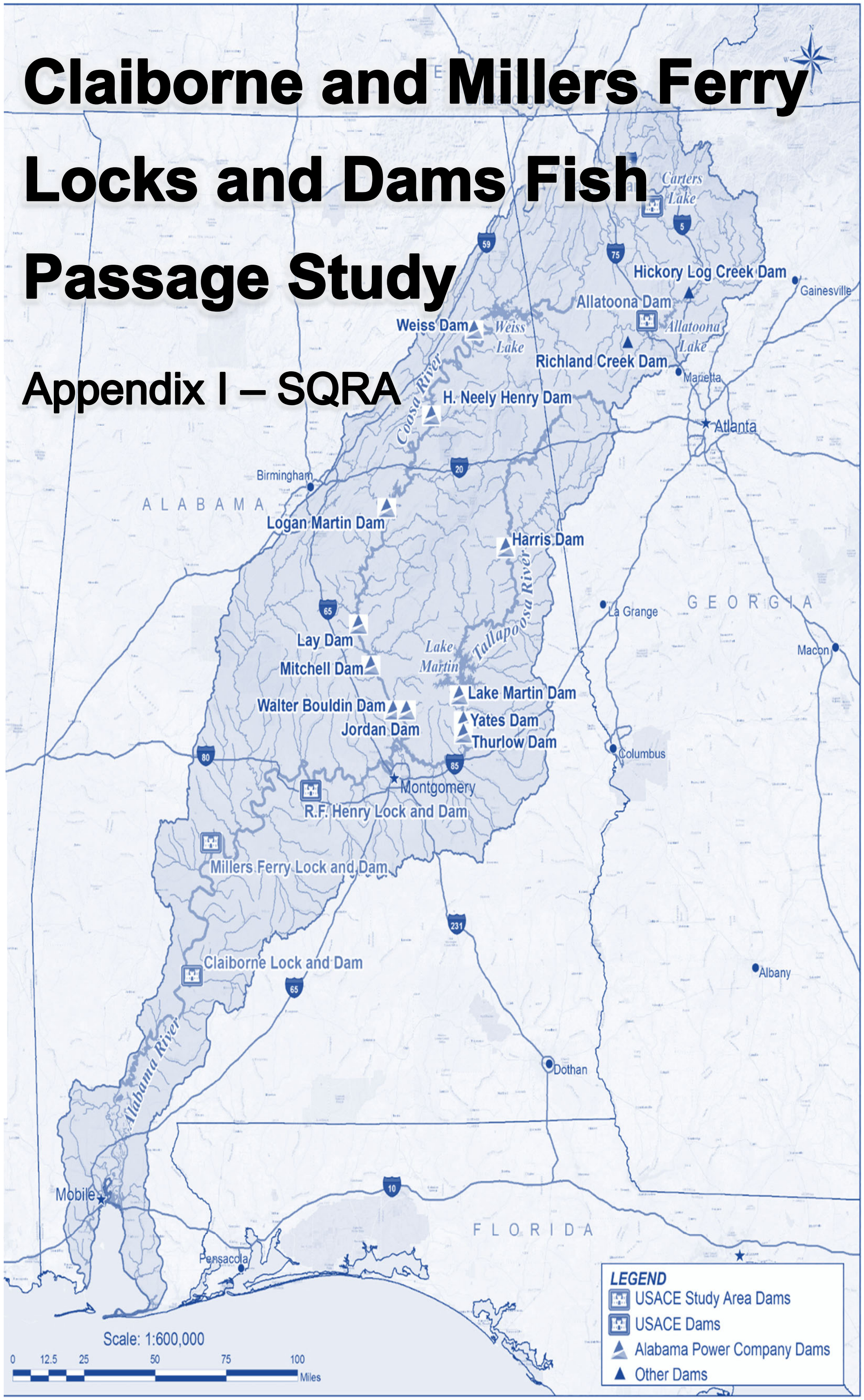
# Claiborne and Millers Ferry Locks and Dams Fish Passage Study

## Appendix I – SQRA




US Army Corps of Engineers®

The Nature Conservancy®

CESAJ-EN 30 November 2023

SUBJECT: Potential Failure Mode Analysis and Screening-Level Risk Assessment – Claiborne and Millers Ferry

## 1. Introduction

The United States Army Corps of Engineers (USACE) Jacksonville District (CESAJ) Risk Cadre along with members of the Project Delivery Team (PDT) from the USACE Mobile District (CESAM) have completed a Screening Level Risk Assessment (SLRA) on the Claiborne Lock and Dam and Millers Ferry Lock and Dam, Fish Passage Feasibility Study. The projects are on the Alabama River in Alabama and are part of a larger system through the Alabama-Coosa-Tallapoosa (ACT) Basin that contains 5 USACE dams and 11 privately owned dams, shown in Figure 1. The tentatively selected plan (TSP) from the district feasibility study (at the time of the risk assessment workshop) developed conceptual designs for channelized fish ladders through or around the right abutment of both dams to encourage migration, spawning, foraging, and nurseries for native fish and mussel species. Figure 2 shows the existing project features of Millers Ferry Lock and Dam and the proposed alignment of the fish ladder bypass. Figure 3 shows the existing project features for Claiborne Lock and Dam and the proposed alignment of the fish ladder bypass. Figure 4 shows the alternative alignment for the fish ladder bypass assessed for the Claiborne Lock and Dam.



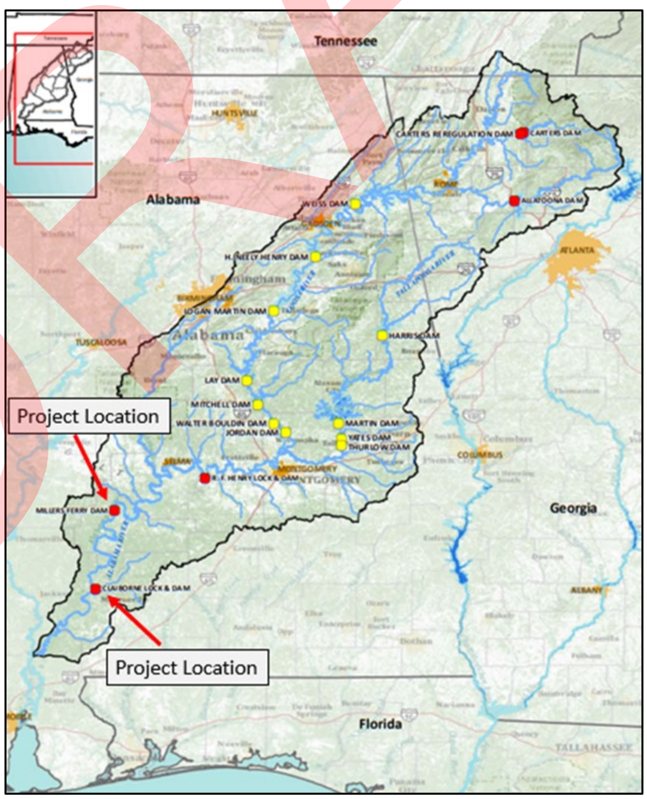



Figure 1 – Project Vicinity within the Alabama-Coosa-Tallapoosa Drainage Basin.

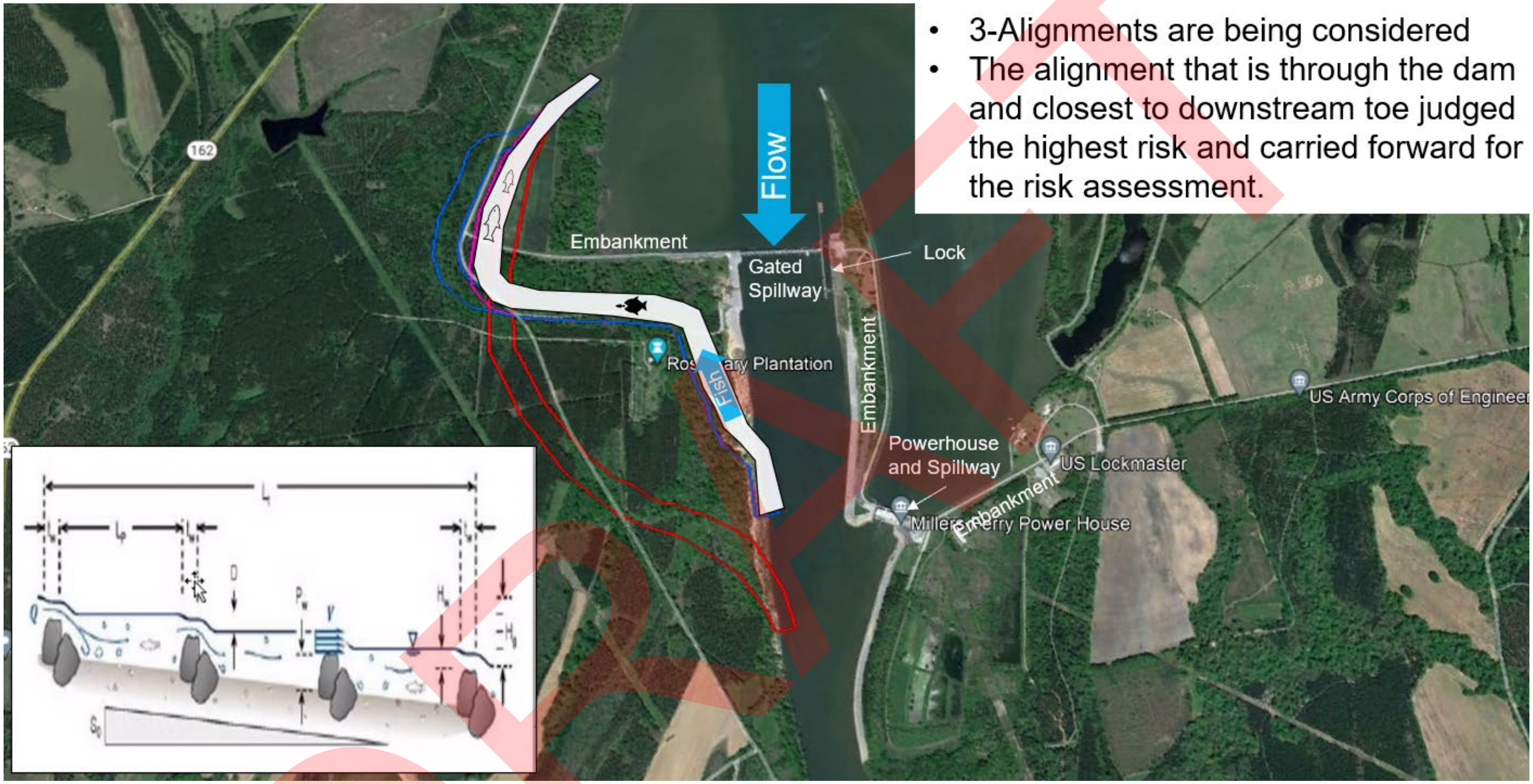



Figure 2 – Millers Ferry Lock and Dam Project Features and Proposed Fish Ladder Bypass.

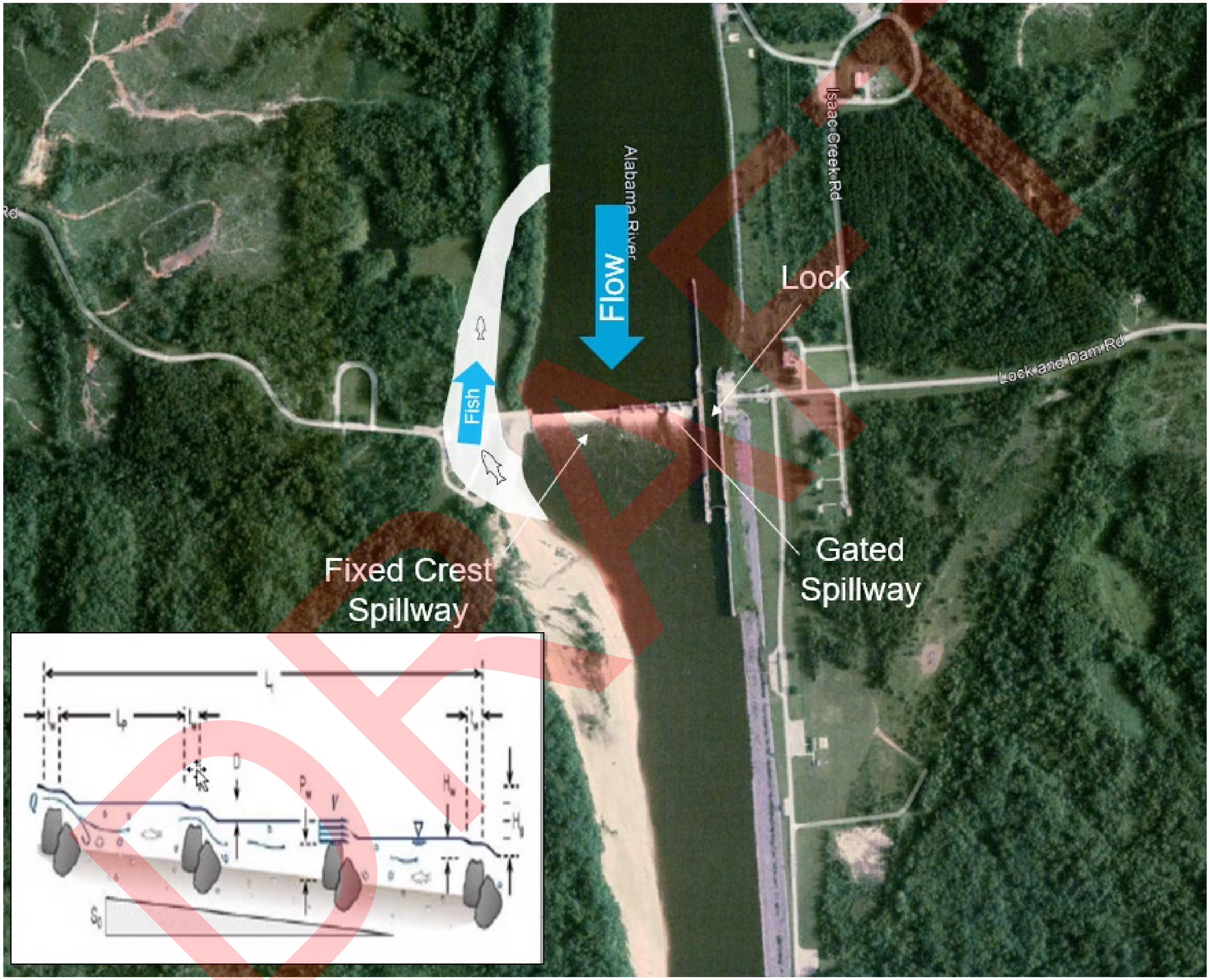



Figure 3 – Claiborne Lock and Dam Project Features and Proposed Fish Ladder Bypass.

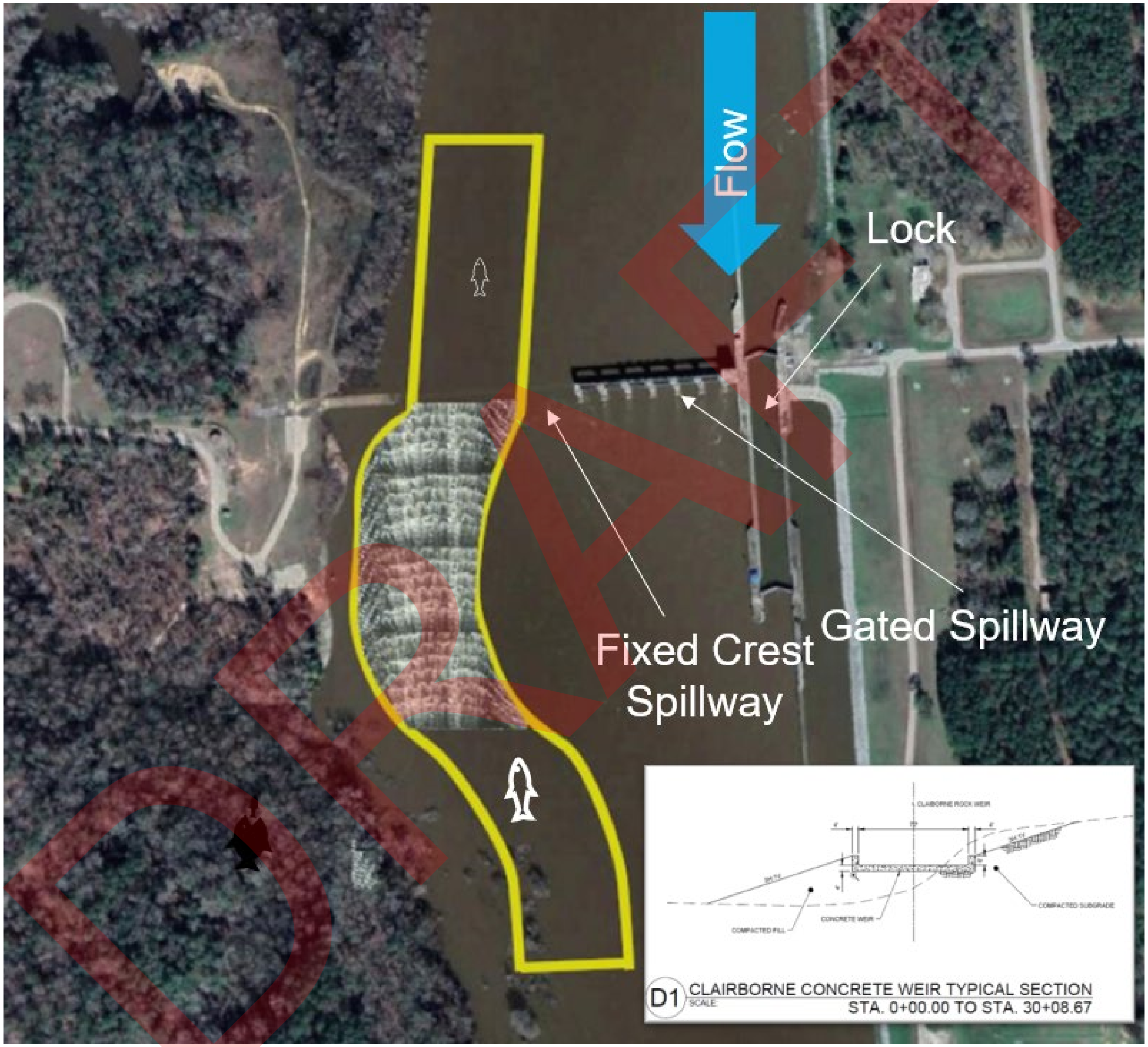





Figure 4 – Claiborne Lock and Dam Project Features and Alternative Proposed Fish Ladder Bypass.

## 2. SLRA

Risk is a measure of the probability and severity of undesirable consequences. Risk assessment is a broad term that encompasses a variety of analytic techniques that are used in different situations, depending upon the nature of the risk, the available data, and needs of decision makers. Risk assessments must also be scalable depending on the needs of the project and maturity of the design. At feasibility, typically only screening level risk assessments can be completed since the design maturity does not allow for more detailed assessment of all project components. SLRA includes mostly qualitative assessment of PFMs with numerical assessment completed when needed to inform the decision. The Potential Failure Mode Analysis (PFMA) leading to a SLRA is a systematic, evidence-based approach for quantifying and describing the nature, likelihood, and magnitude of risk associated with a water impounding structure. The risk assessment process attempts to answer the following questions:

- What can go wrong?
- How can it happen?
- What is the likelihood?
- What are the consequences?
- How can risk be reduced with respect to As Low As Reasonably Practicable (ALARP) Principles?

The facilitated PFMA and resultant SLRA was completed to identify and assess potential failure modes (PFMs) and to use the potential failure mechanisms to better inform the dam modifications to include fish ladders around or over the dams. As part of this process, the team examined the as-built design information, pertinent construction records that were available, geotechnical site characterization data, previous Periodic Assessments (PA) completed for the dams, hydrologic characteristics of the river, and feasibility level design concepts for the fish ladders. The objectives of the risk assessment include the following:

- Verify/adjust design to not violate the “do no harm” concept
- Identify PFMs for the proposed modifications, building on the previously completed PAs
- Complete potential failure mode analysis (PFMA) to inform the TSP design alternatives/ help identify the best alignments for the fish passages
- Brainstorm changes to the TSP to address any potential failure modes that may not be addressed by the conceptual designs
- Identify features that may need to be added to the TSP to reduce potential for significant cost growth in PED.

The following personnel listed in Table 1, from the USACE Jacksonville District (SAJ) and the Mobile District (SAM), completed the SLRA for this project in August 2023.

Table 1 – PFMA Participants.

| Name | Role / Discipline |
|---|---|
| John Kendall, P.E. | SAJ Risk Cadre Lead/Geotechnical Engineer |
| Brian Cornwell, E.I. | SAJ Risk Cadre Hydrology/Hydraulic Engineer |
| Jim Huff, P.E. | SAJ Risk Cadre Geotechnical Engineer |
| Corey Press, P.E. | SAJ Risk Cadre Structural Engineer |
| Joseph Woods | SAJ Risk Cadre Geologist |
| Kip Webber, E.I. | SAJ Risk Cadre Civil Engineer |
| Colin Rawls | SAJ Risk Cadre Economist |
| Cedric Smith | SAJ Economist |
| Tom Terry, P.E. P.G. | RMC Risk Advisor |
| Chris Marr, P.E. | SAM PDT ETL |
| Jonas White | SAM Project Manager |
| Chase Rourke, P.E. | SAM Geotechnical Engineer Lead |
| Ashley Throop, P.E. | SAM H&H Lead |
| Ashley Forwood | SAM Planner |
| Tonya Harrington | SAM Lead Planner |
| Mike FitzHarris, P.G. | SAM DSPM |
| Timothy Sandifer, P.E. | SAM Structural Engineer |
| HuiYing Huang, E.I. | SAM Water Management |
| James Hathorn, P.E. | SAM Water Management Section Chief |

## 3. Findings and Recommendations

This section summarizes the findings and recommendations of the SLRA. Detailed discussions of findings, recommendations, and supporting background information are presented throughout the following sections of this report. The following major findings and recommendations are split apart for the Millers Ferry Lock and Dam (Section 3.1) and Claiborne Lock and Dam (Section 3.2), respectively. Section 3.1 recommendations are for the TSP alignment at the time of the risk assessment (Figure 3). The team conducted a second PFMA for the Claiborne Lock and Dam for a different fish passage alignment alternative through the spillway (Figure 4) and are presented in Section 3.3. This alternative was evaluated since recommendations in Section 3.2 could substantially increase the cost of the TSP, making this second alternative more competitive. All elevations are stated in North Atlantic Vertical Datum of 1988 (NAVD88) unless otherwise specified.

### 3.1 Millers Ferry Lock and Dam

#### 3.1.1 Major Findings

1. At the time of the risk assessment, the TSP has selected a fish bypass around the right abutment; however, three alignments are still being considered and a final alignment has not been selected from the three tentative options.

2. There is evidence of an upstream to downstream erodible sandy gravel layer on the left abutment; however, there is no evidence of this layer on the right abutment where the fish passage will be excavated.
3. The headwater to tailwater differential is highest during low river stages and becomes less as flows increase in the river. The dam is designed to overtop during high river flows, but there is no history of the dam overtopping in its 53 years of service.
4. The project will be a passive design with no monitoring of flows during normal or flood stage operations.
5. A barge is required to install stop logs behind the existing tainter gates of the main dam spillway; however, barges will not be able to access the fish ladder channel.
6. There was evidence of head cutting covered with debris due to tailwater coming up and causing swirling eddies on the downstream right abutment.

3.1.2 Recommendations

1. Once the final alignment is selected, model the fish ladder canal with a debris jam near the upstream end of the fish ladder canal and verify where out of bank flow would go and what areas would be inundated from such an event. (PFM 07)
2. The operations manual should require inspection of the fish ladder channel after a high-water event to verify and/or remove debris from the channel. Revision of the EAP and Surveillance and Monitoring Plan should include inspections of the channel in addition to inspections of the dam. (PFM 07)
3. Require installation of a camera to allow for remote monitoring of possible blockage of sediment buildup in the fish ladder channel during regular use. (PFM 07)
4. If the alignment that goes around the west side of the dam is selected as the preferred alternative, the land mass formed between the right abutment and the fish ladder channel should be treated as a damming surface and protected from overtopping erosion as well as seepage related PFMs.
5. Perform hydraulic modeling to understand the velocities anticipated at sunny day and less frequent event flows. Special care should be taken with the turns in the channel alignment, as outside corners may experience much higher velocities and the freeboard may be less in these areas. (PFM 05)
6. Keep stop logs on site specifically for the fish passage gate and have access to a mobile crane capable of installing them in a timely manner. (PFM 05)
7. Investigate if a sand layer is continuous between the upstream pool and proposed fish ladder channel. If a continuous sand layer is observed, a filter should be incorporated at the exit. (PFM 06)
8. The access bridges must be designed to a sufficient level that allows trucks, mobile cranes, and other flood fighting equipment to access the dam. (PFM 13)
9. Preliminary design concepts show no filter around the embankment penetration and vertical concrete walls transverse to the dam. As design develops, use of

1H:10V battered walls on structure should be added and chamfered concrete to fill in any tight angles in concrete where transverse to the dam. A granular filter should be included on the downstream side of the structure. Also add a sill extending downward and out horizontally or a key on the downstream side tied into the structure to mitigate head cutting and reduce the likelihood for global instability of the gate structure. (PFM 05)

## 3.2 Claiborne Lock and Dam (Fish Ladder Around Right Abutment)

### 3.2.1 Major Findings

1. Unlike the alternative for Millers Ferry, the proposed fish ladder or Claiborne goes around the dam rather than through the dam. By adding a fish channel through the right abutment and weir approximately 1500-ft upstream of the existing dam structure, the island created between the channel and river as well as any additional tie-in to high ground from the northwest end of the weir also become part of the damming surface.
2. The headwater to tailwater differential is highest during low river stages. The dam is designed to overtop during high river flows and head differential approaches zero for events less frequent than an approximate 1/15 AEP and the dam overtops almost every year. Dam overtopping frequently occurs and typically includes flooding on the western side (right abutment) of the dam.
3. There is limited geotechnical information for this area or for the proposed fish ladder alignment.
4. The current conceptual design does not include a structure in the fish ladder bypass that incorporates stop logs or gates.
5. The project will be a passive design with no monitoring of flows during normal or flood stage operations, and no control structures to stop the flow through the fish ladder passage.
6. Based on historic construction records, an artesian condition is expected to exist in the foundation around El. -40 ft.

### 3.2.2 Recommendations

1. The dry island needs to be designed as a damming surface – overtopping, seepage, etc. (PFM 02)
    a. Scour over the island following an overtopping event could shortcut around the weir, allowing uncontrolled flow around the right abutment (PFM 02). This was a PFM not addressed by the current conceptual design and addressing this PFM could add significant cost to the project. To prevent this PFM from developing over the service life of the project, the team brainstormed measures that could be added to the island between the existing right abutment and the fish ladder entrance structure to prevent initiation or progression of this PFM:

   i. Design floodwall and cutoff wall to connect between the existing right abutment and entrance structure.
   ii. Armoring the island (riprap or concrete) to withstand overtopping between the existing right abutment and entrance structure.
   iii. Move the fish ladder canal further from the dam such that natural high ground separates the canal from the dam.

   b. BEP through the island is also identified as a PFM that is not addressed by the current conceptual design (PFM 05). Measures to address this PFM could also add significant cost to the project and should be incorporated into the feasibility plans to capture this effort. To address this PFM, the team should assume that a sand layer exists that may require potentially costly measures in the plan to address the PFM (i.e., cutoff wall or filtered exit) or complete a field exploration and laboratory program to verify what materials will be encountered in the channel excavation and confirm the erodibility of these materials.
   c. The spillway/entrance structure will be part of a damming surface. The design should include water stops at all monolith joints with 1H:10V battered concrete sidewalls. Downstream filters should also be considered to safely manage any seepage that could occur along the concrete discontinuity through the damming surface. (PFM 09)
2. Maintenance will be needed annually to remove debris or to make repairs from high flows or recreational damage. Recommend the channel entrance structure design incorporate a method to stop flow, such as gate recesses or stop logs in the weir. (PFM 03)
3. Currently there are no soil borings in the alignment and there is very limited geotechnical data for the Claiborne Dam. Perform additional geotechnical investigations as necessary to inform design and cost.
4. Require inspection after a high-water event to verify and/or remove debris from the channel. Revision of the EAP and Surveillance and Monitoring Plan should include inspections of the channel in addition to inspections of the dam. Recommend CCTV camera be added to existing monitoring system to inspect the channel for erosion of blockage during/after highwater events. (PFM 03)
5. Flow modeling should incorporate tailwater elevations in flood stage(s) to better understand how the fish ladder channel or out of bank flow interacts with the tailwater. (PFM 03)
6. The project will be a passive design with no monitoring of flows during normal or flood stage operations. Recommend CCTV camera be added to existing monitoring system to inspect the channel for erosion or blockage during/after highwater events. (PFM 03)
7. Recommend stability analyses include a transient case for drawdown of approximately 3.5 ft per day for 10 days to assess historic conditions. (PFM 04)

8. Additional flow modeling is recommended to identify and address any locations which may need armor where the channel and river merge and eddies are likely to occur downstream. (PFM 07)

## 3.3 Claiborne Lock and Dam (Fish Ladder Over Existing Spillway)

### 3.3.1 Major Findings

1. Construction of the fish ladder over the existing spillway could reduce spillway capacity, increasing overtopping frequency of the existing project; however, the dam currently overtops annually so this will not increase the likelihood of failure of the dam.
2. Construction of the fish ladder over the spillway would require approximately 40 feet of fill between bedrock and the concrete fish ladder chute. High magnitudes of settlement could occur if the fill material is not properly moisture controlled and compacted or if the concrete structure does not extend and is founded on bedrock.
3. Current conceptual design likely underestimates material quantities necessary for armoring against all sources of scour erosion that could undermine the fish ladder structure.
4. Adding fill material and the resulting stress increases acting on the existing right abutment walls could lead to additional cracking and differential movements.
5. There are badly jointed and broken zones of the fixed crest spillway's foundation that are likely to produce a lot of water during construction and may need tremie mats to be installed during construction to control seepage for dewatering purposes.
6. Not enough conceptual design information was developed to inform feasibility study level cost estimate without high contingencies and uncertainty.

### 3.3.2 Recommendations

1. PFMA finds high potential for loss of service of fish passage due to several failure modes (e.g., settlement, debris, etc.) and the team should evaluate the need for deep foundation to support the structure on bedrock as well as armoring protection of side slopes.
2. The PDT should review the cofferdam assumptions in the current cost estimate. It is expected that large combiwall (H-pile/sheet pile combination) or cellular type cofferdams would be needed to allow construction in the dry.
3. Better layout of construction sequence and cofferdam elevations is needed to safely build the new fish passage.
4. Assumptions for construction duration should be reviewed considering the frequency and duration of overtopping at the Claiborne dam. The amount of time the project remains inundated throughout the year may be significant and may reduce the allowable construction duration in the dry.

5. All alternatives considered at Claiborne Lock and Dam should include stoplogs/bulkheads or gates to allow for routine maintenance once placed into operation.
6. 3-Dimensional CFD modeling is recommended during preconstruction engineering design (PED) phase to best determine scour potential and stone sizing (PFM 02 and PFM 03).
7. Team should avoid placing additional load on the right abutment wall without a better understanding of why the abutment sections W2 and W3 are cracking.
8. Construction sequencing should be concurrent on the upstream and downstream side of the existing spillway to avoid an unbalanced load or a new load case for the existing spillway.

## 3.4 All Alternatives

The following major findings and recommendations apply to all dams and alignments reviewed for this study:

### 3.4.1 Major Findings and recommendations

1. Paddlefish (one of the species being targeted by the fish bypass) can sense and are averse to metals, including rebar in concrete reinforced structures and metals in gate structures, instinctively causing them to avoid these areas. Design engineers may need to be creative in material selection of the bypass.

## 4. Background

This report is intended to be an attachment to the feasibility study; therefore, this section only summarized background information.

Millers Ferry Lock and Dam (L&D) and Claiborne L&D are classified as significant hazard navigational projects located on the Alabama River, 178.0 and 117.5 river miles, respectively, above the Bankhead Tunnel in Mobile, Alabama. The Mobile District is in the process of identifying various fish ladder alignments that would best fit the Millers Ferry and Claiborne L&D projects. The project objective is to establish a fish passage (fish ladder) around the dams. Environmental considerations are an important aspect for the fish ladder projects. The Alabama sturgeon and Gulf Sturgeon are some of the drivers for the project as these species are listed as threatened or endangered; however, the fish ladder channel would also benefit additional species of fish as can be seen in Figure 5. It has been recommended by the U.S Fish and Wildlife Services that a fish passage would help preserve the remaining fish populations in the river.

The upper pool of Millers Ferry L&D extends 103 river miles creating the William Dannelly Lake. The project is approximately 10 miles northwest of Camden and 30 miles southwest of Selma. The Claiborne L&D and approximately 28 miles of the lower part of the reservoir lie entirely within Monroe County, except for two small reaches totaling approximately one mile are in Clarke County. The remaining 32 miles of reservoir are in Wilcox County.

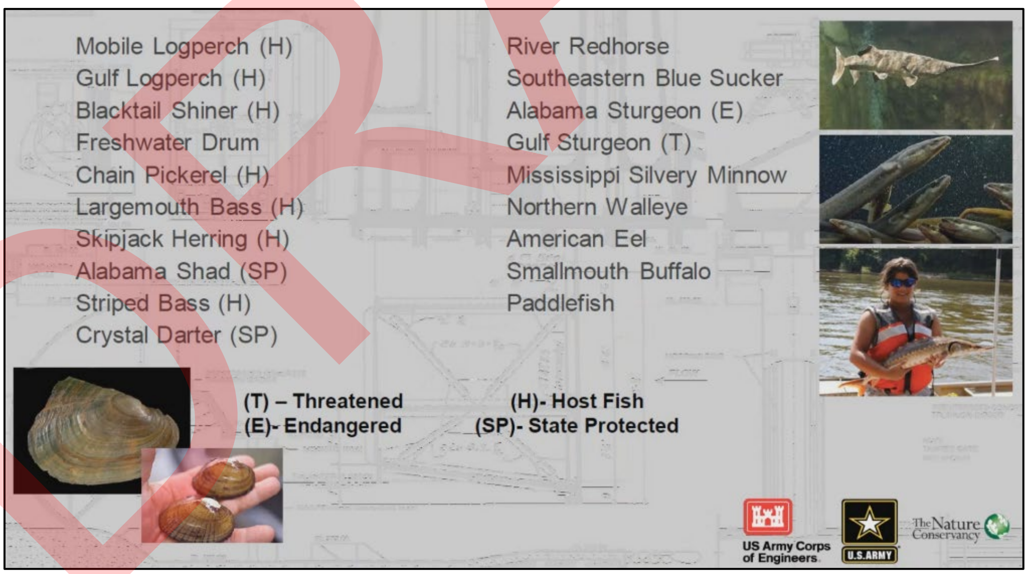


Figure 5 – Target species to benefit from proposed fish ladders.

The following sections summarize background information for the Millers Ferry L&D and Claiborne L&D projects that informed the PFMA and SLRA.

## 4.1 Millers Ferry L&D

### 4.1.1 Location and Purpose

The Millers Ferry L&D project was authorized by the River and Harbor Act of 2 March 1945, House Document 414, 77th Congress, 1st session. Construction began in April 1963 and was completed in 1969 except for recreational facilities. The authorized project purpose is to maintain a navigable channel of 9 feet (ft) deep x 200 ft wide. The project also supports hydroelectric power.

The Millers Ferry L&D is 142 rivers miles above the mouth of the Alabama River and located downstream of R.F. Henry L&D and upstream of Claiborne L&D. The hydropower generation is located on the left bank of the river with a gated spillway along the main channel and an overtopping section on the right bank shown in Figure 2. Millers Ferry L&D is current a DSAC 5, Significant Hazard Dam. No observed overtopping of the dam has observed or recorded to date.

### 4.2.2 Project Description and Pertinent Data

Figure 6 shows the proposed alignments of the fish ladder channel for Millers Ferry L&D. Three alignments were still being considered at the time of the SLRA. The team evaluated the channel shaded white since it results in excavation closest to the existing dam and was therefore judged the highest risk alignment. All three alignments penetrate the existing embankment.


Figure 6 – Millers Ferry L&D fish ladder channel alignments being considered.

The fish ladder will consist of a series of stepped pools formed by boulders and concrete, each rising one foot in elevation above the previous. Each step is proposed to be about 200 feet in length to accommodate the larger fish species and the channel will have 3H:1V side slopes and a 100-ft bottom width. The target slope of the fish ladder is less than 2% to allow velocities to remain below 5 to 6 feet per second (fps) to allow for upstream migration of the fish species.  Figure 7 shows a conceptual illustration of stepped pools to allow fish passage.

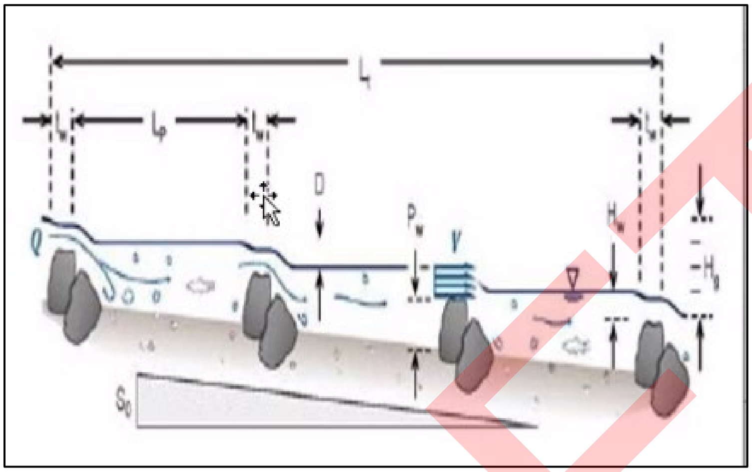


Figure 7 – Conceptual drawing of stepped pools to allow fish passage.

The following Figure 8 shows a topographic map of the dam.  This image shows the location of 2 cross sections cut through the dam and through the fish passage channel. Cross section 1 also cuts through an existing wetland on the downstream toe of the dam.  This wetland is a natural low area that is as deep as 15 feet below the surrounding ground surface.  This area remains wet all year and overflows back into the river after significant local rainfall events.  This figure also shows a photograph of the wetland area.

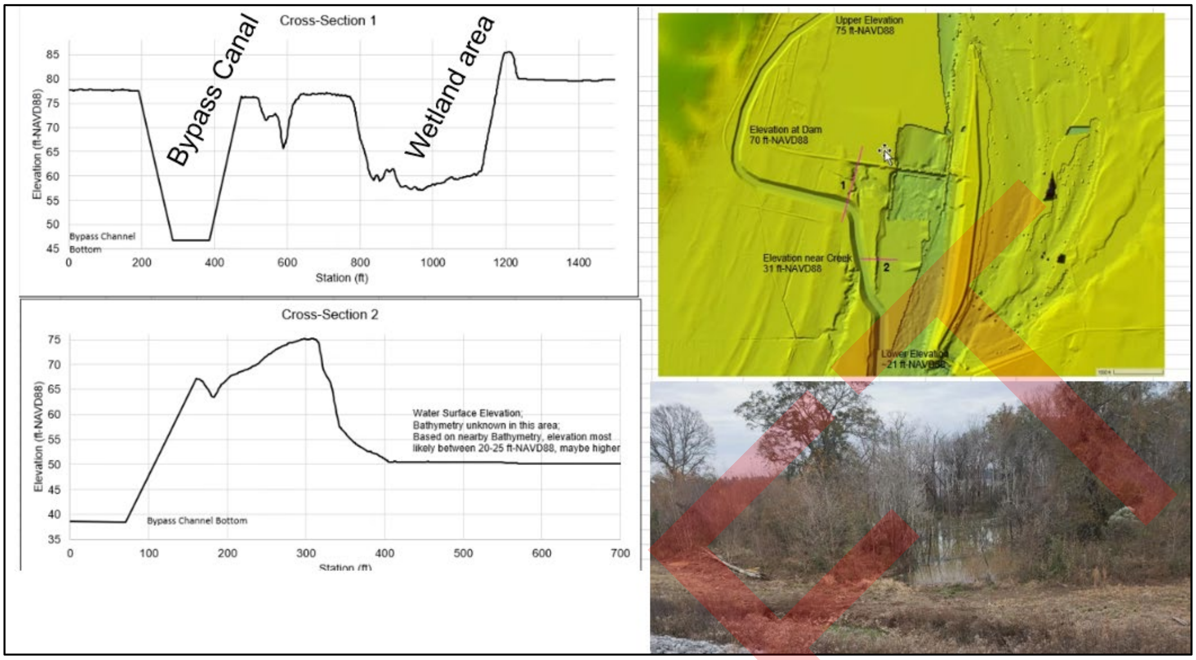



Figure 8 – Cross section through conceptual fish bypass canal and existing wetland area.

Figure 9 below shows the conceptual gate design for the fish passage. The gate will be located in the damming surface near the right abutment. The gate will consist of 4 gates and a bridge that maintains vehicular access to the dam and right bank of the lock structure. Detailed design does not exist for this structure and the conceptual design is for developing cost estimates only. The gate materials are also still unknown. Typical structures would be of reinforced concrete and aluminum or steel gates; however, there is some concern that the paddlefish will not enter through the structure due to heightened electromatic senses and gate materials may be required to be of alternative materials.

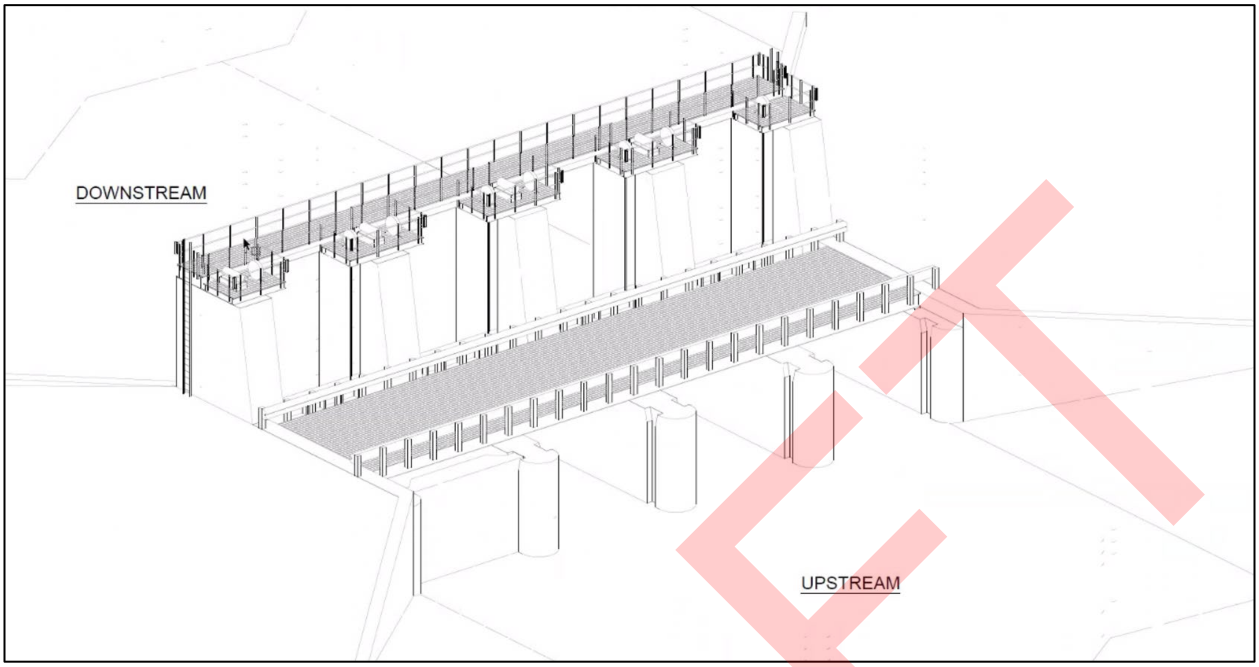



Figure 9 – Conceptual gate design for fish passage.

Table 2 provides pertinent project data for the existing Millers Ferry L&D.

Table 2 - Millers Ferry L&D Pertinent Data.

| **Spillway** | |
|---|---|
| Type | Non-navigable gated concrete |
| Total Length, included end piers | 994 ft |
| Pier dimensions (above sill) | 8-ft wide x 65-ft long x 99-ft tall |
| Number of Tainter gates | 17 |
| Top of pier elevation | EL 113.7 |
| Tainter gate sill elevation (base of pier) | EL 46.2 |
| Closed Tainter gate, top of gate | EL 81.2 |
| Raised Tainter gate low steel elevation | EL 99.0 |
| Tainter gate size | 35-ft tall x 50-ft long |
| Foundation elevations | EL 17.2 to 25.2 |

| **Locks** | |
|---|---|
| Width | 84 ft |
| Chamber length | 600 ft |
| Maximum Lift | 48.8 ft |
| Top of Wall elevation | EL 87.2 |
| Upper sill elevation | EL 61.2 |
| Lower sill elevation | EL 19.2 |
| Upper Emergency gate sill | EL 64.2 |
| Lower emergency gate sill | EL 19.2 |
| Lock filling/emptying time (main & aux): | 9:10 / 12:14 (minutes: seconds) |
| Miter gate size | 22.3-ft tall x 24-ft long |
| Type of emergency dam | Stop logs |
| Type of filling and emptying system | Longitudinal floor culverts |

| **Right Overflow Dike** | |
|---|---|
| Type | Earthfill |
| Length | 3,360 ft |
| Top elevation | EL 85.2 |
| Side slopes | 1 on 2.5 |
| Crest Width | 25 ft |
| Thickness of grouted riprap on slopes | 18 to 24 inches |

| **Left Bank (including Middle Lock Mound)** | |
|---|---|
| Type | Earthfill |
| Length | 5,500 |
| Top elevation | EL 97.2 |
| Side slopes upstream | 1 on 2.5 |
| Side slopes downstream | 1 on 3.0 |
| Crest Width | 32 ft |
| Thickness of grouted riprap on slopes | 24 inches |

4.2.3 History

Millers Ferry L&D was authorized for navigation, hydropower, and recreation benefits. The reservoir upstream is operated for navigation and power generation. The normal pool provides a 9-ft-deep navigation channel to the junction of the Coosa and Tallapoosa Rivers. No flood control is provided as flood risk management was not an authorized purpose for this project. For smaller floods where the pool is held constant, the peak discharge in some cases is slightly higher than natural; however, the effect on the downstream conditions is estimated to be minor. The project completed the first phase of the development of the Alabama-Coosa River to Rome, Georgia.

The project was completed through four award contracts. The third contract was awarded October 1964 for construction of the right overflow dike, right abutment, gated spillway, lock, lock mound, and center non-overflow embankment. Additionally, this contract included some excavation for the upstream lock approach channel, part of the downstream lock approach channel, and some of the powerhouse diversion channel. A total of 2,369,240 cubic yards of material was removed. The channel material was removed by cranes equipped with drag buckets. Two stages of construction were used to place the concrete structure of the lock and spillway. The first stage included construction of the lock, the control station, and a portion of the gated spillway. During this first stage, dewatering of the Ripley formation occurred. Inspection of the borings from the relief wells and observation wells revealed a water bearing bedding plane in the Prairie Bluff limestone. Following this discovery, filter material was placed 6 inches above this fracture in all observations and relief wells. The second stage consisted of construction of the remainder of the spillway. Special care was taken to maintain the integrity of the Prairie Bluff limestone. A minimum of 6 inches of protective covering was placed over foundation material and contractors were required to place concrete on foundation surfaces within 18 hours of exposure. Foundation prep was slowed by excavating corrugations in the Prairie Bluff limestone. Contractors used quilted curing mats to protect the Prairie Bluff formation immediately upon removal of protective cover and excavation of the corrugations. Early during excavation of the lock corrugations, it was found that ridges between corrugations sheared near the bottom offering no resistance to sliding. The result was the loss of approximately 1 foot of critically limited foundation material. To combat this loss the corrugation, configuration and spacing was changed to four on 12-foot centers instead of the required eight in the gated spillway and abutment wall monolith foundations. Final cleanup was slow and expensive. All foundation material was scraped with altered point of a long handle shovel. Once the scraped material was dried to extent as it would not adhere to the rock, it was then blown off with air jets. All foundations were approved by the resident geologist or the assistant geologist prior to placement of the concrete. This contract was completed in June 1969.

The project was fully operational in 1970. The projects' regulated peak outflow is 271,500 cubic feet per second (cfs). The spillway design flood series peak inflow and regulated peak outflow is 840,000 and 814,200 cfs respectively. Elevations for pertinent features of the lock, spillway, embankments, and powerhouse are within Table 2 of this chapter. During normal conditions, all releases are made through the turbine units. During high flows, the spillway gate settings are operated according to the powerhouse operator. The generating units are shut down when the operated head decreases to approximately 14 ft. During normal flow periods, no special water control procedures are required for navigation at Millers Ferry. The normal maximum drawdown is at El. 78.16 ft which provides 13 ft of clearance over the upper lock sill.

*Operation of Lock:* During high flow periods, navigation is discontinued when tailwater reaches El. 81.16 ft, leaving 1.0 foot of freeboard over the lower guide wall. The discharge at El. 81.16 is approximately 220,000 cfs. The lock is not subject to flooding over the lock chamber.

*Operation of Spillway Gates:* The gates are not operated until the inflow exceeds the capacity of the powerhouse (33,600 cfs). The gates are opened in order and increments of openings depend on the inflow and pool elevations. The gate opening schedule has with and without powerhouse operating. Per the gate opening schedule, Gate 1 is the first to open and each gate is opened by 1-foot increments. The gates are numbered in sequence beginning at the left bank or east end of the spillway, adjacent to the lock. When the spillway capacity is reached, a free overflow condition prevails and there is little difference between the water surface upstream and downstream. The river may continue to rise just as it would in the absence of any structure. There is no flood risk management storage in the Millers Ferry project.

#### 4.2.4 Geology and Site Characterization

Millers Ferry L&D project is in the East Gulf Coastal Plain physiographic section. This section consists of Mesozoic and Cenozoic sediments in southwestern Alabama. Near the project area, the topography is characterized by rolling hills and prairie land. The river flows south in wide meanders cutting Tertiary and Cretaceous age rocks. The surficial geology near the project is alluvial deposits. Locally these deposits overlie the Clayton, Prairie Bluff, and Ripley formations.

In the spillway area, the overburden varied in thickness from 35 ft near the abutments to less than 10 ft in the middle of the river. The river overburden consisted of a thin layer of sand and gravel shown in Figure 10. This overburden was excavated to the Prairie Bluff limestone where the foundation of the spillway was placed. Figure 11, Figure 12, and Figure 13 show the geologic cross-sections at B-B, A-A, and D-D respectively.

The right overflow dike has a crest width of 25 ft with 1:2.5 (horizontal:vertical) slopes on the upstream and downstream side, shown in the geologic cross section A-A. The material for the dike was obtained from the upstream borrow site. The bank is protected by 18 inches of grouted riprap on 6 inches of bedding material from El. 85.2 ft to 79.2 ft on the upstream and downstream slopes. From El. 79.2 ft to the toe of the slope on the upstream and downstream face has 24 inches of riprap. Bedding material for the dumped riprap consists of a filter composed of two layers; 4-inch inner layer of sand and gravel and a 5-inch outer layer of gravel. The bedding layer for grouted riprap is a 6-inch layer of sand.

The sections on the right and left side of the river overlie a terraced area. The ground surface elevations varied between 60 ft to 85 ft. The soils on the right bank are principally inorganic silts, lean clays, fat clays, clayey sands, and silty sands. Overburden overlays the gray firm siltstone generally encountered at depths of 15 ft to 55 ft below the existing ground surface. The surface material on the right bank was generally an inorganic silt varying in depth from 29 ft at the river to 3 to 8 ft towards the natural embankment. Below the surface material, interbedded strata of lean clay and inorganic silts, silty sands, and gravel-sand-silt mixture predominate. The borrow area for the dikes was along the right side of the river. The borrow material ranged from lean clays to silty fat clays. The borrow area material was used for construction of the right overflow dike.

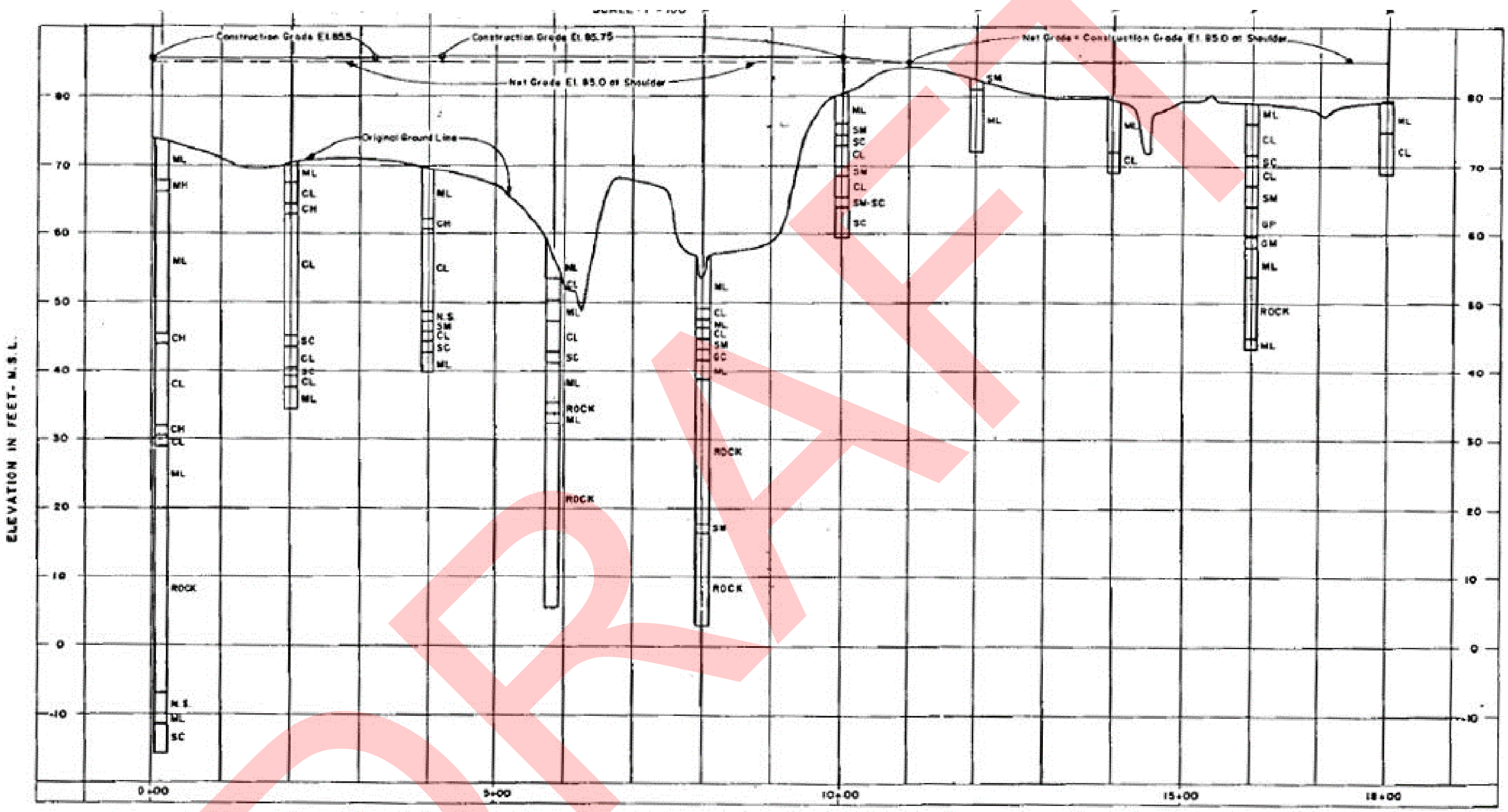



Figure 10 – Borings from Millers Ferry L&D.

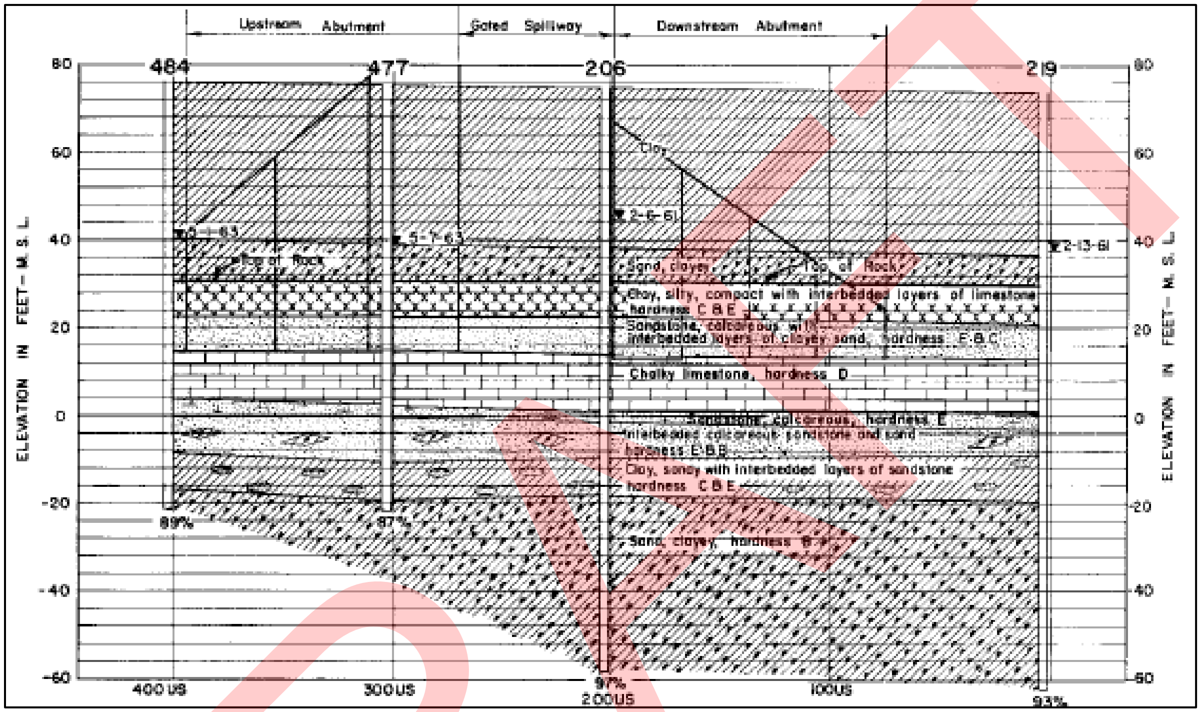



Figure 11 – Geologic cross-section B-B at the right overflow dike.

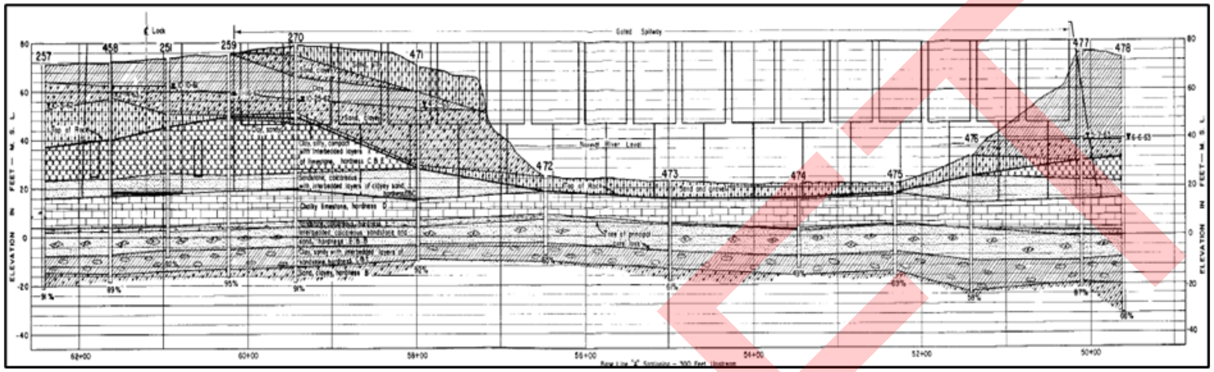


Figure 12 – Geologic cross-section A-A, facing downstream.

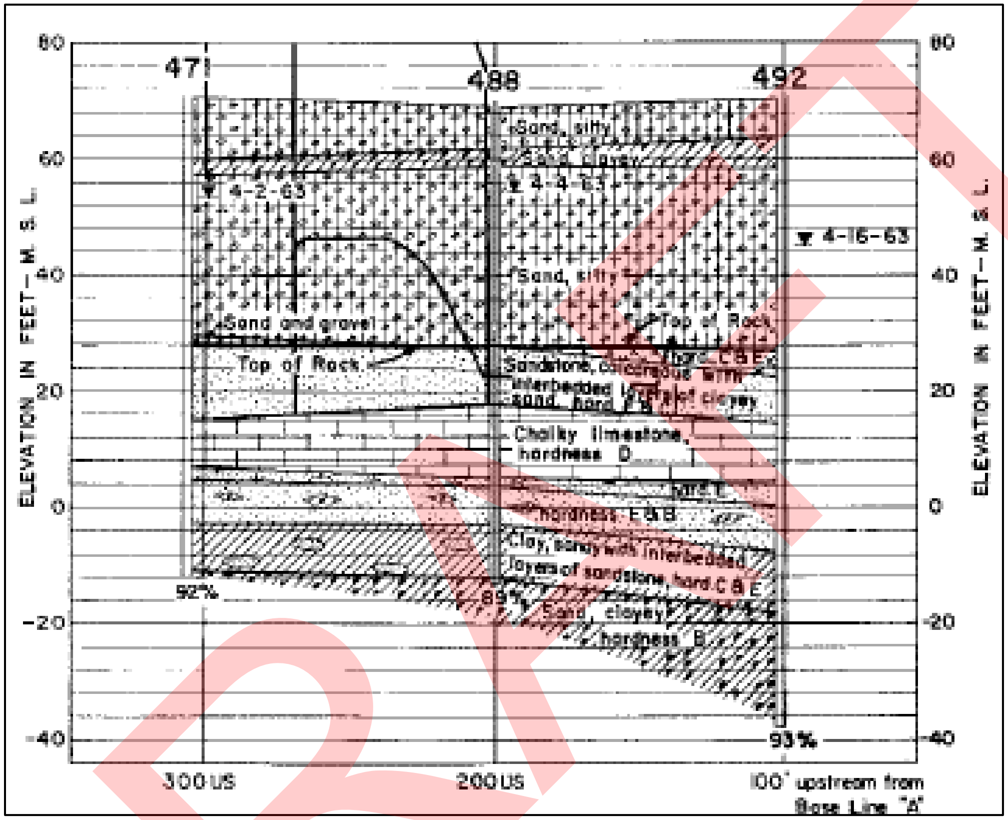



Figure 13 – Geologic cross-section D-D.

## 4.2 Claiborne L&D

### 4.2.1 Location and Purpose

The original Claiborne L&D project was authorized by Congress on 18 June 1878 to provide for a navigational channel 4 ft deep and 200 ft wide from the mouth of the Alabama River to Wetumpka. The project was modified on 13 July 1892 to provide a 6 ft deep channel. Subsequent acts approved in 1905 and 1910 provided for a channel 4-ft deep channel at low water for the use of contracting dikes and dredging. This project was 62 percent complete in 1942, the last year that any new work was performed. In March of 1945, the River and Harbor Act authorized a 9 ft deep navigation channel. The House Document recommended the authorization of a general plan for the basin “in accordance with plans being prepared by the Chief of Engineers.” The basin plan at that time contemplated a 9-ft-deep navigable channel from the mouth of the Alabama River to Rome, Georgia, to be achieved by open river works and locks and dams. P.L. 92-500 provides for Water Quality operation (here, low flow augmentation) and P.L. 78-534 provides for recreation. Water is not managed at the project for recreational purposes.

The first design memorandum for Claiborne, "General Design No. 1", was submitted on 12 April 1963. This report proposed the Claiborne plan to include a navigation lock, gated spillway, fixed spillway, control station, and earth dikes on both banks. Seven other design memoranda dealing with project features were submitted during the next four years.

The Claiborne L&D is primarily a navigation structure. The project's minimum reservoir level at El. 32.1 ft provides navigation depths up to the Millers Ferry L&D. The Claiborne Project also reregulates the peaking power releases from the upstream Millers Ferry Project providing navigable depths in the channel below Claiborne. Other purposes provided by the project include water quality, public recreation, and fish and wildlife conservation and mitigation. Recreation facilities and access to the reservoir are provided, but because of the nature of the project, recreation is typically not considered in water control decisions. There is no flood risk management storage for this project.

### 4.2.2 Project Description and Pertinent Data

Figure 14 shows the proposed alignment of the Claiborne L&D fish ladder channel with Table 3 providing pertinent project data. The bypass will have a 75-foot bottom width with 3H:1V side slopes. Cofferdams upstream and downstream are proposed during construction. Figure 15 presents the typical cross section for the bypass channel. Like Millers Ferry discussed above, the bypass channel will include one-foot stepped pools to allow the fish to progress upwards in elevation around the dam. The slope will be like Millers Ferry at less than 2%. Lastly, Figure 16 shows LiDAR of the area with the proposed fish passage added on the right abutment.

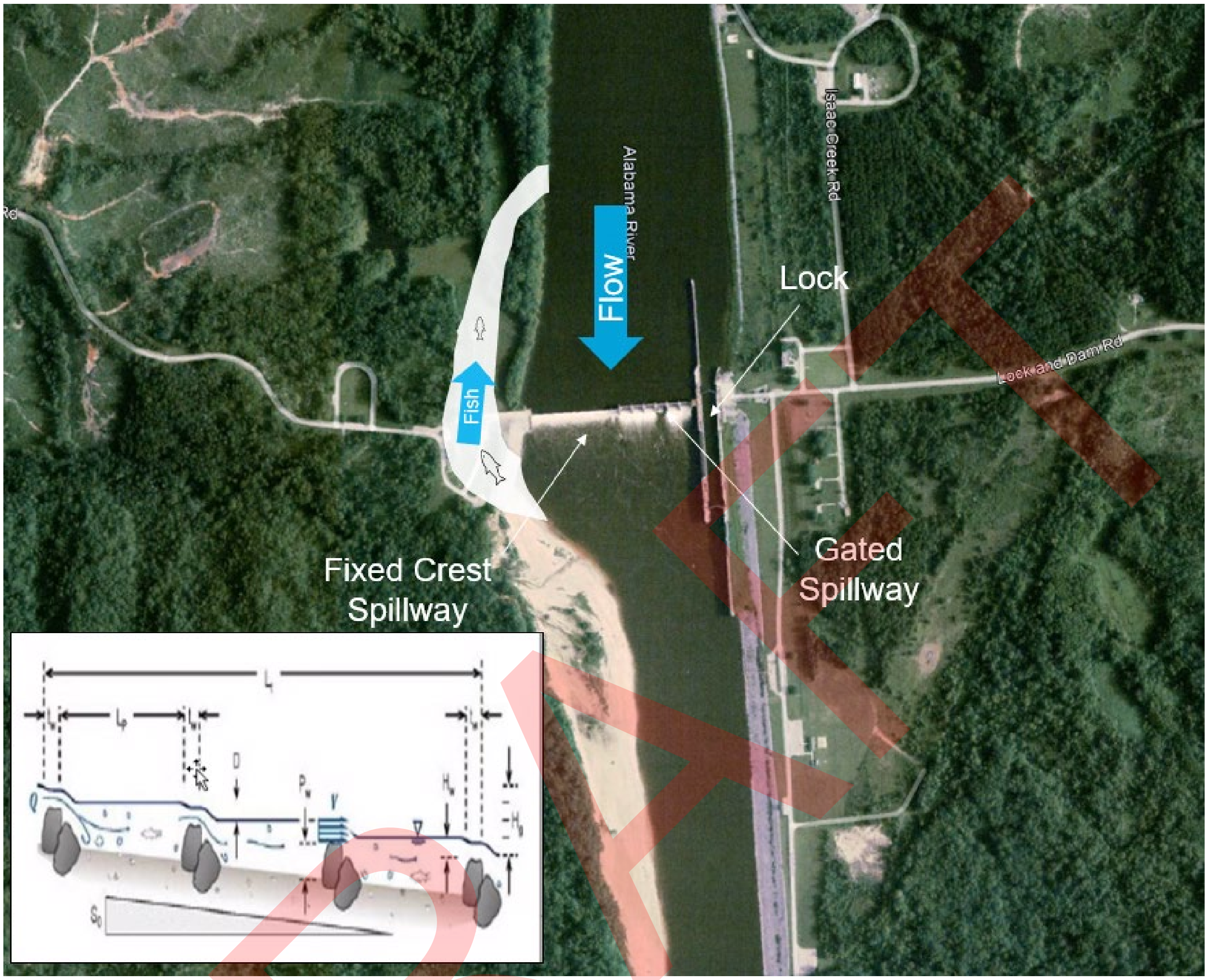



Figure 14 – Claiborne L&D proposed fish ladder channel alignment.

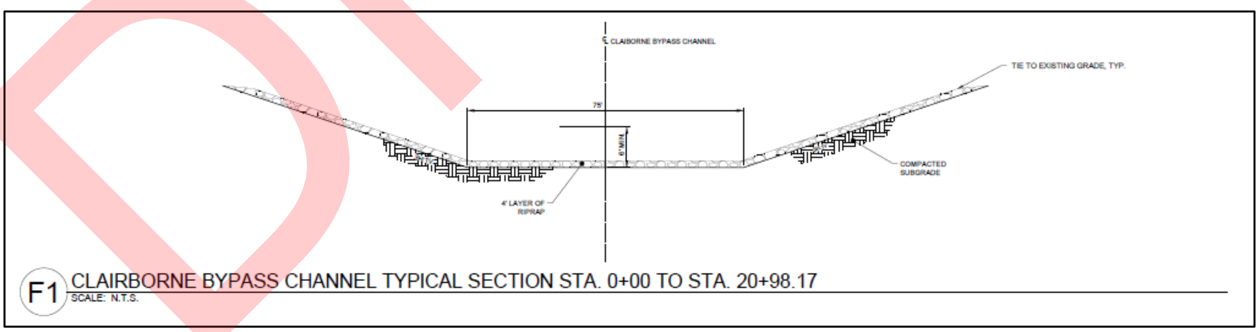



Figure 15– Typical section for the Claiborne bypass channel

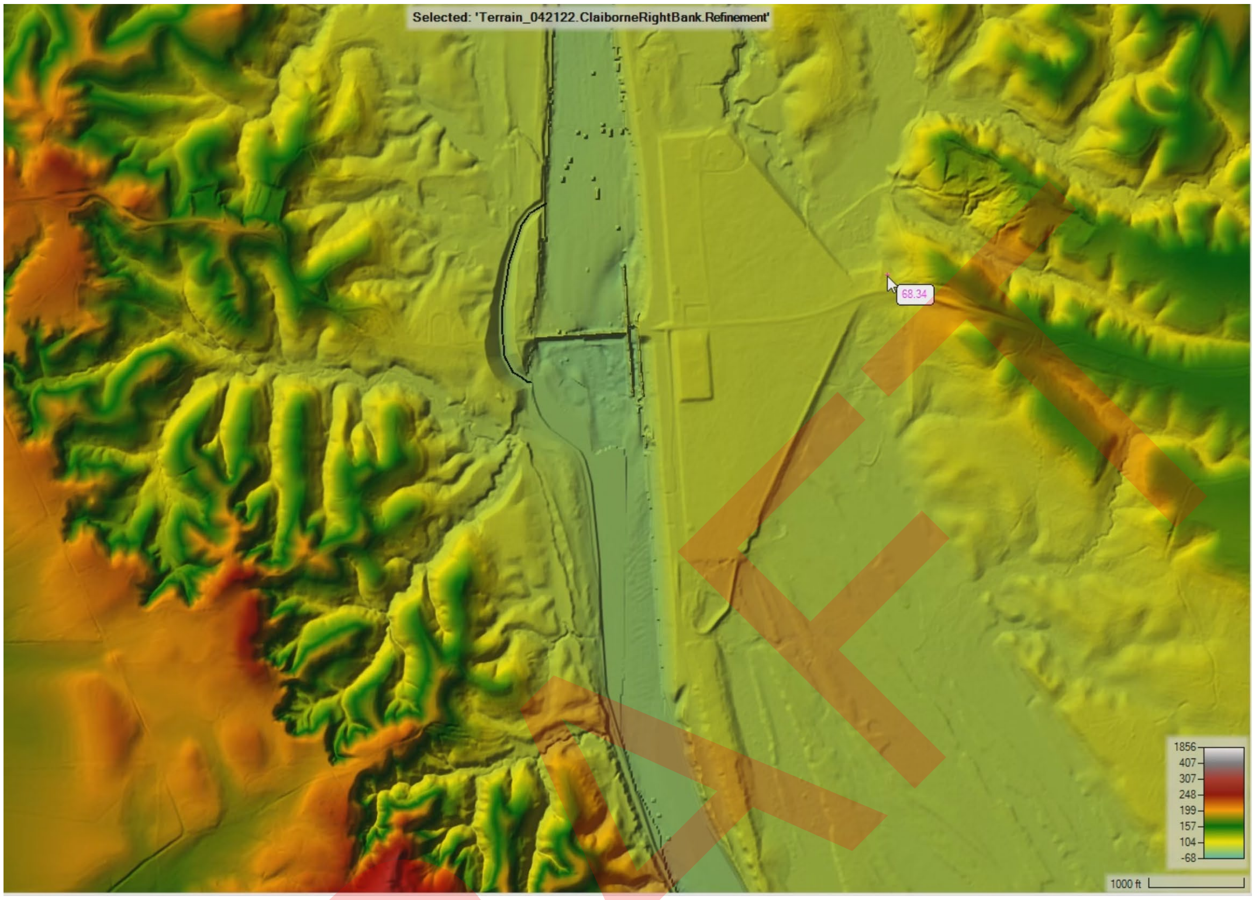



Figure 16 – LiDAR image of Claiborne with proposed fish bypass added on right abutment.

Table 3 - Claiborne L&D Pertinent Data.

| **Gated Spillway** | |
|---|---|
| Type | Non-navigable gated concrete |
| Total Length, included end piers | 416 ft |
| Elevation of crest | El. 15.1 |
| Number of piers, including end piers | 7 |
| Width of piers | 8ft |
| Type of gates | Tainter |
| Number of gates | 6 |
| Length of gates | 60.0 ft |
| Height of gates | 21.0 ft |
| Maximum discharge capacity | 67,111 cfs |
| Elevation of top of gates in closed position | 36.1 ft |
| Elevation of top of gates in open position | 66.1 ft |
| Elevation of trunnion | 48.1 ft |
| Elevation of access bridge | 75.1 ft |
| Elevation stilling basin apron | El. 14.6-17.1 ft |
| Height of end sill | 5.0 ft |

| **Locks** | |
|---|---|
| Width | 84 ft |
| Chamber length | 600 ft |
| Maximum Lift | 48.8 ft |
| Top of Wall elevation | EL 87.2 |
| Upper sill elevation | EL 61.2 |
| Lower sill elevation | EL 19.2 |
| Upper Emergency gate sill | EL 64.2 |
| Lower emergency gate sill | EL 19.2 |
| Lock filling/emptying time (main & aux): | 9:10 / 12:14 (minutes: seconds) |
| Miter gate size | 22.3-ft tall x 24-ft long |
| Type of emergency dam | Stop logs |
| Type of filling and emptying system | Longitudinal floor culverts |

| **Earth Dike** | |
|---|---|
| Type | Earthfill |
| Length | 200 ft |
| Top elevation | EL 40.1 |
| Top width | 25.0 ft |
| Side slopes | 1 on 3 |
| Crest Width | 25 ft |
| Thickness of grouted riprap on slopes | 18 to 24 inches |

| **Left Bank Dike** | |
|---|---|
| Type | Earthfill |
| Length | 2,350 ft |
| Top elevation | EL 60.1 |
| Top width | 32.0 ft |
| Side slopes | 1 on 4 |

#### 4.2.3 History

Claiborne Lock and Dam officially began operation on 15 November 1969. The inflow design flood (IDF) at the project is 814,800 cfs (estimated <1/1000 AEP) and the standard project flood (SPF) is 494,500 cfs (estimated <1/500 AEP). The maximum regulated spillway capacity (for both gated and fixed-crest spillways) is 70,763 cfs. The project's gated spillway crest elevation is 15.0 ft. The crest of the fixed spillway is at 33 ft and is drowned by flows in excess of 57,000 cfs. The right dike at the project is constructed to El. 40.0 ft, providing a 4-ft freeboard above the maximum operating pool of El. 36.0 ft. The right dike is an overflow feature where the structure is expected to overtop, on average, twice a year. The left dike of Claiborne L&D is a non-overflow feature constructed at El. 60.0 ft, except where it slopes down to the lock esplanade (El. 51.0 ft.) at its westernmost extent.

It is anticipated the lock walls will be overtopped by water on an average of once every 3.1 years, for an average period of 3 days. In general, it will be known some 6 to 10 hours prior to the lock flooding that the lock walls will be overtopped with water. As soon as it is known or anticipated the lock walls will be overtopped with water, the lock is prepared for flood condition. The peak design flood of record at Claiborne Lock is El. 58.9 ft which is 7.9 ft above the top of the lock walls. It is estimated the flood elevation of 58.9 ft has a recurrence interval of 62 years. In preparing the lock for flooding, all tools (e.g., oil barrels, etc.) on the first floor in the Lock Control Station and Lock Walls, are carried to higher ground and all electric power to the lock is shut down.

Claiborne L&D has not had any incidents or failures at the project since construction; however, historically there has been erosion on the right streambank downstream of the overflow dike. Figure 17 and Figure 18 are pictures taken in April 1977 and included in Periodic Inspection (PI) No. 6 in 1978 that show the bank failure caused by the erosion. In 1977, a project was designed to remediate the erosion. A combination of riprap, grouted riprap, and bedding material was used to fill in the erosion areas shown in Figure 19. Typically, plastic filter cloth was used beneath the bedding material for areas that did not contain grouted riprap.

During the last PI, the elevated tailwater made it impossible to fully inspect the grouted riprap voids. Previous inspections identified voids/differential movement beneath the grouted portion of the riprap created by some erosion of the underlying fill; however, the water was too high to view during that inspection to evaluate their condition. There had been concerns about embankment material being piped from beneath the concrete overlaying the crest. In 2014, the Mobile District cored several holes into the concrete cap to determine if the feature had been undermined but no voids were detected. Figure 20 shows more current 2023 picture of the differential movement between grounded and ungrouted riprap.


Figure 17 – Right bank erosion, facing upstream towards the fixed crest spillway 1977.

Figure 18 – Right bank erosion area, facing downstream 1977.


Figure 19 – Designed Overtopping flows of the dam with riprap armor.

Figure 20 – Differential movement between grouted and ungrouted riprap (PDT photograph from 2023).

### 4.2.4 Geology and Site Characterization

Claiborne L&D site is in the Southern Red Hill Division of the Gulf Coastal Plain Physiographic Province. The Alabama River meanders gently through this region and has a broad flood plain characterized by gently rolling hills and north facing cuestas developed on harder beds of sedimentary strata. Sediments in the reservoir area consist of typical coastal plain deposits of variably interbedded limestones, clays, sands, and sandstones of Paleocene and Eocene age shown in Figure 21 and Figure 22. Regionally the coastal plain sediments in Alabama dip gently to the south and southwest with only local variations.

All structures at the project site are founded on claystone which in the area is greenish gray, waxy, compact, impervious material that is 20 to 25 ft thick. Claystone is unusually light in weight due to microscopic pore spaces and contains small, interspersed pockets of friable sandstone and sand that are typically a fraction of an inch thick. Throughout much of the lock area, an interbedded sand and stone occurs directly beneath the alluvial overburden and constitutes the top of rock.

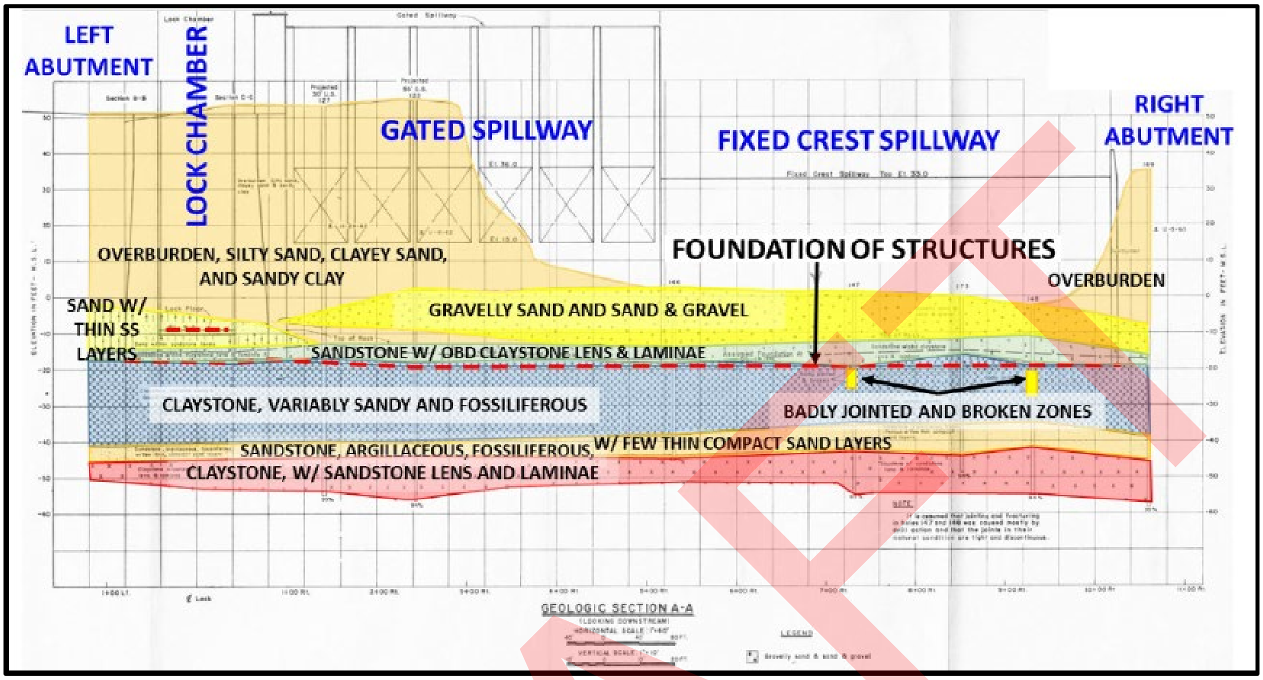



Figure 21 – Geologic Cross-Section A-A along axis showing overburden soils and weak rock above foundation grade (red dashed line).

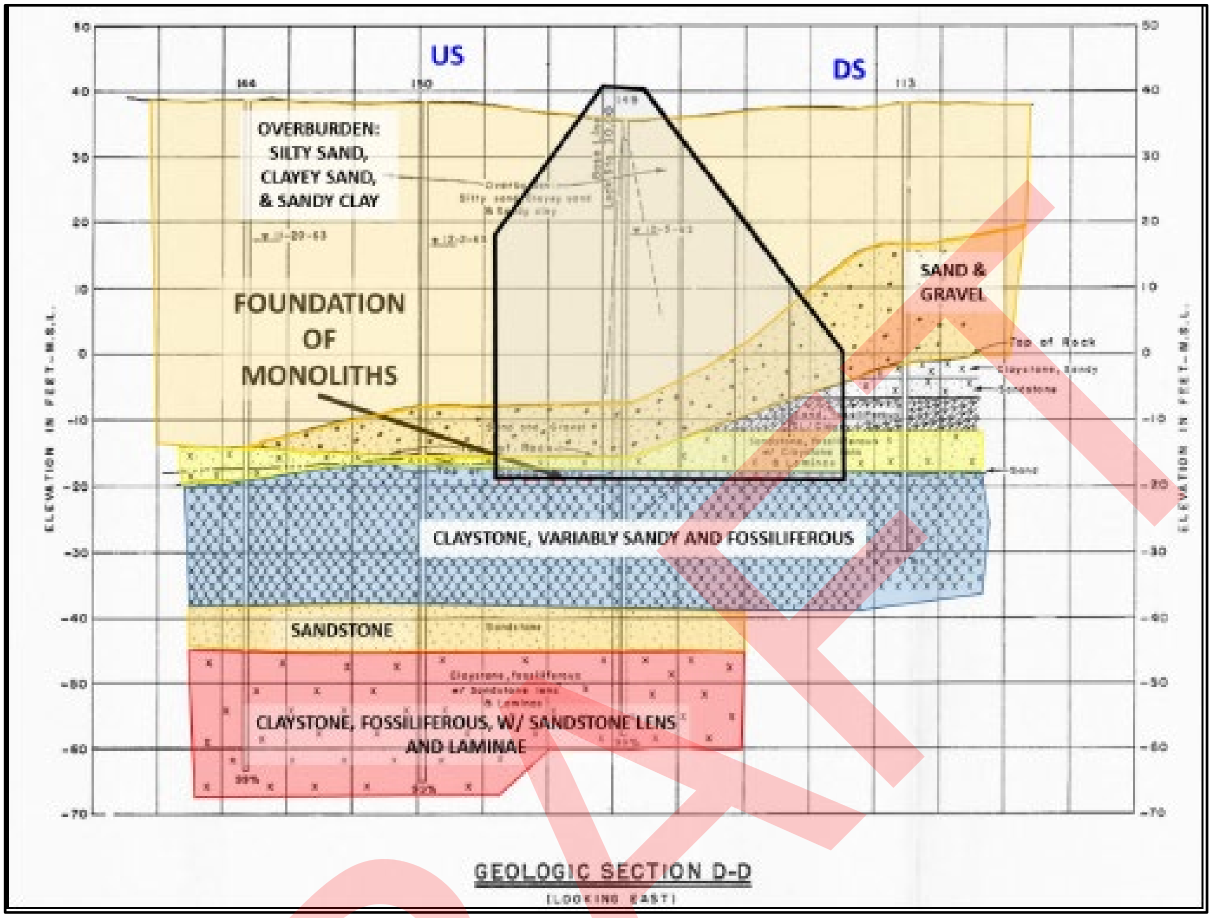



Figure 22 – Geologic cross-section D-D through the fixed crest spillway.

Badly broken and jointed zones up to 5 ft thick were encountered in many of the borings and moderate jointing was encountered in all borings. It is believed that all the joints are tight in their natural condition, although many of them were logged as being “open” due to core breakage.

Although there are preconstruction borings at the dam site from the original design, there are not field exploration test that have been performed in the alignment of the fish ladder bypass channel.

# 5. Hydraulic Loading

The Alabama-Coosa-Tallapoosa (ACT) River Basin is made up of the Coosa, Tallapoosa and Alabama Rivers, and their tributaries and drains northeastern and east-central Alabama, northwestern Georgia, and a small portion of Tennessee. The ACT River Basin drains 22,800 square miles, of which 17,300 square miles are in Alabama. The Alabama River flows through central and southwestern geographic regions of Alabama, originating at the confluence of the Coosa and Tallapoosa Rivers in Montgomery, Alabama, and flowing southwesterly 315 miles to its confluence with the Tombigbee River. Below this junction, the Mobile River conveys flows to the Mobile-Tensaw River Delta, and eventually to Mobile Bay. There are a total of three projects on the Alabama River; primary purposes include support of hydropower generation and provision of navigable depths of 9-ft. from the Port of Mobile, Alabama to Montgomery. The datum referenced in this chapter is the North American Vertical Datum of 1988 (NAVD88). The original project datums of reference are the National Geodetic Vertical Datum of 1929 (NGVD29). The conversion from NGVD29 to NAVD88 for the Claiborne Lock and Dam is +0.11 feet. The conversion from NGVD29 to NAVD88 for the Millers Ferry Lock and Dam is +0.16 feet.

## 5.1 Millers Ferry L&D

### 5.1.1 Background

Millers Ferry Lock and Dam is located 133.0 miles above the mouth of the Alabama River in the southwestern part of Alabama. The project is about 10 miles northwest of the city of Camden, Alabama, and about 30 miles southwest of Selma, Alabama. The lock and dam reside in Wilcox County. The reservoir created by Millers Ferry Lock & Dam extends 105 miles upstream and resides in both Wilcox and Dallas County. The Alabama River's navigability varies with available flow. Navigable depths of nine feet are authorized from the Port of Mobile, Alabama up to Montgomery, Alabama. However, during dry seasons, low flows resulting in 7.5-foot depths and lower provide limited navigation downstream of Claiborne Lock & Dam at river mile 72.5. Construction of Millers Ferry Lock and Dam was completed in the fall of 1969, and construction of the powerhouse was completed in May of 1970.

The Millers Ferry Project serves authorized purposes of hydropower generation and navigation. Other authorized purposes include recreation, fish and wildlife conservation, and wildlife management. This project does not provide flood risk management storage. Components of the Millers Ferry Project include a concrete gravity-type dam with gated spillway and supplemental earth dikes; a navigation lock and control station; a 90-megawatt (MW) power plant; and the William "Bill" Dannelly Reservoir.

The Millers Ferry Dam consists of a 994-foot-long concrete gravity spillway housing 17 50-foot by 35-foot tainter gates; 8-foot-wide piers housing individual electric hoists for gate operation; a spillway stilling basin; a two-section, non-overflow earth dike consisting of lock mound and training dike; and an overflow dike with crest elevation at 85.2 ft (NAVD88). In the closed position, the gates' top elevations are 81.2 ft. The area of the 105-mile reservoir created by the dam is 18,528 acres, and at a full pool elevation of 81.0 feet, the reservoir has a volume capacity of 346,254 acre-ft.

The Millers Ferry Lock is located between the spillway and the lock mound portion of the left-bank earth dike separating the spillway from the powerhouse. The lock chamber is 84-ft wide by 600-ft long. The lower stoplog sill is at elevation 19.2 ft, 13 feet below the minimum tailwater elevation of 32.2 ft, and the upper stoplog still is at elevation 64.2 ft. The filling and emptying of the lock is controlled by reverse tainter valves located in a system of wall and floor intake culverts within the lock chamber. The volume of discharge during emptying procedures can be estimated in acre-ft by multiplying the gross head differential by the surface area of the lock chamber in acres, or 1.264.

The Millers Ferry Powerhouse is a reinforced concrete structure located approximately 0.6 miles downstream of the spillway centerline on the left bank of the Alabama River, and between the left-bank earth dike adjacent the spillway and earthen dike connecting the lock structure to the powerhouse. The powerhouse consists of a 320-foot long by 168.5-foot wide trash gate adjacently located to facilitate the passing of trash, three 80-foot wide generation bays and one 80-foot wide erection bay. A switchyard is located to the east of the powerhouse. Power is generated via fixed-blade propeller turbines, which deliver an output rating of 90 MW.

The project's upper-pool elevation is typically managed to operate at El. 80.8 ft. The normal lower pool elevation is 35.2 ft. Thus, at normal conditions, a static head differential of 45 ft exists. The full discharge capacity of the spillway at El. 80.8 ft is 192,500 cfs, the equivalent of a flood which may be expected to occur once in 10 years. Once the spillway capacity is reached, a free overflow condition will prevail. There will be little difference in the water surface upstream and downstream of the dam. Design criteria for stability against overturning and sliding of the Millers Ferry structures make it imperative that the head, or difference between headwater and tailwater, not exceed 48 ft at any time. Table 2 provides pertinent project data for the existing Millers Ferry L&D.

### 5.1.2 Historic Events

The maximum, historic events at Millers Ferry L&D are shown in Table 4.

Table 4 – Historic Events at Millers Ferry L&D.

| Date | Peak Flow (cfs) | HW Elevation (ft-NAVD88) | TW Elevation (ft-NAVD88) |
|---|---|---|---|
| April 1886*** | 286,500 | N/A | N/A |
| March 1961*** | 284,000 | 86.66** | N/A |
| March 1990 | 269,000 | 83.38 | 82.73 |
| March 1979 | 260,000 | 81.22 | 81.06 |
| March 1929*** | 238,000* | 83.78** | N/A |

* Discharge is an estimate from USGS
**The USGS gage (02427500; Alabama River Near Millers Ferry AL) recorded stage at 26.82 ft above NGVD 29 until March 1979. Stages before this were converted to NAVD88.
***The event occurred prior to construction of the project.

### 5.1.3 Design Flood

The working copy of the Reservoir Regulation Manual (Revised 1990) describes the Spillway Design Flood (SDF) with a peak inflow of 840,000 cfs. The flood of March 1929 was used to precede the SDF with its peak occurring 5 days before the peak of the SDF. This SDF was routed through all the existing Alabama Power Company dams and Jones Bluff and Millers Ferry Dams. The routed SDF resulted in a maximum water surface elevation of 106.8 ft NGVD 29 and a peak outflow of 814,200 cfs. The inflow-outflow hydrographs for the SDF were obtained from the Water Control Manual and is shown in Figure 23. The natural (unregulated Alabama River basin) flow at the dam site is also shown on this figure for reference only and has otherwise no relevance to this risk assessment.

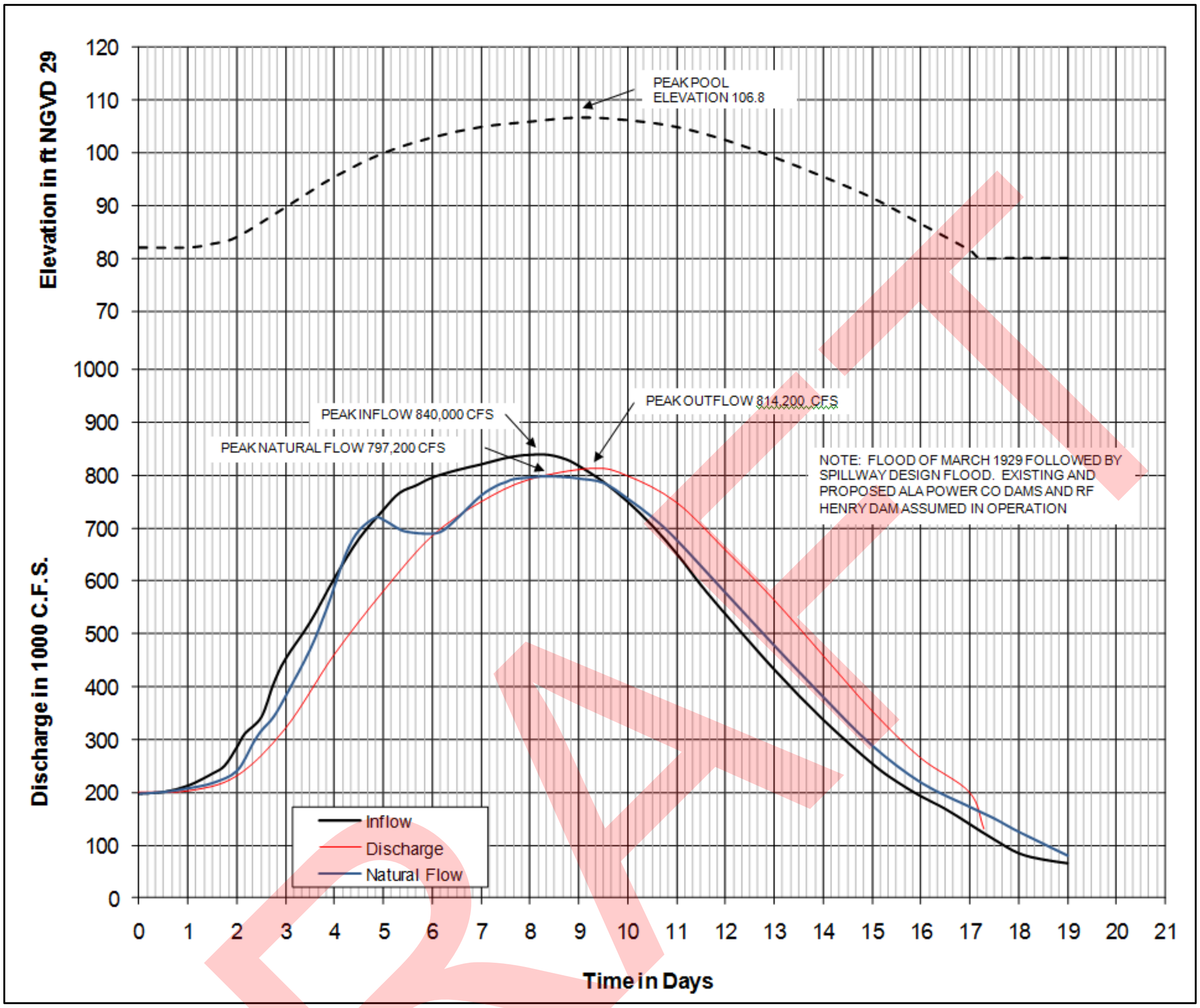



Figure 23 – Inflow-outflow pool hydrographs for the spillway design flood.

## 5.1.4 Hydrologic Hazard Curves

The flow-frequency, stage-frequency, stage-duration and rating curves for Millers Ferry L&D are shown in Figure 24 through Figure 27, below.

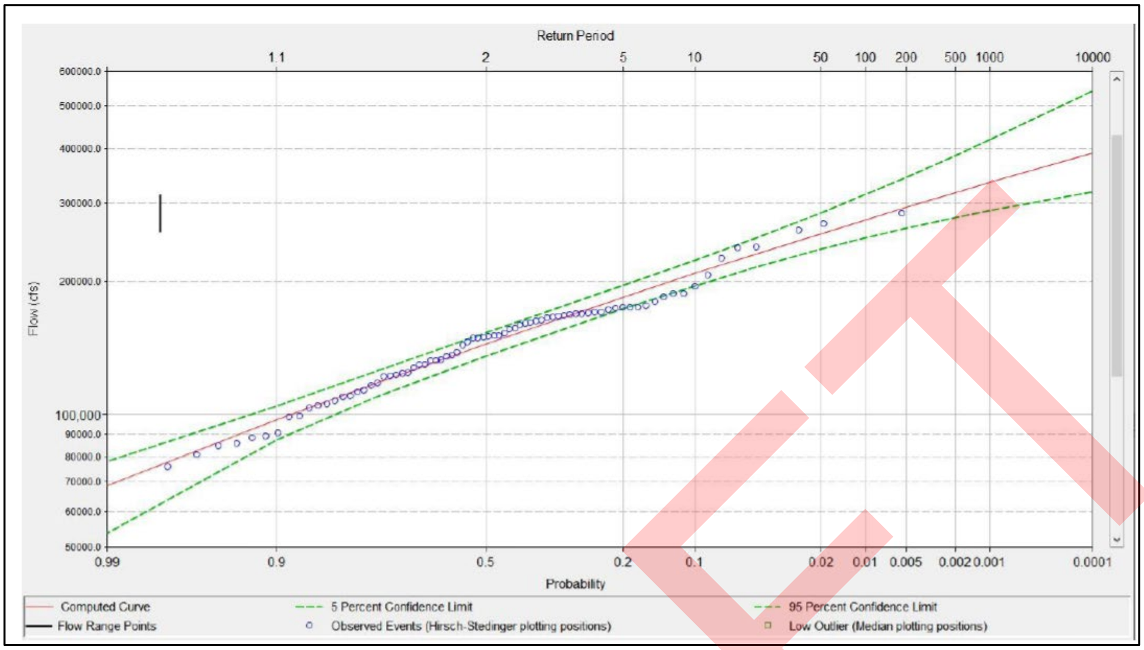



Figure 24 – Flow-frequency curve for Millers Ferry L&D.

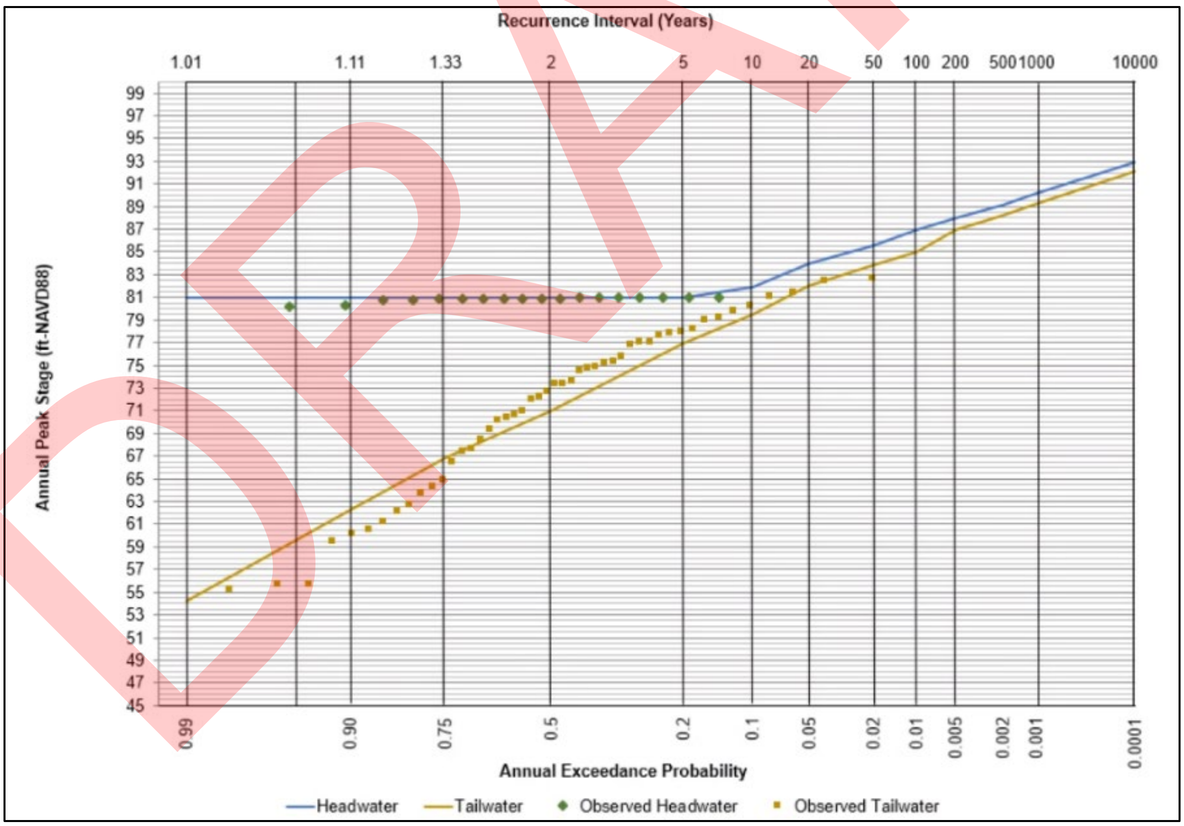



Figure 25 – Stage-frequency curve for Millers Ferry L&D.

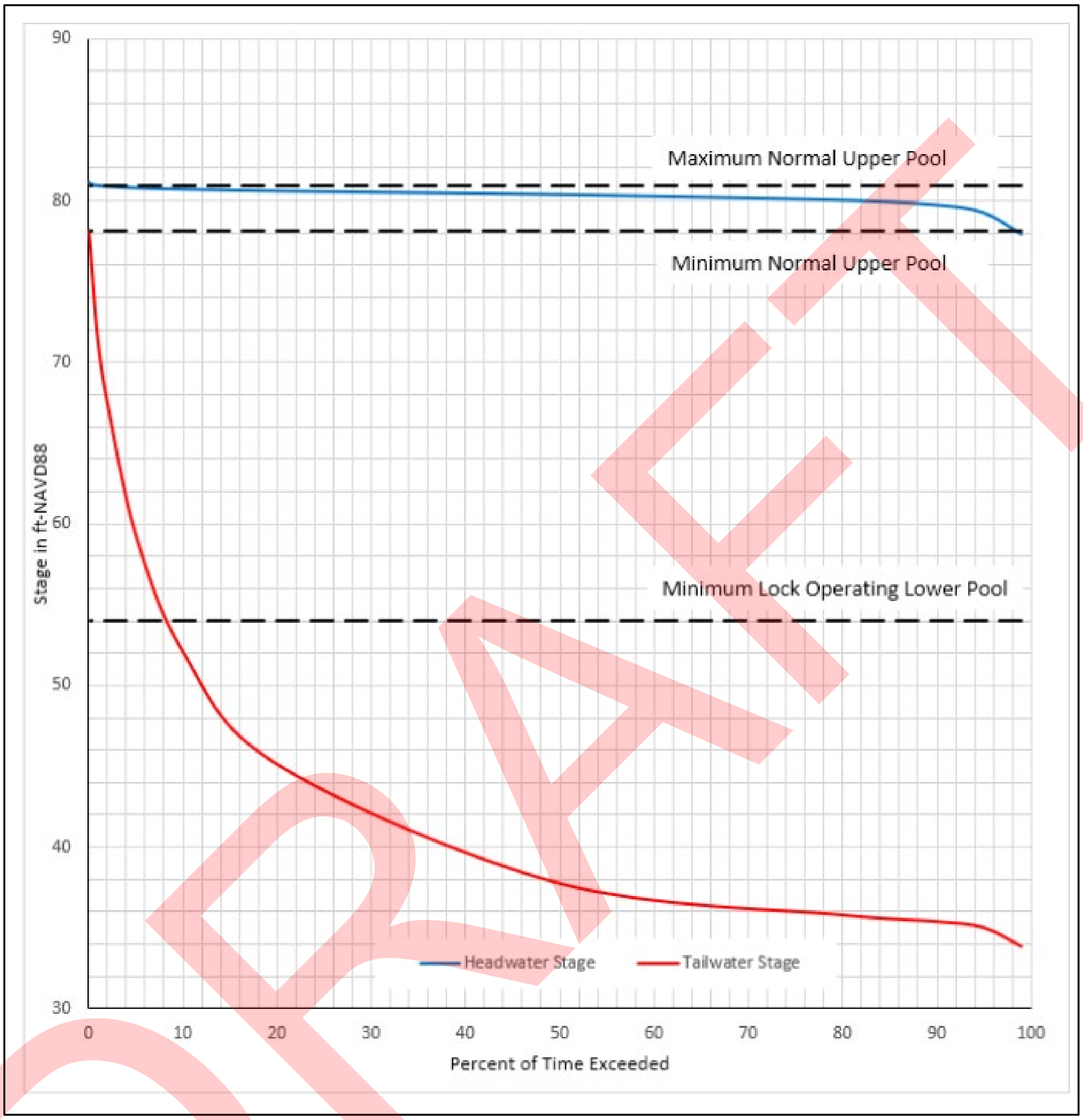



Figure 26 – Stage-duration curves for Millers Ferry L&D.

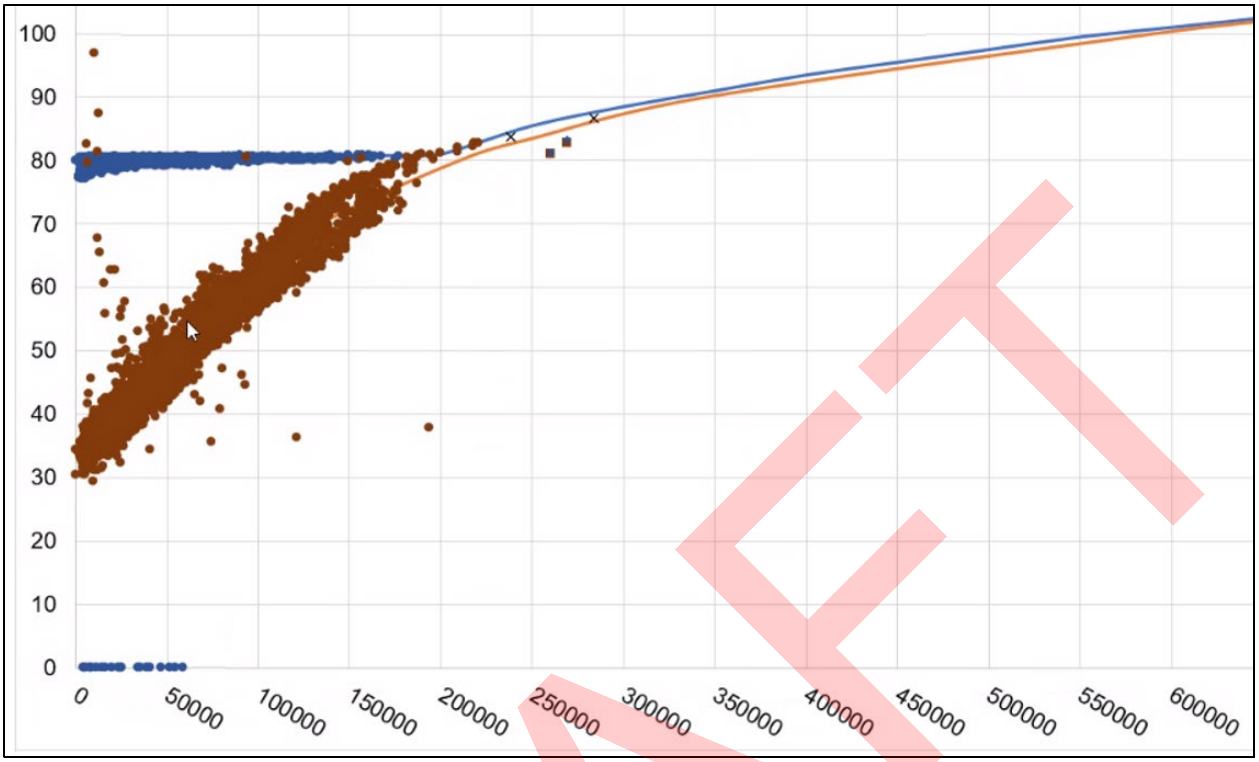



Figure 27 – Discharge rating curve for Millers Ferry L&D.

## 5.2 Claiborne L&D Hydraulic Loading

### 5.2.1 Background

The Claiborne L&D is the third and final project in the direction of flow on the Alabama River. It is located at River Mile (RM) 72.5 on the Alabama River in Monroe County, Alabama, approximately 5.7 miles upstream of Highway 84. The navigation pool extends approximately 60.5 miles upstream to Millers Ferry Lock and Dam, located at RM 133.0. Claiborne L&D is the lowest of three projects that provide navigable depths from the Port of Mobile upstream to Montgomery, Alabama. Robert F. Henry Lock and Dam (RM 236.3) is the upstream-most project in the series. Claiborne Lock and Dam's primary purpose is for navigation and targets re-regulation of flows from peak power releases upstream at Millers Ferry Lock and Dam. The project was completed in 1969 with navigation through the lock permitted on 15 November 1969. The Alabama River Basin drains 22,730 square miles; 21,473 square miles are regulated above Claiborne.

Large storms responsible for flooding in the vicinity of the project occur more frequently during winter and spring than in summer and fall. Typical hydrologic loading is from stalled frontal storms lasting two-to-four days; summer storms are convective-type thunderstorms. Fall months may observe heavy rainfall events accompanied by tropical cyclones. Total travel time of releases from Millers Ferry L&D to reach Claiborne L&D approximately 1.5-to-2 days, unless there is localized rainfall in the intervening basin (downstream of Millers Ferry) that would cause such an event peak to occur sooner at the project and flow for a shorter duration.

The damming surface for the project consists of left (2350-ft) and right (200-ft) dikes, a 500-ft long fixed crest spillway adjacent to the right dike designed for overtopping, gated spillway section with 6 tainter gates (each measuring 21-ft high by 60-ft wide), and an 84-ft by 600-ft (width by length) navigation lock with control station. Left and right dikes tie into graded high ground at elevations 60.11 and 40.11 ft, respectively.

Claiborne L&D is designed to maximize discharge capacity frequently (i.e., >1/1 ACE), as full spillway capacity (including gated and fixed crest structures) is 70,763 cfs which corresponds to the upper end of the normal upper pool range for the project (El. 36.11 NAVD88). Prior to this flow, lock operators raise and maintain all spillway gates to their maximum position at approximately 67,000 cfs. Beyond this condition, water levels, both headwater and tailwater, continue to rise, ultimately reflecting a no-project (i.e., natural river) condition. At El. 51.11 ft, the lock walls are overtopped. At El. 51.61 ft, project personnel are signaled to evacuate.

According to records at USGS 02428400, the lock walls have been overtopped and project personnel has been evacuated a total of 17 times. Prior to evacuation, equipment from the site is removed/elevated in the lock house. Equipment of concern includes control panels from lock control houses, and miscellaneous equipment from the first level of the lock-house (elevated through elevator). Additionally, prior to evacuation, the breaker panel controlling lock-house operations is switched off and a security fence is placed upstream of the left bank esplanade, extending from the left lock wall to the security gate (perpendicular to flow). Navigation through Claiborne L&D is discontinued once tailwater elevations reach 47.11 ft, equating to a flow of approximately 130,000 cfs, and a recurrence interval of 1/1.8 ACE.

5.2.2 Design Flood

The original design flood for Claiborne Lock and Dam was based on regional model studies conducted during the design of Millers Ferry Lock and Dam. The guidance developed at the time (Interim Report, Flood Control at Montgomery, Alabama, Alabama River, 1956) mandated that the standard project flood (SPF) would equate to 60% of the runoff generated from the probable maximum precipitation (PMP). The runoff hydrographs were utilized to develop the inflow design flood (IDF), or the maximum flow the project can pass without damage. Design flow of the IDF at the project location is 814,800 cfs (estimated < 1/1000 ACE), which produces a pool El. of 78.31 ft. The SPF design flow is 494,500 cfs (estimated <1/500 ACE), which is 60.7% of the IDF. The event produces a pool El. of 69.21 ft. Left and right embankment overflow dikes overtop at El. 60.11 ft (280,000 cfs) and 40.11 ft (90,000 cfs), respectively, increasing conveyance at the project during high flow events. When flows approach 67,000 cfs, the spillway gates are raised to their fully opened regulatory positions of 15-ft. As flows approach 160,000 cfs (1/100 ACE) and the water levels continue to rise above the lock wall elevations of 51.11 ft, the spillway gates are raised to their maximum un-regulatory positions, with low-steel of the gates at El. 66.11 ft, and the project is evacuated for safety purposes. The flood of record (March 1961) was determined to produce a headwater elevation of 58.91 ft when routed through the constructed project. During this event, un-regulated opening of the gates provided 7.2 ft of freeboard.

### 5.2.3 Historic Events

Claiborne L&D has been overtopped multiple times since construction was completed in 1969. Historical floods from Table 5, including April 1979, March 1990, and January 2016 overtopped the project, including the lock walls. Based on records from USGS 02429500 and USGS 02428400, the lock walls (El. 51.11 ft) have overtopped a total 17 times. During these overtopping events, the gates were raised out of the water, critical equipment was stored, and navigation was halted. In January 2016, operators and personnel returned to the project 7 days after overtopping once flood waters receded below the lock walls. Because traffic would not be present on the Alabama River during such an event, economic impact was not considered applicable until water levels receeded below those equating to structural overtopping (pool elevation of 51.11 ft). The total time for receding from this level to the maximum navigable tailwater elevation threshold of 47.11 feet – required to comply with miter gate operation limitations (from Water Control Manual) – is estimated at 5 days. Overtopping duration applicable to the right dike during the historic events indicated in Table 5 ranges from 12-25 days.

Table 5 – Historic flow and river stage elevations at Claiborne L&D.

| Date | Peak Flow (cfs) | HW Elevation (ft-NAVD88) | TW Elevation (ft-NAVD88) |
|---|---|---|---|
| March 1929* | 257,000 | 54.71 | N/A |
| March 1961* | 267,000 | 55.26 | N/A |
| April 1979 | 215,000 | 55.11 | N/A |
| March 1990 | 255,000 | 56.71 | N/A |
| January 2016 | 135,000 | 53.88 | 53.62 |

* Event occurred prior to dam construction and based on records at USGS 02429500.

### 5.2.4 Hydrologic Hazard Curves

A flow-frequency curve for Claiborne L&D was generated from a USGS annual peak flow dataset spanning 1976 to 2016 and applicable to the upper pool (Figure 28). Regulation from Millers Ferry is not apparent from the results of a Bulletin 17C analysis on annual peak flows. For this reason, the annual peaks were left unadjusted. In general, the system extending from Claiborne L&D up to Robert F. Henry L&D is regulated, although changes to the regulation have not been made since inception and are predicted to remain in place into the future. This information can be used to advise the risk assessment, but care should be used when it is interpreted for any other purpose.

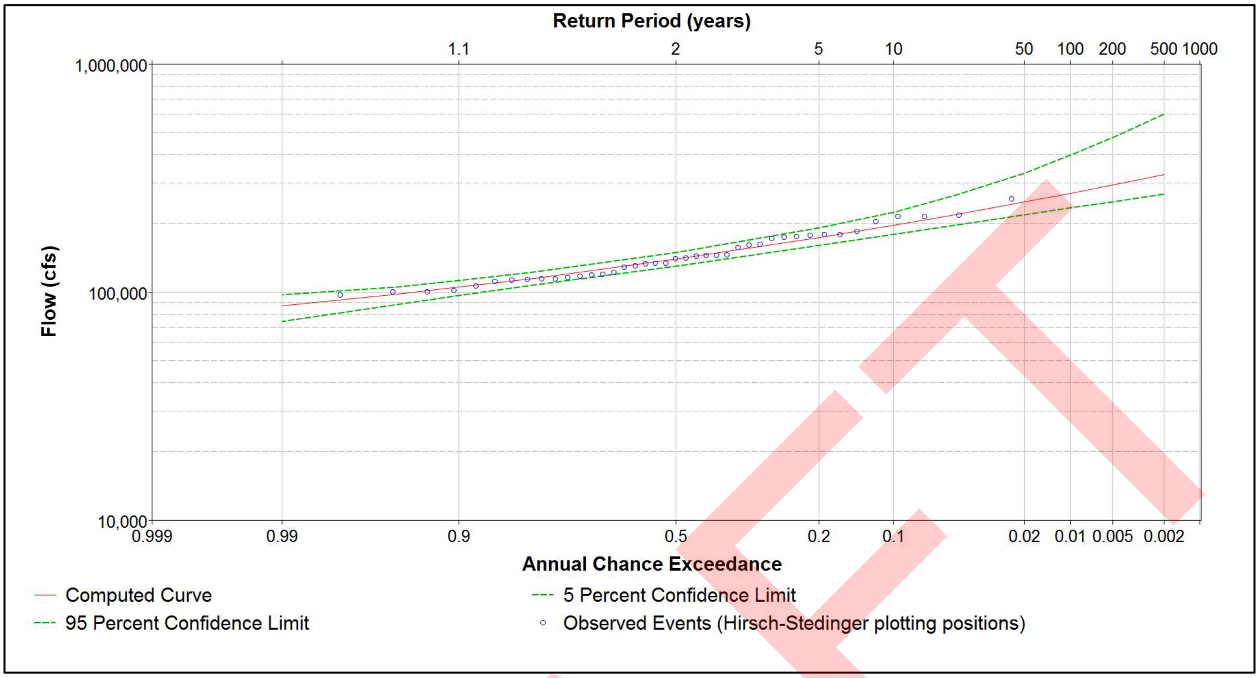



Figure 28 – Flow-frequency curve for Claiborne L&D.

The headwater and tailwater rating curves were obtained from the Water Control Manual, in Appendix F: Plate 7-20, and converted to reference elevations to NAVD88 (NGVD29 + 0.11 ft.). The rating curves are provided in Figure 29. In the figure, several structural elevations are noted as well as discharge differences based on active spillway capacity. From the Water Control Manual, considerable flooding of agricultural lands below the project are expected for a water surface elevation of 40.11 ft. Provided that during high-flow conditions the project will not observe differences in headwater and tailwater, outflows may be approximated as equivalent to inflows. Based on this assumption, the flow corresponding to El. 40.11 ft. can be estimated as 90,000 cfs. Higher inflows may be expected; however, damage will be minimal given the lack of development below the project.

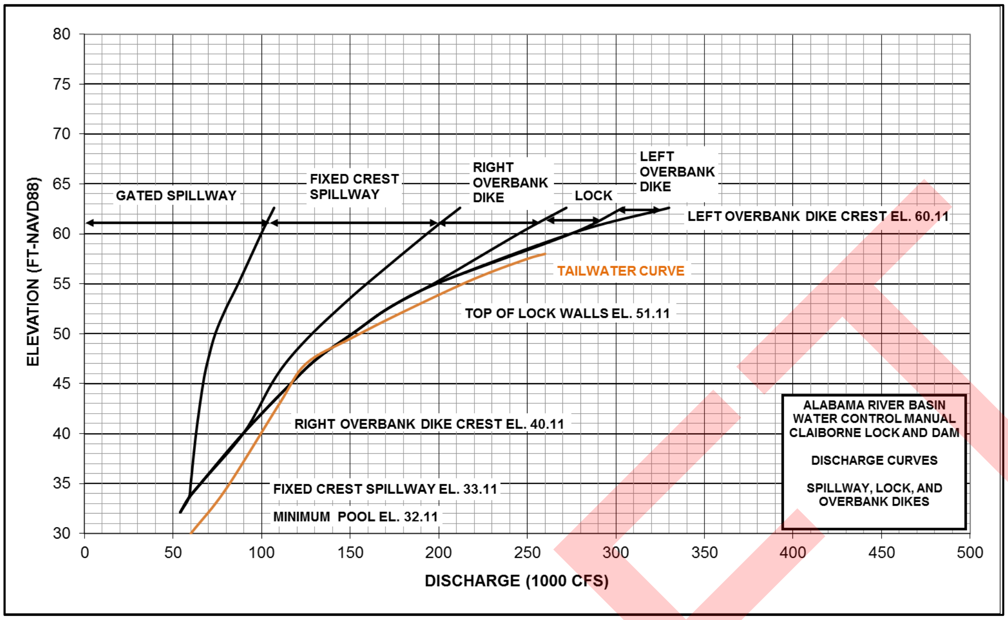



Figure 29 – Headwater and tailwater rating curves for Claiborne L&D.

The stage-frequency curve for the project was created from a general, empirical frequency analysis of annual peak stages from USGS 02428400 and USGS 02428401 utilizing Weibull plotting positions. The plots include best-fit linear models for interpretation of the stage values. The period of record available for analysis of the upper pool included years 1976 to 2018; analysis of tailwater peak stages included years 1972 to 2018. The records were converted to NAVD88; however, gage datums for USGS 02428400 and 02428401 are both 0.0 feet NAVD88. Thus, stage values equate to elevation values.

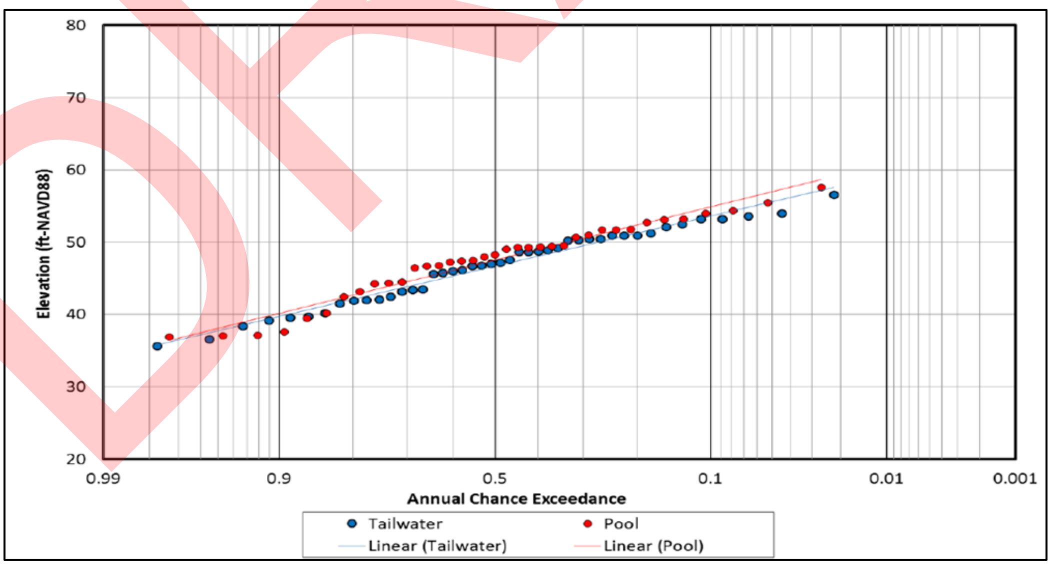



Figure 30 – Stage-frequency curve for Claiborne L&D.

Daily stage-duration curves for both the upstream and downstream pools were created using HEC-DSS and daily pool elevation records from 1996 to 2018 for the both the pool and tailwater stages. This information is shown in Figure 31, which also provides the upstream and downstream navigation pools as reference. In the figure, normal tailwater was based on annual averages over the period of record of daily measurements. This method was pursued for completion of this plot, as the lower pool is unregulated and, on average, tends to fluctuate between wet and dry seasons by approximately 10 feet.

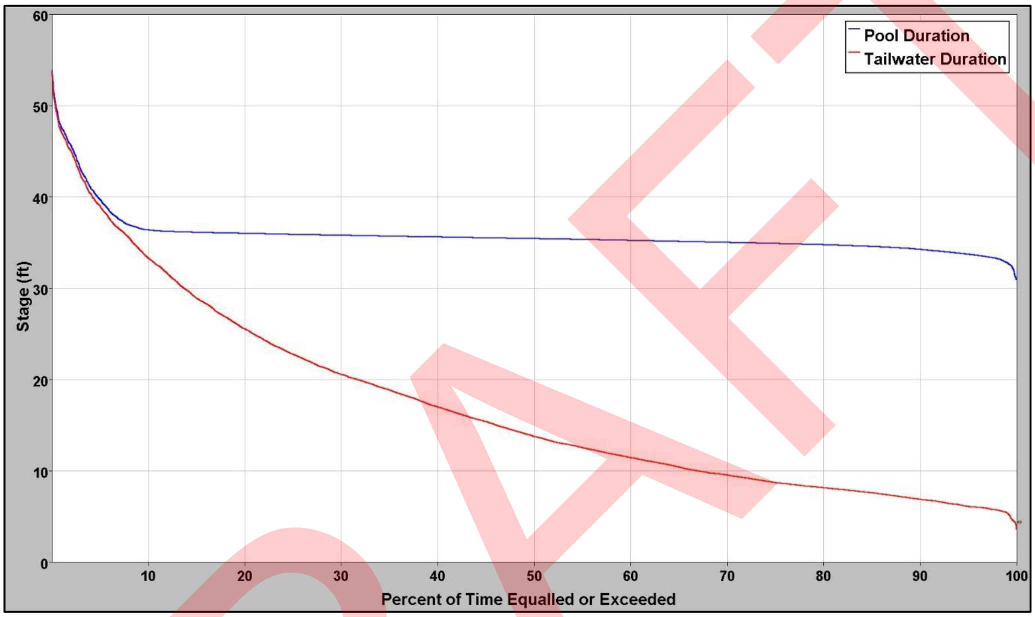


Figure 31 – Stage-duration curves for headwater and tailwater.

Given the high degree of variability in stage at the project as a result of seasonal occurrence, quarterly flow duration curves were developed to better quantify the non-exceedance probability of stage values for both the upper pool and tailwater. As previously mentioned, tailwater stages can fluctuate as much as 10-ft between wet and dry seasons at the project. Even though the upper pool is part of a regulated system, seasonal variability affecting inflows to the project can induce complication in the risk assessment of non-exceedance probabilities of pool stages. Analysis of monthly stage duration decreases the uncertainty in estimation of such probabilities. For clarity purposes, quarterly stage duration curves at the project for pool and tailwater are separated in Figure 32 and Figure 33, respectively.

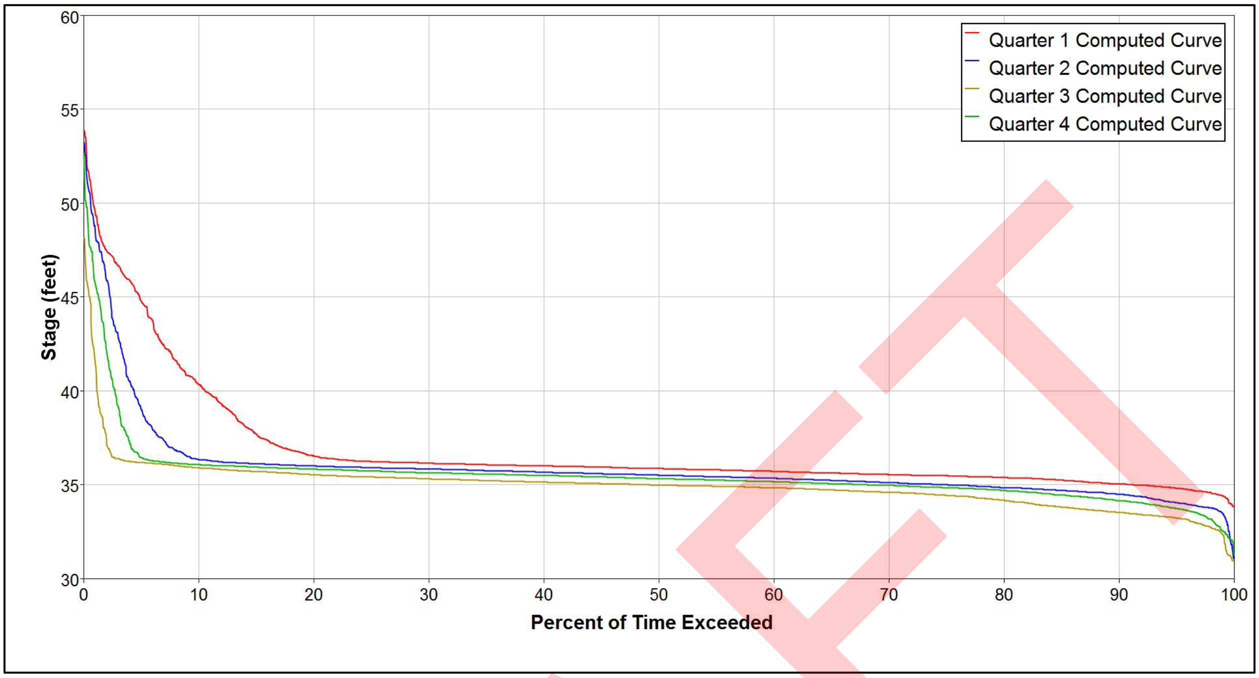



Figure 32 – Quarterly stage-duration curve for upper pool.

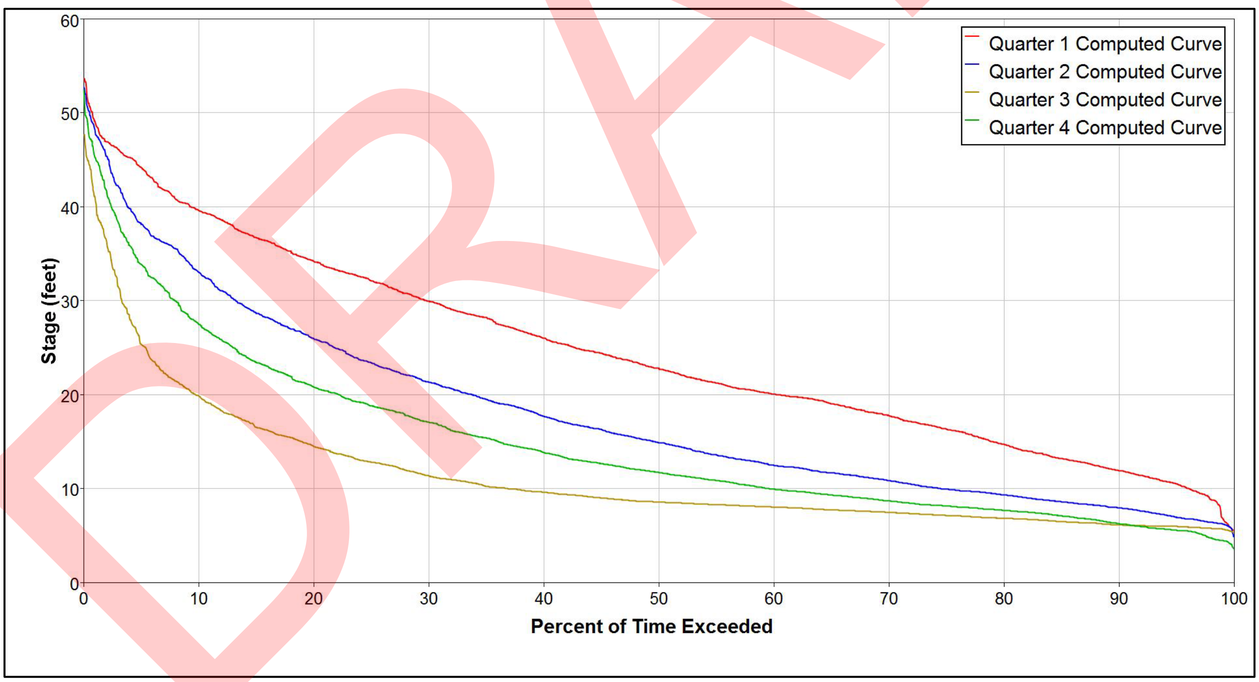



Figure 33 – Quarterly stage-duration curve for tailwater.

A daily flow-duration plot (Figure 34) was created from records at USGS 02428400, located in the upper pool of the project. The plot was created using a daily flow period of record spanning 1975 to 2018. From Figure 34, the 50% flow duration is 16,900 cfs.

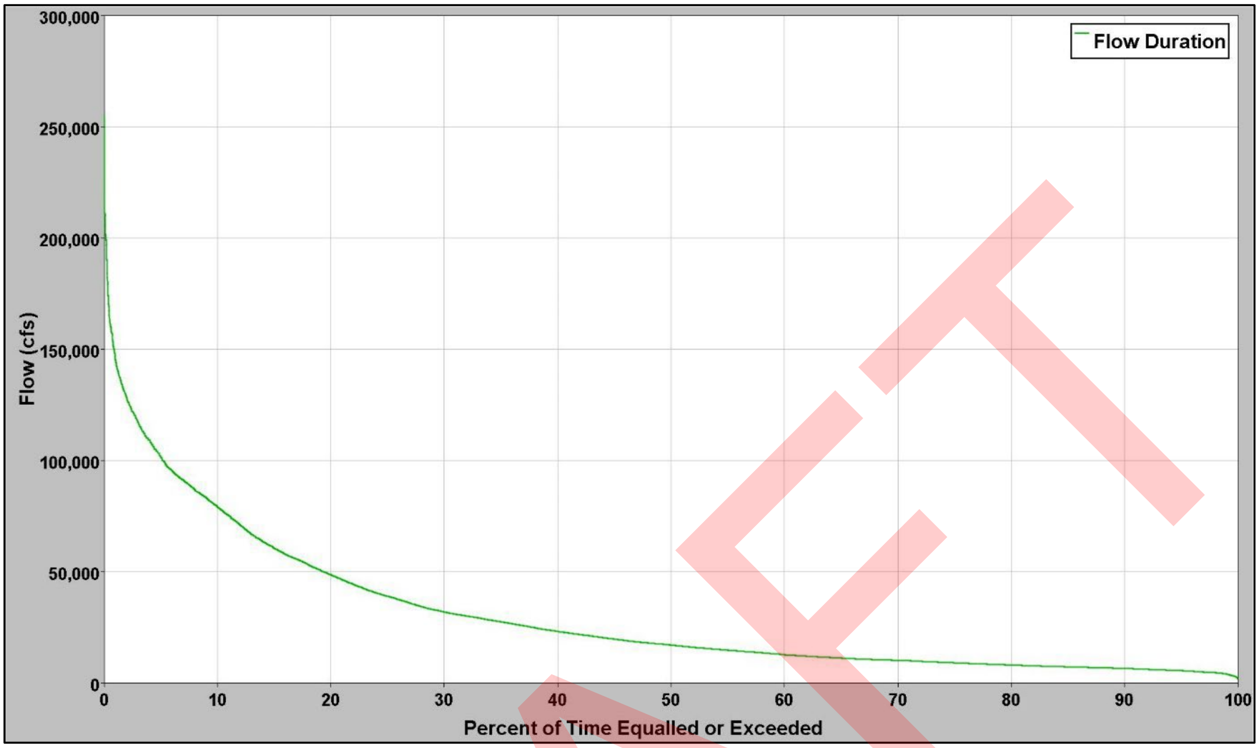



Figure 34 – Annual flow duration curve

## 6. Incremental Consequences and Summary of Previous Risk Assessments

This risk assessment did not include any breach modeling or LifeSIM consequence analysis. Consequences have already been quantified during the August 2018 Periodic Assessment for Millers Ferry L&D and the December 2022 Periodic Assessment and Assessment for the Claiborne L&D. The consequences assessment did not include any prediction of life loss from breach.

Consequences are not further discussed in the risk assessment section of the report as all consequence from failure will be as described in this section and consequences are not a risk driver for these significant hazard structures.

### 6.1 Millers Ferry L&D

The Millers Ferry L&D project is a run-of-river dam that maintains a navigable pool for river traffic but does not store flood waters. At Millers Ferry L&D, differential hydraulic head across the dam decreases with increases in flow. As a result, relatively dry conditions with little inflow will produce the greatest hydraulic head difference across the dam and have the greatest potential for incremental consequences.

The consequences analysis from the last PA determined there are no societal incremental life safety risk estimates for with or without intervention shown in Figure 35. The incremental economic risk estimates with and without intervention are provided in Figure 36. Table 6 and Table 7 show the display the incremental economic risk summary for with and without intervention, respectively.

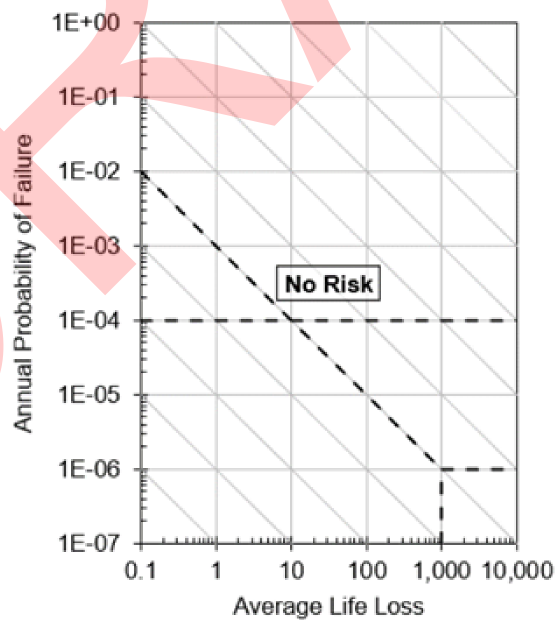



Figure 35 – No societal incremental life safety risk estimates for Miller Ferry L&D.

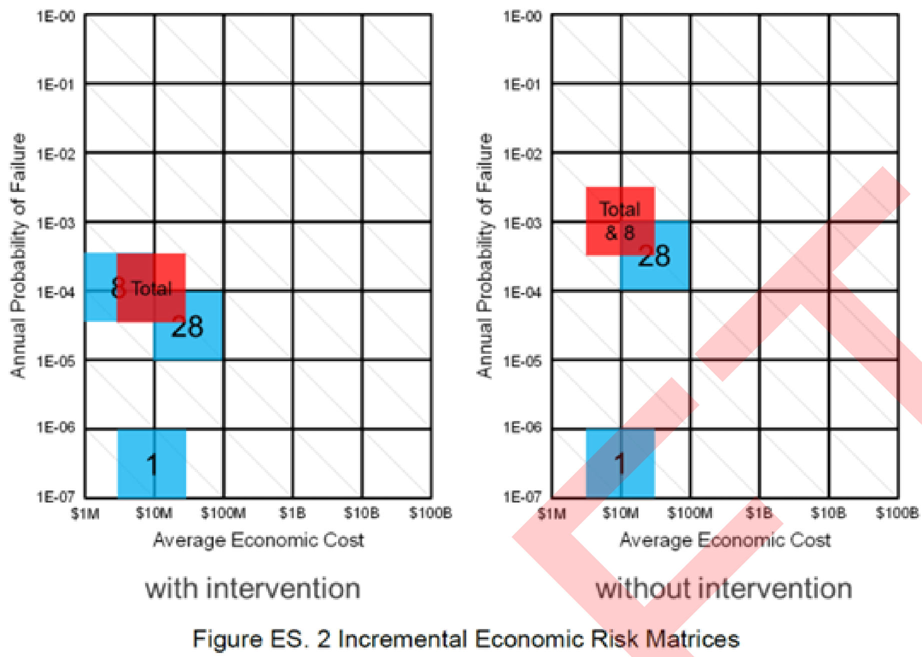



Figure 36 – Incremental Economic Risk Matrices with and without Intervention from PA.

Table 6 – Incremental Economic Risk Summary with Intervention.

| Potential Failure Mode | Annual Probability of Failure, APF (failures/year) | Average Incremental Economic Cost, $\bar{\$}$ ($/failure) | Average Annual Incremental Economic Cost, AAEC ($/year) |
|---|---|---|---|
| PFM RE-01: Overtopping | 1.0E-07 to 1.0E-06 | $3M to $30M | 3.16E+00 |
| PFM S-08: Trunnion Anchor Failure | 3.0E-05 to 3.0E-04 | $1M to $10M | 3.16E+02 |
| PFM CE-28: BEP near the toe at Lock Esplanade | 1.0E-05 to 1.0E-04 | $10M to $100M | 1.00E+02 |
| Total | 3.0E-05 to 3.0E-04 | $3M to $30M | 1.0E+03 |

Table 7 – Incremental Economic Risk Summary without Intervention.

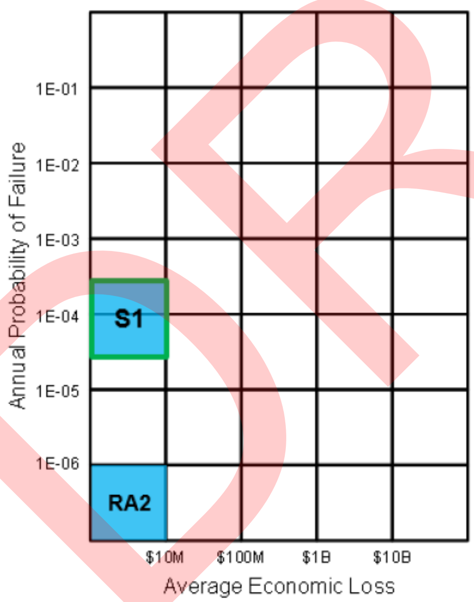
| Potential Failure Mode | Annual Probability of Failure, APF (failures/year) | Average Incremental Economic Cost, $\bar{\$}$ ($/failure) | Average Annual Incremental Economic Cost, AAEC ($/year) |
|---|---|---|---|
| PFM RE-01: Overtopping | 1.0E-07 to 1.0E-06 | $3M to $30M | 3.16E+00 |
| PFM S-8: Trunnion Anchorage Failure | 3.0E-04 to 3.0E-03 | $3M to $30M | 1.0E+04 |
| PFM CE-28: BEP Near the toe at Lock Esplanade | 1.0E-04 to 1.0E-03 | $10M to $100M | 1.0E+04 |
| Total | 3.0E-04 to 3.0E-03 | $3M to $30M | 1.0E+04 |

Since significant property damage due to dam breach is very unlikely, the project was screened out by the Modeling, Mapping and Consequences (MMC) Production Center. Consequences are primarily due to loss of navigable pool or lock closure. The risk estimates above would not be expected to change based on construction of the fish ladder, assuming the recommendations in this report are implemented.

## 6.2 Claiborne L&D

Like Millers Ferry L&D, the Claiborne L&D is a run-of-river dam that maintains a navigable pool for river traffic but does not store flood waters. The project is operated such that flows pass through the spillway or over the dam.

There is presently no dedicated commercial traffic demand, and the lock is classified as low use. Economic consequences associated with lost navigation benefits are negligible and no loss of service PFMs were deemed to be risk drivers during the latest PA. Additionally, the incremental risk associated with a dam breach is considered low. The incremental risk is driven by the potential for a spillway tainter gate to fail due to trunnion friction. The risk matrix in Figure 37 portray the incremental economic risk due to failure or breach associated with the two identified risk drivers. The estimated total APR is between 3E-5 and 3E-4 failures per year, and the weighted average incremental economic loss between \$3M and \$30M per failure. Since significant life safety and property damage due to dam breach is very unlikely, the project was screened out by the MMC Production Center.




Risk-Driver PFMs
PFM S1: Tainter gate trunnion friction failure
PFM RA2: BEP thru right abutment
PFM RA2 APF <1E-7 w/ intervention
Total Risk: Green Box

Figure 37 – Incremental Economic Risk Matrix for Claiborne L&D.

## 7. Potential Failure Mode Analysis

A thorough review of background data, existing plans and specifications, and conceptual fish ladder design documents was initially conducted, followed by a "brainstorming" of the Potential Failure Modes (PFMs). Additional failure modes may exist which were not documented at this time due to the limited design maturity and physical data available. The purpose of this effort was to document which failure modes are likely to occur and ensure the design accommodates sufficient controls to address them or if design features should be added which could have significant impact on cost or completeness.

A failure mode is a unique set of conditions and/or sequence of events which could result in failure, where failure is "characterized by the sudden, rapid, and uncontrolled release of impounded water" (FEMA 2003). A Potential Failure Mode Analysis (PFMA) is the process of identifying and fully describing potential failure modes. A facilitator guided the team members in developing the potential failure modes based on the team's understanding of the project vulnerabilities resulting from the data review and current field conditions.

The PFMs were sorted into two categories based on available information and observations: 1) Likely Risk Driver, and 2) Excluded. PFMs considered likely contributors were developed in more detail.

- Likely Risk Driver – These PFMs are considered likely risk drivers for the project based on the current project details. These PFMs are highlighted for the design team and include recommendations to further reduce risk associated with these PFMs and could affect project cost or alternative selection.
- Excluded – Justification is included why this PFM is not considered a risk driver based on the current design details, hydrologic, and geologic information.

The PFMs identified and analyzed during the PFMA are summarized throughout this section broken out by the PFMAs for each project: Millers Ferry L&D, Claiborne L&D (Right Abutment), and Claiborne L&D Part 2 (Spillway). For each PFMA analysis, the likely risk driver PFMs and excluded PFMs are also broken out. All elevations are stated in North Atlantic Vertical Datum of 1988 (NAVD88) unless otherwise specified.

### 7.1 Millers Ferry L&D Potential Failure Mode Analysis

The potential failure modes identified and analyzed during the PFMA for the Millers Ferry L&D are summarized below in Table 8. Following each heading is a complete description of the potential failure mode, the adverse and favorable factors identified during the session, and assessment of harm to the project. Where necessary, recommendations were identified to further reduce risk and modify the project such that the do no harm concept is not violated by the project. Fifteen (15) PFMs were identified by the team for consideration and one (2) PFM identified as a likely risk driver for the project. The PFMs bolded below resulted in the most critical recommendations for the project. While the remaining PFMs were excluded as likely risk drivers, minor recommendations resulted from several of the PFMs discussed.

Table 8 – Potential Failure Mode (PFM) Summary for Millers Ferry L&D.

| PFM # | PFM Description |
|---|---|
| 01 | Overtopping of the gates |
| 02 | CLE adjacent to new gate structure |
| 03 | Gate failure causes uncontrolled releases |
| 04 | Global instability of the gate structure |
| 05 | Erosion of channel due to excess velocities |
| 06 | BEP through foundation into channel |
| **07** | **Downstream debris cause out of channel flow and flooding** |
| 08 | Sediment buildup in channel cause out of bank flow |
| 09 | Artesian condition uncovered during excavation causing contact erosion progression toward dam |
| 10 | Cofferdam removal during construction causes defect in the damming surface |
| 11 | CLE into structure bypassing the gates |
| 12 | Confluence of three channels causing eddies and eroding the banks |
| 13 | Access restriction caused by new channel prevents access to the dam |
| 14 | Slope instability due to new channel adjacent to existing embankment |
| **15** | **Erosion of “island” bypasses the control structure** |

7.1.1. Likely Risk Driver PFMs:
These PFMs are considered likely risk drivers for the proposed project based on the available information and design details. These PFMs may include recommendations to further reduce risk associated with these PFMs that may affect project costs.

**PFM 07: Downstream debris cause out of channel flow and flooding.**

Description: Float debris during a flood passes through the fish channel gates and causes a debris jam in one of the fish ladder pools. The debris accumulates and restricts flow, causing water to stack up within the channel upstream of the blockage. Water begins to overtop the channel banks and flows across private lands before it makes its way back to the Alabama River. The overland flow erodes the channel banks and forms a new flow path. This erosion from the new flow path undermines the boulders and bypass cannel bottom protection, allowing head cutting to progress upstream. The erosion destabilized the dam embankment toe followed by slope failure and breach. The channel blockage and overflow sequence is illustrated in Figure 38.

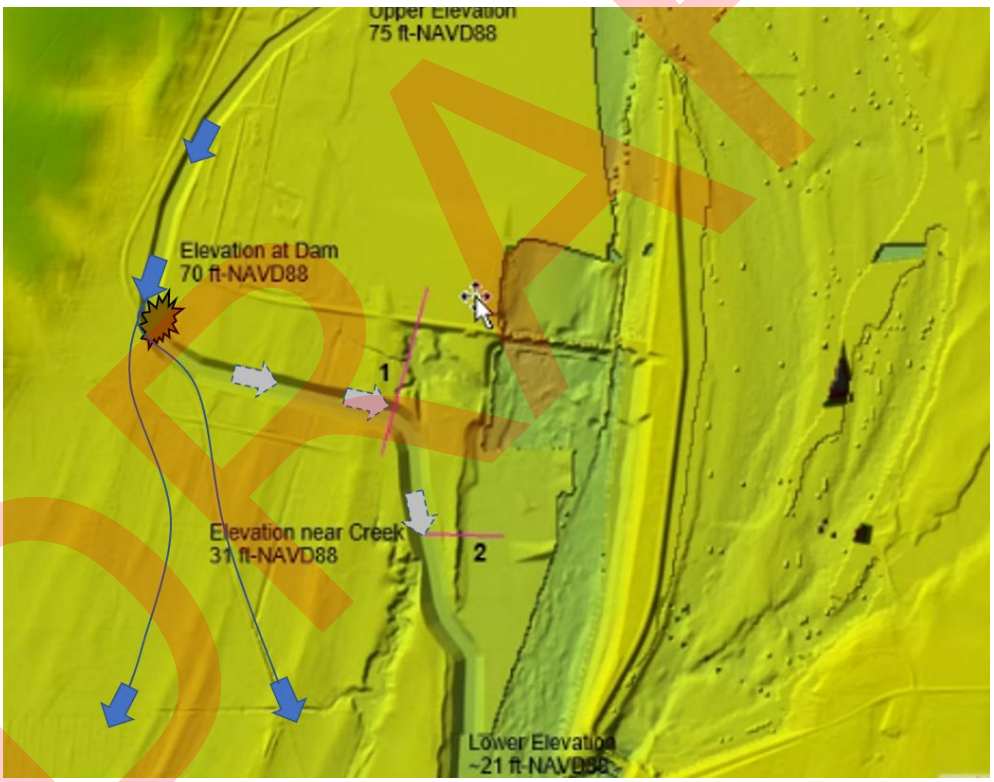



Figure 38 – Example flow paths from debris blockage in fish ladder channel.

Boulders used to build the fish ladder pools will restrict flow velocities. Figure 39 illustrates the use of boulders and rubble to make a series of pools that restricts velocity in the channel to less than 5 to 6 fps and less than 2% slope making a debris jam likely over the service life of this project. The rock steps will tend to trap floating debris and even small debris could get caught between boulders and begin to form a debris dam. Debris has been observed in the Alabama River during past storms evidenced by the

Google Earth Imagery in Figure 40 that shows debris stacked against the spillway and Tainter gates from March 2019. For these reasons, a debris jam is considered likely over the service life of the project.

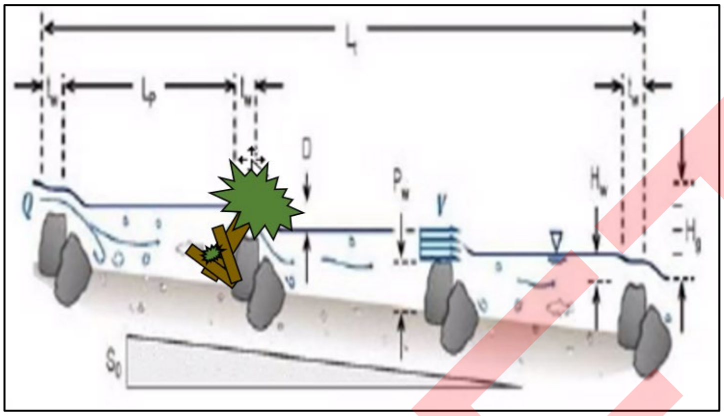


Figure 39 – Boulders create pools that restrict velocities.


Figure 40 – March 2019 Google Earth Imagery showing debris stacked against the spillway and lock gates.

Passing debris in the existing channel does not have history of debris jam. The main river channel is over 1,200 ft wide, and most debris passes through the gates and continues downstream. For debris lodged in the new fish channel, intervention would be easy in that the gates controlling the fish ladder channel could be closed, allowing for the debris to be removed; however, detection of such an occurrence is not likely with remote operation.

If the debris blocked the fish ladder channel and caused out of bank flow, erosion could occur and begin to head cut upstream. This head cutting would tend to follow the flow of water and would progress back to the armored channel or eventually back to the discharge structure. The out of bank flow would cause incremental flooding of the lands around the channel even without breach of the dam. This would be incremental flooding that could cause economic damages to surrounding lands if it remained out of bank for long enough duration. The regional area slopes towards the river and flood water should re-enter the river close to the channel. Such flooding would not be life threatening and economic damages would be low only affecting the areas immediately around the canal.

Intervention would likely be prior to dam failure because it would likely take days or weeks for the blockage to accumulate debris to the point of causing out of bank flow and additional days and weeks for erosion to become significant to progress to near the dam toe, especially considering the channel bottom will be armored to prevent erosion under the expected velocities. Maintenance of the dam is the responsibility of the non-Federal sponsor and is remotely operated most of the time. Therefore, there is a possible scenario where the blockages are not observed for a period of days.

The fish ladder channel bottom width of 100 ft with a 3:1 (horizontal:vertical) slope, as shown in the cross-section in Figure 41 These dimensions would require a lot of material to completely block the flow through the channel. The geometry of the channel should allow flows to pass the blockage without overtopping the banks.

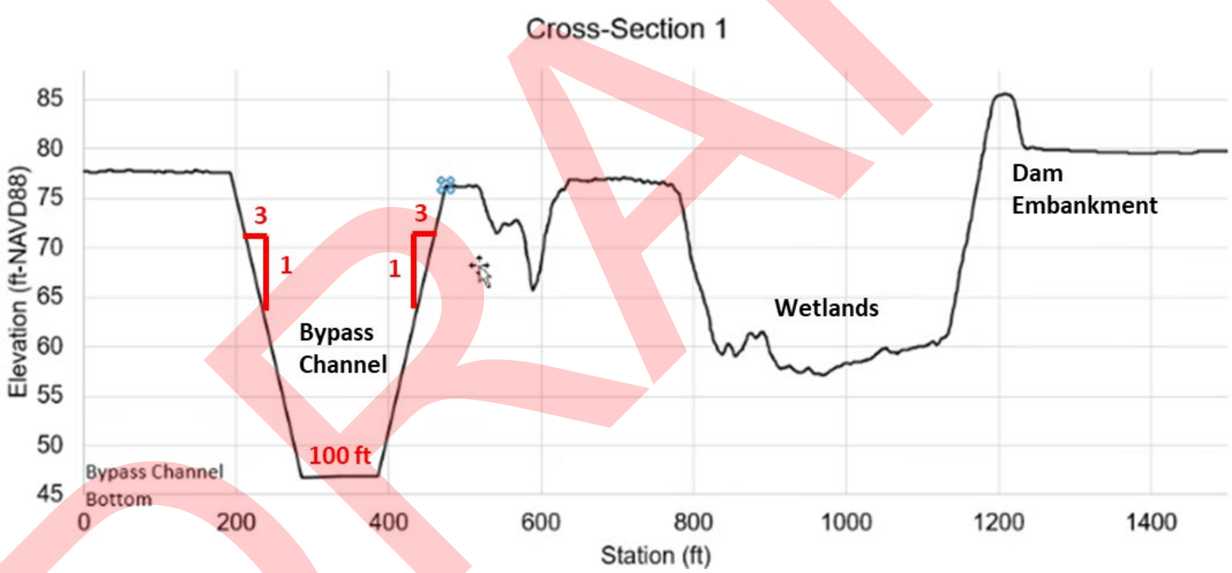



Figure 41 – Cross-section of fish ladder channel dimensions.

While there is a high probability of blockage occurring at some point during the service life of the project, the chance of blockage resulting in dam failure is judged to be remote based on the width of the channel and the very long duration out of bank flow would have to continue to progress to breach. Therefore, this PFM is excluded as a risk driver, but it is an operation and maintenance consideration.

Depending on the location of the debris blockage and out of channel flow, it could still cause some economic damages to the adjacent lands even without failure of the dam. Based on review of the fish ladder channel cross section, if out of bank flow did occur, most of the water would overtop to the wetlands north of the fish ladder channel and south of the embankment, rather than inundating the private property to the south and west where the house is around El. 85 ft.

It is recommended that debris booms be included in the design to limit to quantity of debris floating through the control structure gates. Once alignment is finalized, the channel should be modeled with debris jam and verify where out of bank flow would go and what areas would be inundated from such an event, assuming the debris dam is near the upstream end of the fish ladder. It should also be required to inspect the channel after a high-water event to verify and/or remove debris from the channel. Revision of the EAP and Surveillance and Monitoring Plan should include inspections of the channel in addition to inspections of the dam. Installation of cameras would allow for remote monitoring of possible blockage or sediment buildup in the bypass channel during regular use.

**PFM 15: Erosion of "island" bypasses control structure**

PFM 15 is not actually identified as a risk driver for the alternative evaluated in this risk assessment that includes a gated structure through the damming surface; however, it is documented as a potential risk driver because, during optimization, a decision could be made to route the channel around the right abutment of the dam. If this route were selected, this PFM would become the risk driver for the project and the new damming surface created between the gated structure and the existing right abutment would need to be designed as a dam.

Description: An alignment is selected during optimization that routes the channel around the right abutment of the dam. The main dam and right abutment overtops during a flow event less frequent than an approximate 1/15 AEP flood event. As the flood recedes, overtopping velocities over the new damming surface and into the fish ladder channel begin to erode and head cut back through the earthen embankment allowing uncontrolled release of water around the right abutment, resulting in loss of service and benefits of the fish ladder channel. This failure path is shown in Figure 42.

If an alternative alignment is selected that allows flow around the right abutment, the dry island created between the normal stage river path and the fish ladder will become a new damming surface, shown in Figure 42. This PFM was not evaluated in detail since this is not the TSP alignment and since critical details like gate location are not yet known. If the gate is in line with the existing dam then the "island" would not become a damming surface, likewise, if the gate were at the channel connection to the river, the damming surface shown in Figure 42 would exist. If this alignment were selected, it would need to be evaluated for a variety of PFMs including overtopping, slope instability and all PFMs related to seepage.

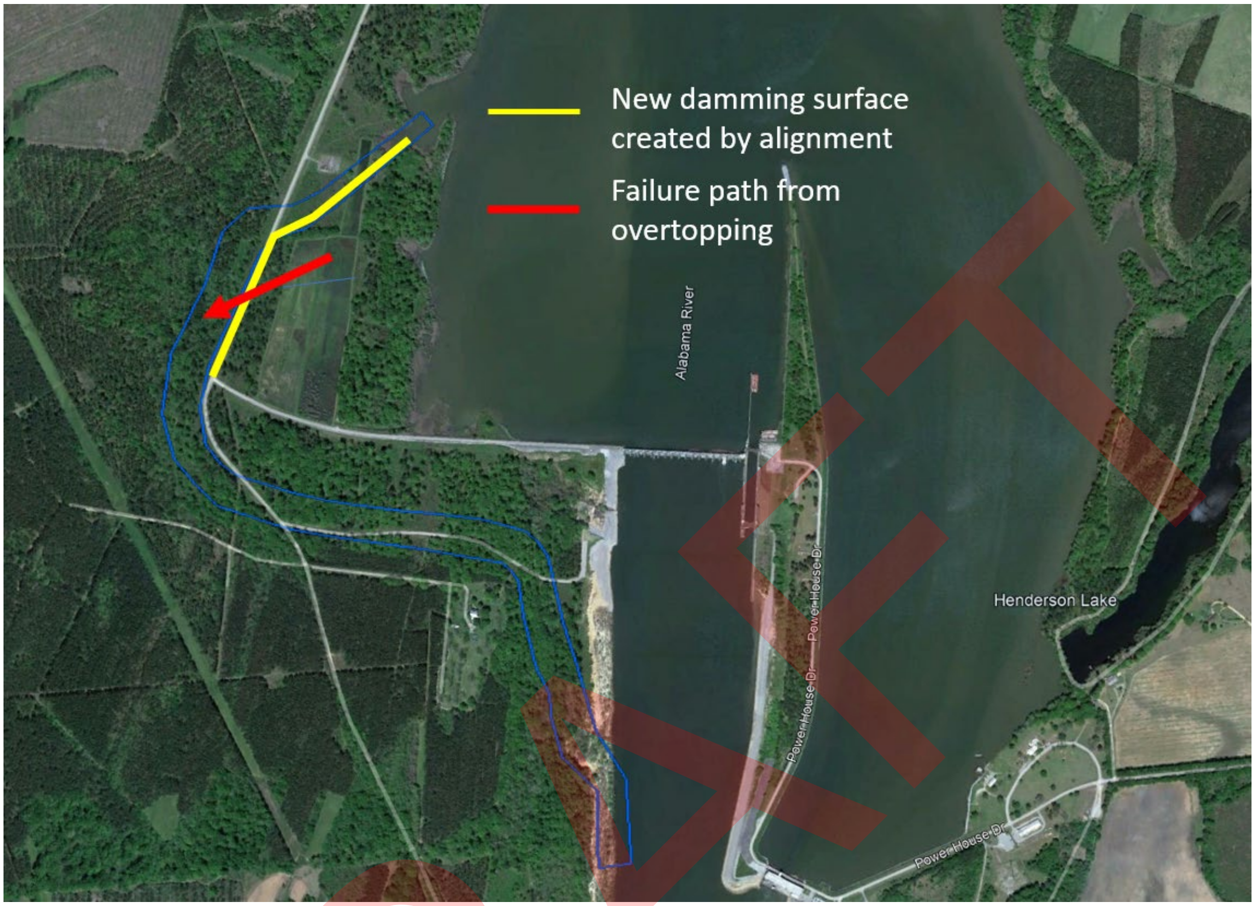



Figure 42 – Location of where new damming surface is needed with alignment.

7.1.2 Millers Ferry Excluded PFMs:
Justification is included why these PFMs are not considered risk drivers for the project based on the current design detail, hydrologic, and geologic information. Minor recommendations have resulted from discussing several excluded PFMs.

**PFM 01: Overtopping of the Fish Bypass Gates**

The current design elevation for the top of the tainter gates is El. 85 ft. The top of the dam embankment is also at El. 85 ft. The proposed gate structure matches the adjacent embankment and therefore it would not be a reduction in the AEP of overtopping. The gate area will be less erodible than the earthen embankment (overflow dike) and therefore would not increase the risk of the overtopping compared to that of the embankment and which was already assessed in the 2022 Periodic Assessment. For these reasons, this PFM was excluded as a risk driver for the proposed fish ladder project.

**PFM 02: CLE adjacent to new structure**

The new control structure alignment is conceptually designed to run through the dam embankment. This design introduces a new discontinuity through the embankment and if not designed and constructed properly, could potentially result in flaws through the embankment, adjacent to the concrete that would allow for preferential seepage.

The embankment has been surcharging the foundation for 50 years reducing the potential for any future settlement or differential settlement. There are two alternatives that are being considered, a slab on grade and a pile founded alternative. The slab size will result in an estimate approximately 1,500 pounds per foot (psf) foundation pressures. Any settlement that occurs after backfilling would be differential to the dam; however, the team considered there to be relatively low potential for differential settlement between the structure and the embankment. Since the embankment has been surcharging the footprint of the new structure since construction in the 1970's, there is little potential that the new structure would increase the effective stress in the foundation under new gates.

The preliminary drawings show vertical sides to the structure which can preclude intimate contact between the clay backfill and the concrete and the footing is sown as a 90-degree connection. This could potentially result in a zone of low stress along the structure and a preferential seepage path shown in Figure 43. Figure 44 shows the location where the flaw could potentially occur from poor compaction adjacent to the structure. The current conceptual drawings also do not show any type of filter for this structure. This leads to conditions where Concentrated Leak Erosion (CLE) would be more likely.

The team also discussed the potential for CLE to occur under the structure. If the structure is founded on piles, any post construction settlement could cause the soil to settle away from the pile founded structure, leaving a gap between the structure and the soil. This would not occur on the alternative where the structure is founded on soil. The team understands that a cutoff wall is being considered under the footprint of the structure.

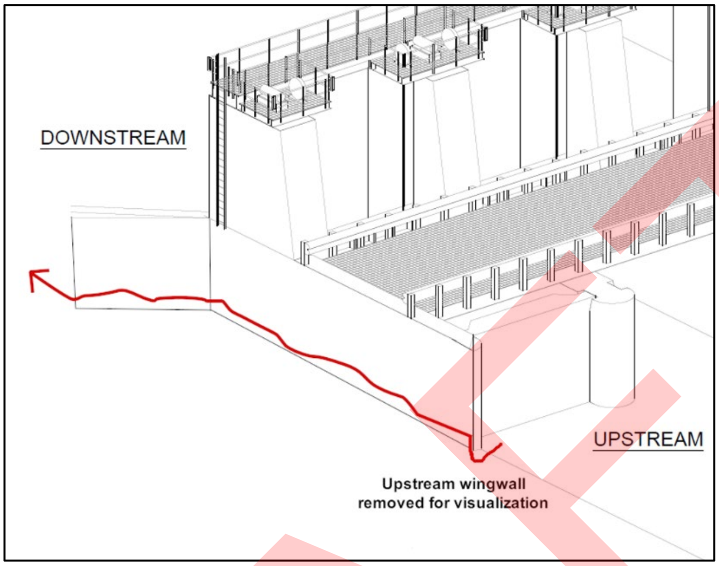



Figure 43 – Failure path along the gated structure.

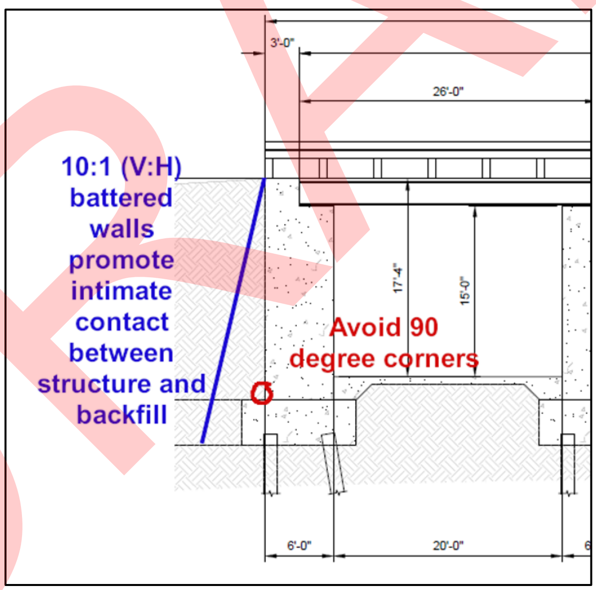



Figure 44 – Potential for compaction concerns along 90-degree corners.



Several recommendations were provided in the Findings and Recommendations Section of this report that are considered standard practice and are expected to be captured in the design during PED. This PFM was excluded as a likely risk driver with the expectation that these standard features would be added. It is recommended to modify the design to 10V:1H battered side walls and any 90-degree corners be chamfered to fill in those corners where compaction will be difficult. The team also recommends that a granular filter be included on the downstream side of the structure to intercept and safely drain any seepage that initiates around the structure. Use EM 1110-2-2902 for design of conduit to include a landside filter relief drain (p. 138 in EM). The critical failure path will occur at the bottom corner of the interface, and it is recommended to avoid 90-degree corners. A properly designed filter downstream will further reduce the likelihood for unfiltered seepage along the new structure. Use of a sheetpile cutoff under the structure should also be included in the design if the structure is founded on plies.

Implementation of these recommendations fully addresses this PFM and would exclude it as a potential risk driver. Failure to add these modifications should require further detailed assessment of this PFM as the design progresses.

**PFM 03: Gate failure causes uncontrolled releases**

This potential failure mode was considered a contributing factor to PFM 05: Erosion of the channel due to excess velocities. This is a highly regulated dam where personnel are generally staffed in the powerhouse. If needed, operations personal could be deployed in the case of a boat strike, debris, or regular failure of the gates. Floating boom logs are expected to be put in to discourage boats. There is backup power on site or anticipated in the design along with stop logs. In the event that a gate was to fail in an open position, flow would be throttled and not sufficient to cause downstream damages; therefore, this PFM was excluded as a risk driver for the project.

**PFM 04: Global stability of the gate structure**

This failure mode could not be fully evaluated due to the conceptual level of design in feasibility phase. The considerations for PED will include differential settlement, overturning, and sliding. Sliding will be limited with the proposed sheet pile cutoff wall on the upstream portion of the structure. Overturning of the gate structure with a maximum head differential of 15 feet is unlikely but must be evaluated. See PFM 02 for contributing factors that lead to possible outcomes for settlement. It is recommended that this PFM be further evaluated in PED; however, the team saw no apparent cost driving features that would need to be added to address this PFM.

**PFM 05: Erosion of the channel due to excess velocities**

Velocities in the fish ladder channel initiates erosion that head cuts back towards the dam and undermines the gate structure, causing overturing or large slope failure, collapse and uncontrolled release of water (breach). The design has not progressed past evaluating total capacity and average velocities at this point in the study. When the design progresses to PED, guidance such as EM 1110-2-1601 Hydraulic Design of Flood Control Channels will be incorporated to ensure the materials used will be non-erodible at channel surface.

The team recommends hydraulic modeling to understand the velocities anticipated at sunny day and less frequent event flows. Special care should be taken with the turns in the channel alignment, as outside corners may experience much higher velocities and the freeboard may be less in these areas. Average velocities within the channel have been modeled at 5 to 6 fps. Table 2.5 from EM 1601 in Figure 45 shows maximum permissible mean channel velocities for clay is 6 fps. Max velocity will likely occur at each step down in the channel where the steps each drop one foot in elevation causing a small plunge pool at each drop structure. The preliminary channel design includes a 4-ft layer of riprap protection on the invert of the channel, separating the clay soils from the potentially erosive channel velocities. Therefore, this PFM was excluded; however, this PFM will need to be further reviewed in PED after riprap sizing is complete and more advanced hydraulic modeling is complete to predict maximum velocities in all areas, including corners where outside river bends will experience faster flow.

Transitioning from headwater to tailwater may also cause eddying and turbulence with enough hydraulic shear stress to initiate undercutting. The interface between the concrete structure and channel would be a critical location which could lead to undercutting and destabilization of the control structure. A recommendation to add a sill extending downward and out horizontally or a sheet pile key on the downstream side tied into the structure could mitigate head cutting to reduce the likelihood for global instability.

**Table 2-5**
**Suggested Maximum Permissible Mean Channel Velocities**

| Channel Material | Mean Channel Velocity, fps |
|---|---|
| Fine Sand | 2.0 |
| Coarse Sand | 4.0 |
| Fine Gravel[1] | 6.0 |
| Earth | |
| Sandy Silt | 2.0 |
| Silt Clay | 3.5 |
| Clay | 6.0 |
| Grass-lined Earth (slopes less than 5%)[2] | |
| Bermuda Grass | |
| Sandy Silt | 6.0 |
| Silt Clay | 8.0 |
| Kentucky Blue Grass | |
| Sandy Silt | 5.0 |
| Silt Clay | 7.0 |
| Poor Rock (usually sedimentary) | 10.0 |
| Soft Sandstone | 8.0 |
| Soft Shale | 3.5 |
| Good Rock (usually igneous or hard metamorphic) | 20.0 |

Notes:
1. For particles larger than fine gravel (about 20 millimetres (mm) = 3/4 in.), see Plates 29 and 30.
2. Keep velocities less than 5.0 fps unless good cover and proper maintenance can be obtained.

Figure 45 – Table 2.5 From EM 1601 suggested channel velocities.

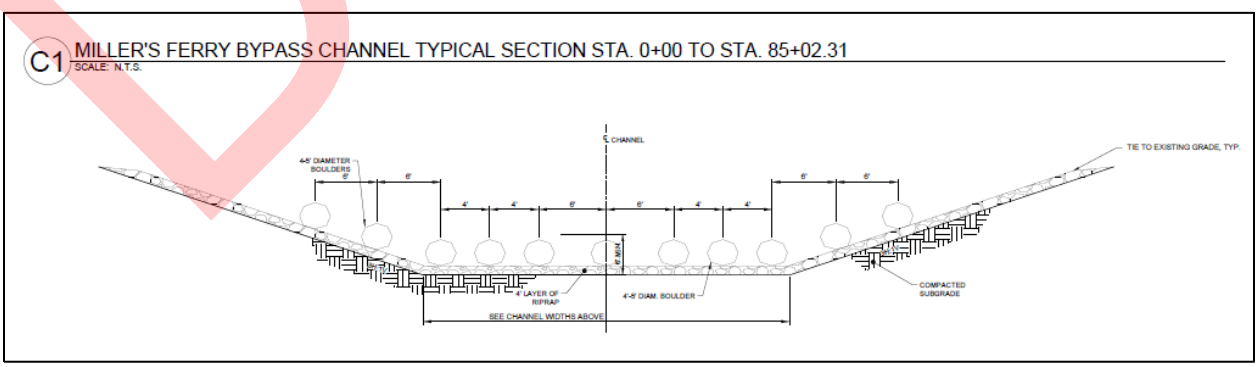



Figure 46 – Typical cross section of fish ladder bypass.

**PFM 06: BEP through foundation into channel.**

There are two sources of head differential and therefore two potential BEP failure paths associated with BEP: one from the upstream pool to the fish bypass and the other from the fish bypass to the downstream pool. The path from dam to bypass channel is the only one that would progress to breach of the dam and is therefore the focus of this PFM. The invert elevation of the fish ladder channel drops at a 2% slope from the dam to the connection to tailwater. The critical location would be just before the turn in the channel where the exit gradients would reach a maximum. This path is measured at length of approximately 730 ft shown in Figure 47. At this location the bypass invert is at around El. 46 ft. The fish ladder channel is designed for a depth of 4-5 ft, resulting in a bypass channel stage of approximately El. 50 ft. The normal pool of the reservoir is 81 ft resulting in a head differential of 31 ft between the upstream and downstream fish ladder channel and average gradient of 0.04. Natural groundwater is between El. 45-50 ft and dewatering would be needed during construction.

The other possible location where the BEP failure path could occur is between the new fish ladder channel and the downstream pool running west to east horizontally shown by the second red line in Figure 48 with an approximate failure path length of 425 ft. While this failure path is shorter in distance, this failure path would not result in the failure of the damming surface and was considered a maintenance issue.

None of the original field investigations encountered sand in this part of the dam foundation; therefore, a continuous sand layer is unlikely; therefore, this PFM was excluded due to the lack of continuous sand and the very low average gradients. However, to be fully evaluated it is recommended to complete borings along the alignment to verify subsurface conditions. If a continuous sand layer is observed, a filter should be incorporated into the design. With the application of this recommendation across clean sand layers exposed during construction, the PFM would be excluded.

Lastly, Figure 48 shows a cross section through the downstream of the dam in the critical location for this PFM. The shallow wetland also exists on the downstream side of the dam in this area. Gradients into the wetland could be higher because it is closer to the dam. Assuming the wetland is near dry with water near the invert at El. 60 ft, the head differential is about 21 ft. However, the wetland is closer starting right at the embankment toe, this results in average gradients around 0.2 assuming a 100 ft seepage path. If a sand layer exists that is exposed by the wetland, these gradients are sufficiently high enough that they should have initiated BEP in the past. Given there is no evidence of BEP form past loading, this further supports exclusion of this PFM.


Figure 47 – Failure path for BEP between dam embankment and fish ladder channel.

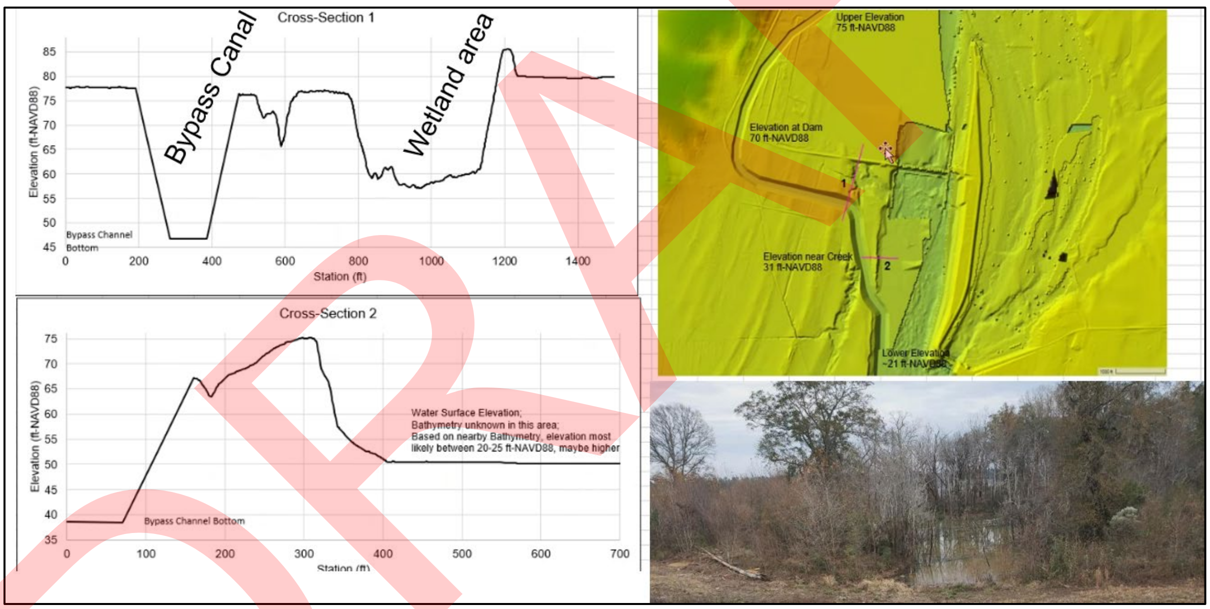



Figure 48 – Cross section through wetland.

**PFM 08: Sediment buildup in channel cause out of bank flow.**

This PFM was judged to be less likely to initiate than the debris blockage and would result in a maintenance issue. Sediments transported from upstream could build up with time and reduces the flow capacity. This could potentially reduce the effectiveness of the project to allow fish to jump the steps. Sedimentation buildup would likely be a slow developing process and would not result in failure of the dam. It was noted by the design team that vegetation would be a more likely concern rather than sediment buildup. This PFM was ultimately excluded as a risk driver because it would likely not result in failure of the dam.

**PFM 09: Artesian condition uncovered during excavation causing contact erosion progression towards dam.**

Artesian conditions may be present at approximately El. -35 ft where the sandstone aquifer exists. Original construction of the dam encountered bedrock at approximately El.-20 ft. Millers Ferry foundation report documents that there were not instances of water emerging from the rock at foundation grades. Water that was coming through the fractures or bedding planes were controlled by trenches and the use of sandbag berms. This was excluded as a risk driver based on the preliminary excavation depths of the bypass channel which will be well above the area for potential artesian head conditions. This PFM should be further reviewed after soil borings are completed and groundwater conditions and potential artesian depths are further tested.

**PFM 10: Cofferdam removal during construction cause defect in the damming surface**

The current design indicates cofferdams are planned at the upstream and downstream entrances to the channel alignment, as well as just upstream of the gated fish ladder structure. Sheet pile cofferdams can leave a transverse defect within the dam during removal. If a sheet pile cofferdam is used upstream to downstream, pulling the sheet piles can leave a preferential seepage path in the clay fill that may progress to CLE over time. It is recommended that use of sheet piles be avoided in a transverse direction to the damming surface, but if it is required, abandonment by cutting off and abandoning in place may be preferential. This feasibility design includes cellular cofferdams that do not cross transvers to the embankment; therefore, this PFM was excluded at this phase of design. Figure 49 shows the conceptual cofferdam design.

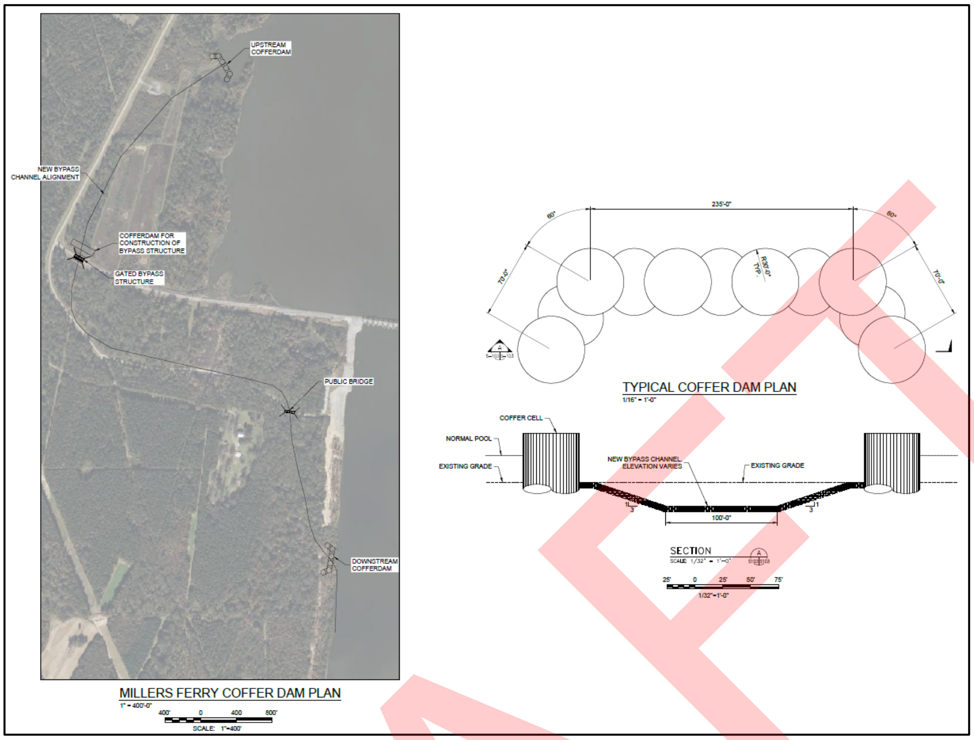



Figure 49 – Typical Cofferdam design plan.

**PFM 11: CLE into gate structure**

Figure 50 shows the CLE failure path for this PFM that could potentially occur by initiating through an open joint in the gated structure, when the gates are closed and there is a head differential across the structure. This PFM could not be evaluated at this level of design; however, the design should include water stops at all cold joints and construction best practices should be monitored with good quality control. Additionally, as it pertains to the foundation, if differential settlement is not addressed there lies the potential for cracks to occur at joints or other places; however, given the dam foundation has been surcharged by the existing fill, differential settlement is not expected to be significant.

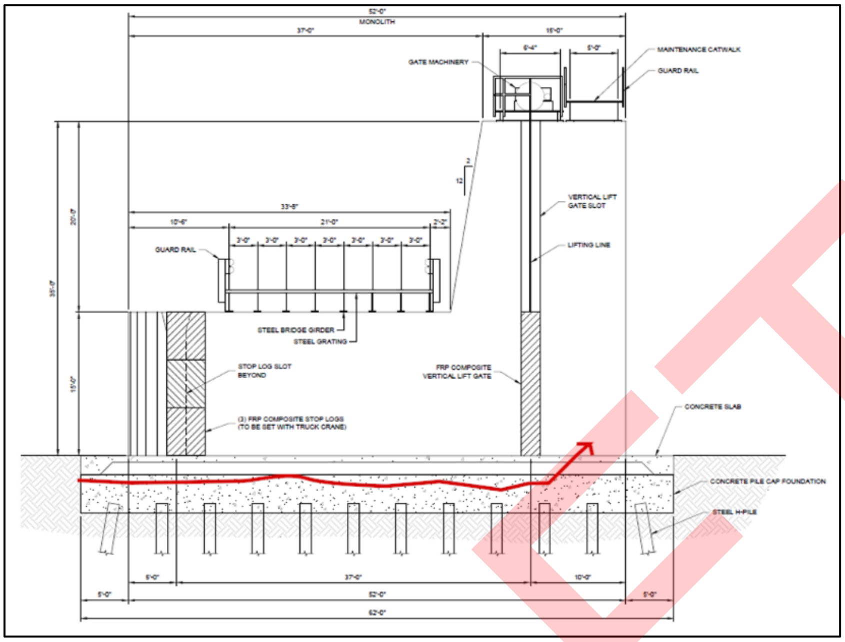



Figure 50 – Failure path for CLE between sheetpile and concrete cap.

**PFM 12: Confluence of three channels cause eddies to occur causing erosion of the banks.**

The fish ladder proposed alignments have a flow of 1,200 cfs exiting into the main river channel, creating a confluence of multiple channels which could result in bank erosion from the eddies, illustrated in Figure 51.

The Powerhouse located on the east side of the river has average discharge flows between 20,000 and 30,000 cfs. The powerhouse discharge capacity is 34,000 cfs with 3 units running. The flow of the fish ladder channels is so much smaller than the powerhouse discharge flows into the river. The east bank downstream of the powerhouse has existing riprap armor for scour protection from powerhouse releases. The normal operations flow through the powerhouse and the spillway is only used during times of high pool; therefore, the majority of the time there will only be confluence of two channels, the fish ladder and the powerhouse. The flow of the fish ladder channel is insignificant compared to the flow generated downstream from the powerhouse. For this PFM to get to failure, the turbulence must be large enough to erode the armoring on the eastern bank. When the spillway does overtop, the powerhouse discharge would still drive the design. This PFM was excluded as a risk driver because the river downstream of the dam is already designed for high velocities from the powerhouse.

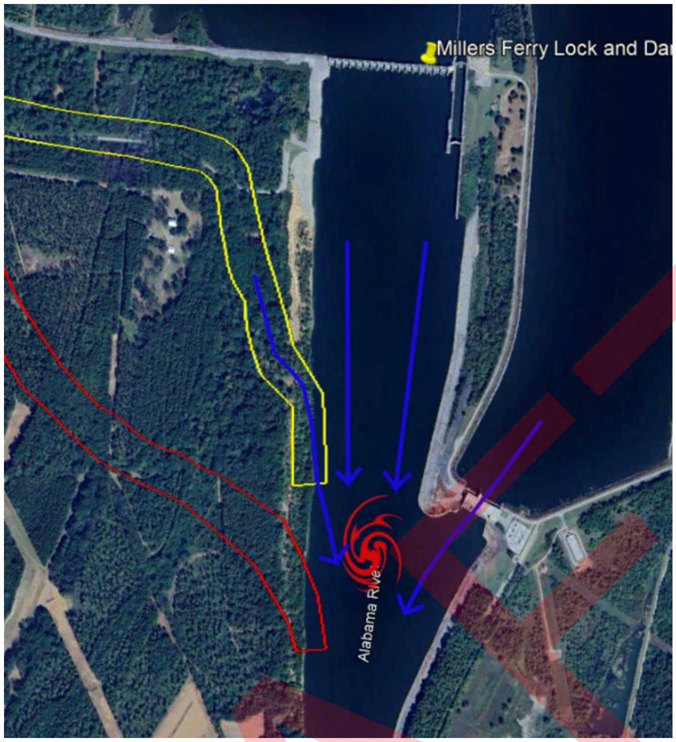



Figure 51 – Confluence of multiple channels resulting in eddies.

**PFM 13: Access restriction caused by channel prevents access to dam.**

This PFM results in the inability to intervene with flood flighting if a PFM were to initiate. The fish ladder channel includes multiple bridges to enable access to areas that would otherwise be isolated by this project shown in Figure 52, excluding this PFM as a risk driver for the project. It is recommended that the bridges be designed to a sufficient level that allows trucks, mobile cranes, and other flood fighting equipment to access the dam.

Figure 52 – Proposed fish ladder alignments through current access road.

**PFM 14: Slope instability due to new channel adjacent to existing embankment**

The PFM explored the potential for the new channel to cause a deep-seated failure plane which causes instability of the dam's embankment. Figure 53 shows the fish bypass channel profile comparing the bypass channel bottom elevation to the elevation of the adjacent wetlands and dam embankment. The PDT has run preliminary design analysis which include seepage and slope stability with the embankment adjacent to the canal and has found it to meet appropriate design factors of safety. These analyses will be revisited during PED to update with additional boring log information. The geology present with an approximate top of rock elevation of 40 ft discourages a deep-seated failure. Furthermore, the adjacent wetlands area has a similar elevation to the channel; however, is much closer to the embankment slope. Therefore, this PFM is excluded.

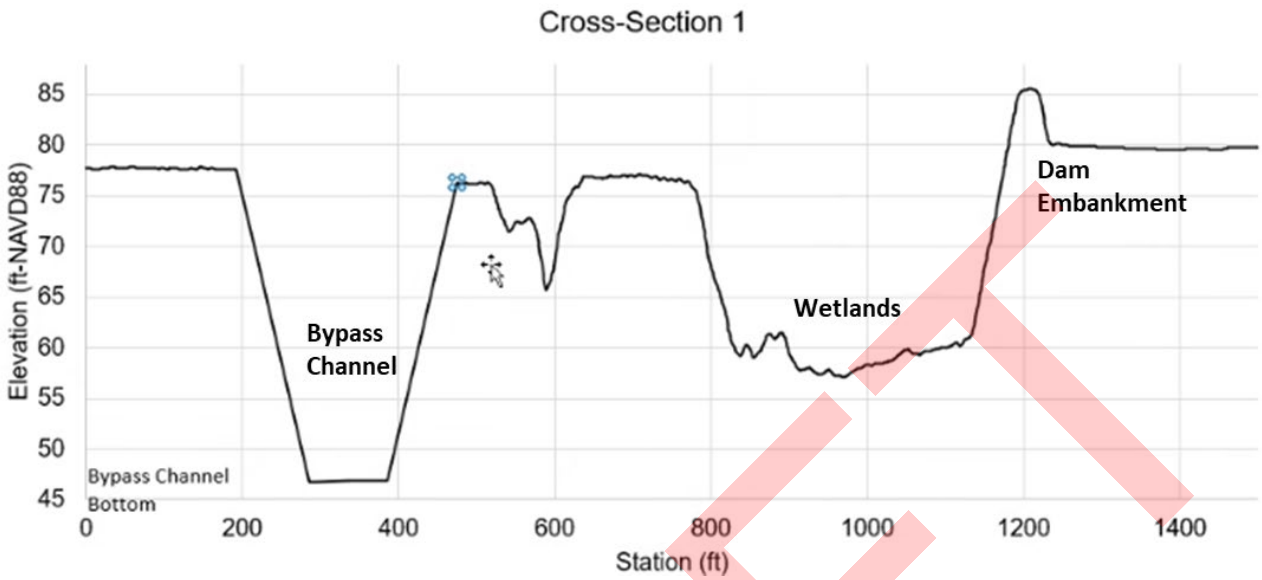



Figure 53 – Profile along the proposed project alignment.

## 7.2 Claiborne L&D Potential Failure Mode Analysis (Right Abutment)

The potential failure modes identified and analyzed during the PFMA for the Claiborne L&D are summarized below in Table 9. Following each heading is a complete description of the potential failure mode, the adverse and favorable factors identified during the session, and assessment of harm to the project. Where necessary, recommendations were identified to further reduce risk and modify the project such that the do no harm concept is not violated by the project. Ten (10) PFMs were identified by the team for consideration two (2) PFMs identified as likely risk drivers for the project bolded below. While the remaining PFMs were excluded as likely risk drivers, minor recommendations resulted from several of the PFMs discussed.

Table 9 - Potential Failure Mode (PFM) Summary for Claiborne L&D.

| PFM # | PFM Description |
|---|---|
| 01 | Overtopping |
| **02** | **Flow over the island cuts new channel bypassing the control weir** |
| 03 | Scour erosion in channel (head cutting, right abutment failure, removal of pool weirs) |
| 04 | Rapid drawdown leads to slope instability |
| **05** | **BEP into channel along the right abutment** |
| 06 | Suffusion from artesian pressures |
| 07 | Confluence of channels causes eddies and erodes the banks |
| 08 | Sediment or debris in fish ladder channel causes water to go out of bank flow |
| 09 | CLE along channel weir |
| 10 | High water during construction causes scour before erosion protection is in place |

7.2.1 Likely Risk Driver PFMs:

These PFMs are considered likely risk drivers for the proposed project based on the available information and design details. These PFMs include recommendations to further reduce risk associated with these PFMs that may affect project costs.

**PFM 02: Flow over the island cuts new channel bypassing control weir**

Description: An overtopping event (approx. 1/3 AEP) inundates the island between the river and the fish ladder bypass. As the flood recedes, water channelizes around the right abutment causing scour erosion. The rate of head cutting is the same or faster than the receding rate of the river. This forms a new channel around the right abutment and breach of the dam. This PFM could initiate anywhere between the right abutment and the entrance weir or even upstream of the weir since the weir does not tie into high ground. Two potential failure paths are shown by the red arrows in Figure 54.

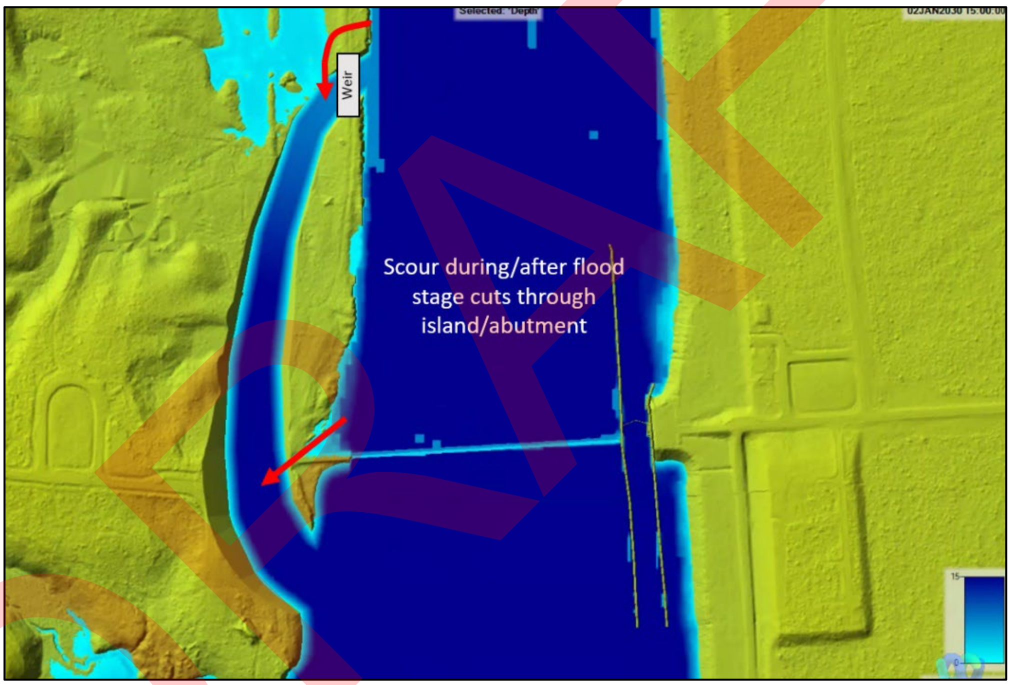



Figure 54 – Scour during overtopping event creates channel through island/abutment.

The elevation of the island at the right abutment is approx. El. 39 ft. This elevation will be inundated annually. Borings near the right abutment found the surficial material to be granular soil (classified as sand and gravel). There are less erodible layers of claystone and sandstone encountered deeper in the foundation. There are no borings in the area identified for this PFM or along the bypass alignment; however, it is judged that there are likely highly erodible materials based on the available borings in this area and knowledge that the surficial material around the site is soil and not rock.

The 1/3 AEP event predicts velocities on the order of 7 to 8 fps which would be erosive to all soil classifications. Top of rock on the right abutment cross section is at about El. -

20 ft Mean Sea Level (MSL). The failure path to make connection to the river is only about 75 ft. Based on the factors above, this PFM was judged a risk driver for the project that is not addressed by the current conceptual design.

It is recommended that that the land mass between the right abutment of the existing dam and the entrance structure be designed as a damming surface including overtopping protection. Adding these features could add substantial cost to the project.

**PFM 05: BEP into the channel along the right abutment**

Description: The falling elevation of the bypass channel results in a head differential between the water behind the main dam and the water elevation in the bypass channel. A continuous erodible sand is exposed by the channel excavation with horizontal seepage exit into the new cut. Gradients are high enough to initiate erosion at the unfiltered exit and piping begins to progress back towards the Alabama River. Clay or clayey sand holds a roof for mechanical progression. There is sufficient flow in the developing pipe for progression and the pipe progresses to a connection with the river, leading to collapse of the embankment. This allows for overtopping to progress the breach. Figure 55 shows a LiDAR cross section of existing topography with the proposed channel added. The failure path is shown on this figure along with the expected geology based on the limited boring data (from boring 149).

The most likely location selected for this PFM is directly adjacent to the dam. At this location, the highest head differential will exist, where the headwater is controlled by the dam while the fish channel is dropping in elevation around the edge of the dam. Additionally, there is an existing sheetpile cutoff wall under the structure in the right abutment that will back up seepage and force seepage to discharge to the west around the side of the wall, and into the channel. Figure 56 illustrates this concept and the potential for more concentrated seepage that could exist around the edge of the existing sheetpile cutoff wall.

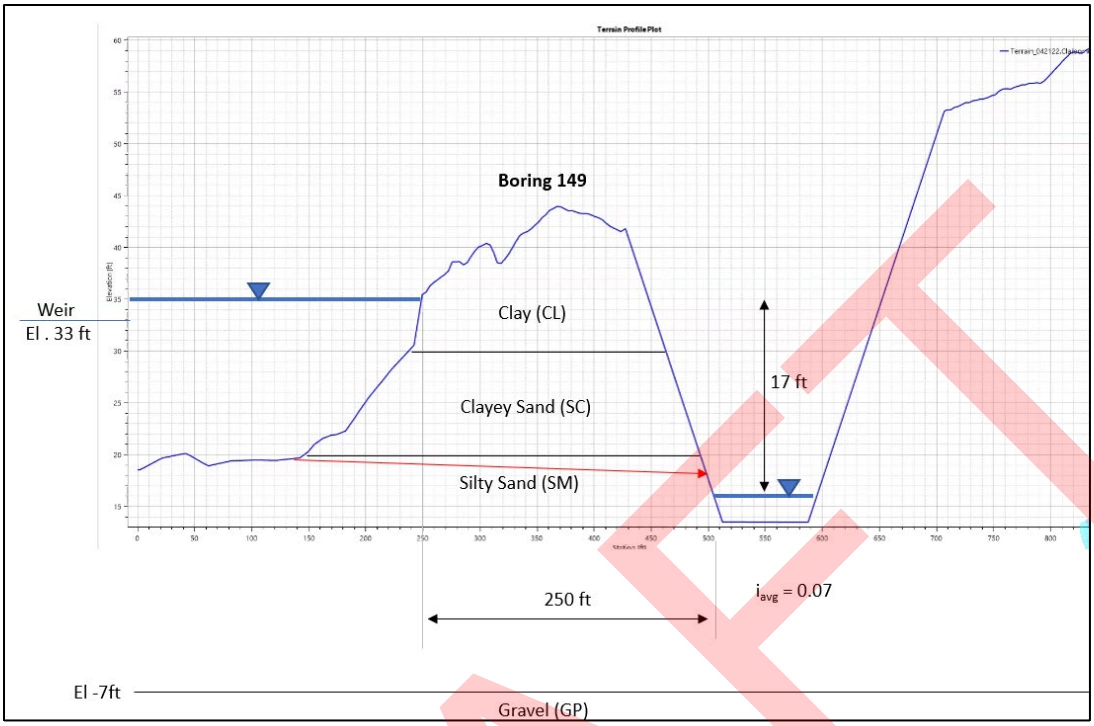



Figure 55 – Lidar cross-section with hydraulic head condition and subsurface conditions at the PMF location.

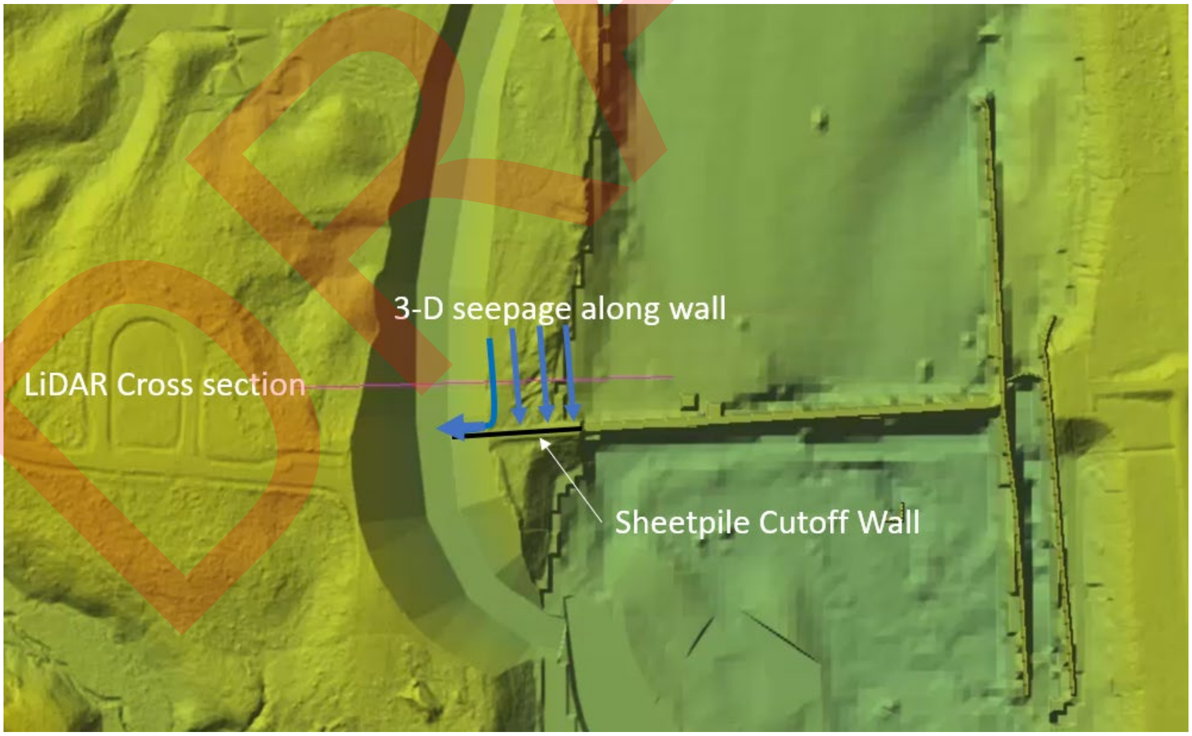



Figure 56 - LiDAR topography with location of cross section cut for PFM.

The current alignment would result in an approximate 250 ft seepage path with approximate 17 ft head differential on a normal sunny day condition (average gradient of 0.07). Less frequent events would cause higher tailwater and would be less likely to initiate and pipe. There is only one soil boring available which shows a silty sand being present near the invert of the river. This allows hydraulic connection to the sand layer in the river and the fish ladder canal would cut into this sand allowing for potential horizontal exit of seepage through this layer. There are no fines content tests to support decisions on how erodible this material would be to piping, with an SM with 40% fines not being susceptible to piping, while a silty sand SM with 15% fines could be susceptible at this gradient. Additionally, the presence of the existing cutoff wall under the dam will cause seepage to concentrate and flow laterally around the end of the wall, increasing the likelihood of initiation and progression at this location. The CL materials above the sand layer are very likely to provide roof support. Schmertmann analysis making gross assumptions shows that if a low Uniformity Coefficient (Cu) sand were to exist in this area, progression probability would be on the order of about 3E-02. Given the AEP of the event is 1 (occurring annually and existing for long sustained periods of time), roof support exists, sands are likely continuous across this width, then the APF for the scenario documented here will likely be controlled by the progression node of the event tree. Based on these factors, this PFM is judged to be a potential risk driver if an erodible sand layer is confirmed along the fish ladder alignment.

To address this PFM in feasibility, it is recommended the team assume that a sand layer exists and include potentially costly measures in the plan to address the PFM (i.e., cutoff wall or filtered exit) or complete field exploration and laboratory program to verify the materials that will be encountered in the fish ladder channel excavation and confirm the erodibility of these materials.

Like PFM 02 above, the land mass between the existing right abutment of the dam and the new bypass channel entrance gate must be designed as part of the damming surface since breach of this land mass would allow flow to pass around the dam. The addition of such features to address overtopping and potential seepage related PFMs could be costly and may result in another alternative being more efficient. The team brainstormed measures that could be added to address these PFMs.

<u>7.2.2 Measures to Mitigate Claiborne Driver PFMs</u>

By adding a channel weir approximately 1,500 upstream of the existing dam structure, the island created between the channel and rive becomes the damming surface as well as any additional tie-in to high ground from the northwest end of the weir. As such, the dry island needs to be designed as a damming surface with regards to overtopping, seepage, etc. A couple measures were brainstormed by the team may help to mitigate PFM 02: Flow over the island cuts new channel bypassing the control weir; and PFM 05: BEP into the channel along right abutment.

The first alternative would construct the crest of the new embankment in-line with the existing dam and extend the fish passage channel further to the south shown in Figure

57. This would move the entrance gate of the bypass channel to be in line with the dam and eliminate the land mass between the gate and existing right abutment. This would be a similar design to Millers Ferry fish ladder bypass.

Figure 57 – Alternative to construct channel in line with crest and extend further south.

The second measure identified would construct a floodwall along the new right abutment damming surface to reinforce the island and connect to higher ground shown in Figure 58. The proposed alignment of the floodwall would include a cutoff to address BEP.

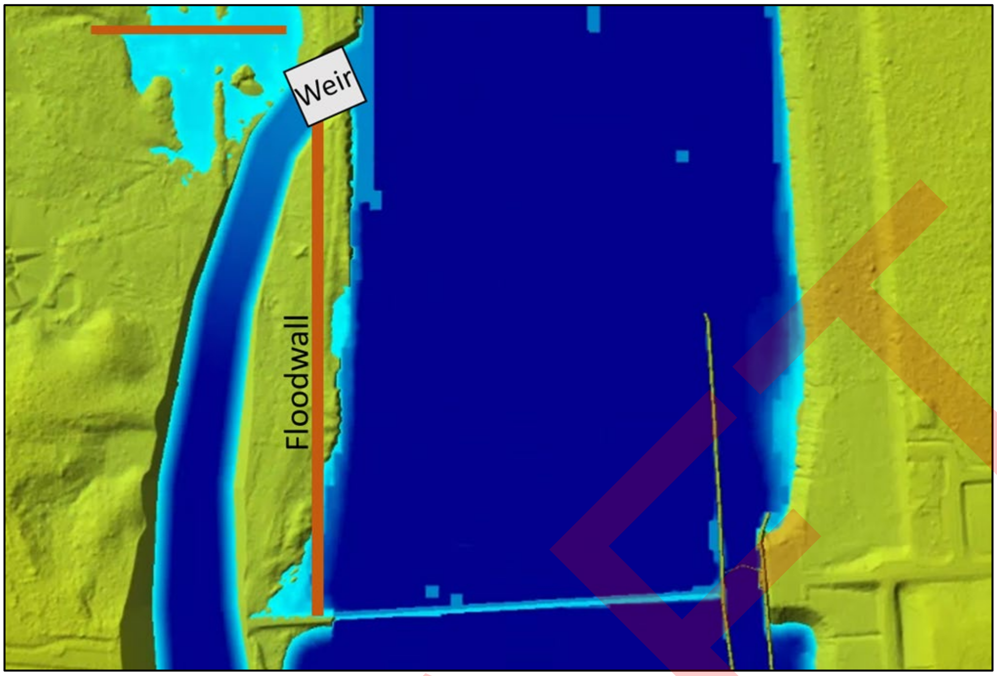



Figure 58 – Alternative to construct a floodwall and connect to higher ground.

### 7.2.3 Excluded PFMs:

Justification is included why these PFMs are not considered risk drivers for the project based on the current design detail, hydrologic, and geologic information. Minor recommendations have resulted from discussing several excluded PFMs.

**PFM 01: Overtopping**

The Claiborne Dam is designed to overtop and overtops most years as seen in Figure 59 from google earth historical imagery. The project is primarily for navigational purposes and there are no flood risk management benefits from the project.

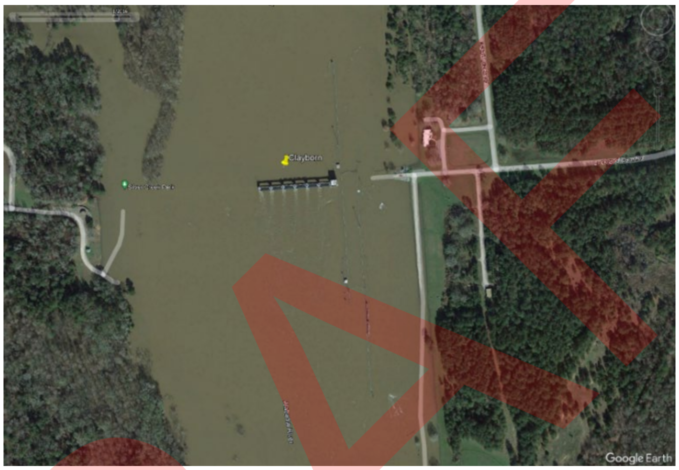



Figure 59 – Google Earth historical imagery showing overtopping of Claiborne L&D.

The upstream fixed crest weir is at elevation 33.1 ft, to match the elevation of the ungated spillway. Therefore, the construction of the fish passage channel will not increase the frequency of overtopping of the dam. Therefore, this PFM was excluded as a risk driver.

**PFM 03: Scour erosion in channel**

A sunny day event allows for velocities within the channel to exceed the design capacity, initiating scour of the channel. This failure mode has two potential paths that could lead to scour erosion of the channel. The first potential failure path occurs when scour progressing upstream in a head cutting manner that leads to the instability of the upstream weir, leading to uncontrolled flow. The second potential failure path is scour of the channel causing erosion to the point of instability of the existing right abutment, allowing for end around flows and breach.

Initial modeling indicates velocities more than 7 fps. This modeling was performed without the proposed drop structures which will reduce average velocity but may include higher velocities at the drops. The models indicate that the velocities reduce with flow, due to the rising tailwater elevation shown in Figure 60.

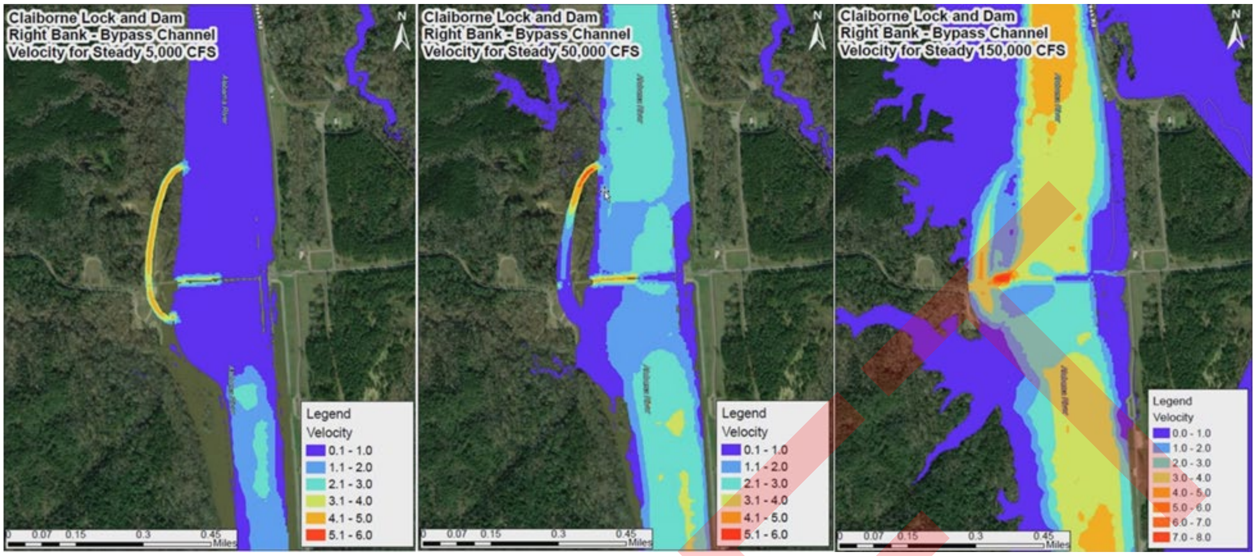


Figure 60 – Bypass Channel velocities exceed 7 fps.

The design team proposes 3-D hydraulic modeling or CFD modeling to better understand the flows within the channel as well as the interaction as the water meets the tailwater at different elevations to understand how the out of bank flow interacts with the channel. The design will include appropriately sized riprap or can use grouted riprap to create weir pools throughout the channel. Figure 61 shows a typical section that includes riprap protection along the channel invert. This PFM cannot be fully evaluated at this time without more suffocated hydraulic models to predict maximum velocities; however, it was excluded given that the feasibility design includes armoring along the channel invert to prevent scour erosion.

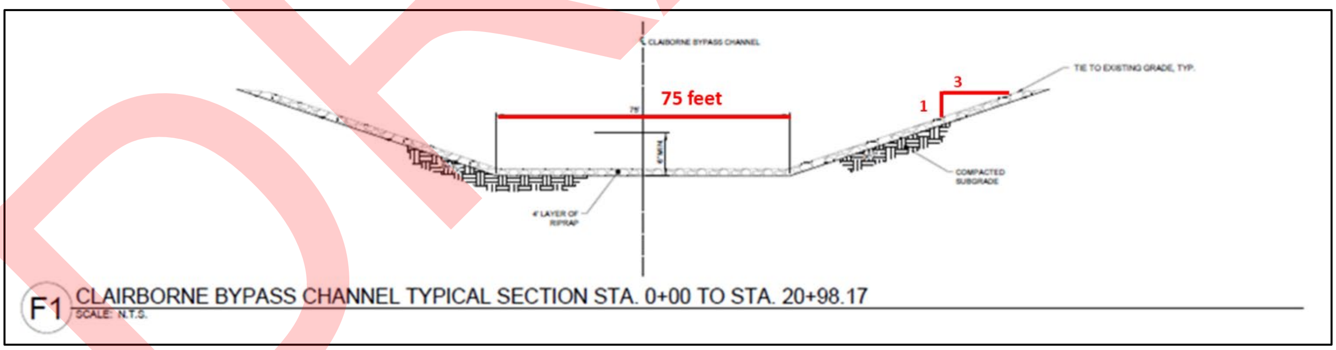


Figure 61 – Fish ladder channel typical section.

Because the project is not staffed, it is recommended that CCTV camera be added to existing monitoring system to inspect the channel for erosion during/after highwater events. Illegal use of the channel by ATV's, which have been known to frequent the area, may cause deviations to the channel's geometry which could potentially cause higher velocities.

Also, the current conceptual design does not include a structure that incorporates stop logs or gates. If erosion occurs that requires maintenance, the flow during normal pool would need to be diverted through other means such as a cofferdam. The team recommends the design incorporate a method to stop the flow of water through the channel, such as stop logs that can be installed in an emergency situation, or for maintenance. Maintenance within the channel would have challenges without a method to stop to the flow of water over the weir. It is anticipated that maintenance will be required at some point during the service life due to the expected annual overtopping events. The bottom width of the fish ladder channel is 75 ft shown by the channel cross-section in with 3:1 (Horizontal: Vertical) sides If stop logs are decided to be included, pillars would need to be constructed to install. Installing the stop logs may also pose a challenge during times of highwater. Installing the stop logs will require a crane and a crew to install them across the width of the weir surface.

A gated structure would also provide ability to maintain the channel but may come at a greater cost. The frequency of maintenance may dictate the cost effectiveness of the method for stopping the flow. If the frequency for maintenance is every year, a gated structure may be worth the investment. If the frequency for maintenance is every 5 years, then stop logs may be an effective solution.

**PFM 04: Rapid drawn down leads to slope instability**

A rapid drawdown after a highwater event results in receding water of approximately 3.5 feet per day (e.g., January 2016). As the water subsides, the earth embankment is slow to release the pore water pressure, resulting in instability of the slope. The most vulnerable slopes include the right abutment sliding into the channel or river, the upstream island failing, or the western slope of the channel failing into and blocking the channel illustrated in Figure 62. Higher banks on the eastern side of the fish ladder channel could make slope instability more likely where the proposed channel slopes are 3H:1V.

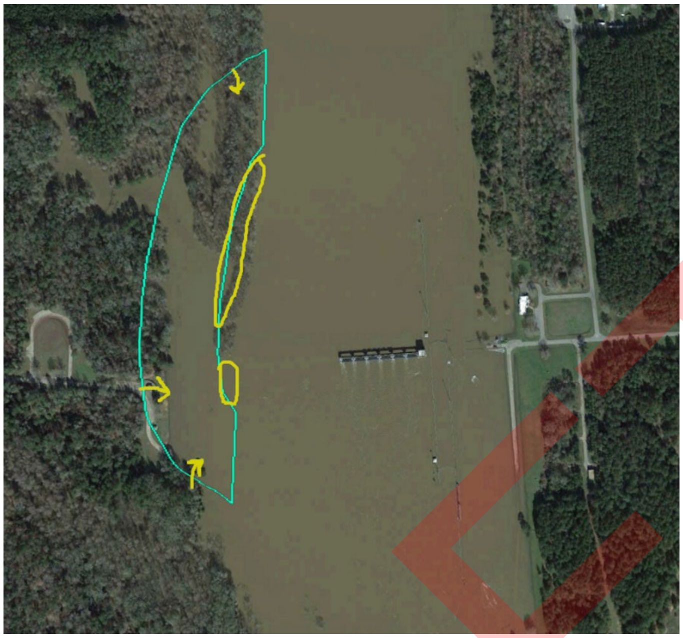


Figure 62 – Slope failure into channel with out of Bank flows.

There is significant uncertainty as to the existing materials along this alignment due to the lack of subsurface exploration in this area. The soil materials may potentially contain (GP) poorly graded gravel or (CL) clay of low plasticity, lean clean which could affect the slopes differently. If slope failure was to occur, the potential for intervention and detection would be unlikely. A slope failure into the channel could potentially release water through the island into the channel creating a partial or full blockage of the channel. This would result in out of bank flows or cause end-around flow around the right abutment of the current structure. Historical erosion on the right streambank downstream of the overflow dike has been observed during past storms.

The hydrograph in Figure 63 show the upstream pool in the reservoir during the 2015-2016 overtopping event. The water level rose from El. 35 to 54 feet and returned to El. 35 ft. It took 9 days to reach the peak of El. of 54 ft and another 10.5 days to go back to down to El. 35 ft. Another example overtopping event occurred in early 2000 between March-April shown by the hydrograph in Figure 64 and shows a 2 peak flood. The typical wet seasons occurs from January to March which can prolong to December or April some years. This coincides with the fish passage season occurring between January and April. The differential between upstream and downstream pools follows closely until restoring back to normal pool.

Historically at this project, this type of highwater event occurs more frequently than a 1/15 AEP event, and slope failures have not been observed along this bank. However, movement has been noted in the annual inspection where the riprap meets the grouted riprap shown in Figure 65 along the downstream stope adjacent to the right streambank; however, it is uncertain if the shelf observed in the photo is a separation from the grouted riprap or a sloughing of the slope. Further evaluation during PED is necessary to exclude this PFM, with the design recommendations above and in PFM 02.


Figure 63 – 2015-2016 Historical overtopping event.


Figure 64 – 2000 historical overtopping events in March and April.

Figure 65 - Sloughing where grouted riprap meats the riprap.

**PFM 06: Suffusion from artesian pressure**

Artesian pressures along the top of rock allow for flow into the invert of the channel, causing suffusion of a gap graded soil. This progresses due to the combined energy from the artesian flows as well as the velocities of water coming down the channel. This failure mode would present itself as a contributor to PFM 03.

The artesian zone noted in the foundation report (USACE, 1969) is described at approximate El. -40 ft under structures which caused a change in the foundation design for lock floor. This is not within the extents of this project and is unlikely to present itself with nearly 40 feet of rock and overburden between this zone and the lowest invert of the channel at El. 3.5 ft. Additionally, this PFM relies on gap graded non-cohesive soils which would be more erodible and unlikely to remain as the channel surface. For these reasons, this PFM was excluded as a risk driver.

The team recommends that construction require dewatering and monitoring of groundwater during excavation. If a layer of clean sand or gravel is identified, recommend further investigation, over excavation, and placement of a graded filter over the area of concern. Construction personnel should be trained to identify artesian conditions and report them as encountered.

**PFM 07: Confluence of channels causes eddies and erodes the banks**

The addition of the fish passage channel creates a confluence of multiple channels resulting in bank erosion from the resultant eddies shown in Figure 66. This PFM was excluded as a risk driver because the river downstream of the dam is already designed for high velocities that are experienced regularly. This PFM may present itself as a maintenance issue with the dredge spoil location adjacent to the canal. Additional modeling is recommended to identify and address any locations which may need armoring.

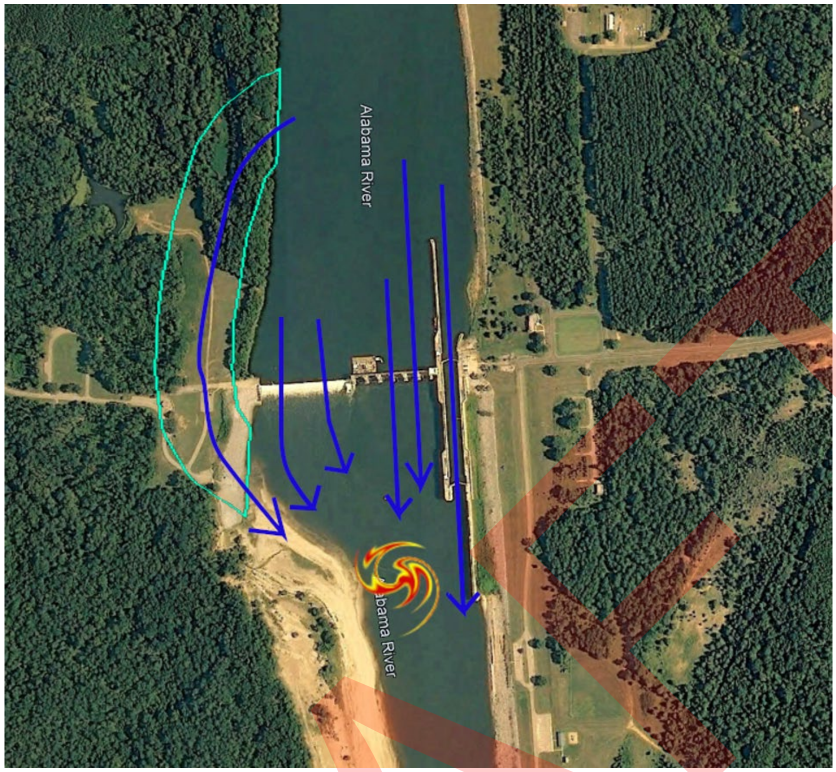



Figure 66 – Confluence of multiple channels causes eddies.

**PFM 08: Sediment or debris in fish ladder channel cause water to go out of bank flow**

Sediment or debris within the fish ladder channel may contribute to PFM 03 by increasing local velocities around an obstruction. However, if an obstruction causes partial or complete blockage, the elevation of water can only match the upstream pool. Backup flow from the channel cannot overtop the island unless the river flow is already at an elevation to overtop the island. Therefore, this PFM would only cause nuisance flooding adjacent to channel which may result in increased maintenance and is excluded as a risk driver for the project. Regular inspections should take place to ensure major obstructions are removed in a timely manner, reducing the potential for blockage.

**PFM 09: CLE along channel weir**

At this phase in the feasibility study, a weir design has not been completed. No evaluation during the risk assessment could be performed; however, the design should include water stops at all cold joints and construction best practices should be monitored with good quality control. Additionally, as it pertains to the foundation, if differential settlement is not addressed, there lies the potential for cracks at joints or other places in the structure. This may inform the foundation design type. Other PFMs around the structure, such as BEP through the foundation, should be evaluated after design details are available and granular filters should be included on the downstream side of any penetration through the damming surface.

**PFM 10: High water during construction causes scour before erosion protection is in place**

During construction, a high-water event occurs causing overtopping of the temporary cofferdams and/or the low point within the island between the channel and river. Bed scour of the channel after completion is reliant on rip rap and other materials to be placed controlling the velocities and erosion anticipated. Figure 67 shows a flood model with a 1/500 AEP event showing 4 to 7 fps in the channel. It should be noted that this model is smooth with a 1 percent slope; whereas the actual construction will be a series of steps that will have drop structures with additional turbulence introduced. These materials may not be in place when the high-water event occurs during construction. Erosion begins similar to an overtopping breach into the partially excavated channel. This bypasses the existing dam's spillway and will allow for uncontrolled release of water.

Staged phases of construction should be implemented to reduce the likelihood of this PFM. The island should be constructed to dam standards up to El. 40 ft along with the construction of the upstream weir, and stop logs installed or gates closed. The western edge of the channel weir is also tied into higher ground as a damming surface.

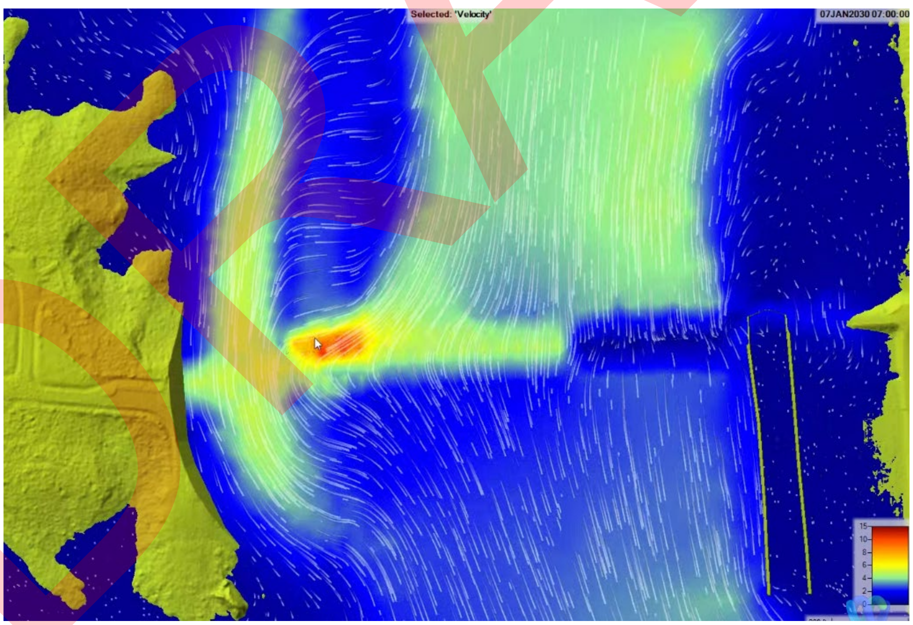


Figure 67 – Flow model shows potential for turbulence in the channel.

Construction of the channel will start from north to south and completed with armor in cells or sections no longer than 450 feet before starting the next cell, repeating until the channel is complete. This phased construction is illustrated in Figure 68. This may incur additional construction costs, especially if multiple mobilizations are required.

This sequence of construction allows for protective measures until the channel is complete and minimizes damage as well as the potential for this PFM to progress. There may be alternate methods of controlling this PFM that should be evaluated for dam safety during PED.

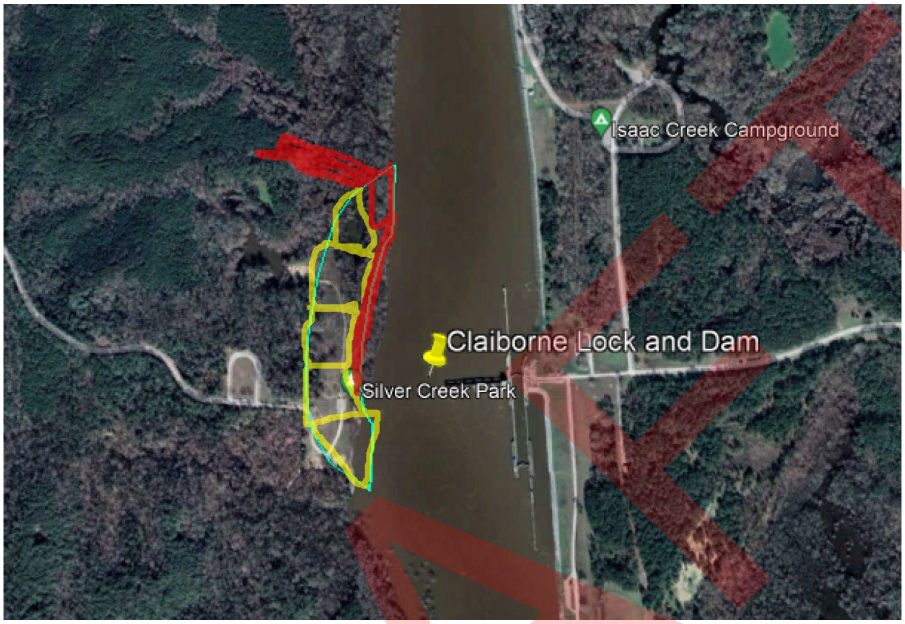



Figure 68 – Construction of the island broken down into phases.

### 7.3 Claiborne L&D Potential Failure Mode Analysis 2 (Spillway)

Due to some of the concerns and recommends associated with the TSP alignment discussed in the previous section, the PFMA team was also asked to assess the alternative that includes a fish ladder over the existing fixed crest spillway.

The potential failure modes identified and analyzed during the second PFMA for the Claiborne L&D are summarized below in Table 10 that focused on the rock arch weir shown by the plan view in Figure 69 and the cross-section in Figure 70. Eleven (11) PFMs were identified by the team for consideration. None of the PFMs were identified as risk driver leading to breach of the existing dam; however, 3 PFMs identified as likely risk drivers leading to loss of function of the fish ladder. These are shown bolded below. The PFMs bolded below resulted in the most critical recommendations for the project. While the remaining PFMs were excluded as likely risk drivers, minor recommendations resulted from several of the PFMs discussed.

Table 10 - Potential Failure Mode (PFM) Summary Part 2 Claiborne L&D.

| PFM # | PFM Description |
|---|---|
| 01 | Overtopping frequency |
| **02** | **Undermining the new structure from spillway flow** |
| 03 | A change in hydrodynamics from the fish ladder creates scour and undermining of existing structure |
| 04 | Water infiltration at interface of new structure and existing |
| 05 | River flow diverted to unmodified section of existing fixed crest weir causing downstream scour |
| **06** | **Loss of function due to settlement** |
| 07 | Cracking of existing dam due to settlement |
| 08 | Internal erosion under dam in unwatered condition |
| 09 | Dam overturning due to unbalanced fill during construction |
| **10** | **Sedimentation and debris in structure causes out of channel flow** |
| 11 | Cofferdam overtops during construction causing scour that leads to failure of existing dam |

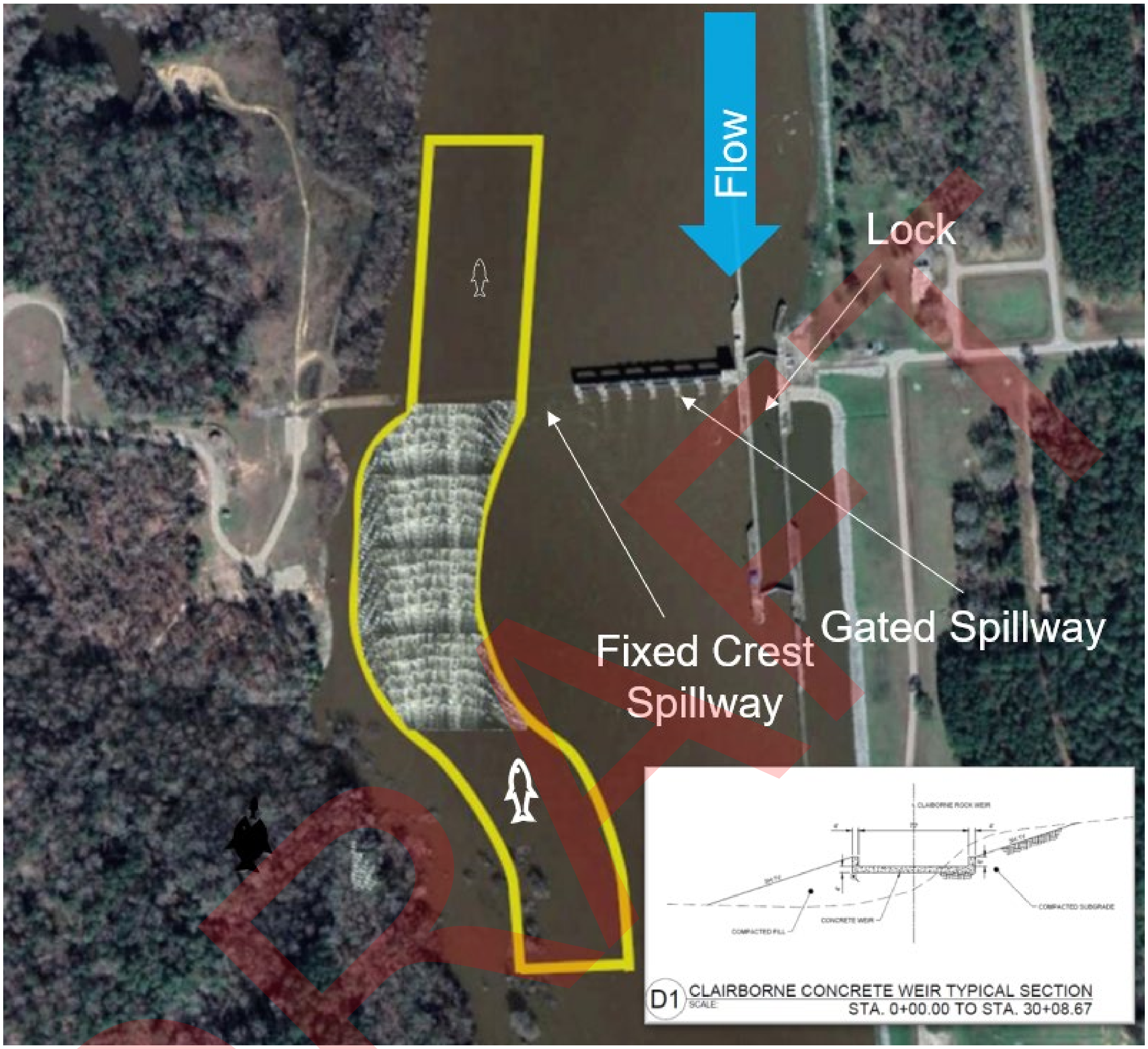



Figure 69 – Claiborne fish ladder alternative over fixed crest weir.

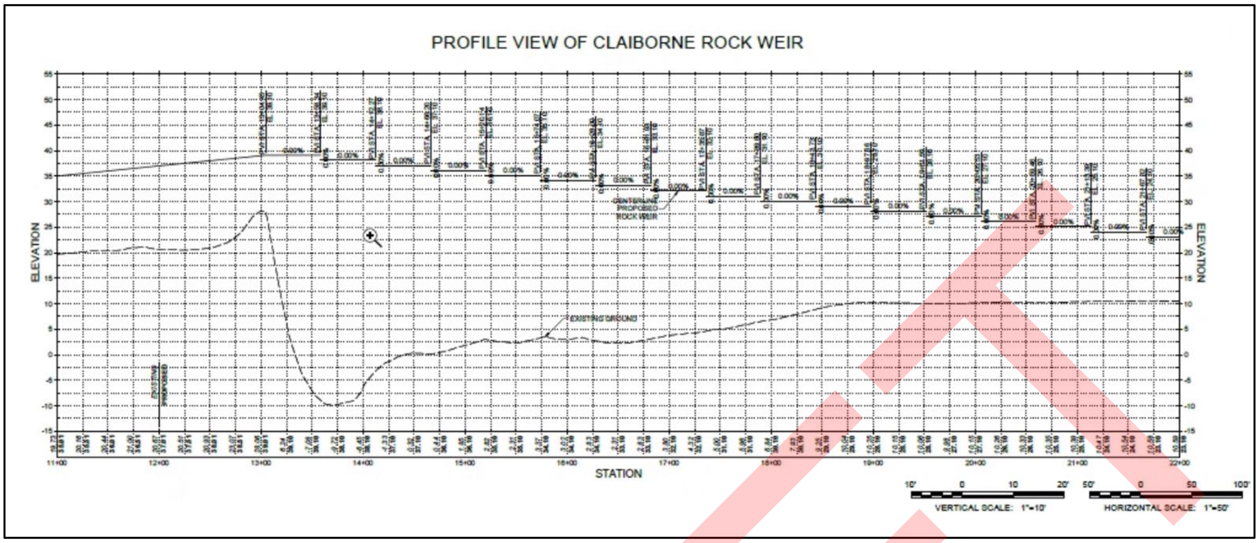



Figure 70 – Profile of fish ladder over fixed crest spillway.

7.3.1 Likely Risk Driver PFMs:

These PFMs are considered likely risk drivers resulting in loss of function of the fish ladder based on the available information and design details. These PFMs include recommendations to further reduce risk associated with these PFMs that may affect project costs.

**PFM 02: Undermining the fish ladder from flow through existing spillway**

Description: Flow through the gated spillway and over fixed crest spillway creates eddies that cause fill material under the concrete channel to erode from scour, leading to differential settlement and tilting of the fish ladder. This causes water to short circuit the fish ladder weir resulting in loss of function. This PFM is illustrated on Figure 71.

The velocities are currently estimated on the order of 7 to 8 fps downstream of the structure. Permissible velocities are on the order of 5 to 7 fps for a compacted clay fill material that is expected to be under the new structure. The PDT is aware of the potential undermining and plan to include riprap and scour protection on the slopes based of velocities from CFD modeling to determine riprap size. This PFM is a loss of service and not a dam safety issue; however, given the 40 ft depth of fill needed to support the structure and the fact that the u-shaped concrete structures will be founded on this fill, there is high potential that any scour erosion could cause tilting of the structures. This would cause unbalanced flow down the fish ladder and will reduce to preclude benefits of fish passage. Therefore, it was identified as a loss of service risk driver for this alternative that is not addressed by the conceptual design.

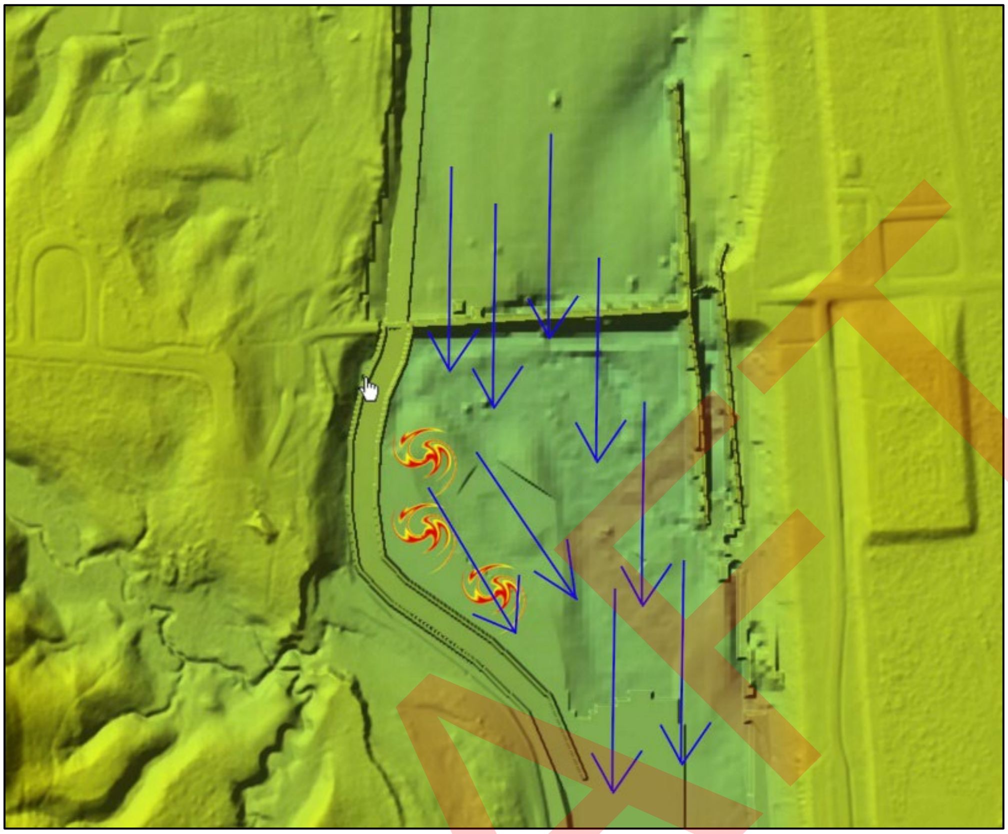


Figure 71 –Eddied currents that may cause scour along the fish ladder passage.

**PFM 06: Loss of function due to settlement**

Description: Construction of the new structures utilizes up to 40 ft of fill in the river channel. Consolidation of the fill causes differential settlement of the concrete u-framed structures. The settlement causes uneven distribution of flow down the fish ladder and overflow over the side of the fish bypass. This further undermines the structure causing tilting and short circuit of water out of the passage and into the river.

Currently in this phase of the design, a fill source has not been identified. If the material is clay, there is likely to be some meaningful magnitude of primary and secondary consolidation over the service life of the project. The fill material will likely be placed directly on bedrock shown by the cross-section in Figure 72; therefore, there will be no significant magnitude of settlement expected from the foundation. However, at 40-ft in fill height, even with good compaction there will be some settlement of the clay fill.

With the u-shaped concrete structures on 40-ft of fill, this was considered a loss of service risk driver for this alternative that is not addressed by the conceptual design.

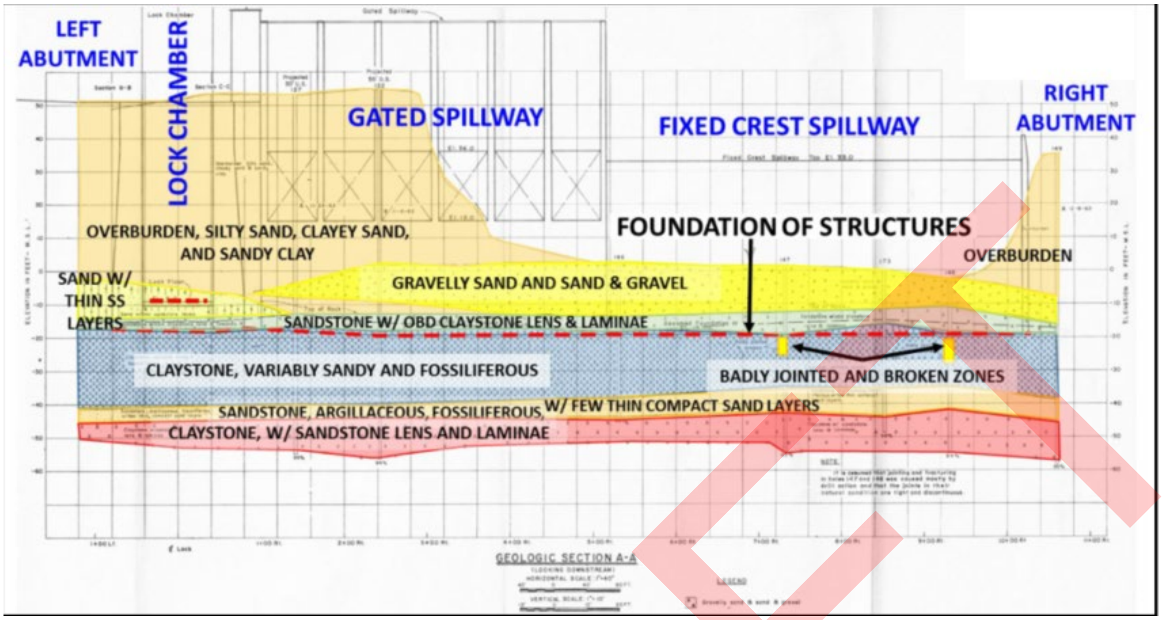



Figure 72 – Geologic cross-section showing overburden soils and weak rock above foundation grade (red dashed line).

**PFM 10: Sedimentation and debris in the structure causes out channel flow**

Description: Sedimentation and or debris in the channel restricts flow leading to out of channel flow over the sidewalls of the structure which erode the side slope protection and lead to undermining of the new structure. Figure 73 shows a debris blockage in the fish ladder channel redirecting flow out of bank and eroding downstream soils. Figure 74 shows the plunge pool erosion the undermines the concrete causing tilting and a point for water to short circuit the intended flow path down the fish ladder.

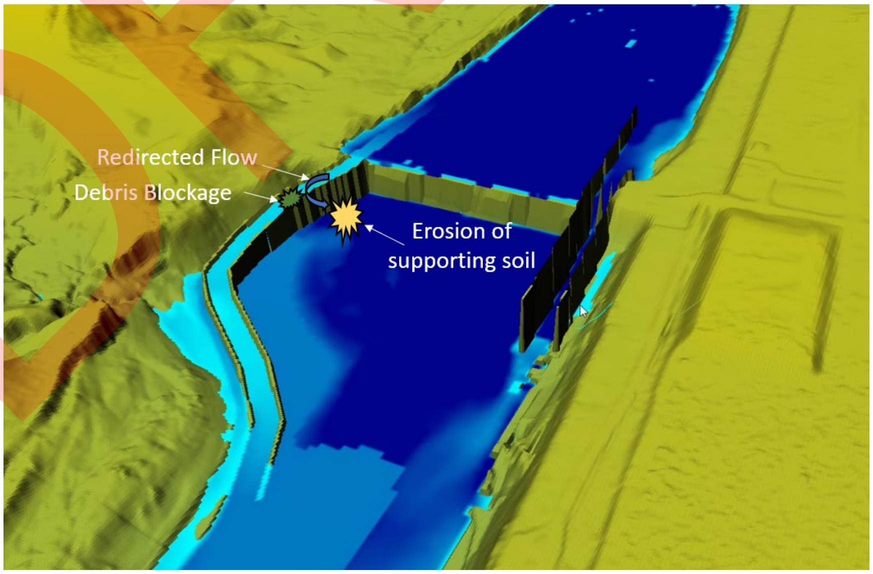



Figure 73 – Plan view of the debris blockage leading to out of channel flow.

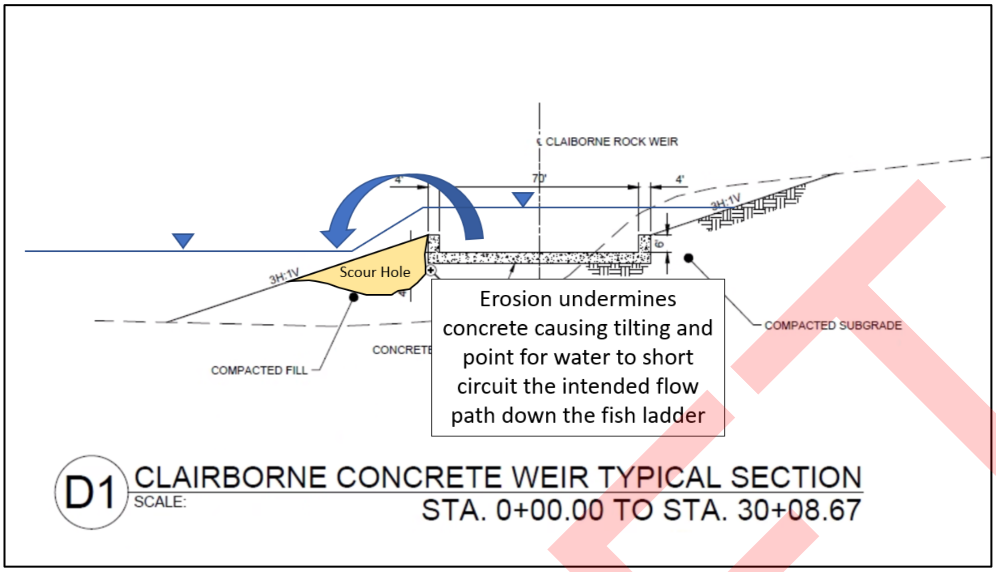



Figure 74 – Cross-section showing the undermining of the slope protection.

This PFM is not a dam safety concern because blockage of the channel would not lead to failure of the dam; however, a debris blockage would lead to loss of function of the new structure. This PFM was judged to be a risk driver for the project that is not addressed by the current conceptual design.

For all three of the risk drivers identified below, the team brainstormed modifications to the design that could address these PFMs. Due to the flow velocities around the structure from the river and due to the potential for settlement and debris jams, it is recommended that if this alternative is further developed that it be modified to include a foundation that extends to bedrock to prevent movement of the structure due of undermining of settlement.

7.3.2 Excluded PFMs:

Justification is included why these PFMs are not considered risk drivers for the project based on the current design detail, hydrologic, and geologic information. Minor recommendations have resulted from discussing several excluded PFMs.

**PFM 01: Overtopping frequency**

The upstream fixed crest weir is at elevation 33.1 ft, to match the elevation of the ungated spillway. The overtopping frequency will increase because of the decreasing capacity of the fixed crest weir in the rock arch weir. While an AEP 1/1 overtopping event increases in frequency, it will not increase the likelihood of failure in the dam. Therefore, this PFM does not increase risk to this project and was excluded.

**PFM 03: A change in hydrodynamics from the fish ladder creates scour and undermining of existing structure**

The new structure will change the hydrodynamics downstream of the dam, which could cause higher velocities that have been historically acting on the structure. These higher velocities could induce erosion in new places that could cause scour and differential movement of the existing dam.

The dam is founded on claystone which should be erosion resistant to the velocities that could occur in the river and therefore this PFM was excluded; however, CFD modeling should still be performed to verify velocities that could occur because of the modification. The PDT recognizes that accurate CFD modeling would better predict the change in flow and help determine the size of armoring needed to prevent scour at new and current structures. The foundation is described as being a claystone with also variable sandy and fossiliferous zones which may be more susceptible to erosion.

The team recommends an appropriate analysis be performed to see if potential scour will occur from a change in hydrodynamics due to a new structure in the channel and to appropriately place a concrete apron or riprap armoring where needed (e.g., downstream toe shown in Figure 75) as well as verify cost estimates to capture potential need for additional armor material.

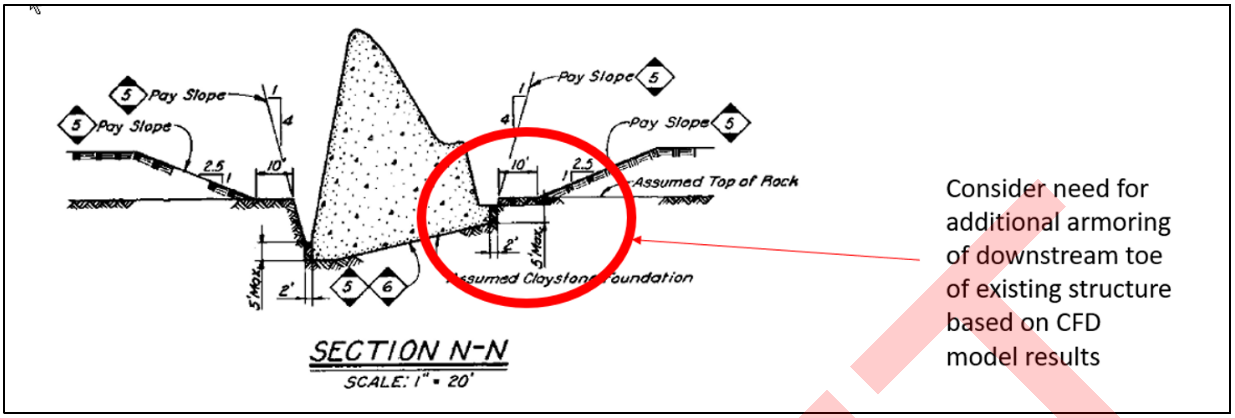



Figure 75 – Additional armoring of existing structure.

**PFM 04: Water infiltration at interface of new structure and existing**

Water entering in between the concrete lined fish ladder and the existing dam shown in Figure 76 scours the supporting fill and causes instability of the fish ladder structure. This PFM could not be fully evaluated at this time as there is currently not a design for evaluation for water stops and concrete keys into the foundation between the two structures.

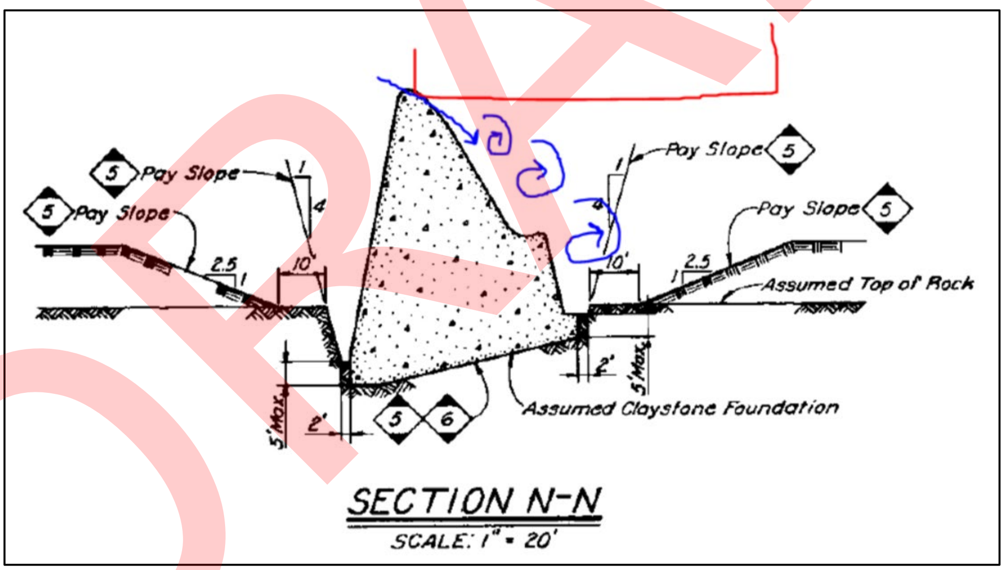



Figure 76 – The water path infiltration at the interface of new structure and existing.

**PFM 05: River flow diverted to unmodified section of existing fixed crest weir causing downstream scour**

This failure mode is similar to PFM 03 but occurs at the existing structure. The additional flows over the existing dam structure due to the restriction of flow through the spillway caused by the fish ladder causes riverbed scour that leads to instability of the structure.

This PFM could not be evaluated at this level of design; however, the performance history of the project has been satisfactory, dive inspections have been completed in 2022 and no dam safety concerns were indicated. The team recommends additional flow modeling be performed to ensure adverse effects don't destabilize the existing structure.

**PFM 07: Cracking of existing dam due to settlement**

Construction of the fish ladder along the existing dam leads to new settlement of the existing dam foundation due to new material being placed next to the existing structure. The addition of 40 feet of fill material and the fish passage concrete structure placed adjacent to the fixed crest spillway induces settlement and causes a crack in the existing dam. The cracking of the existing structural concrete could lead to global instability through scouring or CLE between the soil and concrete structure.

The existing crest spillway is founded on sandstone and claystone which greatly reduces any likelihood of differential settlement. There has been observed cracking at the right abutment wall at sections W2 and W3 documented in Figure 77 and Figure 78 respectively. New settlement from the construction of the structure may exacerbate the existing cracks in the right abutment wall sections. Additionally, the intent will be to fill both sides of the dam evenly so as not to produce an unbalanced load on the existing structure. The potential for induced settlement appears unlikely due to the foundation condition consisting of claystone and sandstone, ultimately excluding this PFM as a risk driver.

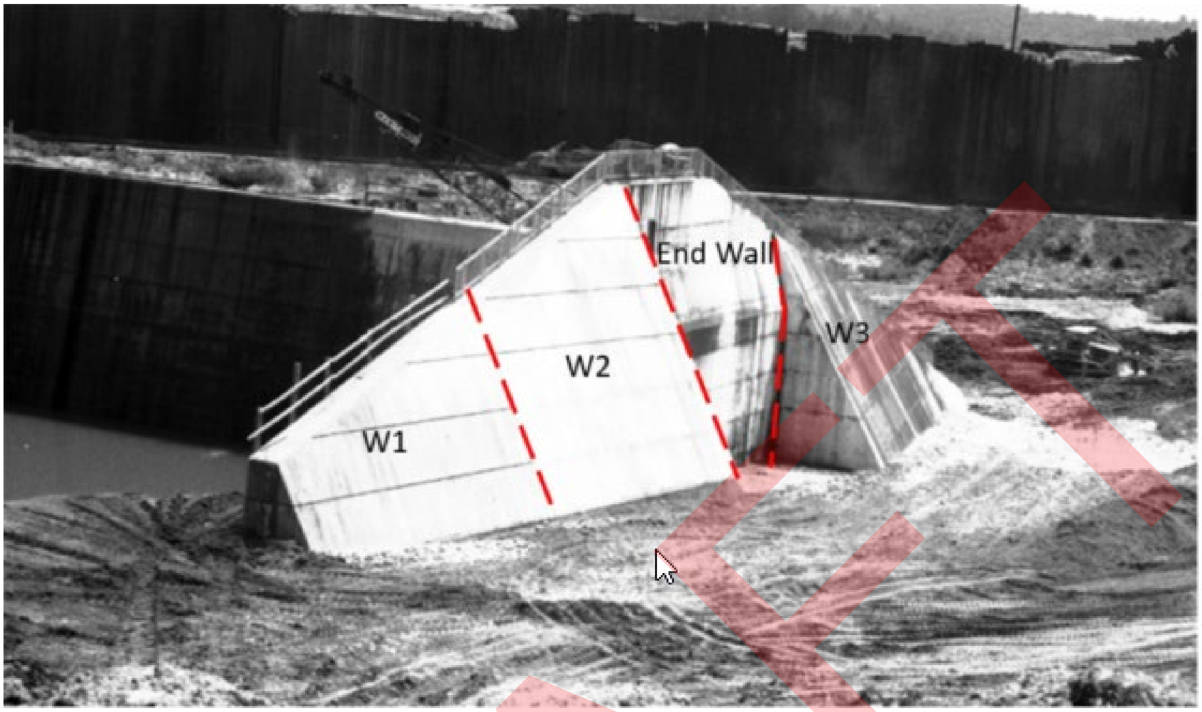



Figure 77 – Right Abutment monolith W2 is cracked, as the end wall which is leaning towards the river.

Figure 78 – Monolith with cracks and leaning towards the river.



**PFM 08: Internal erosion under dam in unwatered condition downstream**

During the construction of the fish ladders, sheet piles are planned upstream and downstream of the existing structure to allow construction in the dry. The unwatering during construction causes gradients to be higher than ever experienced previously (from river, under dam and into unwatered area) and internal erosion initiates through a sand and gravel layer. This circumstance creates a new load test for the dam. If there are erodible materials under the dam, the higher head differential could cause the probability of BEP or CLE to increase.

All construction documents suggest that dam was built on rock and not erodible material. There is evidence of badly jointed foundation material beneath the fixed crest weir that could pose a potential for internal erosion due to the head differential experienced during construction. The contractor during original construction encountered ground water, and the dewatering process required 8 pumps running. This should be considered in the cost estimate to account for significant dewatering activities during construction of the rock arch weir fish passage. This PFM is excluded based on best understanding of the foundation conditions of the dam.

**PFM 09: Dam overturning due to unbalanced fill during construction**

This PFM could not be fully evaluated at the time of the risk assessment. This failure mode would only be a concern if there was intent to build the upstream side prior to constructing the downstream side. There are similarities with PFM 08 unwatering conditions and the potential for a higher gradient. It is assumed that both will be built at the same time and the fill will be brought in equal lifts to avoid an unbalancing force. The construction sequence should be completed such that both sides are constructed concurrent to avoid the potential for this PFM initiating and resulting to the dam overturning.

**PFM 11: Cofferdam overtops during construction causing scour that leads to failure of existing dam.**

The pool elevation exceeds the top of elevation of the cofferdam forming a 30-foot plunge that could lead to excess scour and undercutting within the excavation. Scour then leads to undermining the of the fixed crest spillway and movement of the dam.

Construction techniques have not been identified at this phase of construction to mitigate scour potential in the event of overtopping. It is assumed that excavation will be down to bedrock and the fill material will be placed on top in unison from upstream to downstream for the concrete fixed crest spillway. The top elevation of the cofferdam during original construction was at El. 54 or 57.5 ft MSL. In the event that overtopping occurs, the contractor should have 4-5 days advance warning based on past floods to prepare the construction area and fill the area with water to prevent scour from overtopping. At the time of the risk assessment, this failure mode could not be fully evaluated.

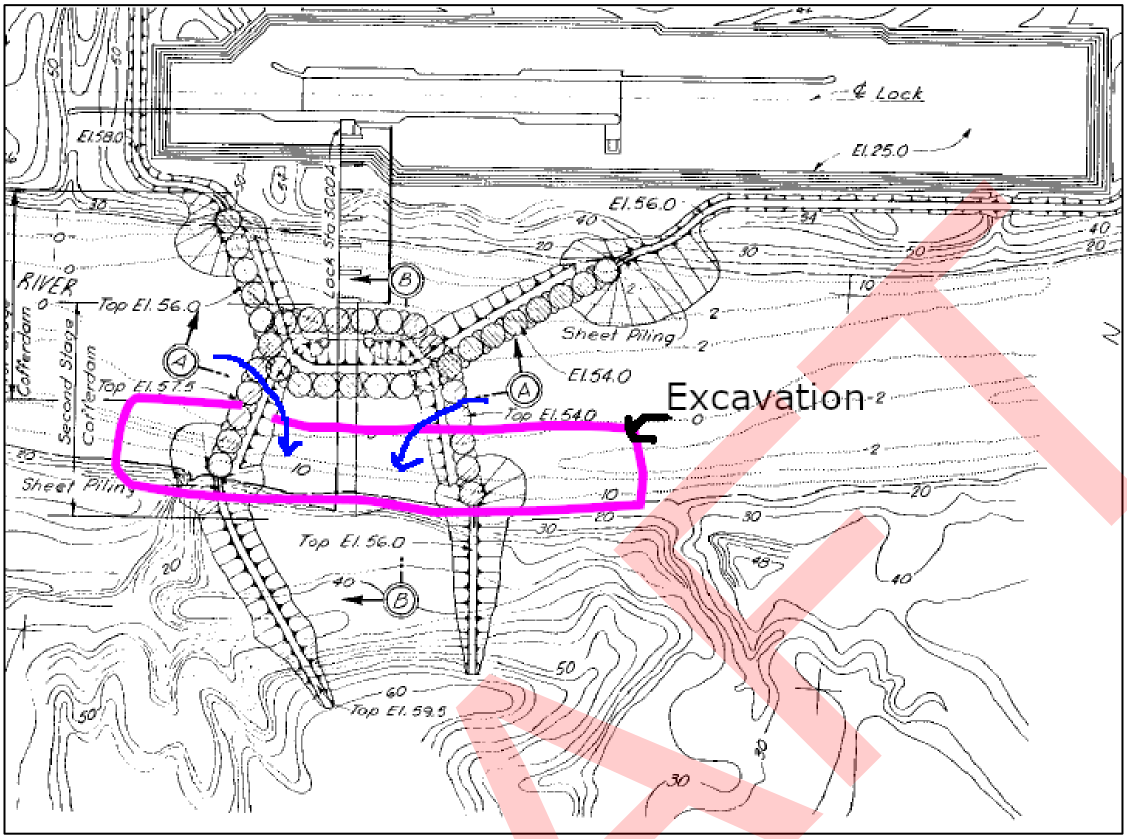



Figure 79 – Area of excavation for new structure.

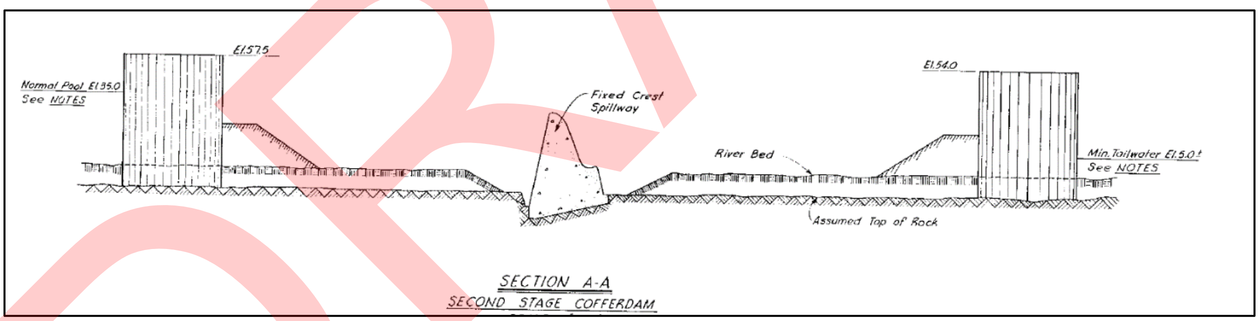



Figure 80 – Cross-section of the second stage cofferdam.

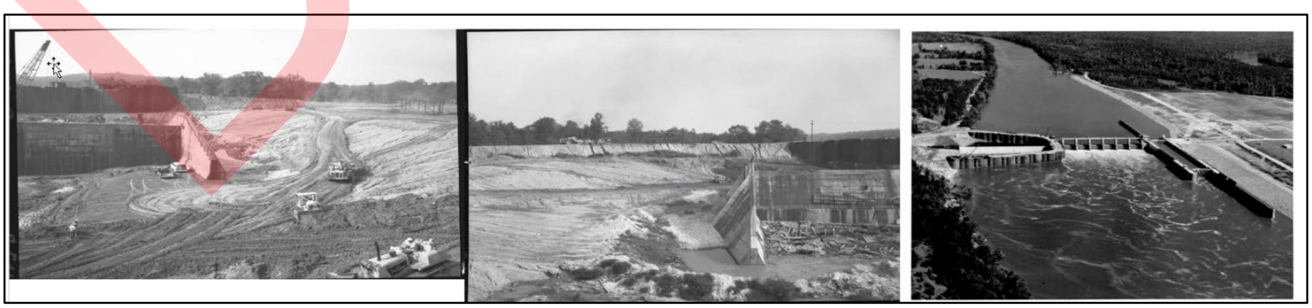


Figure 81 – Construction of fixed weir leading up to the cofferdam.