BASF Corporation

DRAFT
Basis of Design Report
Intermediate 60% Design
Perimeter Barrier Remedy

BASF North Works Site
Wyandotte, Michigan

March 2024

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Prepared By:
Arcadis of Michigan, LLC
28550 Cabot Drive, Suite 500
Novi
Michigan 48377
Phone: 248 994 2240

Prepared For:
BASF Corporation
1609 Biddle Avenue
Wyandotte
Michigan 48192

Our Ref:
30198804

DRAFT

Mandy Giampaolo, P.E.
Principal Engineer

DRAFT

Andrea Krevinghaus, P.E.
Principal Engineer

DRAFT

Jackie Saling, P.E.
Vice President/Project Manager

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Acronyms and Abbreviations

%	percent
AG	above grade
AOC	area of concern
Arcadis	Arcadis of Michigan, LLC
BASF	BASF Corporation
bgs	below ground surface
BOD	basis of design
CAO	corrective action objective
CAZ	critical assessment zone
CCR	Current Conditions Report
CM	construction manager
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
CQAP	Construction Quality Assurance Plan
CSM	conceptual site model
DUWA	Downriver Utility Wastewater Authority
EBCT	empty bed contact time
EGLE	Michigan Department of Environment, Great Lakes, and Energy
EMR	experience modification rate
EOR	engineer of record
ft IGLD 85	feet International Great Lakes Datum of 1985
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
IGLD	International Great Lakes Datum
JPA	Joint Permit Application
JSA	job safety analysis
LWCR	lost workday rate
MDNR	Michigan Department of Natural Resources
µg/L	micrograms per liter
mg/L	milligrams per liter
MI	Michigan
MIOSHA	Michigan Occupational Safety and Health Administration
N/A	not applicable
NAVD88	North American Vertical Datum of 1988
NFPA	National Fire Protection Association
ng/L	nanograms per liter
NPDES	National Pollutant Discharge Elimination System
NREPA	Natural Resources and Environmental Protection Act
O&M	operation and maintenance
OMM	operation, maintenance, and monitoring
OSHA	Occupational Safety and Health Administration
PDI	pre-design investigation
PFAS	per- and polyfluoroalkyl substances
PFOA	perfluorooctanoic acid
PFOS	perfluorooctanesulfonic acid
PLC	programmable logic controller
PM	project manager
POTW	publicly owned treatment works
PPE	personal protective equipment
ppt	parts per trillion
psf	pounds per square foot

psi	pounds per square inch
PZ	piezometer
QA	quality assurance
QAPP	Quality Assurance Project Plan
QC	quality control
QST	QST Environmental
RCRA	Resource Conservation and Recovery Act of 1976
RD	remedial design
RFI	Resource Conservation and Recovery Act Facility Investigation
RSSCT	rapid small-scale column test
S.U.	standard units
SCADA	supervisory control and data acquisition
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
TRIR	total recordable incident rate
USACE	United States Army Corps of Engineers
USC	United States Code
USEPA	United States Environmental Protection Agency
USFWS	United States Fish and Wildlife Service
UTC	Upper Trenton Channel
VE	value engineering
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 Intermediate (60%) Design of the remedy, which consists of a physical containment barrier and a groundwater extraction and above-grade (AG) treatment system to prevent offsite migration of groundwater at the downgradient site perimeter. The physical containment barrier will block groundwater from entering the Detroit River. The groundwater extraction and AG treatment 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 an Intermediate (60%) 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 currently 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 sheet pile wall was installed in the 1990s on the riverside of the existing pier structure/Wakefield wall described above (Figure 2). The sheet pile wall extends approximately 3,243 feet south from the northern site boundary (i.e., the northern end of the Site) 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 sheet pile wall 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 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 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 of COCs exceeding groundwater-surface water interface (GSI) criteria are present in upland groundwater monitoring wells installed along the downgradient site perimeter. 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 sheet pile wall, 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 sheet pile wall. 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 COC concentrations exceed GSI criteria in upland groundwater monitoring wells 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). The potential for groundwater discharge to the river exists 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 sheet pile wall is present, groundwater mounds behind the wall and is deflected to the north and south. A transducer study along the shoreline and aquifer tests at three high-permeability zones identified during the HPT investigation were completed. The results of the transducer study and aquifer tests confirmed a hydraulic no-flow boundary adjacent to the existing bulkhead, a recharge boundary adjacent to the South Dock/Wakefield wall, and high hydraulic connectivity to the Detroit River where rip rap is present at the shoreline.
- A Mann-Kendall stability analysis was performed for select perimeter wells for two compounds: mercury and perfluorooctanesulfonic acid (PFOS). Monitoring wells along the northern and southern boundaries exhibit stable to decreasing trends. Monitoring wells along the eastern shoreline primarily exhibit stable to decreasing trends with a few exceptions. Monitoring wells RFI-MW-5, RFI-MW-10, and CMS-MW-13S exhibit an increasing trend along the perimeter. Based on site conditions, increasing trends do not suggest an expanding plume, but rather reflect the range of concentrations and temporal variability. The perimeter barrier remedy has been designed based on the observed concentration ranges and will effectively treat impacted groundwater along the site perimeter.

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 barrier remedy:

- 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 at the approximate locations shown on Figure 3. Results of the PDI are summarized in the BASF North Works Geotechnical Data Report (Arcadis 2021a).
- 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 2021c). The groundwater model was further

updated during the Intermediate (60%) Design phase to include transient conditions known to be associated with the Detroit River (Appendix B).

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 treatment 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%) Design phase, several data gaps were identified as discussed in the Draft Basis of Design Report for the Preliminary 30% Design Perimeter Barrier Remedy (Arcadis 2022c). To address these data gaps, the following assessments were performed in 2023 in support of the Intermediate (60%) Design:

- A diver survey was conducted in August and September 2023 to evaluate the condition of the existing steel sheet pile bulkhead. Divers visually inspected the existing steel sheet pile bulkhead and collected ultrasonic thickness readings every 50 feet along the existing 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 included in Appendix C and have been incorporated into the evaluation of 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 sheet pile wall, and determine the depth of the basal clay along the alignments of the subsurface barriers and the bulkhead anchor wall. 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 D 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 included in Appendix E 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 F.
- 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 Intermediate (60%) Design Drawings (Appendix G).
- A resin pre-design study (field pilot test) is being conducted to evaluate ion exchange per- and polyfluoroalkyl substances (PFAS) ion exchange treatment. The field pilot test began in July 2023 and is ongoing. As of

February 2024, more than 2 million gallons of groundwater has been extracted, treated, and discharged in accordance with permit requirements to DUWA.

The need for a geotechnical assessment for the foundation design of the AG treatment system building and the need for additional evaluations of the soil-cement design mix to address potential freeze-thaw issues and compositional changes to Portland cement were also identified as potential data gaps during the Preliminary (30%) Design phase. However, based on the new proposed location of the AG treatment system, within an existing building, and the availability of historical geotechnical data for the area, it was determined an additional assessment was not needed for the Intermediate (60%) Design. Likewise, because the soil-cement slurry barrier wall type is no longer proposed, additional freeze-thaw evaluations of the soil-cement design mix are no longer needed; instead, all subsurface barriers are proposed as steel sheet pile.

3 Perimeter Barrier Remedy Basis

Between June 2015 and September 2017, BASF submitted various documents proposing remedy options to prevent potential offsite 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 offsite discharge.
- Site stabilization, a containment barrier, and onsite 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.
- Demolition of the dock and shoreline reconstruction are suggested as an optimal alternative for physical stabilization.

In 2018, BASF and USEPA agreed upon a perimeter barrier remedy, including groundwater extraction and onsite 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 is 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 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, 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 H.

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 4.1, 4.4, 4.5, and 6.3
Volume and type of each medium requiring treatment	Sections 2.2, 6.3, and 7 and Appendix I
Treatment schemes	Sections 6.4 and 6.5.3
Waste management and characterization requirements	Section 7
Operation and maintenance (O&M) requirements	Section 12

3.2 Proposed Remedy Description

After USEPA and BASF agreed upon a path forward, a Remedial Design (RD) Work Plan was submitted in May 2019 (Arcadis 2019a) and provided the framework for design of the perimeter barrier remedy. Based on current site conditions, the technical feasibility and cost benefit of a funnel and gate system were not apparent, and the funnel and gate option was not carried forward as part of the perimeter barrier remedy. A funnel and gate may be considered in the future if site conditions change. Consistent with the Preliminary (30%) Design, this Intermediate (60%) Design includes a physical downgradient perimeter containment barrier with a pump and treat system.

The remedy will include a combination of subsurface and shoreline barriers, along with a series of groundwater collection drains (supplemented with a vertical extraction well, as described in Section 5.1.2) 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 options considered and the basis for barrier selection are detailed in Section 4. The groundwater extraction 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 and finalized by USEPA in correspondence dated January 30, 2024. The finalized CAOs are presented in Table 2.

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 (Onsite)	N/A. The perimeter barrier remedy does not mitigate exposure to groundwater onsite. 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 onsite. 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. Will be addressed as needed by additional interim actions or a final remedy. Additional actions may be determined from RCRA Corrective Action risk assessments that evaluate the exposure of receptors to site-related contaminants.	N/A	N/A
Groundwater (Offsite)	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 offsite migration of groundwater.	Mitigate 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 offsite migration of groundwater.	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 (Onsite)	N/A. The perimeter barrier remedy does not mitigate exposure to onsite 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 onsite 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. Will be addressed as needed by additional interim actions or a final remedy. Additional actions may be determined from RCRA Corrective Action risk assessments that evaluate the exposure of receptors to site-related contaminants.	N/A	N/A
Soil/Sediment (Offsite)	N/A. The perimeter barrier remedy does not address offsite soils.	N/A. The perimeter barrier remedy does not address offsite 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	Mitigate 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 non-cancer values, MI Part 201 GSI, and MI Part 4.	Mitigate 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 non-cancer 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.	Cross media transfer will be addressed through the Human Health and Ecological CAOs.	N/A

Notes:
CAZ = critical assessment zone
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

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

3.4 Performance Standards

In consultation with USEPA and EGLE, BASF developed performance standards concurrently with design development. For the purposes of the Intermediate (60%) 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 prevent the potential for groundwater to enter the Detroit River. Performance standards will assign appropriate value to the presence of the physical containment barrier.
- The groundwater extraction 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), due to 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 while the Site is an operational chemical manufacturing facility. As such, the perimeter barrier remedy is intended to contain and prevent offsite 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 can be adequately accessed and addressed, performance standards associated with these future actions will include specific metrics for the shutdown of the perimeter barrier remedy, if appropriate.

An O&M plan outlining procedures for successfully operating the perimeter barrier system to meet CAOs and associated performance standards will be in place, as described in Section 12. Generally, the CAOs specify that the perimeter barrier remedy must prevent future offsite exposure to site soils and groundwater. This section details the performance standards to achieve these CAOs.

3.4.1 Soils/Sediment

The 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 a physical barrier along the entire eastern shoreline, consisting of new and existing steel sheet pile. The barriers are designed to structurally support site soils, and the design considers the site-specific conditions at the shoreline based on geotechnical, geophysical, and diver inspection data collected to support the Intermediate (60%) Design. The presence of the physical sheet pile barrier will prevent site soils from migrating off the Site and entering the Detroit River. An O&M Plan, as described in Section 12, will be in place that provides for routine inspections to identify and address any issues that could result in loss of site soil through the barrier 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 offsite 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 perimeter barrier remedy includes a physical barrier along the entire eastern shoreline, consisting of new and existing steel sheet pile. The barrier is designed to prevent groundwater discharge to the river.

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, when strong winds or atmospheric pressure changes push water from one end of a body of water to another (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 to develop standards that are technically practicable, 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 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 presence of the physical barrier will prevent the discharge of groundwater to the river and offsite.

The inward hydraulic gradient component of the performance standards, described in the following sections, were developed based on 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 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 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.

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

Table 3. Potential Performance Standard 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 feet	86%	0.06 feet	90%	0.06 feet	96%	0.06 feet
2-Week River Average	74%	0.16 feet	90%	0.16 feet	92%	0.16 feet	97%	0.16 feet
1-Week River Average	76%	0.25 feet	91%	0.25 feet	93%	0.25 feet	98%	0.25 feet
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 provides 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.

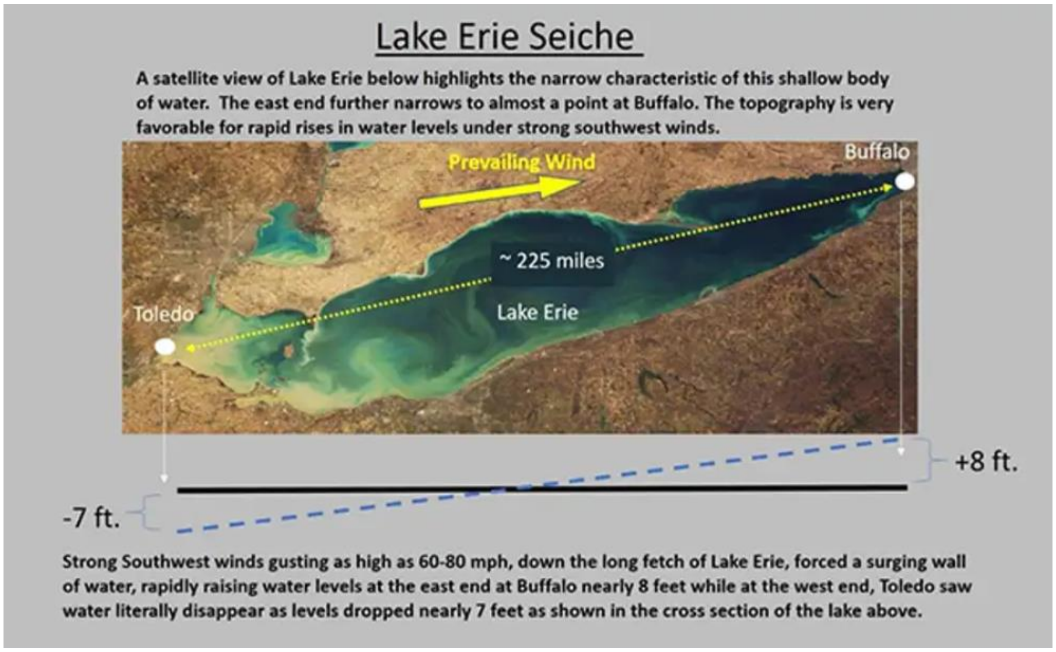
Presence of a Physical Barrier

The perimeter barrier remedy includes a physical barrier along the northern, eastern, and southern shorelines, consisting of new and existing steel sheet pile. The containment barrier has been designed to prevent groundwater discharge to the river and offsite. If there are brief time frames of outward gradients due to seiche events, the barrier will prevent the discharge of groundwater.

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.

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

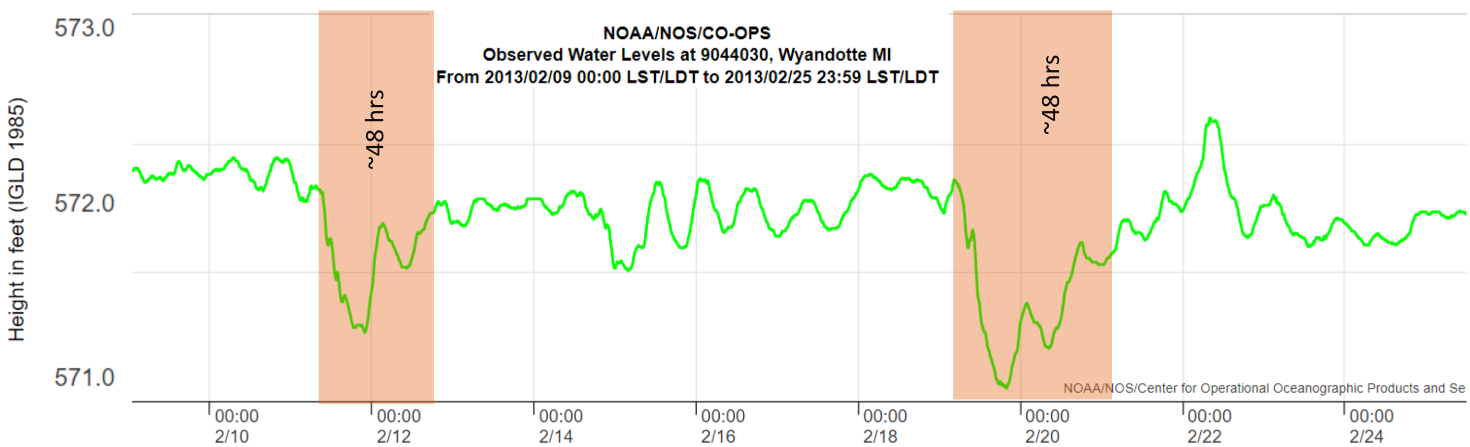
[2] https://tidesandcurrents.noaa.gov/waterlevels.html?id=9044030

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

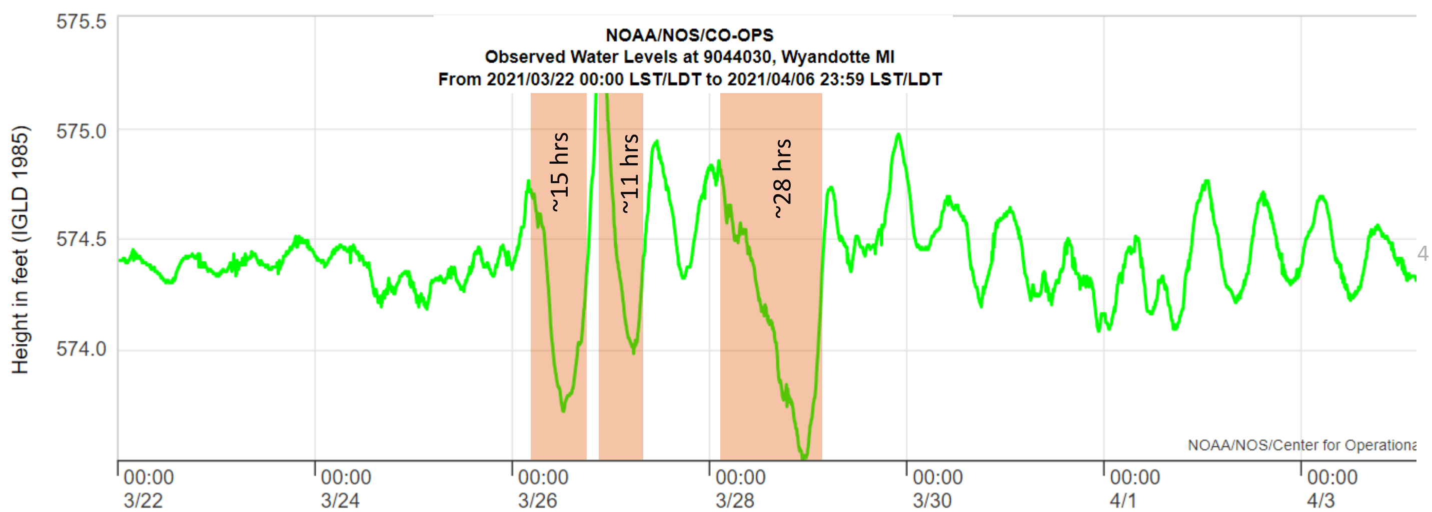
Data from seiche events on the Detroit River over the past 10 years in Wyandotte indicate that seiches generally last 12 to 48 hours and generally occur between October and April. Seiche elevations for two typical months are shown on Exhibits 2 and 3.

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



Note: IGLD = IGLD International Great Lakes Datum

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



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 past 10 years 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 past 10 years 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 past 10 years 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.

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 has 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 offsite 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.

The flow rate of the AG treatment system for the perimeter barrier remedy is 120 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 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 normal and high recharge conditions. Details of the groundwater modeling and how it was used to support the design are described in Section 5.1.1. Table 5 summarizes the average and maximum flow rates and the percentage of time the 120-gpm design flow rate was exceeded.

Table 5. Potential Compliance Scenarios

	0.5 foot Compliance Gradient Requirement					
	Normal Recharge		High Recharge		% of Time Above Design Flow Rate of 120 gpm	% of Time with Inward Gradient
	Average Flow Rate (gpm)	Maximum System Flow Rate (gpm)	Average Flow Rate (gpm)	Maximum System Flow Rate (gpm)		
1-Month River Average	37.6	93.5	56	111	0	96%
2-Week River Average	42.4	132.6	58	153	0.5 - 3%	97%
1-Week River Average	42.6	161	58	181	2.7 – 5.2%	98%
1-Day River Average	43.7	420	58.8	437	9.3 - 13%	98%

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

For the one month of averaging, the modeled expected flow rate does not exceed the design flow rate and maintains an approximate 10 to 33% contingency for extra capacity over the maximum flow rate modeled. The maximum model flow rate to meet the compliance gradient requirement for this scenario is less than two times the average flow rate and is approximately two times the maximum design flow rate of 120 gpm.

For the one-week and two-week averaging scenarios, the maximum flow rate exceeds the current design flow rate up to 5.2% of the time. Increasing the capacity of the system or incorporating influent storage capacity could be considered; however, doing so results in an increased cost and risk associated with permitting. Increasing capacity also increases the difference between the average flow rate and the maximum flow rate by 2.5 to 3 times, which creates operational risks. For these compliance scenarios, the operational and permitting risks increase, as do costs, for a relatively small gain in protectiveness (1 to 2%).

3.4.2.1.2 Performance Standards Recommended 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 prevent the discharge of groundwater to the river and offsite. 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 showed 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, for the purpose of the Intermediate (60%) Design, BASF is proposing to use one month of averaging river elevation and 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 the previous 10 years of Detroit River elevation data.

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 the Drain Compliance Elevation

3.4.2.2.1 Eastern Shoreline

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 will be averaged (i.e., rolling average) and 0.5 foot will be subtracted to calculate the drain compliance elevation.

Along the river, a minimum of one piezometer (PZ) will be installed within each drain for compliance monitoring. For drains with multiple sumps, the PZ will be situated between sumps. For drains with one sump, the PZ will be installed at one end of the drain. For drains longer than 500 feet, PZs will be placed at a maximum spacing of 500 feet between sumps. The bottom of the PZ screens will be set to an elevation of 568 feet International Great Lakes Datum of 1985 (ft 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 drain compliance elevation.

The layout of the proposed drain PZs and stilling wells is shown on Figure 4.

3.4.2.2.2 Northern and Southern Boundaries

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 on 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 of the northern barrier wall. The bottom of the PZ screens will be set to an elevation of 568 ft 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. The previous one month of groundwater elevation data will be averaged and 0.5 foot will be subtracted to calculate the drain compliance elevation.

Along the northern end, one PZ will be installed within the drain for compliance monitoring. The PZ will be situated between the two sumps. The bottom of the PZ screen will be set to an elevation of 568 ft IGLD 1985 to span the 30-year river low and be approximately 1 foot above the drain pipe. A second PZ will be placed in the northeast corner of the Site near a vertical extraction well that will be utilized as part of the extraction system where it was not feasible to install a drain. The bottom of this PZ screen also will be set to an elevation of 568 ft IGLD 85 to span the 30-year river low. During installation, efforts will be made to place the PZs in a location where the geology reflects groundwater conditions and where the PZs can measure the capture from the associated extraction well, avoiding screen placement in low-permeability soils. Once per day, the drain elevation will be logged at the PZs in the drain along northern boundary and near the vertical extraction well and compared to the calculated drain compliance elevation to confirm that the drain elevation is at or below the compliance elevation.

Two PZs will be installed riverward of the southern barrier wall. The bottom of the PZ screens will be set to an elevation of 568 ft 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.

Along the southern end, two PZs will be installed, one within each drain along the southern boundary for compliance monitoring. Each PZ will be at one end of its respective drain. The bottom of the PZ screens will be set to an elevation of 568 ft 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 southern boundary and compared to the calculated drain compliance elevation to confirm that the drain elevation is at or below the compliance elevation.

The layout of the proposed drain and groundwater PZs on the northern and southern ends of the Site is shown on Figure 4.

3.4.3 Project Phases and Objectives

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 operation, maintenance, and monitoring (OMM).

3.4.3.1 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 closing of the downgradient perimeter barrier to minimize 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 will include 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 Start-Up Period

The second phase, the start-up period, will begin after the completion of the containment barrier and the AG treatment system commissioning phase and will end a minimum of 18 months after the completion of the containment barrier. This phase could be longer than 18 months if additional supplemental infrastructure is needed to meet performance standards. 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 containment barrier in place and to optimize AG treatment system operations.

For the extraction system, 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 changeout schedules.
- Confirm equipment maintenance, repair, and/or replacement schedules.

Drain compliance elevation monitoring is required to meet the goals of this phase, but 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 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, the performance standard approach will be assessed to confirm that the proposed averaging time frame and compliance gradient requirements result in a drain compliance elevation that is achievable and protective. As discussed, there is flexibility in the inputs used to calculate the drain compliance elevations while still being protective. It may be appropriate to adjust the length of time used for river averaging and/or the compliance gradient requirement, as long as the appropriate levels of protectiveness can be achieved.

In addition, the need for supplemental infrastructure to meet performance objectives will be evaluated as part of the start-up phase. If the compliance drain elevation cannot be maintained in a portion of the site perimeter, potential supplemental infrastructure could include additional jet grouting, sumps on existing drains, vertical wells, horizontal wells, or new drains.

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 performance standard 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 jet grouting, sumps on existing drains, vertical wells, horizontal wells, or new drains), 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 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 three days per quarter operating out of the required gradient. If the gradient is reestablished within the allowable time frame (i.e., not exceeding three 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 O&M 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 will be 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

Requirement	Description	Applicability	Notes
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
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

Requirement	Description	Applicability	Notes
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 offsite 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 to construction of the perimeter barrier remedy include the following:

- The design of buildings will comply with the 2015 Michigan Building Code, as currently adopted by the City of Wyandotte.
- The design of electric utility services and electrically powered installations will comply with the following:
  - Utility standards designated by Wyandotte Municipal Services;
  - 2015 Michigan Electrical Code, as currently adopted by the City of Wyandotte; and
  - 2017 National Electrical Code, as currently adopted by the City of Wyandotte.
- The design of plumbing and mechanical installations will comply with the following:
  - 2015 Michigan Plumbing and Mechanical Code, as currently adopted by the City of Wyandotte.
- Extraction and conveyance equipment will be designed in accordance with applicable codes, standards, and regulations.

4 Perimeter Containment Barrier

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 815 feet of planned barrier along Perry Place to the property boundary);
- Northern shoreline (existing 3,243-foot-long sheet pile wall);
- South Dock (approximately 1,700 feet long);
- Rip rap area (approximately 850 feet of existing rip rap-covered riverbank between the South Dock and the southern site boundary); and
- Southern boundary (approximately 1,450 feet of proposed barrier along the northern edge of DeSana Drive).

These sections of the proposed barrier alignment are shown on Figure 2 and encompass the extent of concentrations exceeding EGLE generic GSI criteria in upland groundwater monitoring wells. The planned barrier network consists of approximately 4,815 linear feet of new barrier construction along the northern boundary, the South Dock, the rip rap area, and the southern boundary. The existing sheet pile wall along the northern shoreline will be incorporated into the barrier network. No additional barrier construction is planned for the northern shoreline section.

The following sections describe the barrier options (Section 4.1), provide the rationale for the selection of barrier types for each section of the perimeter alignment (Section 4.2), and present the Intermediate (60%) Design of the selected barriers (Section 4.3).

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%) Design BOD Report included descriptions of each subsurface barrier type and the evaluation criteria consisting of geologic considerations, constructability, overall effectiveness, and cost. The subsurface barrier types each included a groundwater extraction and AG treatment system installed on the upland side of the barrier. Appendix I 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 and a groundwater extraction and AG treatment system on the upland side of the barrier. The steel bulkhead wall would be installed along the shoreline, on the outboard side of the existing South Dock structure, to serve as a containment barrier. Interlock sealant would be applied between sheet piles to minimize the potential for seepage.

4.2 Development of Containment Barrier

This section describes which subsurface and shoreline barrier types evaluated were selected for each area of the site perimeter. Design Drawings showing the proposed location for each barrier type selected are included in Appendix G. The proposed barrier types for the containment barrier system in this Intermediate (60%) Design are summarized in Table 7.

Table 7. Barrier Remedy

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

The basis for selection of each barrier type by area was included in the Preliminary (30%) Design BOD Report. As previously discussed, further evaluation of the subsurface barrier options were completed as part of the Intermediate (60%) Design. Appendix I provides a basis for modifications to the selected barrier type for the southeastern portion of the network. These modifications are further discussed below as specific to the barrier remedy area listed in Table 7.

4.2.1 Rip Rap Shoreline

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 4, Appendix G and Appendix I). 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 Intermediate (60%) Design basis for this wall design and the construction measures that will be taken during implementation.

4.2.2 Southern Perimeter

The southern perimeter of the barrier network will consist of a continuation of the driven steel sheet pile wall for similar reasons as stated above for the rip rap shoreline portion of the alignment (Figure 4 and Appendix G). The geophysical findings have shown a lower potential for encountering obstructions through this zone and that this option will be less disruptive and have a smaller footprint during construction.

The steel sheet pile wall will wrap the southeastern corner of the Site and continue from the shoreline barrier to the west for approximately 550 feet and then continue westward along the southern perimeter of the Site for approximately 900 feet. Similar to the northern perimeter, the steel sheet pile wall type was selected for the southern perimeter because of the thinning fill layer and lower potential for obstructions. In addition, the southern perimeter alignment is adjacent to a public roadway, and this wall type offers the advantage of reduced disruption during installation compared to the other subsurface barrier options.

4.3 Barrier Design

This section summarizes the Intermediate (60%) Design for the subsurface and shoreline elements of the barrier network. Additional design details are provided on the Design Drawings (Appendix G), in the Design Calculations (Appendix J), and in the Technical Specifications (Appendix K). A list of final drawings to be included with the Pre-final (95%) Design is included in Appendix L.

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 J (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. An NZ-22 sheet pile was selected. This pile type is Z-shaped and widely available in the United States. Consistent with the recommendations in the Preliminary (30%) BOD Report, the interlocks for this pile shape are the Larssen type, and each joint will be sealed with a hydrophilic sealant.

The potential for section loss due to corrosion was evaluated, and the findings are included in Appendix J (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 50% of the pile's design thickness of 0.48 inch over an assumed 50-year design life is estimated. This estimated corrosion loss would not compromise the steel wall's ability to be an adequate barrier to groundwater flow.

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. A watertight connection to the existing bulkhead is required at its eastern end and will consist of overlapping jet grout columns at the intersection of the two sheet pile walls. The subsurface barrier wall length is approximately 815 feet.

The southern alignment of the proposed sheet pile wall 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.

As the wall extends toward the southern BASF property limit, 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 drain 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 sheet piling is not possible and incorporating another wall technology is required. As shown on the Design Drawings, overlapping jet grout columns beneath the power lines is proposed.

Both the northern and southern alignments will intersect existing subsurface public utilities. Approximate locations of the utility crossings are shown on the Design Drawings and are based on findings from the 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 could include requiring temporary bypass systems or incorporating another wall technology (e.g., jet grouting) to maintain the barrier properties. Conceptual details for addressing the utility crossings are shown on the Design Drawings and will be advanced as solutions are coordinated with the utility owners during the Pre-final (95%) Design phase.

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 2.5 feet around the utility. To provide a continuous barrier at these utility penetrations, the soil zone beneath the utility will be jet grouted. The zone above the utility will be backfilled with controlled low-strength material to within approximately 12 inches of finished grade. For those utilities that can be temporarily bypassed or capped, 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.

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. A supplemental geotechnical investigation was conducted in November 2023 and January 2024 to confirm the top of the clay layer and to collect subsurface data for installation of barrier components. The investigation consisted of SPT borings completed within Perry Place and along James DeSana Drive. Borings were advanced to depths between 20 and 30 feet below ground surface (bgs) along Perry Place and positioned at locations to supplement existing subsurface information along the proposed barrier alignment. SPT borings for the southern barrier alignment extended to depths between 24 and 46 feet bgs. Boring logs from this exploration program are included in Appendix F.

Based on findings from subsurface investigations along the alignments, the pile depths for the northern alignment of the sheet pile wall 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 alignment, the pile depths will range from approximately 17 to 57 feet below proposed grade. 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 from the pre-trenching activities will be staged and loaded for offsite disposal. Suitable materials will be used to backfill the trenches prior to sheet pile installation.

Unlike for other barrier wall types considered, visual confirmation of embedment into the clay layer for a sheet pile wall is not possible. Although multiple subsurface investigations have been completed in the area of proposed barriers, additional investigations along the alignment would provide for greater certainty of the clay profile and allow for greater efficiency in the required pile lengths.

In areas of the alignment where the clay profile is variable, the contractor will be required to conduct a pre-drilling program to refine the clay surface and provide additional information on potential for encountering subsurface debris during installation. It is anticipated that the pre-drilling program will consist of soil borings spaced at 25-foot intervals, advanced to the top of the clay layer, with visual confirmation of the clay surface via sampling. Limits of the pre-drilling program will be provided in the Pre-final (95%) Design for the barrier remedy.

4.3.2 Steel Bulkhead Design at South Dock

The steel bulkhead 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, the rationale for the wall type selection, and discussions of construction methods, constructability, and sequencing of the steel sheet pile bulkhead with an anchor wall at the South Dock. Alternative design elements for the bulkhead that remain applicable to the bulkhead design are also discussed.

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 (3 horizontal units to 1 vertical unit) 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.

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 (Arcadis 2021a). 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 at the approximate locations shown on Figure 3. 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 (Figure 3). 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 soils predominantly consist 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 514 feet and 519 feet North American Vertical Datum of 1988 (NAVD88). Based on the borings drilled in 2020, the top-of-bedrock elevation varies between approximately 509 feet and 516 feet NAVD88.

Appendix J includes a summary of the 2020 soil data collected for the bulkhead design. 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 Intermediate (60%) Design. General descriptions of the soil conditions found during the 2020 investigation program follow:

- 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 below ground surface 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 to 43.5 feet in the upland area and from approximately 15 to 25 feet in the river.

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 pits are included in Appendix E and have been incorporated into the design elements for the new bulkhead as discussed in this section.

4.3.2.2 Bulkhead Wall Intermediate (60%) Design

This section provides descriptions of the bulkhead structural components, the design criteria and assumptions, and the results of calculations performed for the Intermediate (60%) Design of the steel bulkhead wall.

4.3.2.2.1 Bulkhead Structural Components

The steel bulkhead structure will consist of an anchored steel sheet pile wall. Cross sections through the wall system are provided in 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 Intermediate (60%) 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 in the Design Drawings, the tie rods will be installed at regular intervals, perpendicular to the headwall and anchor wall alignments (i.e., approximately perpendicular to the river). As part of the Intermediate (60%) Design, the anchor wall type and alignment have been reviewed, and adjustments to the anchor wall have been incorporated in consideration of potential subsurface obstructions and as needed to facilitate groundwater recovery in the collection drain. These adjustments include installation of an A-frame anchor wall (i.e., continuous grade beam with H-piles) to facilitate groundwater recovery behind the anchor wall and positioning of the anchor wall outside the zones of potential obstructions as identified from the geophysical surveys. Several batter piles are typically tied together near the ground surface by a reinforced concrete block or continuous beam. The tie rods from the bulkhead wall are then connected to the concrete anchor cap/beam. An advantage of an A-frame over a sheet pile anchor wall is that it can be easier to install H-piles through debris than to install sheet pile in the same material. However, pre-drilling some of the piles may be needed should heavy resistance or obstructions be encountered during pile driving.

Walers will be used to transfer loads from the tie rods to the anchor wall and will consist of two parallel steel channel sections installed near the top of the headwall. Details of the pile connections and walers are shown in 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. To achieve this driven depth, the design includes the requirement of driving shoes on the tips of each sheet pile.

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 will be designed to form a low-permeability containment barrier to prevent the flow of groundwater toward the Detroit River. The wall will be designed to withstand structural loads, including lateral earth pressures, hydrostatic pressures, and surcharge loads. In addition to loads developed from lateral earth pressures, other loads considered for the Intermediate (60%) 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 were not considered in the design):

- Hydrostatic Loads: The groundwater extraction system was assumed to maintain an inward groundwater gradient along the bulkhead. In consideration of the proposed compliance gradient requirement of 0.5 foot, it was therefore 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.
- 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. An added surcharge of 90 psf was included in the static and seismic cases 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 1033-year event with a uniform live surcharge load of 100 psf, under post-dredge grades, were developed.

An allowable lateral deflection of 4 inches has been assumed for the headwall and is based on criteria developed for other projects of similar bulkhead height and conditions and barrier requirements. 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.

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 1994). To advance the bulkhead design from the Preliminary (30%) Design phase, a combination of methods and software programs was used. 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. Implementing Plaxis 2D into the Intermediate (60%) Design phase resulted in a more rigorous evaluation using a non-linear approach to soil modeling and considered the effects of soil-structure interaction and staged construction on wall deflections and ground deformations.

Further details on the analysis methodology and assumptions are provided with the bulkhead wall calculations in Appendix J. The assumptions in Appendix J include the following:

- Design cross sections showing the assumed geometry and soil and bedrock stratigraphy;
- Construction sequencing;
- Factors of safety;
- Design loads, including hydrostatic, surcharge, and seismic; and
- Design soil parameters.

4.3.2.2.3 Results of the Intermediate (60%) Design Calculations

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

- Structural demand on the headwall;

- Wall type: regular sheet pile wall or combination wall (i.e., king piles and intermediate sheet pile or similar system);
- Required headwall pile embedment;
- Required anchor load; and
- Anchor wall configuration, including required tie rod diameter/spacing and H-pile structural section/embedment.

The minimum structural requirements for the Intermediate (60%) Design are summarized in Table 8.

Table 8. Summary of Minimum Bulkhead Structural Requirements for Intermediate (60%) 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)	85.7
	Top-of-Wall Elevation (feet NAVD88)	578.0
	Sheet Pile Length (feet)	Approximately 62 to 69 (driven to bedrock; rock shoes required)
	Wale	Continuous double W14×48 or W12×40
	Protective Coating	Epoxy
Anchor Wall	Wall Type	Continuous concrete anchor cap (A-frame with H-piles)
	Anchor Cap	Reinforced concrete, 6 feet wide by 5.7 feet deep
	Top-of-Anchor Wall Elevation (feet NAVD88)	577
	Pile Section and Length (feet)	HP 12×74, ±65 (driven to bedrock)
Steel Tie Rods	Steel Grade	Grade 150 (ASTM A 311 and ASTM A 722)
	Design Capacity (kilo pounds [kips])	288
	Tie Rod Diameter (inches)	1¾ or 2¼
	Tie Rod Length (feet)	Approximately 62 (from face of headwall through anchor wall)
	Protective Coating	Epoxy

Components of the new bulkhead will continue to be evaluated and refined as appropriate for advancing the design to the next phase. Future evaluations will include a load case specific to seiche events, a lateral pile analysis of the anchor wall, and various structural analyses required for completing the anchor system design. Tasks specific to the anchor system include design of the connections and support beams for the double waler system, finalization of bearing plate sizes, and assessment of the overall bending of the concrete cap to determine reinforcement requirements.

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 (CQAP), included in Appendix M, 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. Based on the stiffness of the glacial till, a vibratory hammer is anticipated to be able to drive the sheet piles to the top of bedrock. However, an impact hammer may be used to drive the piles through the glacial till to the top of the bedrock, if necessary. Rock shoes welded to the sheet pile tips will be required to penetrate through the glacial till layer and provide a sound connection to the bedrock.

Difficult conditions may be encountered during H-pile driving for the anchor wall in the upland fill soils, which contain various types of debris, including concrete debris, slabs, and former foundations. 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.

Although the bulkhead wall can be installed from the riverside using deck barges, it may also be possible to position a crane on the upland side for pile installation. Because of the condition of the South Dock, it may not be possible to position the crane on the dock. Instead, the crane would likely be positioned on the upland side of the concrete deck. Based on a maximum deck width of approximately 34 feet, the crane should be within reach of the headwall alignment at that distance.

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 A-frame anchor wall. Details of the wale system and A-frame anchor wall are included in the Design Drawings (Appendix G). Waler installation on the headwall will involve some welding and bolting. At the A-frame anchor wall, the H-piles will be embedded into a concrete grade beam.

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.

The tie rod headwall connection will be near the top of the headwall (approximate elevation of 576 feet NAVD88) 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 concrete cap at approximately 12 inches below the top.

4.3.2.3.3 Backfill Placement

Because the concrete deck of the existing South Dock structure would 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 need to 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. Backfilling behind the new bulkhead will also fill the void behind and on the southern end of the existing bulkhead directly adjacent to the South Dock structure. The contractor will be required to confirm the backfill level below the dock through volume calculations and survey means.

Placement of fill will likely induce consolidation settlement in the underlying clay unit. As a result, the fill surface will move away from the concrete deck above over time. Therefore, filling the gap completely during initial backfilling will not be beneficial; the concrete deck will still be supported by the timber piles. Settlement monitoring of the fill surface will be performed to assess the progress of consolidation in the clay unit and to minimize strain on the tie rods due to settlement of the fill.

Once consolidation settlement is complete or nearly complete, which could take several years, the gap between the fill and the concrete will be closed by pumping flowable, cementitious grout into the gap. To reduce the amount of grout that might enter the void space of the backfill material, the backfill material has been selected such that the fill is progressively finer (i.e., relatively coarse aggregate near the bottom and finer materials such as mixture of gravel and sand near the top of the fill). Details of the backfill materials and placement requirements will continue to be refined in the next design phase.

While it may not be possible to completely close the gap between the fill surface and the concrete, it may not be necessary to do so. Over time, 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 concern because the new bulkhead structure will be 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 Intermediate (60%) Design to address these features. The top of the existing concrete kneewall along the face of the South Dock varies in elevation from approximately 576 to 578 feet. Installation of new tie rods for the bulkhead will require 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 for maintaining the anchoring for the existing Wakefield wall during construction has been considered for this Intermediate (60%) Design. From review of upland subsurface conditions and from the findings of the geophysical surveys, the sheet pile anchor wall in the Preliminary (30%) Design has been changed to an A-frame anchor wall. This change allows for locating the A-frame structure closer to the headwall; however, the structure will require positioning 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 will need to be maintained during construction of the new anchored bulkhead structure until the gap behind the bulkhead is backfilled sufficiently to provide adequate support. Evaluations completed for the Intermediate (60%) Design considered an interim backfill level of 570 feet NAVD88 before installation of the anchor cap and removal of existing anchorage components of the Wakefield wall. The level of backfill considered sufficient will continue to be refined as the bulkhead design advances into the Pre-final (95%) Design phase.

4.3.2.4 Alternative Design Elements and Construction Methods

The anchored wall design presented in this Intermediate (60%) BOD Report is feasible and provides a structural solution to the currently assumed loading conditions. Certain aspects of the design will continue to be evaluated during subsequent phases of the design, including technologies to improve constructability and approaches to lower the structural demand on the wall system. Some of the alternative technologies discussed in the Preliminary (30%) BOD Report have been removed from consideration because they are no longer relevant to the current solution presented for the Intermediate (60%) Design. The alternative technologies and approaches applicable to the Intermediate (60%) Design are described below. Other alternatives may also be evaluated as the design advances into the Pre-final (95%) Design phase.

4.3.2.4.1 A-Frame Combination Batter Pile and Grouted Tieback Anchor Cap

An alternative to the sheet pile anchor wall presented in the Preliminary (30%) BOD Report was an A-frame batter pile structure. As previously discussed in this Intermediate (60%) BOD Report, an A-frame batter pile concrete cap has been designed as the anchor system for the bulkhead at the South Dock. The batter piles consist of steel H-piles driven to refusal in the bedrock and driven at an angle (at a batter) that increases lateral resistance of the structure. One variation of this A-frame type structure is to replace the tension piles with grouted anchors. The benefits or issues of implementing this variation of the A-frame structure will be evaluated further in the Pre-final (95%) Design phase.

4.3.2.4.2 Lightweight Fill as Backfill Behind the Bulkhead Wall

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 H-piles of the A-frame anchor structure. There are lightweight aggregates and lightweight controlled low-strength materials (“flowable fills”) that may be suitable for filling behind the bulkhead wall, under water. The benefits or issues of using lightweight fill as backfill behind the headwall will be further evaluated in the Pre-final (95%) Design phase.

4.3.2.4.3 Buttressing to Increase Passive Resistance

If the sediment removal in front of the headwall is sequenced such that the loading conditions behind the wall are lower than currently assumed (i.e., dredging occurs prior to placement of fill below the concrete deck), placing a sand and gravel buttress in front of the wall is expected to reduce the overall structural demand and the required wall embedment depth. Buttressing in combination with using lightweight fill could potentially reduce the required wall embedment depth and decrease the number or size of the structural elements of the anchor cap (i.e., reduce number of H-piles and size of concrete cap). However, the finished grade in front of the wall will be higher from placement of the buttress material and the localized stability of the buttress would need further review.

4.4 Existing Sheet Pile Wall

The existing 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 when 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 2021c). The groundwater model is discussed further in Section 5.1.1.

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 sheet pile wall 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. Results are summarized in Appendix C, which includes a figure of photograph locations, a photograph log, results of pile thickness measurements, and the diver inspection field notes.

4.4.1 Conditions for 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. Also referenced as the Wainright Wall, the approximate overall alignment of the historical seawall along the existing bulkhead is 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.

4.4.1.1 Former Light Dock Section

4.4.1.1.1 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 sheet pile wall. 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 sheet pile bulkhead and former dock features.

4.4.1.1.2 Findings from Visual Inspections and Diver Survey

Results of visual inspection of the existing bulkhead indicated no signs of rotation and that the sheet piles are in good overall condition. Site observations also indicate 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 attributed to loss of material through open pile lift holes that were not completely sealed with metal plates during construction of the sheet pile 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 24-inch-diameter pipe opening for 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 from 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 C, 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. 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.

4.4.1.2 Former Heavy Dock Section

4.4.1.2.1 Current and Historical Construction Features

Continuing 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 sheet pile wall 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). From 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 was 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 is 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 dock structure (i.e., between the existing steel sheet pile bulkhead and the timber sheet pile wall).

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 sheet pile bulkhead and former dock features.

4.4.1.2.2 Findings from Visual Inspections and Diver Survey

Visual inspections of the existing bulkhead through the former Heavy Dock section showed that the wall alignment is in good condition, as are the visible portions of the sheet pile wall 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 sheet pile 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.

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. 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 depth of impacted area (i.e., above or below the river water surface).

Considering the level of soil loss and erosion behind this section of the existing bulkhead, the design will incorporate placement of fill to the original design grade and measures for managing stormwater runoff along the existing bulkhead. As discussed above, the area beneath the former Heavy Dock structure may not have been filled during construction of the existing sheet pile bulkhead. The level of soil loss in this portion of the existing bulkhead may be a combination of soil loss through lift hole openings and loss of soil vertically into either open voids beneath the concrete dock structure or gaps in the backfill material. Review of previous soil borings completed inland of the timber sheet pile wall indicates either granular material extending from grade or, when encountering concrete at depth, granular material found beneath the concrete to exploration depth.

4.4.1.3 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.

4.4.1.3.1 Current and Historical Construction Features

The bulkhead along the North Central Shoreline is an anchored steel sheet pile wall 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). From 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 sheet pile bulkhead and former seawall features.

4.4.1.3.2 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 sheet pile wall 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 in the survey because a steel plate covered the opening to divert the outfall water toward the river’s mudline. However, based on historical records of the 003 outfall, the diameter is estimated to be 24 inches. 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. However, no leakage through the sheet pile interlocks was noted.

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 C). 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. Repairs through this section generally include placement of metal plates at the lift holes and other noted openings through the sheet piles. The 003 outfall will be further inspected during construction to determine if means are needed for sealing between the pipe penetration and the surrounding sheet piles. 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.

4.4.1.4 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 for 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 wall where repairs are needed are indicated on the Design Drawings (Appendix G). Details for the specific repair measures will be developed and included as part of the Pre-final (95%) Design phase.

Arcadis performed a desktop evaluation of the estimated corrosion rates for the existing bulkhead in support of this 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 estimated above. Review of the data in Appendix C indicates that the average corrosion loss rate over the last 30 years is approximately 8%. The maximum corrosion loss rate, based on the two data points with steel thickness results less than 0.325 inch, is approximately 17%.

From a containment barrier remedy aspect, the anticipated corrosion losses would not compromise the steel wall's ability to act as an adequate barrier to groundwater flow. 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 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 would 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 Section 12.

Findings from review of the historical drawings for the former Heavy Dock section of the existing bulkhead do not clearly indicate that fill is present or was placed beneath the concrete seawall during construction of the current sheet pile wall. This is a concern for the long-term stability of the wall should portions of the historical dock collapse with time. However, based on the existing grade in this area, it is clear that fill has been placed between the historical wooden bulkhead and current sheet pile alignment. Current grades along the existing bulkhead are level with the sheet pile cap, with the exception of noted eroded areas, and extend upland over the historical seawall limits.

Although the historical drawings do not indicate placement of fill below the seawall, there is the potential that backfill materials may have flowed into the pile-supported zone beneath the concrete seawall or materials may have migrated with time into this area. Observations from site inspections of the deeper eroded areas may also indicate that these areas may have developed in part from vertical migration of soils into open voids at depth. At this point, it is not certain whether or not voids are present beneath the concrete seawall. However, given the concern for potential collapse of this structure should voids be present, additional investigations are proposed to collect data regarding the subsurface conditions beneath the former Heavy Dock structures. Investigations for addressing the data gaps are discussed in Section 10.

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 will be designed so that the intersections of the barrier types are properly overlapped or sealed and provide 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, it is proposed to seal the two walls through a series of jet grout columns extending from grade to a 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 and seal these two walls through a series of jet grout columns extending into the clay layer. At the dock’s southern end, the headwall piles will be connected via a sealed interlock to the subsurface sheet pile barrier wall.

5 Groundwater Extraction and Conveyance System

The BOD for the groundwater extraction and conveyance system is described in this section. The layout of the conveyance system is shown on Figure 4. 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 Intermediate (60%) Design and the resulting groundwater extraction system design basis.

5.1.1 Summary of Groundwater Modeling Results

To support the Intermediate (60%) Design of the barrier remedy, the numerical groundwater flow model of the localized groundwater flow system at the Site (Waterloo Hydrogeologic 2002 and Arcadis 2021b) was updated. The groundwater model was updated to reflect the Intermediate (60%) Design of the barrier remedy and transient conditions known to exist in the Detroit River. Model updates included boundary conditions, hydraulic conductivity values, and applied recharge. The drain alignments were adjusted to avoid subsurface obstructions as identified by the geophysical surveys and historical records review. The objective of the effort to update 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 Intermediate (60%) 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 77 groundwater level measurements collected in March 2019 from locations distributed throughout the Site. Validation of the calibrated groundwater model was accomplished by incorporating aquifer storage into the model and simulating the observed groundwater elevation measurements collected at 82 locations between March 2019 and June 2021. 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.

The calibrated steady-state model was used to simulate the perimeter barrier remedy along the northern, eastern, and southern property boundaries. The overall modeled steady-state rate for the extraction system is 31.8 gpm using an average recharge rate of 3.5 inches per year under a 0.5-foot inward gradient from the drain to the Detroit River. The results show that under the 0.5-foot gradient conditions, the groundwater along the perimeter boundary is captured by the extraction system to the top of the lacustrine clay unit. 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 extraction system.

The model was run under transient conditions for a full year (November 1, 2021 to October 31, 2022) to confirm the system can respond to both winter (when the river elevation is low) and summer (when the river elevation is high) conditions, and therefore can accommodate seasonal variability in the Detroit River. Two recharge conditions were evaluated: steady-state calibration (normal) conditions (overall model recharge of 3.5 inches per year, corresponding to a recharge multiplier of 1.0) and high recharge conditions (overall model recharge of 7 inches per year, corresponding to a recharge multiplier of 2.0). The model was run using the proposed performance standard approach of one month of average Detroit River elevations and a compliance gradient requirement of 0.5 foot. The maximum total groundwater extraction rate under normal recharge conditions was 93.5 gpm. The overall system average over the 12-month period was approximately 37.6 gpm. The maximum total groundwater extraction rate under high recharge conditions was approximately 111 gpm. The overall system average over the 12-month period was approximately 56.4 gpm. Because the extraction system will likely be operated at a gradient greater than the drain compliance elevation to ensure compliance, the model was used to evaluate flow rates at a 1-foot gradient. At a 1-foot gradient, the average and maximum flow rates assuming high recharge conditions are 61.3 and 114 gpm, respectively.

The Intermediate (60%) Design for the groundwater extraction system assumes an average extraction flow rate based on the 0.5-foot compliance gradient requirement under normal recharge conditions and a maximum flow rate based on the 1-foot gradient under high recharge conditions. Both the average and maximum flow rates were determined assuming averaging one month of river elevation data as proposed for the performance standard approach.

5.1.2 Groundwater Extraction System

A groundwater extraction system is proposed to capture groundwater behind the proposed perimeter barrier. The system will consist of 10 collection drains and one extraction well capturing groundwater on the northern, eastern, and southern boundaries of the Site. The drain alignments and location of the extraction well are presented in the Intermediate (60%) Design Drawings (Appendix G).

Vertical and horizontal wells were also evaluated for groundwater collection upstream of the perimeter containment barrier. 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 O&M 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 predictably model across the entire perimeter of the Site. The use of vertical wells is reserved for localized areas where drains cannot be feasibly installed and/or to supplement drains, if needed.

Drain lengths and locations required to hydraulically capture groundwater were established based on groundwater model simulations (Appendix B). Drains are generally located in zones of high permeability and aligned to avoid subsurface obstructions where possible. Design parameters considered during evaluation of the drains included feasibility of installation, ease of operation, and long-term OMM requirements. Collection drain design parameters are presented in Table 9A using the 0.5-foot compliance gradient requirement to estimate the average flow rates under normal recharge conditions and a 1-foot gradient to estimate maximum flow rates under high recharge conditions. Both the average and maximum flow rates were determined assuming averaging of one month of river elevation data as proposed for the performance standard approach.

Table 9A. Proposed Drain Lengths and Sump Extraction Rates

Drain ID	Boundary Location	Length (feet)	Sump	Average Sump Rate (gpm)	Maximum Sump Rate (gpm)
1	North	400	1	1.5	4.7
			2	1.9	5.3
2	East	400	3	1.9	5.7
			4	1.8	5.3
3	East	150	5	2.4	5.4
4	East	530	6	2.3	6.1
			7	3.1	7.6
5	East	340	8	1.0	3.5
			9	0.9	3.4

Drain ID	Boundary Location	Length (feet)	Sump	Average Sump Rate (gpm)	Maximum Sump Rate (gpm)
6	East	2025	10	1.6	5.2
			11	5.5	14.2
			12	1.4	4.7
			13	1.2	4.6
			14	1.9	5.7
			15	2.4	6.5
7	East	95	16	2.3	6.0
8	Southeast	875	17	1.4	5.5
			18	1.1	5.1
			19	1.0	4.5
9	South	95	20	0.7	1.8
10	South	230	21	2.5	6.2
Total	NA	5,140	NA	39.8	117

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 37.6 gpm and 114 gpm, respectively.

The drains will consist of perforated pipe installed at a targeted depth interval of approximately 567 ft IGLD 85, which is approximately 2.5 feet below the instantaneous near 30-year historical river low-elevation level of 569.5 feet observed in 1995.[4] Drainage trenches will be backfilled with a high-permeability material (i.e., stone) to promote drainage. Cleanout risers will be provided at both ends of each drain to facilitate maintenance.

To optimize constructability while new drains are being installed 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 (Appendix D) and site maps, the existing utility locations were identified, allowing for strategic design adjustments. The Intermediate (60%) 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. In addition, the conveyance piping path will be diverted around concrete foundations to minimize disturbance or the need for costly removals. This approach aims to streamline construction by reducing conflicts and facilitating a more straightforward installation process, ultimately enhancing efficiency and minimizing disruptions to existing infrastructure. Nevertheless, traditional excavation of trenches may require dewatering, use of trench boxes in narrow pathways, and/or concrete removal, depending on construction methodology.

Based on the simulations run with the groundwater model, it was determined that to meet the proposed performance standards, one extraction well would be needed to supplement the drains. One vertical well will be installed in the northeast corner of the Site, between Drains 1 and 2. This area of the Site has an abundance of subsurface utilities, including tiebacks for the existing bulkhead wall, making installation of a collection drain

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

impractical. Two spare extraction well conveyance lines will be installed on Sump 12 near Drain 6 as a contingency. The well will be installed to a total depth of approximately 10 to 15 feet bgs and screened within the shallow fill. Vertical well design parameters are presented in Table 9B.

Table 9B. Proposed Vertical Well Location and Flow Rate

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

The Intermediate (60%) Design for the groundwater extraction system therefore assumes an average extraction flow rate of 42.3 gpm and a maximum extraction flow rate of 119.5 gpm form the 10 collection drains and one extraction well. The maximum flow rate was rounded to 120 gpm for the purpose of the Intermediate (60%) Design.

A series of PZs will be installed within the drains to monitor groundwater levels and support the evaluation of drainage efficiency. These PZs will be equipped with level transducers, which will control the speed of the sump pumps.

5.2 Conveyance Network

The following sections describe the Intermediate (60%) Design for the sump network and conveyance piping.

5.2.1 Sump Network

Groundwater captured in the drains and extraction well will flow to the adjacent network of sumps. The sump locations are presented on the Intermediate (60%) Design Drawings (Appendix G). Twenty-one sumps will be installed at intervals of up to approximately 350 feet along each segment of the collection drainage network. Sump spacing was determined based on simulations with the groundwater model and maintenance considerations. Sumps will be constructed of perforated high-density polyethylene (HDPE) to allow groundwater from the stone drainage layer of the drains to enter the sump. The sumps will extend to a depth of approximately 5 feet below the drain invert elevation to provide for collection and storage of accumulated solids.

Each sump will be equipped with a primary pump and a redundant pump, both with variable frequency drives to allow for automated flow rate adjustments, increased operational flexibility, and decreased wear on the pumps. 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 monitored by nearby operational PZ(s) as discussed in Section 12.2. Each pump will be sized to accommodate both the average and maximum flow rate for the sump based on the estimated drain flow rates from the groundwater model results. Flow control valves and flow meters will be installed in a valve vault adjacent to each sump to facilitate access by the system operator.

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 a header pipe that runs to the AG treatment system as depicted on the Intermediate (60%) Design Drawings (Appendix G). 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), a network of redundant lateral piping from the sumps to a redundant header pipe will also be installed. The conveyance network will be constructed using buried HDPE and above-grade carbon steel (exterior) and polyvinyl chloride (interior) piping. Pipe sizing was determined using total dynamic head calculations, which account for static head and head loss or frictional loss (e.g., pipe lengths, valves and fittings, instrumentation) for the extraction pumps and conveyance piping within the conveyance network. The total dynamic head calculations are included in Appendix J.

To mitigate freezing risk, subgrade conveyance piping will be buried below the frost line or insulated as needed, and above-grade piping will include heat or steam tracing and insulation. Pipe fouling within the conveyance network will be mitigated by installing cleanouts at intervals of approximately 500 feet. The piping will be cleaned as part of scheduled routine maintenance activities or when increasing pressure or decreased flow is observed in the conveyance network.

6 Above-Grade Treatment System

The BOD for the AG treatment system is described in this section. Piping and instrumentation diagrams and layouts of the treatment building are included in the Intermediate (60%) Design Drawings (Appendix G).

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 main components:

- 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 granular-activated carbon (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, further investigation of potential ion exchange interference compounds was identified as a data gap to be evaluated. Based on the aforementioned treatability study, interference compounds in the process water treated by the resin are expected to be one or two orders of magnitude greater than vendor-recommended levels. To address this issue, a resin pre-design study consisting of a field-scale pilot test system simulating the GAC and ion exchange portion of the AG treatment system was implemented to support the design. Pilot system start-up began on July 5, 2023, and discharge to the local POTW began on July 6, 2023.

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

Groundwater is pumped from the pumping wells to an influent holding 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. 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. Additional resin vessels were installed to achieve compliance with discharge permit requirements while allowing full breakthrough to occur through the lead resin vessel.

Concentration ranges for select compounds observed between July and November 2023 are shown in Table 10. PFOS, perfluorooctanoic acid (PFOA), and TOC concentration ranges are associated with influent sampling locations before and after the bag filter, while chloride, sulfate, and alkalinity are post-GAC concentrations ranges prior to the resin vessels.

Table 10. Resin Pre-Design Study Pilot Test Concentration Ranges

Constituent	Unit	Minimum	Maximum
Perfluorooctanesulfonic acid (PFOS)	ppt	92	180
Perfluorooctanoic acid (PFOA)	ppt	5.0	12
Total Organic Carbon (TOC)	mg/L	3.8	30
Chloride	mg/L	270	690
Sulfate	mg/L	280	320
Alkalinity	mg/L	150	270

Notes:
mg/L = milligrams per liter
ppt = parts per trillion

GAC discharge TOC concentrations during the same time frame have ranged from 0 to 4.3 mg/L, resulting in minimal TOC loading to the resin. Except for one instance where the chloride concentration exhibited a 60-mg/L reduction, concentration reductions for all analyzed potential interference substances across the ion exchange resin vessel have been minimal, with reductions ranging from 0 to 10 mg/L. However, although resin influent loading concentrations for chloride and sulfate have been one order of magnitude less than full-scale design estimates, they are above vendor recommendations overall. Concentrations for these compounds will continue to be monitored to support the Pre-final (95%) Design. Estimated media alkalinity loading concentrations for the AG treatment system are less than current pilot test concentrations; therefore, the pilot test is representative of a worst-case scenario.

Continuous PFOS loading to the first resin vessel has not been observed as of February 2024; therefore, resin usage rates accounting for the site-specific interference compounds cannot yet be estimated. The pilot test will continue to operate and resin performance will continue to be evaluated to inform the Pre-final (95%) Design of the AG treatment system.

6.3 Influent Characteristics

Influent groundwater characteristics and estimated flow rates, determined by groundwater modeling as discussed previously, 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 N). Influent concentration and flow rate estimates were used to determine mass loading and system capacity requirements.

6.3.1 Modeled Flow Rates

As discussed in Section 5, the Intermediate (60%) Design for the groundwater extraction system assumes an average extraction flow rate of 42.3 gpm and a maximum extraction flow rate of 120 gpm.

6.3.2 Contingency Groundwater Storage

Inclusion of contingency groundwater storage was evaluated to provide storage should BASF be directed by DUWA to temporarily cease discharge of treated effluent water. An evaluation of recent DUWA bypass events was completed. Six events ranging from 17 to 63 hours occurred in 2021. These events are understood to be related to increased infrastructure demand during precipitation events. The precipitation events causing bypass were approximately 1.75 inches or greater.

Infrastructure upgrades were completed in 2022, and no bypass events occurred in 2022 and 2023.

Based on review of the DUWA bypass events in 2021 and taking into consideration the DUWA upgrades and lack of bypass events in 2022 and 2023, contingency storage will be designed to accommodate one-day capacity should DUWA mandate no discharge to its facility.

6.3.3 Design Influent Flow Rate and Contingency Storage

Based on the groundwater modeling results, the proposed AG treatment system design flow rate is 120 gpm. The proposed flow rate is based on the proposed performance standard approach, minimizing equipment and infrastructure sizing and cost considerations. If DUWA were to temporarily mandate that BASF could not discharge to DUWA's facility, the storage volume required for 24 hours of operation at the maximum design flow rate is 172,800 gallons. A storage tank with a nominal capacity of 200,000-gallon tank and a minimum operating capacity of 175,000 gallons is therefore proposed for this design.

The extraction system will send influent water to the contingency storage tank when a mandated shutdown occurs. Influent water will then be transferred from the contingency storage tank to the influent equalization tank at a rate of 120 gpm for treatment after the mandated shutdown is lifted. The AG treatment system will return to normal operation (i.e., treating groundwater directly from the extraction sumps) when the groundwater in the contingency storage tank has been treated to a low-level setpoint. The proposed 200,000-gallon contingency storage tank will be located adjacent to the Infinergy Warehouse Building (Infinergy building). Contingency storage options will continue to be evaluated during the Pre-final (95%) Design, and alternate storage options such as multiple smaller tanks instead of one large tank or a retention pond or retention basin may be used in lieu of an AG storage tank.

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 Pre-Design Investigation Report (Arcadis 2021c) and AG System Treatability Study Report (Arcadis 2022b). The monitoring wells were installed in zones with the highest hydraulic conductivity and therefore 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 pump and treat 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 mercury, VOCs, SVOCs, and PFAS (Appendix N). The analytical results for the COCs detected and general chemistry in the blended groundwater samples from December 29, 2021 are presented in Table 11.

Table 11. 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
mg/L = milligrams per liter
µg/L = micrograms per liter
ng/L = nanograms per liter

6.4 Major Unit Processes

Eight major components comprise the AG treatment system and were selected, sized, and designed based on the proposed design flow rates and treatability study findings (Arcadis 2022b). Influent groundwater is collected and stored in an equalization tank to reduce flow and concentration peaks prior to downstream treatment. Neutralization is then performed to reduce pH and prepare the influent water for metals precipitation via coagulation, flocculation, and clarification. Following metals precipitation, process water is fed to a pump-out tank to facilitate treatment via a bag filter, GAC, and ion exchange units to address total suspended solids, SVOCs, VOCs, and PFAS. Further detail is provided in the following sections, in the Design Drawings included in Appendix G, and in the technical specifications included in Appendix K. Average and maximum flow rates are defined above as 42.3 gpm and 120 gpm, respectively.

6.4.1 Influent Equalization

The influent equalization tank will receive untreated groundwater from the extraction system and the contingency storage tank. The equalization tank was designed to dampen variable flow rates and concentrations prior to downstream treatment. The influent equalization tank was sized to maintain groundwater extraction at the design flow rate during routine system maintenance activities such as equipment cleaning and media backwashes, which are expected to take no more than 6 hours. The minimum equalization tank operating volume is approximately 17,000 gallons and has been sized for 20,000 gallons (total).

During normal operation, the tank will operate to maintain a minimum hydraulic residence time of 16.7 minutes (2,000 gallons utilized). Transfer pumps equipped with variable frequency drives will be used to transfer influent water from the equalization tank to downstream operations within the design flow rates. The influent equalization tank parameters are summarized in Table 12.

Table 12. Influent Equalization Tank Design Parameters

Design Parameter	Average	Maximum
Design Equalization Tank Volume (gallons)	20,000	20,000
Equalization Tank Diameter (feet)	14	14
Equalization Tank Operating Height (feet)	15	15
Equalization Tank Freeboard (feet)	2	2
Equalization Tank Total Height (feet)	17	17
Equalization Tank Retention Time (minutes), at Operating Volume (2,000 gallons)	47.3	16.7

Tank level will be maintained using a control loop to communicate with associated 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.

As noted previously, contingency storage is included to manage DUWA bypass events. Details of the contingency storage tank are provided in Section 6.5.2.

6.4.2 pH Neutralization

The treatability study (Arcadis 2022b) included a pH neutralization step to determine the acid dosage required to reduce the groundwater pH level. A reduced pH level 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 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 with sulfuric acid to determine the volume of acid needed to reach the target pH level. Sulfuric acid was selected for pH adjustment due to its effectiveness in pH neutralization with similar groundwater chemistry. The design parameters based on influent water quality and the pH neutralization test are presented in Table 13.

Table 13. pH Neutralization Design Parameters

Design Parameter	Average	Maximum
Initial pH (S.U.)	12.4	12.4
Target pH (S.U.)	9.0	9.0
Sulfuric Acid Dosage to Reach pH 9 S.U. (milliliters per liter of 5-Molar Sulfuric Acid)	1.9	1.9
Sulfuric Acid Concentration (percent by weight)	40%	40%
Sulfuric Acid Usage (gallons/day)	116	328

Sulfuric acid will be delivered to and fed from an 8,700-gallon operational capacity tank using a metering pump. The rate of acid addition will be controlled by a pH sensor in the pH neutralization tank.

6.4.3 Precipitation, Coagulation, and Flocculation

The pH-neutralized process stream 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 in accordance with the anticipated POTW discharge limits. 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 S.U. followed by coagulation and flocculation addition, 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 raw groundwater, with a total mercury concentration range 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. 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, following coagulation and flocculation treatment. The recommended chemical amendments and doses are presented in Table 14.

Table 14. 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 0.9 and 2.6 minutes at the maximum flow rate of 120 gpm and the average flow rate of 42.3 gpm, respectively. The coagulant and metal precipitant will be introduced in the same mixing tank for the full-scale design to prevent O&M issues associated with process pipe fouling. 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 gravity feed into the clarifier's slow mix flocculation tank where the cationic flocculant will be introduced. The flocculation tank is sized to provide approximately 2.6 minutes of retention time at 120 gpm. Retention time is based on equipment specifications for a clarifier at the 120-gpm design flow rate.

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 low-level switch (radar type) to prevent system 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 into a transfer tank. Settled solids will be removed periodically from the bottom of the clarifier using a pneumatic-diaphragm pump and transferred into a sludge holding tank. The design parameters for the clarifier are presented in Table 15.

Table 15. Clarifier Design Parameters

Design Parameter	Value
Influent Solids (mg/L)	79.8
Clarifier Solids (%)	1.5
Minimum Required Plate Area (square feet)	432
Clarifier Plate Area (square feet)	560
Rapid Mix Tank Volume (gallons)	110
Slow Mix Tank Volume (gallons)	326
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 a pump-out tank where transfer pumps will convey process water to bag filters for removal of suspended solids. The transfer pumps will be equipped with a variable frequency drive to maintain a constant level in the pump-out tank. Under certain circumstances, batch operation will be necessary and controlled by switches. The pump-out tank design parameters are presented in Table 16.

Table 16. Pump-Out Tank Design Parameters

Design Parameter	Average	Maximum
Process Flow Rate (gpm)	42.3	120
Retention Time (minutes)	47.3	16.7
Total Pump-Out Tank Volume (gallons)	2,000	2,000
Tank Overall Height (feet)	7	7
Tank Overall Diameter (feet)	7.5	7.5

Bag filtration is used to improve GAC media performance by 1) reducing solids loading, which promotes channeling and preferential flow through the GAC media, reducing overall contaminant removal efficiency, 2) mitigating fouling of the GAC media, which increases system pressures, and 3) reducing the number of GAC backwash cycles required to maintain system flow rates. Five-micron filter bags are proposed but will be modified based on observed conditions and performance following start-up. The bag filters will be provided on a dual housing skid and automatically switch between filter housings based on pressure drop, volume, or time setpoints or have the option of parallel operation. Bags will be changed by the system operator, as needed, and disposed of in accordance with the site-specific Draft Waste Management Plan included in Appendix O.

6.4.6 Liquid-Phase Granular-Activated Carbon

The GAC system was designed to facilitate removal of SVOCs and VOCs. The GAC system will also remove TOC and PFAS, but since TOC does not have a local limit and PFAS removal is targeted during the ion exchange resin treatment, reducing either compound is not a design criterion of the GAC treatment. An RSSCT was performed to evaluate the effectiveness of SVOC and VOC removal using reactivated GAC. The blended influent groundwater collected from the Site was pretreated with sulfuric acid to a pH of 9.0 S.U. and amended with the chemicals described in Section 6.4.3 to remove mercury. Calgon DSR-C reactivated carbon was used for the RSSCT with two columns, resulting in a 40-minute total EBCT. The RSSCT was used to confirm that the reactivated GAC was capable of treating VOCs and SVOCs to below POTW limits. An existing onsite construction water treatment system was used to determine the target EBCT, and the maximum design flow rate of 120 gpm was used to establish the parameters for the GAC design. At the maximum flow rate of 120 gpm, four GAC vessels operated as two parallel trains with two vessels per train are required. When operating at lower flow rates, all four vessels will be used to prevent water stagnation in the unused vessels. Additional backwashing may be used to counter the effects of channeling if suspected. Each train will have the capacity to alternate the lead and lag GAC vessels for each treatment train. Backwashing the GAC vessels will require the use of the pump-out tank and associated pumps, and dirty backwash water will be pumped back to the influent equalization tank. During media changeouts, a single train will continue treating influent water at a maximum flow rate of 60 gpm. The minimum flow rate through the GAC system is 15 gpm to facilitate proper treatment through the ion exchange system, as discussed in Section 6.4.7. The pump-out tank transfer pumps will process water in batch mode, as needed, to maintain the minimum required flow rate through the ion exchange system. The GAC vessel parameters for the Intermediate (60%) Design are based on a 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 17; additional calculations for GAC vessel sizing are presented in Appendix J.

Table 17. Liquid-Phase Granular-Activated Carbon Design Parameters

Design Parameter	Average	Maximum
Design Flow Rate (gpm)	42.3	120
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)	0.7[(1)]	2.1
GAC Weight, Per Vessel (pounds)	5,000	5,000
EBCT, Per Vessel (minutes)	69.8	24.7

Note: [(1)] The hydraulic loading rate at the average flow rate is below the stated minimum of 2 gpm/square foot. 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.

6.4.7 Ion Exchange Resin

Ion exchange resin will be used for the removal of PFAS following GAC treatment. Three parallel resin trains with two vessels in series per train with a minimum flow rate of 15 gpm per vessel were designed to allow for operational flexibility while maintaining the hydraulic loading rates recommended by Purolite. The resin system will have the capacity to alternate the lead and lag vessels for each treatment train and be designed to automatically switch between one, two, or three operating trains, as needed, per the flow ranges presented below:

- One train (two resin vessels) can be used for flow rates between 15 and 30 gpm.
- Two trains (four resin vessels) can be used for flow rates between 30 and 45 gpm.
- Three trains (six resin vessels) can be used for flow rates exceeding 45 gpm.

Identical to the GAC process, the resin trains will be configured in series with a valve tree to enable alternating lead-lag operation. A minimum flow rate of 15 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 15 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 or two trains will continue treating influent water at a maximum flow rate of 40 gpm per train.

Pilot data from the West Tracks system on the Site 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 Purolite recommendations. Elevated concentrations of total dissolved solids, primarily due to chloride and sulfate, were present in the influent water used for the treatability study and are known to affect PFAS treatment performance using resin as discussed in Section 6.2. Resin vessel parameters for the Intermediate (60%) Design are therefore based on a minimum 2-minute EBCT per vessel and hydraulic loading between 6 and 18 gpm per square foot per Purolite specification and consistent with the parameters across the first resin vessel bed in the perimeter pilot system. A summary of the key ion exchange resin process parameters is presented in Table 18; additional calculations for ion exchange resin vessel sizing are presented in Appendix J.

Table 18. Ion Exchange Design Parameters

Design Parameter	Average	Maximum
Design Flow Rate (gpm)	42.3	120
Ion Exchange Resin Vessel Diameter (feet)	1.75	1.75
Ion Exchange Resin Bed Depth (feet)	5	5
No. of Ion Exchange Resin Vessels in Series	2	2
No. of Ion Exchange Resin Vessels in Parallel	2	3
Hydraulic Loading, Per Vessel (gpm/square foot)	8.8	16.6
Ion Exchange Volume, Per Vessel (cubic feet)	12	12
EBCT Per Vessel (minutes)	4.2	2.2

6.4.8 Discharge

Effluent samples will be collected regularly to confirm the AG treatment system is meeting POTW discharge limitations. Effluent flow will be measured using a magnetic flow meter and discharged to the POTW via sanitary sewer lines. A tie-in to the existing sanitary sewer line will be installed and equipped with check and isolation valves to prevent backflow from the site sanitary connection. Anticipated POTW discharge limits are provided in Appendix A of the DUWA Sewer Use Regulations included as Appendix N.

6.5 Ancillary Processes

The following sections present the Intermediate (60%) Design for the AG treatment system pumps, contingency storage tank operation, solids management process, tank mixing, compressed air, and instrumentation and controls, collectively the ancillary processes for the AG treatment system.

6.5.1 System Pumps

Pump and motor sizes were determined using hydraulic calculations for all unit processes as presented in Appendix J. 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. The preliminary pump and motor parameters are presented in Table 19.

Table 19. Pump Design Parameters

Pump ID	Number of Pumps	Description	Pump Type	Design Flow Rate (gpm)	Head (feet)	Horsepower
P-0XX	8	Collection Sump Pump	Submersible	1.0 to 2.5	26 to 27	0.5
P-0XX	34	Collection Sump Pump	Submersible	0.7 to 5.5	27	1
P-0XX	2	Extraction Well Pump	Submersible	2.5	4.3 to 4.4	0.5
P-100A/B and P-101A/B	4	Transfer Pump	Centrifugal	120	12	3
P-200A/B	2	Transfer Pump	Centrifugal	120	170	10
P-500/540	2	Dosing Pump	Diaphragm	0.004 to 0.6	134 (maximum)	1/12
P-530/550	2	Dosing Pump	Peristaltic	0.6 to 2.5	7 (maximum)	1/2
P-510/520/560	3	Dosing Pump	Diaphragm	0.00005 to 0.005	11 (maximum)	1/32
P-600, P-601A/B, P-602A/B	5	Sludge Wasting Pump	Diaphragm	15	15	Not applicable – pneumatic pump
P-605	1	Sump Pump	Submersible	30	22	0.5

6.5.2 Contingency Storage Operation

The contingency storage evaluation is discussed in Sections 6.3.2 and 6.3.3. Under normal operating conditions, water will be collected in the sumps and pumped through the conveyance lines to the influent equalization tank, TK-100, discussed in Section 6.4.1. Pump flow rate will be controlled by gradient differential between the river and associated compliance points for the respective sump. The AG treatment system will be sized to accommodate the modeled maximum flow rate from the sumps. Groundwater from the drainage sumps will be redirected from the equalization tank to the contingency storage tank via automated valves when BASF is notified of a DUWA mandated shutdown. Contingency storage pumps will begin transferring groundwater from the contingency storage tank to the influent equalization tank when the DUWA mandated shutdown has ended. Sump flow will continue to be diverted to the contingency storage tank until the contingency storage tank low-level switch (float) setpoint has been reached. After the low-level setpoint has been reached, groundwater flow from the drainage sumps will be redirected back to the influent equalization tank by closing and opening the necessary valves and turning off the contingency storage transfer pumps. Contingency storage tank parameters are provided in Table 20.

Table 20. Contingency Storage Tank Design Parameters

Design Parameter	Value
Total Volume (gallons)	200,000
Operational Volume (gallons)	175,000
Height (feet)	32
Diameter (feet)	33

6.5.3 Solids Management

Solids from the influent equalization tank and clarifier will be periodically removed. Solids removal will be a manual process from the equalization tank and an automatic process from the clarifier. Solids transfer intervals will be determined and optimized following start-up operation to maintain sufficient total suspended solids removal. Clarifier solids removal will be controlled based on gallons treated and observations of sludge accumulation and discharge from the clarifier sump. The collected solids will be stored in a holding tank prior to dewatering. For solids dewatering, a thickening agent will be added and the solids will pass through a plate and frame filter press prior to storage in a disposal container. Dewatered solids collected in the disposal container will be disposed of in accordance with the site-specific Draft Waste Management Plan (Appendix O). 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, resulting in lower than expected solids generation, reference design standards were used to establish the parameters for the solids management process as listed in Table 21.

Table 21. Solids Management Design Parameters

Design Parameter	Average	Maximum
Holding Tank Influent Flow Rate (gallons/day)	772	2,191
Holding Tank Volume (gallons)	15,000	15,000
Holding Tank Retention Time (days)	19.4	6.8
Holding Tank Underflow Solids (%)	2.0	2.0
Holding Tank Decant Volume (gallons/day)	404	1,146
Filter Press Size (cubic feet)	19	19
Cycles Per Week	1.0	2.5
Cake Solids (%)	35	35
Cake Density (pounds/cubic foot)	75	75
Pressed Sludge (pounds/press cycle)	1,425	1,425
Pressed Sludge (tons/year)	32.9	92.6
Filtrate (gallons/press cycle)	2,819	2,819

6.5.4 Tank Mixing

The inclined plate clarifier's coagulant tank and flocculation 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. Rapid mixing will occur in the coagulation tank, while slow mixing will occur in the flocculation tank. Aggregation of particulates will promote flocculation formation and effective settling in the inclined plate clarifier.

6.5.5 Compressed Air

Existing plant air will be utilized to supply air-operated equipment. Compressed air requirements will be based on two sludge pumps operating to accommodate simultaneous sludge removal from the clarifier and filter press operation. A contingency will be applied to account for simultaneous pneumatic valve operation. Each sludge pump will require an air supply at 20 standard cubic feet per minute of the Infinergy building supplied air pressure of 110 pounds per square inch (psi).

6.5.6 Instrumentation and Controls

The AG treatment system will include a programmable logic controller (PLC) based supervisory control and data acquisition (SCADA) system. The SCADA system will provide monitoring, control, alarming, and data collection. Flow and pump control at the extraction system sumps and extraction wells 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, which ties into the BASF control 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 drains to provide field control of the sump pumps. Each sump control panel will have protective hoods, fan cooling, and locking covers. 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 shutdown of process pumps and 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 a determination as to whether to restart the system. An interlock table describing process alarm conditions and associated process responses is provided in Appendix P.

6.6 Treatment Building Design

Following the Preliminary (30%) Design, existing building and new building options were analyzed to facilitate building selection for the Intermediate (60%) Design. The Infinergy building, located at the northern end of the Site, has been selected to house the AG treatment system. The Infinergy building was selected based on available warehouse space, ease of access for supply trucks, and an existing adequate power supply, with a backup generator, ventilation system, and fire suppression system. The Infinergy building is an approximately 37,000-square-foot pre-engineered metal building with a floating slab-on-grade foundation. It was constructed in 1994 and is currently used for storage.

This section summarizes the code analysis performed to establish criteria requirements to be used as a guide for the design and construction of the AG treatment system building. This section presents specific code and regulatory requirements and identifies design parameters for project preparation in this regard. Further detail is provided in the Design Drawings included in Appendix G and in the technical specifications included in Appendix K.

6.6.1 Codes and Standards

The following applicable codes and standards have been observed for design and will be observed for construction of the AG treatment system within the Infinergy building:

- 2015 Michigan Building Code;
- 2015 Michigan Mechanical Code;
- 2015 Michigan Plumbing Code;
- 2015 Michigan Electrical Code;
- 2017 National Electrical Code;
- 2015 Michigan Energy Code;
- 2018 International Fire Code;
- 25th Edition Guide to the Michigan Gas Safety Standards;
- National Fire Protection Association (NFPA) 13 – Installation of Sprinkler Systems;
- NFPA 14 – Installation of Standpipe and Hose System;
- NFPA 20 – Installation of Stationary Pump for Fire Protection;
- NFPA 70 – National Electric Code;
- NFPA 72 – National Fire Alarm Code;
- NFPA 101 – Life Safety Code; and
- 2010 Americans with Disability Act (ADA) Standards for Accessible Design.

6.6.2 Building Code Analysis

An analysis of the building code was performed, considering the anticipated future occupancy of the designated Infinergy building space allocated for the AG treatment system. Below is a summary outlining the findings derived from this analysis:

- Existing Occupancy Type: S-1, Storage, Moderate Hazard + B, Business;
- Proposed Occupancy Type: F-1, Factory Industrial, Moderate Hazard;
- Construction Type: IIB, Non-Rated;
- Sprinkler System Provided: Yes;
- Allowable Building Height: 75 feet. Existing building height: 45 feet (no change to existing compliance);
- Allowable Number of Stories: 3. Existing number of stories: 1 (no change to existing compliance);
- Allowable Square Footage: 62,000 square feet. Existing square footage: 37,000 square feet (no change to existing compliance);

- Required Separation of Occupancies Between F-1 & S-1 with Sprinkler System: 0-hour fire rating;
- Primary Structural Frame: 0-hour fire rating;
- Occupancy Load: 370 occupants;
- Maximum Common Path of Egress Travel Distance: 100 feet (no change to existing compliance);
- Maximum Egress/Exit Access Travel Distance (w/ Sprinkler System): 250 feet (design complies);
- Minimum of two exits out of the room are required if path of travel is more than 100 feet (no change to existing compliance);
- Water Closets Required/Provided (1 per 100 occupants): 4, 2 Male / 2 Female (no change to existing compliance);
- Lavatories Required/Provided (1 per 100 occupants): 4, 2 Male / 2 Female (no change to existing compliance);
- Drinking Fountains Required/Provided (1 per 400 occupants): 1 (no change to existing compliance);
- Service Sinks Required/Provided: 1 (no change to existing compliance); and
- Emergency Eyewash/Shower: Not Required. Provided: 1 (building complies).

In summary, the Infinergy building complies with the size, height, ingress/egress, and facility requirements for its intended use.

6.6.3 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. It takes into account various factors such as chemical properties and their reactivity, flammability, toxicity, and storage container types. It is assumed in this analysis that there are no chemicals with physical or health hazard classifications currently within the existing Infinergy building. 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.

6.6.3.1 Indoor Storage

The proposed quantities of chemicals to be added within the existing building do not exceed the maximum allowable for indoor control areas for both physical and health hazards and will not require any building upgrades or modifications.

Indoor Storage Chemical List and Actual Quantities:

- Sulfuric Acid, 40% – Water Reactive 1, Corrosive – 4 gallons in piping (use-closed);
- KlarAid IC1251 – Corrosive – 4 gallons in piping (use-closed), 263.3 gallons indoor storage;
- MetClear MR2405 – Corrosive – 4 gallons in piping (use-closed), 209.2 gallons indoor storage;
- Polyfloc AE1703 – Non−Hazardous - 4 gallons in piping (use-closed), 55 gallons indoor storage; and
- ChemTreat P8315 – Non−Hazardous – 4 gallons in piping (use-closed), 55 gallons indoor storage.

Maximum Allowable Quantities (storage + use-closed + use-open) – Corrosive: 500 gallons. Actual: 484.5 gallons (complies)

6.6.3.2 Outdoor Storage

Bulk sulfuric acid (40% – Water Reactive 1, Corrosive – 7,590 gallons) will be stored in a double-walled tank located outside of the Infinergy building. The outdoor control area will be set back more than 20 feet from public streets, public alleys, public rights-of-way, or lot lines. A two-hour fire-resistance-rated wall will be constructed between the tank and the exterior wall of the Infinergy building. The wall will extend at least 30 inches above and to the sides of the storage area.

7 Waste Management and Characterization

Waste management will be conducted in accordance with applicable federal, state, and local waste regulations. Onsite management, sampling, characterization, and disposal of waste are documented in the BASF North Works Draft Waste Management Plan (Appendix O).

Soil generated from trenching and/or barrier wall installation will be pre-characterized for waste disposal in accordance with applicable regulations prior to construction activities. All soils that are not adequate for use as backfill will be disposed of at a licensed waste facility. Groundwater generated during dewatering will be characterized prior to construction activities for disposal or to confirm it can be treated and discharged to DUWA. Non-impacted packaging for materials and equipment will be disposed of with general refuse.

8 Health and Safety Plan

A project-specific health and safety plan (HASP) will be developed for the construction and operation of the proposed remedy at the Site. The HASP will describe the health and safety commitment of all 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 all 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.

All 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 barrier remedy include the following:

- Potential hazards during construction activity 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 the AG treatment system process.

8.2 Site Safety

All personnel working on the project are responsible for completing tasks safely and have the responsibility to stop the work of a coworker or contractor if working conditions or behaviors are deemed unsafe. All BASF and OSHA-required general safety equipment, 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. All 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 performed at the start of the 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 take BASF site-specific safety and site-awareness training. All contractors working at BASF facilities are screened by the Avetta system, requiring a total recordable incident rate (TRIR) 3-year average less than or equal to 1.80, an lost workday rate (LWCR) 3-year average less than or equal to 1.00, and an experience modification rate (EMR) 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 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, a second step design review hazard analysis will be performed to build on the initial hazard assessment and provide guidance for best practices associated with system fail safes, remedy construction, O&M, and regulatory codes. The hazard analysis during the Pre-final (95%) Design phase will include:

- Updated review of the Intermediate (60%) Design hazard analysis;
- Analysis of chemical reaction hazards;
- Analysis of thermal hazards;
- Analysis of dust hazards; and
- Analysis of interlock controls.

The hazard analysis will systematically identify potential concerns and outline engineering and administrative controls that can mitigate the identified hazards.

8.4 Release Containment

Secondary containment will be provided for all fluid-containing vessels and piping carrying chemicals. The following controls will prevent the occurrence of releases to the ground:

- The system will be equipped with alarms to disable the system should a spill or release to the secondary containment occur.
- Emergency stop buttons will be located on the main control panel and the clarifier within the building.
- The AG treatment system will have secondary containment for all process tanks, equipment, and piping within the building. The volume within the containment area exceeds 110% of the volume of the largest tank (equalization tank, which is 20,000 gallons).
- Sensitive 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 double-walled and equipped with an interstitial sensor to detect leaks in the inner wall.

9 Future Considerations

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

- Enhancement of containment barrier through welding of steel plates or jet grouting;
- Addition of supplemental infrastructure (jet grout, sumps on existing drains, vertical wells, horizontal wells, or new drains) to consistently 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;
- Reuse of the treated effluent water to feed BASF’s onsite steam supply system as a potential alternative and/or supplement to POTW discharge; and
- Further separation of stormwater from groundwater by managing and diverting stormwater runoff away from the collection drains.

10 Data Gaps

During the Intermediate (60%) Design phase of the project, it was determined that an assessment of the potential for voids along the existing bulkhead, specifically beneath the former Heavy Dock section, is warranted. An investigation program will be developed for assessing the presence of voids beneath the concrete structure of the seawall. The program may include a combination of soil borings advanced through the concrete and/or additional diver inspections for investigating the soil conditions at depth. Results of the investigation, and the proposed measures to fill any voids identified, will be included the Pre-final (95%) Design.

Additional modeling will also be conducted during the Pre-final (95%) Design phase to further evaluate stormwater runoff at the Site. A two-dimensional stormwater model will be developed considering the rainfall-runoff process as it relates to infiltration. To support the two-dimensional modeling, an updated light detection and ranging (LiDAR) survey will be conducted prior to the Pre-final (95%) Design. The results of the stormwater modeling will be used to confirm the infiltration assumptions in the site groundwater model. The groundwater model infiltration rates will be updated, and scenarios will be simulated to update the extraction system flow rates accordingly.

11 Permit Plan

Various regulatory permits will be required to facilitate construction and operation of the barrier remedy. A permitting schedule was submitted to USEPA on July 24, 2023, following notification of the intent to install the barrier remedy to the relevant permitting agencies (DUWA, EGLE, and the City of Wyandotte). The following is a summary of the anticipated permits required prior to or during construction:

- **National Pollutant Discharge Elimination System (NPDES).** A Construction General Permit will be submitted 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 developed Stormwater Pollution Prevention Plan, a Notice of Intent submittal, and a list of required inspections to verify compliance with the permit. The permit application will be submitted to USEPA several months before the beginning of construction.
- **EGLE/USACE**. A Joint Permit Application (JPA) will be submitted to cover requirements derived from state and federal rules and regulations for construction activities within and near the Detroit River. A summary of the site background, a project description, and site plans will be submitted in the application for review and approval. 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 in 2024 following completion of the Intermediate (60%) Design.
- **DUWA**. Discharging treated groundwater into a DUWA sewage collection system tributary requires an approved Class D Wastewater Discharge Permit. A permit application will be submitted prior to the proposed discharge for evaluation and approval. An alternative to a new permit would be a modification to the existing Wastewater Discharge Permit obtained by BASF. A request to modify the existing permit would be submitted to DUWA for approval in lieu of a new permit application. Arcadis and BASF notified DUWA of the intent to submit for a permit on July 20, 2023, and will continue to hold regular check-in meetings with DUWA as the design progresses. The permit application or modification will be submitted several months before the beginning of construction, tentatively in 2026.

- **Wayne County**. An SESC Permit is required for earthwork within 500 feet of a water of the state. The SESC permit application will consist of approximately nine drawings showing the SESC controls, a short narrative summary of the SESC plan, and the permit form. The permit application will be submitted several months before the beginning of construction, tentatively in 2026.
- **City of Wyandotte**. Construction of the barrier remedy will require permits from the City of Wyandotte Engineering and Building Department. Arcadis/BASF notified the City of Wyandotte of the general construction plans on July 20, 2023. Permit applications, where necessary, will be submitted to the City several months before the beginning of construction, tentatively in 2026.
- **Air Permitting**. Air permitting requirements for construction associated with diesel-operated equipment and/or generators will be assessed and completed, if applicable. At this time, the design does not include active air discharges; however, any air emissions identified prior to construction and system operation will be evaluated and addressed in the Final (100%) Design phase.
- **Spill Control**. All liquids included in the AG treatment system design and operation will be evaluated to determine spill control requirements. Any tanks or drums containing flammable, combustible, or corrosive liquid will be subject to containment standards set by federal, state, OHSA, and NFPA code. The remedy does not involve storage of petroleum products above regulatory thresholds; therefore, it is not anticipated that a Spill Pollution, Control and Countermeasures Plan will be required.

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 offsite sheet pile and collection drain.
- A Notice to Mariners will be provided.

12 Operation, Monitoring, and Maintenance Requirements

This section presents the preliminary OMM requirements for the barrier remedy. A draft OMM manual for the perimeter barrier remedy will be submitted with the Pre-final (95%) Design. The OMM plan addresses routine inspections of barrier components (e.g., exposed bulkhead components and ground surface grades); OMM of the barrier remedy including management of waste streams; collection of performance monitoring data; and compliance monitoring.

12.1 Perimeter Barrier

As discussed in Section 3.4.1, a key component to meet the CAOs associated with soil and sediment is the presence of the physical sheet pile barrier that will prevent site soils from migrating offsite and entering the Detroit River. As discussed in Section 3.4.2, the barrier is also a key feature to meet the performance standards for groundwater by preventing the migration of groundwater toward the Detroit River. Therefore, periodic inspections of visible barrier elements is required to meet the performance standards. For the in-river shoreline barriers (existing and South Dock bulkhead), the sheet piles will be periodically monitored for corrosion, damage, or deterioration to confirm that performance standards continue to be met. Monitoring of the subsurface barriers (i.e., northern and southern ends of the Site) will be conducted through visual ground surface inspections and indirectly through gradient monitoring.

Structural inspections of the Site's border with the Detroit River (all shoreline feature areas with steel sheet pile walls) and all subsurface barrier alignments will be conducted on a regular basis to establish a proactive approach to monitoring the condition of the various barrier components. Visual inspections will occur at least every 5 years. Frequency will be adjusted, if needed, based on inspection reports or drain compliance elevation monitoring of the groundwater extraction system. Visual inspections of the shoreline barriers will be performed from both the landside and waterside. The visual inspections will include features that are visible above the water surface, such as alignment, subsidence, cracking, offsets, spalling, visible steel members, and any observable areas of land surface subsidence along the backside of the sheet pile wall. Visual inspections of the subsurface barriers will include inspecting the ground surface along the alignment of the barriers to identify surface erosion or observable subsidence. Diving inspections of the shoreline barriers below the water surface will also be conducted.

Observations, findings, photographs, and short-term and long-term maintenance recommendations (from both the visual inspections and the diving inspections) will be documented in a report following the inspection. Repairs and/or protective features will be implemented, as needed, to ensure the performance standards of the barrier remedy are maintained. If the compliance drain elevation cannot be maintained in a portion of the perimeter, potential supplemental infrastructure may be installed as discussed in Section 3.4.3.

As discussed in Section 4.3.2.3.3, placement of fill beneath the South Dock concrete deck will likely induce consolidation settlement in the underlying clay unit. As a result, settlement monitoring of the fill surface will be performed to assess the progress of consolidation in the clay unit and to minimize strain on the tie rods due to settlement of the fill. When consolidation settlement is complete or nearly complete, which could take several years, the gap between the fill and the concrete will be closed by pumping flowable, cementitious grout into the gap.

12.2 Groundwater Extraction and Conveyance System

Efficient operation of a groundwater extraction and AG treatment system requires a systematic approach encompassing a spectrum of OMM activities. These tasks are crucial for achieving optimal operation and adherence to performance standards. To maintain the drain compliance elevation, river and site water levels will be monitored routinely through a network of operational and compliance PZs and river level gauges. Compliance PZs will be installed and monitored as discussed in Section 3. Transducers will be installed in the PZs within the drains and sumps to control and adjust pumping rates to maintain the drain compliance elevation at the compliance PZs. The system will routinely compare data and alert operators of any alarm conditions and/or increasing drain levels that could indicate a problem with the conveyance system or extraction equipment and inhibit compliance with the performance standards.

Additional routine OMM activities for the groundwater extraction and conveyance system include, but are not limited to, the following:

- Review of operational and performance data to confirm parameters are within the expected ranges;
- Visual inspection of equipment;
- Maintenance of extraction well pumps, sump pumps, and contingency storage pumps;

- Cleaning of conveyance lines and collection drains to remove accumulated sediment and debris;
- Well development for the vertical extraction well; and
- Annual function testing of level switches and controls.

12.3 Above-Grade Treatment System

To achieve proper treatment of collected groundwater, AG treatment system performance must be monitored and maintained. Influent groundwater conditions may vary greatly depending on pumping rate from each sump and the varying groundwater conditions throughout the Site. Sudden and/or gradual changes in influent water conditions can lead to clarifier upsets and/or different chemical dosing needs. To accommodate unanticipated changes in the influent and respond rapidly, visual performance indicators and jar tests may be used. Jar testing allows the operator/engineer to determine alternate mixing rates and chemical dosages that will result in proper settling for solids removal. Analytical samples may also be taken to confirm treatment results.

The following OMM activities will help facilitate proper evaluation of the AG treatment system operation, identify causes for upset, and maintain proper operation:

Activities anticipated to be completed two to three times per week:

- Assessment of key performance indicators such as pH, colorimeter, and turbidity tests;
- Measurement and recording of system flow rates and pressures; and
- Visual inspection of equipment.

Activities anticipated to be completed weekly:

- Performing clarifier maintenance to prevent solids buildup;
- Backwashing GAC vessels;
- Operating the filter press and emptying the sludge hopper;
- Conducting sludge settling test to assess system performance;
- Checking and refilling chemical feed tanks;
- Changing bag filters; and
- Transferring influent equalization tank sludge to the sludge holding tank.

Activities anticipated to be completed monthly, quarterly, or annually:

- Collect POTW compliance samples (as required by the permit);
- Collect performance samples (monthly):
  - PFAS: influent and resin midfluent;
  - Mercury: influent and clarifier effluent;
- Change out GAC and ion exchange media;
- Perform equipment and instrumentation maintenance; and
- Conduct annual function testing of level switches and controls.

The draft OMM manual submitted with the Pre-final (95%) Design will include schedules and specific procedures for each remedy component.

13 Remedial Project Management

This section describes the proposed approach to carry out the design and implementation of the barrier remedy.

13.1 Value Engineering

As the containment barrier, extraction system, and AG treatment system design progresses, the design elements have and will continue to be evaluated in an effort to reduce overall costs. Value engineering (VE) opportunities implemented for the Intermediate (60%) Design include:

- Proposed a performance standard strategy that is protective and achievable, while reducing operational and permitting risks and considering cost. Input parameters were varied (i.e., river elevation averaging time frames and compliance gradient requirements) to develop a proposed approach that maximizes protectiveness, minimizes operational and permitting risks, and is achievable without inflating costs.
- Moved the AG treatment system into an existing building.
- Incorporated the use of existing heating, ventilation, and air conditioning (HVAC) and electrical infrastructure, within the Infinergy building, to the extent possible.
- Utilized existing aboveground pipe racks for conveyance piping and electrical conduit.
- Designed for an extraction well in lieu of a drain in the northeast corner of the Site to avoid trenching in a utility-dense area with the existing bulkhead tiebacks.
- Strategically designed a drain alignment based on groundwater model and geophysical survey results to efficiently achieve the compliance gradient requirement while also avoiding historical subsurface infrastructure to the extent possible.
- Made adjustments to the anchor wall type and alignment based on results of the geophysical survey to avoid potential subsurface obstructions and as needed to facilitate groundwater recovery in the collection drains.
- Created two design sections for the headwall to optimize the design.
- Refined the soil design parameters after an in-depth analysis of the soil data.
- Refined the clay surface based on the results of additional investigations and subsequently refined the depth and quantity of the subsurface sheet piling.

Potential opportunities for additional VE during the Pre-final (95%) Design or procurement phase include the following:

- Continued evaluation of cost versus function to identify high-cost design elements that may be candidates for a VE study.
- Consideration of material/equipment factors including capital cost; complexity; high-volume, critical materials; difficulty of use in construction; high O&M costs; required specialized skills to construct; and potential for materials and methods to become obsolete.
- Allowance of substituted equipment that meets the design specifications at a reduced cost.
- Use of industry-established design or construction technology.
- Use of pre-designed skids or equipment packages.

13.2 Design and Construction Schedule

A preliminary design and construction schedule is presented in Appendix Q. The Great Lakes Legacy Act Upper Trenton Channel (UTC) sediment dredge project is progressing on a parallel path with the perimeter barrier remedy. Communication between the UTC project team and the perimeter barrier remedy project team is ongoing and will continue throughout the design phase to ensure that both projects proceed, as planned. There is flexibility in the perimeter barrier remedy construction phasing and sequencing that allows the perimeter barrier construction to align with, and not interfere with, the UTC sediment dredge project. Updates to the perimeter barrier remedy schedule specific to the UTC project will be made, if needed, as both projects progress.

13.3 Construction Approach

13.3.1 Cost Estimates

A preliminary cost estimate to construct the containment barrier, extraction system, and AG treatment system based on the proposed Intermediate (60%) Design is between $46.8 million and $81.9 million. The preliminary cost estimate is presented in Appendix R and includes estimated annual OMM cost including performance and compliance monitoring of the barrier remedy. Appendix R also includes a preliminary bill of materials for the AG treatment system.

13.3.2 Procurement Methods and Contracting Strategy

Contractors for construction, start-up, and OMM of the 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 design-bid-build. 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). If warranted (e.g., due to need for an expedited schedule), alternate contracting strategies, such as design-build or sole-source procurement, may be adopted for certain remedial components. Under the design-build approach, contractor procurement and construction elements can be initiated during the design. In this case, USEPA would not review a final design before construction; therefore, a design review process would need to be developed and agreed upon with USEPA to allow adequate time for agency review and approval of the field design and construction submittals.

A Construction/Corrective Action Implementation Work Plan and OMM manual will be submitted as part of the Pre-final (95%) Design. The Construction/Corrective Action Implementation Work Plan will present the strategy and procedures for construction and start-up of the barrier remedy, including the overall management strategy, site management plan, H&S considerations, construction quality assurance procedures, and procedures/sequencing for construction of the remedial components, and will address contractor, labor, and equipment availability concerns. The project management strategy to the extent it is currently known is discussed in Section 13.6. A Draft CQAP is included in Appendix M.

13.3.3 Construction Sequence

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

13.3.3.1 Above-Grade Treatment System Construction

- Infinergy building modifications and selected demolition.
- Installation of helical piers and concrete maintenance pads.
- Installation of AG treatment system equipment (tanks, pumps, mixers, etc.).
- Installation of AG treatment system piping.
- Installation 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.

13.3.3.2 Extraction System and Collection Drain Construction

- Utility location.
- Sump, collection drain, and conveyance network trenching. Installation is anticipated to employ a combination of conventional trench installation methods and use of trench boxes and potentially one-pass trenching or use of a biopolymer slurry, depending on subsurface obstructions.
- Installation of collection drains and conveyance piping within trenching.
- Installation of conduit and wire for extraction system.
- Backfilling of trenches and surface restoration.
- Installation of sumps and extraction wells.
- Mechanical connection of sumps and extraction wells to the conveyance network.
- Electrical connection of sump and extraction well equipment.
- Offsite disposal of waste in accordance with the Draft Waste Management Plan (Appendix O).

13.3.3.3 Barrier Construction

- Debris removal along the headwall and anchor wall alignments where subsurface obstructions are known to exist and cannot be avoided.
- Pre-drilling to along the northern and southern sheet pile walls.
- Pre-trenching and debris removal followed by backfilling with borrow soil along the alignment of the northern and southern perimeter sheet pile walls where subsurface obstructions are known to exist and cannot be avoided, followed by backfilling with borrow soil.

- 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 offsite by steel supplier or fabricator) for the bulkhead and sheet pile walls.
- Installation/driving of bulkhead wall, anchor walls, and subgrade sheet pile walls to specified depths.
- Installation of walers and structural connection elements.
- Installation of tie rods, including soil removal/trenching along tie rod alignments.
- Installation of a cast-in-place concrete cap along the top of the bulkhead wall.
- Backfilling with aggregate fill material behind the new bulkhead wall and under the existing concrete deck.
- Installation/driving of sheet piles 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 and surface restoration materials as appropriate.
- Offsite disposal of excavated soils.
- Complete surface restoration as appropriate.

Refinement of construction methods and sequencing will continue during the Pre-final (95%) Design phase.

13.4 Phasing Alternatives

To accelerate the project, phasing alternatives will continue to be considered and evaluated to meet or fast-track the remedial strategy approach and scheduling. 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 pending EPA approval.

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 the geophysical surveys. Identified utilities include buried and overhead electric, potable water supply, river water supply, steam, 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%) Design phase, the alignments of the barrier walls, collection drains, and conveyance piping have been rerouted, 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 and collection drain. However, crucial utility services 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 offsite excavation or installation, MISS DIG will provide the permit. For onsite activity, the BASF excavation permitting process will be followed, which extensively evaluates historical information, communicates to the contractors and includes a pre-excavation conference with the construction contractor operators and foreman.

13.6 Roles and Responsibilities

The responsibilities and lines of authority of key personnel involved in the RD and implementation of the barrier remedy are summarized in the organizational chart and Table 22 below.

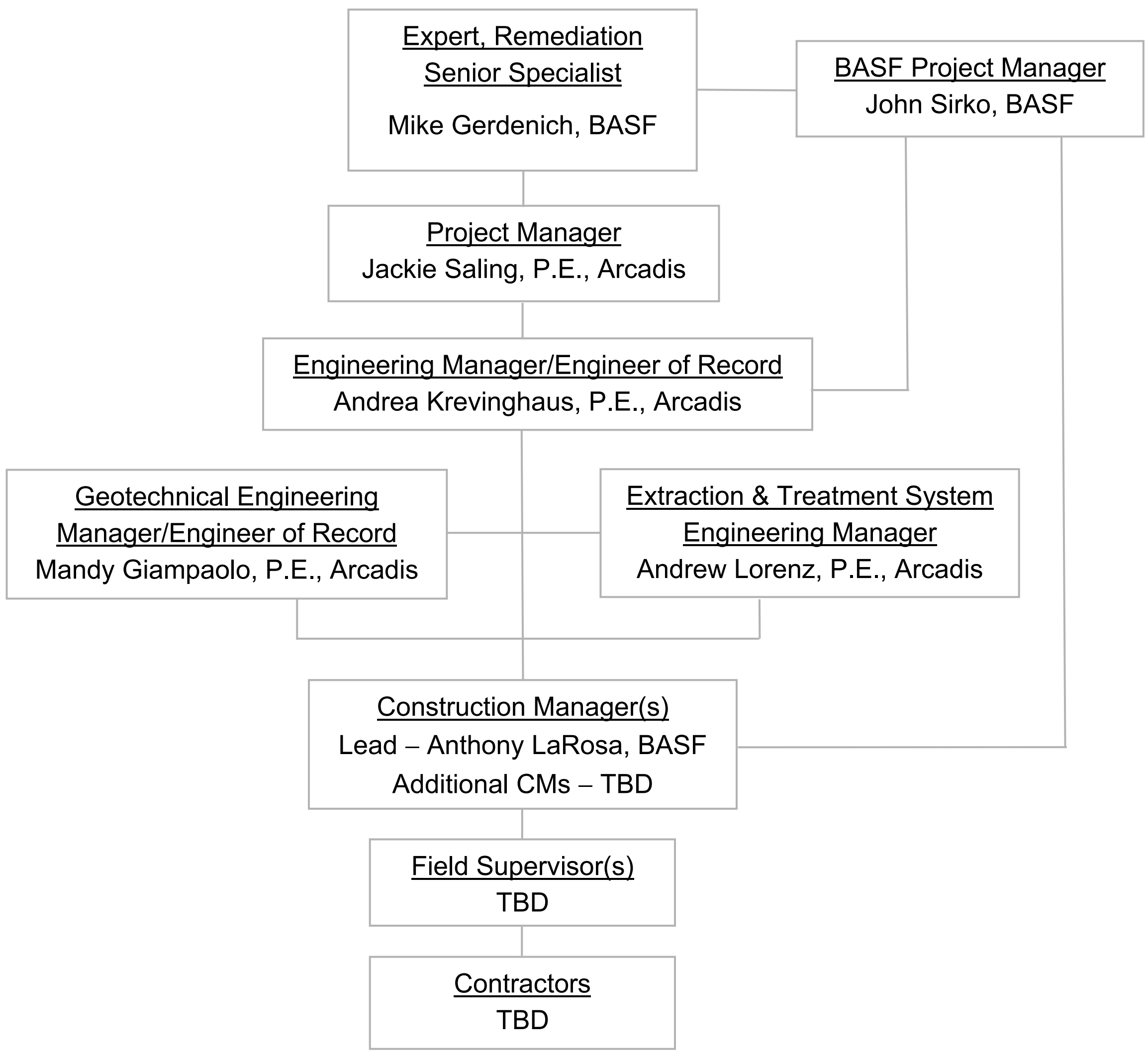


Table 22. Project Roles and Responsibilities

Name	Title	Organization
Mr. Michael Gerdenich	Expert, Remediation Senior Specialist	BASF
Mr. John Sirko	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
Ms. Oonagh McKenna	Project Manger	EGLE
Mr. Ryan Dorn, PE	Designer of Record – Process Discipline	Arcadis
Mr. Rick Hansen, PE	Designer of Record – Civil Discipline	Arcadis
Mr. Matt Lotycz, PE	Designer of Record – Structural Discipline	Arcadis
Ms. Heather Goss	Designer of Record – Architectural Discipline	Arcadis
Mr. Aaron Abb, PE	Designer of Record – Electrical Discipline	Arcadis
Mr. Kevin Erickson, PE	Designer of Record – Controls Discipline	Arcadis
Mr. Alireza Momenihezhad, PE	Designer of Record – Plumbing and HVAC Disciplines	Arcadis

13.6.1 BASF Expert, Remediation Senior Specialist and Project Manager

BASF is the Owner/Operator of the Site. The BASF Expert, Remediation Senior Specialist has overall responsibility for all aspects of the project. The BASF Project Manager (PM) is responsible for confirming that the design and construction of the barrier remedy are completed according to BASF requirements and facilitates coordination between the BASF and Arcadis engineering teams.

13.6.2 Arcadis Project Manager

The Arcadis PM is an authorized representative of the Owner/Operator and has overall responsibility for direction, execution, and successful completion of the project.

13.6.3 Engineers and Designers of Record

The Engineers of Record (EORs) have overall responsibility for the design of the barrier remedy and confirming the remedy was constructed in accordance with the design. The EORs are licensed as professional engineers (PE) of their respective disciplines within Michigan and are Vice Presidents of Arcadis. The EORs are supported by the designers of record, who are also licensed as, or under the direct supervision of, a professional engineer or registered architect within Michigan for their respective disciplines. The duties of the EORs and their supporting designers as they apply to the project include the following:

- Serve as the point of contact for field engineering inquiries and construction activities.
- Implement engineering design and specification requirements.
- Prepare markups of engineering plans and drawings as necessary to document post-construction conditions.

13.6.4 Construction Manager

The Construction Manager (CM) has primary responsibility for the implementation of field activities. Based on the size and complexity of the project, there may be multiple CMs for the different definable features of work, all of whom report to the lead BASF CM. The duties of the CM(s) as they apply to construction work include the following:

- Monitor and report the progress of work and manage project deliverables to achieve on-time completion.
- Coordinate work activities of contractors, as needed with Arcadis personnel, according to the administrative and technical requirements of the project, including procedures and applicable professional standards and construction activities.
- Adhere to the quality requirements of the design documents and the CQAP. Perform quality assurance (QA) inspections designed to confirm that the overall quality control (QC) is functioning in accordance with the CQAP.
- Serve as the primary contact among the engineering managers, field personnel, and contractors for actions and information related to the work.
- Communicate and interface with QC personnel.
- Manage data generated throughout the construction activities.
- Verify necessary permits and licenses.
- Verify QC measures are in accordance with the specified requirements.
- Perform audits and inspections of facilities, equipment, systems, record keeping, testing, sampling strategy, equipment calibration procedures, reporting requirements, compliance with QC document control procedures, and other QC measures.
- Review drawings, specifications, variance reports, design change notices, and nonconformance reports to confirm that proper QC procedures are followed.
- Review working files, QC forms, and checklists.
- Review and approve final QA/QC reports. Coordinate and maintain submittal schedule, photograph log sheet, requests for information, etc.

13.6.5 Field Supervisor

The Field Supervisor is responsible for coordinating, directing, implementing, and supervising construction activities. Based on the size and complexity of the project, there may be Field Supervisors for different aspects of the project, all of whom report to their respective CM. Specific duties of the Field Supervisor include the following:

- Manage work activities in a safe manner and in accordance with the HASP.

- Implement construction activities in accordance with the Design Drawings and technical specifications.
- Administer access to the work area.
- Coordinate and maintain logistics of all components of their tasks, including all personnel and equipment.
- Review all testing results, data, and certification report submittals.
- Perform inspections or supervise testing and sampling activities.
- Document frequency and accuracy of testing and sampling achieved.
- Prepare daily and weekly status reports and estimate future scheduling needs.
- Coordinate, prepare, and complete necessary and applicable field reports.
- Confirm compliance with the site HASP.

13.6.6 Contractor

A qualified contractor(s) will be selected to provide construction services for the project. Multiple contractors may be selected for the project, each responsible for a definable feature of the work. If multiple contractors are selected, a superintendent for each will report to the CM for the respective feature of work. Contractors, along with any subcontractors and vendors, will be required to conform to the CQAP and the requirements of all approved procedures, design drawings, technical specifications, and contract provisions. The contractor's QC inspectors are responsible for field inspection of their construction and operating activities. Construction quality assurance/construction quality control personnel will monitor, oversee, and make onsite observations and inspections of work in progress to determine whether the contractor's work is proceeding in accordance with the CQAP. Contractor personnel are responsible for maintaining a daily log of the project activities. All inspection records, including inspection reports, deficiency reports, and documentation of corrective action re-inspections, will be maintained by the contractor.

13.7 Sample and Data Collection Methodology and Quality Assurance

Sampling and analytical activities conducted have followed QA/QC 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 2020).

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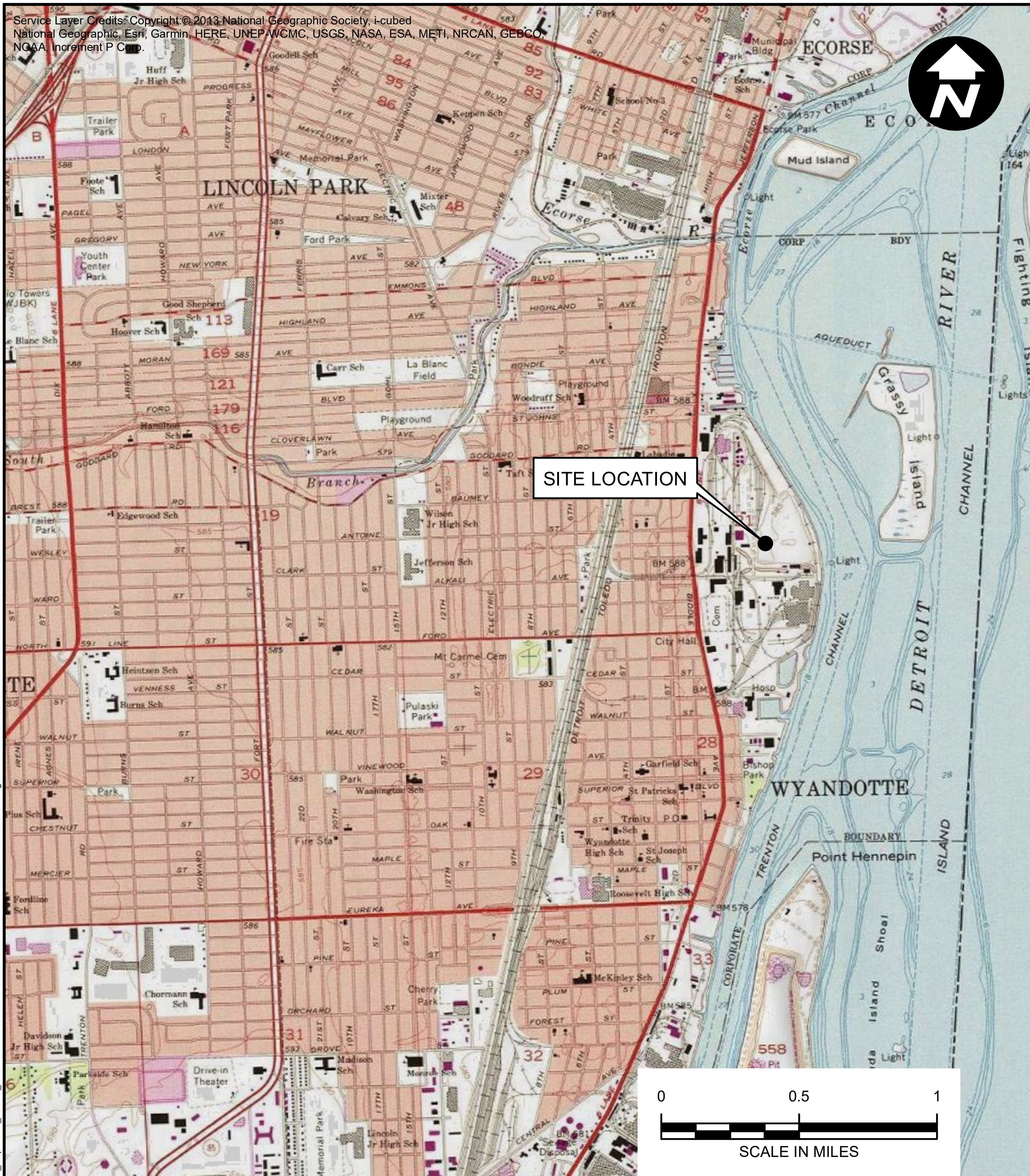
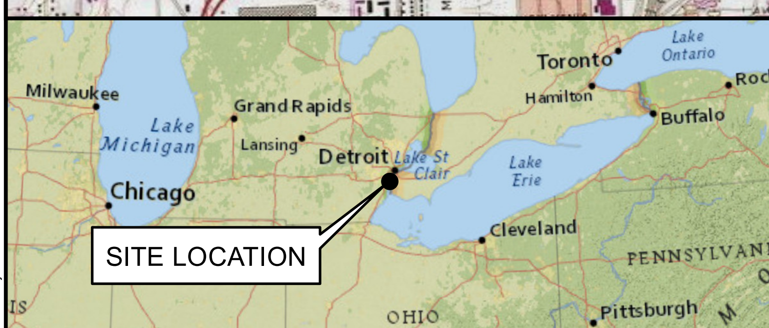
Figures





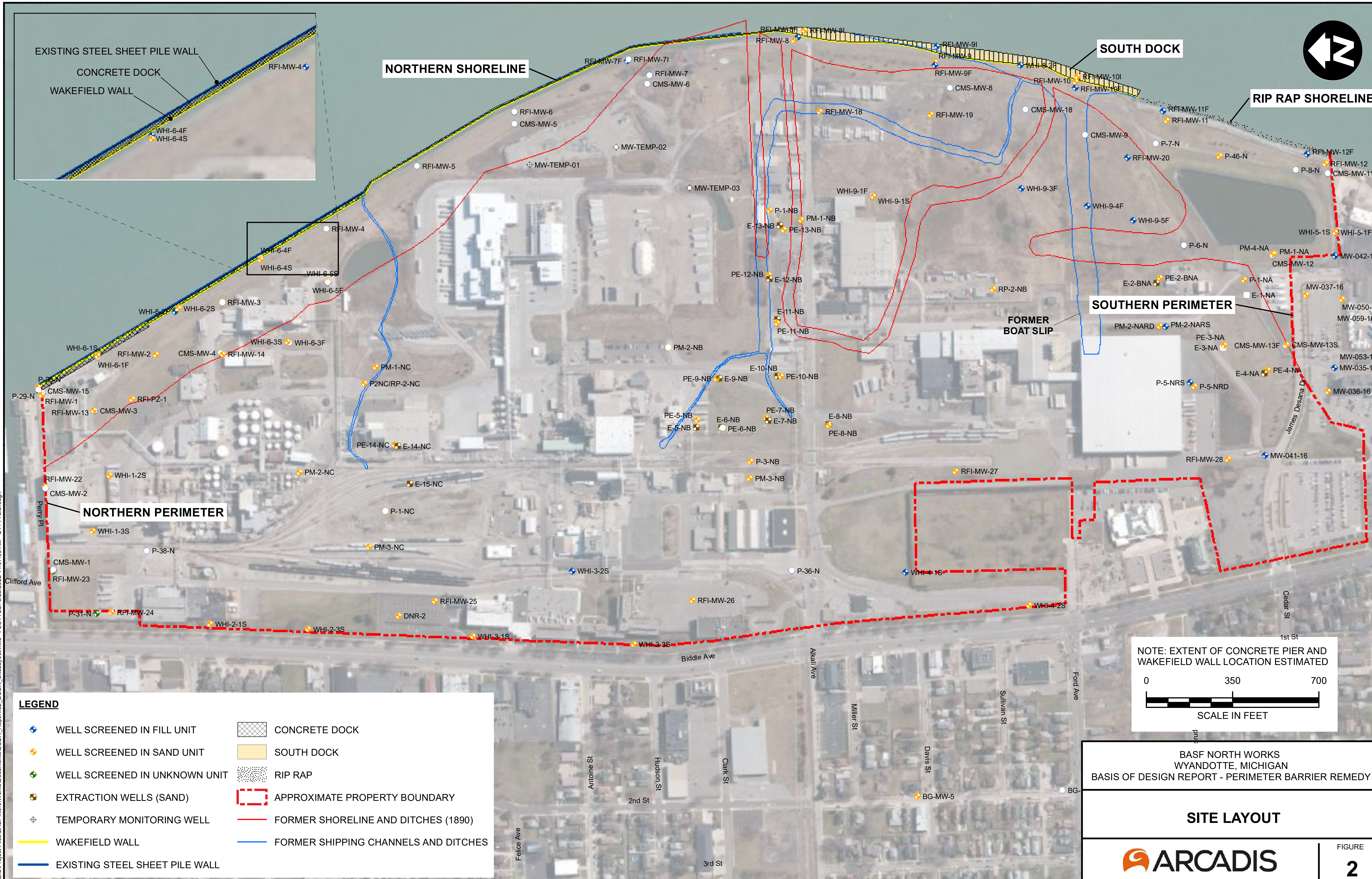
BASF NORTH WORKS
WYANDOTTE, MICHIGAN
BASIS OF DESIGN REPORT - PERIMETER BARRIER REMEDY

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
D:\GIS\Project Files\BASF\NorthWorks\Documents\BODR_Report\01_BODR_SiteLocation.mxd PLOTTED: 8/26/2022 12:24:51 PM BY: TYarbrough



LEGEND

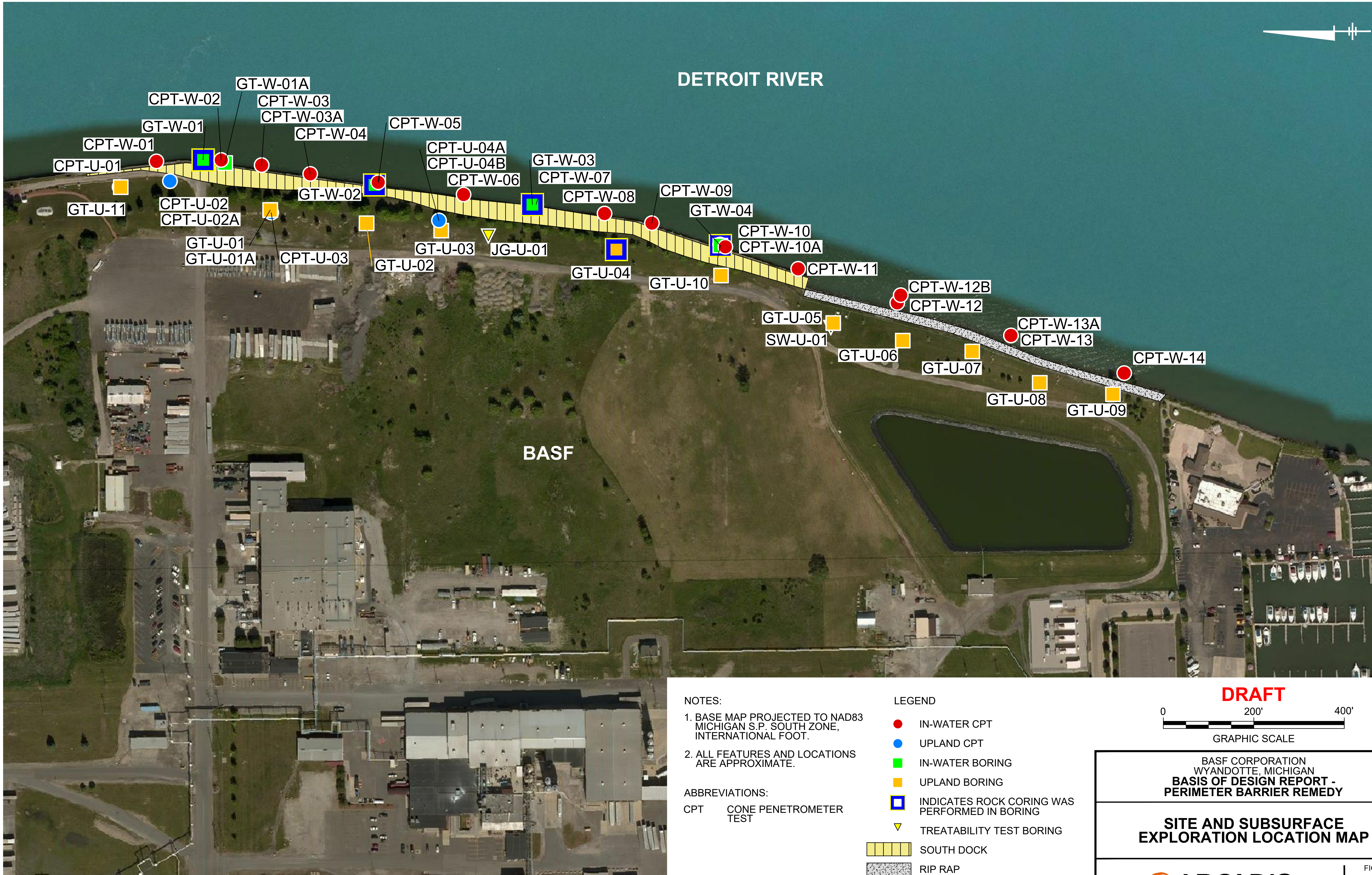
- WELL SCREENED IN FILL UNIT
- WELL SCREENED IN SAND UNIT
- WELL SCREENED IN UNKNOWN UNIT
- EXTRACTION WELLS (SAND)
- TEMPORARY MONITORING WELL
- WAKEFIELD WALL
- EXISTING STEEL SHEET PILE WALL
- CONCRETE DOCK
- SOUTH DOCK
- RIP RAP
- APPROXIMATE PROPERTY BOUNDARY
- FORMER SHORELINE AND DITCHES (1890)
- FORMER SHIPPING CHANNELS AND DITCHES

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

SITE LAYOUT

ARCADIS

FIGURE
2



NOTES:

1. BASE MAP PROJECTED TO NAD83 MICHIGAN S.P. SOUTH ZONE, INTERNATIONAL FOOT.
2. ALL FEATURES AND LOCATIONS ARE APPROXIMATE.

ABBREVIATIONS:

CPT	CONE PENETROMETER TEST

LEGEND

- IN-WATER CPT
- UPLAND CPT
- IN-WATER BORING
- UPLAND BORING
- INDICATES ROCK CORING WAS PERFORMED IN BORING
- TREATABILITY TEST BORING
- SOUTH DOCK
- RIP RAP

DRAFT

0 200' 400'

GRAPHIC SCALE

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

SITE AND SUBSURFACE EXPLORATION LOCATION MAP

ARCADIS

FIGURE
3

CITY: NOVI, MI DIV: ENV DB: G.STEINBERGER 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
PLOTTED: 8/26/2022 2:00 PM BY: STEINBERGER, GEORGE
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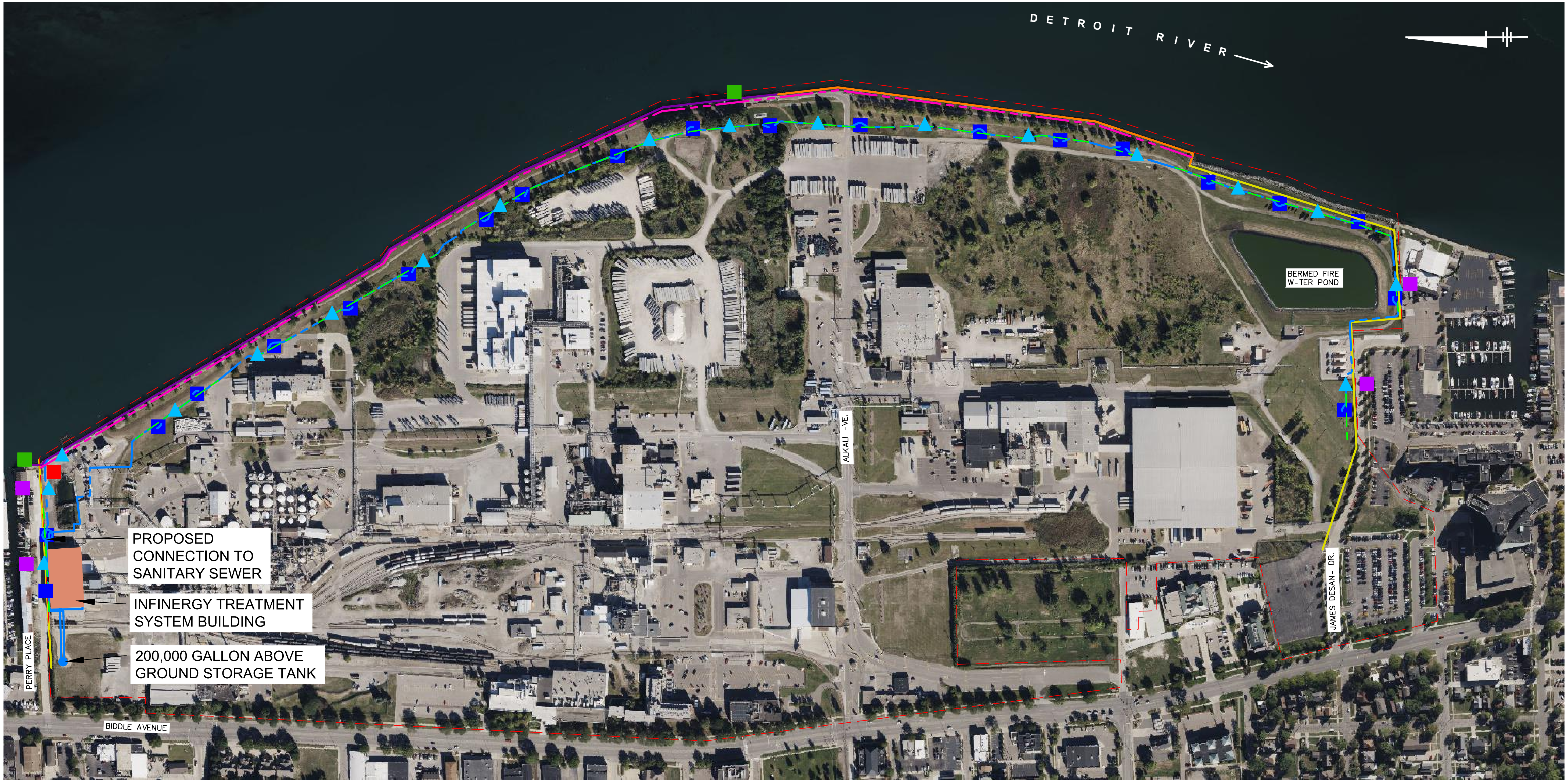

NOTES:

1. BASE MAP PROJECTED TO NAD83 MICHIGAN S.P. SOUTH ZONE, FEET.
2. AERIAL IMAGE PROVIDED BY BING © 2018 MICROSOFT CORPORATION © 2018 DIGITALGLOBE ©CNES (2018) DISTRIBUTION AIRBUS DS.

LEGEND:

- APPROXIMATE SITE BOUNDARY
- PROPOSED SHEET PILE WALL
- EXISTING STEEL SHEET PILE WALL
- WAKEFIELD WALL
- PROPOSED STEEL BULKHEAD
- PROPOSED CONVEYANCE PIPING
- PROPOSED COLLECTION DRAIN
- PROPOSED TREATMENT SYSTEM BUILDING (INFINERGY)
- PROPOSED SUMP LOCATION
- PROPOSED EXTRACTION WELL
- STILLING WELLS
- PROPOSED PIEZOMETERS (WITHIN THE COLLECTION DRAIN OR ADJACENT TO EXTRACTION WELL)
- PROPOSED PIEZOMETERS (RIVERSIDE OF SUBSURFACE BARRIERS)

0 400' 800'
GRAPHIC SCALE

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

PROPOSED BARRIER REMEDY LAYOUT

ARCADIS

FIGURE
4