BASF Corporation

DRAFT
Basis of Design Report
Pre-Final 95% Design
Perimeter Barrier Remedy

BASF North Works Site
Wyandotte, Michigan

August 2025

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Acronyms and Abbreviations

ºF	degree Fahrenheit
%	percent
A	ampere
AASHTO	American Association of State Highway and Transportation Officials
ACI	American Concrete Institute
ADA	Americans with Disabilities Act
AG	above grade
AHU	air handling unit
AOC	area of concern
Arcadis	Arcadis of Michigan, LLC
ASCE	American Society of Civil Engineers
BASF	BASF Corporation
bgs	below ground surface
BOD	basis of design
CAO	corrective action objective
CAZ	critical assessment zone
CCR	Current Conditions Report
cm/sec	centimeters per second
COC	constituent of concern
Consent Order	1994 Administrative Order on Consent (Docket No. V-W-011-94)
CPT	cone penetrometer test
CSM	conceptual site model
DUWA	Downriver Utility Wastewater Authority
E-Stop	emergency stop
EBCT	empty bed contact time
EGLE	Michigan Department of Environment, Great Lakes, and Energy
GAC	granular-activated carbon
gpm	gallons per minute
GSI	groundwater-surface water interface
H&S	health and safety
H:V	horizontal unit to vertical unit

HASP	health and safety plan
HAZWOPER	hazardous waste operations and emergency response
HCV	human cancer value
HDPE	high-density polyethylene
HMI	human machine interface
HNV	human noncancer value
HPT	hydraulic profiling tool
HVAC	heating, ventilation, and air conditioning
I/O	input/output
IGLD 55	International Great Lakes Datum of 1955
IGLD 85	International Great Lakes Datum of 1985
JPA	Joint Permit Application
JSA	job safety analysis
KV	kilovolt
LCC	lightweight cellular concrete
LED	light-emitting diode
MASW	multichannel analysis of surface waves
MCP	modular main control panel
MDNR	Michigan Department of Natural Resources
µg/L	micrograms per liter
µm	micron
mg/L	milligrams per liter
MI	Michigan
MIOSHA	Michigan Occupational Safety and Health Administration
N/A	not applicable
NFPA	National Fire Protection Association
ng/L	nanograms per liter
NPDES	National Pollutant Discharge Elimination System
NREPA	Natural Resources and Environmental Protection Act
OMM	operation, maintenance, and monitoring
OSHA	Occupational Safety and Health Administration
pcf	pounds per cubic foot
PCI	Precast/Prestressed Concrete Institute

PDI	pre-design investigation
PEMB	pre-engineered metal building
PFAS	per- and polyfluoroalkyl substances
PFOA	perfluorooctanoic acid
PFOS	perfluorooctanesulfonic acid
PLC	programmable logic controller
POTW	publicly owned treatment works
PPE	personal protective equipment
psf	pounds per square foot
psi	pounds per square inch
PZ	piezometer
QAPP	Quality Assurance Project Plan
QST	QST Environmental
RCP	remote control panel
RCRA	Resource Conservation and Recovery Act of 1976
RD	remedial design
RFI	Resource Conservation and Recovery Act Facility Investigation
RPM	revolution per minute
RSSCT	rapid small-scale column test
S.U.	standard units
SCADA	supervisory control and data acquisition
scfm	standard cubic feet per minute
SESC	soil erosion and sediment control
Site	BASF Corporation North Works Site in Wyandotte, Michigan
SPT	standard penetration test
SVOC	semi-volatile organic compound
SWMU	solid waste management unit
TOC	total organic carbon
UL	Underwriter's Laboratory
USACE	United States Army Corps of Engineers
USC	United States Code
USEPA	United States Environmental Protection Agency
USFWS	United States Fish and Wildlife Service

V	volt
VAC	volt-alternating-current
VE	value engineering
VFD	variable frequency drive
VOC	volatile organic compound
Woodward-Clyde	Woodward-Clyde Group
WRD	Water Resource Division

1 Introduction

On behalf of BASF Corporation (BASF), Arcadis of Michigan, LLC (Arcadis) has prepared this Basis of Design (BOD) Report for a groundwater remedy at the BASF North Works site in Wyandotte, Michigan (the Site). This BOD Report describes the Pre-Final (95%) Design of the remedy, which consists of a physical containment barrier and a groundwater collection and above-grade (AG) treatment system to manage/mitigate off-site migration of groundwater at the downgradient site perimeter. The physical containment barrier will block groundwater from entering the Detroit River. The groundwater collection system is necessary to prevent groundwater elevations from rising when the containment barrier is installed. The groundwater collection system will be operated to lower the groundwater table and further reduce the potential for migration and will consist of drains, sumps, and a conveyance network that will be constructed to extract site groundwater. Extracted groundwater will be treated before being discharged to the local publicly owned treatment works (POTW), the Downriver Utility Wastewater Authority (DUWA).

This document is being submitted as part of a Pre-Final (95%) Design as requested by the United States Environmental Protection Agency (USEPA) Region 5 for a site-wide downgradient perimeter groundwater remedy.

This BOD Report was developed consistent with applicable USEPA guidance, including:

- Guidance for Scoping the Remedial Design (USEPA 1995a);
- Remedial Design/Remedial Action Handbook (USEPA 1995b);
- EPA Oversight of Remedial Designs and Remedial Actions Performed by Potentially Responsible Parties, Interim Final (USEPA 1990); and
- Handbook on the Benefits, Costs, and Impacts of Land Cleanup and Reuse (USEPA 2011).

2 Site Description and Background

The Site is located on the west bank of the Detroit River in Wyandotte, Wayne County, Michigan, and occupies approximately 230 acres (Figure 1). The Site is bounded by Perry Place to the north, the Detroit River to the east, James DeSana Drive to the south, and Biddle Avenue to the west. The Site is an active industrial property used for manufacturing chemicals and other products. Facility operations and workforce have expanded since issuance of the 1994 Administrative Order on Consent (Docket No. V-W-011-94) (Consent Order) and continue to grow.

At present, approximately 50% of the Site is developed with buildings, paved streets, parking lots, tank farms, and docks. Many of the former site features associated with discontinued processes have been demolished, although concrete surfaces and foundations at or below grade remain. The site boundary with the river is approximately 5,750 feet long and consists of the following shoreline perimeter structures and features:

- A pier structure and a timber Wakefield wall constructed in the early 1900s extend approximately 4,920 feet south from the northern property boundary at Perry Place (Figure 2). The pier structure consists of a soil-covered concrete deck supported on timber piles. The Wakefield wall serves as a bulkhead wall between the upland area and the pier structure and consists of oak sheets with tongue-and-groove joints keyed into the river bottom. The width of the soil-covered concrete deck increases from approximately 3 feet at the northern end of the Site to approximately 34 feet at the southern end of the pier (Arcadis 2019b).

- An anchored steel bulkhead was installed in the 1990s on the riverside of the existing pier structure/Wakefield wall described above (Figure 2). Composed of interlocking steel sheet piles, this bulkhead extends approximately 3,243 feet south from the northern site boundary (i.e., the northern end of the existing pier structure) and was installed as part of a shoreline stabilization project.
- A pile-supported concrete dock (South Dock) extends approximately 1,620 feet south from the anchored steel bulkhead to the rip-rap-protected portion of the riverbank.
- The riverbank south of the existing pier structure is protected by an approximately 830-foot-long rip-rap revetment (Figure 2).

Detailed descriptions of the site physical and hydrogeological settings and distribution of constituents of concern (COCs) are provided in the Resource Conservation and Recovery Act (RCRA) Facility Investigation (RFI) Report (QST Environmental [QST] 1999), the RFI Current Conditions Report (CCR) (Woodward-Clyde Group [Woodward-Clyde] 1994), and the Draft Perimeter Conceptual Site Model (CSM) (Appendix A).

2.1 Site History

A comprehensive description of the site history is presented in the RCRA RFI Report (QST 1999) and the CCR (Woodward-Clyde 1994). Various industrial operations have been implemented at the Site since the late 1800s, and the RFI Report identified several areas of concern (AOCs) and solid waste management units (SWMUs) requiring corrective measures. Detailed information on each AOC/SWMU is provided in the RFI Report (QST 1999) and the CCR (Woodward-Clyde 1994). In addition, land reclamation activities conducted at the Site have led to the widespread distribution of fill material containing site-related constituents. Constituents present in groundwater as a result of past site-related activities include volatile organic compounds (VOCs), semi-volatile organic compounds (SVOCs), and metals (including mercury). Elevated metal concentrations in groundwater are driven by the site geochemistry (e.g., strongly reducing conditions resulting from the presence of organic matter and elevated pH levels associated with fill material in the subsurface). Due to the extent of groundwater impacts at the Site, BASF proposed to address groundwater as a site-wide AOC, managed at the downgradient site perimeter. USEPA concurred with this approach in correspondence to BASF dated August 26, 2016 (USEPA 2016).

2.2 Conceptual Site Model Overview

The hydrogeologic setting at the Site is complex with fill material and built land overlying native fluvial channel deposits and the underlying clay aquitard. The random nature of the fill material, lengthy site history, current site operations, multiple source areas, varying shoreline structures, and implementation of several groundwater recovery systems have resulted in tortuous groundwater flow pathways and contaminant distribution that reaches the site perimeter within more permeable pathways. Based on the historical investigations and recent characterization of the site perimeter and shoreline, the following conclusions are provided:

- There are currently twelve AOCs and eight SWMUs identified at the Site. The source areas at the Site are widely scattered and generally related to former site operations, discharge of waste material to the ground, landfilling, and chemical spills associated with site operations.
- Concentrations in groundwater exceeding groundwater-surface water interface (GSI) criteria are present along the downgradient perimeter monitoring wells. These impacts originated from the upgradient identified

AOCs/SWMUs and have been distributed along the site perimeter by tortuous groundwater flow with higher conductivity zones. As a result of the hydraulic barrier created by the existing steel bulkhead (Arcadis 2021), most of the groundwater flux occurs within higher-conductivity zones where the Wakefield wall and rip rap are present. The highest concentrations in groundwater are observed along the southern half of the eastern shoreline south of the existing steel bulkhead. To a lesser degree, groundwater flux occurs along the northern and southern boundaries. The proposed perimeter barrier remedy encompasses the northern boundary, the entire eastern shoreline, and the southern boundary where groundwater concentrations exceed GSI criteria and the potential exists for groundwater flux to the river.

- Much of the perimeter is composed of lower-conductivity materials consistent with silty sands and silt. Localized zones of higher-conductivity materials are found sporadically along the perimeter and have the potential for increased groundwater flux and contaminant transport. A high-resolution permeability profile was completed along the downgradient site perimeter using the Geoprobe® hydraulic profiling tool (HPT). Groundwater discharge to the river occurs primarily within higher-permeability zones that comprise a small percentage of the overall aquifer.
- Generally, groundwater flow is east toward the river within the fill and fluvial sand units. In the northern half of the Site, where the steel bulkhead wall is present, groundwater mounds behind the wall and is deflected to the north and south. Pumping tests were completed at three high-permeability zones identified during the HPT investigation. The results of the pumping tests confirmed a hydraulic no-flow boundary adjacent to the steel pile wall, a recharge boundary adjacent to the South Dock/Wakefield wall, and high hydraulic connectivity to the Detroit River where the rip rap is present at the shoreline. The pumping test data along with a complete breakdown of the analysis of the pumping test results is provided in the Draft Hydraulic Pre-Design Investigation Report submitted to USEPA on August 27, 2021 (Arcadis 2021).
- A comparison of the perimeter well Mann-Kendall stability analysis results to concentrations tested in the treatability study (which was used as the basis of design of the AG treatment system) shows that the AG treatment system has been designed to handle the current and potential future perfluorooctanesulfonic acid (PFOS) and mercury concentrations that may be recovered by the perimeter barrier remedy.

The pre-design investigations (PDIs) are described in the following section. The CSM is described in more detail in Appendix A.

2.3 Pre-Design Investigations

Subsurface conditions in the upland and near-shore areas of the Site have been investigated extensively. The following PDIs were conducted in 2020 to support the design of the containment barrier:

- Hydraulic PDI, which included a transducer evaluation, HPT investigation, groundwater pumping tests, and calibration/update of the groundwater model. Results of the hydraulic PDI are summarized in the BASF North Works Hydraulic Pre-Design Investigation Report (Arcadis 2021). The groundwater model was further updated during the Intermediate (60%) Design phase to include transient conditions known to exist on the Detroit River and then refined again for the Pre-Final (95%) Design phase (Appendix B).
- Geotechnical PDI, which included 17 geotechnical borings and 17 cone penetrometer test (CPT) explorations in the upland and near-shore areas along the proposed barrier alignments in the South Dock area and along the rip-rap-protected riverbank south of the South Dock. Results of the PDI are summarized in the BASF North Works Geotechnical Data Report included in Appendix C.

In 2021, Arcadis performed a bench-scale treatability study in support of a previously proposed soil-cement wall design. This treatability study evaluated the efficacy of soil/cement and soil/bentonite mix formulations using both native site soil and a clean borrow soil source. The testing procedures and findings of the study are summarized in the treatability study report (Arcadis 2022a).

In 2022, Arcadis performed a treatability study to support the design of the AG treatment system using groundwater collected from select site monitoring wells and blended to represent anticipated influent groundwater characteristics. Results of the study are summarized in the treatability study report (Arcadis 2022b).

During the Preliminary (30%) and Intermediate (60%) Design phases, several data gaps were identified as discussed in the Draft Basis of Design Report for the Preliminary 30% Design Perimeter Barrier Remedy (Arcadis 2022c) and the Draft Basis of Design Report for the Intermediate 60% Design Perimeter Barrier Remedy (Arcadis 2024a). To address these data gaps, the following assessments were performed from 2023 through 2025 in support of the Pre-Final (95%) Design:

- A diver survey was conducted in August and September 2023 to evaluate the condition of the existing bulkhead. Divers visually inspected the existing bulkhead and collected ultrasonic thickness readings every 50 feet along the steel bulkhead at the waterline, mid-water, and near the bottom of the exposed sheeting (i.e., above the sediment). Findings from the diver survey are provided in Appendix D and have been incorporated into the evaluation for the existing bulkhead.
- A geophysical investigation was conducted in August and September 2023 to identify subsurface obstructions and utilities, identify void space behind the existing steel bulkhead, and determine the depth of the basal clay along the alignments of the subsurface barriers and the bulkhead anchor wall alignment. Approximately 130 miles of geophysical surveys were completed using a combination of ground-penetrating radar, electromagnetic, and seismic technologies. Results of the geophysical investigation are summarized in Appendix E and have been used in the design to realign the remedy components around subsurface obstructions and utilities to the extent possible.
- An investigation of the South Dock was conducted in September 2023 to determine the soil thickness on the concrete dock, the top-of-concrete elevation for the soil-covered deck, and the top-of-concrete elevation for the concrete bulkhead at the face of the dock and to facilitate calibration of the geophysical survey results. The investigation included 17 test pits aligned in five transects perpendicular to the Detroit River. Findings associated with the South Dock test pits are provided in Appendix F and have been incorporated into the design elements for the new bulkhead.
- A geotechnical soil boring investigation was conducted in November 2023 and January 2024 to confirm the top of the clay layer, to collect subsurface data for installation of barrier components, and to aid in the calibration of the geophysical survey results. Eight standard penetration test (SPT) borings were completed within Perry Place and on the southern perimeter of the Site along James DeSana Drive. Soil boring logs from the investigation are included in Appendix C.
- A utility assessment was conducted along the proposed remedy alignments to document and obtain an understanding of historical and active utilities. Utilities were evaluated via geophysical surveys and in coordination with Wyandotte Municipal Services and BASF. Known utility information is included on the Pre-Final (95%) Design Drawings (Appendix G).
- A resin pre-design study (field pilot test) is being conducted to investigate ion exchange per- and polyfluoroalkyl substances (PFAS) treatment interferences. The field pilot test began in July 2023 and is

ongoing. As of July 2025, approximately 7 million gallons of groundwater has been extracted, treated, and discharged to DUWA in accordance with permit requirements.

- As discussed in Section 6.6, the location of the AG treatment system has been changed from that proposed in the Intermediate (60%) Design to a new building centrally located along the perimeter of the Site. To support the foundation design for the Water Treatment Building and storage tanks, a geotechnical assessment was completed in May and June 2025 in accordance with the New Building Geotechnical Investigation Work Plan Memorandum (Arcadis 2025a) approved by USEPA. A summary of the investigation and the results are included in Section 6.7.3 and in Appendix C.
- During the Intermediate (60%) Design phase, it was determined that an assessment of the potential for voids along the existing bulkhead, specifically beneath the former Heavy Dock section, was warranted. An investigation program was completed to assess the presence of voids beneath the concrete structure of the seawall in October and November 2024 in accordance with the Heavy Dock Void Space Investigation Memorandum (Arcadis 2024b) approved by USEPA. The program included a combination of soil borings advanced through the concrete to the native clay and geophysical surveys to obtain further information regarding the subsurface conditions beneath the dock. Findings of the investigation are summarized in Section 4.4.1.2 and in Appendix H.
- A limited soil investigation was conducted at the southern boundary of the Site in response to USEPA comments on the Intermediate (60%) Design. The objective of the investigation was to refine the proposed sheet pile embedment depths, if necessary, and verify the basal lacustrine clay properties in the area of the southern property boundary. The investigation was completed in June 2025 in accordance with the Limited Investigation Memorandum (Arcadis 2025b) approved by USEPA. The investigation and available results are summarized in Section 4.3.1.3 and in Appendix I.
- The need for additional evaluations of the soil-cement design mix to address potential freeze-thaw issues and compositional changes to Portland cement was also identified as a potential data gap during the Preliminary (30%) Design phase. However, because the soil-cement slurry barrier wall type is no longer proposed, additional evaluations of the soil-cement design mix are no longer needed; instead, all subsurface barriers are proposed as steel sheet pile. The basis for selecting sheet pile rather than soil-cement slurry is discussed in Section 4.2.

3 Perimeter Barrier Remedy Basis

Between June 2015 and September 2017, BASF submitted various documents proposing remedy options to manage/mitigate potential off-site migration of groundwater at the downgradient site perimeter to adjacent properties and the Detroit River. On April 24, 2018, USEPA provided a letter to BASF outlining the following key requirements for a downgradient perimeter remedy (USEPA 2018a):

- USEPA requires physical stabilization of the Site and a downgradient perimeter barrier to contain groundwater for treatment prior to off-site discharge.
- Site stabilization, a containment barrier, and on-site groundwater treatment are essential for integrating the RCRA corrective action with the Great Lakes Legacy Act project.
- Any proposed remedy must include plans to address sediment under the overhanging dock and to physically stabilize the Site to prevent erosion of fill from the facility to the river. USEPA suggested demolition of the dock and shoreline reconstruction as an optimal alternative for physical stabilization.

In 2018, BASF and USEPA agreed upon a perimeter barrier remedy, including groundwater extraction and on-site groundwater treatment, to facilitate site stabilization, containment, and treatment as requested by USEPA in its April 24, 2018, letter.

On June 5, 2018, USEPA issued a letter indicating concurrence with BASF's proposal to submit a work plan for completion of a Preliminary (30%) Design for a site-wide downgradient perimeter groundwater remedy (USEPA 2018b). BASF's proposal is documented in its May 29, 2018, letter to USEPA (BASF 2018). USEPA was acting under the authority of the Consent Order, which requires that BASF perform corrective action at the Site pursuant to RCRA of 1976 as amended by the Hazardous and Solid Waste Amendments of 1984. A Remedial Design (RD) Work Plan was submitted in May 2019 (Arcadis 2019a) and provided the framework for design of the perimeter barrier remedy. The RD Work Plan (Arcadis 2020) was approved by USEPA and was kicked off following an in-person meeting between USEPA and BASF in February 2020.

A Draft Preliminary (30%) Design was submitted to USEPA in September 2022. In response, USEPA provided a Comprehensive Interim Measure Remedy Selection Letter on May 25, 2023 (USEPA 2023), agreeing to the comprehensive groundwater interim measure proposed by BASF. As an attachment to the Selection Letter, USEPA provided Comments on the Preliminary (30%) Design. A summary of these comments, along with a response to each, is provided in Appendix J.

An Intermediate (60%) Design was submitted to USEPA in March 2024. In response, USEPA provided a comment letter on November 8, 2024 (USEPA 2024). BASF and USEPA worked through responses to these comments over a series of meetings between November 2024 and March 2025, in parallel with the development of the Pre-Final (95%) Design. A summary of these comments, along with the USEPA approved responses and responses pending USEPA approval, is provided in Appendix J.

3.1 Design Criteria

In lieu of preparing a separate Design Criteria Report, the components of a Design Criteria Report as outlined in USEPA's Remedial Design/Remedial Action Handbook (USEPA 1995b) are included in the sections of this BOD Report as referenced in Table 1 below.

Table 1. Design Criteria Components

Required Design Component	Report Section(s)
Corrective action objectives and performance standards	Sections 3.3 and 3.4
Compliance with pertinent regulatory requirements	Section 3.5
Technical factors of importance	Sections 4.3, 5.1.2, 13.1 and Appendix M
Volume and type of each medium requiring treatment	Sections 2.2, 6.3, and 7
Treatment schemes	Sections 6.4 and 6.5
Waste management and characterization requirements	Section 7 and Appendix R
Operation, maintenance, and monitoring requirements	Section 12 and Appendix K

3.2 Proposed Remedy Description

Consistent with the Preliminary (30%) Design and Intermediate (60%) Design, this Pre-Final (95%) Design includes a physical downgradient perimeter containment barrier with a pump and treat system.

The remedy includes a combination of subsurface and shoreline barriers, along with a series of groundwater collection drains (supplemented with a vertical extraction well) and an AG treatment system that discharges treated groundwater to the local POTW. The groundwater collection drains will operate to manage groundwater upland of the containment barrier.

The physical barrier option considered and the basis for barrier selection are detailed in Section 4. The groundwater extraction and conveyance system components of the barrier remedy are described in Section 5, and the AG treatment system design is discussed in Section 6.

3.3 Corrective Action Objectives

Corrective action objectives (CAOs) were developed in coordination with USEPA and the Michigan Department of Environment, Great Lakes, and Energy (EGLE) concurrently with the design. BASF, USEPA, and EGLE had several meetings in June 2023 to discuss the CAOs, which were submitted to USEPA on July 24, 2023. The CAOs are included in Table 2.

As required, the CAOs specify the following:

- COCs;
- Exposure route(s) and receptor(s); and
- Acceptable contaminant level or range of levels for each exposure route.

The CAOs were developed based on the following:

- USEPA and EGLE law, policy, and guidance;
- Threshold criteria: protect human health and the environment, achieve media cleanup objectives, and control sources;
- CSM;
- Current uses and exposures;
- Reasonably expected future uses and exposures; and
- Resource values (ecological, groundwater, etc.).

Note that development of the CAOs took into consideration the critical assessment zone (CAZ) associated with the City of Wyandotte drinking water intake. The perimeter barrier extends across the CAZ where it intersects with the shoreline of the Site. The portion of the CAZ on BASF property is isolated by the barrier; therefore, where the CAZ intersects the shoreline, requirements of EGLE Water Resource Division (WRD) policy WRD-053 will be met. Compliance with the WRD-053 policy will be shown by meeting the performance standards discussed in Section 3.4.

Table 2. Perimeter Barrier Remedy Corrective Action Objectives

Environmental Media	Human Health Residential	Human Health Non-Residential	Ecological Receptors	Cross-Media Transfer	Resource Restoration
Groundwater (On Site)	N/A. The perimeter barrier remedy does not mitigate exposure to groundwater on Site. The health and safety of workers during the construction of the remedy will be covered by MIOSHA/OSHA HAZWOPER and will be part of the Health and Safety Plan for the project.	N/A. The perimeter barrier remedy does not mitigate exposure to groundwater on Site. The health and safety of workers during the construction of the remedy will be covered by MIOSHA/OSHA HAZWOPER and will be part of the Health and Safety Plan for the project.	N/A. No ecological habitat identified on Site.	N/A	N/A
Groundwater (Off Site)	Prevent future human drinking water exposure to site-related COCs in groundwater above federal maximum contaminant levels, MI Rule 57 criteria for drinking water (HCV and HNV), and MI Part 201 criteria by mitigating off-site migration of groundwater; maintain groundwater elevation.	Prevent future human drinking water exposure to site-related COCs in groundwater above federal maximum contaminant levels, MI Rule 57 criteria for drinking water (HCV and HNV), and MI Part 201 criteria by mitigating off-site migration of groundwater; maintain groundwater elevation.	Mitigate future exposure to groundwater exceeding Part 201 GSI and MI Rule 57 criteria by preventing migration of groundwater to the Detroit River; maintain groundwater elevation.	Cross-media transfer will be addressed through the Human Health and Ecological CAOs.	N/A
Soil/Sediment (On Site)	N/A. The perimeter barrier remedy does not mitigate exposure to on-site soil. The health and safety of workers during construction of the remedy will be covered by MIOSHA/OSHA HAZWOPER and will be part of the Health and Safety Plan for the project.	N/A. The perimeter barrier remedy does not mitigate exposure to on-site soil. The health and safety of workers during construction of the remedy will be covered by MIOSHA/OSHA HAZWOPER and will be part of the Health and Safety Plan for the project.	N/A. No ecological habitat identified on Site.	N/A	N/A
Soil/Sediment (Off Site)	N/A. The perimeter barrier remedy does not address off-site soils.	N/A. The perimeter barrier remedy does not address off-site soils.	Prevent future exposure to site soils to ecological receptors in the Detroit River.	N/A	N/A. Upper Trenton Channel Great Lakes Legacy Act sediment dredge project is being completed for resource restoration.
Surface Water	Prevent future discharge of groundwater to the Detroit River and the CAZ in the Detroit River exceeding MI Rule 57 for drinking water, both cancer and noncancer values, MI Part 201 GSI, and MI Part 4.	Prevent future discharge of groundwater to the Detroit River and the CAZ in the Detroit River exceeding MI Rule 57 for drinking water, both cancer and noncancer values, MI Part 201 GSI, and MI Part 4.	Mitigate future exposure to groundwater exceeding Part 201 GSI and MI Rule 57 criteria by preventing migration of groundwater to the Detroit River; maintain groundwater elevation.	Cross-media transfer will be addressed through the Human Health and Ecological CAOs.	N/A

Notes:
HAZWOPER = hazardous waste operations and emergency response
HCV = human cancer value
HNV = human noncancer value
MI = Michigan
MIOSHA = Michigan Occupational Safety and Health Administration
N/A = not applicable
OSHA = Occupational Safety and Health Administration

3.4 Performance Standards

In consultation with USEPA and EGLE, BASF developed performance standards concurrently with design development. For the purposes of the Pre-Final (95%) Design, it has been agreed upon by USEPA, EGLE, and BASF during monthly meetings that the performance standards, with specific metrics, will address the following key elements:

- The physical containment barrier will contain groundwater and mitigate the potential for groundwater to enter the Detroit River. Performance standards assign appropriate value to the presence of the physical containment barrier.
- The groundwater collection system will prevent groundwater elevations from rising upland of the containment barrier. Performance standards will incorporate an inward hydraulic gradient component that is protective of the Detroit River in combination with the physical containment barrier.
- Performance standards will be adaptable to future site conditions and manage risk over the lifetime operation of the barrier remedy.

As summarized in the CSM Overview (Section 2.2), based on the nature and extent of the impacts at the Site, BASF and USEPA agreed to manage site groundwater at the perimeter of the Site. This approach was agreed upon because it is not feasible to address source areas and meet the CAOs while the Site is an operational chemical manufacturing facility. As such, the perimeter barrier remedy is intended to contain and prevent off-site migration of site soil and groundwater and has not been designed to reduce concentrations of COCs in soil or groundwater to below relevant criteria. Therefore, the performance standards presented herein do not include metrics that dictate when the perimeter barrier remedy can be shut down. If the use of the Site changes in the future and the sources are adequately assessed and addressed, performance standards associated with these future actions will include specific metrics for the shutdown of the perimeter barrier remedy, if appropriate.

An Operation, Maintenance, and Monitoring (OMM) plan outlining procedures for successfully operating the perimeter barrier system to meet CAOs and associated performance standards will be in place. Details of the OMM plan are included in the OMM Manual in Appendix K. Generally, the CAOs specify that the perimeter barrier remedy must prevent future off-site exposure to site soil and groundwater. This section details the performance standards to achieve these CAOs.

3.4.1 Soil/Sediment

The only CAO related to soil states that the perimeter barrier remedy will prevent future exposure to ecological receptors in the Detroit River. As described further in this BOD Report, the perimeter barrier remedy includes physical barriers along the entire eastern shoreline, consisting of new and existing steel sheet pile. The barriers are designed to structurally support site soils, considering the site-specific conditions at the shoreline based on geotechnical, geophysical, and diver inspection data collected to support the remedy design. The presence of the physical sheet pile barriers will prevent site soils from entering the Detroit River. An OMM plan will be in place that provides for routine diver inspections to identify and address any issues that could result in loss of site soil through the barriers in the future.

3.4.2 Groundwater and Surface Water

The CAOs related to groundwater and surface water are generally to prevent future human and ecological off-site exposure to site groundwater. The perimeter barrier remedy will achieve these CAOs via a physical barrier that stops discharge of groundwater to the Detroit River and through an inward hydraulic gradient as measured between the river and the groundwater collection drain. As described further in this BOD Report, the remedy includes new and existing steel sheet pile barriers along the entire eastern shoreline.

Several key factors were considered to develop performance standards for the inward hydraulic gradient component of the remedy. The Detroit River elevation is dynamic and can fluctuate by 2 to 3 feet over a short time frame (i.e., within 24 hours). These short-term fluctuations (i.e., seiche events) are unpredictable; however, the long-term elevation trends of the Detroit River are generally cyclical and consistent. It is important to recognize that it is not practical for the perimeter barrier remedy drain elevation to track with the Detroit River instantaneously. Therefore, it is important that standards are achievable and protective. Key considerations are as follows:

- Use average river levels, not instantaneous river levels, that allow for maintaining an inward gradient relative to overall river elevation trends but eliminate chasing major short-term fluctuations in the river (i.e., seiche events).
- Average river elevations using recent past data (not future data) to calculate a compliance elevation. Using future data in the averaging calculation makes it impossible to determine the compliance target or whether compliance is maintained on any given day.
- Select a time frame over which to average river levels that results in achievable changes in gradient day to day, but is still representative of the current conditions.
- Set a compliance gradient requirement (i.e., distance below the calculated river average) that is achievable and also protective of the current conditions.
- Recognize that if there are brief time frames of outward gradients (due to seiche events), the barriers reduce the potential flux to the river to de minimis levels.

The performance standards for the inward hydraulic gradient, described in the following sections, were developed considering these key considerations.

3.4.2.1 Proposed Approach for Inward Gradient

The proposed approach for determining the required compliance drain elevation is to calculate an average river elevation and then subtract a compliance gradient requirement. The measured drain elevation must be at or lower than this calculated drain compliance elevation to achieve the performance standard metric. With this approach, the measured drain elevation will not be compared to the instantaneous river elevation for compliance purposes. Historical data can be used to assess the key input variables (i.e., time over which to average river data and the compliance gradient requirement) to confirm that this method of compliance is adequately protective of the instantaneous condition.

Ten years of daily average and hourly average Detroit River elevation data from 2013 to 2023 for Wyandotte, Michigan[1] were evaluated to assess various time frames for averaging Detroit River elevations (e.g., previous one day, previous one week, previous one month). In addition, various compliance gradient requirements were

[1] https://tidesandcurrents.noaa.gov/waterlevels.html?id=9044030

evaluated (0.1 foot, 0.25 foot, 0.3 foot, and 0.5 foot). These two variables (elevation data and gradient requirements) were assessed in combination to calculate hypothetical compliance drain elevations over the past 10 years. The hypothetical compliance drain elevations were then compared to the daily average river elevations to assess how often an inward gradient is maintained. Although this comparison would not take place for the proposed approach, it is important for development of the performance standards to evaluate the overall protectiveness of the strategy. The maximum daily decrease was also evaluated. The maximum daily decrease is the hypothetical maximum daily reduction in drain elevation that would be required to maintain compliance over the 10 years of data that were assessed. The maximum daily decrease increases when the averaging time frames are shorter and decreases when the averaging time frames are longer. Evaluation of the averaging time is important for assessing whether the daily decrease is achievable along 5,140 feet of drains along the site perimeter and is summarized in Table 3.

Table 3. Potential Compliance Scenarios

Time Frame	0.1-Foot Gradient		0.25-Foot Gradient		0.3-Foot Gradient		0.5-Foot Gradient	
	% of Time with Inward Gradient	Maximum Daily Decrease	% of Time with Inward Gradient	Maximum Daily Decrease	% of Time with Inward Gradient	Maximum Daily Decrease	% of Time with Inward Gradient	Maximum Daily Decrease
1-Month River Average	69%	0.06 foot	86%	0.06 foot	90%	0.06 foot	96%	0.06 foot
2-Week River Average	74%	0.16 foot	90%	0.16 foot	92%	0.16 foot	97%	0.16 foot
1-Week River Average	76%	0.25 foot	91%	0.25 foot	93%	0.25 foot	98%	0.25 foot
1-Day River Average	76%	1.99 feet	92%	1.99 feet	94%	1.99 feet	98%	1.99 feet

The 0.1-foot gradient condition was eliminated because the percentage of time an inward gradient is maintained is too low, as is the 0.25-foot gradient and 1-month of river averaging scenario, which was also eliminated. The 1-day river averaging condition was eliminated because it would not be achievable to pull down the drain elevations to meet a daily decrease of almost 2 feet. The remaining eight conditions result in maintaining an inward gradient threshold of 90% or more and a maximum daily change of 0.25 foot or less. In general, longer averaging time frames are preferred because the potential maximum daily decreases are less and are more likely to be achievable along the 5,140 feet of perimeter drains.

3.4.2.1.1 Lines of Evidence to Support Inward Gradient Thresholds

This section presents lines of evidence to support that an inward gradient threshold that is at least 90% and less than 100% is protective, is achievable, and results in a system design that is reasonable and practical to operate.

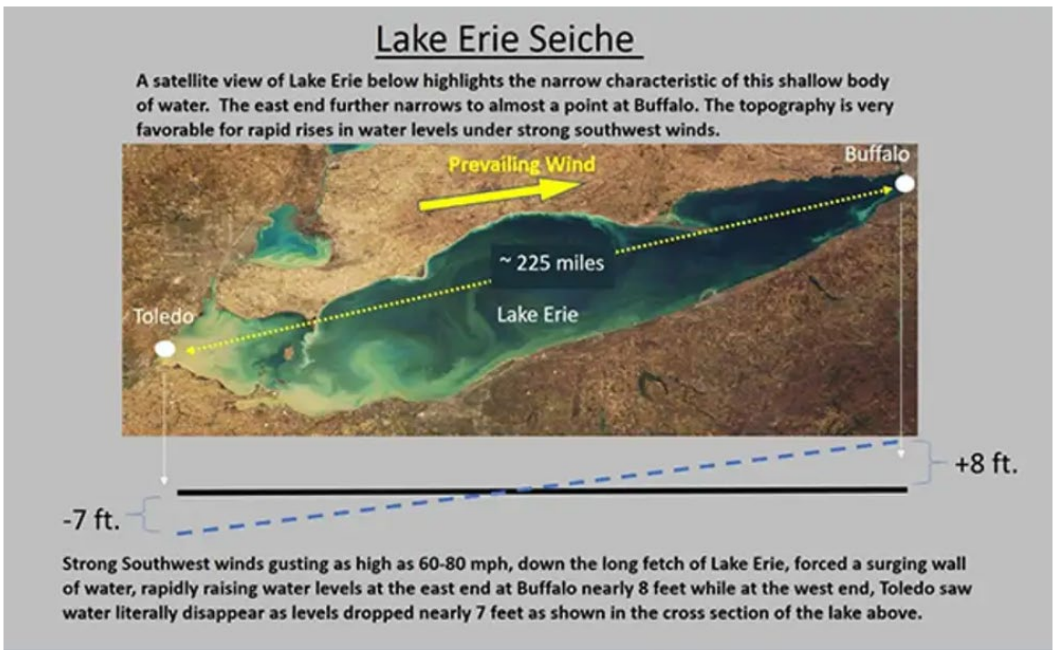
Presence of a Physical Barrier

The perimeter barrier remedy includes a physical barrier along the eastern shoreline, as well as the northern and southern property lines,, consisting of new and existing steel sheet pile. The containment barrier has been designed to prevent groundwater discharge to the river and off Site. If there are brief time frames of outward gradients due to seiche events, the barrier will reduce the potential flux to the river to de minimis levels.

Seiche Intensity and Frequency

A seiche is caused when strong winds or atmospheric pressure changes push water from one end of a body of water to another. Seiches can be extreme on Lake Erie and surface water bodies connected to Lake Erie (i.e., the Detroit River) due to the location, orientation, shape, and bathymetry (Exhibit 1). The quick and drastic changes in water levels associated with seiche events on the Detroit River make it technically impractical for the performance standard compliance approach to be based on the instantaneous river level. It is not practical to lower the drain elevation over a length of 5,140 feet to track with these seiche events.

Exhibit 1. Lake Erie Seiche



Source: Niziol 2020

Ten years of hourly average Detroit River elevation data were evaluated to assess the frequency and varying intensity of seiche events in Wyandotte, Michigan.[2] Table 4 summarizes the overall percentage of time over the course of the previous 10, 5, and 2 years that seiches of various intensity were observed on the Detroit River in Wyandotte.

[2] https://tidesandcurrents.noaa.gov/waterlevels.html?id=9044030

Table 4. Seiche Intensities

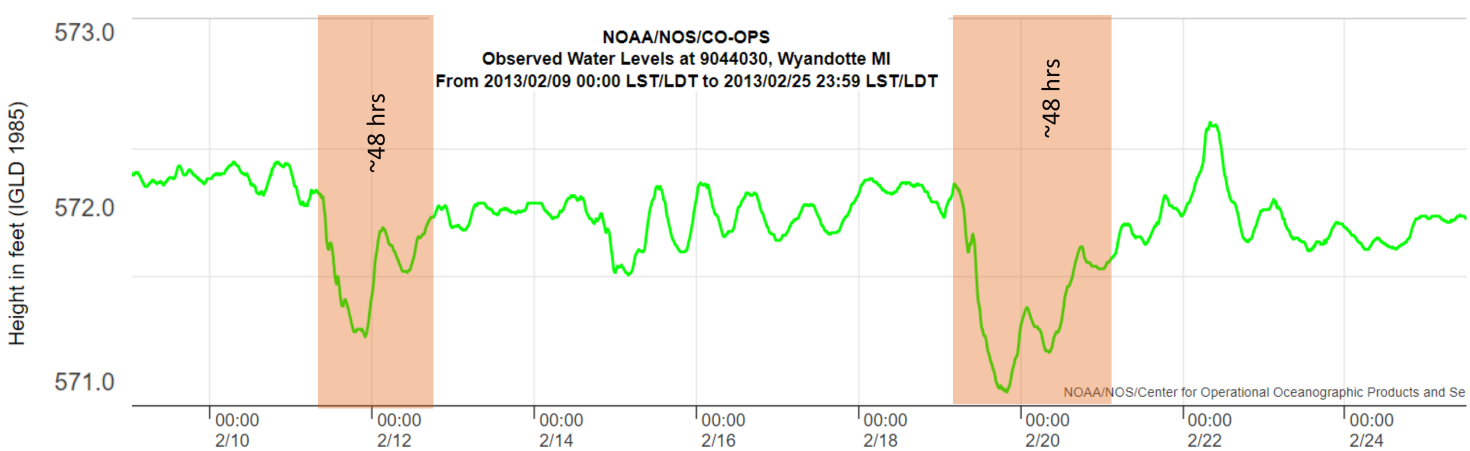
Seiche Intensity, as Measured by Maximum Daily Fluctuation	Number of Days Over 10 Years (1/1/2013 – 11/1/2023)	Overall % of Time	Number of Days Over 5 Years (1/1/2018 – 11/1/2023)	Overall % of Time	Number of Days Over 2 Years (1/1/2022 – 11/1/2023)	Overall % of Time
Less than 0.5 foot	2,881	72.8%	1,547	72.6%	494	73.7%
Greater than 0.5 foot	1,075	27.2%	584	27.4%	176	26.3%
Greater than 0.75 foot	416	10.5%	224	10.5%	70	10.4%
Greater than 1 foot	193	4.9%	107	5.0%	35	5.2%
Greater than 1.5 feet	50	1.26%	34	1.6%	8	1.2%
Greater than 2 feet	15	0.52%	9	0.58%	2	0.40%

This evaluation shows that the magnitude of different seiche events is generally consistent across the 10-, 5-, and 2-year time frames assessed and there is no significant trend suggesting seiches are becoming more or less intense over time. This evaluation indicates that seiches greater than 0.75 foot occurred just over 10% of the time frames evaluated, where seiches greater than 1 foot occurred approximately 5% over the same time frames. Therefore, a 90% inward gradient threshold is consistent with being protective of seiches that are 0.75 foot or less, whereas a 95% inward gradient threshold is consistent with being protective of seiches that are 1 foot or less, and so on.

Seiche Duration

Data from seiche events on the Detroit River over the past 10 years in Wyandotte indicate that seiches generally last 12 to 48 hours and generally occur between October and April. Seiche elevations for two typical months are shown on Exhibits 2 and 3.

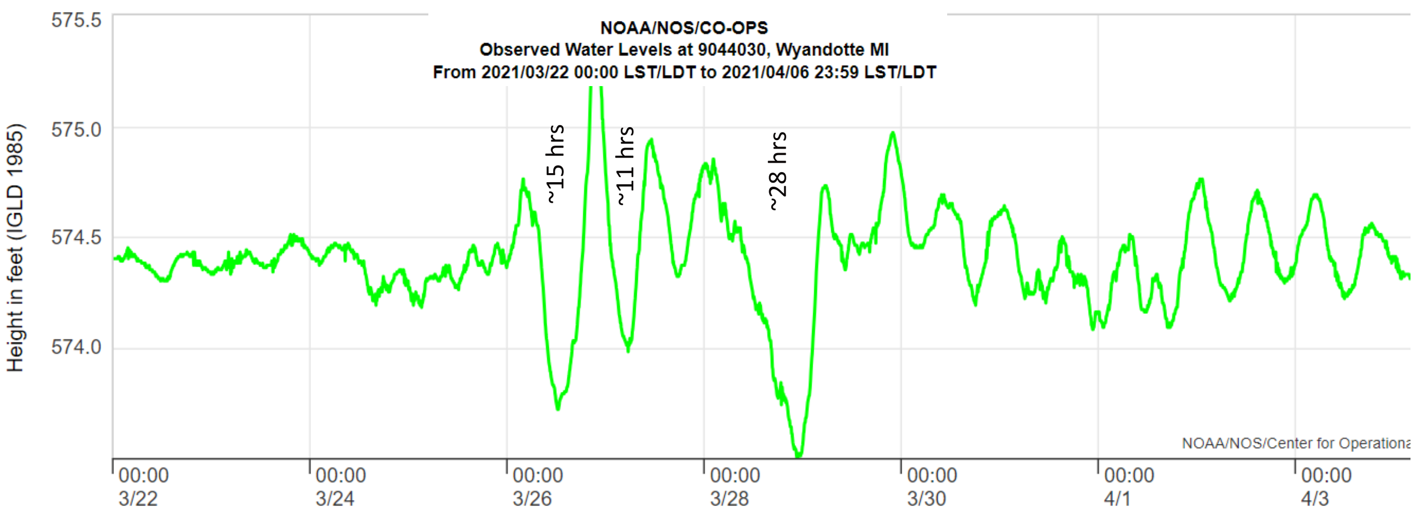
Exhibit 2. February 2013 Detroit River Elevations in Wyandotte, Michigan



Source: https://tidesandcurrents.noaa.gov/waterlevels.html?id=9044030

Note: IGLD = IGLD International Great Lakes Datum

Exhibit 3. March 2021 Detroit River Elevations in Wyandotte, Michigan



Source: https://tidesandcurrents.noaa.gov/waterlevels.html?id=9044030

One of the key considerations for this performance standard approach is that the drains should maintain an inward gradient relative to overall river elevation trends, but would not chase major short-term fluctuations in the river. As such, if gradient is not maintained for longer than 48 hours (i.e., duration of a typical seiche event), it could be an indication that the selected time frame for averaging and/or the compliance gradient is not adequate to reflect the longer-term river trends.

Ten years of daily average Detroit River elevation data were evaluated to assess the number of occurrences where gradient was not maintained for longer than 48 hours using the proposed performance standard approach. Two scenarios were selected for evaluation: (1) averaging the river elevation over 30 days (i.e., one month) with a compliance gradient of 0.3 foot and (2) averaging the river elevation over 30 days (i.e., one month) with a compliance gradient of 0.5 foot. A 30-day averaging time frame was selected for both scenarios because it is preferable to average over a longer time frame; averaging over a longer time frame results in lower maximum daily decreases (i.e., more achievable day-to-day decreases).

For scenario 1 (averaging the river elevation over 30 days with a compliance gradient of 0.3 foot), there were 35 occurrences in the 10-year period evaluated where an inward gradient was not maintained for longer than 48 hours. During that time frame, 18 of the 35 occurrences were related to a seiche event (or multiple seiche events) that was greater than 1 foot, and 22 of the 35 occurrences were related to a seiche event (or multiple seiche events) that was greater than 0.75 foot. These results suggest that for this performance standard compliance scenario, a gradient that is not maintained for longer than 48 hours is not related to a seiche event 40% to 50% of the time. This percentage range indicates that the input parameters for scenario 1 (averaging the river elevation over 30 days with a compliance gradient of 0.3 foot) may not be adequate to reflect the longer-term river trends.

For scenario 2 (averaging the river elevation over 30 days with a compliance gradient of 0.5 foot), there were 11 occurrences in the 10-year period evaluated where an inward gradient was not maintained for longer than 48 hours. During this time frame, 9 of the 11 occurrences were related to a seiche event (or multiple seiche events) that was greater than 1 foot, and 10 of the 11 occurrences were related to a seiche event (or multiple seiche events) that was greater than 0.75 foot. Compared to scenario 1, scenario 2 has far less occurrences of not maintaining gradient for longer than 48 hours (11 versus 35). Over the 10-year period with this scenario, one or two occurrences of a gradient that was not maintained for longer than 48 hours were not related to a seiche event, suggesting that the scenario 2 input parameters (averaging the river elevation over 30 days with a compliance gradient of 0.5 foot) are protective and reflect the longer-term river trends.

Overall, the larger compliance gradient requirement of 0.5 foot reflects the longer-term river trends without chasing short-term fluctuations associated with seiche events. Following the longer-term river trends can also be achieved by shortening the river averaging time frame; however, shortening the averaging time frame must be balanced with ensuring the potential maximum daily decreases (i.e., the daily reduction in drain elevation that would be required to maintain compliance) associated with the averaging time frame can be achieved. Note that the containment barrier will be in place to mitigate the discharge of groundwater if gradient is not maintained for some short duration of time associated with a seiche.

Operational and Permitting Risk and Cost Implications

The performance standard input parameters, specifically the averaging time frame, have a direct influence on the basis for design of the AG treatment system. The shorter the averaging time, the greater the maximum daily decreases and the higher the instantaneous flow rates from the drain sumps will need to be to meet the performance standards. The higher flow rates present both operational and permitting risk and have cost implications.

Operational risks associated with the potential short-term, high flow rates include the following:

- One size does not fit all for baseline/normal flow conditions versus short-term, high-flow events.
- If equipment is sized to handle high-flow events, conveyance piping, pumps, treatment equipment, media beds, etc., might operate outside of the ideal range during baseline/normal flow conditions.
- If equipment is sized for baseline/normal flow conditions, larger infrastructure that would be used infrequently during short-term high-flow events would need to be installed and maintained, increasing the risk of equipment failure and resulting in an increased cost for the remedy.
- Generally, accommodating a larger operational flow rate range would require contingency measures such as large storage tankage or ponds and/or an on-call water trucking vendor for transport and disposal off Site to manage additional flow during high-flow events.
- DUWA has indicated its sensitivity to discharge capacity and it may not be possible to permit a discharge that meets the higher flow requirements.
- Lowering groundwater levels quickly and/or significantly may change the hydrogeologic and/or transport conditions that have been established over time.
- Natural groundwater gradient has been established at the Site over time. The basis of design for the treatment system is from groundwater samples from perimeter monitoring wells under natural gradient conditions.
- After the installation of the barriers, it is important to keep conditions at the perimeter as close as possible to the basis of design conditions (i.e., natural gradient) while achieving an inward gradient that is protective and achievable. The proposed inward gradient approach of averaging the river elevation over 30 days does this.

- Reducing the trench levels quickly and/or significantly to track with a temporary drop in river elevation has the potential to affect groundwater concentrations at the perimeter by mobilizing mass that is not mobile under natural gradient conditions. If the groundwater pumped from the drains at the perimeter of the Site to the treatment system is significantly different in concentrations than the basis of design concentrations, it could create temporary upset conditions, treatment challenges, and/or difficulty meeting DUWA discharge permit requirements – i.e., operational and permitting risk.

The design flow rate of the AG treatment system for the perimeter barrier remedy is 150 gallons per minute (gpm) based on the equipment's ability to treat the expected flow rates under the majority of conditions regardless of the averaging time frame and/or compliance gradient selected. Using one year of data[3] (from November 2021 through October 2022), the groundwater model was employed to estimate the influent flow rate required to meet hypothetical drain compliance elevations for various performance standard compliance scenarios. This evaluation focused on the 0.5-foot gradient requirement, which compared to a 0.3-foot gradient is the most protective relative to seiche durations, and used longer averaging time frames, resulting in smaller potential maximum daily decreases. The groundwater model was run under site-specific recharge conditions obtained from site-specific stormwater modeling. Details of the groundwater and stormwater modeling and how it was used to support the design are provided in Section 5.1.1. Table 5 summarizes the average and maximum flow rates and the percentage of time the 150-gpm design flow rate was exceeded.

Table 5. Potential Compliance Scenarios at a 0.5-Foot Gradient and Under Site-Specific Recharge Conditions

Averaging Time Frame	Average Flow Rate (gpm)	Maximum System Flow Rate (gpm)	Design System Flow Rate (gpm)	% of Time with Inward Gradient
1-Month River Average	46	103	150	96%
2-Week River Average	47	142	150	97%
1-Week River Average	47.5	172	150	98%
1-Day River Average	48.5	469	150	98%

For the one month of averaging, the modeled expected flow rate does not exceed the design flow rate and maintains an approximate 50% contingency for extra capacity over the maximum flow rate modeled. The expected operational range of 46 to 103 gpm is relatively narrow and is within the ideal range of the treatment equipment, minimizing operational risks.

For the two-week averaging scenario, the maximum flow rate does not exceed the design flow rate but has minimal contingency for the maximum flow rate.

For the one-week and one-day averaging scenarios, the maximum flow rate exceeds the current design flow rate. Increasing the capacity of the system and/or incorporating influent storage capacity could be considered; however, doing so results in an increased cost and risk associated with permitting (i.e., DUWA's sensitivity to discharge flow rate). Increasing capacity also increases the difference between the expected average flow rate and the maximum flow rate, which creates operational risks. For these compliance scenarios, the operational risks, permitting risks, and costs increase for a relatively small gain in protectiveness (1 to 2%).

[3] https://tidesandcurrents.noaa.gov/waterlevels.html?id=9044030

3.4.2.1.2 Recommended Performance Standard Approach

In general, for the proposed performance standard compliance approach, longer averaging time frames with a higher compliance gradient requirement are preferred due to the lower potential maximum daily decreases for the drain (i.e., the daily reduction in drain elevation that would be required to maintain compliance).

Several lines of evidence were evaluated to assess various input parameters (i.e., river elevation averaging time frame and compliance gradient requirement) and their associated inward gradient thresholds to identify the combination that is protective and achievable, while minimizing the permitting and operational risks and considering cost. First and foremost, the perimeter barrier system includes a physical barrier between the Site and the Detroit River. During brief times of potential outward gradient associated with seiche events, the presence of the physical barrier will reduce the potential flux to the river and off Site to de minimis levels. An evaluation of seiche intensities indicates that a 90% inward gradient threshold is consistent with being protective of seiches that are 0.75 foot, and a 95% inward gradient threshold is consistent with being protective of seiches that are 1 foot, providing a line of evidence that the inward gradient thresholds are aligned with seiche events. An assessment of individual seiche durations indicates that for a one-month averaging time frame of the river elevation, a compliance gradient requirement of 0.5 foot is necessary for the performance standard approach to be protective of the longer-term river trends. A shorter averaging time frame (i.e., two or three weeks) with a lower compliance gradient requirement (i.e., 0.3 foot) could also be considered to maintain a similar level of protectiveness; however, a shorter averaging time frame results in a greater potential maximum daily decrease for the drain (i.e., the maximum distance the groundwater elevation in the drain would need to be decreased by the extraction system in one day), which may be more difficult to manage operationally. Finally, considering operational and permitting risks while balancing cost, longer averaging time frames allow the groundwater extraction and AG treatment system to operate within its optimal ranges without requiring significantly more/larger equipment, contingencies, or significant tankage for a relatively small gain in protectiveness (2% or less).

Based on the evaluation results and feedback from USEPA, BASF is proposing to average river elevations over one month with a compliance gradient requirement of 0.5 foot. Use of these parameters results in an inward gradient threshold of 96% and a maximum daily decrease of 0.06 foot based on 10 years of Detroit River elevation data from 2013 to 2023.

There are a variety of acceptable inputs (i.e., river averaging time frame and compliance gradient requirement), beyond the scenarios evaluated, that result in a drain compliance elevation that is protective and achievable. Because an appropriate level of protection can be achieved under a variety of scenarios, flexibility can be leveraged to adapt performance standards based on how the system actually operates. The adaptability of this approach for developing performance standards is key to the successful long-term management of the perimeter barrier remedy.

3.4.2.2 Monitoring Details for Inward Gradient

3.4.2.2.1 Eastern Shoreline

The layout of the proposed drain piezometers (PZs) and stilling wells discussed in this section is shown on Figure 3.

Two stilling wells will be installed in the river along the site shoreline. River elevations will be logged twice per day at each stilling well. The previous 30 days of river elevation data from both stilling wells will be averaged and 0.5 foot will be subtracted to calculate the drain compliance elevation.

Along the river, one PZ will be installed at the midpoint of each drain for compliance monitoring. The bottom of the PZ screens will be set to an elevation of 570 feet International Great Lakes Datum of 1985 (IGLD 85) to span the 30-year river low and be approximately 1 foot above the drain pipe. Once per day, the drain elevation will be logged at each PZ in the drains along the river and compared to the calculated drain compliance elevation to confirm that all drain elevations are at or below the compliance elevation.

3.4.2.2.2 Northern and Southern Boundaries

The layout of the proposed drain and groundwater PZs on the northern and southern ends discussed in this section is shown on Figure 3.

At the northern and southern ends, the barrier walls are not directly along the river. Therefore, PZs can be installed on the riverside of the barrier wall for compliance monitoring. Because groundwater elevations will be influenced by the river at these locations, the same averaging approach is proposed using groundwater levels at the northern and southern ends. The groundwater elevation changes are not as drastic as the river elevation changes; therefore, this approach is conservative.

Two PZs will be installed riverward (north) of the northern barrier wall. The PZs will be placed such that they have a corresponding drain PZ and will be spaced equal distances along the northern barrier wall, to the extent possible. The bottom of the PZ screens will be set to an elevation of 570 feet IGLD 85 to span the 30-year river low. During installation, efforts will be made to place the PZs in locations where the geology reflects groundwater conditions, avoiding screen placement in low-permeability soils. Therefore, the locations and depths of the PZs may be shifted based on field conditions. Groundwater elevations will be logged twice per day at each PZ. For each PZ, the previous one month of groundwater elevation data will be averaged and 0.5 foot will be subtracted to calculate the drain compliance elevation for each PZ.

Along the northern end, one PZ will be installed at the midpoint of each drain for compliance monitoring (total of two) and will have a corresponding PZ that is on the riverside (north) of the barrier wall. The bottom of the PZ screen will be set to an elevation of 570 feet IGLD 1985 to span the 30-year river low and be approximately 1 foot above the drain pipe. Once per day, the drain elevation will be logged at the PZs in the drains along the northern boundary and be compared to the calculated drain compliance elevation from the corresponding PZ that is riverside (north) of the barrier wall to confirm that the drain elevation is at or below the compliance elevation.

Three PZs will be installed riverward (south) of the southern barrier wall. The PZs will be placed such that they have a corresponding drain PZ and will be spaced equal distances along the southern barrier wall, to the extent possible. The bottom of the PZ screens will be set to an elevation of 570 feet IGLD 1985 to span the 30-year river low. During installation, efforts will be made to place the PZs in locations where the geology reflects groundwater conditions, avoiding screen placement in low-permeability soils. Therefore, the locations and depths of the PZs may be shifted based on field conditions. Groundwater elevations will be logged twice per day at each PZ. The previous month of groundwater elevation data will be averaged and 0.5 foot will be subtracted to calculate the drain compliance elevation for each PZ.

Along the southern end, one PZ will be installed at the midpoint of each drain for compliance monitoring (total of three) and will have a corresponding PZ that is on the riverside (south) of the barrier wall. The bottom of the PZ screens will be set to an elevation of 570 feet IGLD 1985 to span the 30-year river low and be approximately 1 foot above the drain pipe. Once per day, the drain elevation will be logged at the PZs in the drain along the southern boundary and compared to the calculated drain compliance elevation from the corresponding PZ that is riverside (south) of the barrier wall to confirm that the drain elevation is at or below the compliance elevation.

3.4.2.3 Monitoring Details to Demonstrate Groundwater Collection

This section details the method agreed upon between BASF and USEPA to demonstrate that the collection system combined with the perimeter barriers is effective in containing and collecting groundwater on the Site. The proposed upland PZ locations discussed in this section are shown on Figure 3.

A line of PZs will be installed in the upland generally spaced evenly along the entire barrier and collection drains. The PZs will be located within the capture zone of the drains based on the groundwater model. The groundwater elevations of these upland PZs will be compared to the groundwater elevations of the PZs in the drains to show that groundwater is moving into the drains (i.e., the groundwater elevation in the upland is higher than the drain elevation).

BASF and USEPA agreed that monitoring of these upland PZs would be more frequent during start-up and initial operations and be reduced after routine operational conditions are established. Therefore, groundwater elevations in the upland PZs will be manually measured twice per week for comparison to drain elevations during start-up and initial operations. When routine operational conditions are established, BASF will request approval from USEPA to reduce the frequency of monitoring to weekly, then monthly, and ultimately quarterly. The actual schedule to reduce frequency will depend on site conditions and system operation. No reduction in monitoring frequency will be made without explicit approval from USEPA.

Collection of groundwater elevation measurements from the PZs in the drain will be ongoing for the purpose of system control (i.e., to determine which pumps need to operate to maintain the compliance gradient). As such, groundwater elevations of the drains will be available in real time and can be used to compare to the manual elevations collected in the upland PZs.

3.4.3 Objectives and Performance Standard Requirements for Project Phases

The groundwater management portion of the perimeter barrier system will consist of three phases: 1) AG treatment system commissioning, 2) start-up period, and 3) long-term OMM. A summary of the objective and performance standard requirements for each phase is presented below; additional details for each phase are provided in the OMM Manual (Appendix K). Note that there will be a Phase Zero prior to the system commissioning that includes all preparatory work required to safely and properly start up the system and is expected to take a minimum of one month.

3.4.3.1 Phase 1: Above-Grade Treatment System Commissioning

The first phase, AG treatment system commissioning, will begin a minimum of six months before the completion of barrier construction. The objective of this phase is to achieve reliable operation of the AG treatment system before sealing up the downgradient perimeter barriers to mitigate the risk associated with groundwater management and flooding. During this phase, groundwater pumping and treatment will begin. There will be no compliance gradient requirements during this phase. This phase includes commissioning for mechanical equipment and establishing parameters for treatment performance. Drain PZs will be monitored, as needed, to verify pump operations and controls. POTW permit requirements and discharge limits will apply when treated water is discharged to the sanitary sewer during this phase.

3.4.3.2 Phase 2: Start-Up Period

The second phase, the start-up period, will begin after the completion of the barriers and the AG treatment system commissioning phase. It is anticipated that this phase will last a minimum of 18 months, although this phase could be longer than 18 months if additional supplemental infrastructure is needed to meet performance standards. To expedite the process, the start-up period may begin in some sections of the Site earlier than others as perimeter barriers are completed. The objectives of this phase are to develop operational parameters that maintain compliance with performance standards and to optimize performance standards, as needed. Specifically, the goals during this phase are to identify the operational parameters to maintain the drain compliance elevation with the barriers in place and to optimize AG treatment system operations.

For the drains and sumps, during this phase, BASF will:

- Establish flow rates and elevation setpoints to meet drain compliance elevations.
- Evaluate drain elevation responses to pump setpoints, precipitation, river stages, etc.

For the AG treatment system, during this phase, BASF will:

- Optimize treatment chemistry.
- Establish cleaning/backwash schedules.
- Establish media change-out schedules.
- Confirm equipment maintenance, repair, and/or replacement schedules.

Gradient monitoring is required to meet the goals of this phase; however, there is no requirement to consistently meet the required drain compliance elevation during this phase.

During this phase, the drain compliance elevation basis will be revisited based on the actual operation of the system across the Site. As stated, the proposed performance standard for the inward gradient is the use of one month of river or groundwater data for averaging and subtracting the 0.5-foot compliance gradient requirement to calculate the drain compliance elevation. During the start-up period, each area along the perimeter will be assessed to confirm that the averaging time frame and compliance gradient requirements result in a drain compliance elevation that is achievable and protective. There may be some areas along the perimeter where a higher gradient requirement (e.g., one month of river elevation averaging with a 0.5-foot compliance gradient requirement) is needed to consistently meet the performance standards. There may be other areas along the perimeter where a lower gradient with shorter averaging times is preferred. The OMM Manual (Appendix K) includes a detailed start-up plan with criteria for assessing whether the proposed averaging time frame and compliance gradient requirements are achievable. The OMM Manual also outlines how the assessment will be implemented during start-up.

In addition, the need for supplemental infrastructure to meet performance objectives will be evaluated. If the compliance drain elevation cannot be maintained, potential supplemental infrastructure could include additional extraction wells within existing drains, vertical wells, horizontal wells, or new drains and sumps. The Pre-Final (95%) Design includes the following contingencies to achieve effective containment of groundwater and to account for future changes, to the extent practical:

- A collection trench drain pipe invert depth of 567 feet IGLD 85, approximately 2.5 feet below the instantaneous near 30-year river low-elevation level, which provides the flexibility to maintain a 0.5-foot compliance gradient for any average river elevation above 567.5 feet IGLD 85;

- Low-permeability material to be installed above the stone backfill of the drains, near ground surface, to reduce direct infiltration into the trench;
- Duty standby (redundant) extraction pumps in each sump providing the capacity to pump twice the design flow;
- Two 150,000-gallon storage tanks designed to provide extra storage capacity as needed;
- A treatment system design flow rate of 150 gpm, which is two to three times higher than the average modeled steady-state flow rates for normal and high recharge conditions;
- Additional area inside the Water Treatment Building for future expansion of process equipment if needed;
- Additional area to the south of the Water Treatment Building and removable walls on the south side of the Water Treatment Building for future expansion of the building if needed; and
- Additional area inside the secondary containment berm for the storage tanks for an additional tank to be added in the future if needed.

Prior to completion of the start-up phase, BASF will submit any changes for the performance standard basis to USEPA. Any proposed changes to the performance standards will utilize the same approach outlined in this BOD Report, and BASF will provide a basis for the proposed changes showing that the requested changes are protective and achievable. In addition, if any additional infrastructure is proposed (additional extraction wells within existing drains, vertical wells, horizontal wells, or new drains and sumps), a plan will be submitted to USEPA for approval prior to implementation. The start-up phase will not be considered complete until any proposed updates to the performance standards are approved by USEPA and/or any additional infrastructure is approved, installed, and commissioned.

3.4.3.3 Phase 3: Long-Term Operation, Maintenance, and Monitoring

Upon completion of the start-up phase, the remedy will move into the final phase of long-term OMM. During this phase, maintaining the drain compliance elevation as finalized during the start-up phase is required. For the first year of operation in the long-term OMM phase, quarterly progress reports will be submitted to USEPA detailing the operation of the perimeter barrier system that quarter. Report frequency will be decreased to semiannually for the next year and then annually thereafter. This phase includes an allowance for up to 72 hours per quarter operating out of the required gradient. If the gradient is reestablished within the allowable time frame (i.e., not exceeding 72 hours days per quarter), notification of the upset condition and response activities, along with corrective actions taken and/or updates to standard operating procedures and/or OMM procedures, will be documented in the routine progress report. If the gradient is not able to be maintained consistently, the performance standard basis will be reevaluated and/or the need for supplemental infrastructure will be assessed. Because the perimeter barrier remedy is expected to operate for the foreseeable future, it is critical that ongoing evaluation of performance standards and non-compliance time frames be reevaluated routinely and reflect the current site conditions and risk profile. Proposed changes to the performance standards can be submitted to USEPA anytime during this phase.

3.5 Applicable or Relevant and Appropriate Requirements, Pertinent Codes, and Standards

The barrier remedy is designed in accordance with applicable or relevant and appropriate requirements, codes, and standards. Potential regulatory requirements, including requirement description and applicability, are listed in Table 6.

Table 6. Regulatory Requirements

Requirement	Description	Applicability	Notes
RCRA (42 United States Code [USC], Chapter 82); Hazardous Waste Management (Natural Resources and Environmental Protection Act [NREPA], 1994 PA 451, as amended, Part 111)	Hazardous waste management requirements	RCRA Corrective Action at Michigan facility; waste generation anticipated	Applicable to the management of generated wastes, including tank storage of extracted groundwater characterized as hazardous
Environmental Remediation (NREPA, 1994 PA 451, as amended, Part 201)	Protects the environment and natural resources of the State of Michigan	RCRA Corrective Action at Michigan facility	Applicable to the development of performance standards for groundwater venting to surface water
Clean Water Act (33 USC, Chapter 26)	Regulations for the discharge of pollutants into waters of the United States	Groundwater vents to a surface water of the United States	Applicable to the development of performance standards for groundwater venting to surface water
Safe Drinking Water Act (42 USC, § 300f et seq.)	Program and performance standards for underground injection programs	Potential for underground injection	Class V Underground Injection Control requirements may be applicable to air sparging systems
Clean Air Act (42 USC, Chapter 85); Air Pollution Control (NREPA, 1994 PA 451, as amended, Part 55)	Protect quality of air and promote public health	Potential for producing air emissions of regulated air pollutants (including particulate emissions)	Emission standards may be applicable
Migratory Bird Treaty Act (16 USC, §§ 703–712)	Protects almost all species of native migratory birds in the United States from unregulated “take”	Potential presence of migratory birds	Measures will be taken to evaluate whether migratory birds are present; project activities will be scheduled to not disturb migratory birds, or depredation permits will be obtained if necessary

Requirement	Description	Applicability	Notes
Endangered Species Act (16 USC, Chapter 35)	Requires that federal agencies ensure that any action authorized, funded, or carried out by an agency is not likely to jeopardize the continued existence of any threatened or endangered species and will not destroy or adversely modify critical habitat	Potential presence of federal-listed species	Work will be conducted in disturbed areas and is not anticipated to affect federal-listed species; however, the United States Fish and Wildlife Service (USFWS) and Michigan Department of Natural Resources (MDNR) will be consulted
Fish and Wildlife Coordination Act (16 USC, Chapter 5A)	Requires that activities avoid adverse effect and minimize potential harm, preserve natural and beneficial values, and/or compensate for impacts to fish, wildlife, and their habitats through restoration	Potential impacts to fish/wildlife habitat	USFWS and MDNR will be consulted regarding the impacts on fish and wildlife resources and measures to avoid, minimize, and mitigate the impacts
Bald and Golden Eagle Protection Act (16 USC, Chapter 5A)	Provides for the protection of the bald eagle and the golden eagle by prohibiting, except under certain specified conditions, the taking, possession, and commerce of such birds	Potential presence of bald eagle habitat	USFWS will be consulted regarding potential impacts to bald eagles
Coast Guard Notification	The United States Coast Guard is responsible for disseminating information concerning aids to navigation, hazards to navigation, and other items of marine information of interest to mariners on the waters of the United States	Potential in-river work	Appropriate notifications will be provided to the United States Coast Guard District 9 Commander
Regulations on the Take of Reptiles and Amphibians (Natural Resources Commission and MDNR, Part 487 of 1994 PA 451, FO-224.21)	It shall be unlawful to kill, take, trap, possess, buy, sell, offer to buy or sell, barter, or attempt to take, trap, possess or barter any reptile or amphibian from the wild, or the eggs of any reptile or amphibian from the wild as listed in the order	Potential presence of MDNR endangered or threatened species	Work will be conducted in disturbed areas and is not anticipated to affect federal-listed species; however, USFWS and MDNR will be consulted. If required, a permit will be submitted to protect and relocate any threatened and endangered species identified.
Soil Erosion and Sediment Control (SESC) Permit (NREPA, 1994 PA 451, as amended, Part 91)	Protects the waters of the State of Michigan and adjacent properties by minimizing erosion and controlling off-site sedimentation	Potential land disturbance within 500 feet of lake or stream (applies to work affecting 1 or more acres)	SESC permit coverage will be obtained as applicable

Design codes and standards deemed applicable for construction of the perimeter barrier remedy are included in the Technical Specifications (Appendix L). Sections 6.6 through 6.11 of this BOD Report include relevant building codes and standards for construction of the Water Treatment Building. Codes, standards, and regulations associated with the collection, extraction, conveyance, and barrier components of the remedy are noted on the Design Drawings (Appendix G).

4 Perimeter Barriers

A perimeter containment barrier will be constructed along the northern, eastern, and southern sides of the property perimeter. The barrier alignment is further subdivided into the following (Figure 2):

- Northern boundary (approximately 755 feet of planned barrier along Perry Place to the property boundary);
- Northern shoreline (existing 3,235-foot-long sheet steel bulkhead);
- South Dock (approximately 1,725 feet long);
- Rip-rap area (approximately 830 feet of existing rip-rap-covered riverbank between the South Dock and the southern site boundary); and
- Southern boundary (approximately 1,385 feet of proposed barrier along the northern edge of James DeSana Drive).

These sections of the proposed barrier alignment are shown on Figure 2 and encompass the extent of groundwater concentrations exceeding EGLE generic GSI criteria. The planned barrier network consists of approximately 4,694 linear feet of new barrier construction along the northern boundary, the South Dock, the rip-rap area, and the southern boundary. The existing steel bulkhead along the northern shoreline will be incorporated into the barrier network. The design process for the perimeter containment barrier considered various options, with the ultimate selection of subsurface sheet pile barriers for the northern boundary, southern boundary, and rip-rap area and a new anchored steel sheet pile bulkhead along the South Dock.

The subsurface barriers will be constructed of steel sheet piles embedded a minimum of 3 feet into the underlying clay layer and pile tops cut down to approximately 12 inches below ground surface (bgs). The subsurface barriers will have Larssen interlocks with hydrophilic sealant applied to the sheet pile joints to provide for water tightness.

The South Dock anchored bulkhead will consist of a sheet pile headwall driven into the glacial till layer to provide adequate resistance and stability. The headwall will have Larssen interlocks with hydrophilic sealant applied to the sheet pile joints to provide for water tightness. The deadman anchor wall consists of unsealed steel sheet piles installed inland from the headwall. Approximately every 51 feet along the deadman anchor wall, a 4-foot gap will be left between steel sheet piles to maintain a hydraulic connection with the groundwater recovery trench. Tie rods will connect the headwall to the deadman anchor wall, providing lateral support and reducing structural demand on the headwall. Lightweight cellular concrete (LCC) will be used as backfill under the existing dock to reduce structural demand and earth pressures on the wall components.

The existing steel bulkhead will be integrated into the barrier network. Minor repairs will be made to the existing sheet piles including 1) sealing of lift holes and other openings observed in the sheet piles, 2) sealing of leakage observed through interlocks, tie bolts, and wale bolts, 3) providing a protective coating or metal plates for areas exhibiting corrosion, and 4) placing backfill in areas where soil erosion has occurred behind the wall.

The following sections describe the barrier options considered (Section 4.1), the rationale for the selection of barrier types for each section of the perimeter alignment (Section 4.2), the Pre-Final (95%) Design of the selected barriers (Section 4.3), details on the existing bulkhead (Section 4.4), and the design of barrier intersections (Section 4.5)..

4.1 Barrier Considerations

The barrier types considered as part of the design process include "subsurface barriers" and "shoreline barriers." Subsurface barriers are barriers that are constructed in upland areas, away from the shoreline, and are completely below the ground surface. Shoreline barriers are defined herein as barrier options directly along the shoreline that are in-water and may be exposed or partially exposed above the ground or sediment surface. All are low permeability and a barrier to groundwater flow.

Five subsurface barrier types were evaluated for the proposed subsurface potion of the barrier network. The Preliminary (30%) BOD Report (Arcadis 2022c) includes descriptions of each subsurface barrier type and the evaluation criteria consisting of geologic considerations, constructability, overall effectiveness, and cost. The subsurface barrier types each include a groundwater collection and AG treatment system installed on the upland side of the barrier. Appendix M includes an additional comparison developed for the Intermediate (60%) Design that focuses on the southeastern portion only of the subsurface containment barrier alignment.

The shoreline barrier consists of a steel bulkhead installed along the shoreline, on the outboard side of the existing South Dock structure, to serve as a containment barrier. To support the Pre-Final (95%) Design, a summary table of the South Dock bulkhead was developed to compare the options for the headwall and anchor system for this new structure (Appendix M).

4.2 Development of Containment Barrier

The proposed barrier types for the containment barrier system in this Pre-Final (95%) Design are summarized in Table 7. The basis for selection of each barrier type by area is included in the Preliminary (30%) BOD Report (Arcadis 2022c). Further evaluation of the barrier options was completed as part of the Intermediate (60%) (Arcadis 2024a) and Pre-Final (95%) Design. A basis for the selected barrier type for the southeastern portion of the network is provided in Appendix M. Design Drawings showing the proposed location for each barrier type selected are included in Appendix G.

Table 7. Barrier Remedy

	Barrier Type
Northern Perimeter	Subsurface Barrier – Steel Sheet Pile
Northern Shoreline	Shoreline Barrier – Existing Steel Bulkhead
South Dock	Shoreline Barrier – Steel Bulkhead
Rip-Rap Shoreline	Subsurface Barrier – Steel Sheet Pile
Southern Perimeter	Subsurface Barrier – Steel Sheet Pile

Note that an ex-situ mixed borrow soil-cement wall was proposed in the Preliminary (30%) BOD Report for this portion of the barrier network due in part to the high potential for encountering obstructions in the fill layer. Upon review of recent geophysical studies conducted in this area, it was determined that the potential for encountering obstructions is not as great as originally anticipated and that, with certain construction measures in place, a driven sheet pile wall would be an appropriate technology to implement along the rip rap shoreline (Figure 3, Appendix G, and Appendix M). Additionally, in comparison to the soil-cement wall option, a driven sheet pile wall will be less disruptive and result in a smaller footprint during construction that could impact sensitive adjacent properties.

Section 4.3.1 includes additional detail on the Pre-Final (95%) Design basis for this wall design and the construction measures that will be taken during implementation.

4.3 Barrier Design

This section summarizes the Pre-Final (95%) Design for the subsurface and shoreline elements of the barrier network. Additional design details are provided on the Design Drawings (Appendix G), in the Technical Specifications (Appendix L), and in the Design Calculations (Appendix N).

4.3.1 Subsurface Sheet Pile Walls

This section provides the BOD for the steel sheet piling and includes a summary of subsurface conditions and items specific to the wall properties and alignment.

4.3.1.1 Material Selection

The sheet pile section material has been selected based on drivability through the subsurface, potential for encountering obstructions/debris at depth, corrosion potential, and type of interlock joint. The hydraulic conductivity for this wall type will meet the criteria established from the site groundwater model of 1×10^{-6} centimeters per second (cm/sec) and, when considering the application of a sealant at each interlock joint, is expected to range between 1×10^{-6} and 1×10^{-7} cm/sec.

Evaluations supporting the selection of the sheet pile section for the subsurface wall are included in Appendix N (Design Calculations). For drivability, findings from the SPT borings completed near the proposed subsurface barrier alignment were considered for determining the pile section along with the potential for encountering difficult driving conditions. In addition, consideration in this Pre-Final (95%) Design was given to consistency of pile types across the barrier system for interlock purposes and similar manufacturers. The pile type was changed from an NZ-22 in the Intermediate (60%) Design to an AZ 24-700 sheet pile for this Pre-Final (95%) Design. The AZ pile type is consistent with the sheet piles of the existing bulkhead and for the proposed sheet piles of the South Dock bulkhead. This pile type is Z-shaped and widely available in the United States. Consistent with the recommendations in the Preliminary (30%) BOD Report (Arcadis 2022c), the interlocks for this pile shape are the Larssen type, and each joint will be sealed with a hydrophilic sealant. Calculations for estimating the hydraulic conductivity through the interlocks have been completed and confirm that the sealed, Larssen type interlocks will meet the required conductivity for the wall. These calculations are included in Appendix N (Design Calculations).

The potential for section loss due to corrosion was evaluated, and the findings are included in Appendix N (Design Calculations). Thickness loss rates were determined using Eurocode 3 – Design of Steel Structures – Part 5: Piling (European Committee for Standardization 2007), which provides loss rates based on environmental conditions typical of industrial sites. A section loss of approximately 0.26 inch, or approximately 58% of the pile's design thickness of 0.44 inch over an assumed 50-year design life, is estimated. This estimated corrosion loss would not compromise the steel wall's ability to serve as an adequate barrier to groundwater flow. Because the subsurface barrier is a non-structural wall, this estimated loss of thickness with time is not a concern when evaluating wall strength criteria. As a barrier for groundwater flow, a 58% loss in thickness means that approximately 42% of the pile thickness will remain after 50 years and will continue to perform as an impervious steel section. At the sheet pile interlocks, a combination of the Larssen interlock type and the long-term sealant properties will continue to provide watertight resistance for countering degradation due to corrosion.

4.3.1.2 Barrier Alignment

The alignment of the subsurface barrier is shown on the Design Drawings (Appendix G). For the northern perimeter, the sheet pile alignment is in Perry Place, generally positioned within the southern portion of the roadway to allow access to adjacent properties during construction. For the northern perimeter, the subsurface barrier wall is approximately 755 feet in length. A watertight connection to the existing bulkhead is required at its eastern end. The Intermediate (60%) Design had accounted for this connection to be made via a series of jet grout columns advanced into the clay layer. Further inspections of the existing bulkhead for the Pre-Final (95%) Design, however, have confirmed that it is possible to connect to the AZ-13 sheet piles of the existing bulkhead through an interlock pile connector. A detail of this pile-to-pile connection is shown on the Design Drawings (Appendix G).

For the southern perimeter, the barrier wall alignment extends from the southern end of the new South Dock headwall, continuing along the rip-rap portion of the riverbank, before turning west. The alignment along the rip-rap-protected shoreline is set at an offset distance that allows for construction of the wall using conventional techniques and is protective of the existing shoreline. For the Pre-Final (95%) Design, excluding the portion of the South Dock headwall that will be installed within the bank, the sheet pile wall alignment for the subsurface barrier is offset from the top-of-bank rip rap between 30 and 40 feet. Pile driving equipment will be offset further from the rip-rap bank for installation of this segment of the barrier, which is considered sufficient distance to minimize impacts to the bank from temporary construction loading or pile driving vibrations. Where the sheet piles will be installed through the bank (i.e., for the South Dock return wall), the condition of the bank will be inspected daily while work is in progress for signs of movement or instability. Modifications to construction methods could be required should conditions warrant.

As the wall extends toward the southern BASF property boundary, the alignment then accounts for the existing embankment for the fire water impoundment. This earthen embankment is approximately 11 feet in height and slopes at approximately 4H:1V (4 horizontal units to 1 vertical unit). The sheet pile wall is positioned approximately 30 feet from the embankment toe to allow for operation of the pile driving equipment and for positioning of the collection trench and conveyance piping between the barrier and the embankment. Beyond the fire water impoundment, the wall continues along James DeSana Drive for approximately 900 feet. The majority of the wall will be positioned along the northern edge of the roadway. Where the alignment falls beneath existing overhead power lines that cross James DeSana Drive, installation of full-length sheet piling is not possible and incorporating another wall technology (i.e., jet grouting) was specified in the Intermediate (60%) Design. Based on the advancement of the barrier design and findings from a value engineering (VE) study, in lieu of jet grouting beneath the overhead power lines along James DeSana Drive, the Pre-Final (95%) Design has incorporated installation of spliced sheet piles through this area. The sheet pile section (i.e., AZ 24-700) will be the same as the remainder of the subsurface barrier wall. Installing spliced sheet piles will eliminate the need for jet grouting and allow for sheet pile installation while maintaining the appropriate clearance from the overhead high-voltage lines. As shown on the Design Drawings (Appendix G), splicing of sheet piles is an acceptable approach to barrier installation beneath the power lines.

Both the subsurface barrier walls on the northern and southern perimeters will intersect existing subsurface public utilities. Approximate locations of the utility crossings are shown on the Design Drawings and are based on findings from utility investigations and geophysical surveys completed along the proposed alignments (Appendix G). The utilities that cross the alignment vary in diameter and in depth below current grade. Each utility crossing will be addressed with the utility owners. Options for addressing utilities include temporary bypassing, temporary or permanent relocation, replacement in kind, permanent removal, or protection in place, which would incorporate another wall technology (e.g., jet grouting) to maintain the barrier properties. Details for addressing each utility crossing are shown on the Design Drawings and will be further coordinated with the utility owners prior to construction.

For those utilities that will remain in place during wall installation, it is anticipated that the sheet pile wall will end on either side of the utility, leaving an estimated maximum space of approximately 6 feet around the utility. To provide a continuous barrier at these utility penetrations, the soil zone around the utility will be jet grouted from the required minimum depth of the barrier to within approximately 12 inches of finished grade. Limits of jet grouting will overlap with the adjacent sheet piles to provide a seal between the two wall types. As shown on the Design Drawings, there is only one utility, the water line entering the south side of the Site from James DeSana Drive, that is anticipated to require jet grouting.

For those utilities that can be temporarily bypassed or relocated, an opening in the sheet piles will be made after installation to accommodate placement of a pipe sleeve that is welded to the sheet pile. The annulus between the pipe sleeve and reinstalled utility pipe will be sealed to provide a continuous barrier at the utility penetration. Details for both types of utility crossings are included on the Design Drawings.

4.3.1.3 Pile Depths

The steel sheet piles will be driven into the clay layer to provide a bottom seal for the barrier. A minimum of a 3-foot embedment into the clay layer is proposed for the sheet piles and is considered sufficient for an effective vertical seal along the wall alignment. This minimum embedment into the clay layer was also confirmed to provide sufficient frictional resistance to support the self-weight of the sheet pile. The estimated frictional resistance was evaluated at several locations along the proposed subsurface barrier alignment to represent the varying soil conditions along the north and south walls. In each case, the overlying fill layer was neglected in the estimate of skin friction capacity, and it was assumed that no tip resistance would contribute to the pile's capacity. Given that the sheet piles will be self-supporting and no external loads will be applied to the wall, it can be concluded that the underlying clay layer will have no added vertical stress from wall installation, and therefore, no settlement from the self-weight of the pile is expected. Pile capacity calculations are included in Appendix N.

Based on findings from subsurface investigations along the alignments, the sheet pile depths for the northern subsurface barrier will vary from approximately 12 feet below proposed grade at the western end to 25 feet at the connection to the existing sheet pile bulkhead. For the southern subsurface barrier, the sheet pile depths will range from approximately 13 to 41 feet below proposed grade. Based on the results of the limited investigation performed in the southern alignment (Appendix I), a segment of the clay surface and corresponding pile depths have been modified from the surface and pile depths shown in the Intermediate (60%) Design submittal. In this area, the clay profile in the Intermediate (60%) Design was conservatively shown to follow the multichannel analysis of surface waves (MASW) interpretation. However, from the recent soil borings and laboratory testing results and through review of data from previous soil borings advanced in this area, it was determined that the clay surface is above the MASW interpreted surface. As discussed in the investigation summary report (Appendix I), the data from borings and laboratory testing in this limited area indicate consistent clay properties above and below the MASW interpreted surface, concluding that the MASW technology in this area was not a reliable interpretation of the clay surface. The clay surface was adjusted approximately from barrier station 66+60 to station 70+46. Figures for the clay surface for the complete subsurface barrier alignment are included in Appendix C. These figures include plan views of the wall alignment with subsurface investigation locations and profiles of the alignment with the subsurface investigation locations, the design clay surface and sheet pile limits. The tops of sheet piles will be above the groundwater surface but also at a sufficient depth (approximately 12 inches) below grade for placement of restoration materials.

Prior to pile driving, the alignment of the sheet pile wall will be pre-trenched through the surficial materials (i.e., concrete or asphalt pavement); in zones of known debris or obstructions, the depth of pre-trenching will be increased as required for removal of the debris or obstructions. Excavated materials that meet the requirements of the project specifications will be used to backfill the trenches prior to sheet pile installation and supplemented as needed with imported fill. Excavated materials that cannot be reused as fill from the pre-trenching activities will be staged and loaded for off-site disposal.

4.3.2 Steel Bulkhead Design at South Dock

The steel bulkhead proposed for the South Dock provides stability to the existing historical structures on the shoreline, does not change the footprint of the Site, and is a proven approach for a groundwater barrier along a surface water body.

In addition to design information, this section provides background information, presents the rationale for the wall type selection, and discusses construction methods, constructability, and sequencing of the steel bulkhead with an anchor wall at the South Dock. As part of the Pre-Final (95%) Design, components of the barrier system were reviewed for constructability and VE opportunities. Findings from the VE study are also discussed when related to modifications from the earlier design phases.

4.3.2.1 Background

This section provides background information relevant to the steel bulkhead design, including information regarding the existing structures along the South Dock, planned dredging in the Detroit River adjacent to the Site, and subsurface conditions.

4.3.2.1.1 Existing Structures Along South Dock

The existing South Dock structure consists of a soil-covered concrete deck supported on timber piles. Previous design drawings are available for some of the waterfront structures and are included in the RCRA RFI Response (Arcadis 2017). A 1939 drawing of an outfall structure shows a cross section through the concrete deck at the South Dock (Drawing 92/9, titled “Alterations to Dock for Main Sewer Outfall,” dated October 10, 1939, Michigan Alkali Company, Engineering Department; note that portions of the drawing are barely legible). Observations from recent test pit investigations at the South Dock are discussed in Section 4.3.2.1.3.

In addition to the pier structure, a Wakefield wall (a tongue-and-groove timber sheet pile bulkhead) is present between the concrete deck and the upland area. Based on drawings for similar structures located north of the South Dock, it is assumed that lateral support for the Wakefield wall is provided by steel tie rods connected to timber piles that were driven into the ground in the upland area. At the locations to the north, there are two rows of timber piles that are used as anchors, one approximately 30 feet behind the Wakefield wall and another row approximately 50 feet behind the Wakefield wall (Arcadis 2017).

4.3.2.1.2 Upper Trenton Channel Dredging Project

A sediment remediation project that involves dredging in the Upper Trenton Channel, including along North Works adjacent to the South Dock, is currently being developed by BASF and other non-federal partners in collaboration with USEPA (CH2M HILL, Inc. 2019). The current dredge prism design along the South Dock includes a 10-foot offset from the face of the pier structure and a 3H:1V downward dredge slope away from the shoreline. The new bulkhead design accommodates this current dredge prism design and assumes a 10-foot offset from the face of the headwall starting at the current sediment surface elevation. As part of the dredging project, a cover system will be placed over the dredged surface and up to the face of the new bulkhead structure.

4.3.2.1.3 Subsurface Conditions

Detailed information regarding subsurface conditions and physical characteristics of the geologic units in the alignment of the steel bulkhead wall is provided in the BASF North Works Geotechnical Data Report included in Appendix C. This report summarizes the findings from a geotechnical investigation program completed between June 17 and August 5, 2020 at the Site, generally in the South Dock area. A total of 17 geotechnical borings and 17 CPT explorations were performed in the upland and near-shore areas along the proposed barrier alignments in the South Dock area and along the rip-rap-protected riverbank south of the South Dock. Four of the geotechnical borings and 14 of the CPT explorations were completed in the river, within 50 feet of the shoreline, using barge-mounted equipment. The 2021 geotechnical data report includes details of the investigations completed in 2020, soil descriptions developed from the findings of the investigations, soil profiles depicting the subsurface stratigraphy, and associated boring logs and testing results.

The subsurface predominantly consists of relatively loose and soft soils over bedrock. The overburden soils in the upland range in total thickness from approximately 62 to 65 feet. In the river, the total thickness of the overburden soils ranges from approximately 39 to 41 feet. During the 2020 subsurface investigation, a glacial till layer was encountered overlying the bedrock at elevations between approximately 513 feet and 518 feet International Great Lakes Datum of 1955 (IGLD 55). Based on the borings drilled in 2020, the top-of-bedrock elevation varies between approximately 508 feet and 515 feet IGLD 55.

A summary of the 2020 soil data collected for the bulkhead design is included in Appendix C. This summary includes an analysis of the SPT and CPT borings, geotechnical laboratory testing, and in-situ field vane testing in support of selection of the design soil profiles and soil parameters used in this Pre-Final (95%) Design. General descriptions of the soil conditions found during the 2020 investigation program are as follows:

- Fill was encountered below the surficial coverings at the Site extending to 32 feet bgs. The fill consisted of various types of soil, concrete, crushed stone, gravel, other debris, and distiller blow-off. Concrete slabs were also encountered at some locations bgs in the upland area.
- A very soft, alluvial silt layer was encountered in some of the upland borings at a thickness of 7 to 15 feet. Loose to medium-dense lacustrine sand was encountered in the upland areas, either below the alluvial silt or directly below the fill where no alluvial silt was encountered. The sand layer varied in thickness from approximately 5 to 17 feet. The alluvial silt and the lacustrine sand were not encountered in the in-water explorations.

- A soft to medium stiff layer of lacustrine clay was encountered ranging in thickness from approximately 25.0 to 43.5 feet in the upland area and from approximately 15 to 25 feet in the river. The clay ranged in elevation from approximately 534.0 to 546.5 IGLD 55. Results of hydraulic conductivity testing for eight samples collected from the lacustrine clay ranged from a minimum of 3.5×10^{-8} cm/sec to a maximum of 7.0×10^{-8} cm/sec.

A test pit investigation program was conducted at the South Dock from September 11 to 14, 2023, to determine the depth and thickness of the dock's concrete features. The investigation program consisted of dividing the South Dock into five transects with each transect extending from the existing fence to the landward edge of the concrete dock. Two to three test pits were then excavated along each transect with a goal of collecting the following subsurface information:

- Soil thickness on concrete deck;
- Top-of-concrete elevation for the soil-covered deck and the concrete bulkhead at the face of the dock; and
- Concrete deck thickness.

Findings from the South Dock test pit investigation are included in Appendix F and have been incorporated into the design elements for the new bulkhead as discussed in this section.

4.3.2.2 Bulkhead Wall Pre-Final (95%) Design

This section provides descriptions of the bulkhead structural components, the design criteria and assumptions, and the results of calculations performed for the Pre-Final (95%) Design of the steel bulkhead wall. Design elements that have been modified since the Intermediate (60%) Design are highlighted as well as the basis for the modification.

4.3.2.2.1 Bulkhead Structural Components

The steel bulkhead structure will consist of an anchored steel sheet pile wall (USEPA 2023). Cross sections through the wall system are provided on the Design Drawings in Appendix G. The bulkhead system consists of a headwall on the outboard side (riverside) of the existing South Dock waterfront structure and a parallel anchor wall in the upland area behind the South Dock. Based on assumed loading conditions developed for the Pre-Final (95%) Design, a headwall consisting of driven steel sheet piles will achieve sufficient structural capacity. The structural requirements are provided in Section 4.3.2.2.3.

An anchorage system is needed because of the significant lateral loads that will be imposed on the headwall. The anchors will provide lateral support for the headwall, reduce structural demand on the headwall, and limit wall deflections. As shown on the Design Drawings, tie rods will be installed at regular intervals, perpendicular to the headwall and anchor wall alignments (i.e., approximately perpendicular to the river).

For the Intermediate (60%) Design, the anchor wall type and alignment were reviewed, and it was determined that due to the potential subsurface obstructions and to facilitate groundwater recovery in the collection trench, an A-frame anchor wall (i.e., continuous grade beam with H-piles) was the preferred option. However, during the Pre-Final (95%) Design phase, based on 1) input from the VE study for the South Dock bulkhead, 2) further review of the geophysical survey data, and 3) further review of historical records along the anchor alignment, it was determined that a sheet pile anchor wall (similar to the Preliminary [30%] Design anchor system) can be designed to avoid potential subsurface obstructions and therefore is preferred. The anchor wall has been placed 55 feet to 65 feet upland from the South Dock headwall where data indicate a low potential for encountering obstructions during anchor pile installation. Provisions have been incorporated into the Pre-Final (95%) Design to allow for adjustments to the anchor wall alignment or for pre-drilling sheet pile locations should resistance or obstructions be encountered during pile driving. To maintain groundwater recovery behind the anchor wall, the anchor piles require no embedment into the lacustrine clay, the sheets are not sealed, and the anchor wall layout is segmented, with gaps between each segment. The groundwater model includes the anchor wall and showed that the anchor wall does not interfere with the groundwater collection system's ability to maintain the compliance inward gradient of 0.5 foot. This anchor type was also given preference based on the VE study because it will not require installation of a concrete grade beam below grade, minimizing both excavation and dewatering efforts. Accordingly, other anchor options that utilize a concrete grade beam were not advanced in the Pre-Final (95%) Design.

Walers will be used at both the headwall and anchors to transfer loads from the tie rods to the anchor wall and will consist of two parallel steel channel sections. Details of the pile connections and walers are shown on the Design Drawings and discussed further in Section 4.3.2.2.3.

The combination of soft soils over relatively shallow bedrock provides a challenge in terms of achieving passive resistance for the bulkhead wall. The necessary amount of passive resistance to prevent the toe of a wall from "kicking out" (i.e., excessive movement to the point of wall failure) is typically achieved by increasing sheet pile embedment depth into the subsurface materials. Under the assumed loading conditions developed for the Intermediate (60%) Design, it was determined that sheet piles driven to the bedrock surface would provide sufficient passive resistance for the bulkhead. The loading conditions updated for the Pre-Final (95%) Design allow the sheet piles to be driven into the glacial till for sufficient passive resistance and therefore do not require sheets to be driven into bedrock. Additional passive resistance from buttressing is not needed to meet the design requirements and therefore not included in the Pre-Final (95%) Design.

Another design element specific to reducing seepage through the headwall sheet piles includes application of a sealant in the interlocks prior to installation of the sheet piles. Additionally, the use of Larssen-type sheet pile interlocks and connectors (or similar) will be specified to avoid the use of ball-and-socket interlocks. Larssen interlocks provide a tighter connection than ball-and-socket interlocks and are therefore less prone to seepage through the interlocks.

4.3.2.2.2 Design Criteria and Assumptions

The bulkhead wall is designed to form a low-permeability containment barrier to mitigate the flow of groundwater toward the Detroit River. The wall is designed to withstand structural loads, including lateral earth pressures, hydrostatic pressures, and surcharge loads. Sequencing of bulkhead construction assumes that the headwall will be installed outboard of the South Dock followed by placement of backfill beneath the deck up to the bottom of the concrete. Given that the existing anchor components of the South Dock will likely be encountered and removed for installation of the new anchors, this backfill sequence provides protection of the existing Wakefield wall and South Dock during anchor installation of the new bulkhead. After the new anchor system is installed, backfilling to proposed final grades can be completed.

As discussed in the Intermediate (60%) Design, backfill alternatives for the area beneath the South Dock were to be further evaluated for the Pre-Final (95%) Design. The backfill materials were reviewed in concert with assessment of the VE study findings specific to reducing lateral earth pressures and improving constructability. A fill material with a unit weight that is significantly less than that of aggregates or soil could potentially reduce structural demand on the headwall sheet piles, tie rods, and anchor wall due to its lower lateral earth pressure.

Lightweight aggregates and lightweight controlled low-strength materials (“flowable fills”) were found to be suitable for filling behind the bulkhead wall, under water. However, when considering placement methods, it was determined that use of more flowable slurry materials over an aggregate would provide further assurance that the area beneath the deck would be sufficiently backfilled. An LCC material has both the lightweight benefits and preferred flow characteristics and has been selected for backfill beneath the dock. LCC is a mixture of Portland cement and water slurry, combined with preformed foam, that can act as a strong, lightweight, and durable alternative to soil or aggregate fill. Given this material’s high cement content, use of foam, and superior thermal resistance properties, LCC is considered highly resistant to freeze-thaw cycles. The higher air content provides greater thermal resistance to freezing and with increased air, results in less available water within the mass that would be subject to expansion, which is the root cause of deterioration (expansion of water subjecting the concrete to greater internal stresses).

In addition to loads developed from lateral earth pressures, other loads considered for the Pre-Final (95%) Design are described further below (it should be noted that mooring, breasting, and berthing loads are not expected to occur along the South Dock bulkhead and are not considered in the design):

- Hydrostatic Loads: The groundwater collection system was assumed to maintain an inward groundwater gradient along the bulkhead. In consideration of the inward gradient proposed performance standard of 0.5 foot, it was conservatively assumed that the water levels on either side of the bulkhead were the same (i.e., no water level differential) and the net hydrostatic pressure on the wall was zero. Two additional cases were evaluated to assess the bulkhead performance under unusual hydrostatic loads representative of a low river level and during a river seiche event. For the low river level case, it was assumed that the groundwater extraction system was not operational during a low river level event. For the river seiche case, it was assumed that the groundwater extraction system is operational and meeting the compliance gradient when a seiche event occurs in the river.
- Surcharge Loads: Uniform surcharge loads included 250 pounds per square foot (psf) for construction in the static condition and 100 psf for the long-term seismic condition and for the two unusual hydrostatic cases. An added surcharge of 90 psf was included for each case to account for the weight of the concrete deck; this added surcharge was developed based on the findings from South Dock test pit investigations, review of historical records, and the assumption that the existing timber piles at some point no longer provide support for the deck.
- Seismic Loads: For the pseudo-static seismic condition, lateral loads corresponding to a 1 in 975-year event, under post-dredge grades, were developed.

An allowable lateral deflection of 4 inches has been assumed for the headwall. Given the lack of sensitive structures within the zone of influence behind the sheet pile headwall at the South Dock, this deflection criterion is considered acceptable and is consistent with industry standards for bulkhead design and performance. Considering that the interlocks of the headwall sheet piles will be sealed, a review of sealant properties with respect to the allowable deflection was completed. Based on the interlock sealant’s low modulus and high elasticity, an allowable wall deflection of 4 inches will not minimize the sealant’s effectiveness considering its higher allowable strain when compared to steel. The allowable deflection for a sealed interlock is controlled by the allowable strain for steel and not from the sealant properties.

Design calculations for the headwall and anchor loads were performed in general accordance with the United States Army Corps of Engineers (USACE) guidance document titled “Design of Sheet Pile Walls” (USACE 1994a). A combination of methods and software programs were used to complete this Pre-Final (95%) Design.

The Shoring Suite (developed by CivilTechSoftware) computer program was used to complete a limit-equilibrium analysis for a variety of load combinations, to calculate required pile lengths and bending moments, and to provide an initial estimate of wall deflections. The Plaxis 2D finite element program was then used to corroborate the results from the limit-equilibrium analysis and to estimate the wall deflections and associated ground deformations during the various load phases and proposed construction sequence. Plaxis 2D is a more rigorous evaluation using a non-linear approach to soil modeling and considers the effects of soil-structure interaction and staged construction on wall deflections and ground deformations. For assessing global stability of the bulkhead system (i.e., headwall and anchors), the Geostudio 2D SLOPE/W® computer program was used to confirm the minimum factor of safety against global stability met published criteria. Settlement of the soils underlying the South Dock was estimated using the Settle3 software program by RocScience.

The design soil parameters developed based on the subsurface data collected for the bulkhead design are included in Table 8 below.

Table 8. Bulkhead Design Soil Parameters

Layer	Moist Unit Weight, γ (pcf)	Saturated Unit Weight, γ (pcf)	Effective Friction Angle, ϕ' (degree)	Effective Cohesion, c' (psf)	Undrained Shear Strength, s_u (psf)	Wall Interface Friction Angle, δ (degree)
Existing Fill and General Fill	120	120	32	---	---	17.3
Lightweight Cellular Concrete	90	90	35	350	---	18.9
In-River Fill/Sediment Mix	---	105	26	---	---	14.0
In-River Upper Silty Clay	---	120	27	---	400	9
In-River Lower Silty Clay	---	120	27	---	800	9
Glacial Till	---	135	28	500	1,500	12
Bedrock	---	160	45	---	---	6

Note:
pcf = pounds per cubic foot

Using guidelines provided in USACE 1994a, factors of safety for calculation of wall embedment depths were selected based on the loading case, type of loading, and type of soil. The walls were designed using usual, unusual, and extreme loading cases per USACE design procedures (USACE 1994a). These loading cases correlate with the likelihood of the load occurring. More severe and less likely loading cases are generally assigned smaller factors of safety than less severe loading cases that occur regularly under normal operating conditions.

To avoid compounding factors of safety, the structural components (i.e., the sheet pile sections) were designed using a factor of safety of 1 on the soil strengths to calculate the forces and moments with the factor of safety applied to the structural steel strength. Allowable stresses for steel for the load cases (usual, unusual, and extreme) were calculated per USACE design procedures (USACE 1994a). To calculate required embedment depths, the analyses were then repeated, applying the appropriate factor of safety on the passive earth pressure side.

A global stability evaluation was completed for the Pre-Final (95%) Design of the bulkhead to assess the external stability of the anchor system and confirm there is a sufficient factor of safety against a global system failure. Using guidelines provided in the USACE guidance document titled "Engineering and Design Floodwalls and Other Hydraulic Retaining Walls*"* (USACE 1994b), the target minimum factor of safety for the short-term undrained case is 1.30. For the long-term condition, a factor of safety of 1.40 is required. Global stability analyses were first performed based on the controlling sheet pile embedment determined from the bulkhead analyses for the headwall and anchor piles. If the factor of safety for this initial condition was less than target, the sheet pile tip elevation was lowered until the failure surface was deep enough to reach the target factors of safety.

Further details of the analysis methodology and assumptions are provided with the bulkhead wall calculations in Appendix N.

4.3.2.2.3 Results of the Pre-Final (95%) Design Calculations

Design calculations for the headwall and anchor wall components of the new bulkhead are included in Appendix N. The purpose of the calculations was to determine the following:

- Structural demand on the headwall;
- Wall type;
- Required headwall pile embedment;
- Required anchor load; and
- Anchor wall configuration, including required tie rod diameter/spacing and sheet pile structural section/embedment.

To determine the minimum structural requirements, the bulkhead was divided into four design cases that are representative of the critical sections of the headwall determined by maximum height, limits of LCC backfill, thickness of river sediments (i.e., greatest depth to clay surface), and anchorage requirements. Based on the results for each design case, the minimum structural requirements are based on the controlling, maximum design height scenario. The minimum structural requirements for the Pre-Final (95%) Design are summarized in Table 9.

Table 9. Summary of Minimum Bulkhead Structural Requirements for Pre-Final (95%) Design

Structural Component	Description/Parameter	Type/Value
Steel Headwall	Wall Type	Steel sheet piles
	Steel Grade	Grade 50 (ASTM A 572)
	Minimum Section Modulus (cubic inches per foot)	66.8
	Top-of-Wall Elevation (feet IGLD 55)	577.1
	Pile Section and Length (feet)	AZ 36-700; approximately 62 (driven into glacial till)
	Wale	Continuous double W12×50

Structural Component	Description/Parameter	Type/Value
Anchor Wall	Wall Type	Steel sheet piles
	Steel Grade	Grade 50 (ASTM A 572)
	Top-of-Anchor Wall Elevation (feet IGLD 55)	577
	Pile Section and Length (feet)	AZ 24-700, ±25
Steel Tie Rods	Steel Grade	Grade 150 (ASTM A 311, ASTM A 722, or American Association of State Highway and Transportation Officials [AASHTO] No. M275, Grade 150)
	Design Capacity (kilo pounds [kips])	192
	Tie Rod Diameter (inches)	2¼
	Tie Rod Length (feet)	Approximately 55 (from face of headwall through anchor wall)
	Tie Rod Spacing (feet)	13.79

4.3.2.3 Construction Methods and Constructability Considerations

This section describes the construction methods, constructability considerations, and anticipated construction sequence for the steel bulkhead along the South Dock. The Draft Construction Quality Assurance Plan (Appendix O) describes quality assurance controls to monitor and verify that the steel bulkhead wall is installed in accordance with specification requirements. The use of construction quality monitoring results to adapt the construction methods and/or the construction monitoring methods may be needed to achieve installation of the steel bulkhead wall. These items are further discussed below where appropriate.

4.3.2.3.1 Pile Installation

Sheet piles will likely be driven into the subsurface soils using a vibratory hammer. A crane will be required to lift the piles in position for driving. Driving the piles through the relatively loose and soft native subsurface soils is anticipated to be relatively easy and seamless. A minimum penetration of 2 inches into the glacial till layer is considered a sufficient embedment for stability. Where greater embedment is required into the glacial till, driving shoes welded to the sheet pile tips are required.

Difficult conditions may be encountered during sheet pile driving for the anchor wall in the upland fill soils, which contain various types of debris, including concrete, slabs, and former foundations. Similar to the subsurface barrier, the sheet pile section for the anchor wall was selected based on drivability considerations and a structural strength criterion, exceeding the minimum required section modulus as determined from design evaluations. The anchor wall has been positioned outside the zones of potential obstructions as identified from the geophysical surveys. To the extent possible, the contractor will locate and remove any remaining debris and other obstructions prior to pile driving. The contractor will be prepared to remove subsurface obstructions through a combination of surficial excavation and pre-drilling at pile locations through the fill soils. Based on experience during the 2020 geotechnical investigation, subsurface obstructions are more prevalent in the upland fill soils. CPTs conducted in the upland areas frequently encountered debris and difficult conditions in the fill materials. The CPT probe generally encountered less resistance and fewer obstructions along the river bottom.

The bulkhead headwall will be installed from the riverside using deck barges. For the Intermediate (60%) Design, the possibility of headwall installation from the upland side of the South Dock was considered. Findings from the VE study, however, suggested that pile installation from the riverside would be preferred given the condition of the South Dock and the crane boom constraints when considering the length of the headwall piles.

A sealant will be applied to the sheet pile interlocks prior to sheet pile installation. Various interlock sealants are commercially available and are routinely applied by contractors or fabricators.

4.3.2.3.2 Anchor System Components

After the headwall and anchor piles are in place, walers will be installed on the headwall for load transfer from the tie rods to the sheet pile anchors. Details of the headwall wale system and the anchor wale system are included on the Design Drawings (Appendix G). Waler installation will involve some welding and bolting.

The tie rods will be installed at a relatively shallow depth below the ground surface, within the soil cover on top of the concrete deck of the existing South Dock. This installation will require excavating trenches between the headwall and anchor wall. The riverside edge of the existing concrete deck consists of a “concrete kneewall” that retains the soil on the concrete deck. The contractor will need to cut notches into the concrete kneewall to allow penetration of the tie rod through the concrete bulkhead. Drilling holes through the concrete kneewall is another option for tie rod installation across the dock.

The tie rod headwall connection will be near the top of the headwall (approximate elevation of 575 feet IGLD 55) to keep the tie rods above the concrete deck and the connection points above the river water surface elevation to avoid underwater welding. At the anchor wall, the connection point will continue at the same elevation and intersect the sheet pile anchors at approximately 12 inches below the top.

4.3.2.3.3 Backfill Placement

Because the concrete deck of the existing South Dock structure will remain in place, backfill to fill the void behind the new bulkhead wall, underneath the concrete deck, must be placed either through a tremie pipe hydraulically or using gravity. Access points must be created through the concrete deck by first removing some of the soil on the deck to expose the concrete and then cutting holes into the concrete. The size and spacing of the access points will be determined based on the depth to the existing sediment surface, with a tighter spacing anticipated for those areas of deeper water. There is also the potential of the initial backfilling being conducted via the opening between the new sheet pile headwall and the existing concrete deck. The contractor will be required to confirm the backfill level below the dock through volume calculations and survey means.

As discussed in Section 4.3.2.2.2, backfill materials beneath the deck will consist of LCC. Placement of the LCC will induce consolidation settlement in the underlying clay unit; however, when considering the low density of the LCC, a lower amount of settlement post-construction is expected compared to settlement from placement of traditional backfill materials. Settlement was estimated assuming placement of LCC from the existing sediment surface to the bottom of the concrete deck. For this initial scenario, a maximum total settlement of approximately 0.6 to 1.2 inches was estimated from placement of LCC, with approximately 0.5 inch of settlement occurring immediately in the sediment layer during construction, followed by consolidation settlement of approximately 0.75 to 1 inch in the underlying clay layer post-construction.

The design assumes that the weight of the concrete dock and overlying soils will transfer onto the LCC backfill surface. Settlement evaluations for this scenario estimated an additional maximum vertical displacement of approximately 1.8 to 4.9 inches. For both scenarios evaluated, the estimated settlement ranges are considered minimal and are not expected to impact the performance of the headwall and the anchor systems. Placement of additional LCC beneath the dock post-construction is also not anticipated based on the low estimated settlement. The settlement calculations are included in Appendix N.

Should settlement exceed the estimated ranges, and a gap beneath the deck and the surface of the LCC develop, the existing deck might develop cracks as the timber piles deteriorate, and some minor ground settlement may occur above the concrete deck. This scenario would then be treated as part of maintenance, which may include regrading of the surface. This scenario is not considered a safety or stability concern because the new bulkhead structure is designed to carry the full weight of the backfill and the old concrete deck.

4.3.2.3.4 Existing Subsurface Structures and Stability of Existing South Dock

As previously discussed, there are various existing structures that either will stay in place or will need to be removed during bulkhead construction. Recent investigations at the South Dock have helped to define the limits of the existing structures and have allowed the barrier design to address these features. The top of the existing concrete kneewall along the face of the South Dock varies in elevation from approximately 575.1 to 577.2 feet. Installation of new tie rods for the bulkhead requires cutting notches into this concrete kneewall and backfilling the openings with granular material.

The existing Wakefield wall is anchored via a series of tie rods and existing timber piles. The need to maintain the anchoring for the existing Wakefield wall during construction has been considered in the design of the new bulkhead. The alignment of the sheet pile anchor wall is within the anchor zone (existing timber piles and tie rods) of the Wakefield wall to avoid substantial historical foundations. The stability of the Wakefield wall must be maintained during construction of the new anchored bulkhead structure until the gap behind the bulkhead is backfilled sufficiently to provide adequate support. Based on evaluations completed for this Pre-Final (95%) Design, placement of LLC to the bottom of the concrete deck is required before installation of the anchor cap and removal of existing anchorage components of the Wakefield wall.

4.3.2.4 Alternate Design Elements and Construction Methods

The South Dock anchored wall design presented in the Pre-Final (95%) Design includes the following key elements: 1) a sheet pile headwall driven into the glacial till with Larssen interlocks and hydrophilic sealant, 2) a deadman anchor wall consisting of unsealed steel sheet piles installed inland from the headwall, and 3) LCC as backfill under the existing dock. Alternate design elements considered as part of the Preliminary (30%), Intermediate (60%), and Pre-Final (95%) Designs include rock socketing of king piles, installing a reinforced anchor wall as a slurry wall, installing an A-frame combination batter pile and grouted tieback anchor cap, using lightweight fill as backfill behind the bulkhead wall, and buttressing. The Pre-Final (95%) wall design was selected after considering the various alternate design elements, construction methods, VE evaluations, constructability reviews, and site constraints.

4.4 Existing Steel Bulkhead

The existing steel bulkhead extends approximately 3,243 feet north of the South Dock. It was constructed in the mid-1990s and consists of interlocking steel sheet piles that are approximately 40 to 45 feet deep and embedded into the clay layer. This bulkhead is positioned riverside of the original wooden bulkhead (Wakefield wall) and concrete seawall (where present). Historical drawings do not indicate that the interlock joints were sealed or that corrosion protection measures (i.e., coatings, cathodic protection) were applied to the piles. However, hydraulic testing and groundwater modeling completed as part of the pre-design investigation concluded that this bulkhead is an effective barrier to groundwater flow (Arcadis 2021). The groundwater model is discussed further in Section 5.1.1 and in Appendix B.

4.4.1 Previous Bulkhead Inspections

In May 2018, Arcadis conducted a visual inspection of the shoreline structures at the Site, including the existing bulkhead north of the South Dock. The visible portions of the piling above the water surface were inspected. General findings from this inspection concluded that the existing bulkhead was in good, stable condition. The existing sheet piles appeared to be in proper alignment with no signs of rotation or other failures. The visible portions of the steel bulkhead and its components (pile caps, waler, tie rods) also appeared to be in good condition with no significant signs of corrosion or steel section loss (Arcadis 2018).

An additional visual inspection of the bulkhead was conducted in August and September 2023 to identify and address any areas noted in the 2018 inspection as deficient or in need of repair from a containment barrier perspective. This inspection was completed of the entire existing bulkhead from the northern end of the South Dock to the northern end of Perry Place. It included observations of sheet pile conditions above and below the water surface and estimated the sheet pile thickness using ultrasonic methods at 50-foot intervals along the length of the existing bulkhead. Locations of utility penetrations, surficial corrosion, and other inspection findings were noted and photographed as part of this diver survey. No sediment sampling was conducted as part of the diver survey. Results are summarized in Appendix D, which includes a figure of photograph locations, a photograph log, results of pile thickness measurements, and the diver inspection field notes. Note that the diver inspection included a small section of City of Wyandotte owned sheet pile wall at the end of Perry Place, which is not BASF's responsibility and is not included in the perimeter barrier remedy.

4.4.2 Conditions of Each Existing Bulkhead Section

This section discusses in further detail the historical and current conditions of the existing bulkhead portion of the barrier system. The existing bulkhead portion of the barrier system is approximately 3,243 feet and is typically described by sections that reference the historical facility feature, namely the Light Dock, Heavy Dock, and North Central Shoreline. The Design Drawings (Appendix G) indicate the approximate limits of each historical shoreline feature. The approximate overall alignment of the historical seawall (also referenced as the Wainright Wall) along the existing bulkhead is also shown on the Design Drawings.

Arcadis reviewed available design and record drawings for the existing bulkhead, historical aerial photographs, logs from explorations completed upland of the existing bulkhead, and findings from site inspections and the 2023 underwater diver survey. Findings from this review are further summarized by bulkhead section below.

4.4.2.1 Former Light Dock Section

4.4.2.1.1 Current and Historical Construction Features

The northernmost section of the shoreline is referenced as the former Light Dock and extends from its intersection with Perry Place for approximately 283 feet to the south. Based on review of BASF design drawings (BASF Design Drawing No. T-50805, North Section Seawall Details, October 1995), this section of the existing bulkhead consists of an anchored steel bulkhead. The headwall of this bulkhead includes 40-foot-long AZ-13 sheet piles with tie rods spaced approximately 8.9 feet on center. The tie rods extend to an AZ-13 sheet pile anchor wall that is offset between 30 feet and 70 feet from the bulkhead.

The existing bulkhead through the former Light Dock section was constructed riverside of the former concrete seawall. The concrete seawall was supported by 12-inch-diameter wood piles, spaced 6 feet on center, with 4-inch wood plank piles for lagging. Segments of the concrete seawall were removed to depths required for installation of the tie rod system, and the broken concrete was to be used as fill material.

The Design Drawings (Appendix G) include a plan and profile for this section of the existing bulkhead as well as a typical cross section that depicts the existing steel bulkhead and former dock features.

4.4.2.1.2 Findings from Visual Inspections and Diver Survey

Visual inspection of the existing bulkhead indicated no signs of rotation and that the sheet piles are in good overall condition. Site observations also indicated that fill material is in place behind the existing bulkhead, extending up to the underside of the sheet pile cap throughout most of this Light Dock section. Remnants of the former concrete seawall are exposed at the ground surface and provide an indication of the existing bulkhead tie rod locations. Areas of localized erosion have been observed in this section of the existing bulkhead. This erosion is likely a result of loss of material through open pile lift holes that were not completely sealed with metal plates during construction of the steel bulkhead.

Findings from the diver survey for this section confirmed the locations of three existing pipe penetrations: a 24-inch-diameter pipe opening for the 001 outfall, a 36-inch-diameter pipe opening for the pump house intake, and a 36-inch-diameter outer pipe opening for a 24-inch-diameter inner pipe at the 002 outfall. Other than a number of open lift holes and leakage observed at three waler bolt locations, no other openings, holes, or gaps were noted during the diver survey along the existing BASF bulkhead that will be included as part of the perimeter barrier. No observations of soil leakage through the wall or shoaled materials outside the wall were noted during the diver survey.

Results of the thickness testing for the Light Dock section ranged from 0.338 inch to 0.424 inch. The thickness of steel for the AZ-13 sheet pile section is 0.375 inch, and with the sheet pile manufacturing tolerance of 6%, the steel thickness of the sheet piles could have ranged from 0.353 inch to 0.398 inch. The majority of the thickness measurements collected during the diver survey fall within this tolerance range. As shown on the data charts included in Appendix D, one thickness measurement point within the Light Dock section is below this tolerance range and eight points are above it. The measurement point below the tolerance range could indicate a loss of thickness from corrosion. For those points above the tolerance range, multiple soundings were made to confirm that the probe was reading the steel thickness correctly and that no other anomalies were impacting the measurements. Overall, these thickness measurements support the conclusions from the visual inspection that the sheet piles are in good structural condition.

Anticipated bulkhead repair measures based on the 2023 diver survey and above-water visual observations through the former Light Dock section are shown on the Design Drawings (Appendix G). Repairs through this section will generally include placement of metal plates at the lift holes, sealing between existing pipe penetrations and surrounding sheet piles, sealing of wale bolt connections showing signs of leakage, and placement of fill in eroded areas to the original design grade.

4.4.2.2 Former Heavy Dock Section

4.4.2.2.1 Current and Historical Construction Features

South from the former Light Dock section, the shoreline continues as the former Heavy Dock for approximately 1,275 feet. This section of the existing bulkhead was constructed in 1995. Based on review of design drawings (BASF Design Drawing No. T-50804, North Section Seawall Details, September 1995), it consists of an anchored steel bulkhead that includes 45-foot-long AZ-13 sheet piles with tie rods spaced approximately 8.8 feet on center. The tie rods extend to an AZ-13 sheet pile anchor wall that is positioned between 40 and 70 feet inland of the bulkhead.

The configuration of the former Heavy Dock is shown on BASF Design Drawing No. 2247 (Unloading Dock Reconstruction, February 1917) and No. 40102 (Composite Drawings, May 1990). Based on a review of these drawings, this dock appears to have consisted of two concrete structures that were spaced approximately 27 feet apart, center to center. Between the two concrete structures were rail tracks used for transportation of materials unloaded from vessels moored along the dock. Both concrete structures are shown as being pile supported, and given the 1917 original construction time frame, were likely founded on timber piles. The material beneath the tracks is noted as cinder fill. A typical cross section for the Heavy Dock section is shown on the Design Drawings (Appendix G).

The riverside concrete structure of the former Heavy Dock included a timber sheet pile wall beneath it. The position of this timber sheet pile wall was estimated to be approximately 12 feet from the riverside face of the concrete dock structure and is shown on both BASF drawings referenced for the former Heavy Dock features. This sheet pile wall separated and retained the fill material beneath the tracks from the Detroit River. Beneath the concrete structure east of the timber sheet pile wall, the area would have been open to the surface water of the Detroit River.

The existing bulkhead through the former Heavy Dock section was constructed riverside of the historical concrete dock structure. The outlines of the pile-supported concrete structures are shown on the bulkhead section on BASF Design Drawing No. T-50804. Fill material has been placed above the concrete dock structures to elevate the overall grade to approximate current conditions. The BASF design drawings do not indicate placement of fill below the easternmost concrete hoist foundation of this dock (i.e., between the existing steel bulkhead and the timber sheet pile wall). To investigate the limits of potential voids, a subsurface investigation was conducted in October and November 2024 (Arcadis 2024b). This investigation confirmed the presence of voids beneath the easternmost foundation. It also provided subsurface information on the limits of fill material along and beneath the foundation, and the thickness and general quality of the concrete foundation. The Design Drawings (Appendix G) include a plan and profile for this section of the existing bulkhead as well as a typical cross section that depicts the existing steel bulkhead and former dock features.

4.4.2.2.2 Findings from Visual Inspections and Diver Survey

Visual inspections of the existing steel bulkhead through the former Heavy Dock section showed that the wall alignment is in good condition, as are the visible portions of the steel bulkhead components (steel sheet pile, cap, waler, and tieback rods). Soil erosion has been observed throughout the length of the former Heavy Dock and typically to depths between 6 and 12 inches below the underside of the pile cap. However, several locations of deeper washout holes (6 to 12 inches in diameter) have been observed, exposing portions of the tiebacks and walers of the steel bulkhead. Washout holes extending to depths of 2.5 feet deep were observed. The concrete of the former Heavy Dock structure was not observed in the washout holes, indicating a likely soil cover in this area greater than 2.5 feet thick.

No pipe penetrations were observed during the diver survey of this section of the existing bulkhead. The diver survey did note eight holes in the sheet piles that were visible above the water surface at the time of survey. These openings were typically pile lift holes, approximately 2 inches in diameter, that had not been sealed after wall construction. Leakage through the sheet pile interlocks was observed at four locations above the water surface and at two openings adjacent to the wale bolts. No soil leakage through the wall or shoaled materials outside the wall were noted during the diver survey.

With respect to corrosion or section loss, the diver survey noted nine areas of corrosion across this section, with two of the areas described as containing significant or heavy corrosion. The areas described as having heavy corrosion are typically observed to be pitted or starting to show signs of pitting and scaling, and delamination is present. The areas noted in the diver survey as corroded are typically areas that show discoloration or rust.

Results of the thickness testing for the Heavy Dock section ranged from 0.343 inch to 0.426 inch. As discussed for the Light Dock section, the installed steel thickness for the AZ-13 pile could have ranged from 0.353 inch to 0.398 inch. The majority of the thickness measurements through the former Heavy Dock section fall within this tolerance range. Five thickness measurement points are below this tolerance range and could be indicative of thickness loss from corrosion. For the soundings above the tolerance range, the diving team confirmed that the probe was reading the steel thickness correctly by taking multiple soundings. Overall, the thickness soundings through the former Heavy Dock section support the conclusions from the visual inspection that the sheet piles are in good structural condition.

Anticipated bulkhead repair measures based on the 2023 diver survey and above-water visual inspection through the former Heavy Dock section are shown on the Design Drawings (Appendix G). Repairs through this section generally include placement of metal plates at the lift holes and sealing of interlocks showing signs of leakage. Areas of heavy corrosion will be further assessed for appropriate surface treatments or installation of metal plates, dependent on the depth of the impacted area (i.e., above or below the river water surface). The design will incorporate placement of fill to the original design grade and measures for managing stormwater runoff along the existing bulkhead.

4.4.2.3 North Central Shoreline Section

The southernmost section of the existing bulkhead has been referred to as the North Central Shoreline in previous reports. This section of the existing bulkhead is the longest and estimated to extend south of the former Heavy Dock for approximately 1,840 feet until its intersection with the existing South Dock.

4.4.2.3.1 Current and Historical Construction Features

The bulkhead along the North Central Shoreline is an anchored steel bulkhead constructed between 1990 and 1992. Arcadis reviewed several BASF drawings referenced for this section of the existing bulkhead, including No. 42808 (Dock Renovations – 1988), No. 52197 (1994 Dock Renovations), No. 50369 (1992 Dock Renovations), and No. 40102 (Composite Drawings North Works Dock, May 1990). Based on a review of these drawings, the existing bulkhead appears to have been constructed riverside of a seawall that consisted of a concrete cap over oak sheet piling. Drawings indicate placement of fill between and above the historical seawall.

The existing bulkhead along the North Central Shoreline consists of a combination of AZ-13 and BZ-12.1 sheet piles. The AZ-13 sheet piles extend north from the former Heavy Dock and are shown to be installed to depths of 40 feet. BASF drawings show the BZ-12.1 sheet piles extending from the South Dock, installed to depths of 30 feet and with a steel thickness of 0.375 inch. The tie rods are spaced approximately 7 feet to 9 feet on center and extend to a sheet pile anchor wall that is positioned between 16 and 70 feet inland of the bulkhead.

The Design Drawings (Appendix G) include a plan and profile for this section of the existing bulkhead as well as a typical cross section that depicts the existing steel bulkhead and former seawall features.

4.4.2.3.2 Findings from Visual Inspections and Diver Survey

The North Central Shoreline changes orientation at various points along its length. Findings from previous inspections concluded that the steel bulkhead appears to be in good condition and shows no signs of rotation or other failures. Previous surveys have noted ponding of water along the entire length of this section, causing soil erosion through open lift holes and exposing the lift holes and tie rods/wales in some areas behind the wall. The top elevation of the water ponding noted in the 2018 inspection was equivalent to the underside of the pile cap flange.

One pipe penetration was observed during the diver survey of this section of the existing bulkhead. Designated as the 003 outfall, the size of this penetration was not noted because a steel plate covered the opening to divert the outfall water toward the river's mudline. The diver survey notes indicate that several open lift holes were observed along this section of the existing bulkhead. Water was observed flowing through the lift holes with signs of mineral buildup and soil staining at several holes. Consistent with the other bulkhead sections, these openings were typically 2 inches in diameter and positioned in the sheet pile web above the river water surface. Other openings noted also appeared to be pre-drilled holes, of a similar diameter, but positioned within the tongue-and-groove portion of the sheet pile. These openings were often offset from each other but through the same interlock and observed at approximately 21 locations along this section. Leakage was also observed through the openings for the bulkhead anchorage system. Water was observed flowing from the tie rods and/or bolt connections at approximately 36 locations along this portion of the existing bulkhead. No leakage, however, was noted through the sheet pile interlocks. No observations of soil leakage through the wall or shoaled materials outside the wall were noted during the diver survey of the existing bulkhead.

With respect to corrosion or section loss, the diver survey noted several areas of corrosion across this section, with most of the corrosion observed at open lift holes or at wale bolt connections. Two areas not associated with lift holes were noted as having heavy corrosion. These areas were noted at depths equivalent to the water surface at time of inspection and at a depth near the sediment surface.

Results of the thickness testing for the North Central Shoreline section ranged from 0.31 inch to 0.426 inch. As discussed for the previous existing bulkhead sections, the installed steel thickness for the AZ-13 pile could have ranged from 0.353 inch to 0.398 inch. Assuming the same tolerance for the BZ-12.1 sheet piles installed in this section, the majority of the thickness measurements fall within this tolerance range (see data charts in Appendix D). Twelve thickness measurement points are below this tolerance range and could be indicative of steel thickness loss from corrosion. For the soundings above the tolerance range, the diving team confirmed that the probe was reading the steel thickness correctly by taking multiple soundings.

Anticipated bulkhead repair measures based on the 2023 diver survey and above-water visual inspection through the North Central Shoreline section are shown on the Design Drawings (Appendix G). Repairs through this section generally include placement of metal plates at the lift holes and other noted openings through the sheet piles. Sealing at the pipe penetration for the 003 outfall is included in these repairs but it is recognized that field adjustments to the repair details may be required pending the configuration of the opening through the sheet pile. Repairs will also include sealing of tie rods and wale bolt connections showing signs of leakage and fill placement in eroded areas to the original design grade.

4.4.2.4 Overall Findings and Recommendations

In general, the overall observations from the recent diver survey are consistent with the 2018 findings. The existing sheet piles appear to be in proper alignment with no signs of rotation or other failures. Noted openings above the water line (i.e., at pile lift holes) from both inspections are continual sources of surface water runoff and attributable to the areas of surface erosion and material loss observed behind the majority of the existing bulkhead. Locations on the steel bulkhead where repairs are needed, along with details of the specific repair measures, are indicated on the Design Drawings (Appendix G).

Arcadis performed a desktop evaluation of the estimated corrosion rates for the existing bulkhead in support of the Intermediate (60%) BOD Report. The purpose of the evaluation was to assess the anticipated lifespan of the existing sheet piles from a steel thickness loss aspect. Thickness loss rates were determined using Eurocode 3 – Design of Steel Structures – Part 5: Piling (European Committee for Standardization 2007), which provides loss rates based on environmental conditions typical of industrial sites. The existing sheet pile wall is approximately 30 years old. The estimated corrosion loss over 30 years is approximately 0.05 inch, or 15% of the pile’s original steel thickness of 0.375 inch.

The steel thickness test results from the diver survey indicate that, with the exception of two thickness test results, the sheet piles have not experienced loss of steel in excess of the expected corrosion loss rate noted above. The data in Appendix D indicate that the average corrosion loss rate over the last 30 years is approximately 8% compared to the expected 15% from the desktop evaluation.

From a containment barrier remedy aspect, the anticipated corrosion losses would not compromise the steel wall’s ability to be an adequate barrier to groundwater flow; however, the structural capacity of the steel section and the potential for the bulkhead’s failure from excessive rotation or bending could be of concern. For piles where corrosion losses do progress during the pile lifespan, the expected failures would be localized areas of bulging or cracking/splitting of the sheet pile section. An effective monitoring and maintenance program can optimize the lifespan of the existing bulkhead by identifying and addressing these types of failures on a case-by-case basis. In addition, incorporating a cathodic protection system is an option and will be considered should the corrosion losses impact the performance of the barrier. Monitoring and maintenance requirements will be in place for the existing bulkhead as discussed in the OMM Manual (Appendix K).

Findings from the subsurface investigation of the former Heavy Dock have confirmed the presence of voids beneath the easternmost hoist foundation. Data collected during the investigation confirm that fill material is in place between the existing steel bulkhead and the easternmost hoist foundation, to the depth of the lacustrine clay layer. This material is beneficial to the shoreline barrier as it provides support of and protection to the existing wale and tie rods and is an improvement to the watertightness of the sheet pile. The investigation also provided data regarding the hoist foundation, including the thickness of the concrete, information on concrete resistance based on observations during drilling, and borehole videos showing the presence of voids or cracks in the concrete. Conclusions from these observations indicate minimal deterioration of the concrete over the lifespan of the foundation.

Based on the positive findings from the former Heavy Dock investigation, the good condition of the existing steel bulkhead determined from the diver survey findings, and the stable conditions observed during the recent inspections of the Heavy Dock limits, the former Heavy Dock area will be monitored and maintained as recommended in the OMM Manual (Appendix K). This area, in combination with upland areas of all shoreline barrier walls, will be monitored for settlement, subsidence/voids, and loss of vegetation that would trigger maintenance repair activities specific to the location and observation.

4.5 Barrier Intersections

Construction of the barrier remedy will require design of appropriate transition zones from one barrier type to the next. These transition zones is designed so that the intersections of the barrier types are properly overlapped or sealed and provide for a continuous barrier along the downgradient perimeter of the Site.

There are three intersections along the network. The northernmost connection is between the existing steel bulkhead and the subsurface sheet pile barrier wall. For this intersection, the connection proposed in the Intermediate (60%) Design has been modified based on further inspection of the interlock configuration of the existing sheet pile. It was determined that installation of a connector pile would provide an appropriate transition between the existing and proposed sheet piles in lieu of jet grout columns. The new connector pile would be installed to the same depth as the subsurface barrier piles (i.e., minimum embedment of 3 feet into the clay layer).

For the South Dock area, headwall piles of the new bulkhead will intersect with the existing bulkhead at the dock's northern end. It is also proposed to connect the existing and new piles with a sheet pile connector in lieu of the jet grout columns proposed in the Intermediate (60%) Design. At the dock's southern end, the headwall piles will be connected via a sealed interlock to the subsurface sheet pile barrier wall. Details for each intersection are included on the Design Drawings (Appendix G).

5 Groundwater Extraction and Conveyance System

The proposed groundwater extraction and conveyance system is designed to support the containment barrier remedy, providing effective groundwater capture and management across the Site. The system consists of 23 collection drains, one vertical extraction well, and a comprehensive conveyance network. The extraction system design was informed by detailed groundwater modeling, which simulated steady-state and transient conditions. The updated groundwater flow model demonstrated full capture of groundwater along the perimeter under steady-state conditions, with an average flow rate of 42.4 gpm. Transient modeling evaluated compliance during storm events, predicting that while the system can handle most scenarios, temporary non-compliance may occur during extreme events. Modeling of these conservative scenarios overestimated water volumes, because planned stormwater management measures will reduce water flux.

The groundwater extraction system consists of collection drains with perforated high-density polyethylene (HDPE) pipe installed at targeted depths, backfilled with high-permeability materials, and equipped with cleanout risers for maintenance. Each drain terminates at a sump, which uses a double-barrel design with redundant pumps to minimize downtime due to equipment (pump) failure. The conveyance network consists of buried and above-grade piping to transport groundwater to the AG treatment system. The network incorporates three header pipes for flexibility, including a slipstream option for groundwater requiring less treatment. The collection and conveyance network alignment avoids subsurface obstructions and utilities to the extent possible and leverages one-pass trenching technology to simplify construction and minimize dewatering needs.

The BOD for the groundwater extraction and conveyance system is described in this section. The layout of the conveyance system is shown on Figure 3. Piping and instrumentation diagrams and drain and sump details are provided with the Design Drawings (Appendix G).

5.1 Groundwater Extraction System

This section provides an overview of the groundwater modeling results used to inform the Pre-Final (95%) Design and the resulting groundwater extraction system design basis.

5.1.1 Summary of Groundwater Modeling Results

To support the Pre-Final (95%) Design of the barrier remedy, the numerical groundwater flow model of the localized groundwater flow system at the Site (Appendix B), originally developed by Waterloo Hydrogeologic in 2002 (Waterloo Hydrogeologic 2002), was updated. The groundwater model was updated to reflect the Pre-Final (95%) Design of the barrier remedy, transient conditions known to exist in the Detroit River, and discrete storm events that would stress the perimeter barrier system. Model updates included boundary conditions, hydraulic conductivity values, lacustrine clay surface, and applied recharge, as well as the inclusion of the non-continuous deadman sheet pile anchor wall. The drain alignments were adjusted to further avoid subsurface obstructions and utilities as identified by the geophysical surveys and historical records review. The objective of updating the groundwater flow model was to simulate localized groundwater dynamics at the Site to support the design of the barrier remedy. The model updates and simulation of the Pre-Final (95%) Design of the perimeter barrier remedy are summarized in Appendix B.

The model was calibrated to both steady-state and transient conditions by systematically adjusting the model boundary conditions and input parameters to obtain as close a match as possible between observed and simulated water levels. The model was calibrated under steady-state conditions using 100 groundwater level measurements collected in June 2024 from locations distributed throughout the Site. Sensitivity analyses for parameters adjusted during the calibration process were conducted (recharge, horizontal hydraulic conductivity, and vertical hydraulic conductivity). Validation of the calibrated groundwater model was accomplished by incorporating aquifer storage into the model and simulating groundwater responses to a precipitation event with variation in Detroit River elevations from August 22 through September 6, 2020, when transducers were deployed in 14 site monitoring wells. Based on both quantitative (e.g., calibration statistics) and qualitative (e.g., groundwater flow directions) data, the groundwater flow model was determined to be well calibrated under both steady-state and transient conditions as described further in Appendix B.

The perimeter barrier remedy was evaluated under both steady-state and transient conditions using the updated model. At steady-state conditions, the perimeter remedy exhibits full capture of groundwater along the perimeter boundary under an inward gradient of 0.5 foot below the monthly average elevation of the Detroit River. The steady-state flow rate for the drain system is 42.4 gpm. The perimeter drain induces a vertical gradient in the units below the fill such that deeper groundwater in the sand unit is captured by the drain system.

The model was run under transient conditions to evaluate the system performance under the following:

- Steady-state recharge changing Detroit River elevations for a full year; and
- Two discrete storm events (August 2021 and August 2020) where both recharge and the Detroit River elevation were changed over time.

These scenarios consider how the perimeter barrier remedy would operate during the selected period if it were in place at that time. Each scenario assumes no capping or stormwater management in the vicinity of the collection trenches and sump operation at a 0.5-foot monthly average inward gradient (conservative assumptions).

For the year-long transient run evaluating changes to the Detroit River, the maximum modeled flow rate is approximately 104 gpm. The modeled influent flow rate never exceeds the AG treatment system design capacity.

For the two discrete storm events modeled, the model predicts the system would be able to maintain compliance for one event (August 2021), but would potentially temporarily be out of compliance for the other (August 2020):

- August 2021 Results: This event included one smaller precipitation event and one larger precipitation event with a total rainfall amount of 3.86 inches. The maximum modeled flow rate for the August 2021 event was approximately 376 gpm. The overall modeled average influent flow rate over the approximately 7-day period in August 2021 was approximately 117 gpm. Although the influent flow rate exceeds the AG treatment system design capacity, the excess water would be sent to storage. The estimated volume of water above the treatment system capacity is approximately 204,000 gallons, which is below the combined storage tank volume of 300,000 gallons. Therefore, the system would be able to maintain compliance during this event
- August 2020 Results: This event included back-to-back precipitation events with a total rainfall amount of 4.4 inches. The maximum modeled flow rate was approximately 471 gpm. The overall modeled average influent flow rate over the approximately 15-day period was approximately 127 gpm. Although the modeled influent flow rate exceeds the AG treatment system design capacity, the excess water would be sent to storage. The estimated volume of water above the treatment system capacity is approximately 440,000 gallons, which exceeds the combined storage tank volume of 300,000 gallons. Therefore, the system may be temporarily out of compliance during this event.

These scenarios were modeled under conservative conditions that result in the model overestimating the volume of water the perimeter barrier system would need to manage and do not account for capped drains, stormwater management, and operational buffers (i.e., operating the system such that the drain elevation is maintained below the inward gradient requirement). Realistically, the perimeter barrier system will be able to maintain compliance during the majority of storm events. If the system is temporarily out of compliance, the existing steel bulkhead barrier reduces the potential flux to the Detroit River to de minimis levels.

5.1.2 Groundwater Extraction System Basis

A groundwater extraction system is proposed to capture groundwater flux and mitigate flooding risk of the Site behind the proposed perimeter barrier. The system will consist of 23 collection drains and one vertical extraction well capturing groundwater on the northern, eastern, and southern boundaries of the Site. The drain alignments and the locations of the extraction well are presented on the Design Drawings (Appendix G).

Use of vertical and horizontal wells was also evaluated as the primary method of groundwater collection upstream of the perimeter barriers. However, it is difficult to control drawdown and capture across the length of a horizontal well screen in heterogenous soils. In addition, the inability to design and install a filter pack around the horizontal well screen would likely result in fine sediment entering the wells, conveyance piping, and AG treatment system. The presence of these solids would complicate OMM and overall system management. Therefore, horizontal wells were eliminated from consideration. Similarly, vertical wells were also eliminated as the primary method for groundwater extraction due to the highly heterogenous nature of the soils at the Site. Vertical wells would likely result in varying capture zones that would be difficult to model in localized areas and difficult to design. The one vertical well included in the design is in an area of the Site where installation of a drain is not feasible due to existing site infrastructure.

Considerations for drain lengths and locations included feasibility of installation, ease of operation, and long-term OMM requirements. Drains are generally located in zones of high permeability and are aligned to avoid subsurface obstructions, where possible. After the drains and sumps were placed based on these constraints, groundwater model simulations were run to confirm hydraulic capture of groundwater flux (Appendix B). During the Pre-Final (95%) Design phase, an iterative process was used to optimize drain lengths/locations and sump locations to ensure capture of groundwater flux based on groundwater model simulations. Each drain has one collection sump, installed at the end of the drain, based on the installation method (one-pass trenching) selected to minimize the need for dewatering during construction. Collection drain and sump design parameters are presented in Table 10A.

Table 10A. Proposed Drain Lengths and Sump Extraction Rates

Drain/Sump	Boundary Location	Drain Length (feet)	Average Sump Flow Rate (gpm)	Maximum Sump Flow Rate (gpm)
1	North	188	0.5	1.9
EX-1	Northeast	N/A	2.5	2.5
2	North	204	1.5	5.2
3	East	205	2.4	16.1
4	East	175	1.3	11.0
5	East	129	2.5	12.0
6	East	276	3.6	30.6
7	East	252	2.5	18.6
8	East	160	0.9	10.9
9	East	176	1.2	19.8
10	East	244	1.4	15.3
11	East	295	1.3	11.3
12	East	239	1.2	4.9
13	East	308	1.2	5.5
14	East	248	2.6	22.3
15	East	300	2.8	32.1

Drain/Sump	Boundary Location	Drain Length (feet)	Average Sump Flow Rate (gpm)	Maximum Sump Flow Rate (gpm)
16	East	282	1.7	16.3
17	East	95	8.1	33.9
18	East	333	1.9	25.8
19	East	303	1.4	24.6
20	Southeast	243	1.0	13.3
21	South	107	0.9	10.9
22	South	220	2.2	20.7
23	South	149	1.6	15.3
Total	N/A	5,131	48.2	380.8

Note: Flow rates listed are the instantaneous modeled rates for each individual sump. These rates do not occur at the same time; therefore, the total of the instantaneous rates is slightly higher than the total system average and maximum flow rates of 42.4 gpm and 376 gpm, respectively.

The drains will consist of 6-inch perforated HDPE pipe installed at a targeted depth interval of approximately 567 feet IGLD 85, approximately 2.5 feet below the instantaneous near 30-year river low-elevation level. Drainage trenches will be backfilled with a high-permeability material (i.e., stone) to promote groundwater drainage into the trench. An impermeable material will be placed at the top of the drain to prevent surface water from infiltrating directly into the highly permeable backfill material within the drain. Cleanout risers will be provided at the end of each drain to facilitate maintenance.

To optimize constructability while installing new drains around existing utilities and historical concrete foundations, a detailed route planning process has been employed. Using detailed mapping and surveys from the 2023 geophysics investigation and site maps, the existing utility and subsurface obstruction locations were identified, allowing for strategic design adjustments. The Pre-Final (95%) Design drain and conveyance piping route was selected to circumvent utilities and other subsurface obstructions, either by altering the location, adjusting angles, or using available space between utilities instead of crossing utilities, where possible. In addition, the conveyance piping path was diverted around concrete foundations, to the extent possible, to minimize disturbance or the need for costly removals.

The collection drains will be installed using one-pass trenching technology to simplify construction and minimize the need for dewatering. This technology will allow the trench to be excavated, the sumps and perforated pipes to be installed, and the trench to be backfilled with stone simultaneously using a single piece of equipment. One-pass trenching requires that the construction path be cleared of obstructions and utilities prior to construction, but reduces the need for trench boxes and shoring and is expected to greatly reduce the need for dewatering. This approach aims to streamline construction by reducing conflicts with existing utilities and buried debris, and facilitating a more straightforward installation process, ultimately enhancing efficiency and minimizing disruptions to existing infrastructure. Nevertheless, traditional excavation of trenches may be necessary in some locations if site constraints prohibit one-pass trenching. This installation method will still require dewatering, trench boxes in narrow pathways, and concrete removal.

Based on the groundwater model simulations, it was determined that one extraction well will be needed to supplement the drains and achieve full groundwater capture. One vertical well will be installed in the northeast corner of the Site, between Drains 2 and 3. This area of the Site has an abundance of subsurface utilities, making installation of a collection drain impractical. The well will be installed to a total depth of approximately 15 feet bgs and screened from 10 to 15 feet bgs within the shallow fill. Vertical well design parameters are presented in Table 10B.

Table 10B. Proposed Vertical Well Location and Flow Rates

Well ID	Boundary Location	Nearby Drains	Screen Interval (feet bgs)	Average Flow Rate (gpm)
EW-1	North	2 and 3	10 to 15	2.5

To monitor groundwater levels and support the evaluation of drainage efficiency, a series of PZs will be installed within the high-permeability drainage trenches. These PZs will be equipped with level transducers, which will control the speed of the sump pumps based on the desired compliance elevation.

5.2 Conveyance Network

The following sections describe the Pre-Final (95%) Design for the sump network and conveyance piping.

5.2.1 Sump Network

Groundwater captured in the drains will flow to the corresponding sump at the end of each drain. The sump locations are presented on Figure 3 and on the Design Drawings (Appendix G). Twenty-three sumps, one at the end of each drain, will be installed along the northern, eastern, and southern property boundaries (Figure 3). Each sump will be a double-barrel design, with two identical hydraulically connected, 30-inch-diameter, stainless-steel sump pipes or “barrels.” The bottom 5 feet of each sump “barrel,” above the drain pipe invert, will be constructed with wire-wrap screen sections on two sides to allow groundwater from the stone drainage layer of the drains to enter the sump. Each sump will contain stainless-steel slide rails and a stainless-steel chain for ease of removal and placement of the submersible pumps. Each sump will extend to a depth of approximately 2 feet below the drain invert elevation to provide collection and storage of accumulated solids.

Each sump “barrel” will contain one of the two duty-standby fully redundant pumps associated with each sump. Both pumps will be equipped with variable frequency drives (VFDs) to allow for automated flow rate adjustments. Level switches (floats) will be provided to serve as mechanical safety devices to mitigate the risk of the pumps running dry or water levels daylighting to ground surface. Pump operation will be determined based on groundwater elevations in the drain compared to the compliance elevation as discussed in Section 3.4.2 and in the OMM Manual (Appendix K). Flow control valves and flow meters will be installed in each above-grade enclosure associated with each sump to facilitate access by the system operator. An equipment list for the extraction equipment is provided in Appendix P.

5.2.2 Conveyance Piping

Groundwater collected at the sumps will be pumped through a network of below-grade and above-grade conveyance piping to the AG treatment system. Lateral piping from each sump will connect to header pipes that terminate at the AG treatment system as depicted on the Design Drawings (Appendix G). Three header pipes will be provided to reduce the risk of flooding or gradient loss that could result from extended downtime due to unexpected issues with the conveyance piping (e.g., pipe failure or fouling). The primary header pipe will terminate at the equalization tank; when system treatment capacity is exceeded, groundwater from the primary header pipe will be directed to the storage tanks via manual valves. A secondary header will terminate at the pump out tank, forming a slipstream for groundwater that does not require pH neutralization or metals treatment to comply with POTW limits. If treatment system capacity downstream of the pump out tank is exceeded (flow rate greater than 150 gpm), groundwater from the secondary header will be directed to the exterior storage tanks via manual valves. The slipstream header is included to account for the heterogeneous nature of on-site COCs. If collection sumps are identified that have a neutral pH and mercury concentrations below the treatment goal, operators can direct groundwater from those sumps to the slipstream header. A third header will be a fully redundant spare header going to all three locations (i.e., equalization tank, pump out tank, and storage tanks).

The conveyance piping network will be constructed using buried HDPE and above-grade carbon steel (exterior) and polyvinyl chloride (interior) piping. Based on total dynamic head calculations, which account for static head loss and frictional losses (e.g., pipe lengths, valves and fittings, instrumentation) for the sump pumps and conveyance piping within the conveyance network, pipe sizing was determined to be between 2 and 8 inches. The sump pumps and conveyance piping are designed to accommodate the maximum sump flow rates determined by the groundwater model (Table 10A), although the typical extraction flow rates are expected to be significantly less than the maximum modeled. The total dynamic head calculations are included in Appendix N.

To mitigate freezing risk, subgrade conveyance piping will be buried below the frost line and/or insulated as needed, and above-grade piping will include electric heat tracing and insulation. Pipe fouling within the conveyance network will be mitigated by installing cleanouts at approximately 500-foot intervals and maintaining a 2-inch (minimum) pipe diameter. The piping will be cleaned as part of routine maintenance activities and/or when increasing pressure, decreased flow, or other signs of pipe fouling are observed in the conveyance network.

6 Above-Grade Treatment System

The design for the AG treatment system incorporates comprehensive processes to treat groundwater contaminants effectively. A treatability study conducted in March 2022 established the basis for the treatment system design, which includes pH neutralization, chemical-physical treatment, sludge dewatering, granular-activated carbon (GAC), and ion exchange resin. Findings from the treatability study, along with the resin pre-design study, informed the design of each major process unit to handle site-specific groundwater characteristics, including metals, VOCs, SVOCs, and PFAS.

The design of the AG treatment system comprises several key components: a pH neutralization stage, precipitation, coagulation, flocculation, and clarification stages, followed by filtration through bag filters, GAC vessels, and use of ion exchange resin. The AG treatment system is designed to handle an average flow rate of 48 gpm, with a peak capacity of 150 gpm, and includes upfront storage tanks to moderate expected influent flow fluctuations. The design integrates a programmable logic controller (PLC)-based supervisory control and data acquisition (SCADA) system for monitoring and control. A new 8,800-square-foot, single-story, pre-engineered metal structure will house the treatment equipment.

The BOD for the AG treatment system is described in this section. Design drawings for the AG treatment system and Water Treatment Building are included in Appendix G, specifications in Appendix L, and supporting calculations in Appendix N.

6.1 Treatability Study

In March 2022, a treatment treatability study was performed to support the design of the AG treatment system using groundwater collected from select site monitoring wells and blended to represent anticipated influent groundwater characteristics. The treatability study included the following elements:

- Site groundwater sampling – Groundwater samples for analysis of key constituents were collected from select perimeter monitoring wells (i.e., those located in the zones of highest hydraulic conductivity). These data, along with existing analytical data, were used to evaluate the range of potential influent water quality for treatability testing.
- Treatability test groundwater collection – Groundwater used for treatability testing was collected from select perimeter monitoring wells. Water from these wells was blended to represent reasonably anticipated influent water quality for the full-scale AG treatment system.
- Laboratory treatability testing – Treatability testing included the following: pH neutralization (via acid addition), chemical-physical treatment (using metals precipitant, coagulant, and flocculant), sludge dewatering, and implementation of a GAC rapid small-scale column test (RSSCT).

An evaluation of the results of the treatability study provided the basis for the design of each major process unit (Arcadis 2022b). Groundwater characteristics of the Site and treatability study results are discussed in the following sections.

6.2 Resin Pre-Design Study

During the Preliminary (30%) Design, the need for further investigation of potential ion exchange interference compounds was identified as a data gap. Based on the results of the treatability study, the concentrations of interference compounds in the process water were expected to be one or two orders of magnitude higher than vendor-recommended levels for optimal resin performance. To address this issue, a resin pre-design study consisting of a field-scale pilot test simulating the GAC and ion exchange elements of the AG treatment system was performed to support the design. The pilot system was initiated on July 5, 2023, and discharge to the local POTW was initiated on July 6, 2023. The system remains operational.

Existing pumping wells PW-002 and PW-003 were installed in areas with high hydraulic conductivity as discussed in the Hydraulic Pre-Design Investigation Report (Arcadis 2021). Pumping wells PW-002 and PW-003 are also located in an area of the Site known to contain PFAS and elevated concentrations of potential interference compounds such as total organic carbon (TOC), chloride, sulfate, and alkalinity; therefore, these wells were used for the resin pre-design study.

Operation of the resin pre-design study is ongoing. Groundwater is pumped from PW-002 and PW-003 to an equalization tank where a transfer pump conveys water through bag filters, four vessels filled with 1,000 pounds of Calgon DSR-C reactivated carbon, and three vessels filled with 6.7 cubic feet of Purolite PFA694 resin prior to discharge to the POTW. Purolite PFA694 resin was selected because it is one of the leading PFAS resins commercially available and has been tested and proven to remove PFAS from groundwater at the Site. Empty bed contact time (EBCT) and hydraulic loading parameters for the total GAC system and single resin vessel are consistent with the proposed AG treatment system design criteria. As discussed in Section 6.4.7, resin volume and associated parameters are designed in accordance with vendor recommendations and other successful on-site use parameters. Additional resin vessels were installed to achieve compliance with discharge permit requirements while still being able to observe and establish breakthrough curves for the lead resin vessel.

Concentration data ranges for select interference compounds observed between July 2023 and June 2025 are presented in Table 11. PFOS, perfluorooctanoic acid (PFOA), and TOC concentration ranges are for groundwater collected from sample ports installed upstream and downstream of the bag filter; samples for analysis of chloride, sulfate, and alkalinity were collected downstream of the GAC vessels/upstream of the resin vessels.

Table 11. Resin Pre-Design Study Pilot Test Concentration Ranges

Constituent	Unit	Minimum Concentration	Maximum Concentration
PFOS	ng/L	31	230
PFOA	ng/L	1.8	12
TOC	mg/L	1.7	30
Chloride	mg/L	122	1,200
Sulfate	mg/L	94	320
Alkalinity	mg/L	46	280

Notes:
mg/L = milligrams per liter
ng/L = nanograms per liter

Interference compounds (e.g., TOC, sulfate, chloride) and alkalinity are consistently observed at concentrations above the vendor-recommended levels for the resin. However, based on the data collected during the pilot test, the concentrations do not reduce the ability of GAC and resin to remove PFAS to below DUWA discharge limits. Interference compounds will continue to be monitored as necessary to support the Final (100%) Design.

Continuous PFOS loading to the first resin vessel has not been observed as of June 2025; therefore, resin usage rates accounting for site-specific compounds cannot yet be estimated. The pilot test will continue to operate and resin performance will continue to be monitored to inform the Final (100%) Design.

6.3 Influent Characteristics

Influent groundwater characteristics and estimated flow rates, determined by groundwater modeling as discussed in Section 5 and Appendix B, were evaluated to inform the full-scale system design. Treated groundwater will be discharged to the POTW, and as such, influent chemical characteristics were compared to the anticipated POTW local discharge limits (Appendix Q). Influent concentration and flow rate estimates were used to determine mass loading and system capacity requirements.

6.3.1 Modeled Flow Rates

Based on groundwater modeling, as discussed in Section 5, the average estimated collection system flow rate is approximately 48 gpm, with a maximum flow rate of 376 gpm during peak pumping conditions. Maximum sump flow rates discussed in Section 5.1.2 do not occur simultaneously. The average flow rate was determined based on steady-state modeling, which averages yearly precipitation. The maximum flow rate was determined based on modeling of a 3.86-inch storm event occurring August 9 through 13, 2021. Additional details regarding this storm event are provided in Section 5.4.1 of the Groundwater Modeling Report in Appendix B.

6.3.2 Groundwater Storage

Groundwater storage was evaluated to provide capacity for peak pumping conditions anticipated during storm events, provide equalization of influent flow rates from the collection system, prevent oversizing of the AG process equipment, and provide storage should BASF be directed by DUWA to temporarily cease discharge of treated effluent.

Using the modeled flow rate data, simulations were completed to determine an optimal combination of system flow rate and storage to maximize system uptime and operational flexibility while minimizing site flooding potential and overall capital and operating costs. Steady-state and storm event scenarios were evaluated using treatment capacity flow rates ranging from 80 to 240 gpm; breakpoints within this range were 80, 120, 150, 180, and 240 gpm based on standard equipment sizes.

The analysis using the steady-state conditions identified the following storage needs:

- At a system flow rate of 80 gpm, the minimum storage requirement is 142,000 gallons.
- All higher flow rate scenarios are capable of maintaining inward hydraulic gradient compliance; therefore, no additional storage is required.
- The breakeven flow rate, where no additional storage is required, is estimated to be 100 gpm.

The analysis using the modeled storm event identified the following storage needs:

- At a system flow rate of 80 gpm, the minimum storage requirement is 520,000 gallons.
- At a system flow rate of 120 gpm, the minimum storage requirement is 300,000 gallons.
- At a system flow rate of 150 gpm, the minimum storage requirement is 200,000 gallons.
- At a system flow rate of 180 gpm, the minimum storage requirement is 140,000 gallons.
- At a system flow rate of 240 gpm, the minimum storage requirement is 60,000 gallons.

6.3.3 Design Influent Flow Rate and Storage Capacity

Based on the groundwater modeling results and storage evaluation, the selected AG treatment system design flow rate is 150 gpm. This flow rate was selected based on evaluation of steady-state conditions and optimization of equipment usage based on standard sizing breakpoints listed previously. A storage volume of 300,000 gallons, split between two 150,000-gallon tanks, was selected to provide a 33% safety factor at the design system flow rate. Groundwater will be directed to the storage tanks when influent flow exceeds 150 gpm. Dedicated storage tank pumps will convey groundwater from the storage tanks to the AG treatment system equalization tank at a flow rate of 150 gpm. The AG treatment system will return to normal operation (i.e., treating groundwater directly from the extraction sumps) when the influent flow returns to 150 gpm or less and all groundwater in the storage tanks has been treated. The 150,000-gallon storage tanks will be located inside a secondary containment berm adjacent to the new Water Treatment Building. Pumps, valves, and other appurtenances related to transferring water from the storage tanks will be placed on an enclosed, raised outdoor pad within the secondary containment berm.

6.3.4 Influent Chemical Characteristics

In November and December 2021, Arcadis collected samples from 10 monitoring wells at the perimeter of the Site based on the results of perimeter groundwater monitoring and hydraulic testing as described in the Hydraulic Pre-Design Investigation Report (Arcadis 2021 and AG System Treatability Study Report (Arcadis 2022b). The monitoring wells sampled were installed in zones with the highest hydraulic conductivity and are expected to drive the AG treatment system influent water quality and mass flux. To ensure the water quality characteristics of the groundwater samples were representative of anticipated influent COC concentrations, the samples were blended to replicate the estimated influent flow concentrations of the full-scale treatment system (i.e., higher percentage of water from areas with higher groundwater flow and mass flux). The primary COCs based on anticipated POTW limits are metals (specifically mercury), VOCs, SVOCs, and PFAS (Appendix Q). The analytical results for the COCs detected and general chemistry in the blended groundwater samples from the December 29, 2021, sampling event, are presented in Table 12.

Table 12. Blended Influent Groundwater Characteristics

SVOCs	Concentration (µg/L)
1,4-Dioxane	1.1 J
2,4-Dimethylphenol	0.76 J
2-Methylnaphthalene	0.89
2-Methylphenol	0.92 J
3-Metylhpenol, 4-Methylphenol	22
Carbazole	1.0
Naphthalene	8.5
Phenol	43
VOCs	**Concentration (µg/L)**
1,1-Dichloroethane	2.1
1,2-Dichloropropane	1.1
Acetone	25
Ethylbenzene	1.1
m&p-Xylenes	2.3
o-Xylene	2.1
Toluene	2.4
Metals	**Concentration (mg/L)**
Aluminum	0.6
Arsenic	0.0044 J
Barium	0.36
Boron	0.074
Calcium	870
Chromium	0.0038 J

Cobalt	0.00061 J
Copper	0.0027 J
Iron	0.33
Lead	0.0031 J
Magnesium	7.2
Manganese	0.012
Total Mercury	0.00052
Total Mercury (Low Level)	0.00058
Molybdenum	0.0097
Nickel	0.013
Potassium	58
Selenium	0.0036 J
Silicon	4.6
Sodium	1100
Titanium	0.015
Vanadium	0.0057
Zinc	0.035
Inorganics	**Concentration (mg/L)**
pH (standard units)	12.4
Alkalinity	930
Total Suspended Solids	114
Total Dissolved Solids	5300
Total Organic Carbon	29
PFAS	**Concentration (ng/L)**
Perfluorohexane sulfonic acid (PFHxS)	33
Perfluorohexanoic acid (PFHxA)	9.6
Perfluorooctane sulfonic acid (PFOS)	68
Perfluorooctanoic acid (PFOA)	12
Perfluoropentoanoic acid (PFPeA)	9.4

Notes:
J = estimated value
µg/L = micrograms per liter

6.4 Major Unit Processes

Eight major components comprise the AG treatment system and were designed based on the selected design flow rate and treatability study findings (Arcadis 2022b). During normal operation (i.e., influent flow rates of 150 gpm or less), influent groundwater will be collected and stored in an equalization tank to reduce flow and concentration fluctuations prior to treatment. Neutralization of pH will be performed to reduce pH levels and prepare the influent water for metals precipitation via coagulation, flocculation, and clarification. Following metals precipitation, process water will be fed to a pump out tank to facilitate treatment via filtration, GAC, and ion exchange units to address total suspended solids, SVOCs, VOCs, and PFAS. Treated effluent will be discharged to the POTW in accordance with provisions described by DUWA's permit requirements. Further details are provided in the following sections and on the Design Drawings (Appendix G). A full list of equipment discussed is provided in Appendix P. Technical specifications for equipment and piping are provided in Appendix L. Average and design system flow rates are defined as 48 gpm and 150 gpm, respectively. Unless otherwise stated, the average and design columns in the following tables refer to conditions observed at the identified flow rates.

6.4.1 Influent Equalization

During normal operation (i.e., influent flow rates less than 150 gpm), the equalization tank will receive untreated groundwater from the extraction sump network or storage tanks. The equalization tank was designed to moderate variable groundwater flow rates and concentrations prior to treatment, receive maintenance water from routine media backwashing and clarifier cleaning, and receive other process-related waters including filter press filtrate, decant water from the sludge holding tank, and building sump pump water. The equalization tank has capacity to receive backwash water from two GAC vessels. Backwashing is anticipated to take approximately two hours per vessel; during this time, extracted groundwater will be directed to the storage tanks. Clarifier cleaning may generally be accomplished while water is being directed to the equalization tank. The required equalization tank operating volume is approximately 14,000 gallons but has a specified capacity of 16,100 gallons, representing 15% additional storage capacity.

During normal operation, the equalization tank will operate to maintain a volume of 2,000 gallons, resulting in a hydraulic residence time of 13.3 minutes at the design flow rate, with the remaining 12,000 gallons available for routine process and maintenance activities. Transfer pumps equipped with VFDs will be used to transfer water from the equalization tank to downstream operations. Equalization tank parameters are summarized in Table 13.

Table 13. Equalization Tank Design Parameters

Design Parameter	Average Flow Rate	Design Flow Rate
Process Flow Rate (gpm)	48	150
Design Equalization Tank Volume (gallons)	16,100	16,100
Equalization Tank Diameter (feet)	14	14
Equalization Tank Operating Height (feet)	12' 2"	12' 2"
Equalization Tank Freeboard (feet)	2	2
Equalization Tank Total Height (feet)	16' 10"	16' 10"
Equalization Tank Retention Time (minutes), at Typical Operating Volume (2,000 gallons)	41.7	13.3

The water level within the tank will be maintained using level instrumentation and control loops to communicate with influent and effluent pumps, flow meters, and automated valves. Setpoints will be established for routine operation and alarm conditions to interlock the process equipment as a safety measure to protect equipment and personnel and prevent a release.

6.4.2 pH Neutralization

The treatability study (Arcadis 2022b) included a pH neutralization step to determine the acid dose required to reduce groundwater pH. A reduced pH is necessary to effectively remove metals by coagulation and flocculation, prevent fouling of downstream piping and equipment, and to ensure discharge is compliant with the POTW limit of 5.0 to 11.5 standard units (S.U.). The pH of the blended influent water used for the treatability study measured approximately 12.4 S.U. The blended influent water was titrated using sulfuric acid to determine the volume of acid needed to reach the target pH per unit volume of influent water. Sulfuric acid was selected for pH adjustment based on its effectiveness in pH neutralization with similar groundwater chemistry. The design parameters for pH neutralization, based on influent water quality and results of the titration testing, are presented in Table 14.

Table 14. pH Neutralization Design Parameters

Design Parameter	Average Flow Rate	Design Flow Rate
Process Flow Rate (gpm)	48	150
Initial pH (S.U.)	12.4	12.4
Target pH (S.U.)	9.0	9.0
Sulfuric Acid Dose to Reach pH of 9 S.U. (milliliters per liter of 5 Molar Sulfuric Acid) – from Titration	1.9	1.9
Sulfuric Acid Concentration (percent by weight)	93%	93%
Sulfuric Acid Usage (gallons/day)	52	177

Due to the slope of the titration curve near the target pH, sulfuric acid will be added in up to three locations – inline upstream of the lead neutralization tank, directly into the lead neutralization tank, and directly into the lag neutralization tank. The bulk of the pH adjustment will be performed in the initial inline pH adjustment location with fine adjustment performed in the lead neutralization tank. The lag neutralization tank will serve to confirm the pH level prior to metals treatment with additional pH adjustment performed on an as-needed basis only. Dedicated chemical feed pumps will be used at each sulfuric acid amendment location to optimize acid addition and pH control. The rate of acid addition will be controlled using feedback from a combination of system flow meters (flow-paced) and/or pH sensors (pH-based) at each additional location. Sodium hydroxide and an associated chemical feed pump have been included in the system design to increase the pH if the measured pH drops below the low setpoint.

Sulfuric acid will be delivered by tanker truck and stored in a 5,850-gallon above-ground storage tank located on the northeast side of the Water Treatment Building. Deliveries will occur on an as-needed basis. During offloading, the delivery operator will connect a hose to a dry-disconnect coupling housed within a secure, weather-resistant chemical fill enclosure located north of the building. A dedicated sulfuric acid transfer pump, also located within the enclosure, will convey the acid from the tanker to the storage tank via a closed-loop transfer system designed to minimize operator exposure and environmental risk.

The sulfuric acid tank will be installed 20 feet from the Water Treatment Building to comply with fire code. A dedicated vent line will allow displaced air and vapors to escape the sulfuric acid storage tank during filling. Conveyance piping will be installed directly from the sulfuric acid tank to the three chemical feed pumps described above.

6.4.3 Precipitation, Coagulation, and Flocculation

The pH-neutralized water will be amended with an organosulfide metal precipitant (i.e., MetClear) to precipitate dissolved mercury and reduce mercury concentrations to less than 200 ng/L per anticipated POTW discharge limits. During the treatability testing, the blended influent was tested at various pH points and metal precipitant doses to determine the optimal conditions to precipitate dissolved mercury. Using a 20-mg/L MetClear dose at a pH of 9.0 followed by coagulation and flocculation, the test demonstrated a significant decrease in total and dissolved mercury concentrations, visible indication of mercury precipitation, and effective gravity settling of precipitates within 30 minutes.

Coagulation and flocculation treatment removes precipitated mercury from the process stream. The treatability study tested varying doses of coagulants and flocculants to determine the optimal combination for mercury removal. The blended influent, with a total mercury concentration of 480 to 620 ng/L, was used for jar test screening. As presented in the treatability study report (Arcadis 2022b), all combinations of metal precipitant, coagulants, and flocculants met performance objectives. Following coagulation and flocculation treatment, the total and dissolved mercury concentrations decreased to 23 ng/L and 1.4 ng/L, respectively, which is below the POTW permit limit of 200 ng/L. The recommended chemical amendments and doses are presented in Table 15.

Table 15. Proposed Metal Precipitant Amendments and Doses

Chemical	Stock Solution	Dose
Metals Precipitant	MetClear MR2405 (100%)	20 mg/L
Coagulant	Ferric Iron KlairAid IC1251	10 mg/L
Cationic Flocculant	Polyfloc AE1703	3 mg/L

The metal precipitant and coagulant will be metered into the inclined plate clarifier's rapid mix tank and mixed for approximately 0.73 and 2.3 minutes at the design flow rate of 150 gpm and the average flow rate of 48 gpm, respectively. The coagulant and metal precipitant will be introduced in the same mixing tank for the full-scale design – treatability testing and vendor recommendations indicate that the order and timing between the addition of metal precipitant and coagulant do not significantly affect treatment effectiveness.

Following metals precipitant and coagulant mixing, process water will be gravity fed into the clarifier's slow mix flocculation tank where the cationic flocculant will be introduced. The flocculation tank is sized to provide approximately 2.1 minutes of retention time at 150 gpm.

The chemical metering pumps will be flow-paced based on the influent system flow rate (e.g., higher influent flow rates result in higher chemical amendment flow rates). Each chemical storage tank will be equipped with a drum scale that will generate an alarm condition when the low-level setpoint is reached to prevent operation without the feed chemical.

6.4.4 Clarification

Process water will flow from the flocculation tank via gravity into the inclined plate clarifier. The clarifier was designed using a maximum industry standard loading rate for metals precipitation of 0.50 gpm per square foot of inclined plate area to promote effective settling. Process water will overflow from the clarifier via gravity to the pump out tank. Settled solids will be removed periodically from the bottom of the clarifier using a pair of pneumatic-diaphragm pumps and transferred into a sludge holding tank. The design parameters for the clarifier are presented in Table 16.

Table 16. Clarifier Design Parameters

Design Parameter	Value
Influent Solids (mg/L)	79.8
Clarifier Settled Solids (%)	1.5
Minimum Required Plate Area (square feet)	300
Clarifier Plate Area (square feet)	560
Rapid Mix Tank Volume (gallons)	110
Slow Mix Tank Volume (gallons)	316
Solids Sump Volume (gallons)	469
Clarifier Overall Height (feet)	12.3
Clarifier Overall Length (feet)	11.3
Clarifier Overall Width (feet)	6.4

6.4.5 Pump Out Tank and Bag Filtration

Clarifier effluent will discharge via gravity to the pump out tank. The system will also contain a slipstream header that will discharge groundwater from selected sumps directly to the pump out tank when the sump water is determined to meet pH and mercury requirements without pH neutralization and metals precipitation treatment. From the pump out tank, transfer pumps will convey process water to bag filters for suspended solids removal. The transfer pumps will be equipped with a VFD to maintain a constant level in the pump out tank during normal operation. Under certain circumstances, batch operation may be necessary and will be controlled by the level transducers and/or level switches (i.e., pump on and pump off level setpoints). The pump out tank design parameters are presented in Table 17.

Table 17. Pump Out Tank Design Parameters

Design Parameter	Average Flow Rate	Design Flow Rate
Process Flow Rate (gpm)	48	150
Retention Time (minutes)	41.7	13.3
Total Pump Out Tank Volume (gallons)	2,000	2,000
Tank Overall Height (feet)	8.8	8.8
Tank Overall Diameter (feet)	7.1	7.1

Bag filtration is used to improve GAC media performance by 1) reducing solids loading, which promotes channeling and preferential flow through the GAC media and reduces overall contaminant removal efficiency, 2) mitigating fouling of the GAC media, which increases system pressures, and 3) reducing the number of GAC backwash cycles due to solids buildup in the GAC vessel. Five-micron (µm) filter bags, designed to remove 85 to 95% of particles 5 µm and larger, are proposed but will be modified based on observed conditions and performance following start-up. The bag filters will be installed on a dual housing skid and automatically switch between filter housings based on observed pressure drop, volume treated, and/or time setpoints; an option to use both bag filter units in parallel operation is also included in the design to provide operational flexibility. Filter bags will be changed by the system operator, as needed, and disposed in accordance with the site-specific Waste Management Plan included in Appendix R.

6.4.6 Granular-Activated Carbon

The GAC system was designed for removal of SVOCs and VOCs. The GAC system will also remove TOC and PFAS, but neither compound was considered for design of the GAC treatment system. An RSSCT was performed to evaluate the effectiveness of SVOC and VOC removal using reactivated GAC. For the RSSCT, blended influent groundwater collected from the Site was pretreated with sulfuric acid to a pH of 9.0 S.U. and amended with the metals precipitation chemicals and doses described in Section 6.4.3. Two columns of Calgon DSR-C reactivated carbon were used which had a 40-minute total EBCT. The existing on-site construction water treatment system was used to determine the target EBCT for the RSSCT. The existing construction water system was designed based on the existing West Track system, and both systems have been successfully treating groundwater to below DUWA permit levels. The RSSCT confirmed that the reactivated GAC is capable of treating VOCs and SVOCs to below POTW limits (Arcadis 2022b).

At the design flow rate of 150 gpm, four GAC vessels operating as two parallel trains with two vessels in series per train is required. When operating at lower flow rates, all four vessels will be used to prevent water stagnation in the unused vessels. Additional backwashing can be used to counter observed channeling through the GAC vessels. Each GAC train will have the capability to alternate the lead and lag GAC vessels to optimize GAC usage. Backwashing of the GAC vessels will require the use of a pump out tank and associated pumps, and dirty backwash water will be pumped to the equalization tank. During media changeouts, a single train will generally be able to continue treating process water at a maximum flow rate of 75 gpm. When the GAC changeout requires backwashing and soaking of the replacement GAC, that train will be temporarily disabled. The minimum flow rate through the GAC system is 20 gpm to ensure optimal treatment of the ion exchange system, as discussed in Section 6.4.7. The pump out tank transfer pumps will process water in batch operation, as needed, to maintain the 20 gpm minimum flow rate through the ion exchange system. The GAC vessel parameters for the Pre-Final (95%) Design are based on a target minimum 20-minute EBCT per vessel and a minimum hydraulic loading of 2 gpm per square foot. A summary of the key GAC unit process parameters is presented in Table 18; additional calculations for GAC vessel sizing are included in Appendix N.

Table 18. Granular-Activated Carbon Design Parameters

Design Parameter	Average Flow Rate	Design Flow Rate
Process Flow Rate (gpm)	48	150
GAC Vessel Diameter (feet)	6	6
GAC Bed Depth (feet)	7	7
No. of GAC Vessels in Series	2	2
No. of GAC Vessels in Parallel	2	2
Hydraulic Loading (gpm/square foot)	1.7(1)	2.7
GAC Weight, Per Vessel (pounds)	5,000	5,000
EBCT, Per Vessel (minutes)	30.8	19.7(2)

Notes:
(1) The hydraulic loading rate at the average flow rate is below the stated minimum of 2 gpm/square feet. VOC and SVOC removal will not be significantly affected and as stated above, will be managed by performing GAC vessel backwashes to minimize channeling effects at the lower hydraulic loading rates.
(2) The EBCT at the design flow rate is below the stated minimum of 20 minutes. VOC and SVOC removal is not anticipated to be affected as the EBCT still exceeds typical VOC and SVOC target EBCTs.

6.4.7 Ion Exchange Resin

Ion exchange resin will be used for the removal of PFAS following GAC treatment. Four parallel resin trains with two vessels operating in series per train were designed for operational flexibility while maintaining the vendor-recommended (Purolite) hydraulic loading rates. The resin system will be configured with a valve tree to enable alternating the lead and lag vessels for each treatment train and was designed to automatically switch between one, two, three, or four operating trains, as needed, in accordance with the flow ranges presented below:

- One train (two resin vessels) to be used for flow rates between 20 and 40 gpm.
- Two trains (four resin vessels) to be used for flow rates between 40 and 80 gpm.
- Three trains (six resin vessels) to be used for flow rates between 60 and 80 gpm.
- Four trains (eight resin vessels) to be used for flow rates between 80 gpm and 150 gpm.

A flow rate of 20 gpm is required to maintain the minimum hydraulic loading rate through each resin vessel as recommended by Purolite. If the AG treatment system operates at less than 20 gpm, the pump out tank transfer pump will operate in batch mode through the GAC and resin systems to maintain the minimum hydraulic loading rate. During media changeouts, one, two, or three trains will continue treating influent water at a maximum flow rate of 40 gpm per train.

Pilot data associated with BASF's West Track system showed successful PFAS treatment to below method detection limits with influent PFAS concentrations up to 2,500 ng/L using a 3-minute EBCT, which is greater than the Purolite-recommended EBCT. Elevated concentrations of total dissolved solids, primarily due to chloride and sulfate, were present in the blended influent water used for the treatability study and are known to affect PFAS treatment performance using resin (Section 6.2). Resin vessel parameters for the Pre-Final (95%) Design are based on a minimum 2-minute EBCT per vessel and a hydraulic loading between 6 and 18 gpm per square foot based on Purolite specifications. A summary of the key ion exchange resin process parameters is presented in Table 19; additional calculations for ion exchange resin vessel sizing are included in Appendix N.

Table 19. Ion Exchange Design Parameters

Design Parameter	Average Flow Rate	Design Flow Rate
Process Flow Rate (gpm)	48	150
Ion Exchange Resin Vessel Diameter (feet)	2	2
Ion Exchange Resin Bed Depth (feet)	3	3
No. of Ion Exchange Resin Vessels in Series	2	2
No. of Ion Exchange Resin Vessels in Parallel	2	4
Hydraulic Loading, Per Vessel (gpm/square foot)	7.6	15.6
Ion Exchange Resin Volume, Per Vessel (cubic feet)	9	9
EBCT Per Vessel (minutes)	5.9	2.4

6.4.8 Discharge

Effluent flow will be discharged to the POTW via new and existing sanitary sewer lines equipped with check and isolation valves to prevent backflow from the site sanitary connection. Flow will be measured using a magnetic flow meter. Routine effluent samples will be collected to confirm that the AG treatment system is meeting POTW discharge requirements for COCs, and an effluent pH meter will be used to confirm that POTW pH compliance is met (see Appendix K). Sample frequency will be in accordance with the DUWA discharge permit. Anticipated POTW discharge limits are provided in Appendix A of the DUWA Sewer Use Regulations included in Appendix Q.

6.5 Ancillary Processes

The following sections present the Pre-Final (95%) Design for the ancillary processes for the AG treatment system, including the system pumps, operation of storage tanks, solids management, tank mixing, compressed air, and instrumentation and controls.

6.5.1 System Pumps

Pump and motor sizes were determined using hydraulic calculations for all unit processes as presented in Appendix N. Parameters used for total dynamic head calculations to determine pump size and type include conveyance pipe length, diameter, material of construction, anticipated flow rates, discharge elevation, and head loss through respective unit processes. Flow rates for the collection sumps were estimated using groundwater modeling, while anticipated flow rates for the treatment system are based on the design flow rate of 150 gpm. The pump parameters are presented in Table 20.

Table 20. Pump Design Parameters

Pump ID	Number of Pumps	Description	Pump Type	Design Flow Rate (gpm)	Head (feet)	Horsepower
P-001A/B, P-002A/B, P-003A/B, P-004A/B, P-005A/B, P-006A/B, P-007A/B, P-015A/B, P-016A/B, P-017A/B, P-018A/B, P-019A/B, P-020A/B, P-021A/B, P-022A/B, P-023A/B	32	Collection Sump Pump	Submersible	1.9 to 33.9	67 to 92	4.5
P-008A/B, P-009A/B, P010A/B, P-011A/B, P-012A/B, P-013A/B, P-014A/B	14	Collection Sump Pump	Submersible	4.9 to 22.3	55 to 69	3
P-024	1	Extraction Well Pump	Submersible	2.5	4.0	0.5
P-100A/B, P-110A/B, P-700A/B	6	Transfer Pump	Centrifugal	150	32 to 43	3
P-200A/B	2	Transfer Pump	Centrifugal	150	110	7.5
P-500, P-501, P-502	3	Dosing Pump	Diaphragm	0.041	–	–
P-510, P-520, P-560	3	Dosing Pump	Diaphragm	0.001 to 0.003	–	–
P-530, P-550	2	Dosing Pump	Peristaltic	0.005	–	–
P-540	1	Dosing Pump	Diaphragm	0.18	–	–
P-600, P-601A/B, P-602A/B	5	Sludge Wasting Pump	Diaphragm	15	15	–
P-603	1	Sump Pump	Submersible	30	22	0.5

Note: "-" indicates the design criteria is not applicable for the particular pump type and/or location within the system

6.5.2 Operation of Storage Tanks

The storage volume evaluation is discussed in Sections 6.3.2 and 6.3.3. Storage tank capacity will be split between two equally sized tanks that are hydraulically connected for individual or parallel operation. Groundwater from the drainage sumps will be redirected from the equalization tank to the storage tanks when a high-level setpoint in the equalization tank is reached. Storage tank pumps will begin transferring groundwater from the storage tanks to the equalization tank when the normal operating water level in the equalization tank has been reached. Sump flow will be directed to the storage tanks until:

1) Storage tanks are no longer gaining volume and the discharge flow rate from the storage tanks is greater than the influent flow rate (i.e., the groundwater extraction flow rate from the drainage sumps is below 150 gpm); and

2) The low-level setpoint has been reached in both storage tanks.

After the above conditions have been met, groundwater flow from the sumps will be redirected back to the equalization tank. A high high level switch will prevent overfilling of the storage tanks and shut down the sumps if necessary. To minimize tank maintenance, tank discharge points will be set approximately 1 foot off the bottom of the tank to prevent any settled solids from entering the system. Tank cleaning will be completed on an as-needed basis to maintain the volume of the tanks. Storage tank parameters are provided in Table 21.

Table 21. Storage Tank Design Parameters

Design Parameter	Individual Tank	Total
Total Volume (gallons)	180,000	360,000
Operational Volume (gallons)	150,000	300,000
Height (feet)	38.4	–
Diameter (feet)	30.5	–

6.5.3 Solids Management

Settled solids from the equalization tank and clarifier will be periodically conveyed to the sludge holding tank. Solids removal will be a manual process from the equalization tank and an automated process from the clarifier. Solids transfer intervals will be evaluated following start-up to optimize solids removal and maintain effective equipment and system operation. Clarifier solids removal will be controlled based on gallons treated and observations of sludge accumulation (i.e., depth of sludge blanket) and discharge solids from the clarifier sump. The solids will be stored in the sludge holding tank prior to dewatering to encourage additional settling. The sludge holding tank will have a minimum retention time of 4.75 days at the design flow rate. The holding tank will be equipped with one overflow and two manual decant lines, which will discharge to the building sump for conveyance to the equalization tank. The overflow line will be placed at the 13,000-gallon capacity point along the sidewall, while the decant lines will be placed at the 7,700- and 6,100-gallon capacity points. The overflow location/volume corresponds to a 4.75-day retention time operation at the design flow rate of 150 gpm, while the location/ volume of the decant lines corresponds to a 7-day retention time at 60 and 48 gpm, identified as the flow rates required to maintain 2-foot and 0.5-foot inward hydraulic gradients compared to the river, respectively. To prevent oversizing of the sludge holding tank, a lower retention time at the design flow rate is acceptable because elevated flow rates are not expected to occur regularly (i.e., during storm events).

For solids dewatering, sludge will be drawn off the bottom of the sludge holding tank, a thickening agent will be added to promote floc generation, and dewatering will be performed using a plate and frame filter press. Dewatered solids will be collected in a disposal container and disposed of in accordance with the site-specific Waste Management Plan (Appendix R); filtrate will feed via gravity to the building sump and be transferred to the equalization tank. The treatability study included an assessment of solids from each process, including solids volume generated and solids content (Arcadis 2022b). Due to inconsistencies in solids generation observed during the treatability study that resulted in lower-than-expected solids generation, reference design standards were used to establish the parameters for the solids management processes as listed in Table 22.

Table 22. Solids Management Design Parameters

Design Parameter	Average[1]	Design
Holding Tank Influent Flow Rate (gallons/day)	876	2,739
Holding Tank Volume (gallons)	6,100	13,000
Holding Tank Retention Time (days)	7	4.75
Holding Tank Underflow Solids (%)	2.0	2.0
Holding Tank Decant Volume (gallons/day)	458	1432
Filter Press Size (cubic feet)	12	12
Cycles Per Week	1.5	5
Cake Solids (%)	35	35
Cake Density (pounds/cubic foot)	75	75
Pressed Sludge (pounds/press cycle)	900	900
Pressed Sludge (tons/year)	35.1	116.7
Filtrate (gallons/press cycle)	1,781	1,781

Note: [1]Assumes decant lines are opened such that tank volume is effectively 6,100 gallons.

6.5.4 Tank Mixing

Both the lead and lag pH neutralization tanks will use mixers to provide complete mixing of the sulfuric acid with the influent water and achieve accurate pH readings.

The inclined plate clarifier's coagulant (rapid mix) tank and flocculation (slow mix) tank will be equipped with a rapid vertical mixing unit and picket fence vertical mixing unit, respectively, to distribute chemicals and promote aggregation of dissolved and suspended particles in the process stream. Aggregation of particulates will promote floc formation and effective settling in the inclined plate clarifier. The mixer parameters are listed in Table 23.

Table 23. Mixer Design Parameters

Mixer ID	Number of Mixers	Description	Type	Speed	Horsepower
A-110, A-111	2	pH Neutralization Mixer	Rapid Vertical Mixing	1,750 RPM	2
A-120	1	Coagulant Mixer	Rapid Vertical Mixing	1,750 RPM	1/2
A-121	1	Flocculant Mixer	Picket Fence	125 RPM	1/3

Note:
RPM = revolutions per minute

6.5.5 Compressed Air

Compressed air will be produced by a pair of two redundant air compressors to supply air-operated equipment (e.g., pneumatic valves and pumps, filter press). To ensure each compressor is maintained and operating appropriately, compressor operation will be cycled on a routine (e.g., monthly) basis. The air compressors will be equipped with an air dryer and air filter that can achieve ISO 8573-1 Class 2.2.2 quality air. Compressed air flow rate requirements are based on two sludge pumps and the filter press operating simultaneously. Each sludge pump requires compressed air at 20 standard cubic feet per minute (scfm) at a maximum pressure of 120 pounds per square inch (psi); the filter press also requires a maximum of 20 scfm and 120 psi. Per vendor recommendations, the air compressors are designed such that maximum air demand occurs up to 8 hours per day, which provides contingency for simultaneous pneumatic valve operation.

6.5.6 Instrumentation and Controls

The AG treatment system will include a PLC-based SCADA system. The SCADA system will provide monitoring, control, alarming, and data collection. Flow and pump control at the collection drain sumps and extraction well will be programmed for automatic operation with the ability to operate manually when necessary. The AG treatment system process pumps and instrumentation will be monitored and controlled by the SCADA system. A human machine interface (HMI) will be used to operate equipment and display SCADA information. Each sump will have a control panel, and HMIs will be strategically placed along the collection trench to provide field control of the sump pumps. Each sump control panel will be housed in a secure, lockable sump enclosure to prevent unwanted access to the enclosure internals. A fiber network will be utilized to convey data from the collection network and associated PZs to the SCADA system.

Alarm interlocks based on the monitored process variables will be programmed into the PLC to enable automatic alarms, interlocking, and/or disabling of the conveyance pumps and process equipment. Examples of alarm conditions requiring shutdown include low/high discharge flow, high discharge pressure, low/high tank levels, and motor overloads. Critical alarm conditions (e.g., leak detection) will disable the entire system, requiring operator review of the alarm and determination as to whether to restart the system. Compliance alarms will notify the operator, requiring action to inspect, repair, and restart the system as needed. An interlock table describing process alarm conditions and associated process responses is provided in Appendix S. A process description is provided in Appendix L.

6.6 Water Treatment Building

The Water Treatment Building will be a new, single-story, pre-engineered metal structure designed to enclose AG water treatment process equipment, tanks, and associated systems. The building will have an approximate footprint of 8,800 square feet and will include designated interior spaces such as an electrical room, a small office, and a single-occupant unisex toilet room to support operations. The new building will be located in the central portion of the Site, north of Alkali Avenue (Figure 3). Note that this represents a change from the Intermediate (60%) Design, which had assumed that the current Infinergy building, located at the northern end of the Site, would be retrofitted to house the proposed treatment system.

The BOD for the Water Treatment Building is detailed in this section. The Pre-Final (95%) Design drawings are included in Appendix G, with corresponding specifications in Appendix L and supporting calculations in Appendix N.

6.6.1 Architectural Design

The building will be constructed on a cast-in-place concrete slab, designed to support process loads and integrate with secondary containment features. A monolithic concrete curb will be provided around the interior perimeter of the process area to serve as secondary containment for chemical storage and potential spills. Refer to Appendix N for the design drawings.

The building envelope will consist of insulated metal panel walls and roof, compliant with current energy code requirements and designed for durability and ease of maintenance in an industrial environment. Pre-finished metal exterior panels will be selected for corrosion resistance and compatibility with adjacent structures.

To support equipment installation and maintenance, the building will be equipped with insulated coiling overhead doors and multiple man doors along the exterior walls, located to provide efficient access for personnel, deliveries, and large equipment. Door locations and clearances have been coordinated with mechanical and process layouts to ensure operability.

Interior finishes will be industrial-grade, with exposed structural framing and utility routing, allowing for straightforward maintenance and future system modifications. The unisex toilet room is designed in accordance with Americans with Disabilities Act (ADA) accessibility guidelines.

6.6.1.1 Applicable Codes

This section summarizes the code analysis performed to establish criteria to be used as a guide for the Water Treatment Building. This section presents specific code and regulatory requirements and identifies design parameters for project preparation in this regard. The code analysis is also presented on the architectural design drawings (Appendix G).

The following applicable codes and standards have been observed for design and are observed for construction of the new Water Treatment Building:

- 2021 Michigan Building Code
- 2021 Michigan Mechanical Code
- 2021 Michigan Plumbing Code
- 2023 National Electrical Code
- 2021 Michigan Energy Code
- 2021 International Fire Code
- 25th Edition Guide to the Michigan Gas Safety Standards
- National Fire Protection Association (NFPA) 70 – National Electric Code
- NFPA 72 – National Fire Alarm Code
- NFPA 101 – Life Safety Code
- 2010 ADA Standards for Accessible Design

6.6.1.2 Building Code Review

An analysis of the building code was performed considering the anticipated future Water Treatment Building. Below is a comprehensive summary of the findings derived from this analysis:

- Occupancy Type: F-1, Factory Industrial, Moderate Hazard
- Construction Type: IIB, Non-Rated
- Sprinkler System – Not required
- Allowable Building Height: 55 feet. Proposed building height: 24 feet
- Allowable Number of Stories: 3. Actual number of stories: 1
- Allowable Square Footage: 15,500. Actual square footage: 8,800
- Primary Structural Frame: 0-hour fire rating
- Occupancy Load: 88 occupants
- Building is unoccupied except for maintenance personnel (maximum of 10)
- Maximum Egress/Exit Access Travel Distance (without Sprinkler System): 200 feet
- Minimum of two exits out of the room are required if path of travel is more than 100 feet
- Water Closets Required/Provided: Buildings with less than 15 occupants are required to have only one unisex toilet room
- Drinking fountains are not required for buildings with less than 15 occupants
- Service Sinks Provided: 1
- Emergency Eyewash/Shower Provided: 2
- Water treatment facilities are not required to be ADA compliant, however, the building is ADA compliant.

6.6.1.3 Maximum Allowable Quantity Analysis

The maximum allowable quantity analysis refers to a systematic assessment of the maximum volume or quantity of hazardous chemicals permitted for storage within a facility, specifically indoors, while maintaining compliance with safety regulations and minimizing potential risks. The analysis takes into account various factors such as chemical properties and their reactivity, flammability, toxicity, and storage container types. This analysis is intended to ensure that the stored quantities of chemicals remain within safe limits to prevent fire hazards, environmental contamination, or health risks to occupants and responders in case of accidents or emergencies.

6.6.1.3.1 Indoor Storage

The proposed quantities of chemicals to be added within the Water Treatment Building do not exceed the maximum allowable for indoor control areas for both physical and health hazards.

Indoor Storage Chemical List and Actual Quantities:

- Sulfuric Acid, 93% – Water Reactive 1, Corrosive – 4 gallons in piping (use-closed)
- KlarAid IC1251 – Corrosive – 4 gallons in piping (use-closed), 179 gallons indoor storage
- MetClear MR2405 – Corrosive – 4 gallons in piping (use-closed), 179 gallons indoor storage
- Polyfloc AE1703 – Non-Hazardous – 4 gallons in piping (use-closed), 55 gallons indoor storage

- ChemTreat P8315 – Non-Hazardous – 4 gallons in piping (use-closed), 55 gallons indoor storage
- Sodium Hydroxide, 25% – Corrosive – 15 gallons indoor storage
- Maximum Allowable Quantities (storage + use-closed + use-open) Corrosive: 500 gallons. Actual: 385 gallons (complies)

6.6.1.3.2 Outdoor Storage

Bulk sulfuric acid (93% – Water Reactive Corrosive – 5,175 gallons) will be stored in a stainless-steel tank located 20 feet away from the Water Treatment Building. The tank will be placed in a concrete containment zone. The outdoor control area will be set back more than 20 feet from public streets, public alleys, public rights-of-way, or lot lines.

6.6.2 Structural Design

This section describes the structural design criteria that have been used for the design of the proposed Water Treatment Building, ground-supported tanks, and pipe supports across the Site. The structural design is presented in the Design Drawings (Appendix G). Included in this section are applicable codes and standards, design load criteria, and materials of construction. Codes and standards are subject to change based on building department requirements at the time of design and anticipated construction.

6.6.2.1 Design Criteria

The structural design is based on applicable codes and standards, as well as project-specific assumptions informed by the building's location, use, and configuration. The key structural design criteria and assumptions are summarized below and form the basis for the structural calculations provided in Appendix N.

Project location parameters for use with the American Society of Civil Engineers (ASCE) Hazard tool are as follows:

- Latitude, Longitude: 42.218631, -83.142108
- Risk Category: III (Wastewater)
- Soil Class: E

Concrete classes, strengths, and their corresponding applications are outlined below:

- Class A: Compressive strength, f'_c = 4,500 psi (all applications unless otherwise noted)
- Class B: Compressive strength, f'_c = 3,000 psi (concrete fill, duct banks, unreinforced encasements, curbs, sidewalks, and thrust blocks)
- Concrete reinforcement (rebar): ASTM A615, Grade 60

Steel standards are as follows. Specifications are included in Appendix L:

- Structural tubing: ASTM A 500, Grade B
- Structural pipe: ASTM A 53, Grade B
- W-shapes: ASTM A 992
- Plate and angles: ASTM A 36

- All other shapes: ASTM A 572, Grade 50
- Reinforcing steel: ASTM A615, Grade 60 Mill Finish
- Weldable reinforcing steel: ASTM A706, Grade 60
- Smooth welded wire fabric: ASTM A185 or ASTM A82

Masonry standards are as follows. Specifications are included in Appendix L:

- Structural concrete masonry unit properties:
  - Minimum compressive strength (f'm): 2,000 psi for all masonry work
  - Concrete masonry units: ASTM C90.
- Grout: ASTM C94, f'c = 3,000 psi
- Mortar: ASTM C270, Type S

Dead loads are as follows:

- Design roof dead loads: 20 psf minimum
- Design roof collateral loads: 10 psf
- Reinforced concrete: 150 pcf
- Steel: 490 pcf
- Soil: 125 pcf
- Gravel: 145 pcf

The following live load criteria were used in addition to concentrated loads:

- Roof load: 20 psf + 10 psf pipe load (typical unless otherwise noted)

  See equipment, tank, and pump plan for additional pipe loads on sheet 0154-63W-G6-80738 of the Design Drawings in Appendix G
- Slabs on grade and floor slabs (typical unless otherwise noted): 250-psf slab, 3-kip concentrated, 3-ton forklift and AASHTO H-20 traffic load (not acting simultaneously)
- Walkways, platforms, stairs: 100 psf
- Platform: 125 psf
- Interior room ceiling: 125 psf

Wind load criteria are as follows:

- Wind importance factor: 1.0
- Basic wind speed (3-second gust – ultimate): 115 miles per hour

Snow load criteria are as follows:

- Snow importance factor: 1.1
- Ground snow load, Pg = 20 psf

Seismic load criteria are as follows:

- Seismic importance factor: 1.25

- Occupancy category: III
- Spectral response accelerations: S_s = 10.8%, S_1 = 4.7%
- Soil site class: E (per geotechnical report – CT / Verdantas, August 2025, Appendix C)
- Spectral response coefficients: S_{DS} = 17.3%, S_{D1} = 13.1%
- Seismic design category: B

6.6.2.2 Code and References

The structural design is governed by the following codes and references:

- ASCE 7-16 Minimum Design Loads for Buildings and Other Structures
- ACI 301-20 Specifications for Structural Concrete for Buildings
- ACI 318-19 Building Code Requirements for Structural Concrete and Commentary
- ACI 350-06 Building Code Requirements for Environmental Engineering Concrete Structures
- ACI – The Masonry Society (TMS) 402-16 Building Code Requirements and Specifications for Masonry Structures
- American Institute of Steel Construction 360-16 Manual of Steel Construction
- American Welding Society D1.1 Structural Welding Code – Steel
- International Code Council – 2021 International Building Code
- Precast/Prestressed Concrete Institute (PCI) – MNL 120-10 PCI Design Handbook
- PCI – MNL 122-07 PCI Architectural Precast Concrete
- PCI – MNL 123-88 PCI Design of Connections of Precast Concrete
- 2021 Michigan Building Code
- American Concrete Institute (ACI) 318 Building Code Requirements for Structural Concrete, 2019, Farmington Hills, Michigan
- American Institute of Steel Construction, 360-16, Specification for Structural Steel Buildings, 2023, Chicago, Illinois
- ASCE 7-16 Minimum Design Loads for Buildings and Other Structures, 2017, Reston, Virginia

6.6.2.3 Building Shell Delegated Design

The design of the pre-engineered metal building shell is a variation of a delegated design. The Arcadis Engineer of Record (Arcadis) provides the foundational design criteria in the design documents - this includes the building's general layout, use, location-specific loads (wind, snow, seismic), supports, and performance requirements such as deflection limits and collateral loads. These design criteria are reflected in the design drawings (Appendix G), technical specifications (Appendix L), and calculations (Appendix N).

Rather than fully designing the steel superstructure, however, the Arcadis EOR is delegating the final detailed structural design of the PEMB frame system (primary and secondary members, bracing, and connections) to the metal building manufacturer. The manufacturer will then provide detailed shop drawings and structural calculations sealed by their licensed Professional Engineer (PE), taking engineering responsibility for the structural steel framing system within the defined criteria.

This approach leverages the vendor's proprietary systems and expertise in their product. The Arcadis EOR will verify that the design intent and interface requirements (such as foundation reactions, anchor bolt locations, and coordination with architectural or MEP systems) are clearly defined and accounted for. The Arcadis EOR will review the sealed PEMB submittals for general conformance but does not assume responsibility for checking internal member sizing or proprietary connection design; this is the responsibility of the vendor's designated PE. This division of responsibility is widely accepted in the industry and supported by code, and is outlined in the technical specifications (Appendix L) and Design Drawings (Appendix G).

6.6.2.4 Foundations Delegated Design

New foundations will be required for the Water Treatment Building, as well as the water storage tanks and transformer yard area associated with the Water Treatment Building. Additionally, foundations are required for miscellaneous pipe supports across the Site.

In August 2025, Verdantas provided a report for the geotechnical subsurface investigation titled "Geotechnical Investigation for Proposed BASF NW Treatment Tanks and PEMB Building" (Appendix C) at the Site to obtain soil information. According to the geotechnical report prepared by Verdantas, due to the presence of undocumented fill materials and other unsuitable soils within the near-surface profile, as well as a predominantly very soft to soft native soil strata extending to bedrock, it is recommended that all proposed structures be supported on a deep foundation system or a specialty foundation (soil improvement) systems such as rammed aggregate piers or rigid inclusions. The deep foundation systems and/or specialty foundation (soil improvement) systems will penetrate through the existing fill and weak native soils. The geotechnical report recommended deep foundations consisting of driven steel H-piles to bear directly on competent bedrock and are expected to experience negligible settlement. The geotechnical report also recommended specialty foundation (soil improvement) systems such as Rigid Inclusions, as an option, that would be designed and detailed by a specialty foundation engineer / contractor during the construction phase.

The design of the Water Treatment Building and storage tank foundation (soil improvement) system is being executed through a delegated design process. In this approach, the Engineer-of-Record (Arcadis) establishes performance criteria and design parameters (i.e. foundation design bearing pressure, settlement limits, and modulus of subgrade reaction), which are then used by a specialty foundation engineer / contractor to develop the detailed design of the foundation (soil improvement) system during the construction phase. The specialty foundation engineer / contractor is responsible for preparing stamped design drawings and calculations that meet the required performance criteria, which will be reviewed by the Engineer-of-Record to confirm consistency with the overall project design.

To minimize risk and/or changes during the construction phase, BASF and Arcadis engaged SME, a subject matter expert in soil improvement technologies, to perform a preliminary design of the rigid inclusions during the design phase. This preliminary design for the rigid inclusion soil improvement system has been incorporated into the structural drawings (Appendix G) with the assistance of a registered professional geotechnical engineer (SME Geotechnical Consultants) based on historical soil boring logs in the area of the proposed building and tanks, as well as anticipated tank and building loads. SME will be revising their preliminary design of the rigid inclusions based on the recently provided August 2025 geotechnical report developed by Verdantas and the soil borings conducted at the proposed building and tanks. SME's updated preliminary rigid inclusion design will be included in the 100% design submittal.

6.6.2.5 Miscellaneous Structural Criteria

Notable additional design considerations include:

- Perimeter railings at the office space ceiling floor are steel and are designed and detailed in such a manner that considers long service life and resistance to corrosion.
- Unless specifically noted otherwise, heavy-duty grating will be steel with depth sufficient to withstand a 250-psf slab, 3-kip concentrated, 3-ton forklift and AASHTO H-20 traffic load (not acting simultaneously).
- The office space within the PEMB will consist of 8-inch masonry walls with a precast concrete hollow core plank roof deck with a no grout topping.

These considerations are reflected on the design drawings (Appendix G) and in the technical specifications (Appendix L).

6.7 Electrical Design

The electrical scope of the project consists of power distribution for the new AG treatment system (treatment equipment), the collection and conveyance system (sump pumps, sump enclosures, controls, tanks and aboveground pipe heating), and the Water Treatment Building (lighting and heat). The electrical design is depicted on the design drawings (Appendix G) and outlined in the technical specifications (Appendix L).

The new main switchboard will be tied into the existing 13.8-kilovolt (KV) City of Wyandotte electrical system via a new 13.8-KV-480-volt (V) transformer at the new transformer yard, located on the southwest side of the Water Treatment Building. The new main switchboard will be a 1,600-ampere (A), three-phase, four-wire distribution board with critical and non-critical bus.

All power feeders (and instrumentation and control circuits) from the main switchboard to field equipment will be installed in underground duct banks. Exposed conduit and a cable tray will be used in the Water Treatment Building.

6.7.1 Design Criteria

The electrical design of this project is governed by electrical code requirements adopted by the State of Michigan at the time of completion of this design. Presently, Michigan has adopted the following code requirements:

- 2021 Michigan Building Code
- 2023 National Electrical Code

Electrical design criteria are summarized below.

Electrical Distribution and Utilization:

- 480-V, three-phase 60-Hertz power derived from the existing City of Wyandotte electrical system, transformed to 480 V by a new pad-mounted transformer provided by BASF.

Standby Power:

- Backup power provided by the City of Wyandotte electrical system. Following a power outage, BASF staff will contact the city operator, who will manually turn on the backup power supply.

Power Distribution Equipment:

- 480-volt-alternating-current (VAC), 1600-A, three-phase, four-wire switchboard serving the main control panel, low-voltage panelboard via three phase step-down transformers, and other 480-V, three-phase equipment.
- 480-V switchboard serving sump panels located near sump pumps along the river. These panels then distribute 480-V power to groups of sump pumps and to mini power zones, which step down voltage to 120/240 V, and provide power to control panels, outlets, and enclosure heaters.
- 480-V switchboard providing power to the new Water Treatment Building and the building's associated mechanical and process equipment. Includes a charging station for electric forklifts.

Raceways:

- Schedule 80 polyvinyl chloride conduit below grade, encased in concrete and reinforced under roadways.
- Galvanized rigid steel or intermediate metal conduit for exposed locations.

Pull and Junction Boxes:

- 316 stainless steel.

Cables:

- Standard 600-V, Underwriter's Laboratory (UL)-listed single conductor cables.
- VFD cable: Stranded copper, 2,000 V, with three symmetrical grounds, overall shield.

Lighting:

- Exterior lighting and control are designed for 277 VAC and will be controlled via lighting contactors with photocell or astronomical timer control devices. Light-emitting diode (LED) fixtures will be used. Battery-backed lighting will be required at all areas of common egress from the building.
- Interior and exterior building lighting will consist of LED light fixtures with a target color temperature of 4,000 Kelvin (neutral white).
- Emergency and egress lighting will be provided to facilitate safe exit from the building in case of a power failure. Emergency lighting will rely on battery backup power. Emergency lights will be located so that designated egress paths will have a minimum 1 foot-candle of illumination.
- Exit signs will be LED-type fixtures.

Grounding:

- An equipment grounding conductor will be provided in each raceway, and grounding rings/grid will be provided around the Water Treatment Building, with driven ground rods and electrodes where possible, including grounding connections to outdoor tanks.

6.8 Heating, Ventilation, and Air Conditioning Design

The heating, ventilation, and air conditioning (HVAC) systems have been designed to support the overall Water Treatment Building and meet code as necessary. The HVAC systems will maintain temperature in the space and provide ventilation to all areas as required by code. HVAC design drawings are provided in Appendix G, with corresponding specifications in Appendix L and supporting calculations in Appendix N.

The Water Treatment Building will be heated to provide freeze protection in the winter and will be maintained at approximately 10 degrees Fahrenheit (ºF) above ambient temperature during the summer. Ventilation for the building will be provided during operational hours by a dedicated outside air handling unit (AHU). This AHU will have an electric heating coil to heat the air up to the setpoint during heating season but will not provide any space heating. Temperature during the heating season will be controlled by electric unit heaters. The AHU will be interlocked with the exhaust fan to maintain pressure in the room. During the cooling season, the AHU will still run to provide ventilation but will also run to bring in outside air to "cool" the space. The space will not be traditionally cooled, but rather outside air will be brought in to provide for sufficient air movement so that the space does not overheat and is maintained at approximately 10ºF above the outside air temperature. A second exhaust fan and a louver will be interlocked to allow for additional air flow for occupant comfort during the cooling season.

The electrical room will be served by a dedicated wall-mounted mini-split ductless air conditioning unit. This unit will provide cooling only, no heating.

The office and bathroom will be served by a duct-mounted mini-split unit. Ventilation and exhaust for the office, bathroom, and electrical room will be provided by an energy recovery ventilator ducted into the return duct of the fan coil unit. The mini-split unit will provide heating and cooling for these spaces.

During a shelter-in-place emergency, the designated shelter area is the office and bathroom. This combined space will provide approximately 10 total manhours of oxygen without use of additional mechanical means of oxygen. When the shelter-in-place command is executed, isolation dampers will close all outside air and exhaust dampers and dampers serving the electrical room isolating this space.

The HVAC controls system will be a standalone system serving this building only. It will integrate all HVAC systems via a building-level controller and a laptop that can act as a front end. This front-end system will allow for monitoring of the HVAC system, adjusting control points, monitoring alarms, and tracking utility consumption at the building.

6.8.1 Design Criteria

The mechanical design of this project is governed by the following building codes:

- 2021 International Mechanical Code with Revisions (Michigan Mechanical Code)
- 2021 International Energy Conservation Code with Revisions (Michigan Energy Code)

The following codes and standards were used in the mechanical design of this project:

- ASHRAE Standard 15: Safety Standard for Refrigeration Systems
- ASHRAE Standard 62.1: Ventilation for Acceptable Indoor Air Quality
- ASHRAE Standard 90.1: Energy Standard for Sites and Buildings Except Low-Rise Residential Buildings
- Sheet Metal and Air Conditioning Contractors' National Association (SMACNA) Duct Construction Standards

The following BASF Standards were used in the mechanical design of this project:

- N-R-CL 200 M: Heating, Ventilation, and Air Conditioning (HVAC)
- N-S-CL 205: Heating, Ventilation, and Air Conditioning (HVAC) Best Practices

Outdoor design conditions based on ASHRAE 0.4% cooling and 99.6% heating criteria for Grosse Ile, Michigan (World Meteorological Organization #725373) are as follows:

- Winter heating dry bulb temperature: 3.4°F
- Summer cooling dry bulb temperature: 89.9°F
- Summer cooling mean coincident wet bulb temperature: 74.3°F

Indoor design conditions are as follows:

Office

- Heating dry bulb temperature: 72°F
- Cooling dry bulb temperature: 72°F (no relative humidity control)

Toilet Room

- Heating dry bulb temperature: 61°F
- Cooling dry bulb temperature: 77°F (no relative humidity control)

Electrical Room

- Heating dry bulb temperature: 68°F
- Cooling dry bulb temperature: 85°F (no relative humidity control)

Process Areas

- Heating dry bulb temperature: 50°F
- Cooling dry bulb temperature: 10°F above outside ambient temperature (no relative humidity control)

6.9 Plumbing Design

The plumbing system is designed to support the overall Water Treatment Building and meet code as necessary. The plumbing system will provide domestic water and sanitary connections to fixtures as required by code as well as connections to the process system. Plumbing design drawings are provided in Appendix G, with corresponding specifications in Appendix L and supporting calculations in Appendix N.

The building will have a single water connection connected to the existing site domestic cold water utility system. After the water enters the building and travels through a water meter, the water will be broken out into two separate water systems and associated backflow preventers: the domestic water system and the non-potable water system. The domestic water system will include the tepid water system serving the emergency fixtures.

Domestic cold water will be provided to the toilet room water closet and lavatory, and a service sink in the process area. Domestic hot water for the service sink and the lavatory will be provided by point-of-use domestic water heaters mounted on the walls next to the fixtures.

Tepid water will be provided by a dedicated electric tank type tepid water heater. Tepid water is being provided for all safety showers and eyewash stations. A recirculation loop with a recirculation pump will be used to ensure all fixtures have sufficient tepid water available. Outdoor tepid water heating will have heat trace. The emergency fixtures will be located as required by applicable codes and standards. The emergency fixtures consist of combination emergency showers/facewash fixtures.

The non-potable water system will serve the process system connections (i.e., chemical dilution water), the hose bibbs inside the Water Treatment Building, and the frost-proof wall hydrants on the exterior of the building. The intent for having a separate non-potable water system is to prevent the possibility of any fluid back feeding from the process system into the potable water system.

The sanitary system will connect to the toilet fixtures. The emergency fixtures will drain to the open floor in the process area or to the ground outside. The service sink will drain to the building sump rather than the sanitary system. The sanitary system will exit the building and connect to the lift station adjacent to the building.

The condensate system connected to the HVAC equipment will drain to the sanitary system.

Floor drains and/or trench drains in the process area and storage rooms will be connected together to a combined sump and then pumped to the equalization tank for the AG treatment system. This will prevent accidental discharges of untreated effluent from the Water Treatment Building in the event of a spill/release in the process area. A sump pump will be provided as required. Because the emergency fixtures will be placed in the process area, they will be drained to the sump rather than the sanitary system.

6.9.1 Design Criteria

The plumbing design of this project is governed by the following building codes:

- 2021 International Plumbing Code with Revisions (Michigan Plumbing Code)
- 2021 International Energy Conservation Code with Revisions (Michigan Energy Code)

The following codes and standards were used in the plumbing design of this project:

- American National Standards Institute (ANSI) Z358.1-2014: Emergency Eyewash & Shower Standard

6.10 Controls Design

The control system for the proposed treatment system is designed to execute process logic (as defined in the process control description in Specification 40 61 96 in Appendix L) through a PLC and to provide SCADA capabilities through HMIs. This equipment executes the control loops and interlocking features while providing operators with an interface to view system status/alarms, control equipment (through hand/off/auto switches and process setpoints), and access historical trends. Instrumentation and control design drawings are provided in Appendix G, with corresponding specifications in Appendix L and an interlock table in Appendix S.

Duplicate HMIs will be located throughout the Water Treatment Building and the well field to provide operators with access to system information from multiple locations on the Site. Physical access to these HMIs and the associated network hardware will be controlled through locks on panel doors and/or building/enclosure doors.

A modular main control panel (MCP) located within the electrical room will house the primary PLC and HMI for operating the treatment system along with the motor controls for all pumps and mixers. Modular segments of the panel are designed such that the compartment containing the PLC will have no voltages exceeding 120 V and will be physically separated from the compartments containing the higher voltage equipment. All VFDs will be networked back to the primary PLC utilizing Ethernet switches and CAT6 cabling, reducing the input/output (I/O) requirements for the system. An emergency stop (E-Stop) circuit has been designed to shut down all PLC outputs to control equipment by removing the control power to the outputs. This circuit will also direct connect to the VFD stop command so that activation of any E-Stop will stop all VFDs regardless of the state of the PLC or the operating mode of the VFD.

Twenty-five remote control panels (RCPs) and associated motor control enclosures will be located throughout the well field and in the storage area to drive sump pumps, extraction well pumps, and water storage tank pump motors, and to receive signals from sump, well, and storage tank area instruments. These RCPs will contain remote I/O racks, and the associated motor control enclosures contain VFDs that communicate to the MCP through a single-mode fiber optic network utilizing a star configuration, with wells grouped into smaller ring networks based on how power is fed to them. Where possible, to minimize fiber optic cabling, different strands within the same cable will be used to communicate to each RCP location. These strands will be patched together in patch panels located at each RCP and the MCP.

Up to four sump control panels will be equipped with HMI screens to allow the operator to make changes to the sump operation from the field. The network communications diagram (Appendix G) indicates which sumps will include HMIs.

Several vendor systems will be used to control specific equipment throughout the treatment system. Each vendor system will provide its own PLC and HMI for control of the vendor equipment. Vendor PLCs will be connected to the main treatment system PLC through Ethernet switches and CAT6 cabling. This connection will be used to send vendor status, alarm, and data to the main HMI along with the necessary permissive signals to achieve interlocking control. The main HMI will not be used to directly control vendor equipment, but rather to provide information on the vendor system to the operator in the control room and to include vendor data when assessing historical trends.

Panels containing PLCs will not contain voltages greater than 120 V and will have an uninterruptable power supply. Remote locations will contain RCPs with remote I/O and remote motor control enclosures with VFDs. The separation of voltages is to minimize the electrical hazard to operators accessing the PLC components. All control panels will have their own surge protection equipment to protect the sensitive electronics contained in these panels.

6.10.1 Design Criteria

The following design standards apply to instrumentation and control for the proposed AG Treatment System:

- BASF design guidelines and specifications
- 2023 NFPA 70 National Electrical Code
- Institute of Electrical and Electronic Engineers (IEEE)
- UL
- International Society of Automation (ISA)
- 2023 NFPA 79 Electrical Standard for Industrial Machinery

7 Waste Management and Characterization

Waste management activities will be conducted in compliance with applicable federal, state, and local waste regulations. Procedures for on-site management, sampling, characterization, and disposal of waste are documented in the site-specific Waste Management Plan (Appendix R).

Anticipated waste types include excess soils and debris, groundwater, municipal solid waste and debris, dewatering solids (sludge cake), spent carbon, ion exchange media, bag filters, and cleaning fluids. Based on knowledge of historical soil sampling results, it is not expected that any hazardous waste will be generated during construction or operation of the perimeter barrier remedy. If characteristic or listed hazardous waste is identified during site activities, all applicable regulatory requirements would be addressed through an amendment to the Waste Management Plan.

All waste characterization will follow a documented quality assurance and quality control program, incorporating field duplicates and matrix spike samples for purposes of accuracy and representativeness. Waste will be tracked using manifests and final disposal documentation, including profiles, approvals, and analytical results, which will be retained for project records.

8 Health and Safety Plan

A project-specific health and safety plan (HASP) will be developed for the construction and operation of the perimeter barrier remedy at the Site. The HASP will describe the health and safety commitment of office and field employees, contractors, and site visitors. The HASP will be structured to contain information regarding emergency points of contact and details of the hospital route. The HASP will be supplemented by appropriate Job Safety Analyses (JSAs) for safety-critical tasks conducted on the Site. It is expected that these JSAs will be modified in the field by the personnel conducting the tasks to integrate real-time conditions and hazards at the time of the task. Safety Data Sheets will be available for all materials managed on the Site during construction and operation of the proposed remedy.

Tasks performed under the project HASP will follow the BASF Health and Safety (H&S) Standards of Procedure. All project personnel will be required to sign the certification page included at the end of the HASP acknowledging that they have read, understand, and will abide by the plan. Any supplemental contractor HASP that addresses specific hazards for tasks conducted by the subcontractor will be stored with the project-specific HASP.

8.1 Health and Safety Considerations

During construction and operation of the proposed remedy, H&S protocols will be developed, implemented, and enforced to provide for the safety of project team members and visitors to the Site. Examples of H&S considerations during construction and operation of the perimeter barrier remedy include the following:

- Potential hazards during construction activities conducted in the Detroit River or any water body;
- Potential hazards during excavation and shoring activities (i.e., cave-ins);
- Potential to encounter below- and above-grade utilities;
- Heavy equipment operation risks;
- Fall protection;
- Confined spaces;
- Potential to encounter impacted vapors, soil, and/or groundwater; and
- Handling of chemicals associated with construction and/or operation of the AG treatment system process.

8.2 Site Safety

Personnel working on the project are responsible for completing tasks safely and have the authority to stop the work of a coworker or contractor if working conditions or behaviors are deemed unsafe. BASF and OSHA general safety equipment requirements, including personal protective equipment (PPE) standards, will be identified and followed by all personnel involved in the construction and operation of the proposed barrier remedy. Staff and subcontractors will be required to complete a safe work permit and pre-task plan prior to the start of each workday, describing the tasks to be performed that day with an evaluation of hazards and mitigations for those hazards. Prior to initiation of site activities, all staff and contractors will be required to complete BASF site-specific safety and site-awareness training. All contractors working at BASF are screened by the Avetta system requiring a total recordable incident rate 3-year average less than or equal to 1.80, an lost work day rate 3-year average less than or equal to 1.00, and an experience modification rate for the previous year less than or equal to 1.00. In special circumstances where a contractor has exceedingly unique capabilities but does not meet the requirements, exceptions can be granted with special permitting and surveillance requirements.

8.3 Safety in Design

A hazard analysis was conducted during the Intermediate (60%) Design phase using the BASF step review process to identify hazards associated with the perimeter barrier remedy. The BASF step review process consists of various “steps” dependent on project scale and scope. The first of two step reviews was completed for the Intermediate (60%) Design. Key aspects of the step design review included:

- Plot plans and proximity to risk receptors;
- Chemical and physical hazards;
- Air emissions and water discharge;
- Generation of solid and liquid waste;
- Noise generation;
- Fire protection concepts; and
- Emergency response planning and management.

During the Pre-Final (95%) Design phase, the second of the two step reviews was started and built upon the initial hazard assessment to provide additional guidance for best practices associated with system fail safes, remedy construction, OMM, and regulatory codes. The second step review will continue into the Final (100%) Design phase to systematically identify potential concerns and outline engineering and administrative controls that can mitigate the identified hazards. The second step of the hazard analysis includes:

- Review of the hazard analysis completed in the previous step;
- Analysis of chemical reaction hazards;
- Analysis of thermal hazards;
- Analysis of dust hazards; and
- Analysis of interlock controls.

8.4 Release Containment

Secondary containment will be provided for all fluid-containing tanks and vessels. The following controls will prevent chemical releases to the ground:

- The system will be equipped with alarms to disable the system should a spill or release to one of the secondary containments occur (e.g., leak switches within the containment footprints).

- E-Stop buttons will be located on the MCP and the clarifier within the Water Treatment Building.
- The AG treatment system will have secondary containment for all process tanks, equipment, and piping within the building. The containment will consist of a 6-inch-high curb around the exterior of the building. The volume of the containment area will exceed 110% of the volume of the largest tank (equalization tank, which is 20,000 gallons).
- Energized equipment will be placed on elevated concrete pads to keep all electrical components out of potential water in case of a release.
- The outdoor sulfuric acid tank will be located within a coated concrete secondary containment, capable of holding 110% of the tank volume plus 6 inches.
- The outdoor water storage tanks will be located within a bermed, lined secondary containment area, capable of holding more than 110% of the largest tank volume plus 6 inches to account for the potential addition of a third storage tank in the future.
- The concrete paving in the sulfuric acid truck delivery area will include a chemical-resistant coating and curbs to direct any spills to a sump, which must be manually emptied.

9 Future Design Considerations

Following development of the Final (100%) Design, during remedy operation, optimization of the barrier system will be considered to meet the approved performance standards. The perimeter containment barrier, extraction network, conveyance network, and AG treatment system design will allow for potential adaptive management, optimization, and/or expansion to potentially improve upon the effectiveness of the perimeter barrier remedy to meet the performance objectives. Potential optimization actions may include the following:

- Enhancement of the containment barrier through welding of sheet plates or jet grouting;
- Addition of supplemental infrastructure (vertical wells within existing drains, vertical wells outside drains, horizontal wells, or new drains and sumps) to meet the performance standards;
- Expansion or contraction of the AG treatment system to optimize the process stream capacity based on the potential of increased or decreased influent flow rates or to optimize treatment; and
- Further separation of stormwater from groundwater by managing and diverting stormwater runoff away from the collection drains.

10 Data Gaps

No additional data gaps were identified during the Pre-Final (95%) Design phase. All previous data gaps identified during the Preliminary (30%) and Intermediate (60%) Design phases were addressed as described in Section 2.3.

11 Permit Plan

Various regulatory permits will be required to facilitate construction and operation of the perimeter barrier remedy. In July 2023, notifications of the intent to install the barrier remedy were sent to the relevant permitting agencies (DUWA, EGLE, and the City of Wyandotte). Following is a summary of and schedule for the anticipated permits required prior to or during construction:

- **DUWA**. Discharging treated groundwater into a DUWA sewage collection system tributary requires an approved Discharge Permit. A permit application was submitted to DUWA on July 14, 2025. BASF anticipates a response from DUWA within 90 days of submittal.
- **EGLE/USACE**. A Joint Permit Application (JPA) is currently being finalized and will cover requirements derived from state and federal rules and regulations for construction activities within and near the Detroit River. Arcadis/BASF submitted a JPA Pre-Meeting request on July 20, 2023, and participated in a JPA Pre-Meeting with USACE on August 31, 2023. The JPA application is anticipated to be submitted to EGLE in the fall of 2025 following the submittal of the Pre-Final (95%) Design.
- **City of Wyandotte**. Construction of the barrier remedy will require permits from the City of Wyandotte Engineering and Building Department including, but not limited to, an SESC Permit, a Right-of-Way Permit, a Building Permit, an Electrical Permit, a Plumbing Permit, a Mechanical Permit, and a Utility Permit. Arcadis/BASF notified the City of Wyandotte of the general construction plans on July 20, 2023 and have been in contact with the City regarding permit requirements throughout the Pre-Final (95%) Design phase. The SESC permit application is anticipated to be submitted to the City for review and approval in the fourth quarter of 2025. The permit application will include SESC plans and detailed drawings describing areas of disturbance, proposed changes, drainage patterns, sensitive areas, and stormwater best management practices developed as part of the Pre-Final (95%) Design (Appendix G), as well as other supporting documents as required. The remaining permit applications, where necessary, will be submitted to the City several months before the beginning of construction, tentatively anticipated in 2026.
- **National Pollutant Discharge Elimination System (NPDES) Construction General Permit by Rule.** A Construction General Permit will be submitted to EGLE for approval prior to the start of construction. This permit is required under the Clean Water Act for construction projects that disturb 1 or more acres of land in order to mitigate runoff of sediments and chemicals from construction sites. The permit will include a Notice of Coverage submittal and a list of required inspections to verify compliance with the permit. The permit application requires an approved SESC Permit by the City of Wyandotte to be obtained first. After the SESC Permit is approved, BASF will submit the Notice of Coverage to EGLE several months before the beginning of construction, tentatively anticipated in 2026.
- **Air Quality Permit to Install**. A Permit to Install application for the construction and operation of the AG treatment system is being developed by BASF. A Permit to Install is required to install, construct, reconstruct, relocate, or modify any process or process equipment that may emit an air contaminant. It is anticipated that the Permit to Install application will be submitted to EGLE by early 2026 to allow adequate time for review and approval prior to the start of construction.
- **Spill Control**. All liquids included in the AG treatment system design and operation have been evaluated to determine spill control requirements. The tanks or drums containing corrosive liquid are subject to containment standards set by federal, state, OSHA, and NFPA code. The barrier remedy does not involve storage of petroleum products above regulatory thresholds; however, the remedy does include storage of substances regulated by the State of Michigan Part 31, Part 5 Rules. BASF will update the relevant sections of its site-specific Integrated Contingency Plan and the regulated substances will be stored in compliance with the Part 5 material storage requirements.

Additional activities to be completed prior to or during construction include the following:

- Access agreements will be obtained with adjacent property owners for installation of sheet pile along Perry Place and James DeSana Drive.

- Institutional controls will be recorded with the City of Wyandotte to restrict excavation into the off-site sheet pile and collection drain.
- A Notice to Mariners will be provided.

12 Operation, Monitoring, and Maintenance Requirements

An OMM Manual including schedules and specific procedures for each remedy component is included in Appendix K. The OMM Manual addresses routine inspections of barrier components (e.g., new and existing bulkheads and the subsurface barriers); OMM of the AG treatment system; collection of performance monitoring data for active remedy components; and anticipated compliance monitoring.

13 Remedial Project Management

This section describes the approach for design and implementation of the perimeter barrier remedy.

13.1 Value Engineering

As design of the extraction and conveyance network, AG treatment system, and perimeter barrier progressed through the Pre-Final (95%) Design phase, the design elements continued to be evaluated in an effort to reduce overall costs as part of ongoing VE. Results of continued VE during the Pre-Final (95%) Design phase are summarized below.

Extraction and Conveyance System

- Installing collection drains and sumps using one-pass trenching technology to simplify construction and minimize the need for dewatering:
    - Allows for the trench to be excavated, the sumps and perforated pipes to be installed, and the trench to be backfilled with stone simultaneously using a single piece of equipment.
    - Reduces the need for trench boxes and shoring and is expected to greatly reduce the need for dewatering.
    - Facilitates a more straightforward installation process, ultimately enhancing efficiency and minimizing disruptions to existing infrastructure.
- Minor adjustments have been made to the alignment of drains and conveyance piping to streamline construction by reducing conflicts with existing utilities, infrastructure, and buried debris.
- Standard “off-the-shelf” sump enclosures were selected over custom below-grade vaults to streamline construction, minimize the need for confined space, and possibly allow for off-site construction of piping manifolds during construction.
- A reduced number of sump pump models have been specified for operation of the extraction system, allowing for more streamlined OMM and less potential for downtime.

- In order to minimize waste volumes requiring disposal, reusing excavated soil as fill, to the extent possible, and employing a temporary construction water treatment system to treat and discharge water to DUWA instead of trucking it off Site for disposal has been included in the project.

AG Treatment System

- Addition of a slipstream to eliminate unnecessary treatment (pH neutralization and metals precipitation) of select extracted groundwater in the future where possible.
- Reduced number of pump models needed to facilitate treatment.
- Decreased filter press size required to manage solids based on further design refinement and evaluation.
- Further refinement of the pH neutralization process and equipment to allow for better overall pH control and subsequent streamlined OMM.
- Selection of allowable equipment vendors based on existing site vendors already familiar with BASF technical specifications and requirements to streamline the procurement process.
- Selection of “off-the-shelf” equipment and tanks to avoid costly custom builds and allowance for substituted equipment that meets the design specifications at a reduced cost.
- Use of pre-designed skids or equipment packages where possible (chemical feed skids and clarifier).
- Specification of rigid inclusions for ground improvement for the Water Treatment System building instead of a deep foundation with driven H-piles or micropiles.
- Incorporation of motion-activated lighting to minimize electrical usage.
- Elimination of the required 2-hour fire wall between the Water Treatment System building and the sulfuric acid tank by moving the tank more than 20 feet from the building.

South Dock Bulkhead

- Use of LCC instead of aggregate fill as backfill beneath the existing dock structure to reduce the load on the headwall and the settlement of backfill, and to allow for more streamlined construction (more flowable material).
- Selection of AZ-36 headwall pile, instead of the formerly selected AZ-48 sheet pile, based on the reduced load resulting mainly from the LCC as backfill beneath the existing dock.
- Revised headwall tip elevation based on the revised load conditions. Pile tips were generally raised to the top of the glacial till instead of extending to bedrock, with the exception of the northern end of the headwall, where a minimum of a 2-foot embedment into the glacial till layer is required.
- Selection of a deadman anchor wall instead of an A-frame anchor wall based on further constructability reviews.
- Inclusion of tie rod “crossovers” or angled tie rods to streamline construction by avoiding the historical outfall and intake structures.
- Incorporation of a regrading plan behind the new bulkhead for improved stormwater management in the area and a reduction in the amount of direct infiltration into the drains.

Appendix M includes a comparison of the alternatives evaluated for the South Dock area of the containment barrier alignment.

Northern and Southern Subsurface Sheet Pile Walls

- Minor adjustments to the alignment of subsurface barriers to streamline construction by reducing conflicts with existing utilities, infrastructure, and buried debris.
- Selection of AZ-24 sheet pile, instead of the formerly selected NZ-22 sheet pile, to provide consistency with existing and new bulkhead sheet pile type and interlock and streamline procurement and construction.
- Minor adjustments to pile depths based on the refined design top-of-clay elevation.
- Incorporating the use of spliced sheet piles for installation beneath the overhead electrical lines along James DeSana Drive to eliminate the need for a jet grout barrier through that the area.
- Design of interlock joint connections between the existing bulkhead and new sheet piles. This eliminates jet grout columns originally proposed at the intersections of the existing bulkhead with the northern subsurface wall and with the new South Dock headwall in this area.

Appendix M includes a comparison developed for the Intermediate (60%) Design that focuses on the alternatives evaluated for the southeastern portion only of the subsurface containment barrier alignment.

BASF will continue to evaluate cost versus function to identify any additional high-cost design elements that can be modified or eliminated without compromising performance, reliability, quality, or safety.

13.2 Design and Construction Schedule

A design and construction schedule is provided in Appendix T. To maintain the design and construction schedule provided, BASF assumes the comments provided by USEPA on the Pre-Final (95%) Design will be editorial in nature, will not have design change implications, and will not require discussions with USEPA. If comments received on the Pre-Final (95%) Design are more than editorial nature, additional time will be required to discuss, address, and incorporate comments into the Final (100%) Design. This additional time will directly affect the start date of construction. Similarly, the schedule assumes the Final (100%) Design will be approved by USEPA with no comments. If comments are received on the Final (100%) Design, additional time will be required to discuss, address, and incorporate comments into the Final (100%) Design and will directly affect the start date of construction.

The construction schedule in Appendix T reflects sequencing and durations that are implementable and achievable. The schedule is structured such that the start-up phase of the AG treatment system may begin in some sections of the Site earlier than others as perimeter barriers are completed for the purpose of expediting the start-up phase. Ongoing optimization of the construction schedule will be completed throughout the bidding, procurement, and construction process based on site operations, stakeholder input, procurement lead times, etc. Therefore, the project task schedule and/or sequencing may change as the project progresses through this process.

13.3 Construction Approach

13.3.1 Cost Estimate

The estimated cost to construct the containment barriers, extraction/conveyance network, and AG treatment system based on the proposed Pre-Final (95%) Design is between $72.9 million and $88.2 million. The cost estimate is presented in Appendix U and includes estimated construction, annual OMM, and performance monitoring costs.

13.3.2 Procurement Methods and Contracting Strategy

Contractors for construction, start-up, and OMM of the perimeter barrier remedy will be selected in conformance with BASF purchasing and procurement requirements, and considering factors such as cost, qualifications, and H&S. The current contracting strategy is a design-bid-build approach. Under this approach, the design would be reviewed and approved by USEPA at the Final (100%) Design phase. The design-bid-build approach allows bidding from multiple contractors on well-defined work that can be implemented using standard construction methods (i.e., significant design modifications or constructability issues are not anticipated).

Following USEPA approval of the Final (100%) Design, the design drawings and specifications will be issued for construction bidding. As part of the construction phase, selected vendors for major equipment, including the electrical systems, treatment equipment, and control panels, will be required to prepare detailed fabrication-level shop drawings and supporting calculations. In addition, the PEMB design will be delegated to the selected vendor for production of fabrication drawings.

These submittals must demonstrate full compliance with the design intent and technical parameters outlined in the approved Final (100%) Design documents. All shop drawings and calculations will be submitted for review and approval by the Engineer prior to fabrication. Any deviations from the Final (100%) Design must be clearly identified, justified by the vendor, and formally approved by the Engineer to ensure alignment with project performance, safety, and regulatory requirements.

This submittal and review process facilitates consistency across disciplines, coordination between trades, and mitigation of construction-phase design conflicts.

13.3.3 Construction Sequence

The construction sequence of the new bulkhead, subsurface barriers, extraction and conveyance network, and AG treatment system is generally that the AG treatment system and groundwater extraction and conveyance system will be completed prior to completion of the perimeter barriers. This sequence is important so that groundwater can be managed before the perimeter barriers are completed to avoid flooding of the Site. Therefore, the general construction sequencing is expected to be as follows:

Extraction and Conveyance System Construction

- Utility clearance, site preparation, and clearing and grubbing.
- Collection drain pre-trenching for obstruction removal and installation/grading of work platforms for one-pass trenching.
- Installation of collection drains with one-pass trenching.
- Installation of extraction well and piezometers.
- Installation of conveyance piping, including sump enclosures, clean outs, and air release valves, and hydrostatic leak testing.
- Installation of conduit and wire for extraction and conveyance system.
- Backfilling and surface restoration of collection and conveyance system trenches.

- Mechanical connection of sumps and extraction well to conveyance system.
- Electrical connection of sumps and extraction well equipment.

AG Treatment System Construction

- Procurement of PEMB.
- Site preparation and over excavation for PEMB foundation.
- Installation of PEMB foundation and rigid inclusion system.
- Erection of PEMB and connection of utilities.
- Installation of PEMB interior rooms and electrical/mechanical rough-in.
- Installation of AG treatment system equipment (tanks, pumps, mixers, etc.).
- Installation of AG treatment system piping.
- Installation of new electrical panels and connection to AG treatment system equipment.
- Installation of control panels and connection to AG treatment system instrumentation and equipment.
- AG treatment system start-up and testing to confirm operation.

South Dock Bulkhead Construction

- Pre-trenching and debris removal along the anchor wall alignment where subsurface obstructions are known to exist and cannot be avoided.
- Preparation of the South Dock area for installation of tie rods and placement of backfill under the existing dock, including concrete cutting along the concrete bulkhead at the face of the wharf and creation of access holes through the existing concrete deck.
- Application of sheet pile interlock sealant (may be performed off Site by steel supplier or fabricator) for the bulkhead.
- Installation/driving of bulkhead wall and deadman anchor wall to specified depths.
- Installation of walers and structural connection elements.
- Backfilling with LCC behind the new bulkhead wall and under the existing concrete deck.
- Installation of tie rods, including soil removal/trenching along tie rod alignments.
- Completion of surface restoration as appropriate including stormwater management regrading to direct stormwater away from the collection drains behind the South Dock.

Northern and Southern Subsurface Sheet Pile Wall Construction

- Pre-trenching and debris removal along the alignment of the northern and southern subsurface sheet pile walls where subsurface obstructions are known to exist and cannot be avoided.
- Application of sheet pile interlock sealant (may be performed off Site by steel supplier or fabricator) for the subsurface sheet pile walls.
- Installation/driving of subgrade sheet pile walls to specified depths.
- Removal of the top of the sheet pile to a maximum of 12 inches below final grade.
- Backfilling over the sheet pile with fill, utility reconnection, and surface restoration, as appropriate.

Waste will be managed in accordance with the Waste Management Plan (Appendix R). Refinement of construction methods and sequencing will continue during the Final (100%) Design phase.

13.4 Phasing Alternatives

The schedule for construction provided in Appendix T reflects sequencing and durations that are implementable and achievable. To accelerate the project, phasing alternatives will continue to be considered and evaluated to meet or fast-track the remedial strategy approach and scheduling throughout the Final (100%) Design and construction phase. Alternate construction sequences may be implemented to mitigate the risk of delays or improve scheduling timelines. BASF will continue to have monthly check-in meetings with USEPA to discuss work completed since the last meeting and the schedule for upcoming work. Any actionable feedback requiring phasing alternatives will be implemented accordingly.

13.5 Utility Conflicts

Various utilities have been identified in the work zone based on review of utility maps, site reconnaissance, discussions with the City of Wyandotte and site personnel, and geophysical surveys. Identified utilities include buried and overhead electric, potable water supply, river water supply, storm and sanitary sewer, natural gas, and communication lines. Existing and proposed utility locations are detailed in the Design Drawings (Appendix G).

Based on the geophysical surveys and utility evaluation conducted during the Intermediate (60%) and Pre-Final (95%) Design phases, the alignments of the barrier walls, collection drains, and conveyance piping have been routed, where possible, to avoid conflicting with utilities. Utilities that remain in conflict with the proposed barrier remedy installation have been catalogued and classified based on their importance. Some utilities may be temporarily discontinued or bypassed around construction. For example, the sanitary sewer line in Perry Place may be temporarily bypassed during installation of the sheet pile wall. However, crucial utility services, such as the water line entering the Site from James DeSana Drive, will be preserved and worked around during the installation. In addition, utility surveys and permitting will be conducted prior to construction to confirm the locations of utilities. For off-site excavation or installation, MISS DIG will provide the permit; for on-site activity, BASF will provide the excavation permit through its process that involves extensively evaluating historical information and utility maps, marking active utilities, and holding a pre-excavation conference with the construction contractor operators and supervisor.

13.6 Roles and Responsibilities

The responsibilities of key personnel involved in the RD process are summarized in Table 24.

Table 24. Project Roles and Responsibilities

Name	Title	Organization
Mr. Michael Gerdenich	Expert, Remediation Senior Specialist	BASF
Mr. John Sirko	Project Manager	BASF
Mr. Louis Sreniawski	Project Manager	BASF
Ms. Jacelyn Saling, PE	Project Manger	Arcadis
Ms. Andrea Krevinghaus, PE	Engineer of Record	Arcadis
Ms. Mandy Giampaolo, PE	Engineer of Record	Arcadis
Ms. Valerie Voisin	Project Manger	USEPA
Mr. Marc Messina	Project Manger	EGLE

13.7 Sample and Data Collection Methodology and Quality Assurance

Sampling and analytical activities conducted have followed quality assurance/quality control procedures as detailed in the RFI Quality Assurance Project Plan (Environmental Science & Engineering, Inc. 1996) and its subsequent revisions and addenda (Arcadis 2008), collectively referred to as the existing Quality Assurance Project Plan (QAPP). A QAPP supplement was prepared to address laboratory activities proposed in the RD Work Plan that extend beyond the scope of the existing QAPP (Arcadis 2025c).

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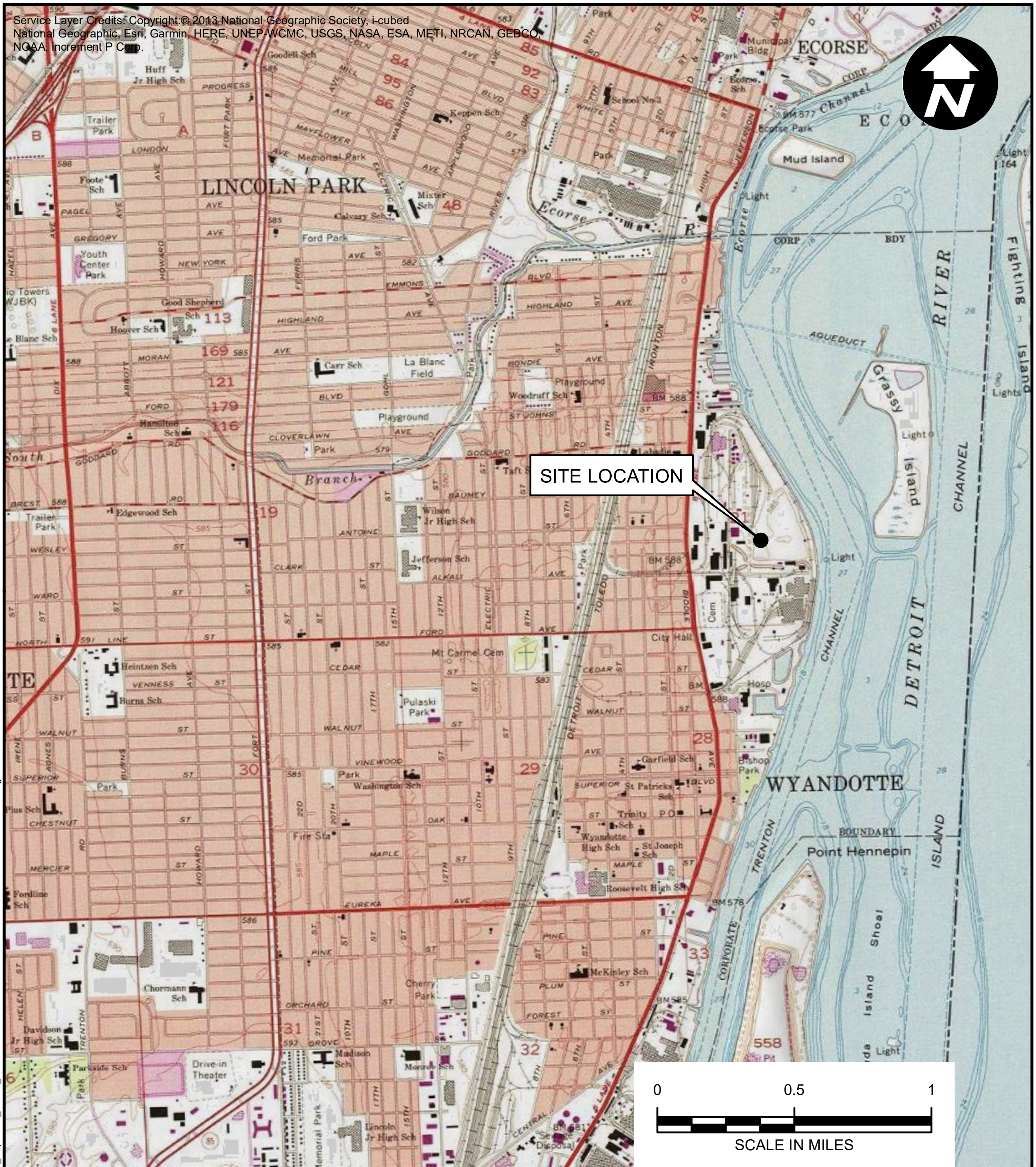
Figures



BASF NORTH WORKS
WYANDOTTE, MICHIGAN
BASIS OF DESIGN REPORT - PERIMETER BARRIER REMEDY

SITE LOCATION MAP

ARCADIS

FIGURE
1

CITY: NOVI, MI DIV: ENV DB: TRY PIC: J. Saling PM: G. Zellmer TM: A. Lorenz TR: T. Matsumoto PROJECT NUMBER: 30133323 COORDINATE SYSTEM: NAD 1983 StatePlane Michigan South FIPS 2113 Feet
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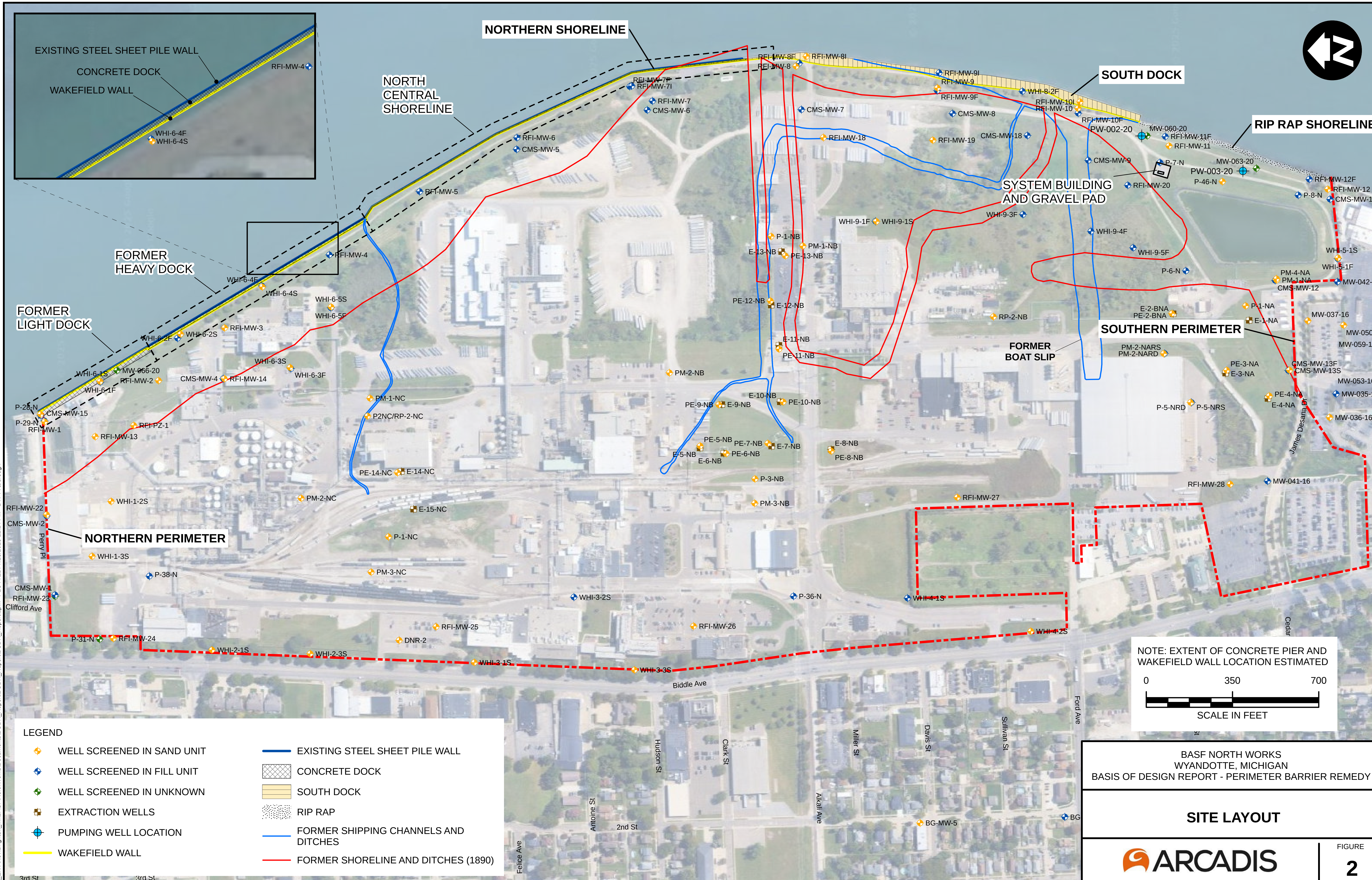

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NOTES:

1. BASE MAP PROJECTED TO NAD83 MICHIGAN S.P. SOUTH ZONE, FEET.
2. AERIAL IMAGE PROVIDED BY BING
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LEGEND:

- APPROXIMATE SITE BOUNDARY
- PROPOSED SUBSURFACE SHEET PILE WALL
- EXISTING STEEL SHEET PILE WALL
- WAKEFIELD WALL
- PROPOSED STEEL BULKHEAD
- PROPOSED CONVEYANCE PIPING
- PROPOSED COLLECTION DRAIN
- PROPOSED TREATMENT SYSTEM BUILDING
- PROPOSED SUMP LOCATIONS
- PROPOSED EXTRACTION WELL
- STILLING WELLS
- PROPOSED PIEZOMETERS (WITHIN THE COLLECTION DRAIN)
- PROPOSED PIEZOMETERS (RIVERSIDE OF SUBSURFACE BARRIERS)
- PROPOSED UPLAND PIEZOMETERS
- CAPTURE ZONE

0 400' 800'
GRAPHIC SCALE

BASF CORPORATION
WYANDOTTE, MICHIGAN
BASIS OF DESIGN REPORT - PERIMETER BARRIER REMEDY

PROPOSED BARRIER REMEDY LAYOUT WITH CAPTURE ZONE

ARCADIS

FIGURE
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