Claiborne and Millers Ferry Locks and Dams Fish Passage Study

Appendix H – Hydrology and Hydraulics

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APPENDIX-H: Hydrology and Hydraulics

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H.1. Study Area

H.1.1. Project Area

The project reach includes two existing USACE lock and dam projects, Claiborne Lock and Dam (Claiborne) and Millers Ferry Lock and Dam (Millers Ferry).

H.1.1.1. Claiborne Lock and Dam

Claiborne is the southernmost lock and dam on the Alabama River and was constructed between 1966 and 1970. The project is primarily a navigation structure , but also reregulates the peaking power releases from the upstream Millers Ferry Project. Other project purposes include water quality, recreation, and fish and wildlife conservation and mitigation. There is no flood risk management storage for this project. Its features include a lock, fixed crest spillway, gated spillway and right and left dikes as depicted on Figure A 2.

H.1.1.2. Millers Ferry Lock and Dam

Millers Ferry is upstream of Claiborne on the Alabama River and was constructed between 1964 and 1970. The project purposes include hydropower and navigation. Other project purposes include recreation, water quality, and fish and wildlife conservation and mitigation. There is no flood risk management storage for this project. Its features include a lock, powerhouse, gated spillway, and right and left dikes as depicted in Figure A 3.

H.1.2. Pertinent Data

H.1.2.1. Claiborne Lock and Dam

GENERAL

Location – Clarke, Monroe, and Wilcox Counties, Alabama; Alabama River, river mile 72.5

Drainage area Millers Ferry to Claiborne – sq. mi.	836
Total drainage area above Claiborne Dam site – sq. mi	21,473
Maximum Static Head (feet)	30

RESERVOIR

Length at elevation 36.0 feet NGVD29 – miles	60.5
Area at pool elevation 36.0 feet NGVD29 – acres	6,290
Total volume at elevation 36.0 feet NGVD29 – acre-feet	102,480

GATED SPILLWAY

Total length, including end piers – feet	416
Elevation of crest – feet NGVD29	15.0
Number of gates	6
Type of gates	Tainter
Size of gates – feet	60x21
Elevation of top of gates in closed position – feet NGVD29	36.0

FIXED CREST SPILLWAY

Length – feet	500
Elevation of ogee crest – feet NGVD29	33.0
Type of stilling basin	Roller bucket

EARTH DIKES

Right Bank Dike	
Total length – feet	200
Total width – feet	25.0
Top elevation – feet NGVD29	40.0
Side slopes	1v to 3h
Left Bank Dike	
Total length including esplanade and ramp – feet	2,350
Total width – feet	32.0
Top elevation – feet NGVD29	60.0
Side slopes	1v to 4h

LOCK

Maximum lift – feet	30.0
Chamber width by length – feet	84 x 600

H.1.2.2. Millers Ferry Lock and Dam

GENERAL

Location – Dallas and Wilcox Counties, Alabama; Alabama River, river mile 133.0

Drainage area R.F. Henry to Millers Ferry – sq. mi.	4,404
Total drainage area above Claiborne Dam site – sq. mi	20,637

RESERVOIR

Maximum operating pool elevation – feet NGVD29	80.8
Length at elevation 80.8 feet NGVD29 – miles	105
Area at pool elevation 80.8 feet NGVD29 – acres	18,528
Total conservation volume between elevation 78.0 – 80.8 feet NGVD29 – acre-feet	102,480

GATED SPILLWAY

Total length, including end piers – feet	994
Number of piers, including end piers	18
Elevation of crest – feet NGVD29	46.0
Number of gates	17
Type of gates	Tainter
Size of gates – feet	50x35
Elevation of top of gates in closed position – feet NGVD29	81.0

EARTH OVERFLOW DIKES

Right Bank Dike (overtopped at approximately 240,000 cfs)	
Total length – feet	3,360
Total width – feet	85.0
Top elevation – feet NGVD29	25.0
Side slopes	1v to 2.5h
Left Bank Dike (overtopped at approximately 525,000 cfs	
Total length including lock mound – feet	5,500
Total width – feet	32.0
Top elevation – feet NGVD29	97.00
Side slopes	1v to 2.5h

LOCK

Maximum lift – feet	48.8
Chamber width by length – feet	84 x 600

POWER PLANT

Number of units	3
Generator rating, 3 units @ 30,000 each – kW	90,000
Maximum static head – feet	48.0

H.1.3. Watershed Characteristics

H.1.3.1. Drainage Area Description

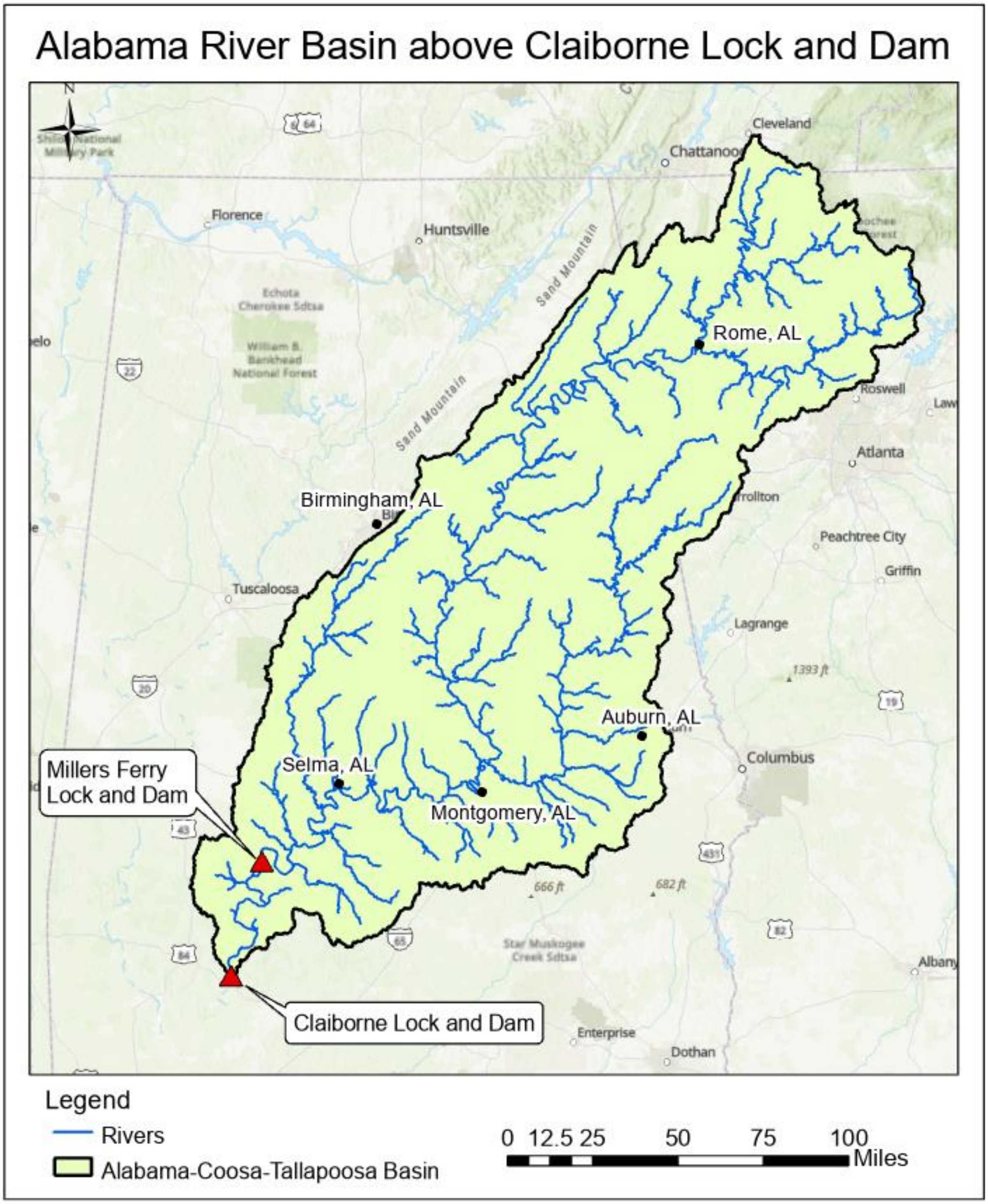
The Alabama-Coosa-Tallapoosa (ACT) River System drains a small portion of Tennessee, northwestern Georgia, and northeastern and east-central Alabama. The Alabama River Basin has its source in the Blue Ridge Mountains of northwest Georgia. The main headwater tributaries are the Oostanaula and Etowah Rivers, which join near Rome, Georgia, to form the Coosa River. The Coosa River in turn joins the Tallapoosa River near Wetumpka, Alabama, approximately 14 miles above Montgomery, Alabama, to form the Alabama River.

The upper and middle ACT basin have several federal and private dams located on the main stem rivers. There are six flood risk management projects located on these systems. They are, Allatoona Dam, Carters Dam, owned and operated by USACE, and Weiss Dam, Logan Martin Dam, H.N. Henry Dam and Harris Dam, owned and operated by the Alabama Power Company. While these provide a great deal of flood protection for moderate flood events directly downstream of each structure, they provide very little peak stage and flow reduction on the Alabama River and are not intended to do so. There are three run-of-river projects located on the Alabama River including Robert F. Henry Lock and Dam, Millers Ferry Lock and Dam, and Claiborne Lock and Dam.

Millers Ferry and Claiborne Lock and Dams are located on the Alabama River at river miles (RM) 142.25 and 81.78, respectively (above the confluence of the Tombigbee and

Alabama Rivers, which form the Mobile River in southwestern Alabama). Above Millers Ferry, the Alabama River Basin has a total drainage area of 20,637 square miles. Claiborne Lock and Dam has a total drainage area of 21,473 square miles (shown on Figure H.1).

Figure H.1: Alabama River Basin map showing drainage basin upstream of Claiborne Lock and Dam.



H.1.4. Available Data

Four (4) United States Geological Survey (USGS) stream gages and one (1) discharge gage from Mobile District Water Management were utilized for the hydrologic and hydraulic analysis of this study. The gages include USGS 02427505 Alabama River at Millers Ferry Dam near Camden, AL, USGS 02427506 Alabama River below Millers Ferry Lock and Dam, USGS 02428400 Alabama River at Claiborne Lock and Dam near Monroeville, AL, USGS 02428401 Alabama River below Claiborne Lock and Dam and Mobile District Water Management Discharge at Millers Ferry Lock and Dam. Table H.1 shows the datum conversion for each location.

Table H.1: Datum conversion from NAVD88 to NGVD29 for each gage.

Location	NAVD88-NGVD29 (FT)
Miller's Ferry Lock and Dam	0.16
Claiborne Lock and Dam	0.09

H.1.5. Hydrology/Runoff Characteristics

H.1.5.1. Temperature

The average daily low and high temperatures in the study area range from the mid to upper-30s/low-40s to upper-50s/low-60s (in ˚F) for the winter months and the high-60s to the upper-80s/low-90s in the summer months. (US Climate Data, 2022)

H.1.5.2. Rainfall

The average annual precipitation at Camden, AL is approximately 57.02 inches. Camden, AL is located ~10 miles southeast of Millers Ferry Lock and Dam. Monthly precipitation averages range from a low of 2.64 inches in October to a high of 6.54 inches in March (US Climate Data, 2022). The average annual precipitation at Jackson, AL is approximately 60.18 inches. Jackson, AL is located ~21 miles southwest of Claiborne Lock and Dam. Monthly precipitation averages range from a low of 3.49 inches in October to a high of 6.03 inches in March.

For Millers Ferry Lock and Dam, synthetic rainfall data for the study area per National Oceanic Administration (NOAA) Atlas 14, present rainfall depths range from 0.463 inches for the 1-year 5-minute storm to 13.4 inches for the 500-year 24-hour storm. For Claiborne Lock and Dam, synthetic rainfall data for the study area per National Oceanic Administration (NOAA) Atlas 14, present rainfall depths range from 0.532 inches for the 1-year 5-minute storm to 15.8 inches for the 500-year 24-hour storm.

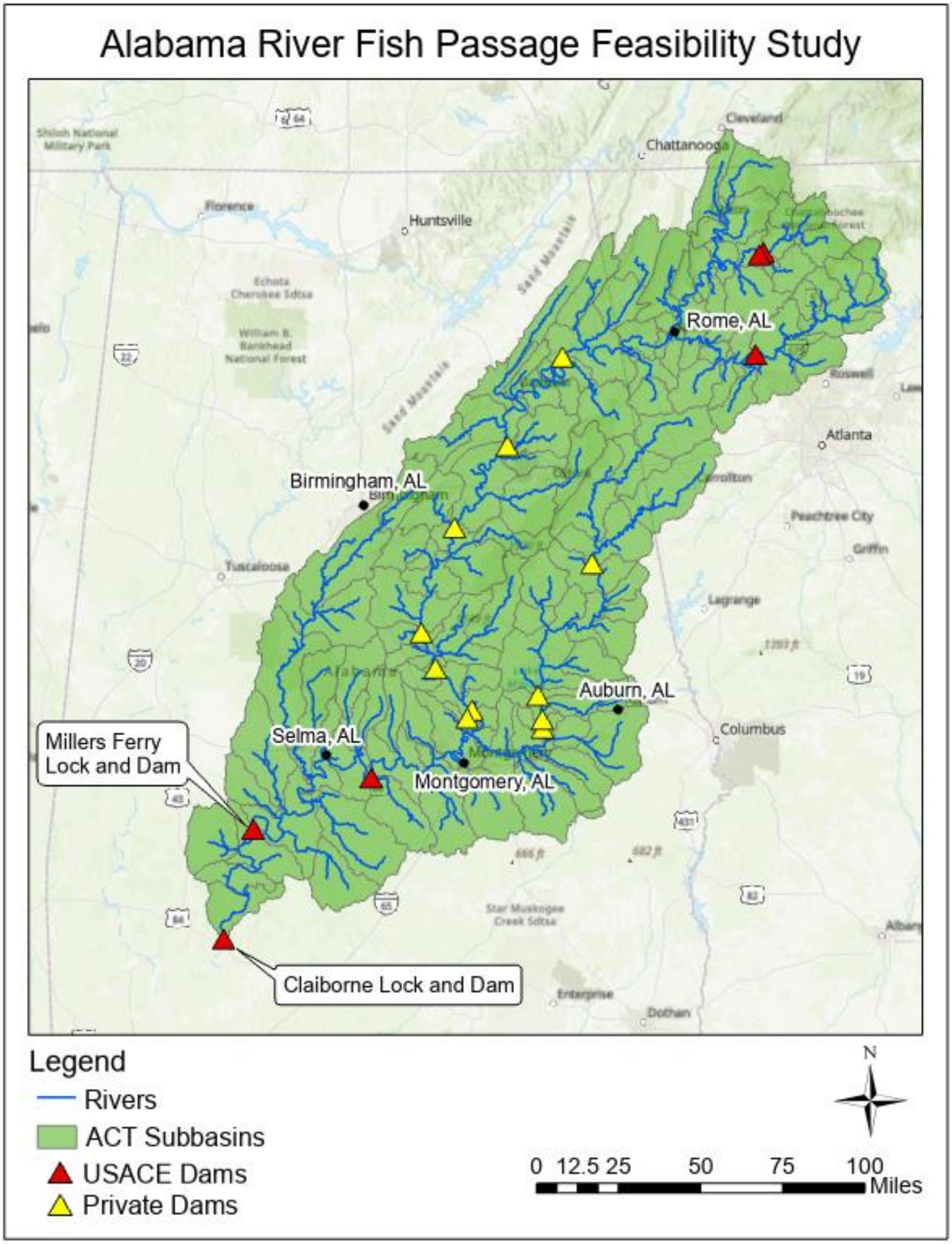
H.1.5.3. Hydrograph Characteristics

The streams which constitute the Alabama River above the study area exhibit wide variations in runoff characteristics from very flashy in the mountainous regions of the Coosa Basin above Rome, Georgia to very slow rising and falling in the lower reaches which includes the stretch of river near Millers Ferry Lock and Dam. A typical hydrograph in the study area increases slowly over several days before reaching a peak flow and recedes at a slower pace. The headwater elevations in Millers Ferry Pool stay relatively constant unless the event is large, such as the 1990 flooding event (Approximately 0.01 AEP). Large events usually occur over several weeks, sometimes lasting over a month.

H.1.5.4. Hydrologic and Hydraulic Characteristics

The Alabama River Basin is a large, diverse basin consisting primarily of broad wooded areas in the upper basin as well as several large urban areas near and upstream of Selma, AL. Overland flow from rain events and stream conveyance in forested and wooded areas found within the upper basin will result in a slow moving flow whereas water will typically convey much faster in the urban areas due to increased land coverage of impervious areas such as asphalt parking lots and roadways. Urbanization within the Alabama River Basin is primarily occurring in areas such as Rome, GA, Montgomery, AL and the south Birmingham, AL metro region.

Figure H.2: Alabama River Basin and contributing rivers and tributaries.

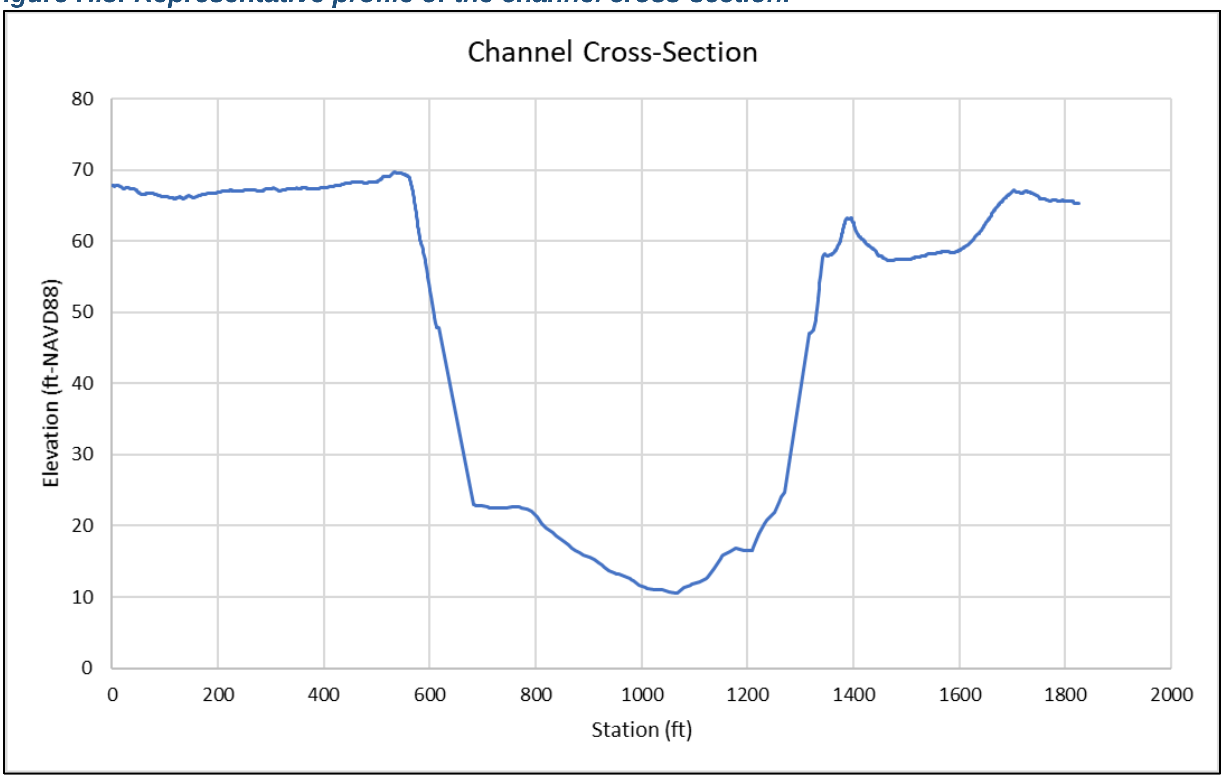


The basin is located over two distinct topographies. The middle and norther portion of the basin is steep and mountainous with narrow floodplains, causing streamflow to be flashier with short, acute high flow events. The southern portion of the basin below Montgomery, Alabama becomes extremely flat with many sections of wide floodplain. Hydrographs in this area of the basin, including the study area, are very slow moving.

The Alabama River channel between Millers Ferry and Claiborne Lock and Dams is generally 30 to 35 feet deep with widths ranging from 400 to 700 feet at bank-full capacity. There is some vegetation on the slopes of the river and the banks along this stretch of river are steep. Figure H.3 shows a typical profile of the channel between Millers Ferry and Claiborne Lock and Dams. The river is fairly clear of debris during normal flowing conditions with some debris built up behind the lock and dams from larger flow events. The floodplain of the Alabama River ranges from cleared farmland to densely vegetated forests. Roughness coefficients (Manning's n-values) used in modeling ranged from 0.035 – 0.045 for the channel section. Roughness on the overbanks and floodplain ranged from 0.03 – 0.15.

The channel is fairly consistent between Millers Ferry and Claiborne Lock and Dams as this channel has historically been dredged for navigation. This dredging is limited to areas under the normal water level of the river within the navigation channel. Historically the area of the river has been dredged up to annually, depending on need. Millers Ferry and Claiborne Pools have not been dredged since 2007 and downstream of Claiborne was dredged between 2014 - 2016. Most dredging stopped along the Alabama River in the late 2000s as this waterway was classified as low use. However, there are plans to dredge below Claiborne Lock and Dam in early 2023.

Figure H.3: Representative profile of the channel cross-section.



H.1.5.5. Land Use

In the Alabama River Basin above the project area, there is a large variety of land use including impervious areas within metropolitan areas and forests throughout the basin. Table H.2 shows the breakdown of percentages for each land use type.

Table H.2: Percentage of Alabama River Basin Land Use Types above Selma, AL

Land Use Type	Percentage of Area Above Selma, AL
Open Water	2.1%
Developed, Open Space	6.4%
Developed, Low Intensity	2.3%
Developed, Medium Intensity	0.8%
Developed, High Intensity	0.3%
Barren Land Rock/Sand/Clay	0.3%
Deciduous Forest	33.3%
Evergreen Forest	17.8%
Mixed Forest	6.1%
Shrub/Scrub	7.5%
Grassland/Herbaceous	4.3%
Pasture/Hay	12.6%
Cultivated Crops	3.0%
Woody Wetlands	3.1%
Emergent Herbaceous Wetlands	0.2%

H.2. Climate Change

H.2.1. Introduction

In 2016, USACE issued Engineering and Construction Bulletin No. 2016-25 (hereafter, ECB 2016-25) which mandated climate change be considered for all federally funded projects in planning stages (USACE, 2016). This guidance was updated with ECB 2018-14 (USACE, 2018), which mandates a qualitative analysis of historical climate trends and assessment of future projects. Even if climate change does not appear to be an impact for a particular region of interest, the formal analyses outlined in the guidance, result in better-informed planning and engineering decisions.

H.2.2. Literature Review

A literature review was performed to summarize climate change literature and highlight both observed and projected assessments of climate change variables relevant to the study area. Since this is an ecosystem restoration project with fish passage as the main objective, the primary variable that is relevant is streamflow. However, this variable is also affected by precipitation and air temperature. Therefore, this review focuses on observed and projected changes in precipitation, air temperature, and hydrology.

H.2.2.1. Temperature

H.2.2.1.1. Observed Temperature

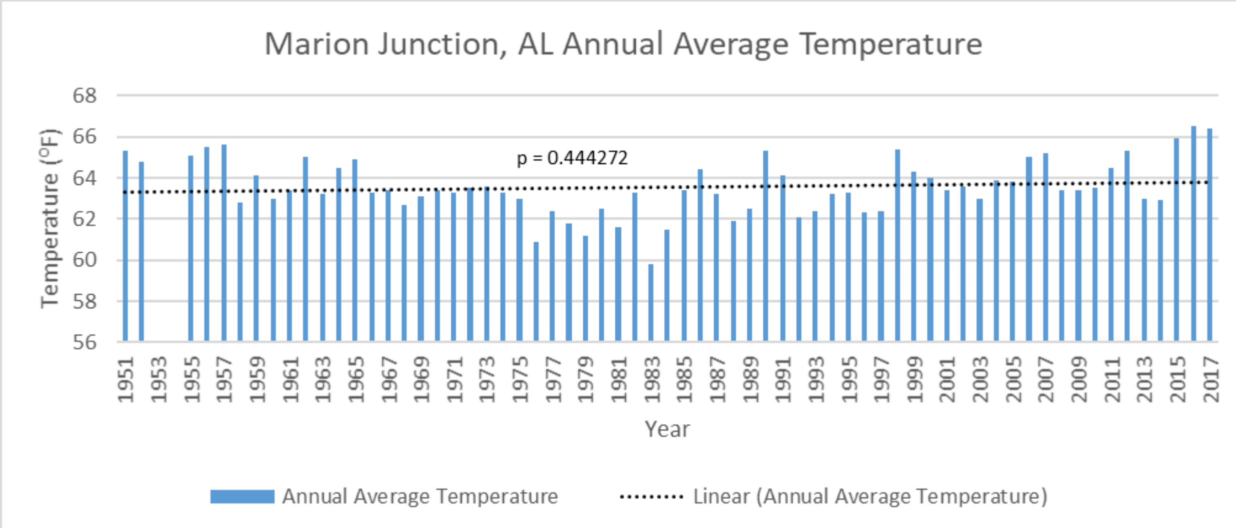
The Fourth National Climate Assessment (USGCRP, 2017) states that observed temperatures in the United States have increased up to 1.9 degrees Fahrenheit since 1895, with an acceleration in increasing temperatures since the 1970s. Warming is projected for all parts of the United States (USGCRP, 2017).

The USACE Institute for Water Resources (IWR) conducted a review in 2015 which summarized the available literature on climate change for the South Atlantic-Gulf Region, including the study area (USACE, 2015). In general, studies have shown that over the last century, a period of warming in the region has been observed since a transition point in the 1970s. This transition period was precluded by an observed cooling period (see Patterson et al., 2012; Laseter et al., 2012; and Dai et al., 2011). The overall warming trend is fairly inconsistent for the region over the last century. The IWR report indicates only mild increases in annual temperature for the region with significant variability. However, there is a clear consensus in general warming since the early 1970s (USACE, 2015).

For the project area, there are a few NOAA gages in proximity of the dam sites with longer than thirty years. The NOAA gage located in Selma, AL (beginning in 1895) was going to be analyzed, however, the dataset has large gaps for the more recent years. The trend from this data shows a decreasing trend, which is inconsistent with the national and regional reports. Therefore, the NOAA gage located in Marion Junction, AL with a record from 1951 - 2017 (continuous record 1955 – 2017) was used to analyze temperature trends in the area.

A statistical analysis was performed on the entire dataset from Marion Junction, AL. Data from the NOAA gage was tabulated in an excel spreadsheet and imported into the USACE Climate Change Preparedness and Resilience Time Series Toolbox. Using the toolbox, a trend analysis was completed to determine the probability value, or p-value for the dataset. The results of this analysis are shown in Figure H.4.

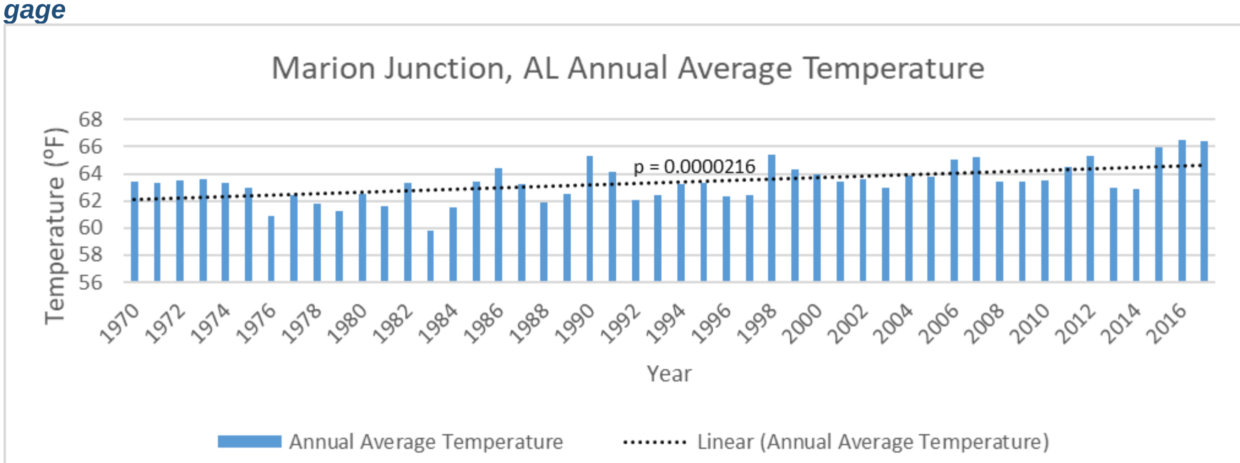
Figure H.4: Annual average temperature and p-value from 1951 - 2017 for Marion Junction, Alabama gage



The alternative hypothesis of an apparent trend is accepted to be true at the 0.05 significance level, meaning that p-values less than 0.05 are indicative of statistical significance and a potential trend within the dataset. This is a threshold commonly adopted within statistical references, but consideration should also be given to trends whose p-values are close to this reference threshold. In this case, the period of record data produces a high p-value of 0.444272; therefore, it is not considered to have a significant increasing or decreasing trend.

However, performing the same test of average annual temperatures from 1970 - 2017 (shown on Figure H.5) produces a p-value of 0.0000216. This would be considered very indicative of a statistically significant upward trend in temperatures.

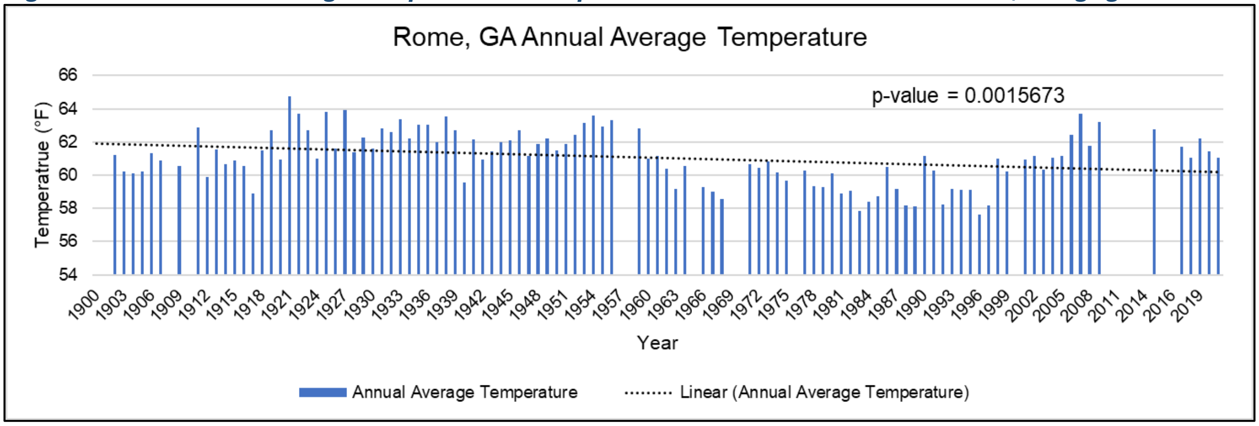
Figure H.5: Annual average temperature and p-value from 1970 - 2017 for Marion Junction, Alabama gage



The temperature gage located in Rome, GA was also analyzed (shown in Figure H.6) This gage is located in the upper portion of the Alabama-Coosa-Tallapoosa Basin. The temperature at this gage may not be representative of the project area but can have an overall effect on the downstream hydrology at the dam sites. The p-value for the entire period of record is 0.0015673, which indicates the downward trend is statistically significant. However, there is a cooling period that occurred in the 1960s to 1980s that may be skewing the data.

Visually, there appears to be an oscillating pattern with the annual average temperature. The temperatures prior to the cooling period (1970s) look similar to temperatures in the early and mid-1900s. Without longer periods of record to compare with, it is difficult to come up with a conclusion.

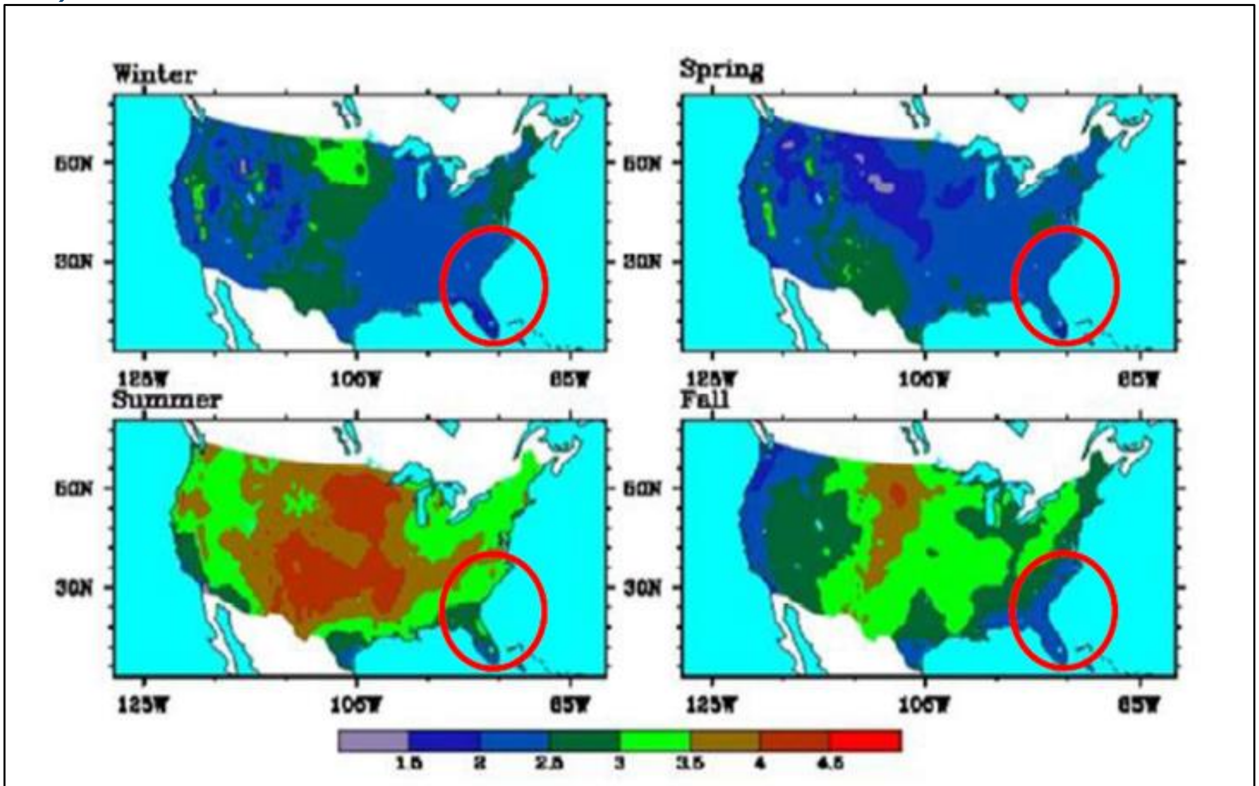
Figure H.6: Annual average temperature and p-value from 1902 - 2018 for Rome, GA gage.



H.2.2.1.2. Projected Temperature

Global Circulation/Climate Models (GCMs) have been used to project future climate conditions in the U.S. including the southeast regions. Results show a significant warming trend at a national and regional scale. Figure H.7 shows the projected changes in seasonal maximum air temperatures from Liu et al. (2013), which is based on a "worst case" greenhouse gas emissions scenario. This shows that, overall, there is a projected warming trend of 2 to almost 4 degrees by 2070.

Figure H.7: Projected changes in seasonal maximum air temperature, ⁰C, 2041 – 2070 vs. 1971 – 2000. The South Atlantic-Gulf Region is within the red oval (Liu et al., 2013; reprinted from USACE, 2015)



H.2.2.2. Precipitation

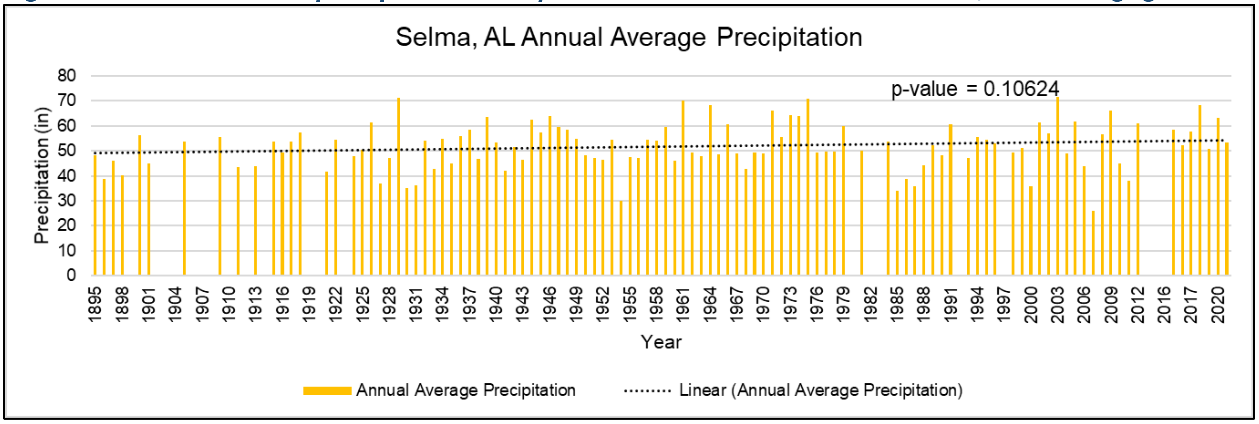
H.2.2.2.1. Observed Precipitation

The IWR report (USACE, 2015) shows there is a general increase in precipitation for the southeast region; however, it is highly variable for the region. Analysis of gridded data spanning years 1950-2000 showed that winter precipitation has consistently increased over the last century (Wang et al., 2009). Other seasons have shown high variability including increases, decreases, and little change in precipitation across the region.

A study by Patterson et al. (2012) did not identify any patterns of precipitation change using monthly and annual trend analysis for a number of climate and streamflow stations within the South Atlantic-Gulf Region (data included 1934 - 2005). However, the study found that more sites exhibited mild increases in precipitation than those that exhibited decreases.

USGS gage data recorded in Selma, AL was tabulated within an excel spreadsheet and imported into the USACE Climate Change Preparedness and Resilience Time Series Toolbox. Using the toolbox, a trend analysis was completed to determine the probability value, or p-value for the dataset. The gage has a large record for precipitation spanning from 1895 – 2021, however, the p-value is 0.10624 which means there is no statistical significance (see Figure H.8). Visually, the dataset seems to be consistent with high and low values being similar throughout the entire record. It appears that there are more low values and extremes in general for precipitation in recent years, even though the trend appears to increase overall.

Figure H.8: Annual total precipitation and p-value from 1895 - 2018 for Selma, Alabama gage

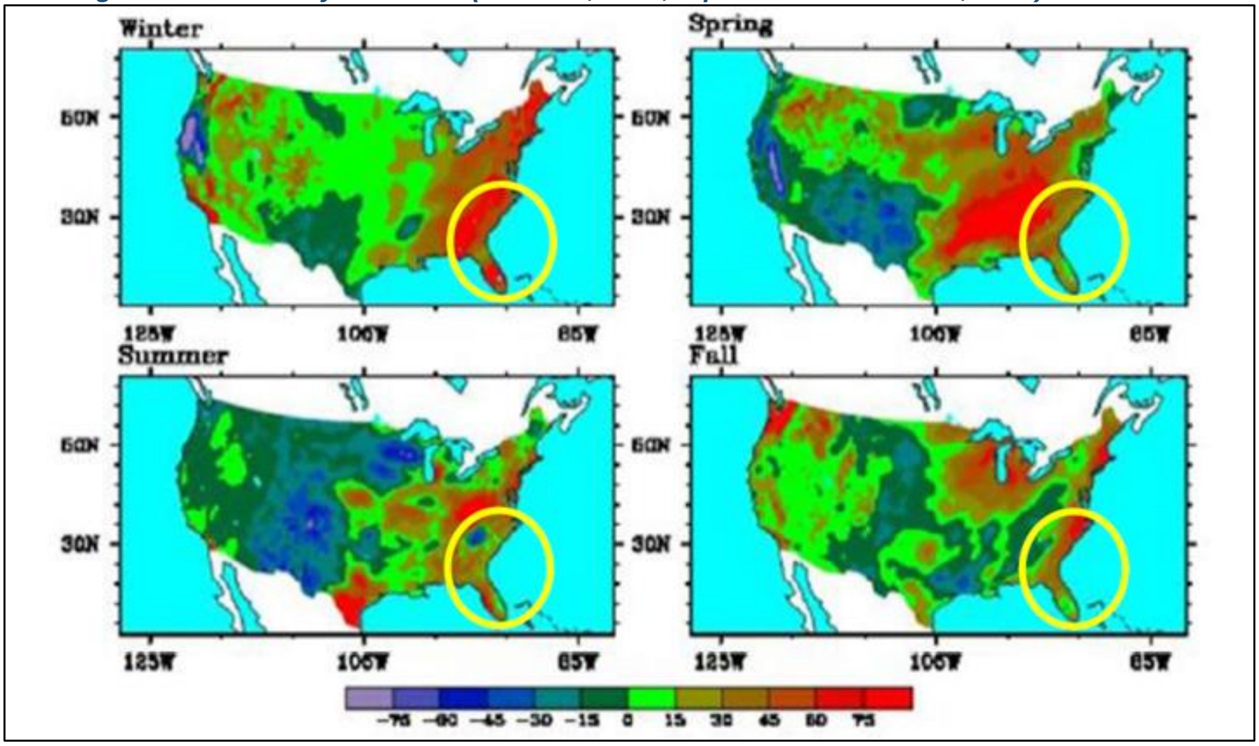


Most studies analyzed by the IWR (USACE, 2015) suggests significance in increasing precipitation severity and frequency trends in observed storms are not definitive. Some of the analyzed literature shows mild increasing trends in these parameters. For instance, Li et al. (2011) investigated anomalous precipitation (based on deviation from the mean) in summer months in the southeastern U.S. and found a greater number of climate stations within the region exhibited increasing trends in the frequency of occurrence of heavy rainfall. Increases were also shown by Wang and Killick (2013), who found that 20% sites analyzed within 56 southeastern watersheds exhibited increasing trends for the 90th quantile precipitation months. There is not a strong consensus regarding trends in extreme precipitation events, however, it is important to remain mindful of the identified increasing trends in intensity and frequency of rainfall within the region.

H.2.2.2.2. Projected Precipitation

Projected future changes in precipitation for the southeast region are variable and lack consensus. Liu et al. (2013) quantified significant increases in winter and spring precipitation associated with a 2055 future condition for the South Atlantic Region. However, other seasons showed almost no increase or a slight decrease in precipitation. Figure H.9 illustrates the projected change in seasonal precipitation. Liu et al. (2013) also project increases in the severity of future droughts for the region, leading to projected temperature and evapotranspiration impacts that outweigh the increases in precipitation.

Figure H.9: Projected changes in seasonal precipitation, 2055 vs. 1985, mm. The South Atlantic-Gulf Region is within the yellow oval (Liu et al., 2013; reprinted from USACE, 2015)



H.2.2.3. Hydrology

H.2.2.3.1. Observed Streamflow

Generalized observations of streamflow trends in the southeast lack a clear consensus, with some models showing positive trends in some areas and others showing negative. Generally, most studies in the southeast area revealed either no trend or a slight negative trend in streamflow. Most notably, studies indicated that the negative trend in streamflow being more consistent for the region since the 1970s (Kalra et al., 2008; and Patterson et al., 2012).

For the Alabama River there is a noticeable decreasing trend for streamflow based on the excel analysis. At the gage upstream of the study area USGS 02420000 near Montgomery, AL, the p-value is 0.004737 indicating the trend is statistically significant (Figure H.10). At USGS 02428400, Alabama River at Claiborne L&D near Monroeville, there is a decreasing trend as well; however, it is not considered statistically significant (p-value of 0.236750; Figure H.11). The gages indicate that there is decreasing trends in streamflow for the Alabama Basin based on the observed data. This could be the result from flood control projects in the upper portions of the basin.

Figure H.10: Annual Peak Streamflow USGS 02420000 Alabama River near Montgomery, AL

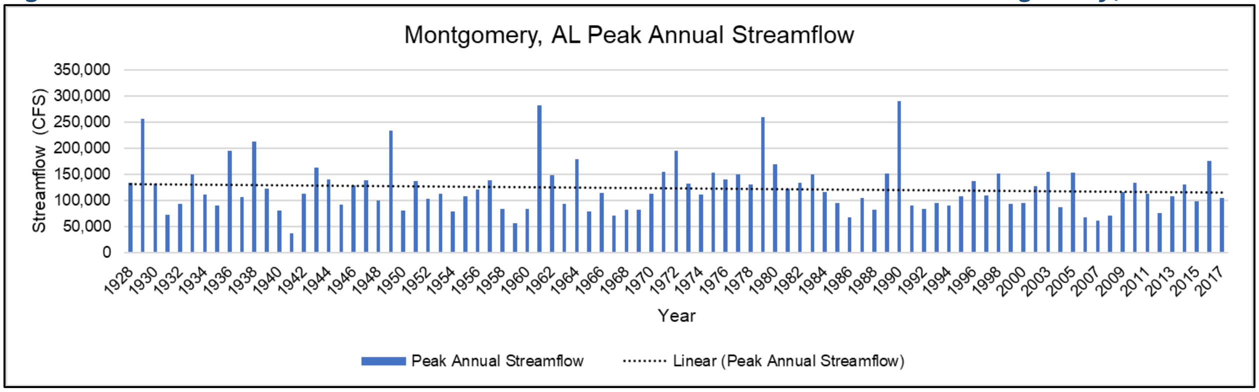
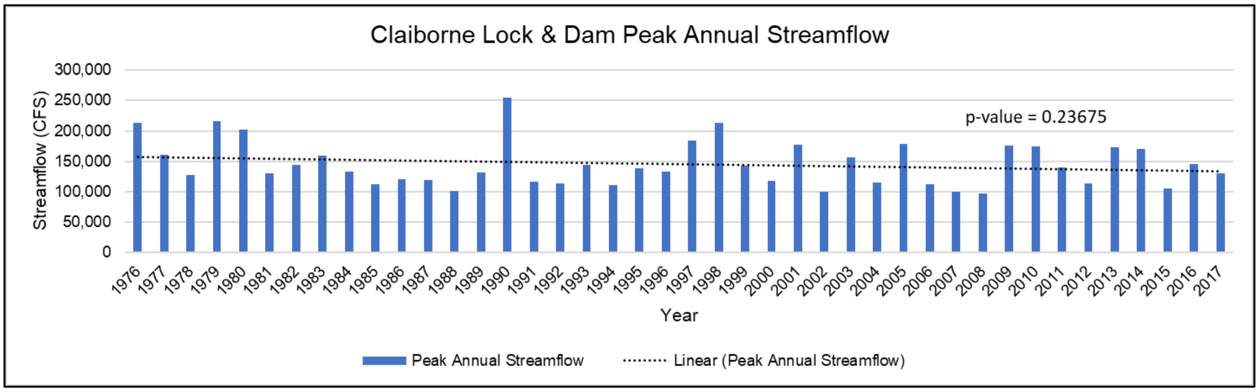


Figure H.11: Annual Peak Streamflow at USGS 02428400 Alabama River at Claiborne L&D near Monroeville



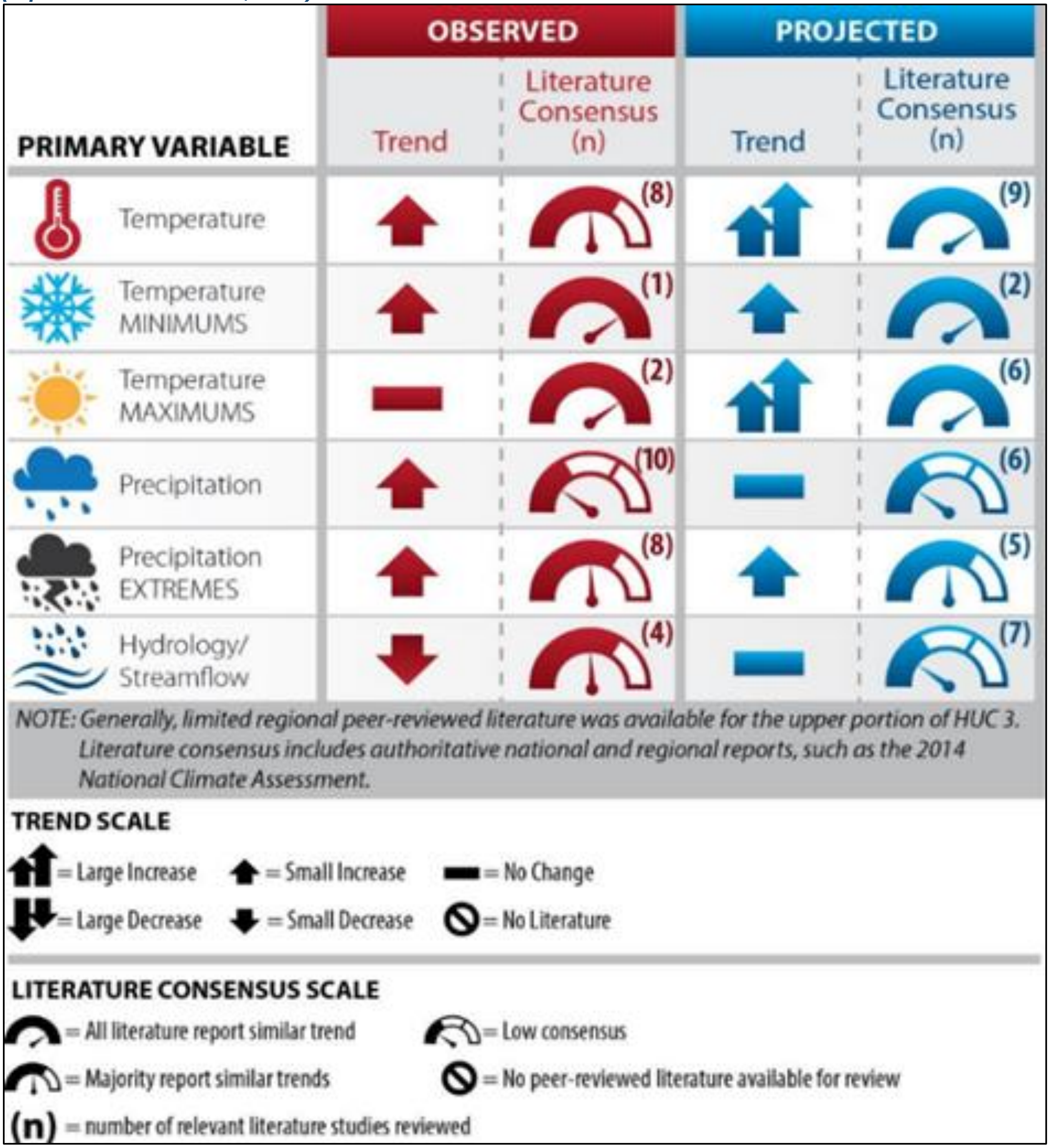
H.2.2.3.2. Projected Streamflow

Review of projected hydrology for the southeast region show that there is very low consensus in projected changes. This is due to the additional uncertainties that are added when coupling climate models to hydrologic models, both of which carry their own uncertainties. Overall, there are little indications of an increasing or decreasing trend in hydrology based on the reviewed literature presented in IWR report (USACE, 2015).

H.2.2.4. Summary

Figure H.12 shows the discussed variables and their overall consensus in trends for both observed and projected scenarios based on the findings of the 2015 USACE IWR literature synthesis. There is evidence that supports an increasing temperature trend from the observed data and less supporting evidence for trends in precipitation or streamflow for a majority of the region. However, there is some evidence that precipitation is increasing, while streamflow appears to be decreasing in some areas within the region.

Figure H.12: Summary matrix of observed and projected climate trends and literary consensus (reprinted from USACE, 2015)



Projections indicate a strong consensus of an increase in projected temperature of approximately 2 to 4 degrees Celsius by the late 21st century. There is some consensus that precipitation extremes may increase in the future, both in terms of intensity and frequency. However, in general, projections of precipitation have been shown to be highly variable across the region. There is not a consensus regarding the directionality of trends in observed streamflow. Very few conclusions can be drawn regarding future hydrology in the region largely due to the amount of uncertainly when coupling climate models with hydrology models.

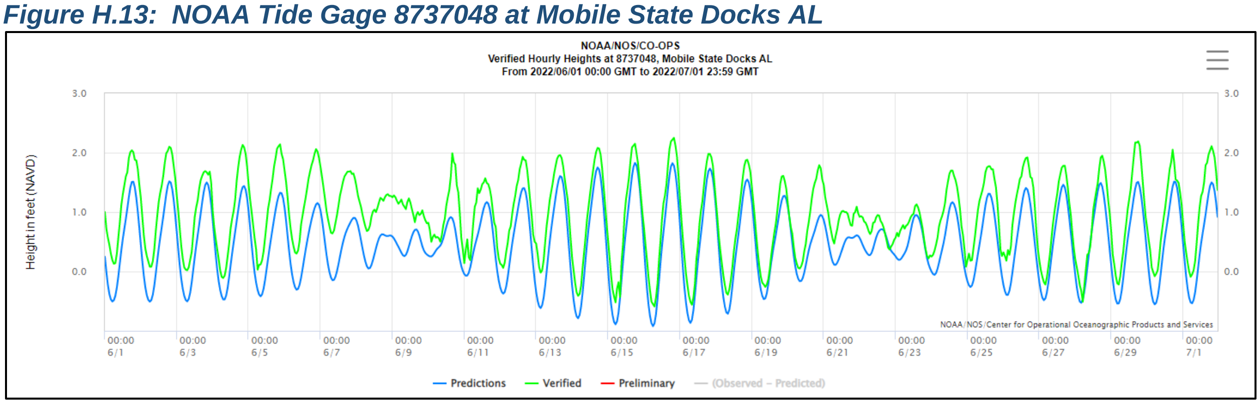
H.2.3. Sea Level Change

Per guidance from ECB 2018-14, for project areas at elevations less than or equal to 50 feet, a determination should be made as to whether sea level change will affect the river stage or performance/operation of the project by increasing (or decreasing) the water surface elevation downstream of the project area. If the project area is affected by sea level change, then policy and procedures outlined in ER 1100-2-8162 will apply.

Millers Ferry Lock and Dam is mostly above this elevation; however, the gated spillway crest is below 50 ft-NAVD88. Since Claiborne Lock and Dam is below Millers Ferry Lock and Dam, an analysis was completed first at Claiborne to determine if sea level change will be a factor at the project. The fixed crest and gated spillway at Claiborne Lock and Dam has elevations at33.1 ft-NAVD88 and 15.1 ft-NAVD88. The tailwater typically fluctuates between 5 to 10 ft-NAVD88 during the summer/autumn months. Claiborne Lock and Dam is located 126.78 miles upstream of the Bankhead Tunnel in Mobile, AL.

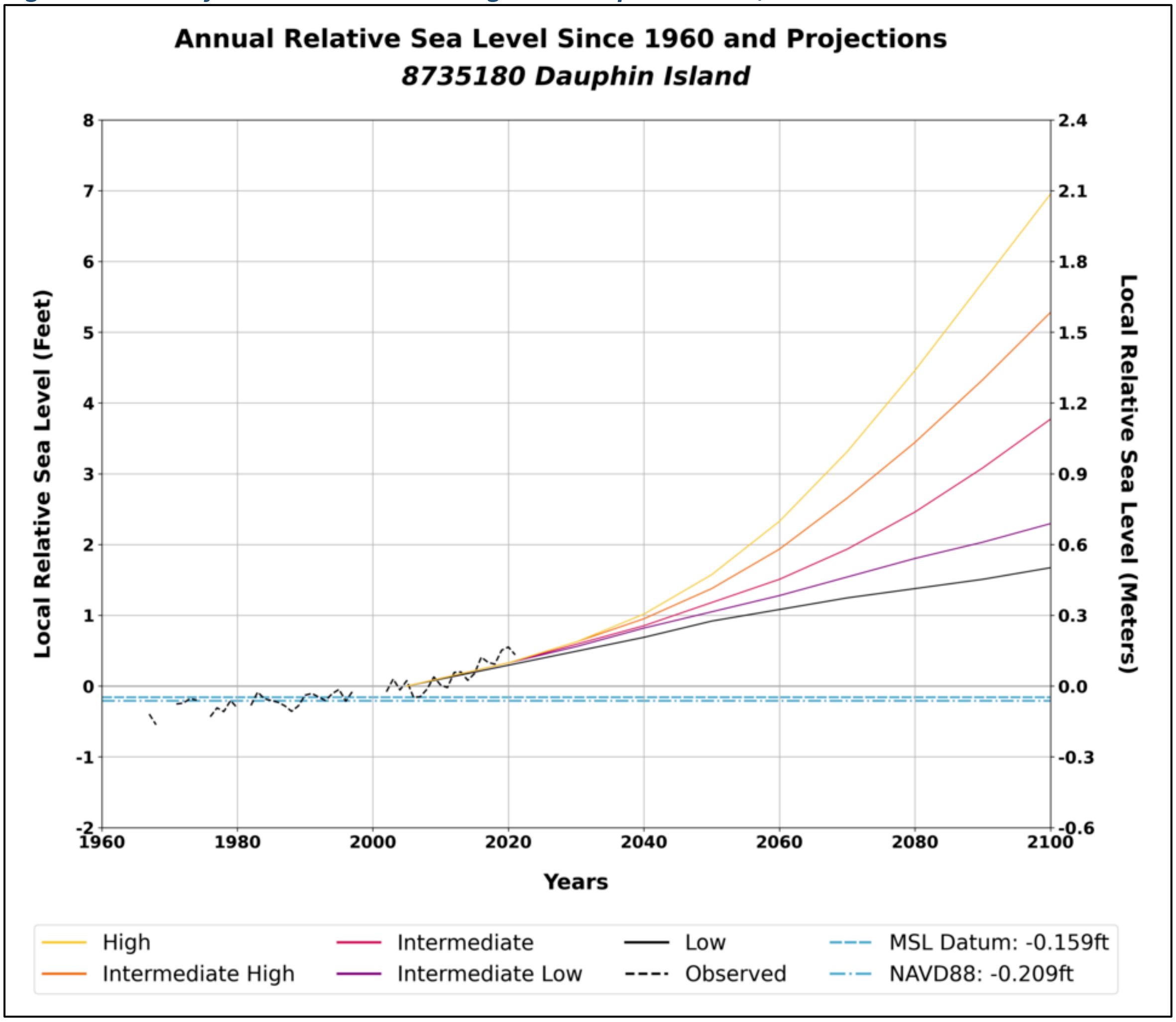
Analysis of the gages around the Claiborne Lock and Dam did not show any effects of current tidal influence. In order to determine what effects sea level change may have on this project, a rough HEC-RAS 2D model was developed. The upper boundary conditions were set to 100 CFS steady flow for both the Alabama River and Tombigbee River. This decision was made due to the lack of bathymetry for the entire system within the model and the river was flowing at bank full (10ft-NAVD88 at Claiborne Lock and Dam tailwater) when the terrain data was obtained. The was determined to be adequate for this level of analysis. The downstream boundary condition was set to NOAA Gage 8737048 Mobile State Docks, Alabama. The period of June 2022 to July 2022 was selected for the existing condition. This period consists of no flood events and is shown below in Figure H.13.

Figure H.13: NOAA Tide Gage 8737048 at Mobile State Docks AL



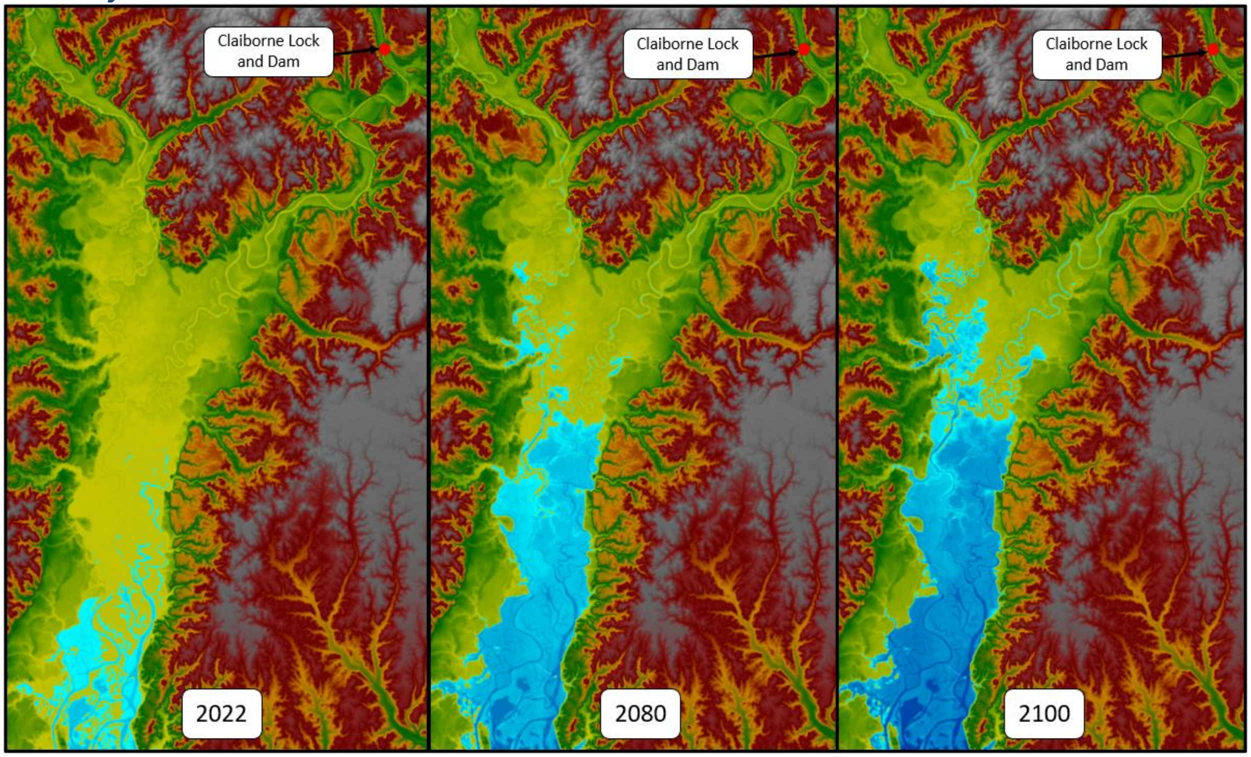
For the Mobile Bay area, only one gage has projections for sea level which is located at Dauphin Island, AL. Based on current trends, the projected path follows the high sea level change curve as seen in Figure H.14. The relative sea level trends between the Mobile State Docks and Dauphin Island are similar, therefore the projections were applied to the Mobile State Docks gage on a 1:1 ratio. The years 2080 and 2100 high projection estimates for relative sea level rise are 4.5 feet and 7.0 feet, respectively. These static projections were added to the existing period to create two sea level rise datasets, and were input into the HEC-RAS 2D model to analyze the upstream effects.

Figure H.14: Projected Sea Level Change for Dauphin Island, AL



Once the models were ran, a comparison of the peak tide for the existing and projected years was compared, shown in Figure H.15. Based on this preliminary analysis, Claiborne Lock and Dam will see little to no tidal impacts related to sea level change while the tailwater is at 10 ft-NAVD88. There is potential to see tidal influence when the tailwater is lower than 10 ft-NAVD88 in the future, but no impacts to the project are anticipated. Since the project area is not affected by sea level change, then policy and procedures outlined in ER 1100-2-8162 will not apply.

Figure H.15: Comparison of Peak Tide between Existing Conditions (2022) and Projected Sea Level Rise for years 2080 and 2100

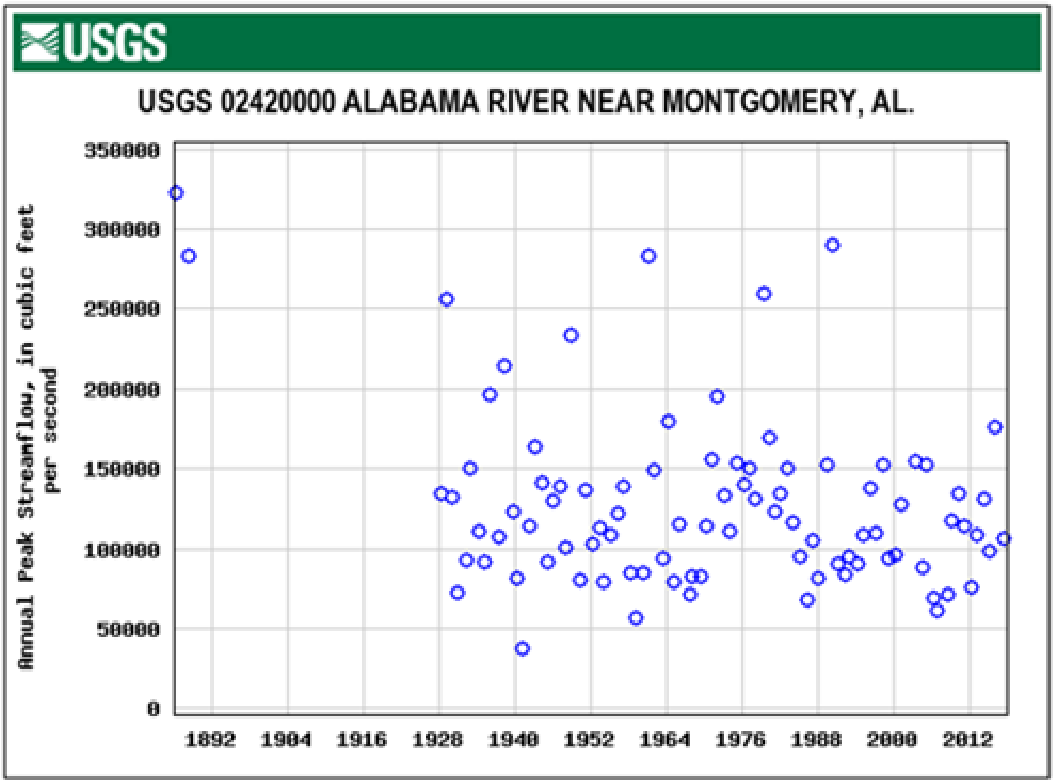


H.2.4. Non-Stationarity Assessment

In accordance with ECB 2018-14, a non-stationarity analysis was performed to determine if there are long-term changes in peak streamflow statistics within the study area and its vicinity. Assessing trends in peak streamflow is considered appropriate as opposed to a focus on precipitation and temperature as one of the primary purposes of this feasibility study is to assess and reduce flooding in the study area. However, trends in these should also be considered as they are both drivers in hydrology.

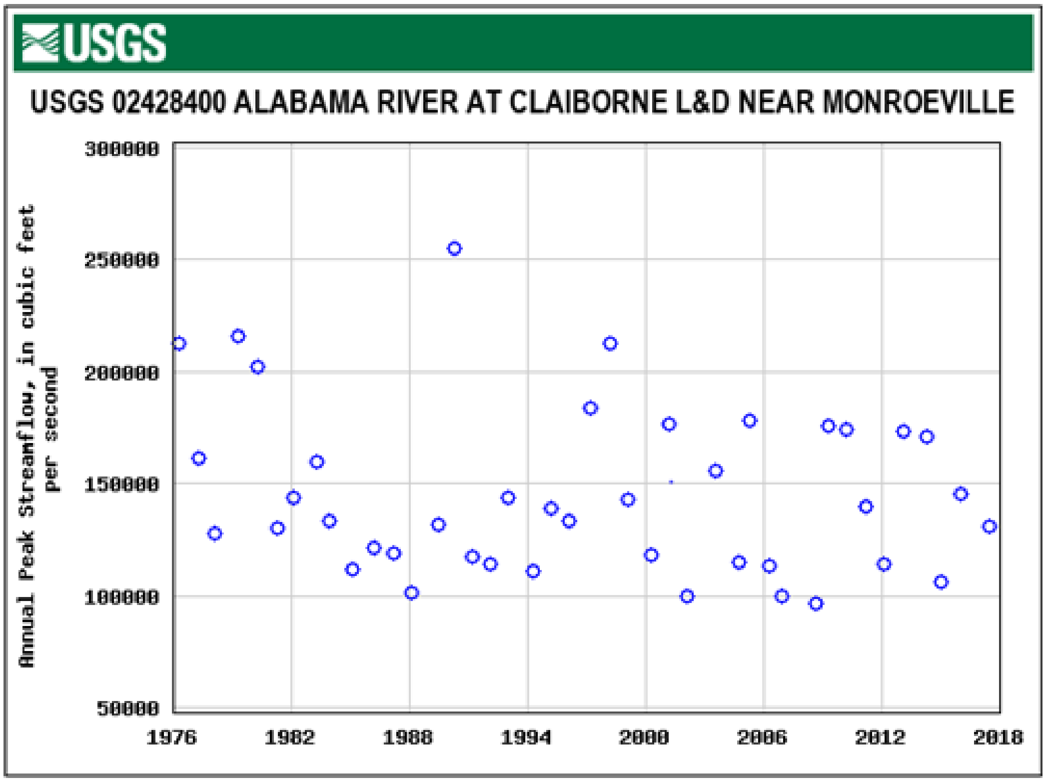
The USACE Non-Stationarity Tool was used to assess possible trends and change points in peak streamflow in the region. USGS 02420000 and USGS 02428400 were used for this analysis. The first gage used in this analysis, USGS 02420000, is located 145 miles upstream of Millers Ferry Lock and Dam on the Alabama River near Montgomery, AL. The gage has a long and nearly continuous record from 1928-2018, includes two historical events, but is missing one year (2003). Figure H.16 shows the time series of Annual Peak Streamflow (APF) for the gage located near Montgomery, AL.

Figure H.16: Annual Peak Streamflow at USGS 02420000 Alabama River near Montgomery, AL.



The second gage used in this analysis was located at Claiborne Lock and Dam, which is located approximately 61 miles downstream from Millers Ferry Lock and Dam. This gage has a continuous record from 1976 to 2017. Figure H.17 shows the time series of APF for the gage located at Claiborne Lock and Dam. To run the USACE Non-Stationarity Tool, it is recommended to have at least 30 continuous years of record. Both of these gages meet that requirement.

Figure H.17: Annual Peak Streamflow at USGS 02428400 Alabama River at Claiborne L&D near Monroeville

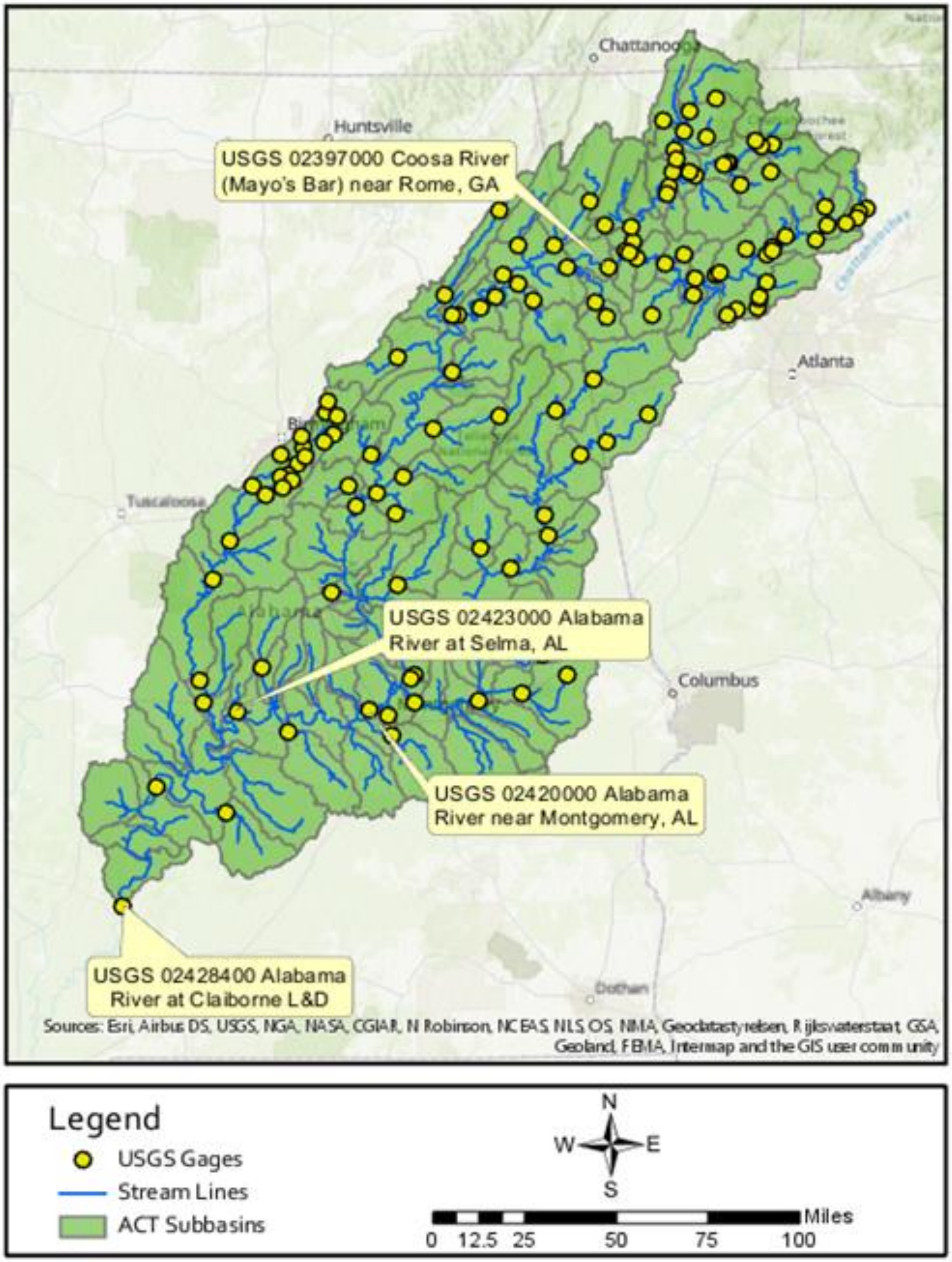


In Figure H.18 the green area encompasses the entire drainage area delineated from Claiborne Lock and Dam and shows all the stream gages available for the entire basin.

The following 16 statistical tests were conducted on the annual peak streamflow time series shown on Figure H.16 and Figure H.17 using the Non-Stationarity Tool:

1. Cramer-von-Mises distribution
2. Kolmogorov-Smirnov distribution
3. LePage distribution
4. Energy Divisive distribution
5. Lombard (Wilcoxon) abrupt mean
6. Pettitt mean
7. Mann-Whitney mean
8. Bayesian mean
9. Lombard (Mood) abrupt variance
10. Mood variance
11. Lombard (Wilcoxon) smooth mean
12. Lombard (Mood) smooth variance
13. Mann-Kendall trend
14. Spearman rank trend
15. Parametric trend
16. Sen’s slope trend

Figure H.18: Study area and locations of the Montgomery, AL gage, Claiborne Lock and Dam gage, Selma, AL gage, and Rome, GA gage used in this analysis



Tests 1-12 are used to detect change points in the distribution, mean, and/or variance of the time series. These non-stationarity tests can be useful in detecting changes in annual instantaneous streamflow peaks driven by natural and human changes in the climate, addition/removal of water control structures, changes in land cover, and any other drivers of non-stationarity. Meanwhile, tests 13-16 are used to analyze monotonic trends. The variety of tests is essential for increasing confidence in the overall non-stationarity analysis. Significant findings in one or two tests are generally not enough to declare non-stationarity.

For this analysis, the continuous period of water years 1976-2014 for the gage located at Claiborne Lock and Dam and water years 1928-2002 for the gage located near Montgomery, AL were used. All sensitivity parameters were left in their default positions. For both gages, there were no non-stationarities detected, as seen on Figure H.19 and Figure H.20. The Alabama River is a regulated system with multiple run-of-river projects and flood control projects. This may be the reason why non-stationarities were not detected. The monotonic trend test indicates that there are no trends for the entire record (not including historical peaks) for both gages, Figure H.21 and Figure H.22.

USGS water year summaries were checked and do not reveal any information that would indicate gage errors or issue with flow recording. For the gage located near Montgomery, AL, the two extremes recorded prior to the period of record were estimated based on high water marks and an extended rating curve. These two extremes were excluded from the non-stationarity analysis.

Figure H.19: Non-Stationarity Tool result for USGS 2420000 located near Montgomery, Alabama

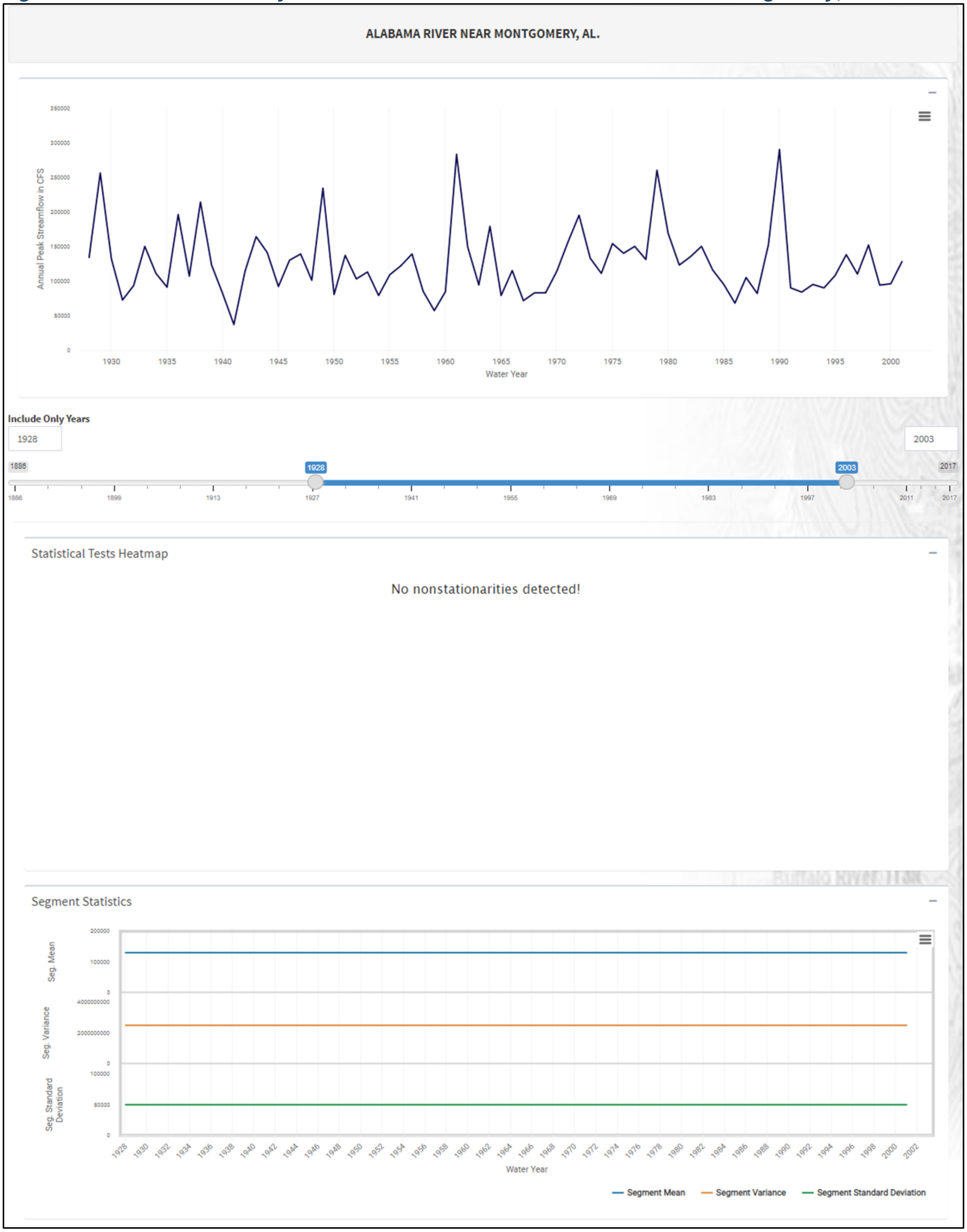


Figure H.20: Non-Stationarity Tool result for USGS 2428400 located at Claiborne Lock and Dam

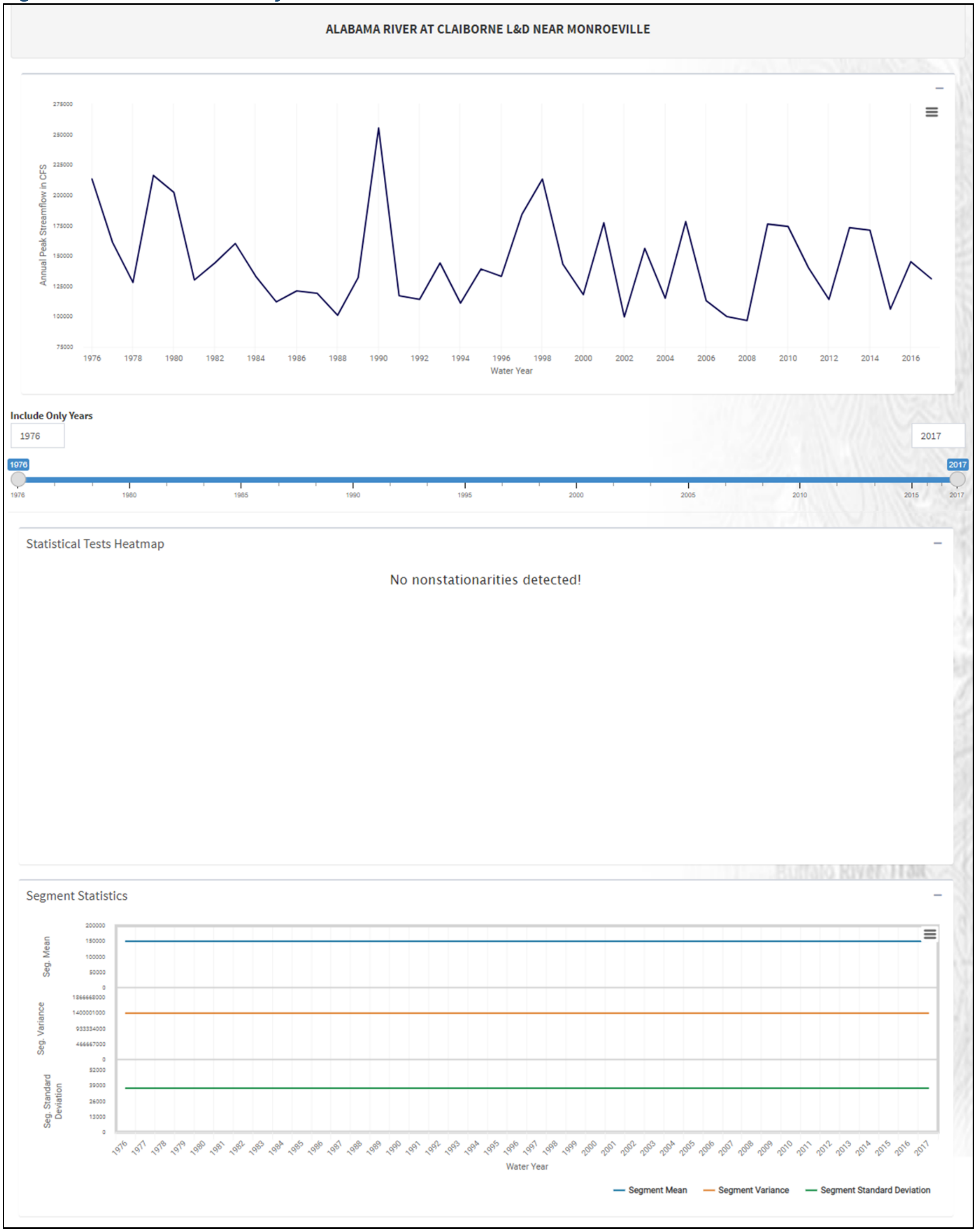
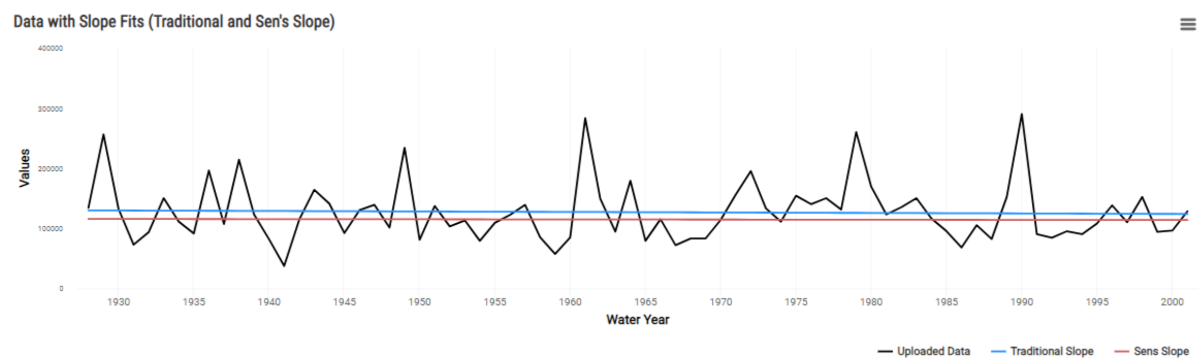


Figure H.21: Monotonic trend analysis for USGS 2420000 located near Montgomery, Alabama

Plot of Maximum Annual Flow/Height with Slope Fits (Traditional and Sen's Slope)

ALABAMA RIVER NEAR MONTGOMERY, AL.



Trend Line Coefficients

Method	Directionality	Slope	Intercept
Traditional Slope	Negative	-82	287159
Sen's Slope	Negative	-26	164872

Trend Hypothesis Test

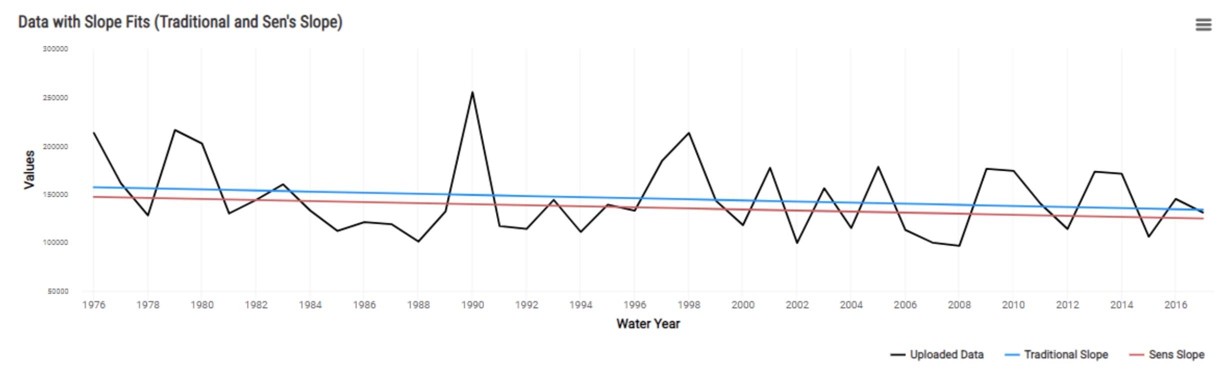
Test	P-Value
t-Test	0.76729
Mann-Kendall	0.88496
Spearman Rank-Order	0.88073

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

Figure H.22: Monotonic trend analysis for USGS 2428400 located at Claiborne Lock and Dam

Plot of Maximum Annual Flow/Height with Slope Fits (Traditional and Sen's Slope)

ALABAMA RIVER AT CLAIBORNE L&D NEAR MONROEVILLE



Trend Line Coefficients

Method	Directionality	Slope	Intercept
Traditional Slope	Negative	-568	1278576
Sen's Slope	Negative	-542	1217438

Trend Hypothesis Test

Test	P-Value
t-Test	0.23675
Mann-Kendall	0.21655
Spearman Rank-Order	0.33438

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

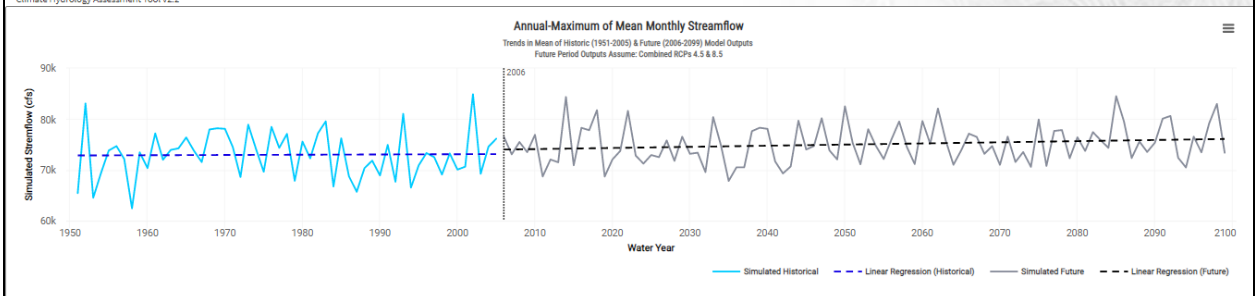
H.2.5. Climate Hydrology Assessment Tool

In addition to the non-stationarity assessment, the USACE Climate Hydrology Assessment Tool (CHAT) version 2.2 was used to assist in the determination of future streamflow conditions. The trends presented in this analysis represent outputs derived from 32 Global Climate Models (GCMs) using different representative concentration pathways (RCPs) of greenhouse gasses that are then translated into a hydrologic response using the United States Bureau of Reclamation (USBR) Variable Infiltration Capacity (VIC) model. The VIC model, forced with GCM meteorological outputs is used to produce a streamflow response for both the hindcast period (1950-2005) and the future period (2006-2099). This dataset is unregulated and does not account for the many flood control structures located on the mainstem rivers within this Hydrologic Unit Code 4 (HUC-4) basin. For this assessment, the middle and lower Alabama Basin were analyzed using a Hydrologic Unit Code 8 (HUC-8) level mean projected annual maximum monthly streamflow.

Figure H.23 shows the CHAT output for HUC 03150203 which includes Millers Ferry Lock and Dam and Figure H.24 shows the CHAT output HUC 03150204 – Lower Alabama which includes Claiborne Lock and Dam. The p-values for the simulated historical data for the HUCs are 0.90239 and 0.89198, respectively. Neither of them are considered statistically significant. The forecast visually indicates a slight upward trend in projected streamflow from years 2000 to 2099 with p-values for the middle and lower Alabama

HUCs at 0.12034 and 0.16389, respectfully. Neither of the future simulated datasets are considered statistically significant.

Figure H.23: CHAT output for HUC 03150203 – Middle Alabama



Trend Lines

Simulated Historical (1951 to 2005)

REGRESSION LINE
Variable Value = 63246 + 4.9251*water year

ADJUSTED R-SQUARED
-0.02

Simulated Future (2006 to 2099)

REGRESSION LINE
Variable Value = 29248 + 22.315*water year

ADJUSTED R-SQUARED
0.02

Statistical Significance Tests (Historical)

Test	p-value
t-Test	0.90239
Mann-Kendall	0.95369
Spearman Rank-Order	0.79332

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

Statistical Signficance Tests (Future)

Test	p-value
t-Test	0.12034
Mann-Kendall	0.10522
Spearman Rank-Order	0.10101

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

Figure H.24: CHAT output for HUC 03150204 – Lower Alabama

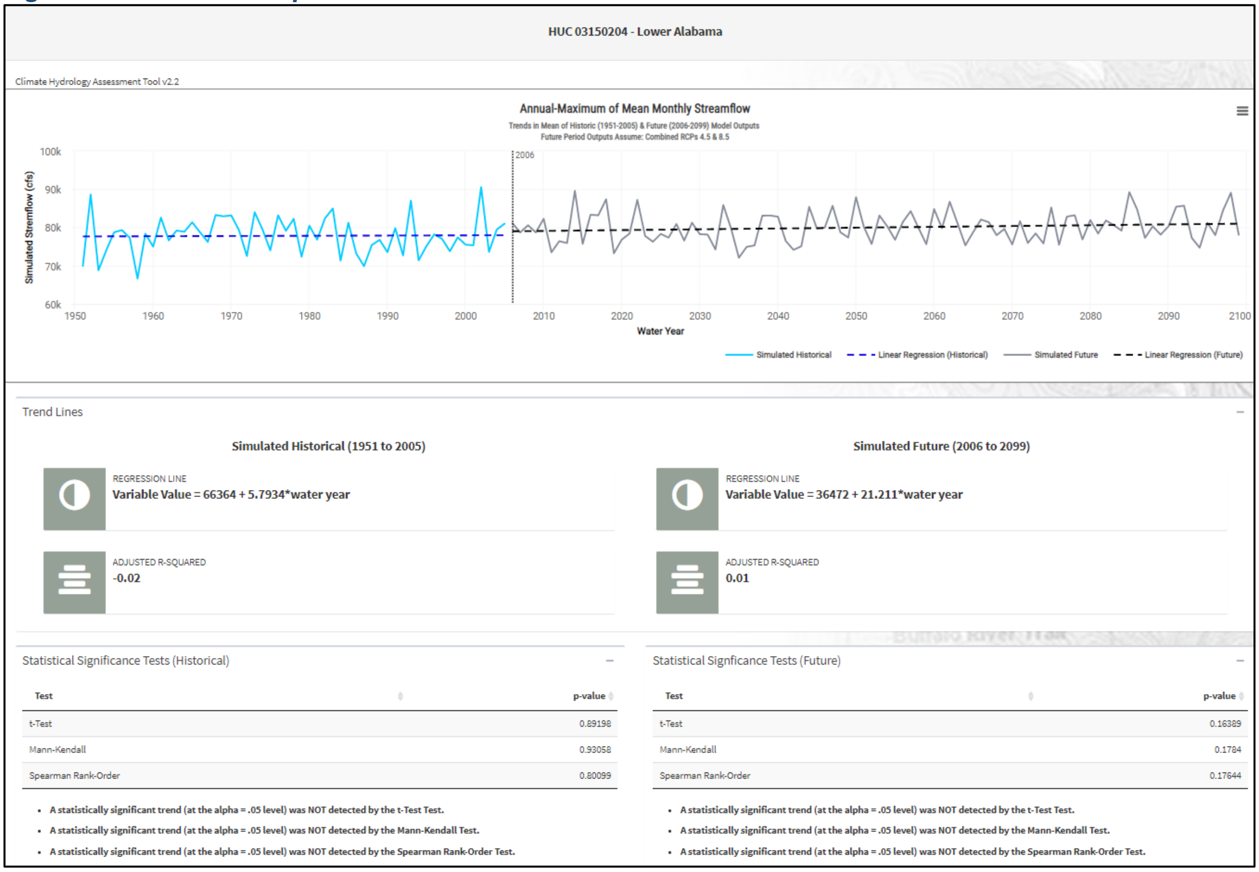


Figure H.25 and Figure H.26 provides the mean value of the 32 projections of future streamflow projections considered through water year 2099 The variability of the spread is fairly consistent for the projected portion of the record: 2000 to 2099. The paragraph below is an excerpt from the CHAT toolbox regarding the streamflow timeseries data.

“The streamflow timeseries displayed in the tool is derived from statistically-downscaled LOCA, CMIP-5 GCM temperature and precipitation outputs. Runoff is generated using an unregulated, distributed precipitation-runoff model and a river network routing model. The river routing model breaks up the rivers within the continental United States (CONUS) into discrete river segments. Runoff displayed in CHAT for a given HUC-8 watershed represents the cumulative runoff which reaches the downstream most routing stream segment associated with the selected HUC-8 watershed. Thus, streamflow displayed is representative of the cumulative flow from all upstream segments, as well as the local runoff contributions to the largest stream segment that transects the selected HUC-8’s downstream boundary.” (USACE, June 2021)

It can be seen on Figure H.25 that there is significant uncertainty in projections of future streamflow. The red shaded area is indicative of the spread in the data produced. It is important to understand that this uncertainty comes from each of the model sources that are used to develop the projected streamflow datasets. GCMs have uncertainty in the bounds of their atmospheric input such as the RCPs. Downscaling the output of these models to a smaller region may not account for some regional effects.

Changes in future conditions that drive the hydrologic model are also a major source of uncertainty. An example of this uncertainty is land use changes, such as increased impervious areas, which can have a major effect on peak streamflow. There are many different land use projections for this region from many sources. Other uncertainties such as changes in temperature extremes and the seasonality of the extreme precipitation can also have a significant effect on the rainfall/runoff transformation. For these reasons, this quantitative analysis should be used with caution, with an understanding that this data should only be considered within the large uncertainly bounds of the analysis

Figure H.25: Projected hydrology for the Alabama River HUC 03150203 – Middle Alabama

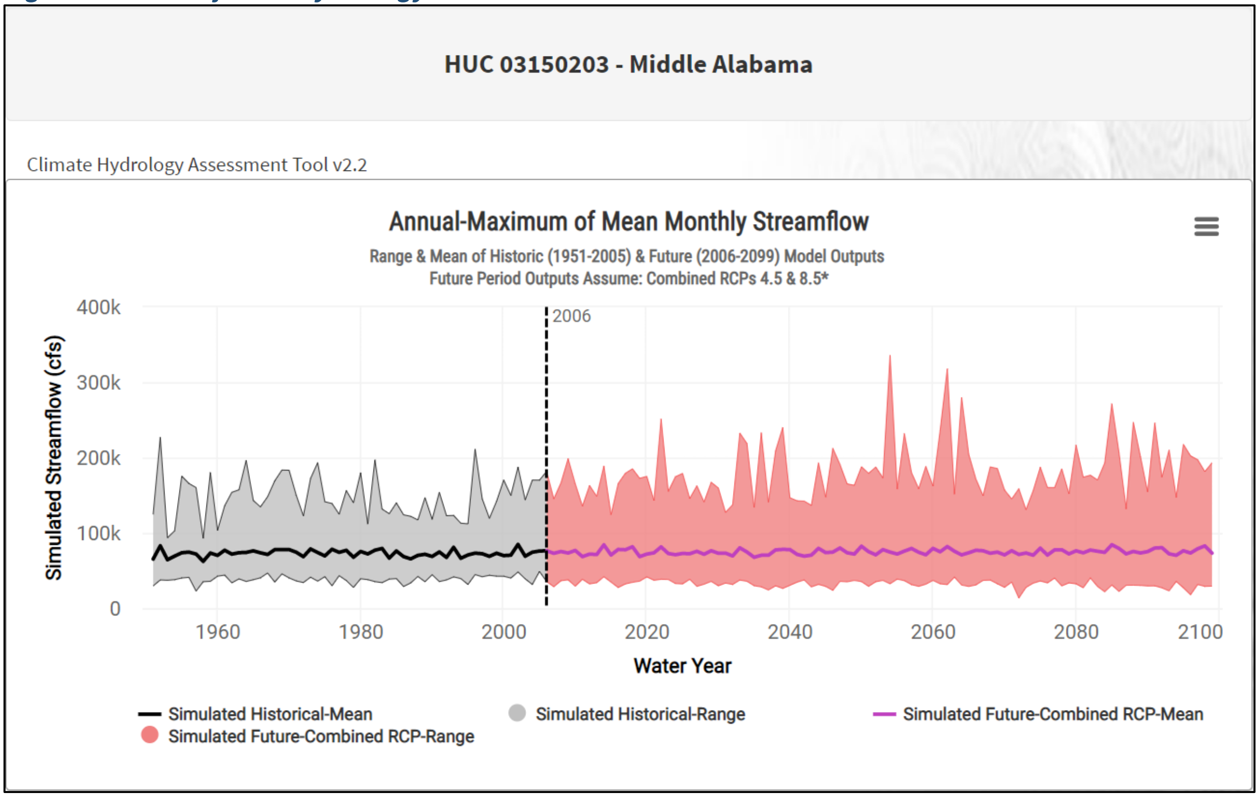
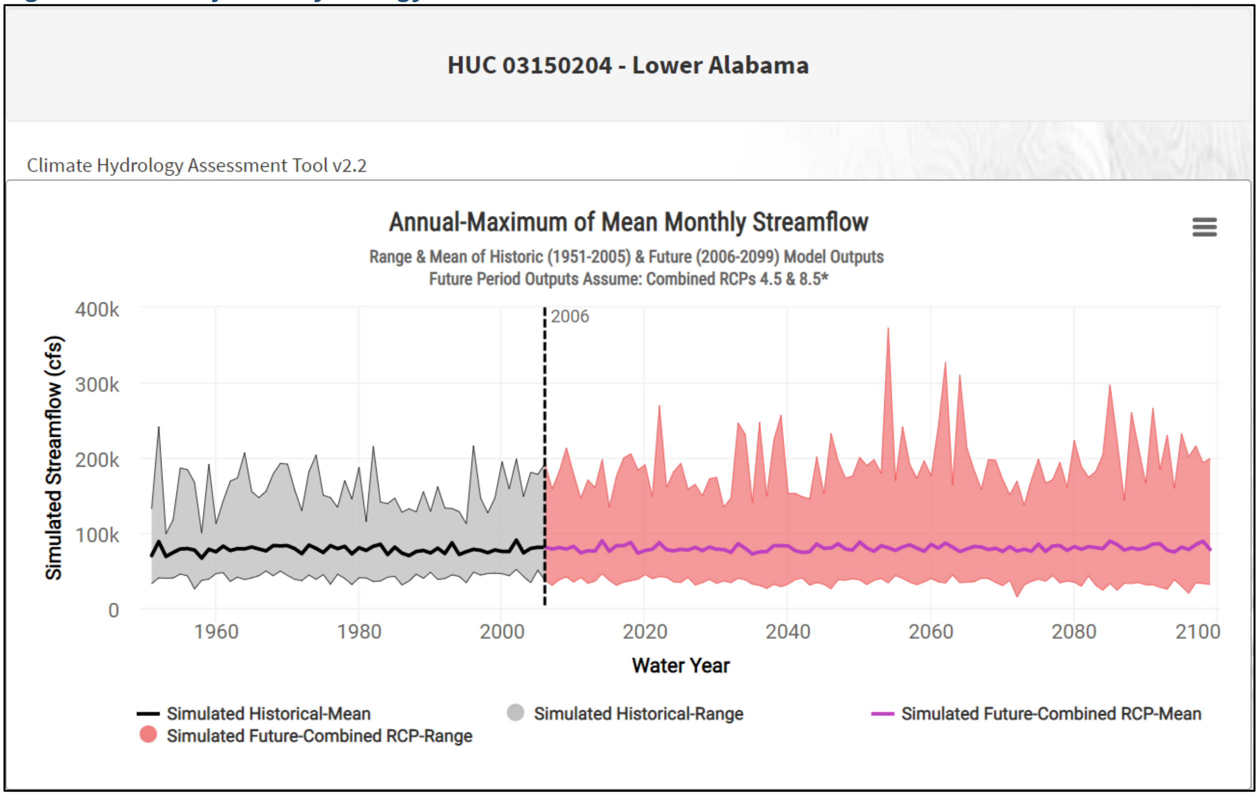


Figure H.26: Projected hydrology for the Alabama River HUC 03150204 – Lower Alabama



H.2.6. Vulnerability Assessment

To understand potential climate change effects and to increase resilience/decrease vulnerability of flood risk management alternatives to climate change, the relative vulnerability of the basin to such factors was analyzed. In accordance with ECB 2018-14, the USACE Watershed Climate Vulnerability Assessment tool was used to identify vulnerabilities to climate change on a HUC-4 watershed scale relative to other HUC-4 basins across the nation. As this study is an assessment of flood risk management alternatives, vulnerability with respect to the Flood Risk Reduction business line is presented in this analysis.

To address vulnerabilities due to climate change, the Vulnerability Assessment tool utilizes two 30-year epochs centered on 2050 (2035-2064) and 2085 (2070-2099) as well as a base epoch. These epochs, while arbitrary, line up well with other national climate change assessments. For each epoch, the tool utilizes the results of 100 combinations of Global Circulation/Climate Models (GCM) run using different Representative Concentration Pathways of greenhouse gas emission to produce 100 traces per epoch for a given watershed. The results of the GCMs are translated into flow and are then sorted by cumulative runoff projections. Traces of the highest 50% of cumulative runoff are categorized as wet and traces with the lowest 50% of cumulative runoff are categorized as dry. This provides two scenarios (wet and dry) for each of the two epochs, excluding the base epoch. Consideration of both wet and dry scenarios reveals some of the uncertainties associated with the results produced using the climate-changed hydrology and meteorology used as inputs to the vulnerability tool.

The tool uses specific indicators of vulnerability relative to the business line being considered. There is a total of 27 indicators in the tool, 9 of which are used to derive the vulnerability score in the Alabama HUC 4 with respect to the Ecosystem Restoration business line. Table H.3 lists the indicators and their descriptions.

Table H.3: Indicator Variables used to derive the Ecosystem Restoration Vulnerability score for the Alabama Basin as determined by the Vulnerability Assessment tool

Indicator Short Name	Indicator Full Name	Description
156_SEDIMENT	Change in sediment load due to change in future precipitation	The ratio of the change in the sediment load in the future to the present load.
221C_MONTHLY_COV	Monthly CV of runoff (cumulative)	Measure of short-term variability in the region’s hydrology: 75th percentile of annual ratios of the standard deviation of monthly runoff to the mean of monthly runoff. Includes upstream freshwater inputs (cumulative).
277_RUNOFF_PRECIP	% change in runoff divided by % change in precipitation	Median of: deviation of runoff from monthly mean times average monthly runoff divided by deviation of precipitation from monthly mean times average monthly precipitation.
297_MACROINVERTEBRATE	Macroinvertebrate index of biotic condition	The sum (ranging from 0-100) of scores for six metrics that characterize macroinvertebrate assemblages: taxonomic richness, taxonomic composition, taxonomic diversity, feeding groups, habits, pollution tolerance.

Indicator Short Name	Indicator Full Name	Description
568L_FLOOD_MAGNIFICATION	Flood magnification factor (local)	Change in flood runoff: Ratio of indicator 571L (monthly runoff exceeded 10% of the time, excluding upstream freshwater inputs) to 571L in base period.
568C_FLOOD_MAGNIFICATION	Flood magnification factor (cumulative)	Change in flood runoff: ratio of indicator 571C (monthly runoff exceeded 1-% of the time, including upstream freshwater inputs) to 571C in base period.
65L_MEAN_ANNUAL_RUNOFF	Mean annual runoff (local)	Mean runoff: average annual runoff, excluding upstream freshwater inputs (local).
700C_LOW_FLOW_REDUCTION	Low flow reduction factor (cumulative)	Change in low runoff: ratio of indicator 570C (monthly runoff exceeded 90% of the time, including upstream freshwater inputs) to 570C in base period.
8_AT_RISK_FRESHWATER_PLANT	% of freshwater plant communities at risk	Percentage of wetland and riparian plant communities that are at risk of extinction, based on remaining number and condition, remaining acreage, threat severity, etc.

Figure H.27 and Figure H.28 shows a comparison of WOWA scores for the ecosystem restoration business line for HUC-4 watersheds nationally, and for the South Atlantic Division only, for the wet and dry scenarios as well as the 2050 and 2085 epochs, respectively. This shows that the WOWA score for the Alabama HUC-4 Basin (highlighted in yellow) is not relatively vulnerable to climate change impacts for the ecosystem restoration business line. Within the wet subset of traces for the South Atlantic Division, there is only one HUC04 watersheds for the 2050 epoch. For the dry subset of traces, there are only two HUC04 watersheds that are considered relatively vulnerable to climate change for the Ecosystem Restoration business line. This further reinforces that the Alabama basin is does not have significant vulnerabilities to the Ecosystem Restoration business line with respect to other watersheds in the United States, or the region.

Figure H.27: Comparison of national vulnerability scores for CONUS HUC-4s

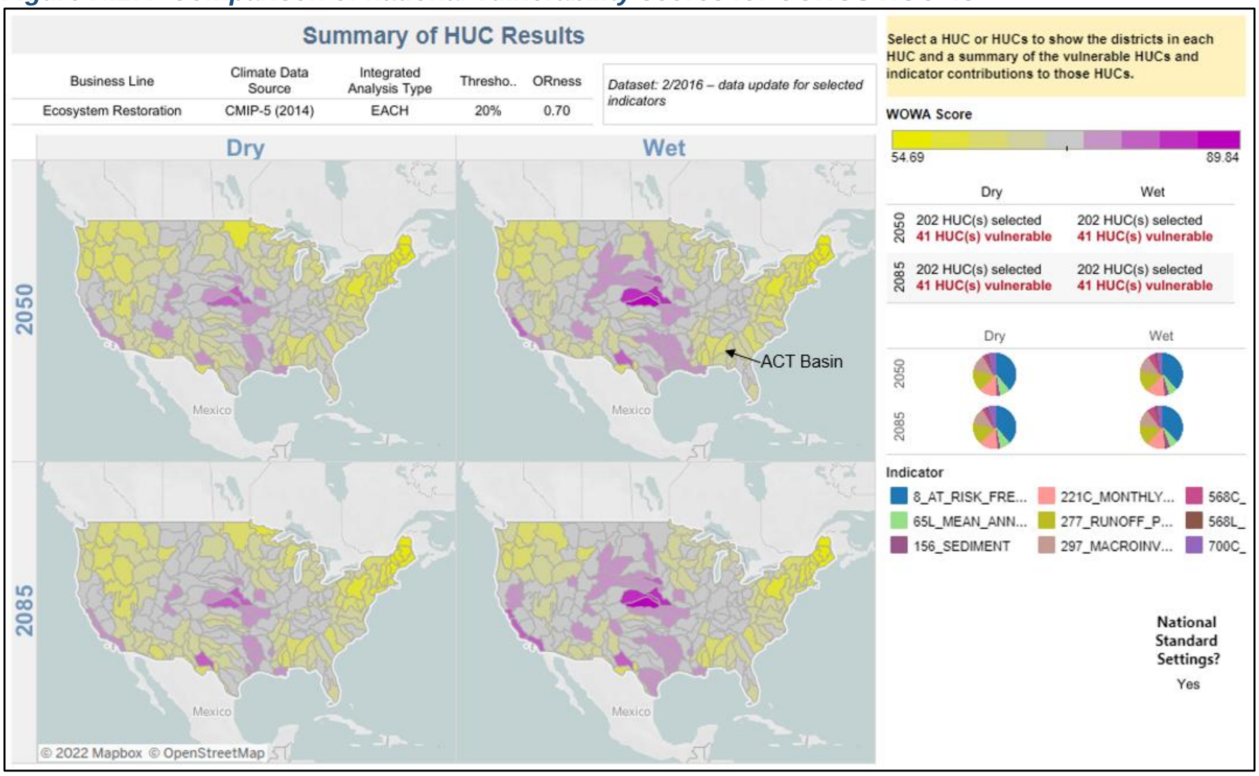
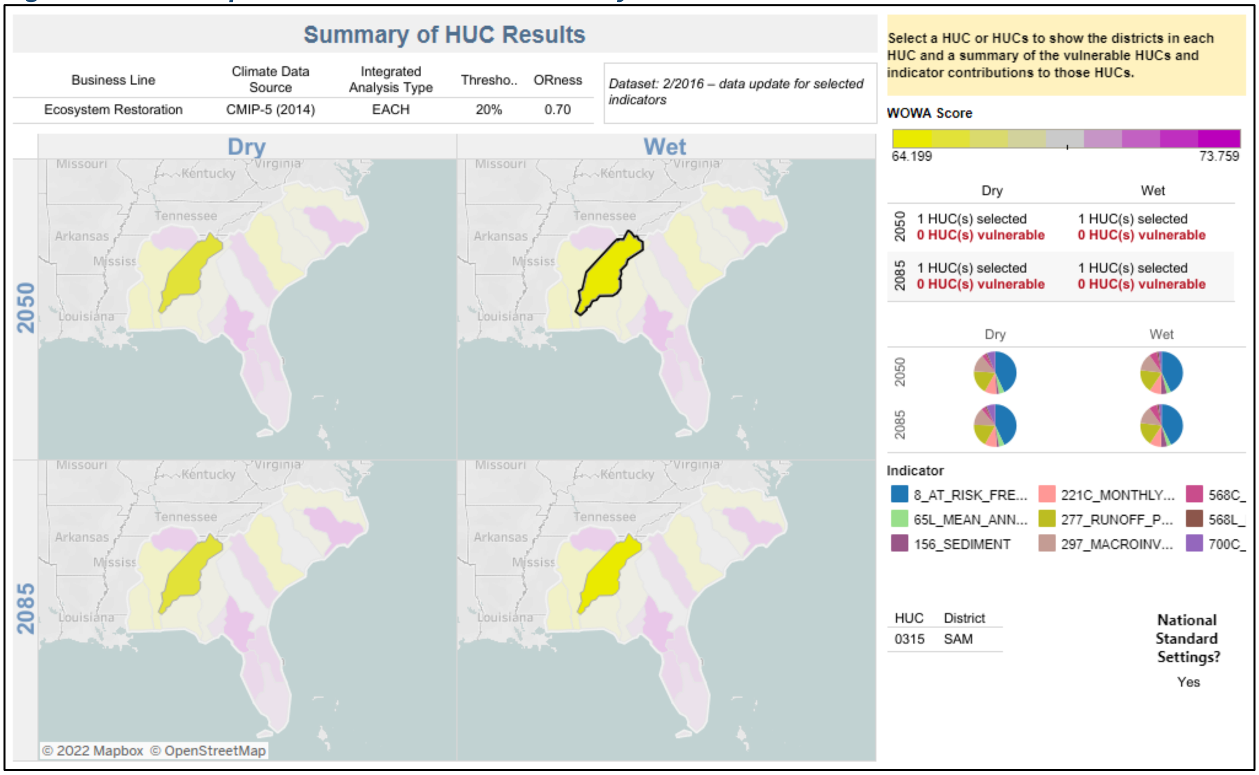


Figure H.28: Comparison of national vulnerability scores for South Atlantic Division HUC-4s



It is important to note that the vulnerability assessment only indicates vulnerability relative to the rest of the nation. It does not state that the basin itself is invulnerable to impacts of climate change on the Ecosystem Restoration business line. The assessment only concludes that it is not in the top 20% of vulnerable basins based on WOWA scores. There are locally significant impacts relative to climate change driven by many different factors. Therefore, it is beneficial to understand the composition of the relevant HUC 04's (Alabama Basin) vulnerability score, in terms of how much each flood risk reduction indicator variable contributes to the vulnerability score for each subset of traces, and for both epochs of time. Figure H.29 and Figure H.30 below shows the dominant indicator at risk freshwater plants is the prevailing indicator variable driving the ecosystem restoration vulnerability score, followed by precipitation runoff for both the dry and wet scenarios. This aligns with the literature review that indicates the potential for more frequent and more severe storms in the southeast.

Figure H.29: Dominate indicators for the Ecosystem Restoration Business Line for the Dry Scenario

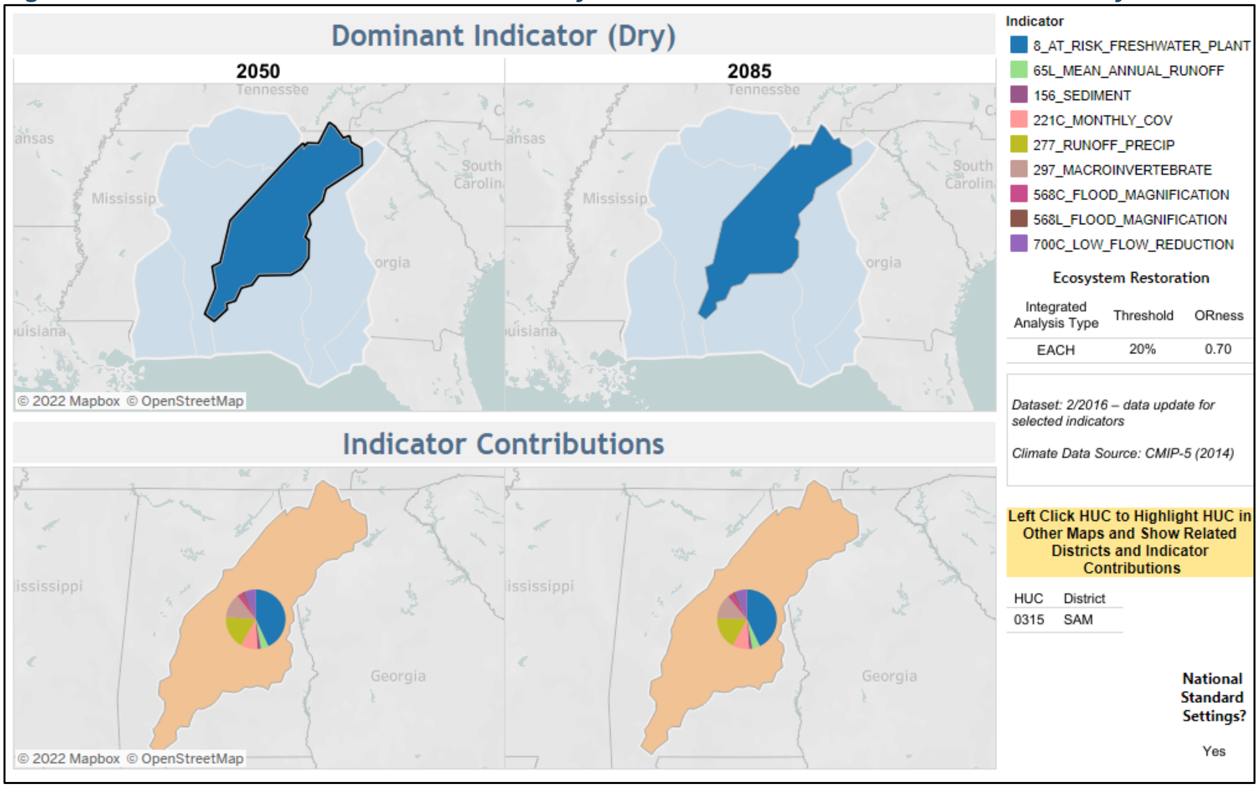
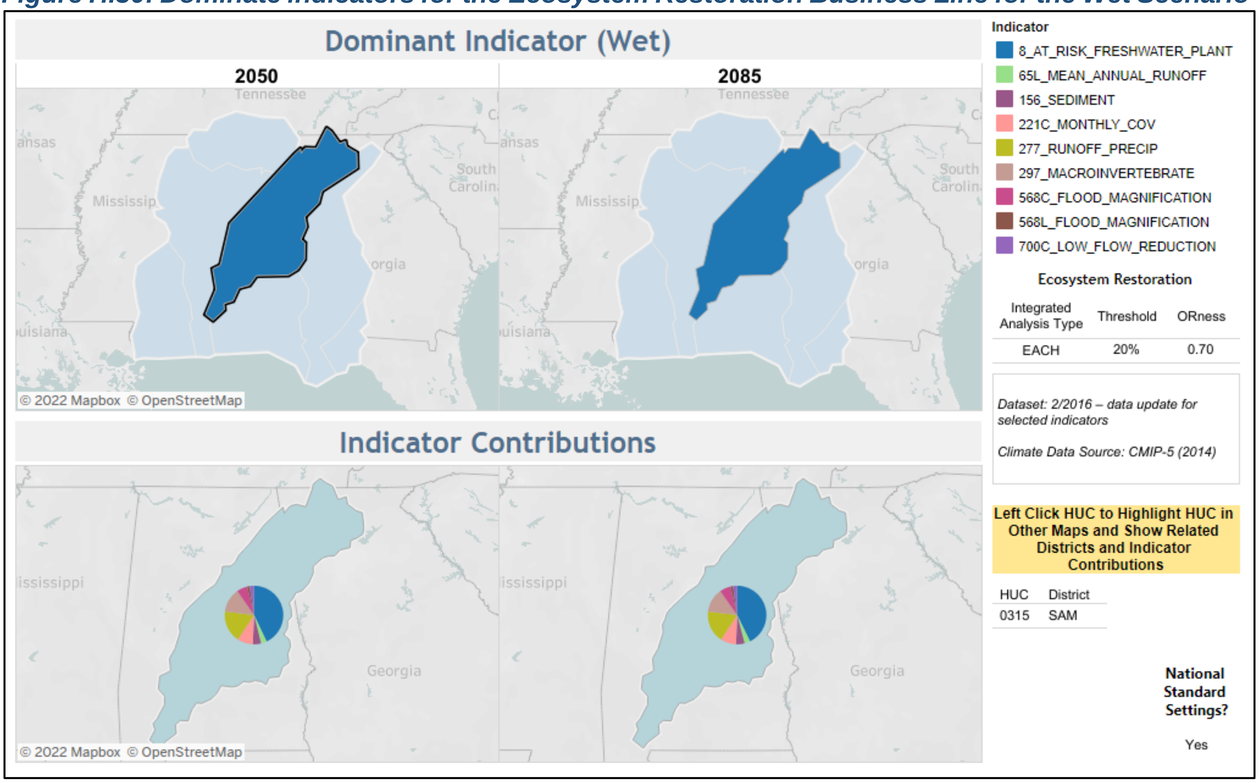


Figure H.30: Dominate indicators for the Ecosystem Restoration Business Line for the Wet Scenario



H.2.7. Climate Change and Impacts on Recommended Plan

The Recommended Plan for this study includes bypass channels around both Millers Ferry Lock and Dam and Claiborne Lock and Dam. Table H.4 shows the risk assessment results for each measure in the recommended plan.

Table H.4: Risk assessment results of each measure in the Recommended Plan

Feature or Measure	Trigger	Hazard	Harm	Qualitative Likelihood
Bypass Channels at Millers Ferry Lock and Dam	Increase in frequency and magnitude of extreme storms	Peak elevations during floods could increase	Damage to bypass channel structure	Highly Unlikely
Bypass Channel at Claiborne Lock and Dam	Increase in frequency and magnitude of extreme storms	Peak elevations during floods could increase	Damage to bypass channel structure	Highly Unlikely

An increase in the magnitude of extreme storms could cause the peak elevations of floods to increase for the same frequency storm. This hazard however is very unlikely to lead to damage of the bypass structure, or any negative effect. The bypass channel at Millers Ferry Lock and Dam will have a gated structure that will be closed during large flood events. The bypass channel at Claiborne Lock and Dam will be overtopped/out of banks frequently, therefore further analysis will be completed to ensure negatives effects will not impact the recommended plan.

H.2.8. Conclusions

Based on the literature review of relevant climate data, there is some consensus that there will be mild increases in the severity and frequency of storms in the region. However, there is no consensus on future changes in hydrology. Observed data from gages near the study area show temperatures have been gradually rising since the 1970s, after a cooling period in the middle part of the century. From the data it is difficult to come to a definitive conclusion on whether temperature is increasing, or if this is a reoccurring pattern. Annual precipitation seems to be variable for the region. From the annual precipitation data there may be more extremes occurring in recent years, such as extreme low annual precipitation values. However, the overall trends appear to be constant or increasing slightly. There is some consensus on peak streamflow for the region decreasing since the middle of the century, however, the literature lacks a clear consensus. For the Alabama Basin, decreasing streamflow could be related to the increase in flood control projects within the region since the late 1940s.

The sea level change analysis completed for Claiborne Lock and Dam did not indicate any anticipated impacts to the project area based on a simplified analysis using tide gages located at Mobile, AL. There are also no anticipated impacts for Millers Ferry Lock and Dam since the project is located in the pool of Claiborne Lock and Dam.

The non-stationarity assessment on the Alabama River Basin was performed using two gages (USGS 02420000 Alabama River near Montgomery, AL and USGS 02428400 Alabama River at Claiborne Lock and Dam). Neither gage displayed non-stationarities, nor were monotonic trends detected.

The USACE CHAT tool indicates that there are no statistically significant trends in the two streamflow datasets analyzed for HUC 03150203 and HUC 03150204. Furthermore, the HUC-4 analysis on streamflow on the Alabama basin only shows an increasing trend in projected streamflow based on GCM model output translated into a hydrologic response. These analyses provide some indication that there will be significant increases in peak annual streamflow in the future as a result of climate change. However, the projections seem to oppose the trend in observed flow. Caution should be used in making any definitive statements on potential future hydrology as there is substantial uncertainty in both the climate and hydrologic models that drive these analyses. The vulnerability assessment helps to further reinforce a lack of evidence in increasing flood risk. Findings of the vulnerability assessment show that the Alabama HUC-4 basin is not considered vulnerable to the ecosystem restoration business line as a result of climate change, with respect to other HUC-4s in the nation.

H.3. Hydrologic and Hydraulic Modeling

Hydrologic analysis and hydraulic modeling were performed on the Alabama River at Millers Ferry Lock and Dam and Claiborne Lock and Dam to support the intermediate evaluation of the focused array of alternatives. The goal of modeling the existing conditions of the study area was to establish a baseline for developing future without project conditions and provide outputs (velocity and hydrographs) for each alternative to the habitat model and ResSim model.

H.3.1. Terrain and Geometric Data

H.3.1.1. Digital Terrain Development

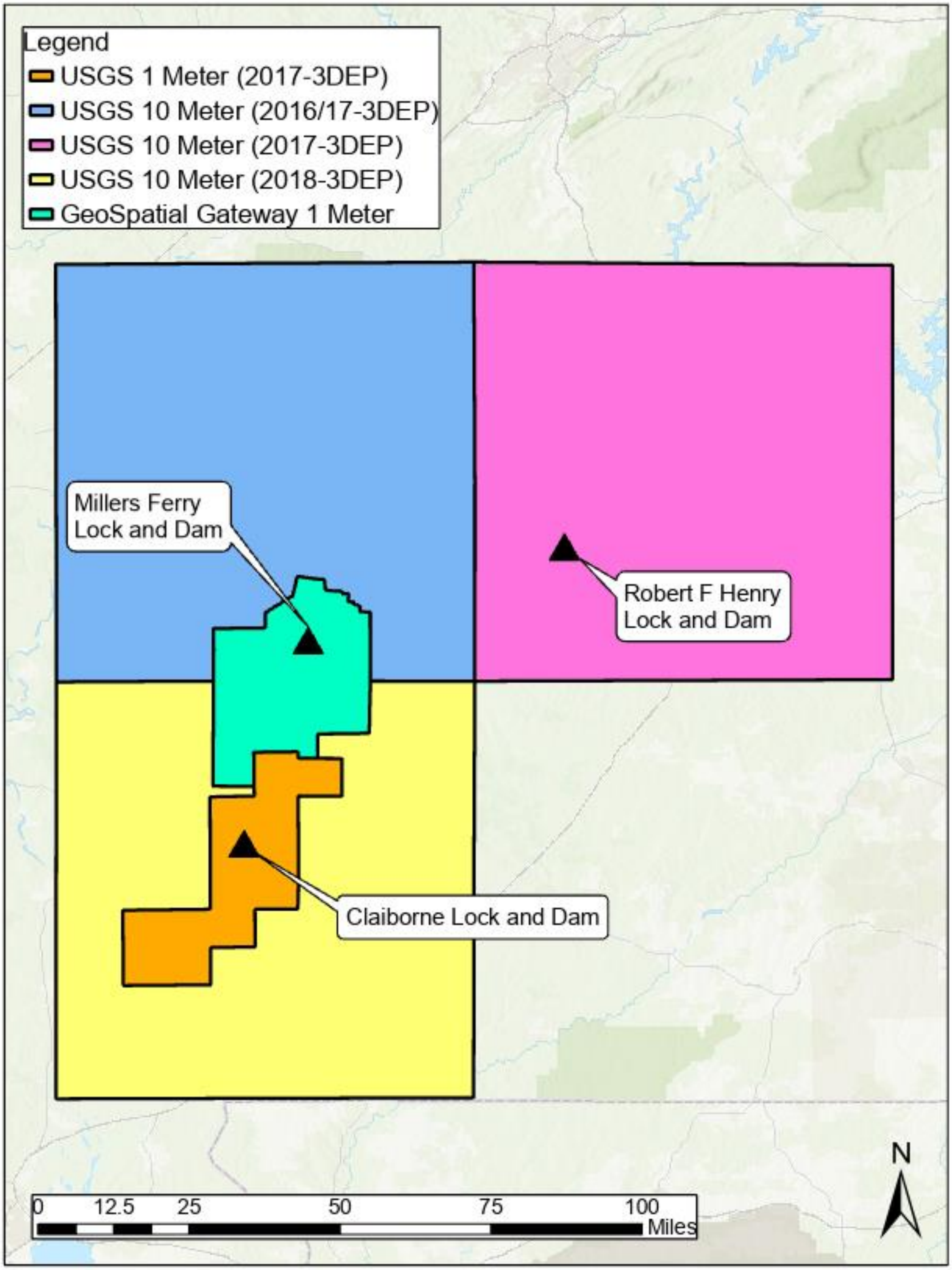
The terrain used in the HEC-RAS model was developed using the USGS National Elevation Dataset (NED), USDA-NRCS Geospatial Data Gateway, and USACE Bathymetry Data (Figure H.31). The survey start and end dates are shown for each data source below in Table H.5. The quality information for these datasets were not listed within the metadata obtained.

The horizontal projection for the combined terrain file was NAD 1983 State Plane Alabama West FIPS 0102 (US Feet). RAS Mapper within the HEC-RAS software was utilized to merge the datasets together and hydraulicly correct areas within the 2D mesh of the model. RAS Mapper was also used to smooth where the bathymetry and terrain met. Figure H.31 shows the various data sources and their extents in the study area.

Table H.5: Digital Elevation Model Survey Dates

Name	Start	End
USGS 1-Meter (2017)	1/2/2018	1/19/2018
USGS 10-Meter (2016/2017)	12/2/2016	3/15/2017
USGS 10-Meter (2017)	2/25/2017	4/2/2017
USGS 10-Meter (2018)	1/14/2018	12/17/2018
Geospatial Gateway 1-Meter	2010	Present

Figure H.31: Data source locations and corresponding extents utilized for the Alabama River Ecosystem Restoration project



H.3.1.2. Bathymetric Data

The bathymetry used in the model was obtained from the USACE Mobile District Operations Division site office in Tuscaloosa, Alabama. The bathymetry stretches from 30 miles downstream of Claiborne Lock and Dam to the downstream side of Robert F. Henry Lock and Dam. Table H.6 shows the survey dates/year is listed for each river or bar section.

Table H.6: Bathymetry Survey Miles and Dates

Bar or River Section Name	Start River Mile	Survey Date/Year
CALIFORNIA BAR	42.9	11/10/2016
MILE 45 BAR	44.3	11/7/2016
LOMBARD LANDING BAR	45.4	10/20/2016
SHACKLEFORD BAR	46.0	12/5/2016
BILLS BAR	47.7	9/24/2015
BAILEY CREEK	49.2	11/14/2016
LOVETTS CREEK	51.0	10/26/2016
HOWARD LANDING	52.1	11/3/2016
MILE 53 BAR	52.6	9/12/2014
DALES FERRY BAR	53.7	9/11/2014
MARSHALL GIN LANDING	55.3	9/30/2015
MILE 57 BAR	56.6	9/11/2014
PIGEON CREEK BAR	58.9	10/26/2016
NANCY HALL LANDING	60.1	9/11/2014
MILE 61 BAR	60.7	9/11/2014
MILE 62 BAR	61.2	12/8/2016
GOSPORT LANDING	62.5	12/8/2016
MILE 64 BAR	63.5	9/10/2014
GALLIARD CREEK BAR	64.7	9/10/2014
STATE DOCKS BAR	65.2	9/10/2014
CLAIBORNE BRIDGE BAR	66.4	12/8/2016
LIMESTONE CREEK BAR	67.6	9/10/2014
BIG FLAT CREEK	68.6	10/20/2015
MILE 70_5 BAR	69.9	10/21/2015
CLAIBORNE LOCK LA	71.3	8/12/2019
CLAIBORNE LOCK UA	72.5	10/16/2007
CLAIBORNE LOCK UA	72.5	2/16/2016
Claiborne Pool	72.6	2007
Millers Ferry Pool	133.0	2007

H.3.2. Hydrologic Model

The hydrology of the Alabama River and upstream drainage area is extremely complex. The drainage area consists of over 21,400 square miles above Claiborne Lock and Dam, 5 flood risk management projects, and several other navigation dams upstream.

The development of synthetic or balanced hydrographs was determined to be the best approach for hydrology within the timeframe and level of detail needed. This consists of scaling an observed flow hydrograph at Millers Ferry Lock and Dam to match peak flow and volume of frequency events determined by a flow-frequency and volume-frequency analysis.

Regression Equations were not utilized for this study due to the size of the basin. The drainage area above Selma, AL is approximately 17,000 sq. miles for the Alabama River. The regression equations have a limitation of the drainage area between 0.44 to 1,344 sq. miles. Depending on which region the area is in, these values vary in-between this range. (USGS, 2003)

H.3.2.1. Flow Frequency Analysis

The Alabama River Basin has several gages throughout, however, only two flow gages were utilized for the flow frequency analysis. Unimpaired flows for Millers Ferry and Claiborne Lock and Dams were obtained from Mobile District Water Management. These datasets were created by removing all storage in the ACT basin in order to make one homogeneous dataset without regulation. The flow frequency at Claiborne Lock and Dam was used for comparison purposes. However, the flow frequency at Millers Ferry Lock and Dam was used to develop balanced hydrographs.

The U.S. Army Corps of Engineers (USACE) Statistical Software Package (HEC-SSP) was used to calculate the frequency flows at these gages. Table H.7 shows the 100-year peak discharges derived from a Bulletin 17C (see England et al., 2017) flow frequency analysis in HEC-SSP. Regional Skew was not available for the study area, therefore the station skew was utilized in the analysis.

Table H.8 shows a full range of frequency flows calculated for the gages. Figure H.32 and Figure H.33 shows the graphical plots for Millers Ferry and Claiborne Lock and Dams.

Using the Stats LPIII Version 2 Expected Probability spreadsheet, the expected probability for the flow frequency analysis was calculated and carried forward in the balanced hydrograph analysis.

Table H.7: 100-Year Frequency Flows using Bulletin 17C

Location	Program	Skew	MSE Error	Period	Historic Period	# of Events	Historical Events	1% Flows (cfs)
Millers Ferry Lock and Dam	HEC-SSP Bulletin 17C	-0.799	0.124	1939 - 2013	75	75	0	253,000
Claiborne Lock and Dam	HEC-SSP Bulletin 17C	-0.857	0.129	1939 - 2013	75	75	0	247,000

Table H.8: Frequency Flow Analysis for Unimpaired Dataset at Millers Ferry Lock and Dam

Site Location	0.5 AEP	0.2 AEP	0.1 AEP	0.04 AEP	0.02 AEP	0.01 AEP	0.005 AEP	0.002 AEP
Millers Ferry Lock and Dam	151,000	190,000	210,000	231,000	243,000	253,000	261,000	271,000
Claiborne Lock and Dam	145,000	186,000	206,000	226,000	237,000	247,000	255,000	264,000

Figure H.32: Millers Ferry Flow Frequency Analysis

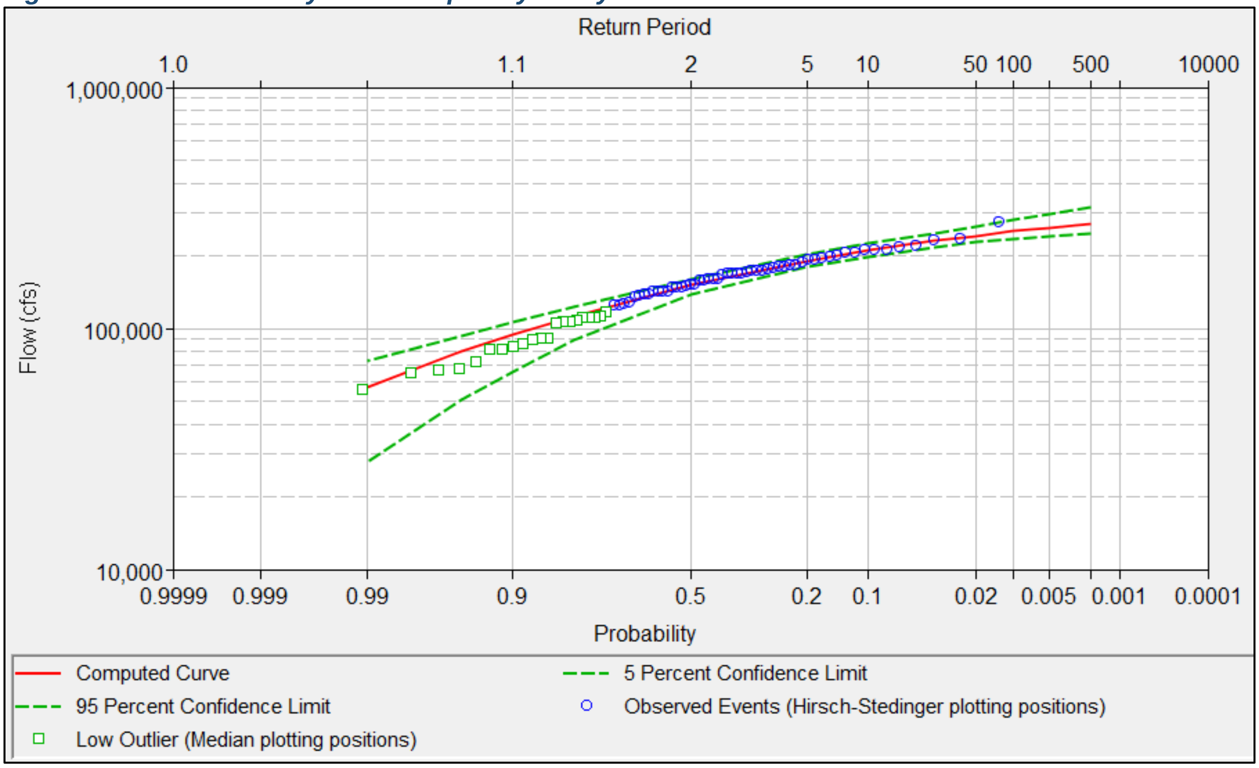
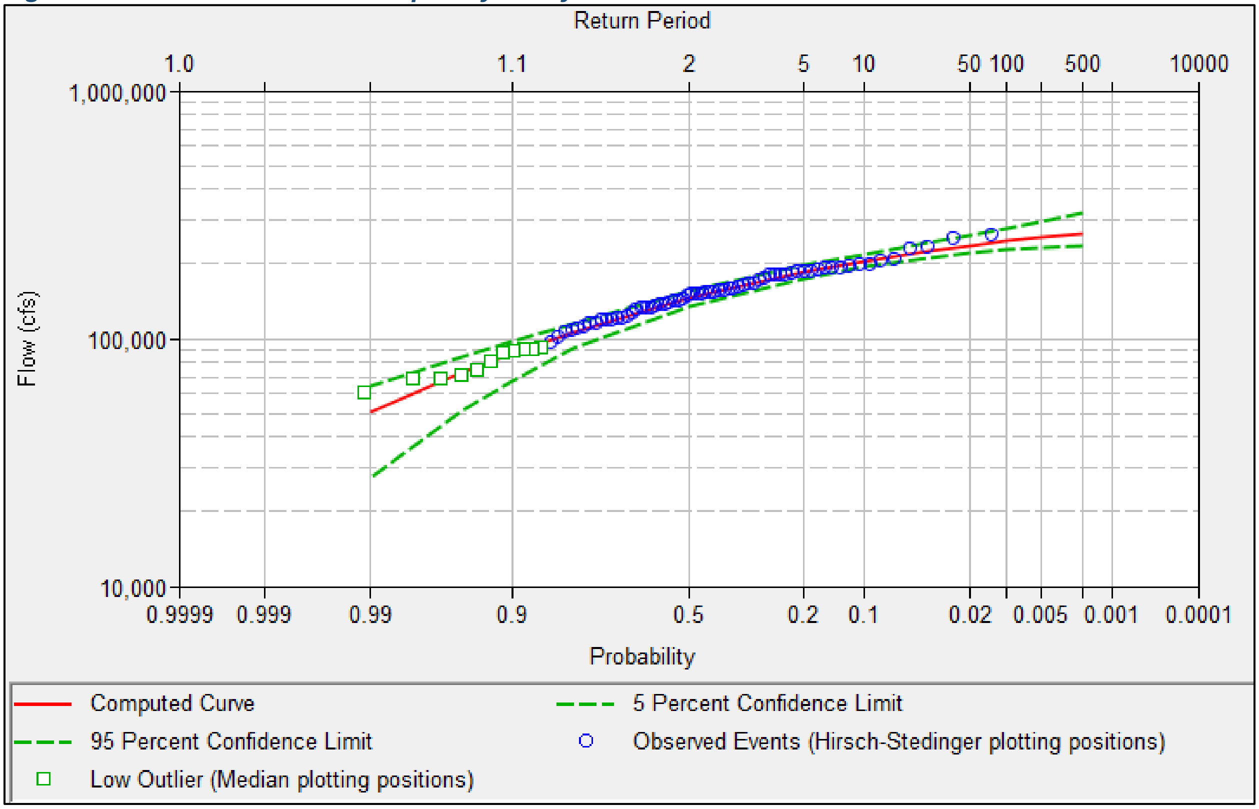


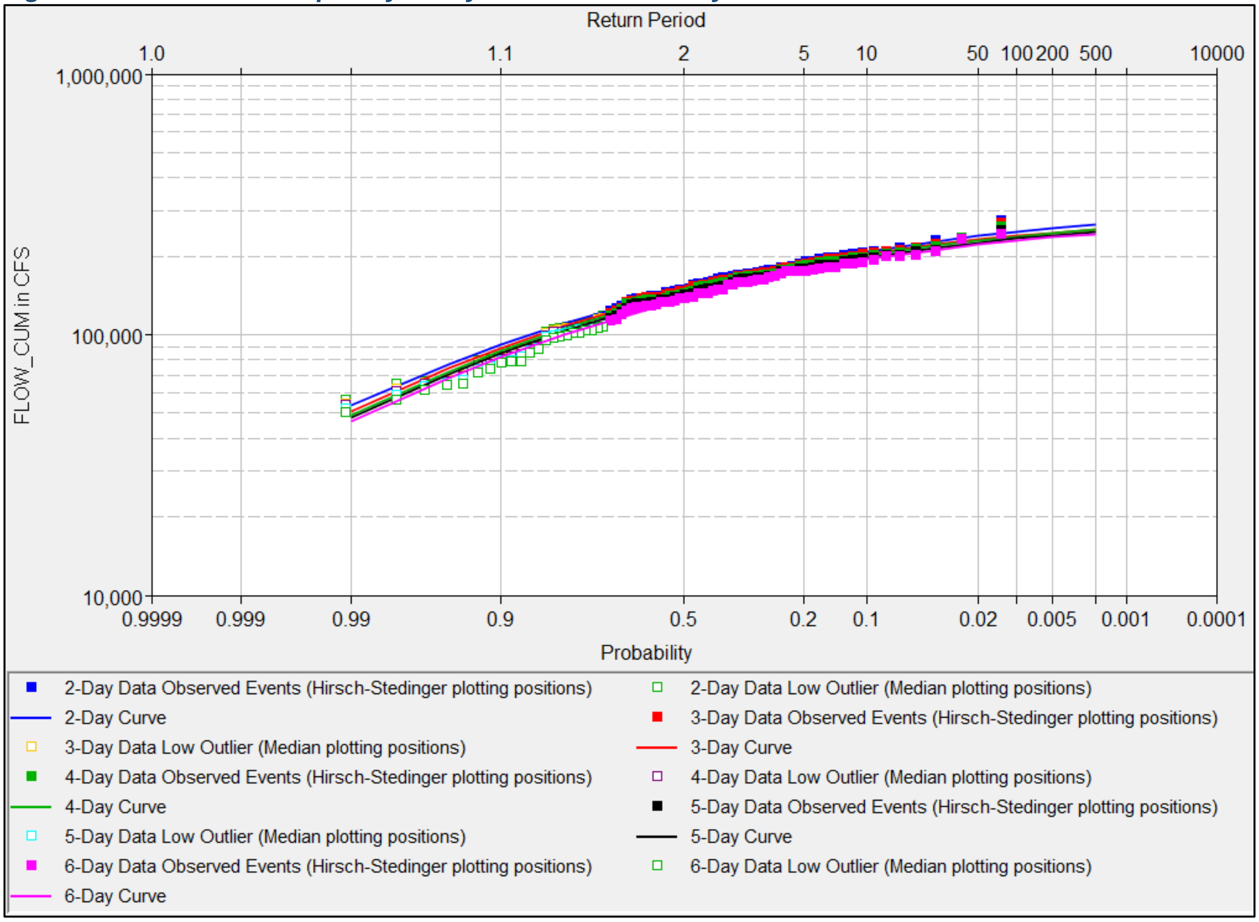
Figure H.33: Claiborne Flow Frequency Analysis



H.3.2.2. Volume Frequency

A volume frequency analysis was performed on the unimpaired dataset for Millers Ferry Lock and Dam using HEC-SSP. After looking at the observed headwater and flow at Millers Ferry Lock and Dam, critical durations of 2 to 6 days were used in this analysis. For this project, the pool elevation does not have a wide fluctuation even with flood wave passing. The highest pool elevation recorded at the project site was 83.4 ft-NAVD88. Figure H.34 shows the volume frequency analysis plot for Millers Ferry Lock and Dam.

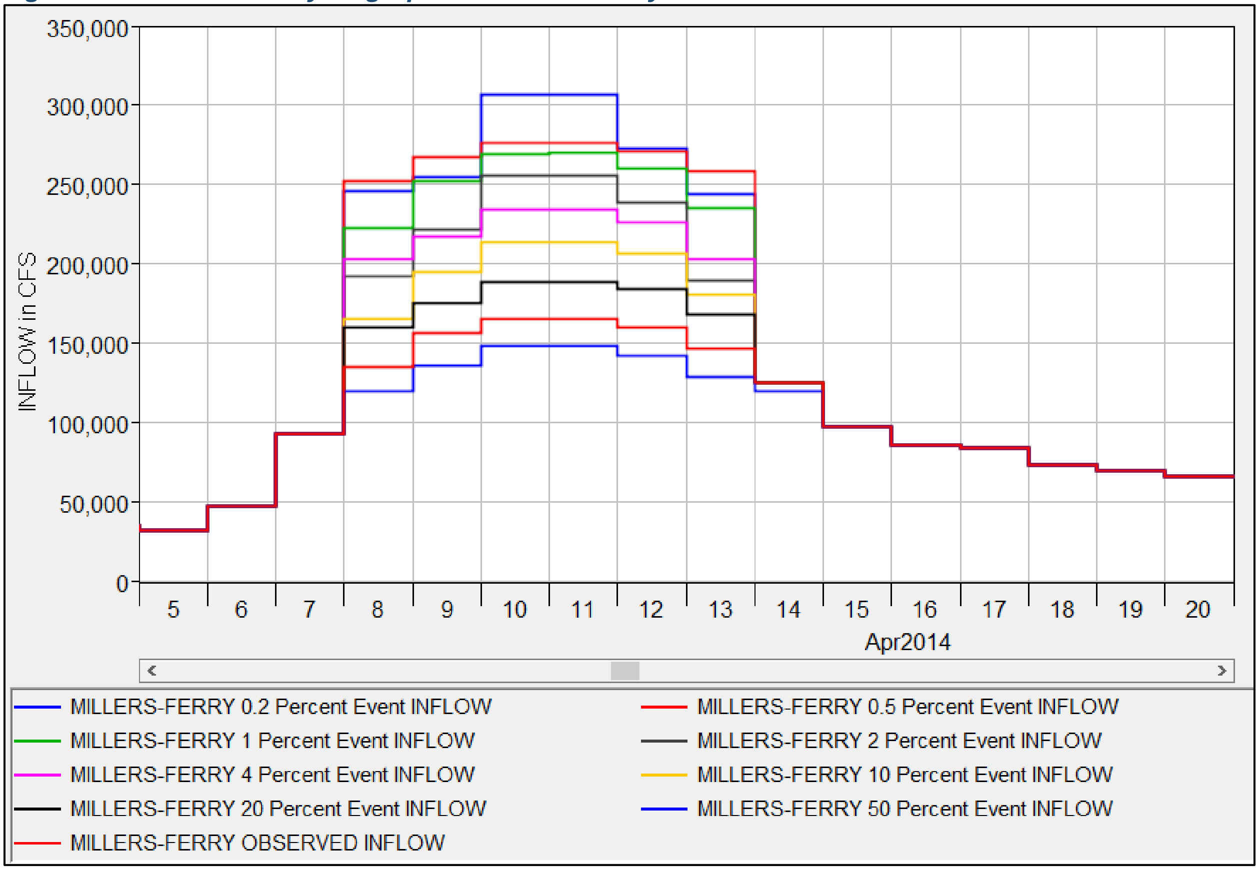
Figure H.34: Volume Frequency Analysis for Millers Ferry Lock and Dam



H.3.2.3. Balanced Hydrographs

Balanced hydrographs were created using the expected probability flow frequency and volume frequency analysis discussed in the previous sections. Within the observed inflow data at Millers Ferry Lock and Dam from 2008 to 2020, the April 2014 event was chosen to scale due to the base flow being more representative of the project area. Figure H.35 shows the balanced hydrographs for Millers Ferry Lock and Dam.

Figure H.35: Balanced Hydrographs for Millers Ferry Lock and Dam



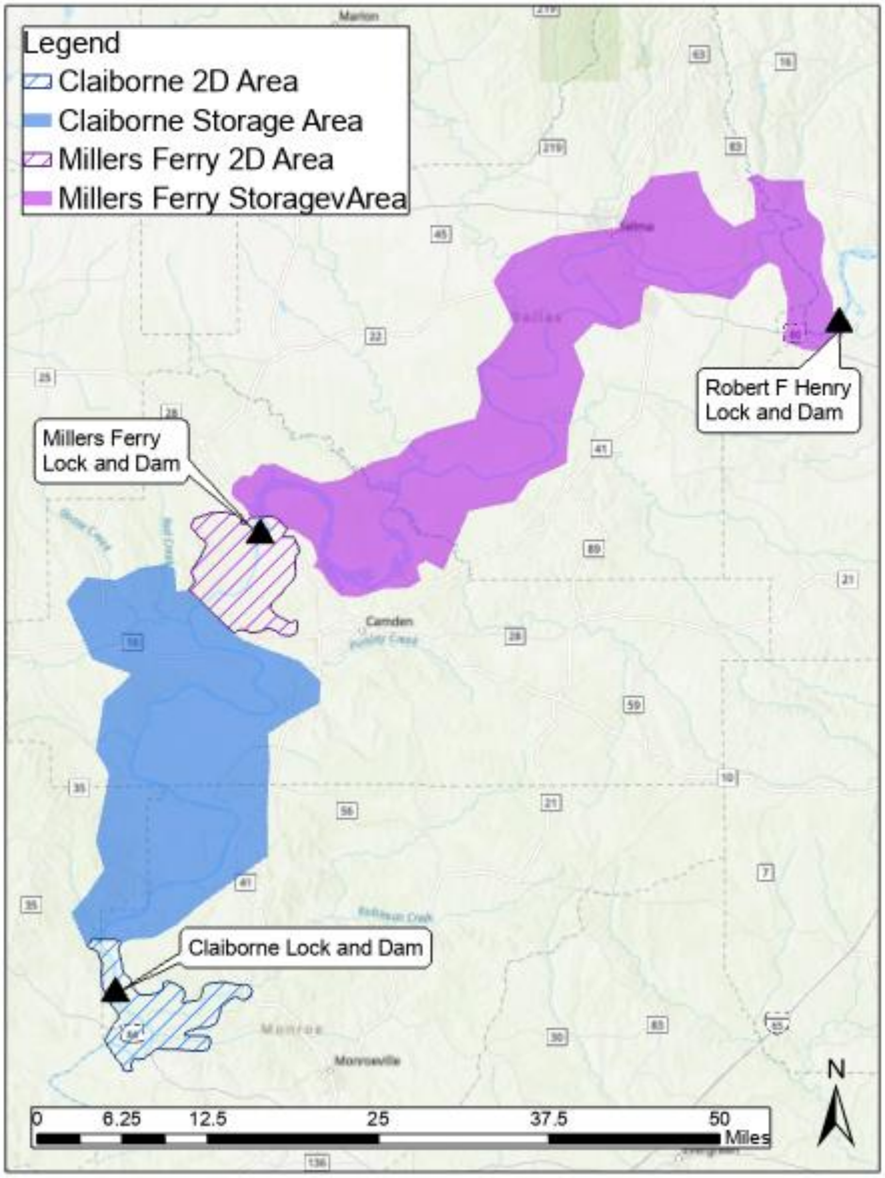
H.3.3. Hydraulic Modeling Approach

There are no existing hydraulic models for the area of interest along the Alabama River between Millers Ferry Lock and Dam to below Claiborne Lock and Dam. . Therefore, a HEC-RAS Version 6.2 Unsteady 2-Dimensional (2D) Model was developed. Reasons supporting 2D modeling included the following:

1. The terrain in the area is extremely flat, meaning water flows in multiple directions as it enters the floodplain.
2. Velocity for the fish passage measures are complex and there is a need for refinement.

The model consists of two separate pieces, one for each dam to shorten model runs for the analysis of alternatives. Each dam has a 2D mesh covering a few miles upstream to a few miles downstream of the structure. The reservoir for each project is covered by a storage area (SA). Figure H.36 shows the hydraulic modeling extents.

Figure H.36: Schematic of the hydraulic modeling extents for the Alabama River and surrounding study area



The Millers Ferry Lock and Dam model has an upstream boundary condition which routes an inflow hydrograph through the storage area covering William "Bill" Dannelly Reservoir (Millers Ferry Lock and Dam reservoir). The storage area is then connected via SA/2D connection to the 2D mesh covering Millers Ferry Lock and Dam. The SA/2D connection has a one-foot weir with a weir coefficient of 3.3. There are two SA/2D connection lines representing the damming structure. One covers the left bank and powerhouse and the other covers the spillway and right overflow bank. Both connections included the gates and incorporated the gate schedules from the water control manual. To simplify gate operations used in the model, the gates were linked together and assumed to all be at the same gate height for each step. The downstream boundary condition was separated as the channel and the floodplain with normal depth at both. The outflow from this boundary condition was used to route through the Claiborne Lock and Dam model.

The Claiborne Lock and Dam model has a similar setup with an upper boundary condition which routes the hydrograph from Millers Ferry Lock and Dam model through the storage area that covers Claiborne's Upper Pool. The storage area then connects to the 2D mesh covering Claiborne Lock and Dam using a SA/2D area connection line. The SA/2D connection has a one-foot weir with a weir coefficient of 3.0. In the 2D mesh, there are three SA/2D Connection lines representing the damming surface. One for each overbank and a third for the gated spillway and fixed crest spillway. The spillways SA/2D area connection included the gates and incorporated the gate schedules from the water control manual. To simplify gate operations used in the model, the gates were linked together and assumed to all be at the same gate height for each step. The downstream boundary condition was separated as the channel and the floodplain with normal depth at both.

H.3.3.1. Boundary Conditions and Tie-ins

The upstream boundary condition for Millers Ferry Lock and Dam is USGS 02427505 Alabama River at Millers Ferry Dam near Camden, AL. The upstream boundary condition for Claiborne Lock and Dam is the model output from the Millers Ferry Lock and Dam model. The downstream boundary condition for Millers Ferry Lock and Dam was set as a rating curve, which was determined by using a combination of calibration events and the Millers Ferry Tailwater Rating Curve. The curve was obtained from the current water control manual (USACE, 2015) for the project, and is shown on Figure H.37. Since the downstream boundary is within Claiborne pool, the rating curve was determined to be the best approach in order to account the backwater effect/pooling. The downstream boundary condition for Claiborne was originally set to a normal depth of 0.000045. However, it was noted that the smaller calibration events were not calibrating well. This could be due to the proximity with the Mobile Bay or backwater. A rating curve was used instead which was determined by using the Claiborne tailwater rating curve with a slope correction factor to account for the downstream extent. Both downstream boundary conditions were sensitive to changes, therefore both were calibrated and validated using a total of seven flood events. Figure H.38 shows the finalized rating curve used in this analysis for Claiborne Lock and Dam.

Figure H.37: Miller's Ferry Tailwater Rating Curve (USACE, 2015)

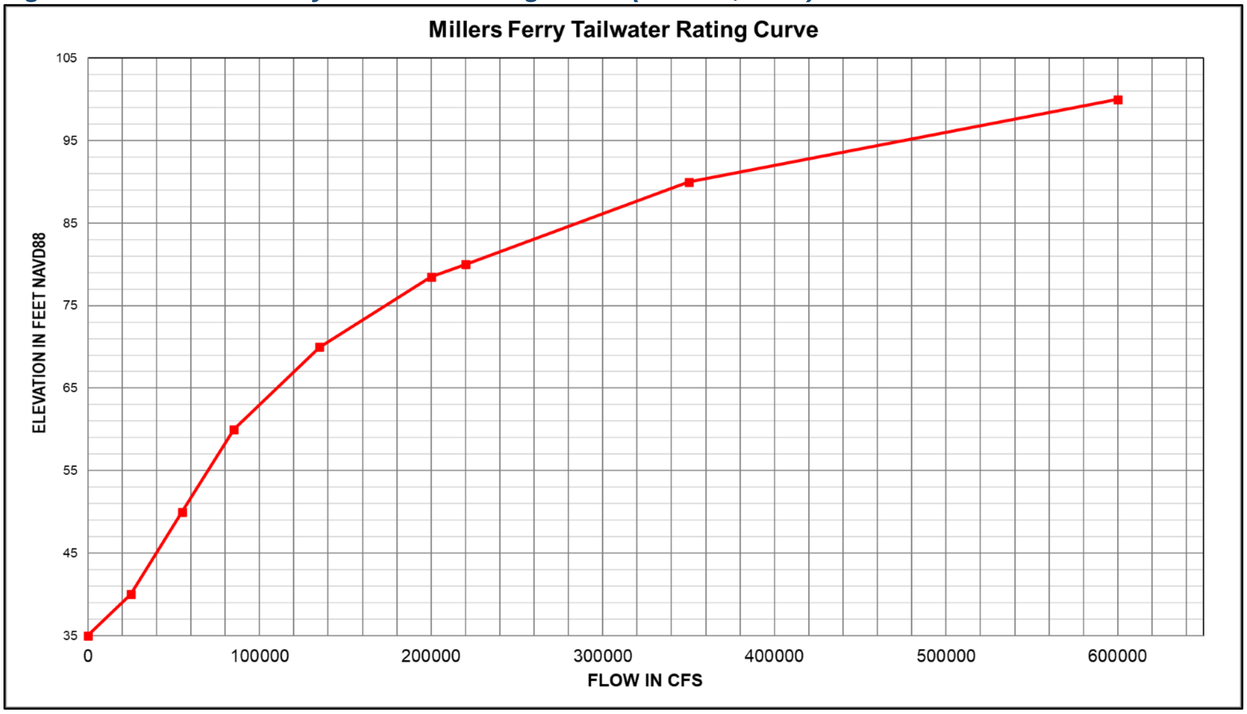
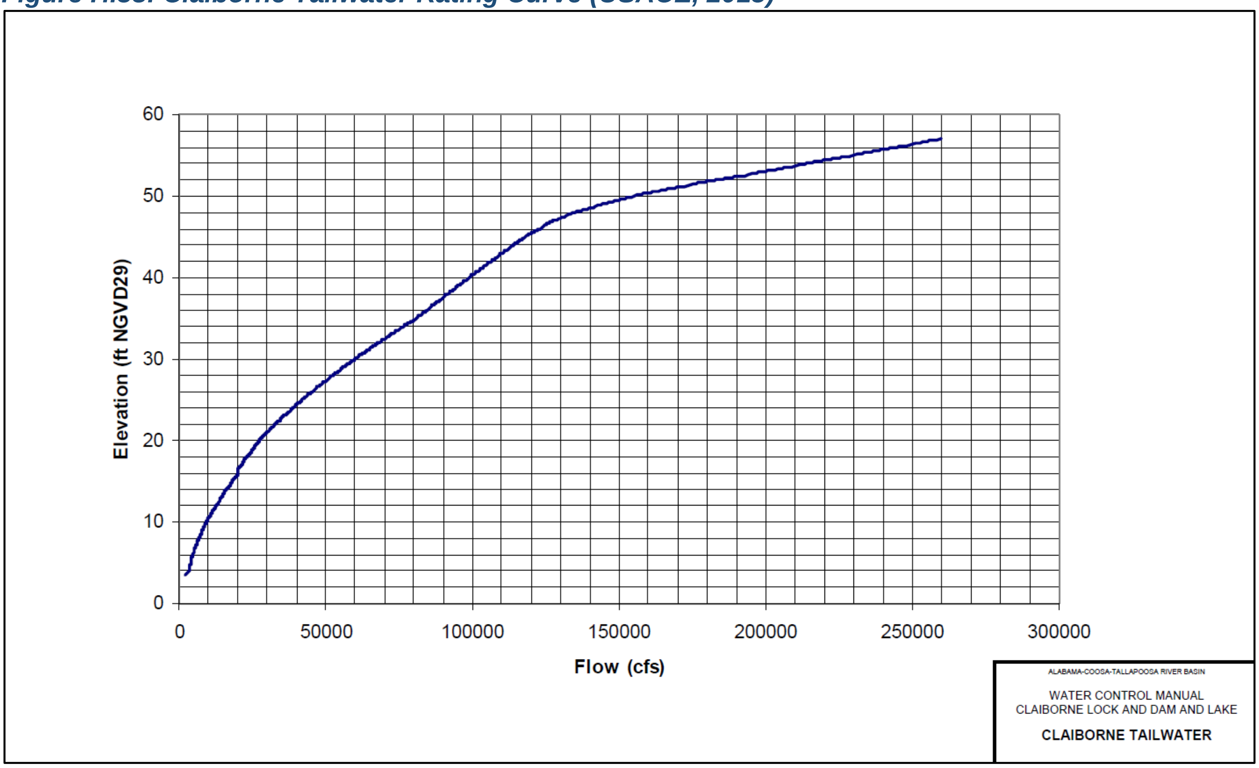


Figure H.38: Claiborne Tailwater Rating Curve (USACE, 2015)



H.3.3.2. Structures

In the model extents, there are two dams that are modeled using SA/2D area connection lines. For Millers Ferry Lock and Dam, there are two SA/2D area connection lines representing the dam face. The connection lines are the left bank/powerhouse and the spillway/lock with the right overflow dike. The Spillway at Miller Ferry Lock and Dam has 17 gates, and the powerhouse has 9 gates with 3 gates per turbine. The weir coefficient for all three SA/2D Area Connection lines is 2.65. The connection lines are plotted below in Figure H.39. Figure H.40 shows the powerhouse in more detail. Figure H.41 shows the gated spillway and right bank. Figure H.42 shows Claiborne Lock and Dam within the SA/2D Connection Editor. The gated spillway is located on the left descending side and the fixed crest spillway is located near the right descending bank.

Figure H.39: Millers Ferry Left Descending Bank and Powerhouse within SA/2D Connection Editor

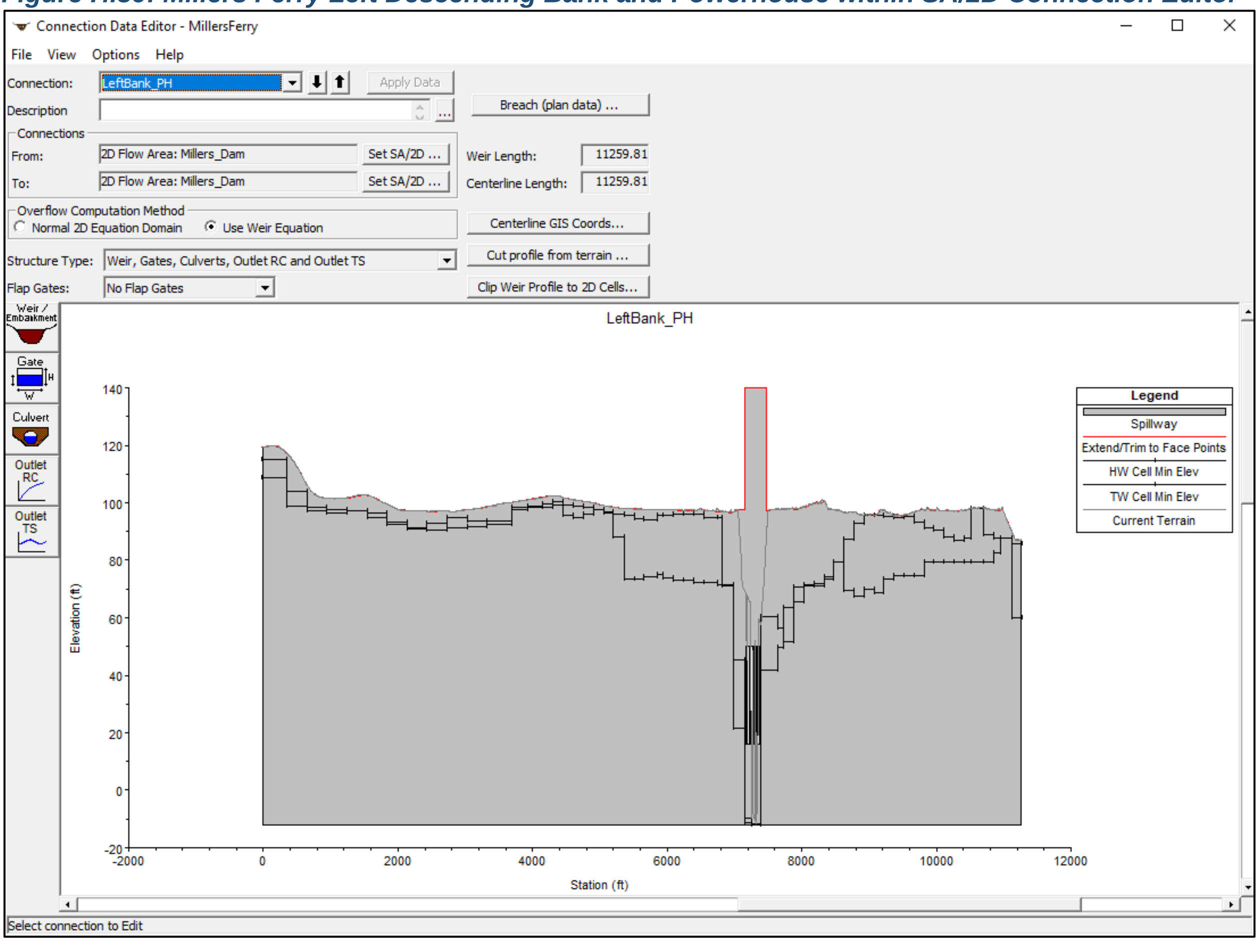


Figure H.40: Millers Ferry Powerhouse within SA/2D Connection Editor

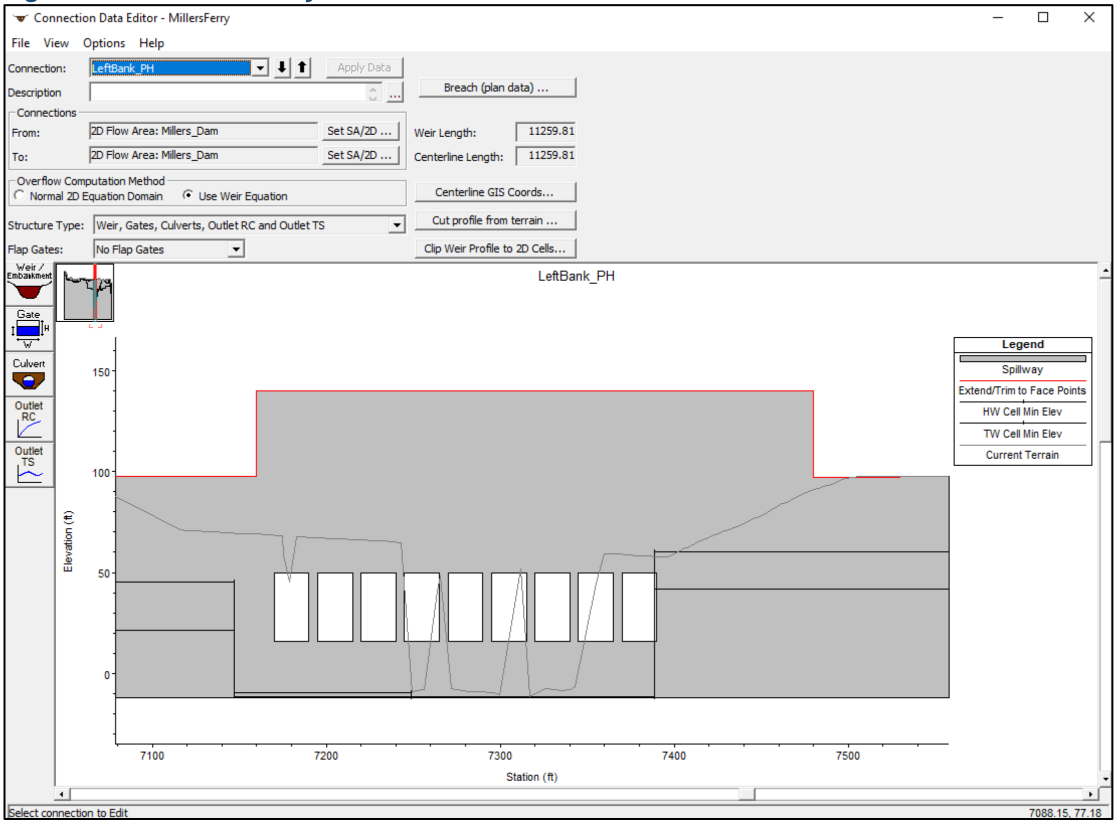


Figure H.41: Millers Ferry Gated Spillway and Right Descending Bank within SA/2D Connection Editor

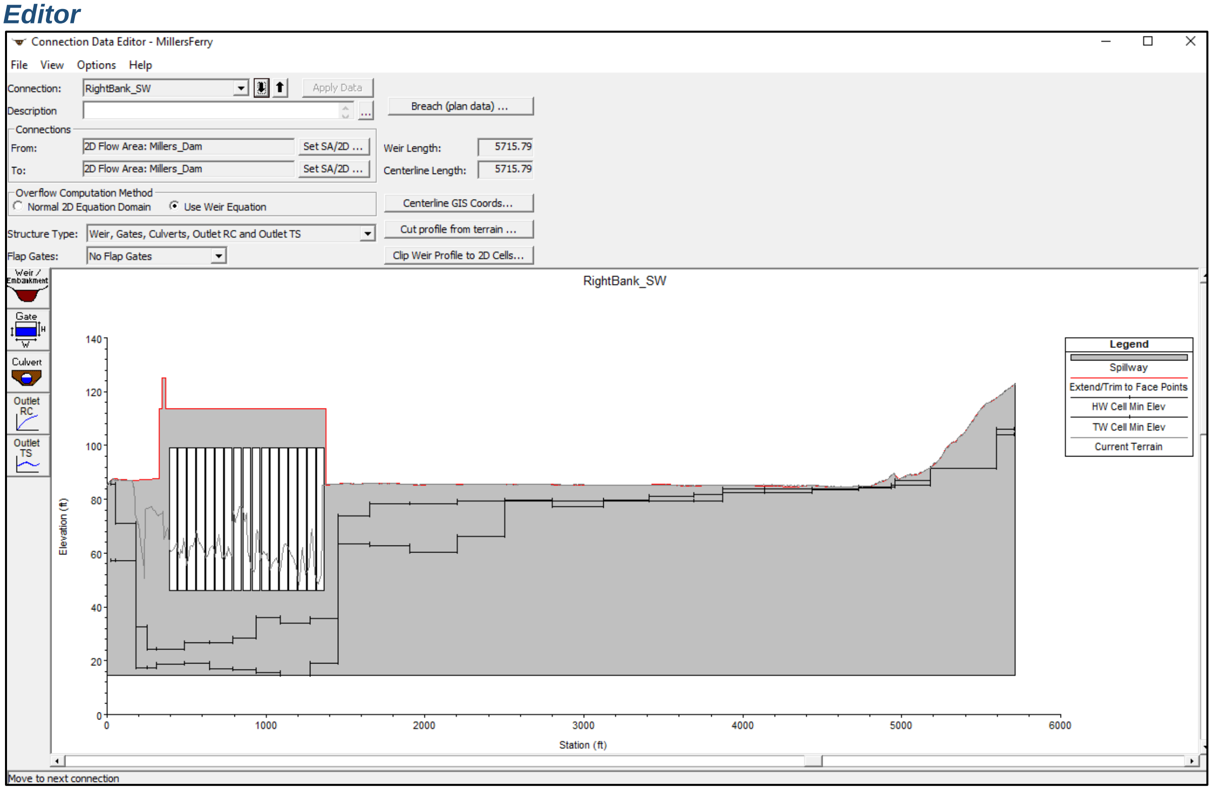
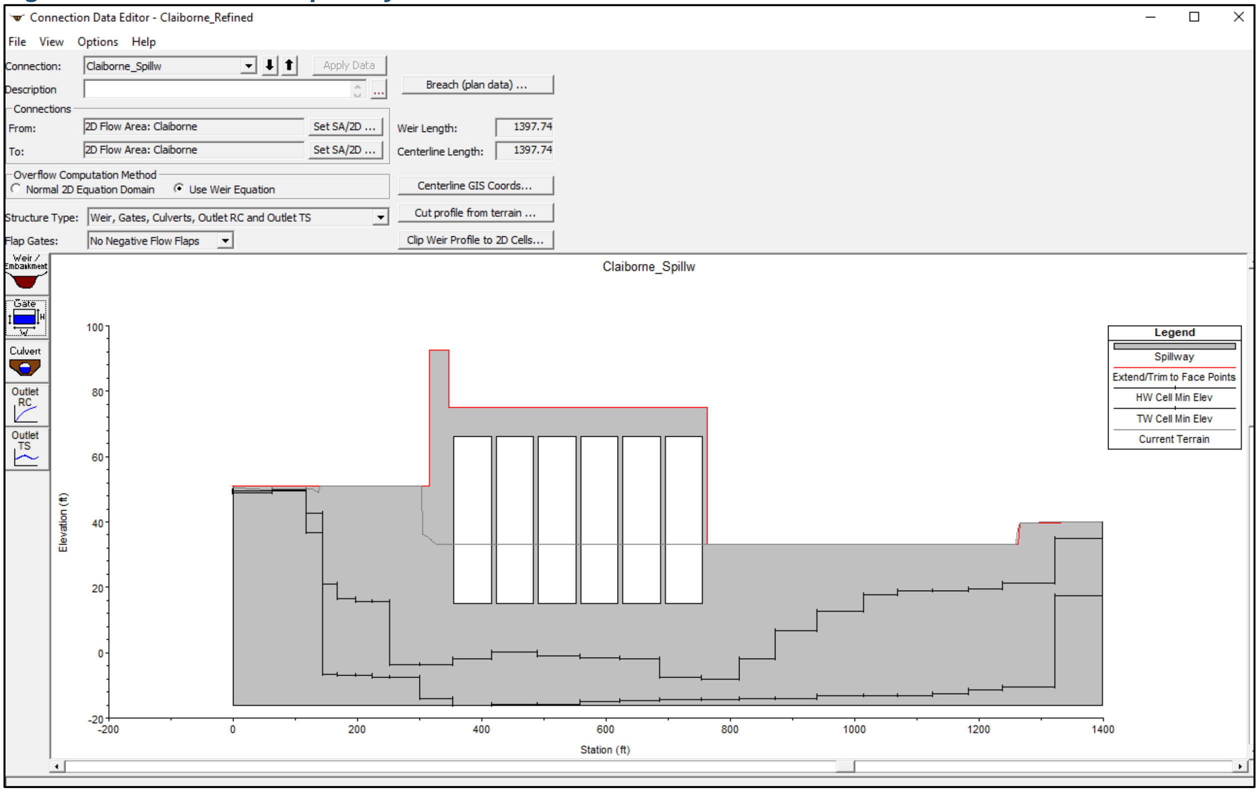


Figure H.42: Claiborne Spillway within SA/2D Connection Editor



H.3.3.3. Channel Roughness Values

Manning's roughness coefficients (Manning's "n-values") were established using guidance from the HEC-RAS *Hydraulic Reference Manual* (Reference 22). Manning's n-values used in the hydraulic computations were chosen based on engineering judgment from field observations of the streams and floodplain areas and utilizing the 2019 NLCD Land Use Dataset. Roughness values used for the study streams varied from 0.035 to 0.045 for the channel and 0.03 to 0.15 for the overbank areas. The lowest value for the overbank areas was for barren land and open fields. The higher values for the overbank areas represented the heavily wooded and forested areas, which accounts for a majority of the downstream extents of both dams. Table H.9 below contains the Manning's n-values associated with the NLCD Dataset imported into HEC-RAS.

Table H.9: Manning's n values by NLCD Land use Type

NLCD Value	NLCD Classification	Manning's n
0	No Data	0.035
42	Evergreen Forest	0.150
52	Shrub - Scrub	0.050
71	Grassland - Herbaceous	0.040
82	Cultivated Crops	0.050
95	Emergent Herbaceous Wetlands	0.100
90	Woody Wetlands	0.120
43	Mixed Forest	0.120

41	Deciduous Forest	0.120
11	Open Water	0.040
81	Pasture - Hay	0.050
21	Developed, Open Space	0.035
22	Developed, Low Intensity	0.080
23	Developed, Medium Intensity	0.120
24	Developed, High Intensity	0.150
31	Barren Land Rock – Sand - Clay	0.030

H.3.3.4. Infiltration Rates

The infiltration rates used in the HEC-RAS model were interpolated using the infiltration table obtained from the HEC-RAS User Manual. This parameter was not sensitive in this model since no rain on mesh was used for calibration.

H.3.4. HEC-RAS Results, Calibration, and Validation for Existing Conditions

To ensure the model is a good representation of the Alabama River near Millers Ferry Lock and Dam and Claiborne Lock and Dam, four events were selected for calibration.

The four events utilized to support the Existing Conditions hydraulic model calibration occurred in September 2012, January 2013, February 2013, and December 2015 with discharges of 80,000, 100,000, 180,000 and 160,000 cubic feet per second, respectively (Table H.10). The events have occurred within the last 20 years to avoid operational changes at either Millers Ferry Lock and Dam or Claiborne Lock and Dam.

In addition to the calibration simulations, three validation runs ensured the composite parameters used reasonably represented hydraulic conditions at the project locations. The validation events occurred in April 2005, February 2014, and April 2016. Additional model results comparing observed data with modeled data for the calibration and validation events can be found in Attachment H-1.

Both calibration and validation events were compared to the observed data for the headwater, tailwater, and discharge at both dams. Table H.10 shows the statistical tests R^2, Nash-Sutcliffe (NSE), Rank Sum Ratio (RSR), PBIAS-P, and PBIAS-V for each event. The values for R^2 range from 0 to 1.0 with the values closer to 1.0 showing a better fit. NSE ranges from negative values to 1.0 with values closer to 1.0 showing a better fit. Most of the statistical tests show the modeled results are representative of the observed values. Millers Ferry Lock and Dam Headwater shows the values do not match well with the model. However, this is most likely due to the simplified operations used within HEC-RAS. The general shape of the hydrograph appears to match well, but the observed data has a variation within each time step that HEC-RAS is not capturing. The modeled headwater elevation is within 0.5 feet for most of the calibration/validation events.

Table H.10: Statistics for Calibration and Validation Events

Calibration/ Validation	Event	Location	Statistics				
			R^2	NSE	RSR	PBIAS-P	PBIAS-V
Calibration	September 2012	Millers Ferry Headwater Elevation	0.27	0.21	0.89	0.15	N/A
		Millers Ferry Tailwater Elevation	0.94	0.92	0.28	-1.09	N/A
		Millers Ferry Discharge	0.98	0.98	0.13	-0.07	4.12
		Claiborne Headwater Elevation	0.42	0.03	0.98	-2.35	N/A
		Claiborne Tailwater Elevation	0.94	0.93	0.27	-3.75	N/A
		Claiborne Discharge	0.93	0.92	0.28	0.66	2.18
	January 2013	Millers Ferry Headwater Elevation	0.29	0.13	0.93	0.20	N/A
		Millers Ferry Tailwater Elevation	0.97	0.96	0.21	-1.17	N/A
		Millers Ferry Discharge	0.99	0.99	0.09	-0.48	0.00
		Claiborne Headwater Elevation	0.91	0.73	0.52	-3.17	N/A
		Claiborne Tailwater Elevation	0.96	0.93	0.26	-2.62	N/A
		Claiborne Discharge	0.97	0.97	0.18	-1.06	1.72
	February 2013	Millers Ferry Headwater Elevation	0.64	1.00	0.01	0.38	N/A
		Millers Ferry Tailwater Elevation	0.96	0.98	0.15	-3.14	N/A
		Millers Ferry Discharge	1.00	1.00	0.06	-0.12	1.65
		Claiborne Headwater Elevation	0.98	0.96	0.21	-3.53	N/A
		Claiborne Tailwater Elevation	0.97	0.93	0.27	-2.82	N/A
		Claiborne Discharge	0.96	0.89	0.33	9.90	13.27
	December 2015	Millers Ferry Headwater Elevation	0.35	0.17	0.91	0.30	N/A
		Millers Ferry Tailwater Elevation	0.97	0.96	0.21	2.91	N/A
		Millers Ferry Discharge	0.99	0.99	0.11	-1.30	1.26
		Claiborne Headwater Elevation	0.97	0.95	0.23	2.54	N/A
		Claiborne Tailwater Elevation	0.96	0.92	0.29	3.65	N/A
		Claiborne Discharge	0.96	0.86	0.37	-11.16	2.38
Validation	April 2005	Millers Ferry Headwater Elevation	N/A	N/A	N/A	N/A	N/A
		Millers Ferry Tailwater Elevation	0.92	0.79	0.46	0.92	N/A
		Millers Ferry Discharge	0.96	0.96	0.21	-1.52	-1.60
		Claiborne Headwater Elevation	0.98	0.96	0.20	-0.17	N/A
		Claiborne Tailwater Elevation	0.97	0.90	0.31	0.53	N/A
		Claiborne Discharge	0.97	0.90	0.32	11.17	11.89
	February 2014	Millers Ferry Headwater Elevation	0.46	0.29	0.84	0.19	N/A
		Millers Ferry Tailwater Elevation	0.95	0.91	0.29	-0.12	N/A
		Millers Ferry Discharge	0.96	0.96	0.19	-0.17	4.01
		Claiborne Headwater Elevation	0.88	0.84	0.40	-0.60	N/A
		Claiborne Tailwater Elevation	0.91	0.82	0.43	0.53	N/A
		Claiborne Discharge	0.91	0.87	0.36	4.29	-0.25
	April 2016	Millers Ferry Headwater Elevation	0.01	-2.69	1.92	0.06	N/A
		Millers Ferry Tailwater Elevation	0.94	0.94	0.25	5.13	N/A
		Millers Ferry Discharge	0.99	0.99	0.11	0.30	1.59
		Claiborne Headwater Elevation	0.82	0.79	0.46	3.07	N/A
		Claiborne Tailwater Elevation	0.90	0.77	0.48	4.67	N/A
		Claiborne Discharge	0.89	0.81	0.43	-2.42	3.18

Using the validated hydraulic model and flows from the flow-frequency analysis as inputs to the model, the frequency simulations were run. The 0.50, 0.20, 0.10, 0.04, 0.02, 0.01, 0.005, and 0.002 annual exceedance probability (AEP) event simulations produced profiles representative of the flooding potential for existing conditions. The stage frequencies for Millers Ferry and Claiborne are shown in Figure H.43 and Figure H.44, respectively. Table H.11 shows the existing conditions flows and estimated elevations.

Table H.11: Estimated Headwater, Tailwater, and Discharge at each dam site for Existing Conditions

Annual Exceedance Probability (AEP)	Millers Ferry Lock and Dam Peak Flow (CFS)	Millers Ferry Peak Headwater Elevation (ft-NAVD88)	Millers Ferry Peak Tailwater Elevation (ft-NAVD88)	Claiborne Lock and Dam Peak Flow (CFS)	Claiborne Peak Headwater Elevation (ft-NAVD88)	Claiborne Peak Tailwater Elevation (ft-NAVD88)
0.5	150,330	80.6	74.4	130,360	50.6	49.8
0.2	189,560	80.8	79.6	158,220	54.9	54.0
0.1	209,660	82.9	81.7	173,220	56.5	55.5
0.04	219,500	85.9	82.5	184,880	57.7	56.7
0.02	236,490	87.5	83.6	190,690	58.2	57.2
0.01	256,320	87.3	85.2	206,600	60.0	58.9
0.005	263,640	88.4	85.8	211,200	60.6	59.5
0.002	277,760	90.0	86.7	214,060	61.0	59.9

Figure H.43: Modeled Stage Frequency at Millers Ferry Lock and Dam

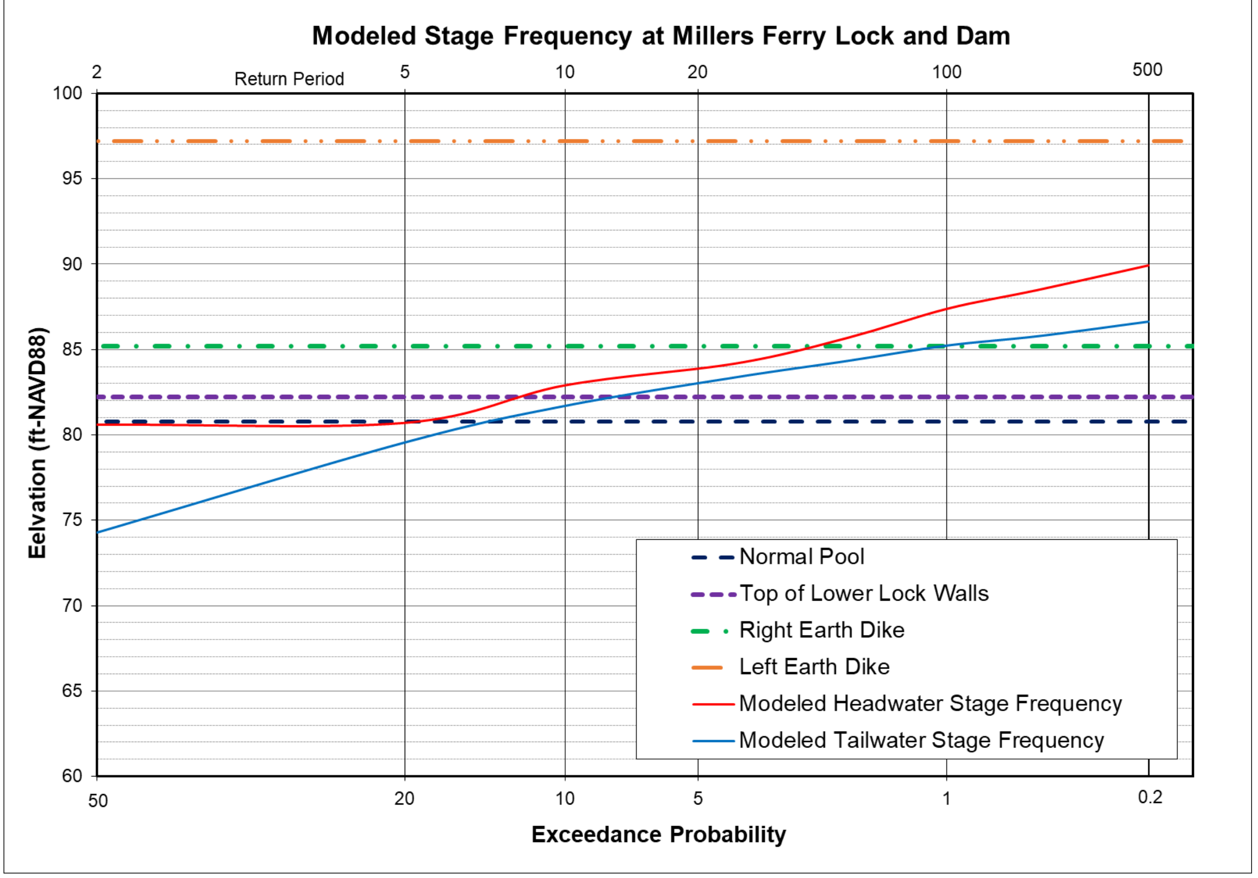
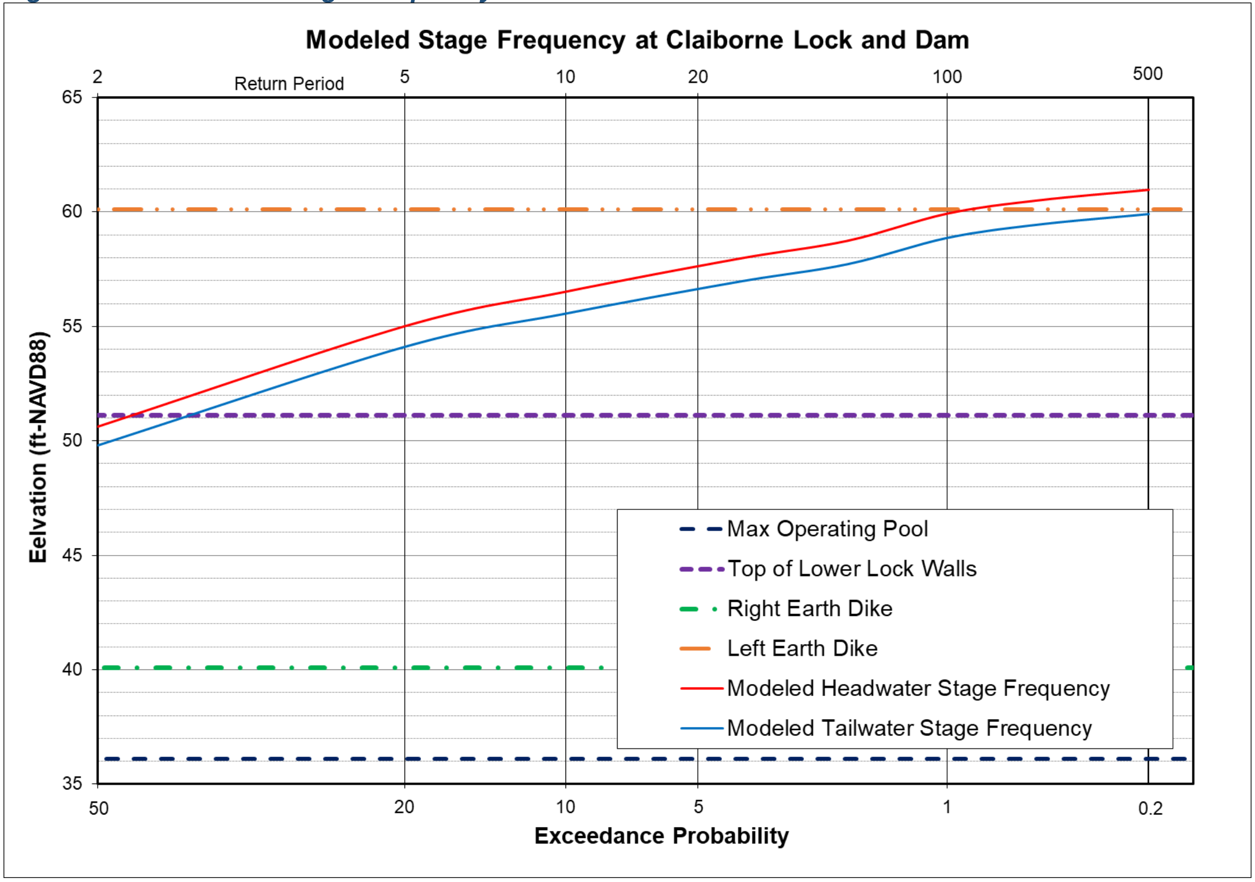


Figure H.44: Modeled Stage Frequency at Claiborne Lock and Dam



H.3.5. Future Without-Project Conditions

As conditions in the basin above Millers Ferry and Claiborne Lock and Dams change over the 50-year planning period, there is uncertainty in the future hydrologic conditions. The primary driver to changes in hydrology for this area were determined to be climate change.

The climate change analysis presented in this report does indicate some consensus that there will be an increase in extreme precipitation events in the southeast, but there is not a strong consensus that this will result in an increase in peak river flows. One of the main reasons for this is there has been, and will continue to be, an increase in temperatures and an increase in the severity and frequency of droughts in the southeast. Since the 1970s, temperatures in the southeast have been gradually increasing. This has caused an increase in soil moisture deficits, increasing groundwater infiltration and evapotranspiration. This is one contributor that is likely to offset the increase in runoff. This is reinforced by the lack of extreme flow events the Alabama River has experienced since the 1990s despite no sharp drop in peak annual precipitation. In the climate change assessment, observed gage data shows there has been a consistent drop in annual peak flows near on the Alabama River both at Montgomery, AL and near Claiborne Lock and Dam.

Due to no anticipated impacts to hydrology, the PDT decided to use existing conditions to represent the future condition. However, for the purpose of testing the selective plan, two climate change datasets created during the Alabama-Coosa Reallocation Study will be used to test the selected plan sensitivity to climate change. The first dataset is considered drier and has lower flows with more droughts. The second dataset is wetter with more frequent flood events.

H.4. Preliminary Design Assumptions and Sizing for Fish Passage Measures

The following sections describe the design assumptions that were used to design the fish passage measures at both dam sites. The primary factors used were depth, slope, width, velocity, and flow going through the channel. These values were determined by using several references including:

- "Nature-like Fishway Passage Design Guidelines for Atlantic Coast Diadromous Fishes" (May 2016)
- "Fish Passage Engineering Design Criteria" (June 2019)
- "Lock and Dam 22 Fish Passage Improvement Project" (2021)
- "Savannah Harbor Expansion Project (SHEP), Georgia and South Carolina, Fish Passage at New Savannah Bluff Lock and Dam (NSBLD) Post-Authorization Analysis Report and Environmental Assessment"

For this region, there are limited resources available on fish passage requirements and data for the target species. For this level of design, two target species were used to determine the design parameters/assumptions. These included Striped Bass and Gulf Sturgeon. The Gulf Sturgeon does not have as much data available, therefore the Atlantic Sturgeon data presented in some of the references was used as a surrogate species. The PDT and non-federal sponsor agreed that the Atlantic Sturgeon and Stiped Bass would be representative of the target species for this project.

H.4.1. Design Assumptions

For both projects the following design assumptions were made:

- The maximum slope recommended for Atlantic Sturgeon is 2%, therefore the PDT moved forward with this as the maximum slope for any fish passage measure.
- A minimum of 5 feet of channel depth is needed to accommodate larger fish species such as sturgeon or paddlefish.
- A minimum channel width for both bypass and rock weir needs to be greater than 50 feet due to size and behavior of larger fish species.
- The mean velocities for channel based on Manning's Equation was considered acceptable within the range between 3 ft/s to 8 ft/s. The PDT wanted to ensure there was water movement, but not enough to induce erosion or was too strong for the target fish species.

H.4.1.1. Millers Ferry Lock and Dam

The following design assumptions were made for Millers Ferry Lock and Dam:

- No minimum flow requirement is needed for Millers Ferry Lock and Dam since the channel will be below normal pool elevation and controlled by a gated structure.

- A maximum flow for the channel at normal pool was calculated to be approximately 10% of flow observed during key migration period (January - March) when flows are higher. These yield a flow range of 3,500 to 5,500 cfs at maximum normal pool capacity for the channel.
- Any tie-ins on right bank will have starting elevation of approximately 75 ft-NAVD88 due bathymetry in those areas.

H.4.1.2. Claiborne Lock and Dam

The following design assumptions were made for Claiborne Lock and Dam:

- Anything below elevation 33.1 ft-NAVD88 (fixed crest weir elevation) has been screened due to concerns of overturning at Millers Ferry Dam.
- The maximum flow for the channel during a mid-sized flood (40 ft-NAVD88, when flood begin to go out of bank) was calculated to be approximately 10% of flow observed during key migration period (January - March) when flows are higher. These yield a flow range of 3500 to 5500 cfs at maximum normal pool capacity for the channel. Since Claiborne is downstream of Millers +/- 500 cfs was added to accommodate any addition/reduction in flows between the two projects.
- A minimum flow through the channel was calculated to be approximately 500 cfs to 2000 cfs which is 10% low flow of 5000 cfs during dry conditions and 10% of typical average flow (20,000cfs) for year for late migration/spawning season (April - June).

H.4.2. Sizing Matrix

After the assumptions for design were determined for each project, a matrix was developed to screen down the sizing based on depth, flow, and velocity of the channel. Table H.12 through Table H.15 show the matrices for the Right Bank Bypass Channels and Rock Weirs at both projects. Items in red did not meet the requirements that were described above. The green items were moved forward as the preliminary sizing for the channel or rock weir. For Millers Ferry Lock and Dam Rock Weir, two bottom width options appeared to work for the assumptions. The PDT decided to move forward with the 75-foot bottom width for consistency with the rock weir design at Claiborne Lock and Dam.

Table H.12: Millers Ferry Lock and Dam Right Bank Bypass Channel Flow and Velocity Estimations

Millers Ferry Lock and Dam Right Bank Bypass Channel - Velocity and Flow Calculations Using Manning's Equation														
Velocity (ft/s)								Flow (CFS)						
			Bottom Width (ft)								Bottom Width (ft)			
Depth (ft)	Elevation (ft-NAVD88)	Slope (ft/ft)	25	50	75	100		Depth (ft)	Elevation (ft-NAVD88)	Slope (ft/ft)	25	50	75	100
~~0~~	~~80.8~~	~~0.0059~~	~~0~~	~~0~~	~~0~~	~~0~~		~~0~~	~~80.8~~	~~0.0059~~	~~0~~	~~0~~	~~0~~	~~0~~
~~1~~	~~79.8~~	~~0.0058~~	~~1.7564~~	~~1.8176~~	~~1.8409~~	~~1.8531~~		~~1~~	~~79.8~~	~~0.0058~~	~~49.179~~	~~96.331~~	~~143.59~~	~~190.87~~
~~2~~	~~78.8~~	~~0.0057~~	~~2.6124~~	~~2.7594~~	~~2.8213~~	~~2.8555~~		~~2~~	~~78.8~~	~~0.0057~~	~~161.97~~	~~309.05~~	~~457.06~~	~~605.36~~
~~3~~	~~77.8~~	~~0.0056~~	~~3.2481~~	~~3.4744~~	~~3.5778~~	~~3.6371~~		~~3~~	~~77.8~~	~~0.0056~~	~~331.3~~	~~614.97~~	~~901.6~~	~~1189.3~~
~~4~~	~~76.8~~	~~0.0055~~	~~3.7653~~	~~4.0592~~	~~4.2026~~	~~4.2876~~		~~4~~	~~76.8~~	~~0.0055~~	~~557.27~~	~~1006.7~~	~~1462.5~~	~~1920.9~~
5	75.8	0.0053	4.2062	4.556	4.7363	4.8465		5	75.8	0.0053	841.25	1480.7	2131.4	2786.7
6	74.8	0.0052	4.5926	4.988	5.2016	5.3355		6	74.8	0.0052	1184.9	2035.1	2902.5	3777.6
7	73.8	0.0051	4.9372	5.3695	5.6127	5.7688		7	73.8	0.0051	1589.8	2668.6	3771.7	4886.2
8	72.8	0.005	5.2482	5.7101	5.9793	6.1558		8	72.8	0.005	2057.3	3380.4	4735.6	6106.5
5.8	75	0.0052	4.519	4.906	5.1133	5.2426		5.8	75	0.0052	1111.3	1917.8	2740.3	3569.8

Table H.13: Millers Ferry Lock and Dam Rock Weir Flow and Velocity Estimations

Millers Ferry Lock and Dam Rock Weir - Velocity and Flow Calculations Using Manning's Equation										
Velocity (ft/s)										
			Bottom Width (ft)							
Depth (ft)	Elevation (ft-NAVD88)	Slope (ft/ft)	25	50	75	100	200	300	400	500
~~0~~	~~80.8~~	~~0.0217~~	~~0~~	~~0~~	~~0~~	~~0~~	~~0~~	~~0~~	~~0~~	~~0~~
~~1~~	~~79.8~~	~~0.0214~~	~~2.9551~~	~~3.0304~~	~~3.0566~~	~~3.0699~~	~~3.0901~~	~~3.097~~	~~3.1004~~	~~3.1024~~
~~2~~	~~78.8~~	~~0.021~~	~~4.4352~~	~~4.6516~~	~~4.7298~~	~~4.7701~~	~~4.8323~~	~~4.8534~~	~~4.8641~~	~~4.8705~~
~~3~~	~~77.8~~	~~0.0206~~	~~5.5115~~	~~5.8985~~	~~6.0433~~	~~6.119~~	~~6.2373~~	~~6.278~~	~~6.2986~~	~~6.311~~
~~4~~	~~76.8~~	~~0.0203~~	~~6.3485~~	~~6.9197~~	~~7.1402~~	~~7.2573~~	~~7.4422~~	~~7.5065~~	~~7.5392~~	~~7.5589~~
5	75.8	0.0199	7.0208	7.7807	8.0829	8.2454	8.5051	8.5963	8.6429	8.6711
6	74.8	0.0196	7.5711	8.5189	8.9063	9.1171	9.4579	9.5788	9.6407	9.6783
7	73.8	0.0192	8.0269	9.1585	9.6327	9.8938	10.321	10.474	10.552	10.6
8	72.8	0.0189	8.4073	9.7164	10.278	10.59	11.107	11.294	11.39	11.449
Flow (CFS)										
			Bottom Width (ft)							
Depth (ft)	Elevation (ft-NAVD88)	Slope (ft/ft)	25	50	75	100	200	300	400	500
~~0~~	~~80.8~~	~~0.0217~~	~~0~~	~~0~~	~~0~~	~~0~~	~~0~~	~~0~~	~~0~~	~~0~~
~~1~~	~~79.8~~	~~0.0214~~	~~73.878~~	~~151.52~~	~~229.25~~	~~306.99~~	~~618.03~~	~~929.09~~	~~1240.2~~	~~1551.2~~
~~2~~	~~78.8~~	~~0.021~~	~~221.76~~	~~465.16~~	~~709.47~~	~~954.02~~	~~1932.9~~	~~2912.1~~	~~3891.3~~	~~4870.5~~
~~3~~	~~77.8~~	~~0.0206~~	~~413.37~~	~~884.78~~	~~1359.7~~	~~1835.7~~	~~3742.4~~	~~5650.2~~	~~7558.3~~	~~9466.5~~
~~4~~	~~76.8~~	~~0.0203~~	~~634.85~~	~~1383.9~~	~~2142.1~~	~~2902.9~~	~~5953.8~~	~~9007.8~~	~~12063~~	~~15118~~
5	75.8	0.0199	877.6	1945.2	3031.1	4122.7	8505.1	12894	17286	21678
6	74.8	0.0196	1135.7	2555.7	4007.8	5470.2	11350	17242	23138	29035
7	73.8	0.0192	1404.7	3205.5	5057.2	6925.6	14449	21994	29546	37100
8	72.8	0.0189	1681.5	3886.5	6166.7	8472.4	17771	27105	36449	45796

Table H.14: Claiborne Lock and Dam Right Bank Bypass Channel Flow and Velocity Estimations

Claiborne Lock and Dam Right Bank Bypass Channel - Velocity and Flow Calculations Using Manning's Equation													
Normal Pool (35 ft)													
Velocity (ft/s)							Flow (CFS)						
			Bottom Width (ft)							Bottom Width (ft)			
Depth (ft)	Elevation (ft-NAVD88)	Slope (ft/ft)	25	50	75	100	Depth (ft)	Elevation (ft-NAVD88)	Slope (ft/ft)	25	50	75	100
0	35.1	0.0144	0	0	0	0	0	35.1	0.0144	0	0	0	0
1	34.1	0.0139	2.3294	2.4106	2.4415	2.4578	1	34.1	0.0139	65.225	127.76	190.44	253.15
2	33.1	0.0135	3.4431	3.6368	3.7185	3.7635	2	33.1	0.0135	213.47	407.33	602.4	797.87
~~3~~	~~32.1~~	~~0.013~~	~~4.2527~~	~~4.549~~	~~4.6844~~	~~4.762~~	~~3~~	~~32.1~~	~~0.013~~	~~433.78~~	~~805.18~~	~~1180.5~~	~~1557.2~~
~~4~~	~~31.1~~	~~0.0125~~	~~4.8956~~	~~5.2777~~	~~5.4641~~	~~5.5747~~	~~4~~	~~31.1~~	~~0.0125~~	~~724.55~~	~~1308.9~~	~~1901.5~~	~~2497.5~~
~~5~~	~~30.1~~	~~0.0121~~	~~5.4285~~	~~5.8798~~	~~6.1126~~	~~6.2547~~	~~5~~	~~30.1~~	~~0.0121~~	~~1085.7~~	~~1910.9~~	~~2750.7~~	~~3596.5~~
Mid-sized Flood (40ft)													
Velocity (ft/s)							Flow (CFS)						
			Bottom Width (ft)							Bottom Width (ft)			
Depth (ft)	Elevation (ft-NAVD88)	Slope (ft/ft)	25	50	75	100	Depth (ft)	Elevation (ft-NAVD88)	Slope (ft/ft)	25	50	75	100
5	35.1	0.0144	5.9167	6.4087	6.6624	6.8173	5	35.1	0.0144	1183.3	2082.8	2998.1	3919.9
6	34.1	0.0139	6.4293	6.9828	7.2819	7.4694	6	34.1	0.0139	1658.8	2849	4063.3	5288.3
7	33.1	0.0135	6.8768	7.4789	7.8177	8.0351	7	33.1	0.0135	2214.3	3717	5253.5	6805.7
~~8~~	~~32.1~~	~~0.013~~	~~7.2708~~	~~7.9108~~	~~8.2837~~	~~8.5282~~	~~8~~	~~32.1~~	~~0.013~~	~~2850.2~~	~~4683.2~~	~~6560.7~~	~~8460~~
~~9~~	~~31.1~~	~~0.0125~~	~~7.619~~	~~8.2879~~	~~8.6899~~	~~8.9586~~	~~9~~	~~31.1~~	~~0.0125~~	~~3565.7~~	~~5743.5~~	~~7977.4~~	~~10240~~
~~10~~	~~30.1~~	~~0.0121~~	~~7.9268~~	~~8.6171~~	~~9.0436~~	~~9.3337~~	~~10~~	~~30.1~~	~~0.0121~~	~~4359.7~~	~~6893.7~~	~~9495.8~~	~~12134~~

Table H.15: Claiborne Lock and Dam Rock Weir Flow and Velocity Estimations

Claiborne Lock and Dam Rock Weir - Velocity and Flow Calculations Using Manning's Equation													
Normal Pool (35 ft)													
Velocity (ft/s)							Flow (CFS)						
			Bottom Width (ft)							Bottom Width (ft)			
Depth (ft)	Elevation (ft-NAVD88)	Slope (ft/ft)	50	75	100	200	Depth (ft)	Elevation (ft-NAVD88)	Slope (ft/ft)	50	75	100	200
0	35.1	0.0168	0	0	0	0	0	35.1	0.0168	0	0	0	0
1	34.1	0.0163	2.6474	2.6703	2.6819	2.6996	1	34.1	0.0163	132.37	200.27	268.19	539.92
2	33.1	0.0158	4.0348	4.1026	4.1376	4.1915	2	33.1	0.0158	403.48	615.39	827.52	1676.6
Mid-sized Flood (40ft)													
Velocity (ft/s)							Flow (CFS)						
			Bottom Width (ft)							Bottom Width (ft)			
Depth (ft)	Elevation (ft-NAVD88)	Slope (ft/ft)	50	75	100	200	Depth (ft)	Elevation (ft-NAVD88)	Slope (ft/ft)	50	75	100	200
5	35.1	0.0168	7.1439	7.4214	7.5706	7.809	5	35.1	0.0168	1786	2783	3785.3	7809
6	34.1	0.0163	7.7744	8.1279	8.3203	8.6314	6	34.1	0.0163	2332.3	3657.5	4992.2	10358
7	33.1	0.0158	8.305	8.735	8.9718	9.3589	7	33.1	0.0158	2906.8	4585.9	6280.3	13103

H.5. Hydraulic Modeling of Final Array of Alternatives

Hydraulic modeling of the final array of alternatives was performed to support the water management and environmental evaluation of the alternatives. There were five alternatives carried forward to the final array including a no action alternative. For this initial modeling approach, each dam was modeling separately, and output was provided to other disciplines for further analysis.

A rating curve was provided for each measure at both dams for use in the HEC-ResSim effort. The rating curves were developed by assuming maximum capacity of the channel and corelated with the respective pool elevation. This is not be confused with depth/elevation within the channel. The main objective of a rating curve was for use within HEC-ResSim by relating how much water would be released with respect to the pool elevation. In order to develop a rating curve, several model runs were completed to estimate the flow. For further information regarding the HEC-ResSim modeling efforts, see Attachment H-3. These model runs included the observed event from December 2015 and two hypothetical events, 0.04 AEP (25-year) and 0.01 AEP (100-year). A range of events was completed in order to observe any differences within how the channel performed and to obtain a wide operating range of the fish passage capacities.

Velocity maps were created for a few different inflows and provided to environmental to be used in the habitat model. The flows were 5,000, 50,000, and 150,000 cfs. The lower flow of 5,000 cfs shows modeled velocities within the channel if the channel is allowed to stay open during low flow/drought conditions. However, this condition may not happen frequently due to hydropower generation. The middle flow of 50,000 cfs is a typical yearly flood during January through March timeframe. This type of flow would likely allow for full power generation and the fish passage channel to be open. The upper flow of 150,000 cfs is close to a 0.5 AEP (2-year) event. During this type of flow conditions, the Claiborne pool would likely be too high for power generation and it is anticipated that a fish passage channel would be completely open. All of the velocity maps generated are shown in Attachment H-2. Further details regarding the hydraulic modeling of alternatives are discussed below.

H.5.1. Alternative 1: No Action Alternative

For the no action alternative, there was no additional hydraulic modeling.

H.5.2. Alternative 3: Fixed Weir Rock Arch – Both Dams

Modeling of the rock weirs involved performing hydraulic modeling with modified terrain data to include the addition of the rock weir structures. For Millers Ferry Lock and Dam, the rock weir was located west of the existing gated spillway. For Claiborne Lock and Dam, the rock weir was located on the western portion of the fixed crest weir. The modified terrain for both structures is shown below in Figure H.45 and Figure H.46.

Figure H.45: Modified Terrain for Millers Ferry Lock and Dam Rock Weir

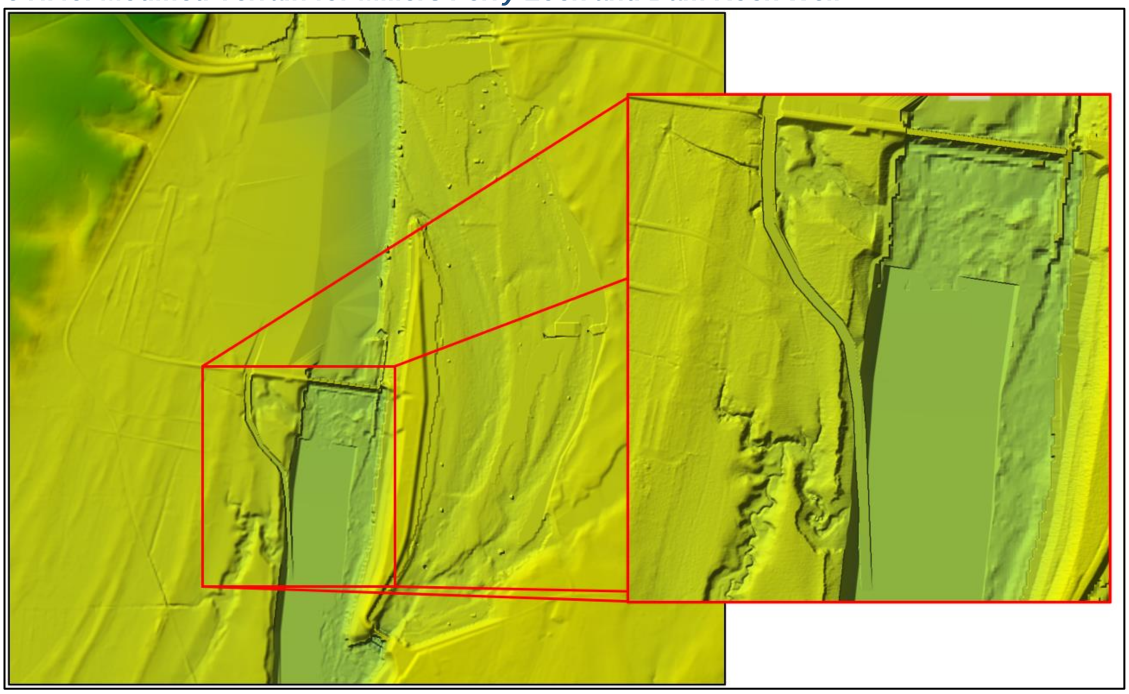
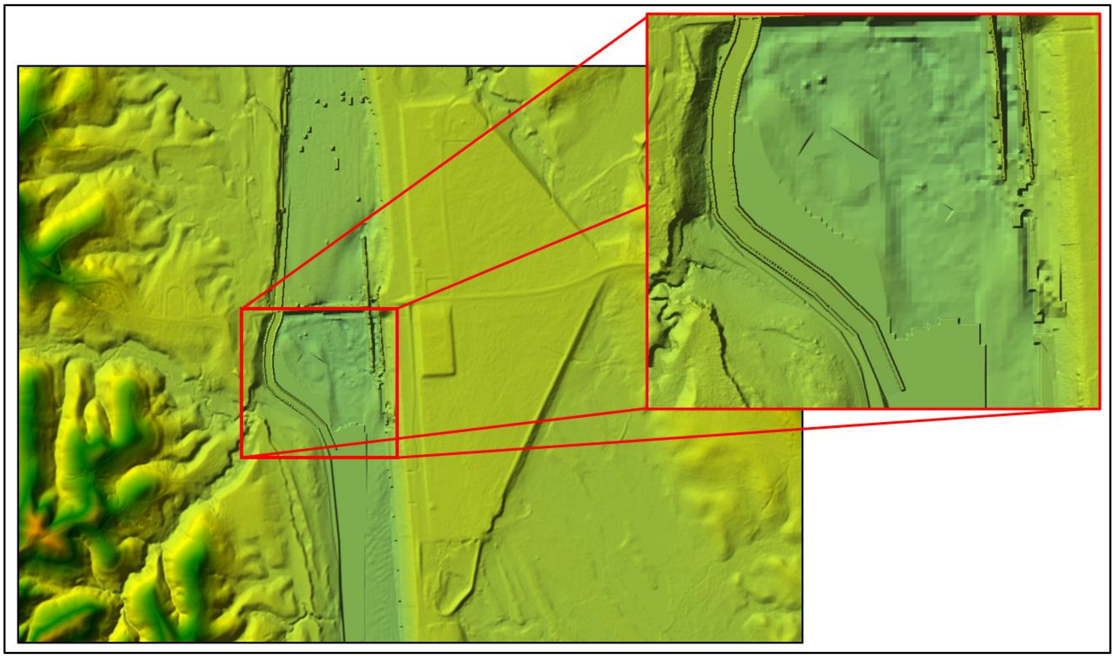


Figure H.46: Modified Terrain for Claiborne Lock and Dam Rock Weir



For Claiborne Lock and Dam, the existing fixed crest spillway would also have a reduced capacity due to the rock weir's size and placement. An updated rating curve for the fixed crest weir and rating curves for both rock weirs were provided for the HEC-ResSim effort. For more information regarding the HEC-ResSim effort, refer to Attachement H-3. The rating curves are shown below in Table H.16, Table H.17, and Table H.18.

Table H.16: Rating Curve for Millers Ferry Lock and Dam Rock Weir

Pool Elevation (ft-NAVD88)	Estimated Flow (cfs)
75	0
76	500
77	1,000
78	1,500
79	1,800
80	2,200
81	2,500
82	3,000
83	3,700
84	4,400
85	4,600
86	5,200
87	5,500
88	5,700

Table H.17: Rating Curve for Claiborne Lock and Dam Rock Weir

Pool Elevation (ft-NAVD88)	Estimated Flow (cfs)
33.1	0
34.8	500
35.5	800
36	1,000
37	1,500
38	2,200
39	3,100
40	4,000
41	5,200
42	6,000
44	7,200
46	7,900
48	8,100
50	8,200
52	8,400
54	8,700
56	9,100
58	9,900
60	10,600
62	11,500

Table H.18: Updated Rating Curve for Claiborne Lock and Dam Fixed Crest Weir

Pool Elevation (ft-NAVD88)	Estimated Flow (cfs)
33.1	0
34.8	2,400
35.5	3,900
36	5,400
37	8,300
38	13,000
39	18,700
40	23,000
41	30,400
42	35,200
44	47,100
46	50,300
48	53,700

50	55,000
52	56,800
54	59,200
56	61,300
58	65,800
60	69,600
62	73,000

H.5.3. Alternative 5d: Natural Bypass Channel – Right Bank at Both Dams

Modeling of the bypass channels involved performing hydraulic modeling with modified terrain data to include the addition of the bypass channels located on the right descending bank for both dams. See Figure H.47 and Figure H.48 below.

Figure H.47: Modified Terrain for Millers Ferry Lock and Dam Right Bank Natural Bypass Channel

Figure H.48: Modified Terrain for Claiborne Lock and Dam Right Bank Natural Bypass Channel

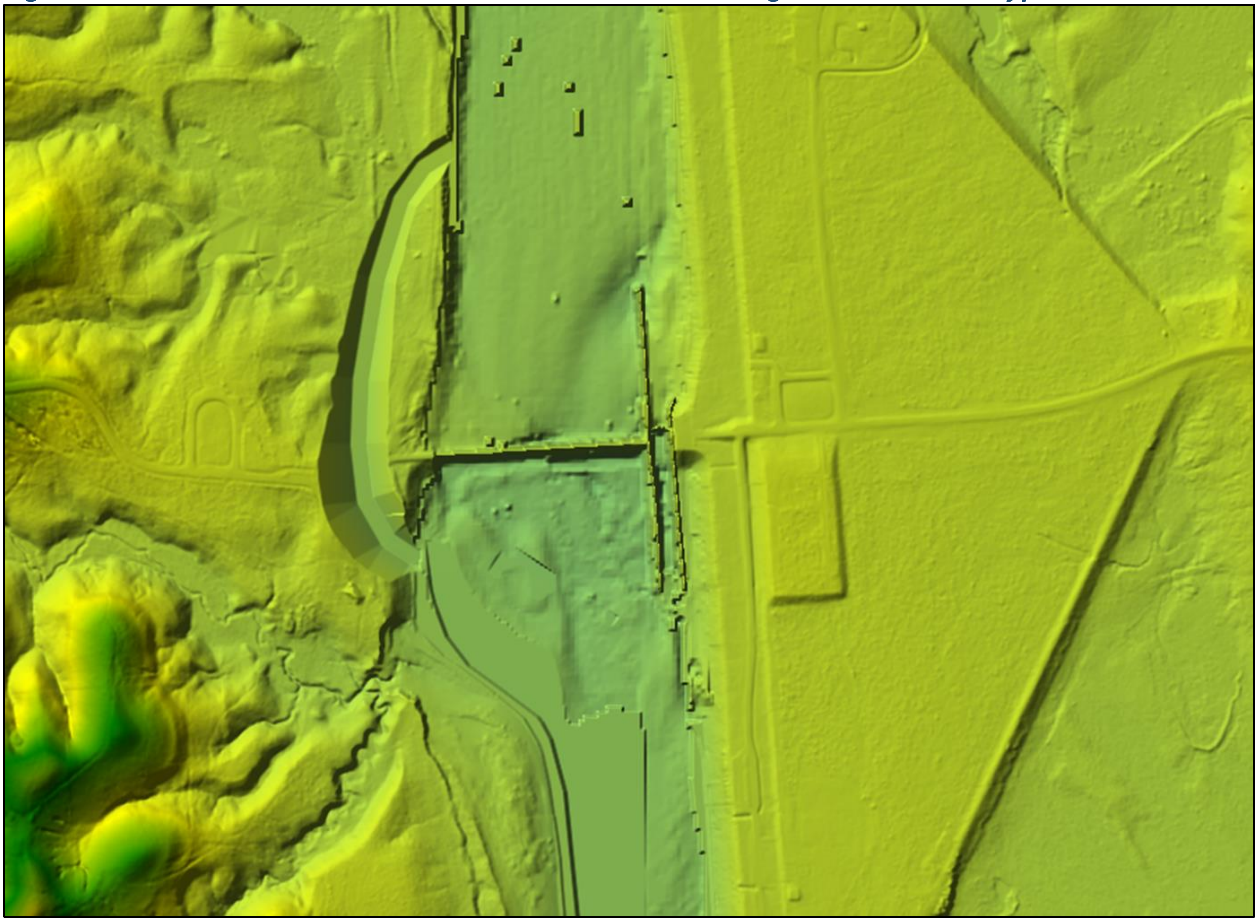


Table H.19: Rating Curve for Millers Ferry Lock and Dam Right Bank Natural Bypass Channel

Pool Elevation (ft-NAVD88)	Estimated Flow (cfs)
75	0
77	200
78	400
79	700
80	1200
80.7	1700
81	1900
82	3200
83	4400
84	5800
85	6800
86	8400
87	12200
88	15600

Table H.20: Rating Curve for Claiborne Lock and Dam Right Bank Natural Bypass Channel

Pool Elevation (ft-NAVD88)	Estimated Flow (cfs)
33.1	0
35	1200
36	2100
38	3700
40	7300
42	9500
44	14000
46	21000
48	27500
50	33800
52	40000
54	45500
56	52000
58	59200
58	59200
60	65800
62	70000

H.5.4. Alternative 12b: Fixed Weir Rock Arch at Claiborne and Natural Bypass on Right Bank at Millers Ferry

This alternative consists of the rock weir at Claiborne Lock and Dam discussed in H.5.2and the right bank bypass channel at Millers Ferry Lock and Dam discussed in H.5.3.

H.5.5. Alternative 13b: Fixed Weir Rock Arch at Millers Ferry and Natural Bypass on Right Bank at Claiborne

This alternative consists of the rock weir at Millers Ferry Lock and Dam discussed in section H.5.2 and the right bank bypass channel at Claiborne Lock and Dam discussed in section H.5.3.

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H.7. Attachment H-1: HEC-RAS Calibration and Validation Plots

H.8. Attachment H-2: HEC-RAS Velocity Maps

H.9. Attachment H-3: HEC-ResSim Modeling