Appendix A

Engineering

Savannah Harbor Expansion Project (SHEP), Georgia and South Carolina, Fish Passage at New Savannah Bluff Lock and Dam (NSBLD) Post-Authorization Analysis Report and Environmental Assessment

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List of Acronyms

1D	1 Dimensional
2D	2 Dimensional
AEP	Annual Exceedance Probability
BC	Boundary Condition
CFS	Cubic Feet per Second
DEM	Digital Elevation Model
DS	Downstream
EIS	Environmental Impact Statement
FEMA	Federal Emergency Management Agency
FIS	Flood Insurance Study
GIS	Geographic Information System
GRR	General Reevaluation Report
HTRW	Hazardous Toxic Radioactive Waste
LOB	Left Overbank
MSL	Mean Sea Level
NAA	No Action Alternative

NAVD	North American Vertical Datum
NGVD	National Geodetic Vertical Datum
NLCD	National LandCover Database
NSBLD	New Savannah Bluff Lock and Dam
PDT	Project Delivery Team
PED	Planning, Engineering, and Design
PI	Periodic Inspection
ROB	Right Overbank
SHEP	Savannah Harbor Expansion Project
SPT	Standard Penetration Test
US	Upstream
USACE	US Army Corps of Engineers
WIIN	Water Infrastructure Improvements for the Nation Act
WSE	Water Surface Elevation

1. General

The Savannah Harbor Expansion Project (SHEP) includes a mitigation feature to provide a fish passage at the New Savannah Bluff Lock and Dam (NSBLD) to address adverse impacts to shortnose and Atlantic Sturgeon. The mitigation feature ensures compliance with the Endangered Species Act. The plan approved in the 2012 SHEP GRR and Final EIS was for construction and operation of a fish bypass around the South Carolina side of the NSBLD. In 2014, Savannah District performed a Periodic Assessment and Inspection of the lock and dam which revealed significant deterioration of the lock and dam, including structural issues. As a result, in May 2014 the Corps closed the lock indefinitely due to safety concerns. In addition, the Corps determined that the condition of the structure could adversely impact the function of the fish bypass around the lock and dam. Savannah District included additional activities in the FY2017 SHEP cost estimate update that would provide structural repairs to reduce the risk of a catastrophic failure of the dam and to ensure proper hydraulic operation of the fish passage.

In December 2016, the Water Infrastructure Improvements for the Nation (WIIN) Act was signed into law, requiring the Corps to study two in-channel fish passage options in lieu of the original design around the NSBLD structure. This report documents the hydraulic modeling used to evaluate alternatives for modification of the SHEP fish passage feature to meet the requirements of the WIIN Act of 2016.

1.1. History and Background

The New Savannah Bluff Lock & Dam is located at river mile 187 on the Savannah River in Richmond County, Georgia and Aiken County, South Carolina, about 13 miles Southeast of the City of Augusta. The dam consists of a lock and gated spillway. The lock chamber is located on the Georgia side of the river and is 56 feet wide and 360 feet long. The maximum lift is about 15 feet. The lock chamber was designed to provide 9 feet of clearance over the miter sills. The dam is a non-navigable moveable gate type. It is 360 feet long and contains five vertical-lift steel gates. Each gate is 60 feet long and located between concrete piers. The three non-overflow gates (middle) are 15 feet high and the two overflow gates (end) are 12 feet high. All five gates can be moved vertically a distance of 35.5 feet above the concrete sill. The operations building is an elevated structure located between the dam and the lock. The dam and operations structure are founded on timber piles, with an interlocking steel sheet pile cutoff wall driven to an impervious stratum of clay underlying the site. The spillway gates are remotely operated from J. Strom Thurmond Dam located approximately 40 miles upstream.

The New Savannah Bluff Lock and Dam project was authorized by the 1930 and 1935 Rivers and Harbors Acts for the sole purpose of improving commercial navigation on the Savannah River between the upper limits of the Savannah Harbor and Augusta, Georgia. Construction began in 1934 and was completed in 1937. In 1979, the last commercial shipment passed through the lock and consequently, maintenance of the navigation channel was discontinued. Funding for proper maintenance of the lock and dam was curtailed and it is currently in caretaker status. According to the most recent

Periodic Inspection (2014) the current condition of the project is poor. Major repairs and rehabilitation are required to assure a safe and reliable project. An additional inspection of the lock and dam structure was conducted in 2016 to develop a list of recommended repairs to support fish passage (Savannah District, 2017).

Figure 1 - New Savannah Bluff Lock and Dam - viewpoint looking upstream from the Georgia side.

Although the project no longer serves commercial navigation, the pool impounded behind the dam provides water supply for multiple users, including two municipalities and several industries. The pool impounded by the dam also supports water-related recreation opportunities such as general boating and fishing, specialized rowing, powerboat race events, regional economic development, and tourism. The lock was also operated to pass migratory anadramous fish species until it was closed in May of 2014 due to safety concerns of the stability of the lower riverside lock wall.

1.2. Pertinent Data

All elevations are reported in feet NGVD 29 for consistency with project drawings, gage data, and manuals. To convert from NGVD 29 to NAVD 88 at the Project Location (Lat 33.372370, Lon -81.941064), subtract 0.78 feet from the NGVD 29 elevation.

DAM:

Type	Non-navigable, movable
Length, feet	360

SPILLWAY:

Total Length, feet	347.5
Elevation of Sill	100.5
Elevation of Apron	90.5
Elevation of Access Walkway	153.5
Height of Piers, feet	67.5

GATES:

Gates (5)	3 non-overflow, 2 overflow
Type	Vertical-lift, crest control
Length, feet	60
Height, Non-overflow, feet	15
Height, Overflow, feet	12
Vertical Movement, feet	35.5
Method of Opening Gates	Fixed Hoists
Crest of Overflow Gates 1 & 5 in Closed Position	112.5
Crest of Non-Overflow Gates in Closed Position	115.5
Bottom of Non-Overflow Gates at Maximum Lift	121.0

LOCK CHAMBER:

Length, feet	360
Width, feet	56
Clearance over Miter Sills, feet	9
Maximum Lift, feet	15
Elevation of Top of Lock Wall	123.5
Elevation of Upper Sill	100.5
Elevation of Lower Sill,	89.5
Size of Culvert at Valves, feet	8 x 10

RESERVOIR:

Local Drainage Area	358 square miles
Total Drainage Area	7,508 square miles
Normal Pool, NGVD 29	114.5
Pool Operating Range, NGVD 29	112 - 115

2. Hydraulics

HEC-RAS was selected as the numerical model to determine likely changes to pool elevations as a result of any proposed modifications to the dam structure itself. HEC-RAS was selected because changes to the pool elevation will be a result of the unique hydraulics of a fish passage structure rather than a change in dam or reservoir operations.

Two levels of hydraulic modeling were conducted for this effort. A screening level evaluation using the existing 1D FEMA effective model of the Savannah River to quickly evaluate likely impacts of possible with-project alternatives. Subsequently, a more detailed analysis using the 2D flow module of HEC-RAS was conducted to better determine project impacts for a focused array of alternatives. The main project report includes additional discussion of alternative screening and plan formulation.

2.1. HEC-RAS 1D Model

The FEMA effective HEC-2 model for Aiken County South Carolina (this is the same model used for the Richmond County effective FIS) was obtained as a starting point for this analysis. The model was imported into HEC-RAS 5.0.3, and the river centerline was geo-referenced to Georgia State Plane East (feet) horizontal coordinate system using aerial imagery. The model uses the NGVD 29 vertical datum, and extends from 2 miles upstream of I-20 at Savannah Rapids Pavilion and downstream approximately 45.5 river miles to the floodplain downstream of the existing NSBLD structure. The model was determined to be sufficient for initial alternative screening, despite the following limitations:

- The existing FEMA effective model was originally developed with HEC-2 in November of 1994. The model did not contain any georeferenced data, and contained very sparse data within the overbanks. As such, this model is not capable of producing suitable visual flood mapping products.

- The existing FEMA effective model was developed to evaluate water surface profiles for large 100 year and 500 year flow events. During these flow conditions, the gates at the NSBLD structure would be fully opened to allow for unobstructed flow through the structure. Therefore, the model assumes the gates are always open. During medium or low flow conditions gates would be closed or partially opened to maintain pool levels, which is not reflected in the model.

The FEMA effective model uses a cross section with piers to represent the gates at NSBLD, indicating that the gates are fully open at all times. The gate cross section was changed to an inline structure for this analysis, with a gate grouping that also remained fully open at 45’ during high flow conditions. The sill elevation is at elevation 100.5’ NGVD 29, as imported from the FEMA model and as verified from original design drawings from USACE archives. Representing the dam structure and gates with an inline structure in HEC-RAS allows for low flow conditions to be modeled in accordance with regular operations of the dam to maintain a pool elevation.

2.1.1. Model Discharges

In addition to providing fish passage for the species of interest, is subject to the constraints of minimizing adverse impacts to water-supply intake structures during low flow conditions, and minimize impacts to residential landowners and nearby property owners during high flow conditions. As such, the screening level flows modeled were the Drought Trigger Level 3 flow of 3,100 cfs, the FEMA 100-year flow of 138,000, and an average high flow condition of 8,000 cfs. Each of these flows were entered in the HEC-RAS steady flow data editor.

Table 1 - Modeled Flow Conditions in 1D Screening

Event	Flow
DROUGHT LEVEL 3	3,100 CFS
AVERAGE HIGH	8,000 CFS
FEMA 100-year	138,000 CFS

The drought flow was used to estimate the impact a rock-weir fish passage structure may have on pool levels during low flows as compared to existing conditions of NSBLD. Similarly, the FEMA 100-year flow was used to determine whether fish passage alternatives would increase water surface elevations to prohibitive levels for high flows in the study area. The average high flow was used to determine what the pool stage would be for normal flow conditions.

2.1.2. Geometry Modifications

The FEMA existing condition model was assumed to be reasonably accurate, and no attempt was made to further calibrate the model to observed data. This was also to preserve model continuity to pursue a no-rise certificate, if possible. However, modifications to the model at the NSBLD were required to represent the proposed fish passage structure. The rock weir with-project condition alternatives can be modeled in two ways within the HEC-RAS 1-D environment:

1) Utilizing a series of cross sections to represent the structure. Manning's n-values and the manning's equation are used to calculate flow and stage over the structure instead of a weir coefficient. The manning's n values for the rock ramp used for this approach will vary based on the site-specific configuration and design, but can be estimated from available literature.

2) Inserting an inline structure element, with US and DS embankment side slopes as a broad crested weir. This requires a weir coefficient, which can be estimated within a range but ideally should be a calibrated parameter and site specific for every structure.

The Cape Fear Dam Removal and Fish Passage project used the methodology found in EM 1110-2-1601 to develop a range of n-values from 0.056 to 0.078 for the Cape Fear rock weir, and ultimately landed on a conservative n-value for the rock ramp of 0.08. A manning's n value of 0.08 for the rock weir was adopted for the NSBLD analysis as well, using methodology #1 above to model to the rock weir. A profile plot of the rock-ramp as represented by cross sections in the HEC-RAS geometry can be seen in Figure 2 below.

The second modeling option above using an inline structure element allows for much easier geometry modification for the various rock-ramp configurations being evaluated in HEC-RAS. In order to run multiple alternative scenarios, a weir coefficient must be determined for the rock ramp inline structure. The weir coefficient is a measure of how hydraulically efficient a structure is, with larger coefficients for more efficient structures. The weir equation as used in HEC-RAS[1] is shown below.

$$Q = CLH^{1.5}$$

Q = Flow rate
C = weir flow coefficient

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[1] HEC-RAS Hydraulic Reference Manual, V4.1, page 6-35

L= weir length
H= weir energy head

A simulation using the modified cross-section methodology with n values of 0.08 was first created to act as a soft calibration target for the weir coefficient. The 100-year WSE output from a set of cross sections with a crest of 110 and US and DS slopes of 10% and 1.3% respectively was computed and the results tabulated. A separate scenario using an inline structure element was also created, with the weir coefficient iteratively adjusted until the computed water surface elevations matched those from the modified cross-section methodology. The resulting inline structure weir coefficient was 0.65, which will was used to run alternative weir configurations varying weir length and elevation. A profile plot of the rock-ramp as represented by cross sections in the HEC-RAS geometry can be seen in Figure 3 below.

It is important to note that since C is not constant for all flow conditions; this causes the inline structure method to under predict water surfaces computed by the cross-section method by approximately 0.65' during drought flow conditions of 3,100 CFS. The inline structure approach was used to model and evaluate various screening level weir configurations going forward.

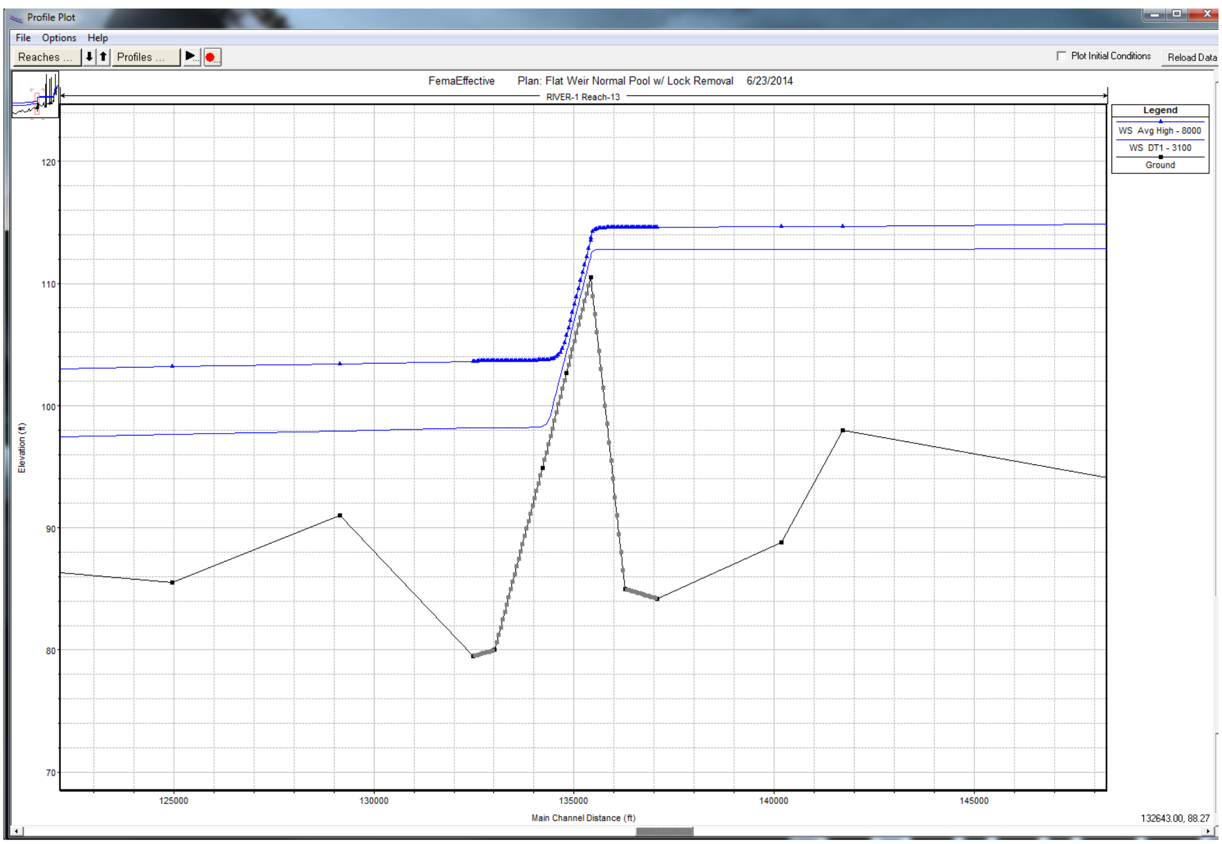


Figure 2 - Rock Ramp using HEC-RAS Cross Sections

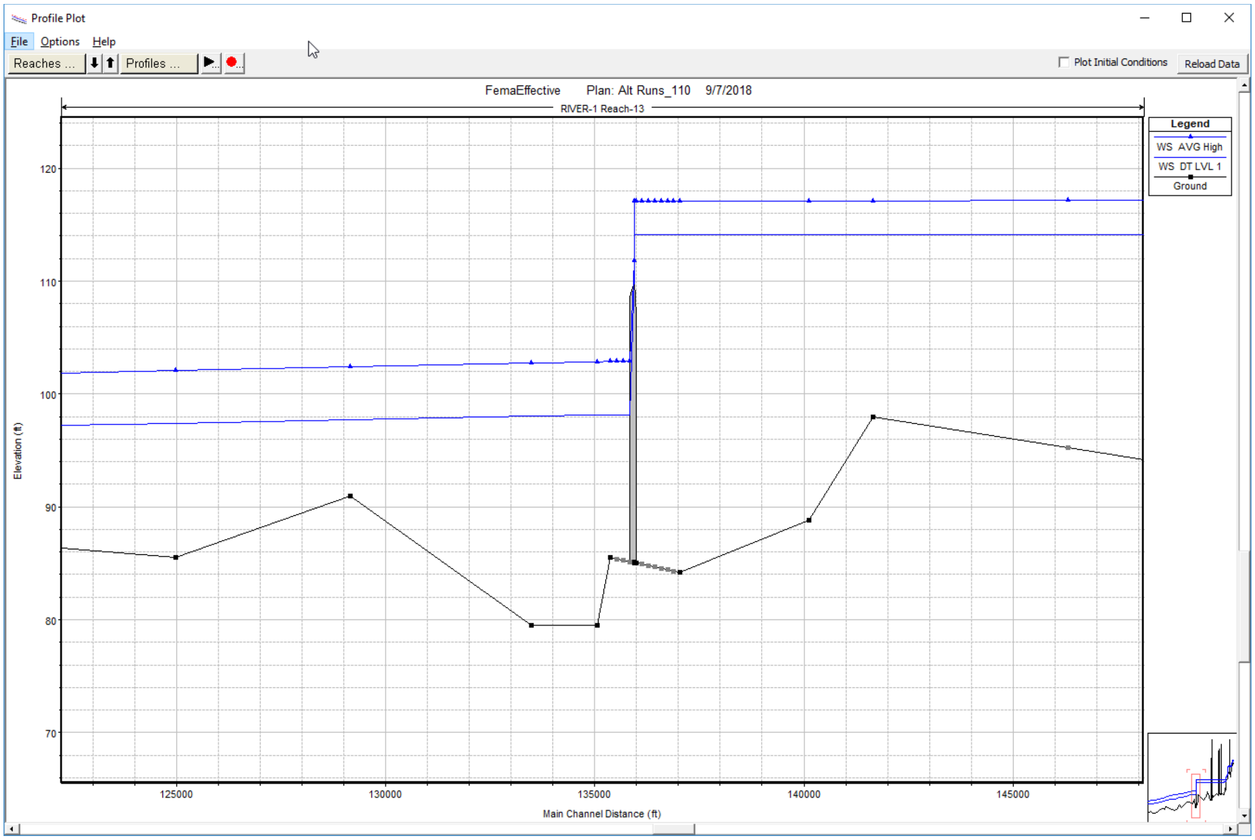


Figure 3 - Rock Ramp using HEC-RAS Inline Structure

2.1.3. Modeled Scenarios

Twenty-four scenarios varying rock-weir crest length and elevation were modeled in the modified 1D HEC-RAS model; modeled weir length varied from 380 feet to 920 feet. Differing combinations of weir length and weir height were considered. The dam structure of NSBLD is approximately 380 feet in length; an assumed total removal of the dam is the basis for selecting 380 feet as the minimum length for the rock weir. Total removal of both the dam and lock would result in a rock weir approximately 500 feet in length, the full width of the river channel. A concrete esplanade to the north of the land-side lock wall could be removed for an additional 50 feet of weir length without impacting the access road through the park. A scenario that includes removal of the lock, dam, and esplanade would results in a rock weir 550 feet in length. Finally, the total distance from the South Carolina riverbank to the north boundary of the park property is approximately 950 feet. A scenario that includes removal of the lock, dam, and esplanade as well as the excavation of a majority of the existing park would result in a weir approximately 920 feet in length. These 24 scenarios were modeled, with variations on the weir crest elevation as summarized in the table below.

Table 2 - Scenarios for the 1D HEC-RAS Model

Scenario	Weir Length (feet)	Crest Elevation (NGVD 29)	Scenario	Weir Length (feet)	Crest Elevation (NGVD 29)
1	380	107	**13**	550	107
2	380	108	**14**	550	108
3	380	109	**15**	550	109
4	380	110	**16**	550	110
5	380	111	**17**	550	111
6	380	112	**18**	550	112
7	920	107	**19**	500	107
8	920	108	**20**	500	108
9	920	109	**21**	500	109
10	920	110	**22**	500	110
11	920	111	**23**	500	111
12	920	112	**24**	500	112

The scenarios listed above were modeled in the 1D HEC-RAS model and their resulting water surface profiles computed. Of interest are the water surface elevations for low flows, which may have an impact to water-supply users upstream, and water surface elevations for high flows which may induce additional flooding. Changes in water surface elevations compared to existing conditions are likely to be greatest near the dam. The first water supply intake upstream of NSBLD, the PCS Nitrogen/Fibrant intake, is therefore most likely to be impacted by lower pool elevations. The raw water pumped from this intake is shared by multiple corporations and is approximately seven miles upstream of NSBLD, as shown in Figure 4 below. The first inhabited structures upstream of NSBLD are along Gum Swamp Road, approximately two miles upstream of the dam. Changes in water surface elevation at these two locations were evaluated to determine if a modeled scenario should be considered for more detailed analysis. The results of the 1D model and the changes in the water surface at PCS Nitrogen/Fibrant and Gum Swamp Road are shown in Table 3 below.

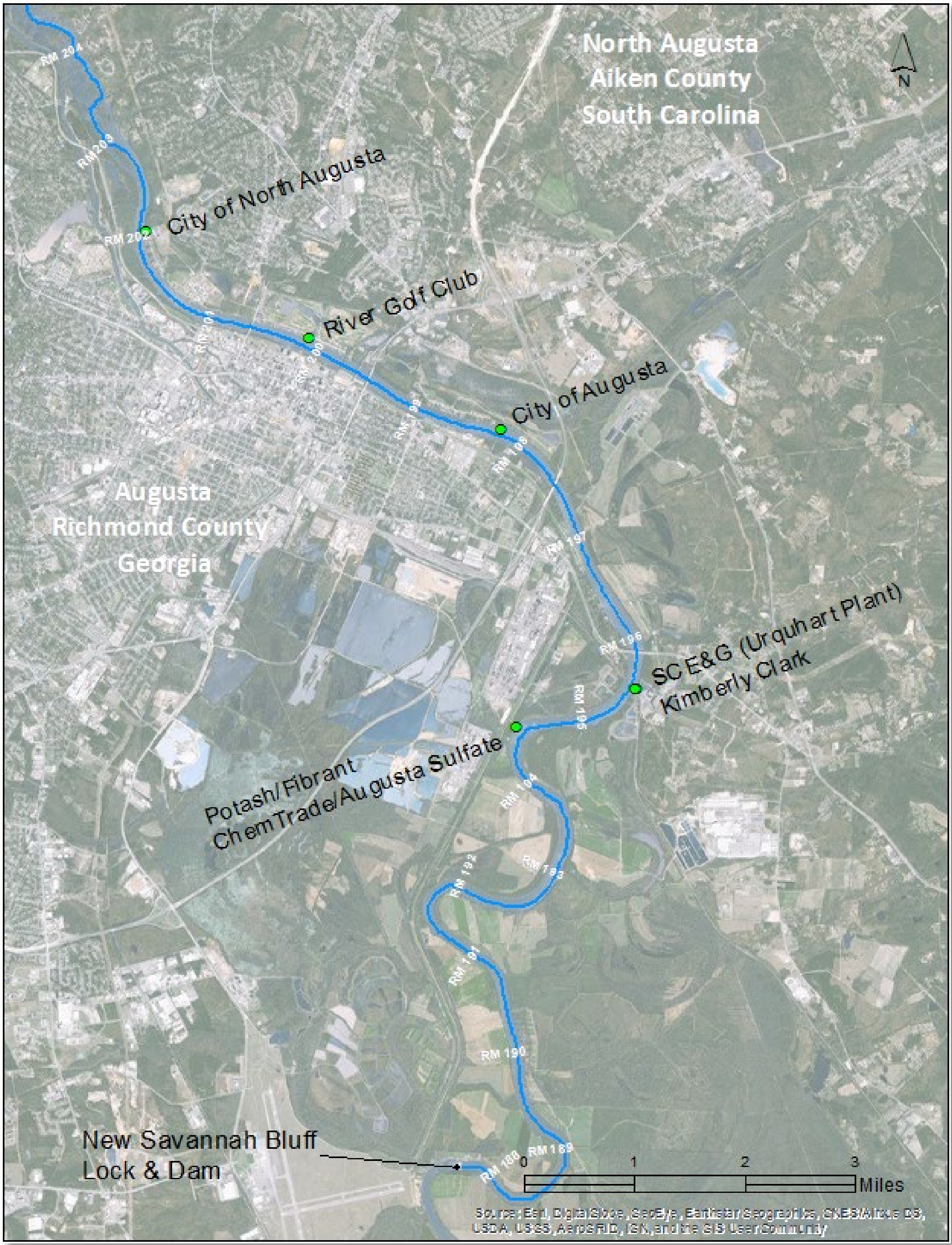


Figure 4 - Water Supply Intake Locations

Table 3 - 1D HEC-RAS Model Results

Scenario	Weir Length (ft)	Weir Elevation (ft NGVD 29)	100-yr WSE (ft NGVD 29)		Drought WSE (ft NGVD 29)	
			Gum Swamp	PCS Nitrogen/Fibrant	Gum Swamp	PCS Nitrogen /Fibrant
Existing Conditions			125.51	130.31	114.5	114.5
1	380’	107	126.86	130.64	112.48	112.66
2	380’	108	126.89	130.65	113.47	113.61
3	380’	109	126.93	130.67	114.46	114.57
4	380’	110	126.96	130.67	115.44	115.53
5	380’	111	126.99	130.69	116.15	116.22
6	380’	112	127.02	130.7	116.64	116.7
7	500’	107	125.87	130.38	111.56	111.8
8	500’	108	125.9	130.39	112.49	112.67
9	500’	109	125.92	130.39	113.44	113.58
10	500’	110	125.94	130.4	114.36	114.47
11	500’	111	125.97	130.41	115.28	115.36
12	500’	112	125.99	130.41	116.05	116.12
13	550’	107	125.84	130.37	111.29	111.55
14	550’	108	125.86	130.38	112.24	112.43
15	550’	109	125.89	130.39	113.18	113.33
16	550’	110	125.92	130.39	114.12	114.24
17	550’	111	125.95	130.4	115.06	115.15
18	550’	112	125.98	130.41	115.9	115.98
19	920’	107	125.52	130.31	110.1	110.48
20	920’	108	125.53	130.31	111.04	111.32
21	920’	109	125.54	130.31	111.98	112.19
22	920’	110	125.57	130.32	112.91	113.07
23	920’	111	125.58	130.32	113.88	114
24	920’	112	125.6	130.33	114.8	114.9

The weir configuration for fish passage that is ultimately adopted must balance maintaining a pool for water supply and minimizing residential flooding impacts, while keeping construction costs reasonable. The most influential parameter to change in 100-year water surface elevation is weir length. Weir height does have some impact within each model grouping, however it is less significant in relation to length.

A weir 380 feet in length, regardless of the crest elevation, would likely be unable to meet project objectives due to increases to the 100-yr water surface. Alternatively, a weir 920 feet in length would better meet project objectives but would likely be much more expensive to construct. The 500 foot wide and 550 foot wide alternatives yield nearly identical results.

Based on the results of the preliminary screening and associated impacts to flooding and water supply, the following scenarios from the 1D analysis were carried forward for a more detailed analysis using HEC-RAS 2D:

Table 4 - Scenarios to be Evaluated in 2D HEC-RAS

Scenario	Weir Length (ft)	Weir Elevation (ft NGVD 29)	Impacts to 100-yr WSE <1ft	Impacts to Drought WSE <1ft	Carried forward to 2D Analysis
4	380’	110	No	Yes	**Yes**
7	500’	107	Yes	No	**Yes**
10	500’	110	Yes	Yes	**Yes**
22	920’	110	Yes	No	**Yes**

The 380’ and 920’ weir lengths serve as the minimum and maximum weir lengths that were considered in the more detailed 2D analysis. The 500’ weir width scenarios serve as a starting point for additional fish passage structure refinement that was carried out in the 2D analysis phase discussed below.

2.2. HEC-RAS 2D Model

A 2D HEC-RAS model was used to further evaluate alternatives from the 1D FEMA effective model exercise discussed above; only the alternatives that seemed most likely to meet project objectives were evaluated with the 2D model. The 2D model was built for the primary purpose of evaluating flooding impacts of the proposed alternatives, though other information can be extracted from the model results as well. Discussion of the data sources and model development is presented below.

2.2.1. Terrain Data

LiDAR point cloud data were obtained for the study area from https://coast.noaa.gov/dataviewer/# and processed by the Savannah District GIS group. See section 3.1 for additional discussion of terrain data. The extents of the terrain model used for this project are presented in Figure 5.

2.2.2. Model Geometry

2.2.2.1. *2D Mesh*

Two 2D flow areas were used to model the study area: one representing the Savannah River and overbanks (Savannah_R), and another representing the leveed area behind the Augusta Levee (Aug_levee_area). The effective FEMA 100-year flood inundation limits were used to determine the extents of the 2D flow areas generally, though smaller tributaries and fingers of inundation were not necessarily included in the mesh. The 2D flow area representing the leveed portions of downtown Augusta was created using a regular cell spacing of 500’ x 500’, with very little modification to the mesh beyond the regular cell spacing; breaklines were not used in the Aug_levee_area 2D flow area.

The Savannah River 2D area was created with a 250’ x 250’ regular cell spacing, with breaklines used to delineate hydraulically significant features such as elevated roadways. The mesh was further refined in the vicinity of NSBLD to accommodate the inclusion of a storage area (SA)/2D connection to represent the dam. The dam itself is represented with a weir and gates, copied directly from the FEMA effective model and validated against as-built drawings for the structure (all elevations were converted to NAVD 88). The 2D flow areas and model geometry used for this analysis are shown in Figure 6.

2.2.2.2. *Augusta Levee*

The 2D flow areas were connected to one another using an SA/2D connection representing the Augusta levee. Levee crest station-elevation data were obtained from the most recent version of the O&M manual, which contains survey data from 2009.

2.2.2.3. *Landuse and Manning’s n Values*

The National Land Cover Database (NLCD 2011) was obtained for the study area and used to inform manning’s n values for the model geometry. A manning’s n value was assigned to each landuse category within the study area, based on several sources, as presented in Table 5. Manning’s n values for natural channels are difficult to quantify outside of a laboratory setting and are subject to the professional judgment and experience of the hydraulic engineer.

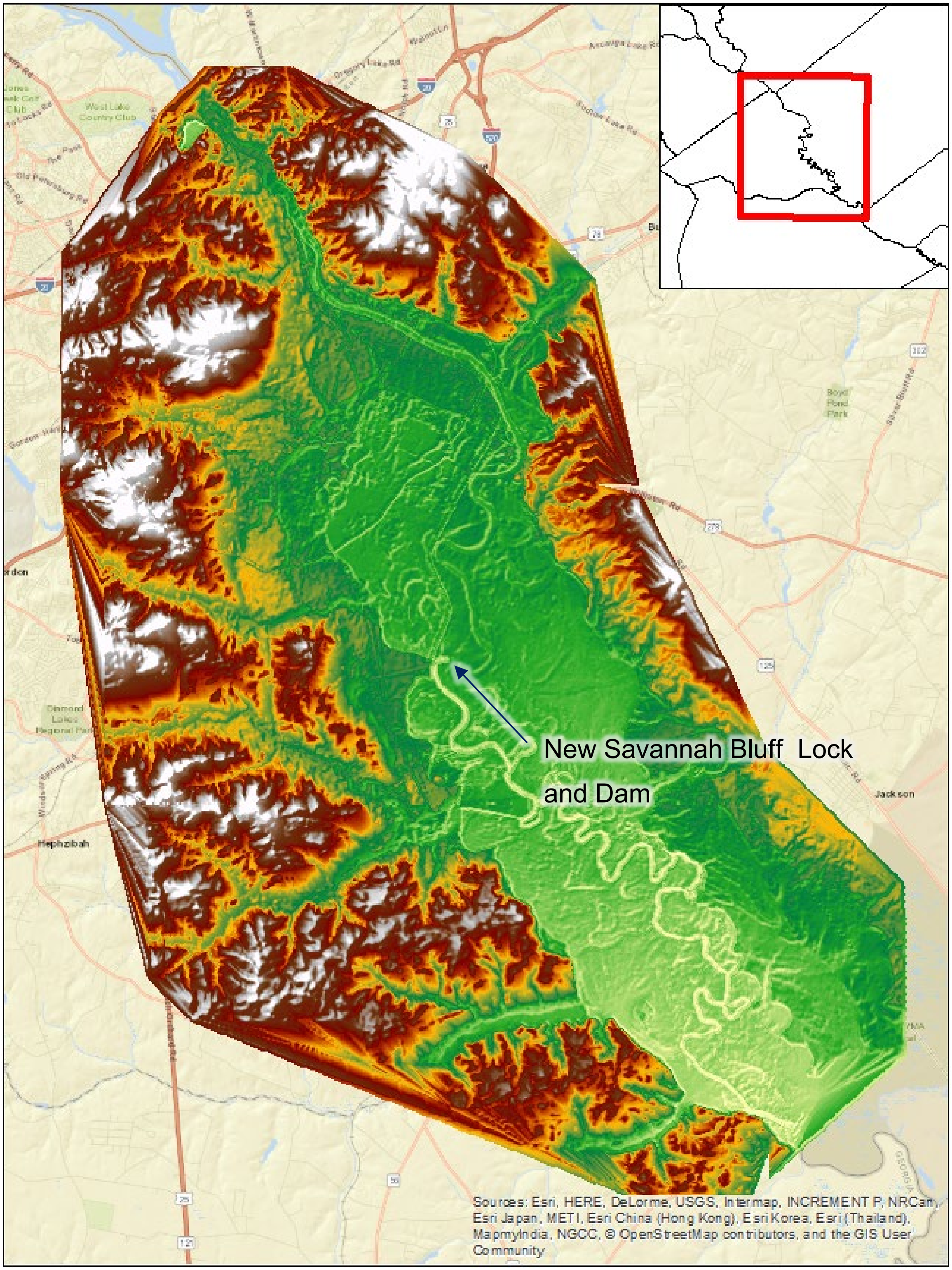


Figure 5 - Terrain Model Extents

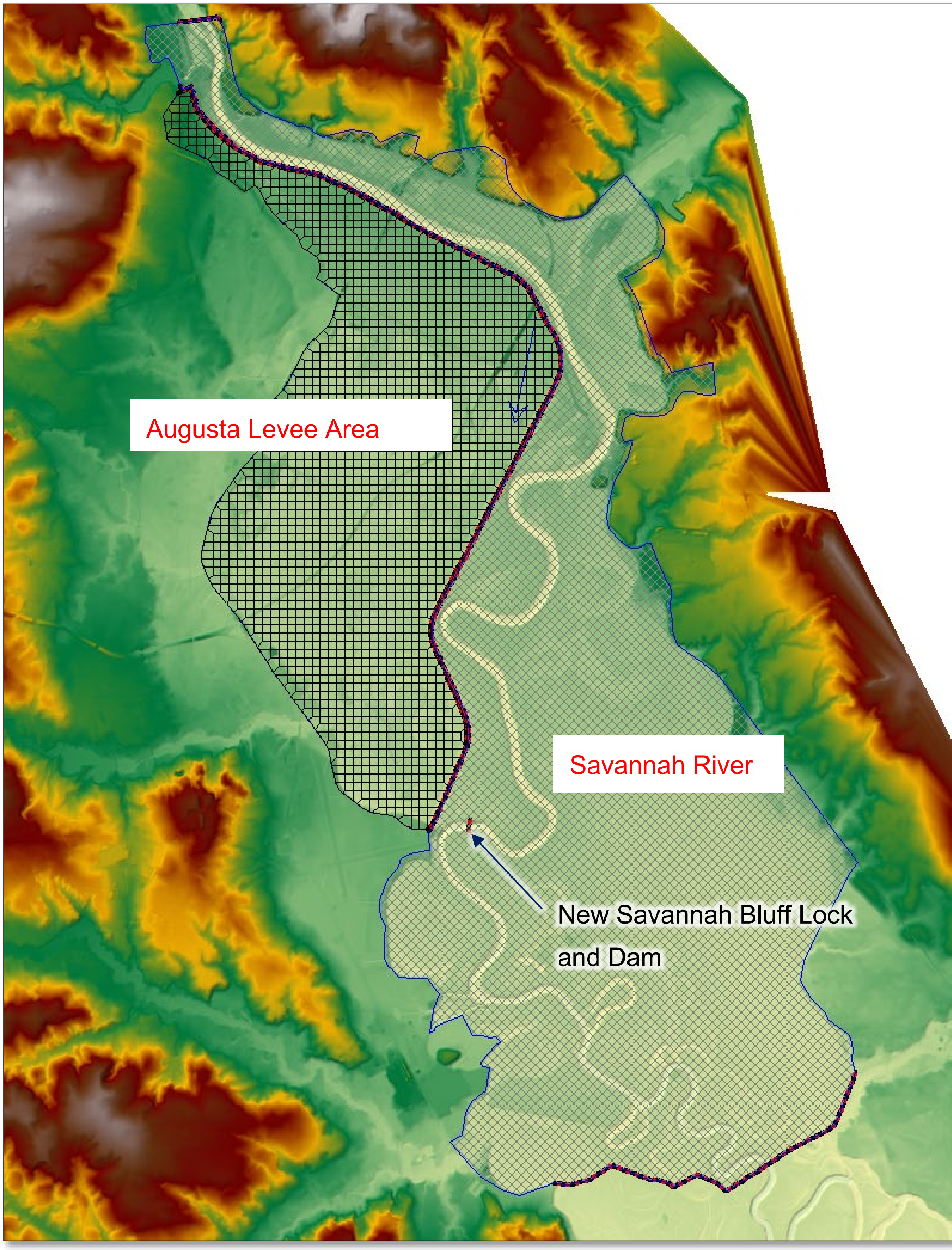


Figure 6 - 2D Model Schematic

Table 5 - Manning's n values by NLCD Landuse

NLCD Value	NLCD Classification	Manning's n	Source
-	default/no data	0.06	Default from HEC-RAS
31	barren land rock/sand/clay	0.03	\1 C.b.4
82	cultivated crops	0.035	\1 D-2.b.2
41	deciduous forest	0.12	\1 D-2.d.5
21	developed, open space	0.013	Jung et al.
22	developed, low intensity	0.05	Jung et al.
23	developed, medium intensity	0.075	Jung et al.
24	developed, high intensity	0.1	Jung et al.
95	emergent herbaceous wetlands	0.1	\1 D-1.a.8
42	evergreen forest	0.12	\1 D-2.d
71	grassland/herbaceous	0.03	\1 D-2.a.1
43	mixed forest	0.12	\1 D-2.d
11	open water	0.03	\1 D-3.a
81	pasture/hay	0.04	\1 D-2.b.3
52	shrub/scrub	0.05	\1 D-2.c
90	woody wetlands	0.1	\1 D-1.a.8
\1 *Open-Channel Hydraulics*, by Chow, 1959. Table 5-6			

The NLCD landuse data described above were associated with the model geometry, and manning's n values assigned to the various landuse types within the footprint of the 2D flow area. Manning's n polygon regions were further used to refine roughness values for the channel portions of the mesh, to better match values used in the effective FEMA model. A polygon with an n value of 0.033 was used to describe the channel from NSBLD to the CSX railroad bridge eight miles upstream. A polygon with an n values of 0.031 was used for the channel portion of the model upstream of the CSX railroad bridge. While the Manning's n values adopted were originally developed for the effective FEMA model, the Manning's n values in Table 5 were validated in the 2D flow domain to ensure modeled river stages matched observed gage data for several stage gages throughout the basin (namely the USGS Gage at Jefferson Bridge, USGS 02196670) as well as at the NSBLD gage, USGS 02197000). The 2D HEC-RAS model was technically reviewed by both David Ford Consulting Engineers as well as by the Hydrologic Engineering Center (HEC), USACE. See Table 5 for the n values that were associated with the various landuse categories within the study region.

2.2.2.4. *Boundary Condition (BC) Lines*

One boundary condition line was used at the upstream side to define inflow to the system. Three boundary condition lines were used on the downstream end for the left overbank (LOB), channel, and right overbank (ROB) flow to allow flow to leave the system. The BC lines can be seen on the upper and lower extents of the HEC-RAS 2D model in Figure 6. The slope of the channel and overbank areas were used to estimate a normal depth slope at the location of the BC lines and normal depth slope was used as the boundary condition type. The model extends several miles downstream from NSBLD, such that any inaccuracies in the normal slope estimates should not impact calculations in the area of interest.

2.2.3. Flow Data

The 2D environment in HEC-RAS is inherently unsteady, in that properties of flow (e.g. depth) vary with time. The focus of the analysis was not on how flow properties vary in time, but on how they vary as a result of changes to the structure at NSBLD. Various constant flow hydrographs were used to evaluate changes to water surface elevations in the main river channel and depths in the overbank areas as a result of the inclusion of a fish passage structure. The range of flow conditions that were evaluated include drought condition flows, average low flows, and a suite of recurrence interval flows. The magnitude of the constant flow hydrographs used in this analysis are presented in Table 6 below. These flows were applied as a constant inflow hydrograph at the upstream boundary condition line described in section 2.2.2.4.

Table 6 - Modeled Flow Conditions in 2D Screening

Event	Flow (cfs)	Description
DROUGHT LEVEL 3	3,600	Drought releases from Thurmond Dam
AVERAGE LOW	5,000	25% percentile mean daily flow
AVERAGE HIGH	8,000	66% percentile mean daily flow
2-Year	33,000	2-yr flow from Aiken County FEMA Model*
5-Year	41,000	5-yr flow from Aiken County FEMA Model*
10-Year	59,800	Aiken County FIS
25-Year	80,000	25-yr flow from Aiken County FEMA Model*
50-Year	103,000	Aiken County FIS
100-year	138,000	Aiken County FIS

* The 2-year, 5-year, and 25-year flow values were included in the FEMA effective model for Aiken County, but are not documented in the supporting FIS.

The drought plan for reservoirs on the Savannah River calls for a minimum flow of 3,600 cfs at NSBLD, which can be met by releases from Thurmond Dam if local and tributary inflows do not meet this level. This flow differs from the 3,100 cfs used as the low flow condition during the 1D model evaluation as there is some latitude in determining what actual flow in the river would be during drought conditions. A worst case scenario of no tributary inflow would trigger a release of 3,600 cfs from Thurmond, as used in the 2D model. A scenario in which tributary inflow equaled or exceeded 500 cfs (primarily from Stevens Creek) would allow Thurmond releases to drop to 3,100 cfs, which is what was used in the 1D model. In either case, a minimum flow of 3,600 cfs is always required at NSBLD.

Average low and average high mean daily flows were obtained from a statistical analysis of the daily flow data at USGS gage 02197000. The average high value of

8,000 cfs was also used in the basis of design for the 2014 SHEP Fish Passage Mitigation Feature discussed elsewhere in this report. The results of the statistical analysis showing the non-exceedance probability for various flow levels at NSBLD are shown in Figure 7 below. The non-exceedance probability, for average daily flows, means that for a certain percentage of time over the observed period of record, mean daily flow values do not exceed that threshold value. For example, over the period of record flow in the river did not exceed 5,000 cfs 25 percent of the time.

Daily non-exceedance flows differ from annual exceedance probability (return interval) flows in that annual exceedance probability is calculated using the highest peak flow that occurs every year in the period of record and not the daily flow values. This provides the probability of a large flow event occurring in any given year. For example, 1% AEP (100-yr) flow has a 1 percent chance of occurring in any given and does not mean that the flow is exceeded 1 percent of the time. Daily non-exceedance and annual exceedance probability measure two different aspects of flow in the river, but both can be used to evaluate changes to the river for with-project alternatives.

The 2-year flow used in the analysis roughly corresponds to the bank-full discharge of the Savannah River near Augusta, though some low lying areas may experience flooding at lower flow levels. Gate operations for the existing conditions at NSBL&D were set to “elevation controlled gates” within the HEC-RAS model, such that the gates were fully opened when the pool elevation exceeded 114.5 at the dam.

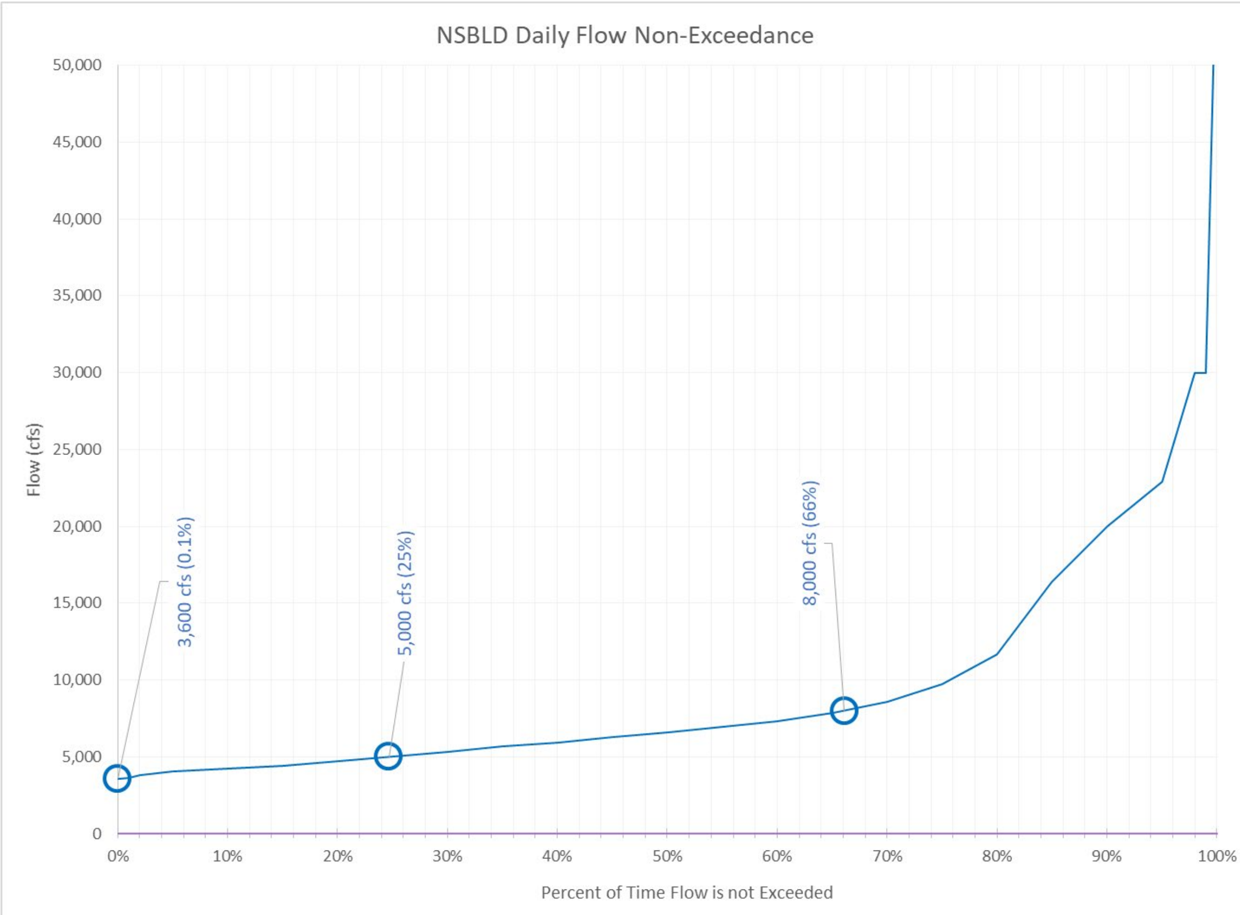


Figure 7 - Flow Non-Exceedance at New Savannah Bluff Lock and Dam

2.3. Fish Passage Alternatives and Results

Any modifications to the structure of New Savannah Bluff Lock and Dam are likely to have an impact on water surface elevations within the pool of the Savannah River upstream of the existing lock and dam. Of primary interest are impacts to flooding, navigation, recreation, and water supply due to changes in pool elevations.

The alternatives that were not screened out in the 1D HEC-RAS evaluation were carried forward to the 2D HEC-RAS model for further analysis (see Table 4). Based on PDT discussion, alternatives in addition to those from the 1D HEC-RAS model screening were analyzed with the 2D HEC-RAS model as well. All alternatives evaluated fall into two broad categories, which are prescribed in the 2016 WIIN Act:

1) Repair of the NSBLD lock wall and modification of the NSBLD structure to facilitate fish passage

2) Removal of the NSBLD structure with construction of fish passage structure

A total of eighteen alternatives were evaluated with the 2D HEC-RAS model, a list of which is presented below. The numbering scheme for the alternatives matches the categories of action presented in the WIIN Act, with alternatives that repair the NSBLD lock wall and structure having leading “1”, while alternatives that would remove NSBLD have a leading “2”. A list of alternatives evaluated with the 2D HEC-RAS model is presented in Table 7 below, and a detailed description of each alternative is presented in the following sections. Alternatives 1-2, 2-3, 2-5, and 2-9 were carried forward from the initial 1D HEC-RAS analysis discussed in Section 2.1. The remainder of the alternatives presented here were developed through ongoing discussion with the PDT and iterative refinement of the alternatives from the 1D HEC-RAS model.

Table 7 - Fish Passage Alternatives for 2D Analysis

Alternative No.	Alternative Description
Existing	Existing conditions at NSBLD as of the date of enactment of the WIIN Act
2012 SHEP GRR (NAA)	Construct a 285’ wide fixed crest weir at elevation 110 around SC side of NSBLD
Upstream	Remove dam, construct fish passage weir upstream of existing dam location
Downstream	Remove dam, construct fish passage weir downstream of existing dam location
1-1	Repair lock wall, retain dam, 200’ wide fish passage ramp on GA side
1-2	Repair lock wall, remove dam, 380' wide fish passage ramp in place of dam
2-1	Remove dam, 500’ wide fixed crest weir @ elevation 105
2-2	Remove dam, 500’ wide fixed crest weir @ elevation 106
2-3	Remove dam, 500’ wide fixed crest weir @ elevation 107
2-4	Remove dam, 500’ wide fixed crest weir @ elevation 107.6
2-5	Remove dam, 500’ wide fixed crest weir @ elevation 110
2-6A	Remove dam, 500’ wide fixed crest weir @ elevation 110 with floodplain bench
2-6B	Remove dam, 500’ wide fixed crest weir @ elevation 107 with floodplain bench
2-6C	Remove dam, 500’ wide fixed crest weir @ elevation 108 with floodplain bench
2-6D	Remove dam, 500’ wide fixed crest weir @ elevation 109 with floodplain bench
2-7	Remove dam, 500’ wide fixed crest weir @ elevation 110 with bypass channel and one 50’ wide gate
2-8	Remove dam, 500’ wide fixed crest weir @ elevation 110 with bypass channel and two 50’ wide gates
2-9	Remove dam, excavate park, 920’ wide fixed crest weir @ elevation 110

For the purposes of this report and in the following sections, all discussion of “normal pool” or “normal flow conditions” correspond to the conditions within the pool for a flow of 5,000cfs. There is a range of flow conditions and pool elevations that would be considered “normal”, but the 5,000cfs level is representative of what might be seen on an average day and provides a stable point against which changes to the pool can be evaluated for all alternatives.

2.3.1. Existing Conditions

The existing conditions model includes the NSBLD structure as described in Section 1.1 and Section 1.2. Gates at the structure are operated to pass high flows and to maintain the pool during low flows. The pool elevation at the dam is maintained between elevation 113.2 and 113.7 NAVD 88 for normal flow conditions, which results in a pool elevation of around114.3 NAVD 88 at 5th Street Bridge. Normal flow conditions here corresponds to 5,000cfs mean daily flow. The pool elevation at NSBLD on the date of enactment of the WIIN Act (16 December, 2016) was 113.98 NAVD88, with an average daily flow of 4,210cfs.. For high flow (flood) conditions the dam gates are opened fully to reduce flooding impacts.

2.3.2. No Action Alternative (NAA) 2012 SHEP GRR Plan

The plan for fish passage that was developed as part of the 2012 SHEP GRR and Final EIS includes the construction of a rock-weir fish passage structure around the south side of NSBLD in South Carolina. This plan leaves in place the NSBLD structure but modifies two of the dam's operational gates to direct additional flow through the fish passage structure. A more detailed discussion of this alternative can be found in the NSBLD Fish Passage Basis of Design (TetraTech, 2014). A schematic of the general site configuration and terrain model used for the 2012 SHEP GRR Alternative can be seen in Figure 8 below. This alternative was not considered as a possibility for construction as it does not meet the requirements of the WIIN Act, but instead serves as a point of comparison against which to judge the remaining alternatives. This alternative is carried forward in the analysis as the NAA as required by National Environmental Policy Act.

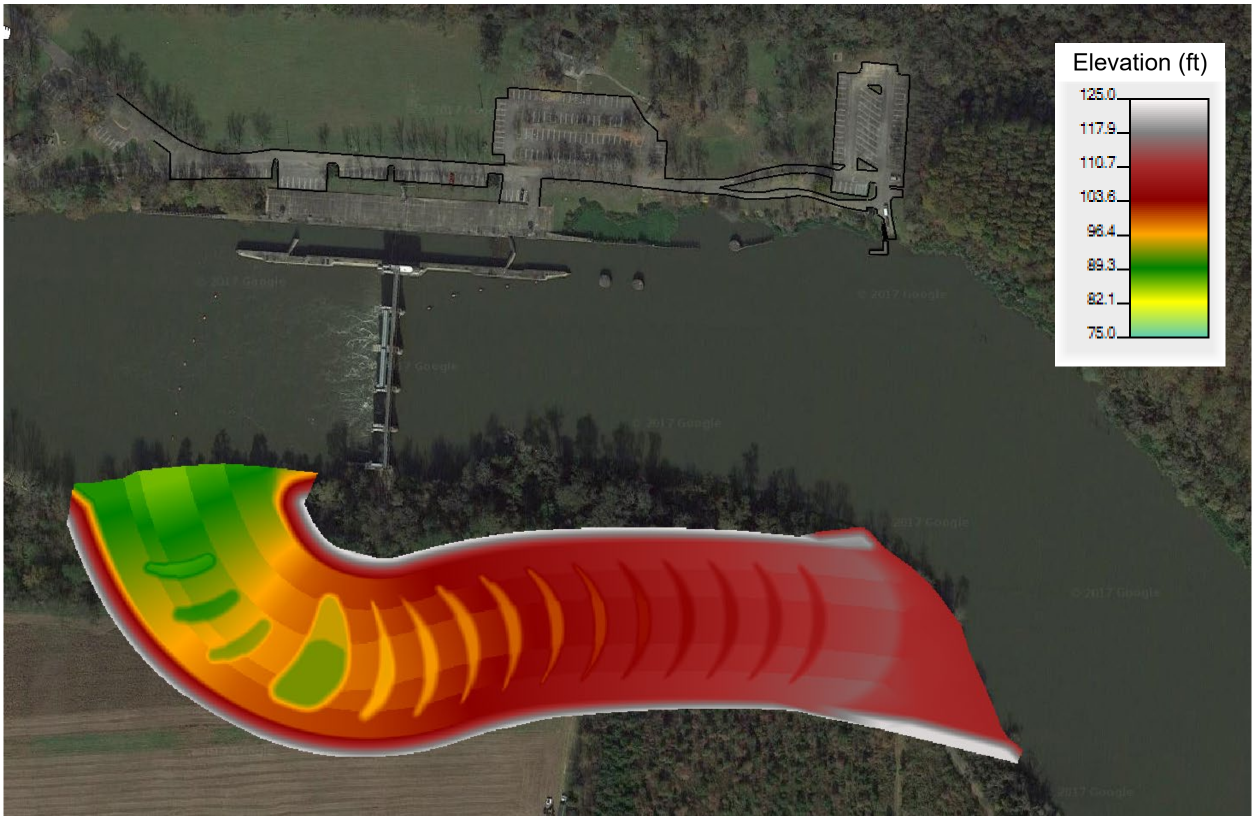
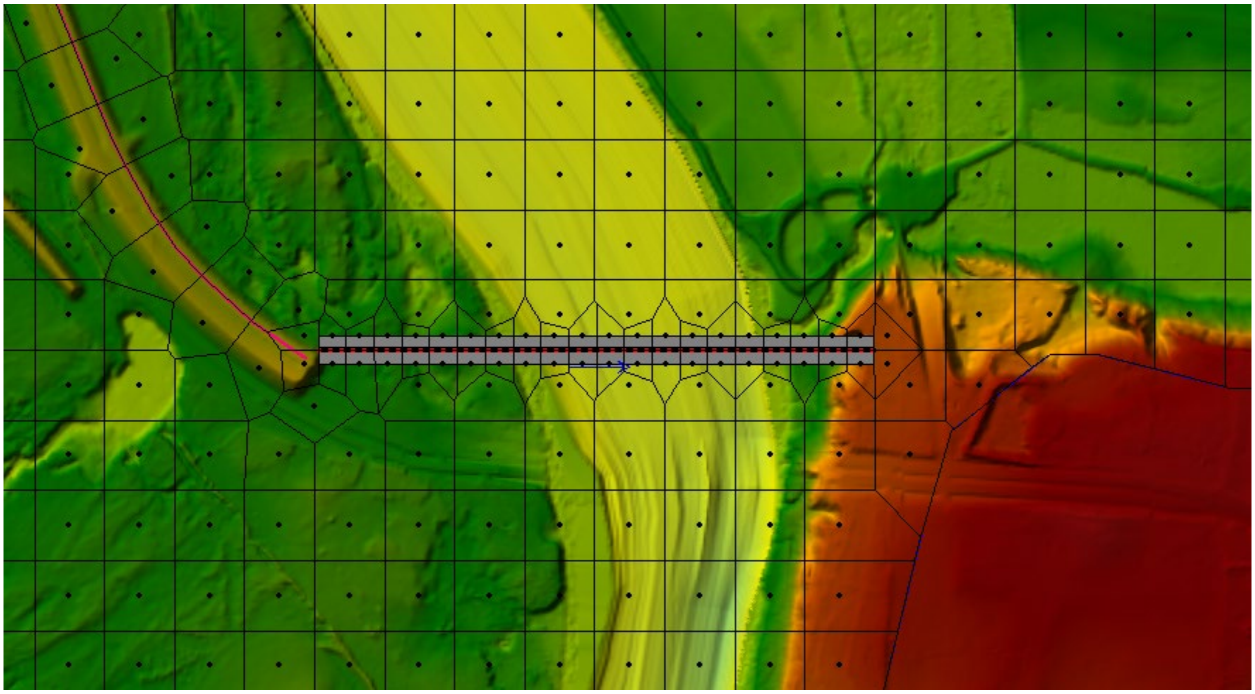


Figure 8 - 2012 SHEP GRR Terrain Schematic

2.3.3. Upstream Location

This alternative evaluated the placement of the fish passage structure at locations upstream of the existing lock and dam structure. Several locations were considered and the width of the weir varied by location to span the width of the river. The height of the weir varies with placement, though generally it followed the trend of the weir crest elevation increasing the further upstream to maintain a pool elevation comparable to existing conditions.

This alternative includes the removal of both lock walls, removal of dam gates and piers, and partial demolition of the dam foundation to elevation 91.2 NAVD 88. A rock ramp the width of the channel will then be placed in the channel at a location upstream of the existing dam (several locations were evaluated). The rock ramp would have an overall slope of 2% from the downstream toe to the ultimate weir crest elevation, and 10% sloping down from the crest to the upstream toe. The weir crest for this alternatives is in a terraced configuration. The terrain model and 2D mesh were modified to incorporate the rock ramp structure into the geometry. A manning's n region was created for the area of the rock ramp and an n value of 0.08 specified, in accordance with the procedure used in the 1D model



Several locations upstream of the current lock and dam location were evaluated, but no suitable locations for placement of a rock weir were found upstream of the dam. The placement of any large structure within the channel reduces the available conveyance to pass high flows (e.g. flood waters). The further upstream a structure is placed, the more dramatic the impacts of reduced conveyance are on water surface elevation, resulting in additional flooding compared to existing conditions. The current location of NSBL&D at the fall line provides the best location where a structure can be placed without adversely impacting water surface elevations during floods. The construction of a fish passage structure upstream of the current location that would result in significant flooding in the overbank areas was deemed infeasible and not considered for further evaluation.

2.3.4. Downstream Location

Several locations downstream of the current lock and dam location were evaluated, but no suitable locations for placement of a rock weir were found downstream of the dam. A larger and taller structure would need to be placed in the river channel downstream of the existing location in order to provide normal pool elevations comparable to existing conditions. A potential location a mile downstream of the existing dam location would need to span the width of the floodplain, resulting in a structure over a mile long. This is because the Fall Line where the structure is currently located provides a breakpoint in elevation where the slope of the river starts to increase noticeably as one moves downstream. The construction of a fish passage structure that would span the full width of the floodplain was deemed infeasible and not considered for further evaluation.

The placement of any large structure within the channel reduces the available conveyance to pass high flows (e.g. flood waters). The further upstream a structure is placed, the more dramatic the impacts of reduced conveyance are on water surface elevation, resulting in additional flooding compared to existing conditions. The current location of NSBL&D at the fall line provides the best location where a structure can be placed without adversely impacting water surface elevations during floods.

2.3.5. Repair Lock Wall, Georgia Side Fish Passage (Alternative 1-1)

This alternative consists of repairing the NSBLD gates and piers and the riverside lock wall. Two of the dam's operational gates would be modified to direct additional flow through the fish passage structure, similar to the 2012 SHEP GRR Plan. The lock chamber and portions of the esplanade will be demolished and a 200' wide fish ramp structure would be constructed on the north side of the remaining lock wall. The fish passage structure would be constructed with boulders and stone sized according to the same specifications that were previously-approved for the bypass in the 2012 GRR. The structure would have a 2% slope upstream to the weir crest, and a 10% slope upstream from the crest to the river bed to the ultimate weir crest elevation of 109.2 NAVD 88 (110 feet NGVD 29). The weir crest is in a terraced configuration with the thalweg located on the north side of the weir. This alternative was not part of the initial screening using the HEC-RAS 1D model, but came up numerous times in discussions amongst the PDT and with others familiar with the project.

The terrain model and 2D mesh were modified to incorporate the rock ramp structure into the geometry. A manning's n region was created for the area of the rock ramp and an n value of 0.08 specified, in accordance with the procedure used in the 1D model. A schematic of the general site configuration and terrain model used for this alternative can be found in Figure 9 below.

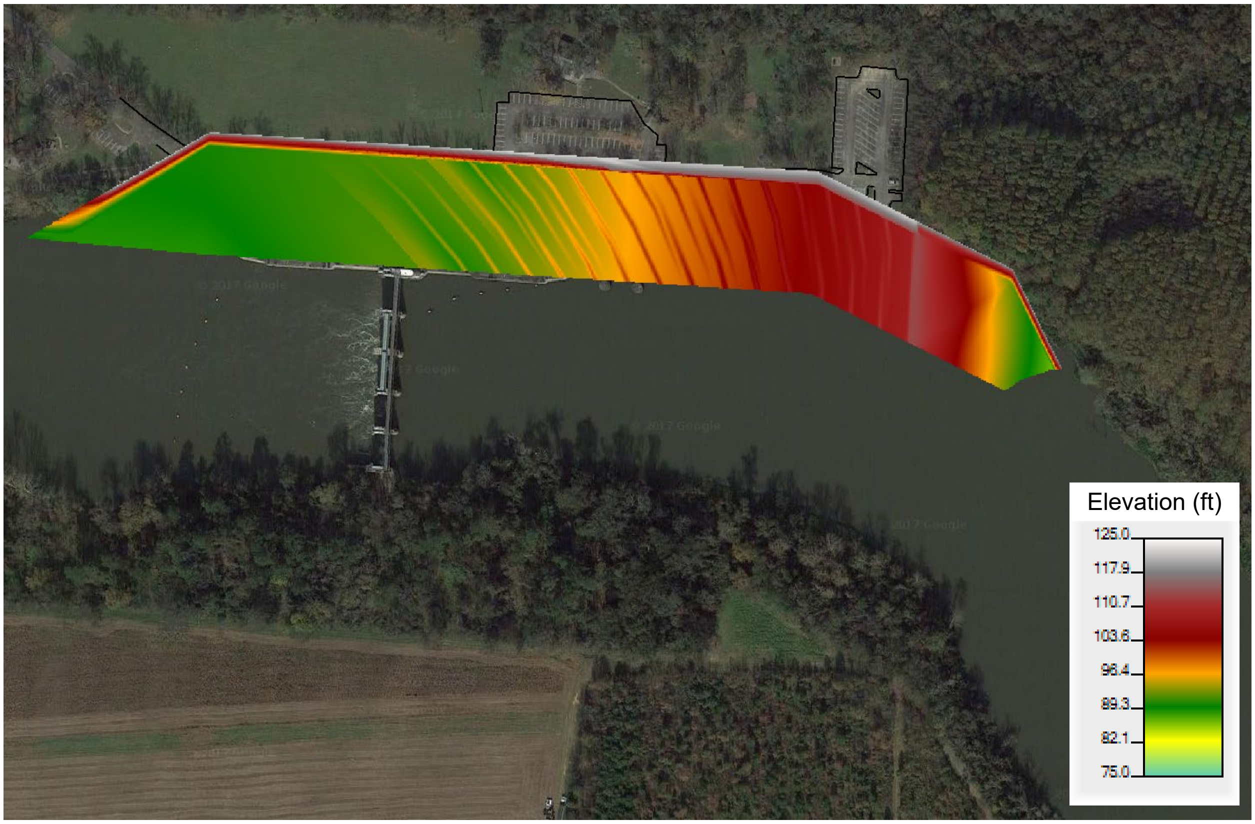


Figure 9 - Alternative 1-1 Layout

This alternative would lower the normal pool elevation near the lock and dam by 1.2 feet, with the impacts attenuating as you move upstream. The pool at 5th St. Bridge would be around elevation 113.5 NAVD 88 (0.8 feet lower than existing) during normal flow conditions. Normal flow conditions here corresponds to 5,000cfs mean daily flow

This alternative would not cause any additional flooding for the 50% through 1% annual percent exceedance (2-year through 100-year) flood events as the dam gates would remain in place and operational during high flows. This alternative was evaluated as part of the “final array” and a cost estimate for construction developed. See additional discussion on plan formulation in the main report and the Cost Engineering Appendix for additional discussion of construction costs.

2.3.6. Repair Lock, 380ft wide Fish Passage (Alternative 1-2)

This alternative includes the repair of the river-side lock wall, removal of dam gates and piers, and partial demolition of the dam foundation to elevation 91.2 NAVD 88. A rock ramp with a crest 380’ in length would be placed on the upstream side of the existing dam location, sloping 2% upstream to the ultimate weir crest elevation of 109.2 NAVD88 (110 feet NGVD29). The weir crest is in a terraced configuration with the thalweg located on the north side of the weir. This alternatives was part of the initial screening using the HEC-RAS 1D model.

The terrain model and 2D mesh were modified to incorporate the rock ramp structure into the geometry. A manning's n region was created for the area of the rock ramp and an n value of 0.08 specified, in accordance with the procedure used in the 1D model.

A schematic of the general site configuration and terrain model used for this alternative can be found in Figure 10 below. This alternative would lower the normal pool elevation near the lock and dam by 1.2 feet, with the impacts attenuating as you move upstream. The pool at 5th St. Bridge would be around elevation 113.5 NAVD 88 (0.8 feet lower than existing) during normal flow conditions (5,000 cfs).

This alternative would likely increase inundation depths in the overbank areas for the 50% and 20% AEP (2-year and 5-year return interval) events for hundreds of parcels in the study area. The 10% AEP through 1% AEP (10-year through 100-year) flow events would not experience any appreciable increase in inundation depths. This alternative was not evaluated as part of the final array due primarily to concerns over increased inundation depths for the 50% and 20% AEP events; a cost estimate for construction was not developed. See additional discussion on plan formulation in the main report.

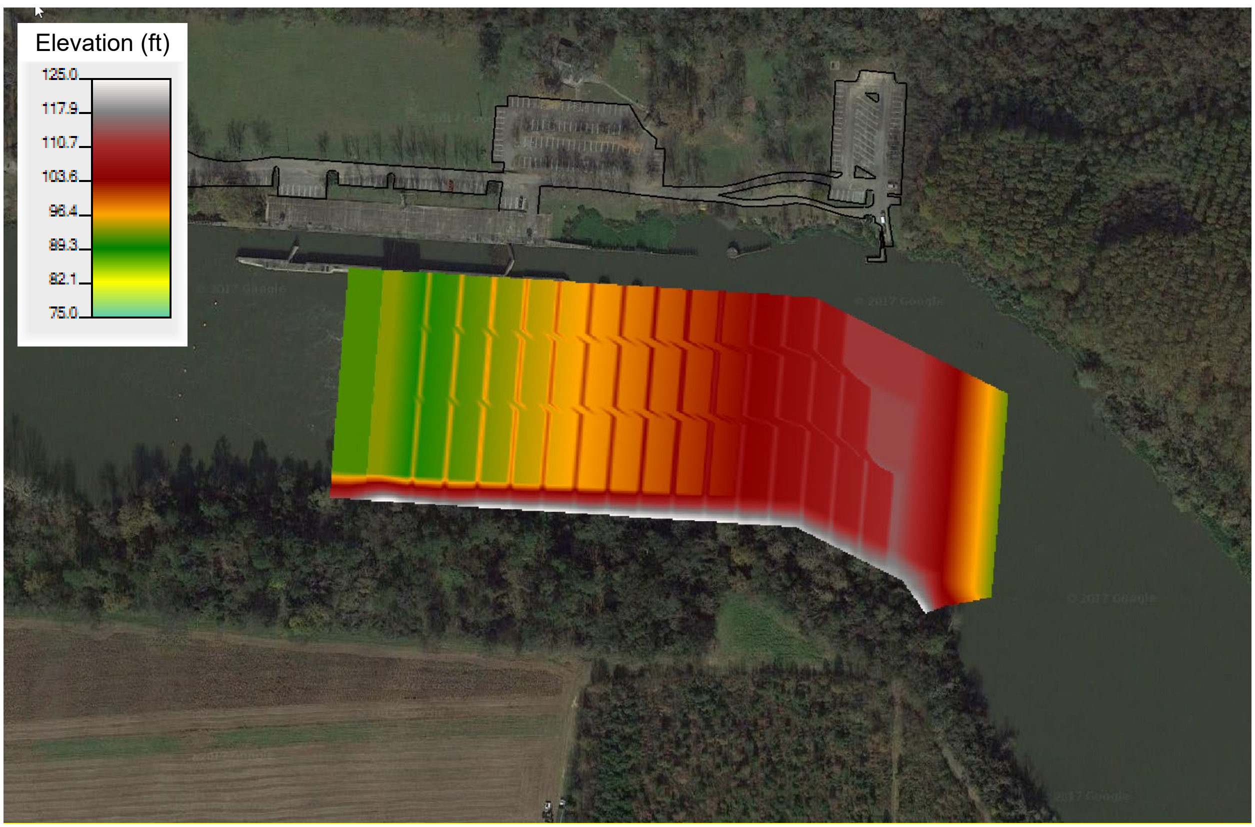


Figure 10 - Alternative 1-2 Layout

2.3.7. Fixed Crest Weir – 500ft wide at elevation 105 (Alternative 2-1)

This alternative includes the removal of both lock walls, removal of dam gates and piers, and partial demolition of the dam foundation to elevation 91.2 NAVD 88. A rock ramp with a crest 500’ in length will then be placed upstream of the existing dam location, sloping 2% upstream to the ultimate weir crest elevation of 104.2 NAVD 88 (105 feet NGVD 29). Several alternatives follow this configuration and vary only in the weir crest elevation. The weir crest is in a terraced configuration with the thalweg located on the north side of the weir. The terrain model and 2D mesh were modified to incorporate the rock ramp structure into the HEC-RAS model geometry. A manning’s n region was created for the area of the rock ramp and an n value of 0.08 specified, in accordance with the procedure used in the 1D model.

A schematic of the general site configuration and terrain model used for this alternative can be found in Figure 11 below.

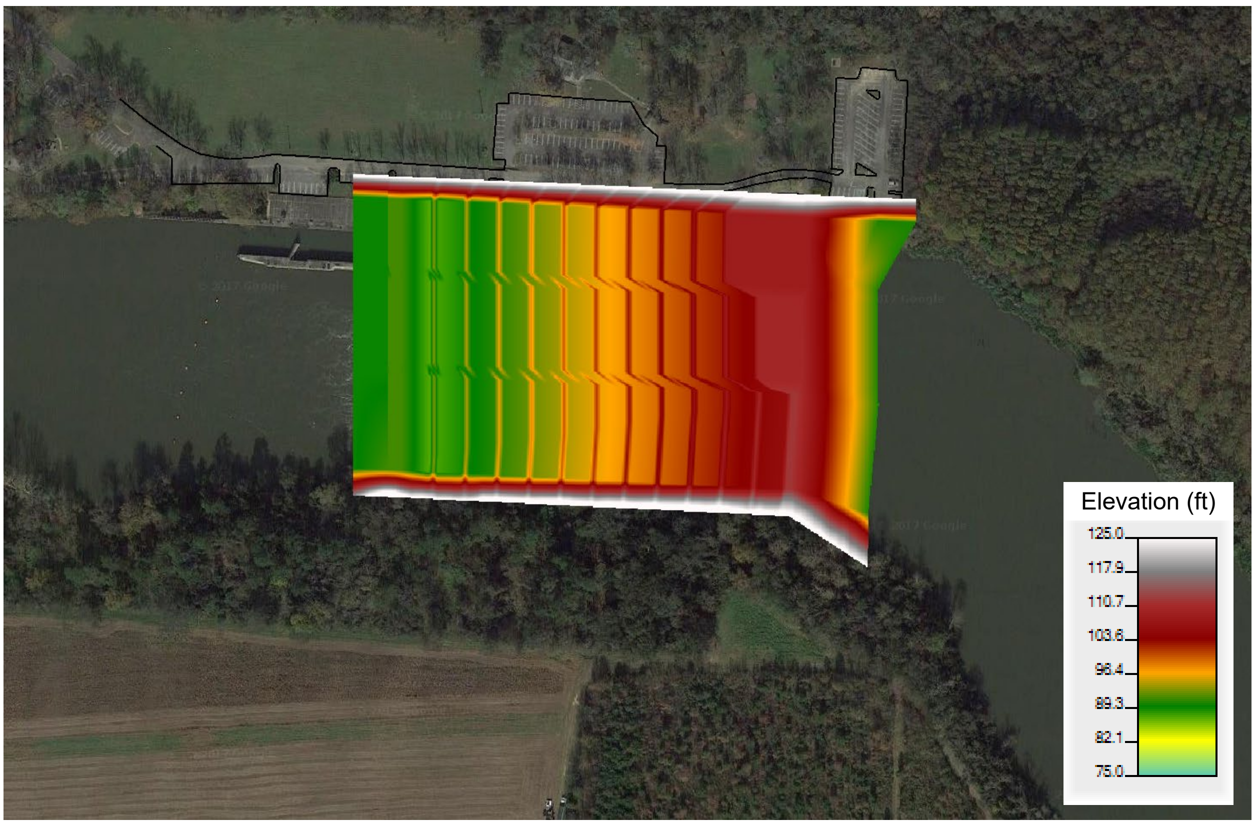


Figure 11 - Alternative 2-1 Layout

This alternative would lower the normal pool elevation near the lock and dam by 6 feet, with the impacts attenuating as you move upstream. The pool at 5th St. Bridge would be around elevation 111.3 NAVD 88 (3.0 feet lower than existing) during normal flow conditions (5,000 cfs).

This alternative would not cause any additional flooding for the 50% through 1% AEP (2-year through 100-year) flood events as the weir crest is low enough to pass high-flows without inducing additional flood damages in the overbank areas. This alternative was not evaluated as part of the final array due primarily to concerns over impacts to water supply and recreational users in the pool; a cost estimate for construction was not developed. See additional discussion on plan formulation in the main report.

2.3.8. Fixed Crest Weir – 500ft wide at elevation 106 (Alternative 2-2)

This alternative includes the removal of both lock walls, removal of dam gates and piers, and partial demolition of the dam foundation to elevation 91.2 NAVD 88. A rock ramp with a crest 500' in length will then be placed upstream of the existing dam location, sloping 2% upstream to the ultimate weir crest elevation of 105.2 NAVD 88 (106 feet NGVD 29). The weir crest is in a terraced configuration with the thalweg located on the north side of the weir. The terrain model and 2D mesh were modified to incorporate the rock ramp structure into the HEC-RAS model geometry. A manning's n region was created for the area of the rock ramp and an n value of 0.08 specified, in accordance with the procedure used in the 1D model.

A schematic of the general site configuration and terrain model used for this alternative can be found in Figure 12 below.

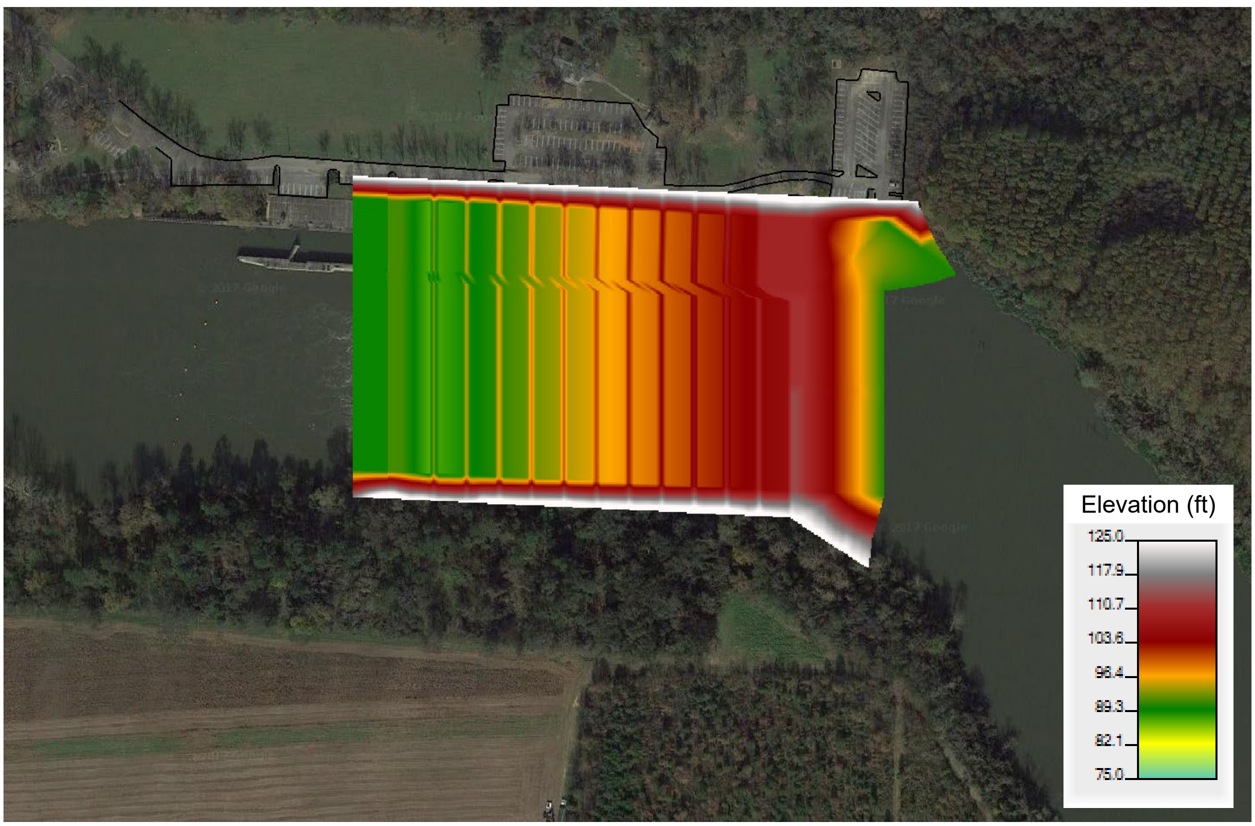


Figure 12 - Alternative 2-2 Layout

This alternative would lower the normal pool elevation near the lock and dam by almost 6 feet, with the impacts attenuating as you move upstream. The pool at 5th St. Bridge would be around elevation 111.3 (NAVD 88) (3.0 feet lower than existing) during normal flow conditions (5,000cfs).

This alternative would not cause any additional flooding for the 50% through 1% AEP

(2-year through 100-year) flood events as the weir crest is low enough to pass high-flows without inducing additional flood damages in the overbank areas. This alternative was not evaluated as part of the final array due primarily to concerns over impacts to water supply and recreational users in the pool; a cost estimate for construction was not developed. See additional discussion on plan formulation in the main report.

2.3.9. Fixed Crest Weir – 500ft wide at elevation 107 (Alternative 2-3)

This alternative includes the removal of both lock walls, removal of dam gates and piers, and partial demolition of the dam foundation to elevation 91.2 NAVD 88. A rock ramp 500' in width will then be placed on the upstream side of the dam, sloping 2% upstream to the ultimate weir crest elevation of 106.2 NAVD 88 (107 NGVD 29). The weir crest for this alternative is in a terraced configuration, with the thalweg located on the north side of the weir. The terrain model and 2D mesh were modified to incorporate the rock ramp structure into the geometry. A manning's n region was created for the area of the rock ramp and an n value of 0.08 specified, in accordance with the procedure used in the 1D model. A schematic of the general site configuration and terrain model used for this alternative can be found in Figure 13 below.

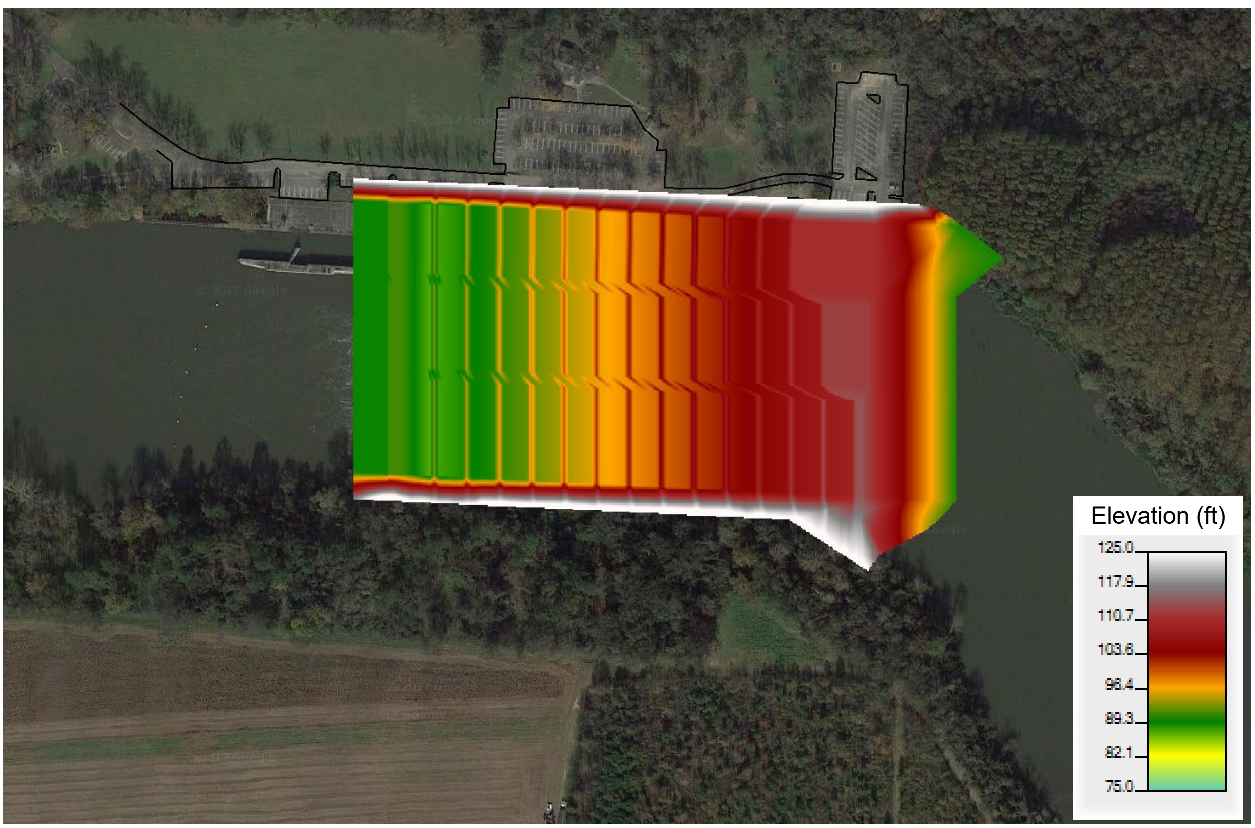


Figure 13 - Alternative 2-3 Layout

This alternative would lower the normal pool elevation near the lock and dam by 4.8 feet, with the impacts attenuating as you move upstream. The pool at 5th St. Bridge would be around elevation 111.6 (NAVD 88) (2.7 feet lower than existing) during normal flow conditions (5,000cfs).

This alternative would not increase flooding depth as compared to existing conditions. This alternative was evaluated as part of the final array and a cost estimate for construction developed because it strikes a balance between minimizing impacts to water supply users and limiting flooding impacts. See additional discussion on plan formulation in the main report and the Cost Engineering Appendix for additional discussion of construction costs.

2.3.10. Fixed Crest Weir – 500ft wide at elevation 107.6 (Alternative 2-4)

This alternative includes the removal of both lock walls, removal of dam gates and piers, and partial demolition of the dam foundation to elevation 91.2 NAVD 88. A rock ramp 500' in width will then be placed on the upstream side of the dam, sloping 2% upstream to the ultimate weir crest elevation of 106.8 NAVD 88 (107.6 NGVD 29). The weir crest for this alternative is in a terraced configuration, with the thalweg located on the north side of the weir. This alternative was not part of the initial screening using the HEC-RAS 1D model, but was developed in an attempt to further refine the weir crest elevation of alternative 2-3 in order to minimize impacts to water-supply and inundated areas.

The terrain model and 2D mesh were modified to incorporate the rock ramp structure into the geometry. A manning's n region was created for the area of the rock ramp and an n value of 0.08 specified, in accordance with the procedure used in the 1D model. A schematic of the general site configuration and terrain model used for this alternative can be found in Figure 14 below.

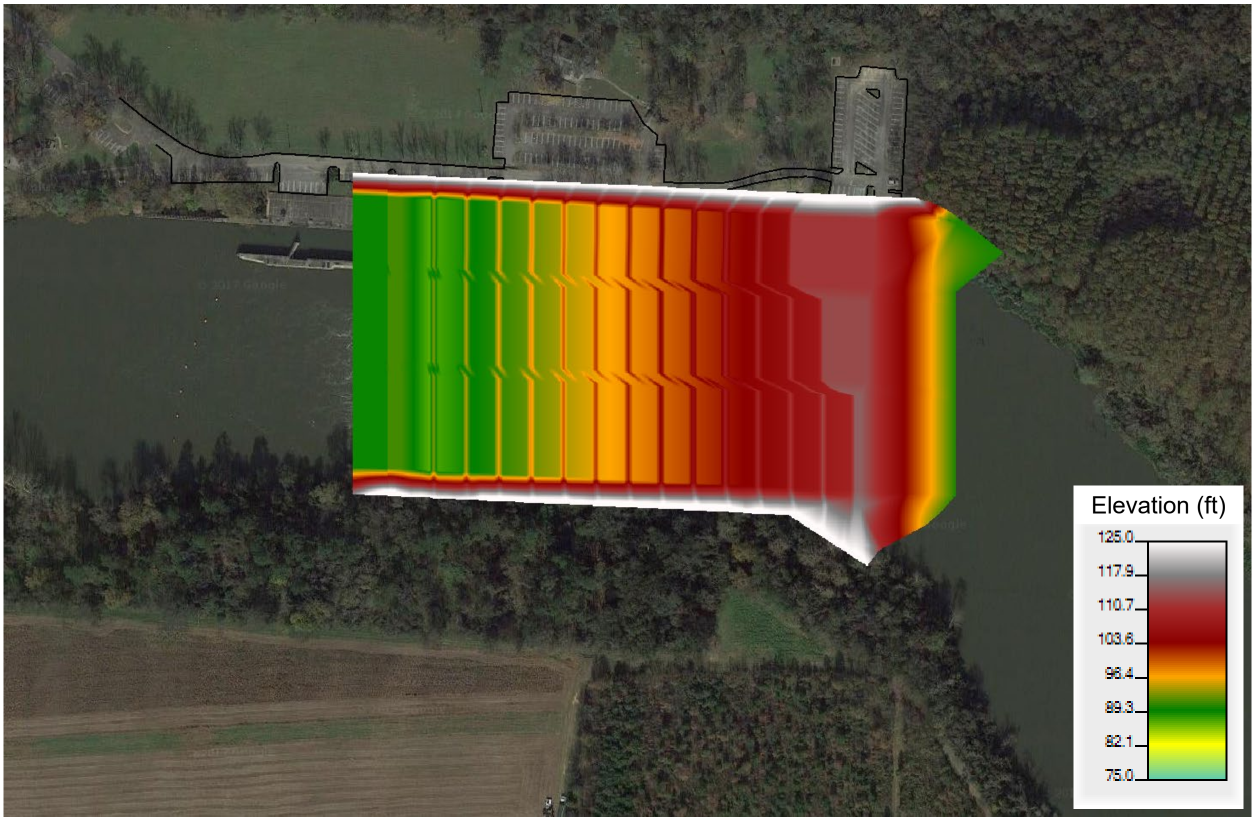


Figure 14 - Alternative 2-4 Layout

This alternative would lower the normal pool elevation near the lock and dam by 3.6 feet, with the impacts attenuating as you move upstream. The pool at 5th St. Bridge would be around elevation 112.1 (NAVD 88) (2.2 feet lower than existing) during normal flow conditions (5,000cfs).

This alternative would likely cause a minor increase flooding depth at dozens of parcels for the 50% and 20% AEP flood events, though larger flows would have the same inundation footprint and depth as under existing conditions. This alternative was not evaluated as part of the final array due primarily to concerns over additional flooding in the overbank areas for the 250% and 20% AEP flows; a cost estimate for construction was not developed. The slight increase in the crest elevation compared to alternative 2-3 result in numerous additional parcels being inundated for the 50% and 20% AEP events with little to no additional benefit to pool elevations during low-flow conditions. See additional discussion on plan formulation in the main report.

2.3.11. Fixed Crest Weir – 500ft wide at elevation 110 (Alternative 2-5)

This alternative includes the removal of both lock walls, removal of dam gates and piers, and partial demolition of the dam foundation to elevation 91.2 NAVD 88. A rock ramp 500' in width will then be placed on the upstream side of the dam, sloping 2% upstream to the ultimate weir crest elevation of 109.2 NAVD 88 (110 NGVD 29). The weir crest for this alternative is in a terraced configuration, with the thalweg located on the north side of the weir. The terrain model and 2D mesh were modified to incorporate the rock ramp structure into the geometry. A manning's n region was created for the area of the rock ramp and an n value of 0.08 specified, in accordance with the procedure used in the 1D model. A schematic of the general site configuration and terrain model used for this alternative can be found in Figure 15 below.

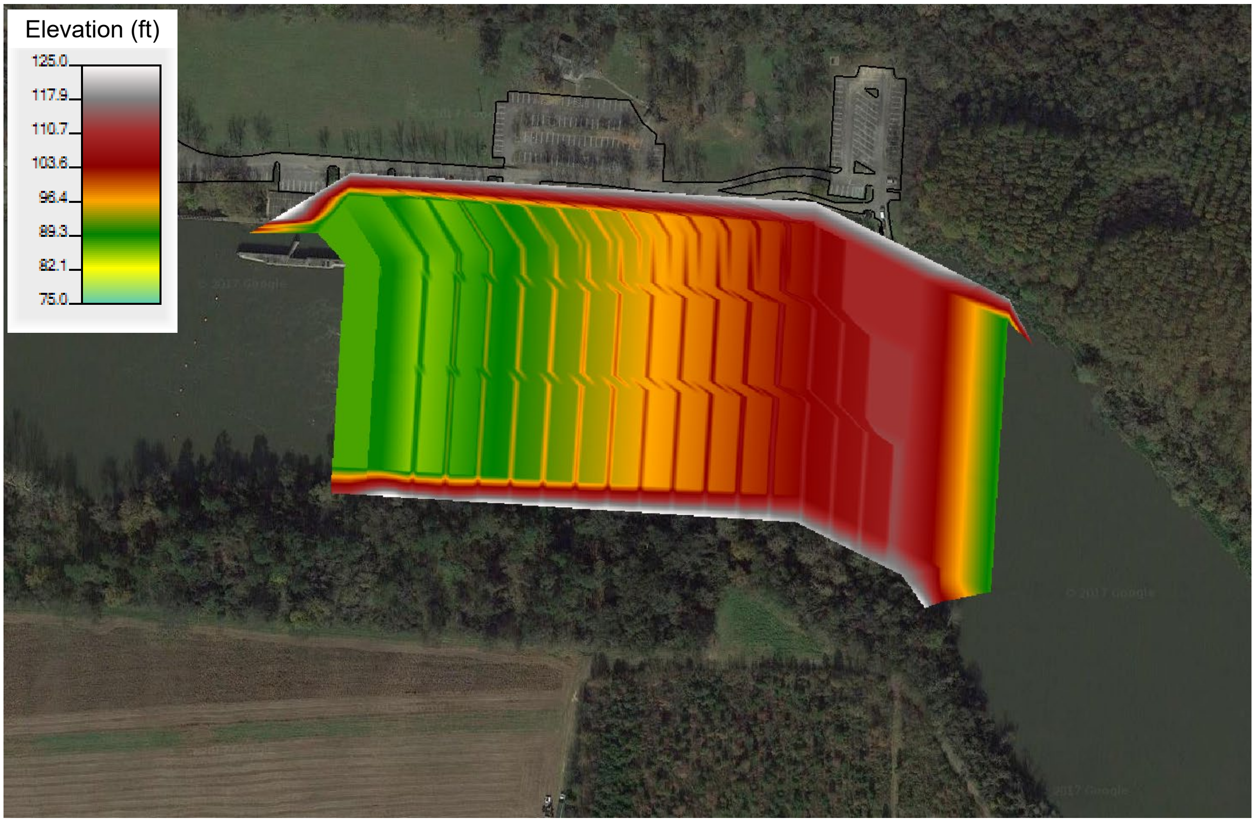


Figure 15 - Alternative 2-5 Layout

This alternative would lower the normal pool elevation near the lock and dam by 1.2 feet, with the impacts attenuating as you move upstream. The pool at 5th St. Bridge would be around elevation 113.4 NAVD 88 (0.8 feet lower than existing) during normal flow conditions (5,000cfs).

This alternative would likely cause an increase flooding depth at hundreds of parcels for the 50% and 20% AEP flood events, though larger flows would have a similar inundation footprint and depth as under existing conditions. This alternative was not evaluated as part of the final array due primarily to concerns over additional flooding in the overbank areas for the 50% and 20% AEP flows; a cost estimate for construction was not developed. The increase in the crest elevation compared to alternative 2-3 resulted in numerous additional parcels being inundated for the 50% and 20% AEP flow events. See additional discussion on plan formulation in the main report.

2.3.12. Fixed Crest Weir at elevation 110 with Floodplain Bench (Alternative 2-6A)

Alternative 2-6A consists of a fixed crest weir with a rock ramp sloping upstream from the existing dam location and a low-lying floodplain bench in the right overbank to provide additional flow conveyance. The lock and dam would be removed, including the foundation down to elevation 91.2 NAVD 88. The weir would be 500 feet in width with an average crest elevation of 109.2 feet NAVD 88 (110.0 NGVD 29). A floodplain bench approximately 275 feet in width would be excavated down to elevation 110 NAVD 88 (approximately 1.8ft higher than the adjacent rock-ramp terrace) on the Georgia side of the existing dam location. The bench would ease the passage of flood waters past that point in the river. The bench was modeled as grass-lined in the HEC-RAS model to provide a hydraulically efficient flow area; though paving may be required to prevent erosion. A schematic of the general site configuration and terrain model used for this alternative can be found in Figure 16 below.

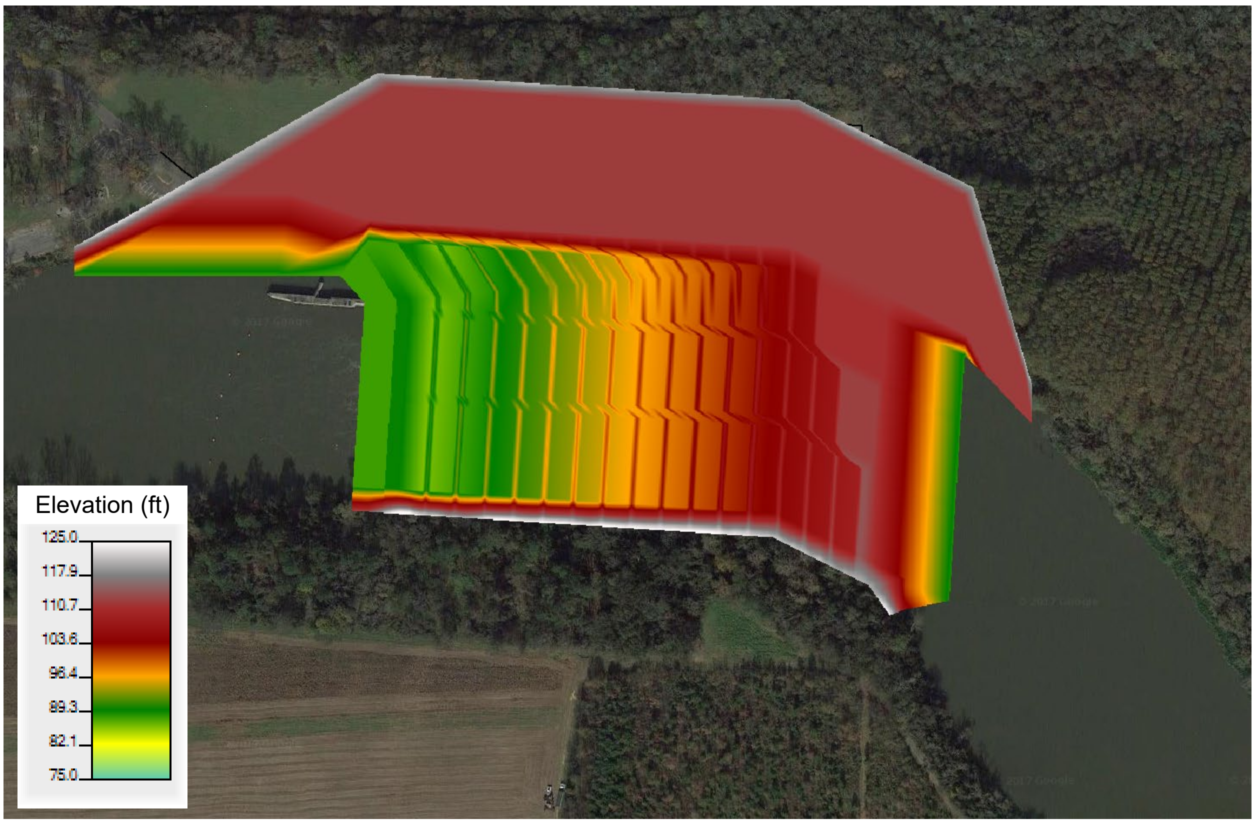


Figure 16 - Alternative 2-6A Layout

A cross section view showing the rock weir terrace configuration as well as the floodplain bench can be seen in Figure 17 below. This configuration is the same for all alternatives with a floodplain bench, with only the crest elevation and normal pool elevation of the terraced rock weir varying by alternative.

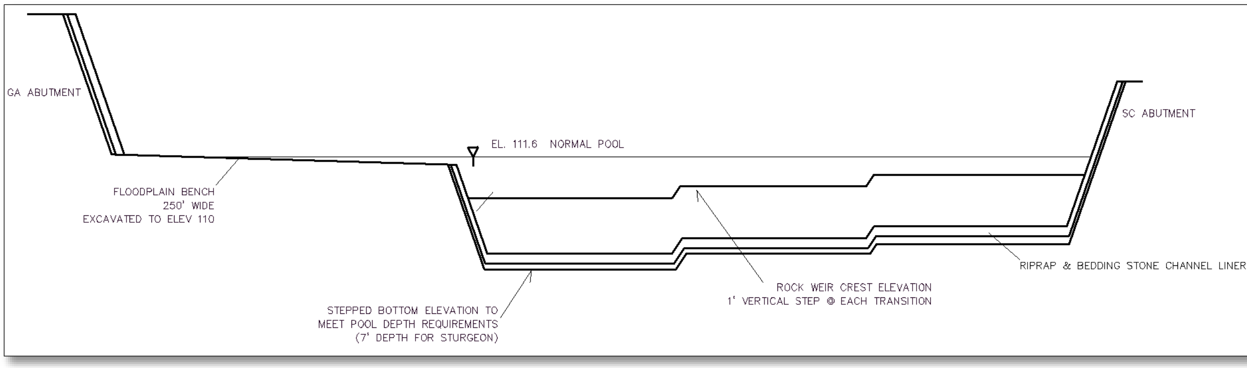


Figure 17 - Floodplain Bench Cross Section

This alternative would lower the normal pool elevation near the lock and dam by 1.6 feet, with impacts attenuating as you move upstream. The pool at 5th St. Bridge would be around elevation 113.2 NAVD 88 (1.1 feet lower than existing) during normal flow conditions (5,000cfs).

This alternative may cause a minor increase in flooding depth at dozens of parcels for the 50% AEP flood event, though larger flows would have the same inundation footprint and depth as under existing conditions. This alternative was evaluated as part of the final array and a cost estimate for construction developed because it strikes a balance between minimizing impacts to water supply users and limiting flooding impacts. See additional discussion on plan formulation in the main report and the Cost Engineering Appendix.

2.3.13. Fixed Crest Weir at elevation 107 with Floodplain Bench (Alternative 2-6B)

Alternative 2-6B is nearly identical to 2-6A, but with a weir crest elevation at 106.2 NAVD 88. The fixed crest weir with a rock ramp would slope upstream from the existing dam location and a low-lying floodplain bench in the right overbank to provide additional flow conveyance. The lock and dam would be removed, including the foundation down to elevation 91.2 NAVD 88. The weir would be 500 feet in width with an average crest elevation of 106.2 feet NAVD 88 (107.0 NGVD 29). A floodplain bench approximately 275 feet in width would be excavated down to elevation 110 NAVD 88 (approximately 4.8ft higher than the adjacent rock-ramp terrace) on the Georgia side of the existing dam location. The bench would ease the passage of flood waters past that point in the river. The bench was modeled as grass-lined in the HEC-RAS model to provide a hydraulically efficient flow area; though paving may be required to prevent erosion. A schematic of the general site configuration and terrain model used for this alternative can be found in Figure 18 below. The terraced rock weir and floodplain bench follows the same configuration seen in Figure 17, with a differing normal pool elevation.

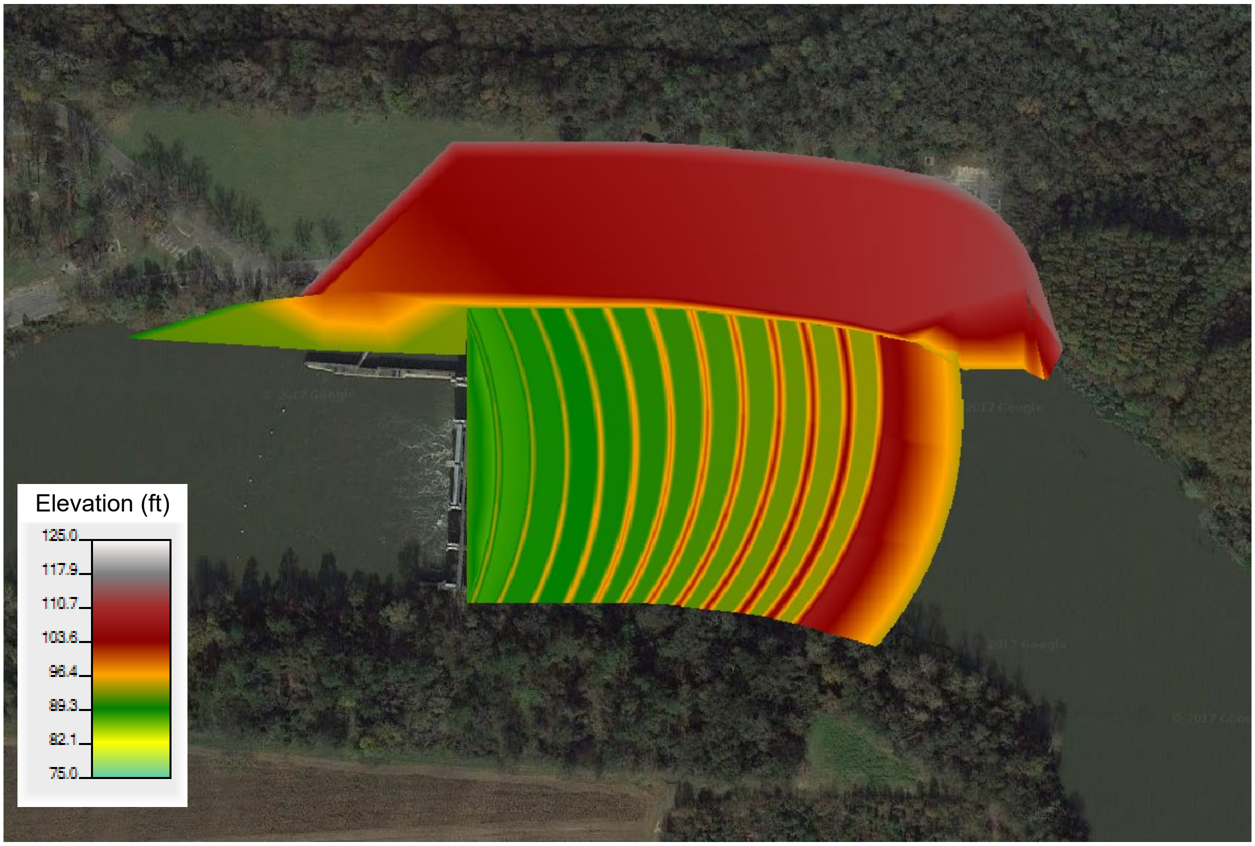


Figure 18 - Alternative 2-6B Layout

This alternative would lower the normal pool elevation near the lock and dam by 4.9 feet, with impacts attenuating as you move upstream. The pool at 5th St. Bridge would be around elevation 111.6 NAVD 88 (2.7 feet lower than existing) during normal flow conditions (5,000cfs).

This alternative would not cause any increase flooding depth or inundation footprint as compared to existing conditions. This alternative was evaluated as part of the final array and a cost estimate for construction developed because it strikes a balance between minimizing impacts to water supply users and limiting flooding impacts. See additional discussion on plan formulation in the main report and the Cost Engineering Appendix.

2.3.14. Fixed Crest Weir at elevation 108 with Floodplain Bench (Alternative 2-6C)

Alternative 2-6C is nearly identical to 2-6A, but with a weir crest elevation at 107.2 NAVD 88. The fixed crest weir with a rock ramp would slope upstream from the existing dam location and a low-lying floodplain bench in the right overbank to provide additional flow conveyance. The lock and dam would be removed, including the foundation down to elevation 91.2 NAVD 88. The weir would be 500 feet in width with an average crest elevation of 107.2 feet NAVD 88 (108.0 NGVD 29). A floodplain bench approximately 275 feet in width would be excavated down to elevation 110 NAVD 88 (approximately 3.8ft higher than the adjacent rock-ramp terrace) on the Georgia side of the existing dam location. The bench would ease the passage of flood waters past that point in the river. The bench was modeled as grass-lined in the HEC-RAS model to provide a hydraulically efficient flow area; though paving may be required to prevent erosion. A schematic of the general site configuration and terrain model used for this alternative can be found in Figure 19 below. The terraced rock weir and floodplain bench follows the same configuration seen in Figure 17, with a differing normal pool elevation.

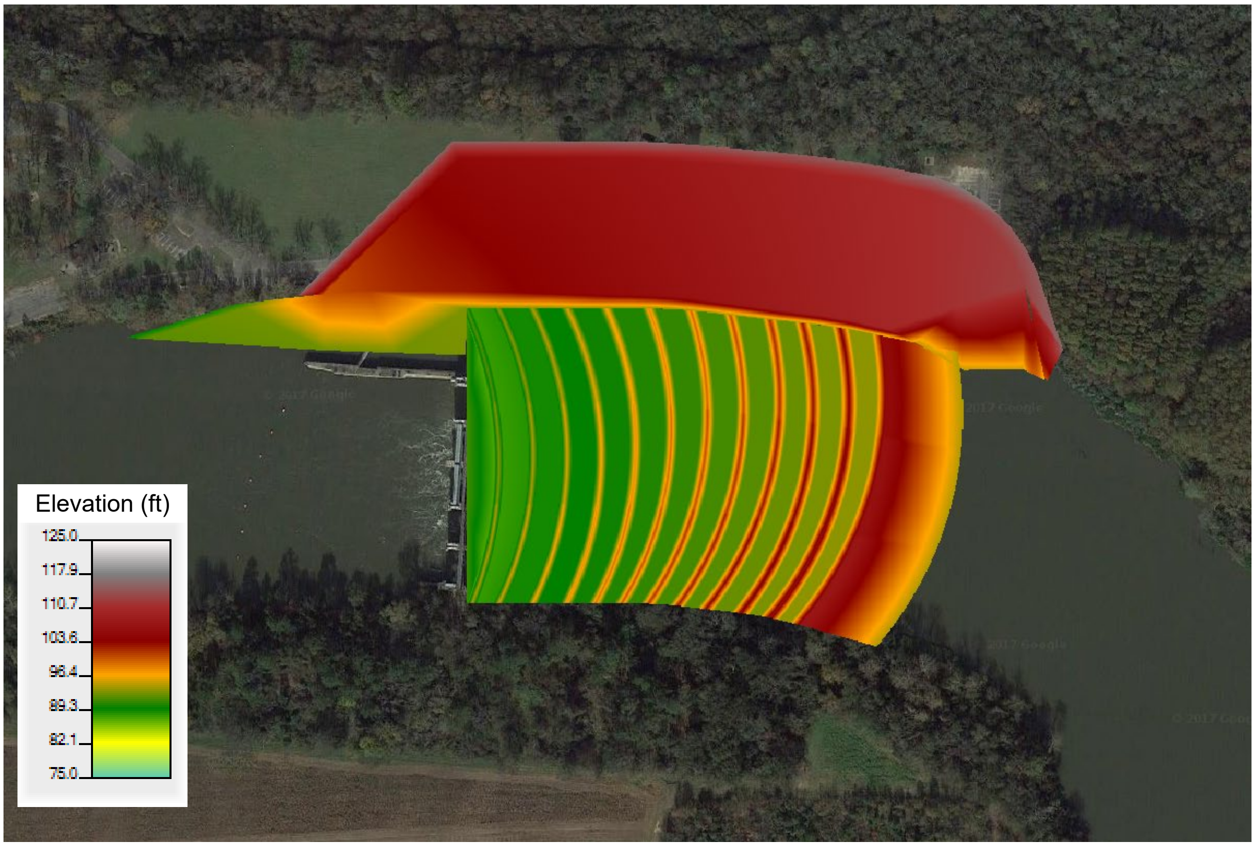


Figure 19 - Alternative 2-6C Layout

This alternative would lower the normal pool elevation near the lock and dam by 3.9 feet, with impacts attenuating as you move upstream. The pool at 5th St. Bridge would be around elevation 112.0 NAVD 88 (2.3 feet lower than existing) during normal flow conditions (5,000cfs).

This alternative would not cause any increase flooding depth or inundation footprint as compared to existing conditions. This alternative was evaluated as part of the final array and a cost estimate for construction developed because it strikes a balance between minimizing impacts to water supply users and limiting flooding impacts. See additional discussion on plan formulation in the main report and the Cost Engineering Appendix.

2.3.15. Fixed Crest Weir at elevation 109 with Floodplain Bench (Alternative 2-6D)

Alternative 2-6D is nearly identical to 2-6A, but with a weir crest elevation at 108.2 NAVD 88. The fixed crest weir with a rock ramp would slope upstream from the existing dam location and a low-lying floodplain bench in the right overbank to provide additional flow conveyance. The lock and dam would be removed, including the foundation down to elevation 91.2 NAVD 88. The weir would be 500 feet in width with an average crest elevation of 108.2 feet NAVD 88 (109.0 NGVD 29). A floodplain bench approximately 275 feet in width would be excavated down to elevation 110 NAVD 88 (approximately 2.8ft higher than the adjacent rock-ramp terrace) on the Georgia side of the existing dam location. The bench would ease the passage of flood waters past that point in the river. The bench was modeled as grass-lined in the HEC-RAS model to provide a hydraulically efficient flow area; though paving may be required to prevent erosion. A schematic of the general site configuration and terrain model used for this alternative can be found in Figure 20 below. The terraced rock weir and floodplain bench follows the same configuration seen in Figure 17, with a differing normal pool elevation.

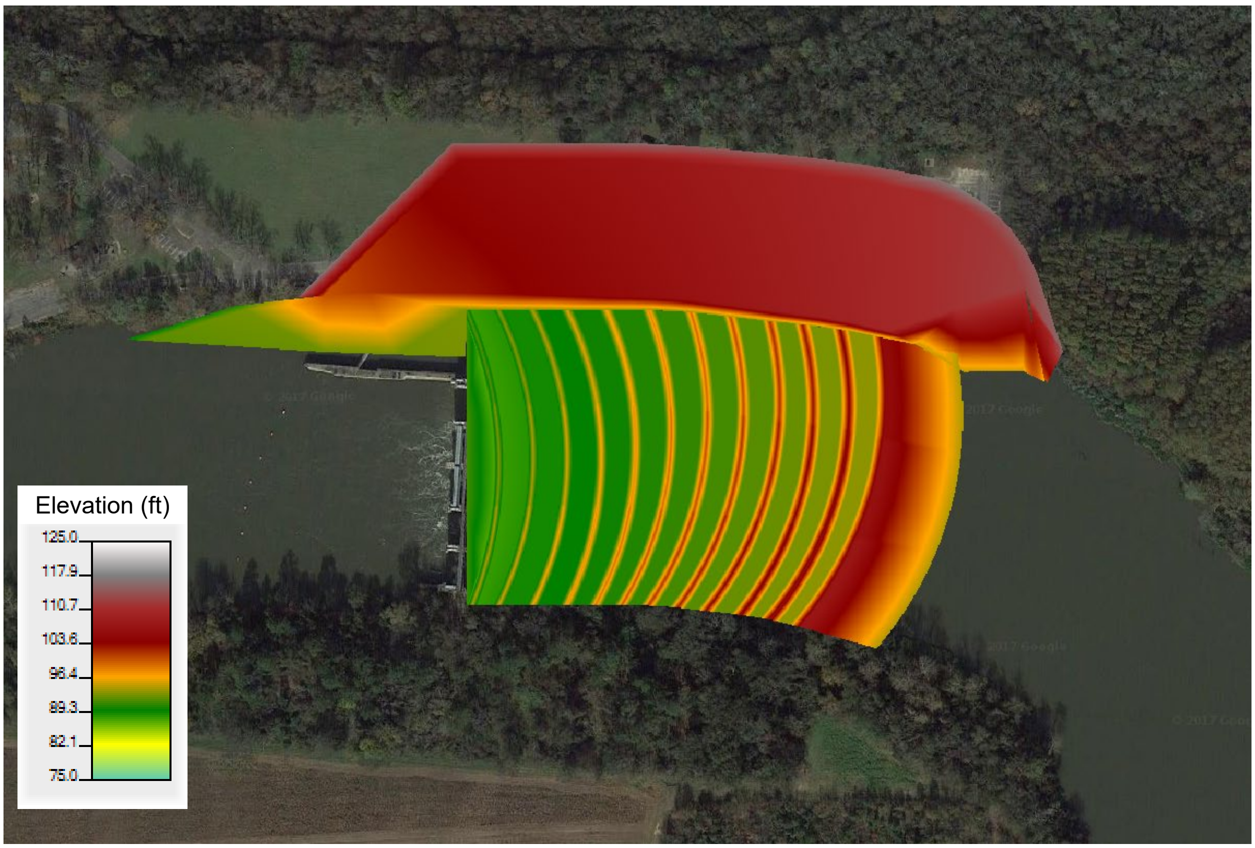


Figure 20 - Alternative 2-6D Layout

This alternative would lower the normal pool elevation near the lock and dam by 2.3 feet, with impacts attenuating as you move upstream. The pool at 5th St. Bridge would be around elevation 112.4 NAVD 88 (1.9 feet lower than existing) during normal flow conditions (5,000cfs).

This alternative would not cause any increase flooding depth or inundation footprint as compared to existing conditions. This alternative was evaluated as part of the final array and a cost estimate for construction developed because it strikes a balance between minimizing impacts to water supply users and limiting flooding impacts. See additional discussion on plan formulation in the main report and the Cost Engineering Appendix.

2.3.16. Fixed Crest Weir with Gated Bypass Channel – 1 Gate (Alternative 2-7)

Alternative 2-7 consists of a fixed weir with a rock ramp at the existing dam site (identical to Alternative 2-5) with an active flood passage structure in an excavated bypass channel through the park on the Georgia side of the river. The bypass channel would be approximately 100 feet wide and excavated to elevation 91.2 NAVD 88. The rock weir would be 500 feet in width with an average crest elevation of 109.2 feet NAVD 88, (110.0 NGVD 29). The lock and dam would be removed, including the foundation down to 91.2 NAVD 88.

The structure in the bypass channel would consist of **one** 50' wide gate, 40' high used to pass high flows. The gated structure would be operated to pass high flows by fully lifting the gate out of the water during high flows and otherwise remain closed to maintain the pool elevation during low and normal flow conditions. A schematic of the general site configuration and terrain model used for this alternative can be found in Figure 21 below.

At the weir, the pool is expected to be 1.3 feet lower than the existing pool elevation for normal flow conditions (5,000cfs). The pool at 5th St. Bridge would be around elevation 113.4 NAVD 88, 0.9 feet lower than existing during normal flow conditions.

This alternative would likely cause an increase flooding depth at hundreds of parcels for the 50% and 20% AEP flood events, though larger flows would have a similar inundation footprint and depth as under existing conditions. This alternative was not evaluated as part of the final array due primarily to concerns over additional flooding in the overbank areas for the 50% and 20% AEP flows; a cost estimate for construction was not developed. See additional discussion on plan formulation in the main report.

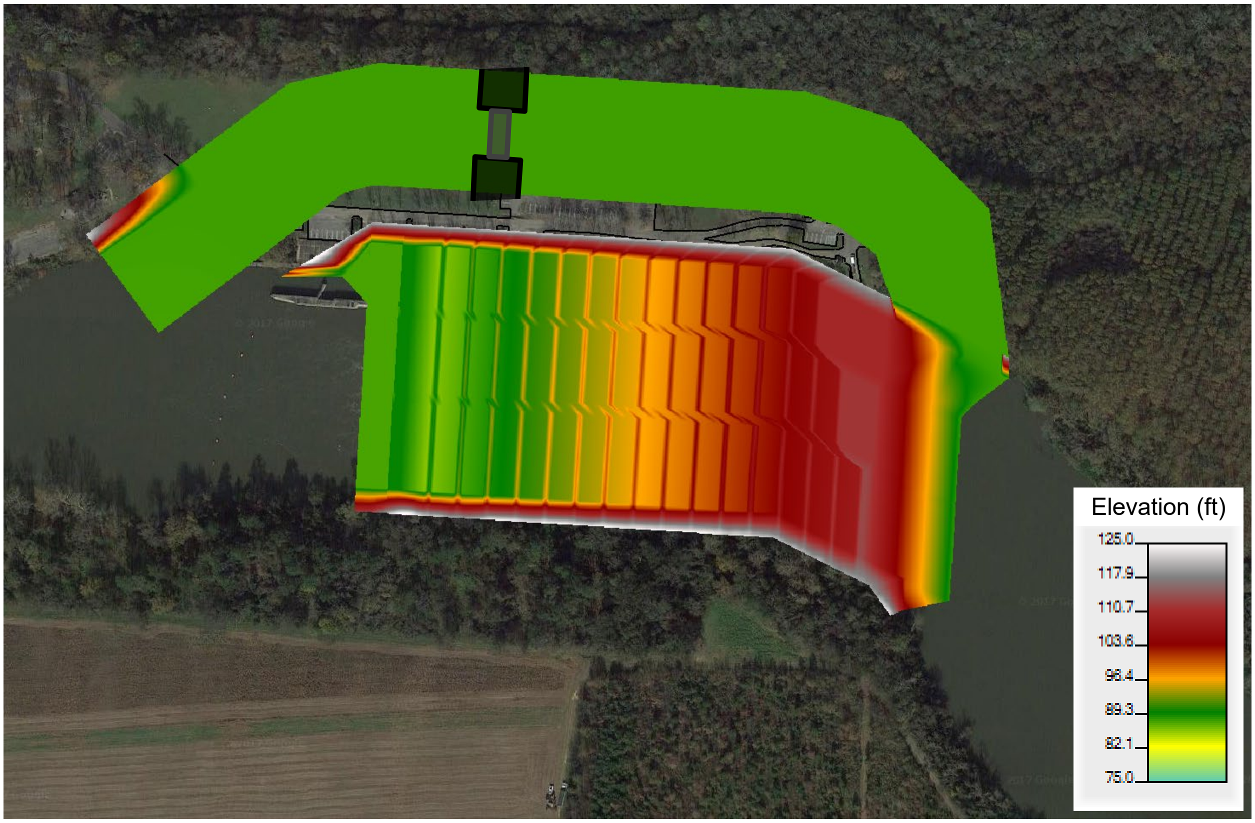


Figure 21 - Alternative 2-7 Layout

2.3.17. Fixed Crest Weir with Gated Bypass Channel – 2 Gates (Alternative 2-8)

Alternative 2-8 consists of a fixed weir with a rock ramp at the existing dam site with an active flood passage structure in an excavated bypass channel through the park on the Georgia side of the river. The fish passage structure would constructed as described in the previously described alternatives, with a weir crest 500 feet in width with an average crest elevation of 109.2 feet NAVD 88 (110.0 NGVD 29). The lock and dam would be removed, including the foundation down to 91.2 NAVD 88. The bypass channel would be approximately 200 feet wide and excavated to elevation 91.2 NAVD 88.

The structure in the bypass channel would consist of **two** 50' wide gates, 40' high used to pass high flows. The gated structure would be operated to pass high flows by fully lifting the gate out of the water during high flows and otherwise remain closed to maintain the pool elevation during low and normal flow conditions.

At the weir, the pool is expected to be 1.3 feet lower than the existing pool elevation for normal flow conditions (5,000cfs). The pool at 5th St. Bridge would be around elevation 113.4 NAVD 88 (0.9 feet lower than existing) during normal flow conditions.

A schematic of the general site configuration and terrain model used for this alternative can be found in Figure 22 below.

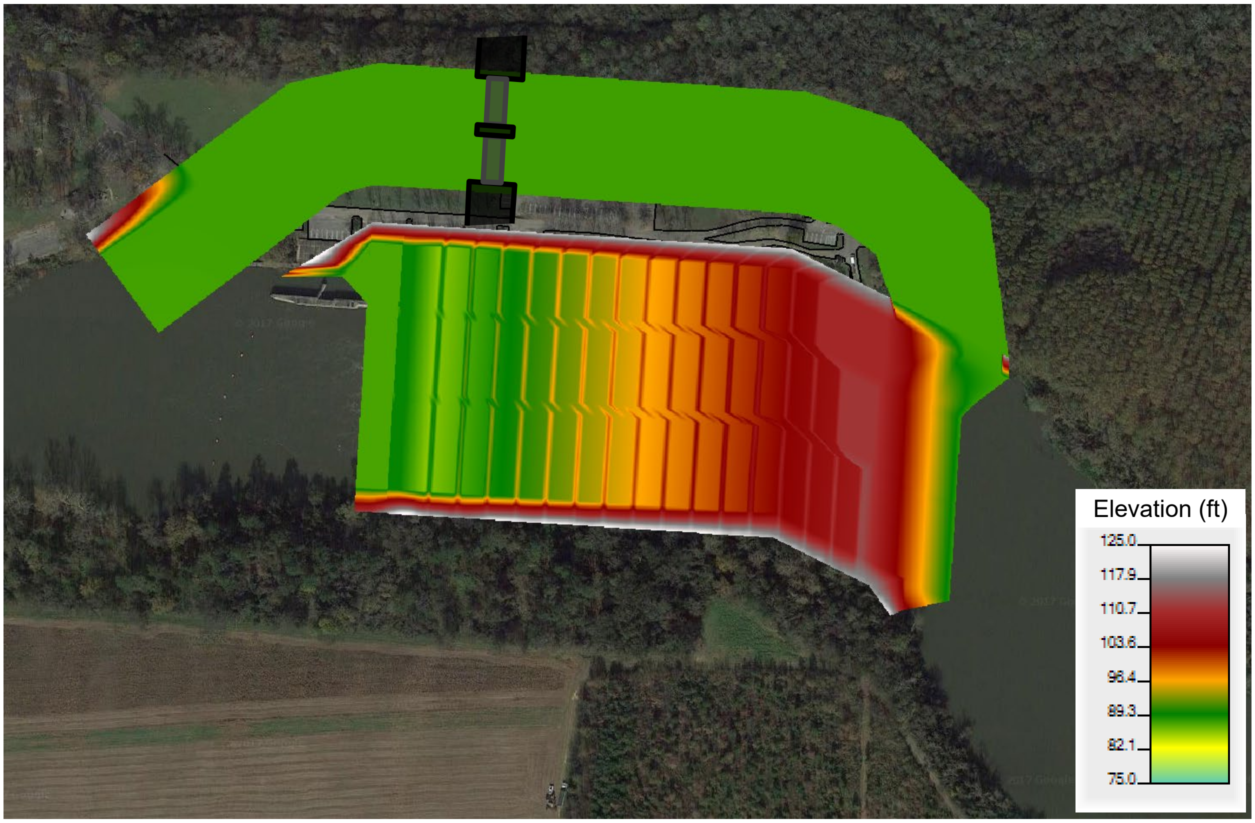


Figure 22 - Alternative 2-8 Layout

This alternative would not cause any additional flooding for the 50% through 1% AEP (2-year through 100-year) flood events as the additional conveyance provided by the two bypass gates is great enough to pass high-flows without inducing additional flood damages in the overbank areas (the single gate in Alternative 2-7 did not provide sufficient additional conveyance to alleviate induced flood impacts). This alternative was evaluated as part of the final array and a cost estimate for construction was developed. See additional discussion on plan formulation in the main report and the Cost Engineering Appendix for detailed construction cost information.

2.3.18. Fixed Crest Weir – 920ft wide (Alternative 2-9)

This alternative includes the removal of both lock walls, removal of dam gates and piers, excavation of the Georgia side park property, and partial demolition of the dam foundation to elevation 91.2 NAVD 88. A rock ramp with a crest 920’ in length will then be placed upstream of the existing dam location, sloping 2% upstream to the ultimate weir crest elevation of 109.2 NAVD 88 (110 feet NGVD 29). The weir crest is in a terraced configuration with the thalweg located on the north side of the weir. The intent of this alternative was to provide a relatively high-elevation weir crest to maintain the pool during low flows while also providing enough conveyance over the weir to minimize additional flooding impacts during high flows.

The terrain model and 2D mesh were modified to incorporate the rock ramp structure into the HEC-RAS model geometry. A manning's n region was created for the area of the rock ramp and an n value of 0.08 specified, in accordance with the procedure used in the 1D model. A schematic of the general site configuration and terrain model used for this alternative can be found in Figure 23 below.

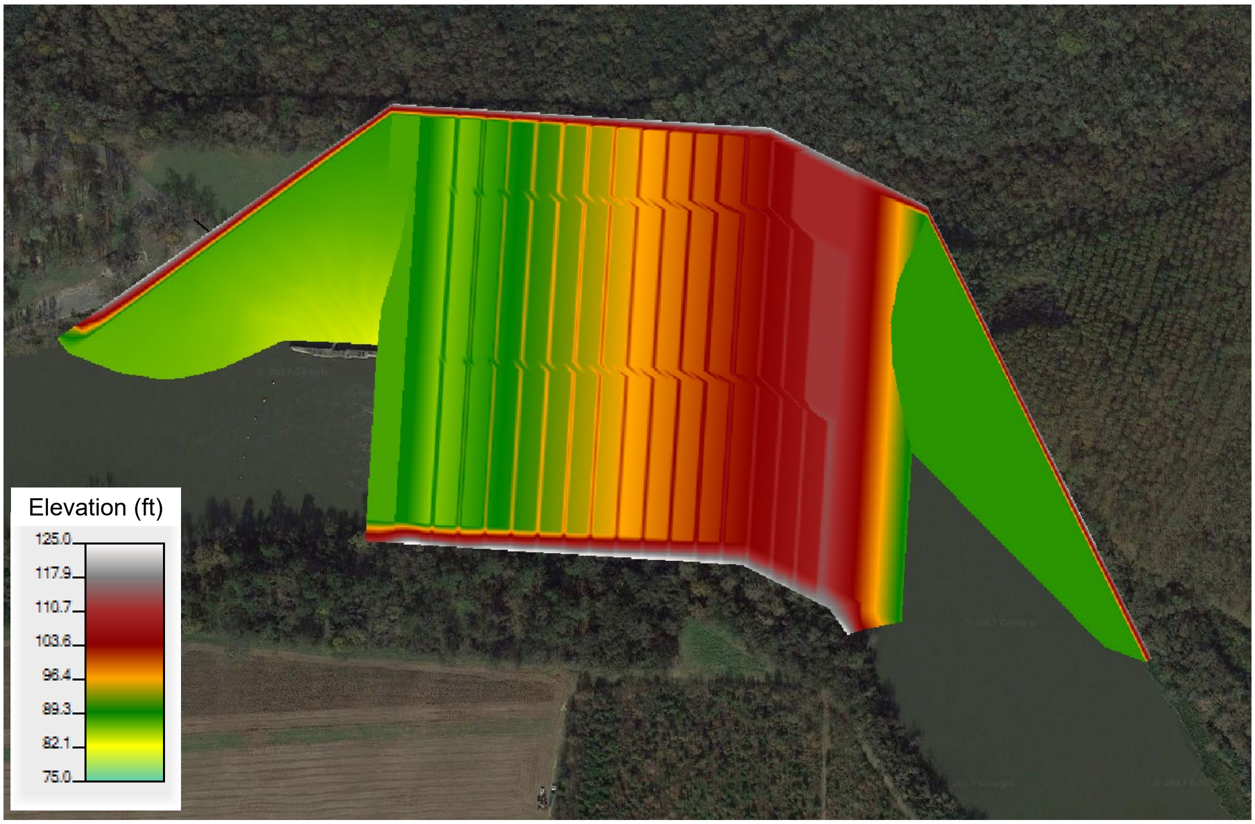


Figure 23 - Alternative 2-9 Layout

At the weir, the pool is expected to be 1.4 feet lower than the existing pool elevation for normal flow conditions. The pool at 5th St. Bridge would be around elevation 113.0 (NAVD 88) (1.3 feet lower than existing) during normal flow conditions (5,000cfs). This alternative has the least impact to pool elevations during low flows, but would still result in a lower normal pool and lower pool during drought flows.

The 920' wide configuration was an attempt to keep a relatively high normal pool under normal flow conditions without inducing flooding. The average weir crest elevation was kept relatively high but also very wide in order to provide enough conveyance to pass high flows. However, flooding was still induced in the overbanks at the weir width/height modeled in this alternative. Several variations on the 920' alternative were also modeled but none were able to provide a higher normal pool without inducing flooding, as compared to the 500' wide alternatives. The n value used to model the rock weir is relatively high (0.08) and makes the structure itself hydraulically inefficient.

This alternative would likely cause an increase flooding depth at hundreds of parcels for the 50% and 20% AEP flood events, though larger flows would have a similar inundation footprint and depth as under existing conditions; the weir simply isn't as hydraulically efficient as the gates in the existing dam structure.

This alternative was not evaluated as part of the final array due primarily to concerns over additional flooding in the overbank areas for the 50% and 20% AEP interval flows; a cost estimate for construction was not developed. See additional discussion on plan formulation in the main report.

2.3.19. Summary of Results

Water surface elevations within the pool are of primary importance in assessing with-project impacts to water supply, recreation, and navigation. The HEC-RAS 2D model was used to compute water surface elevations (and associated inundation, if applicable) for the various alternatives for a range of flow events. A brief summary of the computed water surface elevations at a few key locations can be found in Table 8 below. The locations of interest are immediately upstream of the current NSBLD location, Gum Swamp Road, the Potash/Fibrant water intake, SCE&G, Kimberly Clark, Hicks Raw Water Plant, 5th Street Bridge, and North Augusta Water Intake.

Table 8 - Summary of HEC-RAS Results

Pool Elev @ 3600 cfs (NAVD88)

	Existing	2012 SHEP	Alt 1-1	Alt 1-2	Alt 2-1	Alt 2-2	Alt 2-3	Alt 2-4	Alt 2-5	Alt 2-6a	Alt 2-6b	Alt 2-6c	Alt 2-6d	Alt 2-7	Alt 2-8	Alt 2-9
NSBLD	113.2	112.7	111.6	111.7	106.9	107.0	107.9	109.2	111.6	111.0	107.9	108.8	109.7	111.1	111.1	110.6
Gum Swamp	113.3	112.8	111.7	111.7	107.3	107.4	108.2	109.4	111.7	111.1	108.2	109.1	109.9	111.2	111.2	110.7
Potash et al	113.5	113.1	112.0	112.1	108.5	108.5	109.1	110.1	112.1	111.5	109.1	109.8	110.5	111.6	111.6	110.9
SCE&G	113.5	113.2	112.1	112.2	108.8	108.8	109.4	110.3	112.2	111.7	109.4	110.0	110.7	111.7	111.7	111.0
Kimberly Clark	113.5	113.2	112.1	112.2	108.8	108.8	109.4	110.3	112.2	111.7	109.4	110.0	110.7	111.7	111.7	111.0
Hicks Raw Water	113.7	113.3	112.4	112.4	109.7	109.7	110.1	110.7	112.4	111.9	110.1	110.5	111.1	112.0	112.0	111.3
5th Street Bridge	113.9	113.5	112.5	112.6	110.2	110.2	110.5	111.1	112.6	112.1	110.5	110.9	111.4	112.2	112.2	111.5
N. Augusta	114.0	113.7	112.9	112.9	110.8	110.8	111.1	111.5	112.9	112.5	111.1	111.4	111.8	112.5	112.5	111.9

Pool Elev @ 5000 cfs (NAVD88)

	Existing	2012 SHEP	Alt 1-1	Alt 1-2	Alt 2-1	Alt 2-2	Alt 2-3	Alt 2-4	Alt 2-5	Alt 2-6a	Alt 2-6b	Alt 2-6c	Alt 2-6d	Alt 2-7	Alt 2-8	Alt 2-9
NSBLD	113.2	113.2	112.1	112.1	107.2	107.3	108.4	109.6	112.0	111.6	108.3	109.3	110.2	111.9	111.9	111.3
Gum Swamp	113.4	113.3	112.3	112.3	107.8	107.9	108.8	109.9	112.2	111.8	108.8	109.6	110.5	112.1	112.1	111.5
Potash et al	113.8	113.7	112.8	112.8	109.4	109.5	110.1	110.8	112.7	112.4	110.0	110.6	111.3	112.7	112.7	112.1
SCE&G	113.9	113.8	112.9	112.9	109.8	109.8	110.4	111.1	112.9	112.5	110.3	110.9	111.5	112.8	112.8	112.3
Kimberly Clark	113.9	113.8	112.9	112.9	109.8	109.8	110.4	111.1	112.9	112.5	110.3	110.9	111.5	112.8	112.8	112.3
Hicks Raw Water	114.1	114.0	113.2	113.3	110.7	110.8	111.1	111.7	113.2	112.9	111.1	111.5	112.0	113.1	113.1	112.7
5th Street Bridge	114.3	114.2	113.5	113.5	111.3	111.3	111.6	112.1	113.4	113.2	111.6	112.0	112.4	113.4	113.4	113.0
N. Augusta	114.6	114.6	113.9	113.9	112.0	112.1	112.3	112.7	113.8	113.6	112.3	112.6	112.9	113.8	113.8	113.4

Pool Elev @ 8000 cfs (NAVD88)

	Existing	2012 SHEP	Alt 1-1	Alt 1-2	Alt 2-1	Alt 2-2	Alt 2-3	Alt 2-4	Alt 2-5	Alt 2-6a	Alt 2-6b	Alt 2-6c	Alt 2-6d	Alt 2-7	Alt 2-8	Alt 2-9
NSBLD	113.2	113.2	113.1	112.8	107.8	108.0	109.1	110.2	112.6	111.8	109.1	110.0	110.9	112.5	112.5	111.8
Gum Swamp	113.5	113.5	113.4	113.1	108.7	108.9	109.9	110.8	112.9	112.2	109.8	110.6	111.4	112.8	112.8	112.2
Potash et al	114.4	114.4	114.3	114.0	111.1	111.2	111.7	112.3	113.9	113.4	111.7	112.2	112.8	113.8	113.8	113.3
SCE&G	114.5	114.5	114.4	114.2	111.6	111.7	112.1	112.6	114.1	113.6	112.1	112.5	113.1	114.0	114.0	113.6
Kimberly Clark	114.5	114.5	114.4	114.2	111.6	111.7	112.1	112.6	114.1	113.6	112.1	112.5	113.1	114.0	114.0	113.6
Hicks Raw Water	115.0	115.0	114.9	114.7	112.6	112.7	113.1	113.4	114.6	114.2	113.0	113.4	113.8	114.6	114.6	114.2
5th Street Bridge	115.3	115.3	115.2	115.1	113.3	113.3	113.6	113.9	115.0	114.6	113.6	113.9	114.2	114.9	114.9	114.6
N. Augusta	115.8	115.8	115.7	115.6	114.2	114.2	114.4	114.7	115.5	115.2	114.4	114.6	114.9	115.5	115.5	115.2

Pool Elev @ 50% Annual Chance Exceedance (2-year) (NAVD88)

	Existing	2012 SHEP	Alt 1-1	Alt 1-2	Alt 2-1	Alt 2-2	Alt 2-3	Alt 2-4	Alt 2-5	Alt 2-6a	Alt 2-6b	Alt 2-6c	Alt 2-6d	Alt 2-7	Alt 2-8	Alt 2-9
NSBLD	114.8	114.6	114.6	117.4	113.8	113.8	114.7	115.6	116.7	115.3	114.1	114.5	114.8	115.8	114.5	115.2
Gum Swamp	116.4	116.2	116.2	118.3	115.9	115.9	116.3	117.0	117.8	116.7	115.9	116.2	116.4	117.2	116.2	116.7
Potash et al	119.8	119.7	119.7	120.8	119.7	119.7	119.7	120.0	120.5	119.9	119.5	119.7	119.8	120.1	119.7	119.9
SCE&G	120.2	120.1	120.1	121.2	120.1	120.2	120.2	120.5	120.9	120.4	120.0	120.1	120.2	120.6	120.2	120.4
Kimberly Clark	120.2	120.1	120.1	121.2	120.1	120.2	120.2	120.5	120.9	120.4	120.0	120.1	120.2	120.6	120.2	120.4
Hicks Raw Water	121.6	121.6	121.6	122.4	121.9	121.9	121.6	121.8	122.1	121.7	121.5	121.6	121.6	121.9	121.6	121.7
5th Street Bridge	122.6	122.5	122.5	123.2	122.9	122.9	122.6	122.7	123.0	122.7	122.5	122.5	122.6	122.8	122.5	122.7
N. Augusta	124.0	124.0	124.0	124.5	124.3	124.3	124.0	124.1	124.4	124.1	123.9	124.0	124.0	124.2	124.0	124.1

2.4. Alternative Screening and Final Array

The primary criteria for alternative evaluation are specified in the WIIN Act legislation: cost and impacts to recreation, water supply, and navigation. Flooding is also evaluated as a constraint, in that no alternatives that have high negative impacts from induced flooding should be carried forward. Costs for the various alternatives are discussed in section 12, while impacts to recreation, water supply, navigation, and flooding are discussed below. Alternatives that had little or no impacts to water supply and recreation and that had little to no flooding impacts were carried forward to the final array of alternatives, as discussed in the main report.

2.4.1. Flooding

Impacts of flooding were determined by comparing alternative depth inundation maps for the 2-year and 5-year to depth inundation maps for existing conditions. As the magnitude of flow increases, the conveyance available in the channel becomes a smaller proportion of the overall conveyance available as flow moves out into the overbank areas. The majority of flow is out of the channel at around the 10-year return interval flow and therefore any changes to the existing NSBLD structure has a relatively minor impact to water surface elevations and inundation depths. Therefore, the 2-year and 5-year return interval flows and associated inundations were the primary factors in determining whether or not an alternative would have an adverse impact on flooding.

The number of parcels that showed additional inundated area or greater inundation depth compared to existing conditions were tabulated. Alternatives that impacted a large number of parcels were classified as causing “high” negative impacts to flooding and not carried forward for further consideration. To the extent possible, subjective measures of the severity of flooding were avoided. If a parcel experienced even a 0.01ft increase in inundation depths as a result of the with-project condition, that parcel was counted as “flooded” and would require some type of real estate action. It is important to note that none of the alternatives considered in the focused array of alternatives induced additional flooding that impacted structures.

A list of alternatives that induced flooding and whether or not they were considered further is presented in Table 9 below.

Table 9 - Induced Flooding by Alternative

Alternative	Induced Flooding	Carried Forward
1-1	Yes	**Yes**
1-2	No	No
2-1	No	**Yes**
2-2	No	**Yes**
2-3	No	**Yes**
2-4	Yes	No
2-5	Yes	No

2-6A	Some (94 parcels)	**Yes**
2-6B	No	**Yes**
2-6C	No	**Yes**
2-6D	No	**Yes**
2-7	Yes	No
2-8	No	**Yes**
2-9	Yes	No

2.4.2. Water Supply

Water supply users within the river pool at Augusta are shown in Figure 4. Pool levels that are significantly below existing levels for low and average flow conditions would likely have an impact to water supply users and recreational interests. The HEC-RAS 2D model provided water surface elevations at the water intakes, but additional information regarding intake elevations and pump specifications were required to evaluate impacts of lower water levels. Water supply users self-reported information to USACE regarding required water surface elevations and daily withdrawal volumes. This self-reported information was used to perform an early screening of alternatives, in that if an alternative negatively impacted three or more users the alternative would not be considered for further evaluation. The results of this initial screening based on impacts to water intakes is presented in Table 10 below. After screening, independent verification of information regarding water-intake requirements was needed as the alternative evaluation and plan formulation process progressed.

Table 10 - Water Intakes impacted by Alternative Based on Self-reported Information

Alternative	Intakes Impacted	Carried Forward
1-1	0	**Yes**
2-1	3	**No**
2-2	3	**No**
2-3	2	**Yes**
2-6A	0	**Yes**
2-6B	2	**Yes**
2-6C	1	**Yes**
2-6D	1	**Yes**
2-8	0	**Yes**

A separate report was prepared under task-order to determine existing water-intake capacity and specifications and what, if any, modification to the intake systems would be required to maintain withdrawal capacity (CDM-Smith, 2018). This report summarized the findings of a hydraulic analysis of intake and pump capacity, and proposed mitigation for intakes that would be adversely impacted by proposed fish passage alternatives. The intakes that were identified as need mitigation for the various alternatives are presented in Table 11.

Table 11 - Water Intakes Requiring Modification

Alternative	Water Intakes Requiring Pump/Intake Modification
1-1	none
2-3	City of Augusta, Kimberly Clark
2-6A	none
2-6B	City of Augusta, Kimberly Clark
2-6C	City of Augusta
2-6D	City of Augusta
2-8	none

2.4.3. Recreation
Recreation within the pool is also heavily dependent on maintaining pool levels near existing conditions. Activities such as ironman races, motor boat races, and rowing regattas all require sufficient depth such that obstructions do not impact these events. Alternatives that were expected to have adverse impacts on recreation were not excluded from further consideration as it was assumed these impacts could be mitigated with associated costs accounted for. Generally, any lowering of the normal pool elevation would have an impact to some form of recreation, and all alternatives would lower the normal pool by some amount. Refer to section 2.2.14 in the main report, as well as Appendix F for additional discussion on recreation, impacts, and cost of mitigation.

2.4.4. Navigation
For all alternatives there is effectively no impact to navigation as there is currently no commercial navigation through NSBLD. The lock at NSBLD was closed in 2014 due to safety concerns and the last commercial lockage was in the 1970's. All alternatives considered for fish passage would maintain a minimum 9' deep navigation channel within the pool, which would be required for commercial navigation if it were to be resumed along this portion of the river.

2.4.5. Final Array
The results of the flooding and water intake analysis largely determined which alternatives were kept for consideration in the final array of alternatives. However, impacts to recreation are also important and are discussed in detail in Appendix F and the body of the main report. Alternatives that were kept for the final array had cost estimates developed, as presented in section 12 and discussed in detail in Appendix B. The alternatives composing the final array are presented in Table 12 below.

Table 12 - Final Array of Alternatives

Alternative	Description
Alt 1-1	Repair Dam and Lock Wall, 200’ wide fish passage structure on GA side of the dam through the demolished lock chamber and esplanade
Alt 2-3	Remove Dam, fixed crest fish passage structure 500’ wide at elevation 107
Alt 2-6A	Remove Dam, fixed crest fish passage structure 500’ wide at elevation 110, with excavated floodplain bench
Alt 2-6B	Remove Dam, fixed crest fish passage structure 500’ wide at elevation 107, with excavated floodplain bench
Alt 2-6C	Remove Dam, fixed crest fish passage structure 500’ wide at elevation 108, with excavated floodplain bench
Alt 2-6D	Remove Dam, fixed crest fish passage structure 500’ wide at elevation 109, with excavated floodplain bench
Alt 2-8	Remove Dam, fixed crest fish passage structure 500’ wide at elevation 110, with excavated bypass channel gated flood control structure (2 x 50’ gates)

2.4.6. Recommended Plan

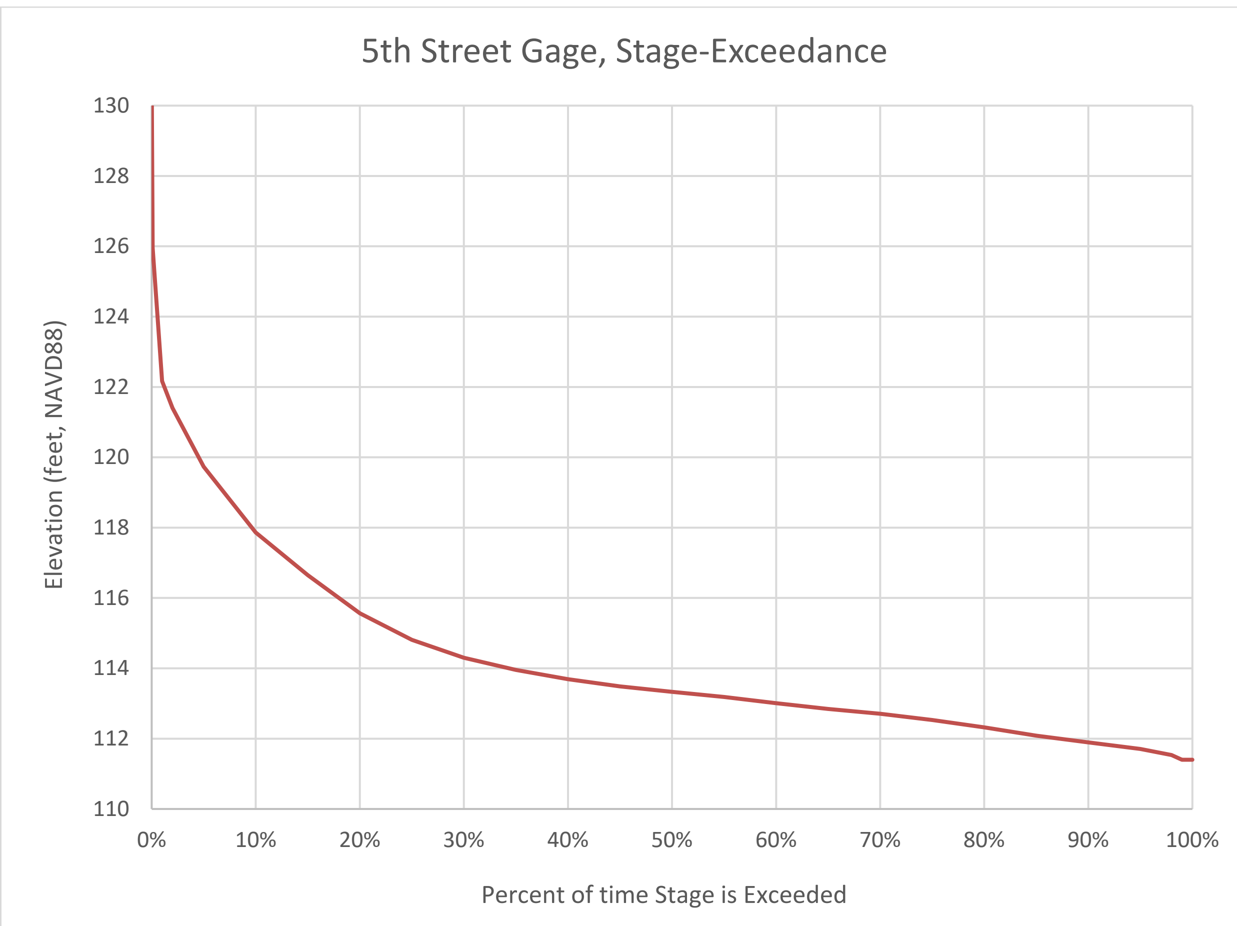
As described in the main report, Alternative 2-6D is the recommended plan. This alternative best meets the project objectives, while adhering to the project constraints. The alternative consists of a fixed crest weir with a rock ramp at the existing dam location and a low-lying floodplain bench in the right overbank to provide additional flow conveyance. The lock and dam would be removed, including the foundation down to elevation 91.2 NAVD 88. The weir would be 500 feet in width with an average crest elevation of 107.2 feet NAVD 88 (108.0 NGVD 29). A floodplain bench approximately 275 feet in width would be excavated down to elevation 110 NAVD 88 (approximately 3.8ft higher than the adjacent rock-ramp terrace) on the Georgia side of the existing dam location.

Water surface elevations in the pool under this alternative are listed in Table 8. However, additional sensitivity runs varying the selected n coefficient for the rock-ramp were also completed for this alternative. The selected n value for the rock ramp was 0.08, as discussed in Section 2.1.2. Sensitivity runs varying this n value by +/- 0.01 were completed for the 3,600cfs, 5000cfs, and 33,000cfs flow conditions. The water surface elevations at locations of interest for these sensitivity runs are shown in Table 13 - Sensitivity Analysis Results for Recommended PlanTable 13 below.

Table 13 - Sensitivity Analysis Results for Recommended Plan

	Pool Elev @ 3600 cfs (NAVD88)			Pool Elev @ 5000 cfs (NAVD88)			Pool Elev @ 2-year (NAVD88)		
	Low n	Selected n	High n	Low n	Selected n	High n	Low n	Selected n	High n
NSBLD	109.6	109.7	109.9	110.1	110.2	110.3	114.6	114.8	114.9
Gum Swamp	109.8	109.9	110.1	110.3	110.5	110.6	116.2	116.4	116.5
Potash et al	110.4	110.5	110.7	110.2	111.3	111.4	119.7	119.8	119.8
SCE&G	110.6	110.7	110.8	111.4	111.5	111.6	120.2	120.2	120.3
Kimberly Clark	110.6	110.7	110.8	111.4	111.5	111.6	120.2	120.2	120.3
Hicks Raw Water	111.0	111.1	111.2	111.9	112.0	112.1	121.6	121.6	121.6
5th Street Bridge	111.3	111.4	11.5	112.3	112.4	112.5	122.5	122.6	122.6
N. Augusta	111.7	111.8	111.9	112.9	112.9	113.0	124.0	124.0	124.0

The values in the table above and more detailed hydraulic model output can be used to develop a stage-frequency curve to better illustrate how pool levels may vary as a result of the recommended plan.



The HEC-RAS model was not built specifically to provide detailed velocity mapping for the rock-ramp structure. Simplifying assumptions that affect computed velocities were made regarding the weir's geometry configuration in order to computer water surface elevations upstream of the structure. Detailed velocity mapping at the crest, terraces, and pools between boulders will be completed during detailed design of the structure. With those limitations in mind, velocities in the project area for this alternative under the 2-year flow condition range from approximately 1ft/s to 10ft/s at the crest. It is unlikely that the detailed design of the project would yield peak velocities much higher than those seen here, though the spatial distribution is likely to differ significantly from that seen here. Figure 24 below shows a velocity map of the project area for the 50% ACE (2-year) flow condition.

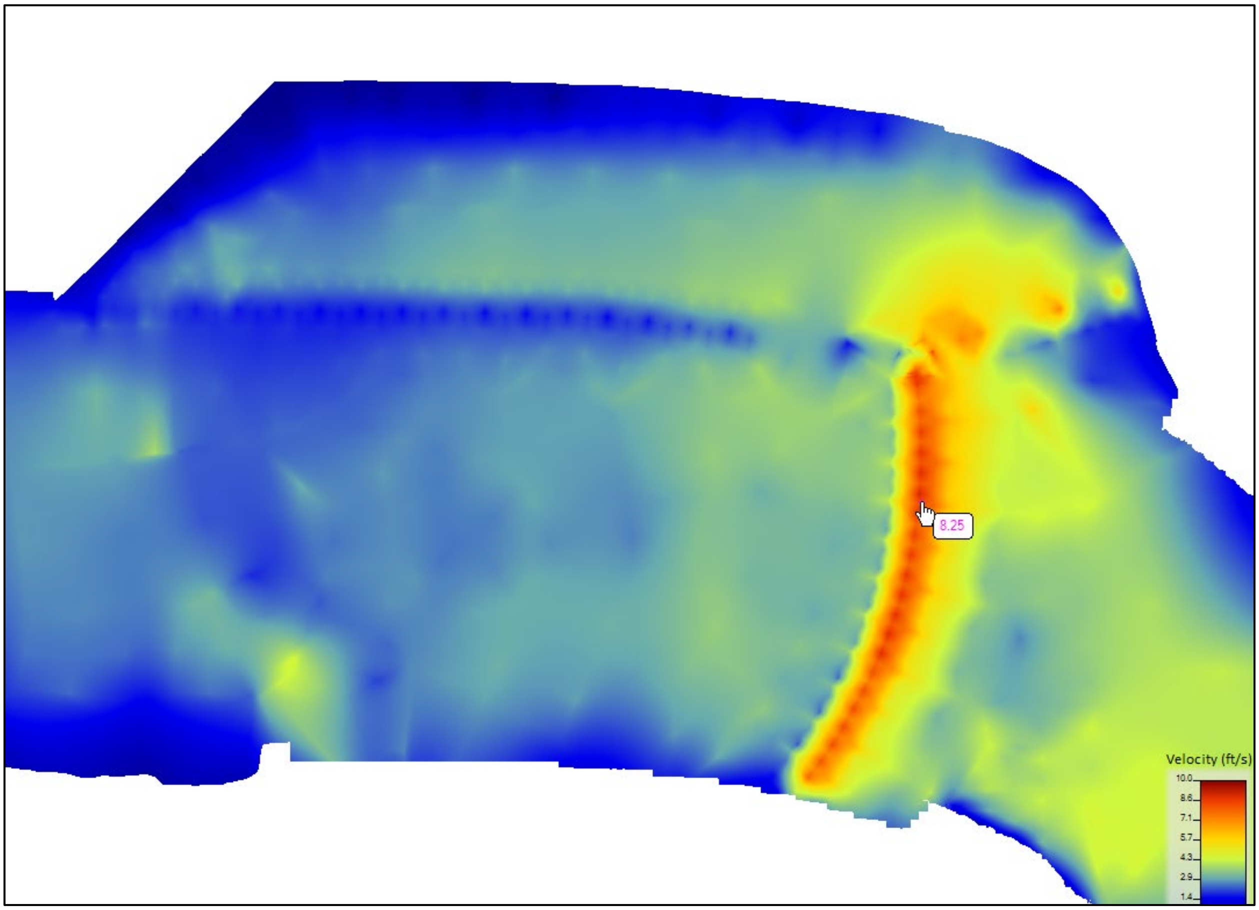


Figure 24 - Velocity Map for the 2-year flow and Recommended Plan

3. Surveying, Mapping, and Other Geospatial Data Requirements

3.1. Terrain Data

LiDAR point cloud data were obtained for the study area from https://coast.noaa.gov/dataviewer/# and processed by the Savannah District office. Point clouds for Aiken and Richmond counties were used to generate a DEM, with vegetation removed. Richmond County data have a horizontal accuracy of plus or minus 4 ft (1.21 m) at the 95% confidence level, the Fundamental Vertical Accuracy (FVA) tested 0.197 ft (0.06 m) RMSE at 95% confidence level. Aiken County data have a horizontal accuracy of plus or minus 11 ft (3.353 m) at the 90% confidence level, and a vertical accuracy of 1.2ft (0.366 m) at 90% confidence level. The extents of the terrain data are shown in Figure 5 in the previous section. Metadata for the original LiDAR dataset can be found at the NOAA website, though it should be noted that the elevation values were converted from meters to feet (NAVD 88) for the purposes of this project.[2]

3.2. Bathymetric Data

The Lidar data obtained from NOAA did not contain elevation data below the water surface of the Savannah River. The terrain model was supplemented with bathymetric data collected by the Hydrographic Survey section at the Savannah District. Bathymetric survey sections were spaced approximately every 1000ft from the Augusta Shoals to approximately one mile downstream of the dam. Additional sections spaced every 50ft were taken in the vicinity of the dam. All survey points were provided in NAVD 88 elevation. The Bathymetric data and topographic data were used to create a new “Terrain” layer within RAS Mapper to cover the entire study domain, as discussed in section 2.2.

4. Geotechnical

4.1. Regional and Site Geology

The New Savannah Bluff Lock and Dam (NSBLD) is located at 33.37256^{O}, −81.94094^{O} on the Savannah River approximately 13 miles below Augusta, Georgia.

The project site is located in the Atlantic Coastal Plain Physiographic Province. The Coastal Plain consists of horizontal to gently dipping Mesozoic and Cenozoic sedimentary rocks; primarily limestone, sandstone, and shale. The Coastal Plain is bound to the west by the Piedmont Physiographic Province. The crystalline Piedmont rocks, remnants of pre-Mesozoic orogenic events, dip seaward and form the basement of the Coastal Plain deposits. Figure 25 shows the physiographic provinces and major fault systems in the region surrounding the NSBLD.

The Fall Line is the boundary between the Piedmont and the Coastal Plain. Its name

[2] https://coast.noaa.gov/htdata/lidar1_z/geoid12b/data/5112/sc2012_scdnr_aiken_m5112_metadata.html

arises from the occurrence of waterfall and rapids that are the inland barriers to navigation on Georgia's major rivers. The Fall Line is a boundary of bedrock geology, but it can also be recognized from stream geomorphology. Upstream from the Fall Line, rivers and streams typically have very small floodplains, if they have any at all, and they do not have well-developed meanders. Within approximately a mile downstream from the Fall Line, rivers and streams typically have floodplains or marshes across which they flow, and within three or four miles they meander. The most pronounced example of this is in the Savannah River's course at Augusta.

The Coastal Plain is a region of Cretaceous and Cenozoic sedimentary rocks and sediments. These strata dip toward the southeast, and so they are younger nearer the coast. At least near the Fall Line, they are ultimately underlain by igneous and metamorphic rocks like those of the Piedmont. The sedimentary rocks of the Coastal Plain partly consist of sediment eroded from the Piedmont over the last 100 million years or so, and partly of limestone generated by marine organisms and processes at sea. One could generalize that buried Triassic rocks in the subsurface are various rift-basin siliciclastics, the Cretaceous strata are sandstones and shales, the Tertiary strata are limestones and shales, and that the Quaternary strata are sands and muds.

The outcrops at the New Savannah Bluff Lock and Dam are mostly Quaternary alluvium composed of unconsolidated sand and gravel located primarily on the river's flood plain. Underlying the alluvium are sediments of Cretaceous to Eocene in age. They are dominantly terrestrial to shallow marine in origin and consist of sand, kaolinitic sand, kaolin, and pebbly sand. The sediments are underlain by metamorphic and igneous rocks including granite, biotite gneiss, granite gneiss, and amphibolite.

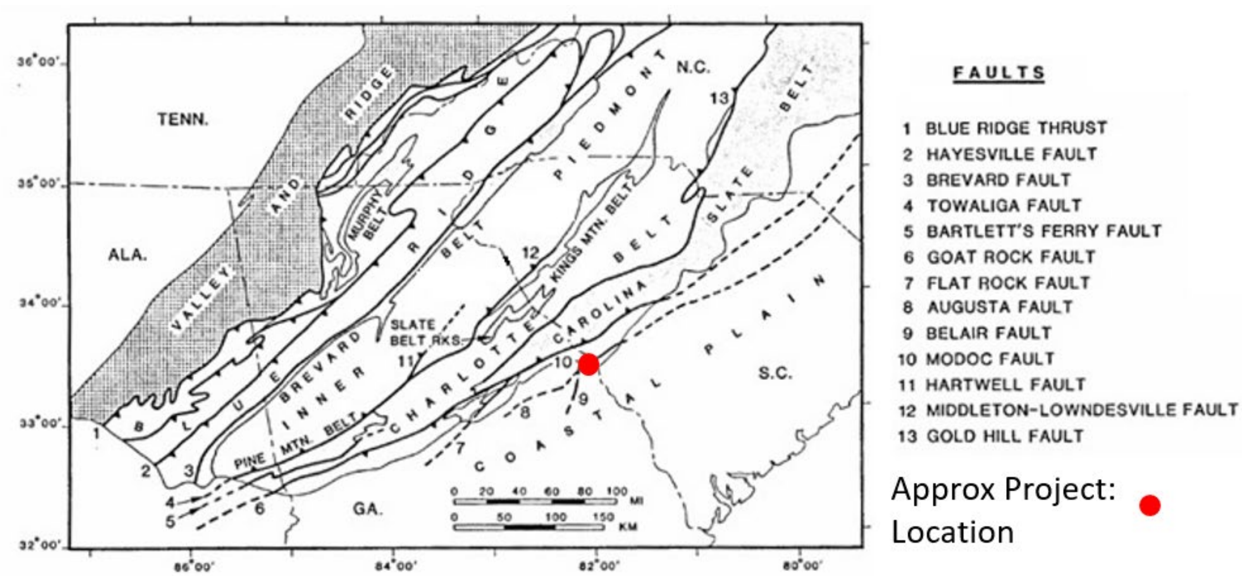


Figure 25 - Physiographic provinces and major fault systems in the region surrounding New Savannah Bluff Lock & Dam

4.2. Seismic Activity

Seismicity of the Coastal Plain is not well understood. The buried crystalline rocks show remnants of late Triassic rifting. Basins formed during the rifting are identified as magnetic lows, while intrusive igneous rocks are identified by magnetic highs. These heterogeneities may alternately concentrate and release stresses beneath the Coastal Plain. The release of stress may cause displacement along reactivated faults that extend up into the overlying sedimentary deposits. Sources of stress may be related to regional compression from the spreading Atlantic basin, the deep intrusion of magma along ancient rifts, or extensional movement caused by a sagging graben.

No earthquake-capable faults are recognized in the region surrounding NSBLD.

Moderate seismicity is experienced in the vicinity of NSBLD despite the lack of identified active faults. The Project is located within the South Carolina Trend, which is a broad, seismically-active source zone extending from the coastal plain of South Carolina to near the eastern border of Tennessee, roughly paralleling the Savannah River. Other regional seismic sources zones include Charleston, Eastern Tennessee, Giles County (Virginia), Central Virginia, and the more distant New Madrid. These source zones have the potential to produce more powerful earthquakes than the South Carolina Trend; in particular, Charleston and New Madrid. The greatest seismic hazard to the project, however, comes from a large magnitude earthquake originating from the Charleston source zone. Figure 26 is a United States Geological Survey (USGS) uniform hazard response spectra map showing the peak ground acceleration (PGA) in acceleration of gravity units (g) with a 2% probability of exceedance (PE) in 50 years. The seismic source zones described above are identified on this map.

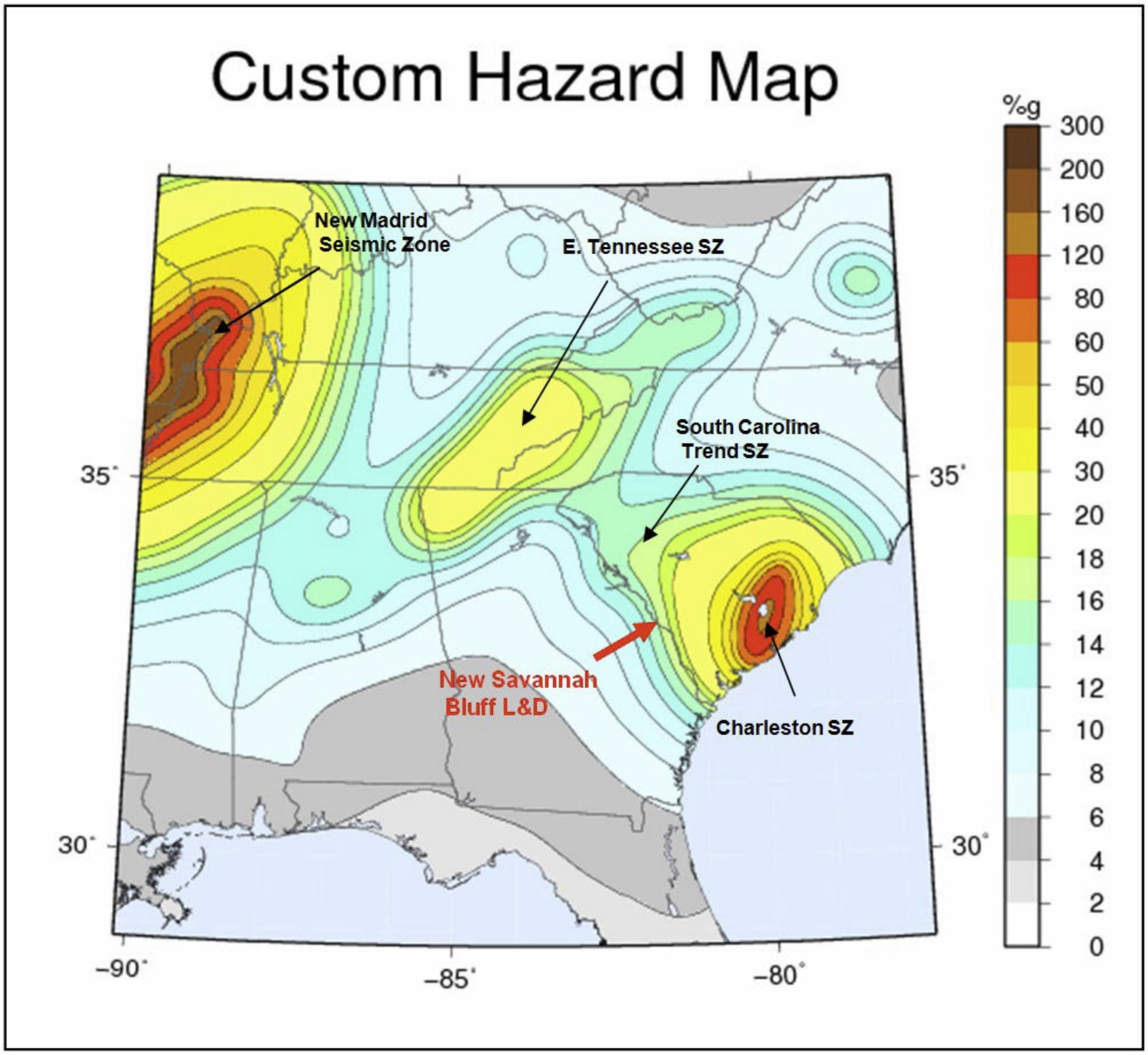


Figure 26 - Peak ground acceleration (PGA) in acceleration of gravity units (g) with a 2% probability of exceedance (PE) in 50 years

4.3. Subsurface Explorations

Available subsurface explorations in the vicinity of the project site were completed prior to this study as indicated below.

- Explorations during the design phase prior to construction of the NSBLD (1934).
- Explorations during the Section 216 Disposition Study (2002).
- Explorations during PED for the Fish Passage Structure at NSBLD (2014).

Information from these investigations was utilized for the conceptual design. However, additional subsurface explorations will need to be conducted during the PED phase prior to construction.

4.3.1. Explorations for Construction of the NSBLD (1934)

Explorations conducted for the purposes of design of the New Savannah Bluff Lock and Dam were found in the project design drawings for construction dated 8 March 1934. The categories used to classify the soils encountered during explorations are clay (C), gravel (G), loam (L), quicksand (QS), sand (S). They are further classified as black (B), chalk (Ch), coarse (Co), fine (F), mud (M), mica (Mi), red (R), yellow (Y), white (W). Borings located upstream of the dam within the river channel are largely sandy soils from the river bottom to approximately elevation 50 ft MSL. There is no other information available from this exploration effort other than what is noted on the drawing (Figure 27).

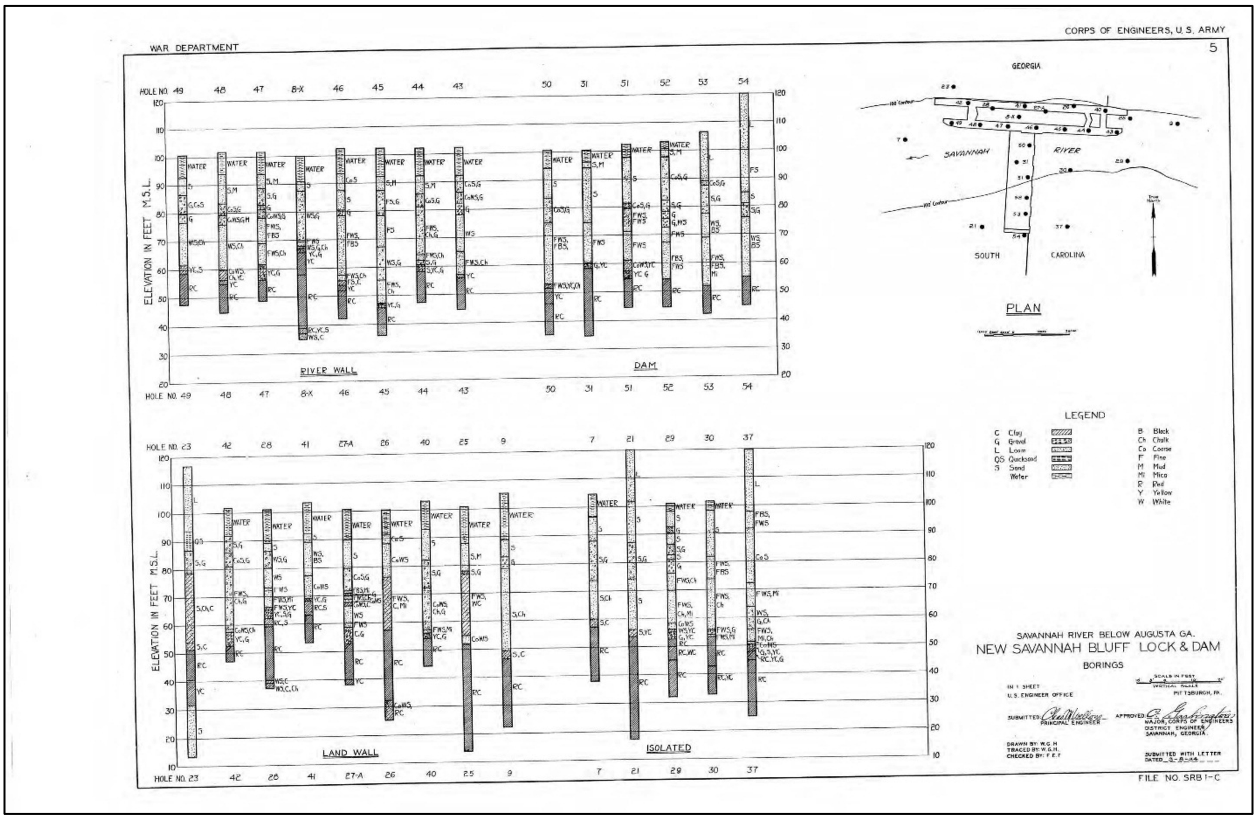


Figure 27 - New Savannah Bluff Lock and Dam Borings (1934)

4.3.2. Explorations for the Section 216 Disposition Study (2002)

Thompson Engineering, in conjunction with the Savannah District, completed subsurface explorations and geotechnical assessment of soil conditions at the NSBLD as part of the Section 216 Disposition Study. Field explorations took place in 2002. The results of the field exploration and laboratory testing programs, and geotechnical information regarding site and soil conditions for the rehabilitation of the lock and dam concluded in two major findings:

1. Lock Floor Repair: There is evidence that the lock floor has voids beneath it, in some areas. The rehabilitation of the floor will require filling with grout beneath the floor.

2. Lock River Wall Erosion Repair: The rehabilitation of the lock river wall is required due to the noted erosion at the mudline near the foundation on the riverside of the wall. The lock walls are supported on timber piles with a sheet pile enclosure at the perimeter. It is noted that there is up to ¼ inch of lateral movement during the filling of the lock chamber. It is further anticipated that there are voids beneath the lock wall, which will require filling.

In support of the 2002 design work, four (4) Standard Penetration Tests were performed: two along the land-side of the lock wall and two from a barge on the river-side of the lock wall. The two land borings were performed approximately ten feet from the land-side edge of the lock wall, and aligned with the lower miter gate (B-1) and the dam stilling basin (B-2). The two marine borings (MB-01 and MB-02) were performed adjacent to the downstream lock wall although the exact locations were not recorded.

Land borings were drilled using a Failing 1500 drill rig. Borings varied in depth from 91 to 100 feet. Top of hole elevations were approximately 123 feet MSL. The borings were advanced using the mud-rotary drilling technique and split-barrel sampling operations were performed mostly on 5.0 foot intervals to the boring termination depths. Boring B-1 contained split-barrel sampling on 5.0 foot intervals to a depth of 30 feet with continuous sampling from 30 feet to boring termination.

Marine borings were drilled using a trailer rig Simco Drilling unit mounted on a barge. Borings were drilled to depths of 71.5 feet below the (2002) mudline elevation. Casing was utilized to advance the boring from the barge deck to the mudline. Due to the encountered gravel strata, the casing was required to depths of 30 to 35 feet below the mudline. The borings were advanced using the mud-rotary drilling technique and split-barrel sampling operations were performed on 5.0 foot intervals to the boring termination depths.

Standard Penetration Test values were obtained using a standard split-barrel sampler with a 2.0 inch outer diameter and a recovery barrel length of greater than 24 inches. At each depth interval, the split-barrel sampler was driven into “undisturbed” soils by repetitive blows of a standard 140 pound hammer free-falling a distance of 30 inches. The final 12 inches of sampler penetration, after first seating six inches, is defined as the SPT value. Test procedures are described in the American Society for Testing and Materials (ASTM) Specification D1586.

SPT values, expressed in units of blows per foot, are presented numerically and graphically opposite the respective depth interval on the Soil Boring Logs in Figures –27 - 30. Soil samples were taken from the borings in accordance with the procedures presented in ASTM D1586 "Standard Method for Penetration Test and Split Barrel Sampling of Soils." The samples recovered were visually classified by a qualified soils technician or engineer, logged, placed in moisture-tight plastic bags or jars, and transported to the laboratory. At the laboratory, all samples were examined by a trained soils technician, and visual classifications were adjusted where necessary. The depths where the samples were taken, the results of the standard penetration tests, and the visual classifications of the soils encountered are presented on the attached soil test boring logs. All testing was performed in accordance with ASTM standards.

Summary of Laboratory Testing Program: The results of the laboratory classification tests performed on selected samples can be found on the soil test boring logs attached. Brief descriptions of the laboratory analyses performed as a part of this study are present in the following paragraphs.

Soil Classifications: Each soil sample recovered during the subsurface exploration program was visually classified in the field and then re-examined in the soils engineered laboratory by a qualified soils technician. The visual classifications were assigned based on an estimate of relative particle size and soil plasticity characteristics. Sieve analyses, moisture contents, and Atterberg Limits tests were performed on selected samples to confirm the visual classifications. All tests were conducted in accordance with ASTM standards. Test results are presented on the soil test boring logs.

Abbreviated Grain Size Tests: Relative grain size distribution was determined by performing U.S. Standard No. 200 washes and sieve analyses. In performing the tests, each soil sample was oven dried and then washed over a U.S. Standard No. 200 Sieve. The material retained on the sieve was then oven dried and weighed. The percent combined silt and clay fraction is defined as the percent by weight of material passing the No. 200 sieve. Test procedures were in accordance with ASTM D1140. Test results are presented on the attached soil test boring logs.

Atterberg Limits Tests: To obtain information regarding soil consistency with variations in soil water content or soil plasticity characteristics, Atterberg Limits Tests are performed. The Atterberg Limits are defined as the Liquid Limit (LL) and Plastic (PL) which are the moisture contents at which the soil sample is in the boundary condition between the liquid and plastic state, and between the plastic and semi-solid state, respectively. The Plastic Index (PI) represents the range of moisture content over which the soil will behave as a deformable material and is determined as the numerical difference between the LL and PL. Test methods are described in ASTM D4318. Test results are presented on the soil test boring logs.

Moisture Content: Additional information regarding soil compressibility characteristics and geologic pre-loading (of fine-grained soils) may be estimated though water content values in conjunction with Atterberg Limits. The water content is defined as the weight of water in a moist soil sample expressed as a percentage of the soil sample’s total oven dry weight. The moisture contents were performed in accordance with ASTM D2216. Test results are presented on the soil test boring logs.

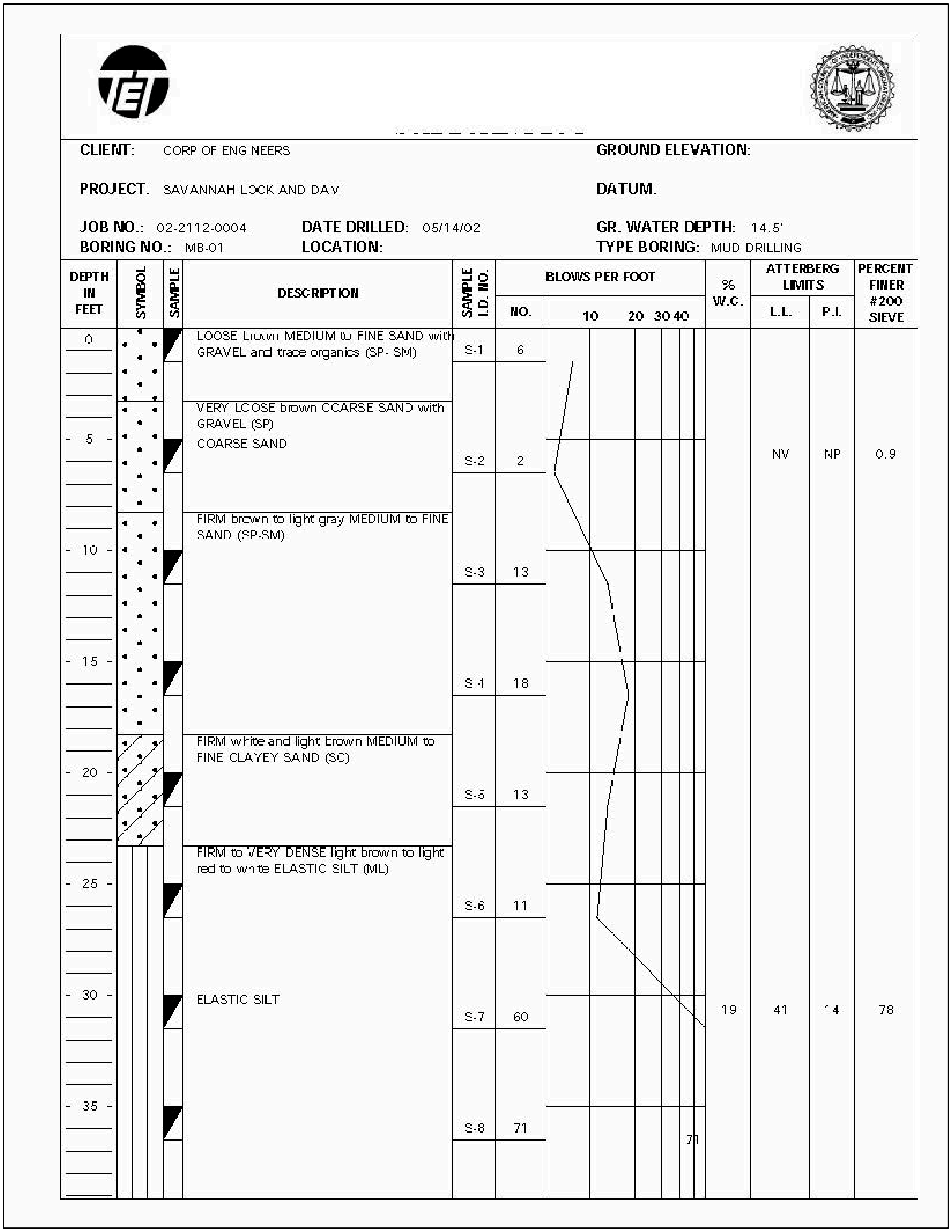


Figure 28 - New Savannah Bluff Lock and Dam Borings (2002)

CLIENT: CORP OF ENGINEERS

GROUND ELEVATION:

PROJECT: SAVANNAH LOCK AND DAM

DATUM:

JOB NO.: 02-2112-0004

DATE DRILLED: 05/14/02

GR. WATER DEPTH: 14.5'

BORING NO.: MB-01

LOCATION:

TYPE BORING: MUD DRILLING

DEPTH IN FEET	DESCRIPTION	SAMPLE I.D. NO.	NO.	% W.C.	ATTERBERG LIMITS L.L.	ATTERBERG LIMITS P.I.	PERCENT FINER #200 SIEVE
40	FIRM to VERY DENSE light brown to light red to white ELASTIC SILT (ML)	S-9	67				
45		S-10	57				
	FIRM white to light brown MEDIUM to FINE CLAYEY SAND (SC)						
50		S-11	30				
	VERY DENSE white, brown and light red CLAY (CL) CLAY						
55		S-12	50/5"	23	48	23	85.14
	FIRM white MEDIUM to FINE CLAYEY SAND (SC)						
60		S-13	26				
	DENSE white MEDIUM to FINE SILTY SAND (SM)						
65		S-14	47				
70		S-15	41				
	B.T. @ 71.5'						
75							

BLOWS PER FOOT: 10 20 30 40

67

Figure 29 - New Savannah Bluff Lock and Dam Borings (2002)

CLIENT: CORP OF ENGINEERS
GROUND ELEVATION:

PROJECT: SAVANNAH LOCK AND DAM
DATUM:

JOB NO.: 02-2112-0004 **DATE DRILLED:** 05/15/02 **GR. WATER DEPTH:** 14.0'
BORING NO.: MB-02 **LOCATION:** **TYPE BORING:** MUD DRILLING

DEPTH IN FEET	DESCRIPTION	SAMPLE I.D. NO.	BLOWS PER FOOT NO.	% W.C.	ATTERBERG LIMITS L.L.	ATTERBERG LIMITS P.I.	PERCENT FINER #200 SIEVE
0	LOOSE white to brown MEDIUM to FINE CLAYEY SAND (SC)	S-1	10				
5	LOOSE to FIRM white MEDIUM to FINE SILTY SAND (SM) SILTY SAND	S-2	10		NV	NP	15
10		S-3	14				
15		S-4	12				
20	HARD white to light brown SANDY CLAY (CL) SANDY CLAY	S-5	44	17	37	14	37
25	VERY DENSE white to light red ELASTIC SILT (MH)	S-6	50/5"				
30		S-7	72				
35	- dark red to light brown to white	S-8	50/5"				

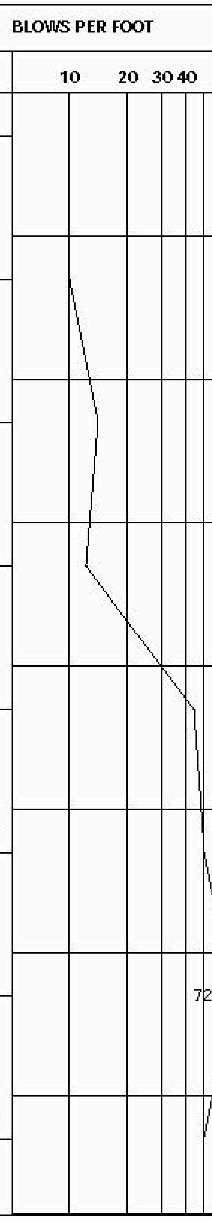


Figure 30 - New Savannah Bluff Lock and Dam Borings (2002)

CLIENT: CORP OF ENGINEERS
GROUND ELEVATION:

PROJECT: SAVANNAH LOCK AND DAM
DATUM:

JOB NO.: 02-2112-0004
DATE DRILLED: 05/15/02
GR. WATER DEPTH: 14.0'

BORING NO.: MB-02
LOCATION:
TYPE BORING: MUD DRILLING

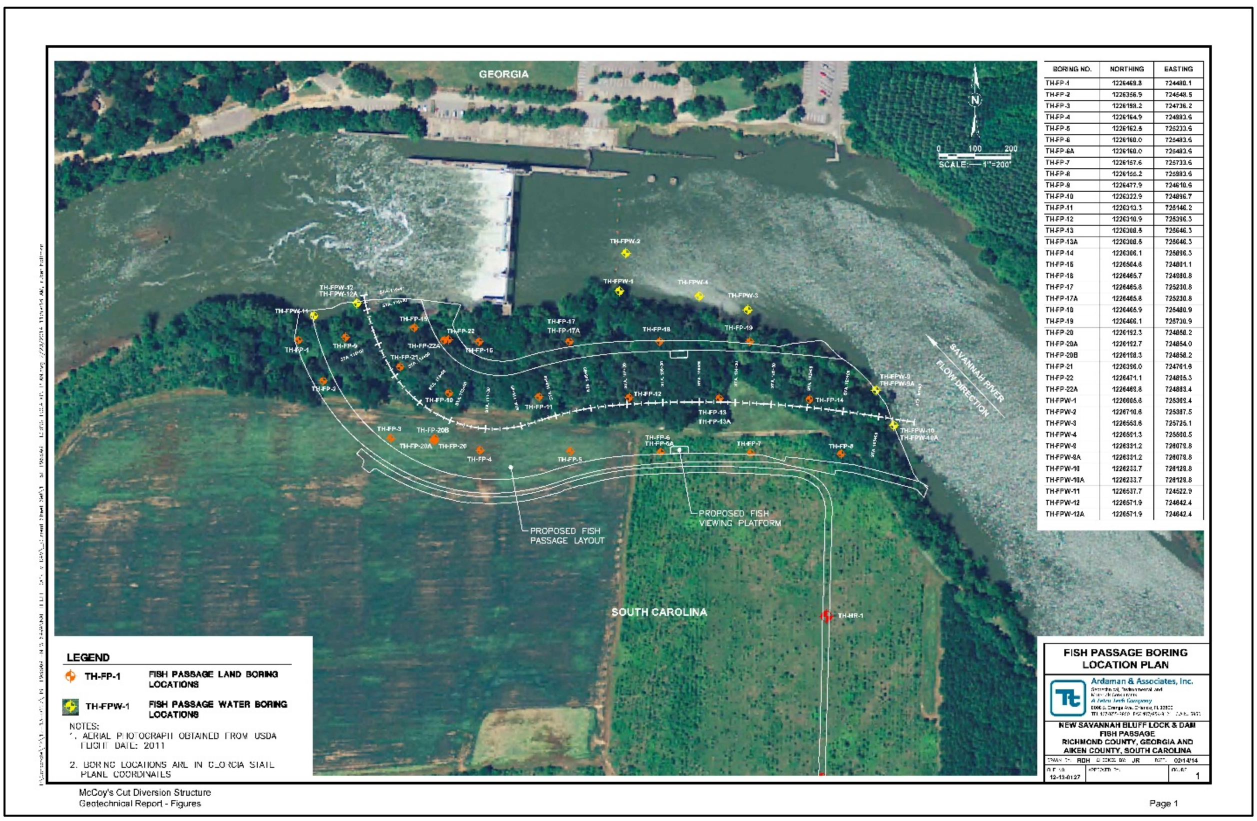
DEPTH IN FEET	DESCRIPTION	SAMPLE I.D. NO.	BLOWS PER FOOT NO.	% W.C.	ATTERBERG LIMITS L.L.	ATTERBERG LIMITS P.I.	PERCENT FINER #200 SIEVE
40	VERY DENSE dark red to light brown ELASTIC SILT (MH) ELASTIC SILT	S-9	50/5"	23	58	23	96
45	- white	S-10	83				
50	DENSE white MEDIUM to FINE SAND (SP-SM)	S-11	41				
55	VERY DENSE white MEDIUM to FINE CLAYEY SAND (SC)	S-12	50/0"				
60	VERY DENSE white MEDIUM to FINE SILTY SAND (SM)	S-13	84				
65	VERY DENSE white MEDIUM to FINE SAND (SP-SM)	S-14	59				
70	VERY DENSE white to light brown to purple ELASTIC SILT (MH)	S-15	89				
	B.T. @ 71.5'						
75							

Figure 31 - New Savannah Bluff Lock and Dam Borings (2002)

4.3.3. Explorations for the Fish Passage (2014)

Explorations for the 2014 Fish Passage design were conducted during development of the plans and specifications (P&S) which was finalized in 2014 as a mitigation feature of the Savannah Harbor Expansion Project. Details from this investigation can be found in the *Subsurface Explorations and Geotechnical Engineering Report* dated April 2014 which is included as part of the 2014 SHEP NSBLD Fish Passage Basis of Design.

The drilling program was conducted in November 2012 and February 2013 by Ardaman & Associates, Inc., a Tetra Tech Company. Laboratory testing of samples collected was conducted at their office laboratory in Orlando, FL. The area investigated focused largely on the South Carolina bank where the proposed fish passage structure and associated haul roads would be constructed (Figure 32). There were 6 water boring locations and 5 land boring locations that are applicable to the in-stream rock weir design. However, they are all located along or adjacent to the southern bank (South Carolina). There are no other borings available that would lend more information on what might be encountered on the northern bank (Georgia). As part of this effort 53 Standard Penetration Test (SPT) borings were advanced to depths of 15 to 75 feet (Elevation 102.3 to 43.0 feet NAVD 88).

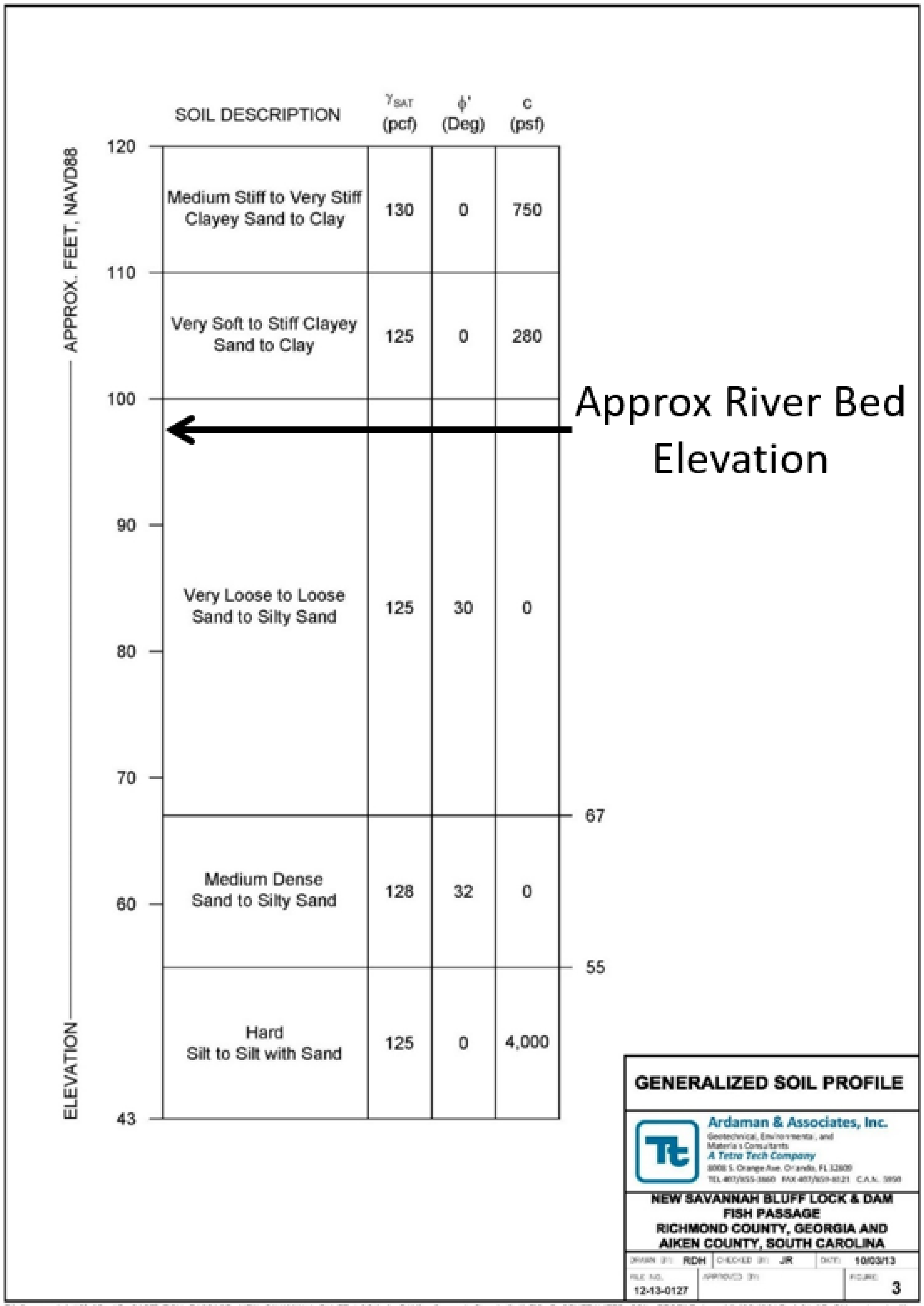


BORING NO.	NORTHING	EASTING
TH-FP-1	1226469.8	724480.1
TH-FP-2	1226356.9	724648.5
TH-FP-3	1226198.2	724736.2
TH-FP-4	1226164.9	724983.6
TH-FP-5	1226162.6	725233.6
TH-FP-6	1226160.0	725483.6
TH-FP-6A	1226160.0	725483.6
TH-FP-7	1226157.6	725733.6
TH-FP-8	1226156.2	725983.6
TH-FP-9	1226477.9	724610.6
TH-FP-10	1226322.9	724896.7
TH-FP-11	1226313.3	725146.2
TH-FP-12	1226310.9	725396.3
TH-FP-13	1226308.6	725646.3
TH-FP-13A	1226308.6	725646.3
TH-FP-14	1226306.1	725896.3
TH-FP-15	1226504.6	724801.1
TH-FP-16	1226465.7	724980.8
TH-FP-17	1226465.8	725230.8
TH-FP-17A	1226465.8	725230.8
TH-FP-18	1226465.9	725480.9
TH-FP-19	1226466.1	725730.9
TH-FP-20	1226192.3	724860.2
TH-FP-20A	1226192.7	724854.0
TH-FP-20B	1226198.3	724858.2
TH-FP-21	1226396.0	724761.6
TH-FP-22	1226471.1	724895.3
TH-FP-22A	1226469.8	724883.4
TH-FPW-1	1226605.6	725369.4
TH-FPW-2	1226710.6	725387.5
TH-FPW-3	1226553.6	725725.1
TH-FPW-4	1226591.3	725580.5
TH-FPW-8	1226331.2	726078.8
TH-FPW-8A	1226331.2	726078.8
TH-FPW-10	1226233.7	726129.8
TH-FPW-10A	1226233.7	726129.8
TH-FPW-11	1226537.7	724522.9
TH-FPW-12	1226571.9	724642.4
TH-FPW-12A	1226571.9	724642.4

Figure 32 - Fish Passage Boring Locations (2014)

A generalized soil profile determined for the Fish Passage project site is shown in Figure 33. The materials encountered in the borings generally consist of medium stiff to stiff clayey sand to clay to an approximate elevation of +110 feet (NAVD 88) underlain by generally very soft to medium stiff clayey sand to clay to an approximate elevation of +100 feet (NAVD 88). Below the clayey sand to clay layer, the borings encountered generally loose sand to an approximate elevation of +67 feet (NAVD 88) underlain by generally medium dense sand to silty sand to an approximate elevation of +55 feet (NAVD 88). The sand to silty sand layer is underlain by hard silt to silt with sand to the boring termination elevation of +43 feet (NAVD 88).

Figure 34 and Figure 35 show the summary of index testing results for the fish passage borings that were within the water (Savannah River). These samples are designated as TH-FPW (-1, -2, -3, -4, -9, -9A, -10, -10A, -11, -12, and -12A). The summarized data includes stratum depths, soil description, SPT N-values, moisture content, and the percent passing the No. 200 sieve.



T:\Corporate\12\12-13-0127\FISH PASSAGE-NEW SAVANNAH BLUFF LOCK & DAM_Current Sheet Set\FIG 3 GENERALIZED SOIL PROFILE.dwg 10/22/2013 4:01:05 PM, ruben.hoffman

Figure 33 - South Carolina Overbank Generalized Soil Profile

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New Savannah Bluff Lock and Dam Fish Passage
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Table 2 Summary of Index Testing Results for Fish Passage Borings (Cont'd)

Boring Name	Stratum Upper Depth (ft)	Stratum Lower Depth (ft)	Sample Number	Soil Description	SPT N-value (blows/ft)	Moisture Content (%)	Percent Passing U.S. Std. No. 200 Sieve (%)	Liquid Limit	Plasticity Index
TH-FPW-1	14.5	16.0	6	CLAY WITH SAND (CL), brown	1	45	76	39	17
	21.0	22.5	8	SAND WITH GRAVEL (SP), fine to coarse, brown	7	17	3	-	-
	31.0	32.5	10	SAND WITH SILT (SP-SM), fine to medium, gray	6	22	6	-	-
TH-FPW-2	12.5	14.0	1	SAND (SP), fine to medium; brown	2	27	4	-	-
	26.0	27.5	8	SAND WITH SILT (SP-SM), fine to medium, gray, with wood	4	25	5	-	-
	36.0	37.5	10	SILTY CLAYEY SAND (SC-SM), pale gray, calcareous	15	14	20	-	-
TH-FPW-3	5.5	7.0	1	SILTY SAND (SM), grayish brown	2	37	19	-	-
	11.5	13.0	4	SAND WITH SILT (SP-SM), gray	6	20	7	-	-
	26.0	27.5	9	SAND WITH SILT (SP-SM), fine to coarse, orangish brown	12	17	8	-	-
TH-FPW-4	18.0	19.5	4	SAND (SP), fine to coarse, orangish brown	4	21	2	-	-
	31.0	32.5	9	CLAYEY SAND (SC), gray	4	32	23	-	-
	36.0	37.5	10	SILTY CLAYEY SAND (SC-SM), pale gray, calcareous	26	18	18	-	-
TH-FPW-9	10.0	11.5	2	SANDY CLAY (CL), brown	WOH	41	63	37	16
	17.5	19.0	7	SILTY SAND (SM), brown	3	24	15	-	-
	21.0	22.5	8	SAND WITH CLAY (SP-SC), fine to coarse, gray, with wood	5	26	8	-	-
	26.0	27.5	9	SAND WITH SILT (SP-SM), brown	8	22	6	-	-
	31.0	32.5	10	SAND WITH CLAY (SP-SC), fine to medium, orangish brown	17	22	9	-	-

Figure 34- Fish Passage Index Testing Results for Water Samples Taken in the Savannah River.

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New Savannah Bluff Lock and Dam Fish Passage
File Number 12-13-0127
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Table 2 Summary of Index Testing Results for Fish Passage Borings (Cont'd)

Boring Name	Stratum Upper Depth (ft)	Stratum Lower Depth (ft)	Sample Number	Soil Description	SPT N-value (blows/ft)	Moisture Content (%)	Percent Passing U.S. Std. No. 200 Sieve (%)	Liquid Limit	Plasticity Index
TH-FPW-9A	9.5	11.5	US-1	CLAYEY SAND TO CLAY WITH SAND (SC to CL), brown	-	33	32	-	-
					-	45	80	-	-
	14.5	16.5	US-2	SANDY CLAY (CL), brown	-	46	61	-	-
					-	39	33	-	-
TH-FPW-10	10.5	12.0	3	SILTY SAND (SM), gray	2	51	44	-	-
	21.0	22.5	8	SAND (SP), fine to medium, orangish brown	8	21	2	-	-
TH-FPW-10A	15.5	17.5	US-1	SAND (SP), grayish brown	-	25	4	-	-
TH-FPW-11	10.0	11.5	5	SAND (SP), fine to medium, orangish brown	5	25	3	-	-
	16.0	17.5	8	SAND WITH GRAVEL (SW), fine to medium, light brown	6	18	3	-	-
	21.0	22.5	9	SAND (SP), fine to medium, gray	7	18	3	-	-
	26.0	27.5	10	SILTY CLAYEY SAND (SC-SM), pale gray, calcareous	14	22	34	-	-
TH-FPW-12	11.5	13.0	3	SAND (SP), fine to medium, orangish brown	-	17	5	-	-
	21.0	22.5	8	SILTY CLAYEY SAND (SC-SM), pale gray, calcareous	14	21	17	-	-
TH-FPW-12A	16.0	18.0	US-1	SAND WITH SILT (SP-SM), fine to medium, orangish brown	-	21	12	-	-

Figure 35 - Fish Passage Index Testing Results for Water Samples Taken in the Savannah River.

4.4. Design Parameters

4.4.1. Design Parameters determined during the Section 216 Disposition Study (2002)

Geotechnical assessment of soil conditions at the project site was based on subsurface information and soil test data obtained at the designated test locations as presented in this report. In evaluating the data, correlations were used which have been previously made between Standard Penetration Test values, soil strength data and behavioral characteristics observed in soil conditions similar to those encountered at the project site. The geotechnical design criteria and recommendations presented in this report are predominately based on guidelines found in Design Manual NAVFAC DM-7 "Soil Mechanics, Foundations, and Earth Structures" prepared by the Department of the Navy and comply with USACE Engineering Regulations.

The general soil profile of the marine borings may be described in terms of approximate depths as shown on the boring logs. All borings generally indicated loose silty sands from existing mudline elevation extending to an approximate depth of 8 feet. Firm silty sands were encountered from a depth of 8 feet to a depth of 21 feet. Stiff clays and very dense silts were encountered from a depth of 21 feet to a depth of 49 feet. Firm sands were encountered from a depth of 49 feet to a depth of 54 feet. Very stiff clays were encountered from a depth of 54 feet to a depth of 59 feet. Very dense sands were encountered from a depth of 59 feet and extending to the maximum boring termination depth of 71.5 feet. Land-side borings displayed a similar profile, but are overlain with additional sandy material to approximate elevation 75 and topped with a medium dense sand/gravel layer to elevation 123.

4.4.2. Design Parameters determined during the 2014 Fish Passage Study

The following engineering properties (Figure 36) were selected for each soil type based on the results of the field exploration and laboratory testing programs performed for Fish Passage. The selected soil properties were used for slope stability analyses in that design.

6.1 Clayey Sands to Clays

The shear strength parameters for the clayey sand to clay soil strata are as follows:

For the medium stiff to stiff clayey sand to clay stratum at EL +120 to +110 ft (NAVD88):

Total (Saturated) Unit Weight: $\gamma_t = 130$ pcf

Shear Strength: c = 750 psf, $\phi = 0°$

For the very soft to medium stiff clayey sand to clay stratum at EL +110 to +100 ft (NAVD88):

Total (Saturated) Unit Weight: $\gamma_t = 125$ pcf

Shear Strength: c = 280 psf, $\phi = 0°$

6.2 Sand to Silty Sand

The shear strength parameters for the sand to silty sand soil strata are as follows:

For the loose sand to silty sand stratum at EL +100 to +67 ft (NAVD88):

Total (Saturated) Unit Weight: $\gamma_t = 125$ pcf

Shear Strength: $\bar{c} = 0$ psf, $\bar{\phi} = 30°$

For the medium dense sand to silty sand stratum at EL +67 to +55 ft (NAVD88):

Total (Saturated) Unit Weight: $\gamma_t = 128$ pcf

Shear Strength: $\bar{c} = 0$ psf, $\bar{\phi} = 32°$

6.3 Silt to Silt with Sand

The shear strength parameters for the hard silt to silt with sand stratum at EL +55 to +43 (ft NAVD) are as follows:

Total (Saturated) Unit Weight: $\gamma_t = 125$ pcf

Shear Strength: c = 4,000 psf, $\phi = 0°$

Figure 36 - Fish Passage Engineering Properties

4.5. Foundation Repairs for Lock and Dam

Provided herein illustrates the recommended foundation design for repairs to the lock and dam, as would be required for Alternative 1-1. Due to the subsurface exploration results and topographic data, global stability is not a concern.

Lock Floor Repair: Voids beneath the lock floor adjacent to the lock river wall have been documented in previous Periodic Inspection reports. At least two (2) dye tests have been conducted where dye was placed into lock floor weep hole via garden hose. Within a short period of time, the dye appeared in the adjacent river. The proposed dye path appears to have traveled beneath the lock floor and adjacent river wall into the river. Some of the dye traveled downstream beneath the river wall and surfaced from beneath the river wall just downstream of the lower miter sill. The dye path provides the void evidence and concerns of grouting beneath the river wall would be pointless and incomplete unless partial grouting beneath the lock floor was also brought into the project.

Grouting beneath the lock floor would first consist of providing a “grout wall” to outline the void areas in question by using a stiffer mix grout for early setting. This “grout wall” can also be used to make a grid within the lock floor to further locate and quantify the void(s). The grouting will be installed into the existing lock floor “weep holes” and be left to set. After the initial set of the grout, the “weep holes” should be re-drilled, grout removed, backfilled with granular material and a containment screen installed over each “weep hole”.

<u>Lock River Wall Erosion Repair</u>: This repair includes a sheet pile wall with a top elevation of +102 ft and tip elevation of +50 ft. This wall will encapsulate No. 57 gravel to an approximate elevation of +82 ft with a five (5) foot overlay of riprap to an approximate elevation of +87 ft. Adjacent to the sheet pile wall, the riprap will be placed on a 3:1 slope and extend to a distance required to meet the existing mudline elevation. The gradations for the No. 57 stone and riprap are provided in Table 14 below:

Table 14 - Stone Gradations

Stone Size	Sieve	Percent Passing by Weight
No. 57 Stone	1 ½”	100
	1”	95-100
	½”	25-60
	#4	0-10
	#8	0-5
	Weight (lbs)	
Riprap	2000	> 90%
	1000	< 50%
	200	< 25%

Based on a review of the soil conditions encountered at the project site, the following soil design parameters were utilized for design of the bulkhead wall planned.

Soil Description	Depth (feet)	Submerged Unit Weight (pcf)	Earth Pressure Coefficients			φ, degrees	C, psf
			Ko	Ka	Kp		
Loose Silty Sands	0 to 8	50	0.56	0.39	2.13	26	0
Firm Silty Sands	8 to 21	55	0.55	0.38	2.22	27	0
Stiff Clay	21 to 24	65	1.00	1.00	1.00	0	3000
Very Dense Silt	24 to 49	65	0.50	0.33	2.5	30	0
Firm Sands	49 to 54	65	0.50	0.33	2.5	30	0
Very Stiff Clays	54 to 59	65	1.00	1.00	1.00	0	3000
Very Dense Sands	59 to 71	65	0.45	0.29	2.83	33	0
Off-site Borrow “Class 5 Rip Rap”	To be determined	75	0.36	0.217	3.83	40	0
Off-site Borrow “No. 57 Gravel”	To be determined	65	0.41	0.26	3.21	36	0
Off-site Borrow “Select Sand”	To be determined	60	0.50	0.33	2.5	30	0

Notes: 1. The phi indicated for the clay strata is for “long term” design and shall be used with a maximum cohesion as noted.

2. The Kp Values should have a factor of safety of 1.2 and 1.5 applied for flexible and rigid walls, respectively.

In general, active earth pressures are used for flexible walls and at-rest earth pressures for rigid walls. Utilizing the aforementioned soil parameters and information obtained within the hydrographic survey, we have performed bulkhead analyses for the proposed wall planned. In summary, we have computed a required pile tip elevation of +50 ft-msl.

Based on the results of the above analyses we recommend that AZ-18 sheet piles be utilized for construction of the bulkhead.

Earthwork Limitations for the Lock River Wall Erosion Repair: Earthwork shall extend 15 feet out from lock chamber structure and continue on a 3:1 slope until the fill material intersects the existing mudline. See sheets S-104 and S-106 for further details.

Special Construction Considerations:

- An implementation of an exploratory program is of utmost importance to determine the limits of the erosion areas to assist in the quantity of materials required. The exploratory program shall include a diver/camera investigation in areas within the lock floor and beneath the lock wall.

- The construction sequence for the repairs, which include underwater grouting, is critical and shall follow as outlined herein. The first grouting task is the lock floor repairs, and shall be completed prior to all other grouting. Second, all grouting repairs beneath the lock wall shall be completed with the last construction task being to install the sheet pile system with associated gravel and riprap erosion protection.

- Lock Floor Repair: As noted above, this repair should be performed first. This procedure shall include installing “grout walls” to outline the proposed eroded area and to form a grid. This procedure will limit the grout movement and bound the grout to within the designated areas and this will also help assist with quantifying materials.

- Lock River Wall Erosion Repair: Riprap material shall be installed by dropping the rock at a fall distance no more than two (2) feet, to minimize damage to the existing sheet pile wall.

- A sheet pile wall is recommended to contain the No. 57 gravel material at a distance of 15 feet from the lock chamber.

- Borrow/Disposal Sites: To be determined during final design.

- Suitability of Material for Slope Protection: The project does not require any excavating of existing material, so all slope protection material shall be from off-site and meet the riprap specifications presented.

Additional Exploration:

- We recommend that a study be performed during detailed design to determine the extent of the voids beneath the lock wall and floor. This study would likely include drilling cores through the lock wall along with using cameras through the underlying sheet pile wall.

4.6. Additional Subsurface Investigation and Testing

Previous investigations are insufficient to conduct a thorough feasibility analysis and develop a sufficiently detailed estimate for the tentatively selected plan. There are two primary areas on the proposed project site that require additional information and testing that will be discussed further below.

Right Abutment of Lock and Dam/Flood Plain Bench

The first area in need of additional exploration is the right abutment on the Georgia side of the Lock and Dam where the parking lot and park are currently located. This location will be the site of the future excavation for the floodplain bench. There is currently only one historic boring in this location from the 1934 design explorations for the Lock and Dam. The location of the boring and the stick log are shown on Figure 37. There are two other borings (B-1 and B-2) that were drilled directly adjacent to the landside lock wall as part of the 2002 lock wall investigation. These logs indicated large layers of gravel that were interpreted to be pervious backfill placed against the lock wall excavation to prevent buildup of excessive hydrostatic pressure against the monoliths. Since the borings are likely not within natural soils they were not considered.

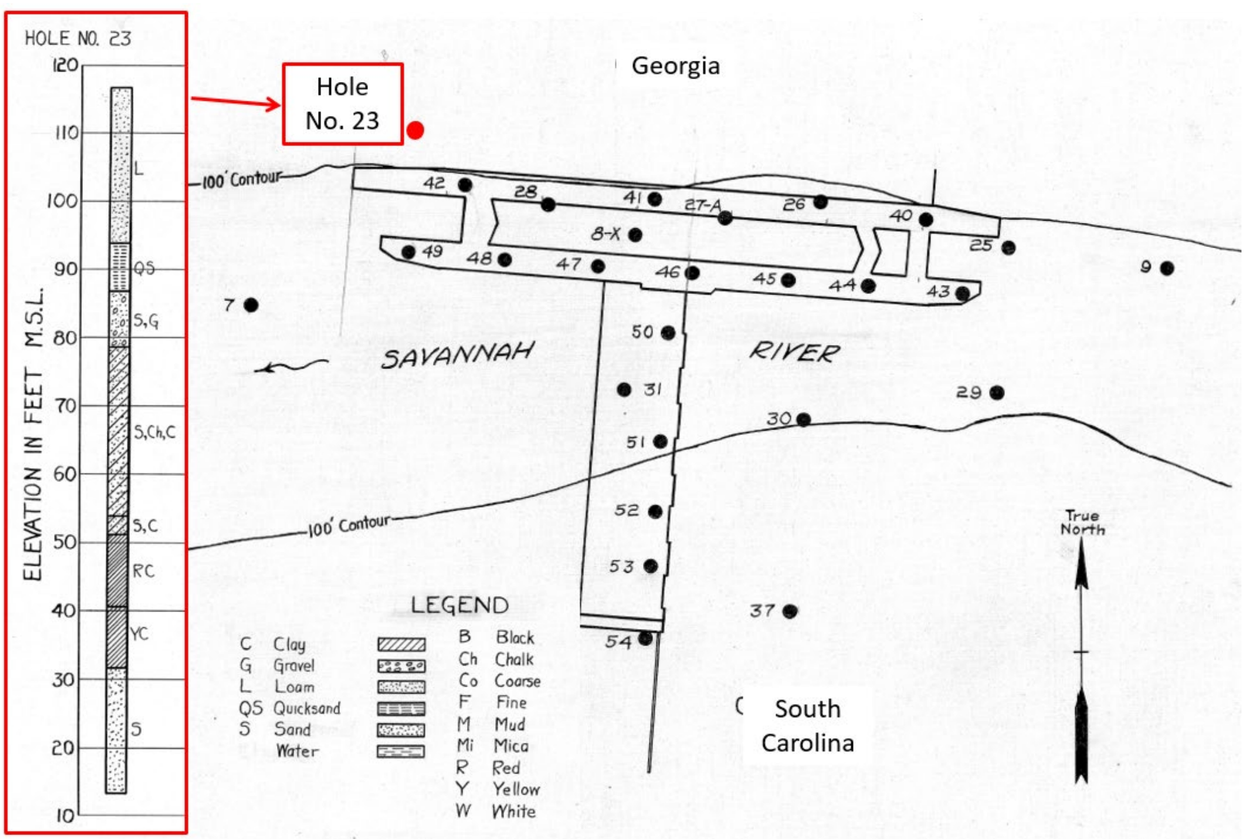

Figure 37 - Plan View of 1934 Pre-Design Borings

The current plan is to excavate in this location approximately 10 feet below the ground surface over an area of approximately 6 acres. Boring No. 23 indicates that the soils in this location consist of “Loam” approximately 20 feet thick underlain by about 10 feet of “Quick Sand”. It is assumed that Loam means sand with variable amounts of silt/clay. A review of historical documents did not reveal a definition for Quick Sand but it is assumed that this implies loose clean sand. The material in this area will be excavated and could possibly be utilized as fill on the bottom of the river channel. The sediments on the riverbed and extending to some depth (discussed in paragraph XX below) are assumed to be loose and likely unsuitable to support the rock weirs currently proposed. As such, these material will have to be densified (if possible) or more likely excavated and replaced. The material to be excavated to form the floodplain bench could provide a convenient replacement for the river sediments. In order to better characterize the soils on the right abutment it is proposed that at least 4 SPT borings to a depth of 20 feet below ground surface be carried out. The preliminary location of these borings will be shown along with the other additional explorations in Figure 39 at the end of this section. The samples collected will be tested for index properties to determine their suitability to act as a filter for the river sediments that will be left in place. This will also help determine the specifications for the geotextile that will provide filter capability between the natural slope material and the bedding stone that will be placed upon it on the right side of the floodplain bench. The SPT blow counts will be corrected and utilized with correlations to develop strength parameters for the cut slopes to determine the feasibility of the currently proposed 1V:2H natural cut slopes. This will include rapid drawdown failure that could occur during periods where the river is high on the floodplain bench then quickly recedes after the flooding has passed. Finally, bulk samples will be collected to determine compaction characteristics using Standard and/or Modified Proctor testing in case the material is utilized as fill.

River Channel Upstream of the Lock and Dam

The second primary exploration area is within the river channel where the rock weirs that make up the fish passage will be constructed. Two marine borings (MB-1 and MB-2) were drilled as part of the 2002 lock wall study. These borings were drilled adjacent to the downstream lock wall (exact location not recorded) and were extended to 71.5 feet below the mudline. The borings are located on the downstream side of the Lock and Dam as opposed to the upstream side where the weirs will be constructed. As such, it is assumed that the sediments near the surface of the borings may not be representative of those that the weir will be founded on since sedimentation is assumed to be more prevalent upstream. However, the data is considered to be valuable for characterizing soils deeper within the foundation. MB-1 and MB-2 indicate relatively clean loose sands from the ground surface to a depth of about 10 feet. The sands continue to a depth of 20 feet with the blow counts increasing in this range. Below the sands both borings encountered dense to very dense plastic soils with high blow counts greater than 40 blows per foot that were about 20 feet thick. These materials were identified on the boring logs as elastic silt (MH). Two Atterberg tests were carried out within this layer with percent finer than #200 greater than 80% and PI values of 14 and 23. Below this layer, extending to the termination of the borings, were layers of dirty sands (SC and SM) with blow counts of 25 or greater with large intervals greater than 40 blows per foot. Boring MB-01 indicated a 10 foot thick layer of very dense lean clay

(CL) in about the middle of the dirty sands. Boring MB-02 encountered a very dense layer of elastic silt (MH) at the termination of the boring.

In addition to the 2002 borings there were eight borings at six locations within the river upstream of the Lock and Dam. These explorations were part of the 2014 design effort for the fish passage. SPT was carried out at all locations. At two of the locations, another boring was drilled adjacent for the purpose of collecting undisturbed samples (identified with an “A” added to the boring designation number). The location of the borings is shown in Figure 38 below relative to the location of the Lock and Dam.

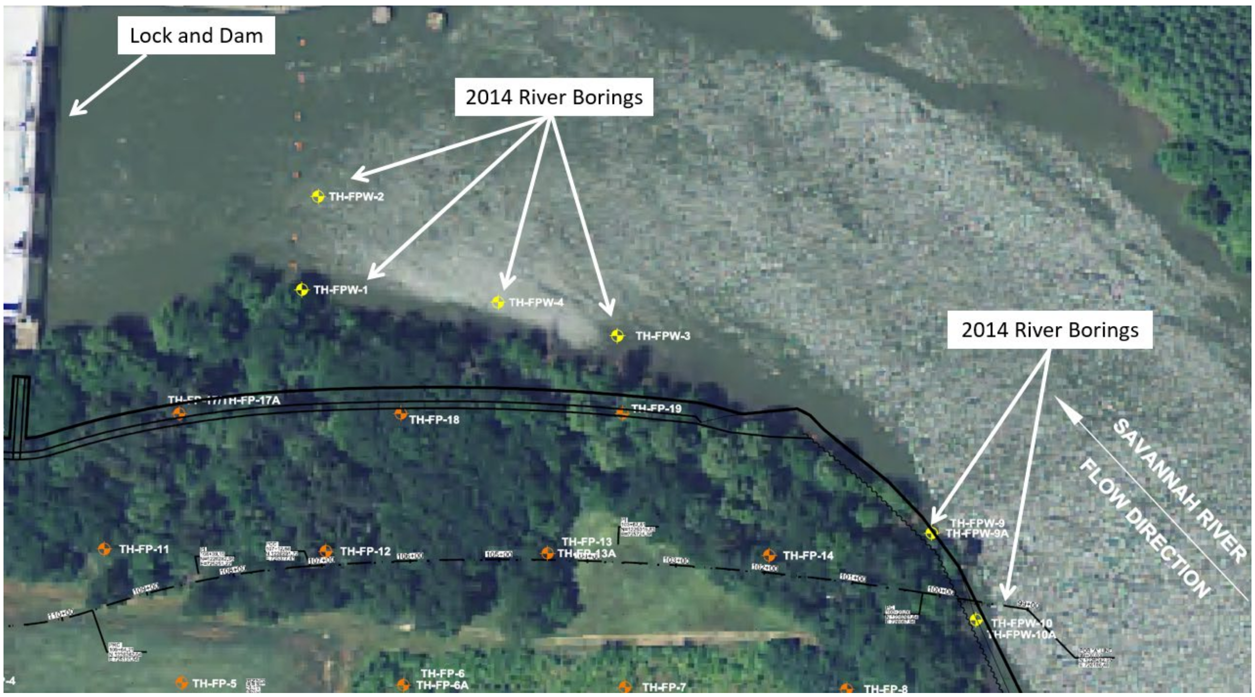

Figure 38 - Plan View of 2014 Fish Passage Upstream Marine Borings

All borings indicate mostly sands with varying amounts of fines from poorly graded clean sand to silty and clayey sand (SP to SM and SC). Blow counts are consistently low (average N-value of ~4) indicating the density of the sands are very loose to loose. All upstream borings except TH-FPW-10 encountered denser sands with low amount but varying fines at a depth of approximately 20 feet below the river bed surface. It is assumed that TH-FPW-10 was terminated just before encountering this denser layer. All the borings terminated within this layer except for TH-FPW-1. Within TH-FPW-1 the layer was classified as poorly graded clean sand (SP) approximately 10 feet thick with one SPT interval indicating an N-value of 13. The boring then terminated in a less dense layer of poorly graded sand with silt (SP-SM) with a lowered blow count of 6. Two exceptions to this stratigraphy occurred in TH-FPW-1 and TH-FPW-9 that both encountered a layer of lean clay (CL) about 7 feet thick. Both layers were soft to very soft with blow counts varying from weight of hammer to 4 blows per foot. In TH-FPW-1 the lean clay was at a depth of 7 feet below the river bed within the predominantly sand foundation. IN TH-FPW-9 the clay layer was at the river bed surface.

The loose and soft foundation soils within the foundation for the proposed rock weirs poses concerns for potential bearing capacity failures and excessive consolidation. Either issue could render the tentatively selected plan unfeasible or excessively expensive as significant amounts of rock or other fill material could be required to reach desired elevations as the foundation soils continue to settle. In order to address these potential problems, 6 SPT borings are proposed to a depth of 75 feet for the borings beneath the largest upstream rock weir and 50 feet for all other borings. These depths should be sufficient to gather data on soils to a depth where approximately 70% of the stress will have dissipated from the applied loading of the rock weirs based on preliminary weir dimensions. SPT will be performed in all borings. Where significant cohesive material is encountered, the barge will be moved to a close by location to allow for sampling of the material with Shelby tubes. Within sandy materials SPT N-values will be correlated to strength and consolidation parameters. The samples collected during the SPT will also provide an opportunity to collect a sample for index testing. Undisturbed samples in significantly thick cohesive materials (greater than 1 foot) will be collected for triaxial testing for strength and consolidation testing to estimate settlement. Finally, representative samples of the foundation sands from several borings will be collected for permeability testing which will be applicable to cofferdam design described in the next section. The boring location plan for all proposed additional explorations is shown in Figure 39.

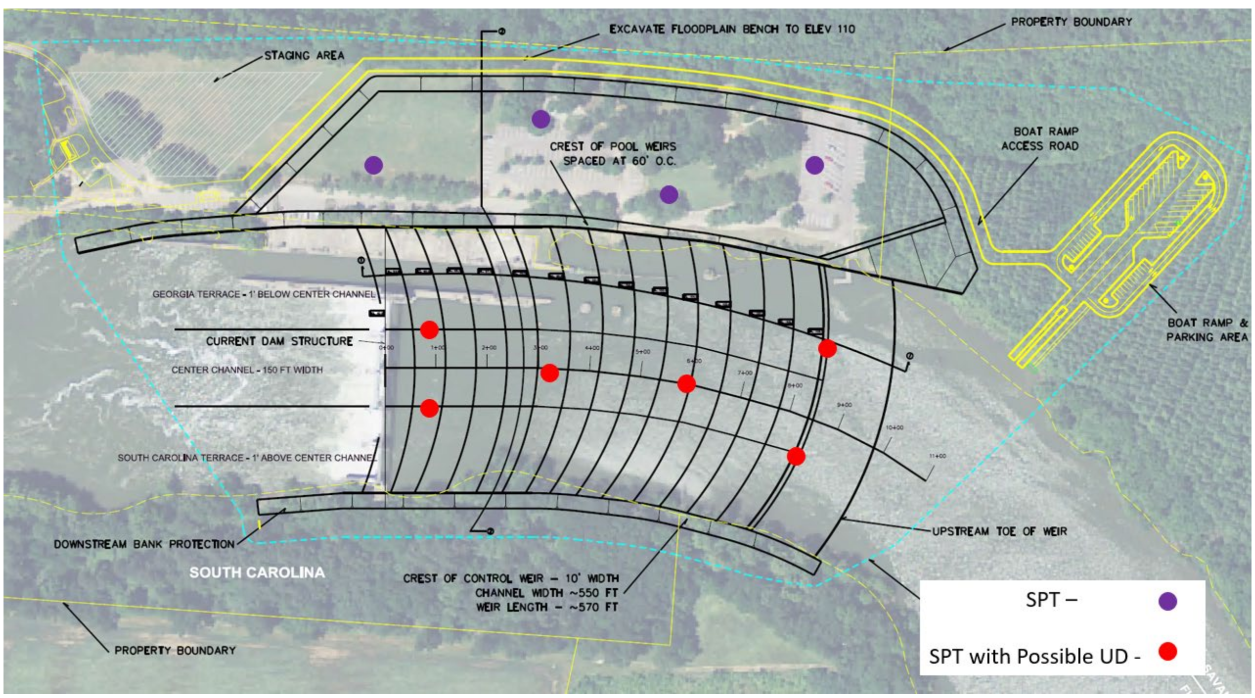

Figure 39 - Plan View of Proposed Additional Explorations for Fish Passage Design

Geotechnical Design and Construction Concerns

The current plan involves constructing the rock weirs in the dry utilizing at least two phases of cofferdams. Sheet piling will be driven into the foundation soils on one side of the river allowing flow to continue relatively unchanged. Unfortunately, there does not appear to be a consistent impermeable barrier at a reasonable depth to found the sheet piles into. Permeability testing described in the previous section will help with designing the length of embedment required to control gradients within the construction area and inform the design of dewatering systems (if necessary). As mentioned previously, soft soils at or near the riverbed surface are a concern. One of the advantages of construction in the dry is the ability to improve loose soils through drying and compaction and/or excavation and replacement with better material. It will also allow for conditioning of the surface soils to make them more suitable to receive the bedding stone currently intended to be placed against the foundation soils. It is not considered feasible at this time to improve foundation soils at depth. If bearing capacity becomes an issue the weir dimensions will likely have to be modified to reduce stresses within the foundation. Consolidation testing will help inform the estimate for required fill and rock accounting for inevitable settlement. The current plan involves embedding permanent sheet piles within the most upstream rock weir to form the actual damming surface. The sheet piles are not anticipated to settle or move by any other mechanism such that if the rockfill on either side of the sheet piles settles excessively the results will not cause a loss of pool upstream of the project. Additional stone could be added if necessary. A summary of the required preliminary analyses is provided in Table 15.

Table 15 – Summary of Required Preliminary Analysis.

Analysis Description	**Design Criteria**
Steady State Slope Stability of Natural Cut Slopes on Right Abutment of Flood Plain Bench	Factor of Safety >= 1.5
Rapid Drawdown Slope Stability of Natural Cut Slopes on Right Abutment of Flood Plain Bench	Factor of Safety >= 1.1
Filter Compatibility between Natural Cut Slopes on Right Abutment of Flood Plain Bench and Geotextile for Particle Retention	Current filter criteria.
Standard/Modified Proctor for Soils Excavated from Right Abutment Flood Plain Bench	No criteria is applicable. Results will inform final estimates for soil and rockfill in the riverbed area.
Filter Compatibility between Soils Excavated from Right Abutment Flood Plain Bench and Surficial River Sediments	Foster and Fell “No Erosion”.

Bearing Capacity of Foundation Soils beneath Rock Weirs	Factor of Safety >= 2.0
Settlement of Foundation Soils beneath Rock Weirs	No criteria is applicable. Results will inform final geometry of weirs and estimates for soil and rockfill in the riverbed area.
Heave Calculation (Critical Gradient) for Sheet Pile Embedment Protecting Temporary Cofferdam Construction	Factor of Safety >=2.0

5. Environmental Engineering

Environmental protection measures will be incorporated into all proposed rehabilitation.

5.1. Environmental Benefits

The environmental benefits for fish passage which are included in all evaluated alternatives are inherent in the design. The structure allows for the upstream migration of a variety of anadromous fish species to include both the shortnose and Atlantic Sturgeon. The structure also allows for fish to return downstream once they have spawned in their historic spawning grounds further upriver at the Augusta shoals. The removal of the dam downstream of the in-line rock weir structure provides an unimpeded river allowing all river flow to pass freely through this location. The dam and sill structure will be removed in a way that allows for fish passage without obstruction to the rock weir located further upstream. Additional discussion regarding environmental considerations can be found in the main report.

6. Civil Design

6.1. Site Selection

The site of the fish passage structure was selected in accordance with the requirements of the 2016 WIIN Act. Numerous project sites were considered, including locations upstream and downstream of the existing dam location. However, hydraulic analysis found that the existing dam location provides the greatest opportunity to maintain the existing pool elevation without adversely impacting flooding in the overbank areas. This is because the location of NSBLD is at a natural breakpoint in the terrain of the region, which provides for the impoundment of a reservoir pool with a relatively small structure across the river. A fish passage structure downstream of the current dam location that would maintain the pool would be significantly larger than that in the proposed alternatives. Conversely, a structure upstream of the existing dam location would result in an increase in inundation depths in inhabited areas during high flow event, due to the lack of a wide floodplain. The proposed alternatives evaluated in the 2D HEC-RAS analysis present variations in the fish passage structure alignment and required appurtenances to meet project objectives.

6.2. Real Estate

Real estate requirements for construction of the project are limited, to the extent possible, to land that is currently owned by the federal government. Several of the project alternatives considered were developed based on the maximum project footprint that would fit within federally owned land surrounding New Savannah Bluff Lock and Dam. However, it is likely that the project ultimately constructed will require additional lands for access roads and the project footprint. These lands have been identified for each alternative and are presented in the drawings in Attachment 1. A detailed discussion of real estate requirements for this project are presented in the Real Estate Appendix.

6.3. Relocations

Significant utility relocations are not anticipated for this project. However, on-site power and sewer lines for parking lots and bath houses may need to be relocated. Additional lands are not required for these relocations, though the exact alignment of new utility lines is dependent on the alternative that is selected for construction.

7. Structural Requirements

An inspection of New Savannah Bluff Lock and Dam was completed in the fall of 2016 and a list of recommend repairs developed. These repairs are the minimum required to ensure the dam structure would be able to maintain the reservoir pool over the life of a proposed fish passage structure; the list of required repairs does not fully rebuild or rehabilitate the structure.

Alternative 1-1 is the only alternative currently being considered that would leave the dam in place and require repair/rehabilitation of the structure, all other alternatives include removal of the dam. If Alternative 1-1 is selected as the recommended plan for construction, the dam will need to be repaired in accordance with the requirements outlined in the 2016 inspection report. Additional repairs may be required beyond those listed in the 2016 report as portions of the dam were inaccessible for inspection. It is anticipated during construction of the repair work that new issues or areas of concern will be uncovered as not all areas were inspected due to being underwater or not accessible by divers due to turbidity and strong current.

<u>Dam Spillway Piers:</u> There is considerable pier cracking. Spillway Piers Nos. 1 through 4 have large open cracks near the water line, at approximately El. 100, that extend to approximately El. 120. It is assumed that the cracks occur below the water down to the base of the Spillway Pier at El. 90. The South Carolina abutment has exhibited similar cracking and was previously repaired. The cracks are up to ¾ inch in width at the surface and pinch down to tight cracks within 3-4 inches (estimated) into the concrete. Any crack repair would be applied to Spillway Piers Nos. 1 through 4. The crack repair would extend from about approximately El. 126 down to base of the piers at El. 90.5 at the low point. Most of the perimeter of the piers are cracked and need repair with the

exception of near and around the gate slots, which are reinforced.

The recommended repair consists of tying the piers together with through steel anchors that would prevent the concrete from coming apart and insure that the pier remains stable. There has been some consideration given to injecting a flexible epoxy resin into the surface cracks to lessen the water intrusion that feeds the alkali-silica reaction (ASR). This is still under study.

<u>Catwalk:</u> Replace all steel plate bearings of the existing catwalk with ¾" elastomeric bearing pads. The paint on the existing steel frames of the catwalk is peeling. After confirming that the paint is devoid of the lead, scrap the framing and repaint the framing.

<u>Sand Blast and Repaint all Embedded Metal and Armor:</u> Lock embedded metal and armor is badly corroded and needs repainting. The embedded metal and armor for the lock and dam apparently has never been repainted and is badly corroded. Repainting would give additional life to these components and would provide a much better appearance.

<u>Retrofitting Top of Spillway Piers to Receive New Gate Hoists</u>: Spillway gate hoists replacements are proposed for gates 1, 4 and 5. Retrofitting tops of spillway piers to receive new gate hoists will be required. This will require some concrete demolition and installation of new concrete and structural steel foundations at the top of spillway piers to support the new gate hoists.

Additional Considerations:

1. The grouting beneath the river wall and lock floor has potential to be a very expensive operation because of: 1) Not knowing the exact amount of voids beneath river wall and lock floor. 2) Past experience of pumping grout below water and into unknown voids has produced large overruns from the estimated volumes because of escaping grout and not being able to confine the grout.

2. Additional exploration should be considered prior to final plans and specifications to better define the amount of voids beneath the river wall and lock floor. This could consist of using a diver to make small openings in the sheet pile wall on the riverside of the river wall and probe beneath the river wall to check for voids. Also a small camera could possibly be inserted to look at timber piles. The openings in the sheet piling would then be closed. Some sample drilling through toes of river wall should be considered.

3. Because of the expense of bringing in great amount of stone backfill, a hydrographic survey of river would be very helpful for making decisions on whether to bring stone by barge or truck.

4. The lock control building has a significant amount of cracking.

5. The cantilevered steel walkway at the roof near the top of the stair is badly

corroded and needs to be repaired. It is suggested that this repair be considered prior to completing final design.

6. During the site visit, it was noticed that the culverts for the water passage in both the land side & water side walls of the lock walls have continuously running water. From the velocity of the water it can be inferred that all 4 butterfly valves & 4 bulkheads need replacement. This continuous unchecked water is aggravating the existing erosion underneath the lock walls.

The estimated cost for these repairs is detailed in the Cost Engineering Appendix. Detailed discussion of the inspection and required structural repairs are presented in the 2016 Inspection Report (Savannah District, 2017).

8. **Electrical and Mechanical Requirements**

The 2016 inspection of NSBLD also identified required repairs to electrical and mechanical systems. These repairs would need to be carried out for Alternative 1-1; the cost of these repairs are detailed in the Cost Engineering Appendix.

8.1. Electrical Repairs for Lock and Dam

<u>Lightning Protection and Grounding</u>: The control building has experienced numerous lightning strikes causing damage to the power and communications equipment. Due to these numerous strikes, additional lightning protection for equipment and personnel safety shall be provided. One solution that has previously been offered was to install the communication and power cables under the lock in lieu of over the lock, as they presently exist. However, due to the specific NFPA 780 requirement for lightning protection systems described below and in order to maintain a “downward” path to ground which can be accomplished with an overhead ground wire, it was decided to retain an overhead grounding conductor across the lock. As detailed below, the existing overhead conductor shall remain in place and a new grounding conductor compatible with the new lightning protection system shall also be added. The existing overhead communications and power cables shall remain overhead since the removal and reinstallation of these cables under the lock is considered to serve only as an aesthetic benefit.

- Install new lightning protection system on Control Building roof consisting of properly spaced air terminal interconnected by roof conductors.

- Provide ground connection to all existing ungrounded metallic structures, devices, and components mounted on the Control Building roof and across the spillway structure.

- Replace existing grounding conductor across spillway and install second lightning protection grounding conductor across spillway catwalk on opposite side of catwalk from existing conductor and make connection to South Carolina side grounding grid.

- Install new grounding grid: Install a new grounding conductor compatible with the new lightning protection system across lock chamber. The lightning protection design shall be in accordance with NFPA 780, Lightning Protection Systems, and, specifically, paragraph 3-9 of NFPA 780 shall be addressed in the design (i.e., “Main conductors shall interconnect all strike termination devices and shall form two or more paths from each strike termination device downward, horizontally, or rising at no more than ¼ pitch to connections with ground terminals, except as permitted by 3-9.1 and 3-9.2”). The new overhead grounding conductor shall be connected to the existing control building tower and land side riser pole so as to maintain a “downward” path away from the control building tower to the new grounding grid installed on the Georgia side of the lock.

<u>Generator Replacement</u>: The 50KW electric generator currently located on the second floor of the operations building will likely need to be replaced as replacement parts are no longer available; the existing generator was manufactured in 1939. A replacement generator of equivalent capacity that has been identified for the project will not fit in the space where the current generator is located. A new building for the generator and supporting equipment has been identified for alternative 1-1. Additional discussion regarding the generator requirements can be found in the 2016 inspection report.

Spillway Electrical Work:

- Spillway Gate Hoists: Remove the existing electric gate hoists for gates 1, 4, and 5 and install new hydraulic gate hoists as per gates 2 and 3. This shall include the installation of limit switches, cable tension sensors, and laser sensors for hoist travel indication on the gates as part of the new hydraulic gate hoist installation package.

- The remote control capability of the gates shall be transferred from J. Strom Thurmond to others. Communications link from dam to Strom Thurmond is a leased telephone line. Transfer of remote control capabilities shall include transfer and setup of computer hardware from Strom Thurmond to others (coordination of transfer of equipment ownership is required). One-line for communication cabling and proposed transfer hardware, including new hydraulic gate hoists. All necessary revisions in programming in order to accommodate the revised control scheme of gates 1, 4, and 5 shall be accomplished.

- Install heaters and thermostats in all spillway gate hydraulic control panels, both new and existing (control panels for gates 2 and 3 do not currently contain heaters and thermostats).

- The control systems for spillway gates 1, 4, and 5 shall be revised, modified, and improved as per gates 2 and 3 and as shown on the drawings. This shall include hardware expansion (additional modules, terminal blocks, etc.) of the existing gate control panel and terminal cabinet located on the first floor of the lock control building.

- Provide separate circuits to each gate control cabinet heater and to each gate control cabinet.

- Remove spillway lighting fixtures from spillway piers that are to be rehabilitated (top of concrete gate piers will be reworked as per gates 2 & 3; this work consists of concrete replacement for new hoisting equipment) prior to start of rehabilitation work on pier. Reinstall lighting fixtures after rehab work is complete.

- Additionally, portions of 16370A Electrical Distribution System, Aerial and 16375A Electrical Distribution System, Underground may be utilized as necessary in order to create one Electrical Work Specification section. The control system work will have to be a created specification section utilizing portions of other specification sections as applicable.

8.2. Mechanical Repairs

Remove and install new gate hoist machinery on gates 1, 4 and 5, as per gates 2 and 3. Install secondary containment on existing and new hydraulic cylinders. Top of concrete gate piers will have to be reworked as per gates 1 and 2. Install hold open struts on gate hoist access covers on existing hoist machines on gate numbers 2 and 3.

Detailed discussion of the inspection and required mechanical repairs are presented in the 2016 Inspection Report (Savannah District, 2017).

9. Hazardous and Toxic Materials.

A Hazardous Building Material survey of New Savannah Bluff Lock and Dam was conducted in April 2017. Material found during the survey include lead joints, lead-based paint, motor oil, and hydraulic oil. Disturbance of lead-based paint during construction/demolition must adhere to OSHA worker protection rules and other application state and federal regulations. A detailed discussion of the survey and its results can be found in the *Hazardous Building Materials Survey of New Savannah Bluff Lock and Dam* dated April 2017.

An asbestos inspection and survey of NSBLD was also conducted in April 2017. Several samples of building materials were identified during the survey that contain or are assumed to contain asbestos, including roofing materials, flange gaskets, and exterior caulking. A detailed discussion of the survey and its results can be found in the *Asbestos Survey of New Savannah Bluff Lock and Dam* dated April 2017.

In addition to these surveys discussed above, a Phase I HTRW investigation will need to be completed during PED for this project.

10. Flood Emergency Plans

New Savannah Bluff Lock and Dam is currently classified as a low hazard dam due to the low storage volume of the reservoir and mostly uninhabited area downstream of the dam; there is no significant risk to loss of life in the case of a dam failure. Construction of the fish passage structure would not increase the storage volume of the reservoir and no increased risk to loss of life is anticipated. The existing emergency action plan for New Savannah Bluff Lock and Dam should be suitable for the with-project condition, with slight modification to the required emergency procedures regarding operation of the existing dam gates.

11.4. For navigation channels, the estimated effect of required overdepth on the frequency and cost of maintenance dredging will be discussed.

11. Access Roads

Access to the fish passage structure from the Georgia side of the river is possible through the use of the existing roadway network. The Butler Creek Bridge provides access to the NSBLD park and project site, and preliminary analysis indicates that this bridge is suitable for all vehicles that do not require a weight permit for other roads in Georgia. It is not anticipated that the bridge would require major upgrades to accommodate vehicle loading during construction. A detailed report of this analysis is currently being developed by the Kansas City District.

Alternatives 2-6d will require the construction of 0.5 miles of new access road as the project footprint for these alternatives requires the demolition of the existing road network within the park.

Temporary access roads for project construction and permanent access roads required for on-going project maintenance have been identified in the conceptual designs for the with-project condition, available in Attachment 1 to this appendix.

12. Cost Estimates

A summary of the Project Costs for each alternative in the final array are presented in Table 12 below. For more detailed cost figures and discussion regarding development of project costs see the Cost Appendix.

Table 16 - Annualized Cost Summary

	Description	Alternatives Summary - 100 yr project life, 3.5% interest rate used to calculate annualized costs							
		SHEP Plan A	**Alt 1-1_2% Slope**	**Alt 2-3_2% Slope**	**Alt 2-6a_2% Slope**	**Alt 2-6b_2% Slope**	**Alt 2-6c_2% Slope**	**Alt 2-6d_2% Slope**	**Alt 2-8_2% Slope**
Initial Cost	04 Dams	$0	$0	$3,834,417	$3,834,417	$3,834,417	$3,834,417	$4,023,316	$4,802,926
	05 Locks	$29,907,405	$38,929,704	$6,890,306	$6,890,306	$6,890,306	$6,890,306	$6,937,586	$8,784,290
	06 Fish & Wildlife Facilities	$32,045,376	$30,673,831	$58,892,630	$91,838,168	$73,536,969	$75,599,723	$71,300,408	$98,229,248
	13 Pumping Plant	$0	$0	$1,581,447	$0	$1,581,447	$312,541	$442,767	$0
	15 Floodway Control and Diversion Structures	$0	$0	$0	$0	$0	$0	$0	$38,682,202
	18 Cultural Resources	$429,336	$709,182	$676,488	$644,274	$644,274	$644,274	$677,173	$665,652
	Construction Estimate Totals	***$62,382,116***	***$70,312,718***	***$71,875,287***	***$103,207,165***	***$86,487,413***	***$87,281,261***	***$83,381,250***	***$151,164,318***
	01 Land and Damages	$307,140	$31,875	$3,598,208	$4,727,819	$138,107	$138,107	$140,178	$0
	30 Planning, Engineering & Design	$2,809,403	$3,483,163	$3,554,546	$5,102,506	$4,274,921	$4,315,622	$4,124,359	$7,262,539
	31 Construction Management	$2,712,264	$3,797,448	$3,592,455	$5,160,471	$4,324,379	$4,364,127	$4,503,538	$7,354,337
	Project Cost Totals	***$68,210,923***	***$77,625,203***	***$82,620,497***	***$118,197,962***	***$95,224,820***	***$96,099,117***	***$92,149,324***	***$165,781,194***
	IDC	$2,711,800	$3,544,000	$3,622,200	$5,201,100	$4,358,400	$4,398,500	$4,202,200	$4,358,400
	Investment Cost	**$70,922,723**	**$81,169,203**	**$86,242,697**	**$123,399,062**	**$99,583,220**	**$100,497,617**	**$96,351,524**	**$170,139,594**
Annualized Cost	Investment Cost	$2,565,000	$2,935,000	$3,118,000	$4,462,000	$3,601,000	$3,634,000	$3,484,000	$6,152,000
	Adaptive Monitoring Costs	$0	$0	$0	$0	$0	$0	$0	$0
	Monitoring Costs	$0	$0	$0	$0	$0	$0	$0	$0
	O&M	$720,000	$710,000	$35,000	$45,000	$45,000	$45,000	$45,000	$320,000
	Major Rehab	$285,000	$285,000	$0	$0	$0	$0	$0	$249,000
	Total Annual Cost	**$3,570,000**	**$3,930,000**	**$3,153,000**	**$4,507,000**	**$3,646,000**	**$3,679,000**	**$3,529,000**	**$6,721,000**

13. Plates, Figures, and Drawings

Concept drawings of the focused array of alternatives and additional figures can be found in Attachment 1. Inundation maps for alternatives 2-6a can be found in attachment 2. The remaining alternatives are not anticipated to induce any additional flooding impacts beyond the existing conditions and therefore no inundation maps were produced for those alternatives.

14. References

CDM-Smith, *New Savannah Bluff Lock and Dam Fish Passage Mitigation Analysis for Impacted Water Users.* July 2018.

Chow, Ven Te. *Open channel hydraulics*. McGraw-Hill Book Company, Inc; New York, 1959.

Jung, Younghun, et al. "An approach using a 1D hydraulic model, Landsat imaging and generalized likelihood uncertainty estimation for an approximation of flood discharge." *Water* 5.4 (2013): 1598-1621.

Jones, Timothy. *Asbestos Survey of New Savannah Bluff Lock and Dam, Augusta, Georgia.* Savannah District, Environmental and Materials Unit. USACE. 2017

Jones, Timothy. Hazardous Building Materials Survey of New Savannah Bluff Lock and Dam, Augusta, Georgia. Savannah District, Environmental and Materials Unit. USACE. 2017

Savannah District, USACE. New Savanna Bluff Lock & Dam Repairs Necessary for SHEP Fish Passage. February 2017.

http://www.habitat.noaa.gov/pdf/Final_Federal_Interagency_Technical_Memorandum_Fish_Passage_Guidelines.pdf

Attachment 1
Concept Design Drawings

Attachment 2
Inundation Maps

Attachment 3
Sedimentation Analysis

Attachment 4
Fixed Weir Pool Simulation – After Action Review