosaics and mapping for 1917 1940, 1949, 1976, 1980, 1985, 1988, 1991, 1995, and 1999. Channel length was determined for the Dam 2 site to the mouth of the Arkansas River reach. The channel lengths for this reach are presented in Figure 5-8. The reach was also broken into two shorter reaches, the first being the Dam 2 site to Yancopin Bridge and the second was Yancopin Bridge to the mouth. Above Yancopin, all of the bends have been revetted. This bank stabilization preserves channel length by preventing future channel migration. Without bank stabilization, continued channel migration can lengthen the channel by increased meandering or shorten the channel by natural cutoffs. Figure 5-9 provides the channel length by reach. Once the channel lengths were measured, the reasons for length changes were investigated. The predominant feature that impacted length was both natural and man-made cutoffs. The man-made cutoffs included Red Fork Cutoff (1945) and Hopedale Cutoff (1946). Both of these cutoffs are located between the Dam 2 site and Yancopin Bridge. Red Fork Cutoff is located at 1949 River Mile 35 and Hopedale Cutoff is at River Mile 30. Other man-made cutoffs included Sawmill Bend in 1960,

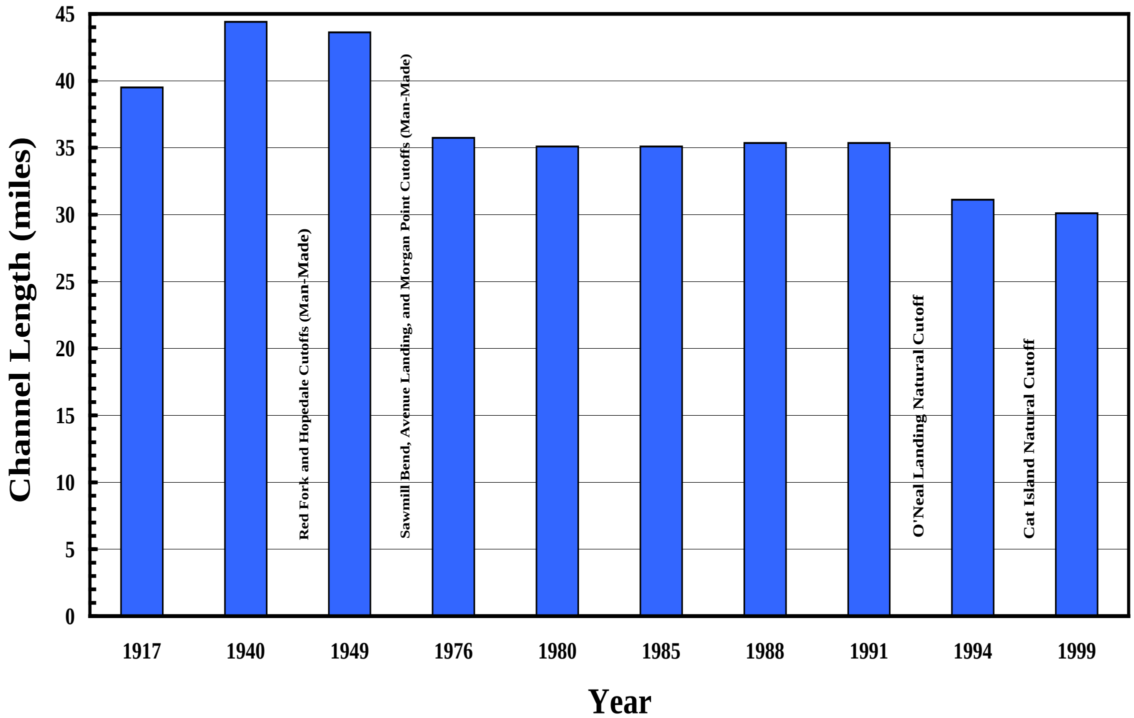

Figure 5 –8 Lower Arkansas River Channel Length

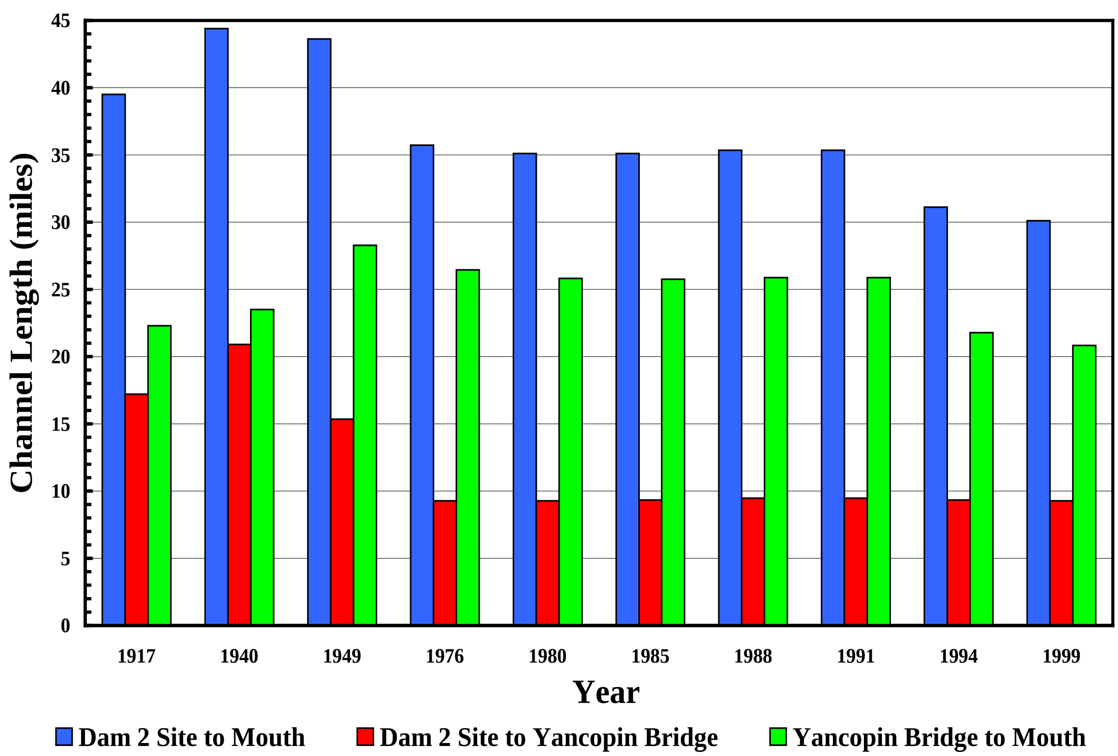

Figure 5 – 9 Lower Arkansas River Channel Length By Reach

Avenue Landing in 1963, and Morgan Point in 1966. Sawmill Bend Cutoff (1949 River Mile 25) is located immediately downstream of House Bend, Avenue Landing Cutoff (1949 River Mile 11) is located between the historic cutoff and the mouth, and Morgan Point Cutoff (1949 River Mile 41.5) is located immediately downstream of the Dam 2 site. The natural cutoffs included Emerson Bend during the early 1980's, O'Neal Landing during the early 1990's and Cat Island during the mid 1990's. All 3 of these cutoffs are located between the historic cutoff and the mouth with Emerson Bend at 1949 River Mile 19, O'Neal Landing at 1949 River Mile 2, and Cat Island at 1949 River Mile 5. However, the data shows that even though some reaches of the river were reducing length due to natural and man-made cutoffs, other reaches were increasing length due to continued channel migration.

Planform geometry is often defined by sinuosity, meander wavelength, and radius of curvature. The term sinuosity describes the degree of meander activity in a stream and is defined as the ratio of the distance along the channel (channel length) to the distance along the valley (valley length). A perfectly straight channel has a sinuosity of 1.0 since the channel length equals the valley length. For this analysis, the sinuosity of the channel was determined for the lower Arkansas River from the Dam 2 site to its mouth for 1917, 1940, 1949, 1976, 1980, 1985, 1988, 1991, 1994, and 1999. Figure 5-10 is a plot of the sinuosity for the lower Arkansas River channel. As was the case with channel length, the lower Arkansas River was divided into two reaches. These reaches were Dam 2 site to Yancopin Bridge and Yancopin Bridge to the mouth. Channel sinuosity by reach is included in Figure 5-11. Sinuosity is a function of channel length. Therefore, the channel cutoffs that occurred on the lower Arkansas River impacted sinuosity just as it impacted channel length. A comparison of the channel length and sinuosity plots show that the greatest sinuosity occurred prior to the channel cutoffs.

Meander wavelength is defined as twice the straight-line distance between two consecutive points of similar condition in a channel. The distance is not measured along the channel but rather in a straight line parallel to the valley. Points of similar condition are considered pools or crossings. Typically meander wavelength is measured between crossings. Figure 5-12 is a sketch that defines various channel geometry parameters including meander wavelength. Figure 5-13 is a plot of the average meander wavelengths as computed for the lower Arkansas River from the Dam 2 site downstream to the mouth. Meander wavelength is measured directly from aerial photography and mapping. For this assessment, wavelength was determined for 1917, 1940, 1949, 1976, 1988, and 1999. As can be seen from Figure 5-13, meander wavelength has not significantly changed on the lower Arkansas River having ranged from less than 13,000 feet in 1917 and 1940 to almost 16,000 feet in 1999. The increase in meander wavelength may be attributed to various man-made and natural cutoffs that occurred on the lower Arkansas River. Specifically, the increase in meander wavelength observed between 1988 and 1999 may be attributed to the two natural cutoffs that occurred on the lower Arkansas at its mouth.

Radius of curvature (sometimes referred to as bend radius) is the radius of a circle that defines the curvature of an individual bend as measured between sequential crossings.

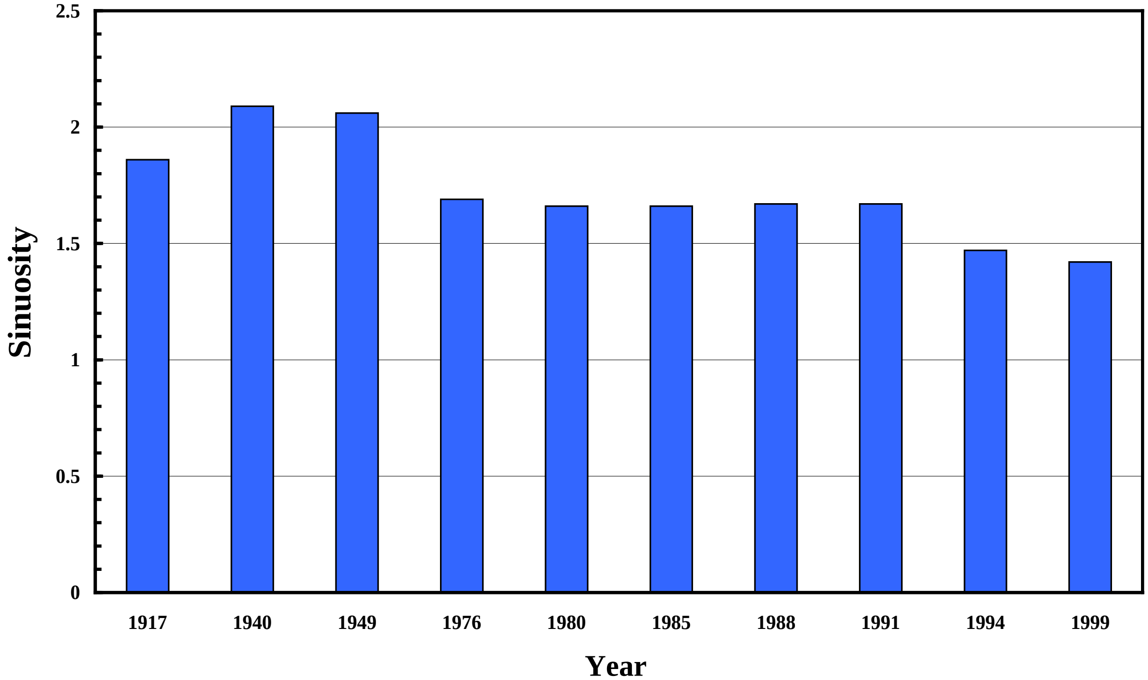

Figure 5-10 Lower Arkansas River Sinuosity

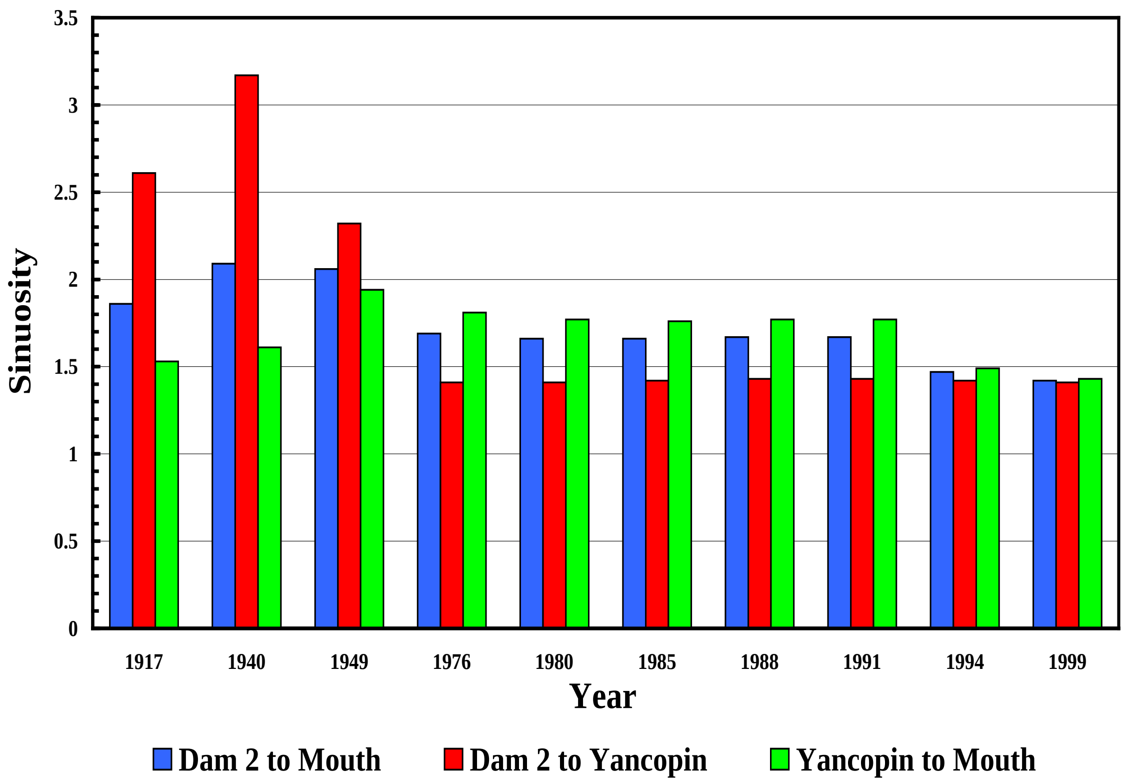

Figure 5-11 Lower Arkansas River Sinuosity By Reach

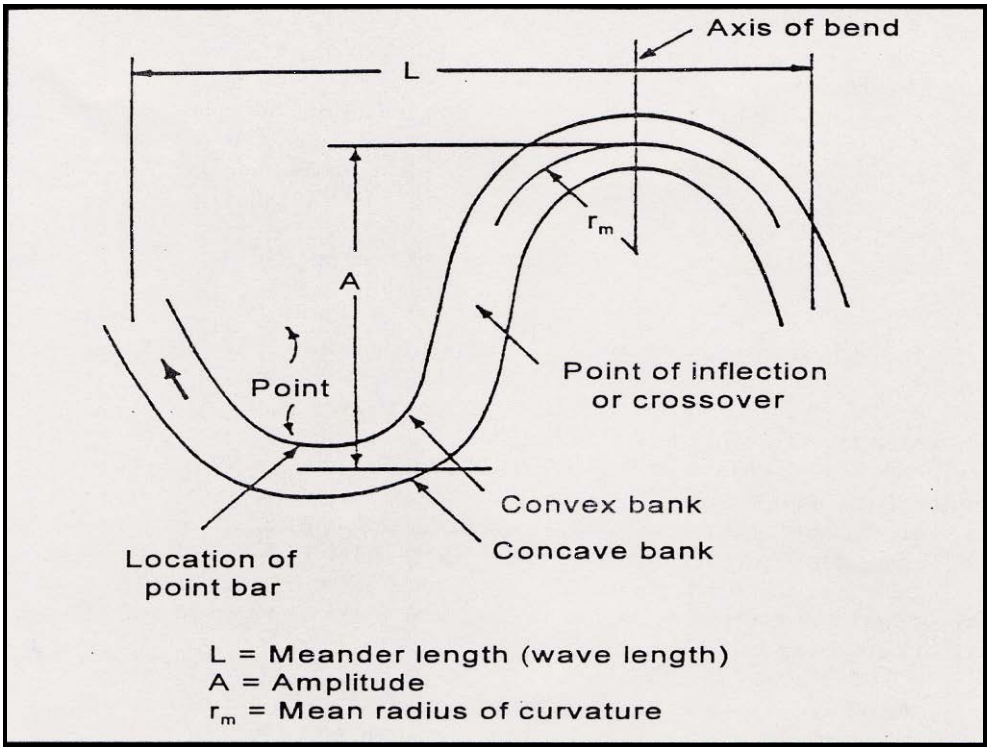

Figure 5-12 Channel Geometry Definition Sketch (after Leopold et al., 1964)

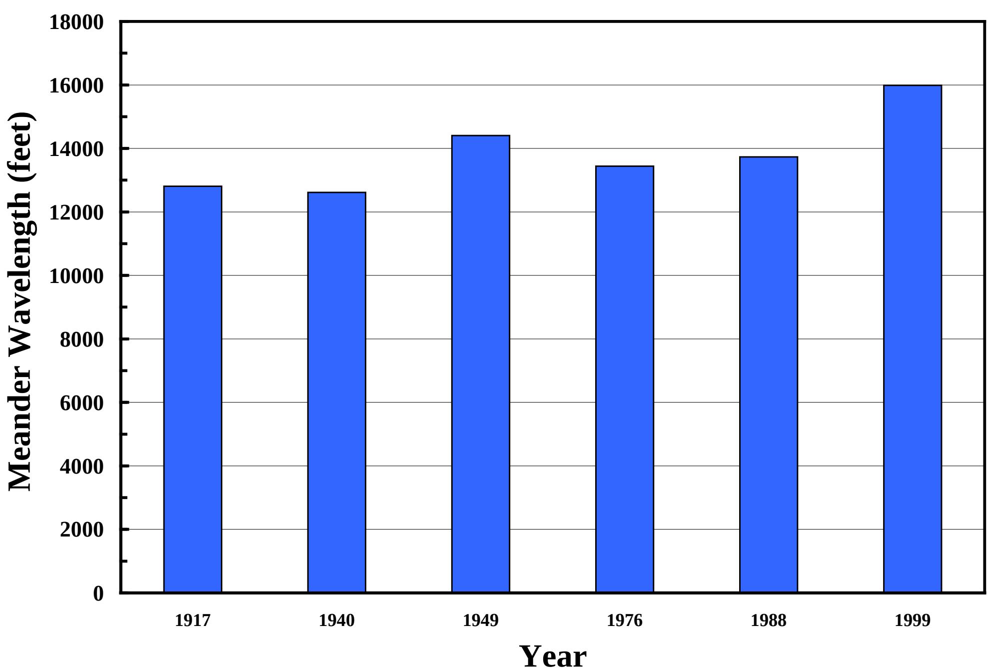

Figure 5-13 Lower Arkansas River Average Meander Wavelength

Radius of curvature is depicted on Figure 5-12. The ratio of bend radius to channel width is very useful in the identification of meander behavior, especially bank caving rates. Figure 5-14 is a plot of the average bend radius for each bend on the lower Arkansas River between the Dam 2 site and its mouth. As with meander wavelength, the average bend radii were determined for 1917, 1940, 1049, 1976, 1988, and 1999. From Figure 5-14, we can see that the average bend radius has continued to increase from approximately 2,500 feet in 1940 to 5,000 feet in 1999. This increase in bend radius can be attributed to the realigning of tighter bends with flatter radius channel cutoffs.

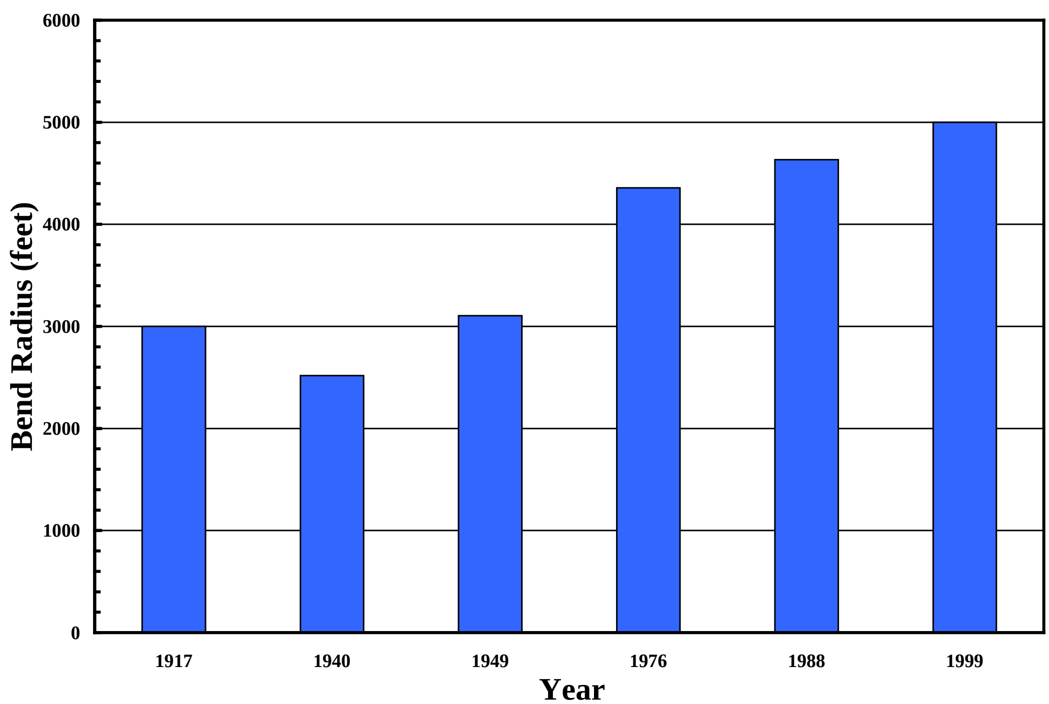

Figure 5-14 Lower Arkansas River Average Bend Radius

5-3 Slope Analysis.

Channel slope (longitudinal profile) of a stream is one of the most important factors in establishing channel morphology. Slope is a good indicator of the energy of a river to do work. Rivers with steep slopes are generally higher energy systems than rivers with flat slopes. Rivers with steep slopes are usually more active with respect to bank erosion, bar building, and sediment movement. Channel slope is often defined by the water surface or by the channel bed. For those streams where sufficient stage data is available, water surface profiles can be developed from stages recorded at the various gage stations. Water surface slope is not constant but frequently varies from low to high stages. Slope also frequently changes over time with corresponding channel changes. The water surface slopes on the lower Arkansas and lower White Rivers are highly dependent upon backwater from the Mississippi River. When stages are high on the Mississippi River, backwater reduces the slope on the lower Arkansas and lower White Rivers. During these times, little energy is available to alter channel conditions. Therefore, the only stages used in this assessment were those not influenced by backwater on the Mississippi River. The stages on the Mississippi River were analyzed from 1976 through 2001. For those periods, the observed stages at Yancopin were compared to the computed stages at the mouth of the Arkansas River. The stages at the mouth were computed by interpolating between the observed stages on the Mississippi River at Arkansas City and at MW5 (mouth of White River). The channel slope on the lower Arkansas River was calculated by dividing the difference in the water surface elevations at Yancopin and the mouth by the channel length between the two. Typically during periods of no backwater, the difference in the stages at Yancopin and at the mouth of the Arkansas River varied between 1.5 and 24.3 feet. Table 5-1 presents the computed channel slopes for the Yancopin to mouth reach of the lower Arkansas River. This table shows how much variability exists in channel slope even during a single hydrologic event. For example, the slopes during 1999 varied from 0.35 feet per mile to 1.06 feet per mile with an average slope of 0.68 feet per mile. The average slopes in 1997 and 1999 were steeper than at any of the other times except for 1988. However, 1988 was a time of severe drought on the lower Mississippi River. Therefore, extreme low stages were experienced on the Mississippi River during the summer and fall of that year. The increased slope during 1997 and 1999 can be attributed to the length reduction resulting from the natural realignment of the channel at Cat Island and O'Neal Landing (at the mouth). This realignment resulted in shortening the lower Arkansas River by 5 miles between 1991 and 1999. The years presented in bold in Table 5-1 are those in which aerial photography was available to measure channel length.

Several researchers have developed empirical relationships between channel slope, discharge, and channel patterns. These relationships provide the river engineer a tool for identifying the change in channel slope or discharge necessary to transition a given stream from meandering to braided and vice versa. Two of these relationships are provided as Figure 5-15 and Figure 5-16. Figure 5-15, as developed by Lane (1957), provides the relationship between channel slope, mean annual discharge and channel patterns. This relationship includes two sloping lines. Channels whose combination of slope and mean annual discharge plot above the higher line are in the braided zone. Those that plot below the lower line are in the meandering zone. The area between the two lines is a transitional zone. For any meandering stream, if the mean annual discharge remains constant and the stream slope increases enough (gets steeper), then the channel can potentially transition from a meandering to a braided stream. The mean annual discharge for the lower Arkansas River as determined at Dam 2 is 51,800 cubic feet per second. The slope on the lower Arkansas River as determined for the 1999 no backwater condition varied from 0.35 feet per mile (0.000066 feet per foot) to 1.06 feet per mile (0.00020 feet per foot) with an average slope of 0.68 feet per mile (0.00013 feet per foot). These points are plotted on Figure 5-15. As can be seen, the minimum slope plots within the meandering range, the average slope plots on the line between the meandering and transitional ranges, and the maximum slope plots in the transitional zone just above the meandering range.

Table 5-1
Arkansas River Water Surface Slope – Yancopin to Mouth

Year	Channel Length (miles)	Stage Difference (feet)			Slope (feet/mile)		
		Minimum	Average	Maximum	Minimum	Average	Maximum
1999	**20.83**	**7.3**	**14.2**	**22.2**	**0.35**	**0.68**	**1.06**
1997	20.83	3.5	12.7	21.8	0.17	0.61	1.05
1996	21.78	4.5	9.7	13.8	0.21	0.44	0.63
1995	21.78	5.3	11.2	18.4	0.24	0.52	0.84
1994	**21.78**	**6.5**	**10.3**	**17.0**	**0.30**	**0.47**	**0.78**
1993	21.78	3.0	6.9	13.2	0.14	0.32	0.61
1992	25.85	6.8	11.5	19.2	0.26	0.45	0.74
1991	**25.85**	**9.9**	**13.4**	**18.9**	**0.38**	**0.52**	**0.73**
1990	25.85	3.5	6.7	13.4	0.14	0.26	0.52
1989	25.85	4.7	10.4	16.2	0.18	0.40	0.63
1988	**25.85**	**16.6**	**18.4**	**24.3**	**0.64**	**0.71**	**0.94**
1987	25.85	8.0	14.9	21.5	0.31	0.58	0.83
1986	25.85	5.4	10.4	15.6	0.21	0.40	0.60
1985	**25.85**	**7.6**	**13.4**	**18.9**	**0.30**	**0.52**	**0.73**
1984	25.85	5.9	10.1	15.4	0.23	0.39	0.60
1983	25.85	4.9	9.2	15.6	0.19	0.36	0.60
1982	25.85	1.5	5.9	13.2	0.06	0.23	0.51
1981	25.85	10.7	15.6	22.2	0.41	0.60	0.86
1980	**25.85**	**4.7**	**9.7**	**15.5**	**0.18**	**0.37**	**0.60**
1978	26.15	6.7	9.8	12.6	0.26	0.37	0.48
1977	26.3	8.7	14.3	21.6	0.33	0.54	0.82
1976	**26.45**	**12.3**	**15.4**	**19.2**	**0.47**	**0.58**	**0.73**

Figure 5-16, as developed by Leopold and Wolman (1957), provides the relationship between channel slope, bankfull discharge, and channel patterns. This figure includes a single sloping line. The combinations of channel slope and bankfull discharge that plot above the line are in the braided zone and those that plot below the line are in the meandering zone. The 1999 minimum, average, and maximum slopes are included on Figure 5-16. The bankfull discharge for the lower Arkansas River was assumed to be 185,000 cfs. This flow is the 2-year frequency discharge at Dam 2. As Figure 5-16 shows, the minimum, average, and maximum slopes for 1999 all fall within the meandering range. A note of caution is warranted about the use of the curves in Figures 5-15 and 5-16. These are empirical relationships developed for streams outside the Arkansas River system and therefore, should be viewed only as general indicators of Arkansas River planform stability. However, these curves do generally indicate that the Arkansas River is in the meandering zone with little indication of transition to braiding unless channel slopes continue to increase. It is important to also note that our Arkansas River channel slopes were computed only for periods of low water when there was no backwater effect from the Mississippi River. The channel slopes at higher stages would probably be flatter due to backwater. Therefore, flatter slopes would move the Arkansas River further into the meandering zone on Figures 5-15 and 5-16.

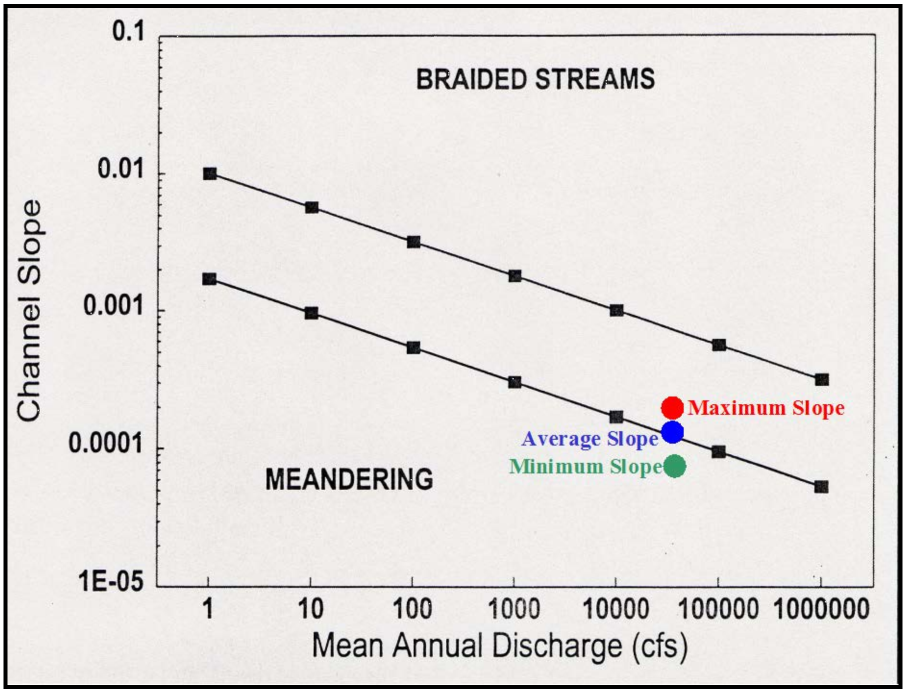

Figure 5-15 Lane's (1957) Relationship Between Channel Patterns, Channel Gradient, and Mean Annual Discharge

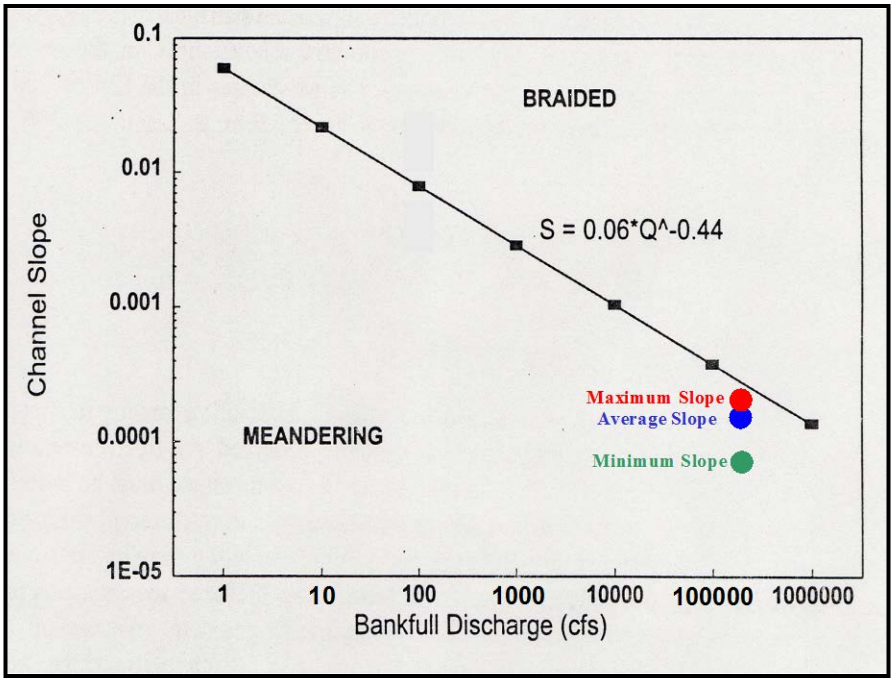

Figure 5-16 Leopold and Wolman's (1957) Relationship Between Channel Patterns, Channel Gradient, and Bankfull Discharge

6. Historic Cutoff.

Prior to 1963, a cutoff channel connected the Arkansas and White Rivers downstream of House Bend. This channel provided an interchange of flow and sediment between the two rivers. At times, the flow was from the White River to the Arkansas River and at other times the flow was in the opposite direction, from the Arkansas River to the White River. To improve conditions for navigation, an earthen closure dam was constructed across the cutoff channel in 1963. Once this closure dam was constructed, this natural low to medium flow connection between the two rivers was severed. Since the historic cutoff channel had developed naturally, the closure resulted in the rivers attempting to reestablish a connection. Much interest has been expressed in the functions that the historic cutoff served, especially the transfer of both water and sediment from the Arkansas to the White River and vice versa. Very little data exists on the historic cutoff. However, the data search conducted for this geomorphic assessment resulted in the locating of Arkansas River hydrographic surveys for 1917, 1940, and 1949. These surveys extend through the historic cutoff. The 1917 survey was conducted on November 7 - 8, 1917. The 1940 survey was conducted from October 10 – 18, 1940. The 1949 survey was conducted from June 14 – 24, 1949. Figure 6-1 is a thalweg plot of the historic cutoff from these surveys. The plot extends from the Arkansas River through the cutoff to the White River. As can be seen on the plot, the historic cutoff was about 3.8 miles long in 1917. The length of the cutoff channel had increased its length to approximately 5.4 miles by 1940 and to about 5.9 miles by 1949. This additional length appears to be the result of down valley migration of the Arkansas River and the increased sinuosity in the historic cutoff. The down valley migration of the Arkansas River moved the point that the historic cutoff channel intersected the Arkansas River farther downstream. The increased sinuosity within the cutoff channel was due to bendway migration within the cutoff. One problem with these surveys is the limited data in some cross sections. Some of the cross sections have as few as three survey points. This limited data leads to the potential to grossly miss the true channel thalweg. The profile from the 1940 and 1949 surveys show a definite downward slope from the Arkansas River to the White River. However, the 1917 survey doesn't show a discernable slope through the cutoff channel. This survey shows four thalweg elevations above 120.0 feet, NGVD. As can be seen on Figure 6-1, these high points are spread throughout the cutoff channel. The historic cutoff channel in 1917 was approximately 3.8 miles long. The high points are located some 0.5 miles, 1.6 miles, 2.6 miles, and 3.2 miles from the Arkansas River. On both the 1917 and 1940 surveys, the direction of flow through the cutoff channel was shown to be from the Arkansas River to the White River. The direction of flow on the 1949 survey is shown from the White River to the Arkansas River. However, observed gage records for Yancopin on the Arkansas River and Benzal on the White River indicate that at the time the 1949 survey was conducted, the flow was more likely to have been from the Arkansas River to the White River. The 1917 survey included two discharge measurements taken on the Arkansas River (approximately 2600 feet upstream and 3600 feet downstream of the historic cutoff) and one in the historic cutoff channel (approximately 250 feet from the Arkansas River). The measured discharge on the Arkansas River upstream of the historic cutoff was 2,753 cubic feet per second (cfs) while the discharge downstream of the cutoff was 1,760 cfs. The measured discharge in

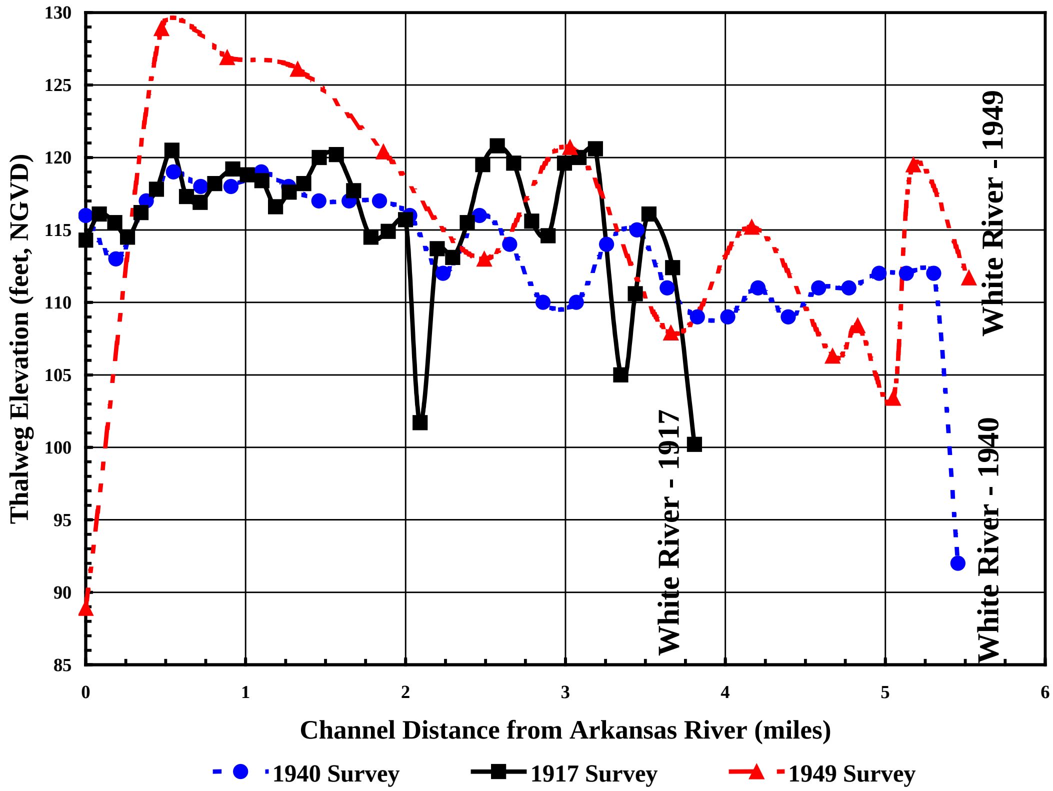

Figure 6-1 Historic Cutoff Thalweg Profiles

the historic cutoff channel was 1,028 cfs. Both the 1917 and 1940 surveys identified the left and right descending top banks of the channel. The procedure used to determine top bank on these surveys is not known. However, the locations as shown on the survey sheets seem reasonable. Therefore, channel width was determined between left top bank and right top bank at each of the cross sections through the historic cutoff. The pertinent channel width data is provided in Table 6-1. The top banks are not identified on the 1949 survey and sufficient survey data is not available to determine top bank. Therefore, channel width is not provided for this survey.

7. Channel Migration Analysis – Camp Bend and House Bend.

Channel migration rates are often dependent upon a complex combination of various hydraulic and geologic factors. These factors include the angle of attack by the flow, channel velocity, and the erodibility of the soils of which the banks are comprised. The angle of attack and channel velocity determine the shear stress that is applied to the river banks by the flow. The erodibility of the material in the banks is a function of soil type. Therefore, the greater the shear stress and the more easily erodible the bank material, the greater the caving rate.

Table 6-1
Historic Cutoff Channel Width Data

Survey	Number of Cross Sections	Minimum Width (feet)	Average Width (feet)	Maximum Width (feet)
1917	38	390	776	1290
1940	28	575	720	1100

Channel Width (feet)	1917 Survey		1940 Survey	
	Number of Sections	% of All Sections	Number of Sections	% of All Sections
500 or Greater	35	92	28	100
600 or Greater	28	74	27	96
700 or Greater	22	58	16	57
800 or Greater	18	47	6	21
900 or Greater	12	32	2	7
1000 or Greater	6	16	2	7

As part of the geomorphic assessment, a channel migration analysis was conducted for Camp Bend and House Bend. Camp Bend is a relatively flat bend (approximately 7000-foot radius) and is located on the right descending bank immediately downstream of the Yancopin Bridge. This bend has experienced bank caving that threatens the integrity of a hunting camp access road. Also, concern has been expressed that continued erosion of this bankline could create a hook at the lower end that will result in a more direct attack of the bank at House Bend in the vicinity of the Melinda outlet channel. House Bend is located on the left descending bank immediately downstream of Camp Bend. House Bend is a tighter radius bend (approximately 5000 feet) and has experienced a higher caving rate than Camp Bend. Caving at House Bend has resulted in the partial draining of Jim Smith Lake. Jim Smith Lake is a perched lake located between the Arkansas and White Rivers. Some concern exists that continued channel migration at House Bend could threaten the containment structure and the historic cutoff closure dam.

The bankline migration analysis included comparing the bankline location at these two bends overtime and determining an annual caving rate. Caving rates can vary from year to year due to varying hydrologic and geologic conditions along a bankline. Also caving rates along a single bend vary greatly depending upon location within the bend. Therefore, caving rates were determined at various locations around each bend. Bankline locations were determined from historic aerial photography that was flown in 1949, 1976, 1988, 1991, 1994, and 1999. Table 7-1 provides average caving rates at Camp and House Bends. Figure 7-1 provides an aerial view of the bankline locations as they existed in 1949, 1976, 1988, 1991, 1994, and 1999. Once caving rates were determined, bankline locations were projected into the future. Bankline projections become less reliable the further the projections are into the future. As discussed earlier, bankline migration rates are dependent upon a complex combination of factors. Unpredictable

Table 7-1
Average Bank Caving Rates at Camp Bend and House Bend

Location	Time	Time Difference (years)	Bankline Migration (feet)	Caving Rate (feet/year)
Camp Bend	September 1999			
		5.00	200	40
	September 1994			
		2.92	125	43
	October 1991			
		3.50	75	21
	March/April 1988			
		11.50	300	26
	October 1976			
House Bend	September 1999			
		5.00	450	90
	September 1994			
		2.92	400	137
	October 1991			
		3.50	350	100
	March/April 1988			
		11.5	500	44
	October 1976			
		26.92	1450	54
	November 1949			

changes in any or all of these factors can greatly alter future migration. Figure 7-2 provides the projected bankline locations for Camp Bend and House bend at 5 years, 10 years, and 20 years into the future. Since the most recent available aerial photography was flown in September 1999, the 5-year bankline projection would be the predicted location in the year 2004, 10- year projection in the year 2009, and the 20-year projection in the year 2019. As shown on this figure, the point at the downstream end of Camp Bend should eventually erode. Available aerial photography indicates that this point has experienced little erosion between 1976 and 1999. However, the 1949 aerial photography shows that the 1999 point was located within the channel (below top bank) as it existed at that time. Therefore, the point appears to be alluvial deposits rather than a geologic hardpoint. Also, this bankline was visually inspected during the October and November 2002 field reconnaissance. This inspection did not reveal any discernable difference in the exposed bank material at this point than any of the other upstream portions of this bank. The erosion that occurs upstream of this point appears to be a direct function of the angle of attack of the flow as it exits the Yancopin Bridge. A detailed geologic investigation would be required to determine precisely what materials exist at the downstream end of Camp Bend but the detailed planform analysis and field

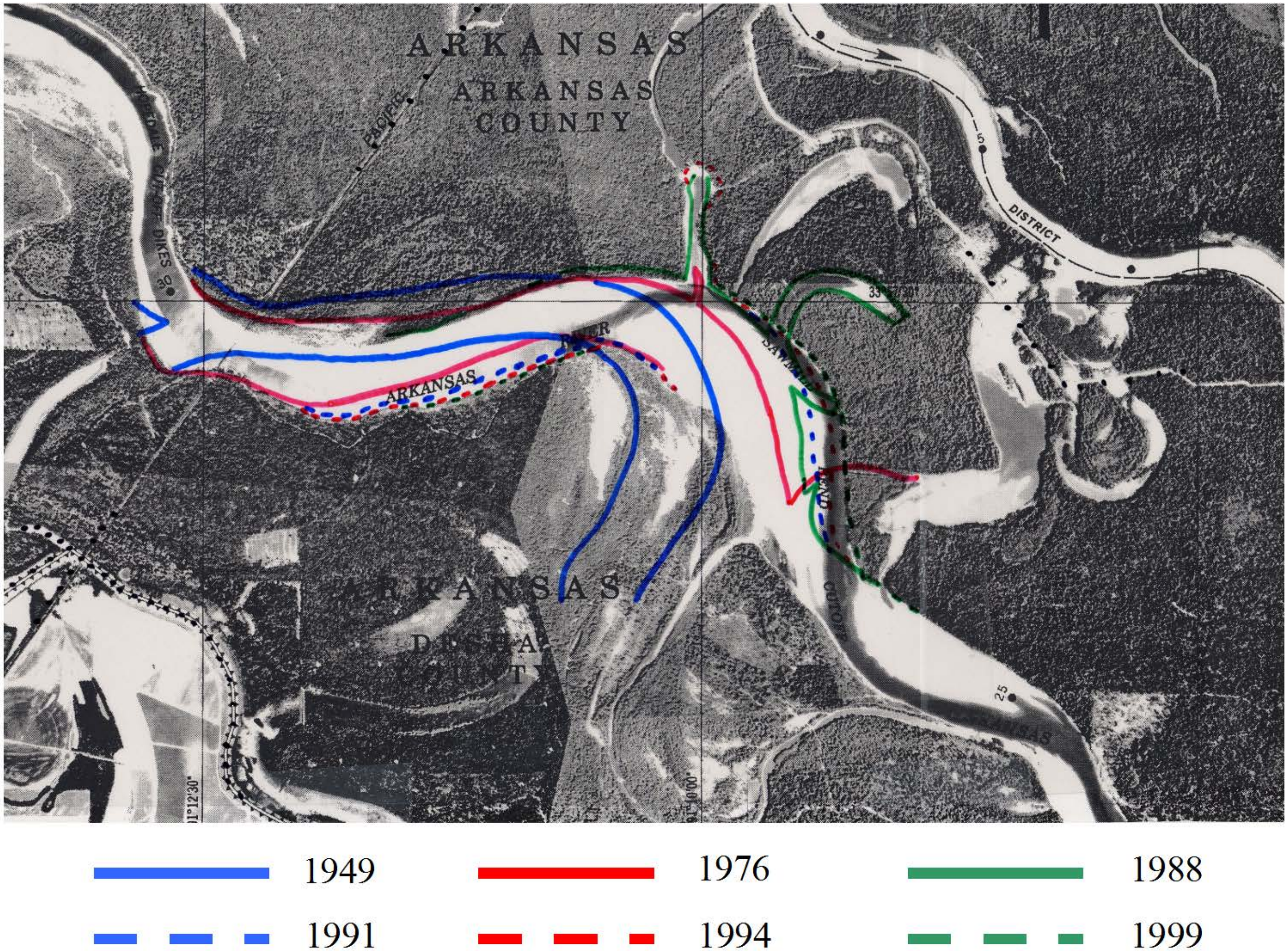

Figure 7-1 Historic Bankline Locations at Camp Bend and House Bend

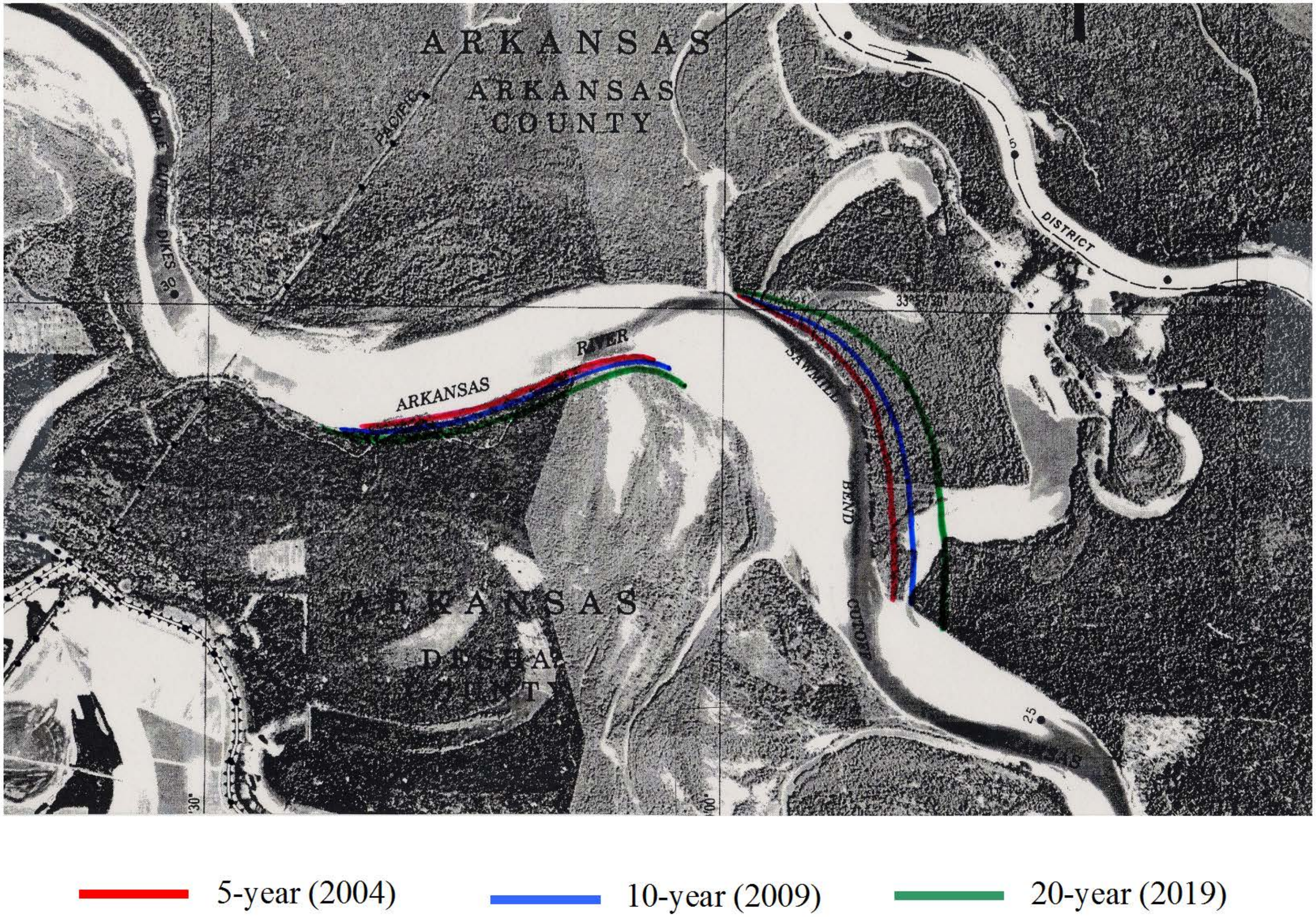

Figure 7-2 Projected Bankline Locations at Camp Bend and House Bend

investigation support the conclusion that the downstream point will erode as shown in Figure 7-2.

At House Bend, the bank retreat that has occurred is typical of alluvial streams. The primary migration is lateral and down-valley. As a result, continued erosion of this bank will result in the bend migrating toward the lower end of the historic cutoff. There is no evidence to suggest that continued bankline migration would threaten the remaining Jim Smith Lake, the containment structure, or the historic cutoff closure dam. However, unless a structure is constructed across the outlet of Jim Smith Lake, erosion of the Arkansas River bankline at this location will continue due to overbank flow from the lake.

8. Results and Discussion

There is no universally accepted definition of a stable river. Often the stability of a river is based on individual perceptions and is frequently debated. However, a practical geomorphic definition of a stable river is one that has adjusted its width, depth, and slope so that there is no significant aggradation or degradation of the bed or significant planform changes (meandering to braided, etc.) within the engineering time frame (generally less than about 50 years). By this definition, a stable river is not static but rather is in a state of dynamic equilibrium. In this state, rivers adjust laterally through bank erosion and bar building. The equilibrium concept is also described by various qualitative relationships. One of the most widely used relationships is the one proposed by Lane (1955). This relationship is:

$$Q\,S \propto Q_s\,D_{50}$$

Where Q is the water discharge, S is the channel slope, Q_s is the bed material load, and D_{50} is the median size of the bed material. This relationship is commonly referred to as Lane's Balance. This concept shows that a change in any one of these variables will result in a change in the other variables such that equilibrium is restored. When a channel is in equilibrium, it will have adjusted these four variables such that the sediment being transported into the reach is transported out, without excessive channel aggradation or degradation. However, the channel is free to migrate laterally. Meandering can be thought of as nature's way of adjusting its energy (slope) to the variables of water and sediment. The occurrence of natural cutoffs is a type of dynamic behavior that is quite common in rivers that are in a state of dynamic equilibrium. As natural cutoffs occur, the river may be gaining additional length by continued meandering with the net result being that the overall reach length, and therefore the slope, remains unchanged. (USACE, 1997)

For this analysis, each of the factors that typically impact channel morphology were investigated. The results of the investigations indicate that channel cutoffs on both the Mississippi River and the lower Arkansas River have had a significant impact on channel morphology on the lower Arkansas River downstream of Dam 2. Definite evidence that the channels on the lower Arkansas River and lower White River have degraded (bed lowering) exists. The specific gage record developed for the Arkansas River at

Yancopin, the annual minimum stage plots for the Arkansas River at Dam 2 tailwater and for the White River at Lock 1 tailwater, the channel thalweg profiles for the lower Arkansas River, and the presence of terraces on both the lower Arkansas River and the lower White River indicate that channel degradation has occurred. The specific gage record at Yancopin was relatively stable from 1932 through the early 1960's. At that point, a downward trend developed. Over the next 15 years, the specific gage record showed a 5 to 6-foot drop. From the mid 1970's until the mid 1990's the Yancopin specific gage record was again relatively stable although there was considerable variability in the data. Beginning in the mid 1990's, the specific gage record took a drastic downward trend. The record shows a 4 to 5 foot drop from the mid 1990's to the end of the period of record in 2001. The specific gage record at Yancopin was developed for low flows only. The higher flow stages are typically associated with backwater on the Mississippi River. Annual minimum tailwater plots were developed for the Arkansas River at Dam 2 and for the White River at Lock 1. Since these structures were put into operation during the 1960's, tailwater stage data is available only for that time to present. The annual minimum tailwater plot for the Arkansas River at Dam 2 shows a definite downward trend that is similar to the trend shown on the Yancopin specific gage record. The annual minimum tailwater plot for the White River at Lock 1 shows an apparent downward trend. However, due to variability in data, the magnitude of this trend is difficult to quantify. The closeness of this gage to the Mississippi River as well as dredging between Lock 1 and the mouth have an impact on low water stages. Therefore, no attempt is made to quantify lowering on the lower White River. Thalweg profiles developed from historic channel surveys on the lower Arkansas River also show that channel degradation has occurred. The surveys available for this assessment were conducted in 1917, 1940, and the 1960's. These profiles show that the bed in 1940 is generally lower than in 1917 and even lower in the 1960's. At Yancopin, the thalweg profiles indicate that the channel degraded approximately 12 feet between 1917 and the 1960's. Further evidence of bed degradation was found during the field investigations. On both the lower Arkansas River and lower White River, numerous terraces were located. Most of these terraces were 10 to 15 feet higher than the new floodplain. Therefore, the specific gage records, minimum tailwater plots, thalweg profiles, and field investigations all support the fact that the lower Arkansas River channel has degraded. As noted above, the Arkansas River appeared to be relatively stable during the mid 1970s to early 1990s, which suggests that the adjustments to cutoffs, the McClellan-Kerr Arkansas River Navigation System, and other changes were essentially complete. However, beginning in the mid 1990s, the stages at Yancopin began to decrease again, which can be attributed to the natural cutoffs at the mouth of the Arkansas River that occurred during the early to mid 1990's. Stage lowering of 4 to 5 feet has been observed since the mid 1990's. However, the question that must be answered is will the stage lowering continue, or has it stabilized? To answer this question, the equilibrium slope concept was employed. The following equation uses equilibrium slope to estimate the magnitude of changes in channel stage.

$$D = (S_o - S_e)\, L_o$$

In this equation, D is the stage change in feet, S_o is the channel slope in feet per mile, S_e is the equilibrium channel slope in feet per mile, and L_o is the channel length in miles.

For this assessment, S_o and L_o are taken at the most recent time for which data is available (1999). This equation was used to compare predicted stage changes with observed changes and to determine if additional stage changes were likely to occur. For the lower Arkansas River (Yancopin to mouth reach), predicted stage changes were computed for the 1976 to 1999 period. Since the available data indicates that the channel was relatively stable from the mid 1970's until about 1990, the channel slope during this time was considered to be the channel's equilibrium slope. Therefore, the channel slope and length in 1999 and the channel slope for each year from 1976 to 1991, except for 1979 and 1988, were used in the stage change equation to determine expected lowering due to the natural cutoffs at the mouth. Suspected inaccuracies in the data in 1979 resulted in not using that year's data. The first step was to select stable hydrologic events for each year with no backwater impact from the Mississippi River. The 1988 data was not used because a good stable hydrologic period could not be found due to the extreme low stages on the Mississippi River during that year. The number of days in the stable hydrologic period for each year ranged from 26 days in 1977 to 47 days in both 1980 and 1987 with an average length of 37 days per year. The slope for each day in each selected hydrologic event was then computed from the observed stage at Yancopin and the computed stage at the mouth of the Arkansas River. The predicted change in stage was then calculated from the equilibrium slope equation. The minimum, average, and maximum slopes and stage change for each hydrologic event (1 per year) were computed. Figure 8-1 is a plot of the predicted stage changes. Since the comparison was made for each year relative to the 1999 condition, a positive stage change represents lowering and a negative stage change represents increased stage. During some of the years, great variability in slope occurs during the single hydrologic events. This variability is shown in Figure 8-1. For instance, in 1987 the predicted stage change ranged from –3.1 feet to 7.8 feet. Variability is also seen between hydrologic events (years). The stage changes ranged from a minimum of –3.7 feet in 1981 to a maximum of 13.0 feet in 1982. The minimum and maximum stage changes represent extremes. The more reasonable expectation would be stage changes in the range of the average values. The average stage change ranged from 1.7 feet in 1981 to 9.5 feet in 1982. The average value of the stage change during the 1976 to 1991 period is 5.1 feet. Therefore, the equilibrium slope analysis, based on the average slope for the 1976 through 1991 period, indicates that the stages at Yancopin should drop about 5.1 feet in order to restore equilibrium. This 5.1 feet of expected stage lowering corresponds well with the Yancopin specific gage record that shows 4 to 5 feet of lowering during the 1990's. Thus, based on the equilibrium slope concept, it can be concluded that the lowering at Yancopin resulting from the natural cutoffs at the mouth should be essentially complete. However, as previously noted, there is considerable uncertainty in these results die to the variability in the data.

The equilibrium slope analysis described above was based on the assumption that the period from about 1976 to 1991 was in dynamic equilibrium, and that the majority of the impacts due to the Mississippi and Arkansas Rivers cutoffs, the McClellan-Kerr Arkansas River Navigation Project, and closure of the historic cutoff were essentially complete. Sorting out the exact contribution of each of these factors is extremely difficult. However, an attempt was made to develop a likely timeline of changes on the Arkansas River and to correlate these changes to the various factors. For this analysis, it was assumed that each one-mile change in length would equate to about a one-foot change in

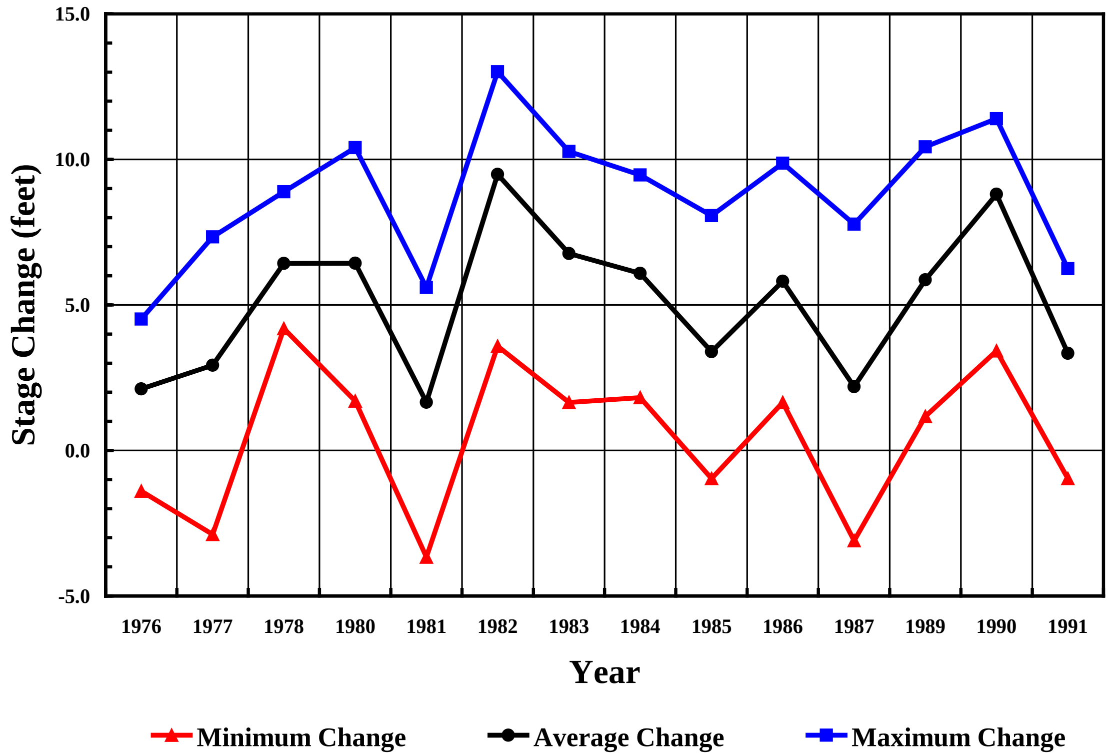

Figure 8-1 Predicted Stage Changes Based on Changes in Channel Slope

stage. This was based on the observed five mile shortening of the lower Arkansas River in the early to mid 1990's and the associated five-foot reduction in stage. Our most detailed, reliable data for both the Mississippi River and the lower Arkansas River begins in the 1930's. Therefore, this assessment will begin at that time and will only focus on the lower flows due to data limitations at the higher flows.

The specific gage record for the Mississippi River at Arkansas City indicates a drop of about 7 to 8 feet at the lower flows from 1930 to 1960. During that same period, the specific gage record for the Arkansas River at Yancopin was relatively stable showing lowering of only 0 to 1 feet. However, the Arkansas River length increased by 6 miles between Yancopin and its mouth during this period. Therefore, between 1930 and 1960, the Arkansas River adjusted to some of the lowering on the Mississippi River by increasing its length. Using the estimated adjustment of 1 foot per mile, the 6 mile increased length on the Arkansas River would account for about 6 feet of lowering on the Mississippi River. Therefore, the 6 mile lengthening seen on the Arkansas River plus the 0 to 1 foot of lowering on the Yancopin specific gage record would account for most of the 7 to 8 feet of lowering that occurred on the Mississippi River. The remaining 1 to 2 feet of lowering appears to have been lagged into the 1960's.

From 1960 through 1975, the specific gage record for the Arkansas River at Yancopin shows a lowering of 4 to 6 feet. Two cutoffs were constructed on the lower Arkansas River during the 1960's between Yancopin and the mouth. These two cutoffs (Sawmill Bend and Avenue Landing) shortened the river channel by about 5 miles. However, during that same period, other reaches of the lower Arkansas River downstream of Yancopin increased in length due to channel migration. This combined shortening and lengthening resulted in a net 1.5 mile reduction in length. The net length reduction can be correlated to another 1.5 feet of lowering. During this period, the specific gage record for the Mississippi River at Arkansas City was relatively stable. Of the 4 to 6 feet of lowering indicated on the Yancopin specific gage record, 1.5 feet can be attributed to the 1.5 mile channel reduction. Another 1 to 2 feet can be attributed to the lag from the pre 1960 period. These factors account for a combined 2.5 to 3.5 feet of the 4 to 6 feet of lowering on the specific gage record. The result is an additional 0.5 to 2.5 feet of lowering that is unaccounted for. During the 1960 to 1975 period, major changes occurred on the lower Arkansas River. During that time, the McClellan-Kerr Arkansas River Navigation Project was put into operation. The impacts of the waterway project, especially Dam 2 operation and the closing of the historic cutoff channel, on the lower Arkansas River channel morphology are not known. Therefore, the unaccounted lowering may represent the response of the lower Arkansas River to the waterway project.

Based on the above scenario, it appears that most of the impacts of the Mississippi and Arkansas Rivers cutoffs, McClellan-Kerr Arkansas River Navigation Project, and the closure of the historic cutoff would have been complete by the mid 1970’s and that future impacts should have been minimal. In fact, this agrees quite well with the lack of any significant trends in the post 1975 period. Once again it must be stated that there is considerable uncertainty in the above scenario, and the results cannot be viewed as absolute. However, this does appear to be a reasonable scenario that is consistent with the observed trends. During the period 1975 to the mid 1990’s, the specific gage record at Yancopin exhibits considerable variability, but there is no discernable trend. It is also important to remember that the channel length during this period did not change. Therefore, it is reasonable to conclude that the Arkansas River had approached a state of dynamic equilibrium during this period.

9. Conclusions.

Based on the data and analyses conducted for this geomorphic assessment, the following conclusions are provided.

1. The lower Arkansas River has experienced significant morphologic changes over the past 75 to 100 years. Channel degradation on the order of 10 to 15 feet has been observed. This lowering can be attributed to a number of factors including flowline lowering on the Mississippi River due to the cutoffs in the 1930’s and early 1940’s, natural and man-made cutoffs on the Arkansas River, and the construction and operation of the McClellan Kerr Arkansas River Navigation System including the closing of the historic cutoff between the Arkansas and White Rivers.

2. The evidence indicates that the majority of the morphologic response along the Arkansas River was completed by the mid 1970’s. From the mid 1970’s to the early, 1990’s, the river appears to have been in a state of dynamic equilibrium. However, stage

lowering occurred again in the mid to late 1990s. During the early to mid 1990's, two natural channel cutoffs occurred near the mouth of the Arkansas River. The analysis suggests that the lowering that occurred during the mid to late 1990's was a result of these cutoffs. Because of the variability in the data, it is difficult to state with certainty what the future response of the Arkansas River will be. However, the analyses conducted as part of this geomorphic assessment indicates that this most recent lowering should be essentially complete but if any future lowering does occur, it should be minimal.

3. Historically, the Arkansas River has adjusted to changes through channel degradation (flowline lowering) and meander migration. Thus, although the Arkansas River exhibits a high degree of meander migration, this migration is simply part of the natural alluvial river adjustment process.

4. Bankline projections indicate that the continued migration of both House and Camp Bends will not threaten the White River, the containment structure, or the historic cutoff closure dam. For this reason, it is suggested that bank stabilization (revetments) along these two bends is not necessary at this time. However, construction of structures across the Arkansas River end and at the headcut at the White River end of Jim Smith Lake are needed to prevent the continued degradation of the lake.

5. The precise impacts of the McClellan Kerr Arkansas River Navigation System (MCKARNS) on the channel morphology of the lower Arkansas River are unknown. This is specifically true of the impact on sediment conditions resulting from the operation of the MCKARNS (including Dam 2) and from the construction of the historic cutoff channel closure dam. However, due to the large volume of sand found within the channel immediately downstream of Dam 2, it is difficult to conclude that the lower Arkansas River downstream of Dam 2 is sediment starved.

6. The Mississippi River at Arkansas City appears to have attained a state of dynamic equilibrium, and therefore, further degradation should be minimal. At Helena, the channel was experiencing degradation from the 1950s to early 1990s. However, stages have been elevated through the mid to late 1990s, possibly resulting from the unusual hydrologic events during this period.

7. The annual minimum stage plot for the White River at Lock 1 tailwater includes so much annual variation that a definite trend is difficult to determine. While some morphological change is noted on the White River downstream Lock 1, the variability of the data for this reach greatly limits the identification of causes of morphological changes and the prediction of future changes.

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