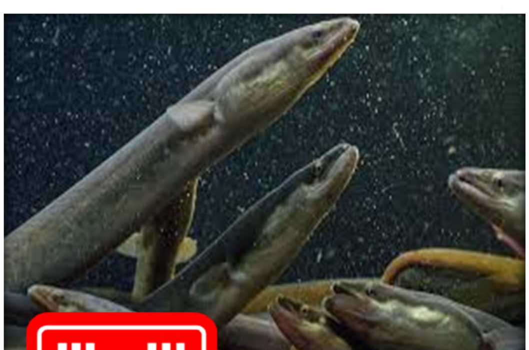
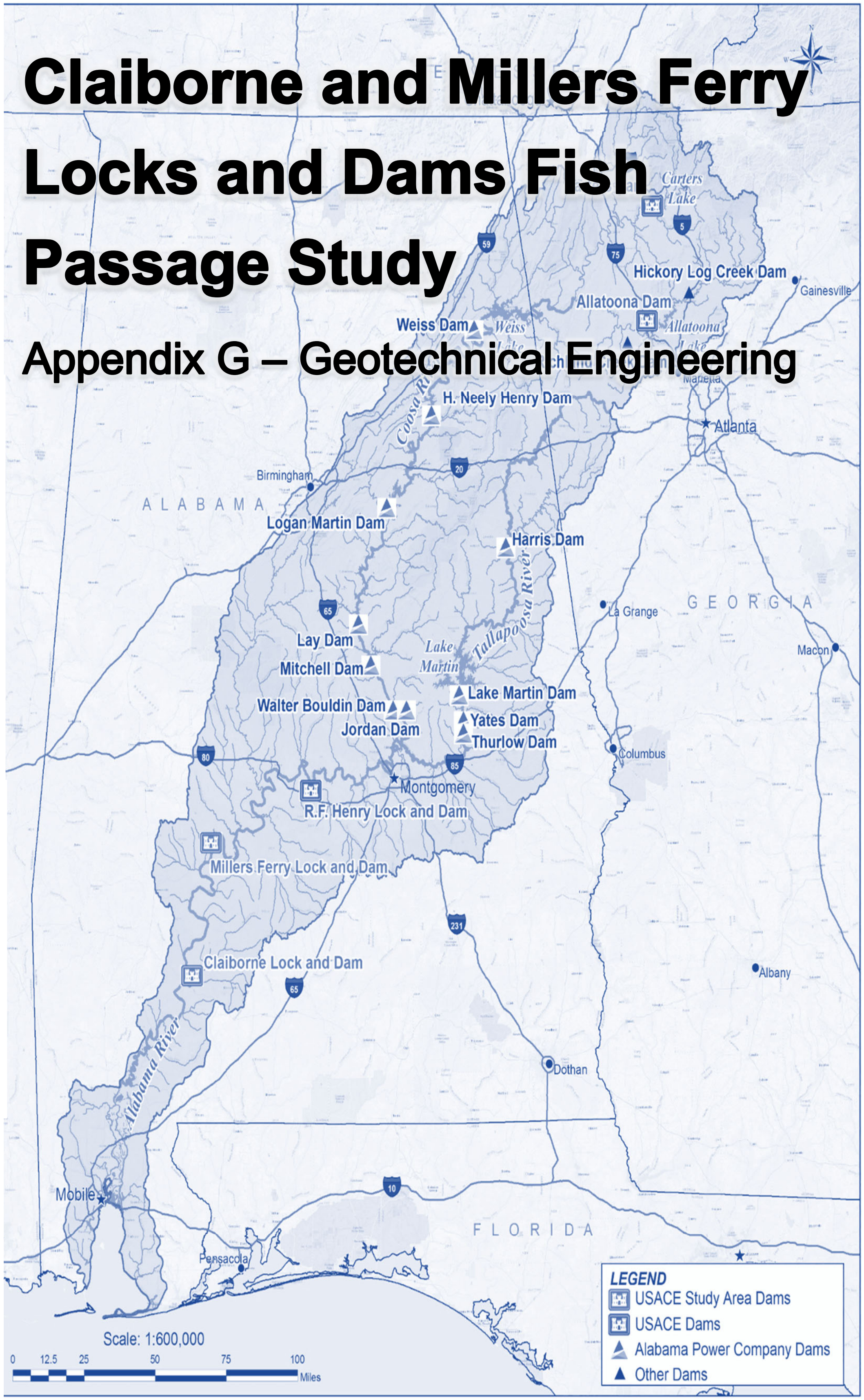
Claiborne and Millers Ferry Locks and Dams Fish Passage Study

Appendix G – Geotechnical Engineering





US Army Corps of Engineers®

The Nature Conservancy®

APPENDIX-G: Geotechnical Engineering

Table of Contents

List of Figures

List of Tables

G.1. Geotechnical Considerations

G.1.1. Millers Ferry Lock and Dam - Site Geology

Millers Ferry Lock and dam is located in the Gulf Coastal Plain physiographic province. The topography in the area is characterized by rolling hills and prairie land. In the reservoir area flows southward in wide meanders. The river traverses the Clayton formation of Tertiary age in the immediate vicinity of the project and chalk and sand formation of Cretaceous age in the remainder of the area.

The overburden soils in the area vary in thickness and composition with an average thickness in excess of 30 feet. The right abutment soils are composed primarily of lean clay and silt, clayey sands with little permeable material. The left abutment soils are composed primarily of lean clays underline by poorly graded sands and gravels which decrease in thickness from 20 feet near the axis line of the dam to zero 1800 feet downstream.

Three geologic formations were penetrated by core at the site. The uppermost is Clayton formation, then Prairie Bluff Formation, and Ripley formation. The Clayton formation consists of primarily black, micaceous, marine clay with strata of silty, sandy, chalky limestone throughout the section. The lower member of the Clayton consists of hard, calcareous, sandstone of variable thickness underlain by poorly cemented silty, fine-grained sands. The Prairie Bluff is a firm, chalky limestone that can be sliced by a pocketknife. With a high-water content, special care shall be needed if exposed to maintain moisture to prevent shrinkage cracks. The thickness of the formation in the area is 10 to 12 feet. Faults were found in four monolith foundations in the landwall of the lock, five monoliths of the spillway and fifteen of the adjacent stilling basin monoliths, also, throughout the powerhouse area. The faults adjacent to the powerhouse were more numerous in the left intake wall, erection bay, and the right tailrace wall areas. The upper Ripley formation is a hard, calcareous sandstone that varies from less than 1 foot to 6 feet with an average of 2 feet. The lower Ripley formation is a calcareous sandstone interbedded with sand and sandstone.

All of the concrete structures, with the exception of the lock floor slabs, were placed on the Prairie Bluff formations.

G.1.2. Millers Ferry Lock and Dam - Subsurface Investigations

For the feasibility study, the USACE Mobile team is using the borings from Design Memorandums for Millers Ferry. Field exploration at the site consisted of obtaining undisturbed and split spoon machine boring samples along the axis of the right bank dike, left bank dike, and during construction core borings were made in the downstream area, three in right abutment wall, and four in the powerhouse area. Four-inch and six-inch cores were taken in proposed locations with 90% to 92% recovery. The borings advance to depths ranging between 10 to 135 feet below existing grade (BEG). The approximate locations are shown on Figure G-1 and Figure G-2, respectively.

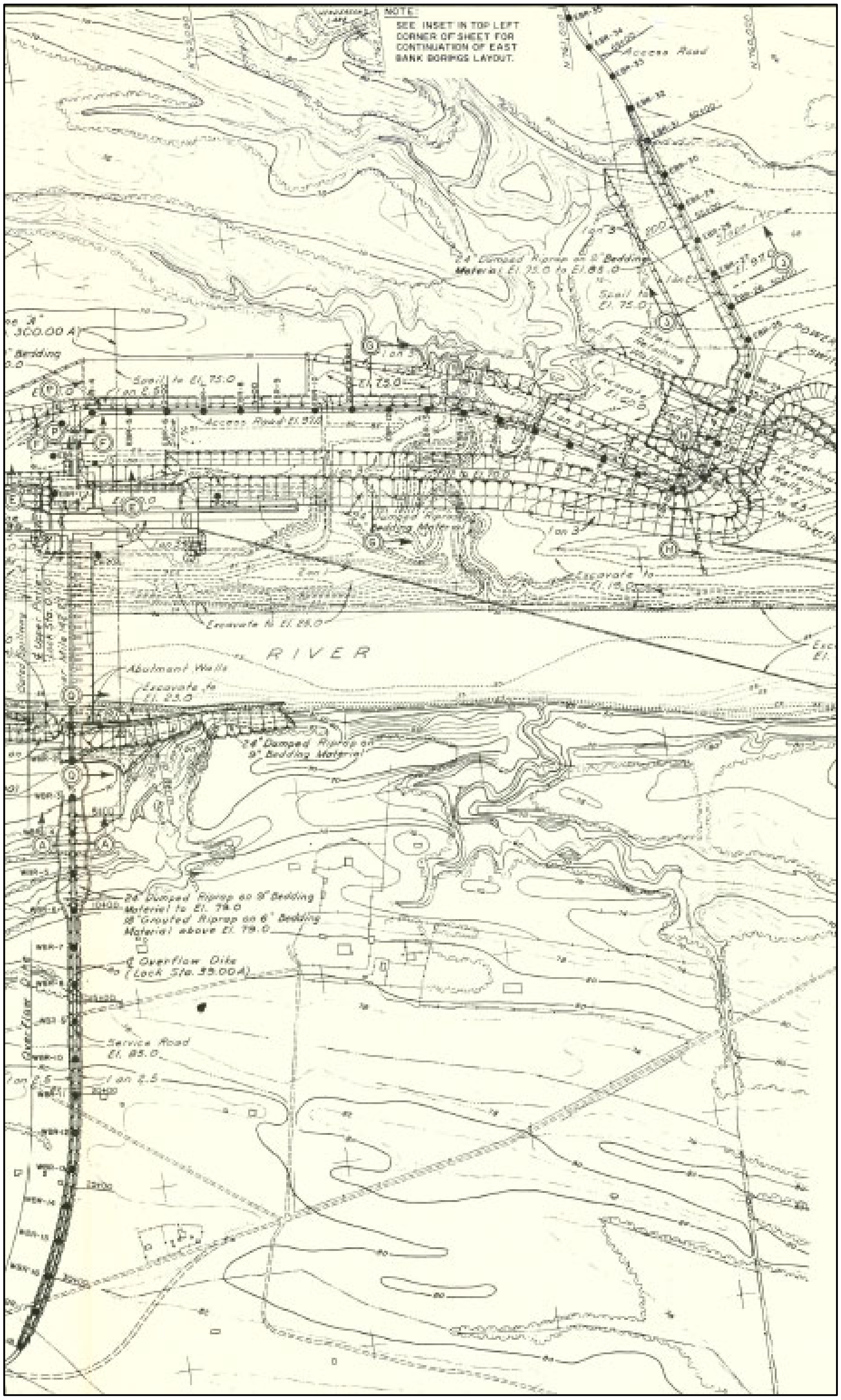


Figure G-1: Geotechnical investigation boring approximate locations for the left and right embankments used for the feasibility level geotechnical analysis

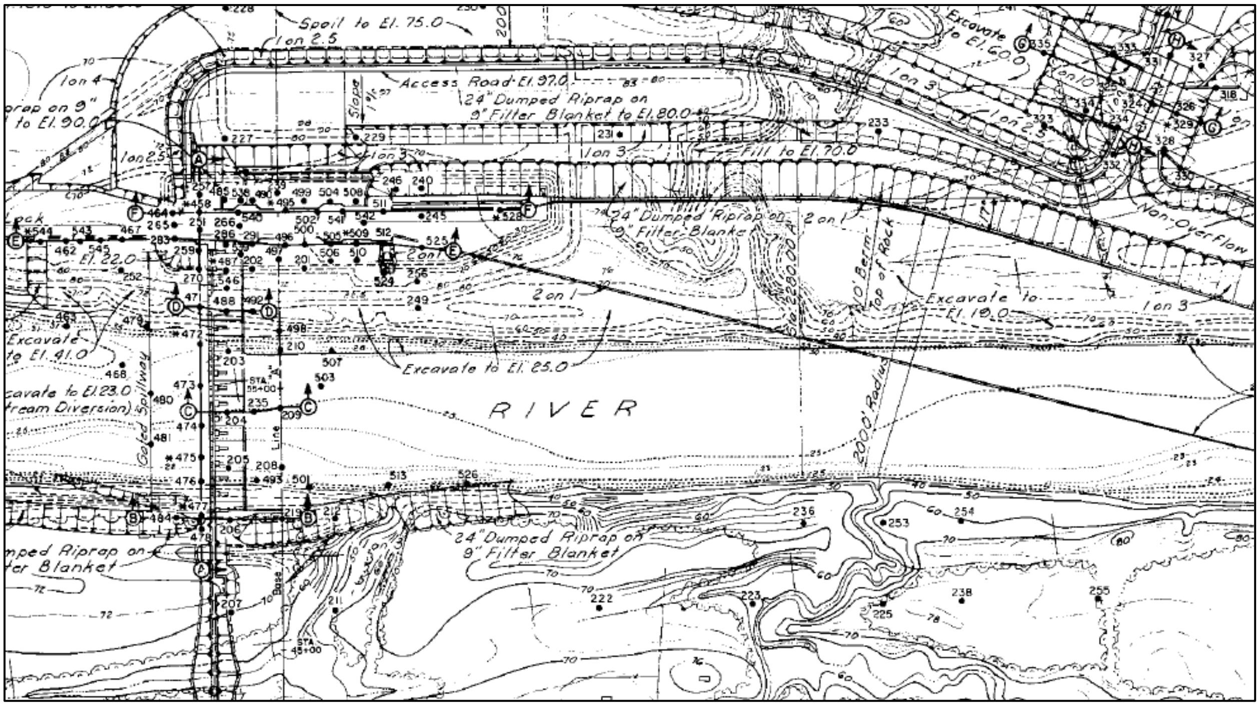


Figure G-2: Geotechnical investigation boring approximate locations for the spillway and powerhouse footprints used for the feasibility level geotechnical analysis

Laboratory testing was accomplished by the South Atlantic Division Laboratory, Marietta, Georgia. Classification, mechanical analyses, moisture and density determinations, compaction, shear, and consolidation tests were part of the test procedures performed. The results of the subsurface investigation are provided in the following sections of this report.

Table G-1: Boring elevations and depths, Millers Ferry Lock and Dam.

Boring #	Location	Boring Elev. (ft-MSL)	Overburden Thickness (ft)	Groundwater Elev. (ft-MSL)	Borehole Depth (ft)
477	Spillway	75.6	44.5	39.6	97.0
472	Spillway	24.8	6.0	--	32.5
475	Spillway	21.7	4.0	--	39.6
487	Spillway	79.4	29.8	56.4	96.7
495	Lock	73.2	40.5	59.2	102.7
458	Lock	72.5	32.5	58.5	90.8
509	Lock	77.2	43.0	60.2	112.2
528	Lock	74.3	41.0	59.3	110.6
324	Powerhouse	75.3	26.7	57.3	131.6
329	Powerhouse	71.0	23.2	57.0	121.4
EBR-22	Powerhouse	75.5	>27.0	58.5	27.0
WBR-1	West Bank	73.5	19.5	36.5	90.0
WBR-2	West Bank	70.4	>36.0	48.4	36.0
WBR-3	West Bank	69.8	>30	48.8	30
WBR-4	West Bank	56.8	32.8	43.8	51.0
WBR-5	West Bank	53.6	15.0	44.6	51.0
EBR-24	East Bank	70.7	>23.0	55.7	23.0
EBR-25	East Bank	70.1	>31.0	54.1	31.0
EBR-26	East Bank	70.9	>53.0	55.9	53.0
EBR-27	East Bank	58.7	>54.0	53.7	54.0
EBR-28	East Bank	69.5	>22.0	54.5	22.0
EBR-7	Central Bank	96.0	49.5	46.0	62.0
EBR-8	Central Bank	96.0	45.0	52.0	55.5
EBR-9	Central Bank	96.0	45.0	52.0	49.5
EBR-10	Central Bank	95.0	45.0	51.0	46.5
EBR-11	Central Bank	70.6	>20.5	52.6	20.5

G.1.3. Millers Ferry Lock and Dam – Subsurface Conditions

Subsurface conditions were analyzed based on observations from past geotechnical exploration conducted as part of the initial study for the project. The exact locations is unknown as no coordinates were provided; however, their approximate locations are based on the Figure G-1 and Figure G-2. Geotechnical investigations were performed by the Mobile District in the 1960's, as detailed in the *Design Memorandum No 10 Geology* (1963) and *Design Memorandum No 14 Design of Earth Embankments* (1964).

Millers Ferry subsurface for the purpose of this section shall be divided into the west bank (right bank), spillway, lock, central bank, powerhouse, and east bank (left bank).

From West to East, the following paragraphs describe the subsurface conditions.

The west bank subsurface contained silts to clays (ML to CH) 0 to 30 feet BEG. The layer has a range of blow counts from 10 to 48. A weak clayey sand layer is 25 feet BEG with weight of hammer blows. Generally, the west bank soils have an average blow count of 20 with the overburden material becoming finer with depth to a sandy lens atop the siltstone in the area. Material taken from boring WBR-3 at 1.0 to 17.0 BEG has a liquid limit (LL) of 50 to 60 and plastic limit (PL) of 25 to 35, and at 20.0 to 22.0 BEG has LL of 30 and PI of 21. The water table is roughly 35 feet BEG. Top of siltstone bedrock elevation ranges from 20 to 40 feet-MSL.

The spillway overburden is composed of mostly sandy clay to sand with gravel near top of rock. Samples were discarded and no blow counts were recorded. Only the field classifications are available. The general trend for material grain size increases with depth. The thickness of the overburden within the footprint of the spillway is 4.0 to 45.0 feet, with overburden being less in footprint of the river. The abutment of the existing riverbanks was stiff silty to sandy clays from 0 to 36 BEG. The bedrock was cored with the upper 20 feet of rock being tightly consolidation uncemented clays with cemented chalky limestone zones.

The central bank subsurface from 0.0 to 46 BEG is mainly silty clays (CL to CH) interbedded with sand lenses (SM to SC). 0.0 to 20.0 BEG is primarily clays with blow counts ranging from 10 to 63. The average blow count for the 0.0 to 20.0 BEG is 26. The groundwater surface was around 44.0 BEG (elev. 55 ft-MSL) at the time of drilling. Bedrock elevation is between 50 and 55 ft-MSL with a 3 to 8 feet thick sand overlying the siltstone.

The powerhouse overburden during drilling was 20 to 30 feet BEG. The overburden boring EMR-22 provides blow counts for the silty sand (SM) material ranging from 9 to 21 with the average at 18. The bedrock is overlayed by a poorly graded gravel at elevation 50 feet-MSL. In addition, borings 324 and 329 samples were discarded, and no blow counts were recorded. Only the field classifications are available with the overall grain size increasing with depth.

The east bank overburden during drilling was greater than 22 feet. The deepest boring depth of 54.0 did not reach bedrock. From the section H-H, the powerhouse adjacent to the east bank bedrock is near elevation -20 feet-MSL. The material on east bank is predominantly lean to fat clay (CL to CH) interbedded with lean silts (ML). The lean to fat clays are 0 to 16.5 BEG with a range of blow counts from 5 to 23 with an average of 16. At elevation 58 feet a weaker clay layer exists with SPT values as low as 1. The thickness of the weak layer varies from 2 to 5 feet. Underlaying the weak layer is very stiff silts to lean clay with blow counts exceeding 100. The bedrock material was not encountered on the east bank with the deepest boring at elevation 15.0 feet-MSL.

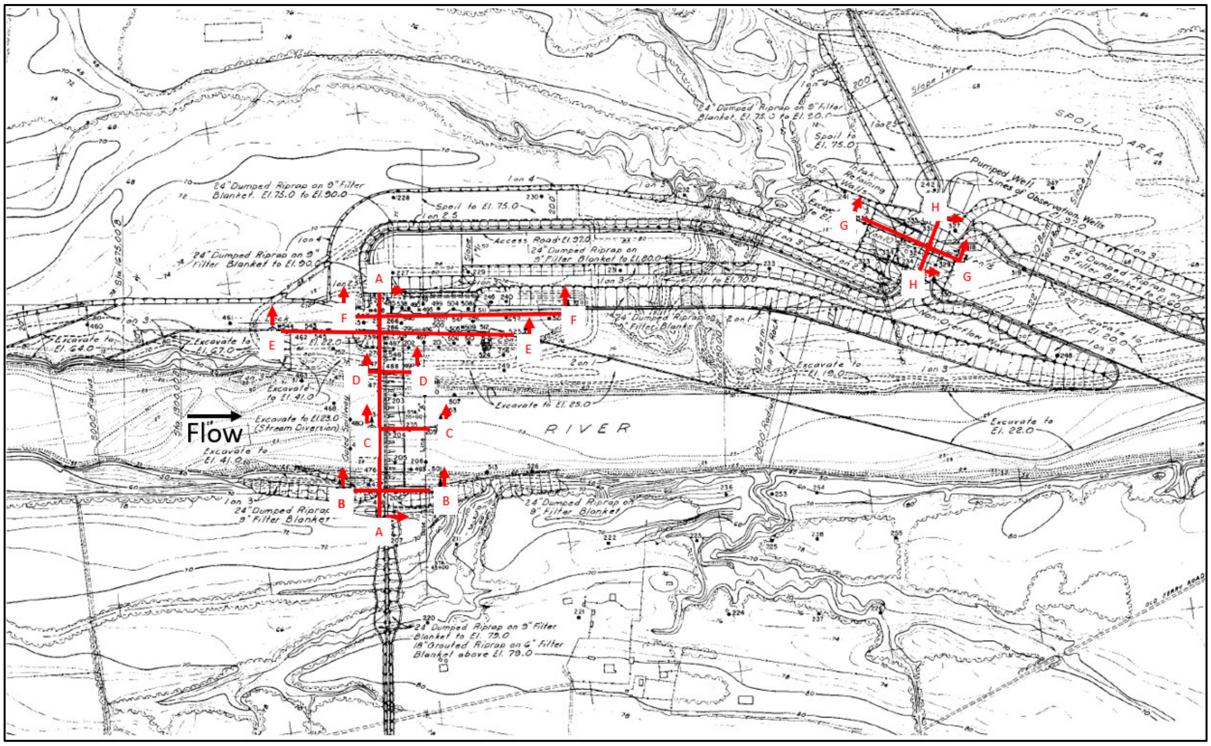


Figure G-3: Plan view of project geologic cross-sections

The plan view of Millers Ferry geologic cross sections are shown in Figure G-3. Fence diagrams of the cross sections are provided in Figure G-4

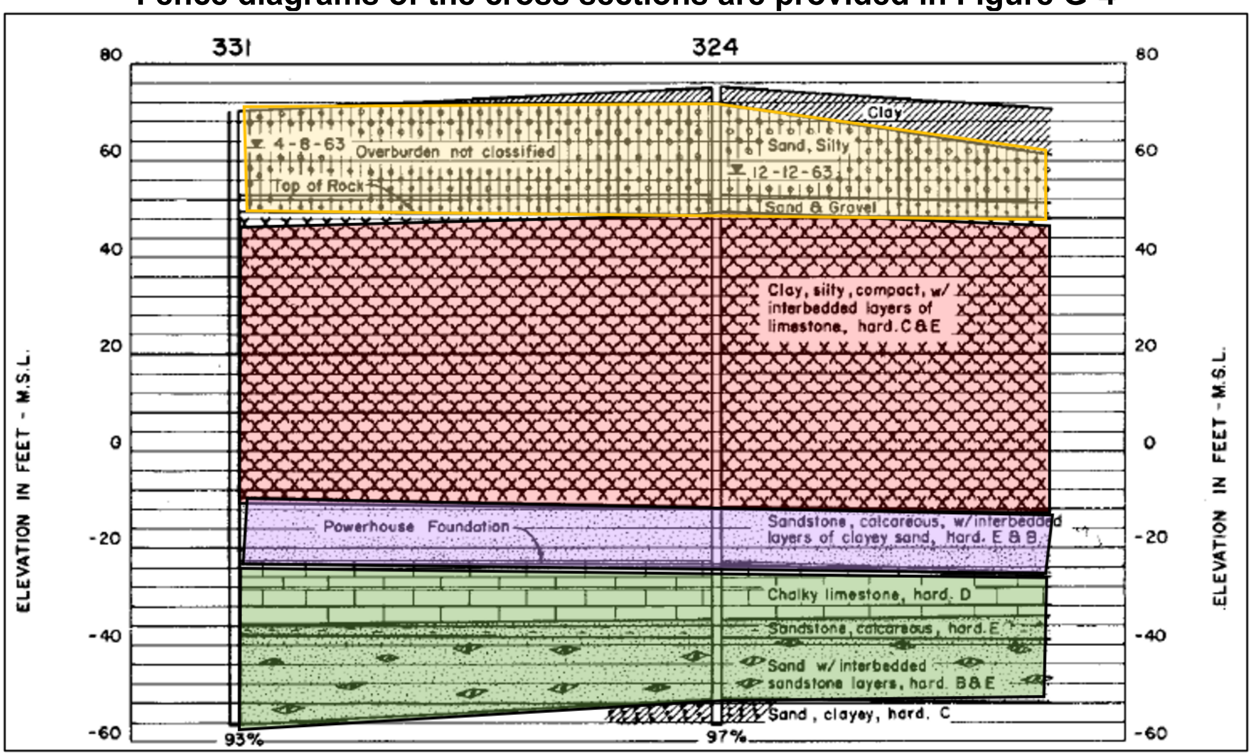


Figure G-6. The legend for material type within the cross-sections is;

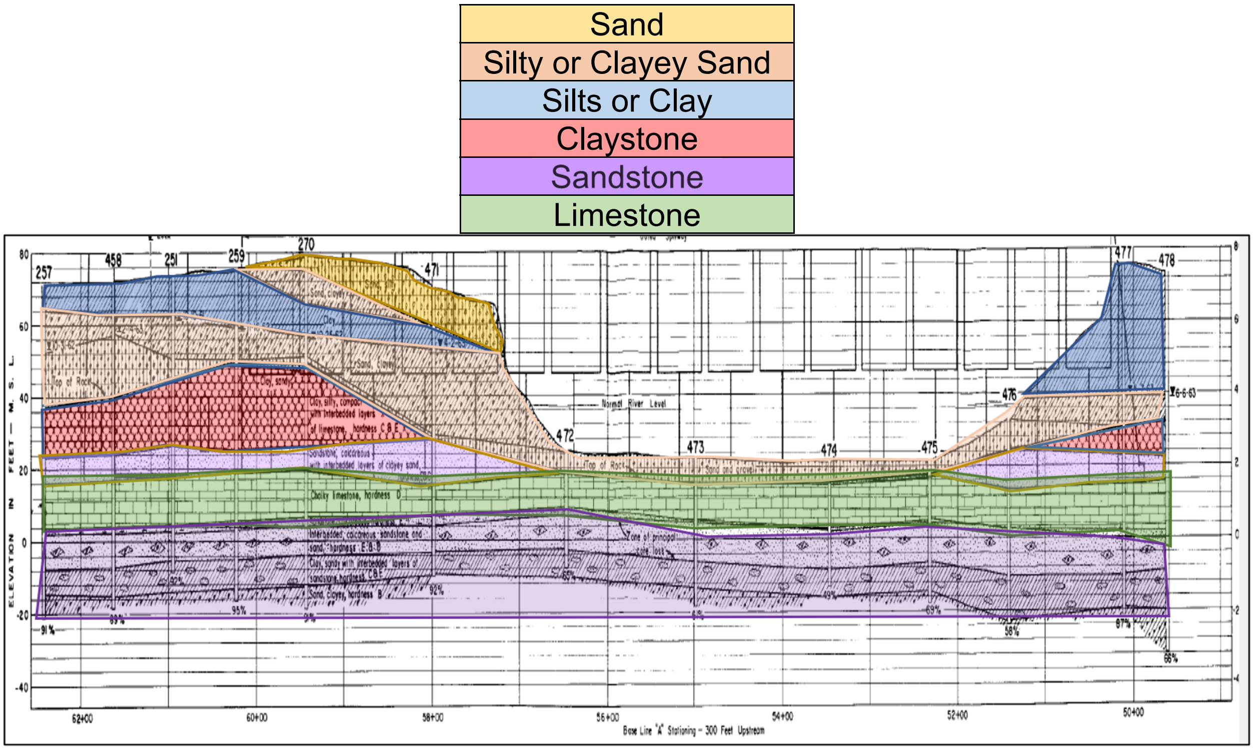


Figure G-4: Fence diagram from central bank to east bank (left to right) with spillway in the middle for the initial subsurface investigation. (A-A)

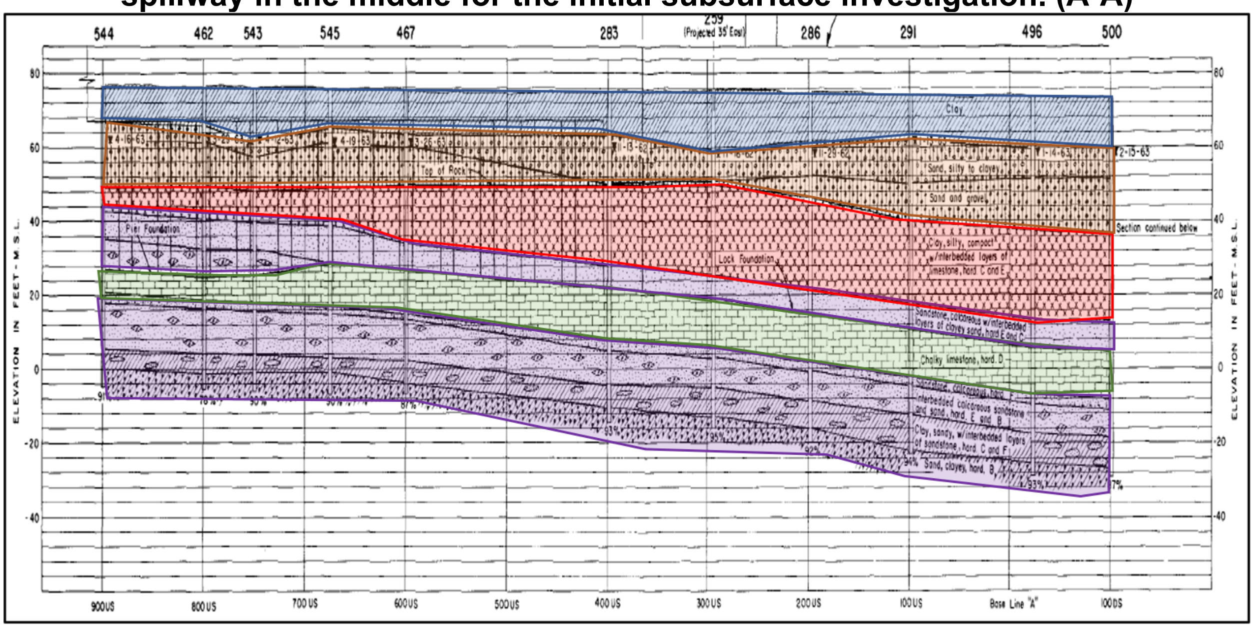


Figure G-5: Fence diagram North to South (Left to right) of the central bank with section taken parallel with flow of the river. (E-E)

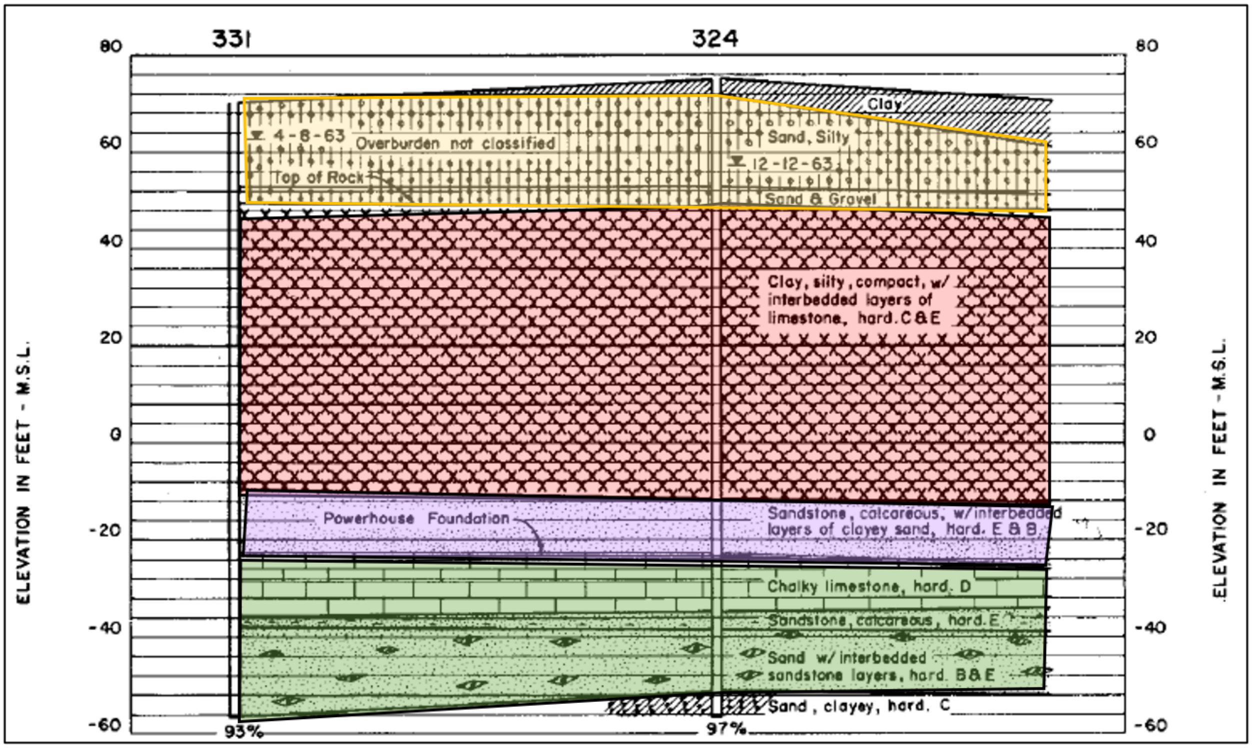


Figure G-6: Fence diagram looking downstream of Powerhouse (H-H)

The subsurface stratigraphy can be described as four major strata: silts and clays, fine sands, clays, and limestone bedrock. The fine sands are mainly on the left portion of the dam from the central bank to the east bank. The west bank is primarily clay, fine sand, and bedrock. The upper clay layers on the west bank (right bank) generally are inorganic silts varying in depth from 29 feet at the bank river to 3 to 8 feet with average depth of 3 feet from stations 0+00 to 28+00 and turns to fat clays from 28+00 to 34+00. The east bank (left bank) is composed on mainly lean clay. The consistency of the upper and lower silt and clay layer is stiff. The consistency for the fine sand is stiff to hard.

G.1.4. Millers Ferry Lock and Dam – Lab Testing Data

The soil laboratory data is from 1964. Testing of the soil includes characteristics, strengths, and gradations.

The design strength and friction angle of the west bank (right bank) embankment material per Q, R, and S triaxial tests are 0.75, 0.50, and 0.00 tons per square foot (tsf) and 3.5, 10.0, and 25.5 degrees. The saturated unit weight 125.5 is pounds per cubic foot (pcf). The foundation soils for the west bank have shear strengths equal to 0.00 to 0.65 tsf and friction angles 0.00 to 26.5 degrees.

The foundation material for the spillway and lock are the chalky limestone with unconfined compressions tests exceeding bearing requirements for the lock and spillway. Beneath the footprint of the lock, the chalky limestone exceeded the required allowable bearing capacity of 18 tons per square foot. Table G-2 summarizes the lab test data for associated sections of embankments.

Table G-2: Triaxial Tests and associated soil parameters

Material	Test Type	Moist Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Friction Angle	Cohesion (tsf)
Embankment	Q	124.6	125.7	3.5	0.75
Embankment	R	124.4	125.4	10.0	0.50
Embankment	S	120.6	125.6	25.5	0.00
Foundation, 6+50	Q	123.6	124.7	0.0	0.65
Foundation, 6+50	R	120.9	122.5	15.5	0.40
Foundation, 6+50	S	122.8	124.0	26.5	0.00
Foundation, 38+15	Q	123.3	125.2	17.0	0.43
Foundation, 38+15	R	120.9	122.6	15.5	0.40
Foundation, 38+15	S	114.6	114.6	25.5	0.00

G.1.5. Millers Ferry Lock and Dam – Construction Techniques

The material for the right overflow dike was from the west bank borrow area or also known as the right bank borrow area site. The material from the borrow source ranged from lean clay to silty fat clays. The borrow material had Q, R, S triaxial tests ran on the undisturbed soil samples. Per the tests ran, the cohesion (shear stress) was 0.5 to 0.7 tons per square foot (tsf) and total friction angle near 24 degrees. From the triaxial analysis, a stability analysis was performed with 1H:3V slopes for three loading conditions. The conditions were construction, sudden drawdown, and steady seepage. The internal friction angle of the fill material was set to 10 & 25 degrees during the slope stability analyses. The factors of safety for construction, sudden drawdown, and steady seepage are equal to 2.0, 2.15, 2.31. The maximum settlement was determined at overflow embankment station 6+50 due to this having the most amount of fill required. At the centerline of the embankment, the calculated settlement of the embankment fill and foundation was 0.60 feet and 0.58 feet.

Compaction of backfill was done by placing 9-inch layers compacted to 90 percent of the maximum density obtained at the optimum moisture content follow CE-55.

G.1.6. Claiborne Lock and Dam - Site Geology

Claiborne Lock and Dam site is within the Southern Red Hills Divisions of the Gulf Coastal Plain physiographic province. The river meanders through this region and has a broad flood plain generally 25 to 40 feet above the normal river level. Outside the river valley is characterized by gently rolling hills and north facing cuestas developed on harder bed of sedimentary strata. Sediments near the project site are typically coastal plain deposits of interbedded limestones, clays, sand, and sandstone of Paleocene and Eocene age. The Tallahatta Formation of Eocene age underlies the site and provides the foundation for all structures. The Tallahatta in the project footprint consists of claystone with subordinate sandstone and sand strata. The claystone is a greenish gray, compact, impervious material 20 to 25 feet thick. The field hardness scale for the claystone is “D”, or that the material can be cut with moderate pressure but cannot be completely penetrated. Beneath the lock area an interbedded sand and sandstone occurs directly beneath the alluvial overburden and constitutes the top of the rock.

The sandstone with abundant claystone lens and lamina was generally hard but data from cores were found broken along the horizontal claystone lamina. These surfaces were extremely soft and slick, potentially resulting in recurring sliding planes. To avoid planes that could lead to differential settling, monoliths of the lock and dam were founded on the hard claystone. Badly broken and jointed zones, up to 5 feet thick, were encountered in many borings with dips varying from 45 to vertical. The slickensides are interpreted as due to swelling and heaving of the rock mass on either side of the joint with change in overburdened load or water content.

At 40 feet BEG a fat clay is encountered on the left portion of the spillway to the lock walls. The fat clay is 20 feet thick. Underlying the fat clay is the gravelly sand and clayey fossiliferous sand above sound rock. The top of bedrock occurs at -12.4 feet-MSL, respectively.

G.1.7. Claiborne Lock and Dam – Subsurface Investigations

Subsurface conditions were analyzed based on observations from past geotechnical exploration conducted as part of the initial study for the project. The exact locations are unknown as no coordinates were provided, but their approximate locations are based on the Figure G-7.

The overburden soils are generally 50 to 70 feet thick in the lock area and 5 to 15 feet within the footprint of the river. The overburden is primarily silty sand, clayey sand, and sandy clay in that order from the surface downward to a predominant sand and gravel bed overlying bedrock. The overburden grain sizes increase with depth. The sand and gravel layer varies from 0 to 21 feet thick with an average thickness of 8 feet. It is continuous across the entire section of the lock and dam perpendicular to the river.

The elevation of the water table in the area is 18 to 19.5 ft-MSL.

The flood plain in the area is about half a mile on the west bank (left bank) and only 100 to 300 feet wide on the east bank (right bank). Historical data states the west bank rises almost vertically to about 35 feet above the river and then levels off to 50 feet-MSL. The east bank rises steeply about 25 feet above the river and levels off near 35 feet-MSL.

The laboratory tests for the claystone that foundations are bearing on has a unit weight of 104 pounds per cubic foot (pcf). The low unit weight is a function of the microscopic pore spacing and interspersed pockets of friable sandstone and sand a fraction of an inch thick.

Table G-3 provides the subsurface data for borings used during the geotechnical design of Claiborne.

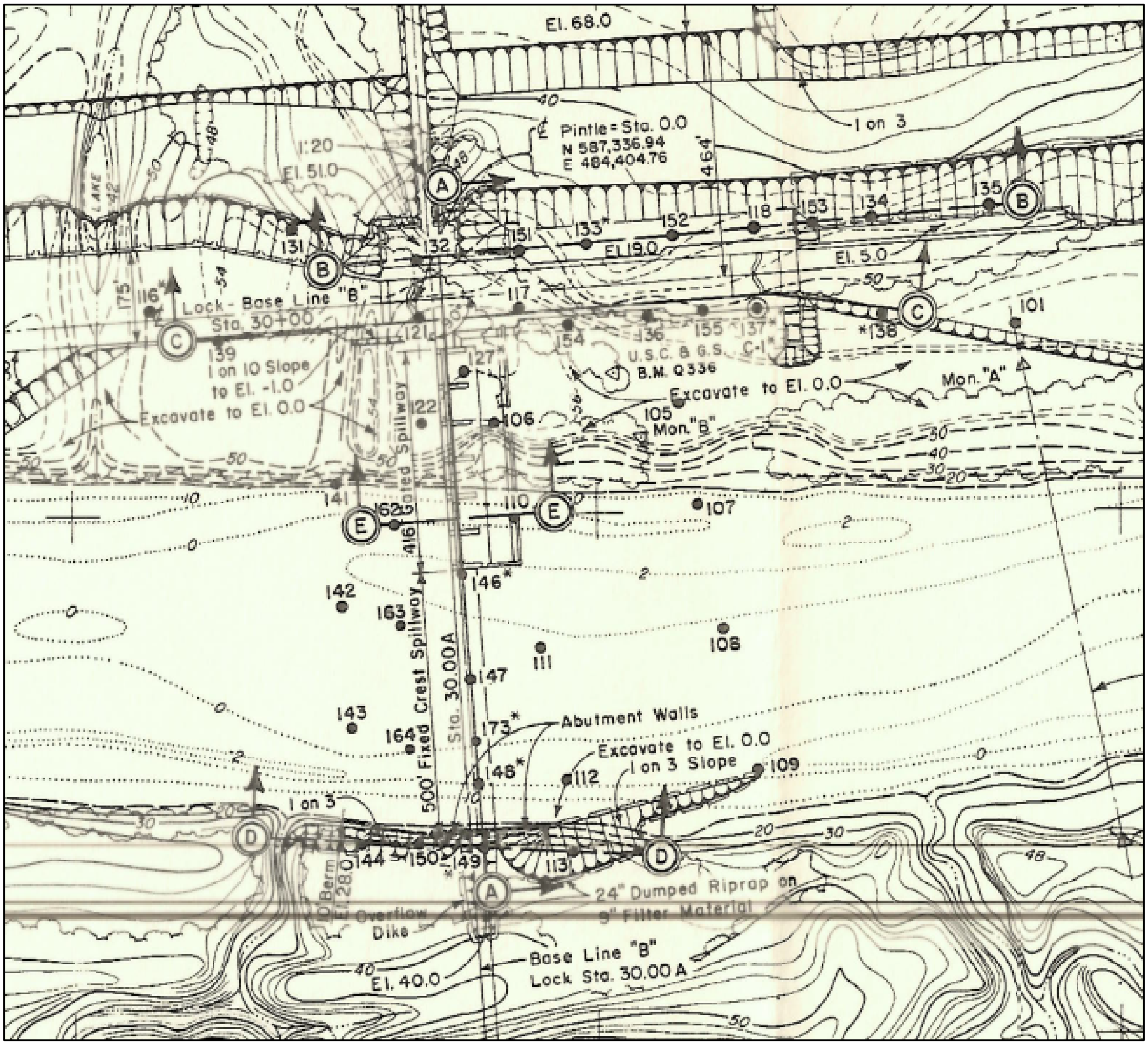


Figure G-7: Boring Log Approximate Locations for Claiborne Lock and Dam

Table G-3: Boring Elevation and Depths, Claiborne Lock and Dam

Boring #	Location	Boring Elev. (ft-MSL)	Overburden Thickness (ft)	Groundwater Elev. (ft-MSL)	Borehole Depth (ft)
116	East Bank	48.0	49.3	28.0	115.7
127	Gated Spillway	53.8	66.2	19.3	108.4
133	East Bank/ Lock	46.9	50.1	18.2	99.2
149	West Bank	35.6	51.5	18.1	93.7
162	Gated Spillway	0.00	22.0	--	43.0
173	Fixed Spillway	-0.7	11.3	River	48.0

G.1.8. Claiborne Lock and Dam – Subsurface Conditions

Subsurface conditions were analyzed based on observations from past geotechnical exploration conducted as part of the initial study for the project. The exact locations are unknown as no coordinates were provided. Geotechnical investigations were performed by the Mobile District in the 1960's, as detailed in the *Design Memorandum No 4 Geology* (1964) and *Foundation Report*.

Claiborne sections within the section shall go east to west, starting with east bank (left bank), lock, gated spillway, fixed crest spillway, and west bank (right bank).

The east bank conditions top to bottom are fine grained sands (SM to SC), gravel and sand (SP to SG), clays, sandstone deposits interbedded with compact fossiliferous sandstone layers, and claystone (foundation material). The samples taken in overburden were discarded and do not have any associated field data (SPT).

The lock has similar subsurface conditions with the east bank and no overburden samples were tested and spilt spoon numbers recorded during the time of drilling. Throughout much of the lock area, an interbed sand and sandstone strata occurs directly beneath the alluvial overburden and constitutes the top of the rock. The sandstone is variable and discontinuous and could not be correlated between borings. In boring 127, a fat clay (CF) was encountered 40 feet BEG to 60 feet BEG.

The gated spillway conditions top to bottom are the fine-grained sands alluvial deposits with thickness of 66 to 70 feet. The gated spillway is where the natural east bank of the river once was. It can be assumed that upstream and downstream of the lock and dam that similar subsurface conditions will exists. No in situ soil strength data was provided within the historical boring logs. Underlying the fine-grained sand deposits is sandstone and claystone bedrock. The top of bedrock is near -8.0 feet MSL.

The fixed crest spillway conditions top to bottom is sand and gravel alluvial deposits with thickness of 11.3 feet. No in situ soil strength data or samples are provided within the historical boring logs. The top of bedrock in the footprint of the fixed crest spillway is -12.0 feet-MSL. The fixed crest spillway is placed in the center of the historical river channel.

The west bank conditions from top to bottoms during initial borings is a 6 feet thick lean clay (CL), a 35 to 40 feet thick fine-grained sand (SM to SC), and interbedded lamina and lenses of claystone and sandstone 50 feet BEG. The elevation of bedrock in this location is -15.0 feet-MSL.

Fence diagrams for the lithology at Claiborne is provided in

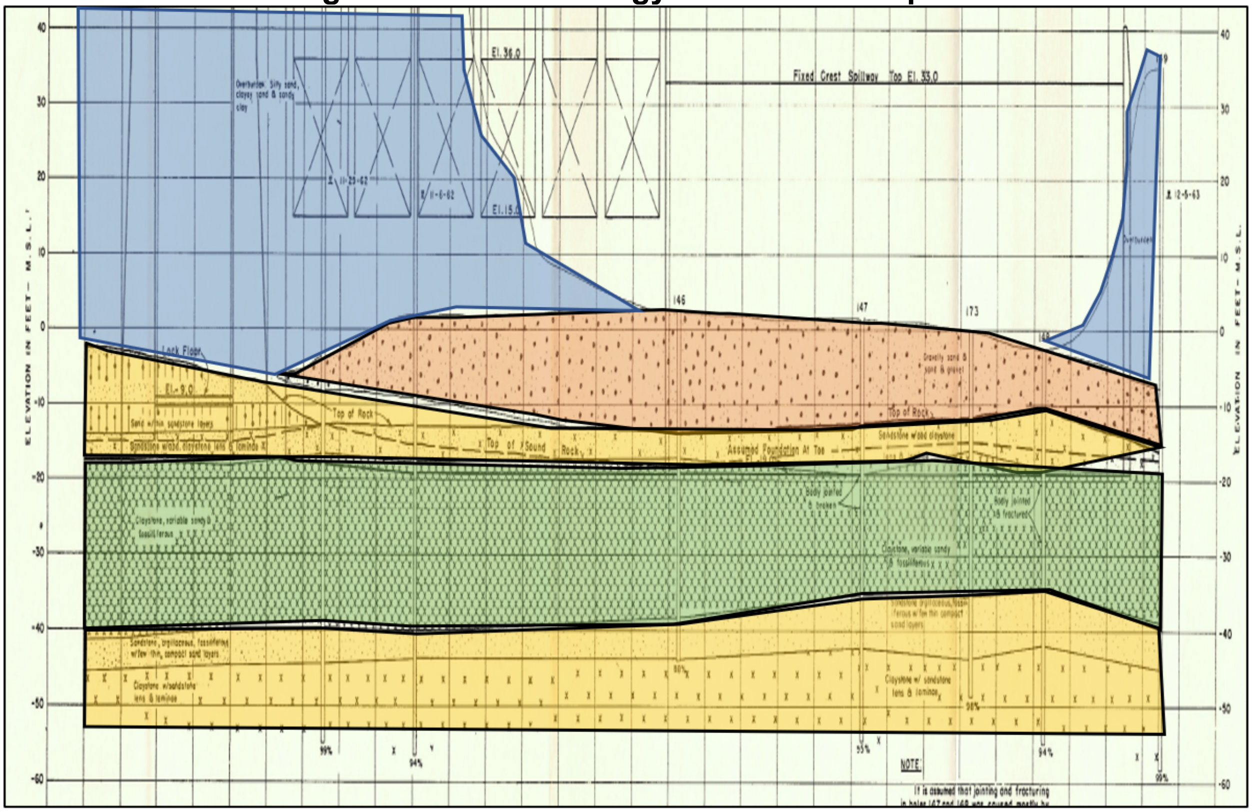


Figure G-8. The overburden soil were discontinuous and alluvial deposits did not connect between borings. The legend for the type of material is;

Overburden: silty sand, clay sand, silts, clays
Gravelly sand, sand and gravel
Sand
Claystone

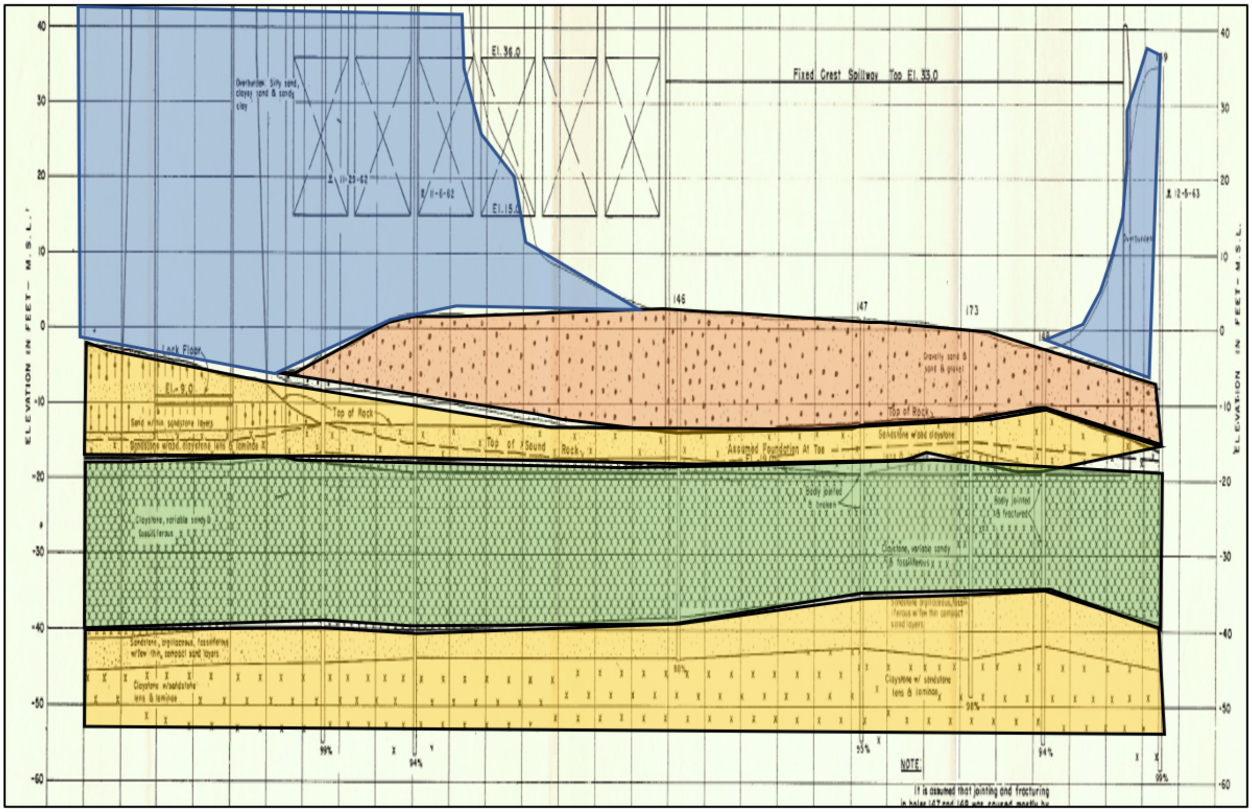


Figure G-8: Fence Diagram looking downstream, Lock on left and fixed crest spillway on the right.

G.1.9. Claiborne Lock and Dam – Lab Testing Data

Laboratory testing was limited on to the foundation material of the concrete structures at Claiborne. The laboratory tests were performed on the claystone material. Of the tests completed, Table G-4 below provides the general design criteria used for the stability analysis performed on the lock, gated spillway, ogee spillway, and abutment walls.

Table G-4: Laboratory Tests and Design Criteria

Direct Shear Tests	1 to 8 tons per sq ft axial Cohesion = 2.75 tons per sq ft Tangent Internal Friction = 0.83
Triaxial Tests	Cohesion = 2.9 tons per sq ft Tangent Internal Friction = 1.0
Unconfined Compression Tests	8 to 49 tons per sq ft
Maximum Allowable Bearing	10 tons per sq ft
Sliding Coefficient FS=1.5	Allowable = 0.4

G.1.10. Claiborne Lock and Dam – Construction Techniques

Excavation of slopes were cut 1V:1.5H. The interbedded sand and sandstone unit that occurs below the overburden was treated overburden and excavated on the same slope down to the underlying sandstone with claystone lens and lamina. During construction, excavation of the overburdened and bedrock was accomplished using mechanical methods.

Two earth dikes on either side of the river were constructed. The dikes are between the abutment wall and high ground on the right bank and between the lock esplanade and high ground on the left bank. If the right dike is overtopped, the difference between headwater and tailwater will be around 1-foot in head. The left bank dike starts at the left bank and slopes down to the lock esplanade. The maximum height differential for the dikes from crest to toe is equal to nearly 20 feet. This location occurs on the left dike and the location is over the level of the normal upper pool. If upper pool elevation exceeds about 40 feet in elevation, the difference between the headwater and tailwater is less than 1-foot, therefore, slope stability analysis of the dike section was not considered necessary.

The right bank was compacted to 90 percent CE-55 maximum density. The left bank upper 3 feet was compacted to 90 percent CE-55 maximum density while the rest of the structure was compacted by controlled movement of construction equipment. Cutoff wall of steel sheet piling was provided on the landside of the abutment wall on the right bank and to the landside of the lock to the right bank. Approximate top and bottom elevations for the right and left sheet pile (seepage mitigation wall) is near elevation 37 feet to top of rock and 47 feet to top of rock. The material used for the construction of the dikes was silty sands, clayey sand, and sandy clays. The slopes for the dike are 1V:3H.

G.1.11. Claiborne Lock and Dam – Right Overflow Embankment Extension

The fish bypass channel at Claiborne along the right (west) bank of the river creates the need for developing a new damming surface. The proposed bypass channel footprint is to remove portions of the right overflow embankment along the western channel bank to create continuity between the upper and lower pools. During design, the bypass channel is proposed to have a stoplog structure to prevent upper pool from entering the bypass channel during operations and maintenance. The stoplog structure will tie high ground along the west to the right bank extension damming surface. The right bank extension damming surface is outlined in red on Figure G-9.

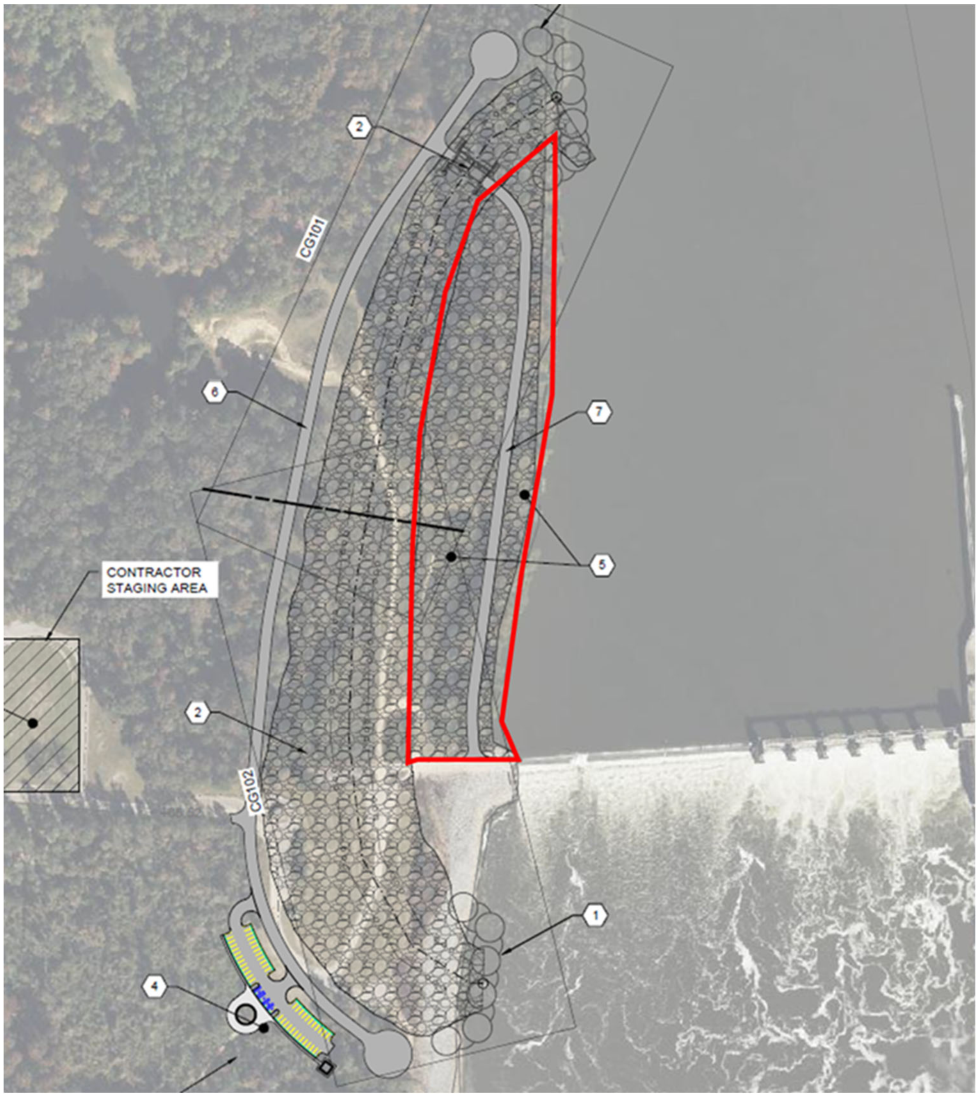


Figure G-9: Proposed extension to right bank creates new damming surface in red

The existing section within the design manuals and general drawings for Claiborne Lock and Dam overflow embankment is shown on Figure G-10 and Figure G-11. The embankment is select pervious fill with a sheet pile extending 100 ft west from the right concrete abutment wall. The sheet pile is driven into top of bedrock. Borings 149 at the center of the overflow embankment had layers of clay and silty sand. Boring 137 was used for determining the depth of bedrock. The top of the bedrock (claystone) was first encountered at elevation -19.0 ft MSL. The design manual and drawings for Claiborne Lock and Dam overflow embankment has the following characteristics:

- Crest elevation: 40.0 ft MSL
- Top and bottom of sheet pile: 35 ft MSL to top of bedrock (assume -19 to -22.0 MSL)
- Trench of compacted impervious fill creates the cap along the sheet pile wall
- Pay slope cut on 1V:2.5H from top of bedrock to existing high ground (40 to 45 ft MSL) and then compacted with select pervious material

- Compacted pervious fill capped with 9 inches of filter material and 24 inches of riprap
- Upstream and downstream slopes 1V:3H

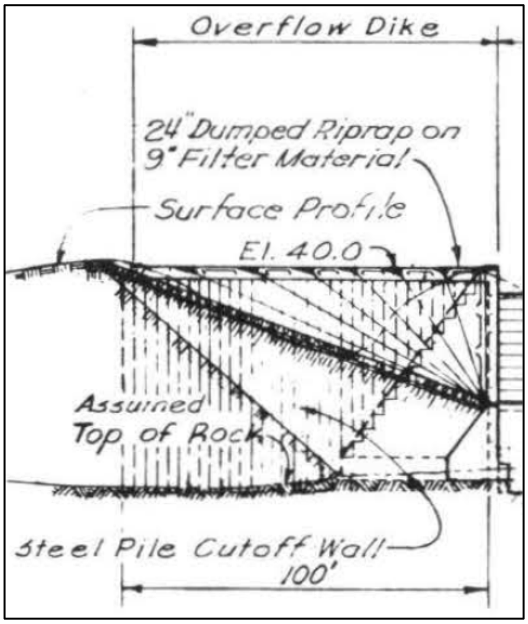


Figure G-10: Transverse section of current right overflow embankment

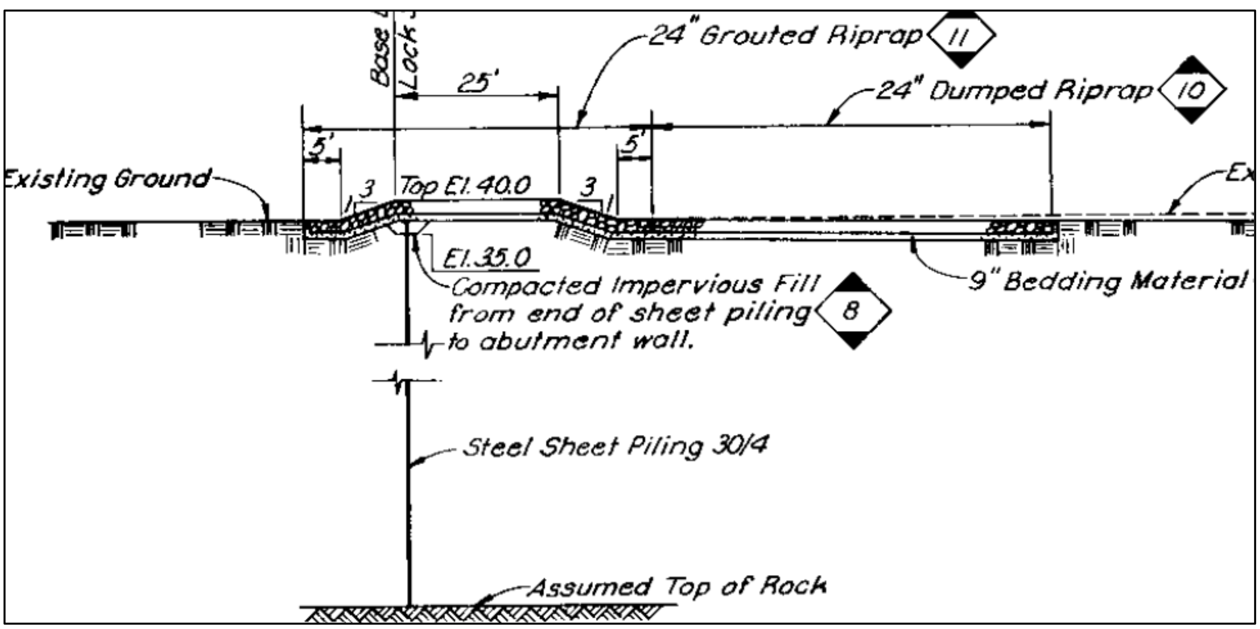


Figure G-11: Cross-section of current right overflow embankment

The proposed location and cross-section for the right overflow embankment extension is preliminary. Geotechnical borings, lab testing, and analysis have not been completed for this location. The boring drilling along the right bank reveal the lithology is a fluvial depositional environment with variable layers of fine- and coarse-grained material. The borings show a sand and gravel layer between elevations -1.9 to -19.1 ft MSL. It is estimated this layer can have significant upward seepage forces within the bypass channel. In addition, above the sand and gravel layer is the presence of the coarse-grained material. The cut (excavation) required to get to optimal bottom elevation of the bypass channel and the elevation which coarse grained material layers occur creates the potential for unwanted seepage and will likely require a dewatering plan during construction. The estimated maximum gradient along the bypass channel, occurring near station 15+00, ranges from 0.07 to 0.1. The gradient was calculated using the measurements for upper pool to bypass channel in empty condition with bank extension width near 200 feet. Due to the hydraulic gradient, the cross-sectional width of the right bank extension, lithology of historic borings, and the criteria for creating a new damming surface the PDT determined installing a sheet pile wall beneath the access road was necessary.

Second, to prevent erosion of the proposed right bank extension armoring will be required. A velocity model ran for Claiborne bypass channel raised the concern between station 14+00 to station 16+00 during overtopping conditions stating localized flow will be significant. The right bank overtops the right embankment once to twice a year on average. The bypass channels at Millers Ferry and Claiborne already require riprap due to the modeled velocity that will flow through the channel. Therefore, the PDT determined to continue the riprap up from the bypass channel slopes to the crest of the extension and towards the river channel bottom due to localized higher velocity in stations 14+00 to 16+00, overtopping occurring annually, riprap already required in the bypass channel, the extension is now a damming surface, cover the sheet pile wall, existing conditions already have 24 inches of riprap, and to prevent undermining within the river channel.

Finally, the sheet pile and riprap along the slopes were proposed to mitigate dam safety concerns for common failure modes such as; overtopping, toe erosion leading to sloughing, protection against shallow slides, global stability, concentrated leak erosion, and backward erosion piping. For cost estimating purposes, the new sheet pile will tie into the existing sheet pile and continue north to tie into the stoplog structure.

For preliminary design and cost estimating purposes the following decisions were made for the cross-section of the right bank extension:

- Crest elevation equal to 40.1 NAVD88
- Top and bottom sheet pile elevations 37.1 and -20 ft NAVD88.
- River and bypass channel slopes 1V:3H
- Entire section of extension covered with bedding and riprap material (Figure 1)
- Riprap along the riverbank shall extend down to river bottom
- 20 feet wide gravel access road along the crest

Additional borings, stability, and seepage analysis are essential to determine final design of the slopes, sheet pile, and construction techniques.

Figure G-12 provides the general section looking upstream along the constructed overflow right embankment. The drawings have the pay slope cut and is backfilled with pervious select fill, 9 inches of riprap bedding for erosion mitigation, and topped with 24 inches of riprap. The same will be true for the current design of right bank embankment extension for cost estimating purposes.

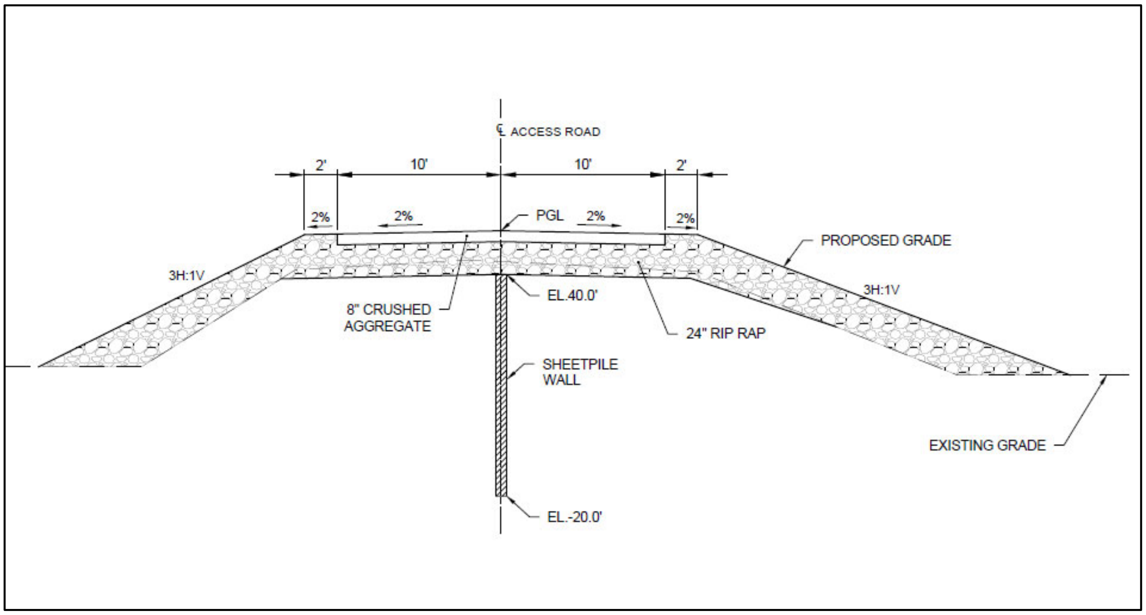


Figure G-12: Proposed cross-section for the right overflow bank extension

G.2. Documentation Supporting Feasibility-level Design

The following sections layout a summary of applicable engineering data needed to support a feasibility-level engineering design as outlined in ER-1110-2-1150 Engineering and Design for Civil Works Projects. There is currently a government estimate and scope of work in place for data collection at the two sites following the centerline of the proposed bypass channel alternative.

G.2.1. Millers Ferry Lock and Dam

The team looked at several fish passage alternatives along the right bank of Millers Ferry. The right portion of Millers Ferry contains an overflow dike with elevation near 85 feet, and the abutment between the gated spillway and earth dike.

G.2.1.1. Site Geology

Overburdened soils generally along the right overflow dike centerline are expected to be 40 to 70 feet thick on fine grained material. The elevation of the bedrock along the centerline increases with right bank dike stationing. 0+00 starts at the interface contact between the concrete of the abutment walls and earth material. The material during construction was removed by mechanical methods. Beneath the overburdened material is a sand and gravel layer 10 to 30 feet thick. The upper elevation of the start of bedrock material along the centerline of the spillway at the right bank is near elevation approxiamately+20 feet. The bedrock is a chalky limestone with hardness D. Per the borings done along the centerline of the overflow dike, the top of bedrock elevation starts from +10 to +55. The slope of the bedrock at the centerline is 0.03 or 3% east to west.

G.2.1.2.Future Subsurface Investigations

To support the feasibility level design for the fish passage channel three borings are proposed shown in Figure G-13.

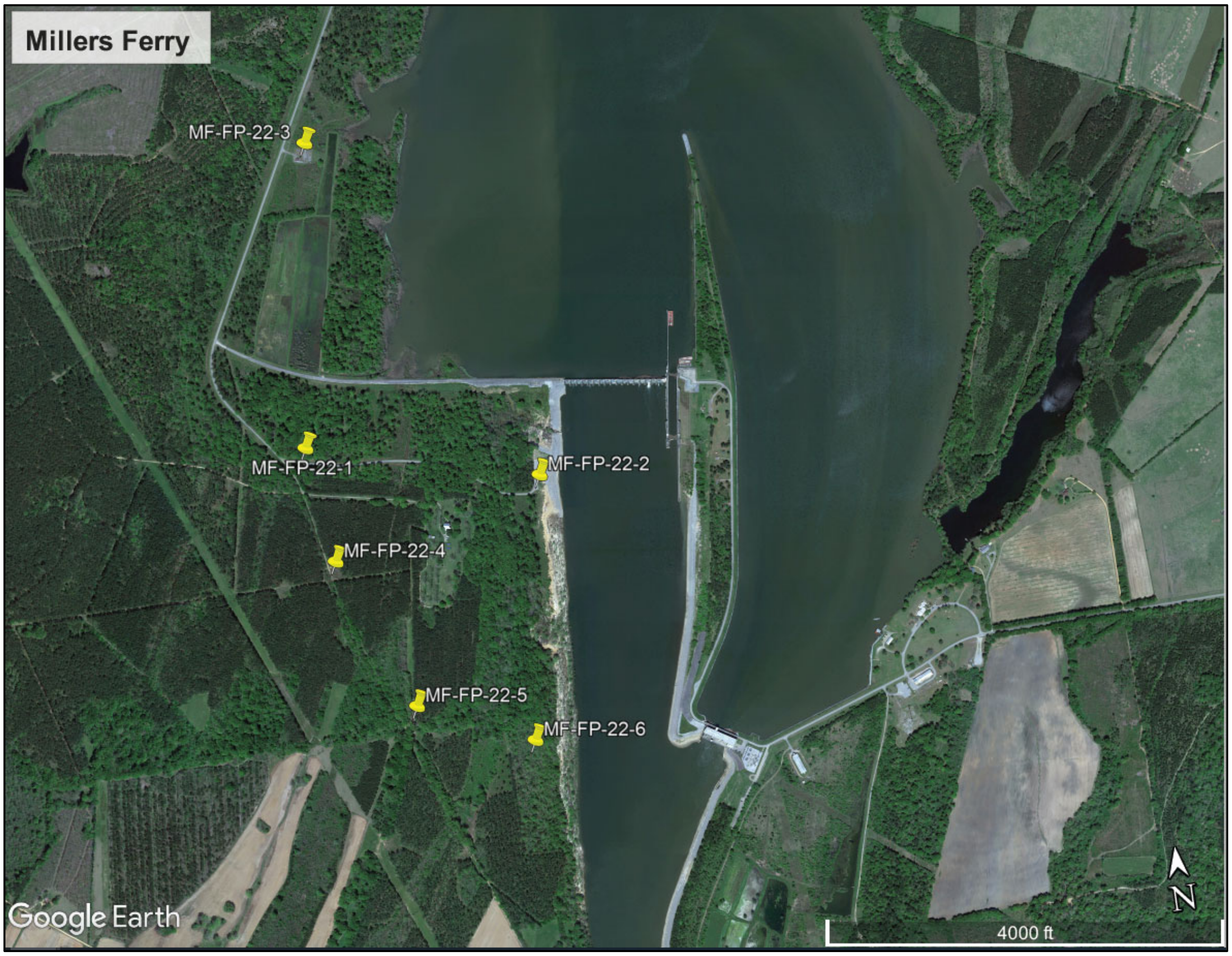


Figure G-13: Proposed geotechnical investigation boring locations at Millers Ferry requested during this feasibility study

The coordinates of the test boring locations are summarized in Table G-5.

Table G-5: Proposed boring locations with ground elevations and depths at Millers Ferry

Boring #	Bypass Station	Lat	Long	Boring Elev. (ft-MSL)	Borehole Depth (ft)
MF-FP-22-1	37+50	32.099886°	-87.410280°	+89	60
MF-FP-22-2	-	32.098266°	-87.402851°	+89	60
MF-FB-22-3	6+50	32.108214°	-87.408934°	+86	60
MF-FB-22-4	51+00	32.096676°	-87.409817°	+88	60
MF-BB-22-5	67+00	32.092420°	-87.407844°	+89	70
MF-BB-22-6	80+00	32.091046°	-87.404177°	+85	70

G.2.1.3. Subsurface Conditions

Geotechnical engineer will determine once boring investigation is completed.

G.2.1.3.1. Past Geotechnical Explorations

Refer to section G.1.1 through G.1.5 of this report.

G.2.1.4. Groundwater Conditions

Current groundwater elevations will be determined once the boring investigation is completed. The boring logs from the past indicated the groundwater elevation along the right bank ranged within elevations approximately 75 to 45 feet during initial investigation. However, the soils could have a higher groundwater elevation now that the river headwater is held at a higher pool elevation than the historic river pool elevation.

G.2.1.5. Lab Testing Program

Currently, the proposed lab testing is to preform USGS classification, Atterberg Tests, sieve analysis, specific gravity, unconfined compression tests, triaxial compression tests, quick direct shear tests, and consolidation tests from the disturbed and undisturbed soil samples collected in the field.

G.2.1.6. Major Subsurface Strata and Initial Material Properties

Geotechnical engineer will input index properties and stress strain properties after laboratory investigation of the proposed borings listed in Table G-5.

G.2.1.7. Geotechnical Engineering Analysis

Geotechnical engineer analysis that will be performed shall include but not limited to seepage analysis, slope stability analysis, risk assessment for the selected alternative, riprap and graduation requirements, and settlement analysis.

The historic groundwater elevation at Millers Ferry is higher than the proposed base of the bypass channel. Further analysis for the groundwater table shall include considerations for dewatering plans, excavation heave potential, and stability analyses to address the factor of safety during construction for the excavated slope.

Slope stability, seepage, and settlement analyses plan to use a computer aided program.

G.2.2. Claiborne Lock and Dam

The team looked at several fish passage alternatives along the right bank of Claiborne. The right portion of Claiborne contains the right dike to elevation approximately 40 feet, the abutment walls on the upstream and downstream side of the ogee spillway, the sheet pile wall within the right dike, and the ogee spillway.

G.2.2.1. Site Geology

Overburden soils generally along the right bank are expected to be 20 to 40 feet thick. Directly below the overburden material is a sand gravel layers that is atop the claystone foundation material. Per the DM's the material within the footprint of the lock and dam was able to be removed entirely by mechanical methods. The elevation of the gravel layer increases upstream of the centerline of the spillway. At the centerline of the spillway the top of the gravel layer is approximately 7 feet and increases up to about 18 feet in elevation nearly 350 feet upstream of the spillway. Bedrock is constant near elevation approximately-16 feet.

G.2.2.2. Future Subsurface Investigations

To support the feasibility level design for the fish passage channel three borings are proposed shown in Figure G-14.

Figure G-14: Proposed geotechnical investigation boring locations requested for Claiborne during this feasibility level study

The coordinates of the test boring locations are summarized in Table G-6 below.

Table G-6: Proposed boring locations with ground elevations and depths at Claiborne

Boring #	Bypass Station	Lat	Long	Boring Elev. (ft-MSL)	Borehole Depth (ft)
CB-FP-22-1	18	31.613985°	-87.554312°	+41.0	40
CB-FP-22-2	14	31.615144°	-87.554272°	+36.0	40
CB-FB-22-3	8	31.616615°	-87.554311°	+128.9	50

G.2.2.3. Subsurface Conditions
Geotechnical engineer will determine once boring investigation is completed.

G.2.2.3.1. Past Geotechnical Explorations
Refer to section G.1.6 through G.1.10 of this report.

G.2.2.3.2. Future Geotechnical Exploration Program
Soil borings were performed to develop a profile of the soil stratigraphy composing the right bank of the Alabama River at Claiborne Lock and Dam. Boring locations were limited due the funding available, unknown centerline and final elevations, presence of right dike, existing borings, aquatic drilling equipment availability, and present topography.

G.2.2.4. Groundwater Conditions
Current groundwater elevations will be determined once boring investigation is completed. The boring logs from the past indicated the groundwater elevation along the right bank was near elevation approximately 18 feet during initial investigation. However, the soils could have a higher groundwater elevation now that the river headwater is held at a higher pool elevation than the historic river pool elevation.

G.2.2.5. Lab Testing Program
Currently, the proposed lab testing is to perform USGS classification, Atterberg Tests, sieve analysis, specific gravity, unconfined compression tests, triaxial compression tests, quick direct shear tests, and consolidation tests from the disturbed and undisturbed soil samples collected in the field.

G.2.2.6. Major Subsurface Strata and Initial Material Properties
Geotechnical engineer will input index properties and stress strain properties after laboratory investigation of the proposed borings listed in Table G-6.

G.2.2.7. Geotechnical Engineering Analysis

Geotechnical engineer analysis that will be performed shall include but not limited to seepage analysis, slope stability analysis, risk assessment for the selected alternative, riprap and graduation requirements, and settlement analysis.

The historic groundwater elevation at Claiborne is higher than the proposed base of the bypass channel. Further analysis for the groundwater table shall include considerations for dewatering plans, excavation heave potential, and stability analyses to address the factor of safety during construction for the excavated slope.

Slope stability, seepage, and settlement analyses plan to use a computer aided program.

Exhibit G-1

Geotechnical Boring Logs and Lab Data