Eagle Creek Flood Basin – Final Design Report

Hancock County, Ohio

August 8, 2023

Prepared for:

Maumee Watershed Conservancy District
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Executive Summary

Hancock County and the City of Findlay, Ohio experience frequent and significant flooding from the Blanchard River and its major tributaries, often flooding agricultural land and the City's streets, homes, and businesses within the floodplain. As a result, the Hancock County Flood Risk Reduction (HCFRR) Program was developed, and Stantec Consulting Services Inc. (Stantec) was contracted by the Maumee Watershed Conservancy District (MWCD) to develop potential alternative solutions to reduce the risk of flooding. The Eagle Creek Flood Basin (ECFB) project is a component of the HCFRR Program.

The ECFB is a flood risk reduction project designed to have storage capacity for excess flows from the 1% Annual Chance Exceedance (ACE) (100-year) storm event on Eagle Creek and to safely pass flood events of greater magnitude. The purpose of the project is to reduce the peak flow rate in Eagle Creek and the Blanchard River during large storm events, thereby reducing the downstream water surface elevations (WSE) and associated flood risk. The primary project goals include the following:

- Reduce WSEs during flooding events along Eagle Creek and the Blanchard River; and
- Reduce flood risk to mitigate potential structural, social, and environmental damages.

A secondary project goal is to improve water quality by creating wetlands, native habitats, and riparian corridors within the inundation area. A tertiary goal is to provide passive recreation opportunities on the project site.

This document is the Final Design Report (FDR) for the ECFB project located in Eagle Township, Hancock County, Ohio. The project footprint is generally bounded by County Road 45 to the south, US 68 to the east, Township Road 76 to the west, and Township Road 80 to the north

The primary flood risk reduction element for the project is an earthen embankment dam forming an in-line, dry-storage basin. Other primary components consist of a principal spillway, an auxiliary spillway, exterior drainage features, and interior drainage improvements and land use design. Figure E1 shows the dam alignment and primary project components. Table E1 provides a design summary of the primary project components.

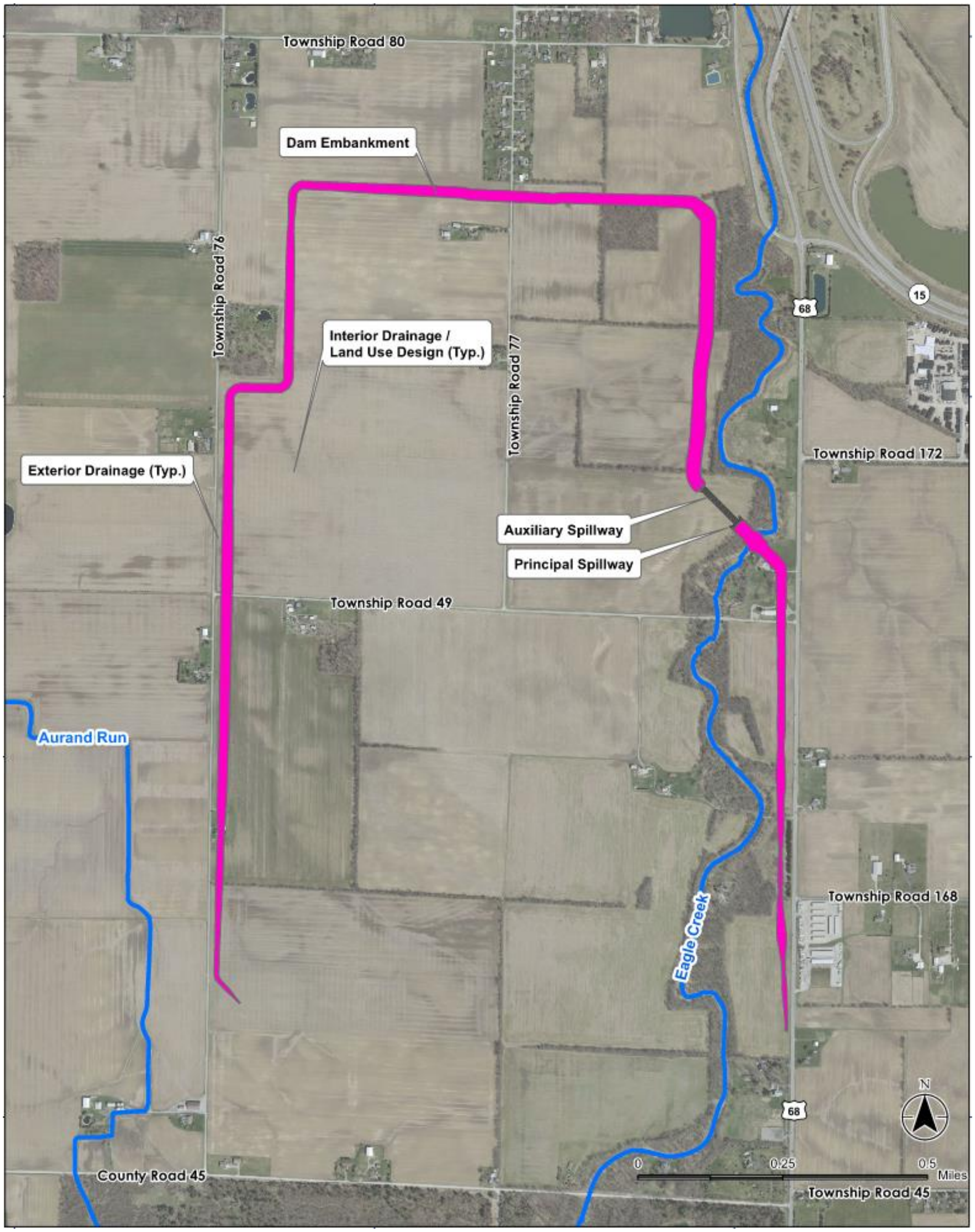

Figure E1. Project Components

Table E1. Eagle Creek Flood Basin Design Components Summary

Inline Dam	
Earthen Embankment	
Crest Elevation	Varies between 812.0 and 813.0 ft
Crest Length	19,533 ft
Top Width	Varies between 12.0 ft and 14.0 ft
Side Slopes	3H:1V
Dry-Storage Reservoir	
Storage Capacity at 100-year Event (WSE at 807.0 ft)	6,945 ac-ft
Storage Pool Area (Elevation 807.0 ft)	910 acres
Storage Capacity at Probable Maximum Flood Event (WSE at 810.0 ft)	9,839 ac-ft
Integrated Spillway Structure	
Principal Spillway: Control Wall with Orifices and Baffled Chute	
Control Wall Orifice Openings	2 @ 9.0 ft x 3.0 ft
Control Wall Gate Settings	2 @ 9.0 ft x 2.4167 ft
Principal Spillway Width	22.0 ft
Baffled Chute Spillway Length	80.0 ft (70.0 ft of baffles)
Baffle Height	Varies between 0.9 ft and 1.2 ft
Debris Rack	22.0 ft W x 13.0 ft H x 29.0 ft L
100-year Event Discharge Capacity (WSE at 807.0 ft)	1,264 cfs
Probable Maximum Flood Discharge Capacity (WSE at 810.0 ft)	862 cfs
Auxiliary Spillway: Integrated Labyrinth Weir	
Spillway Height	13.0 ft
Crest Elevation	807.0 ft
Crest Length (Effective Crest Length)	437 ft (1,672 ft)
Probable Maximum Flood Discharge Capacity (WSE at 810.0 ft)	27,450 cfs
Energy Dissipator: Horizontal Apron (Natural Jump Basin)	
Stilling Basin Length	20.0 ft
Exterior Drainage	
Southwest Trapezoidal Ditch Total Length	3,823 ft
Northwest Trapezoidal Ditch Total Length	3,867 ft
North Trapezoidal Ditch Total Length	4,228 ft
East Trapezoidal Ditch Total Length	4,536 ft
East Stormwater 24” Conduit Total Length	3,714 ft
Interior	
Borrow Area #1 Footprint	63 acres
Borrow Area #2 Footprint	45 acres
Borrow Area #3 Footprint	20 acres

Project Benefits – Model results show that the Eagle Creek Flood Basin project results in a peak flow reduction of about 2,550 cfs (16% decrease) on the Blanchard River during the 1% ACE event which translates to about 2.2 feet of lowering of the base flood elevations near the confluence with Eagle Creek.

The reduction in WSEs along Eagle Creek and the Blanchard River is estimated to remove approximately 1,290 parcels and 1,590 acres from the regulatory floodplain.

Opinion of Probable Cost (OPCC) – An opinion of probable construction costs was developed for the ECFB based on Final Design. Table E2 summarizes the OPCC for the ECFB project. The line items in Table E2 assume a 14% contractor markup and do not factor in potential escalation or construction contingencies.

Table E2. Opinion of Probable Construction Cost Summary

Item #	Description	**100% Final Design
A	General Works, Demolition, and Site Preparation	$2,792,000
B	Dam Embankment Earthwork	$8,902,000
C	Seepage Mitigation	$2,519,000
D	Instrumentation	$127,000
E	Road Modifications and Site Drainage	$2,187,000
F	Stream, Wetlands, Fish, and Wildlife	$2,779,000
G1	Spillways and Outlet Structures	$8,167,000
G2	Mechanical Gates	$200,000
G3	Electrical	$199,000
G4	Permanent Erosion Control	$546,000
H	Interior Features	$383,000
I	Contractor Indirect Costs	$8,293,000
J	Allowances	$-
K	Contractor Markups	$5,282,000
	Total Construction Price	$42,376,000
	Final Design Class 2 Estimate Cost Range	
	-10%	$38,138,000
	10%	$46,614,000

Abbreviations

ACE	Annual Chance Exceedance
APE	Area of Potential Effects
ASTM	American Society for Testing and Materials
AWA	Applied Weather Associates
BCA	Benefit-Cost Analysis
CLOMR	Conditional Letter of Map Revision
CWA	Clean Water Act
DEM	Digital Elevation Model
DTM	Digital Terrain Model
ECFB	Eagle Creek Flood Basin
ESA	Endangered Species Act
FDR	Final Design Report
FEMA	Federal Emergency Management Agency
GEDR	Geotechnical Exploration Data Report
GPS	Global Positioning System
HCFRRP	Hancock County Flood Risk Reduction Program
HEC-HMS	Hydrologic Engineering Center Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Center River Analysis System
HMR52	Hydrometeorological Report No. 52
IMU	Inertial Measurement Unit
LiDAR	Light Detection and Ranging
MSG	The Mannik & Smith Group, Inc.
MWCD	Maumee Watershed Conservancy District
NAD	North American Datum
NAVD88	North American Vertical Datum of 1988
NFIP	National Flood Insurance Program
NHPA	National Historic Preservation Act
NLCD	National Land Cover Data
NOAA	National Oceanic and Atmospheric Administration
NRCS	National Resources Conservation Service
NRHP	National Register of Historic Places

OAC	Ohio Administrative Code
ODNR	Ohio Department of Natural Resources
ODOT	Ohio Department of Transportation
OEPA	Ohio Environmental Protection Agency
OPCC	Opinion of Probable Construction Cost
ORC	Ohio Revised Code
OSHPO	Ohio State Historic Preservation Office
PFDS	Precipitation Frequency Data Server
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
ppsm	Points Per Square Meter
SCS	Soil Conservation Service
SSURGO	Soil Survey Geographic
UH	Unit Hydrograph
USACE	United States Army Corps of Engineers
USBR	United States Bureau of Reclamation
USEPA	United States Environmental Protection Agency
USFWS	United States Fish and Wildlife Service
USGS	United States Geological Survey
WQC	Water Quality Certification
WRDA	Water Resources Development Act
WSE	Water Surface Elevation

1.0 INTRODUCTION

This document is the Final Design Report (FDR) for the Eagle Creek Flood Basin (ECFB) project. The ECFB is designed to provide storage for excess flows during flood events to reduce the peak flow rates in Eagle Creek and thereby the Blanchard River. As a result, downstream water surface elevations and associated flood risk will be reduced. The Flood Basin is anticipated to have storage capacity for excess flows associated with the 1% (100-year) Annual Chance Exceedance (ACE) storm event on Eagle Creek and the capacity to safely pass flood events of greater magnitude.

1.1 PROJECT CLIENT

The Maumee Watershed Conservancy District (MWCD) is a legal subdivision of the State of Ohio created under Section 6101 of the Ohio Revised Code (ORC). The MWCD territory includes 15 counties in northwest Ohio including: Allen, Auglaize, Defiance, Fulton, Hancock, Hardin, Henry, Lucas, Mercer, Paulding, Putnam, Shelby, Van Wert, Williams and Wood. Typical MWCD projects consist of flood risk reduction and drainage improvement studies.

1.2 BACKGROUND

1.2.1 History of Flooding

Hancock County and the City of Findlay, Ohio experience frequent and significant overbank flooding from the Blanchard River and its major tributaries, Eagle Creek and Lye Creek. The Blanchard River and its tributaries can convey small, frequent storms; however, during large rainfall events, flow exceeds channel capacity and overbank flooding occurs in agricultural areas and through the City of Findlay. Historical evidence shows substantial damage during large storms, such as the 4% (25-year) ACE event.

Per the National Weather Service's Advanced Hydrologic Prediction Service, "major flood stage" on the Blanchard River near Findlay occurs when the United States Geological Survey (USGS) gage 04189000 at County Road 140 is at 14.5 feet or greater (updated from 13.5 feet in March 2021). The gage data at this site indicates the Blanchard River has reached or exceeded the former major flood stage 23 times from 1913 to 2022, and of these events, 11 have occurred since 2007. Six events between 2007 and 2017 are among the top 11 stages on record, with four of those events peaking at more than 3 feet over former major flood stage of 13.5 feet. The August 2007 event reached a peak stage near the maximum recorded peak of 18.5 feet in 1913.

The repetitive flooding prompted the Western Lake Erie Study authorization under the Water Resources Development Act of 1999 (WRDA 99). The Hancock County Commissioners and the City of Findlay requested assistance from the U.S. Army Corps of Engineers Buffalo District (USACE) to study and recommend ways to reduce significant flood damages adjacent to the Blanchard River and its tributaries. The County and City began working with the USACE in 2007 to develop a flood risk reduction plan that could be eligible for Federal funding. At that time, the MWCD was requested by the County and City to consider administration of a project or projects recommended by the USACE. The MWCD agreed to consider administration of a project once a formal plan of improvement was presented.

1.2.2 Past Reports and Studies

1.2.2.1 USACE Recommended Plan

In 2008, the USACE began a feasibility study which addressed Flood Risk Management in the Blanchard River Watershed. The feasibility study ultimately resulted in the USACE proposing a 9.2-mile flood diversion channel outside Findlay to the south and west of the city (Western Diversion of Eagle Creek). The diversion channel was proposed to convey flow (4% ACE, 25-year event) from Eagle Creek and discharge into the Blanchard River west of Township Road 130. The project advanced through the planning stages resulting in a Draft Environmental Impact Statement, *Western Lake Erie Basin (WLEB) Blanchard River Watershed Study, Section 441 of the Water Resource Development Act of 1999, General Investigations, Draft Detailed Project Report / Environmental Impact Statement,* dated April, 2015 (USACE 2015) and an unpublished Draft Interim Report, *Feasibility Study / Final Environmental Impact Statement* dated March 2016 (USACE 2016) for the diversion project.

The USACE's final recommended plan, the Western Diversion of Eagle Creek, was presented to the community in 2015, but was deemed unlikely to meet Federal funding requirements because of its inadequate cost benefit ratio and low community support. In early 2016, the County and City requested the assistance of MWCD to review the USACE recommendation and determine if there were other viable mitigation projects. The County and MWCD authorized a Memorandum of Agreement under which MWCD agreed to administer the project review.

1.2.2.2 Proof of Concept (2017) & Proof of Concept Update (2018)

In the Summer of 2016, Stantec Consulting Services Inc. (Stantec) reviewed the USACE Plan's effectiveness (Proof of Concept), considered potential modifications to improve it, and studied other implementable solutions. Stantec ultimately recommended that the MWCD implement a suite of flood-risk reduction projects as an alternative to the USACE's diversion channel that better met the needs of the community.

The proposed set of projects is referred to as the Hancock County Flood Risk Reduction (HCFRR) Program. The recommended HCFRR Program includes multiple flood risk reduction strategies and efforts as documented within the Stantec report titled, *Final Report: Data Review, Gap Analysis, USACE Plan and Alternatives Review, and Program Recommendation,* or *Proof of Concept Report,* dated April 3, 2017 (Stantec 2017) and the follow-up report, *Draft Proof of Concept Update* dated July 9, 2018 (Stantec 2018). Both documents can be found at the Program website: www.HancockCountyFlooding.com.

The HCFRR Program included several independent projects such as hydraulic improvements along the Blanchard River in the City of Findlay, a dry-storage basin on Eagle Creek upstream of the City, and two dry-storage basins near the Village of Mt. Blanchard on the Blanchard River and Potato Run.

The active projects in HCFRR Program include various hydraulic improvements along the Blanchard River. Phase 1 of the Hydraulic Improvements includes the removal of four (4) inline dam/riffle structures and excavation of a floodplain bench on the Blanchard River near Swale Park (between Broad Avenue and the Norfolk-Southern rail bridge). Phase 1 is substantially complete with final completion anticipated in 2024. Phase 2 of the Hydraulic Improvements, currently under design, would include reconstruction of the Norfolk Southern railroad bridge over the Blanchard River with three spans. Phase 2 is also expected to be completed in 2024.

The Eagle Creek dry-storage flood basin concept that was originally developed in the 2017 Stantec study was further refined with the 2018 Proof of Concept Update. Refinements to the concept were made in response to feedback received from the community and project stakeholders in order to reduce potential impacts to residential structures and private property.

1.2.2.3 Eagle Creek Conceptual Design (2019)

Following the Proof-of-Concept Update (Stantec 2018), Stantec advanced the conceptual design of the Eagle Creek dry-storage basin. As part of this study, Stantec collected supplementary field data and performed technical analyses to evaluate additional footprints related to the project. The additional field work and evaluation of alternative footprints is described in the technical memorandum titled, *Eagle Creek Dry-Storage Basin Project Alternatives Review*, dated October 31, 2019 (Stantec 2019). The study results indicated that:

- Each of the footprint options considered would provide a level of flood risk reduction for the downstream communities along Eagle Creek, Lye Creek, and the Blanchard River.
- The degree of water surface elevation (WSE) reduction is dependent on the rate of discharge released from the Eagle Creek basin.
- Construction costs vary significantly based on the selected embankment alignment and downstream discharge criteria with the following trends noted:
    - Construction costs are generally lower for alternatives that utilize a larger reservoir area.
    - Alternatives that incorporate excavation are more expensive than similar options without excavation and the added flood benefits (decreased downstream discharge) derived from storage excavation do not likely warrant the additional costs.

Stantec recommended the following based on the studies and analyses performed for the Eagle Creek flood basin through October 2019:

- Construct the eastern embankment of the dam to the east of Eagle Creek. This is the most cost-efficient option and is anticipated to present fewer dam safety design and operations concerns.
- Implement the largest reservoir footprint feasible.
- Design the principal spillway and downstream flood discharge protection for the acquired project footprint without excavation for additional storage.
- Evaluate the benefits of the Aurand Run secondary spillway in comparison to the achieved principal spillway discharge and other potential flood protection measures downstream.

1.2.2.4 Eagle Creek Preliminary Design Report (2022)

Elements of the Preliminary Design phase included field data collection, hydrologic and hydraulic modeling, environmental and regulatory agency permitting coordination, engineering analyses, and design. Stantec advanced 94 borings during the Preliminary Design Phase to obtain geotechnical data for the proposed flood basin. Hydrologic, hydraulic, structural, and geotechnical analyses were completed to support design of the proposed earthen embankment dam and spillway structures.

The Eagle Creek Flood Basin Preliminary Design Report and associated appendices was submitted to the Ohio Department of Natural Resources (ODNR) Division of Water Resources Dam Safety Group in March of 2022. Division staff completed a review of the Preliminary Design Report and associated appendices. On May 31, 2022, the Chief of ODNR's Division of Water Resources determined the proposed dam meets the criteria for placement in Class I Hazard Classification per OAC Section 1501:21-13-01. Pursuant to OAC Rule 1501:21-5-02, the Preliminary Design Report was approved by the Division of Water Resources Chief.

1.3 PROJECT LOCATION

The ECFB project site (40°58'45.3" N, 83°39'36.9" W) is located in Eagle Township in Hancock County, Ohio, approximately four miles south of the City of Findlay downtown area. Figure 1 shows the location of the project in relation to the City of Findlay and nearby waterways. The project footprint is generally bounded by County Road 45 to the south, US 68 to the east, Township Road 76 to the west, and Township Road 80 to the north. The relief within the project area from the high point of existing ground at County Road 45 (approximately 835 feet) to the invert of the Eagle Creek channel (approximately 785 feet) is 50 feet with an average elevation of 797 feet in the interior of the dam.

1.4 PROJECT PURPOSE

The ECFB is a flood risk reduction project intended to reduce flood elevations in the City of Findlay and Hancock County, Ohio. The purpose of this project is to reduce the peak flow rate in Eagle Creek and the Blanchard River during large storm events, thereby reducing the downstream water surface elevations (WSEs) and associated flood risk. The primary project goals include the following:

- Reduce WSEs during flooding events along Eagle Creek and the Blanchard River; and
- Reduce flood risk to mitigate potential structural, social, and environmental damages.

A secondary project goal is to improve water quality by creating wetlands, native habitats, and riparian corridors within the inundation area. A tertiary goal is to provide passive recreation opportunities on the project site.

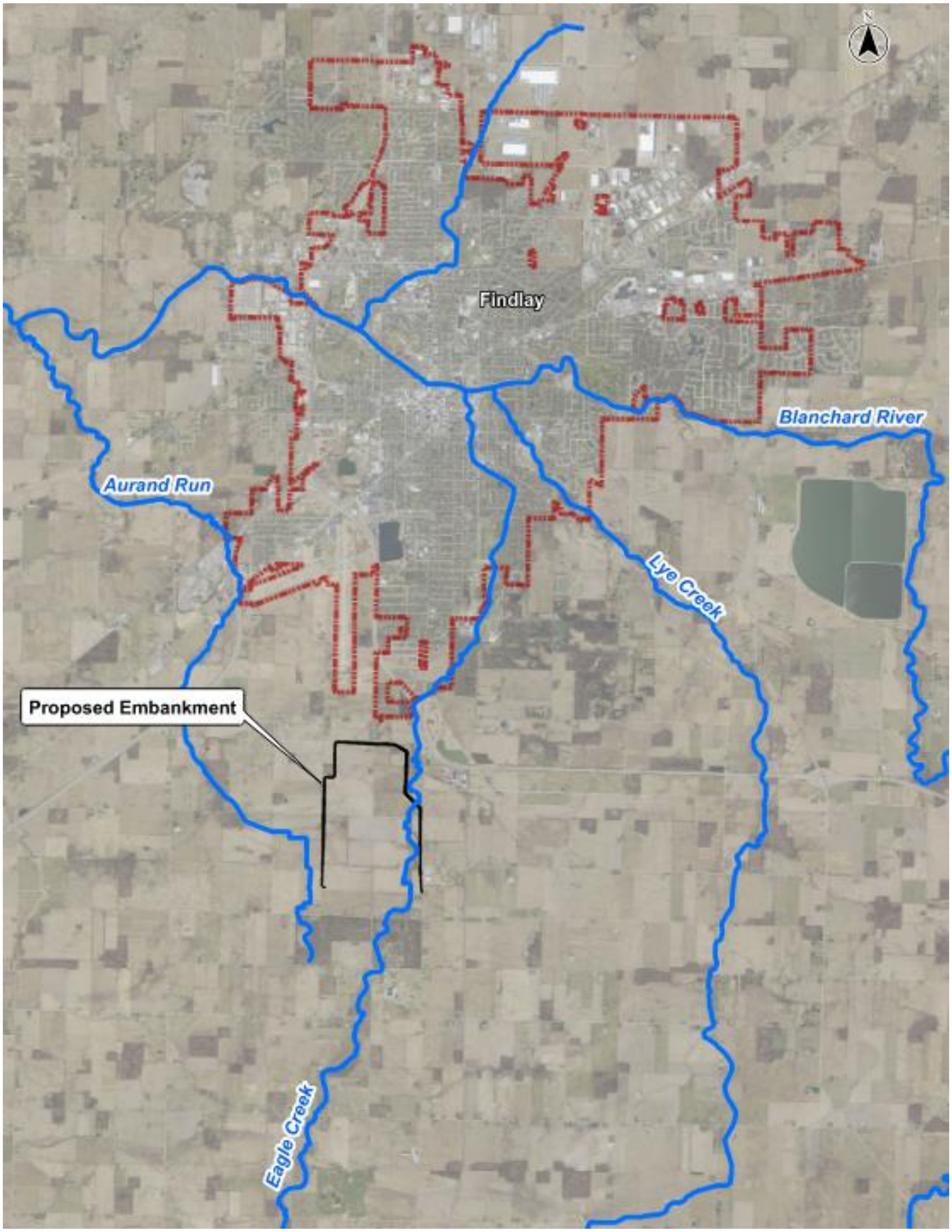

Figure 1. Project Location

1.5 FINAL DESIGN REPORT

Elements of the Final Design phase include supplementary field data collection, hydrologic and hydraulic modeling, environmental and regulatory agency permitting coordination, engineering analyses, and design. This Final Design Report summarizes the following:

- Site Description, including Existing Conditions at the Project Site,
- Field Data Collection and Application of Information,
- Project Components and a summary of their design,
  - Hydrologic, Hydraulic, Geotechnical, Structural, and Geomorphic Analyses and Civil Design
- Summary of Project Impacts and Benefits,
- Construction Considerations,
- Permitting Overview, and
- Opinion of Probable Construction Cost (OPCC).

This report documents the basis of design for the project components and includes a set of drawings and specifications associated with the design.

The Final Design Report follows requirements established under the Ohio Administrative Code (OAC) Rule 1501:21-5-04 and includes the following:

- A report of the field and laboratory investigations of the foundation soils and/or the bedrocks, and the materials that will comprise the dam or levee. Stability and settlement analyses, and seepage and underseepage studies
  - **[Appendix E - Geotechnical Design Report]**.
- The basis, references, calculations, and conclusions relative to hydrologic, hydraulic studies
  - **[Appendix D - Hydrologic and Hydraulic Analysis Report]**
- The basis, references, calculations, and conclusions relative to structural design studies and to the design of spillways and outlet works
  - **[Appendix I - Principal Spillway Technical Memorandum]**
  - **[Appendix J - Auxiliary Spillway Technical Memorandum]**
- A map that shows the locations of borings, test pits, proposed borrow areas, known farm tiles, utility lines, and other areas pertinent to the design and construction of the structure.
  - **[Appendix E - Geotechnical Design Report]**.
- Detailed cost estimates of the Construction of the structure and its appurtenances **[Section 14.0 - Opinion of Probable Construction Costs].**
- Any other studies, investigations, and pertinent design information.
  - **[Appendix A - Design Criteria Document]**

    - **[Appendix G - Dam Embankment Design Technical Memorandum]**
    - **[Appendix H - Exterior Drainage Analysis Report]**
- First filling monitoring plan
    - (Included with the Operation, Maintenance, and Inspection (OM&I) Manual)
- Water Control Plan
    - **[Section 11.5, Appendix B - Design Drawings and Appendix C - Technical Specifications].**
- Steps to minimize erosion during construction
    - **[Appendix B - Design Drawings and Appendix C - Technical Specifications].**

1.6 DESIGN CRITERIA

While the MWCD has not set specific design parameters for the ECFB project, the HCFRR Program consistently has focused on reducing WSEs and associated flood risk during a 1% (100-year) ACE flood event. With this goal in mind, the ECFB is designed to reduce 1% ACE WSE by providing storage without activation of the Auxiliary Spillway.

Project design criteria have been developed to inform the ECFB Project engineering and design effort. The project design criteria are developed based on guidance from the MWCD, agencies such as the Ohio Department of Natural Resources (ODNR) and the US Army Corps of Engineers (USACE), hydraulic modeling results, and other analyses. The engineering and design of the project generally complies with industry standards and guidelines for:

- Dam Hazard Classification Design Requirements,
- Hydraulic Design,
- Geotechnical Design,
- Structural Design,
- Civil Design,
- Transportation Design, and
- Environmental Design.

These design criteria are described in more detail within the project's Design Criteria Document included as Appendix A.

2.0 PROJECT DESCRIPTION

2.1 SITE DESCRIPTION

Existing data related to the project site was compiled and several field investigations and surveys were completed to inform the permitting and design. This section provides an overview of the project site, including results of the field studies that describe existing site conditions for the project area.

2.1.1 Land-Use

Habitats and land uses identified within the Project study area include wetlands (palustrine emergent, palustrine scrub-shrub, and palustrine forested), mixed early successional/second growth deciduous forest, mixed early successional/second growth riparian forest, fallow field, agricultural row crop field, old field, new field, pasture, industrial land, and residential lawn.

The existing land use at the project site is primarily fallow agricultural land. Historically, drain tile has been installed in many areas within the project site to promote drainage. There are seven former residential properties within the interior portion of the proposed basin. Structures related to these properties have either been demolished or are planned to be demolished prior to the start of construction. Additionally, wooded areas exist near the former residential structures and along the Eagle Creek corridor.

2.1.2 Transportation Features

Township Road 49 runs east and west near the center of the project site for the width of the footprint. There is an existing bridge on Township Road 49 crossing Eagle Creek with USGS Gage 04188496 (Eagle Creek above Findlay OH) located at the bridge.

Township Road 77 extends north from Township Road 49 through the northern/central portion of the project footprint. Both roads are two-lane roads (one lane in each direction).

On the exterior of the project footprint, Township Road 76 runs north and south along the western side of the proposed dam, while US-68 runs north and south along the eastern side of the site. County Road 45 is aligned east and west at the southern extents of the project area. Each road has two lanes (one lane in each direction). US-68 is maintained by the Ohio Department of Transportation (ODOT).

The US-68 and State Route 15 Interchange is located just northeast of the project site.

2.1.3 Waterways

The project is located primarily within the Eagle Creek watershed, a tributary to the Blanchard River (8-Digit Hydrologic Unit Code [HUC] 04100008). The Blanchard River flows south to north for the reach located south of the City of Findlay, then flows east to west through the City. Eagle Creek is a tributary to the Blanchard River that flows south to north along the eastern portion of the project site. The downstream end of Eagle Creek at the Blanchard River is in the central portion of the City of Findlay.

The headwaters of the Aurand Run watershed are located southwest of the project site, with the western portion of the project's footprint being situated within the existing Aurand Run watershed. Aurand Run flows south to northwest, west of the proposed dam, flowing into the Blanchard River downstream of the City of Findlay.

2.1.4 Land Ownership

The MWCD has purchased approximately 825 acres of land within the project footprint through July 2023. Figure 2 shows the parcels purchased by the MWCD (yellow polygons with hatching). The remaining parcels required within the project footprint are either identified as "In Discussion" with the property owner (pink polygon) or may be partially impacted and require a flowage and/or construction easement (blue polygon).

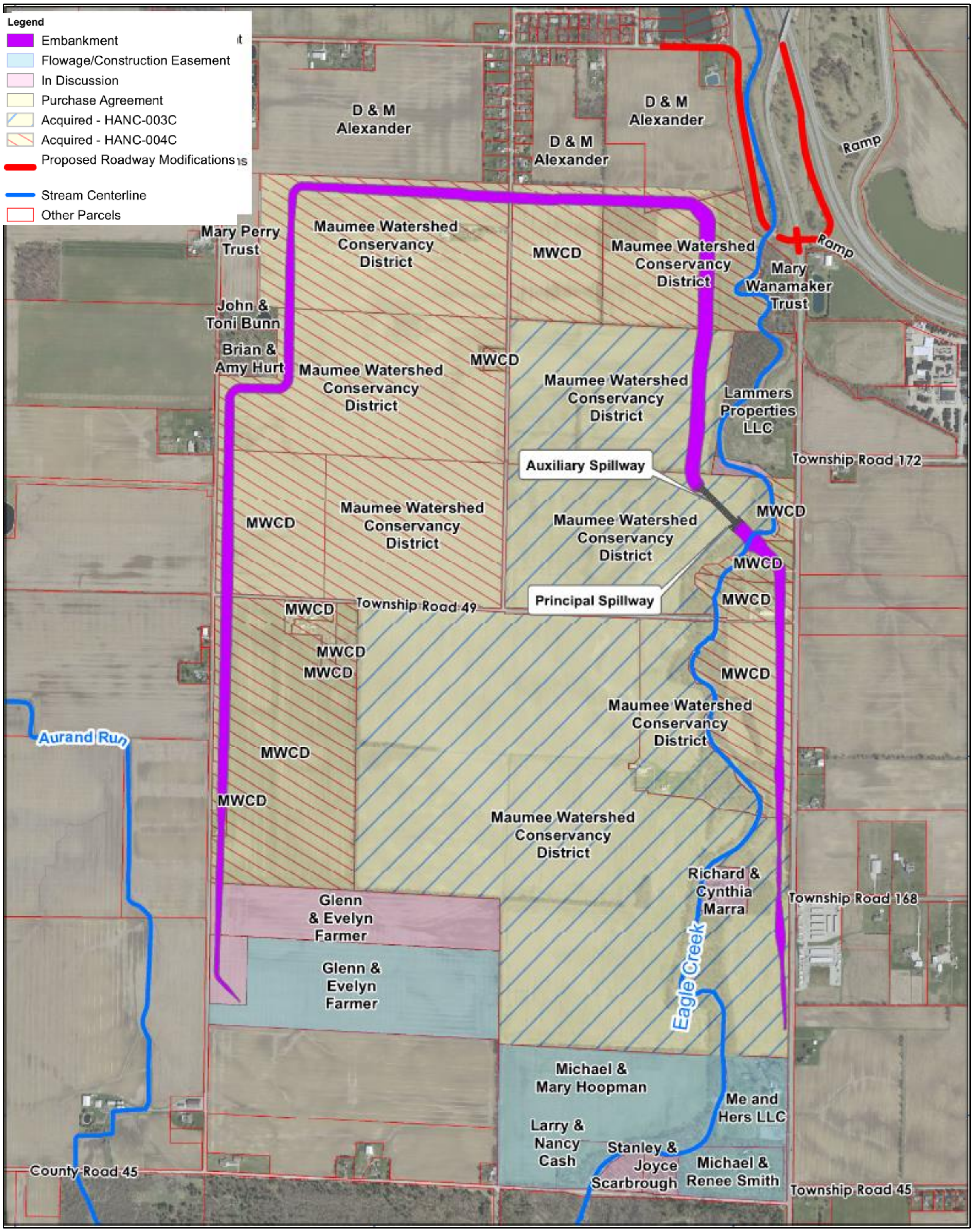

Figure 2. Land Ownership

2.1.5 Existing Utilities

Existing utilities in the project area include: stormwater structures, conduits, and drainage tiles; domestic water wells and domestic septic systems; buried communication, water, and gas lines; and overhead utilities within the impoundment.

2.1.6 Upstream Structure Elevations

A survey of residential and other structures upstream of the project footprint was performed by Bockrath & Associates Engineering and Surveying LLC from September 9 through September 17 of 2021. A total of 38 structures at 12 locations were surveyed to help inform design decisions and report out results of project impacts.

2.1.7 Wetlands and Waterbodies

Stantec conducted a wetland and waterbody delineation survey within the project area as part of the environmental permitting process. (Stantec, 2021a) Stantec performed the wetland and waterbody delineation field surveys on July 25 and 26, August 13 through 16, and September 6, 2019, as well as a supplemental survey on November 9, 2021.

2.1.7.1 Wetlands

The wetland and waterbody delineation field surveys identified 47 wetlands, primarily along the Eagle Creek corridor. The delineated wetlands totaled 9.7 acres and were identified as follows:

- Seventeen (17) Palustrine Forested (PFO),
- One (1) Palustrine Scrub-Shrub (PSS),
- One (1) Palustrine Unconsolidated Bottom (PUB),
- Twenty-Seven (27) Palustrine Emergent (PEM), and
- One (1) mixed PSS/PEM wetland.

The Ohio Rapid Assessment Method (ORAM) for Wetlands scores ranged from 12 to 64.5. There were nine (9) ORAM Category 1, thirty-seven (37) Category 2, and one (1) Category 3 wetlands identified. Four of the wetlands were documented as potentially isolated.

2.1.7.2 Open Waters

One (1) feature was delineated as open waters within the Study Area (Camp Berry property) totaling 3.95 acres.

2.1.7.3 Streams

The wetland and waterbody delineation identified 34 streams totaling 53,699 feet in length. Nineteen (19) of the streams were identified as intermittent (7,156 feet), eleven (11) streams were ephemeral (2,107 feet), and four (4) streams were perennial (44,437 feet). The Stream Evaluation Scores ranged from 21 to 69.

2.1.8 Threatened and Endangered Species

State and federally listed species in Ohio are protected under the Endangered Species Act (ESA) and regulated by the ODNR and the U.S. Fish and Wildlife Service (USFWS) respectively. As part of the Clean Water Act (CWA) Section 404 permitting process, the project is required to comply with Section 7 of the ESA.

Stantec evaluated terrestrial habitats to determine the presence or absence of potentially suitable habitat within the Project area for federally listed and state listed threatened or endangered species as described by the U.S. Fish and Wildlife Service (USFWS) and/or ODNR. Threatened and endangered species habitat assessment field surveys conducted by Stantec within the Project area on July 25 and 26, August 13 through 16, and September 6, 2019. (Stantec, 2019a)

The federally listed threatened and endangered species occurring, or potentially occurring, in Hancock County include the Indiana bat (*Myotis sodalis*; federally endangered), northern long-eared bat (*Myotis septentrionalis*; federally threatened), clubshell (*Pleurobema clava*; federally endangered), rayed bean (*Villosa fabalis*; federally endangered), and bald eagle (*Haliaeetus leucocephalus*; federal species of concern).

The ODNR Division of Wildlife (ODNR 2016) lists the following as state listed species as occurring in, or having the potential to occur within Hancock County: blue-spotted salamander (*Ambyostoma laterale*; state endangered), western banded killifish (*Fundulus diaphanus menona*; state endangered), plains clubtail (*Gomphus externus*; state endangered), purple lilliput (*Toxolasma lividus*; state endangered), black sandshell (*Ligumia recta*; state threatened), pondhorn (*Uniomerus tetralasmus*; state threatened), and Kirtland's snake (*Clonophis kirtlandii*; state threatened).

Stantec documented the presence of potentially suitable habitat for the rayed bean, purple lilliput, and pondhorn mussels within Eagle Creek and/or Aurand Run, within the Project area. However, according to the USFWS and ODNR response letters, no known occurrences of federal or state-listed mussel species occur within the Project area or a 1-mile radius of the Project area. No potentially suitable habitat within the Project Area and no occurrence of the species within a one-mile radius of the Project Area were found for the blue-spotted salamander, western banded killifish, and black sandshell.

2.1.8.1 Bald Eagle Survey

A bald eagle nest survey was completed by Stantec on December 13, 2021. Per USFWS guidance, the survey was completed to validate the location of a known bald eagle nest record located northeast of the

Project and to determine if any additional eagle nests are present within the Project area or its surrounding buffers. Bald eagle nest surveys took place within a 660-foot buffer and 1,000-foot buffer of the Project area, including within the Project's limits of disturbance. No bald eagle nests were identified within the Project area or the 660-foot or 1,000-foot Project area buffers as a result of those surveys. One known bald eagle nest was identified outside of the survey area to the northeast and its location was recorded. The nest is located 1,433 feet from the Project area boundary. Due to the project footprint's distance from the located nest, Stantec received concurrence from the USFWS that the Project is not likely to adversely affect bald eagles or bald eagle nests.

2.1.8.2 Mussel Reconnaissance

Stantec conducted a mussel reconnaissance survey for the ECFB project area and documented the findings in the Freshwater Mussel Reconnaissance Survey on Eagle Creek and Aurand Run. (Stantec, 2019b) The primary objective of the study was to assess the potential presence or probable absence of unionid mussels within the area of direct impact (ADI) for proposed project feature (bridge/culvert/spillway/weir replacements).

Previous reconnaissance surveys were conducted in November 2016 on Eagle Creek. A more recent survey of twelve locations along Eagle Creek, Aurand Run, and an unnamed ditch was conducted on July 25 and October 23, 2019. Each location was surveyed beginning approximately 400 feet downstream of the ADI and ending 200 feet upstream. A total of seven species were observed and no federally or Ohio listed species were found. No mussels (live or shell) were observed at four survey locations, likely due to lack of perennial water and fine sediment dominance. Clear evidence of mussel assemblages was found at two downstream survey locations on Aurand Run and at two upstream survey locations on Eagle Creek. Evidence of mussel assemblages in Eagle Creek is further supported by a 2016 reconnaissance survey which found live mussels at two sites. Surveys at three locations were inconclusive due to lack of visibility in turbid water. It is likely that the upstream limit for mussel occupancy is at some point south (upstream) of Township Road 50. Mussels are likely absent at the two sites on the unnamed ditch and the two most upstream Aurand Run sites.

2.1.8.3 Mussel Relocation

Stantec completed a Group 1 mussel presence/absence survey and salvage on Eagle Creek within the project's ADI between June 16 through June 19, 2023 per the Ohio Mussel Survey Protocols. Live species found were identified and processed by a state permitted malacologist and performed under conditions outlined in Stantec's Ohio Wild Animal Permit #20-080.

Coordination occurred between Stantec and ODNR to identify an appropriate recipient site for mussels relocated from Eagle Creek. Per approval from ODNR, Stantec coordinated with Cuyahoga Valley National Park, Cleveland State University, and Ohio State University to arrange transport of live mussels from Eagle Creek to the Cuyahoga River for mussel community augmentation and reestablishment.

As of the date of this document, Stantec staff are preparing a technical report describing habitat conditions at the survey site, river discharge, methods used to complete the survey, level of effort, species present, and mapping of the survey areas. Additionally, a Group 1 report form will be completed with all required data for ODNR prior to December 31, 2023.

2.1.9 Historic Resources

2.1.9.1 Archaeology

The Mannik & Smith Group, Inc. (MSG) was contracted by Stantec to conduct a Phase I Archaeological Survey of the project area in accordance with the provisions of Section 106 of the National Historic Preservation Act (NHPA). Approximately 920 acres were surveyed from April 5 to 28, 20221 and August 6, 2021. Two types of archaeological investigations were conducted within the project area: an archaeological reconnaissance survey and a geoarchaeological field assessment of the Eagle Creek 100-year floodplain.

Subsurface testing, surface collection and intensive visual inspection of the project area resulted in the identification of 68 previously unrecorded archaeological sites and the re-identification of site 33HK799 from a previous survey. The majority of these sites are small lithic/historic artifact scatters or isolated finds typical of short-term occupations and are not likely to yield additional information about Ohio prehistory or history. The Phase I Archaeological Survey Report dated August 2021, presents the methods and findings of the survey (Mannik & Smith, 2021a).

Sites 33HK991 and 33HK992 were recorded as the Byal site cluster during the Phase I Archaeological Survey and represent the remains of a historic farmstead. Additional investigation of this site cluster was recommended to enable formal determinations of eligibility for listing on the National Register of Historic Places (NRHP). Archaeological Monitoring will be performed during construction at these two sites per the Ohio State Historic Preservation Office (SHPO) approved Archaeological Monitoring Plan.

The Phase I geoarchaeological assessment found that there is low potential for deeply buried archaeological sites outside of the 100-year floodplain, but the burial and preservation of soil layers associated with human activity in the past 12,000 years is a higher potential within the 100-year floodplain. Archaeological deep testing was recommended within the Eagle Creek channel banks during construction.

During the Phase I Archaeological Survey, Sites 33HK1008 and 33HK1011-1014 were recorded as the Eagle Creek site Cluster and may represent an Early Archaic habitation location. The Eagle Creek Site Cluster could not be avoided by the Project and MSG undertook Phase II testing at the sites in the fall of 2021. The Phase II evaluation included a magnetometer survey, coring of geophysical anomalies, and excavation of four geophysical anomalies rated as Excellent or Good. A small, plow-truncated pit dating to the Late Prehistoric Period was identified as the result of anomaly excavations. Despite this discovery, the Phase II investigation demonstrated that the examined sites had experienced significant disturbance, did not appear to represent a single Early Archaic camp or habitation site as initially thought, and had not yielded sufficient data to answer research questions. As a result of the Phase II findings, MSG recommended the sites not eligible for listing in the NRHP (Chidester et al. 2022). The SHPO concurred with these findings in a letter dated April 27, 2022.

2.1.9.2 Historical Architecture

MSG completed a Phase I Architectural / Historical Survey encompassing approximately 2,044 acres to identify significant historic / architectural resources over 50 years of age that may be impacted by the proposed project. The fieldwork was conducted from July 8 to July 10, 2021. Through on-site reconnaissance, MSG identified 17 properties in the Area of Potential Effect that were over 50 years of age and retain some elements of physical integrity, in addition to 9 properties that were previously recorded by MSG in 2015. The significance of these resources was evaluated according to their eligibility for listing in the NRHP. It was determined that none of the 26 properties are eligible for listing in the NRHP due to a lack of integrity caused by many years of alterations. The SHPO concurred with the recommendation. The Phase I Architectural / Historical Survey Report dated July 2021, presents the methods and findings of the survey (Mannik & Smith, 2021b).

2.1.10 Stream Assessment and Geomorphic Conditions

A geomorphic assessment of Eagle Creek was conducted to support design of the ECFB. Site-specific geomorphic field data from Eagle Creek, its tributaries, and the surrounding watershed are important for informing design and operational / maintenance considerations as well as obtaining environmental permits by establishing the existing conditions baseline.

Broad level evaluations of channel slope, shape, and pattern based on aerial photography, topographic mapping, and site observations were used to classify the stream based on the Rosgen Stream Classification System (Rosgen, 1996). A longitudinal profile was surveyed from just downstream of the Camp Berry low-head dam for 12,976 feet downstream. Seven cross sections were surveyed along the profile.

Table 1 presents the stream classification characteristics for Eagle Creek within the project footprint. This reach of Eagle Creek classifies as a C4 stream type which exhibits frequent floodplain access, a gravel bed, low sinuosity, and a moderate width-to-depth ratio. The stream valley is characterized as wide and gently sloped with a well-developed floodplain and terraces adjacent to the creek.

Table 1. Stream Classification Characteristics

Classification Parameter	Value
Entrenchment Ratio	> 3
Width to Depth Ratio	14.19
Slope (ft/ft)	0.0013
Sinuosity	1.1
Channel Bed Materials D50 (mm)	17
Stream Type	C4

The bankfull parameters were established based on field data observations as described above. Table 2 summarizes the existing bankfull channel parameters.

Table 2. XS4 Bankfull Parameters

Bankfull Channel Parameter	Value
Width (ft.)	50
Mean Depth (ft.)	3.6
Maximum Depth (ft.)	5.1
Cross Sectional Area (sq. ft.)	181
Wetted Perimeter (ft.)	53
Hydraulic Radius (ft.)	3.4

2.2 PROJECT COMPONENTS

The ECFB project consists of these primary components: earthen embankment dam (inline, dry-storage basin), an integrated principal and auxiliary spillway structure, exterior drainage features, and interior drainage improvements and land use design. Table 3 provides a design summary of the primary project components. Figure 3 shows the proposed dam alignment and primary project components.

Table 3. Eagle Creek Flood Basin Design Components Summary

Inline Dam	
Earthen Embankment	
Crest Elevation	Varies between 812.0 and 813.0 ft
Crest Length	19,533 ft
Top Width	Varies between 12.0 ft and 14.0 ft
Side Slopes	3H:1V
Dry-Storage Reservoir	
Storage Capacity at 100-year Event (WSE at 807.0 ft)	6,945 ac-ft
Storage Pool Area (Elevation 807.0 ft)	910 acres
Storage Capacity at Probable Maximum Flood Event (WSE at 810.0 ft)	9,839 ac-ft
Integrated Spillway Structure	
Principal Spillway: Control Wall with Orifices and Baffled Chute	
Control Wall Orifice Openings	2 @ 9.0 ft x 3.0 ft
Control Wall Gate Settings	2@ 9.0 ft x 2.4167 ft
Principal Spillway Width	22.0 ft
Baffled Chute Spillway Length	80.0 ft (70.0 ft of baffles)
Baffle Height	Varies between 0.9 and 1.2 ft
Trash Rack	22.0 ft W x 13.0 ft H x 29.0 ft L
100-year Event Discharge Capacity (WSE at 807.0 ft)	1,264 cfs
Probable Maximum Flood Discharge Capacity (WSE at 810.0 ft)	864 cfs
Auxiliary Spillway: Integrated Labyrinth Weir	
Spillway Height	13.0 ft
Crest Elevation	807.0 ft
Crest Length (Effective Crest Length)	437 ft (1,672 ft)
Probable Maximum Flood Discharge Capacity (WSE at 810.0 ft)	27,450 cfs
Energy Dissipator: Horizontal Apron (Natural Jump Basin)	
Stilling Basin Length	20.0 ft
Exterior Drainage	
Southwest Trapezoidal Ditch Total Length	3,823 ft
Northwest Trapezoidal Ditch Total Length	3,867 ft
North Trapezoidal Ditch Total Length	4,228 ft
East Trapezoidal Ditch Total Length	4,536 ft
East Stormwater 24” Conduit Total Length	3,714 ft
Interior	
Borrow Area #1 Footprint	63 acres
Borrow Area #2 Footprint	45 acres
Borrow Area #3 Footprint	20 acres

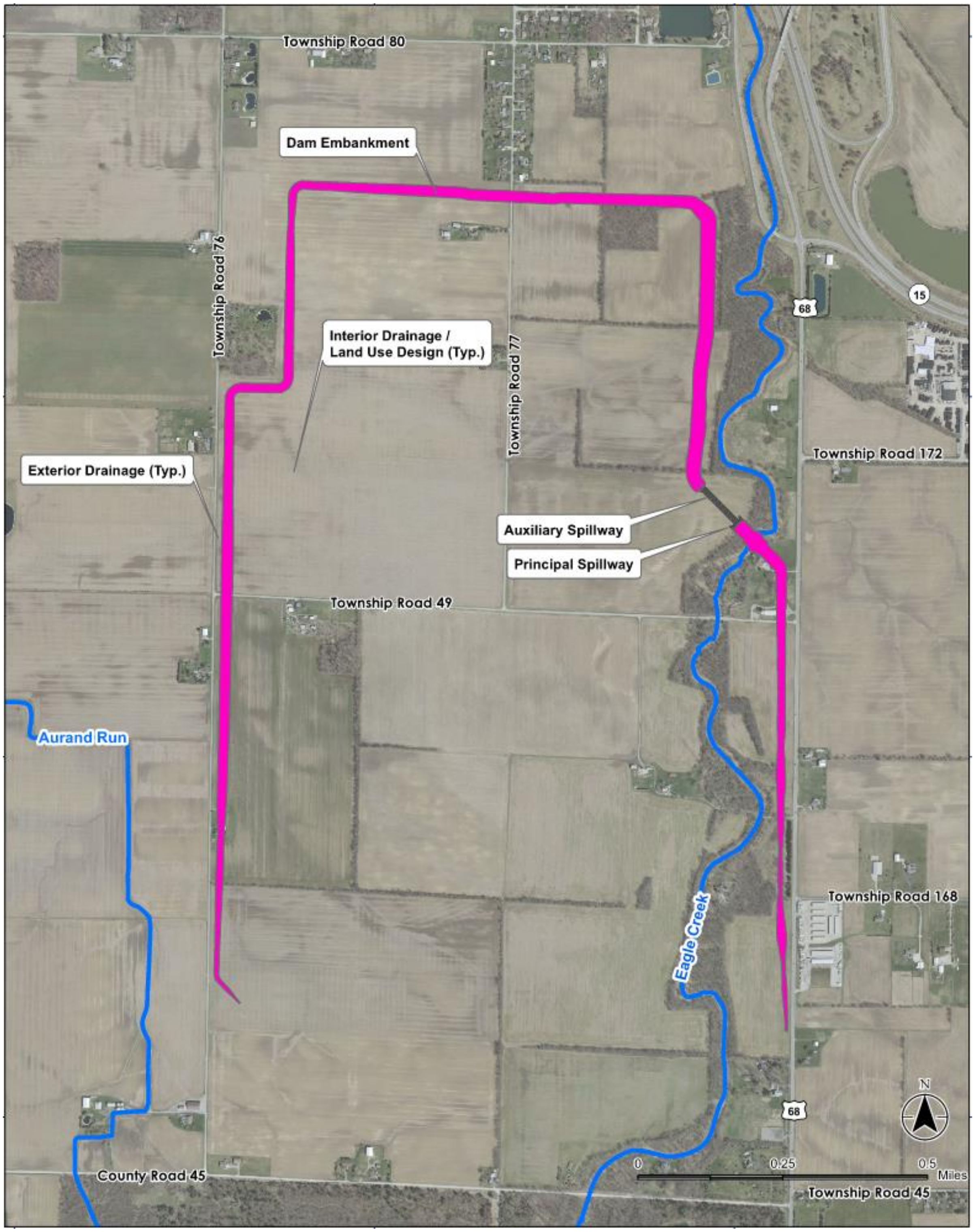

Figure 3. Project Components

2.2.1 Earthen Embankment / Dry Reservoir

The primary flood risk reduction element for the project consists of an earthen embankment that is anticipated to range in height from approximately one foot tall at the upstream tie-in locations up to about 30 feet tall at its intersection with Eagle Creek with a total dam length of approximately 3.70 miles.

The dam alignment and proposed dry basin footprint is bound by Township Road 76 to the west and US-68 to the east. The eastern embankment of the dam is aligned adjacent to the right descending (east) bank of Eagle Creek. To the north, the basin is formed by an embankment parallel to and approximately 4,000 feet to the north of Township Road 49. The impoundment is bound by high ground to the south, approximately 1,750 feet north of County Road 45. The impoundment area of the basin within the dam alignment will be approximately 765 acres.

The basin will remain dry during normal flows (up to approximately 650 cfs) along Eagle Creek and will begin to store flood waters during larger rain events to reduce peak flow rates in Eagle Creek and, ultimately, the Blanchard River downstream.

2.2.2 Principal Spillway

The Principal Spillway consists of a control wall with two rectangular orifices, a baffled chute downstream of the control wall, a debris rack upstream of the control wall, and static gates for maintenance and first filling operations. The Principal Spillway is situated within the realigned Eagle Creek channel that ties into the existing channel at the upstream and downstream ends of the spillway. The Principal Spillway ties into the dam embankment to the south via abutment walls and a slab bridge and is integrated into the Auxiliary Spillway structure to the north.

2.2.3 Auxiliary Spillway

The Auxiliary Spillway is a steel reinforced concrete labyrinth weir with an ogee shaped crest. A labyrinth spillway increases the effective length of the weir within the plan spillway width. A labyrinth weir can pass large discharges at relatively low heads compared to traditional linear weirs of equal width.

The Auxiliary Spillway ties into the embankment to the north via abutment walls and is integrated into the Principal Spillway to the south. Downstream of the labyrinth weir is a natural jump stilling basin comprised of a horizontal riprap apron.

2.2.4 Exterior Drainage

Proposed ditches, conduits, and culverts are designed to convey the runoff along the exterior toe of the dam to a suitable location downstream without impacting the dam embankment or adjacent roadways for specified storm events.

2.2.5 Interior Landuse Design

2.2.5.1 Drainage

The interior drainage features are designed to maintain positive drainage away from the dam embankment and facilitate drawdown of the basin after a filling event by use of grading, swales, and ditches.

2.2.5.2 Wetland Design

A hybrid wetland concept has been designed to utilize the proposed soil borrow areas. Two large wetlands would be situated within the Basin interior making use of the project's borrow pits, and a third wetland would be located along the riparian corridor of Eagle Creek. The wetland / borrow areas are sized to reduce excess excavation in conjunction with the soil material borrow required for the dam embankment.

2.2.6 Secondary Project Components

Secondary project components include stream restoration, bank stabilizations, utility relocations and local road terminations at Township Road 49 and Township Road 77. Construction of the dam embankment will alter existing local transportation routes and affect access to US-68. New transportation alignments that could potentially mitigate the impact to Township Road 49 are outside the scope of this Final Design Report. However, ODOT is currently managing an Interchange Modification design at State Route 15 and US-68. This concept would connect Township Road 80 to US-68 and would act has a replacement for Township Road 49.

2.3 DAM HAZARD CLASSIFICATION

The ODNR, Division of Water Resources requires specific design criteria based on the hazard classification of the dam. Classification of dams is necessary to establish design criteria and adequate safety factors for dams. Per OAC Section 1501:21-13-01 (ODNR 2018), the following parameters are the governing criteria for the classification: height of the dam, storage volume, and potential downstream hazard.

The dam is evaluated on the following criteria and placed in the highest class that any one of these criteria might meet. Table 4 lists the OAC criteria for dam class determination.

Table 4. OAC Section 1501:21-13-01 - Dam Class Determination Criteria

Class	Height of Dam	Storage Volume	Sudden Failure Consequence
I	Greater than 60 feet	Greater than 5,000 acre-feet	• Probable loss of human life • Structural collapse of at least one residence, commercial, or industrial building
II	Greater than 40 feet	Greater than 500 acre-feet	• Disruption of public water supply or wastewater treatment • Flooding of residential, commercial, industrial, or publicly owned structures • Flooding of high-value property • Damage or disruptions to major roads • Damage or disruptions to railroads or public utilities • Damage to downstream class I, II, or III dams or levees
III	Greater than 25 feet	Greater than 50 acre-feet	• Property losses including rural buildings, class IV dams and levees not listed as high-value properties • Damage or disruption to local roads not otherwise listed as major roads
IV	Less Than or equal to 25 feet	Less than or equal to 50 acre-feet	• Property loss limited to dam and rural lands

On May 31, 2022, the Chief of ODNR's Division of Water Resources determined the proposed dam meets the criteria for placement in Class I per OAC Section 1501:21-13-01. Both the storage volume and potential downstream hazard sudden failure consequence criteria place the Eagle Creek Flood Basin dam in the Class I Hazard Classification.

- The storage volume at Elev. 807.0 is 6,945 acre-feet (greater than 5,000 acre-feet – See Section 5.3.2); and
- The potential downstream hazards from a sudden failure include probable loss of human life and structural collapse of at least one residence, commercial, or industrial building.

3.0 HYDROLOGY AND HYDRAULICS

Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) and Hydrologic Engineering Center River Analysis System (HEC-RAS) models for the Upper Blanchard River watershed were leveraged and revised as part of the HCFRR Program Proof of Concept Update (Stantec 2018). These models were used as a starting point to perform analyses of existing and proposed conditions for design of project features.

3.1 HYDROLOGY

This section presents an overview of the hydrologic data and analyses used for the design of the Project. More detail and discussion are provided in the Hydrologic and Hydraulic Analysis Report included as Appendix D.

3.1.1 Watershed Characterization

The hydrologic study area for the ECFB is comprised of approximately 55.0 square miles of the Eagle Creek watershed, and approximately 0.7 square miles of the Aurand Run watershed, upstream of the proposed embankment. The spatial location of the ECFB in relation to the Eagle Creek watershed is presented in Figure 4. Except for a portion of the City of Findlay north of the project area and downstream of the dam embankment, the watershed is sparsely developed and the primary landuse is agricultural row crops.

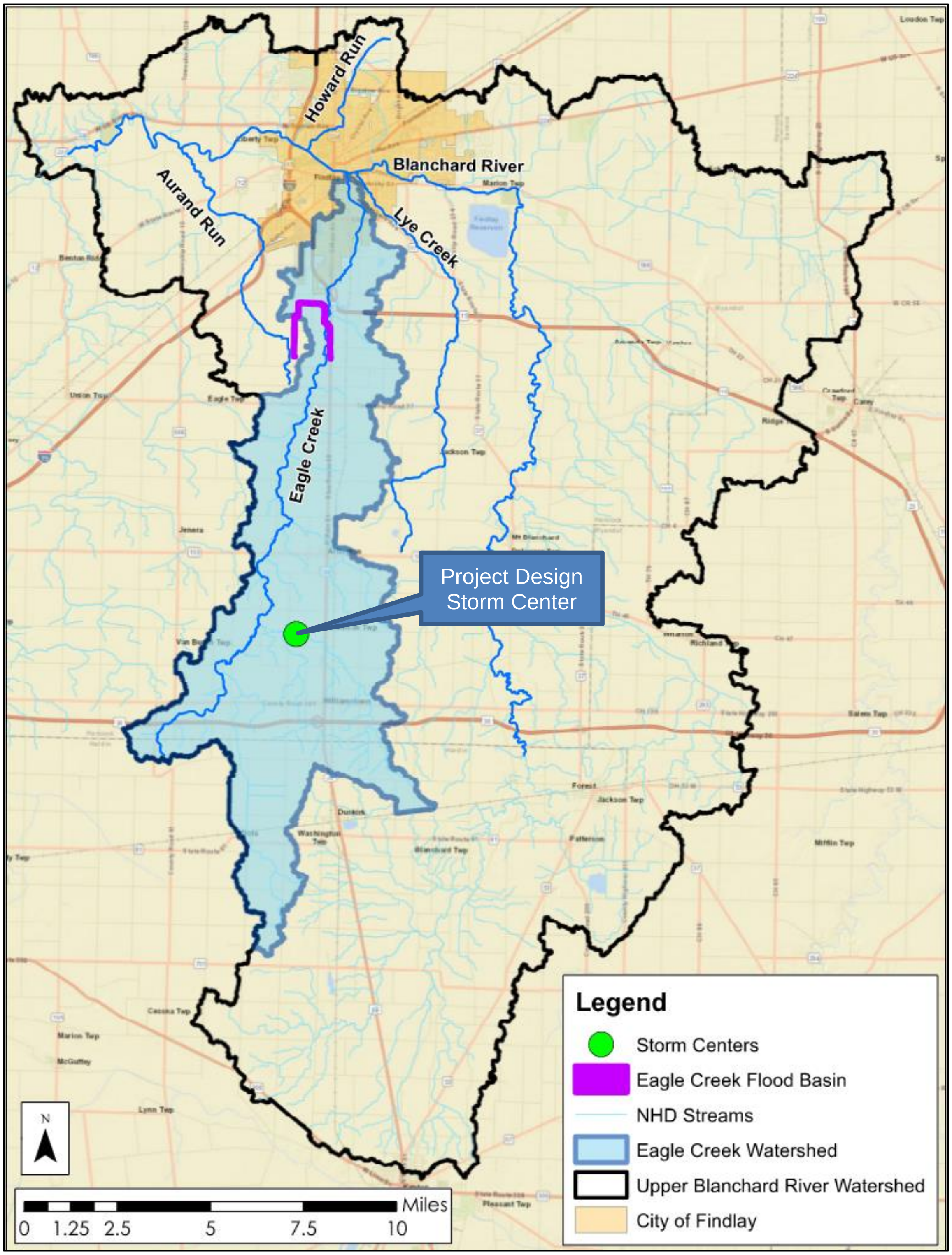

Figure 4. Eagle Creek Watershed

3.1.2 HEC-HMS Model

3.1.2.1 Background

In 2017, Stantec completed a hydrologic analysis of the Upper Blanchard River Watershed for the MWCD as part of the HCFRR Program. The study is based on a model that was originally developed by the United States Army Corps of Engineers (USACE) in 2009 and used by the USACE for their Blanchard River Watershed Study (USACE 2015). For the Hydrologic Evaluation of the Blanchard River (Stantec 2017), the model was updated and modified by Stantec as part of the flood mitigation program. The model included the Upper Blanchard River watershed, down to the confluence of the Blanchard River and Ottawa Creek. This model includes the Eagle Creek and Aurand Run watersheds, tributaries to the Blanchard River.

The updates and modifications are described in the report titled, "*Hydrologic Evaluation of the Blanchard River*" (Stantec 2017). This report (Stantec 2017) is attached as Exhibit A to the Hydrologic and Hydraulic Analysis Report included as Appendix D. The hydrologic analyses performed as part of the Upper Blanchard River watershed evaluation included the following elements:

- Gage frequency analysis on United States Geologic Survey (USGS) gage 04189000 – Blanchard River Downstream of Findlay;
- A site-specific meteorological storm event developed based on large historic storms incorporating spatially varied precipitation and areal reduction factors;
- Custom storm temporal distribution similar to the Huff 3rd quartile rainfall distribution applied in a grid-based pattern; and
- A Hydrologic Engineering Center - Hydrologic Modeling System (HEC-HMS) (version 4.2) model developed with inputs calibrated to USGS gage data from recent storm and flow events.
    - Includes the gage located along Eagle Creek with the proposed ECFB (USGS Gage 04188496 – Eagle Creek Above Findlay)

The calibrated HEC-HMS model developed as part of this previous study (Stantec 2017) was used as the basis for the design analyses.

3.1.2.2 Runoff / Loss Methodology

Subbasin runoff was modeled in HEC-HMS using the SCS curve number approach applied on a grid basis (see Stantec, 2017, Exhibit A to Appendix D). The SCS curve number grid developed as part of the Blanchard River hydrology study was adapted without modification for this project.

3.1.2.3 Transformation Methodology

The selected subbasin transform method was the ModClark grid method. The ModClark grid method was used because it is compatible with the gridded precipitation inputs and produces results with a finer resolution than other methodologies. Associated parameters with this approach are the time of concentration and subbasin retention storage coefficient.

3.1.3 Point Rainfall - Precipitation Data

Precipitation data were used as inputs for the HEC-HMS "Meteorological Models". The modeled rainfall depths were obtained from the National Oceanic and Atmospheric Administration (NOAA) Atlas 14, Volume 2, Version 3 through the NOAA Precipitation Frequency Data Server (PFDS) (NWS 2017). The precipitation depths obtained for each of the recurrence intervals from the PFDS are listed in Table 5. The point rainfalls were used as reported from the PFDS at the storm center with the project's spatial rainfall distribution applied outward from the storm center.

Table 5. Point Rainfall Data

Return Interval	Depth (Inches)
99.9% Annual Chance Exceedance (1-Year), 24-Hour	2.04
50% Annual Chance Exceedance (2-Year), 24-Hour	2.44
20% Annual Chance Exceedance (5-Year), 24-Hour	3.01
10% Annual Chance Exceedance (10-Year), 24-Hour	3.48
4% Annual Chance Exceedance (25-Year), 24-Hour	4.14
2% Annual Chance Exceedance (50-Year), 24-Hour	4.69
1% Annual Chance Exceedance (100-Year), 24-Hour	5.26
0.5% Annual Chance Exceedance (200-year), 24-Hour	5.87
0.2% Annual Chance Exceedance (500-Year), 24-Hour	6.72

3.1.4 Rainfall Distribution

3.1.4.1 Typical Storm

A site-specific rainfall pattern and temporal distribution was developed (Stantec, 2017) for storms that are reasonably expected to occur in the Upper Blanchard River Watershed. The study analyzed historic events that have occurred in the region, and which could be reasonably transposed to the Blanchard Watershed. The outcome of the study recommended a typical storm orientation, shape, and areal reduction factors.

3.1.4.2 Design Model Storm Center

For design of the ECFB Principal Spillway and Auxiliary Spillway structures, the typical storm was modeled in the Eagle Creek watershed HEC-HMS model. The Eagle Creek watershed drainage area, at the confluence with the Blanchard River, is approximately 63.4 square miles. The portion of the watershed draining into the proposed Eagle Creek Flood Basin is approximately 55.0 square miles. The largest peak discharge was determined to result from a storm event centered over the centroid of the Eagle Creek watershed. This storm center location is illustrated in Figure 4.

3.1.5 Design Model Storm Events

The 1% ACE (100-year) storm event was used for sizing the Principal Spillway discharge outlet structure while maintaining a normal pool during the 100-year between elevation 806.8 feet and 807.0 feet. Other discharge estimates were calculated in HEC-HMS for a range of annual return intervals. The frequency and magnitude of expected floods inform design load cases.

3.1.6 Probable Maximum Flood (PMF)

The full (100%) Probable Maximum Flood (PMF) hydrograph was used for determining the Auxiliary Spillway capacity and was a basis for determining the dam embankment crest height based on required freeboard height.

The ODNR Probable Maximum Precipitation (PMP) Application Guidelines stipulate that the PMP Study for the State of Ohio developed by Applied Weather Associates, LLC (AWA) shall be used to determine the PMP values. Probable maximum precipitation is defined as the rainfall depth that approaches the maximum amount of moisture the atmosphere can produce given the current meteorological and atmospheric conditions. The AWA PMP storm depths are used in tandem with the storm temporal distributions prescribed in Hydrometeorological Report No. 52 (HMR52).

The PMF is derived by applying the optimized PMP to the hydrologic model developed for the Eagle Creek watershed. The resulting PMF inflow hydrograph is shown graphically in Figure 5 with a peak discharge of 28,778 cfs. Details related to the PMF development are provided in the Probable Maximum Flood Study report in Exhibit B of Appendix D.

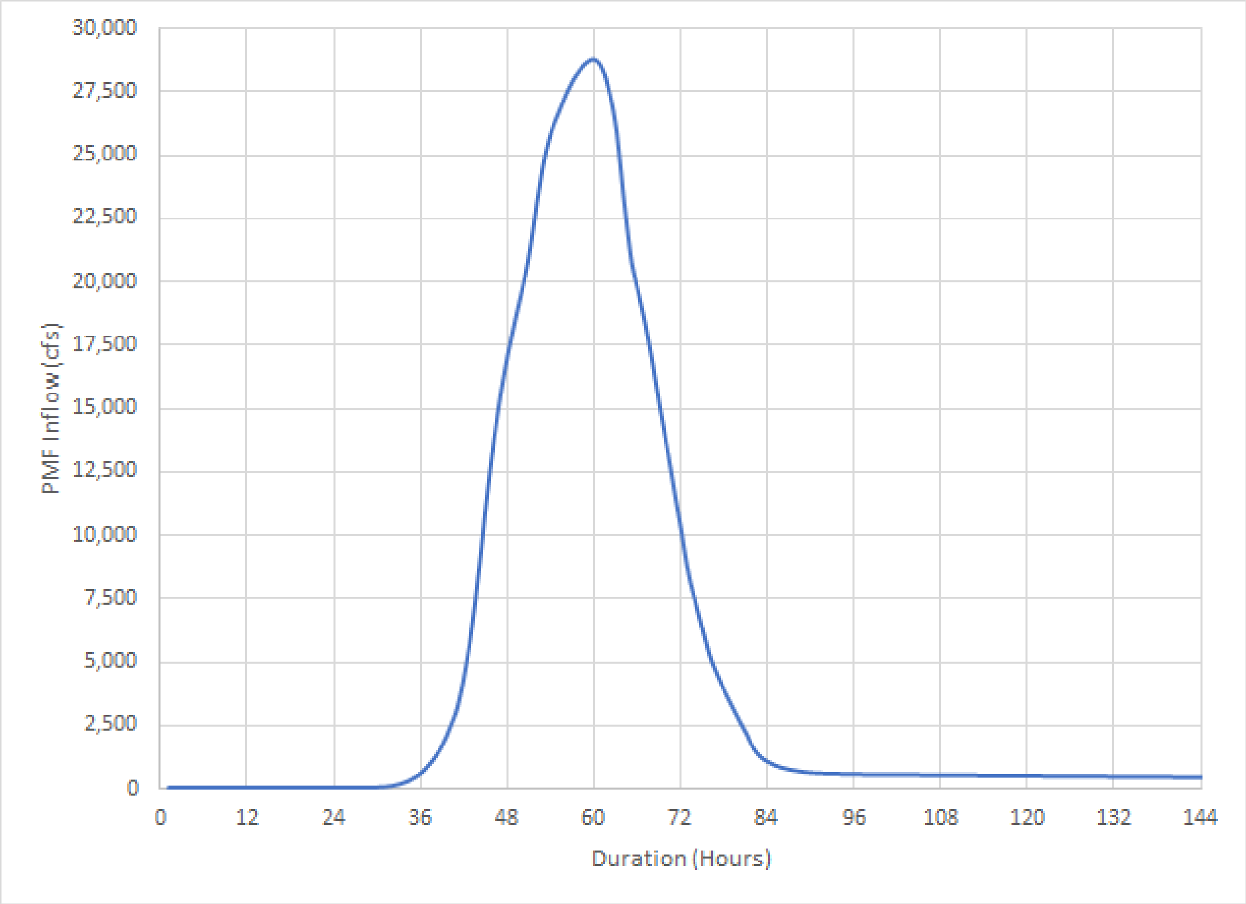

Figure 5. PMF Inflow Hydrograph at the Eagle Creek Flood Basin Project Site

3.2 HYDRAULICS

The Blanchard River watershed Hydrologic Engineering Center - River Analysis System (HEC-RAS) hydraulic model was originally developed by the USACE in 2009 in version 4.0. The model was advanced by the USACE for their Blanchard River Watershed Study (USACE 2015). Stantec leveraged the model and refined it during the concept alternatives phase. This model is used as the basis for Final Design, with a few modifications. The model is a one-, and two-dimensional (1D and 2D) unsteady-state model. The HEC-RAS model was upgraded to HEC-RAS version 6.1 and refined to support various aspects of design. HEC-RAS hydraulic modeling was also supplemented by a three-dimensional computational fluid dynamics (CFD) model to support the design of the Auxiliary and Principal Spillways. Finally, several local, stand-alone models were created to support the design of the exterior drainage system using HEC-HMS, PCSWMM and HY-8. Additional information about these models is presented in the H&H Report included as Appendix D, the Exterior Drainage Analysis Report included as Appendix H, and the Principal Spillway Final Design Report included as Appendix I.

3.2.1 Existing Conditions Peak Discharge

Inflow and lateral flow hydrographs from the HEC-HMS model were used as inputs to be routed through the hydraulic model. Stantec simulated multiple recurrence intervals in the design HEC-RAS model including the 99.9%, 50%, 20%, 10%, 4%, 2%, 1% and 0.2% ACE storm events (1-, 2-, 5-, 10-, 25-, 50-, 100-, and 500-year). The model used the existing terrain that is based on LiDAR and topographic survey data. Existing conditions peak discharge results, extracted from the unsteady HEC-RAS design model, are shown in Table 6. The flow hydrographs at Township Road 49 for each recurrence interval are shown in Figure 6

Table 6. Existing Conditions Peak Discharge at the Project Site (Township Road 49)

Return Interval	Peak Discharge (cfs)
99.9% Annual Chance Exceedance (1-Year), 24-Hour	1,393
50% Annual Chance Exceedance (2-Year), 24-Hour	1,806
20% Annual Chance Exceedance (5-Year), 24-Hour	2,462
10% Annual Chance Exceedance (10-Year), 24-Hour	3,005
4% Annual Chance Exceedance (25-Year), 24-Hour	3,794
2% Annual Chance Exceedance (50-Year), 24-Hour	4,437
1% Annual Chance Exceedance (100-Year), 24-Hour	5,132
0.5% Annual Chance Exceedance (200-year), 24-Hour	5,841
0.2% Annual Chance Exceedance (500-Year), 24-Hour	7,086

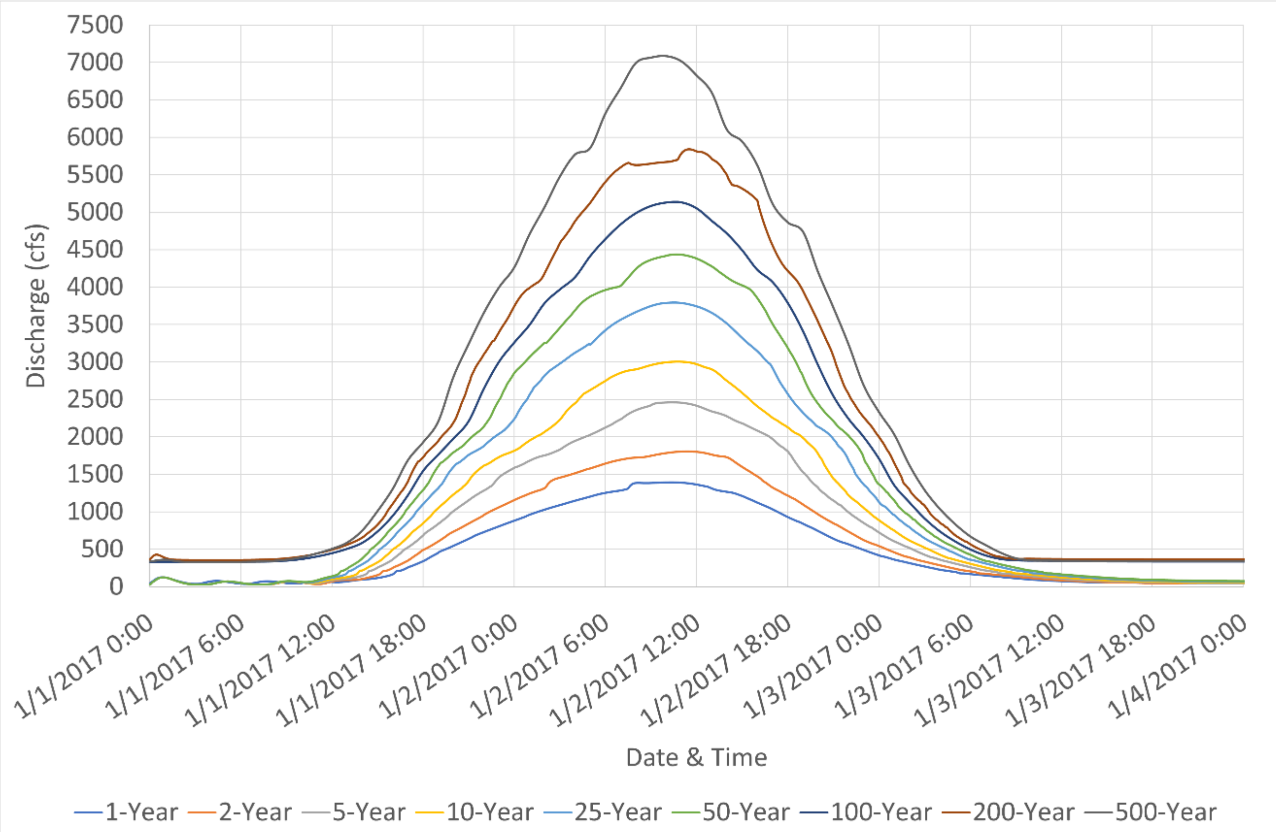

Figure 6. Existing Conditions Flow Hydrographs at the Project Site (Township Road 49)

3.2.2 Summary of HEC-RAS Modeling

3.2.2.1 Upper Blanchard Watershed Model

The Upper Blanchard Watershed model is an unsteady state 1D-2D hybrid HEC-RAS model. The model extents include the Blanchard River from approximately Ottawa Creek at its downstream end to approximately 3 miles upstream of TR-150. The model includes the main tributaries to the Blanchard River, including Aurand Run, Eagle Creek, Howard Run, Lye Creek and Potato Run. The majority of the model is comprised of 1D elements; however, it includes a 2D grid connecting the overbanks between Lye Creek and the Blanchard River upstream of Findlay to calculate flow transfer between the two streams during larger flood events. The model includes both existing and proposed conditions scenarios and is used to evaluate the impact of the ECFB on downstream flow rates and quantify project benefits.

3.2.2.2 Eagle Creek Design Model

The Eagle Creek Design Model is an unsteady state 1D-2D hybrid HEC-RAS model created by truncating the Upper Blanchard River Watershed model and adding additional detail. The model extents are limited to Eagle Creek. The existing conditions Eagle Creek Design Model was updated using LiDAR and bathymetric data in the vicinity of the ECFB project to create a more accurate representation of existing conditions. The existing conditions model was then modified to incorporate a 2D grid representing the ECFB embankment and spillway structures to represent proposed conditions. The Eagle Creek Design Model is used to evaluate the upstream impact of the ECFB and was used to support the design of proposed project channel grading and erosion protection components as well as provide hydraulic loading cases for the embankment design.

3.2.2.3 Eagle Creek Tailwater Model

The Eagle Creek Tailwater Model is a quasi-steady state 2D HEC-RAS model extending from the Auxiliary and Principal Spillway at the upstream end to US-68 at the downstream end. The Eagle Creek Tailwater Model was used to simulate a wide range of constant flow rates to establish tailwater rating curves for the Principal Spillway and Auxiliary Spillway calculations and models.

3.2.2.4 Principal Spillway Design Model

The Principal Spillway design model is a steady state 1D HEC-RAS model which represents the Principal Spillway control wall and baffled chute. The model was calibrated using CFD modeling results and was used to develop the Principal Spillway rating curve which was then used by the Eagle Creek Design model to route flow through the spillways. The rating curve results were also used to provide hydraulic loading cases for structural and geotechnical design of the Principal and Auxiliary Spillway structures and evaluate whether the structure facilitates fish passage.

3.2.2.5 Eagle Creek CLOMR Model

The Eagle Creek Conditional Letter of Map Revision (CLOMR) Model is a steady state 1D HEC-RAS model converted from the Eagle Creek Design Model. The model extents are from Eagle Creek at the Blanchard River to just upstream of County Road 26. The base model geometry and inputs are refined for the CLOMR to be a 1D steady-state model for consistency with effective FEMA models and as a simplified tool for floodplain regulation in the future. The Eagle Creek CLOMR model was submitted to FEMA and was used to document areas of modeled increases in Base Flood Elevations and changes to the Special Flood Hazard Areas due to the proposed ECFB project. The Eagle Creek CLOMR Model is used to evaluate the upstream impact of the ECFB.

3.2.3 CFD Modeling

CFD modeling was completed to evaluate the Principal and Auxiliary Spillway structures. The CFD models were developed using the Flow-3D HYDRO computer program. The CFD models extended approximately 175 ft upstream and 220 ft downstream of the Principal Spillway control wall and included four cycles of the Auxiliary Spillway labyrinth weir. The CFD model simulations covered a range of discharge conditions, and the results were used to validate spillway rating curve calculations for the Auxiliary Spillway and calibrate the 1D HEC-RAS Principal Spillway Design model used for creating rating curves. The CFD model results were also used to confirm that the spillways sufficiently dissipate energy, confirm that fish passage was facilitated, and was used to supplement the Eagle Creek Design model results for sizing erosion protection downstream of the spillways.

3.2.4 Exterior Drainage Models

Multiple stand-alone models were created to support the design of the exterior drainage ditches and culverts. For the North, Northwest and Southwest Ditches, an unsteady 1D HEC-HMS model was created to compute and route local runoff and compute design discharges for ditches and culverts which comprise the system. For the East Ditch, an unsteady 1D PCSWMM model was created to route local runoff and compute design discharges for the ditches, culverts and storm sewers which comprise the system. The HY-8 culvert modeling software was then used to support the design of individual culverts along the exterior drainage system.

4.0 GEOTECHNICAL

4.1 GEOLOGY / SITE OBSERVATIONS

4.1.1 General

The *Physiographic Regions of Ohio* map (Ohio Department of Natural Resources (ODNR), 1998) indicates that the project site is located in the Central Ohio Clayey Till Plain which has a surface of clayey till, well-defined moraines with intervening flat-lying ground, and intermorainal lake basins. This region contains a few large streams and has moderate relief (100 feet) with elevations of 700 to 1,150 feet. The Columbus Escarpment is approximately one to two miles north of the ECFB site.

4.1.2 Soil Geology

According to the *Quaternary Geology of Ohio* map (ODNR, 1999a), the project site is predominantly underlain by clayey till deposited during the Late Wisconsinan Age. The clayey till originated as flat to gently undulating ground moraine.

The soil survey (*Web Soil Survey of Hancock County, Ohio*, NRCS, 2021) indicates that the site is underlain predominantly by Blount silt loam. These soils consist of silt loam, silty clay, and clay loam with low to moderately high capacities to transmit water.

The *Drift Thickness Map of Ohio* (ODNR, 2004) suggests a range of soil cover near the project site between 0 and 50 feet.

The surficial geology at the site consists of unconsolidated Quaternary glacial till and alluvium. The till is comprised of an unsorted mix of silt, clay, sand, gravel, and boulders of glacial origin (ODNR, 2005). The alluvium is derived from reworked glacial deposits and is present in river valleys (USGS, 1995).

4.1.3 Bedrock Geology

Bedrock mapping *(Reconnaissance Bedrock Geology of the Arlington, Ohio Quadrangle*, ODNR, 1999b) and Descriptions of Geologic Map Units (ODNR, 2011) indicate that overburden soils in the vicinity of the project site are underlain by sedimentary bedrock from the Tymochtee Dolomite Formation of the Silurian System. The Tymochtee Dolomite Formation is composed of olive gray to yellowish brown dolomite with shale laminae. This bedrock is described as thin to massively bedded, with thicknesses ranging from 0 to 140 feet.

According to the Abandoned Underground Mine Locator (ODNR, 2021), mapped underground mines have not been identified in the project vicinity.

The *Ohio Karst Areas* map (ODNR, 2006) does not indicate known karst areas in the vicinity of the sites. Probable karst areas are located east of the project sites in Wyandot and Seneca Counties.

4.1.4 Regional Hydrogeology

Groundwater occurs within unconsolidated surficial deposits and the underlying dolomite. Groundwater within the surficial deposits is generally unconfined (USGS, 1995). However, artesian or confined, conditions exist in places where interbedded clay or silt compose local confining units. Horizontal hydraulic gradients are often influenced by local topography, and are generally oriented towards local drainages, streams, and rivers. Hydraulic conductivities of the glacial deposits are highly variable depending on local lithology (USGS, 1995). Vertical hydraulic conductivity is highly dependent on the presence and thickness of clay-rich till (Bugliosi, 1990).

Groundwater within the dolomite is generally under confined conditions with flow occurring through fractures, bedding planes, and solution cavities (USGS, 1995). Note that solution cavities were not identified in the project exploration program. The dolomite is recharged from the overlying surficial aquifer system in areas where water levels in the surficial aquifer system are higher than those in the dolomite. Groundwater may discharge to the surficial aquifer system locally when water-level differences are reversed. Hydraulic conductivity within the bedrock is generally less than in the overlying unconsolidated sediments. Groundwater flow direction in the bedrock aquifer is generally to the north, towards Lake Erie (Sprowls, 2008), but is also influenced locally by drainages and streams when the bedrock is near the surface.

4.1.5 Local Hydrogeology

In conjunction with the exploration program (Section 4.2), Stantec performed a hydrogeological and geological review of the project site. The review included the following:

- Summaries of the geology and hydrogeology of the region and at the site.
- Development of a three-dimensional (3-D) Lithological Model to support the identification of suitable thicknesses of borrow material, and potential areas susceptible to seepage from stored water through a coarse-grained unit below the embankment.
    - The 3-D Lithological Model is based on geotechnical soil borings advanced at the site (as discussed in Section 4.2), groundwater level measurements collected from on-site piezometers, and surface water measurements from a USGS surface water gauging station.
- Estimation of aquifer properties (e.g., horizontal hydraulic conductivity) to support the understanding of the hydrogeological framework.
- Hydrographs, water level contour maps, and hydraulic gradient estimations were completed to support the characterization of groundwater flow at the site.

A technical memorandum summarizing the hydrogeological and geological review is provided as Exhibit D of the Geotechnical Design Report (Appendix E).

4.1.6 Seismic

A review of the seismic data available in the project vicinity included the database developed by the ODNR, Division of Geological Survey (ODNR, 2022). Overall, Ohio has relatively limited seismic activity. However, within a 100-kilometer (approximately 62 miles) radius of the proposed dam site, there have been 75 earthquake epicenters with magnitudes ranging between 2.0 to 5.4. The available data included events that occurred from 1804 to present day.

The proposed dam is classified as a Class I structure per ODNR and Ohio Revised Code definitions. According to the NRCS Technical Release Number 60 (NRCS, 2005), the project site is in Seismic Zone 2, and therefore requires an evaluation of liquefaction potential and the presence of nearby faults. Liquefaction potential of the site subsurface soils is discussed in Section 4.4.11 and the presence of nearby faults are discussed in the Geotechnical Design Report (Appendix E).

4.2 EXPLORATION

In addition to recent historical exploration programs, Stantec advanced 120 borings and excavated six (6) test pits during the Preliminary and Final Design Phases to obtain geotechnical data for the proposed flood basin. Seventy-two (72) borings were located along the proposed embankment alignment, and 15 borings at the proposed integrated spillway structure. The alignment and structure locations shifted during design and the boring locations may or may not directly coincide with the currently proposed geometry. The borings are still considered representative of the project site subsurface conditions.

Ten (10) borings were located at regular intervals approximately 400 feet upstream from the toe of the proposed embankment to evaluate the thickness and continuity of the upper fine-grained soil layer. Twenty-three (23) borings and six (6) test pits were located within the interior of the basin to classify and quantify potential borrow materials.

Water pressure testing was performed in fifteen (15) borings to provide data for estimating permeability and flow regimes through the bedrock. Twenty-nine (29) open standpipe piezometers were installed, and twenty-five (25) were outfitted with water level transducers to collect pressure and temperature readings. Rising head and falling head slug tests were performed in the 25 installed piezometers that were outfitted with transducers.

The soil samples obtained during the exploration were subjected to laboratory testing performed by Stantec and GeoTesting Express. Soil testing was performed to characterize the soil type, shear strength, hydraulic conductivity, and other material properties.

In general, the laboratory analyses consisted of natural moisture content determinations, particle size analysis (sieve and hydrometer), Atterberg limits, specific gravity, unit weight, consolidated-undrained triaxial compression, unconsolidated-undrained triaxial compression, consolidated-drained direct shear, unconfined compression, consolidation, falling head permeability, soil water characteristic curve, dispersive clay (double hydrometer, pinhole, and crumb), soil resistivity, and standard Proctor testing.

Additional detail regarding the field exploration (including boring locations and boring logs) and laboratory testing/data is presented in the Geotechnical Exploration Data Report (GEDR), which is provided as Exhibit C of the Geotechnical Design Report (Appendix E).

4.3 GROUNDWATER SAMPLING

The field team observed damage to the piezometer transducers during data collection, including corrosion of the stainless-steel cables suspending the transducers, malfunctions of the transducers themselves, and black residue within the open standpipes and on the transducer equipment. These observations led to a field program to test groundwater samples retrieved from the installed piezometers.

Groundwater sampling and testing was completed during two events. The purpose of the first sampling event was to determine if the black residue observed on the piezometer transducers, as well as the petroleum hydrocarbon odors noted during the geotechnical exploration, were naturally occurring. Groundwater samples were analyzed for the following constituents:

- Total Petroleum Hydrocarbon (TPH), Middle and Heavy Distillates/Diesel Range and Extended Range Organics
- TPH Fingerprinting

Groundwater analytical results indicated that the observed residues in the piezometers were not a result of naturally occurring hydrocarbons in the groundwater. During the first sampling event, the field geologist noticed a strong rotten egg smell (commonly associated with sulfur-reducing bacteria) and significant deterioration of the metal cables used to suspend the transducers in the piezometers. Groundwater geochemical results for Oxidation Reduction Potential (ORP) also indicated highly reducing conditions in the sampled piezometers. Based on the ORP results and the onsite observations, corrosion of the transducer cables and the black residue in the piezometers was likely caused by naturally occurring hydrogen sulfide. Further groundwater sampling and testing was warranted to support this hypothesis. Additional characteristics of interest included sulfates, sulfides, and hydrogen sulfide in groundwater.

The purpose of the second sampling event was to further characterize groundwater conditions, to support material selection for the seepage cutoffs and spillway foundations, considering the potential corrosive nature of hydrogen sulfide. Groundwater samples were collected and analyzed for hydrogen sulfide, sulfate, sulfide, dissolved gases (ethane, ethene, methane), ferrous iron, iron, manganese, and total organic carbon (TOC). Results from the second sampling event confirmed the presence of hydrogen sulfide with the highest concentrations occurring in the area of the proposed seepage cutoff wall (near B-3.52) and on the west side of the basin (near B-3.14). Hydrogen sulfide was not detected in the surface water samples. Groundwater geochemical results for oxidation reduction potential also indicated highly reducing groundwater conditions (i.e., anaerobic). Anaerobic conditions along with the presence of organic compounds provide a favorable environment for sulfur reducing bacteria. Concentrations of sulfate were detected in all seven piezometers sampled which has the potential of being converted to hydrogen sulfide. Attempts were made to correlate the occurrence of hydrogen sulfide to geologic strata (i.e., upper soils, lower coarse grain material, or dolomite) but no strong correlation was observed. Based on these results, alternatives to metal structures (e.g., vinyl sheet piles) and concrete additives are recommended in the construction of the seepage cutoffs and spillway foundations

Details from the groundwater sampling events can be found in Exhibit M of the Geotechnical Design Report (Appendix E).

4.4 SOIL PARAMETERS

The subsections below summarize the key materials and selected soil parameters used in the design analyses. Detailed descriptions and parameter derivations are provided in the Geotechnical Design Report (Appendix E).

4.4.1 Key Materials

The geotechnical data obtained from the explorations and the hydrogeological and geological review were used to select representative soil layers for foundation soil characterization. Four soil layers were identified:

- Upper Fine-Grained material
- Upper Coarse-Grained material
- Lower Fine-Grained material
- Lower Coarse-Grained material

The key materials represented in the geotechnical analyses are identified in Table 7.

Table 7. Identification of Materials

Material Name	General Description
Embankment Fill	Compacted fill soil used to construct the dam embankment and assumed to be borrowed from the Upper Fine-Grained material; this represents embankment Fill Type 1 as shown on the Drawings and defined in the Specifications
Upper Fine-Grained	Overconsolidated lean clay with varying amounts of sand, low to medium plasticity, brown to orange-brown, firm to hard
Upper Coarse-Grained	Silty and clayey sand, non-plastic to low plasticity, gray and brown, medium dense to dense
Lower Fine-Grained	Overconsolidated lean clay with varying amounts of sand and gravel, gray, low plasticity, hard to very hard
Lower Coarse-Grained	Silty sand with varying amounts of gravel, non-plastic, gray or gray and black, dense
Filter	Filter Sand and ODOT No. 7 Coarse Aggregate (ODOT CMS 2019 703.01) used for chimney, blanket, and toe drain or other filter/drain elements of the dam embankment

4.4.2 Density Parameters

Density parameters for the foundation and proposed embankment materials are needed for stability and settlement analyses are summarized in Table 8.

Table 8. Density Parameters

Material Name	G_s	w (%)	e	γ_d (pcf)	γ_m (pcf)	γ_{sat} (pcf)
Embankment Fill	2.70	18	0.53	110	130	132
Upper Fine-Grained	2.69	22	0.70	99	121	124
Upper Coarse-Grained	2.70	24	0.70	99	123	125
Lower Fine-Grained	2.70	17	0.49	113	132	132
Lower Coarse-Grained	2.70	17	0.46	115	135	135
Filter	2.65	13	0.38	120	136	137

G_s = specific gravity of the solids
w = natural, gravimetric water content
e = void ratio
γ_d = dry unit weight
γ_m = moist unit weight
γ_{sat} = saturated unit weight

4.4.3 Saturated Soil Permeability

The saturated permeability parameters for the foundation and proposed embankment materials are needed for seepage analyses and are provided in Table 9.

Table 9. Saturated Permeability Parameters

Material Name	k_v (cm/sec)	k_h (cm/sec)	k_h/k_v	*Inputs for SEEP/W*	
				k_x (ft/sec)	k_y/k_x
Embankment Fill	1.0×10^{-7}	5.0×10^{-7}	5	1.6×10^{-8}	0.2
Upper Fine-Grained	3.5×10^{-8}	3.5×10^{-6}	100	1.1×10^{-7}	0.01
Upper Coarse-Grained	1.3×10^{-5}	1.3×10^{-3}	100	4.3×10^{-5}	0.01
Lower Fine-Grained	2.8×10^{-7}	2.8×10^{-5}	100	9.2×10^{-7}	0.01
Lower Coarse-Grained	1.1×10^{-5}	1.1×10^{-3}	100	3.6×10^{-5}	0.01
Filter	3.0×10^{-2}	3.0×10^{-2}	1	9.8×10^{-4}	1

4.4.4 Unsaturated Soil Permeability

Unsaturated permeability parameters are needed in seepage analyses and are discussed and defined in the Geotechnical Design Report (Appendix E). The selected design values are presented in Table 10.

Soil-Water Characteristic Curve (SWCC) testing was performed on samples of the Embankment Fill and Upper Fine-Grained material. In the absence of specific laboratory testing, the SWCC can be characterized by the four parameters listed in Table 10.

Table 10. Unsaturated Permeability Parameters

Material Name	Unsaturated Permeability Parameters				
	α (cm^{-1})	Units for SEEP/W Input α (psf)$^{(1)}$	n	θ_s	θ_r
Embankment Fill	One laboratory SWCC test – see Figure 7				
Upper Fine-Grained	Average of two laboratory SWCC tests – see Figure 7				
Upper Coarse-Grained	0.021	97	1.61	0.41	0.067
Lower Fine-Grained	0.030	68	1.37	0.33	0.129
Lower Coarse-Grained	0.021	97	1.61	0.32	0.067
Filter	0.035	58	3.19	0.28	0.058

(1) Dividing the unit weight of water by α results in a parameter with units of pressure.

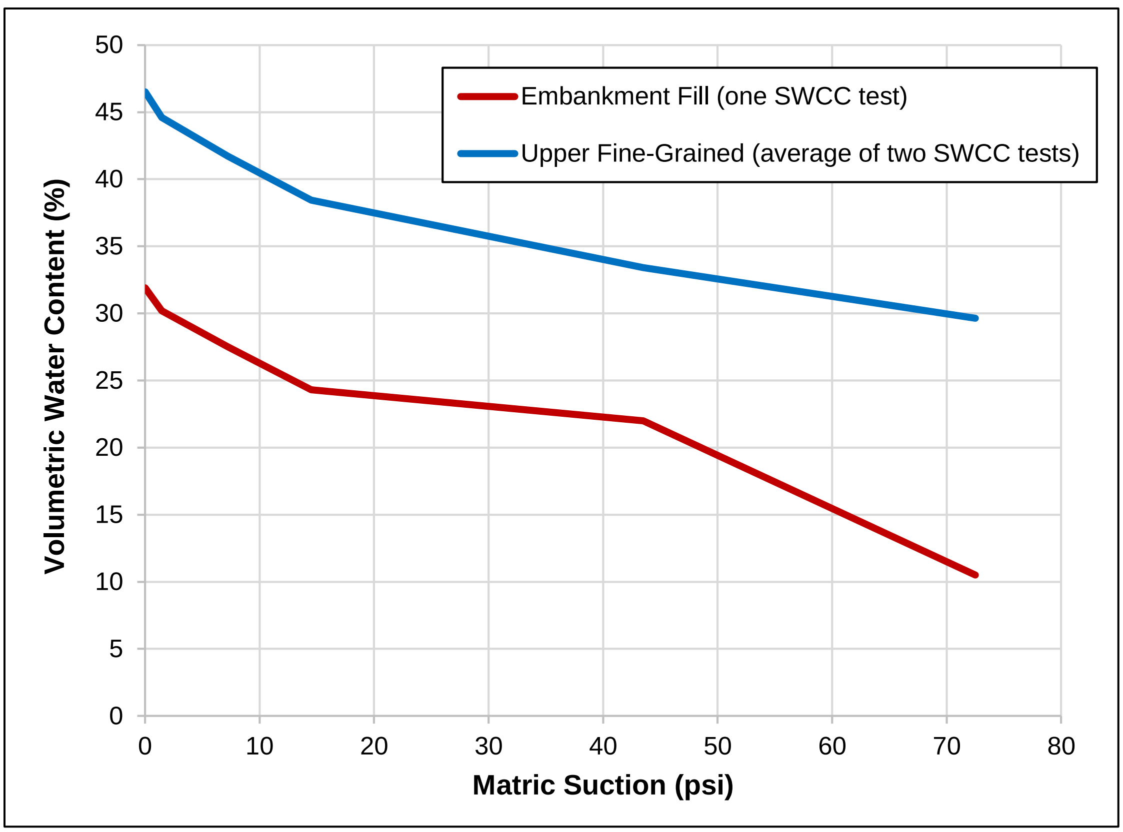

Figure 7. Laboratory Soil-Water Characteristic Curves

4.4.5 Dispersive Clays

Selected soils, which might be associated with a potential failure mechanism of seepage along an internal crack, were tested for dispersive clay properties. Six samples were subjected to crumb, double hydrometer, and pinhole tests. The tested materials were judged to be non-dispersive.

4.4.6 Gradation Characteristics

Gradation envelopes were developed from the available laboratory testing results for the embankment and foundation materials for use in evaluating filter compatibility See the Geotechnical Design Report (Appendix E) for details on the soil gradation characteristics.

4.4.7 Drained Strengths for Static, Long-Term Conditions

Drained strength parameters for the embankment and foundation materials under static, long-term conditions are summarized in Table 11. In general, laboratory shear strength test results were used to assign selected shear strength parameters to the project soils.

Table 11. Drained Shear Strength Parameters for the Analysis of Static, Long-Term Conditions

Material Name	ϕ' (deg)	c' (psf)
Embankment Fill	33	0
Upper Fine-Grained	34	0
Upper Coarse-Grained	34	0
Lower Fine-Grained	34	0
Lower Coarse-Grained	37	0
Filter	33	0

4.4.8 Undrained Strengths for Static, Short-Term Conditions

Undrained strength parameters for the embankment and foundation materials under static, short-term conditions are summarized in Table 12. In general, laboratory shear strength test results were used to assign selected shear strength parameters to the project soils.

Table 12. Undrained Shear Strength Parameters for the Analysis of Static, Short-Term Conditions

Material Name	Consolidated Undrained (CU)		Unconsolidated-Undrained (UU)
	ϕ (deg)	c (psf)	Undrained Shear Strength, s_u (psf), ϕ = 0 deg
Embankment Fill	25	100	2,900
Upper Fine-Grained	20	400	700
Upper Coarse-Grained	20	400	700
Lower Fine-Grained	25	1,000	1,200
Lower Coarse-Grained	37	0	N/A
Filter	33	0	N/A

4.4.9 Consolidated-Undrained Strengths for Rapid Drawdown Conditions

The upstream slope of the embankment dam will be subject to a rapid drawdown loading condition when a retained flood pool drops quickly after a storm event. The parameters for the embankment and foundation materials during rapid drawdown are summarized in Table 13.

Table 13. Shear Strength Parameters for the Analysis of Rapid Drawdown Conditions

Material Name	Drained Strength		Isotropically Consolidated, Undrained Strength	
	ϕ' (deg)	c' (psf)	ϕ (deg)	c (psf)
Embankment Fill	33	0	25	100
Upper Fine-Grained	34	0	20	400
Upper Coarse-Grained	34	0	20	400
Lower Fine-Grained	34	0	25	1,000
Lower Coarse-Grained	37	0	37	0
Filter	33	0	33	0

4.4.10 Undrained Strengths for Earthquake Conditions

Pseudo-static slope stability analysis is used to evaluate the seismic stability of the dam. The methodology assumes the seismic (earthquake) soil strength parameters are reduced to 80 percent of the static undrained strength parameters. Table 14 provides the reduced undrained shear strength parameters for use in the pseudo-static slope stability analyses.

Table 14. Seismic Shear Strength Parameters

Material Name	ϕ_{EQ} (deg)	c_{EQ} (psf)
Embankment Fill	0	2,320
Upper Fine-Grained	16	320
Upper Coarse-Grained	16	320
Lower Fine-Grained	20	800
Lower Coarse-Grained	31	0
Filter	27	0

4.4.11 Liquefaction/Cyclic Softening Susceptibility

An initial screening was performed to determine if the soils were potentially subject to liquefaction/cyclic softening during an earthquake event. See the Geotechnical Design Report (Appendix E) for details on the assessment and methodology. Table 15 summarizes the results of the liquefaction/cyclic softening screening.

Table 15. Results of Liquefaction/Cyclic Softening Screening

Material	Susceptible Material
Upper Fine-Grained	None
Upper Coarse-Grained	Sand-Like, Susceptible to Liquefaction from Station 118+00 to 159+00
Lower Fine-Grained	Sand-Like, Susceptible to Liquefaction from Station 99+00 to 155+00
Lower Coarse-Grained	Sand-Like, Susceptible to Liquefaction from Station 62+00 to 102+00 and 143+00 to 176+00

A triggering analysis was completed to evaluate if the susceptible materials would be predicted to liquefy during a design earthquake event (see Section 5.10). Liquefaction is not expected to be triggered during the design earthquake event at the project site.

4.4.12 Compressibility

The compressibility parameters for the foundation materials are summarized in Table 16. Compressibility parameters for the foundation materials are needed to compute expected foundation settlements. The selected compressibility parameters are based on the results of laboratory consolidation tests on undisturbed samples.

Table 16. Compressibility Parameters

Material Name	Initial Void Ratio, e_0	Compression Index, C_c	Recompression Index, C_r	Representative Preconsolidation Pressure (tsf)	Coefficient of Consolidation, c_v (ft^2/day)
Upper Fine-Grained	0.70	0.25	0.034	1.4	0.031
Upper Coarse-Grained	0.70	0.12	0.0073	4.4	0.096
Lower Fine-Grained	0.49	0.14	0.011	1.4	0.15
Lower Coarse-Grained[(1)]	N/A	N/A	N/A	N/A	N/A

[(1)] Lower Coarse-Grained material is free-draining and will consolidate during construction; these parameters not required for design.

4.4.13 Corrosivity

Soil resistivity tests were performed on bulk and undisturbed samples. The results indicated soil resistivity between 1,236 and 2,706 ohm-cm.

Corrosivity potential of the soil to ductile-iron pipes was estimated using ANSI/AWWA C105/A21.5 (ANSI/AWWA 2005). Based on the results of the laboratory resistivity testing and according to ANSI/AWWA C105/A21.5 (ANSI/AWWA 2005), the soil is expected to be corrosive to ductile-iron pipe. Polyethylene encasement should be used for ductile-iron pipe systems, as outlined by ANSI/AWWA C105/A21.5 (ANSI/AWWA 2005).

As discussed in Section 4.3, groundwater sampling for the project identified the presence of hydrogen sulfide. Hydrogen sulfide can corrode metals and concrete structures. Based on the results, the groundwater on the project site should be considered corrosive. Alternatives to metal structures (e.g., PVC (vinyl) sheet piles) and concrete additives are recommended for seepage cutoffs and spillway foundations.

4.5 BEDROCK PROPERTIES

The bedrock was described as gray dolomite, slightly weathered, fractured to highly fractured, slightly rough, and thin to medium bedded. Fractured zones and water loss were noted in the bedrock until the termination depths. Therefore, bedrock was modeled as “Fractured Bedrock” in the seepage and stability analyses.

Table 17 summarizes the saturated permeability parameters selected for the Fractured Bedrock. Because groundwater in the applicable borings was encountered above the top of rock, the bedrock materials were modeled with only saturated permeability parameters in the seepage model. The saturated permeability parameters were selected based on the results from the water pressure testing and slug testing and comparisons to typical values of bedrock permeability.

Table 17. Saturated Permeability Parameters

Material Name	k_v (cm/sec)	k_h (cm/sec)	k_h/k_v	*Inputs for SEEP/W*	
				k_x (ft/s)	k_y/k_x
Fractured Bedrock	1.7×10^{-3}	1.7×10^{-3}	1	5.6×10^{-5}	1

For the stability analyses, the Fractured Bedrock was assumed to be much stronger than the overburden soils and was modeled as "impenetrable". Slip surfaces do not pass through "impenetrable" materials. For settlement analyses, the bedrock was assumed to be incompressible.

4.6 SOIL MATERIAL BORROW STUDY

As part of the geotechnical exploration for the project, Stantec conducted a series of borings on the interior of the proposed embankment alignment. Twenty-three (23) borings were advanced, and six test pits were excavated to evaluate potential borrow soils. An additional ten borings were conducted to evaluate continuity of an upper layer of fine-grained soil, but also provided data to inform the borrow study. Laboratory testing was performed on selected samples to support the characterization of the materials observed during the exploration.

The Soil Material Borrow Study, in Appendix F, is a technical memorandum that provides a summary of the geotechnical exploration and assessment of the native soils within the basin for use as borrow material for the embankment construction. The results of the Soil Material Borrow Study were compared against the anticipated embankment fill specifications to evaluate suitability.

Based on the boring observations and laboratory testing results, the Upper Fine-Grained material (as defined in Section 3.2 of the Soil Borrow Material Study) observed on site generally meets the anticipated embankment requirements (Section 5.1 of the Soil Borrow Material Study) for use as earthen embankment Fill Type 1 (as defined in the Embankment Design Technical Memorandum (Appendix G)) for this project.

4.6.1 Earthwork Materials

The following earthwork materials are defined for the construction of the project.

4.6.1.1 Unsuitable Materials

Unsuitable fill materials include topsoil, frozen materials, construction materials and materials subject to decomposition, clods of clay and stones larger than three (3) inches, organic material (including silts), which are unstable, and inorganic materials (including silts) too wet to be stable. Unsatisfactory soils also include satisfactory soils not maintained within specified limits of optimum moisture content at the time of compaction, as defined by ASTM D698 or ASTM D1557.

4.6.1.2 Fill Type 1

Fill Type 1 are soil materials used in the construction of the dam embankment, below spillways or other concrete structures, or other features as shown on the Drawings, with compaction requirements compared to ASTM D698 or ASTM D1557 as specified in technical specification 31 20 00. - Earth Moving.

- Materials placed as Fill Type 1 consist primarily of clay soils that are suitable for dam embankment construction and free of unsuitable material. The gravel content shall be less than 15 percent by weight.
- Fill Type 1 materials exhibit a classification of CL, CL-ML, CH, or SC as determined in accordance with ASTM D2487. Materials classifying as ML or MH may be mixed with other materials if the resulting mixture meets one of the specified Fill Type 1 material classifications above.
- Materials placed as Fill Type 1 exhibit an average Plasticity Index (PI) that does not exceed 40. For this provision, the Plasticity Index shall be measured in accordance with ASTM D4318 using representative samples.
- Materials exhibiting a Plasticity Index (PI) less than 12 shall not be used as Fill Type 1, unless otherwise permitted in writing by the Engineer. For this provision, the Plasticity Index shall be measured in accordance with ASTM D4318 using representative samples.

4.6.1.3 Fill Type 2

Fill Type 2 are soil materials used in the construction features as shown on the Drawings, with compaction requirements compared to ASTM D698 as specified in technical specification 31 20 00. - Earth Moving.

- Materials placed as Fill Type 2 shall be free unsuitable materials. The gravel content shall be less than 15 percent by weight.
- Fill Type 2 materials shall exhibit a classification of CL, CL-ML, CH, SC, ML, or SM as determined in accordance with ASTM D2487.
- Materials placed as Fill Type 2 shall exhibit an average Plasticity Index (PI) that does not exceed 40. For this provision, the Plasticity Index shall be measured in accordance with ASTM D4318 using representative samples.

4.6.1.4 Fill Type 3

Fill Type 3 are soil materials used to raise existing grades in areas other than those specified as requiring Fill Type 1, or 2 materials as shown on the Drawings and as specified in technical specification 31 20 00. - Earth Moving.

- Materials placed as Fill Type 3 shall be free of unsuitable material and free of organic soil classifications as determined in accordance with ASTM D2487.

4.6.1.5 Topsoil

Topsoil or amended soil fill is material capable of supporting vegetation that is placed directly over the subsoil.

5.0 DAM EMBANKMENT

The earthen embankment dam ranges in height from approximately 1 foot tall at the tie-in locations at the upstream end of the Basin, up to about 30 feet tall at the embankment's intersection with Eagle Creek. The dam alignment is approximately 3.70 miles long, with a crest elevation that varies between 812.0 feet and 813.0 feet. Fill soils required for earthen embankment construction are anticipated to come from within the interior of the basin.

Hydrologic, hydraulic, and geotechnical analyses were completed to support design of the earthen embankment dam. Specifically, analyses were completed to evaluate the required storage capacity and embankment geometry to meet the design objectives and criteria defined in the project Design Criteria Document (Appendix A). The design criteria were informed by ODNR, Division of Water, Dam Safety Program regulations and guidelines and standards published by the USACE and USBR. A stand-alone Dam Embankment Design Technical Memorandum is included as Appendix G that describes the analyses completed for the dam embankment in greater detail.

5.1 GENERAL ARRANGEMENT

To provide maintenance and monitoring access to the embankment, provide the necessary flood protection, and meet the project design criteria, the embankment dam geometry will consist of the following:

- Crest elevation: 812.0 to 813.0 feet (maximum PMF WSE + calculated wind/wave run-up + hydrologic uncertainty)
- Embankment side slopes: 3H:1V (maximum slope for access, monitoring, and maintenance)
- Crest width: 12 feet minimum (for vehicle access for maintenance and monitoring), and widens to 14 feet where the embankment is taller than 25 feet
- Crest surface cross slope: 2 percent minimum (to provide surface drainage)
- Excavation (stripping) as needed to remove vegetation and topsoil under the dam footprint
- Inspection (cutoff) trench: 10 feet wide with minimum depth equal to height of embankment up to a maximum depth of 6 feet; backfilled with compacted fill
- Minimum 15 feet wide offset (bench) between the downstream toe and the exterior drainage ditch (for monitoring and access)
- Minimum 50 feet wide offset (bench) upstream of the dam embankment to provide positive drainage away from the interior toe

In select areas of the embankment dam, the typical cross section also includes a seepage cutoff, chimney drain, blanket drain, and/or toe drain. The typical embankment geometry including a chimney, blanket, and toe drain is provided in Figure 8.

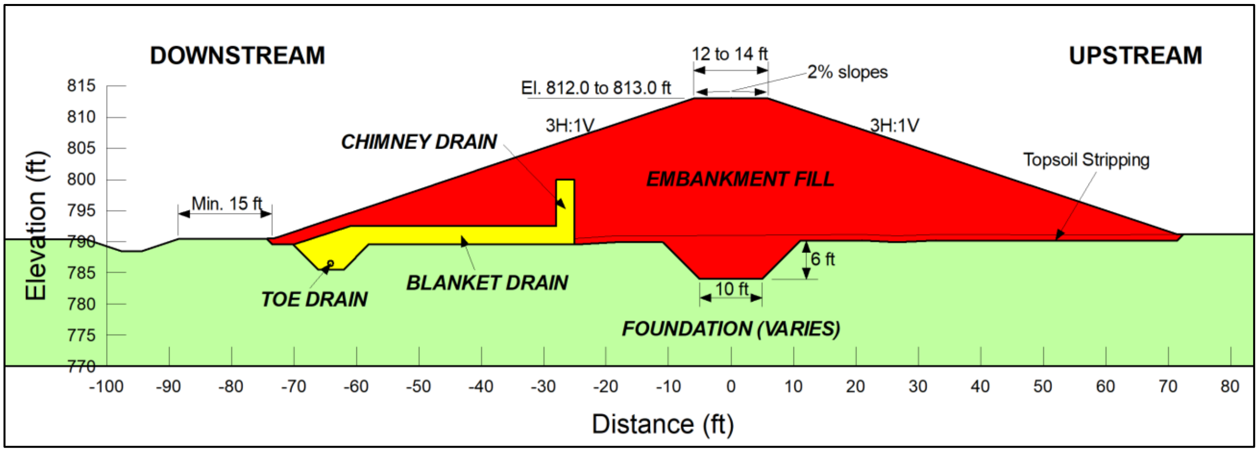

Figure 8. Typical Embankment Cross Section

5.2 DESIGN OBJECTIVES

The objective of the dam embankment design is to provide a stable, seepage and settlement resistant, cohesive alignment that is able to provide storage capacity for up the 1% (100-year) ACE storm event on Eagle Creek and to safely pass flood events of greater magnitude. The embankment will provide enough freeboard to safely pass 100% of the PMF discharge while accounting for wind and wave runup and additional hydrologic uncertainty.

5.3 ALIGNMENT

5.3.1 Design Assumptions

In addition to the design flood event, the design considered the following qualitative criteria provided by project stakeholders:

- Reduce the footprint of the storage facility,
- Reduce the number of parcels impacted by construction,
- Reduce the number of structures impacted by construction,
- Reduce the acreage of agricultural land impacted by construction,
- Reduce the risk of flooding to structures and roadway crossings upstream and downstream of the basin, and
- Incorporate cost saving considerations during the design process.

MWCD provided Stantec with guidance on the selected general alignment of the proposed Flood Basin embankment at the onset of the Preliminary Design phase, based on existing purchase agreements and assumed property acquisitions. From the general alignment provided by MWCD, Stantec then proceeded to lay out the details of the embankment in relation to private property lines, existing roadways, and the necessary exterior drainage ditch. The general layout for the dam alignment was such that a 15-foot minimum exterior maintenance bench is set at the downstream embankment toe.

Downstream of the maintenance bench, exterior drainage channels were sized to convey the 4% ACE (25-year) storm event and checked to verify the 1% ACE (100-year) storm did not encroach on the exterior maintenance bench. See Section 5.4 for additional details related to exterior drainage.

5.3.2 Area-Capacity-Elevation Data

The area-capacity-elevation data was calculated based on the dam alignment as shown in Figure 9. The terrain is based on LiDAR data collected in 2016 as part of the HCFRR Program by Kucera International. Project features such as interior drainage grading, borrow pits, and proposed wetlands were incorporated into the design surface.

The cumulative design storage volume within the dry reservoir is 6,945 acre-feet (2.26 billion gallons) at an elevation of 807.0 feet (normal pool elevation at Auxiliary Spillway crest). A summary of the stage-storage relationship for the reservoir is provided in Table 18 and key dam design elevations highlighted in Table 19. A plot of the reservoir stage-storage curve is presented as Figure 10.

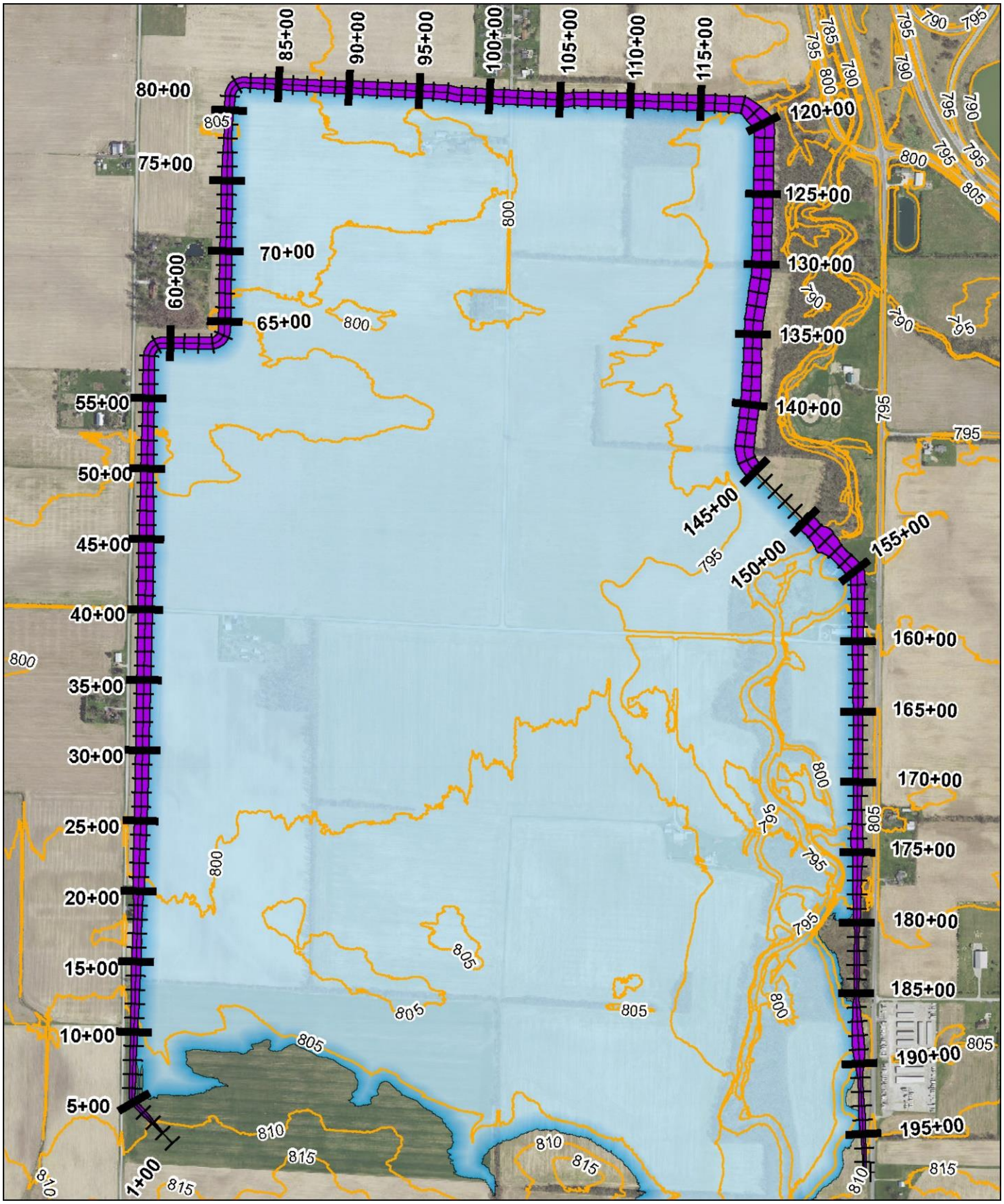

Figure 9. Dam Embankment Alignment

Table 18. Reservoir Stage-Storage Curve

Stage (ft)	Total Storage (ac-ft)	Stage (ft)	Total Storage (ac-ft)
784.0	0	798.0	845
785.0	1	799.0	1,246
786.0	2	800.0	1,723
787.0	4	801.0	2,284
788.0	7	802.0	2,922
789.0	12	803.0	3,620
790.0	18	804.0	4,375
791.0	28	805.0	5,191
792.0	43	806.0	6,051
793.0	63	807.0	6,945
794.0	107	808.0	7,872
795.0	183	809.0	8,834
795.5	234	810.0	9,839
796.0	310	811.0	10,889
796.5	413	812.0	11,984
797.0	533	813.0	13,122

Table 19. Dam Design Elevations

Feature	Elevation (Feet, NAVD88)	Notes
Auxiliary Spillway Crest	807.0	Set at the elevation of the 1% Annual Chance Exceedance (100-Year) event.
PMF Maximum Water Surface Elevation (WSE)	810.0	Maximum WSE above the Auxiliary Spillway during the Probable Maximum Flood (PMF) event.
Dam Crest	Varies from 812.0 to 813.0	maximum PMF WSE + calculated wind/wave run-up + hydrologic uncertainty.

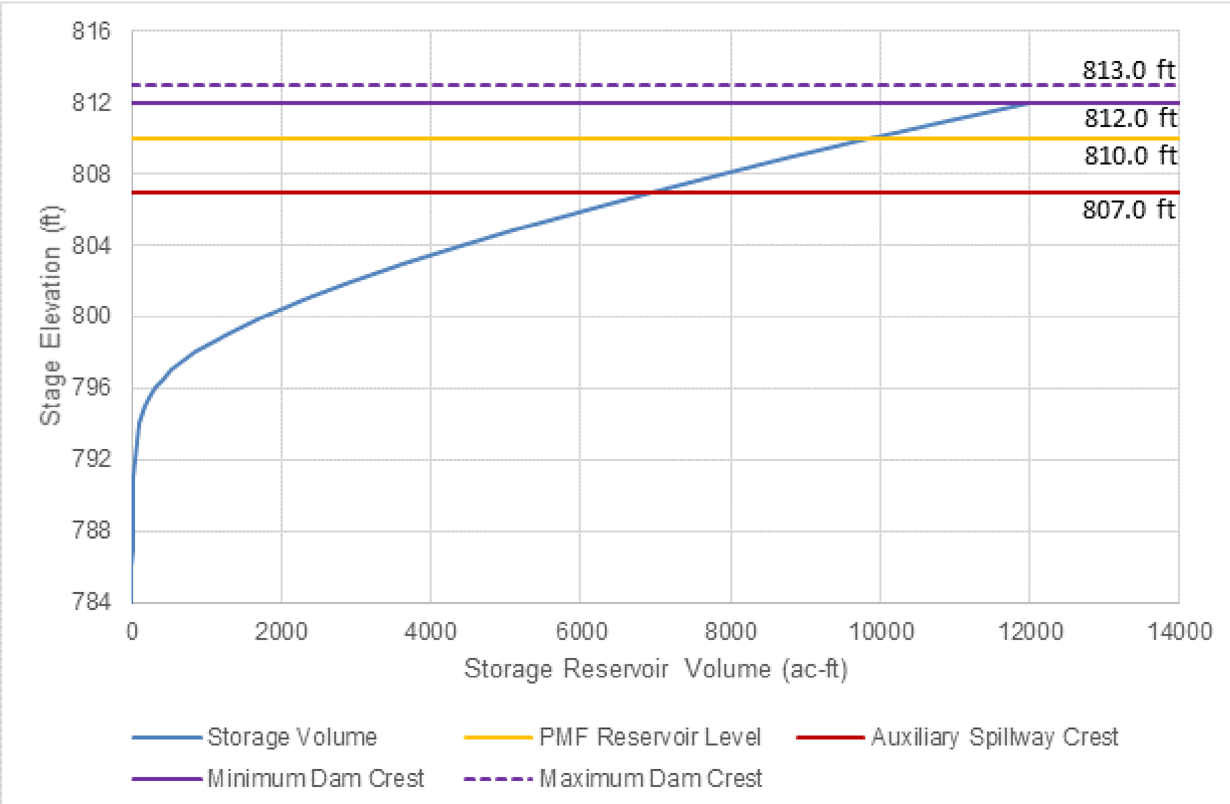

Figure 10. ECFB Reservoir Stage-Storage Curve

5.4 EXTERIOR DRAINAGE

Construction of the dam embankment will modify existing watershed drainage paths. Flow will need to be captured and conveyed away from the dam embankment to reduce the risk of ponded water along the embankment toe. Proposed ditches, conduits, and culverts are designed to convey the runoff along the exterior toe of the dam to a suitable location without impacting the dam embankment or adjacent roadways for specified storm events. The exterior drainage ditches are positioned such that the top of bank is typically 25 feet away from private property lines and at least 8 feet away from existing edge of pavement (Township Road 76 and US-68). Placement of ditches within existing road right-of-way was confirmed after consultation with the Hancock County Engineer and the Eagle Township Trustees.

Existing conditions drainage on the project site flows to either Aurand Run or Eagle Creek. Pre- and post-project conditions were analyzed to confirm that the post-project peak discharges entering Aurand Run and the Aurand Run ditch were equal to or less than the existing flow rates at the same locations.

5.4.1 Open Channel Design Approach

The exterior drainage ditches, located along the north and west sides of the dam embankment, are designed to convey the 4% ACE (25-year) flood event. The trapezoidal channels are lined with grass on both the channel bottom and the 3H:1V side slopes. The ditches have varying bed slopes, typically ranging between 0.2% and 0.3%, with the minimum slope of 0.16%. The relatively flat slopes are due to the minimal grade change along existing terrain. The ditches were checked against the 1% ACE storm event to confirm that the exterior maintenance benches were above the channel WSEs.

5.4.2 Dual Drainage Design Approach

The exterior drainage corridor located along the east side of the eastern dam embankment and parallel to US-68, was sized with a dual-drainage design consisting of a 24-inch diameter storm sewer underneath an open channel ditch. The 24-inch storm is designed to accept flow from an existing 20-inch drain tile at the upstream end of the project site at US-68.

An inlet structure is located at the upstream end of the corridor and manholes with grated inlets are typically located every 300 feet along the corridor. These structures allow flow to enter the storm sewer. Once the capacity of the storm sewer is exceeded, the water flows through the drainage ditch designed to convey the remainder of the flow. The combined eastern drainage ditch and interconnected storm sewer were designed to meet freeboard requirements for the 4% ACE event. The drainage ditches and culverts were checked with the 1% ACE event to mitigate potential water on the exterior maintenance bench.

The 24-inch conduit is approximately 3,714 feet long at a slope of between 0.30% and 0.32% and will be constructed of pre-cast concrete. Backfill for the storm sewer will utilize low-strength mortar (LSM) to reduce the potential for a seepage corridor along the exterior toe. The overflow ditch is trapezoidal and lined with a 6-inch section of uniform section fabric lining on the bottom and grass on the 3H:1V side slopes. The ditch has varying bottom widths and slopes, with some sections as flat as 0.2%.

Two existing box culverts convey flow from east of US-68 to Eagle Creek. These culverts were checked with the 4% ACE (25-year) storm event to verify that the proposed exterior drainage ditches did not increase flooding upstream of US-68. Both of the existing culverts analyzed were headwater controlled and the proposed ditch did not result in negative effects upstream. These culverts will not be modified as part of the project.

5.4.3 Proposed Culverts

Three existing culverts under Township Road 76 were checked with the 4% ACE (25-year) event. The existing culverts were not large enough to convey the design flow without the roadway overtopping. Therefore, a 9-foot by 4-foot box culvert is proposed to convey the 4% ACE (25-year) event without overtopping TR-76. This proposed culvert will convey flows from the northwest and southwest drainage ditches to the existing Aurand Run ditch. The culvert does not require a road raise of TR-76, but the road will be re-paved in this area due to the removal of TR-49 and required modifications of TR-76. The re-paved road will accommodate the culvert dimensions and will be constructed directly above the culvert.

Three new culverts are required at dam access points that cross the exterior drainage ditch: southwest access, north access, and Township Road 49 Principal Spillway access. A 5-foot by 3-foot box culvert at a 0.2% slope is designed for the southwest access crossing. A 112-foot long, 30-inch diameter culvert at a 0.3% slope is designed under the north access cul-de-sac crossing. A 16-foot by 4-foot three-sided concrete culvert at a 0.54% slope is designed for the Township Road 49 Principal Spillway access crossing. These culverts are designed to pass the 4% ACE (25-year) event and were checked with the 1% ACE (100-year) event to confirm no significant overtopping of roadways occur.

The Exterior Drainage Analysis Report is included in Appendix H.

5.5 FREEBOARD

The reservoir stage for the 1% ACE (100-year) event was used to develop the Auxiliary Spillway crest elevation of 807.0 feet. Once the water surface elevation rises above 807.0 feet, the Auxiliary Spillway will activate, and flows will be routed through the spillway and into Eagle Creek downstream. The maximum reservoir stage of 810.0 feet for the PMF event was used to develop the embankment crest elevation considering freeboard.

5.5.1 Freeboard Criteria

The freeboard was calculated in coordination with ODNR Dam Safety. The Ohio Administrative Code (OAC) 1501: 21-13-07 “Freeboard Requirements for Dams” states: “Sufficient freeboard shall be provided to prevent overtopping of the top of the dam due to passage of the design flood and other factors including, but not limited to, ice and wave action.” The United States Bureau of Reclamation (USBR) Design Standards No. 13, Embankment Dams (USBR 2021) states that for new embankment dams, “the design crest is selected as the higher of either: (1) the maximum reservoir water surface (MRWS) elevation plus 3 feet or (2) the MRWS plus the runup and setup that would be generated by a wind with a 10-percent probability of exceedance.” Because the dimensions of the ECFB embankment are not typical of a USBR facility, it was determined that the minimum 3-foot freeboard criteria is not reasonable in this case. The minimum freeboard criteria uses a direct approach focused on potential failure modes and overall project risk. The approach to freeboard methodology was approved by ODNR on 10/18/2022. The freeboard methodology and calculations are discussed in detail in the technical memorandum “Eagle Creek Flood Basin – Freeboard Determination” dated October 24, 2022, and is included as Exhibit A to Appendix G (Dam Embankment Design Report). A summary of the analysis is presented in the following section.

5.5.2 Freeboard Analysis

Wave runup and wind setup calculations were performed according to USBR Design Standards No. 13, Embankment Dams with a few adjustments. Because the Eagle Creek Flood Basin does not maintain a pool under normal conditions, the 100-year event maximum WSE is treated as the normal reservoir water surface (NRWS) for the purposes of freeboard calculations. Also, the maximum recorded wind speed was applied rather than a 100-mph wind velocity. Recorded overland wind speeds were adjusted to represent over-water wind speeds using the ratio of wind speed over water to wind speed over land as a function of wind speed over land (R_L), following guidance in USBR Design Standards No. 13., Embankment Dams. The required freeboard was analyzed under two conditions: normal operation (100-year elevation) with maximum wind speed, and PMF operation with a typical wind speed. Wind speeds used in the calculations are based on historical data from the Findlay Airport with a period of record from 1/1/1973 to 6/27/2022. The wind speeds used in the freeboard analysis are summarized in Table 20.

Table 20. Wind Speed

Reservoir Level Simulation	Parameters	Overland Velocity (mph)	Adjusted Overland Velocity (mph)	R_L (based on figure B-2 from USBR Design Standards)	Over-water Velocity (mph)
MRWS	Hourly 2-minute wind speed with 10% chance of exceedance	17.0	*20.0	1.2	24.0
NRWS	Highest recorded 2-minute wind speed in 50-year record (mph)	64.0	64.0	0.9	57.6

* Overland Velocity associated with Hourly 2-minute Wind Speed increased by 15% to correlate to parallel dataset

Wave runup and wind setup calculations were performed at multiple locations along the length of the ECFB dam by computing multiple fetch length and depth values. The multiple calculations provide justification for variation of the dam crest. The locations of the fetch calculations are presented in Figure 11. In addition to wave runup and wind setup calculations, hydrologic uncertainty was accounted for during design. Stantec considered the potential for future changes in hydrology of the watershed and its impact on the 1% ACE and PMF reservoir levels.

A summary of freeboard calculations at various locations along the ECFB dam is presented in Table 21. The computed dam crest elevation was rounded up to the nearest half-foot increment which results in a design dam embankment crest elevation of 812.5 feet north of Township Road 49 and 812.0 feet south of Township Road 49. At the embankment that tie-in with the Principal Spillway abutment, the crest increases to an elevation of 813.0 feet. Exhibit A to Appendix G includes a calculation package for the dam crest calculations.

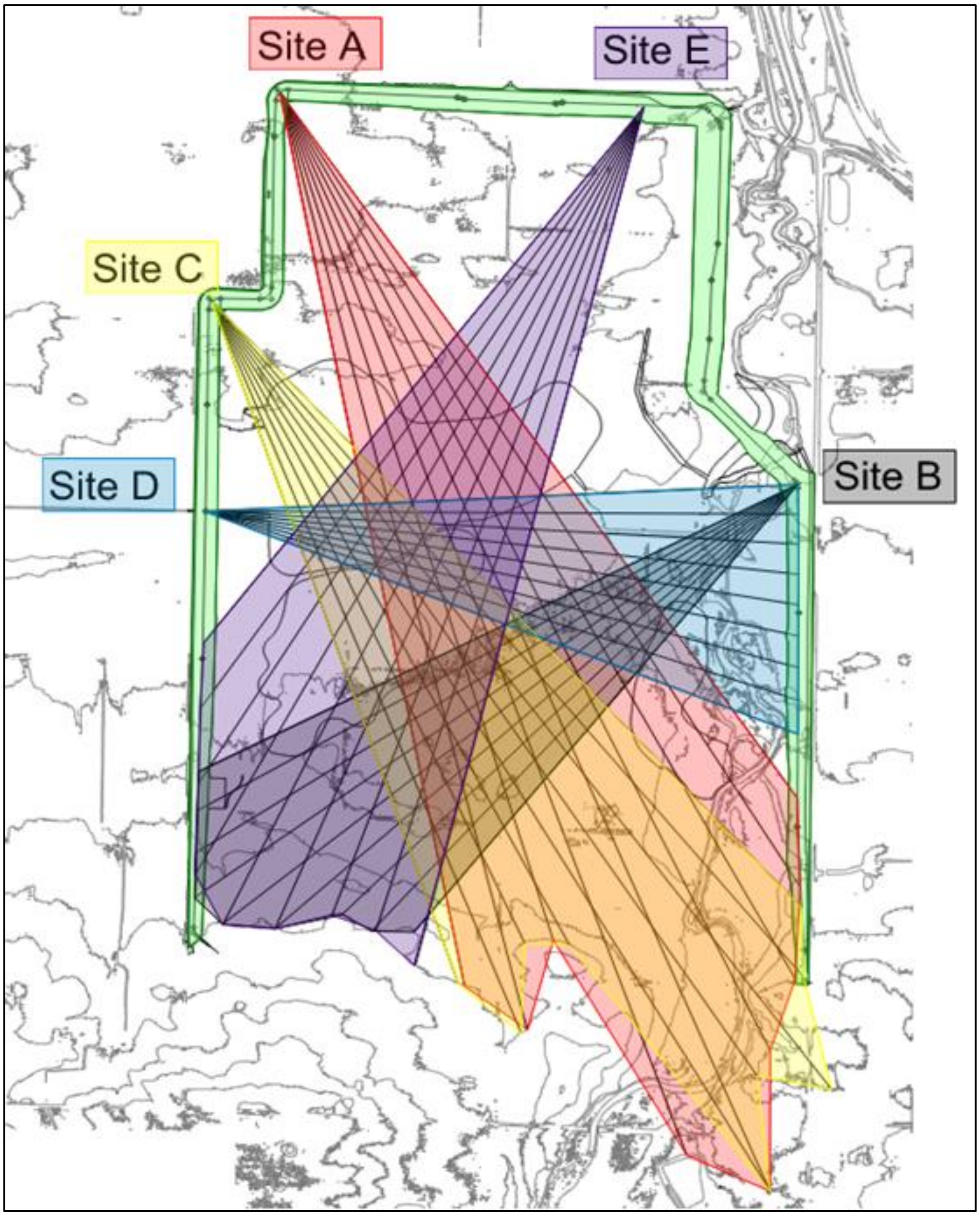

Figure 11. Locations of Fetch Analysis

Table 21. Dam Crest Calculations Summary

Parameter	Fetch A		Fetch B		Fetch C		Fetch D		Fetch E	
	Normal	PMF	Normal	PMF	Normal	PMF	Normal	PMF	Normal	PMF
Reservoir Water Surface (feet)	807.0	810.0	807.0	810.0	807.0	810.0	807.0	810.0	807.0	810.0
Wind and Wave Calculation (feet)	4.5	1.8	3.4	1.4	4.0	1.6	3.2	1.3	4.0	1.6
Hydrologic Uncertainty (+20% additional precipitation)	0.65	0.50	0.65	0.50	0.65	0.50	0.65	0.50	0.65	0.50
Total Freeboard (feet)	5.1	2.3	4.1	1.9	4.7	2.1	3.9	1.8	4.7	2.1
Calculated Dam Crest Elev. (Reservoir WSE + Freeboard) (feet)	**812.1**	**812.3**	**811.1**	**811.9**	**811.7**	**812.1**	**810.9**	**811.8**	**811.7**	**812.1**

5.6 STABILITY

The embankment slopes are designed to meet the target factors of safety for slope stability according to USACE guidance and the project Design Criteria Document (Appendix A). Geotechnical analyses for slope stability considered the location and depth of the exterior ditches when evaluating slope stability cases. Slope stability was evaluated using Spencer's limit equilibrium method as implemented in the GeoStudio SLOPE/W 2018 R2 software.

5.6.1 Load Cases and Acceptance Criteria

The USACE provides guidance for analyzing the stability of slopes of new earth dams in EM-1110-2-1902 (USACE, 2003). This guidance is followed for static analyses (Case No. 1 through 5). The factors of safety for the pseudo-static analyses (Case No. 6) are based on Hynes-Griffin and Franklin (1984). Table 22 provides the load cases considered and the required minimum factors of safety.

Table 22. Minimum Required Slope Stability Factors of Safety

Case No.	Load Case	Required Minimum Factor of Safety	Pore Pressure Conditions and Shear Strength Parameters	Slope
1	End of Construction (No Pool)	1.3	Pore pressures for normal groundwater level[(1)], total stress (undrained) shear strengths	Upstream and Downstream
2	Long-term (Normal Pool = No Pool)[(1)]	1.5	Pore pressures for normal groundwater level[(1)], effective stress (drained) shear strengths	Downstream
3	Flood (Maximum Headwater/Tailwater Differential = 100-Year Flood)[(2)]	1.4	Pore pressures for 100-Year Flood[(3)] water level, effective stress (drained) shear strengths	Downstream
4	Flood (Maximum Headwater Elevation = PMF)	1.4	Pore pressures for normal groundwater level[(1)], flood surcharge to maximum headwater elevation, total stress (undrained) shear strengths	Downstream
5	Rapid Drawdown (Maximum Differential to No Pool)	1.3	Pore pressures for 100-Year Flood[(3)] water level, drawdown to no pool, rapid drawdown strengths	Upstream
6	Pseudo-Static (k_h = 0.5*PGA) (Normal Pool = No Pool)[(1)]	1.0	Pore pressures for normal groundwater level[(1)], undrained seismic strengths	Upstream and Downstream

[(1)] The dam does not retain a pool under normal conditions; the normal water level is assumed to equal the current groundwater levels at the site.
[(2)] The maximum head differential occurs during the 100-year flood, as the tailwater rises above the dam toe during higher inflow events.
[(3)] The 100-year flood is conservatively assumed to reach steady-state conditions for analysis purposes.

5.6.2 Analysis Cross Sections

Twelve cross sections were selected to represent the conditions in various reaches along the dam alignment for the design seepage and stability analyses. Details of the twelve cross sections are summarized in Table 23. Figure 12 shows the selected analysis cross section locations. The subsurface profile at each cross section was developed using available boring information and the 3-D lithological model developed from the hydrogeological and geological review (See Appendix E).

Table 23. Summary of Analysis Cross Sections

Station	Approx. Representative Station Range	Approx. Embankment Height in Station Range (ft)(1)	Modeled Embankment Height (ft)(1)	Boring(s) Considered for Subsurface Profile	Final Design Seepage Control and Drainage Features
40+15	2+07 to 47+13	0 to 16	16	B-3.11	Chimney, Blanket, and Toe Drains(2)
69+75	47+13 to 82+60	7 to 13	12	B-3.20	None
92+70	82+60 to 92+78	11 to 14	14	B-3.24 through B-3.27	Toe Drain
100+75	92+78 to 107+50	13 to 15	15	B-3.30 through B-3.32	Chimney, Blanket, and Toe Drains
114+15	107+50 to 118+00	13 to 17	16	B-2.10 and B-3.35	Chimney, Blanket, and Toe Drains, Vertical Sand Drains
118+85	118+00 to 119+38	17 to 21	21	B-3.36 through B-3.38	Chimney, Blanket, and Toe Drains, Vertical Sand Drains
126+30	119+38 to 133+00	19 to 23	21	B-3.41	Chimney, Blanket, and Toe Drains, Vertical Sand Drains
139+40	133+00 to 145+05	18 to 21	21	B-3.43 through B-3.45	Chimney, Blanket, and Toe Drains, Vertical Sand Drains
152+00	149+72 to 156+41	14 to 27	27	B-3.48 through B-3.53	Chimney, Blanket, and Toe Drains, Sheet Pile Cutoff
157+65	156+41 to 174+56	8 to 14	13	B-3.85	Toe Drain, Vertical Sand Drains
176+20	174+56 to 185+56 and 190+66 to 197+40	2 to 13	9	B-3.57 through B-3.59 and B-4.18 through B-4.20	Toe Drain, Sheet Pile Cutoff(2)
189+00(3)	185+56 to 190+66	6 to 13	12	B-3.60	Toe Drain

(1) Embankment height is measured from the existing ground surface

(2) Parametric seepage analyses were also performed considering no seepage control and drainage features to represent varying design segments

(3) This cross section evaluated for seepage only

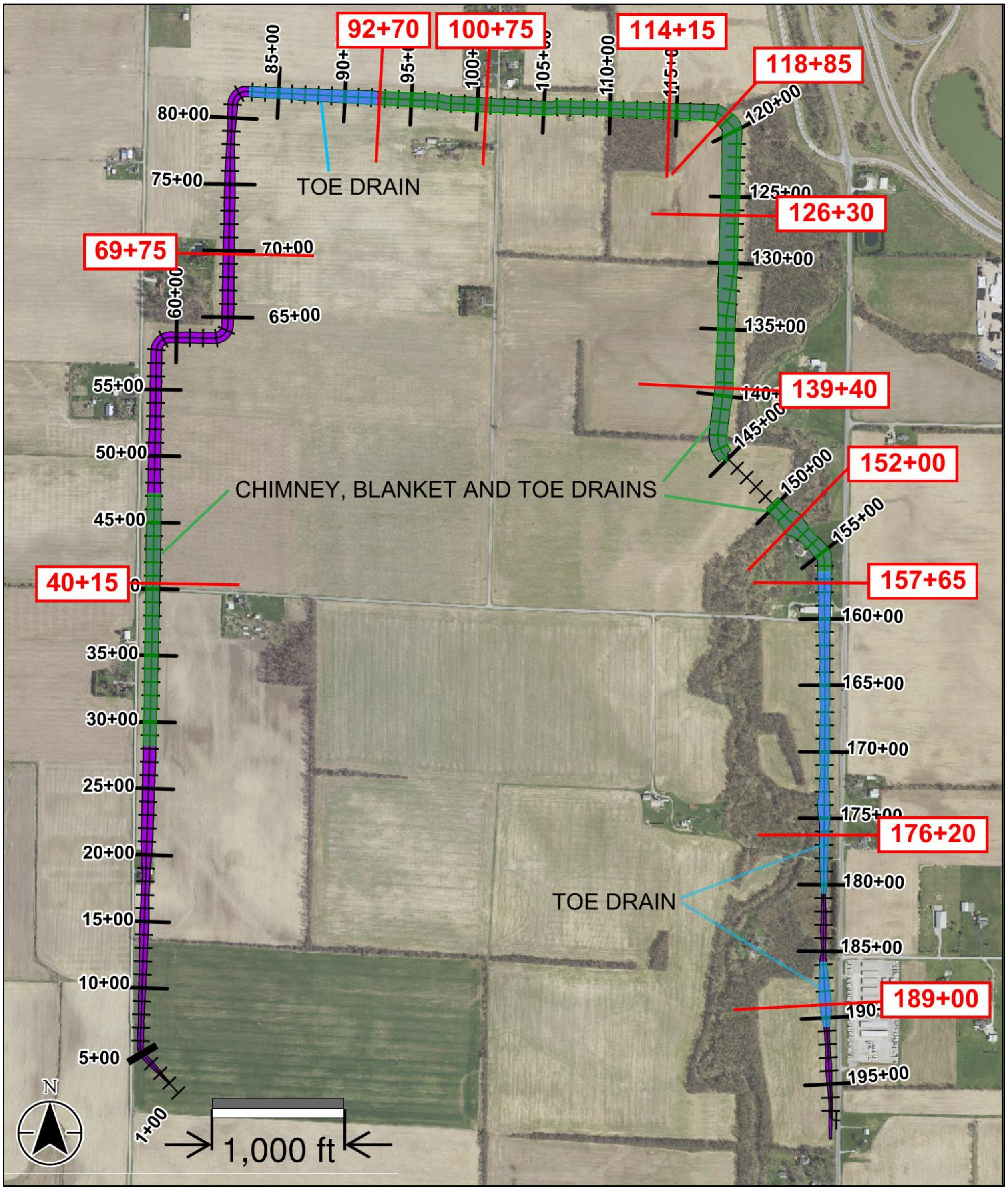

Figure 12. Analysis Cross Section Locations

5.6.3 Stability Results

Table 24 summarizes the results of the slope stability analyses for the evaluated conditions. The slope stability analyses considered a global search for failure surfaces that encompass the full width of the dam crest, partial width of the crest, and shallow failure surfaces. The minimum factors of safety from the various searches are reported in Table 24 and are well above acceptance criteria. The high factors of safety indicate that steeper embankment slopes may be stable; however, the 3H:1V design side slope are required for access, monitoring, and maintenance.

The pore pressures used for the stability analyses in the sections at Station 152+00 and Station 176+20 are based on the seepage analysis results that model a sheet pile seepage cutoff. No strength was given to the sheet pile wall for the stability analyses. The pore pressures used for the stability analyses in the sections which model vertical sand drains are based on the seepage analyses that model the vertical sand drains.

Slope stability analysis results are included within Exhibit H of the Geotechnical Design Report (Appendix E).

Table 24. Slope Stability Analysis Results

Case No.	1		2	3	4	5	6	
Load Case[1]	End of Construction		Long-Term	Flood – Max. Differential	Flood – Max. Headwater	Rapid Drawdown	Pseudo-Static	
Analyzed Conditions	No pool, undrained strengths		No pool, drained strengths	100-year pool, drained strengths	PMF pool, undrained strengths	Max. Diff. to No Pool, rapid drawdown strengths	No pool, undrained seismic strengths	
Slope	D/S	U/S	D/S	D/S	D/S	U/S	D/S	U/S
Required Factor of Safety	1.3	1.3	1.5	1.4	1.4	1.3	1.0	1.0
40+15	2.1	2.1	2.1	2.1	2.1	1.8	1.7	1.9
69+75	2.1	2.2	2.1	2.1	2.1	1.9	2.0	2.0
92+70	2.1	2.1	2.1	2.1	2.1	1.9	1.8	2.0
100+75	2.1	2.1	2.1	2.1	2.1	1.7	1.7	1.9
114+15	2.1	2.1	2.1	2.1	2.1	1.7	1.7	1.9
118+85	2.1	2.1	2.1	2.1	2.1	1.7	1.8	1.9
126+30	1.8	2.1	2.0	2.0	2.1	1.7	1.7	1.9
139+40	1.9	2.1	2.0	2.0	2.2	1.7	1.7	1.9
152+00	2.0	2.1	2.0	2.0	2.2	1.8	1.9	2.0
157+65	2.2	2.2	2.2	2.2	2.2	1.9	2.0	2.0
176+20	2.5	2.5	2.5	2.5	2.5	2.4	2.3	2.3

Note: Cells that are shaded green indicate that factors of safety met acceptance criteria.

5.7 SEEPAGE

A numerical seepage model was utilized to estimate pore pressures for the seepage exit factor of safety calculations and the slope stability analyses. Seepage analyses were completed using GeoStudio SEEP/W 2018 R2, finite element software. The embankment seepage control features (drains, cutoffs) are designed to meet the target factors of safety for exit gradients according USACE and USBR guidance and the project Design Criteria Document (Appendix A). Geotechnical analyses for seepage considered the location and depth of the exterior ditches when evaluating seepage cases.

5.7.1 Acceptance Criteria

Factors of safety are used to assess soil stability in areas where seepage water exits near the ground surface. Two definitions can be used:

- $FS_{exit-SF}$ = factor of safety for soil stability at a seepage exit, computed using the seepage force method. For the design and assessment of dams, USACE (1986) criteria for soil heave are based on the seepage force definition ($FS_{exit-SF}$) and are applicable to all soil types.
- $FS_{exit-TS}$ = factor of safety for soil stability at a seepage exit, computed using the total stress method. USBR (2014) describes assessing uplift (heave) of a confining soil layer (where more permeable material underlies less permeable material) using the total stress definition ($FS_{exit-TS}$).

Calculated factors of safety were compared to the acceptance criteria at multiple points along the downstream toe of the dam embankment and spillway structures, including at the bottom of exterior ditches and the toe of proposed fill locations. The project acceptance criteria for exit gradient factors of safety using both methods are summarized in Table 25.

Table 25. Project Acceptance Criteria for Exit Seepage

Location	Required Minimum Seepage Force Factor of Safety ($FS_{exit-SF}$)	Required Minimum Total Stress Factor of Safety ($FS_{exit-TS}$)
Toe of Dam	3.0	2.0
Toe of Exterior Bench and Bottom of Exterior Ditch	1.5	1.5

5.7.2 Seepage Control Measures

Initial geometry criteria (embankment crest width and side slopes) were provided by MWCD for maintenance considerations. Subsequently, the chimney, blanket, and toe drains were added to the design as documented in the Dam Embankment Design Technical Memorandum (Appendix G).

The top of filter elevation (800 feet) in the internal drainage system was selected to intercept seepage that might otherwise exit on the face of the embankment (as predicted by seepage analyses). The top of filter elevation also considered the potential for internal erosion of the embankment fill through cracks. The chimney and blanket drains shown in the analysis cross sections are included whenever the base of the embankment is below elevation 800 feet.

Next, locations along the embankment alignment that require a toe drain were considered. The toe drain is integrated with the chimney and blanket drains to provide collection and a filtered exit for collected seepage. Additional reaches of the dam include a toe drain (without internal drains) to facilitate seepage collection and reduce uplift pressures associated with underlying alluvial, coarse-grained soils. The designed drains were then incorporated into the analysis cross sections to evaluate seepage models for the design embankment geometry.

Additional seepage control measures outlined in EM 1110-2-1901 (USACE, 1986) were considered for the embankment, principal spillway, and auxiliary spillway. Seepage control measures, where required, were designed to meet the exit gradient factors of safety. Vertical sand drains and sheet pile walls are recommended along various portions of the dam alignment based on the project site and results of the seepage analyses. The locations of these seepage control measures are shown in Figure 13.

Due to low preliminary exit gradient factors of safety calculated in several cross sections, vertical sand drains will be installed under the toe drain along portions of the embankment. The vertical sand drains were designed in accordance with the guidance found in EM 1110-2-1914 (USACE, 1992) and will consist of 2-foot diameter bored holes filled with Filter Sand, which will extend from below the toe drain to the top of bedrock. The vertical sand drains will be spaced 25 feet apart (center-to center). Vertical sand drains are included along the following station ranges:

- Station 107+33 to 144+58
- Station 156+41 to 174+66

PVC sheet pile walls will be installed to create seepage cutoffs under select portions of the embankment. A sheet pile wall driven through the inspection trench to the top of bedrock is included in the following station ranges:

- Station 149+96 to 156+41
- Station 174+56 to 180+61

A sheet pile wall will be built in the foundation of the combined spillway structure, from Station 144+67 to 149+96.

The locations and extents of the proposed vertical sand drains and sheet pile walls are shown in Figure 13.

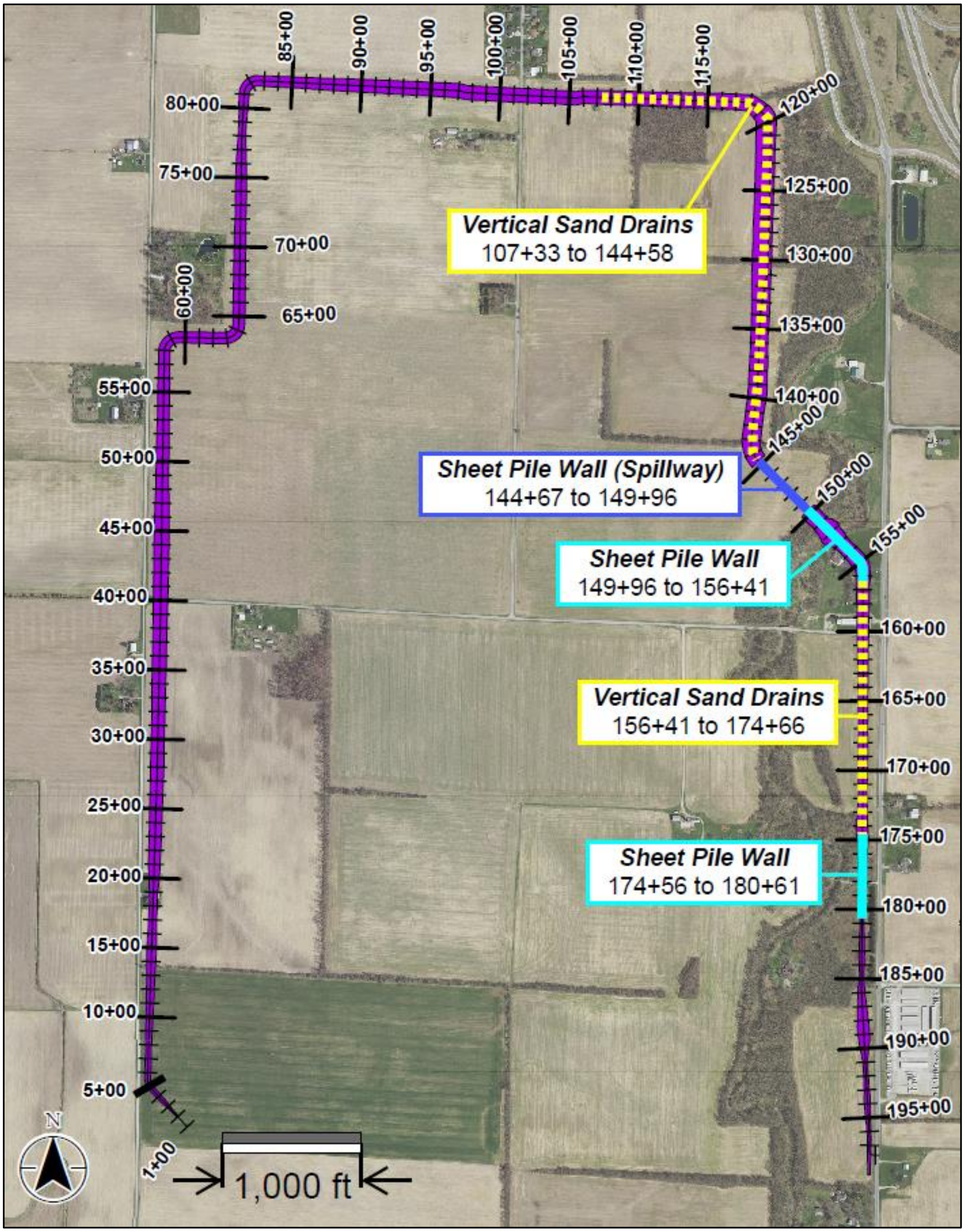

Figure 13. Seepage Control Measures (Vertical Sand Drains and Sheet Pile Walls)

5.7.3 Analysis Results

Factors of safety for exit seepage at the downstream toe of the dam were calculated at the 12 analysis sections. The results of the exit seepage factor of safety calculations are summarized in Table 26. Seepage analysis results are included within Exhibit G of the Geotechnical Design Report (Appendix E).

Table 26. Seepage Exit Analysis Results

Section	Location	Calculated Seepage Force Factor of Safety ($FS_{exit-SF}$)	Required Minimum Seepage Force Factor of Safety	Calculated Total Stress Factor of Safety ($FS_{exit-TS}$)	Required Minimum Total Stress Factor of Safety
40+15	Bottom of Exterior Ditch	3.5	1.5	N/A	N/A
	Toe of Dam	21.3	3.0	N/A	N/A
69+75	Bottom of Exterior Ditch	N/A[1]	1.5	2.8	1.5
	Toe of Dam	N/A[1]	3.0	3.5	2.0
92+70	Bottom of Exterior Ditch	N/A[1]	1.5	2.2	1.5
	Toe of Dam	N/A[1]	3.0	2.6	2.0
100+75	Bottom of Exterior Ditch	4.9	1.5	1.7	1.5
	Toe of Dam	15.5	3.0	2.6	2.0
114+15 (Vertical Sand Drains)	Bottom of Exterior Ditch	2.1	1.5	1.6	1.5
	Toe of Dam	31.9	3.0	2.5	2.0
118+85 (Vertical Sand Drains)	Bottom of Exterior Ditch	14.7	1.5	1.6	1.5
	Toe of Dam	34.4	3.0	3.0	2.0
126+30 (Vertical Sand Drains)	Toe of Fill	10.3	1.5	1.8	1.5
	Toe of Dam	18.7	3.0	2.1	2.0
139+40 (Vertical Sand Drains)	Toe of Fill	16.1	1.5	1.6	1.5
	Toe of Dam	24.5	3.0	2.0	2.0
152+00 (Sheet Pile Wall)	Toe of Fill	7.2	1.5	N/A	N/A
	Toe of Dam	146.6	3.0	2.6	2.0
126+30 (Vertical Sand Drains)	Bottom of Exterior Ditch	2.5	1.5	1.5	1.5
	Toe of Dam	60.3	3.0	3.9	2.0
176+20 (Sheet Pile Wall)	Bottom of Exterior Ditch	N/A[1]	1.5	9.6	1.5
	Toe of Dam	N/A[1]	3.0	29.8	2.0
189+00 (Vertical Sand Drains)	Bottom of Exterior Ditch	50.4	1.5	2.2	1.5
	Toe of Dam	N/A[1]	3.0	4.1	2.0

[1] Seepage gradients are negative. The seepage is thus downward, and piping/heave is not expected.

Note: Cells that are shaded green indicate that factors of safety met USACE (1986) recommendations. Cells shaded red indicate that factors of safety were below the recommendations.

5.8 SETTLEMENT

Settlement analyses are conducted in general accordance with USACE EM 1110-1-1904 (USACE, 1990). Compressibility parameters were established from laboratory consolidation test results. Vertical stresses were computed based on Boussinesq equations using ranges in Poisson's ratio consistent with the soil materials encountered. Primary consolidation was calculated. Secondary consolidation, which is typically 2% to 5% of total settlement in over-consolidated soils, was neglected in the settlement calculations. Settlement calculations were conducted using spreadsheets or Settle3 software by Rocscience.

5.8.1 Acceptance Criteria

Embankments were designed with overbuild and camber to maintain project design grades following primary consolidation settlement.

5.8.2 Analysis Results

Settlements were computed along the dam baseline, at the locations of the soil borings, using the embankment fill height. Considering the embankment height and subsurface profile at each boring advanced in the vicinity of the dam alignment, and considering the unit weight and compressibility parameters, a maximum embankment settlement of 5 inches was estimated. A variable overbuild profile is recommended to accommodate the estimated long-term settlements of the embankment. The variable overbuild recommendations are as follows:

- Station 2+07 to 100+00: overbuild embankment 3 inches above design crest elevation
- Station 100+00 to 100+10: transition from 3 to 6 inches overbuild above design crest elevation
- Station 100+10 to 145+05 (Auxiliary Spillway abutment): overbuild embankment 6 inches above design crest elevation
- Station 149+72 (Principal Spillway abutment) to 197+40: overbuild embankment 3 inches above design crest elevation

Figure 14 shows the predicted total settlement at the boring locations and the required overbuild profile of the embankment. Results indicate a gradual variation in settlement along the dam profile and limited concern for differential settlement.

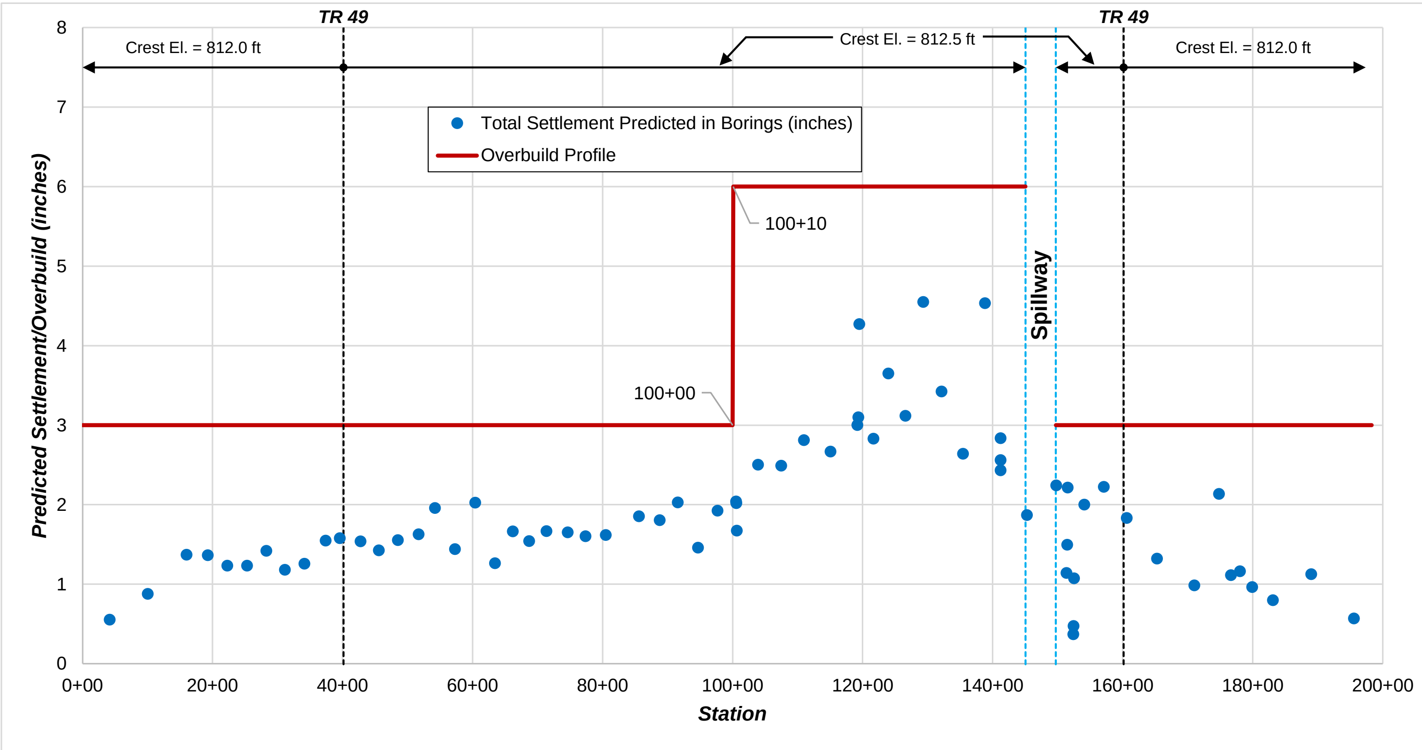

Figure 14. Embankment Settlement Profile

The results of the settlement calculations are provided in Exhibit I of Appendix E.

5.9 FILTER COMPATIBILITY

Filter compatibility was evaluated using the empirical methodology defined by the USACE in EM 1110-2-2300 (USACE, 2004). Table 27 summarizes the filter compatibility calculations. The Filter Sand is used to filter the Embankment Fill (Fill Type 1) and Upper Fine-Grained material. ODOT No. 7 Coarse Aggregate serves as a filter against the Filter Sand where specified on the project design drawings. Detailed calculations are provided in Exhibit J of Appendix E.

Table 27. Filter Compatibility Check

Base Material	Filter Material	Stability Check	Permeability Check	Segregation Check	Additional Filter Checks (See Exhibit J)
Embankment Fill/Upper Fine-Grained Material	Filter Sand	Pass	Pass	Pass	Pass
Filter Sand	ODOT No. 7 Coarse Aggregate ODOT CMS 2019 703.01)	Pass	Pass	Pass	Pass

5.10 LIQUEFACTION TRIGGERING

A screening analysis was performed to determine if the foundation soils are susceptible to liquefaction/cyclic softening during an earthquake event (Section 4.4.11). The results indicate that various foundation materials characterized as sand-like are susceptible to liquefaction along the dam alignment, from approximately Station 62+00 to 176+00. A liquefaction triggering analysis was completed for these susceptible zones of material. Factors of safety against liquefaction were calculated for the 55 borings located near the dam alignment between Station 62+00 and 176+00. Calculations were performed according to the procedures outlined in Exhibit F of Appendix E.

The seismic inputs used for the liquefaction triggering calculations are summarized below:

- Earthquake event with 975-year return period (5 percent chance of occurrence in 50 years)
- Earthquake magnitude of 4.9
- Site-adjusted peak ground acceleration (PGA) at the ground surface = 0.05356 g

A factor of safety against liquefaction greater than 1.4 indicates liquefaction is not expected. At the 55 evaluated boring locations, the minimum calculated factor of safety against liquefaction was 2.2 in Boring B-4.21. Liquefaction is not expected to be triggered during the design earthquake event at the project site. The results of the triggering analyses are provided in Exhibit F of Appendix E.

6.0 PRINCIPAL SPILLWAY

6.1 GENERAL ARRANGEMENT

The Principal Spillway is comprised of a:

- realigned/relocated reach of Eagle Creek upstream of the embankment,
- a cast-in-place concrete structure to control downstream discharge,
- energy dissipation downstream of the structure, and
- a realigned/relocated reach of Eagle Creek downstream of the embankment tying back into the existing Eagle Creek channel.

Figure 15 shows the general arrangement of the Principal Spillway. A new Eagle Creek channel alignment is proposed, directing flow from the existing Eagle Creek (approximately 540 feet downstream of TR 49) to the Principal Spillway structure. This new channel is designed using geomorphic data collected on Eagle Creek, with an emphasis on flow conveyance, sediment transport, and aquatic habitat creation to offset impacts to Eagle Creek and provide future system stability.

The Principal Spillway alignment crosses the embankment approximately 250 feet northwest of the existing Eagle Creek channel, within a realigned reach of new Eagle Creek channel. During construction, normal flow can be maintained through the existing Eagle Creek channel until construction of the Principal Spillway is substantially complete. The proposed spillway construction is off-line of the existing Eagle Creek Channel (away from active flow. The realignment of Eagle Creek simplifies construction phasing and reduces overall construction cost and schedule duration. The overall aquatic use designation of Eagle Creek will be maintained upstream and downstream of the Principal Spillway throughout the construction process.

6.2 DESIGN OBJECTIVES

The Principal Spillway is designed with the primary objective of maximizing flood storage while limiting the water surface elevation (WSE) in the basin to a normal pool elevation of up to 807.0 ft during a 1% ACE (100-year) event.

A secondary objective of the Principal Spillway incorporates a fish passage system. Development of the fish passage design considers conditions in the spillway relative to physiological capabilities and migratory behaviors of relevant fish species as measures to minimize or mitigate for adverse effects on aquatic resources.

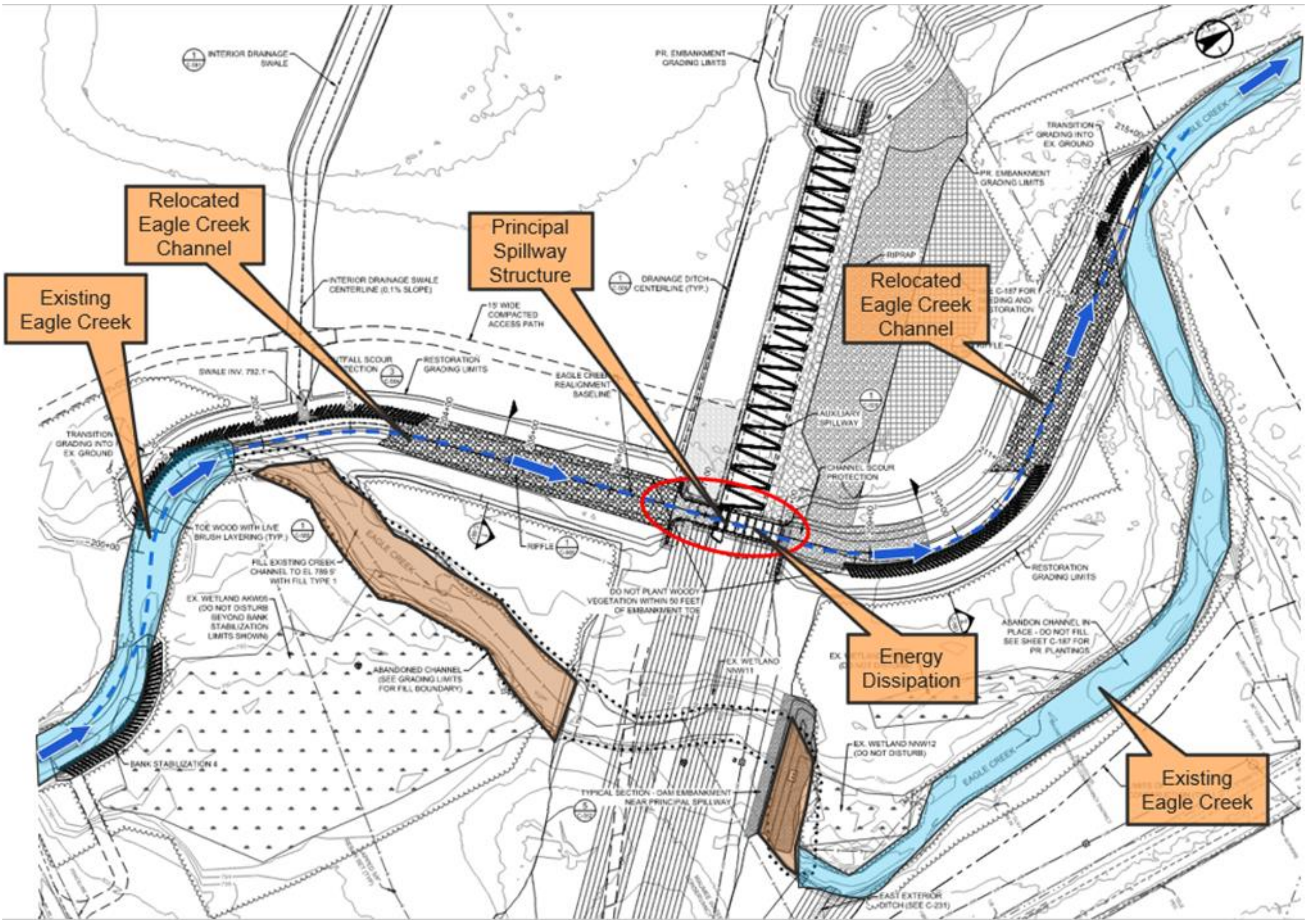

Figure 15. Principal Spillway General Arrangement

6.3 SPILLWAY GEOMETRY

The Principal Spillway configuration consists of an upstream inlet channel, a control wall with orifice openings, a baffled concrete chute, and a downstream outlet channel that ties into the existing Eagle Creek channel. The left side of the Principal Spillway is integrated into the labyrinth weir Auxiliary Spillway and the right-side ties into the adjacent earthen embankment dam. The presence of one integrated structure instead of two reduces risk associated with seepage pathways, one of the potential failure modes identified at the onset of Preliminary Design, by reducing the number of contacts between the embankment and concrete structures.

The upstream control wall includes a 2-ft thick, reinforced concrete headwall and integrated debris rack. The headwall is 26.6-ft tall and 22-ft-wide with two (2) formed rectangular orifices to restrict flow. The openings are both 3-ft high by 9-ft wide rectangular orifices at the invert of the channel’s finished grade (EL. 784.15 ft) with a combined capacity of approximately 1,264 cubic feet per second (cfs) when the upstream headwater is at a full normal pool, elevation 807.0 ft. Two gates are planned to be set to a position such that the two rectangular orifices have openings of 2-ft, 5-inches high by 9-ft wide.

The orifice openings are planned to be kept clear by a sloping debris rack measuring 22 ft wide, 13 ft tall, and 29 ft long, placed between the abutments. Downstream of the wall is a 22 ft wide, flat rectangular concrete chute measuring 80 ft long. The concrete chute contains seven baffle walls spaced 10 ft on center to provide energy dissipation and fish passage. The baffle walls range in height from 1.2 ft to 0.9 ft above the channel bottom to maintain a slope of 0.5% as measured at the invert of the baffle notch. Each baffle wall includes a 0.5 ft deep notch, measuring 1.5 ft wide. The first baffle wall downstream of the control wall and orifice openings is designed with a steel plate on its upstream face to reduce the risk of abrasion.

Design drawings for the Principal Spillway are included in the Final Design Drawings, Appendix B.

As noted, the Principal Spillway will facilitate fish passage. Fish passage refers to the act, process, or science of moving fish over a stream barrier (e.g., dam). A fish passageway or fishway is the combination of elements (structures, facilities, devices, project operations, etc.) necessary to ensure safe, timely, and effective movement of fish past a barrier (16 U.S.C. 811 1994). To facilitate the passage of fish during migratory periods of certain species present in the area, the Principal Spillway provides a minimum flow depth during low flow conditions while not exceeding certain velocities during increased average monthly flows. A detailed analysis of the fish passage concept is included in the document titled, "*Eagle Creek Flood Basin - Aquatic Resource Connectivity Review*" (Stantec, 2022a).

6.4 HYDRAULIC DESIGN

6.4.1 Design Criteria

6.4.1.1 Flood Control Performance

The Principal Spillway is designed to detain excess flood waters during the 100-year storm event to a maximum WSE below 807.0 ft to achieve the desired flood attenuation without activating the Auxiliary Spillway. The maximum downstream design flowrate was determined to be approximately 1,250 cfs at this pool elevation during Preliminary Design. During Final Design, the maximum discharge of the Principal Spillway was calculated to be 1,264 cfs at a WSE of 807.0 ft.

6.4.1.2 Fish Passage

Fish passage design criteria (e.g., depth of flow and velocity) were developed based on the physiological requirements and behaviors of target fish species that may be impacted by impeded migratory pathways in Eagle Creek. Target species were identified using Ohio Environmental Protection Agency (OEPA) fish community survey data from the Blanchard River Watershed and are Channel Catfish, White Sucker, and Smallmouth Bass (OEPA, 2007). To facilitate passage of these species during their primary spawning season of April to June, the velocity throughout the Principal Spillway must:

- maintain a minimum flow depth of 0.37 ft;
- be traversable by a fish with a swim speed of 3.61 ft/s for a distance of 10 ft or at a speed of 2.66 ft/s for a distance of 30 ft; and
- provide sheltered resting areas for fish approximately every 10 ft based on the swim speeds and distances being used in the evaluation.

The goal of the Principal Spillway configuration is to facilitate fish passage 75% of the time during months of migration. This means that ideally the minimum depth will be provided for flows as low as the 90% exceedance flow (1.8 cfs) in the driest month (June), and the maximum velocity will not be exceeded for flows as high as the 15% exceedance flow (147.5 cfs) in the wettest month (April).

6.4.2 Design Approach and Methodology

The Principal Spillway control wall orifices and baffled chute were designed to control flood discharge, dissipate energy and facilitate fish passage through the hydraulic structure. The hydraulics were evaluated using a steady-state Hydrologic Engineering Center River Analysis System (HEC-RAS) model, version 6.1 (USACE, 2021) and a computational fluid dynamics (CFD) model. The HEC-RAS model includes the baffled chute and control wall orifices of the PS. Gate coefficients and roughness coefficients of the HEC-RAS model were calibrated to the results of a CFD model of the Principal Spillway at various discharges. The CFD model analysis is discussed with more detail in the ECFB Hydrologic and Hydraulic Analysis Report, Appendix D.

The HEC-RAS model was used to size the inlet control wall orifices and develop a headwater / discharge rating curve and evaluate multiple discharges and the resulting velocities and flow depths with respect to fish passage. Sizing of the baffles was achieved through an iterative process in conjunction with sizing of the control wall orifices and the tailwater modeled in the downstream channel. The Principal Spillway Final Design Memo in Appendix I further describes the assumptions / parameters that influenced the iterative process.

6.4.3 Control Wall Orifices

The control wall orifices were designed using the HEC-RAS model and CFD models as discussed in Section 6.4.2 and an unsteady HEC-HMS model, version 4.8 (USACE 2021). The HEC-RAS model was used to estimate a velocity and depth of flow through the orifices for evaluating fish passage and to develop a rating curve for reservoir routing. The HEC-HMS model then used the rating curve from HEC-RAS to evaluate the performance during the 100-year flood event to determine whether the resulting WSE maximized the available reservoir storage but remained below 807.0 ft. Sizing of the orifices was achieved through an iterative process between HEC-RAS and HEC-HMS.

The width of each orifice was set at 9 ft to facilitate fish passage. The effective height of the orifices was set at 2.417 ft (2 ft 5 in) to impound water upstream of the dam during the 100-year event to within the desired range of 806.8 ft and 807.0 ft. The control wall orifices will each be fitted with slide gates. Based on calculations, the gates will be set to an opening height of 2.417 ft, however, to add additional operational flexibility, the orifices (and resulting maximum gate opening) were designed to be 3.0 ft high. This added orifice height provides the operator some flexibility to change operations of the structure in the future without structural modifications to the control wall or Auxiliary Spillway.

6.4.3.1 Flood Control Performance (2.417 ft Opening)

The control wall orifice configuration with a gate height of 2.417 ft results in a maximum 100-year WSE of 806.8 ft which is within the desired range and corresponds to a maximum discharge through the Principal Spillway of 1,255 cfs. Figure 16 presents the Principal Spillway rating curve graphically and Table 28 summarizes the control wall / baffled chute rating curve and presents a summary of the HEC-RAS model results. Each of these results are based on gate openings of 2.417 ft. The results of the rating curve calculations show the Principal Spillway discharge increases with increasing upstream WSE until the Auxiliary Spillway activates at an upstream WSE of 807.0 ft. After the Auxiliary Spillway activates, the tailwater on the Principal Spillway increases, causing a reduction in discharge through the control wall orifices. The 100-year hydrograph routing through the ECFB is presented in Figure 17.

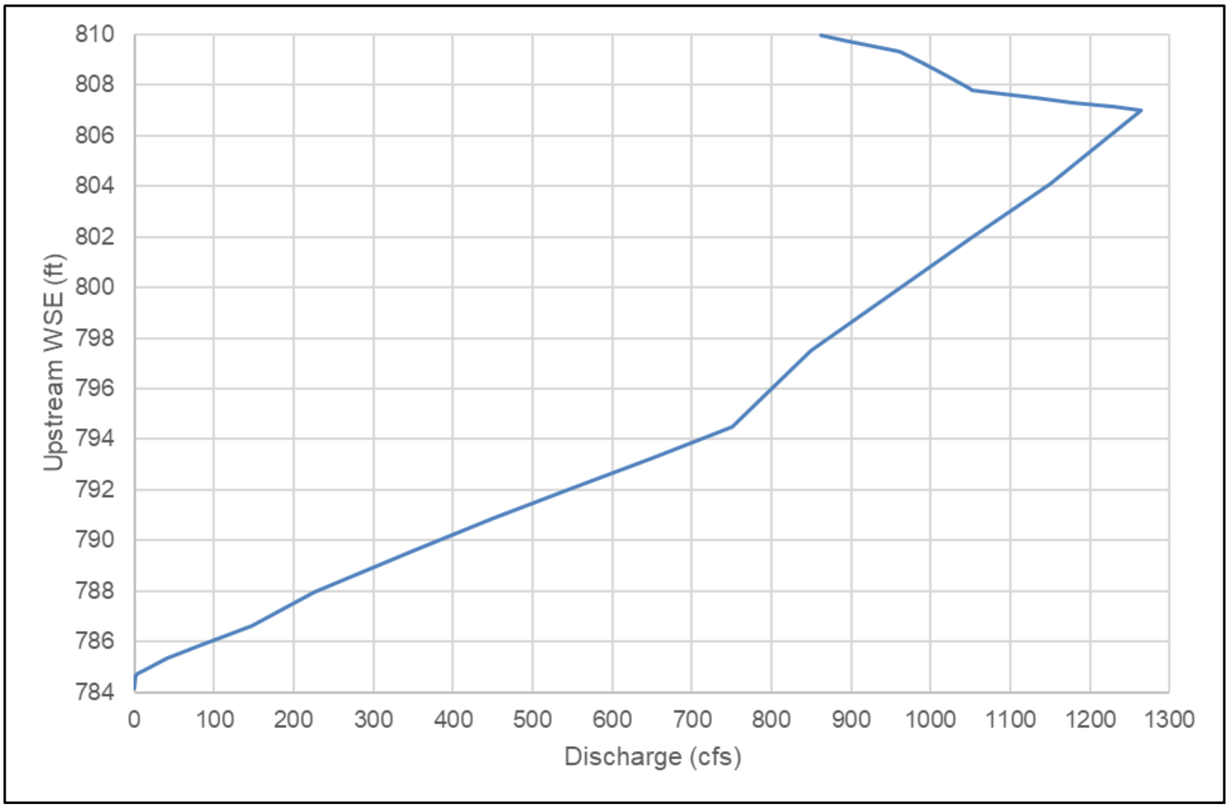

Figure 16. Principal Spillway Rating Curve (2.417 ft Opening)

Table 28. Principal Spillway Rating Curve and HEC-RAS Model Results (2.417 ft Opening)

Profile Description	Discharge (cfs) [1]	Control Wall Upstream WSE (ft)	Baffled Chute Downstream WSE (ft) [2]	Sluice Coefficient Calibrated to CFD [4]	Control Wall Orifice Velocity (ft/s)	Orifice Flow Depth (ft)
No flow (for rating curve)	0	784.15	783.80	n/a	n/a	n/a
June, 90% Exceedance Discharge	1.8	784.67	784.00	0.633 [3]	0.19	0.52
June, 85% Exceedance Discharge	2.7	784.70	784.03	0.633	0.27	0.55
June, 75% Exceedance Discharge	5	784.76	784.10	0.633	0.46	0.61
	10	784.86	784.20	0.633	0.78	0.71
	40	785.33	784.78	0.633	1.88	1.18
April 25% Exceedance Discharge	76.6	785.79	785.28	0.633	2.59	1.64
April 15% Exceedance Discharge	147.5	786.61	786.13	0.633 [3]	3.39	2.417
April 10% Exceedance Discharge	225.6	787.98	786.89	0.633	5.19	2.417
	350	789.62	787.98	0.633	8.04	2.417
	450	790.86	788.72	0.633	10.34	2.417
	550	792.06	789.38	0.633 [3]	12.64	2.417
Bankfull Discharge	650	793.24	789.98	0.633	14.94	2.417
	750	794.48	790.53	0.633	17.24	2.417
	850	797.51	791.04	0.633 [3]	19.54	2.417
	1,050	801.94	791.95	0.711 [3]	24.13	2.417
	1,150	804.09	792.30	0.736 [3]	26.43	2.417
807.0 ft HW (~100-yr Discharge)	1,264	807.00	792.93	0.756 [3]	29.05	2.417
Aux Spillway TW of 794.0 ft	1,229	807.13	793.45	0.733	28.25	2.417
Aux Spillway discharge of 500 cfs	1,212	807.20	793.73	0.722	27.86	2.417
Aux Spillway discharge of 1,000 cfs	1,180	807.32	794.27	0.701	27.12	2.417
807.5 ft HW	1,132	807.50	795.02	0.670	26.02	2.417
807.8 ft HW (~500-yr Discharge)	1,052	807.80	796.69	0.619 [3]	24.18	2.417
Aux Spillway discharge of 5,000 cfs	1,047	807.89	797.34	0.615	24.07	2.417
Aux Spillway discharge of 10,000 cfs	1,016	808.42	798.99	0.590	23.35	2.417
Aux Spillway discharge of 15,000 cfs	989	808.89	800.10	0.569	22.73	2.417
Aux Spillway discharge of 20,000 cfs	961	809.34	800.97	0.548	22.09	2.417
Aux Spillway discharge of 25,000 cfs	890	809.78	801.71	0.528	20.46	2.417
810.0 ft HW (~PMF Discharge)	862	810.00	802.07	0.518 [3]	19.81	2.417

[1] Discharges for HW 807.0 ft and higher are rounded to nearest 1 cfs

[2] Downstream WSE for Principal Spillway is based on results of 2D tailwater modeling and augmented with results of CFD modeling corresponding to HW less than 807.0 ft.

[3] Control wall gate sluice coefficient calibrated directly to a matching CFD simulation

[4] Control wall gate sluice coefficients interpolated between values calibrated based CFD simulations (see footnote 3)

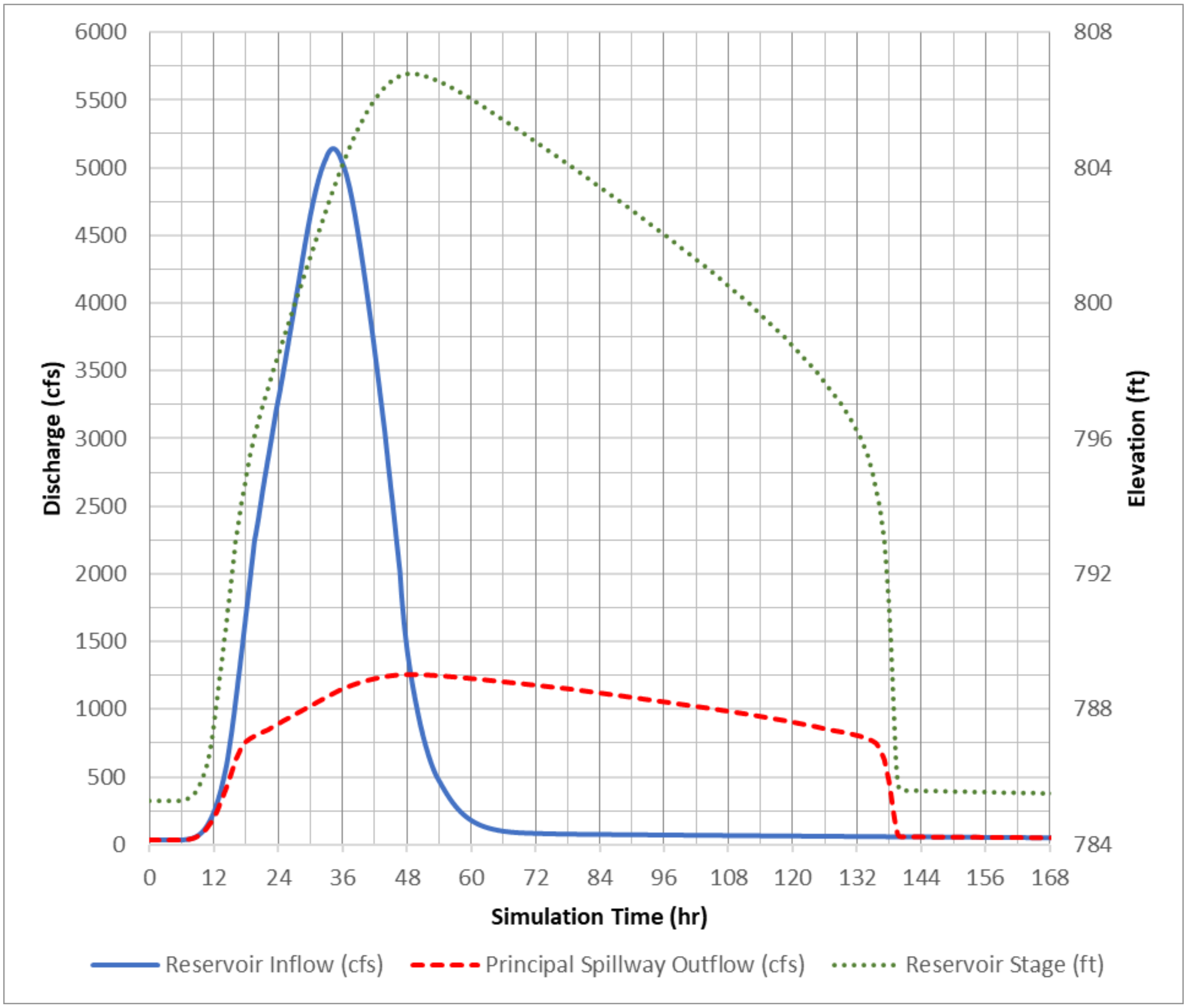

Figure 17. Principal Spillway HEC-HMS 100-year Hydrograph Routing (2.417 ft Opening)

6.4.4 Maximum Gate Opening (3 ft) Performance

As discussed in Section 6.4.3, the control wall orifice gates are intended to be set to an opening height of 2.417 ft (2 ft 5 in). For additional operational flexibility, the gates may be opened to a maximum height of 3 ft. Reasons for opening the gate to 3 ft might include performing maintenance, releasing debris, or adapting to future flood mitigation needs. CFD modeling was performed for the Principal Spillway during interim design iterations using varied gate opening heights. A limited number of discharges were evaluated for a gate opening height of approximately 3 ft. Based on the preliminary modeling, the Principal Spillway headwater elevations for a maximum 3 ft gate opening height are expected to be within approximately 0.5 ft of the designed gate opening height rating curve presented in Table 28 up to a discharge of approximately 650 cfs. At discharges higher than 650 cfs, the maximum 3 ft gate opening height is capable of releasing more flow from the PS orifice openings at the same headwater elevation. A control wall orifice configuration with a gate height of 3 ft is expected to result in a 100-year discharge of approximately 1,500 cfs through the PS, an increase of about 20% compared to the 2.417 ft opening.

6.4.4.1 Fish Passage

Per the design criteria, fish need a minimum depth of flow of 0.37 ft to be able to traverse the control wall orifices. The notches in the baffled chute are designed to maintain this minimum depth for flows as low as the 90% exceedance flow (1.8 cfs), including through the control wall orifices. As shown in Table 28, the design maintains a minimum depth of 0.52 ft, so fish passage is facilitated for the minimum depth criteria.

At the 15% exceedance discharge, using the 10 ft swim speed and time criteria, if there is a path through the control wall orifices with a velocity of less than or equal to the threshold velocity of 2.8 ft/s, then fish can traverse the control wall orifices in less than the 2.77 seconds threshold. Based on the CFD results, there is an area through the control wall cross section measuring approximately 0.5 ft wide and on either side of each of the orifices where the velocity is 2.8 ft/s or less which is sufficient for facilitating fish passage. Figure 18 shows the CFD results at this location.

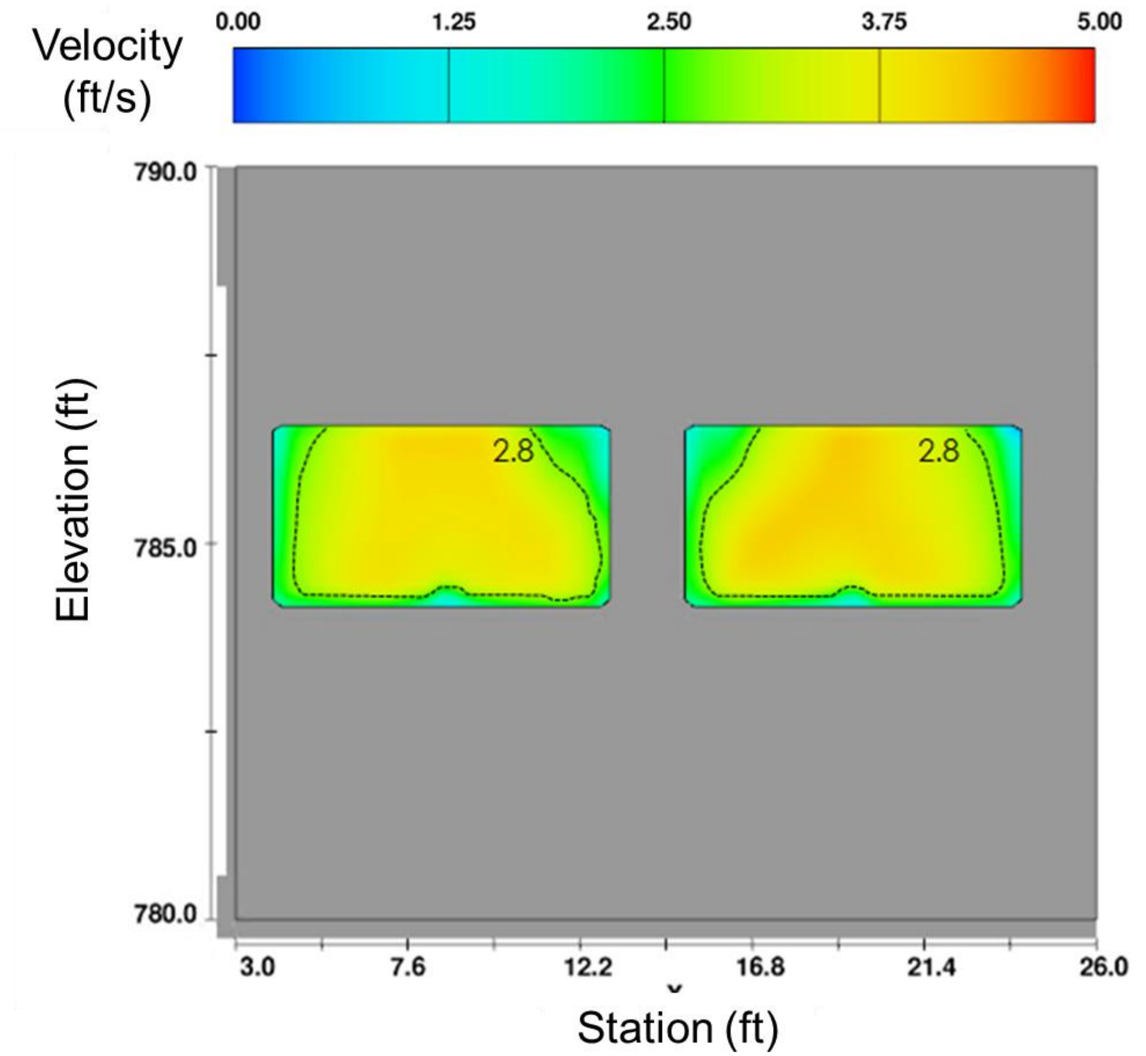

Figure 18. 15% Exceedance Discharge CFD Results Velocity Magnitude at Control Wall

Based on the CFD model velocity results and HEC-RAS model depth results, the control wall orifices are capable of facilitating fish passage for the 90% exceedance through the 15% exceedance discharges for the months of April, May and June. Table 29 presents the results demonstrating that fish are capable of traversing the control wall orifice based on velocity criteria.

Table 29. Control Wall Fish Passage Calculations

Description	Traversal Through Orifice
Fish Swim Speed Criteria (ft/s)	3.61 (10 ft)
Fish Swim Speed Time Criteria (s)	2.77
Max Flow Velocity Threshold to Facilitate Fish Passage (ft/s)	2.60
Swim Speed Relative to Ground (ft/s)	0.81
Traversal Distance (ft)	2.0
Traversal Time (s)	2.47
CFD Model Cross Section Results Show Path Available with Velocity Less than Max Velocity Threshold?	Yes

6.4.5 Baffled Chute

6.4.5.1 Energy Dissipation

Velocities through the Principal Spillway will be super-critical during discharges exceeding approximately 350 cfs. To prevent scour damage to the downstream outlet channel, the baffled chute is designed to dissipate energy within the Principal Spillway concrete structure so that flow is sub-critical at the end of the concrete section.

Energy dissipation for the control wall and baffled chute is provided within the baffled concrete section of the structure. The hydraulic design for the energy dissipation is based on HEC-14, section 7.2.2 (FHWA 2006). This methodology was developed for internal baffles in box culverts which behave similarly to the rectangular chute with an open-top.

First, a baffle spacing of 10 ft on center was selected to facilitate fish passage. HEC-14 recommends a ratio of baffle spacing to baffle height of 10 which results in an average baffle height of 1 ft. The 70 ft Principal Spillway chute length was determined based on structural layout and site grading and results in 7 baffles evenly spaced at 10 ft. This exceeds the minimum of 5 baffles recommended by HEC-14. The calculations determine the maximum velocity from the baffled section by computing normal depth flow properties with an effective conduit roughness caused by the baffles.

The calculations determined that the baffled chute configuration would result in sub-critical flow with a maximum velocity of 6.7 ft/s for the maximum discharge at the downstream end of the baffles corresponding to an upstream headwater WSE of 807.0 ft or approximately the 100-year event. This is a conservative velocity estimate because the elevated downstream tailwater experienced during an 807.0 ft upstream WSE event will result in a lower velocity value closer to 6.3 ft/s according to HEC-RAS modeling of the baffled chute. The HEC-14 energy dissipator calculations are included as Exhibit A of Appendix I.

The results of the CFD model simulations for the 807.0 ft upstream WSE simulation were used to verify that energy dissipation was achieved within the baffled chute. Figure 19 presents a velocity magnitude profile from the CFD model showing a stable hydraulic jump within the length of the Principal Spillway baffled chute. The CFD results for the 807.0 ft simulation indicate that the Froude number at the downstream end of the baffled chute is approximately 0.4 which is subcritical. Due to the magnitude of velocities observed downstream of the control wall, the first baffle is designed with a steel plate on its upstream face to reduce the risk of abrasion.

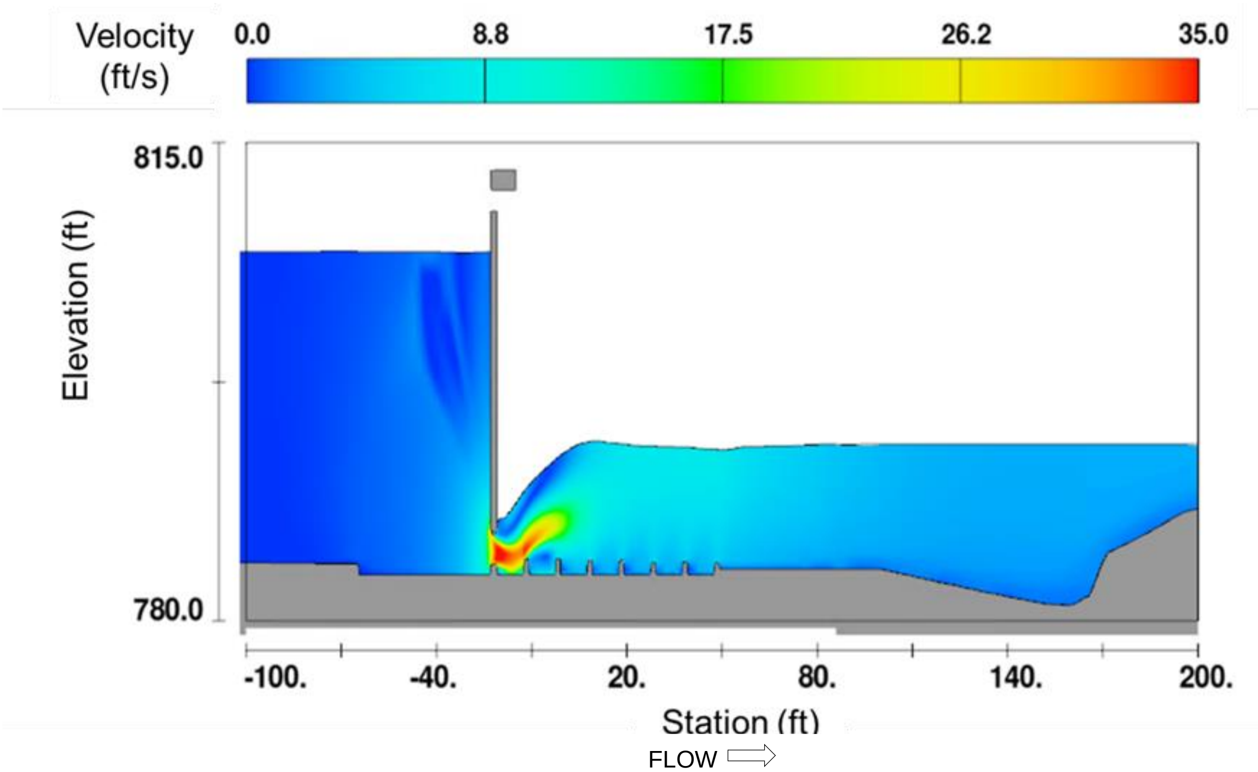

Figure 19. 807.0 ft Upstream WSE CFD Simulation Profile Through Principal Spillway

6.4.5.2 Fish Passage

Per the design criteria, the fish species analyzed need a minimum depth of flow of 0.37 ft to traverse the baffled chute. The notches in the baffles are designed to maintain this minimum depth for flows as low as the 90% exceedance flow (1.8 cfs). Table 30 summarizes the velocity and depth results of the baffled chute portion of the HEC-RAS model which are relevant to fish passage.

Table 30. Principal Spillway Baffled Chute HEC-RAS Model Summary

Profile Description	*Discharge (cfs)	Baffled Chute Downstream WSE (ft)	Max. Avg. Velocity Across Baffles (ft/s)	Max. Avg. Velocity Between Baffles (ft/s)	Minimum Flow Depth (ft)
June, 90% Exceedance Discharge	1.8	784.00	2.96**	0.09	0.41
June, 85% Exceedance Discharge	2.7	784.03	1.59	0.12	0.54
June, 75% Exceedance Discharge	5.0	784.10	2.20	0.22	0.57
April 25% Exceedance Discharge	76.6	785.28	3.66	1.76	1.42
April 15% Exceedance Discharge	147.5	786.13	3.71	2.39	2.27
April 10% Exceedance Discharge	225.6	786.89	3.99	2.87	3.04

*Data for these discharges used for fish passage design

**Flow is contained within baffle notch, increasing velocity

As shown in Table 30, the Final Design maintains a minimum depth of 0.41 ft, so fish passage is facilitated for the minimum depth criteria.

The fish can maintain the 10 ft swim speed criteria of 3.61 ft/s for 2.77 seconds and can maintain the 30 ft swim speed criteria of 2.66 ft/s for 11.29 seconds. The area upstream and downstream of each baffle should create sheltered areas allowing the individual baffles and the space between baffles to be evaluated independently for fish passage. Each feature is traversable if the calculated swim duration is less than the duration threshold developed from the swim speed criteria. It is not necessary that the average channel velocity allows for fish passage, instead it is only necessary that a “path” of acceptably low velocity be available to facilitate fish passage. For this reason, the CFD model results were used to verify that fish passage is facilitated.

At the 15% exceedance discharge, using the 10 ft swim speed and duration criteria, if there is a path over the baffles with velocities equal or less than the threshold velocity of 3.0 ft/s, then fish will be able to traverse each baffle in less than the 2.77 seconds threshold. Based on the CFD results, the last (downstream) baffle will be most challenging for fish passage due to the velocities observed. The last baffle has an area over the top of the baffle measuring approximately 7 ft wide and 0.5 ft tall where the velocity is 3.0 ft/s or less, which is sufficient for facilitating fish passage. Figure 20 presents the CFD results at the most downstream baffle.

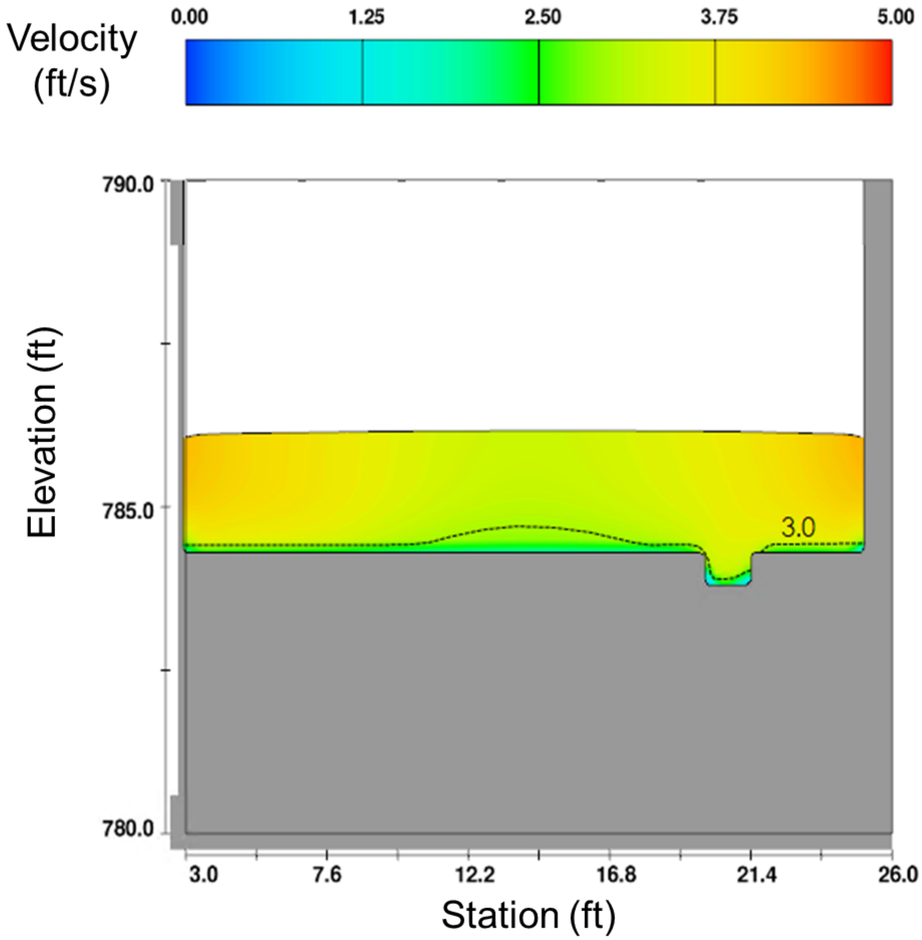

Figure 20. 15% Exceedance Discharge CFD Results Velocity Magnitude at Last Baffle

At the 15% exceedance discharge, using the 30 ft swim speed and duration criteria, if there is a path between the baffles with a velocity of less than or equal to the threshold velocity of 1.9 ft/s, then fish will be able to traverse the distance between each baffle in less than the 11.29 seconds threshold. Based on the CFD results, the distance between the last and second to last baffles will be most challenging for fish passage, but it has an area through the chute cross section measuring approximately 12 ft wide and 1 ft tall where the velocity is 1.9 ft/s or less, which is sufficient for facilitating fish passage. Figure 21 presents the CFD results at this location.

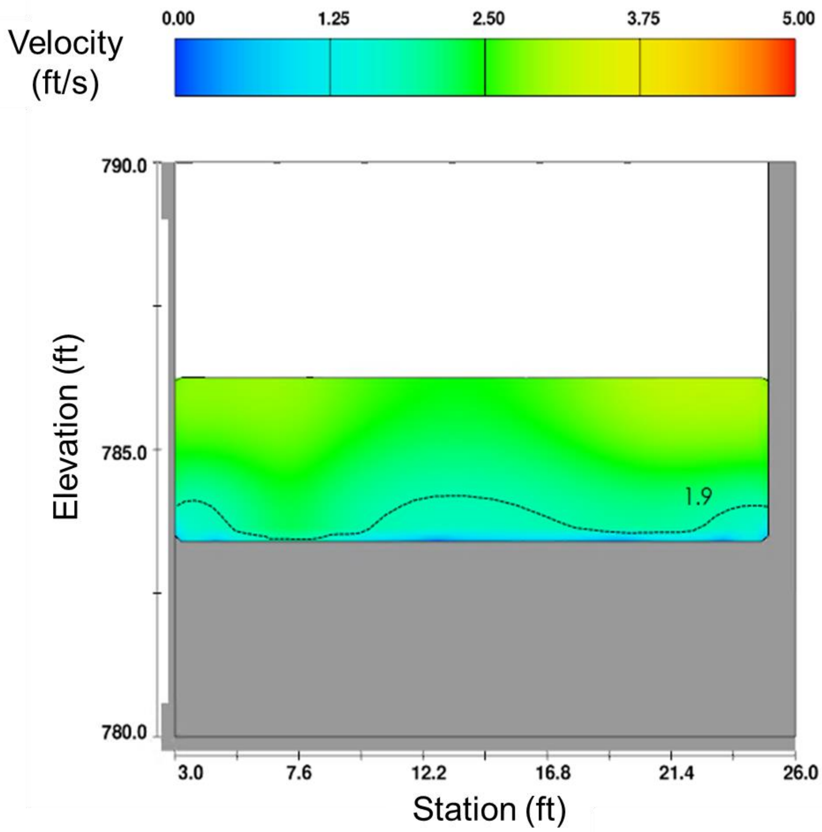

Figure 21. 15% Exceedance Discharge CFD Results Velocity Magnitude Between Last and Second to Last Baffles

Considering the above, the hydraulic model results indicate that the baffled chute facilitates fish passage for the 90% exceedance through the 15% exceedance discharges for the months of April, May and June. Table 31 presents results demonstrating that fish are capable of traversing the baffled chute based on velocity criteria.

Table 31. Baffled Chute Fish Passage Calculations

Description	Traversal Over Baffles	Traversal Between Baffles
Fish Swim Speed Criteria (ft/s)	3.61 (10 ft)	2.66 (30 ft)
Fish Swim Speed Time Criteria (s)	2.77	11.29
Max Velocity Threshold to Facilitate Fish Passage (ft/s)	3.0	1.9
Swim Speed Relative to Ground (ft/s)	0.61	0.76
Traversal Distance (ft)	1.5	8.5
Traversal Time (s)	2.46	11.23
CFD Model Cross Section Results Show Path Available with Velocity Less than Max Velocity Threshold?	Yes	Yes

6.4.6 Channel Armoring

The relocated channel downstream of the Principal Spillway outlet is armored with riprap to reduce the risk of scour and erosion due to baffle chute exit velocities. The size and thickness of the riprap blanket downstream of the Principal Spillway chute was determined using Equation 3-3 from USACE Engineering Manual 1110-2-1601*, Hydraulic Design of Flood Control Channels*.

Flow velocity and depth values were extracted from the HEC-RAS modeling results as described in the *Hydrologic and Hydraulic Analysis Report, Appendix D*. Where the CFD model domain included the area downstream of the spillway, results from the CFD model were also used. The resulting depth-averaged velocity and depth from the CFD and HEC-RAS model results that produced the larger riprap thickness was used for design.

A grid of computation points was established along the channel slopes, and a set of computation regions were used along the channel bottom. This approach accounts for the variations in channel side slope and channel bottom geometry. The minimum required riprap based on the hydraulic model results was determined at the computation locations.

The thickness of the riprap layer is determined using the USACE EM 1110-2-1601 criteria (Section 3-2e) which states that thickness should not be less than the spherical diameter of the upper limit D100 stone or less than 1.5 times the spherical diameter of the upper limit D50 stone, whichever results in the greater thickness. It is expected that riprap will be placed in the dry condition (not below water) and additional thickness corrections are not required.

The Threshold design criteria described in Chapter 8 of the National Resources Conservation Services *Part 643 Stream Restoration Design National Engineering Handbook* was used to determine the extents of the riprap armoring. ODOT Type D riprap has been included from the outlet of the Principal Spillway chute to a point downstream where channel velocities do not exceed 3.3 ft/s. Figure 22 indicates that during a long duration flood event, poor grass cover is adequate to withstand velocities up to 3.3 ft/s.

The riprap armoring includes a thickened riprap edge at the downstream end of the concrete spillway. This thickened edge was incorporated with the goal of providing a filtered outlet to address potential uplift pressures. Additional information regarding the geotechnical considerations can be found in Section 6.5.1. The design detail of the thickened edge is included in Figure 23.

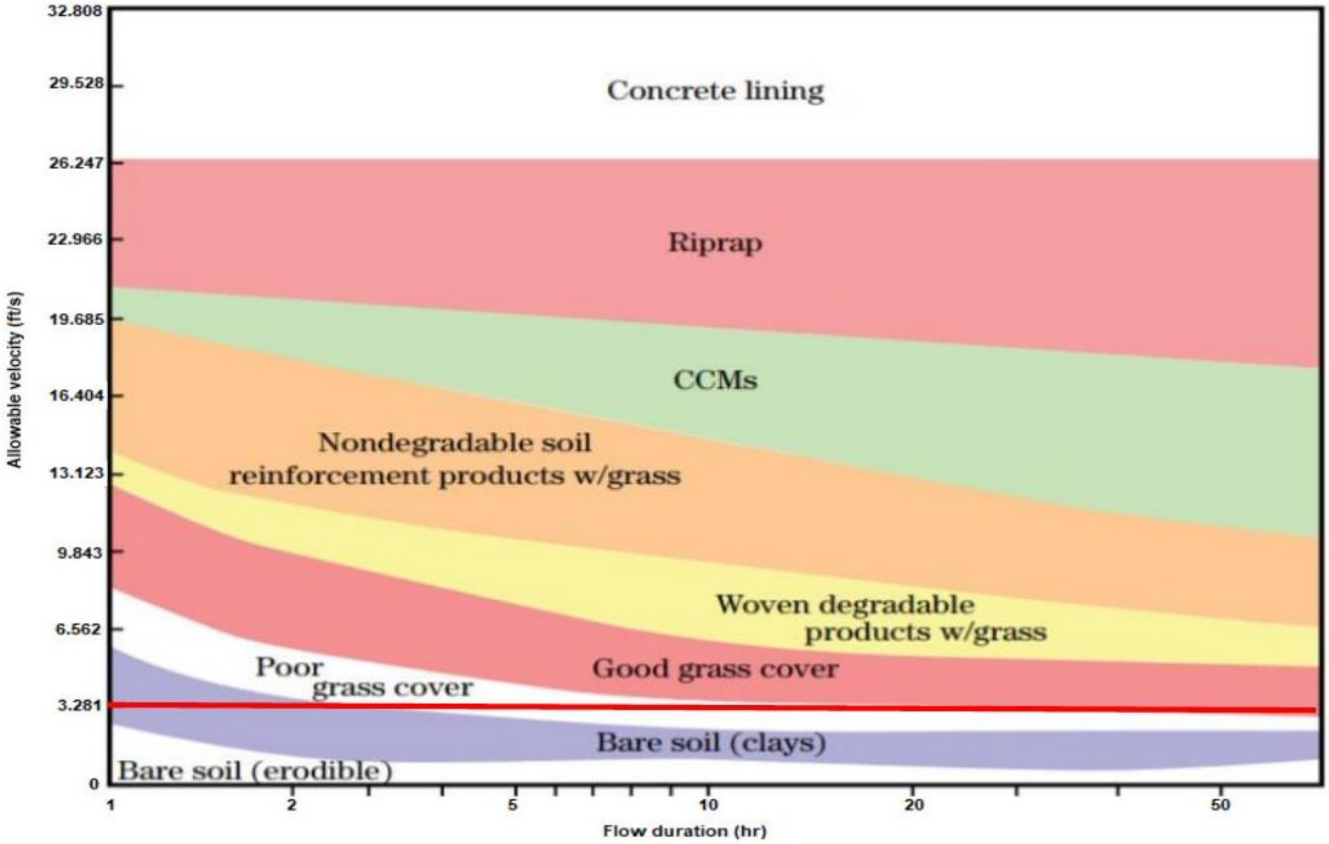

Figure 22. Velocity Threshold Design Criteria

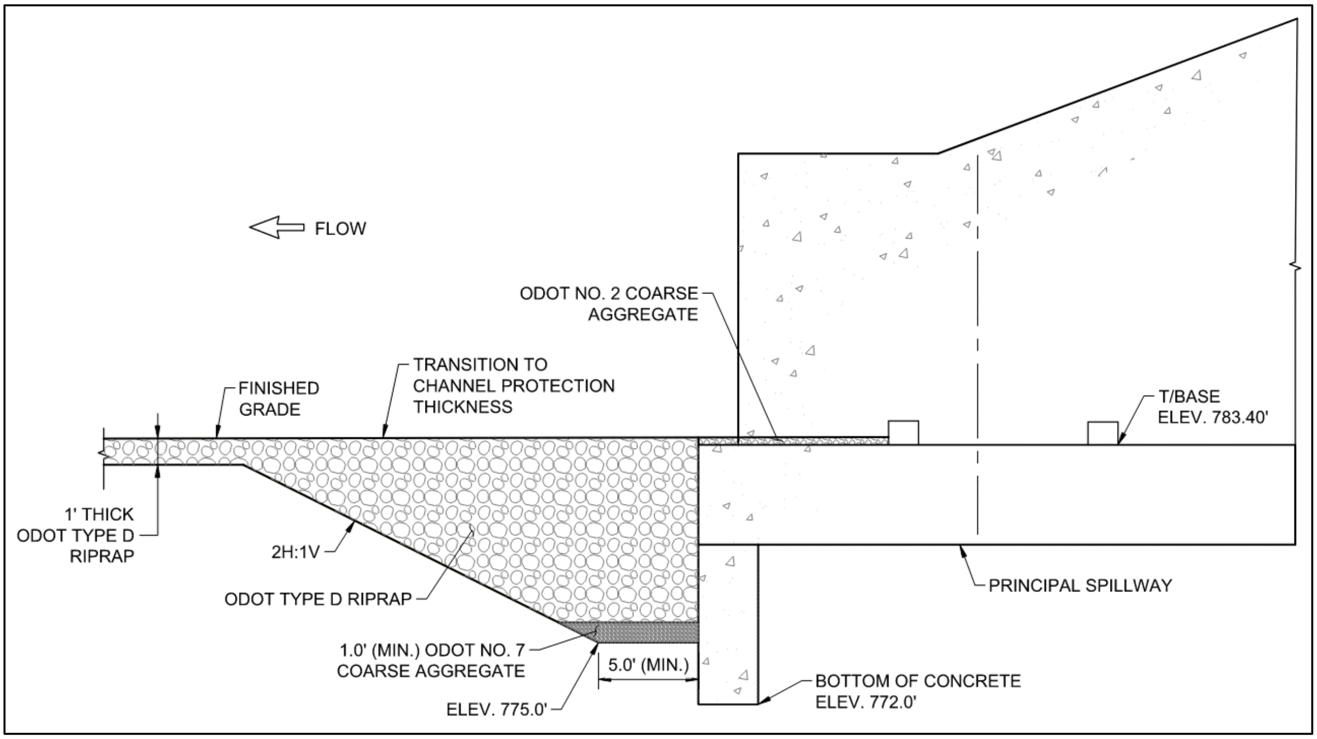

Figure 23. Principal Spillway Downstream Riprap Armoring Detail

6.4.7 Debris Rack

As a natural system, Eagle Creek has the potential to convey large debris such as trees to the Principal Spillway. To maintain proper function during a flood event, it is critical that the Principal Spillway continue to pass flow as designed under debris loading. This will be achieved through a sloping debris rack structure upstream of the control wall.

6.4.7.1 Debris Rack Dimensions

The dimensions of the debris rack were computed based on the methodology described in Chapter 10 of USBR Design of Small Dams (DSD), 3rd Edition (USBR 1987). The debris rack was designed using the 807.0 ft upstream WSE (~100-year) event discharge (1,264 cfs). The debris rack is a rectangular structure with a 22 ft width to fit within the Principal Spillway structure abutment walls. Debris rack bar thickness was assumed to be 4-inches.

During ECFB storage operations, the inlet of the Principal Spillway will be submerged and inaccessible. According to DSD guidance, the velocity through a clean debris rack should be limited to 2 ft/s. Following DSD guidance, the maximum head loss through the debris rack was computed assuming that the debris rack is 50% clogged. Based on the structure width and design discharge, the debris rack slope and vertical height were varied to achieve the desired velocity. Table 32 summarizes the results of the debris rack calculations. Detailed calculations are included in Exhibit A.2 of Appendix I.

Table 32. Principal Spillway Debris Rack Dimensions

Parameter	Control Wall / Baffled Chute
100-year Design Discharge (cfs)	1,264
Rack Shape (ft)	Rectangular
Bar Thickness (in)	4
Clear Spacing Between Bars (in)	14
Rack Top Width (ft)	22
Rack Vertical Height (ft)	13
Rack Slope (H:V)	3.50
Rack Horizontal Length (ft)	29
Maximum Head Loss at 50% Clogged (ft)	0.27

6.4.7.2 Debris Rack Drag Force and Debris Loading

The drag force of the debris acting on the debris rack during a flood event was calculated using methodology described in Section 4.6 of the Federal Highway Administration's Hydraulic Engineering Circular No. 9 (HEC-9): "Debris Control Structures Evaluation and Countermeasures" third edition (FWHA 2005). The drag force for 30% debris blockage and 70% debris blockage was calculated. Using the 100-year discharge through the Principal Spillway and the dimensions of the rack discussed in Section 6.4.7.1, a water velocity, coefficient of drag, and area of open space between debris was found for each blockage scenario. Equation 4.1 of HEC-9 was then used to calculate the drag force exerted by the debris on the debris rack for each blockage scenario. Results are summarized in Table 33.

Table 33. Drag Force Calculation Results Summary

Blockage Scenario	Drag Force (lbs)	Pressure on Debris Rack (lbs/ft^2)
30% Blockage	11,500	15
70% Blockage	19,100	25

A debris loading calculation was also performed to assess the weight of debris on the debris rack as the flood pool recedes. A conservative approach was taken in determining the type, volume, and density of debris for this calculation. All debris was assumed to be solid waterlogged shagbark hickory wood. By using the dry and green weights of shagbark hickory wood and assuming a water content of 80%, the weight density of the debris was calculated. The volume of debris was calculated as the volume of open space above the debris rack within the Principal Spillway where debris can reasonably accumulate. Lastly, the void ratio within the debris pile was estimated to be 80%. Multiplying the weight density of debris, the volume of debris, and the ratio of debris within the debris pile (20%) resulted in a debris load of 78,499 lbs. These calculations are summarized in Table 34 and included in Exhibit A.2 of Appendix I. Because the debris load was significantly higher than the calculated drag force for both blockage scenarios, the weight of the debris was utilized to design the debris rack.

Table 34. Debris Loading Results Summary

Parameter (Unit)	Value
Weight Density of Debris (lb/ft^3)	62.8
Volume of Debris Pile (ft^3)	6,252
Void Ratio	0.8
Debris Load on Rack (lbs)	**78,499**

6.5 GEOMORPHIC CONSIDERATIONS

6.5.1 Upstream / Downstream Spillway Channel

The upstream and downstream spillway channels were designed using restoration and Natural Channel Design (Rosgen, D.L. 2011) principles adapted to design constraints. The primary design goals for the channel are to reduce the risk of future erosion and migration of the channel, promote sediment transport, and provide aquatic habitat.

The spillway channel ties into the thalweg elevation of the existing Eagle Creek channel at both upstream and downstream tie-ins. The upstream tie-in is located within the backwater of an existing low-head dam. To account for the impacts of the low-head dam on sediment deposition in the channel, the upstream tie-in was checked against the longitudinal profile of the channel upstream and downstream of the dam-influenced reach. The spillway channel alignment includes “gentle” meander bends with radii that will reduce shear stresses on the outside of the bends.

Geomorphic data collected on Eagle Creek upstream and through the project site, from the dam at Camp Berry to US-68, were used to develop typical riffle and pool cross sections. Cross section measurements were collected at four naturally occurring riffles. Two of the riffles were determined to be most stable and have the best bankfull indicators and were used to further estimate geomorphic characteristics of Eagle Creek. A HEC-RAS one-dimensional (1-D) model was used to estimate a bankfull discharge of 630 cfs. A detailed discussion of the geomorphic data collection and analysis is included in the report titled, “*Eagle Creek Flood Basin – Geomorphic Assessment*” (Stantec, 2022b). A typical riffle cross section was designed to approximate the field measured cross section width to depth ratio and convey the estimated bankfull discharge. This typical section is approximately trapezoidal, with a top width of 51.5 ft., a bottom width of 32 ft., and an average depth of 3.49 ft.

The slope of the spillway channel (0.13%) approximates the bankfull slope of Eagle Creek as measured in the field. While the spillway channel is shorter than the reach of Eagle Creek that it replaces, the steeper concrete spillway allows these lower slopes to be used in the design of the stream channel.

Outer bends, where shear stresses will be greater and the threat of future erosion higher, will be reinforced using rock toe and toe wood. This toe of slope reinforcement will reduce the risk of channel migration in the vicinity of the Principal Spillway and dam embankment. The bank protection features utilize logs and brush anticipated to be generated during clearing and grubbing for construction of the channel, spillway, and embankment. This wood provides valuable habitat and decreases the velocity of flow immediately adjacent to the banks through increased roughness. The banks above the rock or wood will be built back using soil wrapped live brush layering. This bioengineering approach provides reinforcement of newly constructed slopes with erosion control fabric, as the live branches and native seed incorporated in the soil lifts are established.

6.5.2 Sediment Transport

Sizing the Principal Spillway to pass the bankfull discharge unregulated will allow the Principal Spillway to maintain appropriate sediment transport competence such that sedimentation within the Principal Spillway will be reduced. A critical shear stress of 1.02 lbs/ft^2 was determined to be sufficient to transport the largest particles observed within the active bed. The bankfull shear stresses for the control wall and baffled chute were estimated to be 1.2 lbs/ft^2. Refer to the report titled, "*Eagle Creek Flood Basin – Geomorphic Assessment*" (Stantec, 2022b) for a detailed description of the sediment transport model and results.

6.6 GEOTECHNICAL CONSIDERATIONS

To characterize the subsurface conditions of the Principal Spillway, borings B-4.15 through B-4.17 were conducted in the vicinity of the Principal Spillway location as shown in Figure 24. Details of the exploration and logs of the borings are available in the Geotechnical Design Report, Appendix E.

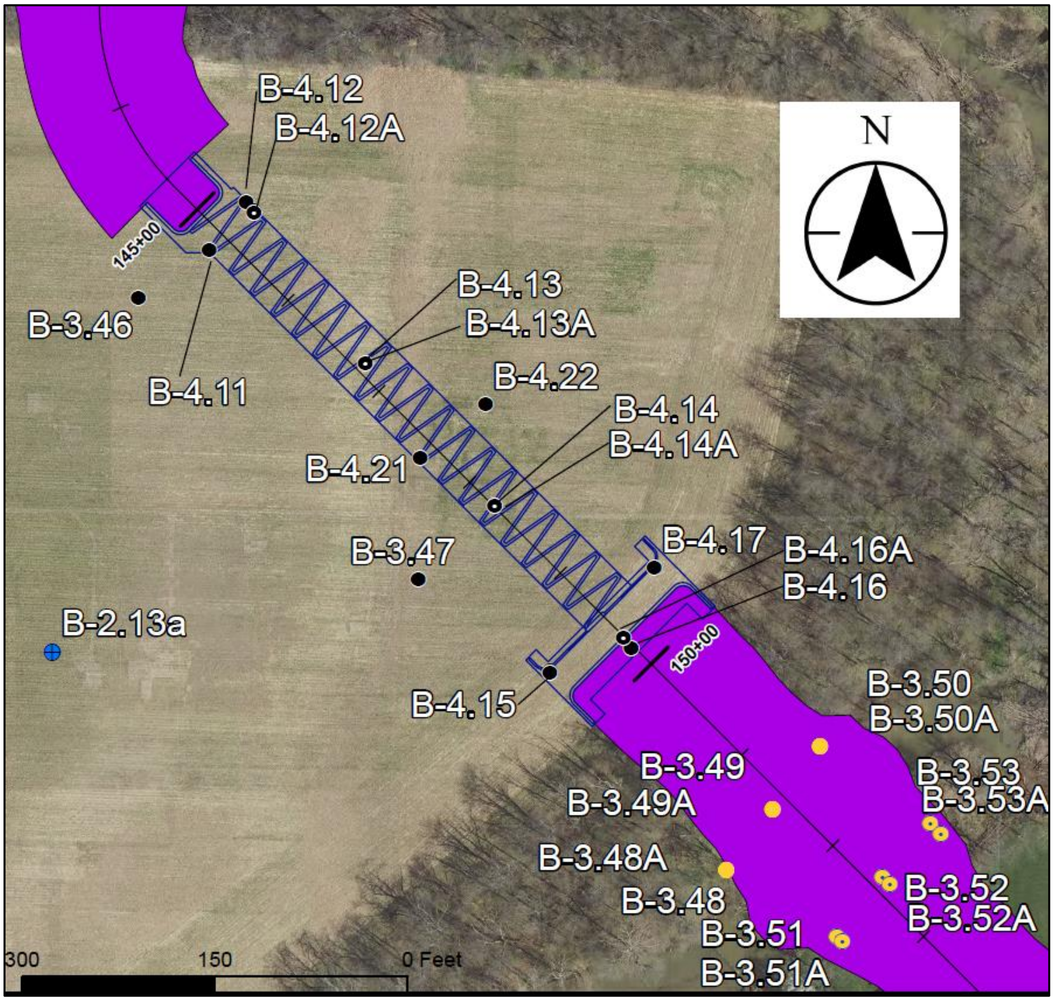

Figure 24. Boring Layout at the Spillway Location

The subsurface soils encountered at the Principal Spillway location include alluvial and glacial till deposits near Eagle Creek. Soils encountered consisted of Upper Fine-Grained, Upper Coarse-Grained, Lower Fine-Grained, and Lower Coarse-Grained horizons as defined in the Geotechnical Design Report (Appendix E). The Principal Spillway orifice elevation is approximately 784 feet, with the bottom of the concrete foundation slabs at elevation 778.1 feet to 778.4 feet. The encountered top of rock elevation is approximately 774 feet to 777 feet for both the Principal and Auxiliary Spillways. Figure 25 shows the subsurface stratigraphy at the Principal Spillway location.

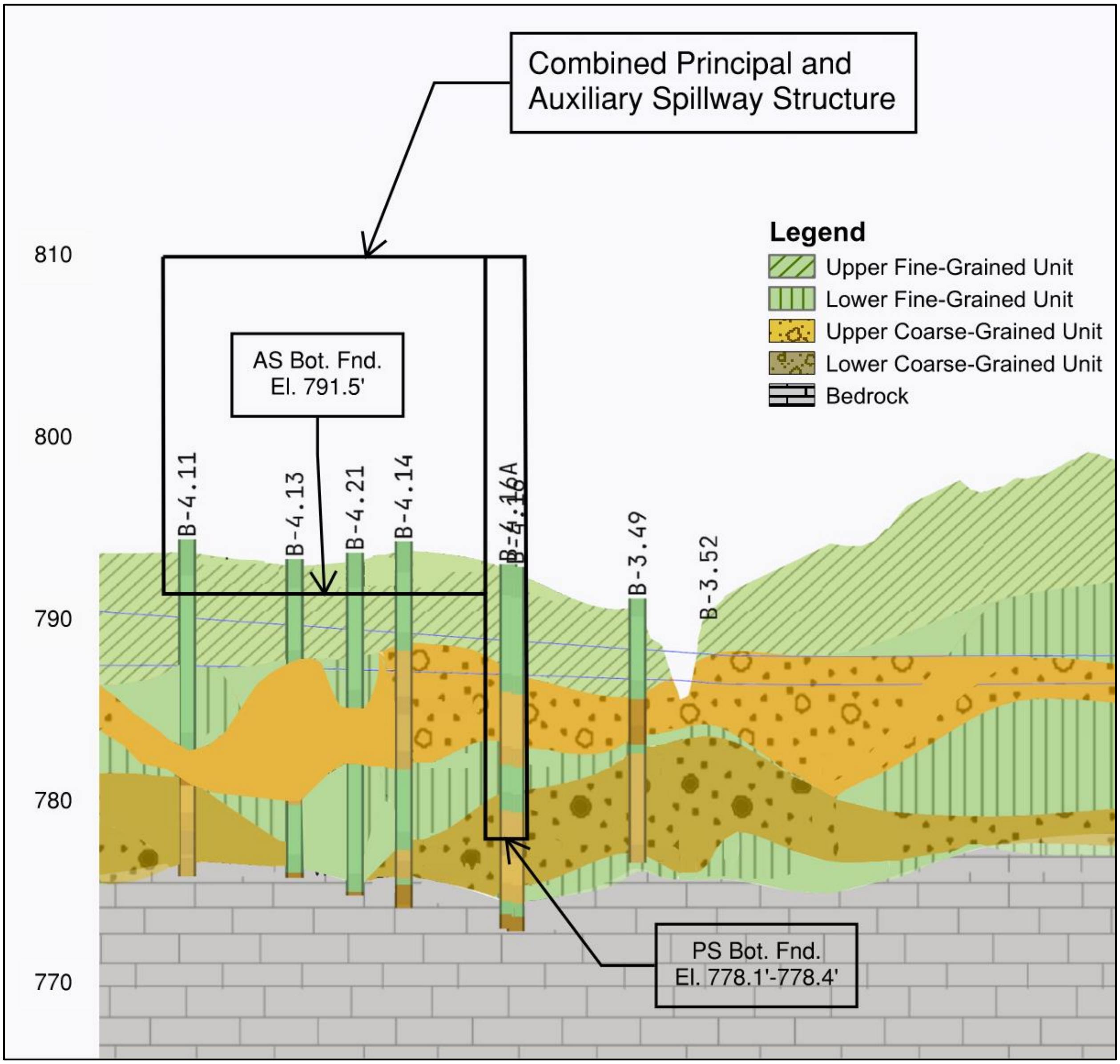

Figure 25. Subsurface Profile – Integrated Labyrinth/Principal Spillway

6.6.1 Foundation and Lateral Earth Pressure Parameters

The subsurface information and test data were used to develop the following geotechnical parameters for design of the Principal Spillway:

- Allowable bearing capacity of 5,000 pounds per square foot (psf). This estimate was based on the size and elevation of the structure's foundation.

- Subgrade modulus of 125 lbs/in^3 based on guidance from UFC (2005), Table 4-1.
- Interface friction angle of 26.5 degrees between concrete and ODOT Item 304 Aggregate Base. This value is based on recommendations in NAVFAC (1986), Table 1.
- Undrained shear strength parameters for native Lower Fine-Grained soil: 25-degree friction angle (ϕ) and 1,000 psf cohesion (c), based on laboratory test results and characterization.
- The spillway walls were designed for at-rest earth pressure conditions. Lateral loads applied to the Principal Spillway structure were calculated considering compacted fill placed against the spillway walls. The drained friction angle (ϕ') of the compacted fill is 33 degrees. The estimated at-rest earth pressure coefficient is $K_0 = 0.455$.
- Frost depth for the project site is 36 inches.

6.6.2 Groundwater Corrosivity

As discussed in Section 4.4.13, groundwater at the project site could be corrosive to concrete and steel due to the presence of naturally occurring hydrogen sulfide. To account for this in the design, microcrystalline additives are included in the requirements for subsurface concrete. Additionally, seepage cutoff walls will be PVC sheet piles, rather than steel.

6.6.3 Seepage and Uplift

Details of the seepage analyses performed on the Principal Spillway are available in the Geotechnical Design Report, Appendix E. Seepage analyses were completed using GeoStudio SEEP/W 2018 R2, finite element software. To support the load cases analyzed for structural stability, three uplift pressure profiles for the Principal Spillway were developed using SEEP/W results.

To reduce uplift, a 3-foot layer of Fill Type 1 is included below the upstream riffle armor, extending 150 feet upstream of the Principal Spillway toe.

The downstream key of the Principal Spillway is proposed to extend approximately 3 feet into bedrock (note that top of rock is approximately elevation 775 feet). This key provides a barrier to reduce the risk of soil scour progressing upstream and undercutting the Principal Spillway slab. With this key (concrete wall), an erosion pipe at the downstream toe of the spillway cannot form to initiate backward soil erosion under the slab. The key wall will otherwise restrict downstream movement of eroded soil from below the Principal Spillway slab.

The design includes a zone of riprap armoring to prevent erosion at the downstream edge of the spillway slab. As additional protection from seepage near the toe of the Principal Spillway, the riprap will be bedded on a 1-foot (minimum) thickness of ODOT No. 7 Aggregate at the top of rock. This layer will serve as a filter on the top of rock to lower the risk of erosion due to upward seepage.

6.6.4 Settlement

Settlement of the Principal Spillway foundation was evaluated using the program Settle3 by Rocscience. The maximum estimated total (ultimate) settlement at the base of foundation elevation was about 0.4 inches, with a maximum estimated differential settlement of 0.3 inches. The Principal Spillway foundation slabs are at least 5 feet thick and connected with stainless steel dowels to keep them aligned with one another. The estimated settlements are acceptable. Details of the settlement analyses are provided in the Geotechnical Design Report, Appendix E.

6.6.5 Ground Improvement

Ground improvement is not required to meet design criteria for the Principal Spillway structure. To improve constructability after dewatering, the design includes over-excavation below the Principal Spillway foundation slabs to allow for a 6-inch layer of ODOT 304 Aggregate Base to be placed and compacted. This material is well graded, with fines, and will provide a firm, compacted pad to begin concrete construction and backfill activities.

6.6.6 Seismic Design

6.6.6.1 Site Class

The Seismic Site Class was estimated per the American Society of Civil Engineers (ASCE) Standard 7-16 design manual. The concrete Principal Spillway structure is expected to have a fundamental period of vibration lower than 0.5 seconds. Standard Penetration Test (SPT) data from the geotechnical exploration was used to characterize the site as Class C.

6.6.6.2 Liquefaction Triggering

The borings advanced near the proposed Principal Spillway structure location were included in the liquefaction triggering analyses for the embankment (Section 5.10). Liquefaction is not expected to be triggered during the design earthquake event at the project site.

6.7 STABILITY

The Principal Spillway components analyzed for stability included the primary structure and associated wingwalls, abutments, and debris rack. The analyzed forces included overturning and bearing stress, sliding forces, and floatation forces.

6.7.1 Acceptance Criteria

All structural elements of the Principal Spillway (except the debris trash rack) are assumed to be critical structures as failure could directly or indirectly lead to a loss of life. The debris rack is classified as a normal structure as any potential failure scenario is unlikely to cause a loss of life.

The following acceptance criteria are based on EM 1110-2-2100. The load cases to be evaluated are divided into categories as listed in Table 35.

Table 35. Acceptance Criteria for Hydraulic Structures

Loading Combination	Position of Resultant Force (Percent of Base in Compression)	Sliding Safety Factor (Friction Only)	Floatation Safety Factor
Usual	Middle third of the base: 100% compression	≥2.0	≥1.3
Unusual	75% of Base in compression	≥1.5	≥1.2
Extreme Flood	Resultant within base, and all other acceptance criteria must be met	≥1.1	≥1.1

6.7.2 Load Combinations

Table 36 summarizes load conditions for the Principal Spillway structure. Table 37 summarizes load conditions for the typical retaining wall. The load combinations listed below are to be considered a representative sample.

Table 36. Principal Spillway Monolith P3 – Load Conditions

Usual Load Cases		
U1	Usual Condition – Dry/No Flow (Groundwater to top of foundation) Headwater Elevation – 783.40 Tailwater Elevation – 783.40	D+H+E+U
U2	Usual Condition – Normal Operating Headwater Elevation – 794.00 Tailwater Elevation – 790.30	D+H+E+U
Unusual Load Cases		
UN1	Unusual Condition – Maximum headwater / tailwater difference Headwater Elevation – 807.13 Tailwater Elevation – 793.00 Uplift Per Seepage Analysis	D+H+E+U
Extreme Load Cases		
E1	Extreme Condition – Maximum Flood Level at Reservoir (PMF) Headwater Elevation – 810.00 Tailwater Elevation – 802.11 Uplift per Seepage Analysis	D+H+E+U
Notes		
D	Dead Load: Includes concrete (C), backfill (E),	
H	Hydrostatic Load:	
EH, EV	Earth / Backfill / Sediment / Siltation: Include horizontal and vertical loads	
U	Uplift Load	

Table 37. Monolith P1 and P5 Retaining Wall – Load Conditions

Usual Load Cases		
U1	Usual Condition – (dry / no flow) At-Rest Soil Loading Groundwater at Top of Foundation	D+H+E+U
U2	Usual Condition – At-Rest Soil Loading Headwater EL. 794.00, Tailwater EL. 790.30	D+H+E+U
Unusual Load Cases		
UN1	Unusual Condition – At-Rest Soil Loading + Equipment Surcharge Groundwater at Top of Footing	D+H+E+U+L
UN2	Unusual Condition – Water to EL. 793.00 Headwater EL. 807.13, Tailwater EL. 793.00	D+H+E+U
Extreme Load Cases		
E1	Extreme Condition – At-Rest Soil Load – Post Maximum Flood Level at Reservoir Saturated Earth behind walls	D+H+E+U
E2	Extreme Condition – At-Rest Soil Load – Post Maximum Flood Level at Reservoir Headwater EL. 810.00, Tailwater EL. 802.11	D+H+E+U
Notes		
D	Dead Load: Includes concrete (C), backfill (E),	
H	Hydrostatic Load:	
EH, EV	Earth / Backfill / Sediment / Siltation: Include horizontal and vertical loads	
U	Uplift Load	
L	Live Loads: Vehicle / equipment surcharge	

6.7.3 Stability Analysis Results

The following is a summary of the stability analyses conducted for the Principal Spillway, including all associated structures. Each section was evaluated for Usual, Unusual, and Extreme flooding loading conditions representing potential conditions the structure will experience during its design life. Refer to Exhibit B.1 in Appendix I, Principal Spillway Technical Memorandum, for additional details regarding the stability calculations and results.

6.7.3.1 Maintenance Bridge

Due to the location of the Maintenance Bridge, no stability analysis is required as it is located above the maximum PMF elevation.

6.7.3.2 Monolith P2 and P4

Monoliths P2 and P4, shown in Figure 26, are similar in height on both sides of the Principal Spillway channel and therefore only one stability analysis was performed. Stability is calculated about the C/L of the Principal Spillway channel. Stability about the dam baseline is not necessary as there is not a way for an imbalance of loading to occur in that direction. For purposes of this analysis water in the channel is ignored in one check of stability with load cases being similar to a retaining wall, in addition to the stability analyses matching those of P3 which considers water within in the channel. Representative 1-ft sections were taken at the low side of the monoliths and another at the high side of the monolith. If both sides are stable, the full monolith is considered stable. A summary of Monoliths P2 and P4 stability results are shown in Table 38.

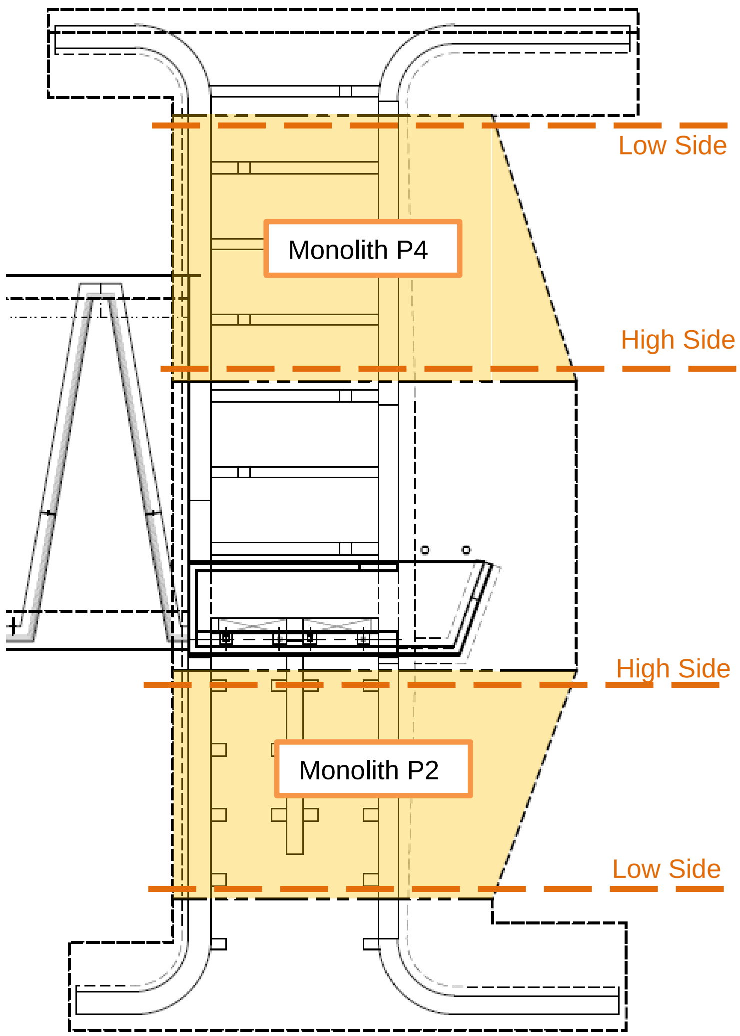

Figure 26. Monolith P2 and P4 Plan

Table 38. Monolith P2 and P4 – Stability Summary

Location	Load Comb.	e (ft)	σ @ Heel (psf)	σ @ Toe (psf)	% Base in Comp.	Sliding SF	Sliding SF Req'd	Floatation SF	SF Req'd.
High Side	U1	-7.36	3,290	300	100	2.26	2.0	3.11	1.3
	U2	-5.74	3,250	690	100	2.48	2.0	3.32	1.3
	UN1	-8.64	2,810	30	100	1.63	1.5	2.17	1.2
	UN2	-4.67	2,770	850	100	1.55	1.5	2.45	1.2
	E1	-3.81	810	2,040	100	1.15	1.1	1.84	1.1
	E2	-3.81	810	2,040	100	1.15	1.1	1.84	1.1
Low Side	U1	-6.29	1,630	90	100	3.28	2.0	2.01	1.3
	U2	-3.86	1,680	490	100	4.14	2.0	2.01	1.3
	UN1	-4.83	1,240	230	100	2.49	1.5	1.32	1.2
	UN2	-3.92	1,140	320	100	1.66	1.5	1.32	1.2
	E1	-2.79	840	360	100	1.47	1.1	1.35	1.1
	E2	-2.79	840	360	100	1.47	1.1	1.35	1.1

6.7.4 Center Monolith P3

The center monolith (P3) of the control wall and baffled chute is the large central section of the Principal Spillway shown in Figure 27.

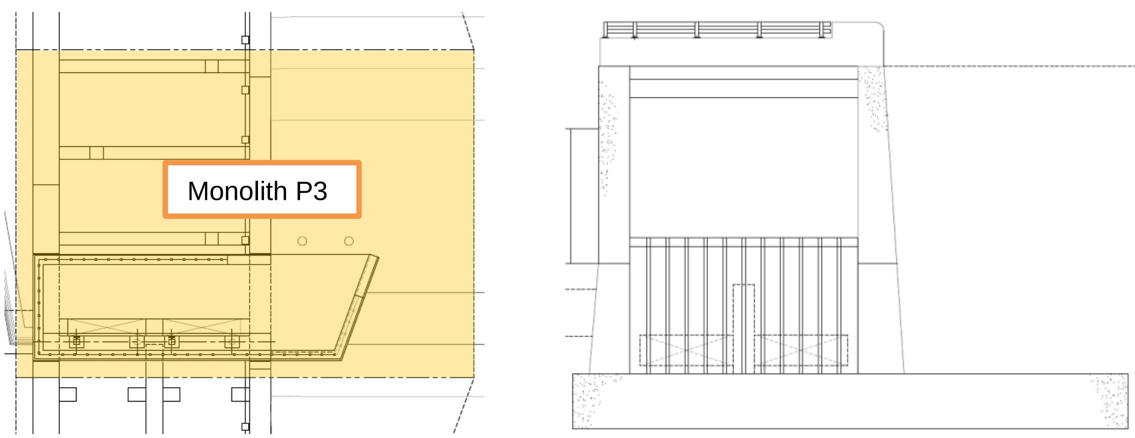

Figure 27. Center Monolith (P3) Plan and Section

Stability analysis for the center monolith structure was performed using Excel and was checked in two directions. Stability was evaluated along the plane of the channel and along the plane of the dam baseline. Results of the analyses are summarized in Table 39.

Table 39. Center Monolith (P3) – Stability Summary

Stability About:	Load Comb.	E (ft)	σ @ Heel (psf)	σ @ Toe (psf)	% Base in Comp.	Sliding SF	Sliding SF Req'd	Floatation SF	SF Req'd.
Channel	U1	-5.25	2,310	2,240	100	2.05	2.0	3.43	1.3
	U2	-4.08	2,480	2,430	100	2.22	2.0	3.43	1.3
	UN1	-3.06	2,250	2,210	100	1.5	1.5	2.48	1.2
	E1	-1.96	1,940	1,920	100	1.74	1.1	1.74	1.1
Dam Baseline	U1	-0.97	2,280	2,270	100	-*	2.0	3.43	1.3
	U2	-0.84	2,450	2,460	100	55.46	2.0	3.64	1.3
	UN1	-0.25	2,230	2,230	100	7.02	1.5	2.72	1.2
	E1	-0.10	2,310	2,430	100	9.6	1.1	2.89	1.1

* - denotes not lateral load imbalance in load case.

6.7.5 Monolith P1 and P5 (Section F and Section G)

The east and west abutment or retaining walls have two more traditional cantilever style retaining walls. Section F Walls are similar on each side and Section G walls are also similar. Figure 28 shows the retaining wall design locations.

Principal Spillway

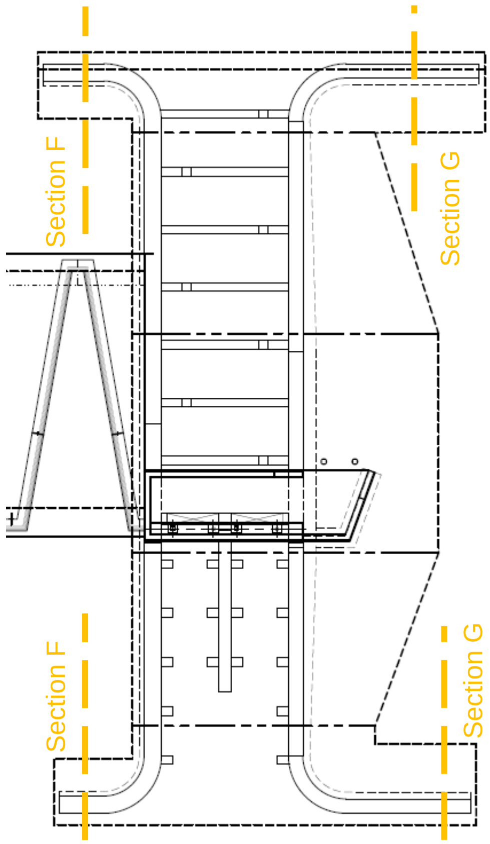

Figure 28. Retaining Wall Design Locations

The stability analysis was performed using Excel spreadsheets. Results of the analyses are summarized in Table 40.

Table 40. Retaining Wall – Stability Summary

Wall Section	Load Comb.	Reaction in Middle 1/3 (Within the base for E1)	σ @ Heel (psf)	σ @ Toe (psf)	% Base in Comp.	Sliding SF	Floatation SF	SF Req'd.
F	U1	Yes	1,250	1,230	100	5.08	4.27	1.5
	U2	Yes	850	840	100	7.04	1.74	1.5
	UN1	Yes	1,200	1,180	100	4.86	4.27	1.3
	UN2	Yes	760	750	100	4.02	1.54	1.3
	E1	Yes	1,240	1,240	n/a	6.60	4.27	1.1
	E2	Yes	540	540	n/a	2.87	1.20	1.1
G	U1	Yes	1,640	1,650	100	2.39	5.75	1.5
	U2	Yes	1,200	1,220	100	1.69	2.23	1.5
	UN1	Yes	1,550	1,560	100	2.25	5.75	1.3
	UN2	Yes	1,080	1,120	100	1.32	1.94	1.3
	E1	Yes	1,630	1,660	n/a	2.76	5.75	1.1
	E2	Yes	650	680	n/a	1.12	1.32	1.1

6.8 STRUCTURAL DESIGN

Structural design calculations are included as Exhibit B to Appendix I.

6.8.1 Methodology

6.8.1.1 Maintenance Bridge

The Maintenance Bridge is a single span slab bridge constructed of cast-in-place concrete. The Maintenance Bridge will span the 22-ft-wide Principal Spillway channel and will be supported by the west pier and east retaining wall. Maintenance staff will have access to the top of the bridge from the proposed embankment crest on the east side of the Principal Spillway. The Maintenance Bridge will utilize a modified Standard ODOT BR-2-15 railing system and is wide enough to allow for the use of a small maintenance vehicle (ATV sized) on the bridge. A gate will be placed at the start of the concrete slab to stop unauthorized vehicles from accessing the bridge.

The Maintenance Bridge is assumed to have a fixed end condition on the retaining wall achieved by running dowel bars into the top of the retaining wall and a propped support condition on the west side. The top of the pier and bridge slab will have a concrete bond breaker applied to allow the bridge to slide along the top of the pier freely to account for any longitudinal movement. The design of the Maintenance Bridge will include loading imparted from the slide gate actuators, either pedestrian live load of 90 psf or H5 vehicular live loading, snow load and dead load of the structure. In addition to typical live loading previously described, an Extreme loading condition was evaluated assuming each of the slide gates are unable to move and the slide gate actuators are considered “stalled” imparting a significantly higher load that will be combined with snow and dead loads.

Maintenance Bridge structural design calculations can be found in Exhibit B – Structural Calculations on pages 56 thru 119.

6.8.1.2 Control Wall

The Control Wall is located perpendicular to the Principal Spillway channel centerline on P3 spanning 22-ft between the west pier wall and the east retaining wall of the structure and anchored to the foundation slab. The Control Wall has two 9-ft by 3-ft openings centered in the Principal Spillway channel with 2-ft between the opening. These openings have slide gates to close off the opening with the actuators being supported on the Maintenance Bridge above.

The Control Wall is assumed to have pin supports on the west pier, east retaining wall and foundation slab. Load cases were developed based on water levels provided by hydraulic analysis at the time of design. The Usual loading condition assumed headwater at an elevation of 802.40 ft and no tailwater, the Unusual load condition was considered headwater at elevation 807.00 ft and no tailwater, and finally the Extreme loading condition was considered to be headwater at elevation 810.00 ft with tailwater at elevation 802.11 ft.

Control Wall structural design calculations can be found in Exhibit B – Structural Calculations on pages 120 thru 154.

6.8.1.3 Debris Rack

The debris rack for the Control Wall and Principal Spillway channel is designed assuming 18-inches clear between members. The design includes a concrete wall in the center of the Principal Spillway channel supporting primary debris rack members spanning perpendicular to the Principal Spillway channel centerline while small debris rack members would span continuously between those members. Spacing of the bar members allowed for an 18-inch clear opening.

Design loading for the debris rack was calculated per Section 6.4.7.2. Debris Rack structural design calculations can be found in Exhibit B – Structural Calculations on pages 3 thru 55.

6.8.1.4 Retaining Walls

The Principal Spillway channel retaining walls located on both the east and west sides of the channel were designed to be cantilevered from the foundation slab. Design of the walls utilized at-rest soil pressures with parameters provided in the geotechnical sections. Retaining walls on all monoliths P1 thru P5 were designed assuming no lateral support from concrete appurtenances within the channel, most notably the Control Wall.

Design loading for the Principal Spillway channel retaining walls utilized three different loading conditions. The Usual case which included groundwater to the top of the foundation slab (EL. 783.40 ft) and Principal

Spillway channel dry, an Unusual condition which assumed groundwater to top of the foundation slab (EL. 783.40 ft) and a 300 psf live load surcharge on the retained soil, and finally an Extreme loading condition which is considered a post-flood condition where ground water reaches maximum PMF elevation of EL. 810.00 ft and the Principal Spillway channel is dry.

Retaining Wall structural design calculations can be found in Exhibit B – Structural Calculations on pages 155 thru 334.

6.8.2 Load Combinations

6.8.2.1 Maintenance Bridge

Maintenance Bridge Load Cases	
1	1.2 * Dead Load + 1.6 * Pedestrian Live Load + 0.2 * Snow Load
2	1.2 * Dead Load + 1.6 * Snow + Pedestrian Live Load
3	1.2 * Dead Load + 1.2 * Extreme Load + 0.5 * Snow
4	1.2 * Dead Load + 1.6 * H5 Vehicular Load + 0.5 * Snow

6.8.2.2 Control Wall

Control Wall Load Cases	
1	1.2 * Dead Load + 1.6 * Usual Water Load
2	1.2 * Dead Load + 1.6 * Unusual Water Load
3	1.2 * Dead Load + 1.3 * Extreme Water Load

6.8.2.3 Debris Rack

Debris Rack Load Cases	
1	1.2 * Dead Load + 1.6 * Debris Load

6.8.2.4 Retaining Walls

Retaining Wall Load Cases	
1	1.6 * Dead Load + 1.6 * Earth Load* + 1.6 * Live Load – See Note
Note: Load cases follow Army Corps of Engineers load cases for unusual due to inclusion of live load surcharge behind walls.	

6.9 MECHANICAL DESIGN

The ECFB is designed to operate with two static orifice openings of 9 feet wide by 2 feet 5 inches tall. To increase operational flexibility, facilitate testing of a first filling event, and enhance maintenance capabilities, the structural openings were each increased to 9 feet wide by 3 feet high and the structure outfitted with two fabricated stainless-steel gates mounted on the Principal Spillway Control Wall. Each gate will be operated by an electric operator (screw stem type) with double stems that can mounted on the Maintenance Bridge at a deck elevation of 813.0 feet.

The gates can be operated by the electric operator, via a corded drill rack, or by hand wheel operation. Gate speed during opening and closing operation will be approximately 0.5 feet per minute. A transparent pipe stem cover with graduated markings corresponding to the gate travel in feet and inches will be included to monitor the opening height of the gates. The typical gate setting is based on the design orifice opening of 2 feet 5 inches.

The gates are designed for a maximum head of 26 feet corresponding to maximum reservoir pool elevation of 810.0 feet. Besides hydrostatic head, operating forces on the gate from the gate operator under both normal conditions and breakdown torque conditions (BDT) (overload case) are considered.

6.10 CONSTRUCTION CONSIDERATIONS

The following construction considerations are noted:

- Dewatering of excavated areas will be required to sufficiently enable construction of the Principal Spillway.
- Foundation preparation will require care during excavation to identify unsuitable conditions or weak bedding planes that could impact stability.
- Concrete placement that qualifies as mass concrete will require monitoring to control heat of hydration and reduce crack potential.
- Due to a concentration of H2S found at project site, concrete with an H2S resisting additive is required in locations as specified in the contract specifications.
- Horizontal joints in the retaining wall stems may be required to reduce placement height to minimize aggregate separation, improve access for adequate vibration, and reduce potential for form bulging.
- Joint preparation will require attention to proper installation of water stops, dowels, and reinforcement. Joint alignment and water-tight integrity are critical for spillway construction.
- Fill placement and compaction methods must be reviewed and monitored to ensure wall movement does not occur during construction.

7.0 AUXILIARY SPILLWAY

The Auxiliary Spillway is a secondary structure used to safely discharge flows exceeding the 1% ACE (100-year) flood event. The spillway has additional discharge capacity to safely pass flows up to the design flood (PMF). A summary of the design configuration is presented below. Detailed design discussion and calculations are included in the Auxiliary Spillway Design Technical Memorandum in Appendix J.

7.1 GENERAL ARRANGEMENT

The Auxiliary Spillway is comprised of a concrete labyrinth weir with a downstream riprap armored stilling basin. The labyrinth weir geometry provides for greater discharge capacity per length of spillway compared to a linear spillway with the same structure width. The stilling basin discharges at existing grade, at which point flow will continue downstream to Eagle Creek. The Auxiliary Spillway crest is set to elevation 807.0 ft, which is approximately the 100-year water surface elevation.

The structure is to be constructed at-grade with a cast-in-place concrete abutment wall that transitions back to the earthen embankment dam on the left (northwest) side. The labyrinth weir structure is integrated with the Principal Spillway on the right (southeast) side with a shared training wall. Figure 29 shows a typical section of the labyrinth spillway. Figure 30 shows a plan view of the AS.

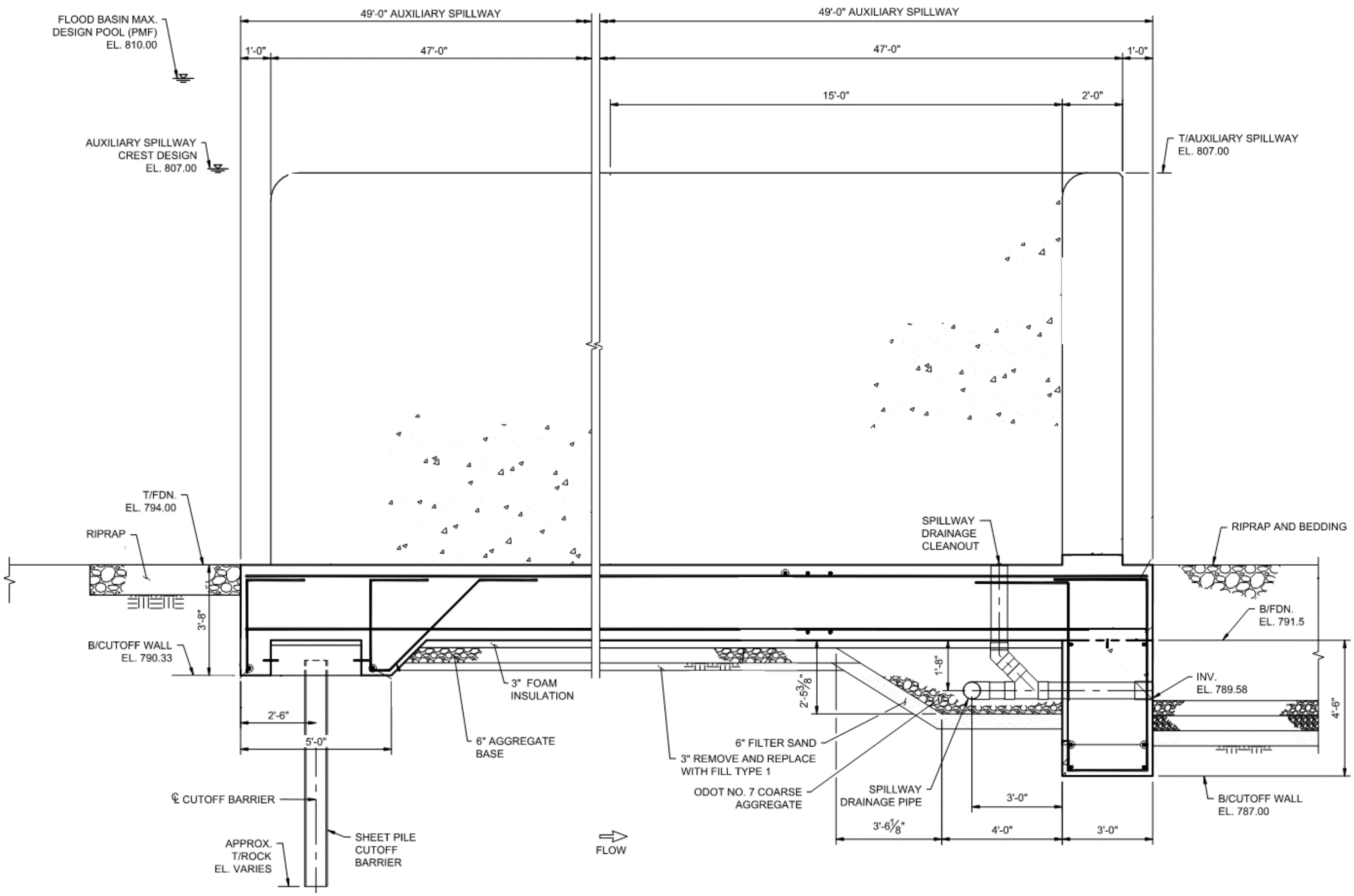

Figure 29. Typical Labyrinth Spillway Section

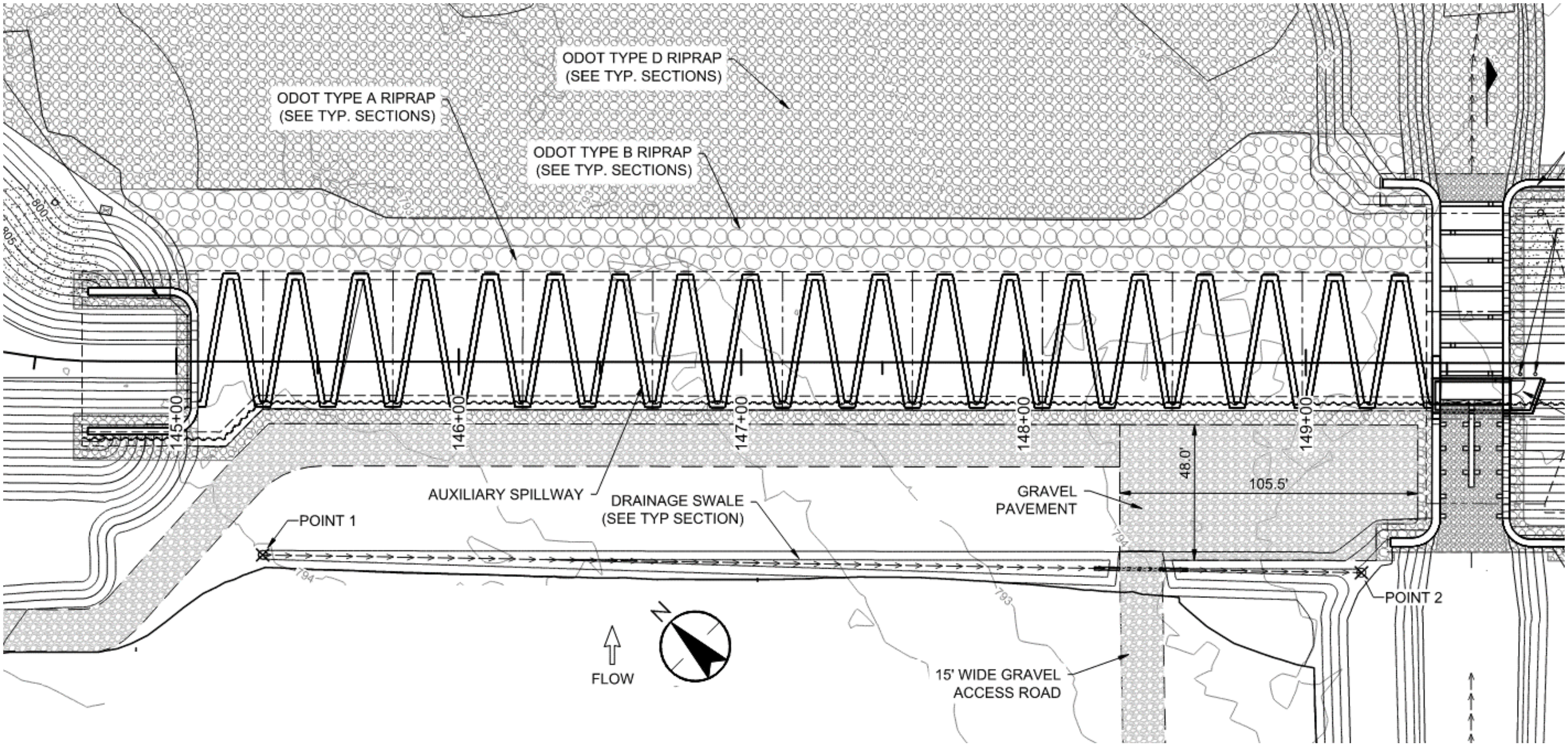

Figure 30. Auxiliary Spillway Plan View

7.2 DESIGN OBJECTIVES

The maximum discharge capacity of the Auxiliary Spillway is 27,450 cfs and is designed to pass the Probable Maximum Flood (PMF) producing a peak water surface elevation of 810.0 ft. The maximum pool elevation of 810.0 ft during the PMF equates to a design head of 3 ft above the Auxiliary Spillway structure crest with sufficient freeboard to the dam crest without overtopping due to passage of the design flood and other factors including wind and wave run up. The PMF study used in the design of the Auxiliary Spillway is described within the ECFB Hydrologic and Hydraulic Analysis Report, Appendix D.

7.3 LOCATION

The Auxiliary Spillway is situated in the northeast section of the ECFB dam embankment. The labyrinth weir is integrated with the Principal Spillway and is aligned in the northwest-southeast direction, perpendicular to direction of flow in Eagle Creek. The location is approximately 250 feet west of the existing Eagle Creek channel.

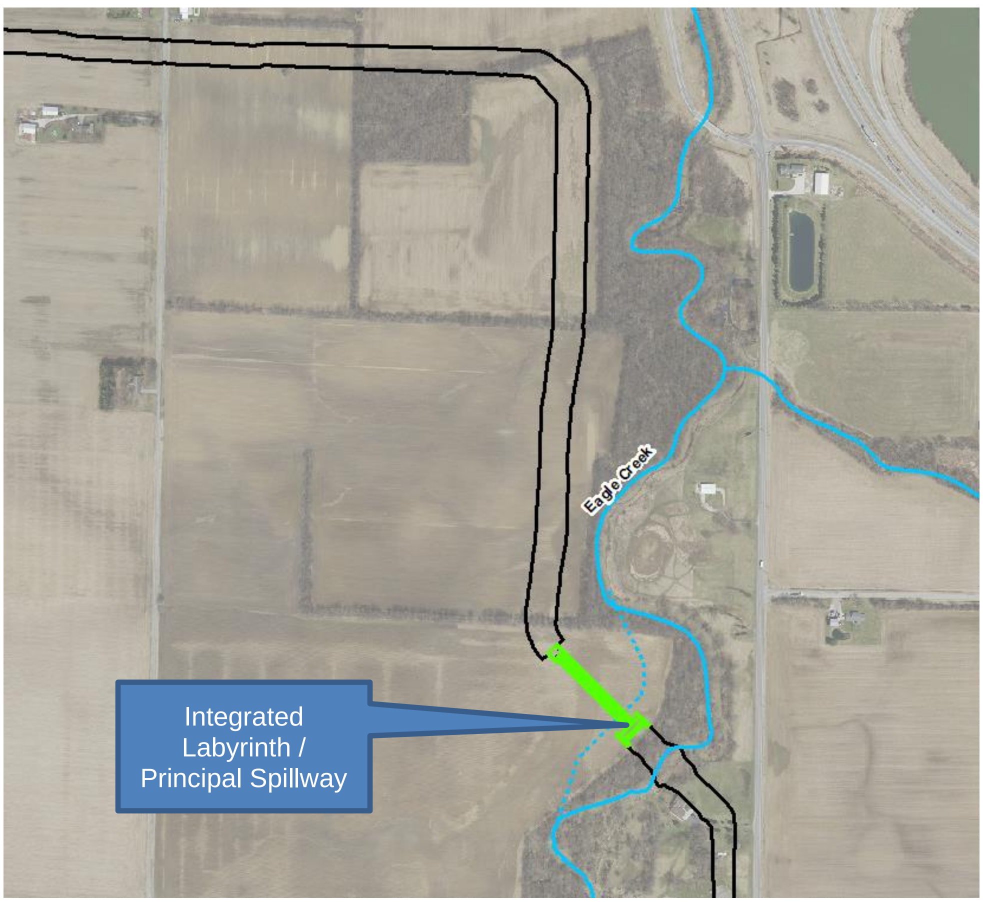

Figure 31. Auxiliary Spillway Location

7.4 WEIR GEOMETRY

A labyrinth spillway is a linear weir folded in plan-view to increase the effective length of the weir. A labyrinth weir can pass larger discharges at relatively low heads compared to traditional linear weirs of equal structure width. At low heads, a labyrinth weir behaves similarly to a linear weir. As head increases, the discharge efficiency begins to decline as nappe collision and submergence regions develop. Due to their hydraulic performance and geometric versatility, labyrinth weirs are well-suited for a variety of applications. In this case, a labyrinth weir was selected because low overtopping head reduced upstream impacts and the more compact shape of a labyrinth weir provided greater value in terms of estimated construction cost when compared to other alternatives which were considered.

The weir is a 13-ft-tall, 2-ft-thick reinforced concrete wall, comprising nineteen (19) labyrinth cycles. Each cycle is 47 ft deep and 23 ft wide and together produce a total structure width of 437 ft with an effective crest length of 1,672 ft. The reinforced concrete apron slab is 2.5 ft thick and 49 ft deep (in the direction of flow). Design drawings for the Auxiliary Spillway are included in the Final Design Drawings, Appendix B.

7.5 HYDRAULIC DESIGN

The hydraulic design of the Auxiliary Spillway includes the geometry of the spillway, an energy dissipation component, and the transition into the downstream channel. The Auxiliary Spillway is designed to activate for flood events greater than the 1% ACE (100-year) event and to convey flows safely up to the PMF event. Sufficient freeboard, as required by the State of Ohio Dam Safety regulations (OAC Section 1501:21-13-03), is included as part of the designed embankment crest elevation. The crest elevation is discussed in Section 5.5.

7.5.1 Labyrinth Crest Length and Rating Curve

Detailed computations for the labyrinth spillway design are provided in Exhibit A of Appendix J. The discharge capacity of the labyrinth weir was calculated using guidance from the American Society of Civil Engineers (ASCE) *Hydraulic Design and Analysis of Labyrinth Weirs I: Discharge Relationships*, by B.M. Crookston and B.P. Tullis (2013). Discharge over the crest is dependent upon the discharge coefficient, weir length, and upstream driving head. Equation 1 (Crookston and Tullis, 2013) represents the discharge relationship of the weir.

The inputs required (labyrinth weir geometry) are used to calculate several labyrinth weir ratios for the desired output (discharge). The weir ratios must be within an acceptable range based on experimental data from physical modeling and previous design studies. Weir geometry inputs used in the calculations are shown in Table 41 and computed weir geometries are shown in Table 42. The computations assume an ogee-shaped weir crest. Detailed computations for the labyrinth spillway design are provided in Exhibit A of Appendix J.

Table 41. Labyrinth Geometry Inputs

Description	Symbol	Integrated Labyrinth	Unit
Discharge	Q	27,400	cfs
Max pool elevation	EL_{pool}	810.0	ft
Spillway crest elevation	EL_{crest}	807.0	ft
Upstream slab elevation	EL_{slab}	794.0	ft
Wall height	P	13.0	ft
Wall thickness	t	24.0	in
Number of cycles	N	19	
Cycle width	w	23.0	ft
Apex inside width	A_D	24.0	in
Apex outside width	A_U	5.365	ft
Cycle Depth	B	47.0	ft

Table 42. Labyrinth Geometry Computations

Description	Symbol	Integrated Labyrinth	Unit
Design head	H_T	3.0	ft
Total spillway width	W_t	437.0	ft
Cycle half width	C	7.82	ft
Effective cycle depth	D	45.0	ft
Sidewall angle	α	9.855	degrees
Actual sidewall length	L_A	45.67	ft
Effective sidewall length	L_e	43.99	ft
Total sidewall length	L_T	49.36	ft
Total effective crest length	L_C	1,672	ft
Discharge coefficient	$C_{d(\alpha°)}$	0.5265	
Total discharge capacity	Q_T	27,450	cfs

Figure 32 shows a general schematic of a single weir cycle.

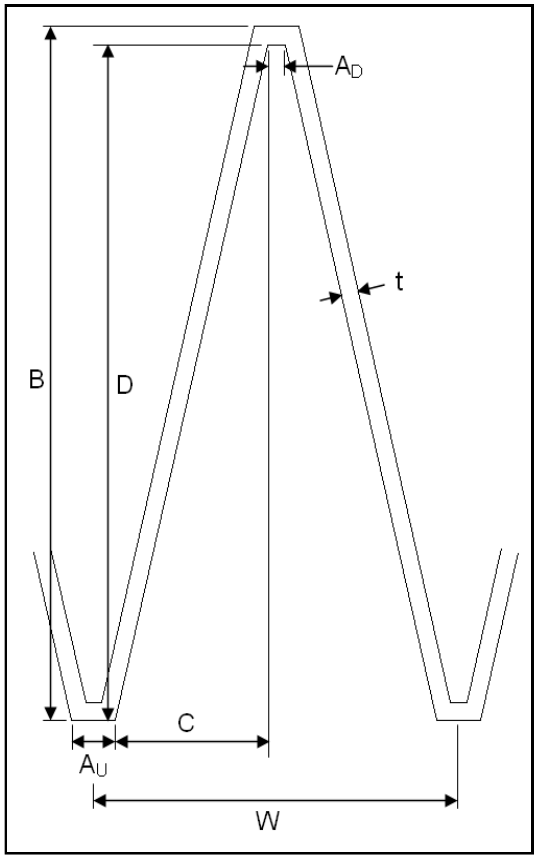

Figure 32. Labyrinth Weir Single Cycle Schematic

A discharge rating curve was computed for the labyrinth weir using the selected weir geometry and Equation 1 (Crookston and Tullis, 2013). The rating curve is shown in Table 43 and Figure 33. The rating curve was compared to Computational Fluid Dynamics (CFD) modeling results at a headwater elevation of 810.0 ft and 807.8 ft. The average discharge over a single cycle in the CFD model was calculated and multiplied by 19 to compare with the computed rating curve. The CFD model results were within 2% of flow at both comparison points verifying the calculated rating curve. The CFD model analysis is discussed in more detail in the ECFB Hydrologic and Hydraulic Analysis Report, Appendix D. Performance of the spillway during simulated design flood events is presented in Section 8.0.

Table 43. Labyrinth Spillway Rating Curve Table

Pool Elevation (ft)	Integrated Labyrinth Discharge, Q (cfs)
807.00	0
807.25	626
807.50	1,962
807.75	3,766
808.00	5,901
808.25	8,275
808.50	10,818
808.75	13,479
809.00	16,218
809.25	19,006
809.50	21,818
809.75	24,637
810.00	27,450

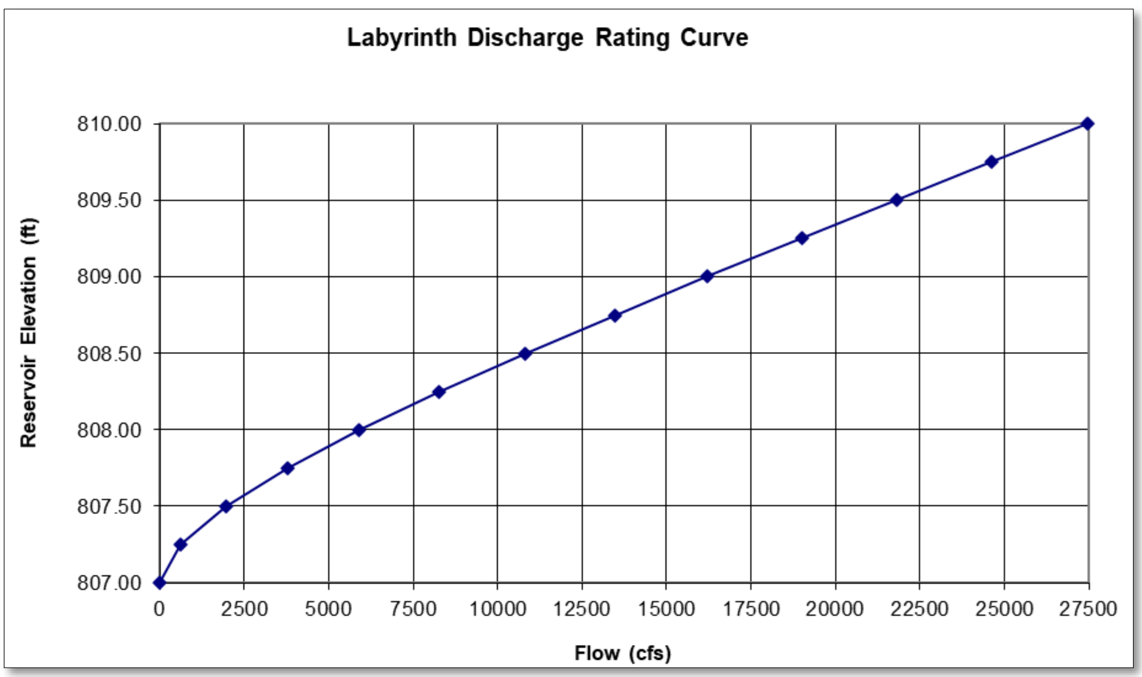

Figure 33. Labyrinth Spillway Rating Curve

7.5.2 Energy Dissipation

The purpose of the stilling basin is to reduce the flow velocities, suppress waves, and dissipate energy to reduce the potential for erosion.

The labyrinth weir is designed as a vertical drop to a horizontal stilling basin. Energy is first dissipated by the drop from the spillway crest to the stilling basin floor. The stilling basin is then designed considering the relative residual energy at the base of the labyrinth weir. The relative residual energy can be calculated using Equation 2 from Matos and Chanson's *Discharge Capacity and Residual Energy of Labyrinth Weirs* (2006). The equation is based on the relative total upstream head (H / P) and the magnification ratio (L / W). The relative residual energy is then used to calculate the residual energy at the base of the labyrinth. A summary of the residual energy calculations for the PMF event is in Table 44.

Table 44. Residual Energy at Base of Labyrinth

Description	Symbol	Measurement	Unit
Total discharge	Q	27,450	cfs
Spillway height	P	13.0	ft
Total upstream head	H_0	16.0	ft
Total head over crest	H	3.0	ft
Relative total upstream head	H/P	0.23	
Labyrinth spillway width	W	437	ft
Effective length of crest	L	1,672	ft
Magnification ratio	L/W	3.83	
Relative residual energy	H_1/H_0	0.47	
Residual energy at base	H_1	7.45	ft

A hydraulic jump is used to dissipate the energy within the basin, and it is defined by equation 2-26 in EM 1110-2-1603 (USACE, 1992). The flow depth at the toe is calculated using solver in Excel. Using the calculated sequent depth, the sequent velocity, Froude number, and conjugate depth can be calculated. The conjugate elevation and tailwater elevation are used to determine if the hydraulic jump is stable. Tailwater elevations were extracted from the HEC-RAS modeling results. The model used to develop these results is described in Appendix D. From EM 1110-2-1605 (USACE, 1987), the hydraulic jump is stable if the ratio of the tailwater depth to the conjugate depth is greater than one. To calculate the length of the hydraulic jump, Equation 2-29 from EM 1110-2-1603 is used.

The required length of the stilling basin was checked using Figure 12 from Engineering Monograph 25 (USBR, 1984). Since the calculated Froude number (Table 45) is less than 2, the minimum ratio of basin length to conjugate depth (L/d_2) is 4, which results in a 20 ft long basin. Typically, a horizontal apron, or natural jump basin, is constructed of concrete; however, given the lower velocities it was determined that a riprap apron meets the design criteria. See Section 7.5.3 for details regarding sizing of riprap downstream of the Auxiliary Spillway. A summary of the stilling basin calculations is presented in Table 45.

Table 45. Labyrinth Stilling Basin Calculations

Description	Symbol	Measurement	Unit
Unit discharge	Q	62.8	cfs
Residual energy	H_1	7.45	ft
Sequent depth	d_{1_super}	4.97	ft
Sequent velocity	v_{1_super}	12.65	ft/s
Froude number	F_1	1.00	
Conjugate depth	d_2	4.97	ft
Conjugate elevation	EL_{d2}	798.97	ft
Tailwater elevation	EL_{TW}	802.45	ft
Tailwater depth	d_{TW}	8.45	ft
Difference in elevation	$EL_{TW} - EL_{d2}$	3.49	ft
Tailwater ratio	d_{TW} / d_2	1.70	
Length of hydraulic jump	L_j	17.4	ft
Ratio of basin length to d_2	L / d_2	4	
USBR basin length	L_b	19.9	ft

Detailed calculations for the labyrinth spillway and energy dissipator design are provided in Exhibit A of Appendix J.

The jump stability calculations for shallow overtopping of the Auxiliary Spillway suggests that the downstream tailwater may not be high enough to produce a stable jump, however the USBR calculations do not take the roughness of the riprap and shallow flow routing downstream of the stilling basin into account. Results of the CFD modeling show that the jump will be stable even during shallow overtopping.

7.5.3 Downstream Armoring

When flow is discharged through the Auxiliary Spillway, it travels overland before entering into the Eagle Creek channel. Flows will be significant across this relatively flat transition area between the Auxiliary Spillway and the Eagle Creek channel. Scour mitigation measures are designed to reduce the risk of erosion of the native soils and the formation of a headcut that could potentially undermine the integrity of the labyrinth spillway structure.

A riprap blanket is included downstream of the Auxiliary Spillway concrete slab for a sufficient distance to reduce the risk of headcutting. A thickened riprap toe is included at the downstream end of the riprap blanket. Downstream of the riprap toe, a low-maintenance grass seed with turf-reinforcement mat is included between the riprap toe and Eagle Creek channel.

A plunging affect occurs immediately downstream of the Auxiliary Spillway due to flow falling from the top of the labyrinth cycles. This plunging has the potential to scour and erode the underlying soils adjacent to the spillway without armoring. This is primarily a concern at the ‘nose’ of the labyrinth cycles because other areas discharge on top of the concrete spillway foundation. The thickness of the riprap blanket downstream of the Auxiliary Spillway was established to account for the potential plunging/jetting affect and additional potential for scour.

The size and thickness of the riprap blanket downstream of the Auxiliary Spillway was determined using Equation 3-3 from USACE Engineering Manual 1110-2-1601*, Hydraulic Design of Flood Control Channels*. Velocity and depth values were extracted from the HEC-RAS modeling results as described in the *Eagle Creek Flood Basin – Hydrologic and Hydraulic Analysis Report,* Appendix D. Where the CFD model domain included the area downstream of the spillway, results from the CFD model were also used. The more critical combination of depth-averaged velocity and depth from the CFD and HEC-RAS model results were used for design.

A plunging affect occurs immediately downstream of the Auxiliary Spillway due to flow falling from the top of the labyrinth cycles. This plunging has the potential to scour and erode the underlying soils adjacent to the spillway without armoring. This is primarily a concern at the ‘nose’ of the labyrinth cycles because other areas discharge on top of the concrete spillway foundation. The thickness of the riprap blanket downstream of the Auxiliary Spillway was established to account for the potential plunging/jetting affect and additional potential for scour.

A grid of calculation points distributed in the area downstream of the Auxiliary Spillway was used to determine the size and extents of riprap downstream of the spillway. Velocities immediately adjacent to spillway are higher and require larger riprap. The design includes a layer of ODOT Type A riprap for 9-ft downstream of the labyrinth concrete foundation. A layer of ODOT Type B riprap then extends downstream from the Type A riprap for variable lengths. The variation in length is due to the direction of flow and areas of higher velocities occurring downstream of the labyrinth abutment walls. Beyond this zone, the design includes a layer of Type D riprap for approximately 100-ft. A typical detail is shown on Figure 34. The thickness of the riprap layer is determined using the USACE EM 1110-2-1601 criteria (Section 3-2e).

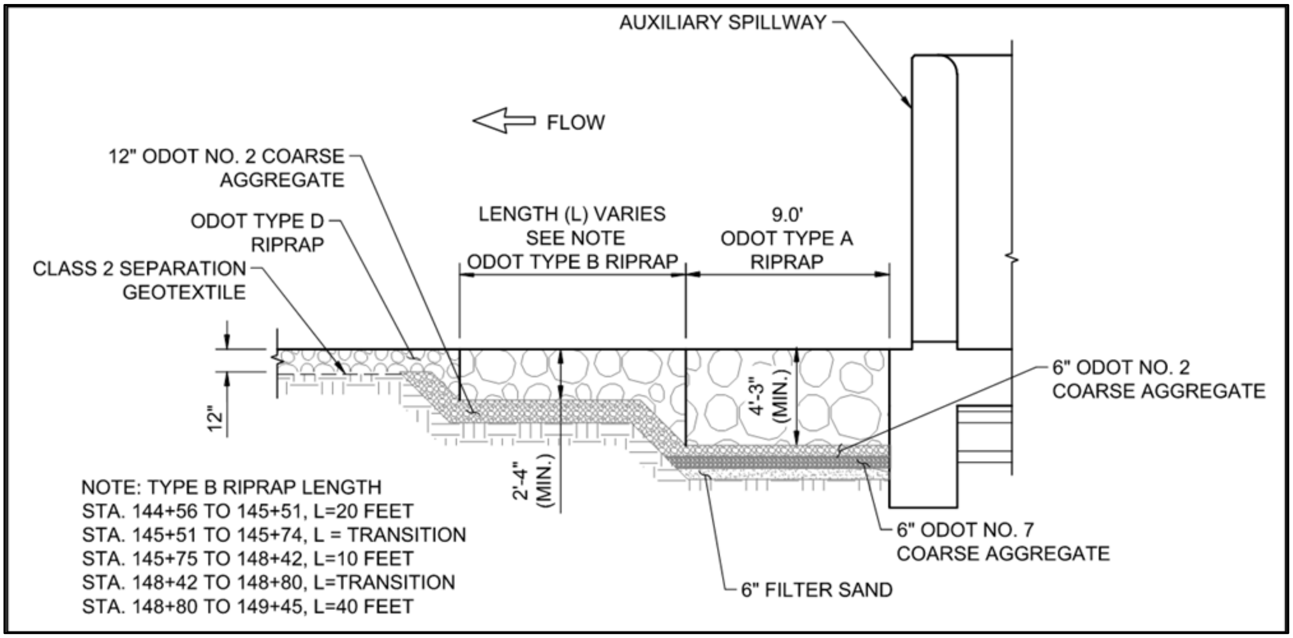

Figure 34. Auxiliary Spillway Riprap Armoring

The ODOT Type D riprap blanket terminates with a thickened riprap toe treatment. The purpose of the thickened toe is to reduce the risk of a headcut forming due to scour at the transition between riprap and grass. Should scour develop at this transition area, the riprap will ‘launch’ and form a stable slope to the point where velocities are reduced, and scour is mitigated.

The recommended design procedure outlined in US Army Corps of Engineers Technical Report HL-95-11 *Toe Scour and Bank Protection Using Launchable Stone, dated September 1995* was used to determine the end treatment dimensions. The riprap end treatment detail is shown on Figure 35. Calculations for riprap sizing and end treatment detail can be found in Exhibit A.2 of Appendix D.

The design also includes a turf reinforcement mat with low-maintenance turf-type grass downstream of the thickened toe. The Threshold Design Criteria described in Chapter 8 of the National Resources Conservation Services *Part 643 Stream Restoration Design National Engineering Handbook* was used to determine the extents of riprap armoring. Where velocities are less than 5 ft/s, turf reinforcing mat (TRM) is used. TRM extends from the riprap toe to the edge of the riparian planting zone along the channel. The relationship between flow duration and nondegradable soil reinforcement products with grass is shown in Figure 36. Although the table indicates turf reinforcement mat should be able to withstand velocities above 5 ft/s, this value was chosen due to the critical nature of the Auxiliary Spillway and potential impacts of a headcut forming due to erosion.

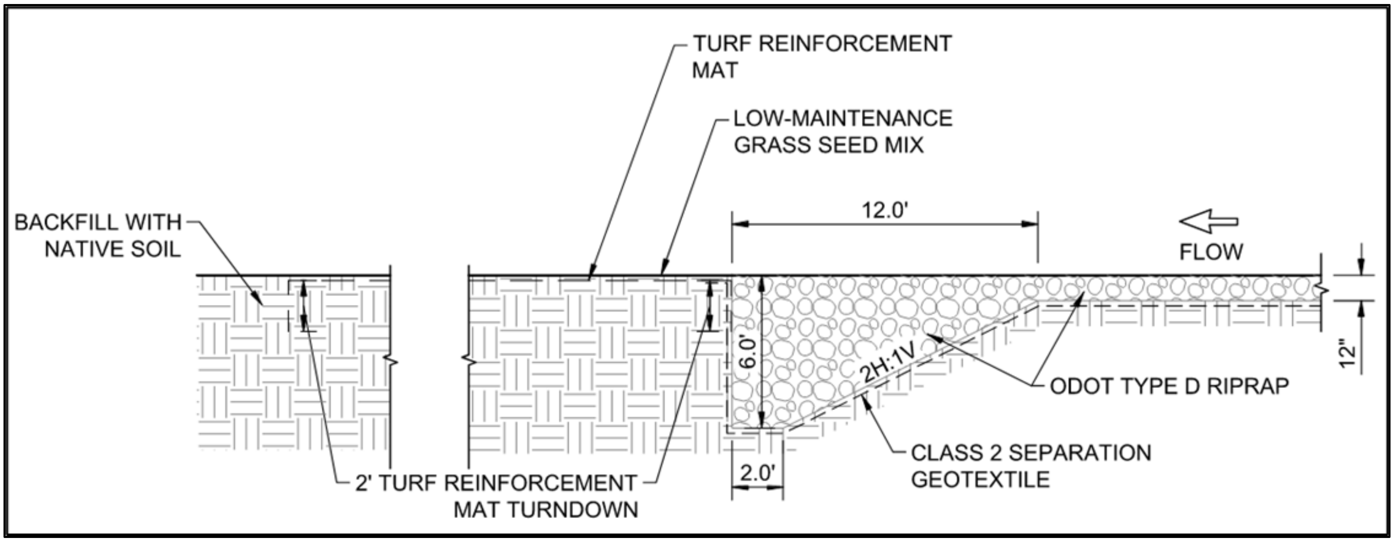

Figure 35. Riprap Blanket End Treatment

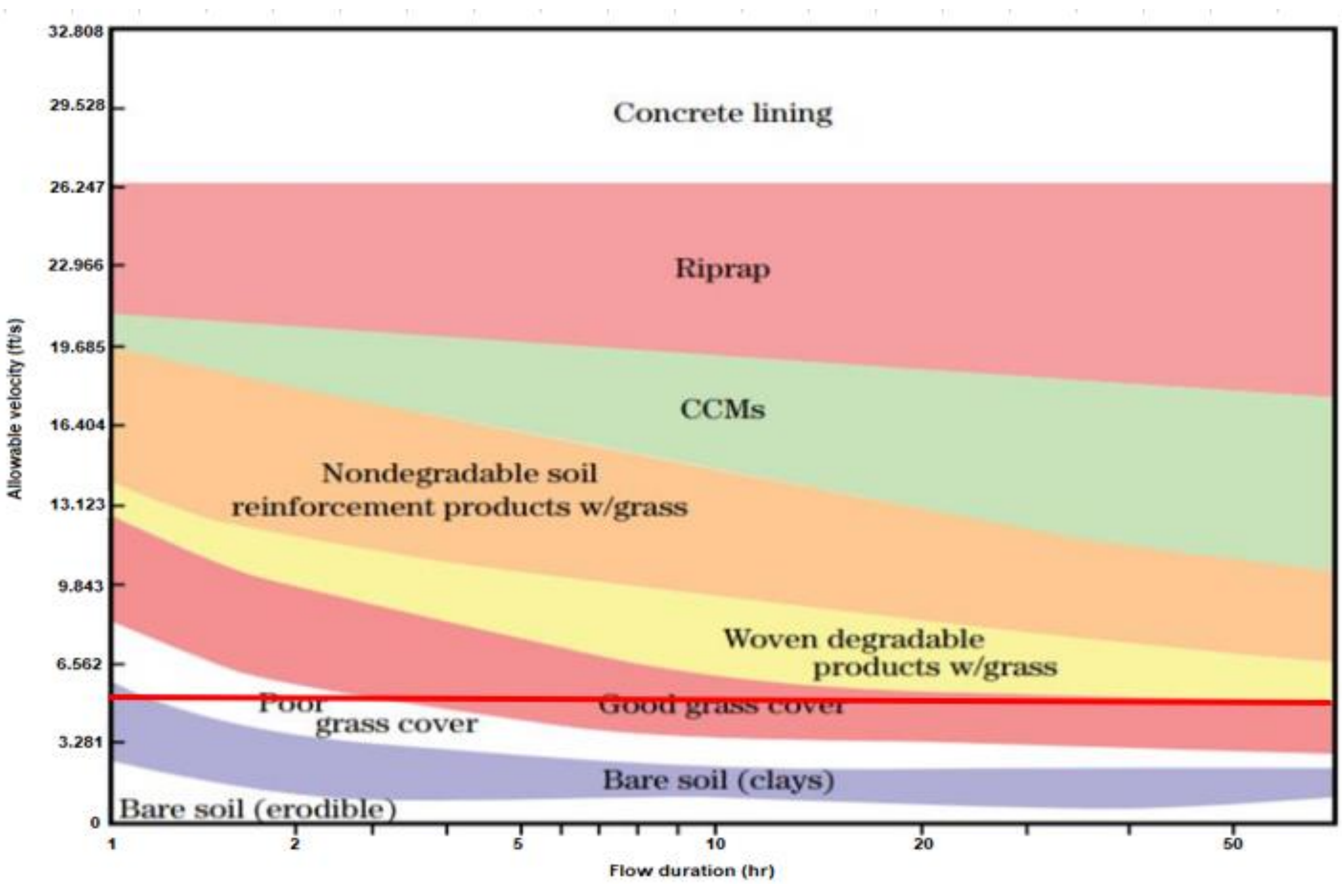

Figure 36. Velocity Threshold Design Criteria

7.6 GEOTECHNICAL CONSIDERATIONS

To characterize the subsurface conditions of the Auxiliary Spillway, borings B-3.46 through B-3.47, B-4.11 through B-4.14, and B-4.21 through B-4.22 were conducted in the vicinity of the Auxiliary Spillway location as shown in Figure 24. Details of the exploration and logs of the borings are available in the Geotechnical Design Report, Appendix E.

The subsurface soils encountered at the Auxiliary Spillway location include alluvial and glacial till deposits near Eagle Creek. Soils encountered consisted of Upper Fine-Grained, Upper Coarse-Grained, Lower Fine-Grained, and Lower Coarse-Grained horizons as defined in the Geotechnical Design Report (Appendix E). The base slab of the Auxiliary Spillway is founded at elevation 791.5 feet, which is about 1 to 2.5 feet below the existing site grade. The encountered top of rock elevation is approximately 774 feet to 777 feet for both the Principal and Auxiliary Spillways. Figure 25 shows the subsurface stratigraphy at the Auxiliary Spillway location.

7.6.1 Foundation and Lateral Earth Pressure Parameters

The subsurface information and test data were used to develop the following geotechnical parameters for final design of the Auxiliary Spillway:

- Allowable bearing capacity of 2,100 psf. This estimate was based on the size and elevation of the structure's foundation.
- Subgrade modulus of 100 lb/in^3 based on guidance from UFC (2005), Table 4-1.
- Interface friction angle of 17 degrees between concrete and native Upper Fine-Grained soil, and 26.5 degrees between concrete and ODOT Item 304 Aggregate Base. These values are based on recommendations in NAVFAC (1986), Table 1.
- Undrained shear strength parameters for native Upper Fine-Grained soil: 20-degree friction angle (ϕ) and 400 psf cohesion (c), based on laboratory test results and characterization.
- The abutment walls were designed for at-rest earth pressure conditions. Lateral loads applied to the Auxiliary Spillway abutment were calculated considering compacted fill placed against the abutment walls. Based on laboratory test results and characterization, the drained strength friction angle (ϕ') of the compacted fill is 33 degrees. The estimated at-rest earth pressure coefficient is $K_0 = 0.455$.
- Frost depth for the project site is 36 inches. Because the Auxiliary Spillway foundation is at the final ground surface and only 2.5 feet thick, a layer of structural foam insulation is recommended.

7.6.2 Groundwater Corrosivity

As discussed in Section 4.4.13, groundwater at the project site could be corrosive to concrete and steel due to the presence of naturally occurring hydrogen sulfide. To account for this in the design, microcrystalline additives are included in the requirements for subsurface concrete. Additionally, seepage cutoff walls will be PVC sheet piles, rather than steel.

7.6.3 Seepage and Uplift

The Auxiliary Spillway will be located in an area with alluvial, coarse-grained soil deposits. These subsurface materials are susceptible to seepage below and around the structure. Details of the seepage analyses performed on the Auxiliary Spillway are available in the Geotechnical Design Report, Appendix E.

A PVC sheet pile wall is included as a seepage cutoff below the structure, extending to the top of rock. The sheet pile is specified as a box profile with a section width of 23.9 inches, depth of 7.1 inches, and thickness of 0.25 inches. The sheets will include a soft PVC joint gasket that is coextruded with the sheet to reduce potential seepage through the joints (interlocks). The sheet pile wall is located under the Auxiliary Spillway foundation slab. As discussed in Section 7.6.4, minimal settlements are anticipated, reducing the risk of damage to the sheet piles. However, if settlements do occur, the connection at the top of the sheet pile to the concrete foundation includes a layer of structural foam that will allow for some compression.

The Auxiliary Spillway and abutment include an underdrain pipe on the downstream end of the slab to relieve uplift pressures at the toe and provide a filtered exit for potential seepage. The underdrain pipe is a perforated 6-inch diameter Schedule 80 PVC, encased in a graded aggregate filter. Unperforated 6-inch Schedule 80 PVC outlet pipes are provided at each downstream apex of the labyrinth. Cleanout of the underdrain system will be possible through the outlets, as well as from an additional drain system cleanout on the left side of the spillway abutment. Figure 37 presents the details of the seepage cutoff and underdrain system.

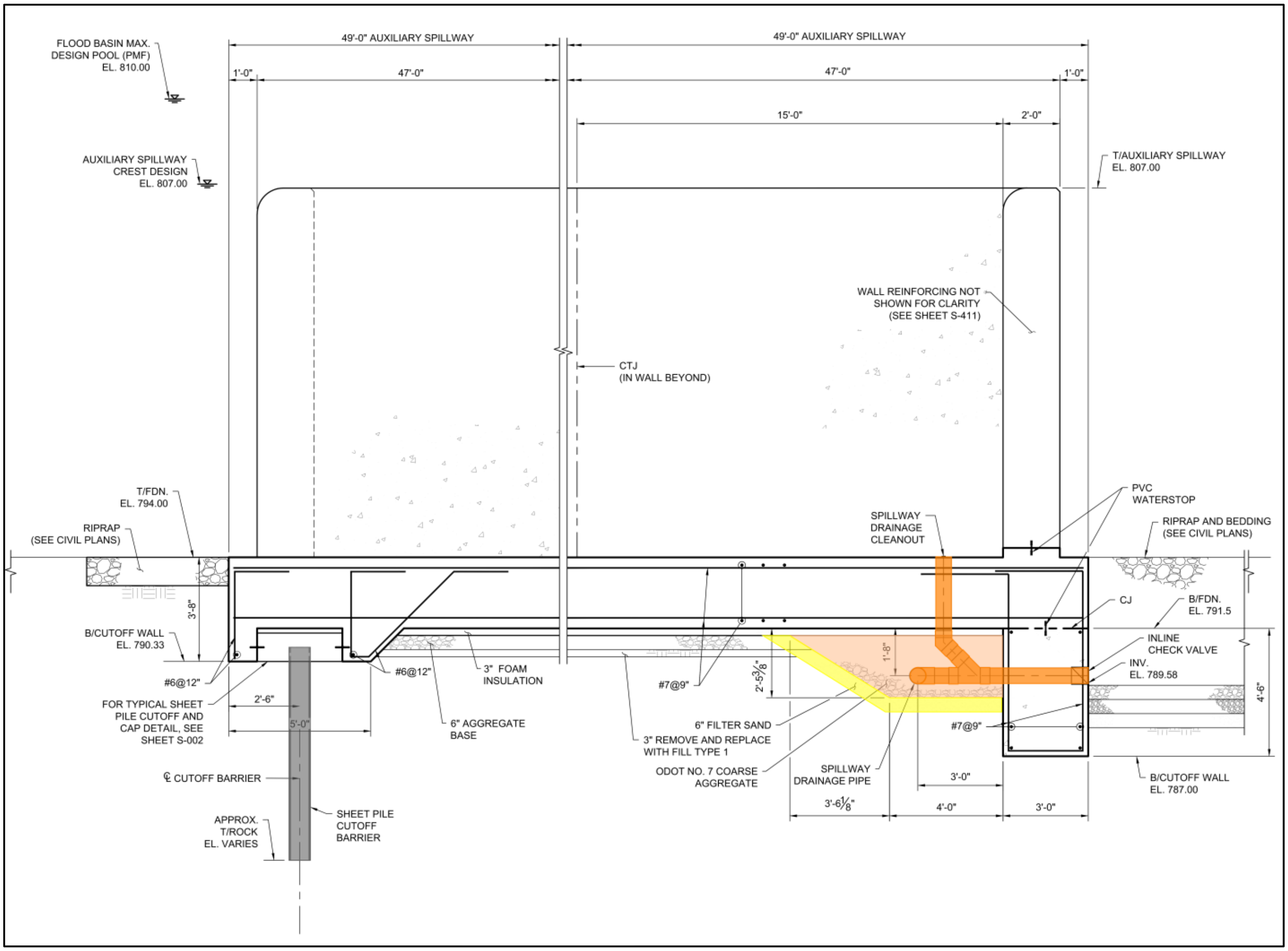

Figure 37. Auxiliary Spillway Seepage Cutoff and Underdrain System

Seepage could also develop along the soil-structure interface along the sides of the spillway abutment. To mitigate this risk, the soil side of the spillway abutment walls will be battered outward to improve soil compaction against the concrete. Additionally, a sand filter-drain will be incorporated into the backfill and tied into the embankment filter-drain system. The filter-drain will be provided with a pipe outlet to relieve pressures and drain seepage from along the spillway contact. The chimney, blanket, and toe drain system for the embankment will connect to the downstream Auxiliary Spillway abutment wall. The chimney drain will be extended from elevation 800.0 ft down to the bottom of the foundation slab, to provide a diaphragm filter for seepage that may develop along the concrete-soil interface. The gradation of the drainage material (Filter Sand) was selected to filter and retain the embankment soils (Fill Type 1), thereby protecting against soil erosion along the wall. As additional protection, the spillway abutment includes curved walls that effectively lengthen the seepage path along the concrete-soil interface.

Seepage analyses were completed using GeoStudio SEEP/W 2018 R2, finite element software. To support the load cases analyzed for structural stability, four uplift pressure profiles for the Auxiliary Spillway were developed using SEEP/W results.

Factors of safety for exit gradients downstream of the Auxiliary Spillway were calculated using the seepage force and total stress methods. Exit gradients were evaluated assuming the seepage cutoff and drainage system are effective. The underdrain pipe was assigned total head boundary conditions equal to the drain outlet pipe elevation or the tailwater elevation, whichever was higher for the case being evaluated. Acceptable factors of safety were calculated along the downstream toe of the Auxiliary Spillway. Details of the seepage analyses performed on the Auxiliary Spillway are available in the Geotechnical Design Report, Appendix E.

The uplift profiles were developed according to the structural design load cases. Here, the seepage cutoff was modeled as 50% effective, and separate uplift profiles were developed for a 50% functional and a 0% functional underdrain system. Descriptions for how this was modeled, and discussion of the results, can be found in the Geotechnical Design Report, Appendix E. Based on the analyses, the sheet pile wall and drainage system meet criteria for the seepage analyses. Downstream of the spillway, the armoring includes a 10-ft-wide graded filter consisting of Filter Sand, ODOT No. 7 Coarse Aggregate, and ODOT No. 2 Coarse Aggregate below the erosion control riprap. Additionally, a 3-ft wide by 7-ft-deep concrete wall is included below the downstream toe of the Auxiliary Spillway. This wall reduces the risk for undermining the downstream toe from turbulent flows overtopping the Auxiliary Spillway. See Figure 34 for a detail.

7.6.4 Settlement

Settlement of the Auxiliary Spillway and abutment foundation was evaluated using the program Settle3 by Rocscience. Settlement estimates were calculated in Settle3 at various elevations below the bottom of the foundation to evaluate the influence of removing the upper soils and replacing with engineered backfill with limited compressibility. Removal of soil to elevation 790.5 ft (1-ft below the bottom of foundation) for the labyrinth slab was modeled. For this design, the maximum estimated total (ultimate) settlement was about 0.9 inches for both the labyrinth and abutment slabs. However, the maximum differential settlement was estimated to be 0.4 inches across the abutment foundation. The Auxiliary Spillway foundation slabs are 2.5 feet thick and connected with stainless steel dowels to keep them aligned with one another. The estimated settlements were judged to be acceptable if the upper 1-ft of soil for the labyrinth slabs and upper 3-ft of soil for the abutment slab is removed and replaced with the foam insulation, ODOT 304 Base Aggregate, and/or compacted fill to the bottom of slab elevation.

To evaluate the settlements that may occur during construction of the structure, the Settle3 model included a time step at 0.1 years after the load placement. Based on the results, 70 to 90 percent of the total settlement occurs during the first 0.1 year, with only about 0.25 inches settlement remaining after the first 0.1 years. This indicates that most settlement will occur during construction of the spillway, and the remaining settlement is tolerable.

Details of the settlement analyses are provided in the Geotechnical Design Report, Appendix E.

7.6.5 Seismic Design

7.6.5.1 Site Class

The Seismic Site Class was estimated per the American Society of Civil Engineers (ASCE) Standard 7-16 design manual. The concrete Auxiliary Spillway structure is expected to have a fundamental period of vibration lower than 0.5 seconds. Standard Penetration Test (SPT) data from the geotechnical exploration was used to characterize the site as Class C.

7.6.5.2 Liquefaction Triggering

The borings advanced near the proposed Auxiliary Spillway structure location were included in the liquefaction triggering analyses for the embankment (Section 5.10). Liquefaction is not expected to be triggered during the design earthquake event at the project site.

7.7 STABILITY

The overflow weir of both the labyrinth Auxiliary Spillway and the abutment walls of the labyrinth weir were analyzed for stability. The analyzed forces included overturning and bearing stress, sliding forces, and floatation forces.

7.7.1 Acceptance Criteria

All structural elements of the Auxiliary Spillway are assumed to be critical structures as failure in the spillway could directly or indirectly lead to a loss of life. The following acceptance criteria are based on EM 1110-2-2502. The load cases to be evaluated are divided into categories as listed in Table 46.

Table 46. Acceptance Criteria for Hydraulic Structures

Loading Combination	Position of Resultant Force (Percent of Base in Compression)	Sliding Safety Factor (Friction Only)	Floatation Safety Factor
Usual	Middle third of the base: 100% compression	≥2.0	≥1.5
Unusual	75% of Base in compression	≥1.5	≥1.3
Extreme Flood	Resultant within base, and all other acceptance criteria must be met	≥1.1	≥1.1

7.7.2 Load Combinations

Table 47 summarizes load conditions for the Auxiliary Spillway structure. The load combinations listed below are to be considered a representative sample.

Table 47. Auxiliary Spillway – Load Conditions

Usual Load Cases		
U1	Usual Condition – Dry/No Flow (Groundwater to top of foundation) Headwater Elevation – 794.00 Tailwater Elevation – 794.00	D+H+E+U
Unusual Load Cases		
UN1	Unusual Condition – Maximum head/tail water difference – Headwater near Auxiliary Spillway Crest – EL. 807.13 – Tailwater at EL. 792.93 – Uplift per Geotech analysis with Drain working	D+H+E+U
UN2	Unusual Condition – Maximum head/wail water difference – Headwater near Auxiliary Spillway Crest – EL. 807.13 – Tailwater at EL. 792.93 – Uplift per Geotech analysis with Drain not working	D+H+E+U
UN3	Unusual Condition – Maximum head/wail water difference – Headwater near Auxiliary Spillway Crest – EL. 807.13 – Tailwater at EL. 792.93 – Uplift @ headwater before cut-off wall, then 50% right after cut-off wall to tailwater elevation.	D+H+E+U
Extreme Load Cases		
E1	Extreme Condition – Maximum Flood Level at Reservoir (PMF) – Headwater at EL. 810.00 – Tailwater at EL. 802.11 – Uplift per Geotech analysis with Drain working	D+H+E+U
E2	Extreme Condition – Maximum Flood Level at Reservoir (PMF) – Headwater at EL. 810.00 – Tailwater at EL. 802.11 – Uplift per Geotech analysis with Drain not working	D+H+E+U
Notes		
D	Dead Load: Includes concrete (C), backfill (E),	
H	Hydrostatic Load:	
EH, EV	Earth / Backfill / Sediment / Siltation: Include horizontal and vertical loads	
U	Uplift Load	

7.7.3 Stability Analysis Results

The following is a summary of the stability analyses conducted for the overflow weir of the labyrinth spillway and the abutment walls. Each section was evaluated for Usual, Unusual, and Extreme flooding loading conditions representing potential conditions the structure will experience during its design life. Refer to Exhibit B of Appendix J for the stability calculations and results.

7.7.4 Auxiliary Labyrinth Spillway

The stability analysis for the structure was performed using Excel spreadsheets. Results of the analyses are summarized in Table 48.

Table 48. Auxiliary Labyrinth Spillway – Stability Summary

Load Comb.	e (ft)	σ @ Upstream (psf)	σ @ Downstream (psf)	% Base in Comp.	Sliding SF	SF Req'd	Floatation SF	SF Req'd.
U1	2.50	850	870	100	13.32	2.0	3.39	1.3
UN1	1.42	1,160	1,180	100	4.16	1.5	4.17	1.2
UN2	1.00	930	940	100	3.68	1.5	2.20	1.2
UN3	2.60	1,040	1,070	100	3.94	1.5	2.94	1.2
E1	3.44	1,310	1,360	100	4.93	1.1	3.47	1.1
E2	2.15	1,210	1,240	100	4.68	1.1	2.70	1.1

7.7.5 Auxiliary Labyrinth Spillway Abutment

The stability analysis for the integrated labyrinth spillway abutment structure was performed using Excel spreadsheets. Results of the analyses are summarized in Table 49.

Table 49. Auxiliary Spillway Integrated Labyrinth Abutment – Stability Summary

Load Comb.	E (ft)	σ @ Upstream (psf)	σ @ Downstream (psf)	% Base in Comp.	Sliding SF	SF Req'd	Floatation SF	SF Req'd.
U1	1.22	1,980	2,510	100	9.82	2.0	9.42	1.3
UN1	1.66	1,890	2,610	100	6.97	1.5	7.45	1.2
UN2	1.37	1,750	2,290	100	6.22	1.5	4.17	1.2
UN3	2.14	1,720	2,610	100	6.70	1.5	5.80	1.2
E1	2.84	1,570	2,760	100	7.13	1.1	5.21	1.1
E2	1.58	1,600	2,170	100	6.14	1.1	3.10	1.1

7.8 STRUCTURAL DESIGN

Structural design calculations are included in Exhibit B – Structural Calculations of Appendix J.

7.8.1 Methodology

7.8.1.1 Auxiliary Labyrinth Spillway

The Auxiliary Labyrinth Spillway is a cast-in-place concrete structure made up of 9 independent units containing 2 cycles each of the labyrinth spillway. Joints between adjacent slabs alternate between contraction joints and expansion joints. The labyrinth walls have been designed to withstand the loading imparted from the Usual, Unusual and Extreme conditions. In general, the Unusual condition controlled the design of the reinforcing. The downstream apex does not have a joint in the concrete, but two joints are designed approximately a third of the way up the wall to allow for movement. The upstream apex has either the contraction joint or expansion joint with a bulbed PVC waterstop. The upstream apex does not require reinforcing steel to cross over the joint as the water pressure will act to press the cycles together. Each unit is founded on a concrete slab with a layer of insulating foam and aggregate base. The insulating foam has been included under the slab to reduce the potential for frost heave.

Auxiliary Labyrinth Spillway structural design calculations can be found on pages 10 thru 75 of Exhibit B – Structural Calculations, in Appendix J.

7.8.1.2 Auxiliary Labyrinth Spillway Abutment

The Auxiliary Labyrinth Spillway Abutment is a cast-in-place concrete structure at the northwest side of the Auxiliary Spillway. The abutment has a U-shaped concrete wall atop a mat foundation of the same thickness of the Auxiliary Spillway. The first cycle of Labyrinth Spillway connects to the abutment wall at the upstream apex of the labyrinth cycle. Insulating foam was included underneath the labyrinth cycle of this mat foundation but is not necessary after the U-shaped abutment walls begin as earthen fill will reduce the risk of frost heave. Stresses in the concrete and reinforcing steel were determined based on the Usual, Unusual and Extreme loading conditions.

Auxiliary Labyrinth Spillway Abutment structural design calculations can be found on pages 76 thru 119 of Exhibit B – Structural Calculations, Appendix J.

7.8.2 Load Combinations

7.8.2.1 Auxiliary Labyrinth Spillway

Auxiliary Labyrinth Spillway Load Cases	
U1	1.2 * Dead Load + 1.6 * Usual Uplift
UN1	1.6 * Dead Load + 1.6 * Unusual 1 Uplift + 1.6 * Unusual 1 Water
UN2	1.6 * Dead Load + 1.6 * Unusual 2 Uplift + 1.6 * Unusual 2 Water
UN3	1.6 * Dead Load + 1.6 * Unusual 3 Uplift + 1.6 * Unusual 3 Water
E1	1.2 * Dead Load + 1.3 * Extreme 1 Uplift + 1.3 * Extreme 1 Water
E2	1.2 * Dead Load + 1.3 * Extreme 2 Uplift + 1.3 * Extreme 2 Water
E3	1.2 * Dead Load + 1.3 * Extreme 3 Uplift + 1.3 * Extreme 3 Water

7.8.2.2 Auxiliary Labyrinth Spillway Abutment

Auxiliary Labyrinth Spillway Abutment Load Cases	
U1	1.2 * Dead Load + 1.6 * Usual Uplift + 1.6 * Soil Load + 1.6 * Live Load
UN1	1.2 * Dead Load + 1.6 * Live Load + 1.6 * Soil Load
E1	1.2 * Dead Load + 1.35 * Soil Load + 1.0 * Uplift

7.9 CONSTRUCTION CONSIDERATIONS

The following construction considerations are noted:

- Dewatering of excavated areas will be required to sufficiently enable construction of the Auxiliary Spillway.
- Foundation preparation will require care during excavation to identify unsuitable conditions or weak bedding planes that could impact stability.
- Due to a concentration of H_2S found at project site, concrete with an H_2S resisting additive is required in area as specified in the contract specifications.
- Joint preparation will require attention to proper installation of water stops, dowels, and reinforcement. Joint alignment and water-tight integrity are critical for spillway construction.
- Fill placement and compaction methods must be reviewed and monitored to ensure wall movement does not occur during construction.
- Water management requirements during construction will require the Auxiliary Spillway to be constructed in two phases. The first phase will include the base slab for the labyrinth weir walls and the full abutment. The second phase will be to construct the labyrinth weir walls after completion of the embankment closure sections.

8.0 RESERVOIR ROUTING

Hydrologic and hydraulic modeling was performed to route certain design hydrographs through the reservoir. The Principal Spillway is designed such that the reservoir stores excess flows associated with the 1% ACE (100-year) event without the Auxiliary Spillway activating. The Auxiliary Spillway is designed to activate for events larger than the 1% ACE storm event. A HEC-HMS model was developed to simulate the 1% ACE and the PMF events. Inflow hydrographs were routed through the spillways for each event to determine the water surface elevations in the reservoir. The reservoir routing hydrographs for the 1% ACE event are shown in Figure 38 and the reservoir routing hydrographs for the PMF event are shown in Figure 39. The hydrographs represent the integrated Principal Spillway and labyrinth Auxiliary Spillway structures.

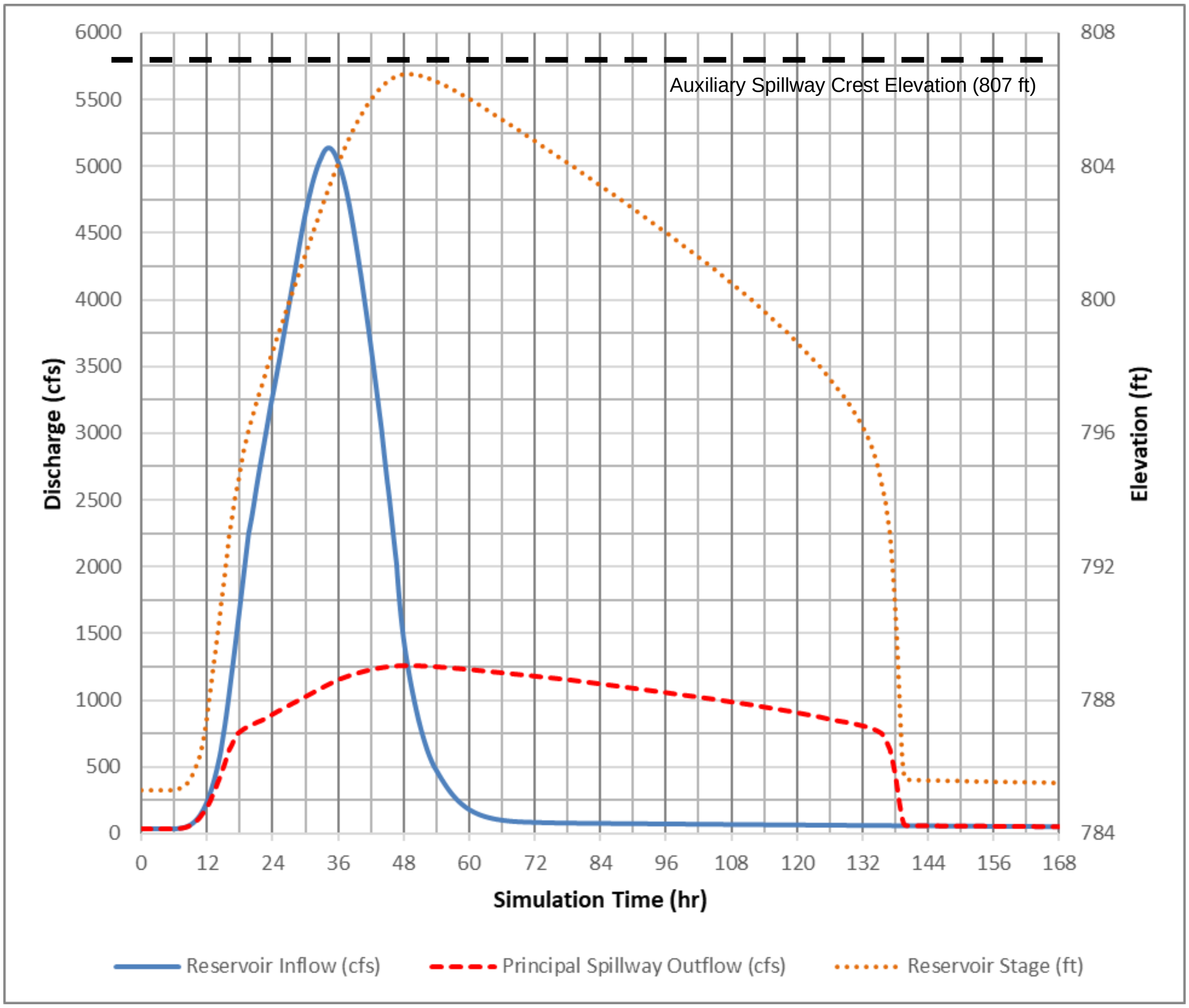

Figure 38. Reservoir Routing – 1% ACE (100-year) Event

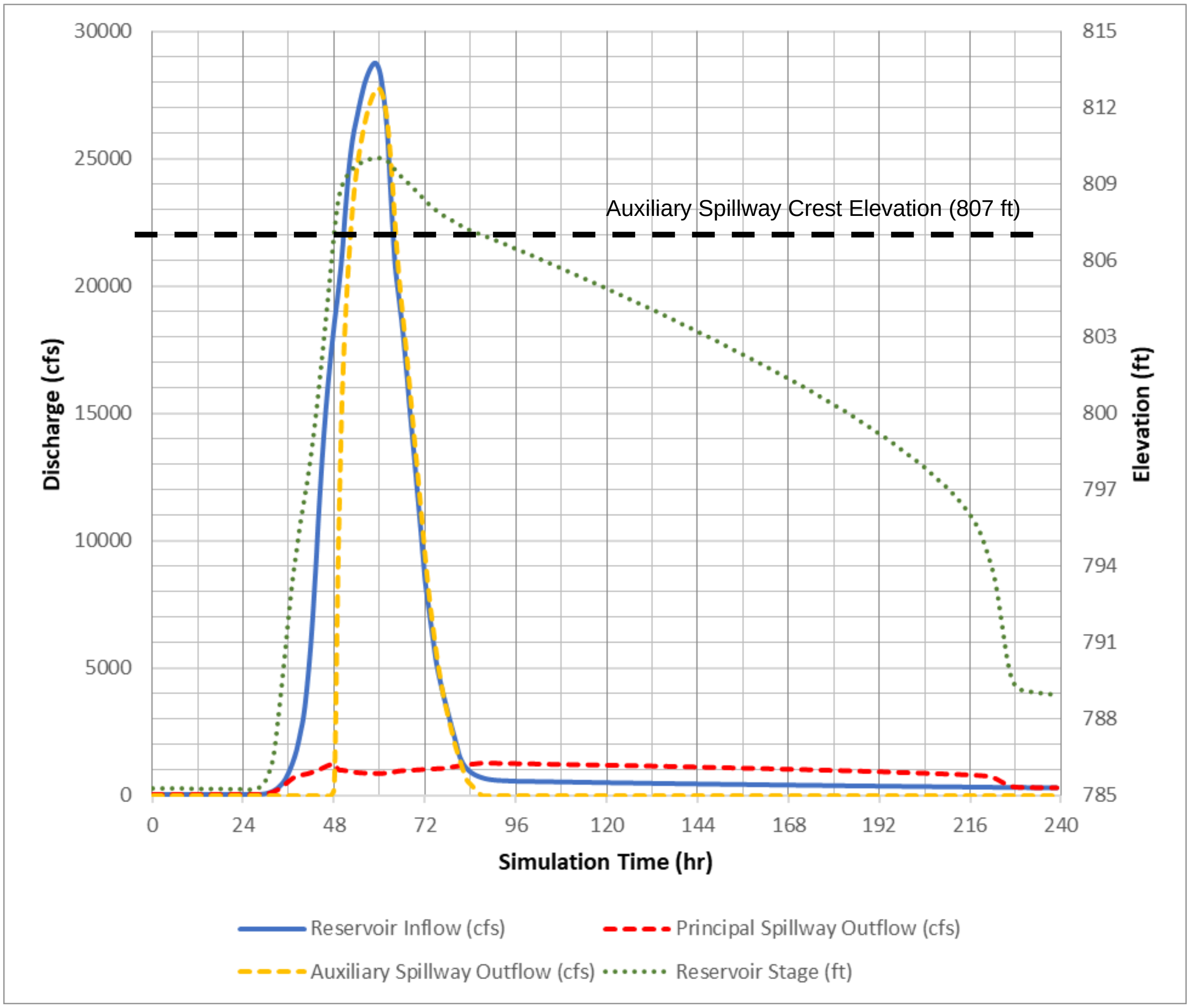

Figure 39. Reservoir Routing – PMF Event

The maximum modeled reservoir inflow, Principal Spillway outflow, and reservoir stage for each event is summarized in Table 50.

Table 50. Peak Inflow-Outflow-Stage Summary

Peak Reservoir Flows and Stages	100-Year	PMF
Reservoir Inflow (cfs)	5,132	28,778
Principal Spillway Discharge (cfs)	1,255	1,263
Auxiliary Spillway Discharge (cfs)	0	27,450
Reservoir Stage (ft)	806.8	810.0

8.1 UPSTREAM IMPACTS

The approximate surface areas and storage volumes are presented in Table 51. The proposed project inundation areas are shown in Figure 40. Additional upstream impacts are presented in the Hydrologic and Hydraulic Analysis Report, Appendix D.

Table 51. Inundation Areas and Storage Volumes

Reservoir Elevation (ft)	Surface Area (acres)	Storage Volume (acre-ft)
807.0	910	6,945
810.0	1,026	9,839

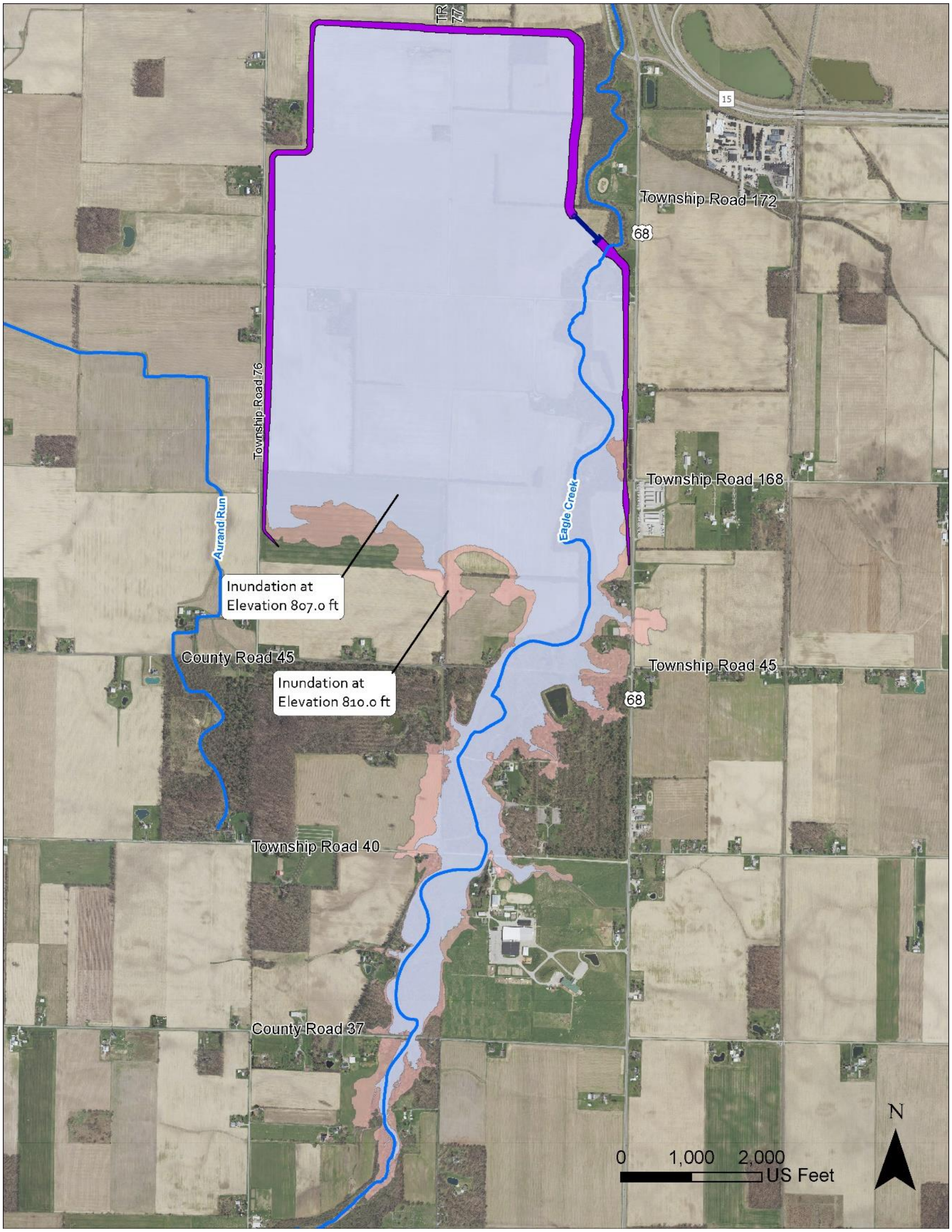

Figure 40. Reservoir Inundation Extents

8.2 DOWNSTREAM BENEFITS

Hydraulic model results show that the ECFB project results in a peak flow reduction of about 2,550 cfs (16% decrease) on the Blanchard River during the 1% ACE WSE which translates to about 2.2 feet of lowering of the base flood elevations near the confluence with Eagle Creek.

The reduction in WSEs along Eagle Creek and the Blanchard River is estimated to remove approximately 1,290 parcels and 1,590 acres from the regulatory floodplain.

Stantec reviewed transportation impacts due to flooding at 18 locations across the watershed. Many of the transportation impacts are expected to be reduced as a result of the ECFB. Table 52 shows the approximate depth of flooding at multiple locations across the watershed for existing conditions and the with-project conditions during a 1% ACE flood event. Additional detail on the downstream benefits is presented in the Hydrologic and Hydraulic Analysis Report, Appendix D.

Table 52. Transportation Impacts and Benefits

Bridge/Intersection	Reach	1% ACE Depth above Bridge/Intersection (feet)	
		Ex. Cond.	ECFB
US 68 near TR 172	Eagle Creek	1.0	0.0
6th Street / Westview	Eagle Creek	0.8	0.0
S. Blanchard St. / E. Lincoln	Eagle Creek	4.0	1.6
E. Sandusky / S. Blanchard St.	Eagle Creek	3.6	1.2
CR 180 near SR 37	Lye Creek	0.4	0.4
SR37 near Williams St.	Lye Creek	1.4	0.8
Fishlock Ave. Bridge	Lye Creek	3.3	2.3
E. Sandusky Bridge	Lye Creek	5.2	3.2
SR 568 near CR 236	Blanchard River	2.8	2.2
E. Main Cross / Warrington Ave.	Blanchard River	2.0	0.0
S. Blanchard St. / E. High St.	Blanchard River	5.9	3.8
E. Main Cross / MLK Pkwy	Blanchard River	1.3	0.0
Main St.	Blanchard River	4.0	1.8
Defiance Ave. / Univ Townhouses	Blanchard River	4.7	2.9
Broad Ave. / Findlay St.	Blanchard River	2.5	0.8
Broad Ave. / Howard St.	Blanchard River	1.2	0.0
CR 223 / US 224	Blanchard River	2.9	2.0
CR 140 / US 224	Blanchard River	0.3	0.0
CR 139	Blanchard River	0.0	0.0

9.0 INTERIOR BASIN DESIGN

9.1 INTERIOR DRAINAGE

The goals of the interior drainage design are to maintain positive drainage away from the dam embankment and facilitate drawdown of the basin after a filling event by use of grading, swales, and ditches. These features are incorporated into the design to address drainage associated with the interior access bench and maintenance zone, the proposed borrow pits, and isolated low-lying areas.

Because the interior of the basin is designed to be fully inundated, interior ditches were not designed to any particular frequency storm event or sized to convey particular flow rates. Rather, ditches are designed to provide positive drainage from higher elevations to lower elevations to maintain flood storage capacity within the basin. Both the existing ground and designed ditches are at shallow slopes, so localized ponding is expected. In general, the basin will dewater in the days following a filling event so that storage is available for the next event. Specific areas and considerations are described in the following sections.

9.1.1 Dam Interior Access Bench and Maintenance Zone

A minimum 50-foot-wide grass corridor along the interior toe of the dam embankment is designed to allow for maintenance and access for inspection of the dam. The area within this maintenance zone is graded to drain away from the toe of the dam at a minimum slope of 2%. Features such as roads, recreational trails, ditches, and culverts are permitted, but no water should pond within the maintenance zone. This area will be cleared of existing trees and woody vegetation, and no such plantings are proposed.

9.1.2 Proposed Borrow Pits / Wetlands

Borrow soils for dam embankment fill material will be sourced from the interior of the basin. The borrow pits will be graded and vegetated. An alternative design for the project includes converting the borrow areas into wetlands as their final condition. The location and maximum depths of the borrow pits / wetlands were selected to reduce the risk for potential seepage paths below the dam embankment. Seepage considerations include locating borrow pits / wetlands a minimum of 400 feet upstream of the dam embankment toe and limiting the excavation depths to reduce the risk of exposing the underlying coarse-grained soils. The wetland design requires that a portion of the wetlands interact with groundwater and should not drain overland by gravity. Higher elevations within the wetlands are designed to inundate less frequently and are able to dewater completely following a basin filling event.

Following a basin filling event, the basin's pool elevation will lower until it reaches the outer rim of the wetlands. Shallow flow passing over the low points along the wetland rim could result in erosion if not properly controlled, therefore a controlled outlet was designed for each of the two largest wetlands. Wetlands 1 and 2 drain through a swale with riprap protection installed at the upstream end to reduce the risk of uncontrolled head cutting as the basin drains. Details of the proposed wetlands are shown on the design drawings, Appendix B.

More detail related to the design of the wetlands can be found in the Stantec report titled, "*Eagle Creek Flood Basin – Interior Wetland Design*" (Stantec 2023).

9.1.3 Isolated Low-Lying Areas

Portions of the basin interior will require grading and ditches to convey runoff toward Eagle Creek and the Principal Spillway. Some areas along the west side of the basin currently drain to the west toward the Aurand Run watershed and will be regraded to direct runoff into the proposed wetlands.

The northeast corner of the flood basin is the lowest area in the overbank terrain within the basin's footprint. This area currently drains to the east toward Eagle Creek. The proposed embankment cuts off the natural overland drainage path for this low-lying area and runoff must now be directed to the south (against existing grade) toward the Principal Spillway channel. Due to seepage considerations, excavated drainage ditches must be kept at least 400 feet from the toe of the dam and their depth must be limited to reduce the risk of exposing the underlying coarse-grained soils. Given these limitations, the area in the northeast corner, generally below elevation 794 feet, cannot be drained to the Principal Spillway overland by gravity. Because frequent ponding is expected to occur in this area, a 50-foot-wide bench starting at elevation 798 feet is proposed to extend from the interior embankment toe and along the ponding area to keep the ponded water away from the upstream dam toe. Beyond the 50-foot-wide bench, the low-lying area may be filled with excavated soils unsuitable as Fill Type 1 or Fill Type 2. This area is referred to as the 50-foot Priority Fill Type 3 Placement Zone. If additional capacity is needed after the Priority Zone has been filled, Type 3 Fill may be placed until the 2% final grade slope intersects existing grade. Otherwise, the area beyond the bench may remain an open, shallow pool (until infiltrated or evaporated).

9.2 WETLAND DESIGN

The basin interior likely supported large wetland complexes in the past and is well suited for wetland restoration and creation opportunities due to its suitable soils, good growing conditions, and abundance of potential sources of wetland hydrology. The interior was evaluated considering excavation for embankment borrow material, wetland size, potential water quality benefits, and / or a combination of these. The final wetland design seeks to maximize the created wetland area and water quality potential while reducing excess excavation. The proposed layout is in Figure 41.

The interior wetland design is described in the Stantec report titled, "*Eagle Creek Flood Basin – Interior Wetland Design*" (Stantec 2023).

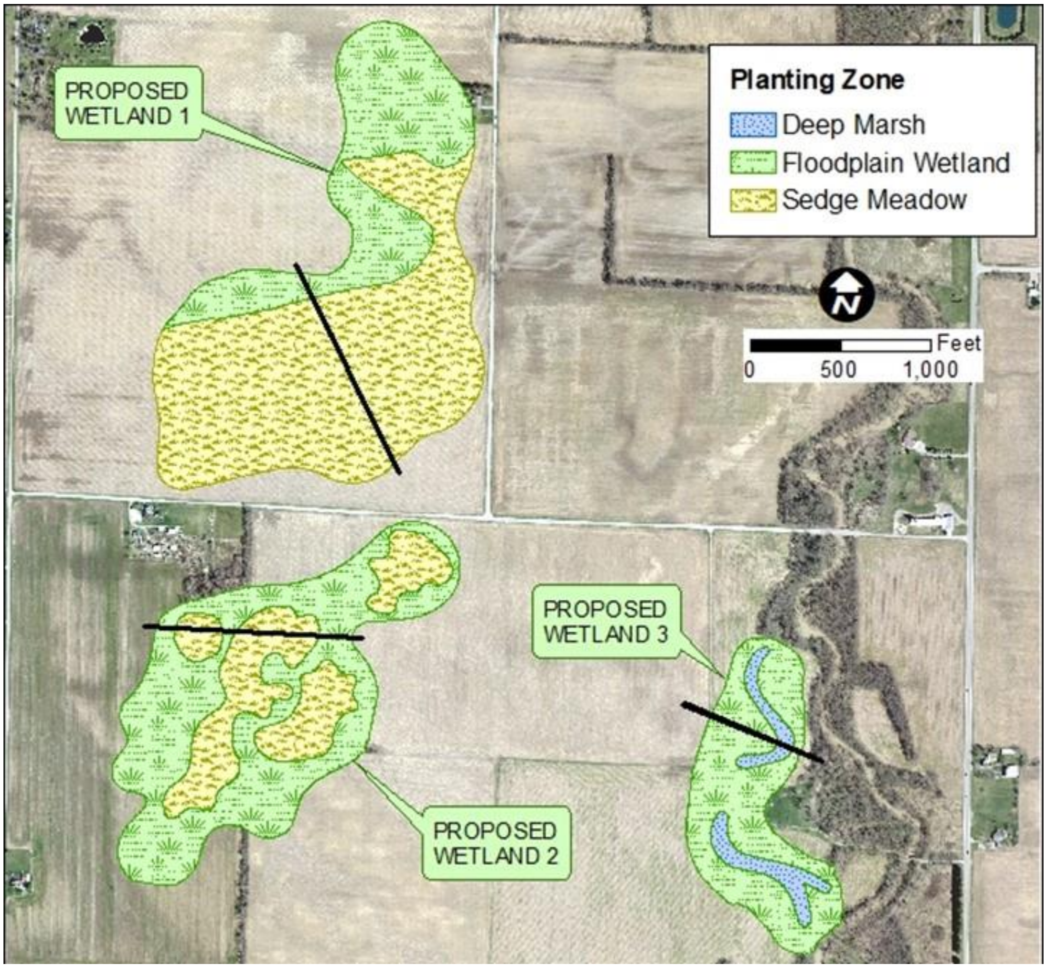

Figure 41. Proposed Wetlands and Planting Zones

9.3 RECREATIONAL OPPORTUNITIES

Passive recreation is anticipated to be an important component of the basin interior post-construction. Construction of the wetlands as described above will return areas of high-value wildlife to the community and will provide an aesthetically pleasing viewshed within. To allow local residents and visitors to the site to fully utilize these features, a public trail system is proposed to access the wetlands and adjacent naturalized areas. Road access with parking is currently proposed at the southwest corner of the site leading to a 12-foot wide, approximately 3.5-mile-long pedestrian trail winding across the site and along the wetlands. The trail is proposed as a mowed turf grass trail to reduce potential maintenance and loss of trail paving material (e.g., mulch, gravel) associated with flooding during basin activation. Park benches are proposed as part of this amenity, along with informational kiosks describing the functions and values of the wetlands and the creatures that inhabit them (Figure 42). This component of the design may evolve in the future as MWCD coordinates with the City of Findlay.

Figure 42. Ground-Level Rendering of Trail near Wetland 1

9.4 SITE ACCESS

Access for the maintenance, inspection, observation, and operation of the various flood mitigation structures is provided by way of exterior and interior maintenance benches. Multiple access points are available from the perimeter of the dam embankment to the crest for visual maintenance, observation, and inspection. Additionally, gravel access paths will be provided from the dam embankment crest and Township Road 49 to the spillway for operations, maintenance, and inspection activities. The Township Road 49 bridge is anticipated to remain in place for access to the spillway site from the east side of Eagle Creek.

An entry drive in the southwest corner of the site is anticipated for use by the general public. The access road will extend from Township Road 76, over the embankment tie-in, and to the north on the interior of the basin to a designated parking area at the head of the trail system. The parking area sits above the 100-year flood elevation of 807.0 feet.

To discourage foot and vehicle traffic near the spillway structures, fencing, safety rail, and barrier gates are designed near embankment access ramps and along the perimeter of the spillway structure.

10.0 ROADWAYS AND UTILITIES

The Eagle Creek Flood Basin project will impact publicly owned roadways and public and privately owned utilities. These impacts are summarized below.

10.1 ROADWAYS

Two local roads will be terminated during construction. Township Road 77 will be partially removed from Township Road 49 to just north of the dam embankment near a row of residential structures. A 42-foot radius cul-de-sac is designed at the site of Township Road 77 termination to allow for residents, school buses, and other vehicles to turn around.

Township Road 49 will be fully decommissioned between US-68 and Township Road 76. Township Road 49 currently provides access to/from residential areas west of US-68 to US-68. Transportation alignments that could potentially mitigate the impact to Township Road 49 are outside the scope of this Report. However, MWCD is currently coordinating with the Ohio Department of Transportation (ODOT) District 1 on proposed improvements to the US-68/SR-15 interchange. The project would connect Township Road 80 to US-68 in order to provide direct access from the residential areas west of US-68 to US-68. Township Road 49 and Township Road 77 will remain open until the Township Road 80 has been connected to US-68. Construction of the ECFB is anticipated to be closely coordinated with the US-68/SR-15 interchange project.

10.2 UTILITIES

10.2.1 Existing Utilities

Existing utilities will be removed or relocated where they cross the proposed dam alignment. Features within the limits of borrow areas and between 200 feet upstream and 50 feet downstream of the dam embankment shall be demolished including septic systems and utilities. Existing agricultural drain tiles are known to be located on site. Drain tile is to be removed and disposed of when encountered while performing earthwork. Unless otherwise noted, drain tiling shall be completely removed within 50 feet of the footprint of the dam embankment or within borrow areas.

A known fiber optic communication line parallel to Township Road 49 is to be relocated through the upper layer of the dam embankment on both the east and west sides of the flood basin. The remaining fiber optic line within the interior of the basin is to remain. The Contractor is to coordinate with the Utility. See sheets C-151 and C-152 of the design drawings, Appendix B, for more information.

Existing utilities may be relocated, abandoned, or remain in place where they are adjacent to the dam embankment, or are within the impoundment. Impacted utilities may include: stormwater structures, conduits, and drainage tiles on the interior or exterior of the basin; domestic water wells and domestic septic systems within the basin; buried communication, water, and gas lines within the basin or within the footprint of the dam embankment; and overhead utilities within the impoundment. Coordination with the utility owners is ongoing to determine the extent of utility relocation, if necessary. Water wells shall be decommissioned if encountered or otherwise noted in accordance with ODNR guidelines and applicable codes.

10.2.2 Proposed Utilities

Electric components are proposed in the southwest corner near the access road / parking area from existing service along Township Road 76, and also to the spillway area from existing service along US-68. Single phase overhead and underground electric lines will supply power to lighting fixtures and will provide power to the slide gates at the spillway. Coordination with USGS is ongoing, but power will be available at the spillway for a gage, if necessary.

11.0 CONSTRUCTION CONSIDERATIONS

The construction permit through ODNR Division of Water Resources Dam Safety will follow OAC Rule 1501:21-9-01 and construction requirements will follow OAC Rules 1501:21-5-01 through 1501:21-5-07.

11.1 PROJECT CONTROL

Project control points were established to assist with design and construction activities. The coordinates and elevations for the control points are listed on Sheet G-109 in Appendix B.

11.2 CONSTRUCTION ACCESS

Construction access will likely be available from the southwest corner of the project site at Township Road 76, from the west at the intersection of Township Road 49 and Township Road 76, and from the east at Township Road 49 and US-68.

11.3 STAGING / CONTRACTOR FIELD OFFICE

Staging is anticipated to occur within the footprint of the project site. Approximately 765 acres of land are within the proposed impoundment area inside the dam alignment that contain several level areas suitable for large machinery. The Contractor’s field office is expected to be located in the southeast corner of the site off of US-68. Area of approximately one acre should be planned for the equipment staging and maintenance area, if necessary.

11.4 CONSTRUCTION SEQUENCING

Stantec developed a conceptual construction sequencing plan and estimated construction schedule associated with the design. The sequencing plan shown in Table 53 considers diversion of Eagle Creek flow, construction of the spillway structure, embankment and exterior drainage ditches, and impacts to adjacent transportation networks.

Table 53. Summary Conceptual Construction Sequencing

ECFB Issued for Bid	November 2023
ECFB Bid Due Date	January 2024
ECFB Construction Contract Award	February 2024
ECFB Issued for Construction	February 2024
Mobilization / Erosion Control / Staking / Tree Clearing	February - March 2024
Construct Temporary Cofferdam	April 2024
Strip, Stockpile Topsoil	April - May 2024
Excavate Exterior Drainage Ditches	May - October 2024
Dam Embankment Construction (Authorized Areas)	May - November 2024
Inspection Trench / Foundation Preparation	June - July 2024
Construct Spillway Slabs	August - December 2024
ODOT Interchange End Construction	October - November 2024
Install culverts & stormwater pipe / backfill	October 2024 - January 2025
Spillway Walls / Abutments	December 2024 - April 2025
Construct base course and asphalt pavement	March - April 2025
Spillway Channels	March - June 2025
Dam Embankment Construction (Restricted Areas)	April - August 2025
Divert Eagle Creek Flow / Construct Remaining Embankment	June - July 2025
Construct Labyrinth Weir Walls	August - November 2025
ECFB End Construction	November 2025

11.5 CARE OF WATER

The contractor will be responsible for the control, collection, and removal of surface water and ground water in the areas of project excavations as needed to perform the construction activities. Prior to commencing construction activities, the contractor will provide the Engineer an Excavation Dewatering Plan detailing the approach for capture, control and discharge of surface waters and groundwater, a Spill Response Plan in accordance with the Section 401 Water Quality Certification, and a Control of Water Plan detailing the approach for control and discharge of surface waters, including Eagle Creek, for the construction of the Work.

11.5.1 Control of Water Plan

The selected location of the Principal Spillway allows for the existing Eagle Creek channel to flow in its natural course during the majority of construction. Once the Auxiliary Spillway base slab and north abutment wall, Principal Spillway, and proposed channel is constructed, flows will be diverted to the relocated Eagle Creek channel and through the Principal Spillway. Once flow is diverted, a section of the existing Eagle Creek channel will be filled to promote positive drainage away from the dam embankment and remaining embankment section will be completed.

The following is a general approach to construction sequencing plan and care of water within Eagle Creek during construction of the spillway and dam embankment. The Contractor is responsible for the development of the Care of Water Plan and the means and methods. Figure 43 identifies the general construction sequencing assumed for the integrated spillway structure.

Phase 1A

1. Maintain and do not disturb the existing Eagle Creek channel from top of bank to top of bank until completion of the Principal Spillway and Eagle Creek Realignment Channel.
2. Construct Principal Spillway, including approach channel, control wall, stilling basin, and retaining walls. Construct the Auxiliary spillway base slabs and north abutment wall. Do not construct Auxiliary Spillway weir wall until completion of dam embankment.
3. Construct the Eagle Creek Realignment downstream channel (STA 208+07 to 214+96).
4. Embankment dam construction may proceed in parallel with Steps 2 and 3. Do not place embankment fill within the existing Eagle Creek channel.
    a. The slopes of any embankment gap must not exceed 4H:1V slopes to allow for adequate benching of material.
    b. Adequate provisions are required to protect the filter from contamination and for later tie-in.
5. Construct the Eagle Creek realignment upstream channel from STA 202+88 to 206+73.

Phase 1B

6. Divert flow from the existing Eagle Creek channel to the Eagle Creek realignment channel.
    a. Complete Eagle Creek realignment channel from STA 200+24 to 202+88.
    b. Divert flow from the existing Eagle Creek channel using clean granular fill, water-filled dam, or aggregate filled geotextile bags. Do not place fine grained soils into open water.

Phase 2A

7. Complete construction of the dam embankment and filling of the existing Eagle Creek channel.

Phase 2B

8. Complete the Auxiliary Spillway weir walls.

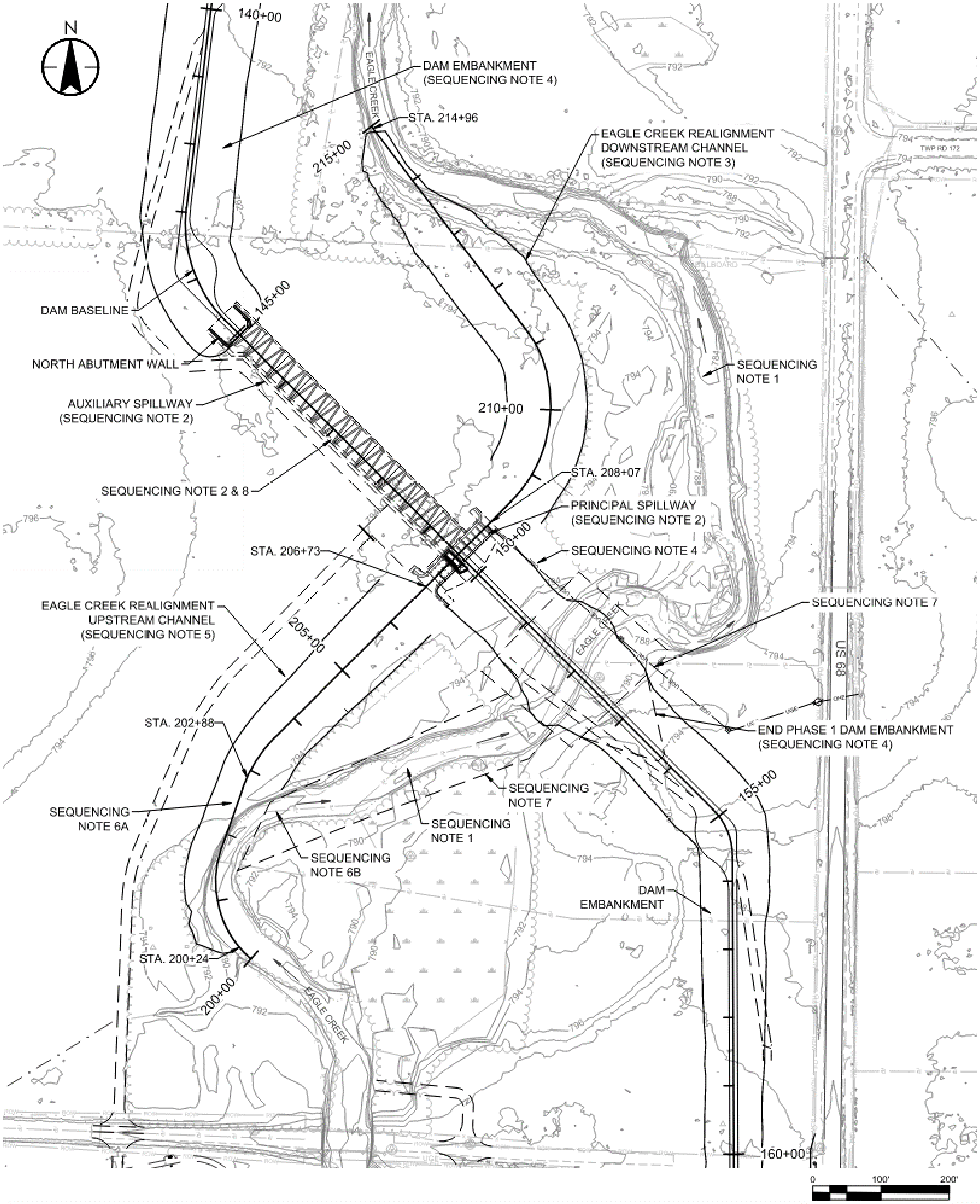

Figure 43. Diversion of Streamflow Sequencing Overview

11.6 TECHNICAL SPECIFICATIONS

Project specifications are included in Appendix C.

12.0 INSTRUMENTATION AND MONITORING

Pursuant to the Ohio Revised Code section 1521.062, the owner of a dam shall monitor, maintain, and operate the structure and its appurtenances safely in accordance with state rules, terms and conditions of permits, orders, and other requirements issued. The ECFB project's Operation, Maintenance, and Inspection (OM&I) Manual is a living document detailing the operation, maintenance, and inspection procedures necessary for the continued safe operation and use of the dam and its appurtenances. As part of the OM&I Manual, Permanent Instrumentation Plan drawings show the locations of project Piezometers, Drain Outlets, and Survey Monuments / Structure Monitoring Points (SMP). The OM&I Manual also includes the Instrumentation Monitoring Plan which contains further discussion on the standpipe piezometers and toe drain outlets data collection procedures.

In order to maintain the integrity of the dam, consistent monitoring is required. The schedule in Table 54 is established to document the frequency of monitoring required for instrumentation, survey monuments and the dam embankment.

Table 54. Monitoring Schedule

Item	Specific Items	Frequency
Piezometers	Document readings per Instrumentation Monitoring Plan	- Quarterly - Within 48 hours of receiving 2-inches of rainfall during a 24-hour period - After a flood operations event (Every 12 hours while pool elevation is 802 feet or higher)
Toe Drain Outlets	Document readings per Instrumentation Monitoring Plan	- Quarterly - Within 48 hours of receiving 2-inches of rainfall during a 24-hour period - After a flood operations event (Every 12 hours while pool elevation is 802 feet or higher)
Staff Gauge	Record pool elevation (NAVD88)	- Routine Inspections - Detailed Inspections - Within 48 hours of receiving 2-inches of rainfall during a 24-hour period - After a flood operations event (Every 12 hours while pool elevation is 802 feet or higher)
Survey Monuments / SMP	Record elevation (NAVD88) at each survey monument / SMP	Annually for the first 5 years after construction. Then every 5 years after.
Embankment	Visually Monitor Area Downstream of Embankment	- Within 48 hours of receiving 2-inches of rainfall during a 24-hour period - After a flood operations event (Every 12 hours while pool elevation is 802 feet or higher)

12.1 PIEZOMETERS

12.1.1 Standpipe Piezometers

Thirteen standpipe piezometers are proposed to be installed at the ECFB. The piezometer location along the dam embankment stationing and height of the monitoring well casing above ground surface is listed in Table 55.

Table 55. Summary of Planned Piezometers

Piezometer	Station (ft)	Height (ft)
PZ-1	39+57	3'-2"
PZ-2	69+75	3'-2"
PZ-3	92+70	3'-2"
PZ-4	101+00	3'-2"
PZ-5	114+15	3'-2"
PZ-6	118+85	3'-2"
PZ-7	126+30	3'-2"
PZ-8	139+70	3'-2"
PZ-9	144+75	3'-2"
PZ-10	150+10	3'-2"
PZ-11	152+00	3'-2"
PZ-12	157+65	3'-2"
PZ-13	176+50	3'-2"

12.1.2 Observation Frequency

A manual water level reading, using a water level indicator, is to be recorded a minimum of four times a year (every three months) at each piezometer, and within 48 hours of receiving 2-inches of rainfall during a 24-hour period. Piezometer readings should be recorded after a flood operations event (elevation 802 feet) and every 12 hours while the reservoir pool elevation is 802 feet or higher.

12.1.3 Threshold / Action Levels

Static groundwater levels within and around the Flood Basin commonly change with the seasons and from fluctuations of the reservoir water surface during rainfall events. These groundwater fluctuations should be gradual and correct over time to a static average for the dam. Piezometer threshold levels are used to highlight water level readings that could indicate developing dam safety issues. "Threshold Limits" indicate conditions requiring heightened attention and increased surveillance, while higher "Action Limits" identify conditions that warrant investigation and possible mitigation.

12.2 TOE DRAIN SYSTEM

12.2.1 Toe Drain Outlets

The toe drain system includes 15 outlets to the downstream exterior ditch or channel graded to Eagle Creek. The pipe outlets are designed with animal guard and discharge 1 foot above the ditch invert elevation to facilitate flow measurement and monitoring. Trench blocks constructed of Fill Type 1 within the toe drain alignment isolate the different toe drainpipe segments so that various reaches of pipe can be monitored separately. The pipes outlet to a concrete pad that will protect the end of the pipe and reduce erosion potential from pipe flows. The inspection team is to observe the flow from the toe drain outlets and document the flowrate from each outlet. Observers should note changes in volume of flow and turbidity (clear, cloudy, muddy, etc.) of the water.

The toe drain outlet spatial and embankment station locations are also presented in Table 56.

Table 56. Summary of Planned Drain Outlets

Drain Outlet	Northing	Easting	Invert Elev. (feet)	Drainpipe Start Station	Drain Outlet Station	Drainpipe Length
DO-1	480,488.59	1,645,557.73	794.14	28+08	40+08	1,200
DO-2	480,492.84	1,645,557.78	794.14	47+11	40+28	683
DO-3	484,231.34	1,647,349.07	796.20	82+62	92+68	1,006
DO-4	484,194.40	1,648,317.14	794.12	92+88	102+30	942
DO-5	484,194.27	1,648,321.39	794.12	105+90	102+50	340
DO-6	484,062.25	1,650,035.06	788.86	106+10	119+00	1,290
DO-7	484,059.61	1,650,038.39	788.86	129+40	119+20	1,020
DO-8	482,027.51	1,650,068.26	787.90	129+60	138+85	925
DO-9	482,023.26	1,650,068.12	787.90	144+50	139+05	545
DO-10	481,045.37	1,650,560.92	786.47	149+83	152+05	222
DO-11	481,042.37	1,650,563.93	786.47	156+90	152+25	465
DO-12	480,537.71	1,650,774.09	793.28	166+85	157+10	975
DO-13	478,794.03	1,650,752.12	799.27	167+05	174+45	740
DO-14	478,789.78	1,650,752.12	799.27	180+61	174+65	596
DO-15	477,300.79	1,650,749.79	800.40	185+58	189+50	392
				190+64	189+50	114

12.2.2 Observation Frequency

A manual water level reading should be recorded from each drain outlet a minimum of four times a year (every three months), and within 48 hours of receiving 2-inches of rainfall during a 24-hour period. Drain outlet readings should be recorded after a flood operations event (elevation 802 feet) and every 12 hours while the reservoir pool elevation is 802 feet or higher.

12.2.3 Action Levels

Observers should be looking for sediment exiting the toe drain outlets, or a discharge exiting the toe drain outlet exceeding 15 gallons per minute.

12.3 STAFF GAUGE

One staff gauge is installed on the upstream side of the ECFB. The gauge indicates water surface elevation upstream of the Principal Spillway control wall that the Owner can use to observe the reservoir pool and calculate subsequent discharge based on the spillway rating curve.

The staff gauge is located on the upstream side of the Principal Spillway, attached to the northwest wall (left when looking downstream).

12.3.1 Observation Frequency

In addition to recording water levels during Routine and Detailed Inspections, a manual water level reading should be recorded within 48 hours after rain events of 2 inches or more in a 24-hour period after a flood operations event (elevation 802 feet), and every 12 hours while the reservoir pool elevation is 802 feet or higher.

12.3.2 Action Levels

Action should start to be taken when the reservoir pool is observed to be at elevation 805 feet or higher. When the reservoir pool is observed to be at elevation 805 feet, a “WATCH” is initiated per the Emergency Action Plan.

12.4 SURVEY MONUMENTS

Survey monuments located on the dam embankment and spillway structures can be used to monitor potential settlement. The Permanent Monitoring Instrumentation Plan included in Appendix B of the OM&I Manual shows the location and numbering of the embankment survey monuments and structural monitoring points (SMP). The stationing associated with the survey monuments are presented in Table 57. The stationing associated with the SMPs are presented in Table 58.

Table 57. Summary of Survey Monuments

Survey Monument	Northing	Easting	Design Crest Elevation (feet)	Station
M-1	476,975.71	1,645,551.40	812.12	5+00
M-2	477,972.61	1,645,566.98	812.12	15+00
M-3	478,972.11	1,645,598.67	812.12	25+00
M-4	479,971.84	1,645,621.64	812.12	35+00
M-5	480,971.68	1,645,638.72	812.62	45+00
M-6	481,971.57	1,645,653.68	812.62	55+00
M-7	482,351.72	1,645,711.34	812.62	59+00
M-8	482,378.26	1,646,154.93	812.62	63+50
M-9	483,515.53	1,646,211.94	812.62	75+00
M-10	484,197.51	1,646,281.37	812.62	82+00
M-11	484,150.29	1,647,577.36	812.62	95+00
M-12	484,089.22	1,648,574.41	812.62	105+00
M-13	484,058.48	1,649,823.07	812.62	117+50
M-14	483,869.75	1,650,025.83	812.62	120+50
M-15	482,919.97	1,650,003.64	812.62	130+00
M-16	481,627.87	1,649,882.72	812.62	143+00
M-17	480,778.68	1,650,629.46	812.62	154+50
M-18	479,747.37	1,650,688.60	812.12	165+00
M-19	478,747.39	1,650,683.41	812.12	175+00
M-20	477,747.41	1,650,676.40	812.12	185+00
M-21	477,048.31	1,650,709.74	812.12	192+00
M-22	476,508.95	1,650,732.76	812.12	197+40

Table 58. Summary of Structural Monitoring Points

Structural Monitoring Point (SMP)			
Instrument ID	**Station**	**Offset**	**Top of Conc. Elevation (feet)**
SMP-1	144+90.00	24.75' LT.	810.50
SMP-2	144+90.00	24.75' RT.	810.50
SMP-3	145+19.13	0' LT./RT.	794.00
SMP-4	145+42.13	0' LT./RT.	794.00
SMP-5	145+88.13	0' LT./RT.	794.00
SMP-6	146+34.13	0' LT./RT.	794.00
SMP-7	146+80.13	0' LT./RT.	794.00
SMP-8	147+26.13	0' LT./RT.	794.00
SMP-9	147+72.13	0' LT./RT.	794.00
SMP-10	148+18.13	0' LT./RT.	794.00
SMP-11	148+64.13	0' LT./RT.	794.00
SMP-12	149+10.13	0' LT./RT.	794.00
SMP-13	149+46.13	4.5' RT.	813.00
SMP-14	149+70.88	4.5' RT.	813.00
SMP-15	149+46.13	56' LT.	794.00
SMP-16	149+70.88	56' LT.	798.00
SMP-17	149+46.13	56' RT.	794.00
SMP-18	149+70.88	56' RT.	798.00

12.4.1 Observation Frequency

Elevations should be recorded annually for the first five years after construction completion for each survey monument and SMP on the Survey Monument and SMP Elevation Logs. Elevations should be recorded every 5 years thereafter.

12.4.2 Action Levels

An Engineer should be notified if settlement occurs such that observed embankment elevations at any monument are below the design crest elevation. An Engineer should be notified if more than 0.04 feet (0.5 inches) of cumulative settlement occurs at any SMP.

13.0 PERMITTING

The ECFB Project requires a number of permits prior to construction of the Project from both State and Federal agencies. Authorization of work proposed by the project must be received through the application of a Clean Water Act (CWA) Individual Section 404 permit from the USACE and a CWA Individual Section 401 Water Quality Certification from the Ohio Environmental Protection Agency (OEPA) before construction can begin. The project is subject to review and consultation with the Ohio State Historic Preservation Office (SHPO) and other stakeholders under Section 106 of the National Historic Preservation Act (NHPA) as part of Section 404 permitting process.

13.1 CLEAN WATER ACT

The Project includes the placement of fill within the jurisdictional waters of the U.S., as defined in the *Clean Water Act Jurisdiction Following the U.S. Supreme Court's Decision in Rapanos v. United States and Carabell v. United States* and regulated under Section 404 of the Clean Water Act. Due to these impacts, the Project will require the following authorizations:

- Section 404: Authorization from the USACE for the discharge of dredged or fill material into waters of the U.S.
- Section 401: A Water Quality Certification (WQC) or waiver from the OEPA.

The OEPA issued the Final WQC for the project on January 27, 2023.

The Section 404 Individual Permit Application was first submitted to the USACE on May 2, 2022. A revised Minimization, Restoration and Mitigation Plan for the Eagle Creek Flood Basin Project was sent to the USACE on November 21, 2022. An Unvalidated Permit was issued by the USACE on June 23, 2023.

13.2 NATIONAL FLOOD INSURANCE PROGRAM

Hancock County is a participating community of the National Flood Insurance Program (NFIP) and as such has ordinances related to management of the regulatory floodplain. The Project proposes to place fill within the regulated Floodway and 1% Annual Chance Exceedance Floodplain and therefore must comply with Resolution #261-11 – Hancock County (Unincorporated), Ohio, *Special Purpose Flood Damage Reduction Regulations* (Revised Effective June 2, 2011).

The Project will result in an increase in water surface elevations upstream of the Project for the 1% ACE event and therefore will require approval from the Federal Emergency Management Agency (FEMA) through a Conditional Letter of Map Revision (CLOMR). A CLOMR is submitted to FEMA for authorization of a proposed modification to the NFIP regulatory flood boundaries caused by a project. The CLOMR application (MT-2 Form) was submitted to FEMA on November 23, 2022. FEMA provided their first round of comments on the application on January 26, 2023. Stantec provided a revised submittal to FEMA on March 15, 2023.

FEMA provided additional comments on March 28, 2023. Stantec provided supplemental information to FEMA as recently as June 30, 2023..

Following approval of the CLOMR and prior to construction, a Floodplain Permit application will be submitted to Hancock County.

13.3 OHIO DAM SAFETY

The project will require a construction permit through the ODNR Water Resources Dam Safety Program. The permit application is a two-step process. The first step was completed with the submittal of the preliminary design report on March 8, 2022. Division staff completed their review of the preliminary design report and recommended that it be approved. On May 31, 2022, the Chief of ODNR's Division of Water Resources approved the preliminary design report pursuant to OAC Rule 1501:21-5-02. The Chief also determined the proposed dam meets the criteria for categorization as Class I per OAC Section 1501:21-13-01. As a next step, an application will be submitted to ODNR with the necessary statutory filing fee and surety bond, and final design report package (including plans and specification and a detailed cost estimate). Construction of the dam cannot begin until a permit has been issued by the Chief of the Division of Water Resources.

13.4 NATIONAL POLLUTANT DISCHARGE ELIMINATION SYSTEM

The Clean Water Act amendment of 1987 requires the U.S. Environmental Protection Agency (EPA) to require National Pollutant Discharge Elimination System (NPDES) permits of storm water discharges associated with construction activities. Construction sites disturbing one or more acres of land are required to obtain NPDES permit coverage.

A Construction Stormwater General Permit (CGP) will be required for the project. A Stormwater Pollution Prevention Plan (SWPPP) will be required to be completed prior to the submittal of a Notice of Intent (NOI) application to obtain coverage under the CGP. The NOI application must be submitted at least 21 days prior to the initiation of construction activities. Although the SWPPP does not need to be submitted to Ohio EPA to obtain coverage under the CGP, it must be retained at the construction site at all times during the construction activity. However, the local governing authority may require approval of an SWPPP or a sediment and erosion control plan prior to initiation of construction activities.

13.5 AGENCY CONSULTATION

As a Project that requires Federal permit approvals, consultation is required to demonstrate compliance with Section 106 of the NHPA and Section 7 of the ESA.

Under Section 106 of the NHPA, Phase I archaeological surveys are completed to identify the locations of potential cultural resource sites within the project area, to make preliminary recommendations regarding NRHP eligibility, and to allow for the possibility of modification to project design in order to preserve sites that are potentially eligible for the NRHP. A Phase I Archaeological Survey was completed for the project area. The Phase I Archaeological Survey Report dated August 2021, presents the methods and findings of the survey (Mannik & Smith, 2021a). The findings and recommendations were submitted to the SHPO for review. The SHPO concurred with the findings of the report and recommended that the identified potential cultural resource site clusters be subject to additional testing or avoidance. The SHPO also concurred with the deep testing recommendations for Eagle Creek in the northern portion of the project area at the location of the Eagle Creek embankment crossing.

The Eagle Creek Site Cluster could not be avoided by the Project and MSG undertook Phase II testing in the fall of 2021. As a result of the Phase II findings, MSG recommended the sites not eligible for listing in the NRHP (Chidester et al. 2022). The SHPO concurred with these findings in a letter dated April 27, 2022.

Archaeological Monitoring will be performed during construction at Sites 33HK991 and 33HK992 (Byal site cluster) and within the Eagle Creek channel banks where excavation occurs. The Archaeological Monitoring Plan was approved by SHPO and is implemented into the project's Technical Specifications.

Under Section 106 of the NHPA, a survey is also required to identify historic properties within the area of potential effect (APE) that may be directly or indirectly impacted by a federal undertaking. A survey was conducted to identify historic / architectural resources within the APE. A Phase I Survey was completed for the project area. The significance of resources within the APE was evaluated according to their eligibility for listing in the NRHP. It was determined that none of the 26 identified properties are eligible for listing in the NRHP due to a lack of integrity caused by many years of alterations. The SHPO concurred with the recommendation and no further action is necessary related to the historic / architectural resources. The Phase I Architectural / Historical Survey Report dated July 2021, presents the methods and findings of the survey (Mannik & Smith, 2021b).

Section 7 of the ESA requires consultation with the USFWS when proposed work may affect a listed endangered or threatened species, or a designated critical habitat. Stantec is following the requirements of Section 7 through the development of the proposed project and is coordinating with the USFWS, as needed. The USFWS concurred that the project is not likely to result in the take of bald eagles based on the location of an existing nest that was identified near the project site. The USFWS stated that no future coordination is necessary at this time relative to this project and bald eagles.

14.0 OPINION OF PROBABLE CONSTRUCTION COSTS

A Class 2 opinion of probable construction costs (OPCC) was developed for the ECFB based on Final Design drawings. In accordance with the Association for the Advancement of Cost Engineering's (AACE) cost estimate classification system (Recommended Practice 18R-97), the Class 2 cost estimate developed retains a stated accuracy range of -10% to +10% of final installed construction costs (Q1 2023 dollars). More specifically, the AACE defines a Class 2 cost estimates as follows.

> AACE International Class 2 Cost Estimate - Class 2 estimates are generally prepared to form a detailed control baseline against which all project work is monitored in terms of cost and progress control. Typically, engineering is from 30% to 70% complete Class 2 estimates involve a high degree of deterministic estimating methods. Class 2 estimating efforts are characterized by significant line-item detail. Typical accuracy ranges for Class 2 estimates are -5% to -15% on the low side, and +5 to +20% on the high side, depending on the technological complexity of the project. (*AACE International Recommended Practices and Standards).*

Stantec has no control over the costs of labor, materials, competitive bidding environments, unidentified field conditions, financial and/or commodity market conditions, or any other factors likely to affect the OPCC of this project, all of which are and will unavoidably remain in a state of change, especially in light of high market volatility attributable to Acts of God and other market forces or events beyond the control of the parties. As such, this OPCC is to be considered a "snapshot in time" estimate and is based on normal market conditions, defined by stable resource supply/demand relationships, and does not account for extreme inflationary or deflationary market cycles. As with any cost estimate performed prior to procurement of services (equipment, construction, etc.), Stantec cannot and does not make any warranty, promise, guarantee or representation, either express or implied that proposals, bids, project construction costs, or cost of O&M functions will not vary significantly from Stantec's good faith Class 2 OPCC.

14.1 OPCC PRICING METHODOLOGY

The Class 2 OPCC was prepared using a detailed pricing methodology with a detailed crew analysis and budget quotes. Using a simplified high level work breakdown structure (WBS), major project scope elements were organized into a two-tiered template to focus the estimating effort to items of significance defined as cost drivers.

The project team, using draft 100% design or other available project information, determined the OPCC quantity basis. Quantities are derived from the design drawings included in Appendix B. Major items such as earthwork volumes were calculated using digital models. For example, existing and proposed surfaces were prepared in Civil3D to estimate the quantity of excavation and embankment placement required. Additionally, expected shrinkage factors were accounted for after considering the geotechnical properties of the soil and required compaction values.

Other furnished quantity inputs were developed using average areas, volumes, and profiles by the design team and remain a source of cost estimate deviation until future design refinement allows for rigorous verification of the quantity basis. Contingency allowances were applied to minimize the risk of cost deviation in relation to future quantity refinement.

14.1.1 OPCC Exclusions

As developed, the Class 2 OPCC excludes the following program costs:

- Property purchase or land rights expenses
- Property or consumption taxes
- Water rights and use fees
- Owner internal project management costs
- Corporate administrative and governance overheads
- Facility capital costs
- Interest During Construction (IDC)
- Unconventional environmental mitigation measures
- Exposure to hyper-inflationary or hyper-deflationary market conditions
- Exposure to landslides due to bank erosion impacts
- Costs associated with improvements to local infrastructure
- Mitigation of flooding impacts for cleared forest lands
- Mitigation of wildlife habitat loss excessive stream flow releases
- Owner insurance coverage policies
- Overly prescriptive permit conditions or specifications
- Risk to build cofferdams/dikes to extreme river flow conditions
- Severe weather impacts
- Exposure to swelling soils or rebound consequences
- Design Costs
- Owner Sunk Costs

14.1.2 OPCC Assumptions

As developed, the Class 2 OPCCs consider the following assumptions or qualifications:

- Pricing basis = 1st quarter of 2023.
- Suitable material aggregate pits will be located within 20 miles of the project site
- Sufficient and qualified craft labor resources are available without significant wage premiums
- Sufficient and viable construction equipment resources are available without major premium
- Industry standard commercial terms will be applied to all procurements
- Owner has sufficient and qualified personnel to manage the project to stated cost & time objectives
- Sufficient supply of qualified contractors will tender bid proposals
- The contracting strategy will maximize competition and promote project objectives
- No external or internal delays to achieving the project approval
- Stable resource market conditions and minimal geo-political disruptions

14.1.3 Pricing Basis

Pricing reflects the estimator's opinion as to the probable costs that a "prudent" contractor would include in his tender to construct the defined facilities. Unless specifically stated, the OPCC does not capture framework costs borne by the owner for pre-construction activities or for expenses related to the management and support of field construction activities. The OPCC is intended to be an indication of fair market value and is not necessarily a predictor of lowest bid. Fair market value is assumed to be a mid-range tender considering four or more competitive bids. Finally, OPCC pricing is predicated on the contractor's compliance with all contract specifications and design parameters during field execution activities.

Past, relevant bids, detailed estimates, and manufacturers' product quotes were reviewed to develop unit costs. Estimators also referenced the publicly available Ohio Department of Transportation (ODOT) historical bid database, which was filtered to identify recent (2022, where possible) projects with similar quantities for corresponding line items.

14.1.4 Direct Cost Development

Direct costs representing the project's fixed physical scope are estimated against the WBS to organize the estimate details. Software functionality allows the direct cost detail to be decomposed to multiple sub-levels, which are referred to as item activities. Class 2 cost estimates derive pricing under a crew productivity analysis per line item.

14.1.5 Indirect Cost Development

For Class 2 OPCCs, indirect costs are estimated in a bottoms-up fashion to determine actual resource needs in relation to the proposed construction duration schedule. Indirect Costs vary between contracts and what contractors consider and indirect cost. The following is a listing of typical indirect costs to be considered in a cost estimate. Some of the following items may be line items in the estimate and therefore shift to a direct cost line item. The indirect costs together with profit, bonding and insurance and contingencies are spread to direct cost items to make a bid price for a specific line item in the bid form or cost estimate. The following list are typical indirect costs and only the ones that apply to the Eagle Creek Flood Basin were priced into the estimate.

- Contractor Management and Supervision
- Accounting and Time Keeping
- Purchasing and Warehousing
- Contractor Engineering
- Surveying
- Quality Control
- Administration
- Computers
- Telephone
- Professional Services
- Association Dues
- Travel & Conference
- Permits
- Photos
- Employee Moves
- Special Business Tax
- Office Supplies
- Engineering Supplies
- Vehicle Licenses
- Miscellaneous.

- Safety / Safety Personnel
- PPE - First Aid, Hard hats, Etc.
- Signs
- Watchman
- Flagmen
- Fire Protection
- General Job Services
- Commercial Power
- Shop Power
- Auxiliary Field Lighting
- Water supply
- Potable Water supply
- Air supply
- Radios
- Building Maintenance
- Fuel for Heat
- Road Maintenance
- Railroad Siding Maintenance
- Yard Rent & Maintenance
- Periodic and Final Clean up
- Storm Drainage & Snow Removal
- Latrines
- Job office Rent
- Service Vehicles
- Fuel Truck
- Grease Truck
- Mechanic Trucks
- Tire Truck
- Busses
- Ambulance
- Flatbed Service Truck
- Lowboy
- Service Crane
- Small Tools & Supplies
- Camp Operation
- Camp Labor
- Camp Food
- Outside board & lodging
- Camp Income for Outside People
- Camp Supplies
- Per Diem in lieu of Board & lodging
- Camp Power
- Transportation & Travel
- Special Insurance
- Builder Risk
- Strike Insurance
- Marine Insurance
- Railroad Insurance
- Bonds
- Liquidated Damages
- Office & Shop Installation
- Site Prep
- Job Office setup
- Equipment Shop Setup Fuel, Storage Parts
- Carpenter Shop
- Warehouse
- Change Houses
- Powder Magazine
- Owners Engr Office
- Service Facilities Installation
- Commercial Power
- Power Distribution
- Water & Distribution
- Air System
- Communications
- Access & Haul Roads
- Rail Siding
- Camp Facilities Installation
- Start up Camp
- Camp Setup
- Site Prep
- Water Supply
- Wastewater
- Camp Power
- Unload & Setup
- Rail Freight
- Barge Freight
- Ferry
- Other
- Over the road cost
- Unload cost
- Set up Crusher
- Set up any Plants
- Shut down Plants Cost
- Startup Plants
- Demobilization

14.1.6 Labor Rate Development

All-inclusive craft labor rates all built-up from local Davis-Bacon wage determinations (for Hancock County, Ohio) to include all applicable fringes (i.e., health and welfare, vacation, training, and union dues,) and tax burdens (I.e., workers compensation, payroll taxes). The rates for the OPCC were based on a contractor working a 10-hour shift five days a week.

14.1.7 Equipment Rate Development

All-inclusive rolling equipment rates are determined from published equipment rates such as Equipment Rate Blue Book and other sources. Equipment rates vary from contractor to contractor and each contractor has different methods of ownership calculations to write of plant cost to the project. The options to would be the following for each piece in the report:

- Buy and sell the equipment when the project is complete
- Rent the equipment
- Use an owned piece of equipment
- Buy new and keep the equipment at the end of the project.

14.1.8 Cost Escalation Analysis

The Class 2 OPCC has a 1st quarter of 2023 pricing basis. The OPCC shown in this Final Design Report does not consider escalation to the Notice To Proceed, or to the mid-point of construction.

14.1.9 Allowances and Contingency

Allowances are added to the OPCC to anticipate expenses for known but undefined scope items. This allowance is not a contingency. Certain elements of the work have not yet been fully defined and were separated as allowances. The utility relocation allowance accounts for contractor coordination associated with relocation of overhead electric utilities. Stantec has initiated contact with utility owners and assumes that this infrastructure will need to be removed and relocated ahead of construction or during construction, but the work required is still yet to be defined pending utility coordination

Although unknown risks or unforeseen market conditions, quantity and cost estimating accuracies in productions and equipment rates may occur, the OPCC presented in this Final Design Report excludes contingency from the cost estimate. The OPCC also excludes an allowance for the owner's costs or management reserve, which represents the owner's contingency for changed field conditions and is not included in the above contingency.

Costs are presented in 2023 dollars to reflect the current market. The methods for development of this cost estimate, including detailed quantity take-offs and unit cost derivations, are consistent with a Class 2 estimate, as defined by the Association for the Advancement of Cost Engineering (AACE) International, and should be considered to have an expected accuracy range of -10% to +10%.

14.2 OPINION OF PROBABLE CONSTRUCTION COSTS

Table 59 summarizes the OPCC for the ECFB project. The line items in Table 59 assume a 14% contractor markup and do not factor in potential escalation or construction contingencies.

Table 59. Opinion of Probable Construction Cost Summary

Item #	Description	**100% Final Design
A	General Works, Demolition, and Site Preparation	$2,792,000
B	Dam Embankment Earthwork	$8,902,000
C	Seepage Mitigation	$2,519,000
D	Instrumentation	$127,000
E	Road Modifications and Site Drainage	$2,187,000
F	Stream, Wetlands, Fish, and Wildlife	$2,779,000
G1	Spillways and Outlet Structures	$8,167,000
G2	Mechanical Gates	$200,000
G3	Electrical	$199,000
G4	Permanent Erosion Control	$546,000
H	Interior Features	$383,000
I	Contractor Indirect Costs	$8,293,000
J	Allowances	$-
K	Contractor Markups	$5,282,000
	Total Construction Price	$42,376,000
	Final Design Class 2 Estimate Cost Range	
	-10%	$38,138,000
	10%	$46,614,000

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APPENDIX A

Design Criteria Document

APPENDIX B

Design Drawings

APPENDIX C

Technical Specifications

APPENDIX D

Hydrologic and Hydraulic Analysis Report

APPENDIX E

Geotechnical Design Report

APPENDIX F

Soil Material Borrow Study

APPENDIX G

Dam Embankment Design Technical Memorandum

APPENDIX H

Exterior Drainage Analysis Report

APPENDIX I

Principal Spillway Technical Memorandum

APPENDIX J

Auxiliary Spillway Technical Memorandum