Eagle Creek Flood Basin – Preliminary Design Report

Hancock County, Ohio

March 7, 2022

Prepared for:

Maumee Watershed Conservancy District
1464 Pinehurst Drive
Defiance, Ohio 43512

Prepared by:

Stantec Consulting Services Inc.
11687 Lebanon Road
Cincinnati, Ohio 45241

Revision	Description	Author		Quality Check		Independent Review	
A	Draft	D. Bolubasz	1/21/22	D. Hayson J. Menninger	2/9/22 2/11/22	D. Dalton	2/11/22
0	Final	D. Bolubasz	3/7/22	D. Hayson J. Menninger	3/7/22	D. Dalton	3/7/22

Prepared by Danielle Bolubasz
(signature)

Danielle Bolubasz, PE

Reviewed by David Hayson
(signature)

David Hayson, PE, SI

Approved by _______________________
(signature)

John Menninger, PE

Table of Contents

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

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 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 Preliminary Design Report (PDR) 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, interior drainage improvements and land use design, and exterior drainage features. Figure E1 shows the proposed embankment alignment and primary project components. Table E1 provides a design summary of the primary project components.

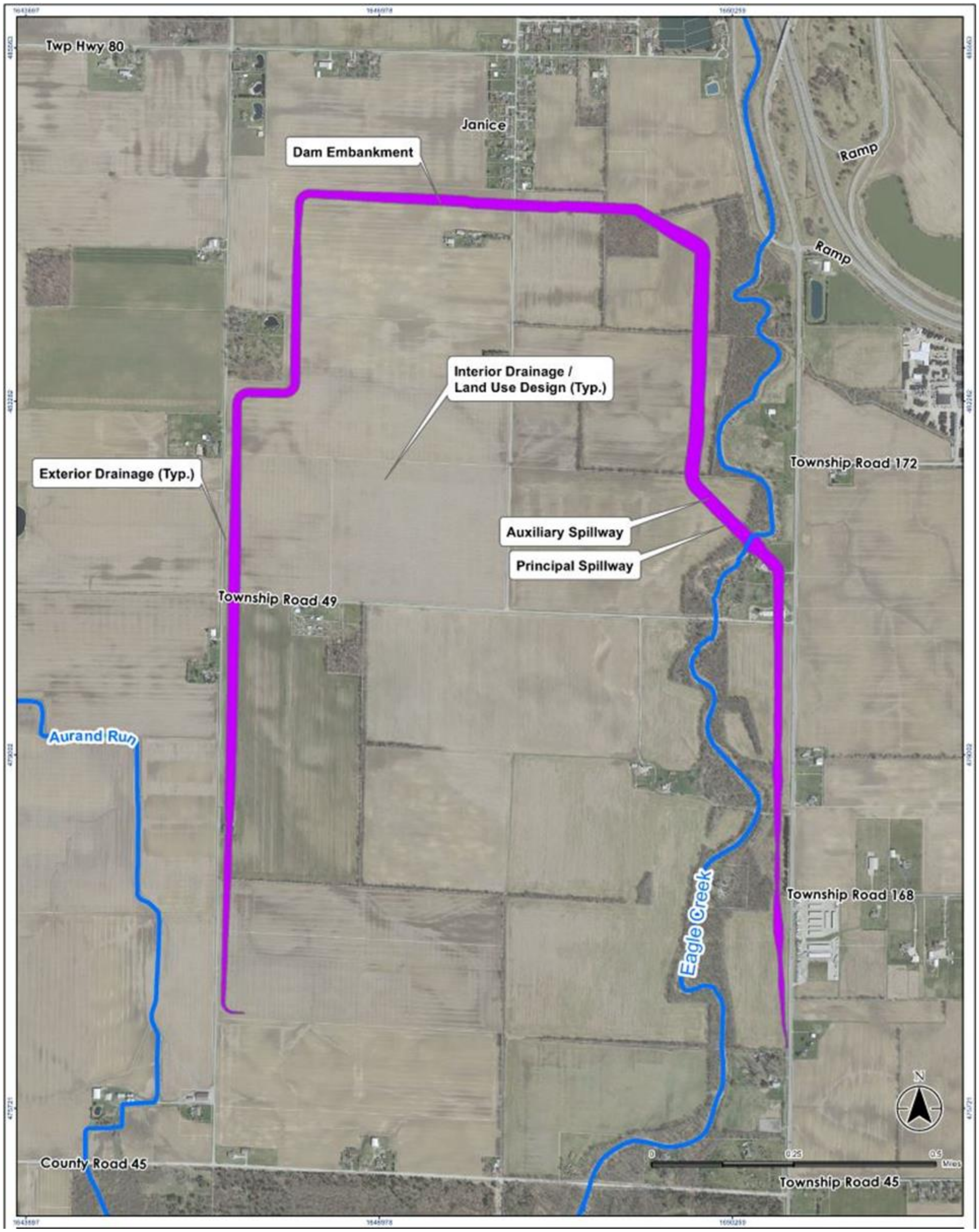

Figure E1. Project Components

Table E1. Eagle Creek Flood Basin Design Components Summary

Inline Dam	
Earthen Embankment	
Crest Elevation	813.0 ft
Crest Length	19,699 ft
Top Width	12.0 ft
Side Slopes	3H:1V
Dry-Storage Reservoir	
Storage Capacity at 100-year Event (WSE at 807.0 ft)	6,900 ac-ft
Storage Pool Area (Elevation 807.0 ft)	906 acres
Storage Capacity at Probable Maximum Flood Event (WSE at 810.0 ft)	9,776 ac-ft
Integrated Spillway Structure	
Principal Spillway: Control Wall with Orifices and Baffled Chute	
Control Wall Center Orifice	1 @ 6.0 ft x 3.5 ft
Control Wall Side Orifices	2 @ 6.0 ft x 3.0 ft
Principal Spillway Width	20.0 ft
Baffled Chute Spillway Length	80.0 ft (50.0 ft of baffles)
Baffle Height	1.0 ft
Trash Rack	20.0 ft W x 13.0 ft H x 40.0 ft L
100-year Event Discharge Capacity (WSE at 807.0 ft)	1,215 cfs
Probable Maximum Flood Discharge Capacity (WSE at 810.0 ft)	1,264 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: USBR Type I Basin	
Stilling Basin Length	21.0 ft
Exterior Drainage	
Southwest Trapezoidal Ditch Total Length	3,967 ft
Northwest Trapezoidal Ditch Total Length	3,927 ft
North Trapezoidal Ditch Total Length	4,275 ft
East Trapezoidal Ditch Total Length	3,563 ft
East Stormwater 24” Conduit Total Length	3,739 ft
Interior Wetlands	
Wetland #1 Footprint	77 acres
Wetland #2 Footprint	45 acres
Wetland #3 Footprint	20 acres

Dam Classification – The Ohio Department of Natural Resources, 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. The proposed flood basin is anticipated to be classified as a Hazard Class I Dam per Ohio Administrative Code (OAC) Section 1501:21-13-01 based on both the storage volume and potential downstream hazard sudden failure consequence criteria.

Earthen Dam– The basin will be formed by a 3.75 mile long earthen embankment dam on three sides. The dam ranges in height from approximately 1 foot tall at the tie-in locations at the upstream end of the Basin to about 30 feet tall at the embankment's intersection with Eagle Creek. Fill soils required for earthen embankment construction are anticipated to come from within the interior of the basin.

The embankment dam geometry will consist of the following:

- Crest elevation = 813 feet minimum (freeboard requirements for design flood pool)
- Embankment side slopes = 3H:1V (maximum slope for access, monitoring, and maintenance)
- Crest width = 12 feet minimum (vehicle access for maintenance and monitoring)
- Crest surface slope = 2 percent minimum (to provide surface drainage)
- 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 E2.

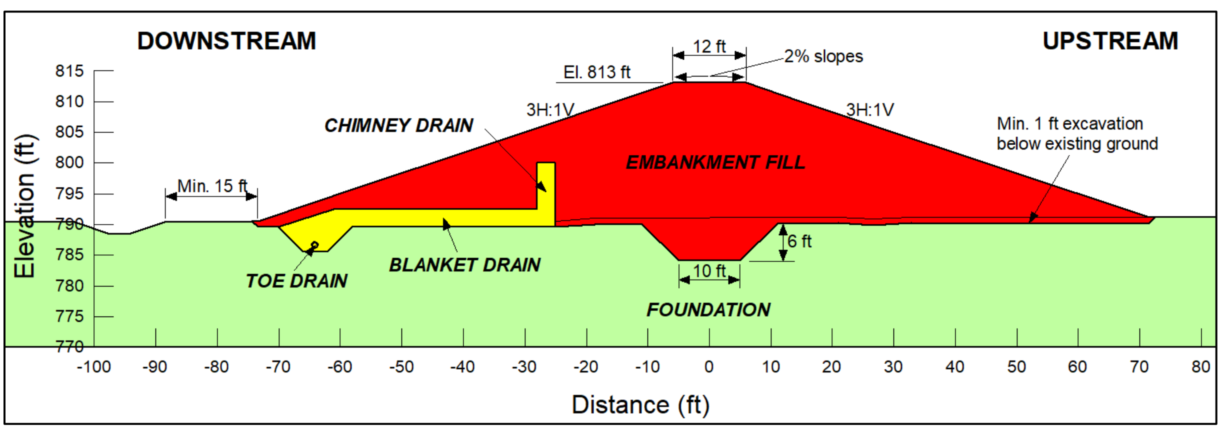

Figure E2. Typical Embankment Cross Section

Principal Spillway – The Principal Spillway replaces a section of the dam embankment and is the primary structure designed to convey Eagle Creek flows downstream. The spillway is situated within a realigned reach of Eagle Creek and will pass normal Eagle Creek flow (up to 630 cfs) through a static control wall with three rectangular orifices. When a large storm event occurs, the Principal Spillway will restrict the flow passing through the control wall and start to fill the basin. The ECFB is designed to store up to the 1% ACE (100-year) flood event, while discharging no more than 1,250 cfs downstream into Eagle Creek.

As a natural system, Eagle Creek has the potential to convey large debris such as trees to the Principal Spillway. A sloping trash rack is provided upstream of the control wall to maintain proper function during a flood event and to allow the Principal Spillway to continue to pass flow as designed under debris loading.

Downstream of the control wall is a baffled chute designed for energy dissipation and depth/velocity control associated with fish passage criteria. 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.

Auxiliary Spillway – The Auxiliary Spillway is a secondary structure, used to discharge flows exceeding the 1% ACE (100-year) flood event, once the basin's storage volume is at capacity. The spillway has additional discharge capacity to safely pass flows up to the design flood (Probable Maximum Flood). The crest of the labyrinth weir will be at elevation 807.0 feet, approximately the 1% ACE (100-year) water surface elevation

The Auxiliary Spillway is a 13-foot tall, steel reinforced, concrete labyrinth weir with an ogee shaped crest. Downstream of the labyrinth weir will be an USBR Type I natural jump stilling basin which will dissipate energy and provide the transition for flow downstream to Eagle Creek.

Exterior Drainage - Proposed ditches, conduits, and culverts are designed to convey storm event runoff away from the embankment and to a suitable location downstream without impacting the dam embankment or adjacent roadways.

Interior 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.

Wetlands - The borrow sources for the dam will be reclaimed as two large wetlands on the interior of the basin with a third wetland along the riparian corridor of Eagle Creek. The wetlands are sized to reduce excess excavation in conjunction with the soil material borrow required for the embankment. Construction of the wetlands is expected to return areas of high-value wildlife to the community and provide an aesthetically pleasing viewshed within.

Recreational Features – Passive recreation is anticipated to be an important component of the basin interior post-construction. A variety of passive recreational features are currently under consideration by project stakeholders. Some features under consideration include a public pedestrian trail system to access the wetlands and adjacent naturalized areas, fitness stations, playground equipment, wildlife/birding platforms, and kiosks describing the functions and values of the wetlands and the creatures that inhabit them.

Project Impacts - Two local roads will be terminated during construction. Township Road 49 will be fully decommissioned between US-68 and Township Road 76. 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 cul-de-sac is planned at the site of Township Road 77 termination to allow for residents, school buses, and other vehicles to turn around. Transportation alignments that could potentially mitigate the impact to Township Road 49 are outside the scope of this Preliminary Design Report. However, past studies considered conceptual alternatives for relocation of Township Road 49.

Existing utilities will be relocated or abandoned where they cross the proposed dam alignment, are adjacent to the dam embankment, or are within the impoundment.

Segments of certain roadways upstream of the project footprint are expected to be inundated for some duration during large flood events.

Project Benefits – Model results show that the Eagle Creek Flood Basin project results in a peak flow reduction of about 2,700 cfs (17% decrease) on the Blanchard River during the 1% ACE WSE which translates to about 2.1 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 1,740 parcels and 1,680 acres from the regulatory floodplain.

Opinion of Probable Cost (OPCC) – An opinion of probable construction costs was developed for the ECFB based on Preliminary Design. A contingency of 15% was applied to the base construction cost to account for potential design refinements between Preliminary and Final Design. The contingency may cover unforeseen administrative and legal fees as well as obstacles that may arise throughout detailed design. Table E3 summarizes the OPCC for the ECFB project.

Table E3. Preliminary Opinion of Probable Construction Cost

Description	Cost
Base Construction Cost	**$32,650,000**
Estimate Contingency (15%)	$4,900,000
Total Construction Cost: Single Point Estimate (w/ 15% Contingency)	**$37,550,000**
Owner Project Allowances	**$40,400,000**
Total Project Cost: Single Point Estimate (February 2022 Dollars)	**$77,950,000**
Escalation (present to mid-point of construction)	$4,130,000
Total Project Cost: Single Point Estimate (With Escalation)	**$82,080,000**
Total Project Cost Range (-10% to +20%)	**$76M - $94.3M**

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
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 Preliminary Design Report (PDR) for the Eagle Creek Flood Basin (ECFB) project. The ECFB is designed to provide storage 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 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 22 times from 1913 to 2021, and of these events, nine have occurred since 2007. Six events between 2007 and 2017 are among the top eleven stages on record, with four 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 by September 2022. 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.

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.

1.3 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.

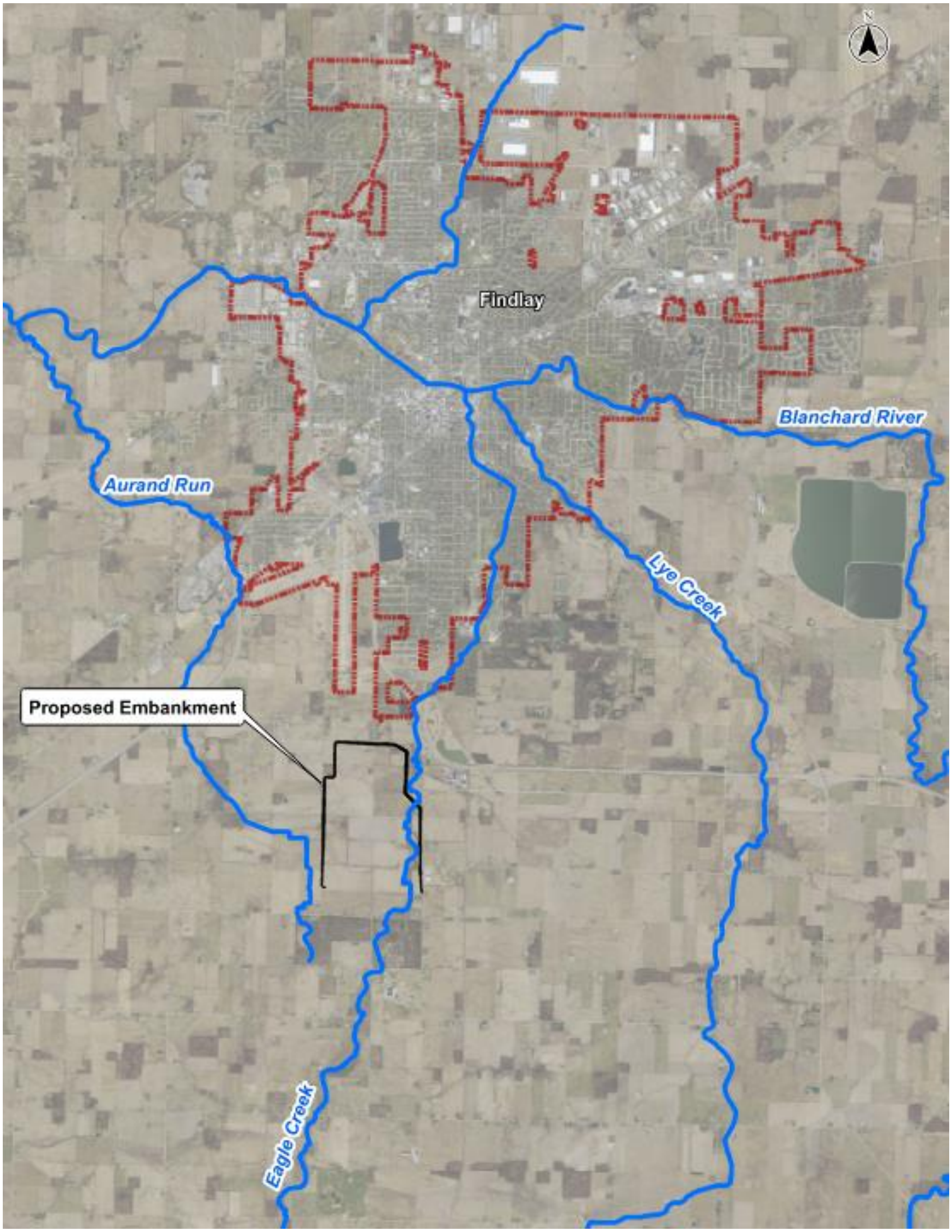

Figure 1. Project Location

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.

1.5 PRELIMINARY DESIGN REPORT

Elements of the Preliminary Design phase includes field data collection, hydrologic and hydraulic modeling, environmental and regulatory agency permitting coordination, engineering analyses, and design. This Preliminary 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
- Preliminary Opinion of Probable Construction Cost (OPCC).

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

The Preliminary Design Report follows requirements established under the Ohio Administrative Code (OAC) Rule 1501:21-5-02 and includes the following:

- General description of the dam **[Section 5.0 -** Dam Embankment**]**, proposed dam classification **[Section 2.3]**, purpose of dam **[Section 1.4 -** Project Purpose**]**, and impact of dam to human life, health and property **[Appendix D -** Hydrologic and Hydraulic Analysis Report**]**;
- Maps showing the location of the proposed structure, county and township lines, reservoir extents, utility locations, topography, and affected structures **[Appendix** L2 – Project Vicinity Figure**]**;

- Written report of the surficial surface **[Appendix E -** Geotechnical Design Report**]**;
- Cross sections of dam accurately showing proposed elevations, pool levels, and top width **[Appendix** L1 - Preliminary Design Drawings**]**;
- Log of borings in the foundation and borrow areas **[Appendix E -** Geotechnical Design Report **(Exhibit C – Geotechnical Exploration Data Report]** and results of any seismic and resistivity subsurface investigations **[Appendix E -** Geotechnical Design Report**]**;
- Preliminary design assumptions **[Section 5.3.1]**, tentative conclusions **[Section 8.0 -** Reservoir Routing**]**, and references **[Section 14.0]**;
- Diversion of stream flow **[Section 11.4]**;
- Reservoir inundation map **[Appendix D -** Hydrologic and Hydraulic Analysis Report**]**; and
- Preliminary cost estimate for structure and appurtenances **[Section 13.0]**.

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% Annual Chance Exceedance (ACE) flood event. With this goal in mind, the ECFB is to be 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 in advance of the design efforts related to the ECFB to inform the permitting and design processes. 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 agricultural and drain tile has been installed in many areas within the project site to promote drainage. There are seven existing residential structures within the interior portion of the proposed basin. Additionally, wooded areas exist near the 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 flows south to north along the eastern portion of the project site, flowing into the Blanchard River 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 north, 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 671 acres of land within the project footprint through 2021. Figure 2 shows the parcels purchased by the MWCD (yellow polygons). 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).

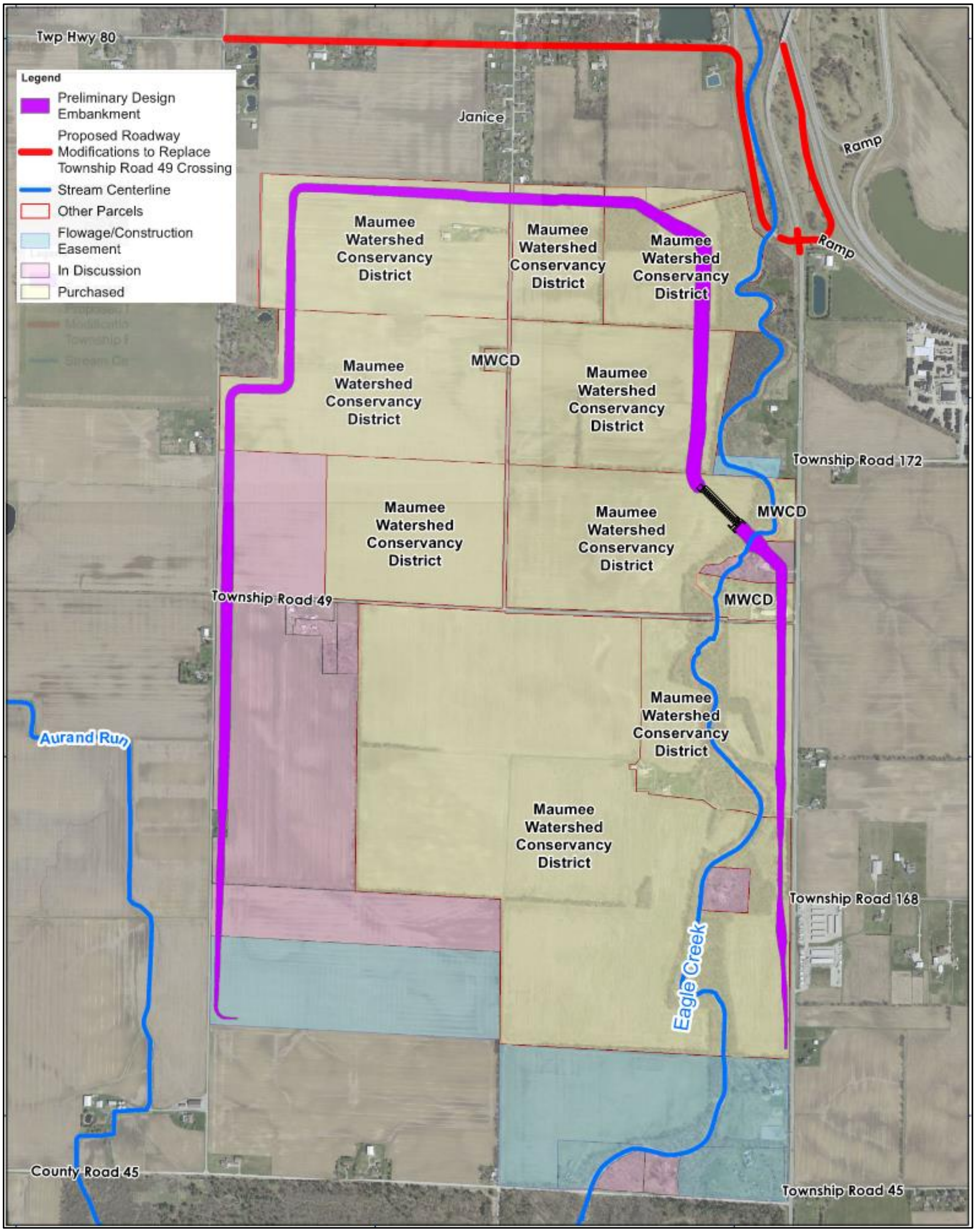

Figure 2. Land Ownership

2.1.5 Existing Utilities

Existing utilities in the project area may 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 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. As part of the wetland and waterbody delineation.

2.1.7.1 Wetlands

The wetland and waterbody delineation field surveys that occurred in 2019 and 2021 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.

2.1.7.4 Methods and Results

The Wetland and Waterbody Delineation Report, dated December 13, 2021, presents the methods and findings of the wetland and waterbody delineation survey. The report is included in Exhibit A of Appendix B - Field Surveys.

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.

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.

The Threatened and Endangered Species Habitat Assessment Report dated October 30, 2019, presents the methods and findings of the survey. The report is included in Exhibit B of Appendix B - Field Surveys.

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. The concurrence letter and confirmation from USFWS is provided as Exhibit C to Appendix B - Field Surveys.

2.1.8.2 Mussel Reconnaissance

Stantec conducted a mussel reconnaissance survey for the ECFB project area. 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. More discussion on the survey findings is in the Freshwater Mussel Reconnaissance Survey on Eagle Creek and Aurand Run report in Exhibit D of Appendix B - Field Surveys.

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.

Sites 33HK991 and 33HK992 were recorded as the Byal site cluster and represent the remains of a historic farmstead. Sites 33HK1008 and 33HK1011-1014 were recorded as the Eagle Creek site Cluster and may represent an Early Archaic habitation location. These site clusters were recommended for monitoring, additional testing, or avoidance. Additional investigations of these site clusters may be necessary to enable formal determinations of eligibility for listing on the National Register of Historic Places (NRHP).

The 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 may be recommended in certain areas as the design is further refined.

The Phase I Archaeological Survey Report dated August 2021, presents the methods and findings of the survey. The report is included in Exhibit E of Appendix B - Field Surveys.

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 Ohio State Historic Preservation Office (SHPO) concurred with the recommendation.

Additional detail of the Phase I History / Architecture Survey dated September 2021 is in Exhibit F of Appendix B - Field Surveys.

2.1.10 Stream Assessment and Geomorphic Conditions

A geomorphic assessment of Eagle Creek was conducted to support Preliminary Design of the Eagle Creek Flood Basin. 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 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 embankment alignment and primary project components.

Table 3. Eagle Creek Flood Basin Design Components Summary

Inline Dam	
Earthen Embankment	
Crest Elevation	813.0 ft
Crest Length	19,699 ft
Top Width	12.0 ft
Side Slopes	3H:1V
Dry-Storage Reservoir	
Storage Capacity at 100-year Event (WSE at 807.0 ft)	6,900 ac-ft
Storage Pool Area (Elevation 807.0 ft)	906 acres
Storage Capacity at Probable Maximum Flood Event (WSE at 810.0 ft)	9,776 ac-ft
Integrated Spillway Structure	
Principal Spillway: Control Wall with Orifices and Baffled Chute	
Control Wall Center Orifice	1 @ 6.0 ft x 3.5 ft
Control Wall Side Orifices	2 @ 6.0 ft x 3.0 ft
Principal Spillway Width	20.0 ft
Baffled Chute Spillway Length	80.0 ft (50.0 ft of baffles)
Baffle Height	1.0 ft
Trash Rack	20.0 ft W x 13.0 ft H x 40.0 ft L
100-year Event Discharge Capacity (WSE at 807.0 ft)	1,215 cfs
Probable Maximum Flood Discharge Capacity (WSE at 810.0 ft)	1,264 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: USBR Type I Basin	
Stilling Basin Length	21.0 ft
Exterior Drainage	
Southwest Trapezoidal Ditch Total Length	3,967 ft
Northwest Trapezoidal Ditch Total Length	3,927 ft
North Trapezoidal Ditch Total Length	4,275 ft
East Trapezoidal Ditch Total Length	3,563 ft
East Stormwater 24” Conduit Total Length	3,739 ft
Interior Wetlands	
Wetland #1 Footprint	77 acres
Wetland #2 Footprint	45 acres
Wetland #3 Footprint	20 acres

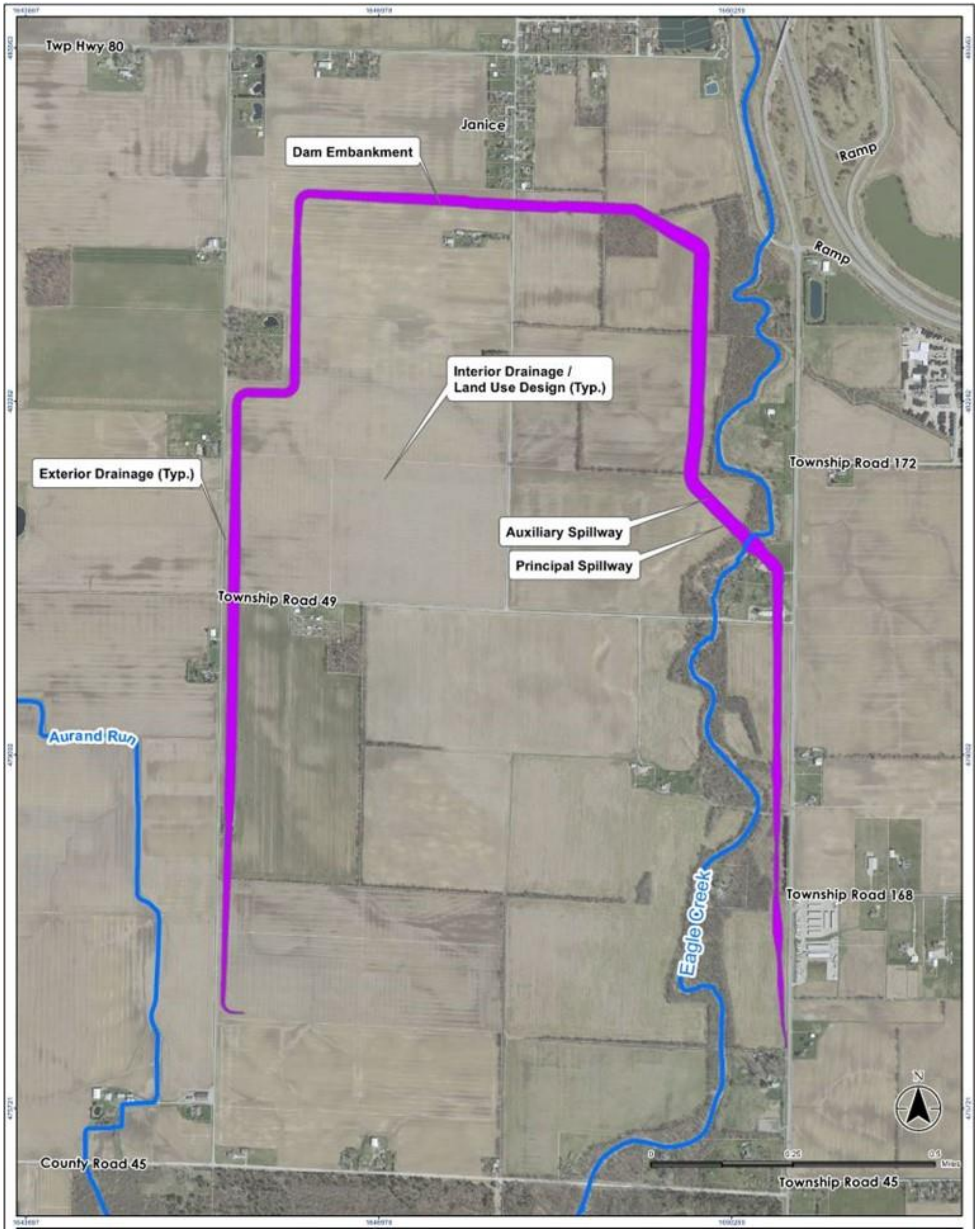

Figure 3. Project Components

2.2.1 Earthen Embankment / Dry Reservoir

The primary flood risk reduction element for the project will consist 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 embankment length of approximately 3.75 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 proposed to be 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 anticipated to be bound by high ground to the south, approximately 1,750 feet north of County Road 45. The footprint of the basin within the earthen embankment will be approximately 765 acres.

The basin will remain dry during normal flows along Eagle Creek and will begin to store flood waters during large storm events to reduce peak flow rates in Eagle Creek and, ultimately, the Blanchard River downstream.

2.2.2 Integrated Spillway Structure

2.2.2.1 Principal Spillway

The Principal Spillway consists of a static control wall with three rectangular orifices, a baffled chute downstream of the control wall, and a trash rack upstream of the control wall. 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.

2.2.2.2 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 a given 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 will be an USBR Type I natural jump stilling basin.

2.2.3 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.

2.2.4 Interior Design

2.2.4.1 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.

2.2.4.2 Wetland Design

A hybrid wetland concept will be utilized in the design, with two large wetlands on the interior making use of the project's borrow pits, and a third wetland along the riparian corridor of Eagle Creek. The wetlands are sized to reduce excess excavation in conjunction with the soil material borrow required for the embankment.

2.2.5 Secondary Project Components

Secondary project components include utility relocations and local road terminations at Township Road 49 and Township Road 77. Construction of the dam embankment would alter existing local transportation routes and affect access to US-68. Transportation alignments that could potentially mitigate the impact to Township Road 49 are outside the scope of this Preliminary Design Report. However, past studies considered conceptual alternatives for relocation of Township Road 49. These concepts have previously been developed by Stantec and presented to ODOT for review. Select alternatives were recommended for further evaluation. The next step in the roadway design will likely include a feasibility study and possibly an interchange modification study in coordination with ODOT.

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

The proposed flood basin is anticipated to be classified as a Hazard Class I Dam per Ohio Administrative Code (OAC) Section 1501:21-13-01 based on both the storage volume and potential downstream hazard sudden failure consequence criteria.

- The storage volume at Elev. 807.0 is 6,900 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. A preliminary Dam Breach Analysis was conducted and is summarized the Hydrologic and Hydraulic Analysis Report included as Appendix D.

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 mi^2 of the Eagle Creek watershed, and approximately 0.7 mi^2 of the Aurand Run watershed, upstream of the proposed embankment. presented in Figure 4. Except for a portion of the City of Findlay north of the project area at the downstream end, the watershed is sparsely developed and the primary landuse is agricultural row crops.

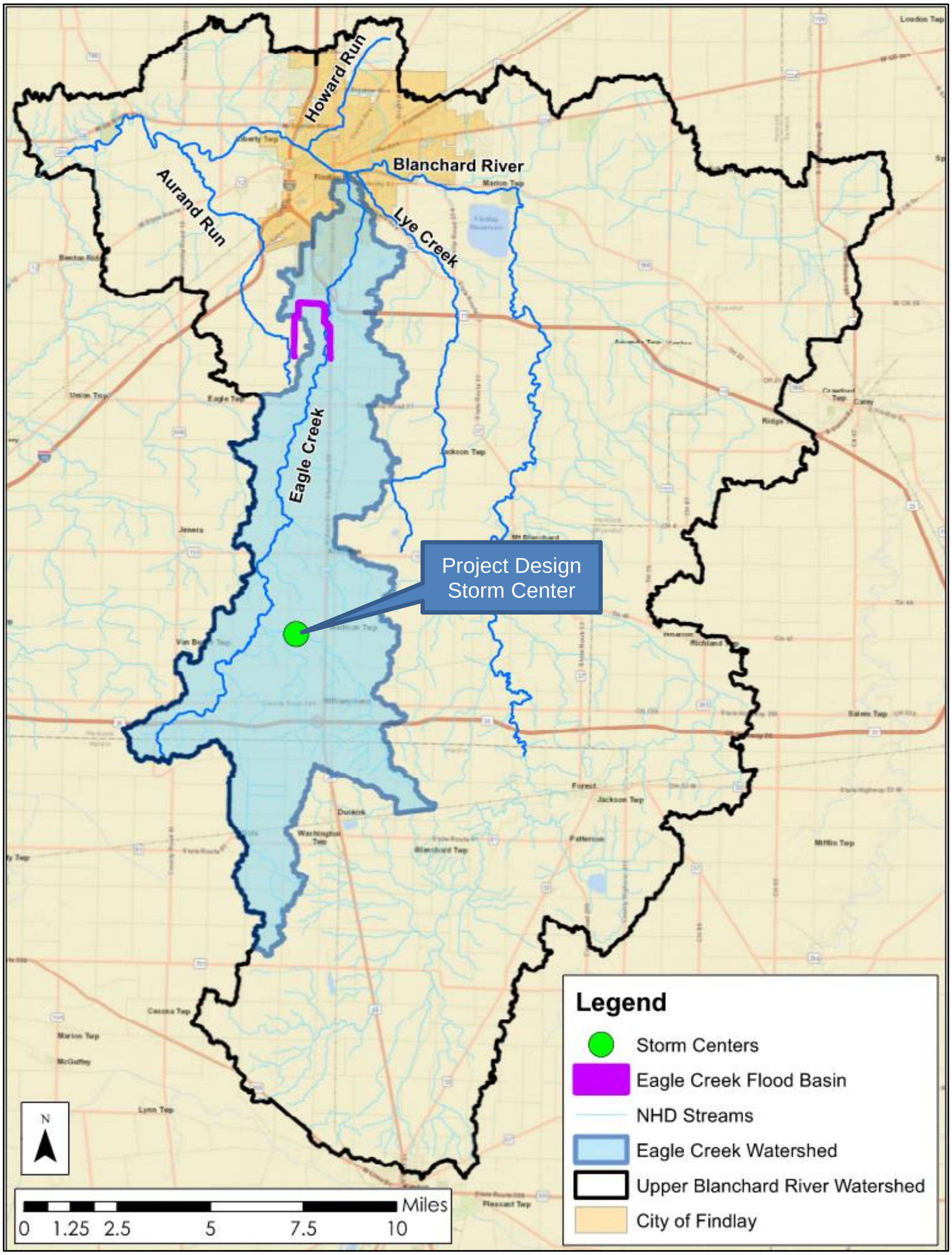

Figure 4. Storm Center Locations Used in Hydrologic Modeling

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 leveraged based on the modified modeled 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 extensively 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 details of this effort to leverage the model 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 (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 be transposed to the Blanchard Watershed. The outcome of the study recommended a typical storm orientation, shape, and spatial reduction factors, i.e., 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 A of Appendix D.

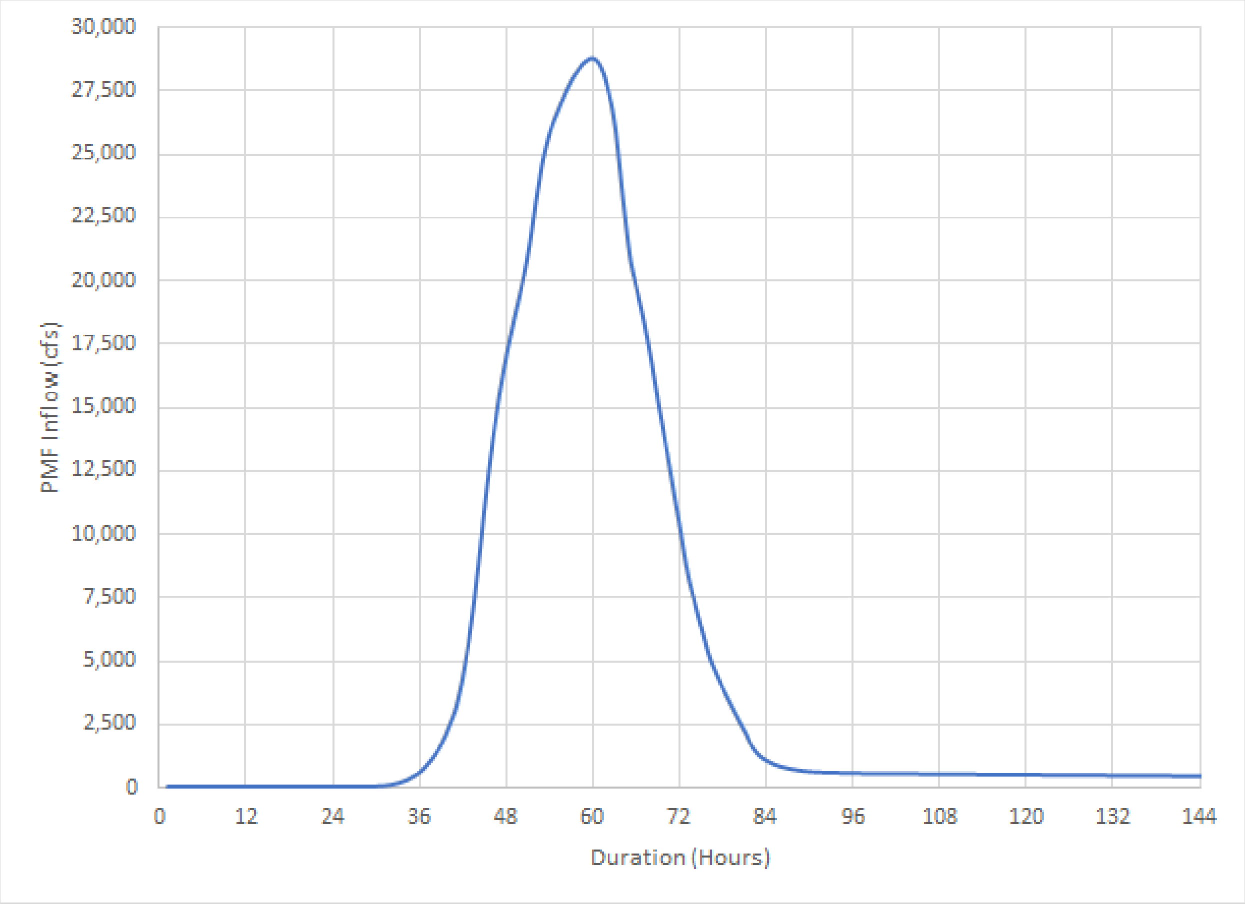

Figure 5. PMF Inflow Hydrograph at the Eagle Creek Flood Basin Project Site

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 Preliminary Design, with a few modifications. The model is a one-, and two-dimensional unsteady-state model. The HEC-RAS model was upgraded to HEC-RAS version 6.1 and refined for design purposes.

3.2.1 Existing Conditions Peak Discharge

Inflow hydrographs from the HEC-HMS model were used in the hydraulic modeling. 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. Existing conditions peak discharge results are shown in Table 6. The inflow 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	1,392
50% Annual Chance Exceedance (2-Year), 24-Hour	1,801
20% Annual Chance Exceedance (5-Year), 24-Hour	2,458
10% Annual Chance Exceedance (10-Year), 24-Hour	2,996
4% Annual Chance Exceedance (25-Year), 24-Hour	3,785
2% Annual Chance Exceedance (50-Year), 24-Hour	4,499
1% Annual Chance Exceedance (100-Year), 24-Hour	5,138
0.5% Annual Chance Exceedance (200-year), 24-Hour	5,983
0.2% Annual Chance Exceedance (500-Year), 24-Hour	7,046

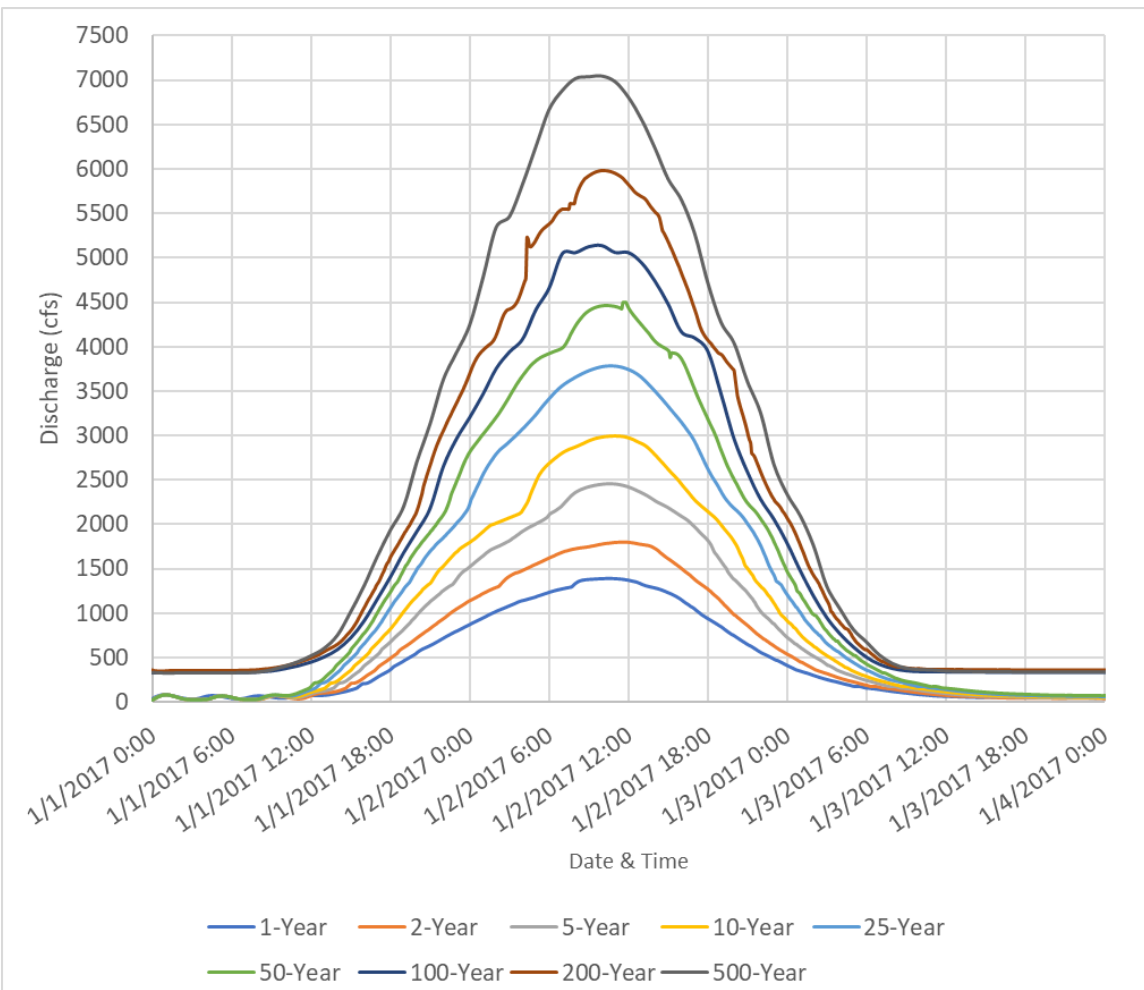

Figure 6. Existing Conditions Inflow Hydrographs at the Project Site (Township Road 49)

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 proposed 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 preliminary design 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 in 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 a relatively limited amount of 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 high hazard 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 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 94 borings during the Preliminary Design Phase to obtain geotechnical data for the proposed flood basin. Seventy-one borings were located along the initially proposed embankment alignment and at five selected embankment cross sections. The borings along the alignment were spaced at approximately 200-foot intervals in areas with a proposed embankment height greater than 10 feet, and 400-foot intervals in areas with a proposed embankment height less than 10 feet. Borings at the selected cross sections were initially located along the centerline and at the upstream and downstream toes of the initial embankment alignment. The cross sections were selected near initially proposed structure locations and areas of taller embankment fills. The alignment and structure locations shifted during preliminary 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 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. Thirteen borings were located within the interior of the basin to classify and quantify potential borrow materials.

Water pressure testing was performed in 15 borings to provide data for estimating permeability and flow regimes through the bedrock. Twenty-five open standpipe piezometers were installed and 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.

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 in the Geotechnical Design Report (Appendix E)

4.3 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).

4.3.1 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 project drawings
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	Fine aggregate (ODOT Fine Aggregate for Mortar or Grout – Natural Sand (ODOT CMS 2019 703.03)) and coarse aggregate (ODOT No. 7 Coarse Aggregate (ODOT CMS 2019 703.01)) used for chimney, blanket, and toe drain or other filter elements of the dam embankment.

4.3.2 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	2.71	17.5	0.52	111.4	130.9	132.6
Upper Fine-Grained	2.69	22.3	0.70	98.5	120.5	124.4
Upper Coarse-Grained	2.70	21.9	0.66	101.8	124.1	126.3
Lower Fine-Grained	2.70	16.4	0.45	116.1	135.1	135.1
Lower Coarse-Grained	2.70	16.6	0.45	116.2	135.5	135.5
Filter	2.65	12.5	0.38	120.0	135.0	137.0

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

4.3.3 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

4.3.4 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.

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	0.030	68	1.37	0.34	0.129
Upper Fine-Grained	Use average of two laboratory SWCC tests – see Geotechnical Design Report (Appendix E)				
Upper Coarse-Grained	0.021	97	1.61	0.40	0.067
Lower Fine-Grained	0.030	68	1.37	0.31	0.129
Lower Coarse-Grained	0.021	97	1.61	0.31	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.

4.3.5 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.

4.3.6 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.

4.3.7 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

4.3.8 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	ϕ (deg)	c (psf)	Consolidation State
Embankment Fill	0	3,000	Unconsolidated-Undrained (UU)
	27	100	Consolidated-Undrained (CU)
Upper Fine-Grained	20	400	Consolidated-Undrained (CU)
Upper Coarse-Grained	20	400	Consolidated-Undrained (CU)
Lower Fine-Grained	20	1,400	Consolidated-Undrained (CU)
Lower Coarse-Grained	37	0	N/A
Filter	33	0	N/A

4.3.9 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

	Drained Strength		Isotropically Consolidated, Undrained Strength	
Material Name	ϕ' (deg)	c' (psf)	ϕ (deg)	c (psf)
Embankment Fill	33	0	27	100
Upper Fine-Grained	34	0	20	400
Upper Coarse-Grained	34	0	20	400
Lower Fine-Grained	34	0	20	1,400
Lower Coarse-Grained	37	0	37	0
Filter	33	0	33	0

4.3.10 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. If required during Final Design, seismic shear strengths may be further refined for residual strength following liquefaction triggering analyses.

Table 14. Seismic Shear Strength Parameters

Material Name	ϕ_{EQ} (deg)	c_{EQ} (psf)
Embankment Fill	0	2,400
Upper Fine-Grained	16	320
Upper Coarse-Grained	16	320
Lower Fine-Grained	16	1,120
Lower Coarse-Grained	31	0
Filter	27	0

4.3.11 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. Based on soil index properties, pockets of Upper Coarse-Grained, Lower Fine-Grained, and Lower Coarse-Grained materials were judged to be sand-like and susceptible to liquefaction. A triggering analysis should be completed in the Final Design to evaluate if the susceptible materials would be predicted to liquefy during a design earthquake event.

4.3.12 Compressibility

The compressibility parameters for the foundation materials are summarized in Table 15.

Table 15. Compressibility Parameters

Material Name	Initial Void Ratio, e_0	Compression Index, C_c	Recompression Index, C_r	Representative Preconsolidation Pressure (tsf)
Upper Fine-Grained	0.70	0.34	0.086	1.7
Upper Coarse-Grained	0.66	0.13	0.033	2.7
Lower Fine-Grained	0.45	0.16	0.041	6.3
Lower Coarse-Grained	N/A	N/A	N/A	N/A

4.4 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 16 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.

Table 16. 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

4.5 SOIL MATERIAL BORROW STUDY

As part of the geotechnical exploration for the preliminary design of the project, Stantec conducted a series of borings on the interior of the proposed embankment alignment. A total of 13 borings were advanced 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.

5.0 DAM EMBANKMENT

The basin will be formed by an earthen embankment dam on three sides and is anticipated to range 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 embankment is approximately 3.75 miles long, with a crest elevation at 813 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 embankment design is in accordance with USACE guidance and ODNR, Division of Water, Dam Safety Program regulations. 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 = 813 feet minimum (freeboard requirements for design flood pool)
- Embankment side slopes = 3H:1V (maximum slope for access, monitoring, and maintenance)
- Crest width = 12 feet minimum (vehicle access for maintenance and monitoring)
- Crest surface slope = 2 percent minimum (to provide surface drainage)
- One foot (minimum) of excavation (stripping) to remove vegetation and topsoil under the dam footprint
- Inspection trench = minimum 6 feet deep by 10 feet wide, 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 7.

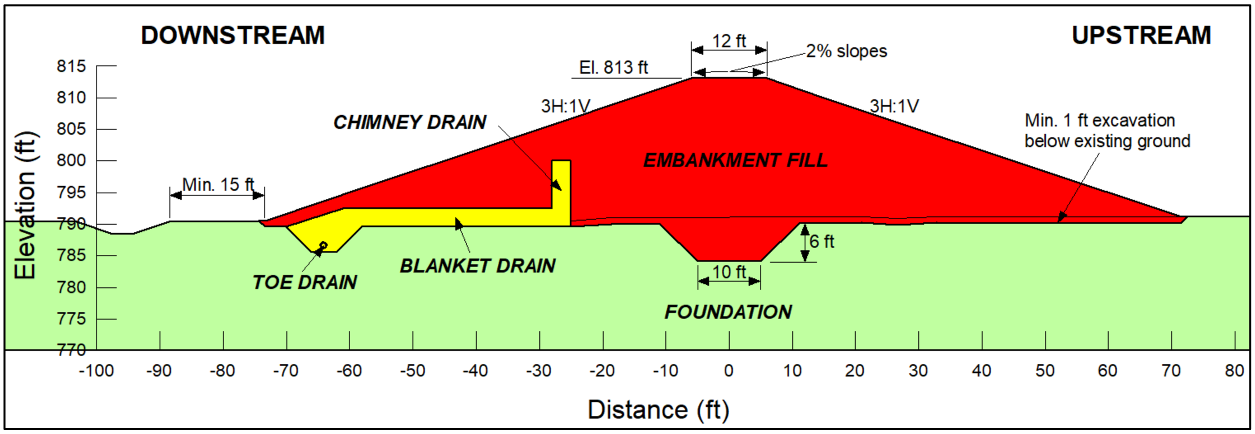

Figure 7. Typical Embankment Cross Section

The Dam Embankment Design technical memorandum, found in Appendix G, describes the dam features and design considerations in detail.

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.

5.3 ALIGNMENT

5.3.1 Preliminary Design Assumptions

In addition to the design flood event, project stakeholders have provided the following qualitative criteria for the project team to consider through the planning and design processes, particularly related to the dam alignment:

- Reduce the footprint of the proposed storage facility,
- Reduce the number of parcels potentially impacted by construction,
- Reduce the number of structures potentially 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. The exterior drainage ditch is 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). Based on preliminary conversations with the Hancock County Engineer and the Eagle Township Trustees, designed ditches within the existing road right-of-way was assumed to be acceptable.

5.3.2 Area-Capacity-Elevation Data

The area-capacity-elevation data was calculated based on the alignment set during Preliminary Design as shown in Figure 8. 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 6,900 acre-feet (2.25 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 in Table 17 and key dam design elevations highlighted in Table 18. A plot of the reservoir stage-storage curve is shown in Figure 9.

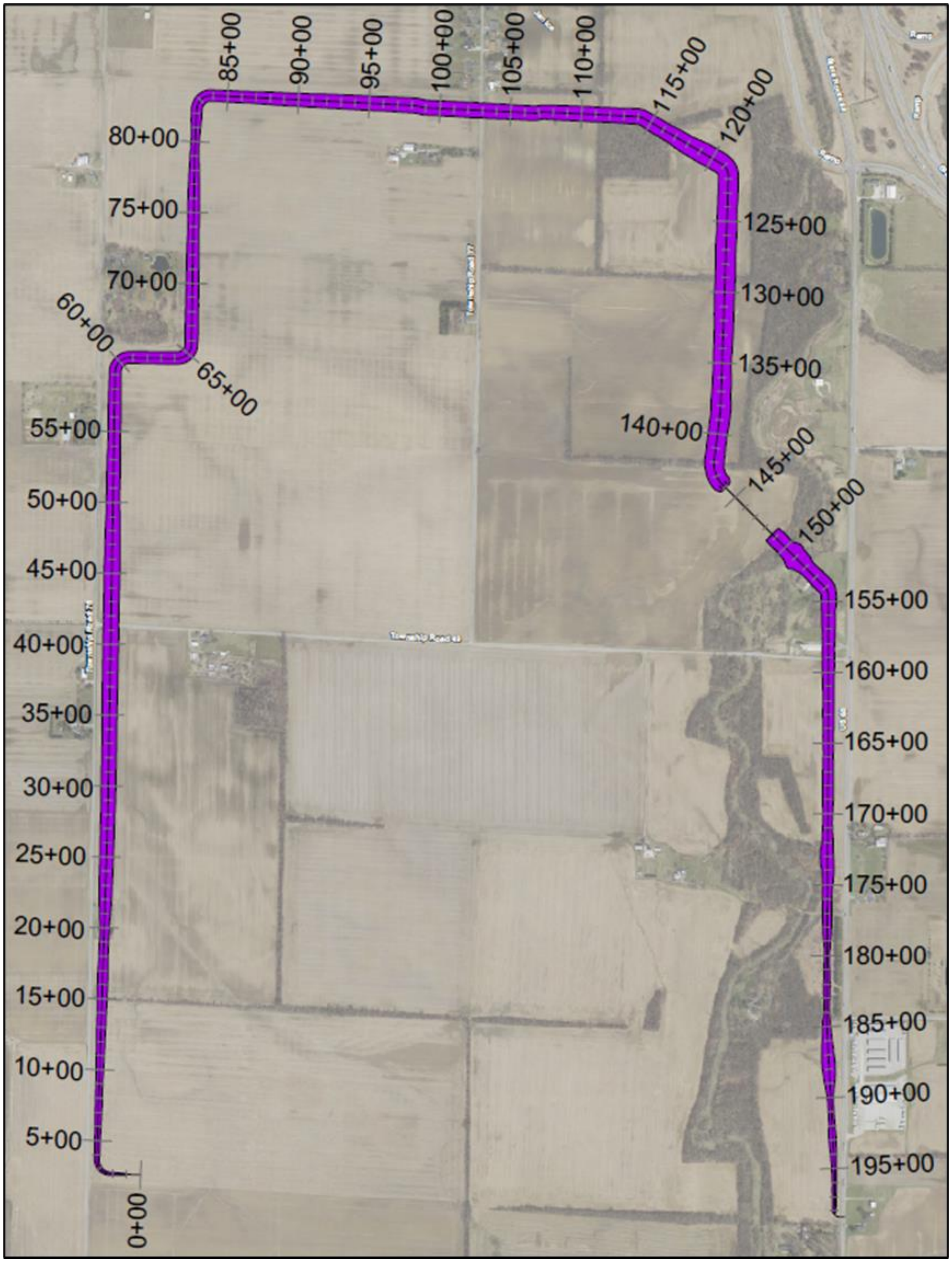

Figure 8. General Alignment provided by MWCD for Preliminary Design

Table 17. Reservoir Stage-Storage Curve

Stage (ft)	Total Storage (ac-ft)	Stage (ft)	Total Storage (ac-ft)
784.0	0	798.0	853
785.0	0	799.0	1,245
786.0	1	800.0	1,714
787.0	3	801.0	2,270
788.0	6	802.0	2,904
789.0	10	803.0	3,596
790.0	16	804.0	4,346
791.0	25	805.0	5,157
792.0	40	806.0	6,011
793.0	60	807.0	6,900
794.0	103	808.0	7,821
795.0	180	809.0	8,778
795.5	230	810.0	9,776
796.0	311	811.0	10,820
796.5	408	812.0	11,909
797.0	535	813.0	13,040

Table 18. 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	813.0	Peak WSE during PMF plus 3.0 feet of freeboard.

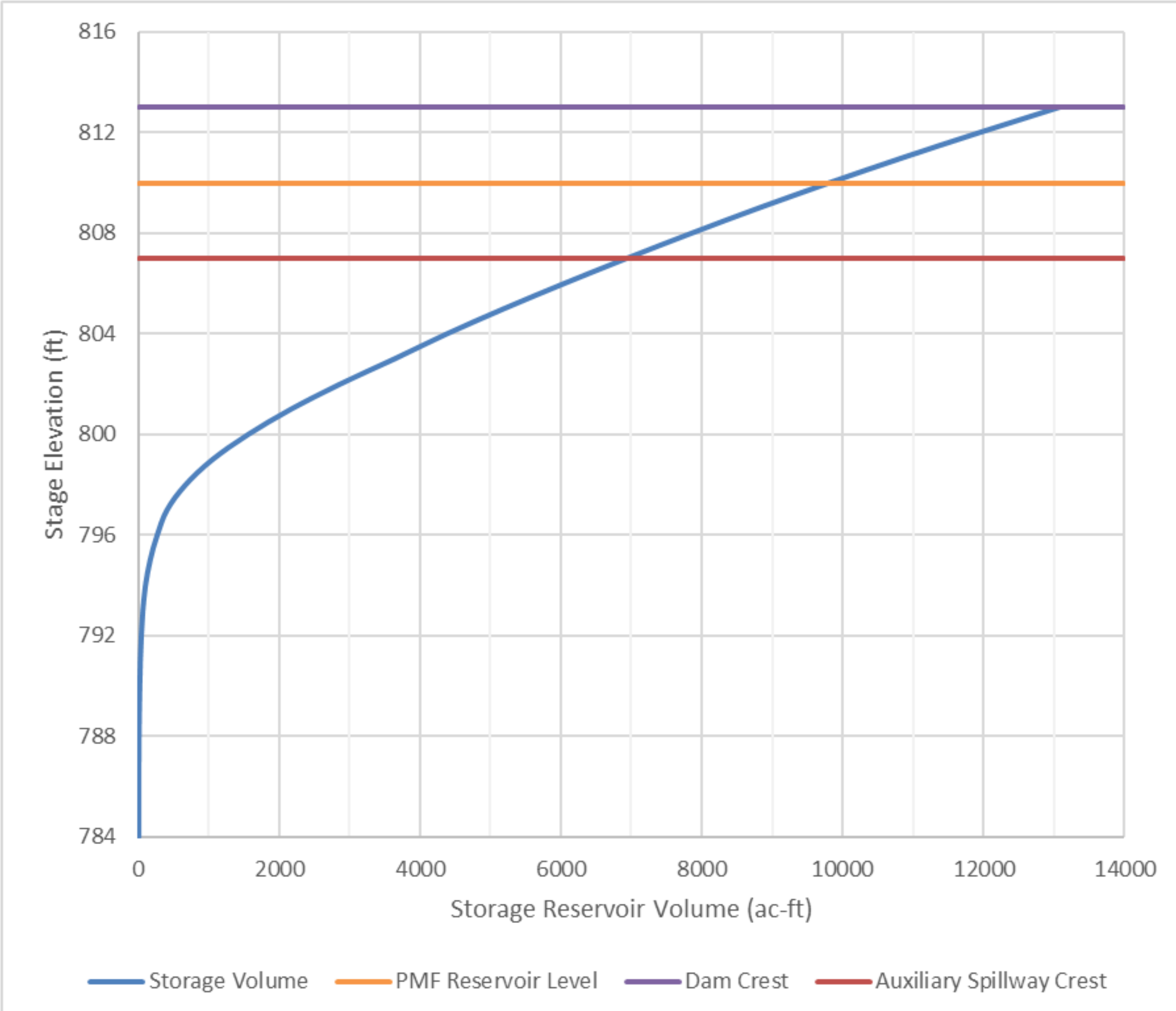

Figure 9. ECFB Reservoir Stage-Storage Curve

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.

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 channels are designed trapezoidal and 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%, 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 system consisting of a 24-inch conduit in parallel with an overflow ditch. The conduit is designed to convey the 10% ACE (10-year) flood event without flowing full and without surcharging the road surface during the 4% ACE (25-year) event. The 24-inch conduit is approximately 3,740 feet long at a constant slope of 0.34% and will be constructed of pre-cast concrete. The overflow ditch is trapezoidal and lined with concrete 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%.

There are several existing box culverts under US-68 that typically 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

Several 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 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. As a result of the 9’ by 4’ culvert, Township Road 76 will require a road raise of approximately 1 foot. The road will be modified over a distance of 450 feet to tie into existing grade.

Several new culverts are required at dam access points convey the required flows. There are three access points along the dam embankment: southwest access, north access, and Township Road 49 Principal Spillway access. Three (3), 30-inch diameter culverts at a 1.0% slope are proposed for the southwest access crossing. Two (2), 36-inch diameter culverts at a 0.3% slope are proposed for the north access crossing. Two 9 by 4-foot box culverts at a 0.55% slope are proposed for the Township Road 49 Principal Spillway access crossing. The culverts are designed to pass the 4% ACE (25-year) event and were checked with the 1% ACE (100-year) event to confirm no 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

Freeboard criteria for the embankment design was determined based on United States Bureau of Reclamation (USBR) Design Standards No. 13, Embankment Dams (USBR 2021). The criteria 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." Additionally, according the USBR Design Standards, freeboard should be evaluated for the normal reservoir water surface (NRSW) plus runup and setup that would be generated by a 100-mph wind velocity.

Because the Eagle Creek Flood Basin does not maintain a pool under normal conditions, the 100-year event maximum WSE is treated as the NRSW for the purposes of freeboard calculations and the maximum recorded wind speed was applied rather than a 100-mph wind velocity. 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. The wind speeds used in the freeboard analysis are summarized in Table 19 and discussed in more detail in the Embankment Design Technical Memorandum included as Appendix G.

Table 19. Wind Speed

Parameters	Overland Velocity	R_L (based on figure B-2)	Over-water Velocity
Daily 2-minute wind speed with 10% chance of exceedance (mph)	30.0	1.05	31.5
Highest recorded 2-minute wind speed in 20-year record (mph)	53.9	0.9	48.5

5.5.2 Freeboard Analysis

Wave runup and wind setup were calculated using equations 8 and 10 in the USBR Design Standards, resulting in a required dam crest of 811.22 feet for the NRWS condition and 812.75 feet for the MRWS condition. USBR design guidance recommends a minimum of 3 feet of freeboard for embankment dams. Stantec recommends a dam crest elevation of 813.0 feet to comply with this requirement. This crest elevation will allow for hydrologic uncertainty due to climate change and the potential for antecedent events, as well as the risk of spillway malfunction including loss of capacity due to debris buildup. A summary of freeboard calculations is provided in Table 20.

Table 20. Freeboard Calculations Summary

Parameter	USBR Eqn	Normal	PMF
Reservoir Water Surface (ft)		807.00	810.00
Fetch (mi)		1.85	2.05
Hourly Wind Velocity (mph)		48.51	31.5
Significant Wave Height (ft)	Eqn 2	2.20	1.40
Wave Period (sec)	Eqn 4	2.30	2.01
Surf Similarity Factor	Eqn 7	1.17	1.28
Wave Runup (ft)	Eqn 8	3.71	2.58
Wind Setup (ft)	Eqn 10	0.51	0.17
Total Wind-Generated Wave (ft)		4.22	2.75
Required Dam Crest Elev (ft)		811.22	812.75
Dam Crest Elev (ft)		813.00	813.00
Freeboard to Wave Runup (ft)		1.78	0.25
Overtopping		NO	NO

A detailed discussion on freeboard and the calculations package is included as Exhibit A to the Dam Embankment Design Technical Memorandum (Appendix G).

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 21 provides the load cases considered and the required minimum factors of safety.

Table 21. 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_{rock}) (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

Four cross sections were selected to represent the conditions in various reaches along the dam alignment in the design seepage and stability analyses. Figure 10 shows the selected analysis cross section locations and the reaches they represent. 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). Additional information on each of the cross sections is described in the subsections below.

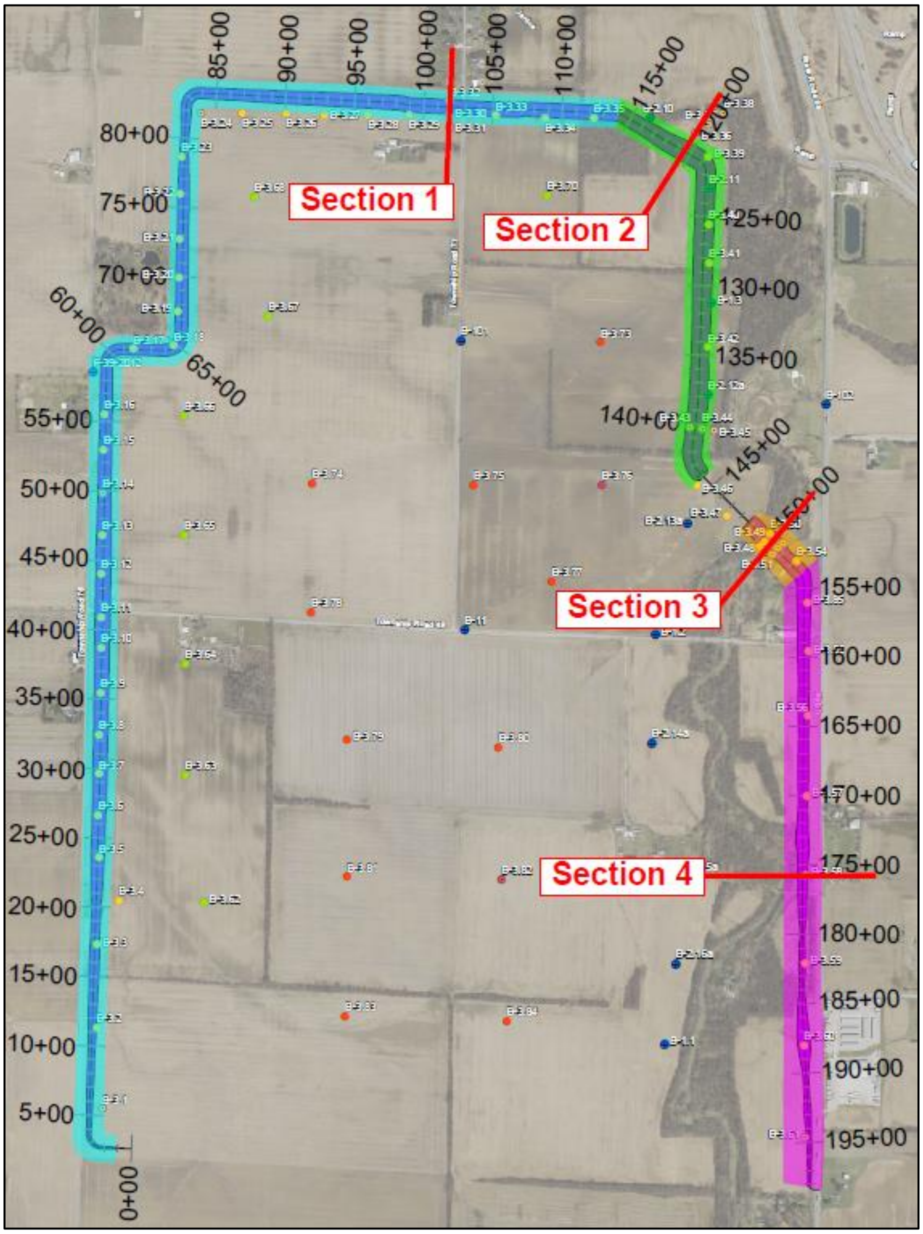

Figure 10. Analysis Cross Section Locations

5.6.3 Stability Results

Table 22 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 22. The design load cases met acceptance criteria. Graphical outputs from the slope stability analyses and GeoStudio reports are included in Exhibits H-1 and Exhibit K to the Geotechnical Design Report (Appendix E). The pore pressures used for the stability analyses in Section 4 are based on the seepage analysis results which model a sheet pile seepage cutoff. No strength was given to the sheet pile wall for the stability analyses.

Table 22. Slope Stability Analysis Results

Case No.	Load Case[(1)]	Analyzed Conditions[(1)]	Slope	Calculated Factor of Safety				Required Minimum Factor of Safety
				Sect. 1	Sect. 2	Sect. 3	Sect. 4	
1	End of Construction	No pool, undrained analysis	Downstream	2.0	1.5	2.0	2.3	1.3
			Upstream	1.9	1.5	2.1	2.2	
2	Long-term	No pool, drained analysis	Downstream	2.1	2.0	2.0	2.3	1.5
3	Flood - Maximum Differential	100-year pool, drained analysis	Downstream	2.0	2.0	2.0	2.3	1.4
4	Flood - Maximum Headwater Elevation	PMF pool, undrained analysis	Downstream	2.2	2.1	2.2	2.3	1.4
5	Rapid Drawdown	Maximum Differential to No Pool, undrained analysis	Upstream	1.8	1.6	1.8	2.1	1.3
6	Pseudo-Static	No pool, undrained analysis	Downstream	1.7	1.7	1.8	1.9	1.0
			Upstream	1.8	1.6	1.8	2.0	

(1) Refer to Table 21 for additional details on the analyzed conditions for each load case.

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

5.7 SEEPAGE

A seepage model is needed 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 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

For the design and assessment of dams, the USACE criteria for soil heave (USACE, 1986) are summarized in Table 23. These values are based on the seepage force definition ($FS_{exit\text{-}SF}$) and are applicable to all soil types.

Table 23. USACE Criteria for Exit Seepage at Dams (EM 1110-2-1901, USACE, 1986)

For Design of:	Requirement
Upstream seepage blanket, Downstream seepage berm	$FS_{exit\text{-}SF} \geq 3.0$ at downstream toe of dam
Downstream seepage berm	$FS_{exit\text{-}SF} \geq 1.5$ at toe of downstream berm

5.7.2 Analysis Results

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 considerations for exit seepage and seepage cutoffs were evaluated.

Factors of safety for exit seepage at the downstream toe of the dam were calculated using the seepage force method for each of the four analysis sections. The results of the exit seepage factor of safety calculations are summarized in Table 24.

Table 24. Seepage Exit Analysis Results

Section	Location	Calculated Factor of Safety	Required Minimum Factor of Safety
1	Bottom of Exterior Ditch	1.5	1.5
	Toe of Dam	3.1	3.0
2	Toe of Exterior Bench	2.6	1.5
	Toe of Dam	2.3	3.0
3	Toe of Exterior Bench	8.2	1.5
	Toe of Dam	261.0	3.0
4 (with cutoff)	Bottom of Exterior Ditch	53.3	1.5
	Toe of Dam	N/A(1)	3.0

(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.

One location returned a factor of safety slightly below acceptance criteria. To increase the factor of safety at this location, a weighted berm could be added at the toe of the dam. This is a logical solution for this project because fill is readily available. An increase of 1.5 feet is necessary. The increased elevation in the exterior bench can be incorporated during Final Design.

Seepage cutoffs are recommended under portions of the embankment and are assumed to consist of steel sheet pile walls. The Final Design will consider cost comparisons for steel sheet piles, vinyl sheet piles, slurry walls, or other solutions. The locations and extents of the proposed seepage cutoffs are shown in Figure 11.

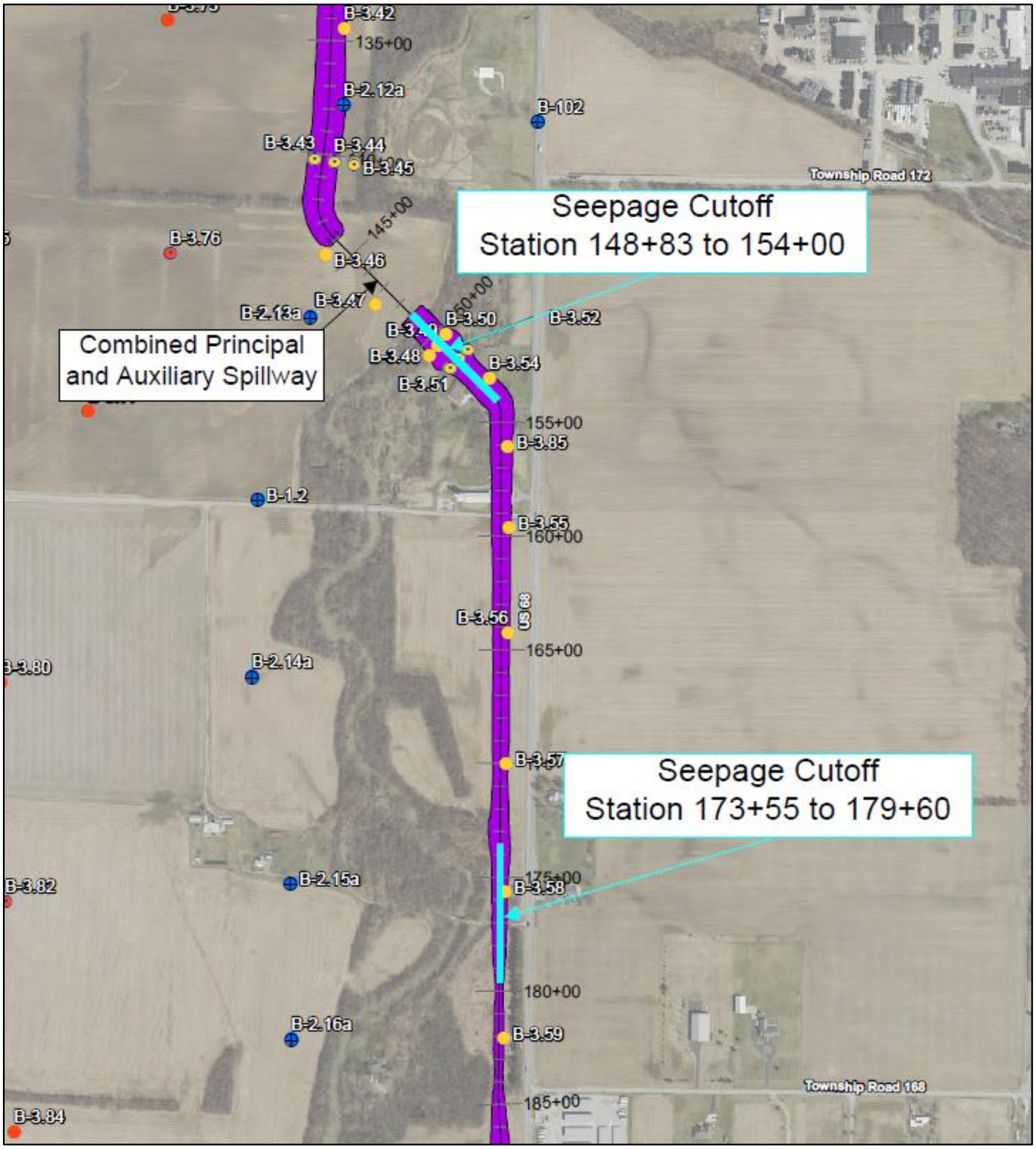

Figure 11. Locations of Seepage Cutoffs

Seepage analysis results are included within Exhibit G of the Geotechnical Design Report (Appendix E).

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 and comparisons to published correlations. Vertical stresses were computed based on Boussinesq equations using ranges in Poisson's ratio consistent with the soil materials encountered. Primary consolidation is 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 Settle3 software by Rocscience.

5.8.1 Acceptance Criteria

Allowable total and differential settlements of structures are consistent with the operating conditions of the facility. Total settlement should not exceed 1 inch with up to ¾ inch of differential settlement at structural spillway facilities.

Embankments should be overbuilt to accommodate the predicted settlement without the crest settling below the design grade.

5.8.2 Analysis Results

Settlement of the embankment was estimated considering a typical, representative embankment height of 20 feet. Based on the developed compressibility parameters, the Upper Fine-Grained, Lower Fine-Grained, and Upper Coarse-Grained materials are compressible. The Lower Coarse-Grained material is assumed to consolidate as the embankment is built, and thus will not contribute to the post-construction settlement. To estimate settlement, the boring logs along the dam alignment were reviewed to evaluate the greatest encountered thickness of compressible soil. A total of 20 feet of compressible material was observed in boring B-3.23, with 12.5 feet of Upper Fine-Grained material underlain by 7.5 feet of Lower Fine-Grained material.

Considering an embankment height of 20 feet, unit weight and observed compressibility parameters, and the subsurface profile at boring B-3.23, a maximum embankment settlement of 8 inches was estimated. This estimate is based on a typical, representative embankment height and a conservative estimate of compressible foundation soil thickness. The embankment should be overbuilt by up to 8 inches to elevation 813.7 feet to allow for this settlement and maintain the necessary freeboard. The results of the settlement calculations are provided in Exhibit I of Appendix E.

6.0 PRINCIPAL SPILLWAY

6.1 GENERAL ARRANGEMENT

The Principal Spillway consists of the following components:

- Realigned/relocated reach of Eagle Creek upstream of the embankment
- a passive structure to control inflow (no actively operated gates or valves),
- 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 12 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 contributing watershed upstream of the Principal Spillway is approximately 55 square miles.

The Principal Spillway structure is located approximately 250 feet northwest of where the ECFB embankment crosses the existing Eagle Creek channel (about 700 feet north of TR 49) and provides a hydraulic connection to the realigned/relocated channels upstream and downstream of the embankment. The realigned/relocated Principal Spillway channel slope and structure invert elevation are set to be similar to the existing Eagle Creek channel.

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, considered an alternative during the preliminary design process, incorporates a fish passage system through the Principal Spillway. Development of the fish passage design alternative 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.

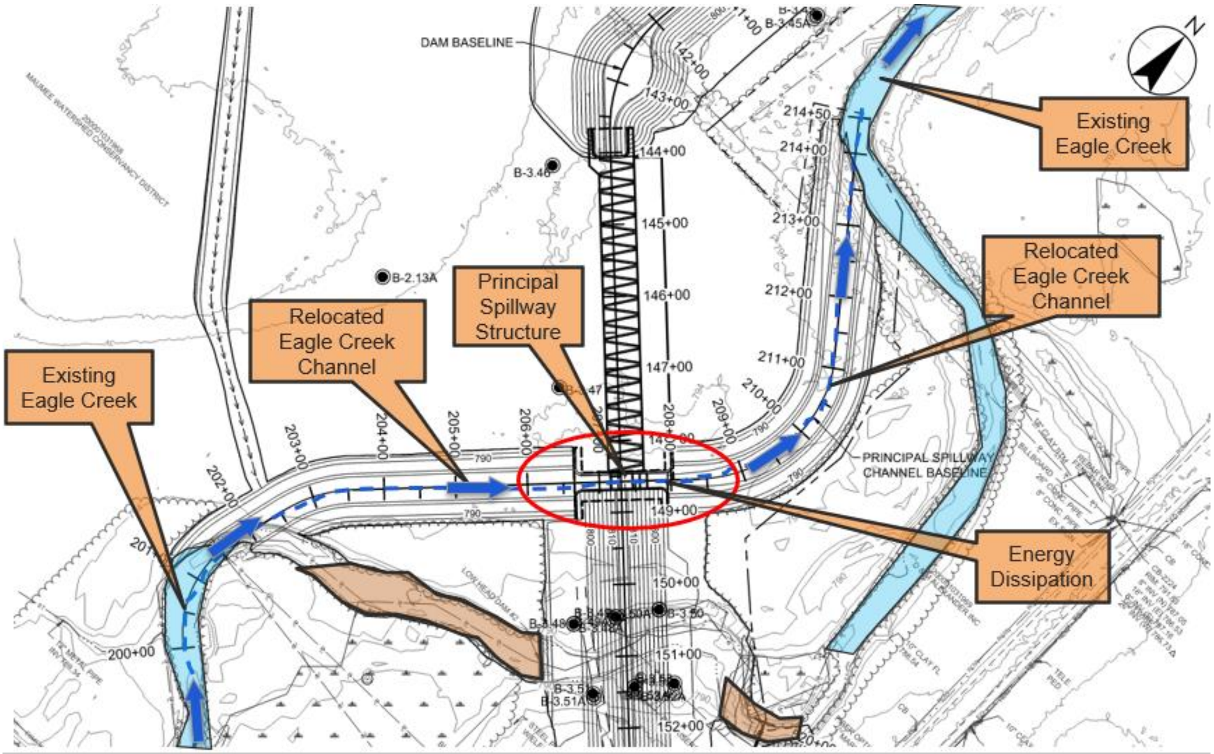

Figure 12. Principal Spillway General Arrangement

6.3 LOCATION AND ALIGNMENT

The location and alignment of the Principal Spillway structure and its approach channels, presented in Figure 12, are located approximately 250 feet west of the existing Eagle Creek channel. The alignment was selected to facilitate construction of the structure outside of the existing channel. During construction, normal flows can be maintained through the existing Eagle Creek channel until construction of the Principal Spillway is substantially complete. This off-line arrangement (away from active flow) is located on higher ground and will require smaller cofferdams to keep the site dry during construction. These features of the design simplify construction phasing and should reduce overall cost and schedule duration in comparison to an alignment within the existing channel.

Environmental concerns associated with the net reduction in Eagle Creek channel length of approximately 458 linear feet were considered, however, analysis shows the overall aquatic use designation of Eagle Creek will likely be maintained upstream and downstream of the Principal Spillway throughout the construction process.

6.4 RECOMMENDED DESIGN

The upstream control wall with baffled concrete channel includes a reinforced concrete headwall and integrated trash rack. The cast-in-place control wall is a 26.5-foot tall, 20-foot-wide structure perpendicular to the channel with formed orifices to restrict flow, and an 80-foot-long concrete discharge channel with baffles to provide energy dissipation and fish passage.

The control wall is located between two large cast-in-place concrete abutments or retaining walls located on both the west and east side of the Principal Spillway channel. A cast-in-place concrete mat foundation provides support for the retaining walls and control wall. A rectangular stainless steel trash rack is located on the upstream side of the control wall and is supported by the control wall, mat foundation, and opposite retaining walls. The retaining walls on the west and east side of the channel direct flow into the Principal Spillway and through the trash rack.

To allow access for maintenance and to remove debris from the trash rack, a cast-in-place concrete slab bridge spans between the two retaining walls and is located above the control 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. Preliminary design of the slab bridge, using standard Ohio Department of Transportation (ODOT) design guides, estimates the slab bridge to be a maximum of 18 inches thick with a #9 main reinforcing bar spaced every 8 inches.

This design is largely dependent upon the Principal Spillway facilitating 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 would need to provide 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*" (Appendix N). Designing for fish passage may increase overall project cost; however, not facilitating the passage of fish may be unacceptable from an environmental permitting standpoint.

The recommended Principal Spillway configuration for the ECFB consists of a control wall and baffled chute integrated into the labyrinth weir Auxiliary Spillway alternative. A summary of the reasons this spillway was recommended is presented below, with more detail provided in the Spillway Type Recommendation Memo (Exhibit C to the Principal Spillway Technical Memorandum (Appendix I).

- Fish passage will likely be a significant permitting driver as part of the anticipated Clean Water Act, Section 404 Individual Permit (IP). As part of the IP process, other regulatory agencies are engaged and are able to provide project reviews. The Ohio Department of Natural Resources (ODNR) Division of Water will likely have fish passage as a necessary consideration for the project.
    - The recommended configuration would provide an opportunity for fish passage. Incorporating fish passage into the design should reduce the risk of schedule delays and/or not being granted the necessary permits.
- 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.

- There are potential cost efficiencies and value engineering that may be realized during final design, in addition to the potential mobilization savings.
    - The Integrated Spillway approach allowing for fish passage may reduce the potential mitigation requirements and associated mitigation costs related to direct and indirect impacts for the project.
- Access for operations and maintenance are likely to be better for the Integrated Spillway option with the ability to be in close proximity to the proposed trash rack.
    - The Principal Spillway wall structure may allow for additional operational considerations and flexibility compared to a typical culvert design.

The recommended Principal Spillway configuration consists of a 2 ft thick reinforced concrete control wall with three 6.0 ft wide rectangular orifice openings. The middle opening will have an invert elevation of 784.15 ft and a height of 3.5 ft. The side orifice invert elevations are set 0.5 ft higher and will be 0.5 ft shorter. The orifice openings will be kept clear by a sloping trash rack between the abutments measuring 20 ft wide, 13 ft tall and 40 ft long. Downstream of the wall will be a 20 ft wide rectangular concrete chute sloped at 0.5% and measuring 50 ft long. The concrete chute will contain five, 1 ft tall baffle walls spaced 10 ft on center. Each baffle wall will include a 0.5 ft deep notch, measuring 1.5 ft wide. A new Eagle Creek channel alignment, directing flow to the Principal Spillway, is proposed. This new channel was 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.

Design drawings for the Principal Spillway are included in the Preliminary Design Drawings, Appendix L.

6.5 HYDRAULIC DESIGN

6.5.1 Flood Storage Design Criteria

The primary design objective of the PS is to detain water during the 100-year storm event to a maximum WSE between 806.8 ft and 807.0 ft. This elevation range utilizes the available storage capacity within the basin to achieve the desired flood attenuation without activating the auxiliary spillway. The maximum downstream design flowrate was determined to be 1,250 cfs at this pool elevation. The development of the design flowrate (1,250 cfs) is driven by the Auxiliary Spillway crest set to 807.0 ft and the available reservoir capacity, assuming no additional storage via excavation.

6.5.2 Control Wall / Baffled Chute Design

The control wall and baffled chute configuration replaces a segment of the earthen dam embankment. The structure is comprised of a control wall containing rectangular orifices followed by a baffled, rectangular, concrete chute designed to facilitate fish passage and dissipate energy before transitioning to the downstream outlet channel. The first 80 feet of the outlet channel, downstream of the concrete chute, will be lined with riprap to reduce the risk of scour.

6.5.2.1 Baffled Chute / Energy Dissipation

The baffled chute was designed using a steady-state HEC-RAS model, version 6.1 (USACE, 2021). The model was used to evaluate multiple discharges and the resulting velocities and flow depths with respect to fish passage. The results of the HEC-RAS model were used to create a tailwater rating curve for sizing the inlet control wall orifices. 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.

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 energy is designed to be dissipated 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 Principal Spillway is provided within the baffled concrete section of the structure. Calculations to check that appropriate energy dissipation is provided by the baffled chute utilized methodology from Federal Highway Administration (FHWA) Hydraulic Engineering Circular 14 (HEC-14): Hydraulic Design of Energy Dissipators for Culverts and Channels, section 7.2.2 (FHWA 2006). This methodology is intended for calculating the increased roughness provided by internal baffles in box culverts, but was determined to still be appropriate for an open-top chute.

A baffle spacing of 10 feet on center was determined to facilitate fish passage. According to HEC-14, a ratio of baffle spacing to baffle height of 10 should be used, resulting in a baffle height of 1 ft. According to the HEC-14 methodology, a minimum of five baffles should be included. At a centerline spacing of 10 feet, this results in the 50 ft baffled chute length at a slope length of 0.5%.

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 6.8 ft/s for the probable maximum flood (PMF) discharge at the downstream end of the baffles. This is a conservative velocity estimate because the elevated downstream tailwater experienced during a PMF event will result in much lower velocity values, closer to 3.4 ft/s according to HEC-RAS modeling of the baffled chute. The minimum of five baffles was determined to be sufficient based on HEC-14 methodology. The HEC-14 energy dissipator calculations are included as Exhibit A within Appendix I.

The width of the chute was set at 20 feet as the minimum requirement to meet velocity criteria for fish passage at the higher flow threshold. This width was determined through an iterative design approach using the baffled chute HEC-RAS model and HEC-14 hand calculations.

For lower flows, the baffles include a 1.5-foot wide by 0.5-foot-deep notch to concentrate and constrict flow to meet minimum depth requirements for fish passage. The controlling condition is a minimum depth of flow of 0.37 ft during the 90% exceedance June discharge of 1.8 cfs.

The upstream and downstream invert elevations of the baffled chute were determined to be 783.65 ft and 783.40 ft respectively. These invert elevations are 0.5 ft below the invert elevations upstream and downstream channel so that the invert of the notch in the downstream-most baffle and middle orifice in the control wall match the invert elevation of the upstream and downstream channel.

Table 25 summarizes the results of the baffled chute HEC-RAS model which are used as a tailwater rating curve for sizing of the control wall orifices and determining velocity and depth results relevant to fish passage. Figure 13 presents WSE profile results from the baffled chute HEC-RAS model.

Given the unique design and non-uniform flow from the orifices, the hydraulic performance of the baffle chute should be confirmed using a 3-dimensional computational fluid dynamics model in Final Design.

Table 25. Principal Spillway Baffled Chute HEC-RAS Model Summary

Profile Description	Discharge (cfs)	Baffled Chute Upstream WSE (ft) [2]	Baffled Chute Downstream WSE (ft)	Maximum Velocity Across Baffles (ft/s)	Maximum Velocity Between Baffles (ft/s)	Minimum Flow Depth (ft)
June, 90% Exceedance Discharge	1.8	784.90	784.90	0.27	0.07	0.80
June, 85% Exceedance Discharge	2.7	785.00	785.00	0.31	0.10	0.90
June, 75% Exceedance Discharge	5.0	785.10	785.10	0.46	0.17	1.00
April 25% Exceedance Discharge	76.6	786.66	786.60	1.85	1.27	2.53
April 15% Exceedance Discharge	147.5	787.68	787.60	2.40	1.83	3.54
April 10% Exceedance Discharge	225.6	788.51	788.40	2.90	2.32	4.35
	350.0	789.73	789.60	3.43	2.88	5.57
	450.0	790.45	790.30	3.87	3.31	6.28
	550.0	791.07	790.90	4.28	3.71	6.89
Bankfull Discharge	650.0	791.60	791.40	4.68	4.09	7.41
	750.0	792.12	791.90	5.03	4.43	7.92
	850.0	792.55	792.30	5.40	4.78	8.33
	1,050.0	793.30	793.00	6.10	5.44	9.07
Approximate 100-year Discharge[1]	1,250.0	793.79	793.40	6.90	6.16	9.52
Approximate PMF Discharge[1]	1,275.0	802.65	802.60	3.53	3.36	18.52

[1] 100-year and Probable Maximum Flood (PMF) discharge do not need to exactly match final values for the baffled chute HEC-RAS model because it is used only as a tailwater rating curve for computing the orifice sizes.

[2] Baffled Chute Upstream WSE serves as the tailwater rating curve for sizing the control wall orifices

Principal Spillway

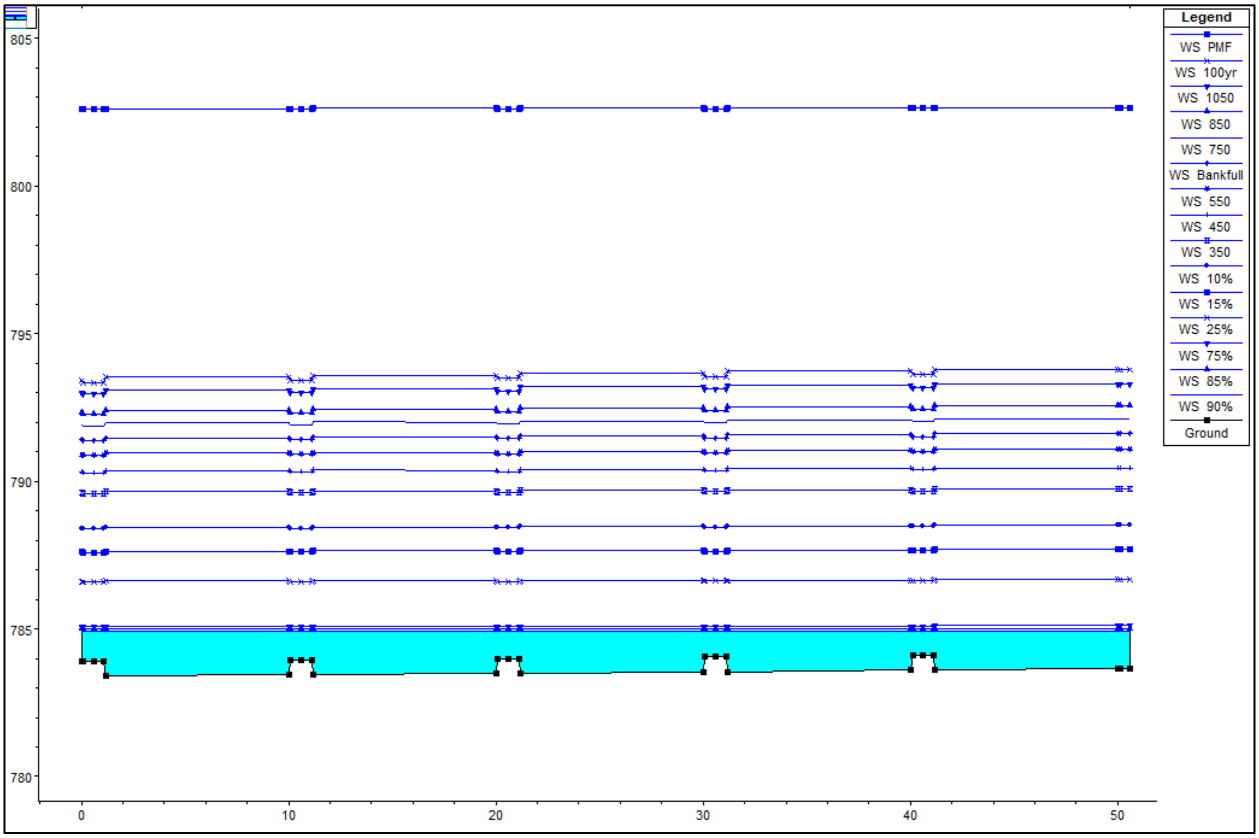

Figure 13. Baffled Chute HEC-RAS Model Water Surface Elevation Profile Results

6.5.2.2 Control Wall Orifices

The control wall orifices were designed using a combination of a steady state HY-8 model, version 7.70.10 (FHWA 2021), and an unsteady HEC-HMS model, version 4.8 (USACE 2021). The HY-8 model was used to calculate 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 HY-8 to evaluate the performance during the 100-year flood event to determine whether the resulting maximum WSE met the target of 807.0 ft. Sizing of the orifices was achieved through an iterative process between HY-8, HEC-HMS, and in conjunction with the baffled chute HEC-RAS model.

The middle orifice was set flush with the invert of the upstream channel at 784.15 ft. This invert elevation is 0.5 ft higher than the invert of the baffled chute located immediately downstream of the control wall. The bottom of the side orifices was set at elevation 784.65 ft, which is 0.5 ft above the invert of the middle orifice and 1.0 ft above the invert of the downstream baffled chute. This was done to mirror the 1 ft tall baffle height designed as part of the baffled chute design calculations.

The width of each orifice was set to 6 feet to meet velocity criteria for fish passage. This width was determined through an iterative design approach using the HY-8 model and the downstream baffled chute HEC-RAS model. The minimum flow depth of 0.37 ft required to facilitate fish passage through the middle orifice during the 90% exceedance discharge was achieved due to the first baffle located 10 feet downstream of the control wall.

The crown of each orifice was set to the same elevation, such that the side orifices are 0.5 ft shorter than he middle orifice. The orifice heights, 3.5 feet for the middle orifice and 3.0 feet for the side orifices, were determined through the iterative design approach described above and achieved the target upstream water surface elevation.

This results in a maximum 100-year WSE of 806.85 ft which is within the desired range and corresponds to a maximum discharge through the Principal Spillway of 1,215 cfs. Table 26 summarizes the control wall / baffled chute rating curve and presents a summary of the HY-8 model results. Figure 14 presents the Principal Spillway rating curve graphically. Performance of the spillway during simulated design flood events is presented in Section 8.0. Results of the HY-8 model is included with Exhibit A of Appendix I.

Table 26. Control Wall / Baffled Chute Principal Spillway Rating Curve and HY-8 Model Results Summary

Profile Description	Discharge (cfs)	Orifice Upstream WSE (ft)	Orifice Downstream WSE (ft)	Middle Orifice Velocity (ft/s)	Middle Orifice Flow Depth (ft)
No flow (for rating curve)	0	784.15	784.15	0.00	0.00
June, 90% Exceedance Discharge	1.8	784.90	783.65	0.25	0.75
June, 85% Exceedance Discharge	2.7	785.00	784.90	0.29	0.85
June, 75% Exceedance Discharge	5.0	785.10	785.00	0.46	0.95
April 25% Exceedance Discharge	76.6	786.75	785.10	1.96	2.51
April 15% Exceedance Discharge	147.5	787.84	786.66	2.60	3.50
April 10% Exceedance Discharge	225.6	788.88	787.68	3.96	3.50
	350.0	790.61	788.51	6.14	3.50
	450.0	791.91	789.73	7.89	3.50
	550.0	793.25	790.45	9.65	3.50
Bankfull Discharge	650.0	794.65	791.07	11.41	3.50
	750.0	796.18	791.60	13.16	3.50
	850.0	797.76	792.12	14.92	3.50
	1,050.0	801.58	792.55	18.51	3.50
100-year Discharge	1,215.0	806.84	793.30	21.40	3.50
PMF Discharge	1,263.0	809.90	793.70	22.16	3.50

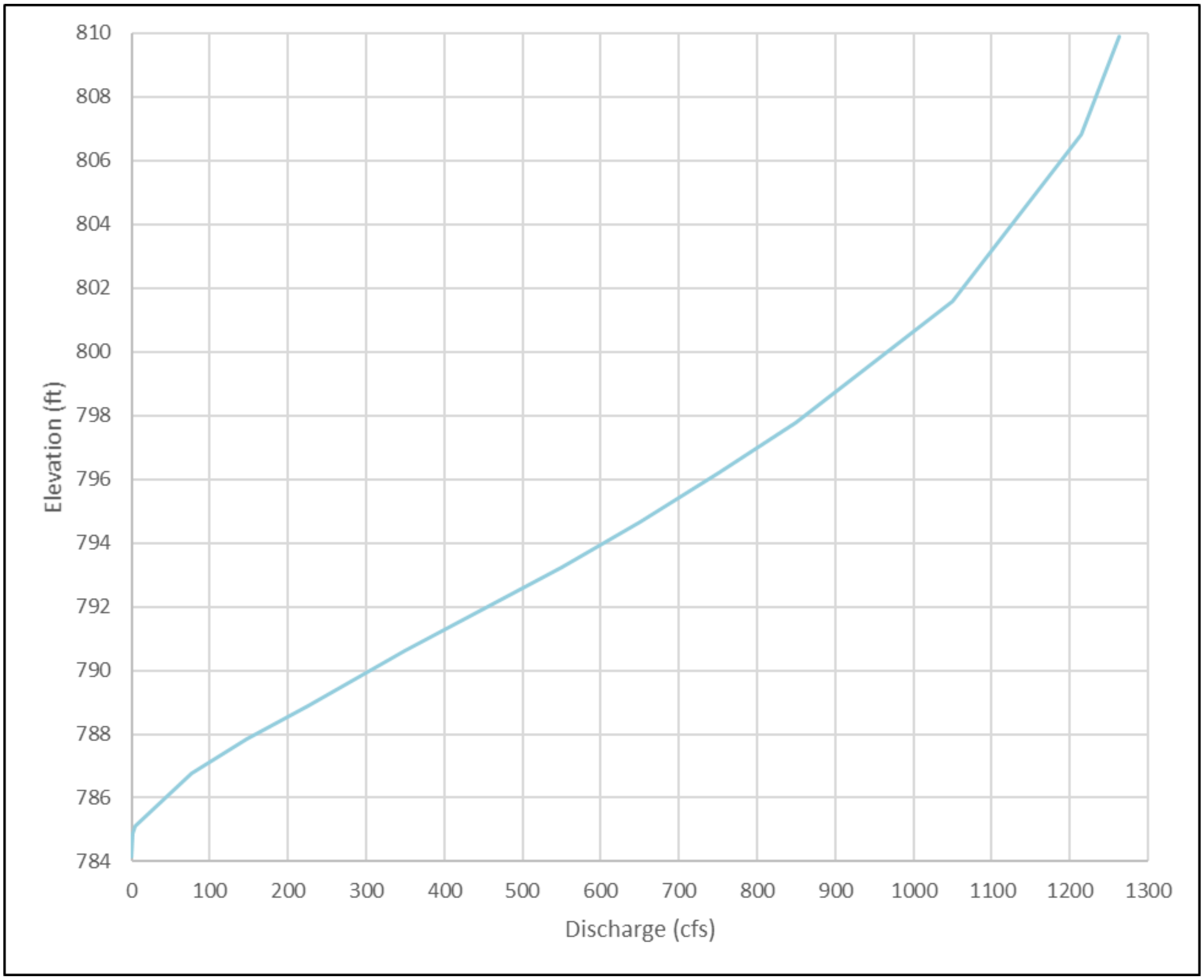

Figure 14. Control Wall / Baffled Chute Principal Spillway Rating Curve

6.6 TRASH 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 trash rack structure upstream of the Principal Spillway. The dimensions of the trash rack were computed based on the methodology described in Chapter 10 of USBR Design of Small Dams (DSD), 3rd Edition (USBR 1987).

The trash rack was designed using the 100-year event discharge (1,215 cfs). The trash rack is a rectangular structure with a 20 ft width to fit within the planned integrated structure abutment walls. Trash rack bar thickness was assumed to be 4-inches. Final bar thickness will be determined during the next phase of design.

During basin storage operations, the inlet of the Principal Spillway will be submerged and inaccessible. According to DSD guidance, the velocity through a clean trash rack should be limited to 2 ft/s. Following DSD guidance, the maximum head loss through the trash rack was computed assuming that the trash rack is 50% clogged. Based on the structure width and design discharge, the trash rack slope and vertical height were varied to achieve the desired velocity. Table 27 summarizes the results of the trash rack calculations.

Table 27. Principal Spillway Trash Rack Dimensions

Parameter	Control Wall / Baffled Chute
100-year Design Discharge (cfs)	1,215
Rack Shape (ft)	Rectangular
Bar Thickness (in)	4
Clear Spacing Between Bars (in)	12
Rack Top Width (ft)	20
Rack Bottom Width (ft)	20
Rack Vertical Height (ft)	13
Rack Slope (H:V)	3.08
Rack Horizontal Length (ft)	40
Maximum Head Loss at 50% Clogged (ft)	0.26

6.7 FISH PASSAGE DESIGN

The recommended Principal Spillway is designed to facilitate fish passage.

For configurations to facilitate fish passage, baffles are required to provide sheltered areas for fish to rest, maintain minimum flow depths at low flows, and keep maximum velocities within acceptable ranges. 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.

Table 28 presents calculations for checking that fish are capable of traversing the baffled chute based on velocity criteria. Table 29 presents calculations for checking that fish are capable of traversing the control wall orifice based on velocity criteria. For details on the development of the fish passage design, refer to Appendix I, Principal Spillway Technical Memorandum.

Table 28. 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
15% Exceedance Discharge Max Flow Velocity (ft/s)	2.40	1.83
Swim Speed Relative to Ground (ft/s)	1.21	0.83
Traversal Distance (ft)	1.5	8.5
Traversal Time (s)	1.21	10.28
Traversal Time Less Than Swim Speed Time Criteria?	Yes	Yes

Table 29. Control Wall Fish Passage Calculations

Description	Traversal Through Middle Orifice
Fish Swim Speed Criteria (ft/s)	3.61 (10 ft)
Fish Swim Speed Time Criteria (s)	2.77
15% Exceedance Discharge Max Flow Velocity (ft/s)	2.60
Swim Speed Relative to Ground (ft/s)	1.01
Traversal Distance (ft)	2.0
Traversal Time (s)	1.98
Traversal Time Less Than Swim Speed Time Criteria?	Yes

6.8 GEOMORPHIC CONSIDERATIONS

Pebble counts were performed at the surveyed riffle cross sections to support discharge calculations and along the reach to classify the stream. A bar sample was collected to support sediment transport competence/entrainment calculations

The geomorphic assessment resulted in design targets for the proposed project reach of Eagle Creek and identified potential causes of future instabilities. Within the proposed project reach, it was found that Eagle Creek has a bankfull discharge of 630 cubic feet per second (cfs) and a bankfull slope of 0.13%. The creek classifies as a Rosgen C4 stream type and was found to be slightly incised while still maintaining floodplain access throughout much of the reach. To reduce the risk of future channel migration, it is recommended that the project reach within the proposed embankment be inspected for woody debris accumulation and that the woody debris be periodically removed, continuing with the work administered by Hancock Soil & Water Conservation District. It is also recommended that the proposed Principal Spillway / embankment crossing be designed to limit the restriction of flows up to 630 cfs (the estimated bankfull discharge) to help mitigate the risks of debris accumulation, sedimentation, and channel migration post-construction.

Stantec applied the BANCS framework in order to characterize erosion in the watershed tributary to the project area, and to help estimate the required operation and maintenance demands. BANCS is an empirically driven model which estimates bank erosion rates using two field assessments. These are:

(1) the Bank Erodibility Hazard Index (BEHI), a measure of bank stability, and

(2) Near-Bank Stress (NBS), a measure of the erosivity of stream flow against a given stream bank.

Stantec collected BEHI and NBS data for Eagle Creek and its tributaries. Multiple techniques were employed to inspect the 55.3 miles of creeks, streams, and ditches which are tributary to the proposed basin. Based on the assessment of streambank erosion throughout the contributing watershed, Eagle Creek and its tributaries are estimated to generate approximately 16,300 tons of sediment per year. The analysis also identified four eroding banks along the approach channel to the proposed dam embankment crossing that are recommended to be stabilized to reduce the risk of bank migration which may affect the stability of the proposed dam embankment. It is recommended to stabilize approximately 1,000 linear feet (LF) of eroding bank using natural stabilization techniques such as root wads with live branch layers.

Based on the existing and proposed conditions sediment transport models, sedimentation within the basin is predicted to increase on average 19% and 13% for a wet and dry year, respectively. The greatest amount of sedimentation predicted was during a wet year which resulted in 4,500 tons (1,700 cubic yards) of sedimentation. This volume of sediment evenly spread across the basin area results in a sediment depth of seven one-hundredths of an inch (0.07 inches / year). Sediment accumulation within the basin over a period of 50 years, assuming a wet year for every year, results in accumulation of approximately 3.5 inches of sediment leading to a loss of 85,000 cubic yards of active storage. This volume reduces the active storage volume by 0.8%. The results show there is expected to be a minimal impact to the active storage volume due to sediment.

The methodology, data, and conclusions are presented in the Geomorphic Assessment Report located in in Appendix C.

6.8.1 Upstream / Downstream Spillway Channel

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

The proposed realigned/relocated Eagle Creek channel ties into the approximate 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 proposed realigned/relocated Eagle Creek channel alignment includes “gentle” meander bends, with radii that will decrease 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. A detailed discussion of the geomorphic data collection and analysis is included in Appendix C, Geomorphic Assessment Report. 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 0.13% slope of the proposed realigned/relocated Eagle Creek channel approximates the bankfull slope of Eagle Creek as measured in the field. While the proposed realigned/relocated Eagle Creek channel is shorter than the reach of Eagle Creek that it replaces, the steeper 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 proposed bank protection structures utilize logs and brush anticipated to be generated during clearing and grubbing for construction of the new Eagle Creek 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 branch 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.

6.8.2 Sediment Transport

Sizing of the Principal Spillway considered sediment transport competence such that sedimentation within the spillway will not occur. 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 was estimated to be 1.2 lbs/ft^2. Since the calculated shear stress is greater than 1.02 lbs/ft^2, it is assumed that the Principal Spillway configuration has the competence to transport the largest bedload particles and will therefore not be prone to sedimentation. Refer to Appendix C for a detailed description of the sediment transport model and results.

6.9 GEOTECHNICAL CONSIDERATIONS

To characterize the subsurface conditions of the Principal Spillway, borings B-3.46 through B-3.54 were conducted in the vicinity of the potential spillway location as shown in Figure 15. Details of the exploration and logs of the borings are available in the Geotechnical Design Report, Appendix E.

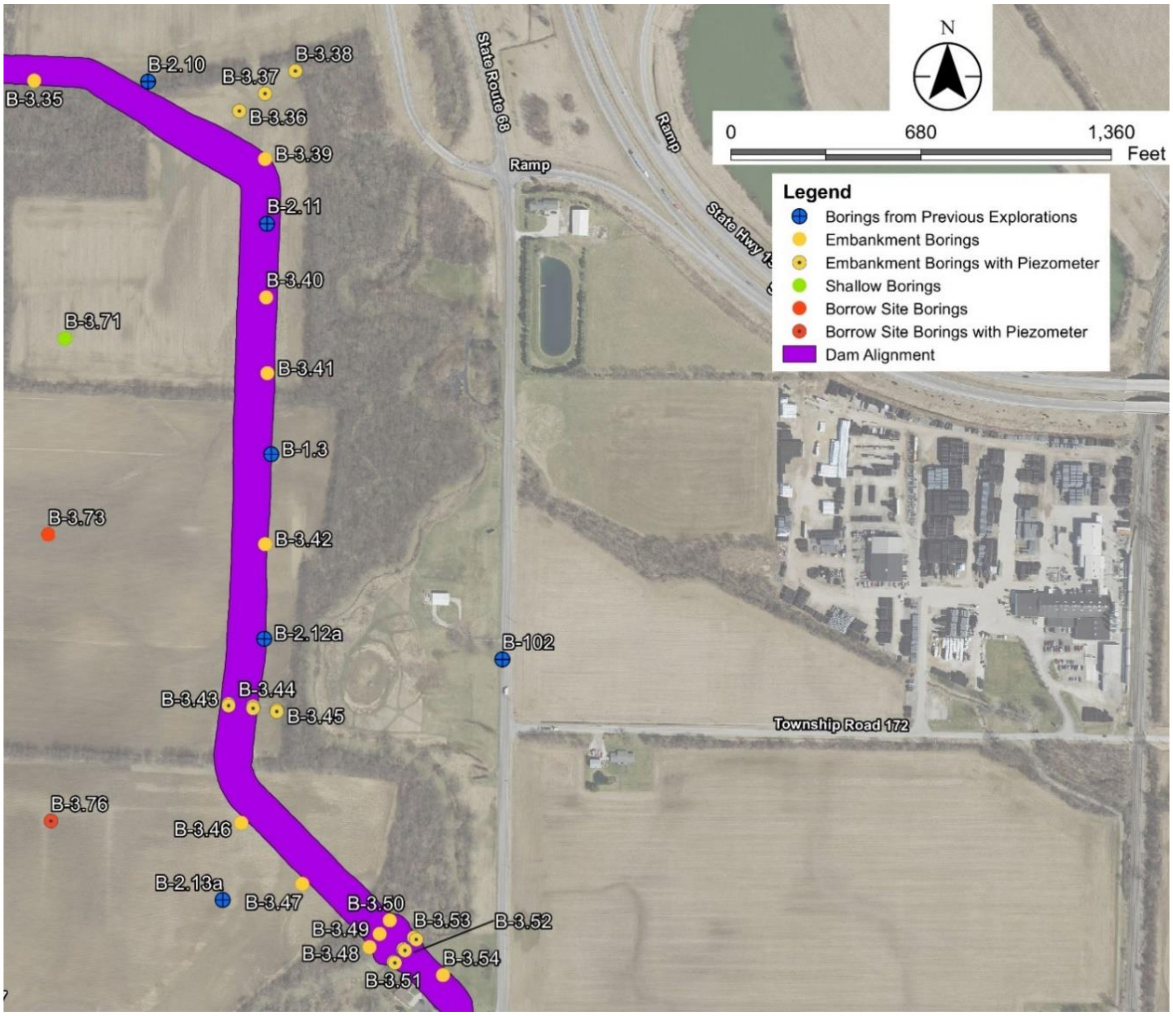

Figure 15. Boring Layout in Potential Principal Spillway Locations

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. The lowest Principal Spillway orifice elevation is approximately 784 feet. The encountered top of rock elevation is approximately 775 to 776 feet. Figure 16 shows the subsurface stratigraphy in the proposed spillway location.

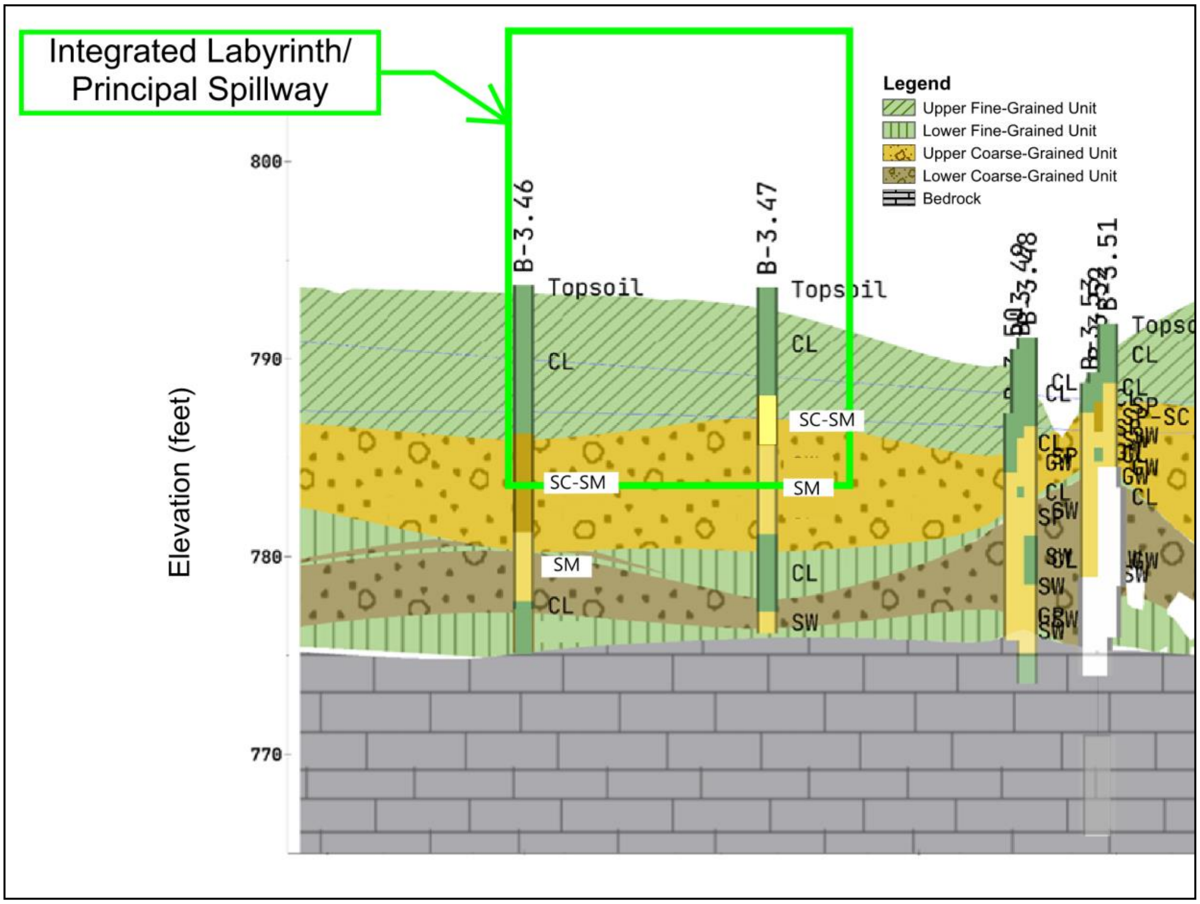

Figure 16. Subsurface Profile – Integrated Labyrinth/Principal Spillway Alternative

6.9.1 Foundation

The geotechnical recommendations and parameters for Preliminary Design include:

- Allowable bearing capacity of 3,000 pounds per square foot (psf). This estimate was based on preliminary sizing and foundation elevation of the structure.
- Subgrade modulus of 100 lbs/in^3 (native soil) or 125 lbs/in^3 (compacted fill). These values are based on guidance from UFC (2005), Table 4-1.
- Interface friction angle between concrete and native soil of 17 degrees and between concrete and compacted fill of 26.5 degrees. These values are based on recommendations in NAVFAC (1986), Table 1.
- Shear strength parameters of native soil: 20-degree friction angle (ϕ) and 400 psf cohesion (c).
- Frost depth for the project site is 36 inches per the Design Criteria Document, Appendix A.

Foundation recommendations and parameters will be further developed during final design. The parameters listed above may be modified as the Principal Spillway details are finalized, or additional geotechnical data is obtained.

6.9.2 Seepage

As shown in Figure 16, the Principal 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. A sheet pile cutoff wall is included as a seepage cutoff below the structure. Detailed seepage analyses will be conducted during Final Design to select cutoff type, size, depth, and other details.

There could also be potential for seepage along the soil-structure interface along the sides of the spillway. To mitigate this risk, the soil side of the spillway 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. Details of the sand filter-drain will also be developed during Final Design.

6.9.3 Settlement

Based on settlement analyses of the dam embankment, less than two inches of settlement of the spillway is expected. Detailed spillway settlement analyses will be conducted during Final Design, after structural loads and spillway locations are further developed. Based on the preliminary estimate, settlement is anticipated to be accommodated through design. Settlement considerations that may be evaluated include differential settlement across the structure, adequate foundation soil compaction, and the need for ground improvement or pile foundations to meet settlement criteria.

6.9.4 Seismic Design 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.

6.10 STABILITY

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

6.10.1 Acceptance Criteria

All structural elements of the Principal Spillway (except the trash rack) are assumed to be critical structures as failure could directly or indirectly lead to a loss of life. The trash 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 1110-2-2502. The load cases to be evaluated are divided into categories as listed in Table 30:

Table 30. 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	≥1.5	≥1.5
Unusual	Middle third of the base: 100% compression	≥1.3	≥1.3
Extreme Flood	Within middle half of the base, and all other acceptance criteria must be met	≥1.1	≥1.1

6.10.2 Load Combinations

Table 31 summarizes load conditions the Principal Spillway structure. Table 32 summarizes load conditions for the typical retaining wall. The load combinations listed below are to be considered a representative sample. For preliminary design and to be conservative while checking stability, the toe drains were never considered effective. Therefore, those load cases will not be included until final design.

Table 31. Principal Spillway – Load Conditions

Usual Load Cases		
U1	Usual Condition – (dry / no flow) At-Rest Soil Loading Ground Water at Top of Foundation	D+H+E+U
Unusual Load Cases		
UN1	Unusual Condition – (No tailwater) At-Rest Soil Loading Max. headwater without Tailwater (top of foundation) – Effective Toe Drain + Flood	D+H+E+U
UN2	Unusual Condition – (No tailwater) At-Rest Soil Loading Max. headwater without Tailwater (top of foundation) – Ineffective Toe Drain + Flood	D+H+E+U
UN3	Unusual Condition – (100-yearPool Elevation) At-Rest Soil Loading – Effective toe drain + Flood	D+H+E+U
UN4	Unusual Condition – (100-year Pool Elevation) At-Rest Soil Loading – Ineffective toe drain + Flood	D+H+E+U
Extreme Load Cases		
E1-F	Extreme Condition – Maximum Flood Level at Reservoir (Max PMF) – Effective toe drain + Flood	D+H+E+U
E2-F	Extreme Condition – Maximum Flood Level at Reservoir (Max PMF) – Ineffective toe drain + Flood	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 32. 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
Unusual Load Cases		
UN1	Unusual Condition – At-Rest Soil Loading + Equipment Surcharge Groundwater at Top of Footing	D+H+E+U+L
Extreme Load Cases		
E1-F	Extreme Condition – At-Rest Soil Load – Post Maximum Flood Level at Reservoir Groundwater varies linearly from Headwater to Tailwater	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	

6.10.3 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 the stability calculations and results.

6.10.3.1 Slab Bridge

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

6.10.3.2 Center Monolith

The center monolith of the control wall and baffled chute option is the large central section of the Principal Spillway shown in Figure 17.

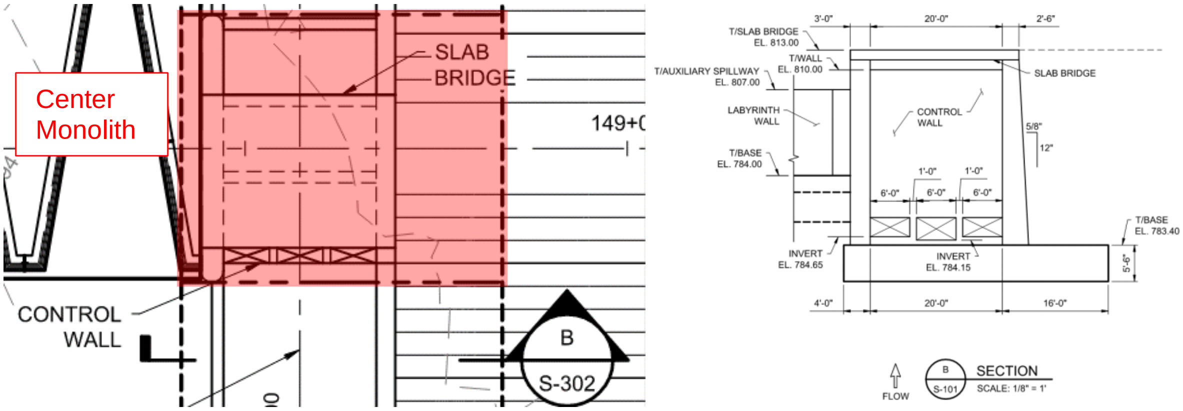

Figure 17. Center Monolith Plan and Section

Stability analysis for the center monolith structure was performed using Mathcad and was checked in two directions. Stability was evaluated along the plane of the channel and along the plane of the dam baseline. By inspection due to the center monolith's location and considering surrounding structures sliding will not be a stability concern and was not evaluated. Results of the analyses are summarized in Table 33.

Table 33. Center Monolith – Stability Summary

Stability About:	Load Comb.	e (ft)	σ @ Heel (psf)	σ @ Toe (psf)	% Base in Comp.	Reaction in Middle 1/3 (1/2 for E2-F LC	Floatation SF	SF Req'd.
Channel	U1	-1.272	2596	2573	100	Yes	10.355	1.5
	UN2	0.003	2584	2585	100	Yes	10.355	1.3
	E2-F	-1.213	2596	2573	-	Yes	10.355	1.1
Dam Baseline	U1	-0.113	2336	2334	100	Yes	10.355	1.5
	UN2	2.313	1574	1603	100	Yes	2.226	1.3
	E2-F	2.01	1435	1458	-	Yes	1.862	1.1

6.10.3.3 Retaining Walls

The east and west abutment or retaining walls were divided into four different wall types for the purpose of the stability analysis. Figure 18 shows the retaining wall design locations.

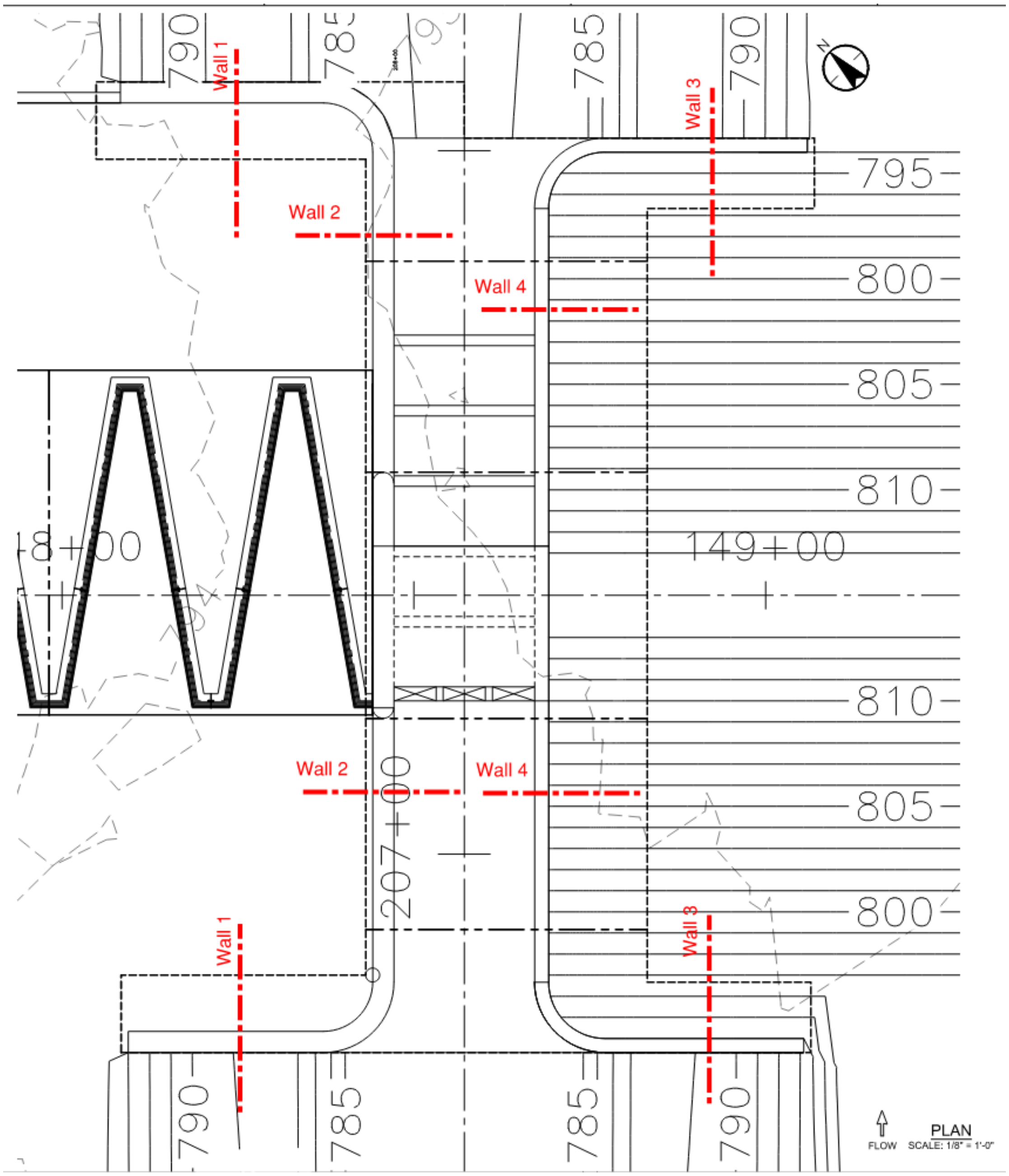

Figure 18. Retaining Wall Design Locations

The stability analysis was performed using Excel spreadsheets. Due to the locations of Walls 2 and 4 shown in Figure 18, sliding was not a stability concern because any sliding movement would push against the opposite retaining wall. Results of the analyses are summarized in Table 34.

Table 34. Retaining Wall – Stability Summary

Wall	Load Comb.	Reaction in Middle 1/3 (1/2 for E1-F LC)	σ @ Heel (psf)	σ @ Toe (psf)	% Base in Comp.	Sliding SF	Floatation SF	SF Req'd.
1	U1	Yes	966	2135	100	2.62	10.00	1.5
	UN1	Yes	414	2687	100	1.68	10.00	1.3
	E1-F	Yes	-190	2659	n/a	1.18	3.45	1.1
2	U1	Yes	1010	452	100	n/a	5.26	1.5
	UN1	Yes	656	806	100	n/a	5.26	1.3
	E1-F	Yes	30	780	n/a	n/a	1.8	1.1
3	U1	Yes	1761	2323	100	2.08	11.91	1.5
	UN1	Yes	1120	2964	100	1.52	11.91	1.3
	E1-F	Yes	355	2932	n/a	1.16	4.08	1.1
4	U1	Yes	2694	2254	100	n/a	10.92	1.5
	UN1	Yes	2141	2894	100	n/a	9.97	1.3
	E1-F	Yes	250	3215*	n/a	n/a	2.7	1.1

The toe pressure in Wall 4 under the extreme loading condition is calculated to exceed the allowable bearing capacity, however, this is in a limited area under an extreme load condition. The result at this location will be accounted for and addressed during Final Design.

6.11 SERVICEABILITY

6.11.1 Trash Rack

The trash rack for the control wall and baffled chute option was preliminarily designed assuming a maximum 4-inch-wide main member size spaced with 12 inches clear between members. The maximum loading assumed in the design of the trash rack was based on a case where the trash rack has been 100% blinded by debris and the flood waters reach the 100-year pool elevation of 807.0 ft with no tailwater. This load case would result in the maximum load being placed on the trash rack members. If flood waters were to rise higher than elevation 807.0 ft, then tailwater would begin to rise because water would begin to flow over the auxiliary spillway weir crest.

Preliminary design of the trash rack main members showed that the span as configured, approximately 42 feet, would be too long for any single member without additional supports. For the purpose of Preliminary Design, the main member was assumed to be a stainless steel HSS 8x4x1/2 with support beams spaced every 5 feet. The support beams would run between the east and west abutment or retaining walls. Based on the assumed design loadings, the 20-foot-long stainless-steel support beams are required to be a W30x90 using a smaller W24x68 at the ends where the loading is less. The total deflection varies per support beam, but the largest deflection was 0.351 inches or equivalent to L/684.

6.12 CONSTRUCTION CONSIDERATIONS

Construction specifications and details for the Principal Spillway will be furthered during Final Design. The following construction considerations are noted:

- Dewatering of excavated areas will be required to sufficiently enable construction of the relocated Eagle Creek Channel. The services of a specialist dewatering contractor may be needed.
- Excavation should be performed by mechanical means only; blasting will not be permitted.
- Fill placement and compaction methods adjacent to the retaining walls must be reviewed and monitored to ensure wall movement does not occur during construction.
- Limiting total and differential settlement between the Principal Spillway and the Auxiliary Spillway may require foundation improvements and additional foundation protection requirements during construction.

7.0 AUXILIARY SPILLWAY

The Auxiliary Spillway is a secondary structure used to 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 recommended 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 situated in the northeast section of the ECFB dam embankment, integrated with the Principal Spillway structure. The structure would be constructed at-grade and replace the earthen embankment dam. Cast-in-place concrete retaining walls transition back to the embankment.

A labyrinth spillway is a linear weir folded in plan-view to increase the effective length of the weir within a given spillway width. A labyrinth weir can pass large discharges at relatively low heads compared to traditional linear weirs of equal 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.

The labyrinth Auxiliary Spillway structure consists of a 2-foot-thick reinforced concrete wall, forming nineteen (19) labyrinth cycles, each measuring 47 feet deep and 23 feet wide, for a total structure width of 437 feet. The crest of the labyrinth weir will be at elevation 807.0 feet, approximately the 1% ACE (100-year) water surface elevation, and the base will be at elevation 794.0 feet, for a total height of 13 feet. The labyrinth weir will sit on a reinforced concrete apron slab 2.5 feet thick and 47 feet long.

Downstream of the labyrinth weir is a 21-foot long USBR Type I Natural Jump concrete slab stilling basin to dissipate energy so that the flow exiting the downstream channel is sub-critical. The stilling basin requires no baffles or end sill for sufficient energy dissipation. The stilling basin daylights at existing grade, at which point flow will continue downstream to Eagle Creek. The structure includes semi-circular upstream approach walls and downstream retaining walls

Figure 19 shows a typical section of the labyrinth spillway.

Auxiliary Spillway

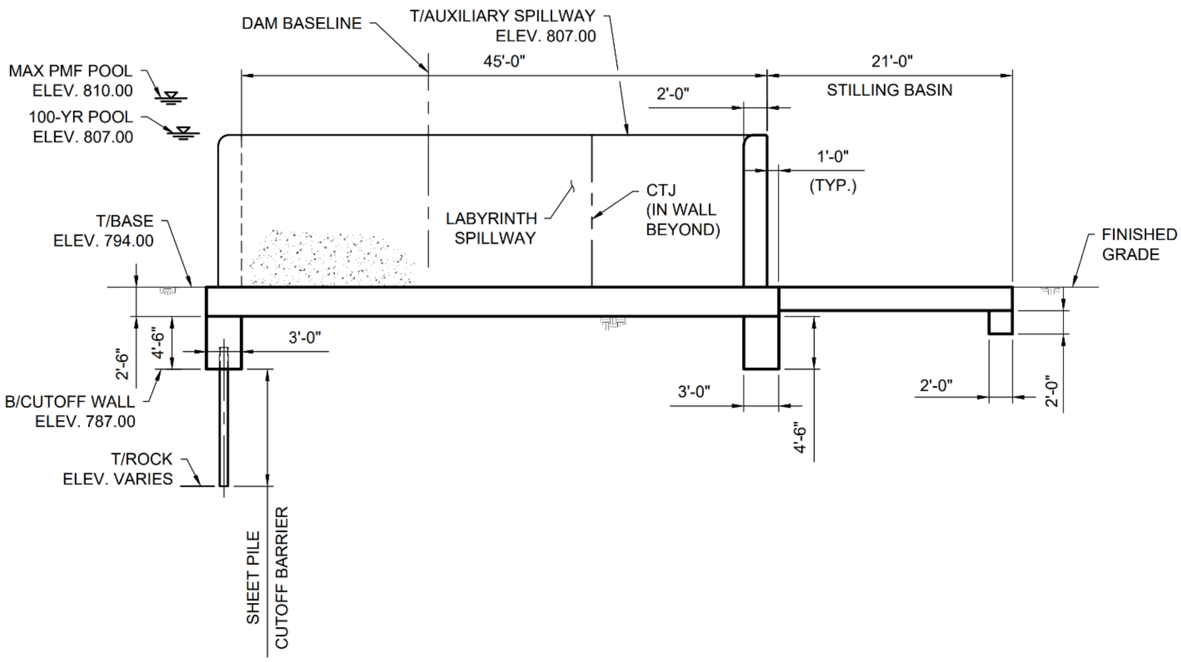

Figure 19. Typical Labyrinth Spillway and Stilling Basin Section

7.2 DESIGN OBJECTIVES

The design objective of the Auxiliary Spillway is to maintain discharge capacity to pass the Probable Maximum Flood (PMF), approximately 27,450 cfs, 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 Auxiliary Spillway is designed to pass the PMF up to an elevation of 810.0 feet. The maximum pool elevation of 810.0 feet during the PMF equates to a maximum design head of 3 feet above the structure crest. 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 HYDRAULIC DESIGN

The hydraulic design of the Auxiliary Spillway includes the geometry of the spillway, an energy dissipation component, and the natural 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 crest elevation consideration. The crest elevation is discussed in the Dam Embankment Design Technical Memorandum, Appendix G.

7.3.1 Labyrinth Crest Length and Rating Curve

Crest length was calculated for the integrated labyrinth spillway using the effective spillway length and selected discharge coefficient, a rating curve was developed. Detailed computations for the labyrinth spillway design are provided in Exhibit A of Appendix J. The labyrinth weir was designed 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 35 and computed weir geometries are shown in Table 36. The computations assume an ogee-shaped weir crest.

Table 35. 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 36. 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 20 shows a general schematic of a single weir cycle.

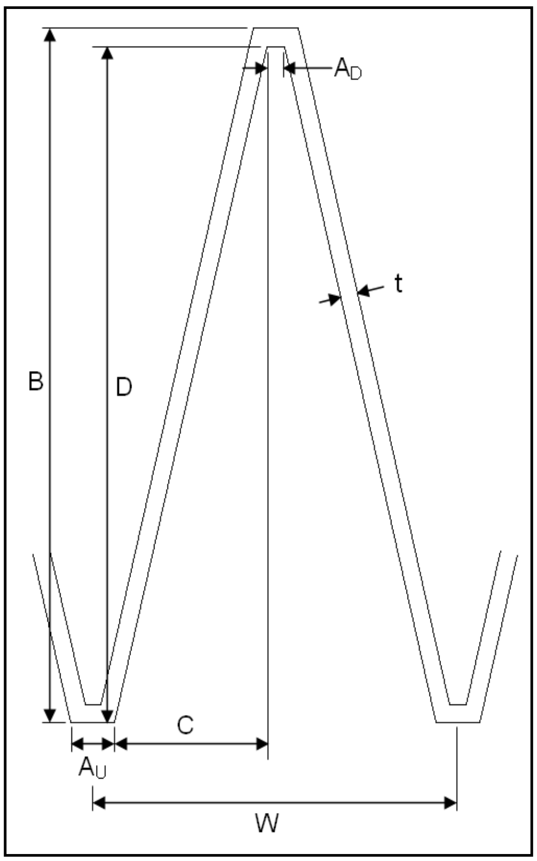

Figure 20. Labyrinth Weir Single Cycle Schematic

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 37 and Figure 21. Performance of the spillway during simulated design flood events is presented in Section 8.0.

Table 37. 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

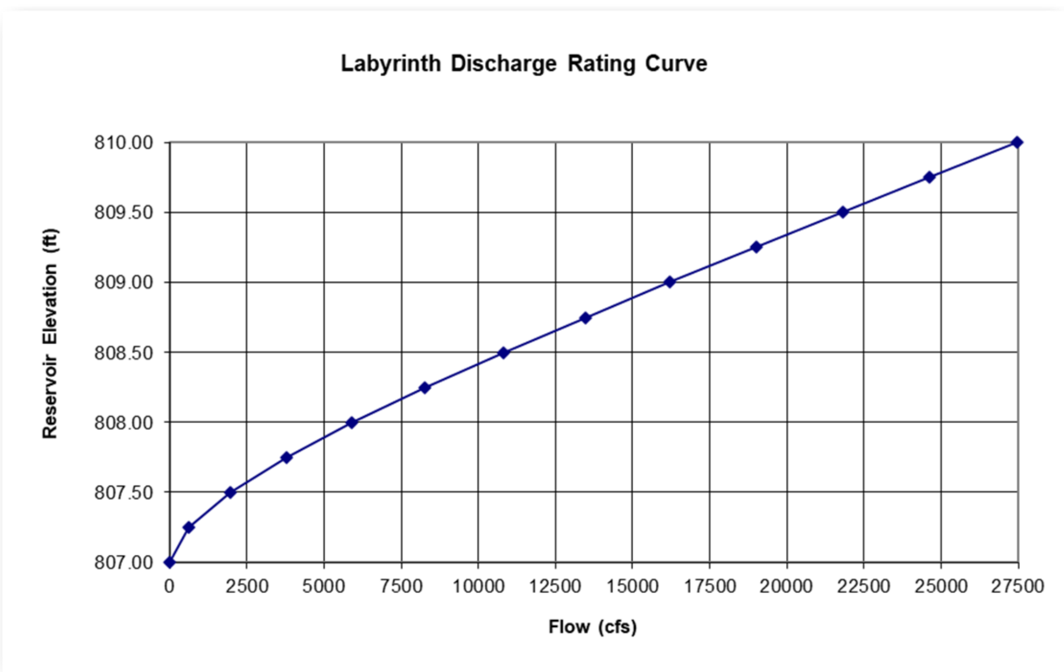

Figure 21. Labyrinth Spillway Rating Curve

7.3.2 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. The stilling basin is 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 38.

Table 38. Residual Energy at Base of Labyrinth

Description	Symbol	Integrated Labyrinth	Unit
Total discharge	Q	27,400	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 is found using Figure 12 from EM 25 (USBR, 1984). Since the Froude number is less than 2, the minimum ratio of basin length to conjugate depth (L/d_2) is 4. A USBR Type I natural jump stilling basin (no baffles, no end sill) was selected and the required length is 21 feet. A summary of the stilling basin calculations is in Table 39.

Table 39. Labyrinth Stilling Basin Calculations

Description	Symbol	Integrated Labyrinth	Unit
Unit discharge	Q	62.7	cfs
Residual energy	H_1	7.45	ft
Sequent depth	d_{1_super}	4.81	ft
Sequent velocity	v_{1_super}	13.04	ft/s
Froude number	F_1	1.05	
Conjugate depth	d_2	5.11	ft
Conjugate elevation	EL_{d2}	799.11	ft
Tailwater elevation	EL_{TW}	802.52	ft
Tailwater depth	d_{TW}	8.52	ft
Difference in elevation	$EL_{TW} - EL_{d2}$	3.40	ft
Tailwater ratio	d_{TW} / d_2	1.66	
Length of hydraulic jump	L_j	18.0	ft
Ratio of basin length to d_2	L / d_2	4	
USBR basin length	L_b	20.5	ft

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

7.3.3 Hydraulic Design Geometry Summary

Table 40 provides a summary of the key hydraulic design geometry for the integrated labyrinth AS option.

Table 40. Auxiliary Spillway Components / Dimensions

Crest Elevation (ft)	Spillway Type	Approximate Footprint Length (ft)	Approximate Height (ft)	Stilling Basin length (ft)
807.0	Labyrinth Integrated into PS	437 (19 cycles, 47 feet deep)	13	21.0 (USBR Type I)

7.4 GEOTECHNICAL CONSIDERATIONS

The geotechnical considerations for the integrated Principal Spillway and Auxiliary Spillway are described in Section 6.9. Additional information is provided in the ECFB Geotechnical Design Report, Appendix E.

7.5 STABILITY

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

7.5.1 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 41.

Table 41. 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	≥1.5	≥1.5
Unusual	Middle third of the base: 100% compression	≥1.3	≥1.3
Extreme Flood	Within middle half of the base, and all other acceptance criteria must be met	≥1.1	≥1.1

7.5.2 Load Combinations

Table 42 summarizes load conditions for the AS structure. The load combinations listed below are to be considered a representative sample. For preliminary design and to be conservative while checking stability, the toe drains were not considered effective. The load cases with drains will be evaluated in Final Design.

Table 42. Auxiliary Spillway – Load Conditions

Usual Load Cases		
U1	Usual Condition – (dry / no flow) At-Rest Soil Loading Ground Water at Top of Foundation	D+H+E+U
Unusual Load Cases		
UN1	Unusual Condition – (No tailwater) At-Rest Soil Loading Max. headwater without Tailwater (top of foundation) – Effective Toe Drain + Flood	D+H+E+U
UN2	Unusual Condition – (No tailwater) At-Rest Soil Loading Max. headwater without Tailwater (top of foundation) – Ineffective Toe Drain + Flood	D+H+E+U
UN3	Unusual Condition – (100-year Pool Elevation) At-Rest Soil Loading – Effective toe drain + Flood	D+H+E+U
UN4	Unusual Condition – (100-year Pool Elevation) At-Rest Soil Loading – Ineffective toe drain + Flood	D+H+E+U
Extreme Load Cases		
E1-F	Extreme Condition – Maximum Flood Level at Reservoir (Max PMF) – Effective toe drain + Flood	D+H+E+U
E2-F	Extreme Condition – Maximum Flood Level at Reservoir (Max PMF) – Ineffective toe drain + Flood	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	

7.5.3 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.

7.5.4 Auxiliary Spillway Integrated Labyrinth Weir

The stability analysis for the structure was performed using Excel spreadsheets. Results of the analyses are summarized in Table 43.

Table 43. Auxiliary Spillway Integrated Labyrinth Weir – Stability Summary

Load Comb.	e (ft)	σ @ Heel (psf)	σ @ Toe (psf)	% Base in Comp.	Sliding SF	Floatation SF	SF Req'd.
U1	0.17	890	890	100	27.56	1.83	1.5
UN2	3.99	800	810	100	2.85	1.33	1.3
E2-F	2.60	770	780	100	3.08	1.21	1.1

7.5.5 Auxiliary Spillway Integrated Abutment

The stability analysis for the integrated labyrinth spillway abutment structure was performed using Excel spreadsheets. Results of the analyses are summarized in Table 44.

Table 44. Auxiliary Spillway Integrated Labyrinth Abutment – Stability Summary

Load Comb.	E (ft)	σ @ Heel (psf)	σ @ Toe (psf)	% Base in Comp.	Sliding SF	Floatation SF	SF Req'd.
U1	0.029	2234	2234	100	55.58	4.88	1.5
UN2	2.543	1555	2187	100	5.36	2.5	1.3
E2-F	2.249	1335	1771	100	5.0	1.7	1.1

7.6 SERVICEABILITY

Serviceability concerns for the reinforced concrete structures relate to concrete durability, shrinkage, crack control, volume changes, and wall deflections. Durability, shrinkage, and crack control are achieved primarily through reinforcement placement, high reinforcement ratios, and use of high load factors that account for both strength and serviceability. Volume changes are addressed primarily with placement sequence, mix design, surface reinforcement, and material specifications. Expanded guidance related to placement sequence and joint locations will be addressed as part of constructability review during Final Design.

7.7 CONSTRUCTION CONSIDERATIONS

Construction specifications and details for the Auxiliary Spillway will be furthered during Final Design. The following construction considerations are noted:

- Dewatering of excavated areas will be required to sufficiently enable construction of the Auxiliary Spillway. The services of a specialist dewatering contractor may be needed.
- Limiting total and differential settlement between the Principal Spillway and the Auxiliary Spillway may require significant foundation improvement.

- The length of the Auxiliary Spillway structure will require a long duration for segments of the foundation to be exposed. Protection of the foundation from freeze / thaw, precipitation and surface runoff will be an important consideration during construction.

- Staging of the Auxiliary Spillway must consider the potential for a flood event during construction. Care of water plans may consider the potential to utilize a “gap” in the Auxiliary spillway to reduce risks for dam overtopping during construction 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 was designed such that the reservoir stores 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 22 and the reservoir routing hydrographs for the PMF event are shown in Figure 23. The hydrographs represent the integrated Principal Spillway and labyrinth Auxiliary Spillway structures.

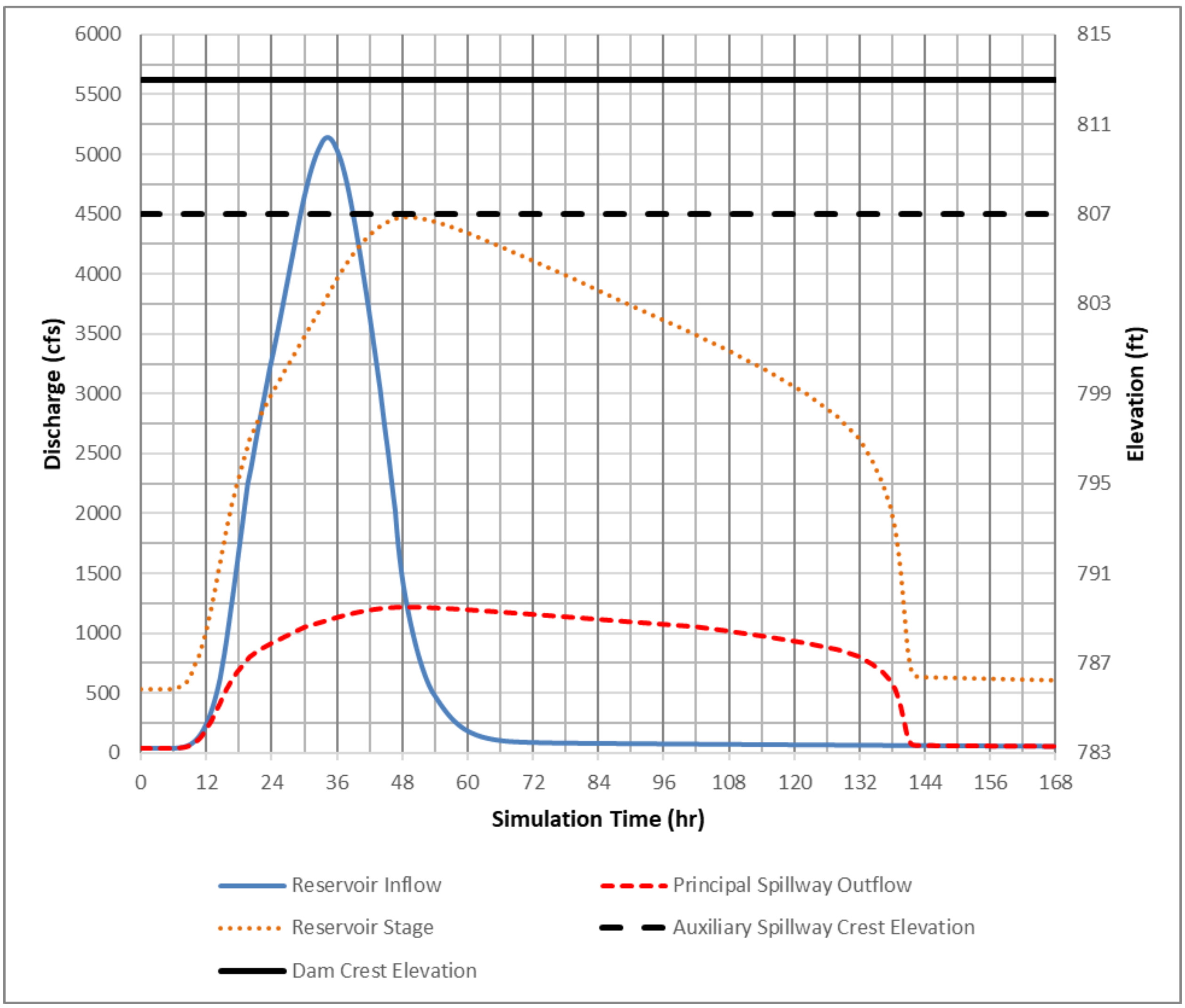

Figure 22. Reservoir Routing – 1% ACE (100-year) Event

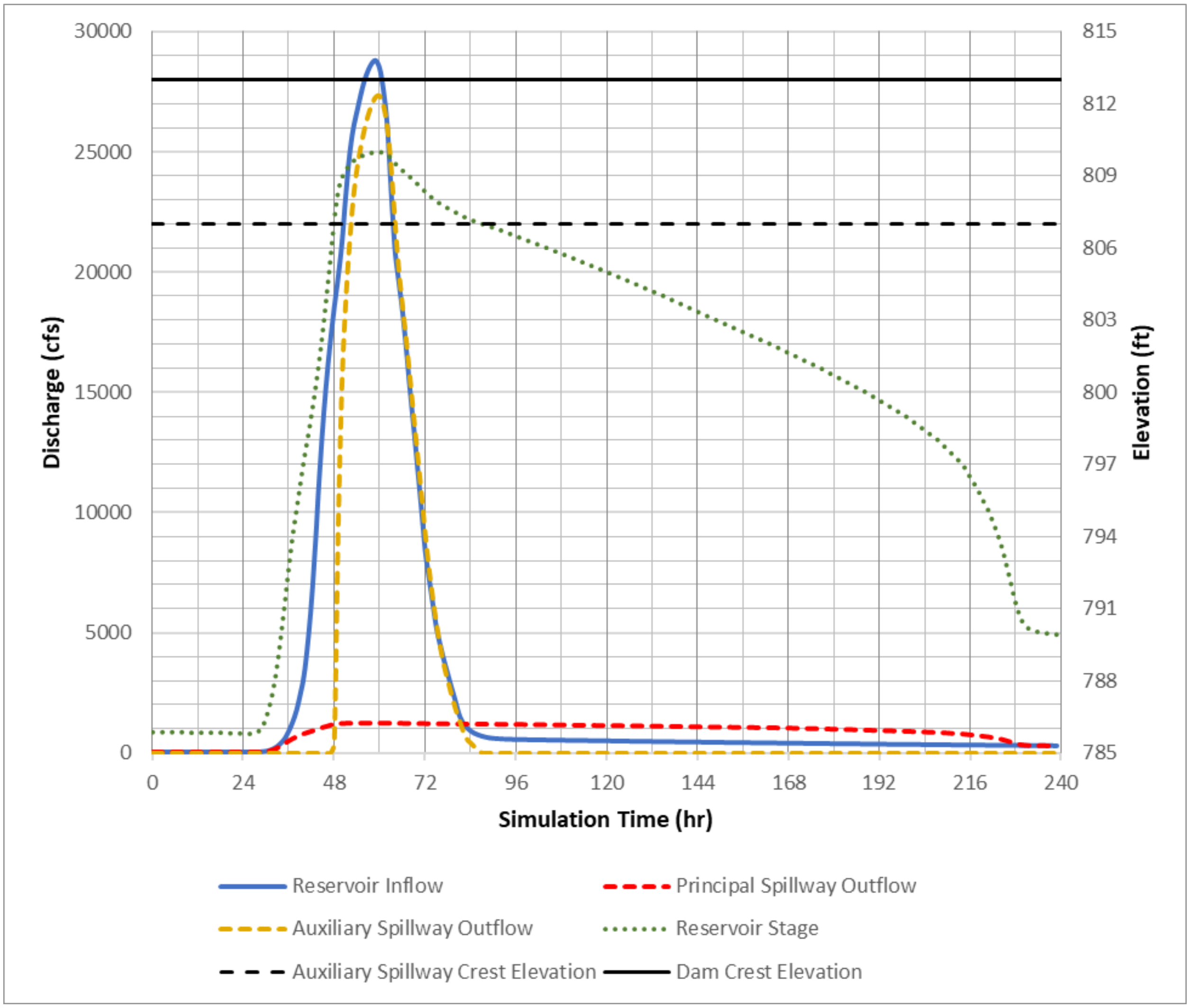

Figure 23. Reservoir Routing – PMF Event

The maximum modeled reservoir inflow, principal spillway outflow, and reservoir stage for each event is summarized in Table 45.

Table 45. Peak Inflow-Outflow-Stage Summary

Peak Reservoir Flows and Stages	100-Year	PMF
Reservoir Inflow (cfs)	5,137	28,778
Principal Spillway Discharge (cfs)	1,215	1,264
Auxiliary Spillway Discharge (cfs)	0	27,333
Reservoir Stage (ft)	806.85	809.99

8.1 UPSTREAM IMPACTS

The approximate surface areas and storage volumes are presented in Table 46. The proposed project inundation areas are shown in Figure 24. Additional upstream impacts are presented in the Hydrologic and Hydraulic Analysis Report, Appendix D.

Table 46. Inundation Areas and Storage Volumes

Reservoir Elevation (ft)	Surface Area (acres)	Storage Volume (acre-ft)
807.0	906	6,900
810.0	1,029	9,776
813.0	1,182	13,040

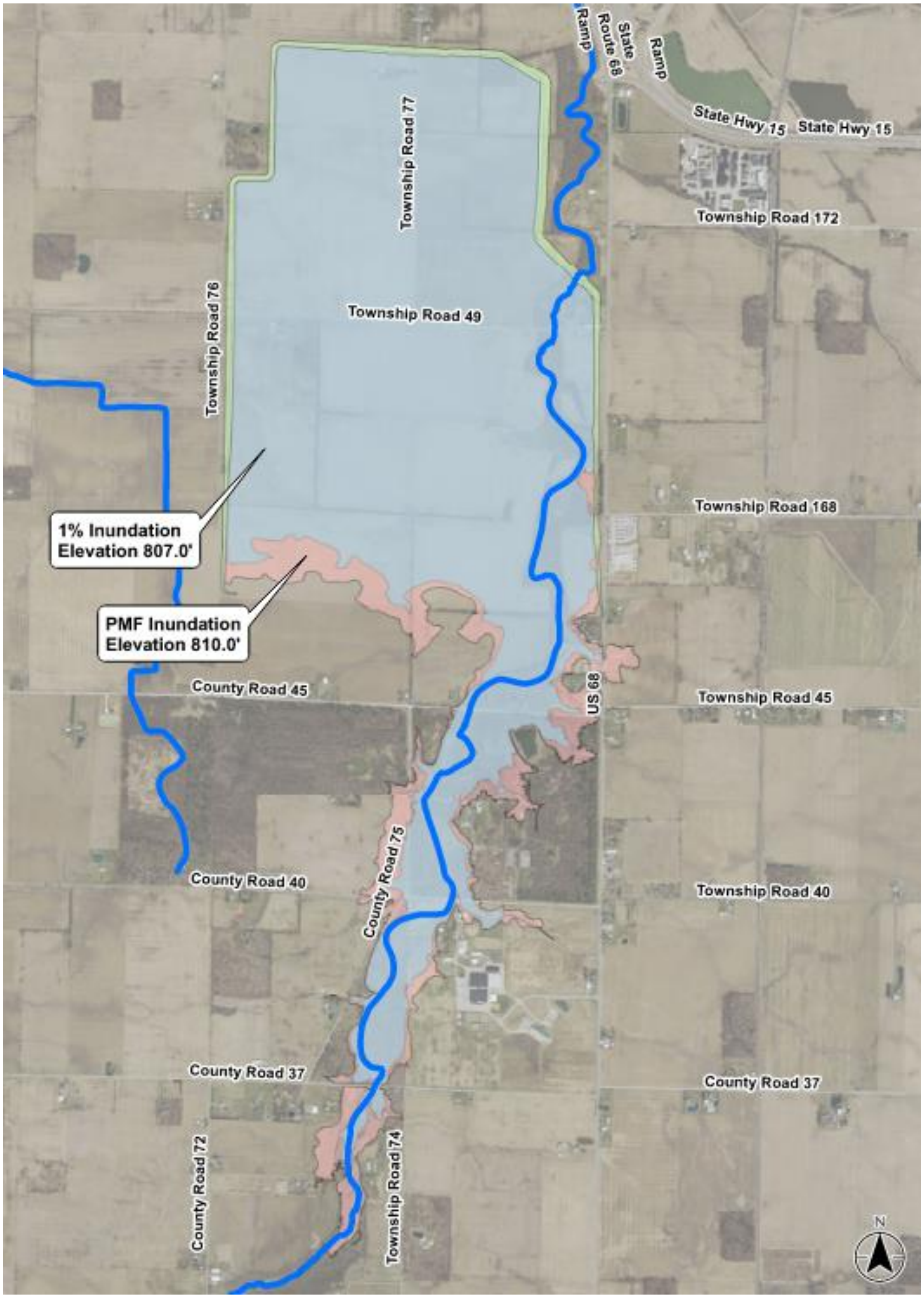

Figure 24. Reservoir Inundation Extents

8.2 DOWNSTREAM BENEFITS

Hydraulic model results show that the ECFB project results in a peak flow reduction of about 2,700 cfs (17% decrease) on the Blanchard River during the 1% ACE WSE which translates to about 2.1 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 1,740 parcels and 1,680 acres from the regulatory floodplain.

Stantec reviewed transportation impacts due to flooding at 19 locations across the watershed. Many of the transportation impacts are expected to be reduced as a result of the ECFB. Table 47 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 47. Transportation Impacts and Benefits

Bridge/Intersection	Reach	1% ACE Depth above Bridge/Intersection (feet)	
		Ex. Cond.	ECFB
SR 68 near TR 172	Eagle Creek	1.6	0.0
6th Street / Westview	Eagle Creek	0.8	0.0
S. Blanchard St. / E. Lincoln	Eagle Creek	4.1	1.7
E. Sandusky / S. Blanchard St.	Eagle Creek	3.3	1.0
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	5.3	3.4
SR 568 near CR 236	Blanchard River	2.8	2.2
E. Main Cross / Warrington Ave.	Blanchard River	2.0	0.3
S. Blanchard St. / E. High St.	Blanchard River	5.9	3.9
E. Main Cross / MLK Pkwy	Blanchard River	4.4	2.4
Main St.	Blanchard River	3.9	1.9
Defiance Ave. / Univ Townhouses	Blanchard River	4.6	3.0
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	1.9
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 were incorporated into the design to address drainage associated with the interior access bench and maintenance zone, the proposed borrow pits and wetlands, 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, though, the basin will dewater in the days following a filling event so that storage is available for the next event. Figure 25 presents the primary interior drainage strategies. 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 is designed to allow for maintenance and access for inspection of the dam. The area within this maintenance zone will be 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 be allowed to pond within the maintenance zone. One ditch is provided along the public access road at the southwest corner of the site to convey runoff between the road and embankment. 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 for dam embankment fill material will be sourced from the interior of the basin. The borrow pits will be graded, vegetated, and converted into wetlands as their final condition. The location and maximum depths of the borrow pits and wetlands were selected to reduce the risk for potential seepage paths below the dam embankment. Seepage considerations include locating borrow pits and wetlands a minimum of 400 feet upstream of the dam embankment toe and limiting the borrow/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.

Wetland 2 finishes draining through two 24-inch culverts into the Wetland 1 rather than over an uncontrolled low point along the edge. Wetland 1 finishes draining through a swale with riprap protection installed on the upstream end to reduce the risk of uncontrolled head cutting as the basin drains. Details of the proposed wetlands are shown on the preliminary design drawings.

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

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 depth must be limited to reduce the risk of exposing the underlying coarse-grained soils. Given these limitations, the area in the northeast corner 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 100-foot-wide bench starting at elevation 796 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 100-foot-wide bench, the low-lying area may be filled with spoil material up to elevation 794 feet or may remain an open pool (until infiltrated or evaporated).

9.1.4 Future Considerations

The interior drainage is subject to revision in final design as needs arise. Considerations may include future use of the basin interior such as site access and walking trails. Figure 25 presents a layout of the proposed interior drainage grading and features.

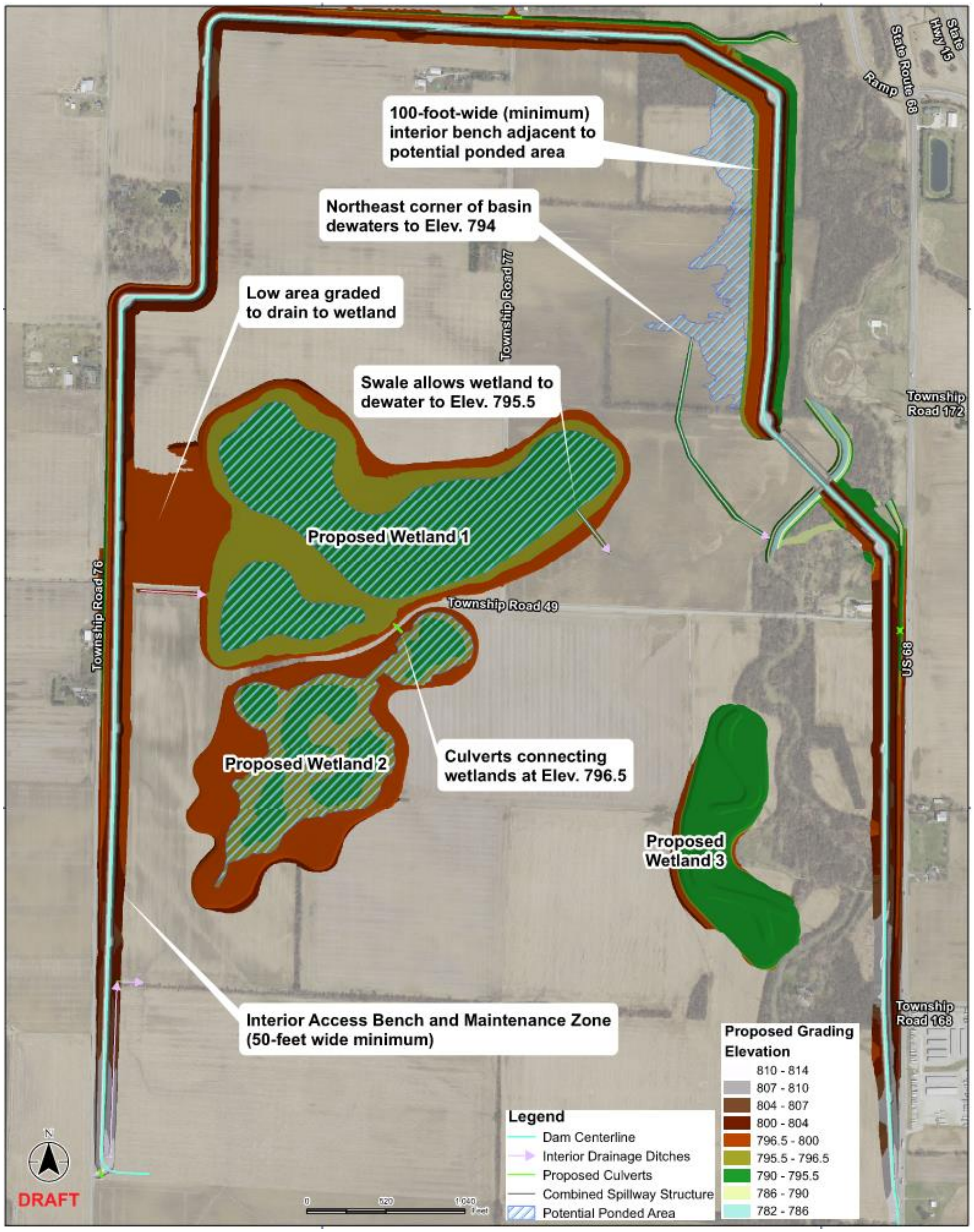

Figure 25. Proposed Interior Drainage Design

9.2 WETLAND DESIGN

The proposed basin interior is well suited for wetland restoration due to its suitable soils, good growing conditions, and abundance of potential sources of wetland hydrology. Three interior concepts were explored that considered excavation for embankment borrow material, wetland size, potential water quality benefits, and / or a combination of these. The recommended option is a hybrid concept, where the created wetland area and water quality potential is maximized while excess excavation is minimized. The proposed layout is in Figure 26.

Another anticipated component of the basin interior (post-construction) is passive recreation. A public trail system is proposed to access the wetlands and adjacent naturalized areas. The wetlands are expected to attract wildlife to the area and will provide an aesthetically pleasing landscape to visitors to view. A 12-foot-wide trail will wind through the site and will contain wildlife / birding platforms and informational kiosks describing the functions and values of the wetlands and creatures that inhabit them. The recreational component will continue to evolve in future stages of design. The Interior Wetland Design Report is included in Appendix K.

9.3 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. Township Road 49 roadway and bridge are anticipated to remain in place for access east and west across the site.

Multiple access points are available from the perimeter of the dam embankment to the crest for visual maintenance, observation, and inspection.

An entry drive in the southwest corner of the site is anticipated for use by the general public. The primary 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 potential trail system.

To discourage foot and vehicle traffic near the spillway structures, fencing, rail, or gates are anticipated to be installed along the perimeter of the spillway structure.

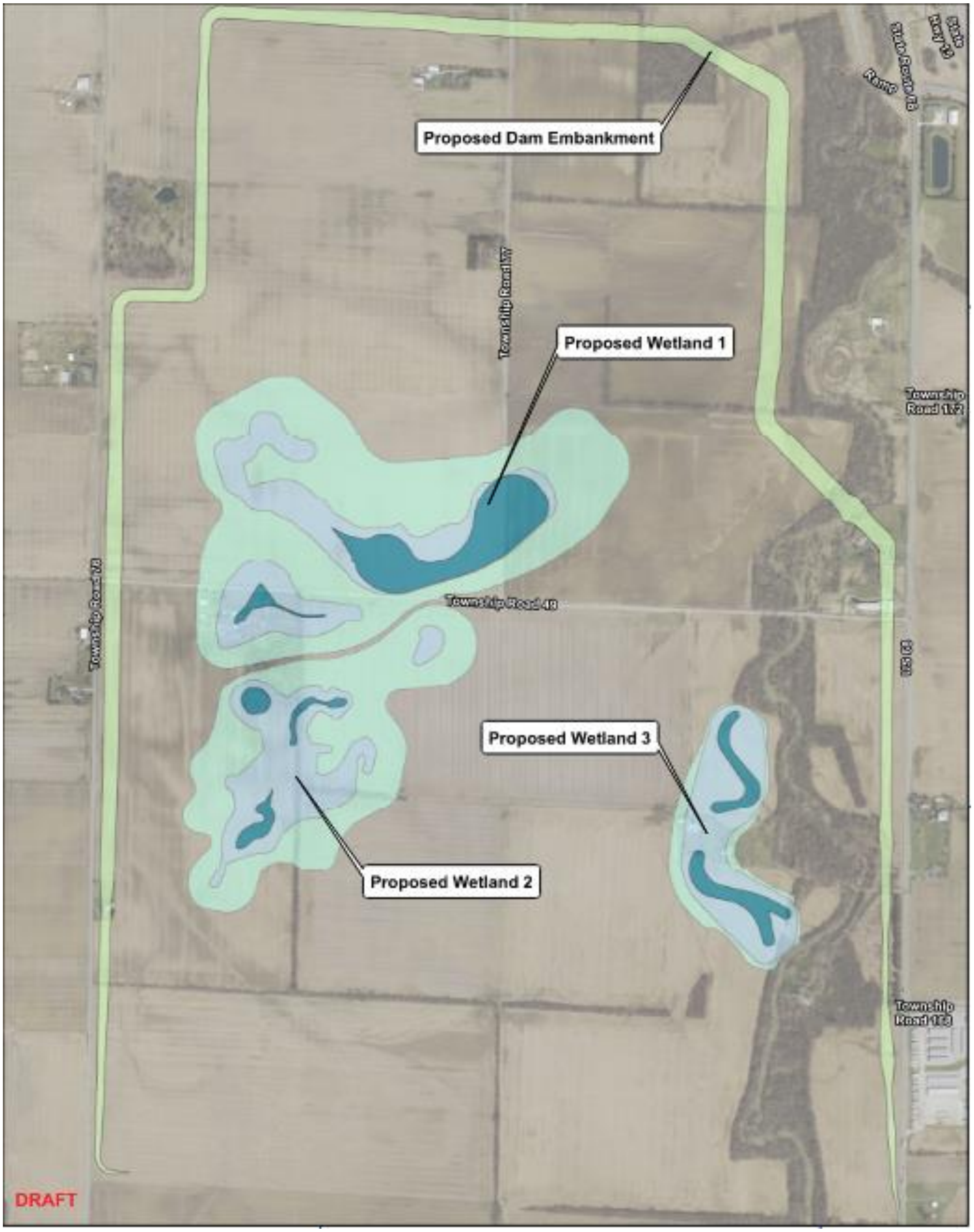

Figure 26. Proposed Wetlands and Planting Zones

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 49 will be fully decommissioned between US-68 and Township Road 76. 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 planned at the site of Township Road 77 termination to allow for residents, school buses, and other vehicles to turn around.

Transportation alignments that could potentially mitigate the impact to Township Road 49 are outside the scope of this Preliminary Design Report. However, past studies considered conceptual alternatives for relocation of Township Road 49. These concepts have previously been developed by Stantec and presented to ODOT for review. Select alternatives were recommended for further evaluation. The next step in the roadway design will likely include a feasibility study and possibly an interchange modification study in coordination with ODOT

10.2 UTILITIES

Existing utilities will be relocated or abandoned where they cross the proposed dam alignment, 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.

11.0 CONSTRUCTION CONSIDERATIONS

The construction permit 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 future design and construction activities. The coordinates and elevations for the control points are listed on Sheets 10 and 11 in Appendix L.

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

Stating is anticipated to occur within the footprint of the project site. Approximately 765 acres of land are within the proposed embankment area with several level areas suitable for large machinery. Area of approximately one acre should be planned for the equipment staging and maintenance area next to the production plant, if necessary.

11.4 DIVERSION OF STREAM FLOW

The selected location of the Principal Spillway allows for the existing Eagle Creek channel to flow its natural course during the majority of construction. Once the proposed channel is built, flows will be diverted to the relocated Eagle Creek channel and through the Principal Spillway. Once construction of the Principal Spillway is complete, a section of the existing Eagle Creek channel will be filled to promote positive drainage away from the dam embankment.

The following is a conceptual sequencing plan for construction:

1. Maintain Eagle Creek flow along its original course during construction of the Principal and Auxiliary Spillway.

2. Construct Principal and Auxiliary Spillways with the exception of labyrinth walls from STA 146+00 to 146+50. This 50-foot gap (approximately the width of the existing Eagle Creek channel) allows for additional conveyance capacity in the event of a flood during construction. Further analysis of the temporary flood conditions will be completed during Final Design.

3. Construct the Eagle Creek Realignment downstream channel (STA 207+94 to 214+55).

4. Construct the entirety of the dam embankment up to the banks of Eagle Creek, leaving the stream open to continue to pass flow.
    a. The slopes of the embankment gap must not exceed 4H:1V slopes to allow for adequate benching of material.
    b. Adequate provisions will be required to protect the filter from contamination and for later tie-in.
5. Construct the Eagle Creek realignment upstream channel from STA 203+50 to 206+75, leaving a wedge of soil in place between Eagle Creek and the realigned channel.
6. Divert flow from the existing Eagle Creek channel to the Eagle Creek realignment channel and through the Principal Spillway.
    a. Complete Eagle Creek realignment channel from STA 200+90 to 203+50.
    b. Place fill in the upstream end of the existing channel to be abandoned to divert flow to the Eagle Creek realignment channel
7. Complete construction of the dam embankment and filling of the channel to be abandoned.
8. Complete combined spillway structure labyrinth walls STA 146+00 to 146+50.

Figure 27 identifies the general construction sequencing assumed for the integrated spillway structure.

11.5 TECHNICAL SPECIFICATIONS

A draft list of technical specifications is included in Appendix M.

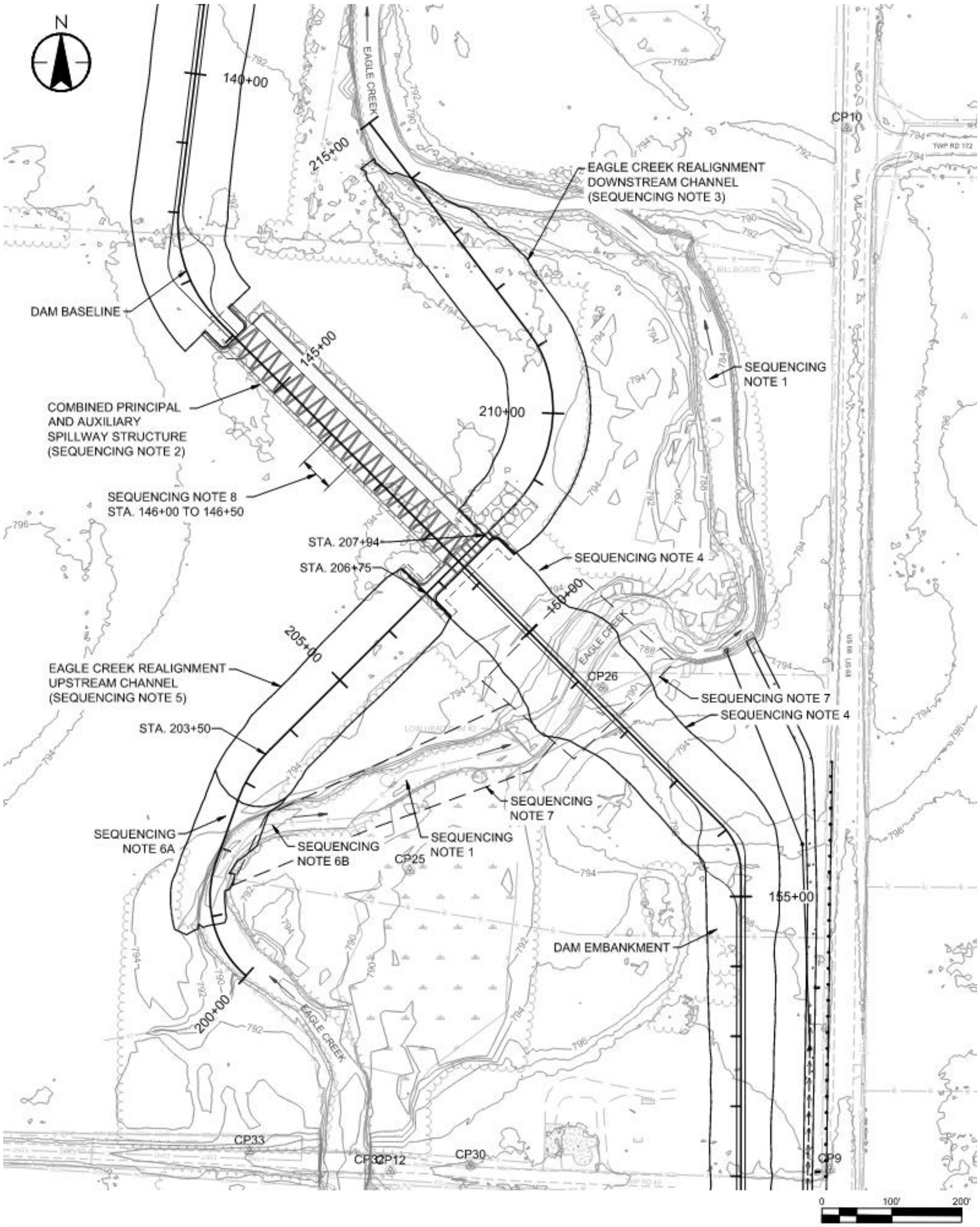

Figure 27. Diversion of Streamflow Sequencing Overview

12.0 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.

12.1 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 design team participated in a virtual pre-application meeting with members of the USACE, OEPA, and MWCD on January 7, 2022. The presentation included discussion on the following information:

- Project background,
- Project's purpose and need,
- Recent project history,
- Overview of alternatives considered,
- Anticipated project schedule,
- Potential direct and indirect impacts associated with the preferred alternative and
- Potential mitigation options.

The Permit Application is currently being developed based on guidance from the respective agencies and is anticipated to be submitted for review to the USACE and OEPA in the second Quarter of 2022.

12.2 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) will be completed by Stantec at the conclusion of Preliminary Design.

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

12.3 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 is the submittal of a preliminary design report to determine the suitability of the proposed site and the classification of the dam. Following approval of the preliminary design report, an application will be submitted with the necessary statutory filing fee and surety bond, final design report, 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.

12.4 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.

12.5 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 summary report associated with this survey, dated August 2021 is included as Exhibit E of Appendix B. 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 reporting associated with a Phase II Archaeological Survey for the recommended additional testing is ongoing and will be submitted to the SHPO upon completion.

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. Additional detail of the Phase I History / Architecture Survey dated September 2021 is in Exhibit F of Appendix B. 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.

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.

13.0 OPINION OF PROBABLE CONSTRUCTION COSTS

An opinion of probable construction costs was developed for the ECFB based on Preliminary Design. To develop this OPCC, Stantec identified quantities of materials in conjunction with a review of unit rates from past, relevant bids, and industry standard cost estimating resources.

Quantities are derived from the design drawings included in Appendix L. 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.

Past, relevant bids and detailed estimates were reviewed to develop unit costs. Prior projects used for unit cost development include but are not limited to the Blanchard River Phase 1 Hydraulic Improvements in Findlay, Ohio (bid in 2018) and Clear Fork Reservoir Dam Modifications in Richland County, Ohio (to be issued for bid in 2022). Estimators also referenced the publicly available Ohio Department of Transportation (ODOT) historical bid database, which was filtered to identify recent (2021, where possible) projects with similar quantities for corresponding line items. The industry standard cost estimating resource used for this OPCC was RSMeans: a construction-industry cost estimating software which evaluates local market conditions to provide a database of benchmarked unit costs.

At this point in Preliminary Design, certain elements of the work have not yet been fully defined and were separated as allowances. The utility relocation allowance accounts for an existing fiber optic line as well as 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, but the work required is still yet to be defined pending utility coordination. The Township Road 49 Relocation will be required to provide continued access to US-68. Coordination with ODOT is ongoing and project design is pending at this time.

Costs are presented in 2022 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 +20%.

13.1 PROJECT CONTINGENCIES

Multiplier markups were applied to estimate costs for certain elements of the work. A contingency of 15% was applied to the base construction cost to account for potential design refinements between Preliminary and Final Design. The contingency may cover unforeseen administrative and legal fees as well as obstacles that may arise throughout detailed design. This contingency should be considered separate from an owner’s contingency used for budgeting purposes that may also address potential uncertainty in market conditions and potential changes during constructions.

13.2 MOBILIZATION, DEMOBILIZATION, AND PREPARATORY WORK

Site mobilization/demobilization was estimated at 5% of base construction cost. Construction phase services were estimated at 5% of total construction cost (including contingency). Additional costs were included for preparatory work such as survey staking and construction layout.

13.3 OPINION OF PROBABLE COSTS

Table 48 summarizes the OPCC for the ECFB project. Some owner's allowances such as land and a portion of the engineer's fees have already been paid for, however, these items are included for reference for comparison to past ECFB project OPCCs. These items do not include an additional factor of uncertainty (-10% to +20%) on the bottom line.

Table 48. Preliminary Opinion of Probable Construction Cost

Description	Cost
Temporary Works, Demolition, and Site Preparation	$2,360,000
Basin Earthwork	$12,330,000
Seepage Mitigation	$2,800,000
Road Modifications and Site Drainage	$1,960,000
Stream, Wetlands, Fish, and Wildlife	$4,320,000
Spillways and Outlet Structures	$8,880,000
Base Construction Cost	**$32,650,000**
Estimate Contingency (15%)	$4,900,000
Total Construction Cost: Single Point Estimate	**$37,550,000**
Lands and Structures	$28,670,000
Utility Relocations	$1,000,000
Township Road 49 Relocation (Alt 1A)	$2,000,000
Engineering, Design, and Permitting, Cultural Resources	$6,500,000
Construction Phase Services	$2,230,000
Owner Project Allowances	**$40,400,000**
Total Project Cost: Single Point Estimate (February 2022 Dollars)	**$77,950,000**
Escalation from Current PL to NTP	$2,630,000
Escalation During Field Construction	$1,500,000
Total Project Cost: Single Point Estimate (With Escalation)	**$82,080,000**
Total Project Cost Range (-10% to +20%)	**$76M - $94.3M**

14.0 REFERENCES

Ohio Department of Natural Resources (ODNR). *Division of Soil and Water Resources – Dam Safety, Section 1501:21*. Ohio Administrative Code (OAC). May 24, 2018. <https://codes.ohio.gov/ohio-administrative-code/1501:21>.

Stantec Consulting Services Inc. *Final Report: Data Review, Gap Analysis, USACE Plan and Alternatives Review, and Program Recommendation (Proof of Concept Report)*. April 3, 2017.

Stantec Consulting Services Inc. *Draft: Proof of Concept Update.* July 9, 2018.

Stantec Consulting Services Inc. *Eagle Creek Dry-Storage Basin Project Alternatives Review*. October 31, 2019.

Stantec Consulting Services Inc. *Hydrologic Evaluation of the Blanchard River*. November 8, 2017.

U.S. Army Corps of Engineers (USACE), Buffalo District. 2015. *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*" (Buffalo, New York. April.

U.S. Army Corps of Engineers (USACE). 2016. Draft *Feasibility Study / Final Environmental Impact Statement*. Interim Report in Response to the Western Lake Erie Basin (WLEB) Blanchard River Watershed Study. Buffalo, New York. March.

U.S. Army Corps of Engineers (USACE). 2021. *HEC-HMS*. Hydrologic Engineering Center, Institute for Water Resources, Version 4.8. Davis, California.

U.S. Army Corps of Engineers (USACE). 2021. *HEC-RAS*. Hydrologic Engineering Center, Institute for Water Resources, Version 6.0. Davis, California.

U.S. Army Corps of Engineers (USACE) and U.S. Environmental Protection Agency (USEPA). 2020. *The Navigable Waters Protection Rule: Definition of "Waters of the United States."* Federal Register, Vol. 85, No. 77: Rules and Regulations. April 21, 2020.

U.S. Environmental Protection Agency (USEPA). 2015. *Draft Environmental Impact Study Statement (CEQ# 20150102)*. Blanchard River Watershed Study, City of Findlay, Hancock County, Ohio. May.

APPENDIX A

Design Criteria Document

APPENDIX B

Field Studies

APPENDIX C

Geomorphic Assessment Report

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

APPENDIX K

Interior Wetland Design Report

APPENDIX L1

Preliminary Design Drawings

APPENDIX L2

Project Vicinity Figure

APPENDIX M

Draft Technical Specification List

APPENDIX N

Aquatic Resource Connectivity Review