



a division of Englobe

Preliminary Design Report

WATERSMART A HAZEN COMPANY

Municipal District of Greenview No. 16 Greenview Industrial Gateway

Water Intake and Raw Water Transfer Station

Date: December 18, 2024

Project No. 5946-001-00

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December 18, 2024
File: N:\5946\001-00\L01-1.0

Attention: Alison Regan, P.Eng., SCMP
Director of Projects

Dear Ms. Regan:

Re: Greenview Industrial Gateway
Preliminary Design Report – Water Intake and Raw Water Transfer Station

MPE a division of Englobe (MPE) is pleased to submit a digital copy of the above-mentioned report. We thank you for the opportunity to be of service and to have prepared this report on your behalf and look forward to assisting you in implementing your plans for the future.

If you have any inquiries regarding our report or if clarification is required, please contact the undersigned at 780-719-2986 or ikagoro@mpe.ca.

Yours truly,

MPE a division of Englobe

A handwritten signature in black ink, appearing to read "Ivan Kagoro", is written over the printed name and title.

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IK/vv
Enclosure

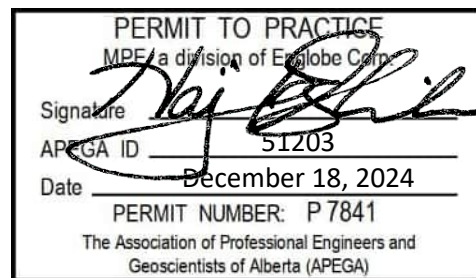
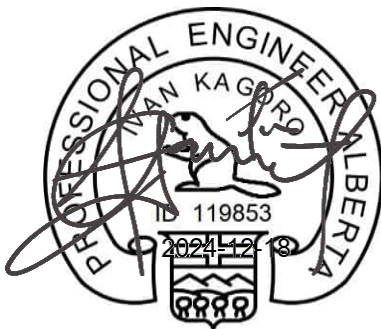
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Should any questions arise regarding content of this report, please contact the undersigned.

MPE a division of Englobe

Prepared by:



Ivan Kagoro, P.Eng.

Professional Seal

Corporate Permit

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1.0 INTRODUCTION

1.1 Purpose and Scope

MPE a division of Englobe (MPE) has developed this Preliminary Design based on the outcome and acceptance of various technical memoranda, assessments and field investigations. This report included the following:

- Prepare preliminary design drawings of the selected alternative including intake components and associated protection measures.
- Include a revegetation/restoration plan for the site.
- Prepare and submit an application package with design drawings to Alberta Environment and Protected Areas (AEPA) for Water Act and EPEA approvals.
- Prepare a disturbance footprint plan for submittal under the Historical Resources Act, compiled and submitted by others.
- Prepare a design information package for use in First Nations Consultation (FNC), by others.
- Prepare specifications identifying scope of work for the Contractor.
- Compile a complete set of 'For Approval' Documents.
- Preparation of a Preliminary Class D cost estimate.

To achieve these objectives, work was completed under several separate, yet interrelated tasks. The tasks were as developed in the project proposal and include.

- Water Intake Concept Technical Memorandum by MPE.
- Pump Station Concept Technical Memorandum by MPE.
- Preliminary Hydrotechnical Design by Northwest Hydraulic Consultants (NHC).
- Geotechnical Investigation Report – On Land by Englobe Corp.
- Preliminary Assessment of Construction Berm Hydraulics for Rock Cofferdam by NHC.
- Preliminary Assessment of Hydraulics for Sheet Pile Cofferdam by NHC.
- Initial Ice Design of an Ice Platform for Geotechnical Drilling in the River by NHC.

The interrelationships among the various reports are notable. The preliminary hydrotechnical design formed the basis for the water intake concept design and location of the pump station concept. The on-land geotechnical report provides conceptual requirements for the pump station concept construction

requirements and the intake civil/landscape finishes. NHC also completed preliminary assessments of the hydraulics associated with a rock or sheet pile cofferdam. This winter, river geotechnical investigations are being conducted to confirm the most feasible cofferdam option and the initial ice platform design provides requirements for completing this investigation.

This preliminary design report includes all the aforementioned reports appended for reference.

1.2 Water Demand

The water demand for the Greenview Industrial Gateway was estimated for each lot as 10,000 m³ per day based on similar industrial use in the area. There are currently a total of six lots proposed that will be serviced over three phases. The storage ponds require a minimum of 18-weeks (4.5 months) of storage assuming raw water withdrawal from the Smoky River would not be possible during this period due to low flow conditions. The corresponding total storage capacity required is 7,560,000 m³, or 2,520,000 m³ per phase. The total estimated annual water demand for all six lots is projected to be 21,840,000 m³. The maximum daily withdrawal from the Smoky River is 109,728 m³/d (1,270 L/s) with a maximum annual withdrawal of 24,000,000 m³.

Each phase will be designed for a flow rate of a Design Flow Rate of 355 L/s for the ultimate flow of 1,065 L/s.

1.3 System Overview

The intake and pumping system will generally comprise of the following:

- A four-chamber intake structure embedded in the Smoky River banks.
 - Each chamber will have an intake port fitted with bar screens and an isolation sluice gate.
- A fish screen in each chamber and conveyance piping to a common wetwell.
- Erosion protection at the intake and opposite banks.
- An underground wetwell chamber and aboveground pump station.
- Four vertical turbine pumps each capable of 355 L/s.
- A pump station building housing the pumping system, and electrical equipment.
- Road access from the pump station and intake.

2.0 PROCESS MECHANICAL

2.1 Design Standards & References

All process equipment design shall conform to all applicable local, provincial, and/or federal codes, standards, regulations, and references in effect at the time of quote including but not limited to:

- Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage, Alberta Environment and Parks; 2013.
- Wastewater Systems Standards for Performance and Design; Alberta Environment and Parks; 2013.

The design will comply with the requirements of the following organizations at a minimum:

- CSA, Canadian Standards Association.
- NEC, National Electric Code.
- NEMA, Standards of National Electrical Manufacturers Association.
- ANSI, American National Standards Institute.
- ASTM, American Society for Testing and Materials.
- AISI, American Iron and Steel Institute.
- AGMA, American Gear Manufacturer's Association.
- AISC, American Institute of Steel Construction.
- AWS, American Welding Society.
- ASME, American Society of Mechanical Engineers.
- AWWA, Applicable Standards.
- CSA, Canadian Standards Association.
- CEC, Canadian Electrical Code.
- IEEE, Institute of Electrical and Electronic Engineers.
- EEMAC, Electrical and Electronic Equipment Manufacturers Association of Canada.

2.2 Raw Water Intake

The following provides a general description of the major process mechanical components of the raw water intake.

2.2.1 Trash Bars

Each intake chamber inlet port will be equipped with a trash bar to screen out large debris and large fish from entering the chamber. Specifications for the trash bars are summarized in **Table 2.1**.

Table 2.1: Trash Bar Specifications

Parameter	Specification
Bar Orientation	Horizontal
Bar Material of construction	Stainless Steel
No. of Bar Screens	4
Bar Thickness	20 mm (3/4")
Bar Spacing	75 mm (3")
Port Opening	3.1m Long X 1.3m Wide
Minimum Effective Area	3.98 m ²
Maximum Approach Velocity	0.098 m/s

2.2.2 Fish Screen System

Each intake chamber will be equipped with a fish screen designed to meet Fisheries and Oceans Canada (DFO) requirements for safely shielding fish from entering the wetwell. The two main components of the fish screen system include the screen and the air backwash system. Specifications are provided in **Table 2.2**.

Table 2.2: Fish Screen System Specifications

Parameter	Specification
Screen	
Location	Intake
Type	Cylindrical
Number of Screens	4
Slots	0.254mm
Material of Construction	304 stainless steel
Maximum Approach Velocity	0.035 ft/s
Design Flow Rate	355 L/s
Installation Depth	Approx 7.5 m
Collapse Rating	10.66 psi
Outlet	AWWA C-207 Class D Flange
End Connection	Endplate x Endplate
Effective Screen Area	11.57 m ²
Open Area	6.77 m ²
Size	1.07 m diameter x 3.8 m long
Air Backwash System	
Location	Pump Station (Indoors)
Capacity	78 cfm @ 175 psi
Distance between Air Receiver and Screen	Approx. 50 m
Airline Size	100 mm (4")
Air Line Material	Schedule 10 Stainless Steel
Installation Depth	7.5 m
Maximum Air Receiver Recharge Time	15 minutes
Compressor Type	Rotary Screw
Power	600V/3pH/60 Hz
Motor	15 HP TEFC Electric
Enclosure	Sound proof to 67 dBa
Controls	Controls to be performed by Main PLC No integra control panel required Control logic to be provided for integration into main PLC
Mode of Operation	1. Manual start of air burst system to run for specified time period 2. Automatic operation frequency and duration all adjustable via the Main control panel
Accessories	Four On/Off Actuated Butterfly Control Valve Pressure Transducer Pressure Gauge Air Filter/Regulator

2.2.3 Fish Return System

Fish smaller than the trash bar spacing of 75 mm (3") would enter the intake chamber and be guided by a crowder system to a fish pump that would pump the fish through a pipeline to the downstream end of the intake. Specifications for the Fish Return system are provided in **Table 2.3**.

Table 2.3: Fish Return System

Parameter	Specification
Type	Submersible Fish Friendly Axial or Bowl Pumps
Design Flow	35.5 L/s
Design Head	TBD
Suction Size	TBD
Discharge Size	TBD
Minimum Free Bowl Passage	100 mm (4")
Impeller Type	Screw
Power Requirements	600V/3pH/60 Hz
Speed	800 rpm
Material of Construction	304 Stainless Steel
Enclosure	IP 68
Start Method	Soft Start
Installation	Vertical
Discharge Piping Material	304 SS
Controls	Via main PLC
Mode of Operation	Pumps to run continuously during operation of the corresponding intake screen.

Design of the crowder system would require fluid dynamics modeling using a physical laboratory model that should be completed at the commencement of detailed design.

2.2.4 Sluice Gates

Sluice gates will be mounted on the inside of the port opening at each intake chamber to allow for isolating the river for maintenance or downtime. Specifications are summarized in **Table 2.4**.

Table 2.4: Sluice Gate Specifications

Parameter	Specification
Type	Self-contained frame, double rising stem, AWWA C561-14
Design Flow	35.5 L/s
Dimensions	3.1 m long by 1.3 m wide
Material of Construction	316 Stainless Steel
Installation Depth	2.94 m
Mounting	Wall Thimble
Operator	Yoke mounted tandem gearboxes and handwheel

2.3 Pumping Station

The following provides a general description of the major process mechanical components of the pumping station.

2.3.1 Raw Water Pumps

Proposed raw water pumping for the GIG project consists of four (4) identical 1,250 HP vertical turbine pumps, as outlined in Technical Memorandum #3. The pumping capacity is such that, with one pump out of service, the remaining pumps can handle peak flow requirements. At full build-out, it is expected three pumps will be staged accordingly to operate during the reservoir fill period.

Each pump would be sized to provide 355 L/s and controlled by variable frequency drivers (VFDs). The required head of the pumps and corresponding sizing would need to be revised during detailed design once a new location of the raw water ponds is established and the raw water pipeline alignment and profile finalized. Following that, initial pump selections and subsequent analysis can be conducted (i.e. system curve, net positive suction head available, variable speed curves, intake, and transient).

General specifications for the raw water pumps are summarized in **Table 2.5**.

Table 2.5: Raw Water Pumps

Parameter	Specification
Installation Type	Vertical Turbine
Shaft Diameter	400 mm (16")
Impeller Material	316 SS
Design Flow	355 L/s
Design Head	194 m (TBC)
Efficiency	TBD
Motor Type	Vertical Hollow Shaft
Motor Size	1250 HP (TBC)
Power	600V/3ph/60 Hz
Accessories	Inverter Duty, Premium Efficiency
Starting Method	VFD

2.3.2 Piping & Valving

General specifications for the process piping and valving are presented in **Table 2.6**.

Table 2.6: Piping & Valving

Parameter	Specification
Pipe Maximum Velocity	3 m/s
Pipe Material	Sch. 40 A106B Seamless Carbon Steel
Pump Discharge Piping	Size: 400 mm dia. Check Valve Pump Control Valve Isolation butterfly valve
Transmission Main Header	Size: 900 mm dia. Air Release Valve
Hydraulic Backwash Header	Size: 900 mm dia. Pressure control valve Check valve Butterfly valve
Hydraulic Backwash Supply Lines	Four (4) 900mm dia. main header Each with flow control valve
Wetwell Inlet Piping	Knife Gate Valve
Flanges	ASME B16.5 Class 3004

3.0 CIVIL

3.1 River Intake Isolation – Cofferdam

Isolation of portion of the Smoky River is required to allow for construction of the intake structure and piping either using a rockfill cofferdam or a sheet pile cofferdam. Englobe Corp. is conducting river geotechnical works in February 2025 to further the design of the cofferdam.

For this pre-design report, a rock cofferdam is assumed in conjunction with Technical Memorandum #2 and berm hydraulic analysis, and will have the following characteristics:

- Material will be a mixer of riprap and small well grained rock.
- Top of berm elevation of about 577.23 m
- The inside and outside side slopes would be a minimum of 2:1.
- The total width of the cofferdam top of berm would be a minimum of 6.0 m.
- Minimum distance between the inside toe of the cofferdam and the intake structure will be about 8.0 m.
- The wet side of the cofferdam will be armored with riprap that has a median diameter of 0.8 m and a thickness of 1.5 m.
- A 2.0 m thick riprap apron will be placed at the base to the riprap slope extending out 5.6 m to protect against scour.
- A non-woven geotextile will be placed under the riprap slope.
- Erosion protection with Class 3 rock riprap on the exposed surface of the cofferdam.
- Temporary Erosion protection with Class 2 rock riprap on the east bank on the opposite side of the intake.

3.2 Intake Structure

Civil siteworks for the intake structure would include the following.

- Permanent erosion protection with Class 2 rock riprap on east bank opposite of the intake structure.
- Permanent scour erosion protection with Class 3 rock riprap around the intake structure base.
- Permanent erosion projection protection around the banks upstream and downstream of the intake structure.

- Imported fill from the raw water pond site would be used to fill the space between the intake and the bank up to the intake access elevations.
- The top of the fill will be designed to have a smooth cemented surface sloped towards the river and act as access for the intake structure.

3.3 Pump Station

Civil siteworks for the intake structure would include the following.

- Access road to the site is proposed to be 8 m wide gravel road.
- Drainage around the pump station will be positive with a minimum of 2% towards the river and designed to *AEPA Stormwater Management Guidelines* (1999).
- A designated parking lot will be located directly west of the new facility and will also be granular surfaced. The lot will be sufficiently sized to accommodate trucks as well as provide parking for several passenger vehicles.
- To provide adequate site security, a 1.8-meter-high perimeter chain link fence fitted with security wire will enclose the pump station.
- An electric cantilevered gate will be located at the access point and will be locked when maintenance personnel are not present.
- The fence will also include single man gates and one 7.3 m (24') cantilever gate for delivery truck access.

4.0 ARCHITECTURAL

4.1 Design Standards & References

The design of the building for the proposed Raw Water Transfer Station will follow the following codes and standards.

- National Building Code – 2019 Alberta Edition (ABC).
- National Energy Code of Canada for Buildings 2017 (NECB).

4.2 Building Code Review

Preliminary Building code review is summarized in **Table 4.1** in accordance to the NBC(AE), fire protection, occupant safety and accessibility Division B Part 3 of the NBC(AE).

Table 4.1: Preliminary Building Code Review

Code Review	Comments	Reference
Applicable Code	National Building Code: Alberta 2019 Edition	Division B, Part 3
Importance Category	Post-Disaster	Div. B, Part 4 Table 4.1.2.1
Major Use and Occupancy	Group F Division 3, up to 2 Storeys	3.2.2.85
Construction	Combustible	3.2.2.85
Sprinklering	Not Sprinklered	3.2.2.85
High Buildings Classification	Not applicable	3.2.6.1
Fire Suppression		
Sprinkler System	Not required	3.2.2.85
Standpipe	Not required	3.2.5.8
Fire alarm	Not required	3.2.4.1
Fire Ratings/Separation		
Floor Assemblies	There will be fire separations and, if of combustible construction, shall have a fire-resistance rating not less than 45 min.	3.2.2.85
Roof Assemblies	No requirement	3.2.2.85
Supporting Assemblies	loadbearing walls, columns and arches supporting an assembly required to have a fire-resistance rating shall have a fire-resistance rating not less than 45 min, or be of non-combustible construction	3.2.2.85
Service Rooms (Gas Fired)	A fire separation having a 1-hour fire resistance rating	3.6.2.1.1
Exits		
Maximum Travel Distance		
Served by 2 Exits	30 m	3.4.2.5.1f
Served by 1 Exit	15 m	Table 3.4.2.1.-A
Mezzanine	15 m	3.4.2.2.2
Accessibility Design	Not required	3.8.2.1

4.3 Building Envelope

The building envelope for the proposed raw water pump station is to be designed to meet the requirement of the NECB Division B, Part 3.

For this project, the following building envelopes are proposed and its proposed Overall Thermal Transmittance (U-Value) of the building envelope:

Table 4.2: Proposed Building Envelope U-Values

Component	Exterior Wall Construction Assembly	Roof Construction Assembly
Proposed Assembly	100 x 200 x 400 Concrete Block Wall (Standard Gray)	Ceiling Finish – 28 ga. Prefinished metal liner
	38 Air Space	12 mm Plywood sheathing
	100 Type 2 EPS Insulation	150 Micrometre Poly Vapour Barrier
	Peel and Stick Air and Vapour Seal	400 mm thick Loose-Fill Insulation
	200 x 200 x 400 Concrete Block Wall (Painted)	Pre-Engineered Wood Trusses
	Strapping and Drywall finish (Lab and Control Room)	16mm Plywood sheathing
		Underlayment
		24 Gauge Standing Seam Metal Roof
U-Value	0.24 W/(m²-K)	0.136 W/(m²-K)

The U-Value of the proposed wall construction assembly does not meet the requirement for the Prescriptive Path in accordance to the NECB. Simple Trade-off Path Method will be used to comply with the NECB. The remaining building components including fenestrations (windows, louvers, etc.) and doors will meet the following requirements:

- Maximum Overall U-value of Fenestrations = 2.2 W/(m²-K).
- Maximum Overall U-value of Doors = 2.2 W/(m²-K).

4.4 Building Finishes

The proposed building finishes are chosen based on MPE's experience for similar buildings and aim to balance durability, aesthetic, and cost.

4.4.1 Exterior Finishes

MPE proposes the following finishes for the exterior of the building:

- **Walls:** Concrete block wall, split face. Colour to be chosen by the Client.
- **Roof:** 24-gauge prefinished standing seam metal roof. Colour to be chosen by the Client.
- **Doors:** Metal door painted to colour chosen by the Client.
- **Fascia, soffit:** Prefinished metal.

4.4.2 Interior Finishes

MPE proposes the following Interior finishes.

Table 4.3: Proposed Building Interior Finishes

Room	Floor	Base	Walls	Ceiling
Control Room	Epoxy	Rubber Base	Drywall	Ceiling Tiles
Pump House	Epoxy	Epoxy	Block Wall/Painted	Prefinished metal liner
Electrical Room (Low Voltage)	Epoxy	Epoxy	Block Wall/Painted	Prefinished metal liner
Electrical Room (Medium Voltage)	Epoxy	Epoxy	Block Wall/Painted	Prefinished metal liner

5.0 STRUCTURAL

5.1 Codes and Standards

The structural design of the RWWTF will conform to the following Codes and Standards:

- National Building Code - Alberta 2019 Edition (NBC(AE)).
- Structural Commentaries (User's Guide – NBC 2015, Part 4 of Division B).
- CAN/CSA A23.3-14, Design of Concrete Structures.
- CAN/CSA O86-14, Engineering Design in Wood.
- CAN/CSA S304-14, Design of Masonry Structures.
- CAN/CSA S16-14 Design of Steel Structures.
- CAN/CSA S157-05 Strength Design in Aluminum.
- CISC/ICCA 2013, Crane-Supporting Steel Structures: Design Guide.

For the purpose of determining specified snow, wind, and seismic loads, buildings shall be assigned an Importance Category based on intended use and occupancy. The pump station's primary use is raw water pumping; it is considered as a building that is essential in the event of a disaster. In accordance with NBC(AE) Division B Part 4 Table 4.1.2.1., the building is categorized as **Post-disaster**.

5.2 Design Loads

Design loads include dead loads, live loads, snow loads, wind loads, seismic loads, and lateral earth pressure. Loads are determined based on the building material and the use of the building. The following sections will outline the loads that will be used for the design of the proposed pump station in accordance with the following requirements:

- National Building Code - Alberta 2019 Edition (NBC(AE)), Part 4 of Division B.
- Structural Commentaries (User's Guide – NBC 2015, Part 4 of Division B).

Dead loads are gravitational loads including the self-weight of the structure, finishes and basic mechanical and electrical components inside the building and will vary depending on the material.

Live loads are used and occupancy loads based on the assigned usage of the spaces within the building. Minimum live loads are provided for common usage in Table 4.1.5.3 of the NBC(AE). All equipment loads and water inside all the concrete tanks will also be considered as live loads for this project.

Snow load is calculated based on the climatic data provided in Division B, Appendix C Climate and Seismic Information for Building Design in Alberta Table C-2 of the NBC(AE) for Greenview, Alberta. The following parameters will also be used to determine the design snow load for the building.

Table 5.1: Building Snow Loads

	Ultimate Limit State Design		Service Limit State Design	
	Full Load Case	Partial Load Case	Full Load Case	Partial Load Case
Ground Snow Load, S_s	1.9 kPa			
Associated Rain, S_r	0.1 kPa			
Importance Factor, I_s	1.25		0.9	
Basic Roof Snow Load Factor, C_b	0.8			
Wind Exposure Factor, C_w	1	1	1	1
Slope Factor, C_s	1			
Accumulation Factor, C_a	1	1.17	1	1.17
Design Roof Snow Load, S	2.0 kPa	2.3kPa	1.5 kPa	1.7 kPa

In addition to the basic design roof snow load, for multi-level roofs, drifting load of snow on a roof adjacent to a higher roof shall be taken as trapezoidal, in accordance with Section 4.1.6.5 of the NBC(AE).

Wind loads for the Greenview will be determined using the Static Procedure in accordance with Section 4.1.7.3 of the NBC(AE). The specified external pressure or suction due to wind is also based on the climatic data provided in Division B, Appendix C Climate and Seismic Information for Building Design in Alberta Table C-2 of the NBC(AE) for Greenview, Alberta. The following parameters will also be used to determine the design wind load for the building.

Table 5.2: Building Wind Loads

	Ultimate Limit State Design				Service Limit State Design			
	Roof		Wall		Roof		Wall	
Hourly Wind Pressure, q	0.36 kPa							
Importance Factor, I _w	1.25				0.75			
Exposure Factor, C _e	0.92							
Topographic Factor, C _t	1							
C _p C _g	-2	-1.3	1.46	0.97	-2	-1.3	1.46	0.97
External Wind Pressure (kPa)	-0.83	-0.54	0.61	0.4	-0.5	-0.32	0.36	0.24
Internal Wind Pressure (kPa)	0.25		-0.37		0.15		-0.22	
Net Specified Pressure (kPa)	-1.08	-0.79	0.98	0.77	-0.65	-0.47	0.98	0.78

Earthquake Loads: The deflections and specified loading due to earthquake motions will be determined according to the requirements of Section 4.1.8.2 to 4.1.8.22 of the NBC(AE). Earthquake load for the pump station and intake structure shall be based on the seismic data provided in Table C-3 Seismic Design Data for Selected Locations in Alberta, in Division B, Appendix C of the NBC(AE) for Greenview, Alberta.

5.3 Water Intake Structure

- The proposed intake preliminary structure design will be constructed with cast in place concrete. The upcoming geotechnical investigation of the subsurface conditions of the Smoky River will confirm the type of foundation suitable to avoid uplift and sliding.
- The design will account for the total intake face length of 19.2 m which includes a 3.0 m leading edge..
- The overall footprint of the concrete intake structure is approximately 126 m² which is a length and width of 14.0 m and 9.0 m, respectively.
- Additionally, there will be concrete walls that extend from the south side and north side of the structure towards the bank of the river.
- Fill material will be used to cover the structure up to the access hatch elevation and outward to the concrete walls. This will allow for ease of access to complete maintenance and will lessen the steepness of the nearby slope.
- The proposed concrete mix design for the Intake structure is to meet the requirement for exposure class of F-1 according to CSA A23.1, Concrete Materials and Methods of Concrete

Construction, where concrete is exposed to freezing and thawing in a saturated condition, but not to chlorides.

- However, if further geotechnical investigation determined that the soil surrounding the intake structure contains excessive sulphate content, concrete mix design to meet the exposure class of S-1 or S-2 will be required. “S” classes are designed to resist sulphate attack of the concrete structure.

5.4 Pump Station

The proposed pump station will consist of underground reinforced concrete wetwell structure supported by a shallow foundation system, completed with reinforced concrete slabs. The superstructure is proposed to be of pre-engineered wood trusses supported by reinforcement concrete block walls on top of the main floor concrete slab. The following provide characteristics for the structural portion of the pump station.

- The proposed wet well would have a floorplan area of approximately 130 m² and provide retention of approximately 650 m³ to 1500 m³ at minimum and maximum water levels, respectively.
- Bottom slab elevation and top slab elevations are established at 567.14 m and 581.50 m geodetic, respectively. This will give a total approximate wet well depth of slightly more than 14.4 m.
- A minimum 2% slope from the intake outlet to the wetwell inlet shall be maintained.
- Within the rectangular wetwell there are four (4) proposed pumping bays that will be separated by partitions.
- The bottom slab of the wetwell will also be sloped towards the pumping bays to aid in passing sediment.
- Accommodation can be made for a shallow wash out sump and solids handling pump to accommodate cleanout and maintenance.

A CFD modelling will be completed as part of detailed design of the pump bays and wetwell. The CFD modelling should include phasing scenarios to determine the fluid dynamics as additional pumps are added.

6.0 HEATING, VENTILATION & AIR CONDITIONING (HVAC)

6.1 General

The proposed pump station heating, ventilation, and air conditioning system will be comprised of a gas fired forced air furnace, heat-recovery ventilation system, exhaust fans, and associated louvres and dampers.

6.1.1 Design Standards & References

The design shall conform to all applicable local, provincial, and/or federal codes, standards, regulations, and references in effect at time of quote including but not limited to:

- Alberta Building Code (including National Energy Code); 2019.
- Alberta Fire Code; 2019.
- Safety Code Act.
- Occupational Health & Safety Act.
- Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems; (2012).
- National Plumbing Code of Canada (2015) and as amended by the Province of Alberta Regulation.
- Occupational Health and Safety Regulation.

The design will comply with the requirements of the following organizations, at a minimum:

- CSA, Canadian Standards Association.
- NEC, National Electric Code.
- NEMA, Standards of National Electrical Manufacturers Association.
- ANSI, American National Standards Institute.
- ASTM, American Society for Testing and Materials.
- AISI, American Iron and Steel Institute.
- AGMA, American Gear Manufacturer's Association.
- AISC, American Institute of Steel Construction.
- AWS, American Welding Society.
- ASME, American Society of Mechanical Engineers.
- AWWA, Applicable Standards.

- CSA, Canadian Standards Association.
- CEC, Canadian Electrical Code.
- IEEE, Institute of Electrical and Electronic Engineers.
- EEMAC, Electrical and Electronic Equipment Manufacturers Association of Canada.
- NFPA (National Fire Protection Association).
- ACGIH (American Conference of Governmental Industrial Hygienists).
- American National Standards Association – International Safety Equipment Association.
- SMACNA (Sheet Metal and Air Conditioning Contractors' National Association).

The design will comply in particular with the following standards and manuals, at a minimum:

- ASHRAE 62.1, Ventilation for Acceptable Indoor Air Quality; 2016.
- ASHRAE 55, Thermal Environmental Conditions for Human Occupancy, 2017.
- ACGIH Industrial Ventilation: A Manual of Recommended Practice for Design; 2016.
- ANSI/SMACNA 006, Duct Construction Standards Metal and Flexible; 2006.
- ANSI/ISEA Z358.1, Emergency Eyewash and Shower Equipment; 2014.
- NFPA 10, Standard for Portable Fire Extinguishers; 2013.
- NFPA 90.A, Standard for the Installation of Air-Conditioning and Ventilating Systems; 2015.
- NFPA 820, Standard for Fire Protection in Wastewater Treatment & Collection Facilities; 2016.

6.2 Design Basis

The following will be followed for the design of the building mechanical systems for the pump station.

- Heating in all the rooms, including the control room, electrical rooms and pump room will be performed by a forced air furnace.
- A forced furnace complete with a heat recovery ventilator will perform ventilation in occupied areas (i.e. the control room).
- Ventilation systems will maintain pressure gradients between the process areas and the occupied areas of the building.
- A potable water cistern will be provided to service sinks and toilets.
- Floor drains and vents will be gravity and sized to applicable codes. Drains will be routed to the wetwell.

- Potable water distribution service fire protection is not available. Therefore, the proposed pump station will not be fitted with a sprinkler system. Fire extinguishers located in accordance with the ABC, AFC, NFPA 10, and other applicable NFPA standards will provide fire protection. The pump room will use 10 kg, cartridge operated, A:B:C dry chemical extinguishers while the electrical rooms will be fitted with 10 kg, pressurized CO2 extinguishers.

7.0 ELECTRICAL

7.1 General

The electrical and controls preliminary design summarizes the proposed design basis for the electrical servicing, emergency power generation requirements, and plant controls for the GIG project.

7.1.1 Design Standards & References

The design shall conform to all applicable local, provincial, and/or federal codes, standards, regulations, and references in effect at time of quote including but not limited to:

- Alberta Building Code (including National Energy Code); 2014.
- Alberta Fire Code; 2014.
- Safety Code Act.
- Occupational Health & Safety Act.
- Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems; (2012).

The design will comply with the requirements of the following organizations, at a minimum:

- CSA, Canadian Standards Association.
- NEC, National Electric Code.
- NEMA, Standards of National Electrical Manufacturers Association.
- ANSI, American National Standards Institute.
- ASME, American Society of Mechanical Engineers.
- CSA, Canadian Standards Association.
- CEC, Canadian Electrical Code.
- IEEE, Institute of Electrical and Electronic Engineers.
- EEMAC, Electrical and Electronic Equipment Manufacturers Association of Canada.
- NFPA (National Fire Protection Association).

The design will comply with the following standards and manuals at a minimum:

- NFPA 820, Standard for Fire Protection in Wastewater Treatment & Collection Facilities; 2016.

7.2 Electrical Servicing

The proposed Raw Water Transfer Station will be supplied by a single 25 kV High Voltage Utility feed into a customer-owned outdoor switch yard. The switch yard will contain the following:

- A 25 kV service entrance switchgear.
- Two (2) pad-mounted 25 kV:5 kV power distribution transformers (one [1] duty, one [1] redundant).
- A 25 kV: 600V step down transformer.
- An exterior 300 kW (approximate) outdoor standby generator located within the fenced compound but outside of the switch yard. The standby generator will be sized to satisfy essential building loads only and will not be sufficient to power the main process equipment. It is assumed that a temporary operational shutdown will be acceptable during utility power loss.

The Medium Voltage (MV) electrical room located inside the Raw Water Transfer Station will contain the 5 kV Main-Tie-Main switchgear and VFDs for powering the distribution pumps. The 5 kV switchgear will be designed to accommodate full future buildout of the station to be fed from either 25 kV transformer with two (2) distribution pumps fed from each of the respective main busses. A Main-Tie-Main electrical service allows for the power for the distribution pumps to be fed from either 25 kV transformer simultaneously, or individually. During maintenance, a Main-Tie-Main electrical service also allows for half of the electrical distribution system to be isolated and shut off for maintenance, while the other half remains energized and operational. This allows for planned maintenance to be performed without needing to fully shut down the Raw Water Transfer Station.

Each pump will include a motor protection relay for monitoring and safety interlocks ensuring the motor is not run outside of safe operating conditions. Each VFD will be specific to have an “active front end” or 24 pulse configurations, ensuring harmonics are limited to well within the IEEE 519 recommended requirements.

A separate Low Voltage (<750 V) electrical room will house the 600 V and 208 V distribution equipment for Raw Water Transfer Station building loads such as HVAC, smaller pumps, compressors, HVAC, along with network equipment and control panel. The outdoor standby generator will be connected to an

automatic transfer switch located in the electrical room to provide standby power for the 600 V and 208 V distribution.

The wiring will be primarily run using RPVC, rigid metal, or armored teck/tray cable run in cable tray. Where possible, conduit will be run hidden in walls, below grade, or embedded in-slab.

Lighting will be low maintenance, energy efficient LED fixtures throughout the facility. Interior lighting levels will be to Illuminating Engineers Society (IES) Standards and the National Energy Code for Canada for Buildings with a minimum of 250 lux in service rooms and 100 lux in plant rooms and an ideal level of 500 lux in these areas. Emergency lighting will be provided throughout. External security lighting will be controlled by astronomical time clocks. Exterior process lighting will be controlled by personnel operated switches.

Smoke and heat detectors will be provided in the electrical and blower rooms for onward transmission to the SCADA system. A full fire alarm system is not proposed.

8.0 CONTROLS & AUTOMATION

The proposed Raw Water Transfer Station will house the control system for the raw water intake and transfer pumping system. A PC in the control room is proposed to act as the primary operator's interface and will house the Supervisory Control and Data Acquisition (SCADA) system. The SCADA system would also be accessible remotely through a secure cellular or satellite-based internet connection.

The Raw Water Transfer Station and Raw Water Intake equipment will be hard-wired and controlled via a programmable logic controller (PLC) in the main control panel. Because of the remoteness of the raw water storage ponds, water level feedback from the raw water ponds will need to be via a cellular or satellite internet connection. The system is proposed to be capable of automated operation based on a control narrative including but not limited to, flow setpoint and pressure-based pump control, river and wetwell level monitoring, and automated intake backflush.

9.0 PRELIMINARY FINANCIAL ANALYSIS

Technical Memorandum #2 (**Appendix E**) and Technical Memorandum #3 (**Appendix F**) provide detailed preliminary capital and phased costs estimated for the intake structure and pumping station, respectively. **Table 9.1** presents a summary of these costs.

Table 9.1: Summary of Full Build-out and Phased Preliminary Cost Estimates (July 2024)

DESCRIPTION		FULL BUILD-OUT CAPITAL COST	PHASED COSTS		
			Phase 1: 355 L/s	Phase 2: 710 L/s	Phase 3: 1065 L/s
INTAKE STRUCTURE					
1	General Items	\$11,594,300.00	\$11,376,000.00	\$177,900.00	\$289,700.00
2	Coffer Dam	\$45,551,800.00	\$45,551,800.00	\$	\$
3	Civil/Site Work	\$15,438,700.00	\$15,438,700.00	\$	\$
4	Structural	\$10,827,300.00	\$10,827,300.00	\$	\$
5	Electrical	\$1,207,500.00	\$885,500.00	\$262,300.00	\$427,200.00
6	Equipment	\$4,269,800.00	\$3,136,300.00	\$923,200.00	\$1,503,700.00
	TOTAL	\$88,889,400.00	\$87,215,600.00	\$1,363,400.00	\$2,220,600.00
PUMP STATION					
1	General Items	\$8,507,300.00	\$7,773,300.00	\$755,300.00	\$717,700.00
2	Civil/Site Work	\$13,512,000.00	\$13,512,000.00	\$	\$
3	Structural	\$17,663,500.00	\$17,618,400.00	\$36,800.00	\$59,900.00
4	HVAC Mechanical	\$904,100.00	\$904,100.00	\$	\$
5	Electrical	\$13,974,800.00	\$12,686,800.00	\$2,098,100.00	\$
6	Transfer Pumping System	\$9,935,700.00	\$6,375,600.00	\$2,899,500.00	\$4,723,000.00
7	Air Backflush	\$724,500.00	\$724,500.00	\$	\$
	TOTAL	\$65,221,900.00	\$59,594,700.00	\$5,789,700.00	\$5,500,600.00
GRAND TOTAL		\$154,111,300.00	\$146,810,300.00	\$7,153,100.00	\$7,721,200.00

Note that preliminary costing for the access road from the Confor Road to the raw water transfer station is not included and has been part of the Pipeline project.

10.0 IMPLEMENTATION SCHEDULE

10.1 General

Based on the use of a rockfill cofferdam to isolate a portion of the Smoky River to allow for the construction of an intake for the GIG project, the following reviews potential start dates and construction durations in relation to the restricted activity period (RAP). The RAP runs from August 1 to July 15 for the following year, resulting in an allowable (non-RAP) two-week period to complete any work in the river between July 16 to July 31.

The discussion following refers to seasons which will be defined as the 12-month period from January to December.

10.2 Cofferdam Construction

This section only discusses the Cofferdam construction during the first season. A reasonable period for a Rock cofferdam construction is assumed at 3 weeks during low flows. Two options were reviewed with the Team regarding the possible start dates for the construction of the cofferdam; these are:

- Option 1: Start construction on July 16 during the onset of the non-RAP period.
- Option 2: Start construction at the beginning on September 1 during low river flows.

Table 10.1 compares the two options.

Table 10.1: Comparison of Options 1 and 2

Construction Start	Option 1: July 16 Start	Option 2: September 1 start
River Hydraulics	High flows: ~ 479 m ³ /s High Velocities: ~ 3.43 m/s NHC notes that velocities could be 1.5 times higher.	Lower Flows: ~ 155 m ³ /s Lower Velocities: ~ 1.87 m/s NHC notes that velocities could be 1.5 times higher.
Constructability	Difficulties with constructability would be apparent due to the high flows and velocities.	Constructability would likely be within manageable conditions typically encountered.
Cost	Construction cost would likely be very high due to unknown risks presented by the fast-moving water.	Construction costs would likely be manageable based on expectations.
Safety	Increased safety risk of work personnel when working in fast moving water	Safety risks would be decreased working in slower moving water.
Construction Duration	Cofferdam construction is expected to extend beyond the two-week non-RAP period So construction during the RAP period would be expected. Construction of cofferdam likely to take longer due to difficulties presented by fast-moving waters.	Entire construction would be in the RAP period. Construction of cofferdam likely to be reasonable.
Aquatic Impact	Negative impact from rockfill material being washed away or displaced by the fast-flowing waters River peak velocities are higher than the scour velocity of 2.5 m/s, so erosion of the riverbed is expected.	Lower flows allow for manageable construction of the cofferdam. River peak velocities would be lower than the scour velocities.
Fish Migration	Fish migration will be impacted for the White Sucker in July and the Bull Trout in August.	The cofferdam construction and constriction would not impede fish passage during the first season of construction.

For both options, construction of the cofferdam during the RAP period is expected and could be longer for Option 2. However, Option 2 construction seems to provide a more feasible and favorable opportunity to construct the cofferdam with less risks compared to Option 1.

10.3 Intake Structure Construction Period

Following our team discussions, it is assumed that the rockfill cofferdam would remain for the duration of construction of the intake structure. Estimating the total duration of construction from cofferdam start to cofferdam removal following completion of the intake structure is difficult. However, for discussion purposes, a reasonable minimum construction period of 18 months could be used

For Option 1, assuming a July 16, 2026, start date, construction would be completed around January 16, 2028. For Option 2: a start date of September 1, 2026, would see construction completed around March 1, 2028. In both cases, constriction would be present during the sensitive fish migration months between May and August 2027. Boulders/fish migration enhancement measures would be needed as detailed in the Preliminary Assessment of Construction Berm Hydraulics Draft Report by NHC (i.e. boulders spaced at 0.2 m). Erosion protection, as recommended by NHC, would also be needed on the outer face of the rockfill cofferdam to protect the cofferdam from damage from the fast-moving waters during this period.

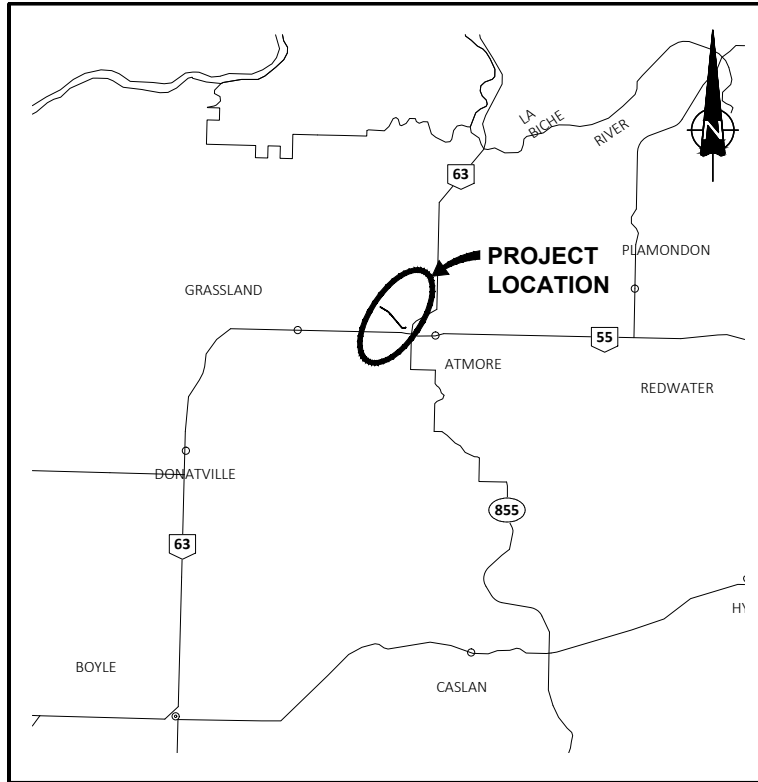
Option 1 would have added disturbance of the river during the six-week sensitive fish migration period between July 16, 2026, and August 31, 2026.

10.4 Conclusion

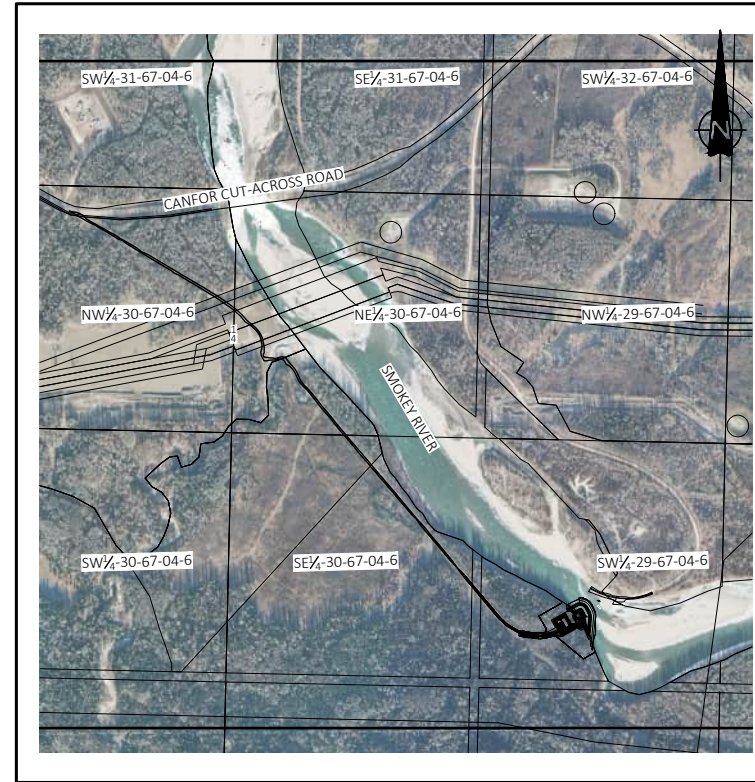
A mid-July start satisfies the requirements for entering the river during the non-RAP however the duration of the Cofferdam construction would extend into the RAP period. Also, a mid-July start presents various risks to workman safety, fish migration, aquatic habitat and more, due to the fast-moving waters at that time. A cofferdam construction start on September 1 would be preferred.

Appendix A

Preliminary Drawings



LOCATION PLAN



SITE PLAN

LIST OF DRAWINGS

GENERAL	
00	TITLE COVER
CIVIL	
C1.1	OVERALL PLAN
C2.1	ACCESS ROAD IMPROVEMENT STA 1+000 TO STA 1+800
C2.2	ACCESS ROAD IMPROVEMENT STA 1+900 TO 2+500
C2.3	ACCESS ROAD IMPROVEMENT STA 2+600 TO 3+300
C3.1	INTAKE AND TRANSFER STATION EROSION PROTECTION
C3.2	INTAKE AND TRANSFER STATION
C3.3	TRANSFER STATION SECTIONS
C3.4	TRANSFER STATION SECTIONS
C4.1	INTAKE AND PIPELINES
C5.1	COFFER DAM AND EROSION PROTECTION PLAN
C5.2	COFFER DAM AND EROSION PROTECTION SECTIONS
C6.1	GENERAL SPECIFICATIONS
C6.2	GENERAL SPECIFICATIONS
C6.3	GENERAL SPECIFICATIONS
C6.4	GENERAL SPECIFICATIONS
C6.5	GENERAL SPECIFICATIONS
C6.6	GENERAL SPECIFICATIONS
PROCESS	
P2.1	PLAN VIEW
P2.2	SECTION VIEW
P3.1	MAIN FLOOR PLAN
ELECTRICAL	
E2.1	SINGLE LINE DIAGRAM

WATERSMART A HAZEN COMPANY GREENVIEW INDUSTRIAL GATEWAY WATER INTAKE AND RAW WATER TRANSFER STATION FOR PRELIMINARY DESIGN REPORT 5946-001-00



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 4. THE CONTRACTOR SHALL CALL ALBERTA ONE-CALL AT 1-800-242-3447 AT LEAST THREE (3) WORKING DAYS PRIOR TO BEGINNING ANY EXCAVATION OR REMOVALS.
 5. THE CONTRACTOR IS RESPONSIBLE FOR COORDINATING UTILITY LINE CROSSING WITH UTILITY COMPANIES.
 6. THE CONTRACTOR IS RESPONSIBLE FOR PROVIDING ADEQUATE BARRICADES, CONSTRUCTION SIGNAGE, TRAFFIC CONTROL, AND FLAG PERSONS.
 7. THE CONTRACTOR IS RESPONSIBLE FOR NOTIFYING RESIDENTS AND BUSINESSES AFFECTED BY CONSTRUCTION PROCEDURES AT LEAST SEVEN (7) DAYS PRIOR TO COMMENCING CONSTRUCTION.
 8. THE CONTRACTOR IS RESPONSIBLE FOR MEETING OCCUPATIONAL HEALTH AND SAFETY REGULATIONS.

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GREENVIEW INDUSTRIAL GATEWAY WATER INTAKE AND RAW WATER TRANSFER STATION CIVIL OVERALL PLAN		
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DRAWN	T.N.Q., C.C.	SCALE 1:7500
DATE	NOVEMBER 2023	DRAWING C1.1



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GREENVIEW INDUSTRIAL GATEWAY
WATER INTAKE AND RAW WATER TRANSFER
STATION
CIVIL
ACCESS ROAD IMPROVEMENT
STA 1+000 TO STA 1+800

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DATE	NOVEMBER 2023	DRAWING	C2.1

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WATER INTAKE AND RAW WATER TRANSFER
STATION
CIVIL
ACCESS ROAD IMPROVEMENT
STA 1+900 TO 2+500

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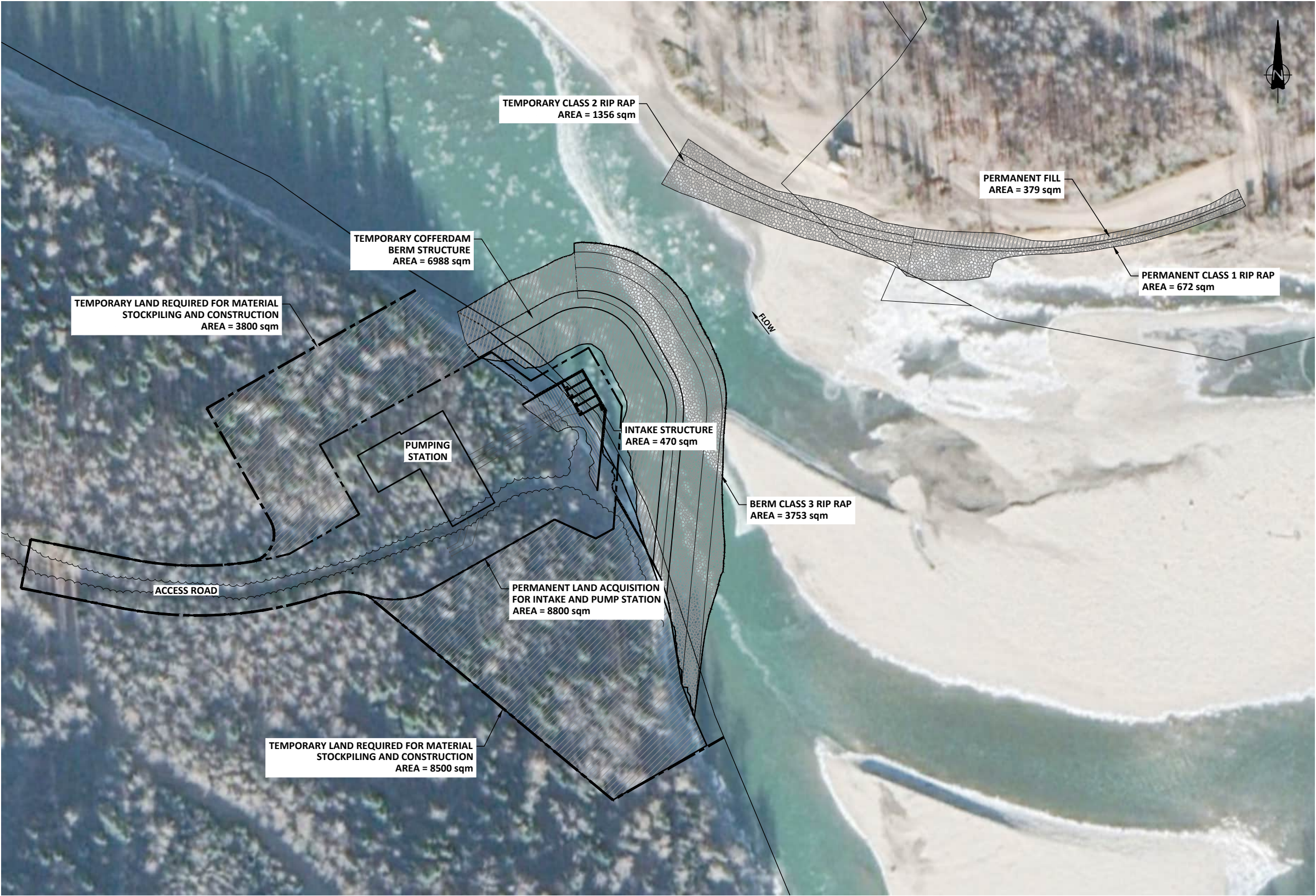
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WATER INTAKE AND RAW WATER TRANSFER
STATION
CIVIL
ACCESS ROAD IMPROVEMENT
STA 2+600 TO 3+300

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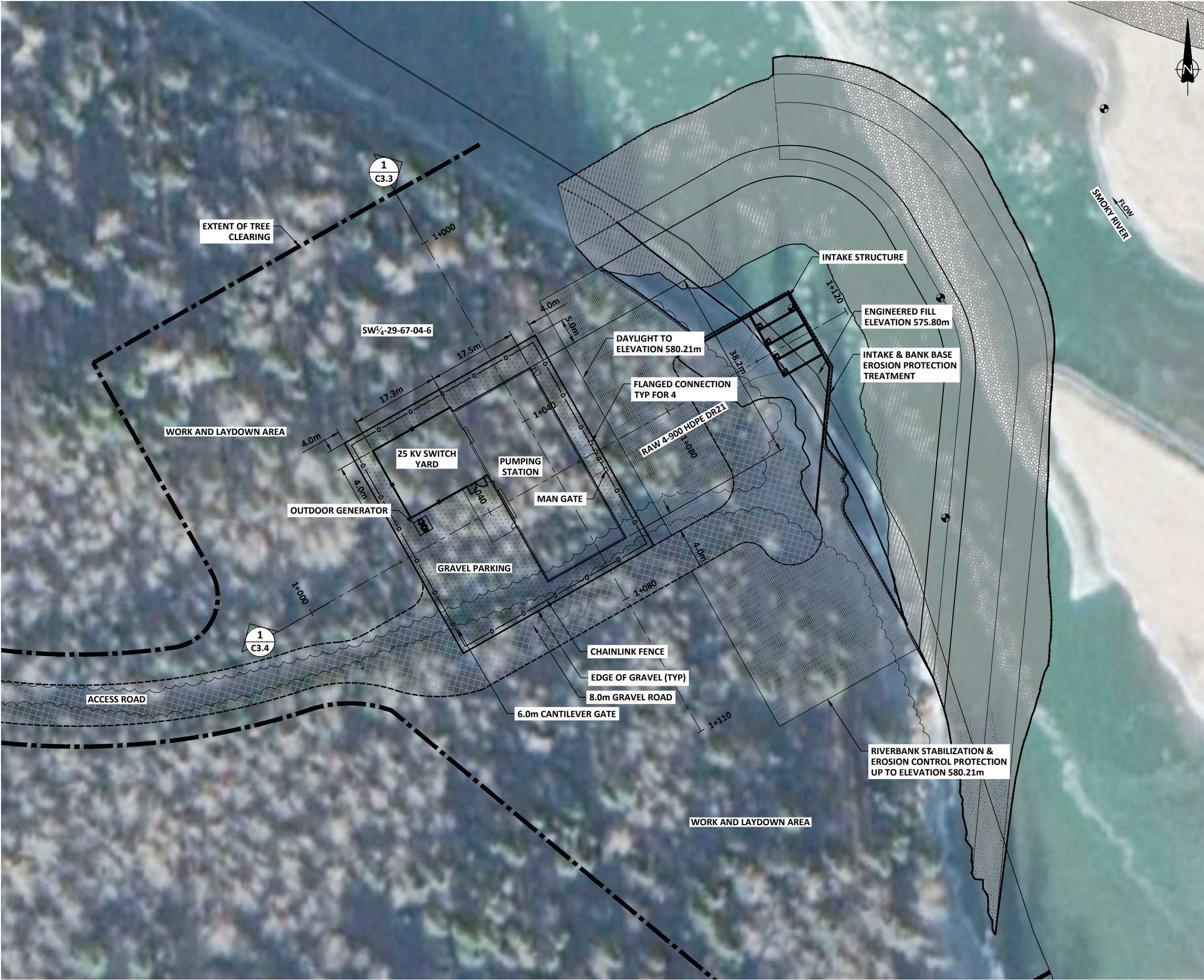
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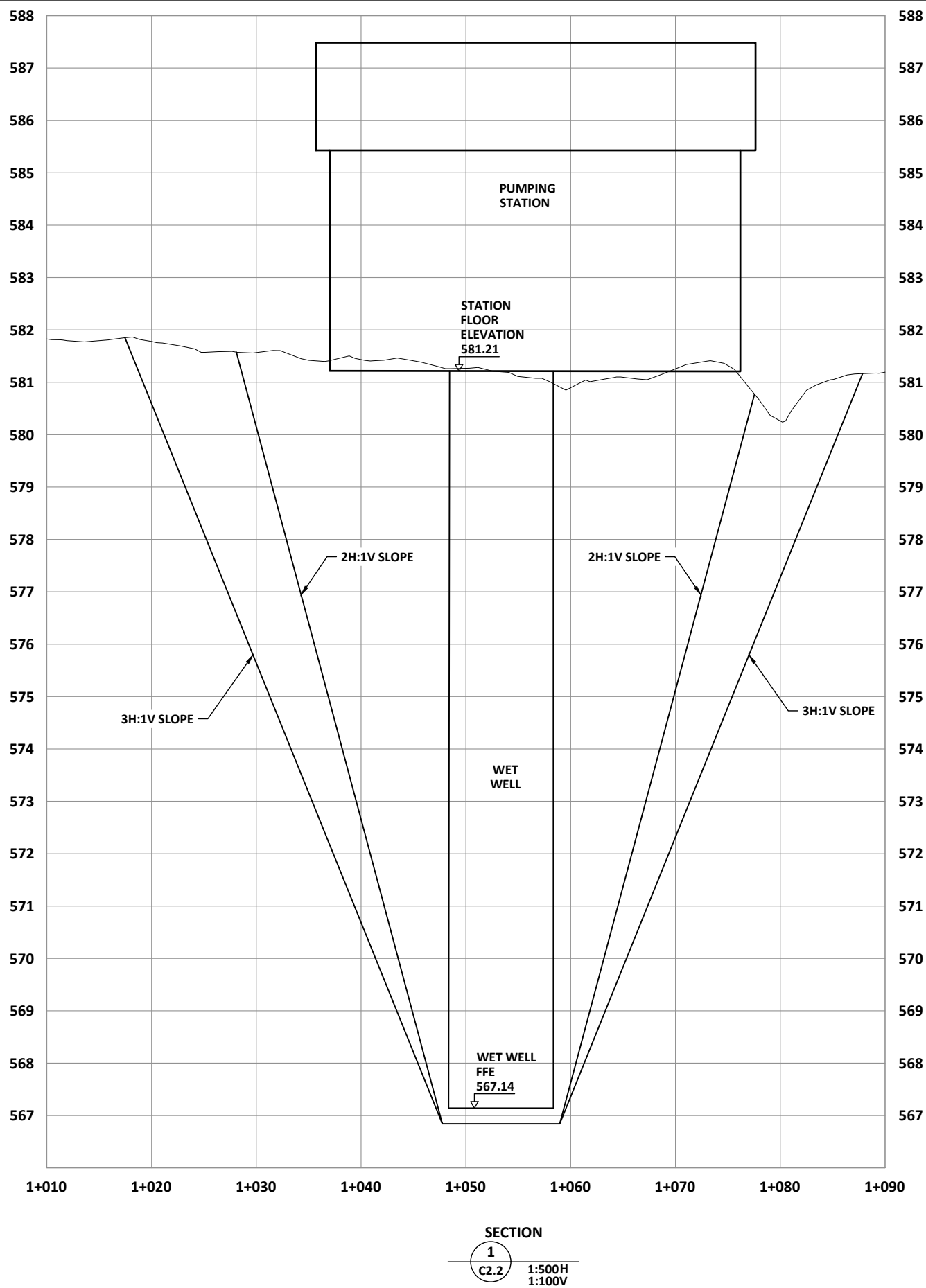
GREENVIEW INDUSTRIAL GATEWAY
WATER INTAKE AND RAW WATER TRANSFER
STATION
CIVIL
INTAKE AND TRANSFER STATION EROSION
PROTECTION

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 - THE CONTRACTOR IS RESPONSIBLE FOR MEETING OCCUPATIONAL HEALTH AND SAFETY REGULATIONS.
 - ALL DISTURBED AREAS TO BE LANDSCAPED, TOP SOILED AND SEEDED WITH FORESTRY GRASS MIXTURE.

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GREENVIEW INDUSTRIAL GATEWAY WATER INTAKE AND RAW WATER TRANSFER STATION CIVIL INTAKE AND TRANSFER STATION		
DESIGNED	I.A.K., C.C.	JOB 5946-001-00
DRAWN	T.N.Q., C.C.	SCALE 1:1500
DATE	NOVEMBER 2023	DRAWING C3.2



- NOTES:
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 3. ALL BOLD ITEMS INDICATE WORK TO BE DONE AND ALL LIGHT ITEMS INDICATE EXISTING CONDITIONS.
 4. THE CONTRACTOR SHALL CALL ALBERTA ONE-CALL AT 1-800-242-3447 AT LEAST THREE (3) WORKING DAYS PRIOR TO BEGINNING ANY EXCAVATION OR REMOVALS.
 5. THE CONTRACTOR IS RESPONSIBLE FOR COORDINATING UTILITY LINE CROSSING WITH UTILITY COMPANIES.
 6. THE CONTRACTOR IS RESPONSIBLE FOR PROVIDING ADEQUATE BARRICADES, CONSTRUCTION SIGNAGE, TRAFFIC CONTROL, AND FLAG PERSONS.
 7. THE CONTRACTOR IS RESPONSIBLE FOR NOTIFYING RESIDENTS AND BUSINESSES AFFECTED BY CONSTRUCTION PROCEDURES AT LEAST SEVEN (7) DAYS PRIOR TO COMMENCING CONSTRUCTION.
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ISSUE	YY-MM-DD	REVISION

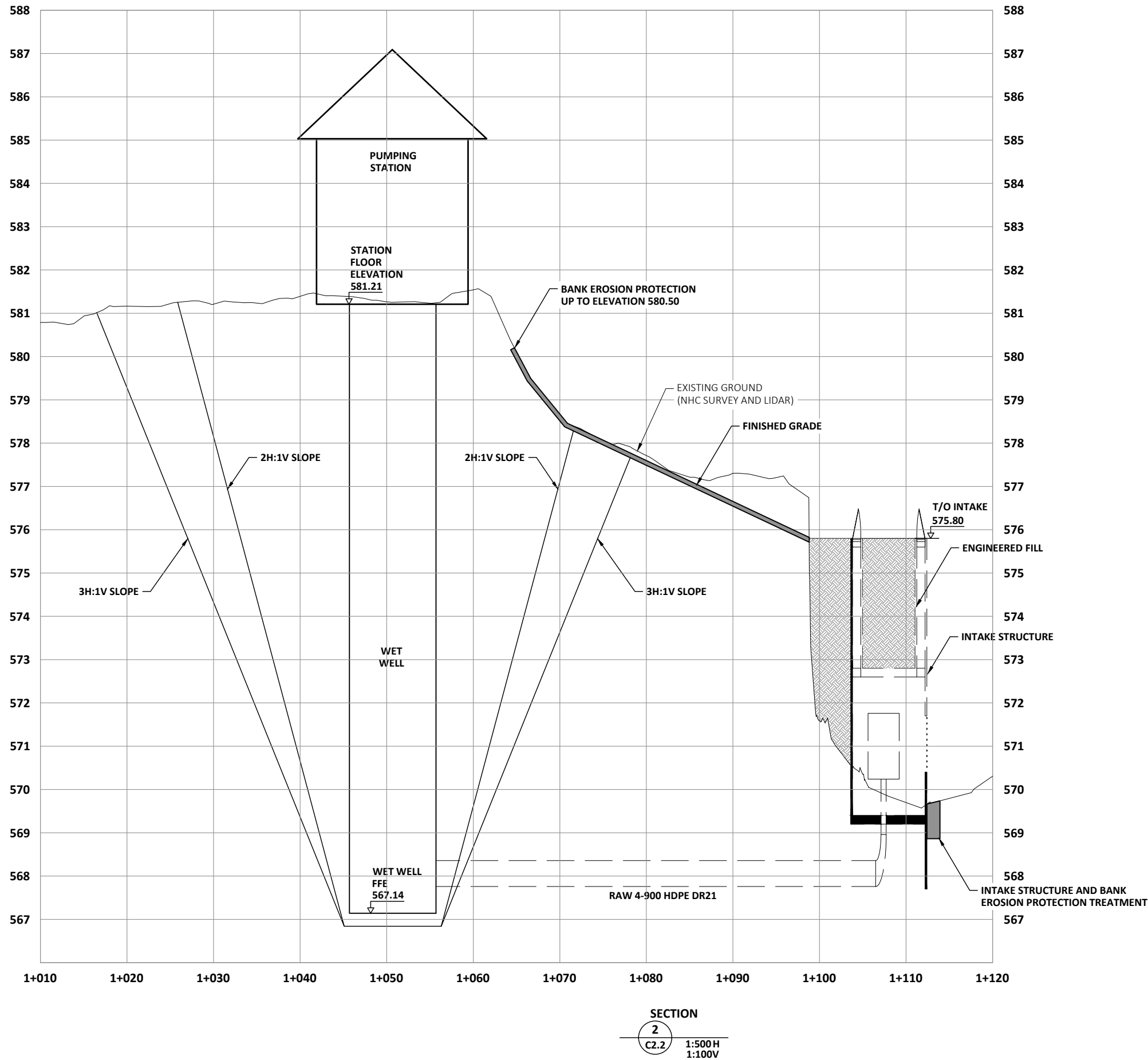
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WATERSMART A HAZEN COMPANY

**GREENVIEW INDUSTRIAL GATEWAY
WATER INTAKE AND RAW WATER TRANSFER
STATION
CIVIL
TRANSFER STATION SECTIONS**

DESIGNED	I.A.K., C.C.	JOB	5946-001-00
DRAWN	T.N.Q., C.C.	SCALE	1:1500
DATE	NOVEMBER 2023	DRAWING	C3.3



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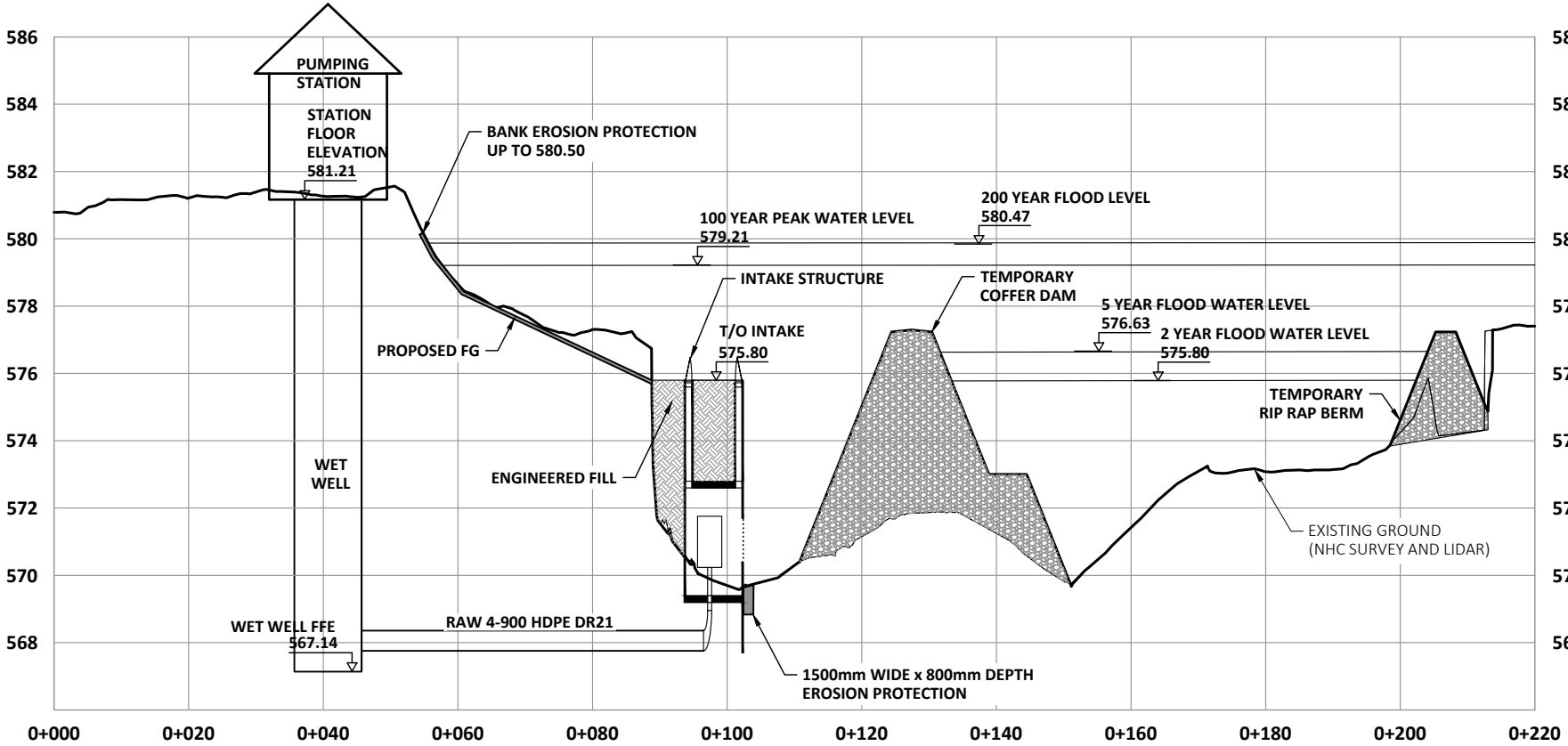
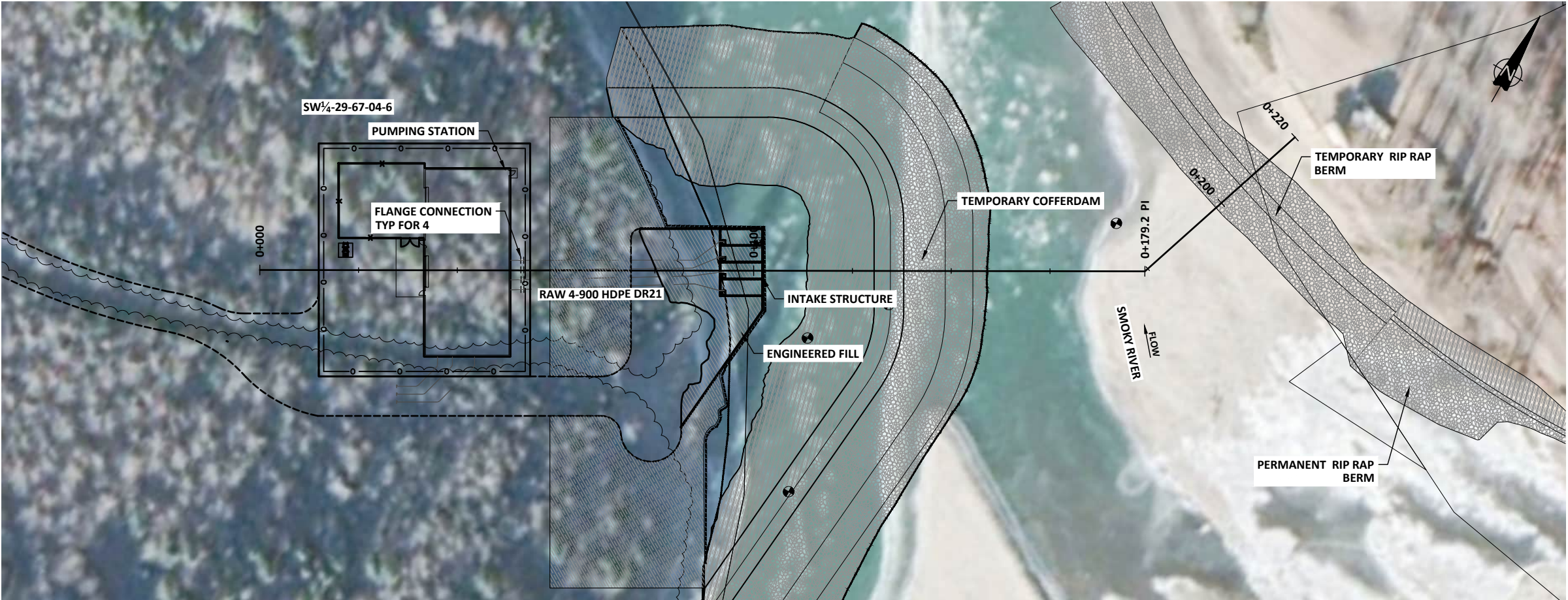
MON DD, YYYY



WATERSMART A HAZEN COMPANY

**GREENVIEW INDUSTRIAL GATEWAY
WATER INTAKE AND RAW WATER TRANSFER
STATION
CIVIL
TRANSFER STATION SECTIONS**

DESIGNED	I.A.K., C.C.	JOB	5946-001-00
DRAWN	T.N.Q., C.C.	SCALE	1:1500
DATE	NOVEMBER 2023	DRAWING	C3.4



PLAN AND PROFILE
1
C4.1 1:1000H
1:200V

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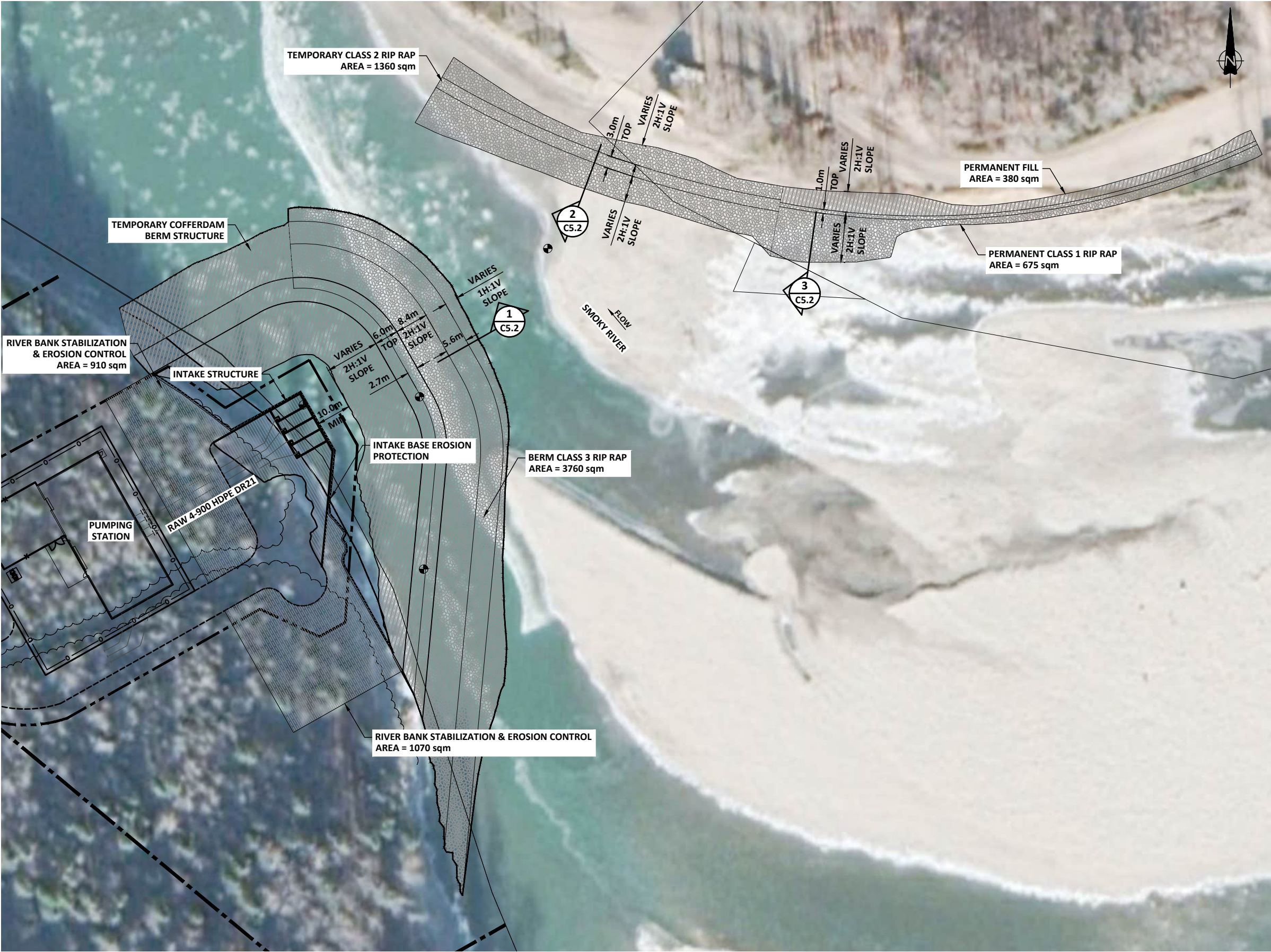
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WATERSMART A HAZEN COMPANY

GREENVIEW INDUSTRIAL GATEWAY
WATER INTAKE AND RAW WATER TRANSFER
STATION
CIVIL
INTAKE AND PIPELINES

DESIGNED	I.A.K., C.C.	JOB	5946-001-00
DRAWN	T.N.Q., C.C.	SCALE	1:1000
DATE	NOVEMBER 2023	DRAWING	C4.1



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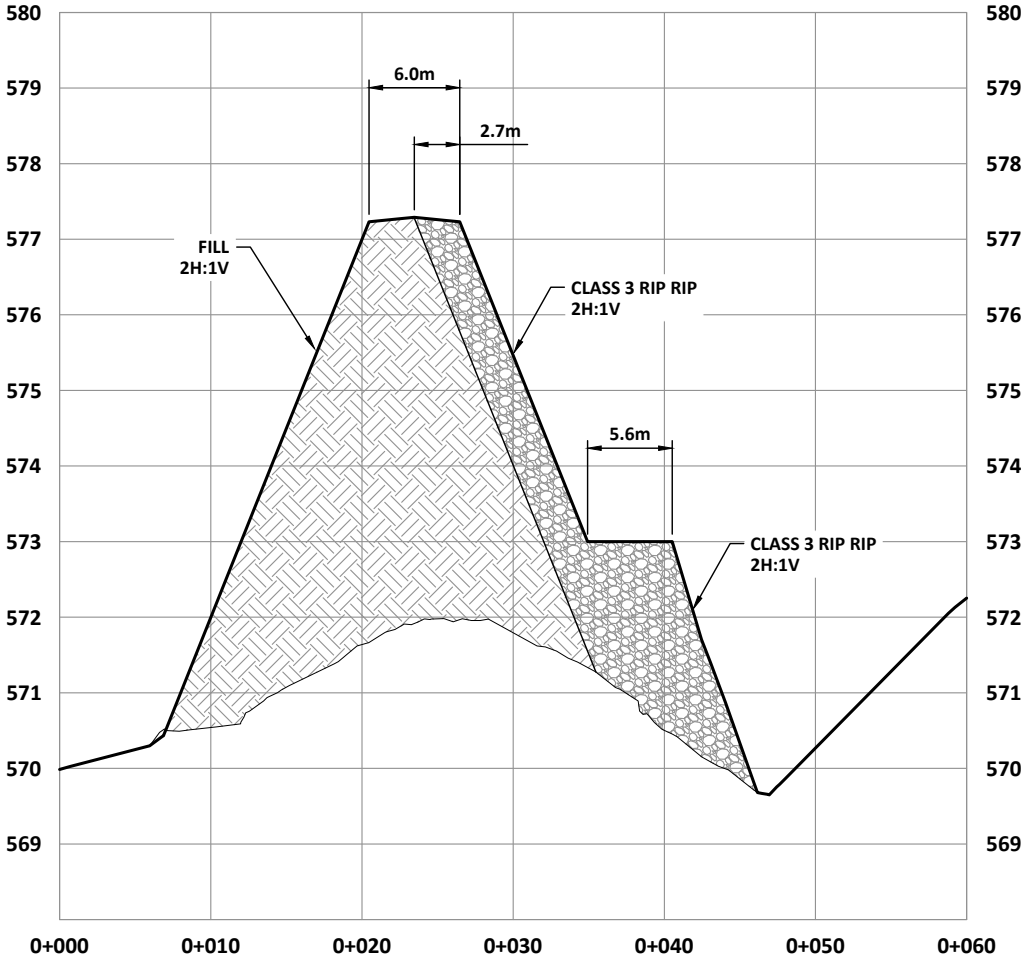
WATERSMART A HAZEN COMPANY

GREENVIEW INDUSTRIAL GATEWAY
WATER INTAKE AND RAW WATER TRANSFER
STATION
CIVIL
COFFER DAM AND EROSION PROTECTION
PLAN

DESIGNED	I.A.K., C.C.	JOB	5946-001-00
DRAWN	T.N.Q., C.C.	SCALE	1:1000
DATE	NOVEMBER 2023	DRAWING	C5.1

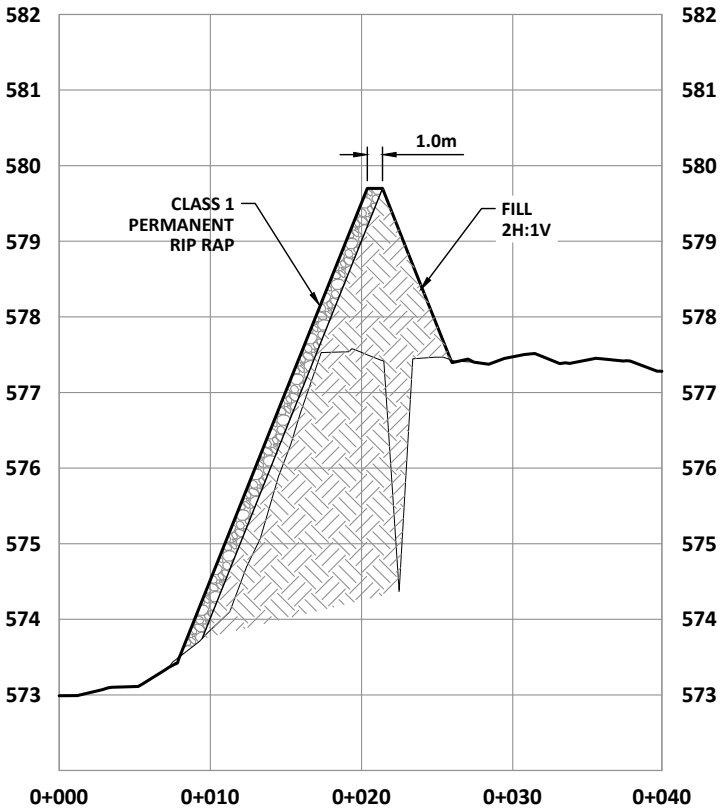
DIGITAL DRAWINGS BEST VIEWED IN ©ADOBE ACRobat READER

THIS DRAWING MAY HAVE BEEN MODIFIED FROM ITS ORIGINAL SIZE. ALL SCALE NOTATIONS INDICATED ARE BASED ON 11"x17" FORMAT DRAWINGS



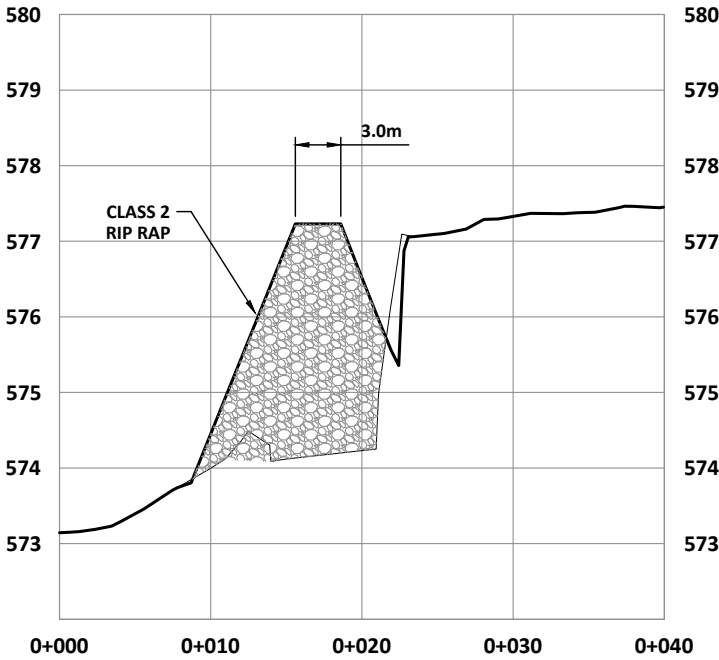
TEMPORARY COFFER DAM

1
CS.1 1:500H
1:100V



PERMANENT RIP RAP

3
CS.1 1:500H
1:100V



TEMPORARY RIP RAP

2
CS.1 1:500H
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WATERSMART A HAZEN COMPANY

**GREENVIEW INDUSTRIAL GATEWAY
WATER INTAKE AND RAW WATER TRANSFER
STATION
CIVIL
COFFER DAM AND EROSION PROTECTION
SECTIONS**

DESIGNED	I.A.K., C.C.	JOB	5946-001-00
DRAWN	T.N.Q., C.C.	SCALE	AS SHOWN
DATE	NOVEMBER 2023	DRAWING	C5.2

CLEARING & GRUBBING

1. GENERAL
- 1.1. REMOVE ALL CONCRETE, LOGS, TREES, BRUSH, STUMPS, ROOTS AND ALL OBJECTIONABLE MATERIAL ABOVE THE GROUND OR ON THE GROUND SURFACE.

1.2. REMOVE ALL ROOTS AND OTHER DELETERIOUS MATERIALS TO 0.3M BELOW THE GROUND SURFACE.

1.3. TEMPORARILY STOCKPILE MATERIALS WITHIN THE SITE UNTIL CONDITIONS ARE SUITABLE FOR DISPOSAL.
2. DISPOSAL
- 2.1. BURN COMBUSTIBLE MATERIALS ON SITE WHEN PERMITTED. OBTAIN PERMITS AND AUTHORIZATION IN ACCORDANCE WITH THE REGULATOR REQUIREMENTS PRIOR TO BURNING.

2.2. PROVIDE CONTINUOUS SUPERVISION DURING BURNING OPERATION AND CONDUCT BURNING OPERATIONS IN SUCH A MANNER TO PROTECT SURROUNDING VEGETATION AND PROPERTY FROM DAMAGE.

2.3. WHEN BURNING IS NOT PERMITTED BY REGULATORY REQUIREMENTS, EXCAVATE, LOAD, TRANSPORT AND DISPOSE OF ALL MATERIALS TO THE NEAREST WASTE DISPOSAL SITE AS DESIGNED IN THE CONTRACT DOCUMENT OR TO THE NEAREST QUALIFIED SANITARY LANDFILL SITE.

2.4. BURY BURNED DEBRIS AND NON-COMBUSTIBLE MATERIALS IN DESIGNATED WASTE AREA WITH A MINIMUM COVER OF 1M OF WASTE FILL. FINISH TO A NEAT, LEVELED APPEARANCE, CONFORMING TO SURROUNDING LANDSCAPE.

STRIPPING

1. GENERAL
- 1.1. THIS SECTION APPLIES TO STRIPPING REQUIREMENTS FOR PIPELINE EASEMENT AND RIGHT OF WAYS, BORROW AREA, ACCESS ROADS, MATERIAL HAULAGE ROADS AND STOCKPILE AREAS WITHIN THE SITE
2. DEFINITIONS
- 2.1. FOR THE PURPOSE OF CONSTRUCTION IN THIS CONTRACT, THE FOLLOWING DEFINITIONS APPLY:

"TOPSOIL" IS DEFINED AS THE UPPERMOST PART OF THE SOIL, ORDINARILY MOVED IN TILLAGE, OR ITS EQUIVALENT IN UNCULTIVATED SOILS, AND NORMALLY RANGING IN DEPTH FROM 50 mm TO 450 mm.

"SUBSOIL" IS DEFINED AS MATERIAL THAT LIES IMMEDIATELY BENEATH THE TOPSOIL AND EXTENDING TO ROT DEPTH. SUBSOIL MAY BE UP TO 1.5M IN DEPTH.

"OVERBURDEN" IS DEFINED AS SOIL MATERIAL THAT LIES BETWEEN THE SUBSOIL AND THE MATERIAL THAT IS DESIGNATED TO BE UTILIZED FOR CONSTRUCTION.

"STRIPPING" IS DEFINED AS THE EXCAVATION OF TOPSOIL, SUBSOIL, AND OVERBURDEN, INCLUDING MATERIALS IN FROZEN CONDITION.
3. PROTECTION OF EXISTING FACILITIES
- 3.1. LOCATE UTILITY LINES, FENCING, SURVEY REFERENCE POINTS, INSTRUMENTATION, CULVERTS, AND ALL OTHER EXISTING FACILITIES BEFORE COMMENCEMENT OF WORK. PROTECT THESE ITEMS FROM DAMAGE.
4. EXCAVATION
- 4.1. STAY ON DESIGNATED HAUL ROADS AND DO NOT DISTURB GRASSED OR NATURAL AREA NOT PART OF THE WORK. DO NOT DRIVE ON UNDISTURBED AREA EXCEPT FOR THE PERFORMANCE OF STRIPPING OPERATION.

4.2. STRIP AND STOCKPILE MATERIALS SEPARATELY AND PREVENT CONTAMINATION

4.3. STRIP AND STOCKPILE MATERIALS FROM TEMPORARY CONSTRUCTION ACCESS ROADS, BORROW AREA AND WASTE FILL AREA REQUIRED FOR PERFORMANCE OF THE WORK

4.4. CONDUCT THE STRIPPING OPERATIONS FAR ENOUGH IN ADVANCE OF EXCAVATION TO ENSURE THAT UNDESIRABLE MATERIAL DOES NOT BECOME MIXED WITH THE TOPSOIL.

4.5. SUSPEND STRIPPING OPERATIONS DURING RAIN OR WET GROUND CONDITION.

4.6. SUSPEND STRIPPING OPERATIONS DURING HIGH WINDS GREATER THAN 80 KM/HR, WHICH MAY RESULT IN CONTAMINATION OR LOSS OF TOPSOIL.

4.7. PROVIDE PROPER DRAINAGE OF SURFACE WATER FROM STRIPPED AREA TO PREVENT PONDING AND INFILTRATION IN AREAS WHERE FILL IS TO BE PLACED.

4.8. USE EQUIPMENT WITH PRECISE DEPTH CONTROL SUCH AS A GRADER WHEN STRIPPING SHALLOW DEPTH TOPSOIL.

4.9. EXCAVATE ALL INITIAL FROZEN MATERIAL. SUBSEQUENT FROST REMOVAL WILL NOT BE PAID.
5. STOCKPILE
- 5.1. UNLESS OTHERWISE DESIGNATED, STOCKPILE STRIPPED MATERIALS ADJACENT TO BORROW AREAS, WASTE AREAS OR ALONG THE PIPELINE EASEMENT. CHOOSE STOCKPILE LOCATIONS SUCH THAT THEY WILL NOT INTERFERE WITH FUTURE CONSTRUCTION ACTIVITIES.

5.2. STOCKPILE TOPSOIL SEPARATELY FROM OTHER MATERIALS.

5.3. MAINTAIN A MINIMUM SEPARATION OF 1M BETWEEN STOCKPILES.

5.4. STOCKPILE FROZEN MATERIAL STRIPPED FROM BORROW AREAS SEPARATELY FROM OTHER MATERIALS.

5.5. ADOPT MEASURES TO PREVENT DRIFTING OF TOPSOIL.

5.6. KEEP DRAINAGE COURSES CLEAR OF STOCKPILED MATERIAL.

5.7. STOCKPILE MATERIAL AT SLOPES LOWER THAN 2H:1V.
6. DISPOSAL OF STRIPPING MATERIAL
- 6.1. REHANDLE AND PLACE ALL STOCKPILED STRIPPING TOPSOIL MATERIAL UNIFORMLY ON ALL AREA DISTURBED DURING CONSTRUCTION.

- 6.2. REHANDLE AND PLACE ALL STOCKPILES STRIPPING MATERIAL FROM BORROW AREAS IN COMPLETED BORROW AREAS. PLACE, SPREAD AND COMPACT OVERBURDEN AND THEN SUBSOIL MATERIAL ON BORROW AREA SUBGRADE PRIOR TO THE PLACEMENT OF TOPSOIL.

TOPSOIL PLACEMENT

1. PREPARATION OF SUBGRADE
- 1.1. REMOVE DEBRIS, ROOTS AND OTHER DELETERIOUS MATERIALS AND HAUL TO DESIGNATED DISPOSAL AREAS.

1.2. GRADE AREA TO BE RECLAIMED TO ELIMINATE UNEVEN AREAS. GRADE TO ENSURE PROPER DRAINAGE.

1.3. SCARIFY ALL AREAS TO BE RECLAIMED TO A MINIMUM DEPTH OF 300 mm. SCARIFY ENTIRE SUBGRADE AREA ONCE IN THE LONGITUDINAL DIRECTION AND ONCE IN THE PERPENDICULAR DIRECTION. DISC AREA WHEN LARGE CLAY LUMPS ARE PREVALENT.
2. TOPSOIL PLACEMENT
- 2.1. EXCAVATE, LOAD AND HAUL TOPSOIL FROM STOCKPILES AND PLACE IN WASTE AREAS, DRAINAGE ITCHES AND ROAD EMBANKMENTS.

2.2. PLACE SUBSOIL AND TOPSOIL IN DRY WEATHER ON DRY UNFROZEN SUBGRADE.

2.3. DISTRIBUTE MATERIAL TO UNIFORM THICKNESS OVER THE ENTIRE AREA. DISC MATERIAL AND REMOVE ALL ROCKS LARGER THAN 75mm.

2.4. MANUALLY SPREAD TOPSOIL AROUND STRUCTURES, FENCES OR OTHER OBSTRUCTIONS.

2.5. GRADE TO ENSURE POSITIVE DRAINAGE ON SLOPES AND AWAY FROM STRUCTURES.

2.6. LEAVE SURFACE SMOOTH, UNIFORM AND COMPACT TO 90% STANDARD PROCTOR DENSITY TO PREVENT SETTLEMENT WHEN WATERED. OBTAIN MINIMUM DEPTH OF TOPSOIL AFTER COMPACTION OF 100mm.

EARTH EXCAVATION

1. DEFINITIONS
- 1.1. "COMMON EXCAVATION" IS DEFINED AS ALL EXCAVATION WITHIN THE PROJECT WORK AREA.

1.2. "BORROW EXCAVATION" IS DEFINED AS ALL IMPORTED EXCAVATION FROM DESIGNATED BORROW AREA EXCLUDING STRIPPING AND EXCAVATION OF FROZEN MATERIAL.

1.3. "WASTE EXCAVATION" IS DEFINED AS ALL EXPORTED EXCAVATION FROM THE PROJECT AREA TO THE DESIGNATED WASTE AREAS THAT IS NOT REQUIRED FOR THE COMPLETION OF THE PROJECT WORK EXCLUDING STRIPPING.
2. PREPARATION
- 2.1. NOTIFY OWNER AT LEAST 2 DAYS PRIOR TO BEGINNING EXCAVATION OPERATIONS.

2.2. PRIOR TO COMMENCING EXCAVATION:

CONTACT ALL APPROPRIATE UTILITY COMPANIES AND ESTABLISH EXACT LOCATION AND CURRENT STATUS OF ALL UTILITIES, VOLTAGE OF UNDERGROUND AND OVERHEAD POWER LINES AND PRESSURE OF NATURAL GAS LINES.

NOTIFY OWNER IF ANY UTILITY LINES HAVE BEEN OMITTED FROM OR INCORRECTLY INDICATED IN THE CONTRACT DOCUMENTS.
3. PROTECTION OF EXISTING FACILITIES
- 3.1. LOCATE UTILITY LINES, FENCING, SURVEY REFERENCE POINTS, INSTRUMENTATION, CULVERTS, AND ALL OTHER EXISTING FACILITIES BEFORE COMMENCEMENT OF WORK. PROTECT THESE ITEMS FROM DAMAGE.
4. SHORING AND BRACING
- 4.1. IF REQUIRED TO PROVIDE SAFE WORKING CONDITIONS AND TO PREVENT CAVE-INS AND LOOSE SOIL FROM FALLING INTO EXCAVATIONS, PROTECT EXCAVATIONS BY TEMPORARY SHORING, BRACING, OR OTHER SUITABLE METHODS.

4.2. WHERE THE EXCAVATION IS MADE TO ACCOMMODATE STRUCTURES, SUFFICIENT MATERIAL SHALL BE REMOVED TO ALLOW FOR THE PROPER PLACING AND BRACING OF FORMS.
5. EXCAVATION
- 5.1. STRIP TOPSOIL (AND SUBSOIL WHERE REQUIRED) IN ACCORDANCE WITH STRIPPING SECTION AND STOCKPILE IN THE AREA DESIGNATED ON THE DRAWINGS.

5.2. DISPOSE OF UNSUITABLE ORGANIC SOILS AND OTHER UNSUITABLE, NATIVE OR UNCONTAMINATED MATERIALS IN THE DESIGNATED WASTE AREA.

5.3. REMOVE AND DISPOSE OF ALL WATER, SNOW AND SURFACE ICE PRIOR TO EXCAVATION.

5.4. SCHEDULE AND COORDINATE THE WORK SUCH THAT EXCAVATIONS ARE TRIMMED TO GRADE PRIOR TO BECOMING FROZEN.

5.5. EXCAVATE TO THE REQUIRED LINES, GRADES AND ELEVATIONS.

5.6. PREVENT LOSS OF SOIL AND SLOUGHING OF SLOPES IF SPRINGS OR SEEPAGE ARE ENCOUNTERED WITHIN EXCAVATION.

5.7. REMOVE BOULDERS, LOOSE BEDROCK, SOIL BLOCKS AND OTHER FRAGMENTS THAT MAY SLIDE OR ROLL INTO EXCAVATED AREAS, WHICH, IN THE OPINION OF THE OWNER OR THE CONTRACTOR, ARE UNSAFE OR APPEAR TO ENDANGER PERSONS, WORK OR PROPERTY. THE FACT THAT SUCH REMOVAL MAY ENLARGE AN EXCAVATION BEYOND THE REQUIRED EXCAVATION LINES SHALL NOT RELIEVE THE CONTRACTOR FROM THE NECESSITY OF DOING SUCH SCALING AND REMOVAL.

6. COBBLES AND BOULDERS
- 6.1. COBBLES AND BOULDERS TOO LARGE TO BE LEFT IN FILL MATERIALS MUST BE REMOVED FROM THE FILL PRIOR TO COMPACTION. IN SOME CASES THE COBBLES AND BOULDERS CAN BE TOO LARGE TO BE REASONABLY MOVED BY HAND AND MUST BE REMOVED BY MACHINERY. WHEN COBBLES AND BOULDERS ARE REMOVED BY MACHINERY, REASONABLE CARE, SATISFACTORY TO THE OWNER, MUST BE TAKEN TO LIMIT THE QUANTITY OF SUITABLE FILL REMOVED WITH THE COBBLE OR BOULDER.

6.2. THE CONTRACTOR IS ENCOURAGED TO MAKE APPROPRIATE USE OF COBBLES AND BOULDERS MATERIALS WHERE SUITABLE AND ACCEPTABLE. STOCKPILE EXCESS COBBLES AN D BOULDERS AT LOCATIONS WITHIN THE SITE LIMITS DESIGNATED.
7. TOLERANCE
- 7.1. EXCAVATE ALL SURFACE TO WITHIN ±20MM OF THE LINES, GRADES AND ELEVATIONS SHOWN IN THE CONTRACT DOCUMENTS.

COMPACTED EARTH FILL

1. TYPES OF FILL
- 1.1. COMPACTED IMPERVIOUS FILL.

1.2. COMPACTED FILL.

1.3. TAMPED IMPERVIOUS FILL.

1.4. WASTE FILL.
2. DEFINITIONS
- 2.1. "SUITABLE MATERIALS" IS DEFINED AS MATERIALS OBTAINED FROM BORROW EXCAVATIONS, FREE OF ORGANIC, WET OR FROZEN MATERIALS, THAT IS SUITABLE FOR COMPACTED EMBANKMENT CONSTRUCTION.

2.2. "UNSUITABLE MATERIAL" IS DEFINED AS ORGANIC, WET OR FROZEN MATERIAL FROM CANAL OR BORROW EXCAVATIONS THAT IS NOT SUITABLE FOR COMPACTED EMBANKMENT CONSTRUCTION.

2.3. "COMPACTED FILL" IS DEFINED AS SUITABLE MATERIAL OBTAINED FROM COMMON EXCAVATIONS, FREE OF ORGANIC, WET OR FROZEN MATERIALS, AND PLACED ON CANAL EMBANKMENTS.

2.4. "WASTE FILL" IS DEFINED AS ORGANIC, WET OR FROZEN MATERIAL FROM CANAL EXCAVATION THAT IS NOT SUITABLE FOR EMBANKMENT CONSTRUCTION, AND PLACED ON CANAL EMBANKMENTS OUTSIDE OF COMPACTED IMPERVIOUS OR COMPACTED FILLS.
3. FILL MATERIALS
- 3.1. COMPACTED AND TAMPED IMPERVIOUS FILL MATERIAL IS FINE GRAINED MATERIALS HAVING A MINIMUM OF 50% PASSING THE 0.075MM SIEVE SIZE AND CLASSIFIED AS A LOW TO MEDIUM PLASTIC CLAY BASED ON THE UNIFIED CLASSIFICATION SYSTEM AS MODIFIED BY PFRA.

3.2. REMOVE TREE ROOTS, SOD OR OTHER ORGANIC MATERIALS.

3.3. DO NOT USE FROZEN MATERIAL IN THE FILL.

3.4. REMOVE COBBLES AND ROCK FRAGMENTS HAVING MAXIMUM DIMENSIONS GREATER THAN 75MM.
4. DENSITY CONTROL
- 4.1. COMPACT ALL FILL MATERIAL, EXCEPT WASTE FILL, TO A DRY DENSITY EQUAL TO OR GREATER THAN 95% OF THE MAXIMUM DRY DENSITY OBTAINED IN THE STANDARD PROCTOR COMPACTION TEST PERFORMED IN ACCORDANCE WITH ASTM D698.

4.2. TAMP FILL MATERIAL TO 98% OF THE MAXIMUM DRY DENSITY OBTAINED IN THE STANDARD PROCTOR COMPACTION TEST PERFORMED IN ACCORDANCE WITH ASTM D698. TAMPED FILL AREA AROUND ALL BUILDING STRUCTURE.

4.3. COMPACT WASTE FILL MATERIAL TO 85% OF STANDARD PROCTOR MAXIMUM DRY DENSITY TEST PERFORMED IN ACCORDANCE WITH ASTM D698. ALL WASTE FILL TO BE LOCATED AS DIRECTED BY ENGINEER.
5. MOISTURE CONTROL
- 5.1. MAINTAIN MOISTURE CONTENT FOR COMPACTED FILL MATERIALS WITHIN -1% TO 2% OF OPTIMUM MOISTURE CONTENT AS DETERMINED BY ASTM D698 TEST PROCEDURES.

5.2. MAINTAIN MOISTURE CONTENT FOR TAMPED FILL MATERIALS WITHIN -1% TO 2% OF OPTIMUM MOISTURE CONTENT AS DETERMINED BY ASTM D698 TEST PROCEDURES.

5.3. WHEN THE MOISTURE CONTENT IN THE FILL MATERIAL IS LOWER THAN THAT SPECIFIED FOR PLACEMENT, WATER SHALL BE ADDED AND MIXED WITH THE MATERIAL TO ACHIEVE UNIFORM MOISTURE CONTENT IN THE MATERIAL TO CONFORM TO THE REQUIREMENTS.

5.4. WHEN THE MOISTURE CONTENT IN THE FILL MATERIALS IS HIGHER THAN THAT SPECIFIED FOR PLACEMENT, THE MATERIAL SHALL BE DRIED BY SCARIFYING, DISKING AND HARROWING TO ACHIEVE UNIFORM MOISTURE CONTENT IN THE MATERIAL THAT CONFORMS TO THE REQUIREMENTS.

5.5. MOISTURE CONTENT CONTROL ON WASTE FILL WILL NOT BE REQUIRED.
6. PLACEMENT AND COMPACTION
- 6.1. DRAIN AND CLEAN ALL EARTH FOUNDATIONS OF LOOSE THAWED, FROZEN, SOFT, OR OTHER DELETERIOUS MATERIAL INCLUDING ICE, SNOW AND ORGANIC MATERIALS AND TOPSOIL.

6.2. WORK THE SURFACE TO OBTAIN A SUITABLE BOND WITH THE EARTH FILL IMMEDIATELY PRIOR TO PLACING THE FIRST LAYER.

6.3. WHEN THE SURFACE OF THE PREPARED FOUNDATION OR THE COMPACTED FILL MATERIAL IS TOO DRY OR TOO SMOOTH TO BOND PROPERLY WITH THE LAYER OF FILL MATERIAL TO BE PLACED THEREON, MOISTEN THE SURFACE AND WORK WITH A DISC, SCARIFIER, OR OTHER EQUIPMENT, TO PROVIDE A SATISFACTORY BONDING SURFACE BEFORE THE SUCCEEDING LAYER OF FILL MATERIAL IS PLACED.

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WATERSMART A HAZEN COMPANY

GREENVIEW INDUSTRIAL GATEWAY
WATER INTAKE AND RAW WATER TRANSFER
STATION
CIVIL
GENERAL SPECIFICATIONS

DESIGNED	I.A.K., C.C.	JOB	5946-001-00
DRAWN	T.N.Q., C.C.	SCALE	NTS
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6.4. WHEN THE SURFACE OF THE PREPARED FOUNDATIONS OR THE COMPACTED FILL MATERIAL IS TOO WET FOR PROPER COMPACTION, REMOVE IT AND ALLOW IT TO DRY, OR WORK IT WITH A HARROW, DISC OR OTHER EQUIPMENT TO REDUCE THE MOISTURE CONTENT TO THE REQUIRED AMOUNT; THEN COMPACT THE FILL MATERIAL BEFORE THE SUCCEEDING LAYER OF FILL MATERIAL IS PLACED THEREON. 6.5. MAINTAIN SLOPES AT LESS THAN 1V:1H FOR EARTH FOUNDATIONS ON WHICH FILL IS TO PLACED. 6.6. PLACE COMPACTED FILL MATERIAL IN CONTINUOUS HORIZONTAL LAYERS NOT EXCEEDING 150MM IN THICKNESS WHEN COMPACTED. SPREAD, BLEND, DISC, BLADE, SMOOTH AND COMPACT EACH LIFT TO PROVIDE A HOMOGENOUS FILL WITHOUT STRATIFICATION. COMMENCE PLACEMENT OF FILL AT THE LOWEST ELEVATION OF FOUNDATION. USE SHEEP'S FOOT TYPE COMPACTION EQUIPMENT. 6.7. PLACE WASTE FILL MATERIALS IN CONTINUOUS HORIZONTAL LIFTS NOT EXCEEDING 300MM IN THICKNESS SUCH THAT THERE WILL BE NO VOIDS OR BRIDGING OF MATERIAL. SPREAD AND COMPACT EACH LIFT BY COMPLETE COVERAGE OF TRACKED EQUIPMENT. BLADE THE COMPACTED WASTE FILL EMBANKMENT TO A SMOOTH, UNIFORM, FREE-DRAINING SHAPE. 6.8. JOIN NEW FILL TO EXISTING SLOPES BY TERRACING OR EXCAVATING INTO SLOPES TO REMOVE ALL DRIED AND LOOSE MATERIAL. 6.9. SCHEDULE FILL PLACEMENT OPERATIONS SUCH THAT THE FOUNDATION AREAS OR PREVIOUSLY COMPACTED EARTH FILL DOES NOT FREEZE AND THAT COMPACTED EARTH FILL IS NOT PLACED ON FROZEN SUBGRADE. REMOVE AND REPLACE ANY SUCH FROZEN LAYERS OF COMPACTED EARTH FILL AT NO COST TO THE OWNER. SCARIFY EACH LIFT OF IMPERVIOUS FILL TO A MINIMUM DEPTH OF 70 MM FOLLOWING COMPACTION, USING A DISC OR OTHER MINISTER APPROVED EQUIPMENT TO ENSURE COMPLETE BOND BETWEEN THAT LIFT AND THE OVERLYING LIFT. 6.11. REROUTE CONSTRUCTION TRAFFIC OR INCREASE FILL THICKNESS OVER SOFT FOUNDATIONS IN AREAS WHERE FILL SURFACE STARTS RUTTING. IF RUTTING HAS OCCURRED, SCARIFY, REGRADE AND MOISTURE CONDITION IMPERVIOUS FILL SURFACE PRIOR TO PLACEMENT OF OVERLYING FILL. 6.12. RE-COMPACT OR REMOVE ANY PORTION OF THE FILL WHICH HAS SUFFERED A REDUCTION IN DENSITY DUE TO FROST, RAIN OR ANY OTHER REASON BEFORE PLACING SUCCEEDING LAYERS. PROTECT COMPACTED FILL MATERIAL AND FOUNDATIONS PREPARED FOR THE FILL FROM FREEZING. 6.13. REMOVE ANY NON-CONFORMING MATERIALS, WHICH ACCUMULATE ON THE SURFACE OF ANY LAYER OR PREPARED FOUNDATION BEFORE ANY MATERIAL IS PLACED FOR THE SUCCEEDING LAYER. 7. COMPACTION EQUIPMENT 7.1. SUPPLY NECESSARY COMPACTION EQUIPMENT CAPABLE OF MEETING THE SPECIFIED COMPACTION REQUIREMENTS. 7.2. HAULING EQUIPMENT IS NOT ACCEPTABLE FOR COMPACTION. 7.3. THE OWNER'S REPRESENTATIVE RESERVES THE RIGHT TO ORDER THE DISCONTINUATION OF ANY COMPACTION EQUIPMENT THAT DOES NOT PRODUCE THE SPECIFIED COMPACTION REQUIREMENTS OR CAUSES EXCESSIVE BREAKAGE AROUND STRUCTURES. 8. TOLERANCE 8.1. MAKE CHANGES IN GRADE NATURAL. BLEND SLOPES INTO LEVEL AREAS. 8.2. COMPACT ALL SURFACES TO WITHIN 50 MM FROM THE LINES, GRADES AND ELEVATIONS.	1.4. THE CONTRACTOR SHALL MAINTAIN THE SUB-BASE MATERIAL TO THE SPECIFIED SECTION, GRADE AND CONDITION REQUIRED FOR THE PLACEMENT OF OTHER MATERIALS. THE CONTRACTOR SHALL BE RESPONSIBLE FOR PROVIDING INTERIM DRAINAGE TO PREVENT DAMAGES TO THE WORK OR THE CAUSING OF UNSTABLE CONDITIONS DUE TO HIGH MOISTURE CONTENTS. 1.5. DO NOT PLACE THE BASE MATERIAL UNTIL THE SUB-BASE MATERIAL HAS BEEN INSPECTED, SURVEYED, PROOF ROLLED, TESTED AND APPROVED BY THE ENGINEER. 2. COMPACTION OF SUB-BASE MATERIALS 2.1. GRANULAR SUB-BASE MATERIALS SHALL BE COMPACTED BY ROLLING WITH A PNEUMATIC TIRED ROLLER, VIBRATORY ROLLER OR OTHER APPROVED EQUIPMENT. 2.2. DURING COMPACTION, WATER SHALL BE ADDED BY AN APPLICATOR IN SUCH QUANTITIES THAT THE MOISTURE CONTENT WILL BE MAINTAINED AT THE OPTIMUM LEVEL AS DETERMINED BY STANDARD PROCTOR TEST. IF THE MOISTURE CONTENT EXCEEDS THE OPTIMUM MOISTURE CONTENT, THE MATERIAL SHALL BE AERATED BY MECHANICAL MEANS OR WORK SHALL CEASE TEMPORARILY UNTIL THE MATERIAL HAS DRIED SUFFICIENTLY TO REACH THE OPTIMUM MOISTURE CONTENT. 3. TOLERANCES 3.1. THE FINAL SURFACE SHALL BE EVEN AND UNIFORMLY SHAPED AND COMPACTED WITHIN A TOLERANCE OF ±15 MM OF ESTABLISHED GRADE BUT NOT UNIFORMLY LOW OR HIGH. BASE GRANULAR MATERIALS 1. PLACEMENT OF BASE MATERIAL 1.1. PROCESS, HANDLE AND TRANSPORT AGGREGATES TO AVOID SEGREGATION, CONTAMINATION AND DEGRADATION. 1.2. DO NOT PLACE GRANULAR MATERIALS ON SNOW, ICE OR FROZEN SURFACES. 1.3. DO NOT PLACE THE BASE MATERIAL UNTIL THE SUB-BASE MATERIAL HAS BEEN INSPECTED, SURVEYED, PROOF ROLLED, TESTED AND APPROVED BY THE ENGINEER. 1.4. PLACE THE BASE MATERIAL UNIFORMLY ON THE APPROVED SUB-BASE MATERIAL TO COMPACTED DEPTHS SPECIFIED. DO NOT PLACE THE BASE MATERIALS IN LAYERS EXCEEDING 150 MM COMPACTED DEPTH. SHAPE EACH LAYER TO A SMOOTH CONTOUR AND COMPACT TO THE SPECIFIED DENSITY BEFORE PLACING THE NEXT LAYER. AREAS THAT BECOME SEGREGATED DURING SPREADING SHALL BE REMOVED AND REPLACED AS REQUIRED AT THE EXPENSE OF THE CONTRACTOR. COMPACT THE FINAL LAYER OF THE BASE MATERIAL TO PROPER GRADE AND CROSS-SECTION. 1.5. THE CONTRACTOR SHALL MAINTAIN THE BASE MATERIAL TO THE SPECIFIED SECTION, GRADE AND CONDITION REQUIRED FOR THE PLACEMENT OF OTHER MATERIALS OR AS REQUIRED BY THE ENGINEER. THE CONTRACTOR SHALL BE RESPONSIBLE FOR PROVIDING INTERIM DRAINAGE TO PREVENT DAMAGES TO THE WORK OR THE CAUSING OF UNSTABLE CONDITIONS DUE TO HIGH MOISTURE CONTENTS. 2. COMPACTION OF BASE MATERIAL 2.1. GRANULAR BASE MATERIALS SHALL BE COMPACTED BY ROLLING WITH A PNEUMATIC TIRED ROLLER, VIBRATORY ROLLER OR OTHER APPROVED EQUIPMENT. 2.2. DURING COMPACTION, WATER SHALL BE ADDED BY AN APPLICATOR IN SUCH QUANTITIES THAT THE MOISTURE CONTENT WILL BE MAINTAINED AT THE OPTIMUM LEVEL AS DETERMINED BY STANDARD PROCTOR TEST. IF THE MOISTURE CONTENT EXCEEDS THE OPTIMUM MOISTURE CONTENT, THE MATERIAL SHALL BE AERATED BY MECHANICAL MEANS OR WORK SHALL CEASE TEMPORARILY UNTIL THE MATERIAL HAS DRIED SUFFICIENTLY TO REACH THE OPTIMUM MOISTURE CONTENT. 2.3. COMPACT BASE MATERIAL TO 98% OF STANDARD PROCTOR DENSITY, -1% TO 2% OF OPTIMUM MOISTURE CONTENT. 3. TOLERANCES 3.1. THE FINAL SURFACE SHALL BE EVEN AND UNIFORMLY SHAPED AND COMPACTED WITHIN A TOLERANCE OF ±10MM OF ESTABLISHED GRADE BUT NOT UNIFORMLY LOW OR HIGH.	1. PRESSURE TESTING 1.1. SUBJECT THE NEWLY LAID PIPE TO HYDROSTATIC TESTS AFTER BACKFILLING. 1.2. PROVIDE ALL LABOUR, EQUIPMENT AND MATERIALS REQUIRED TO PERFORM THE HYDROSTATIC AND LEAKAGE TESTS 1.3. THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE SUPPLY OF ALL WATER, MATERIALS, EQUIPMENT AND FITTINGS REQUIRED FOR PIPE PRESSURE TESTING. 1.4. SUBMIT WRITTEN HYDROSTATIC TESTING PLAN TO THE OWNER'S REPRESENTATIVE AT LEAST 48 HOURS PRIOR TO STARTING THE TESTS. TESTS ARE TO BE PERFORMED IN THE PRESENCE OF THE OWNER'S REPRESENTATIVE. DO NOT CONDUCT TESTS UNTIL AT LEAST 5 DAYS AFTER PLACING CONCRETE OR 2 DAYS IF HIGH EARLY STRENGTH CONCRETE IS USED FOR THRUST BLOCKS. 1.6. OPEN ALL VALVES NECESSARY TO TEST SECTION OF PIPE. 1.7. TEST PIPELINE IN SECTIONS NOT TO EXCEED 500 M IN LENGTH UNLESS OTHERWISE AUTHORIZED BY OWNER'S REPRESENTATIVE. 1.8. EXPEL AIR FROM MAIN BY SLOWLY FILLING MAIN WITH POTABLE WATER. INSTALL CORPORATION STOPS AT HIGH POINTS IN MAIN WHERE NO AIR-VACUUM RELEASE VALVES ARE INSTALLED. REMOVE STOPS AFTER SATISFACTORY COMPLETION OF TEST AND SEAL HOLES WITH PLUGS. TO COMPENSATE FOR INITIAL PIPE STRETCH AND TO EXPEL ALL ENTRAPPED AIR, THE PIPE SHALL BE PRESSURIZED UNTIL PRESSURE IS MAINTAINED BEFORE THE TEST PERIOD IS STARTED 1.9. FILL CONCRETE PIPE AT LEAST 24 HOURS BEFORE TESTING TO ALLOW WATER ABSORPTION BY PIPE MATERIAL. 1.10. THOROUGHLY EXAMINE EXPOSED PARTS AND CORRECT FOR LEAKAGE AS NECESSARY. 1.11. FOR PVC PIPES, APPLY HYDROSTATIC TEST PRESSURE OF 1035 KPA OR 1.5 TIMES THE NORMAL OPERATING PRESSURE, WHICHEVER IS GREATER, BASED ON ELEVATION OF LOWEST POINT IN THE MAIN AND CORRECTED TO ELEVATION OF TEST GAUGE, FOR A PERIOD OF 2 HOURS. TEST PRESSURE NOT TO EXCEED THE PRESSURE RATING OF THE PIPE OR FITTINGS. 1.12. DEFINE LEAKAGE AS AMOUNT OF WATER SUPPLIED IN ORDER TO MAINTAIN TEST PRESSURE FOR 2 HOUR FOR JOINTED PIPES AND 4 HOURS FOR POLYETHYLENE PIPE. 1.13. AFTER THE TEST PERIOD, A MEASURED AMOUNT OF "MAKE_UP" WATER SHALL BE ADDED TO RETURN THE PIPE TO THE TEST PRESSURE. THE AMOUNT OF "MAKE_UP" WATER SHALL NOT EXCEED THE ALLOWANCE GIVEN IN THE FOLLOWING TABLE. 1.14. FOR PVC PIPES, DO NOT EXCEED ALLOWABLE LEAKAGE AS DEFINED IN AWWA C605 USING THE FOLLOWING FORMULA FOR DUCTILE IRON PIPE AND THE VALVES. Q = $\frac{L \cdot D \cdot (P \cdot 0.5)}{795,000}$ WHERE L = LENGTH OF PIPE SECTION BEING TESTED (M) D = NOMINAL PIPE DIAMETER (MM) P = TEST PRESSURE (KPA) NOTE:1.0 PSI = 6.89 KPA 1.15. STEEL PIPELINES SHALL BE PRESSURE AND LEAKAGE TESTED IN ACCORDANCE WITH AWWA M11 AND THE MANUFACTURERS RECOMMENDATIONS, UNLESS OTHERWISE SPECIFIED. 1.16. FOR FUSED POLYETHYLENE PIPE THE HYDROSTATIC TEST CONSISTS OF TWO STEPS: THE INITIAL EXPANSION PHASE AND THE TEST PERIOD. THE MAIN SHALL INITIALLY BE PRESSURED UP TO 1.5 TIMES THE STANDARD PRESSURE RATING OF THE PIPE WITH PRESSURE MEASURED AT THE LOWEST ELEVATION IN THE SECTION BEING TESTED. WATER SHALL BE ADDED EACH HOUR DURING THIS PERIOD TO MAINTAIN THE PRESSURE, THEREBY COMPENSATING FOR THE EXPANSION OF THE PIPE. THE INITIAL EXPANSION PERIOD SHALL BE NO LESS THAN 3 HOURS. AT THE END OF THE THREE HOUR INITIAL PERIOD, THERE SHOULD BE A NOTICEABLE DROP IN THE AMOUNT OF MAKE-UP WATER REQUIRED. FOLLOWING THE EXPANSION PERIOD, A 1 TO 3 HOUR TEST PERIOD WILL COMMENCE. THE PIPE SHALL BE PRESSURED UP TO THE TEST PRESSURE AND AT THE END OF THE TEST PERIOD, THE VOLUME OF MAKE-UP WATER MEASURED. THE AMOUNT OF MAKE-UP WATER SHOULD NOT EXCEED THE VOLUME IN THE FOLLOWING TABLE:
1	24-12-18	FOR PRELIMINARY DESIGN REPORT
ISSUE	YY-MM-DD	REVISION
		MON DD, YYYY
MPE a division of Englobe		
WATERSMART A HAZEN COMPANY		
GREENVIEW INDUSTRIAL GATEWAY WATER INTAKE AND RAW WATER TRANSFER STATION CIVIL GENERAL SPECIFICATIONS		
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TABLE NO. 2 MAXIMUM MAKE-UP WATER ALLOWANCE FOR EXPANSION UNDER TEST PRESSURE AT 73.4F (23.0C) PER 100 FT (30.5 METRES) OF PIPE

NOMINAL PIPE SIZE		ALLOWABLE MAKEUP VOLUME (1hr TEST)		ALLOWABLE MAKEUP VOLUME (2hr TEST)		ALLOWABLE MAKEUP VOLUME (3hr TEST)	
M	(mm)	(U.S. gal)	(litres)	(U.S. gal)	(litres)	(U.S. gal)	(litres)
3	75	0.10	0.38	0.15	0.57	0.25	0.95
4	100	0.13	0.49	0.25	0.95	0.40	1.50
6	150	0.30	1.10	0.60	2.30	0.90	3.40
8	22	0.50	1.90	1.00	3.80	1.50	5.70
10	250	0.70	2.70	1.3	4.90	2.10	8.00
11	280	1.00	3.80	2.00	7.60	3.00	11.40
12	300	1.10	4.20	2.30	8.70	3.40	12.90
14	350	1.40	5.30	2.70	10.20	4.20	15.90
16	400	1.70	6.40	3.30	12.50	5.00	18.90
18	450	2.20	8.30	4.30	16.30	6.50	24.60
20	500	2.70	10.20	5.50	20.80	8.00	30.30

FOR TEST TEMPERATURES OTHER THAN 23 DEGREES C., THE FOLLOWING CORRECTION FACTOR SHOULD BE MULTIPLIED BY THE EXPANSION ALLOWANCE VOLUME:

TEMPERATURE	CORRECTION FACTOR
23	1.00
21	0.87
18	0.75
16	0.66
10	0.58
5	0.42
0	0.30

TEST TEMPERATURE IS THE AVERAGE BETWEEN THE AMBIENT AIR TEMPERATURE AND THE TEST TEMPERATURE OF THE WATER INTRODUCED INTO THE MAIN.

- 1.17. FOR POLYETHYLENE PIPE, UNDER NO CIRCUMSTANCES SHOULD THE TOTAL TIME A
- 1.18. TIMES THE PRESSURE RATING EXCEED EIGHT (8) HOURS. IF THE TEST IS NOT COMPLETE WITHIN THE TIME PERIOD FOR WHATEVER REASON, THE TEST SECTION SHOULD BE PERMITTED TO "RELAX" FOR AN ADDITIONAL EIGHT HOUR PERIOD PRIOR TO STARTING THE NEXT TEST SEQUENCE.
- 1.19. AFTER TESTING, LOCATE AND REPAIR DEFECTS IF LEAKAGE IS GREATER THAN AMOUNT SPECIFIED.
- 1.20. REMOVE JOINTS, FITTINGS AND APPURTENANCES FOUND DEFECTIVE AND REPLACE WITH NEW SOUND MATERIAL AND MAKE WATERTIGHT.
- 1.21. REPEAT HYDROSTATIC TEST UNTIL ALL DEFECTS HAVE BEEN CORRECTED.
- 1.22. REPEAT TEST UNTIL LEAKAGE IS WITHIN SPECIFIED ALLOWANCE FOR FULL LENGTH OF WATERMAIN.

SWABBING

1. DISASSEMBLE PIPING AT LAUNCH SITE AND RECEIVING SITE, OR TEMPORARILY MODIFY PIPING AS REQUIRED IN ORDER TO FACILITATE LAUNCHING AND RETRIEVAL OF THE SWABS.
2. INSERT SWAB AND CONNECT PUMPING EQUIPMENT TO THE LAUNCH POINT. FORCE THE SWAB THROUGH THE SYSTEM WITH WATER PRESSURE UNTIL IT EMERGES FROM THE RECEIVING POINT.
3. MONITOR PRESSURE CAREFULLY. DO NOT EXCEED SAFE OPERATING PRESSURE FOR THE WATER LINE.
4. PROTECT AGAINST EROSION AT THE SITE OF THE RECEIVING POINT. SUBMIT PLAN TO OWNER'S REPRESENTATIVE WELL IN ADVANCE OF SWABBING OPERATIONS.
5. DISPOSE OF WASTE AT AN APPROVED LANDFILL LOCATION.
6. REASSEMBLE HYDRANTS, AND/OR REMOVE PIPING MODIFICATIONS AS REQUIRED AFTER COMPLETION OF SWABBING. ALL DISASSEMBLY AND REASSEMBLY OF HYDRANTS IS TO BE DONE BY QUALIFIED PERSONNEL ONLY AND UNDER SUPERVISION OF THE OWNER'S REPRESENTATIVE. ALL REASSEMBLY MUST BE IN ACCORDANCE WITH THE HYDRANT MANUFACTURER'S SPECIFICATIONS.
7. ONCE SWABBING OF THE WORK HAS BEEN COMPLETED, FLUSH AND DISINFECT.

GRASS SEEDING

1. PRODUCTS

1.1. 11 KG/HA MEADOW BROME GRASS
1.4 KG/HA ALFALFA
36 KG/HA OATS COVER CROP
28 KG/HA 11-51-0 FERTILIZER
- 1.2. APPLY SEED MIXTURE TO MEET THE FOLLOWING APPLICATION RATES:

.1 BROADCAST APPLICATION RATE 34 KG/HA.
- 1.3. PROVIDE SEED AND SEED MIXTURE THAT ARE FREE OF ALL PROHIBITED NOXIOUS WEED SEEDS.
- 1.4. PROVIDE CANADA NO. 1 GRADE SEED IN ACCORDANCE WITH THE GOVERNMENT OF CANADA SEEDS ACT AND SEEDS REGULATIONS. DO NOT PROVIDE SEEDS WHICH ARE WET, MOULDY, OR OTHERWISE DAMAGED.
- 1.5. PROVIDE FERTILIZER IN ACCORDANCE WITH GOVERNMENT OF CANADA FERTILIZER ACT AND FERTILIZERS REGULATIONS. FERTILIZER IS TO BE COMPATIBLE WITH THE SEED MIXTURE AND THE SOIL CONDITIONS.

CARE OF WATER

1. GENERAL

1.1 REGULATORY REQUIREMENTS

.1 COMPLY WITH THE CONDITIONS OF THE PERMITS FOR THE PROJECT OBTAINED BY THE OWNER.

.2 MAKE ARRANGEMENTS WITH THE OWNER, LANDOWNERS, OR OTHER AGENCIES THAT MAY BE AFFECTED BY DISPOSAL OF WATER, SNOW, OR ICE. OBTAIN ANY PERMITS REQUIRED IN ADDITION TO THOSE OBTAINED BY THE OWNER.

1.2 SITE CONDITIONS

.1 PROJECT AREA IS IN MELFORT, SASKATCHEWAN.

.2 THE SITE IS LOCATED WHERE GROUNDWATER IS PRESENT AND SEEPAGE INTO EXCAVATIONS AND AREAS OF FILL PLACEMENT MAY OCCUR DURING CONSTRUCTION.
2. PRODUCTS

2.1 EQUIPMENT AND MATERIALS

.1 PROVIDE ALL PUMPS, HOSES AND RELATED EQUIPMENT AND POWER SOURCES REQUIRED FOR CARE OF WATER.

.2 MAINTAIN PUMPS IN GOOD OPERATING CONDITION AT ALL TIMES.

- .3 HAVE AT THE SITE AT ALL TIMES, AT LEAST ONE STANDBY PUMP FOR EACH CATEGORY OF PUMP REQUIRED FOR CARE OF WATER.
- .3 INSTALL A REPLACEMENT PUMP OR PUMPS OF EQUAL CAPACITY BEFORE REMOVING A PUMP OR PUMPS FOR MAINTENANCE.
- 2.2 DESIGN OF CARE OF WATER PROVISIONS

.1 INCLUDE PROVISIONS FOR HANDLING GROUNDWATER, RAINSTORM RUNOFF, SNOW, SNOWMELT, AND ICE THAT MAY ENTER THE WORK AREAS IN THE DESIGN OF THE CARE OF WATER MEASURES.

.2 DESIGN DEWATERING SYSTEMS THAT ARE CAPABLE OF LOWERING AND MAINTAINING THE GROUNDWATER LEVEL TO PERMIT CONSTRUCTION OF THE WORK TO BE CONDUCTED IN THE DRY, AND ON A STABLE FOUNDATION, WITH NO LOSS OF FOUNDATION MATERIALS OR MATERIALS FROM EXCAVATED SURFACES.

.3 DESIGN PROTECTIVE WORKS, INCLUDING ENCLOSURES, INSULATION, AND HEATING SYSTEMS, TO ENSURE THAT DEWATERING MEASURES OPERATE CONTINUOUSLY AND DO NOT BECOME FROZEN DURING COLD WEATHER.
- 2.3 SUBMITTALS

.1 PROVIDE THE FOLLOWING SUBMITTALS.

.2 A CARE OF WATER PLAN, INCLUDING SITE SPECIFIC DRAWINGS, OUTLINING THE CARE OF WATER PROVISIONS DESIGNED AS SPECIFIED IN CLAUSE 1.3 AT LEAST 14 DAYS PRIOR TO COMMENCING WORK AT THE JOBSITE.

.3 A COPY OF EACH PERMIT OBTAINED (IN ADDITION TO THOSE OBTAINED BY THE ENGINEER) UPON THE ENGINEER'S REQUEST.

.4 A COFFERDAM PLAN FOR THE INSTALLATION AND REMOVAL OF THE COFFERDAMS REQUIRED FOR THE CONSTRUCTION OF THE WORKS.
3. EXECUTION

3.1 GENERAL

.1 DESIGN, CONSTRUCT AND MAINTAIN TEMPORARY WORK, CONSTRUCT RELATED PERMANENT WORK, AS REQUIRED FOR CARE OF WATER, INCLUDING ALL NECESSARY COFFERDAMS, CHANNELS, FLUMES, DRAINS, SANDPOINTS, WELLS AND SUMPS AND OTHER TEMPORARY DIVERSION AND PROTECTIVE WORKS AND FURNISH ALL MATERIALS REQUIRED THEREFOR. FURNISH, INSTALL, MAINTAIN AND OPERATE ALL NECESSARY PUMPING AND OTHER EQUIPMENT, FOR DEWATERING THE VARIOUS PARTS OF THE WORK AND FOR MAINTAINING THE FOUNDATION AND OTHER PARTS OF THE WORK FREE OF WATER, ICE AND SNOW FROM WHATEVER SOURCE.

.2 MAINTAIN ALL SUMPS, TRENCHES AND DISCHARGE LINES TO ENSURE PROPER CONTAINMENT AND FREE FLOW OF WATER TO AND FROM THE PUMPS AND OTHER DIVERSION AND PROTECTIVE WORKS AT ALL TIMES.

.3 OBTAIN PERMITS, IN ADDITION TO THOSE OBTAINED BY THE OWNER.

.4 REPAIR DAMAGE TO ANY PART OF THE WORK CAUSED BY WATER OR FAILURE OF PROTECTIVE WORKS AT NO EXTRA COST TO THE OWNER.

.5 BE RESPONSIBLE FOR ADDITIONAL EXCAVATION AND SUBSEQUENT BACKFILL MADE NECESSARY BY WATER, SNOW, OR ICE.

.6 ENSURE PROCEDURES FOR "CARE OF WATER" DO NOT CAUSE POLLUTION IN THE AREA. LOCATE AND CONTROL DISCHARGES OF WATER TO AVOID CAUSING DAMAGE TO PROPERTY, POLLUTION OF WATER COURSES, NUISANCE ON ROADS, OR INJURY TO THE PUBLIC OR WILDLIFE.

.7 REMOVE OR LEVEL ALL COFFERDAMS, DRAINAGE DITCHES OR OTHER TEMPORARY WORK AFTER HAVING SERVED ITS PURPOSE SO AS NOT TO INTERFERE IN ANY WAY WITH THE OPERATION OF EXISTING DRAINAGE SYSTEMS.

.8 MAKE PROVISIONS FOR HANDLING RESIDUAL WATER, STORM RUNOFF AND SNOWMELT THAT MAY ENTER THE EXCAVATIONS FROM TIME TO TIME.

.9 MAKE ARRANGEMENTS WITH THE MINISTER, LANDOWNERS AND AGENCIES WHICH MAY BE AFFECTED BY DISPOSAL OF WATER SNOW AND ICE.

3.2 ENVIRONMENTAL PROTECTION

.1 DO NOT USE CARE OF WATER MEASURES THAT CAUSE POLLUTION.

.2 PROVIDE AND MAINTAIN SEDIMENTATION PONDS TO REDUCE THE LEVEL OF SUSPENDED SOLIDS TO WITHIN ACCEPTABLE LIMITS. DISPOSE OF SEDIMENTS IN WASTE DISPOSAL AREAS.

.3 DO NOT CAUSE DAMAGE TO PROPERTY OR NUISANCE ON ROADS, OR INJURY TO THE PUBLIC OR TO WILDLIFE DUE TO DISCHARGE OF WATER FROM THE CARE OF WATER MEASURES.

.4 DURING DEWATERING OR OTHER CARE OF WATER OPERATIONS, DO

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- NOT ALLOW WATER TO ENTER OR RETURN TO A WATERCOURSE UNLESS THE WATER ENTERING OR RETURNING IS OF EQUAL OR BETTER QUALITY THAN THE WATER IN THE WATERCOURSE.
- .5TEST WATER QUALITY IN THE WATERCOURSE TO ESTABLISH ITS QUALITY WITHIN 21 DAYS AFTER COMMENCEMENT OF THE CONTRACT. CARRY OUT FREQUENT WATER QUALITY TESTING DURING DEWATERING OR OTHER CARE OF WATER OPERATIONS TO COMPARE THE WATER QUALITY. TAKE NECESSARY MEASURES TO MODIFY THE WATER TO EQUAL OR BETTER QUALITY PRIOR TO ALLOWING IT TO ENTER OR RETURN TO THE WATERCOURSE.
- .6PROVIDE AND MAINTAIN SEDIMENT PONDS OR OTHER MEANS TO MODIFY THE WATER QUALITY PRIOR TO ALLOWING IT TO ENTER OR RETURN INTO THE WATERCOURSE OR CANAL. DISPOSE OF SEDIMENTS IN WASTE DISPOSAL AREAS.

BUILDING AND STRUCTURE EXCAVATING

1. GENERAL
- 1.1 INTENT
- .1READ THIS SECTION IN CONJUNCTION WITH OTHER SECTIONS FOR LOCATION, USE AND PLACEMENT OF BUILDING AND STRUCTURE EXCAVATION SPECIFIED HEREIN.
- 1.2 SECTION INCLUDES
- .1THIS SECTION INCLUDES REQUIREMENTS FOR EXCAVATING FOR BUILDINGS AND STRUCTURES INSIDE PERIMETER OF EACH BUILDING OR STRUCTURE, AND WITHIN A NOMINAL DISTANCE OUTSIDE PERIMETER OF EACH BUILDING OR STRUCTURE, AS REQUIRED FOR FOUNDATION AND OTHER SUBSTRUCTURE CONSTRUCTION.
2. PRODUCTS
- 1.1 NOT APPLICABLE.
3. EXECUTION
- 3.1 EXCAVATION
- .1EXCAVATE TO ELEVATIONS AND DIMENSIONS INDICATED ON DRAWINGS WITHIN A TOLERANCE OF ±25 MM, AND EXTENDING A SUFFICIENT DISTANCE FROM FOOTINGS AND FOUNDATION WALLS TO PERMIT PLACING AND REMOVAL OF CONCRETE FORMWORK, INSTALLATION OF SERVICES, OTHER REQUIRED CONSTRUCTION, AND FOR INSPECTION.
- .2IN EXCAVATING FOR FOOTINGS AND FOUNDATIONS, TAKE CARE NOT TO DISTURB BOTTOM OF EXCAVATION. EXCAVATE BY HAND TO FINAL GRADE JUST BEFORE CONCRETE REINFORCEMENT IS PLACED.
- .3PROTECT BOTTOM OF EXCAVATIONS AND SOIL AROUND AND BENEATH FOOTINGS FROM FROST.
- 3.2 EXISTING RESEVOIR SLAB EXCAVATION
- .1EXCAVATE EXISTING MATERIAL FROM SLAB AS NEEDED FOR BUILDING CONSTRUCTION.
- .2MAXIMUM ALLOWABLE VEHICLE SIZES ON SLAB:
- 2 AXLE GVW 7,250 KG (5,000 KG MAX AXLE LOAD)
- .3NO PILE OF MATERIAL SHALL EXCEED 1000MM IN HEIGHT NOR EXCEED A TOTAL MASS OF 8,000KG.
- .4VEHICLES SHALL NOT OPERATE WITHIN 7000MM OF EACH OTHER.
- .5PRESSURE WASH EXPOSED SLAB TO REMOVE ALL DEBRIS. REMOVE WATER. SHOT BLAST TOTAL AREA TO A 6MM SURFACE PROFILE

TRENCH EXCAVATING AND BACKFILLING

1. GENERAL
- 1.1 DETAIL DRAWINGS
- 2319.01 TYPICAL TRENCH BEDDING AND TRENCH BACKFILL REQUIREMENTS
- 1.2 SECTION INCLUDES:
- .1THIS SECTION INCLUDES REQUIREMENTS FOR EXCAVATING, BACKFILLING, AND COMPACTING TRENCHES REQUIRED FOR INSTALLATION OF UNDERGROUND SERVICES, OCCURRING WITHIN AND OUTSIDE PERIMETER OF BUILDINGS OR STRUCTURES AND WITHIN AND OUTSIDE TOWN LIMITS. UNDERGROUND SERVICES MAY INCLUDE PIPES, DUCTS, DUCTBANKS, CONDUITS, CABLE AND WIRE.
- 1.3 COORDINATION
- .1COORDINATE WORK SPECIFIED IN THIS SECTION WITH OTHER RELATED WORK SPECIFIED IN OTHER SECTIONS.
2. PRODUCTS
- 2.1 FILL MATERIALS
- .1REFER TO SECTION 02265 FOR GRANULAR MATERIAL PRODUCT SPECIFICATIONS.

3. EXECUTION
- 3.1 EXCAVATION
- .1EXCAVATE TRENCHES TO THE LINES, GRADES AND ELEVATIONS SHOWN ON THE CONTRACT DOCUMENTS. FOR PIPE TRENCHES, COMPLY WITH PIPE TRENCH WIDTH SCHEDULE.
- .2GRADE AND SHAPE PIPE TRENCH TO GIVE UNIFORM AND EVEN BEARING FOR EACH LENGTH OF PIPE. DIG BELL HOLES AT EACH JOINT AS REQUIRED.
- 3.2 PLACEMENT AND COMPACTION OF FILL MATERIAL
- .1BACKFILL TRENCHES USING FILL MATERIALS AS SPECIFIED IN BACKFILLING SCHEDULE.
- .2PLACE FILL MATERIALS IN LAYERS NOT EXCEEDING LOOSE THICKNESS SPECIFIED IN BACKFILLING SCHEDULE.
- .3UNIFORMLY COMPACT EACH LAYER OF FILL TO MINIMUM PERCENTAGES OF STANDARD PROCTOR DENSITY SPECIFIED IN BACKFILLING SCHEDULE.
- 3.3 UTILITY CROSSINGS
- .1INSTALL CROSSINGS TO THE LINES, GRADES AND ELEVATIONS SHOWN ON THE CONTRACT DOCUMENTS.
- .2COMPLY WITH REQUIREMENTS OF CROSSING PERMIT ISSUED BY UTILITY COMPANY.
- 3.4 PIPE TRENCH WIDTH
- .1EXCEPT AS OTHERWISE SPECIFIED, MINIMUM AND MAXIMUM TRENCH WIDTHS, UP TO A POINT 300 MM ABOVE TOP OF PIPE, SHALL BE AS SPECIFIED IN PIPE TRENCH WIDTH SCHEDULE.
- .2MAXIMUM TRENCH WIDTHS INDICATED IN PIPE TRENCH WIDTH SCHEDULE EXCLUDE AN ALLOWANCE FOR SHORING.
- .3TRENCH WIDTH AT ANY POINT SHALL NOT BE LESS THAN TRENCH WIDTH AT ANY DEPTH BELOW SUCH POINT.
- 3.5. PIPE TRENCH WIDTH SCHEDULE

NO. OF PIPES	MINIMUM TRENCH WIDTH	MAXIMUM TRENCH WIDTH
SINGLE PIPE, 850mm DIAMETER OR LESS	300mm GREATER THAN EXTERNAL PIPE DIAMETER	450mm GREATER THAN EXTERNAL PIPE DIAMETER OR 750mm TOTAL TRENCH WIDTH, WHICHEVER IS GREATER
SINGLE PIPE, GREATER THAN 850mm DIAMETER	300mm GREATER THAN EXTERNAL PIPE DIAMETER	600mm GREATER THAN EXTERNAL PIPE DIAMETER
MULTIPLE PIPE SIDE BY SIDE	300mm GREATER THAN EXTERNAL HORIZONTAL DIMENSION ACROSS OUTER EDGES OF PIPE	600mm GREATER THAN HORIZONTAL DIMENSION ACROSS OUTER EDGES OF PIPES.

ALL PIPE SHALL BE INSTALLED WITH TYPE 2 BACKFILL WITH THE EXCEPTION OF WASTE FILL AND LANDSCAPE AREAS

3.6. BACKFILLING AND FILLING SCHEDULE

LOCATION	FILL MATERIAL	MAX. LIFT THICKNESS	MINIMUM COMPACTION	MOISTURE CONTENT
PIPE ZONE (TO 300mm ABOVE PIPE TYPE 2)	ZONE 4 MATERIAL	150mm	95%	-1% TO +2%
INTERMEDIATE PIPE ZONE TYPE 2	SUITABLE NATIVE MATERIAL	300mm	95%	-1% TO +2%
FINAL PIPE ZONE (1000mm BELOW SUBGRADE) TYPE 2	SUITABLE SITE MATERIAL	150mm	98%	-1% TO +2%

3.7. BACKFILLING SCHEDULE - STAINLESS STEEL PIPE

LOCATION	FILL MATERIAL	MAX. LIFT THICKNESS	MIN. LIFT THICKNESS	MOISTURE CONTENT
PIPE ZONE- PIPE BEDDING	ZONE 1 GRADED SAND	150mm	95%	±2%
PIPE ZONE- HAUNCHING	ZONE 1 GRADED SAND	150mm	100%	±2%
PIPE ZONE- INITIAL BACKFILL	ZONE 1 GRADED SAND	150mm	100%	±2%
INTERMEDIATE ZONE	SUITABLE NATIVE MATERIAL	300mm	95%	±2%
FINAL ZONE- TO 1.0m BELOW GRADE	SUITABLE NATIVE MATERIAL	300mm	98%	±2%

ALL STRUCTURES SUCH AS VALVES, FITTINGS, HYDRANTS, PRECAST STRUCTURES AND AIR RELEASE VALVES SHALL BE BACKFILLED ACCORDING TO SECTION 02318 ITEM 3.3

RESTORATION OF SITEWORK

1. GENERAL
- 1.1 INTENT
- .1READ THIS SECTION IN CONJUNCTION WITH OTHER SECTIONS FOR LOCATION, USE AND PLACEMENT OF RESTORATION OF SITEWORK SPECIFIED HEREIN.
- 1.2 DETAILED DRAWINGS
- .1NOT APPLICABLE.
2. PRODUCTS
- 2.1 NOT APPLICABLE.
3. EXECUTION
- 3.1 RESTORATION - GENERAL
- .1RESTORE ALL EXISTING AREAS AND SITEWORK DAMAGED OR DISTURBED DUE TO EARTHWORK OR OTHER WORK OF THIS CONTRACT, BACK TO THEIR ORIGINAL CONDITION OR BETTER.
- 3.2 LANDSCAPE WORK
- .1PROTECT THE INTEGRITY OF THE EXISTING LANDSCAPE FEATURES BY IMPLEMENTING CONSTRUCTION PROCEDURES THAT WILL MINIMIZE DAMAGES.
- .2RESTORE ALL LANDSCAPE FEATURES DAMAGED OR DISTURBED BY THE WORK, BACK TO THEIR ORIGINAL CONDITION OR BETTER. ALL COSTS ASSOCIATED WITH THIS WORK SHALL BE BORNE BY THE CONTRACTOR.
- .3MAINTAIN ALL TRESS WITHIN THE WORK SITE.
- .4MINIMIZE DAMAGE TO TREES, PLANTS AND SHRUBS DURING THE COURSE OF CONSTRUCTION.
- .5ATTEND TO DAMAGED TREES, PLANTS OR SHRUBS BY QUALIFIED PERSONNEL.
- .6ALL GRASSED AREAS STRIPPED SHALL BE RESTORED WITH TOPSOIL AND DRYLAND SEED MIXTURE.
- .7TOPSOIL AND SOD RECONSTRUCTION LIMITS WILL BE LAID OUT BY THE OWNERS REPRESENTATIVE WHEN ROUGH GRADING HAS BEEN COMPLETED.
- .8ALL LANDSCAPE WORK SHALL BE COMPLETED TO THE SATISFACTION TO THE OWNER.
- 3.3 CONCRETE STRUCTURE WORK
- .1PROTECT THE INTEGRITY OF EXISTING CONCRETE STRUCTURES. THIS INCLUDES UTILIZING SUITABLY SIZED EQUIPMENT AND IMPLEMENTING CONSTRUCTION PROCEDURES THAT WILL MINIMIZE CONCRETE DAMAGE
- .2REPLACE ANY CONCRETE STRUCTURES DAMAGED OR DISTURBED, OUTSIDE OF THE CONSTRUCTION LIMITS. ALL COSTS ASSOCIATED WITH THIS WORK SHALL BE BORNE BY THE CONTRACTOR.
- 3.4 GRAVEL WORK
- .1RESTORE ANY GRAVEL DAMAGED OR DISTURBED OUTSIDE OF THE CONSTRUCTION LIMITS, BACK TO THEIR ORIGINAL CONDITION OR BETTER. ALL COSTS ASSOCIATED WITH THIS WORK SHALL BE BORNE BY THE CONTRACTOR.

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WATERSMART A HAZEN COMPANY

GREENVIEW INDUSTRIAL GATEWAY
WATER INTAKE AND RAW WATER TRANSFER
STATION
CIVIL
GENERAL SPECIFICATIONS

DESIGNED	I.A.K., C.C.	JOB	5946-001-00
DRAWN	T.N.Q., C.C.	SCALE	NTS
DATE	NOVEMBER 2023	DRAWING	C6.4

BUILDING AND STRUCTURE BACKFILLING

1. GENERAL
- 1.1 INTENT
- .1 READ THIS SECTION IN CONJUNCTION WITH OTHER SECTIONS FOR THE LOCATION, USE AND PLACEMENT OF BUILDING AND STRUCTURE BACKFILLING SPECIFIED HEREIN.
- 1.2 SECTION INCLUDES
- .1 THIS SECTION INCLUDES REQUIREMENTS FOR:
- .1 BACKFILLING, FILLING, AND COMPACTION INSIDE PERIMETER OF EACH BUILDING OR STRUCTURE AND OUTSIDE PERIMETER OF EACH BUILDING OR STRUCTURE, ADJACENT TO FOUNDATIONS.
- .2 GRANULAR BASE CONSTRUCTION, ABOVE SUBGRADE, FOR FLOOR SLABS SUPPORTED ON GRADE, WITHIN PERIMETER OF BUILDING OR STRUCTURE.
2. PRODUCTS
- 2.1 MATERIALS
- .1 REFER TO SECTION 02250 FOR MATERIAL PRODUCT SPECIFICATIONS.
3. EXECUTION
- 3.1 PLACEMENT AND COMPACTION OF FILL MATERIALS
- .1 BACKFILL EXCAVATIONS AND FILL TO REQUIRED SUBGRADE ELEVATIONS USING FILL MATERIALS SPECIFIED IN BACKFILLING AND FILLING SCHEDULE.
- .2 PLACE FILL MATERIALS IN LAYERS NOT EXCEEDING LOOSE THICKNESS SPECIFIED IN BACKFILLING AND FILLING SCHEDULE.
- .3 COMPACT EACH LAYER OF FILL TO THE MINIMUM PERCENTAGES OF STANDARD PROCTOR DENSITY AND WITHIN MOISTURE CONTENT RANGE SPECIFIED IN BACKFILLING AND FILLING SCHEDULE.
- .4 ENSURE THAT ADEQUATE PERMANENT OR TEMPORARY HORIZONTAL BRACING IS IN PLACE PRIOR TO BACKFILLING AGAINST WALLS.
- .5 TAKE CARE TO PREVENT DAMAGE TO OR DISPLACEMENT OF WATERPROOFING, INSULATION, WEEPING TILE INSTALLATION, PIPE, CONDUIT AND OTHER WORK.
- 3.2 GRANULAR BASE CONSTRUCTION UNDER SLABS
- .1 DIRECTLY UNDER FLOOR SLABS SUPPORTED ON GRADE, PROVIDE A MINIMUM OF 150 MM COMPACTED LAYER OF 20 M
- 3.3. BACKFILLING AND FILLING SCHEDULE

LOCATION	FILL MATERIAL	MAX. LIFT THICKNESS	MIN. LIFT THICKNESS	MOISTURE CONTENT
UNDER EXTERIOR SLABS-ON-GRADE	SUITABLE NATIVE MATERIAL	150mm	98%	WITHIN ±2% OF OPTIMUM MOISTURE CONTENT
AGAINST EXTERIOR SIDE OF FOUNDATION AND UNDER LANDSCAPED AREAS	SUITABLE NATIVE MATERIAL	150mm	95%	WITHIN ±2% OF OPTIMUM MOISTURE CONTENT
AGAINST EXTERIOR SIDE OF FOUNDATION AND UNDER GRAVELED, CONCRETE OR PAVED AREAS	SUITABLE NATIVE MATERIAL	150mm	98%	WITHIN ±2% OF OPTIMUM MOISTURE CONTENT
LANDSCAPED AREAS	SUITABLE NATIVE MATERIAL	150mm	95%	WITHIN ±2% OF OPTIMUM MOISTURE CONTENT

GRANULAR MATERIALS

1. GENERAL
- 1.1 INTENT
- .1 READ THIS SECTION IN CONJUNCTION WITH OTHER SECTIONS FOR LOCATION, USE, AND PLACEMENT OF GRANULAR MATERIALS SPECIFIED HEREIN.
- .2 THIS SECTION IS INTENDED TO BE USED AS A REFERENCE SECTION; IT IS NOT A "SECTION OF WORK". ALL MATERIALS SPECIFIED IN PART 2, PRODUCTS, MAY NOT NECESSARILY BE REQUIRED.
2. PRODUCTS
- 2.1 MATERIAL QUALITY
- .1 “GRAVEL” IN GENERAL MEANS A MIXTURE OF NATURAL GRAVEL, CRUSHED GRAVEL OR CRUSHED STONE, AND NATURAL OR CRUSHED SAND, MEETING THE GRADATION LIMITS SPECIFIED BELOW FOR EACH TYPE.
- .2 USE ONLY CLEAN, SOUND, HARD, DURABLE PARTICLES, FREE FROM SILT, CLAY, SOFT SHALE, FLAKY PARTICLES, TOPSOIL, ORGANIC MATTER AND OTHER DETRIMENTAL MATERIAL.
- .3 ENSURE GRANULAR MATERIALS ARE NOT GAP GRADED AND HAVE A SMOOTH GRADATION CURVE WITH NO EXCESS OR DEFICIENCY OF ANY PARTICULAR GRAIN SIZE WITHIN THE REQUIRED RANGE.
- .4 WHERE BLENDING IS REQUIRED, THOROUGHLY MIX THE GRANULAR MATERIALS IN SUCH A MANNER THAT A HOMOGENEOUS MATERIAL OF THE SPECIFIED GRADATION IS ACHIEVED PRIOR TO PLACING OF THE MATERIAL INTO THE WORK OR STOCKPILES.
- .5 “CRUSHED GRAVEL” MEANS ANGULAR SHAPED PARTICLES OF CRUSHED GRAVEL OR STONE, WASHED, MEETING THE GRADATION LIMITS SPECIFIED. ENSURE MINIMUM OF 50% BY WEIGHT, OF MATERIAL RETAINED ON 5 MM SIEVE HAS AT LEAST ONE FACE RESULTING FROM FRACTURE.
- .6 RIPRAP AND BEDDING MATERIALS MEANS CLEAN, SOUND, HARD DURABLE ROCK PARTICLES, FREE FROM SILT, CLAY, FLAKY PARTICLES, ORGANIC MATTER AND FOREIGN SUBSTANCES. ENSURE THE RATIO OF MAXIMUM DIMENSION TO MINIMUM DIMENSION OF INDIVIDUAL STONES DOES NOT EXCEED 3.0.
- 2.2 GRANULAR MATERIALS
- .1 GRANULAR MATERIALS TO MEET GRADATION LIMITS SPECIFIED BELOW FOR EACH TYPE:

GRANULAR MATERIAL TYPE	SIEVE SIZE	% PASSING BY WEIGHT
ZONE 1 - A WELL GRADED SAND, TYPICALLY USED AS A FINE FILTER BENEATH STRUCTURES CONCRETE SLABS AND SMALL EMBANKMENT TOE DRAINS:	10mm 2mm 1mm 0.2mm 0.07mm	100 65-95 35-80 5-25 0-5
ZONE 4 - A WELL GRADED GRAVEL MATERIAL TYPICALLY USED AS A BEDDING MATERIAL:	20mm 10mm 5mm 2mm	100 20-60 5-30 2-10
ZONE 7 - WASH ROCK - TYPICALLY USED WHEN SPECIFIED AS BEDDING MATERIAL IN EXTREMELY WET OR UNSTABLE CONDITION:	20mm 10mm 5mm 2mm	100 35-95 5-25 0-10

- .2 GRANULAR MATERIALS TO MEET GRADATION LIMITS SPECIFIED BELOW FOR EACH TYPE:

SUB-BASE COURSE	
SIEVE DESIGNATION	PERCENT BY WEIGHT PASSING
	TYPE 8
50.0mm	100.0
2.0mm	0-90.0
400um	0-60.0
160um	0-25.0
71um	0-15.0
PLASTICITY INDEX	0-6.0

BASE AGGREGATE AND BASE MIX	
SIEVE DESIGNATION	PERCENT BY WEIGHT PASSING
	TYPE 33
31.5mm	100.0
18.0mm	75.0-100.0
12.5mm	50.0-75.0
5.0mm	32.0-52.0
2.0mm	20.0-35.0
900um	15.0-25.0
400um	8.0-15.0
160um	6.0-11.0
71um	
PLASTICITY INDEX	0-6.0
FRACTURED FACE %	50.0 MAXIMUM
LIGHTWEIGHT PIECES %	5.0 MAXIMUM
ALL OTHER DELETERIOUS MATERIAL %	2.0 MAXIMUM

3. EXECUTION
- 3.1 GENERAL
- .1 DRAIN, CLEAN AND MAINTAIN FOUNDATION AND SUBGRADES FREE FROM DEBRIS, SNOW, ICE, WATER, TOPSOIL OR ANY LOOSE OBJECTIONABLE MATERIAL. DO NOT PROCEED WITH GRANULAR MATERIAL, RIPRAP, GRAVEL ARMOUR PROTECTION OR ROAD GRAVEL PLACEMENT UNTIL THE OWNER HAS INSPECTED AND APPROVED THE FOUNDATIONS AND SUBGRADE AREAS.
- .2 PLACE GRANULAR MATERIALS, RIPRAP, GRAVEL ARMOUR PROTECTION AND ROAD GRAVEL TO THE LINES, GRADES AND ELEVATIONS SPECIFIED IN THE CONTRACT DOCUMENTS.
- .3 SUSPEND ALL GRANULAR MATERIAL, RIPRAP, GRAVEL ARMOUR PROTECTION AND ROAD GRAVEL PLACEMENT AT ANY TIME WHEN SATISFACTORY WORK CANNOT BE CONDUCTED DUE TO RAIN, FLOODS, SNOW OR OTHER UNSATISFACTORY CONDITIONS.
- .4 SELECT TEMPORARY STOCKPILE SITES THAT MINIMIZE POTENTIAL FOR CONTAMINATION WITH UNDERLYING SOILS.
- .5 STOCKPILE MATERIAL IN A MANNER THAT MINIMIZES SEGREGATION.
- 3.2 PLACEMENT
- .1 GRANULAR BEDDING OR GRANULAR BACKFILL MATERIALS
- .1 PLACE COURSE FILTER, FINE FILTER AND PITRUN GRAVEL MATERIAL, IN LAYERS NOT EXCEEDING 150 MM IN THICKNESS WHEN COMPACTED, TO THE LINES, GRADES AND ELEVATIONS SHOWN IN THE CONTRACT DOCUMENTS. COMPACT TO A MINIMUM DENSITY OF 95% STANDARD PROCTOR DENSITY. COMPACT EACH LAYER BEFORE PLACING THE SUCCEEDING LAYER. MAINTAIN MOISTURE CONTENT AT APPROXIMATELY 80% OF MAXIMUM.

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WATERSMART A HAZEN COMPANY

GREENVIEW INDUSTRIAL GATEWAY
WATER INTAKE AND RAW WATER TRANSFER
STATION
CIVIL
GENERAL SPECIFICATIONS

DESIGNED I.A.K., C.C.

JOB 5946-001-00

DRAWN T.N.Q., C.C.

SCALE NTS

DATE NOVEMBER 2023

DRAWING C6.5

- .2

IF ANY GRANULAR BEDDING MATERIAL IS TOO DRY TO ALLOW ADEQUATE COMPACTION, APPLY WATER INTO THE MATERIAL UNTIL UNIFORM DISTRIBUTION OF MOISTURE IS OBTAINED. CONTROL WATER APPLICATION ACCURATELY IN AMOUNTS SO THAT FREE WATER WILL NOT APPEAR ON SURFACE DURING OR SUBSEQUENT TO ROLLING OR TAMPING.
- .3

IF THE MATERIAL IS TOO WET, DRY AND SPREAD MATERIAL IN THIN LIFTS ON SUBGRADE AND PERMIT TO DRY UNTIL THE MOISTURE CONTENT IS REDUCED TO THE SPECIFIED MOISTURE CONTENT.
- .4

PROVIDE TAMPING WITH HAND OPERATED MECHANICAL TAMPERS SUCH AS VIBRATORY PLATE TAMPERS, JUMPING JACKS OR WALK-ALONG DOUBLE DRUM ROLLERS. DO NOT USE LARGE COMPACTION EQUIPMENT IN TAMPED BACKFILL ZONES.
- .2

RIPRAP

.1

PLACE RIPRAP IN SUCH A MANNER TO ENSURE THAT THE COMPLETED RIPRAP IS STABLE, WITHOUT TENDENCY TO SLIDE, AND SUCH THAT THERE ARE NO LARGE UNFILLED SPACES WITHIN THE RIPRAP. USE SMALL ROCKS TO FILL THE VOIDS BETWEEN THE LARGER ROCKS. DO NOT DROP RIPRAP MORE THAN 1.5 M. DO NOT END DUMP MATERIALS ON SLOPE AND PUSH INTO PLACE.

.2

DO NOT MOVE ANY EQUIPMENT OVER THE RIPRAP THAT IS ALREADY IN PLACE.
- .3

GRAVEL ARMOUR SLOPE PROTECTION

.1

PLACE GRAVEL ARMOUR SLOPE PROTECTION SO THAT THE FINISHED LINES OF ARMOUR ARE SMOOTH, REGULAR AND UNIFORM.

.2

PLACE 200 mm MINIMUM THICKNESS OF ARMOUR, MEASURED PERPENDICULAR TO THE SLOPE, ON UNLINED SECTIONS.

.3

PLACE 300 mm MINIMUM THICKNESS OF ARMOUR, MEASURED PERPENDICULAR TO THE SLOPE, ON LINED SECTIONS.

.4

PLACE GRAVEL ARMOUR SLOPE PROTECTION ON PVC MEMBRANE LINER BY MEANS OF OVERHEAD EQUIPMENT SUCH AS CLAMSHELLS, DRAGLINES, BACKHOES AND GRADALLS, OR OTHER MECHANICAL MEANS THAT WILL NOT RESULT IN DIRECT OR INDIRECT EQUIPMENT LOADING ON THE LINER. DO NOT DROP ARMOUR MORE THAN 1 m. DO NOT END DUMP ARMOUR ON SLOPES AND PUSH INTO PLACE.

.5

DO NOT PLACE GRAVEL ARMOUR SLOPE PROTECTION ON SUBGRADE OR PVC MEMBRANE LINER COVERED WITH SNOW OR ICE. REMOVE SNOW OR ICE PRIOR TO PLACEMENT OF ARMOUR AT NO EXTRA COST TO THE OWNER.

.6

DO NOT PLACE CONTAMINATED MATERIAL. OBTAIN EVEN DISTRIBUTION OF VARIOUS SIZES OF PARTICLES THROUGHOUT THE MASS BY SELECTIVE LOADING AT THE PIT OR STOCKPILES BY CONTROLLED DUMPING OF SUCCESSIVE LOADS DURING PLACEMENT OF BY ANY OTHER METHODS OF PLACEMENT THAT WILL AVOID SEGREGATION OF MATERIALS.

.7

COMMENCE PLACEMENT OF ARMOUR ON THE SIDE SLOPES AT THE BOTTOM AND PROCEED UP THE SLOPE. DO NOT BLADE THE ARMOUR MATERIAL OVER THE BANKS.

3.3

TOLERANCES

.1

GRAVEL ARMOUR PLACEMENT

.1

PLACE GRAVEL ARMOUR ON EARTH SUBGRADE TO A MINIMUM THICKNESS OF 150 mm AT ANY LOCATION AND AN AVERAGE THICKNESS OF 200 mm, AND 210 mm OVER 1 KM.

.2

PLACE GRAVEL ARMOUR ON PLASTIC MEMBRANE LINED SLOPES AND BOTTOM TO A MINIMUM THICKNESS OF 290 mm. ENSURE THE AVERAGE THICKNESS IS BETWEEN 300 mm AND 315 mm.

.3

PLACE GRAVEL TO THE TOP OF THE BANK, WHERE DESIGNATED IN THE CONTRACT DOCUMENTS. WHERE GRAVEL ARMOUR IS DESIGNATED TO BE PLACED AT A LEVEL BELOW THE TOP OF THE BANK, PLACE ARMOUR TO A MAXIMUM OF 50 MM ABOVE THE DESIGN ELEVATIONS SHOWN IN THE CONTRACT DOCUMENTS.

.2

RIPRAP AND BEDDING GRAVEL PLACEMENT

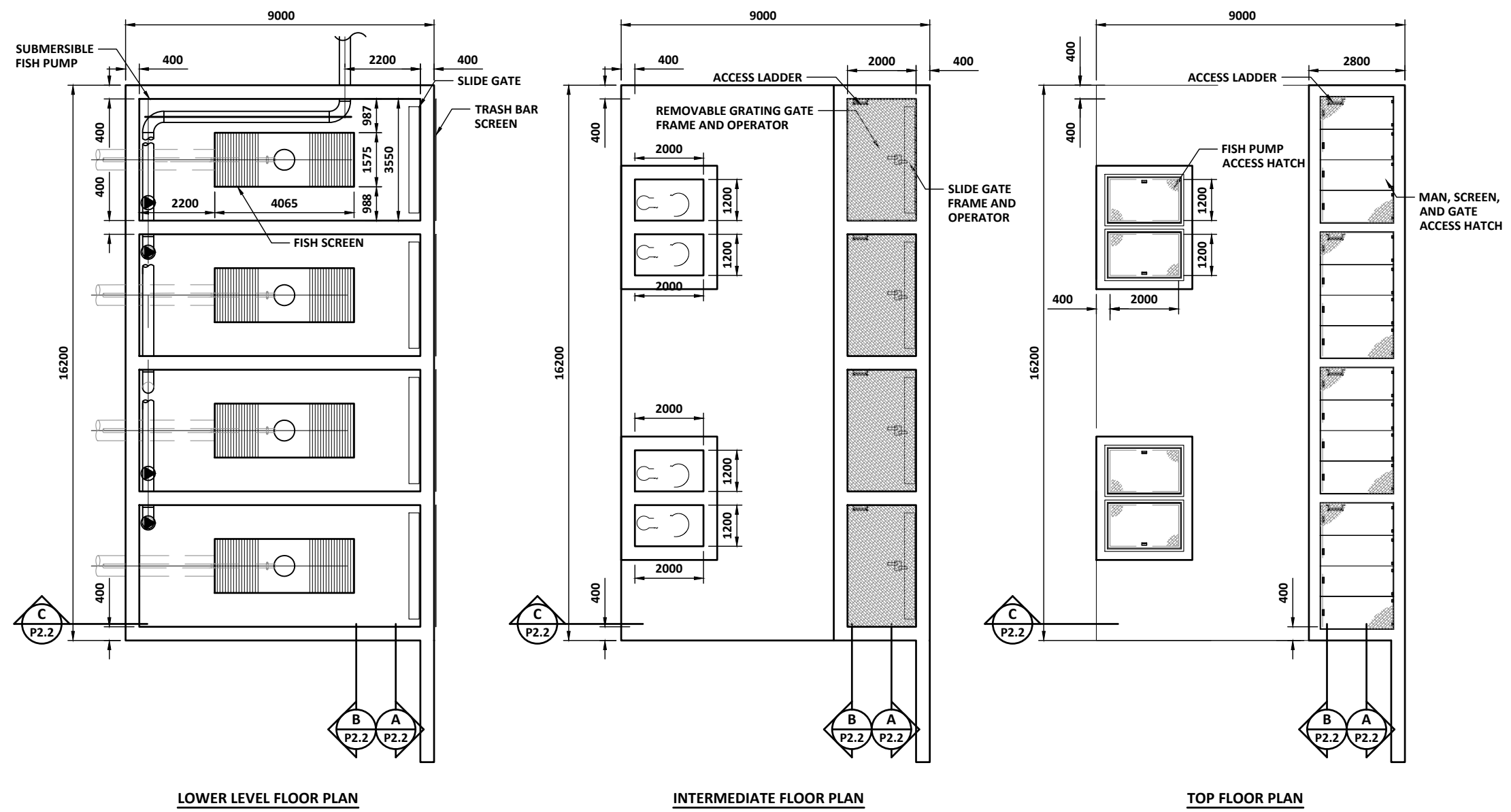
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PLACE RIPRAP AND BEDDING GRAVEL TO THE THICKNESS AND GRADES SHOWN ON THE CONTRACT DOCUMENTS TO A THICKNESS NOT TO EXCEED 120% OF THE SPECIFIED THICKNESS AND HEIGHT.

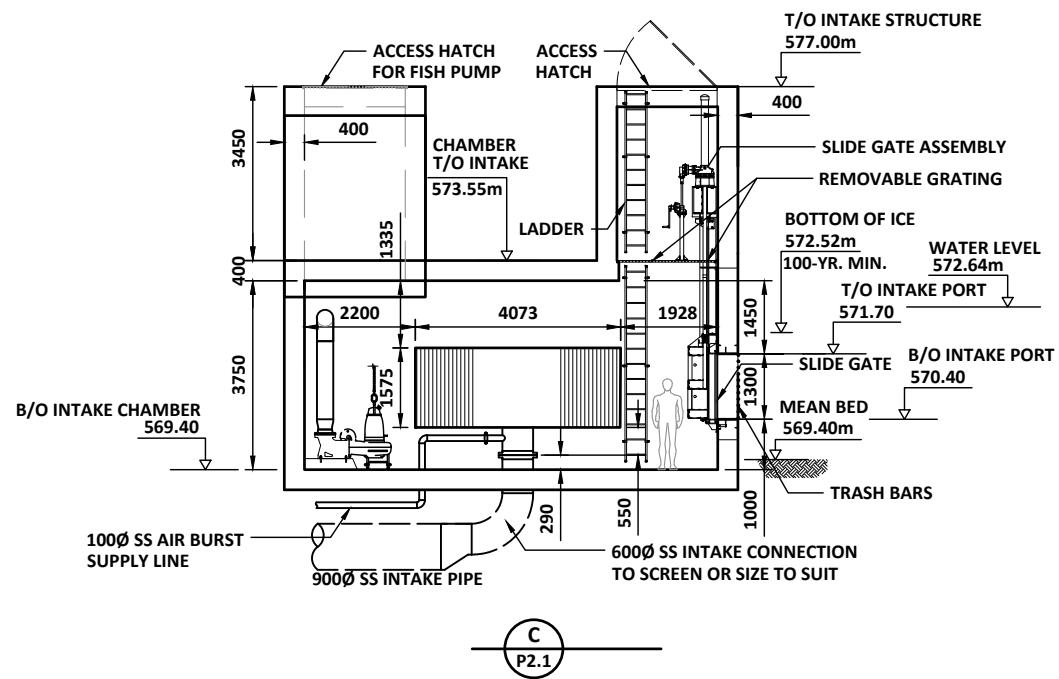
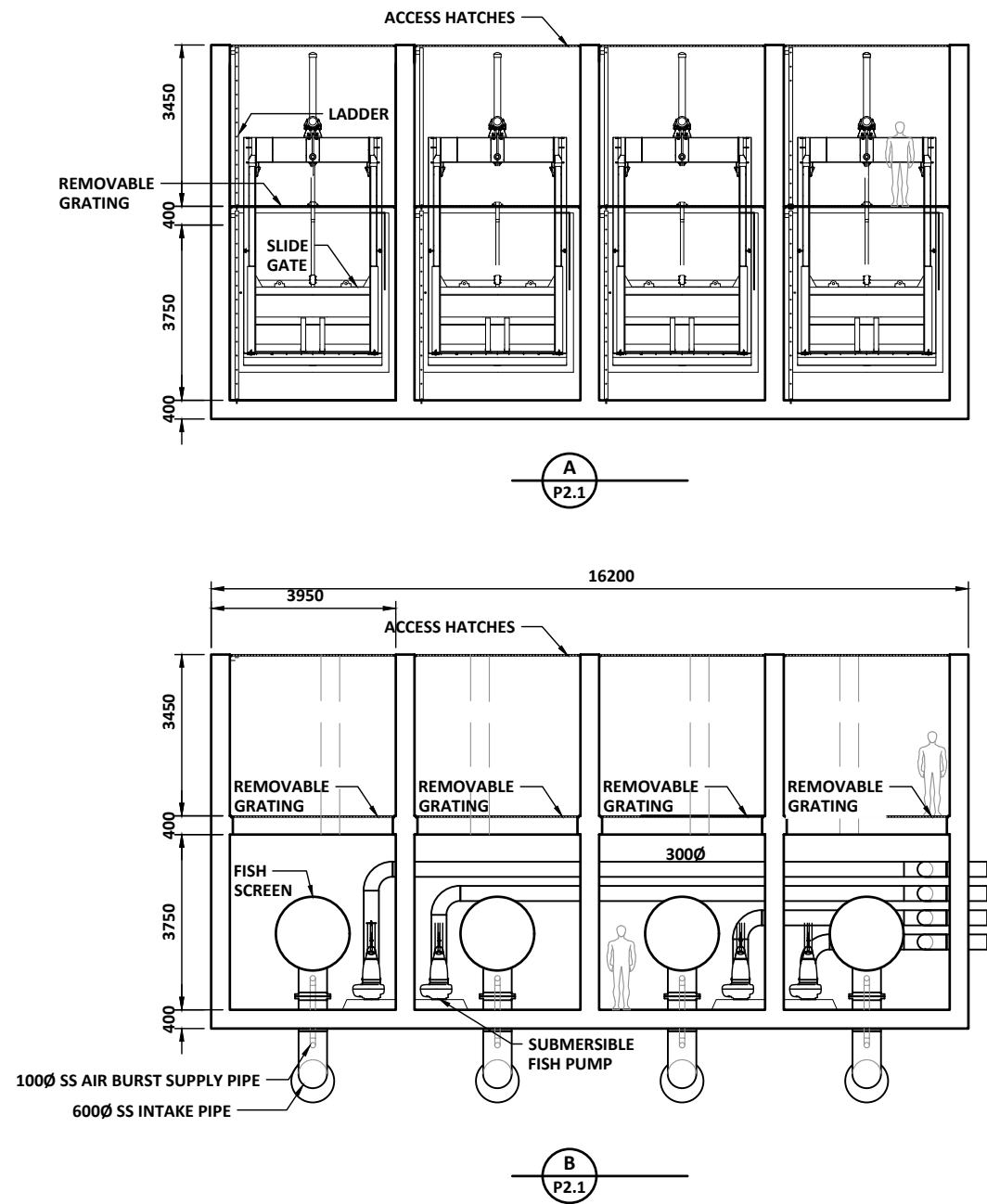
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WATERSMART A HAZEN COMPANY		
GREENVIEW INDUSTRIAL GATEWAY WATER INTAKE AND RAW WATER TRANSFER STATION CIVIL GENERAL SPECIFICATIONS		
DESIGNED	I.A.K., C.C.	JOB 5946-001-00
DRAWN	T.N.Q., C.C.	SCALE NTS
DATE	NOVEMBER 2023	DRAWING C6.6

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WATERSMART A HAZEN COMPANY		
GREENVIEW INDUSTRIAL GATEWAY WATER INTAKE AND RAW WATER TRANSFER STATION PROCESS PLAN VIEW		
DESIGNED	I.A.K.	JOB 5946-001-00
DRAWN	D.F.F., J.M.M.	SCALE 1:150
DATE	NOVEMBER 2023	DRAWING P2.1



THE EXISTENCE, LOCATION AND ELEVATION OF ALL UTILITIES AS SHOWN ON ANY PLANS MAY BE BASED ON INFORMATION RECEIVED FROM THE RESPECTIVE AUTHORITIES AND ARE NOT GUARANTEED BY THE ENGINEER. NO RESPONSIBILITY IS IMPLIED OR ASSUMED BY THE ENGINEER AS TO THE LOCATION AND ELEVATION OR ANY OMISSIONS. THE CONTRACTOR OR ANY THIRD-PARTY IS RESPONSIBLE FOR DETERMINING THE EXISTENCE, LOCATION AND ELEVATION OF ALL SUCH UTILITIES AND MUST CONTACT THE VARIOUS UTILITY COMPANIES FOR ON SITE INFORMATION PRIOR TO COMMENCEMENT OF ANY OPERATIONS.

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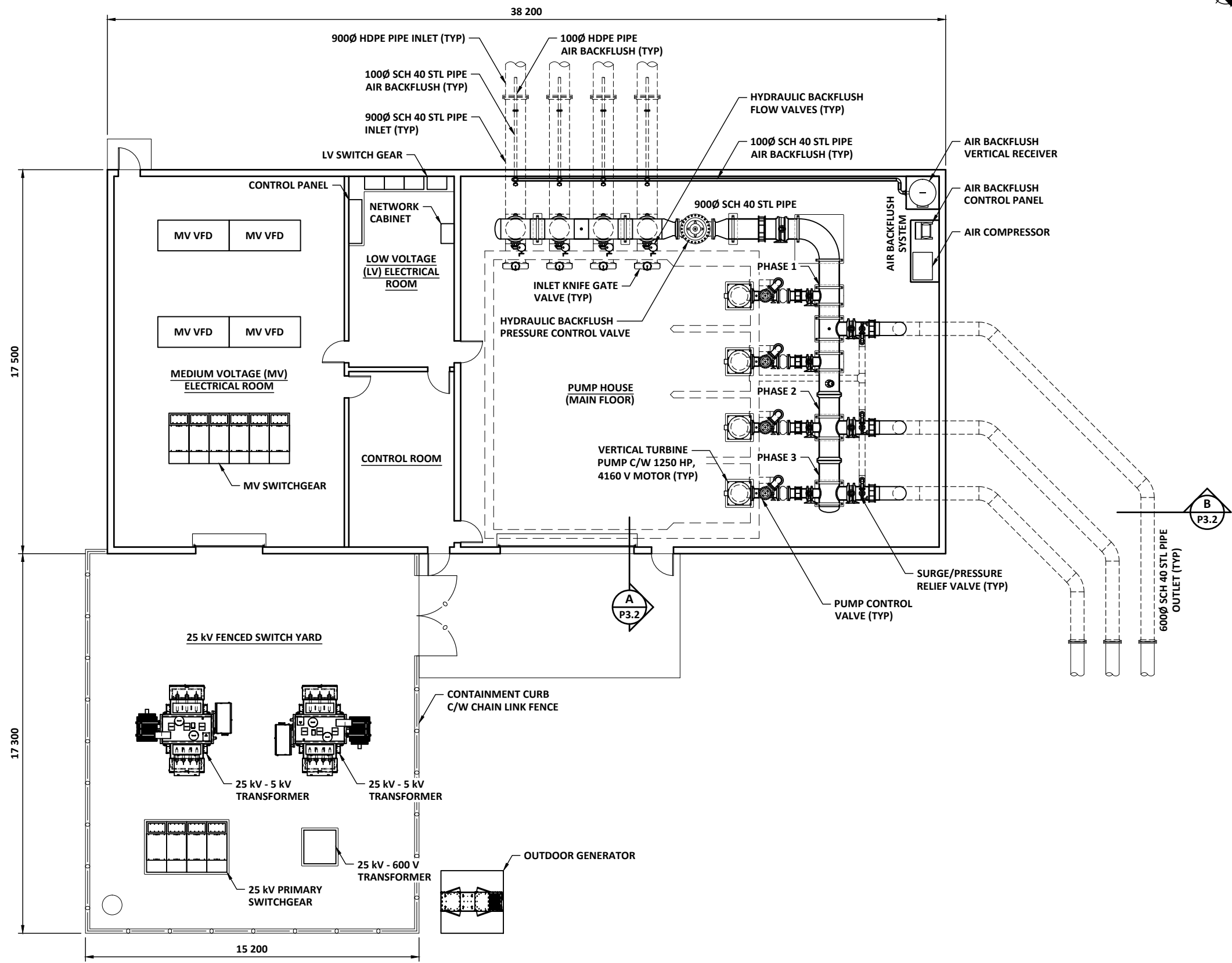
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WATERSMART A HAZEN COMPANY

GREENVIEW INDUSTRIAL GATEWAY
WATER INTAKE AND RAW WATER TRANSFER
STATION
PROCESS
SECTION VIEW

DESIGNED	I.A.K.	JOB	5946-001-00
DRAWN	D.F.F., J.M.M.	SCALE	1:150
DATE	NOVEMBER 2023	DRAWING	P2.2



- NOTES:
- EXISTING (LIGHT) SHALL BE CALIBRI LIGHT AND PROPOSED (BOLD) SHALL BE CALIBRI BOLD. ALL LEADERS MUST BE 30°. HORIZONTAL AND VERTICAL LEADERS ARE ACCEPTABLE BUT MUST BE AVOIDED WHEREVER POSSIBLE. DIMENSION STYLES HAVE MEASUREMENT SCALE FACTOR OF 1000 TO SHOW MILLIMETRES.
 - SYMBOL TEXT SHALL BE CALIBRI 1.35mm. LEADERS, NOTES, AND DIMENSIONS SHALL BE 1.65mm CALIBRI. TITLES SHALL BE CALIBRI 1.95mm.
 - NOTES SHALL HAVE A LINE SPACE FACTOR OF 1.1 AND BE SEPARATED BY A RETURN. TYPE NOTES AS USUAL AND AFTERWARDS ADD THE RETURN BETWEEN NOTES BY HOLDING SHIFT AND PRESSING ENTER.

THE EXISTENCE, LOCATION AND ELEVATION OF ALL UTILITIES AS SHOWN ON ANY PLANS MAY BE BASED ON INFORMATION RECEIVED FROM THE RESPECTIVE AUTHORITIES AND ARE NOT GUARANTEED BY THE ENGINEER. NO RESPONSIBILITY IS IMPLIED OR ASSUMED BY THE ENGINEER AS TO THE LOCATION AND ELEVATION OR ANY OMISSIONS. THE CONTRACTOR OR ANY THIRD-PARTY IS RESPONSIBLE FOR DETERMINING THE EXISTENCE, LOCATION AND ELEVATION OF ALL SUCH UTILITIES AND MUST CONTACT THE VARIOUS UTILITY COMPANIES FOR ON SITE INFORMATION PRIOR TO COMMENCEMENT OF ANY OPERATIONS.

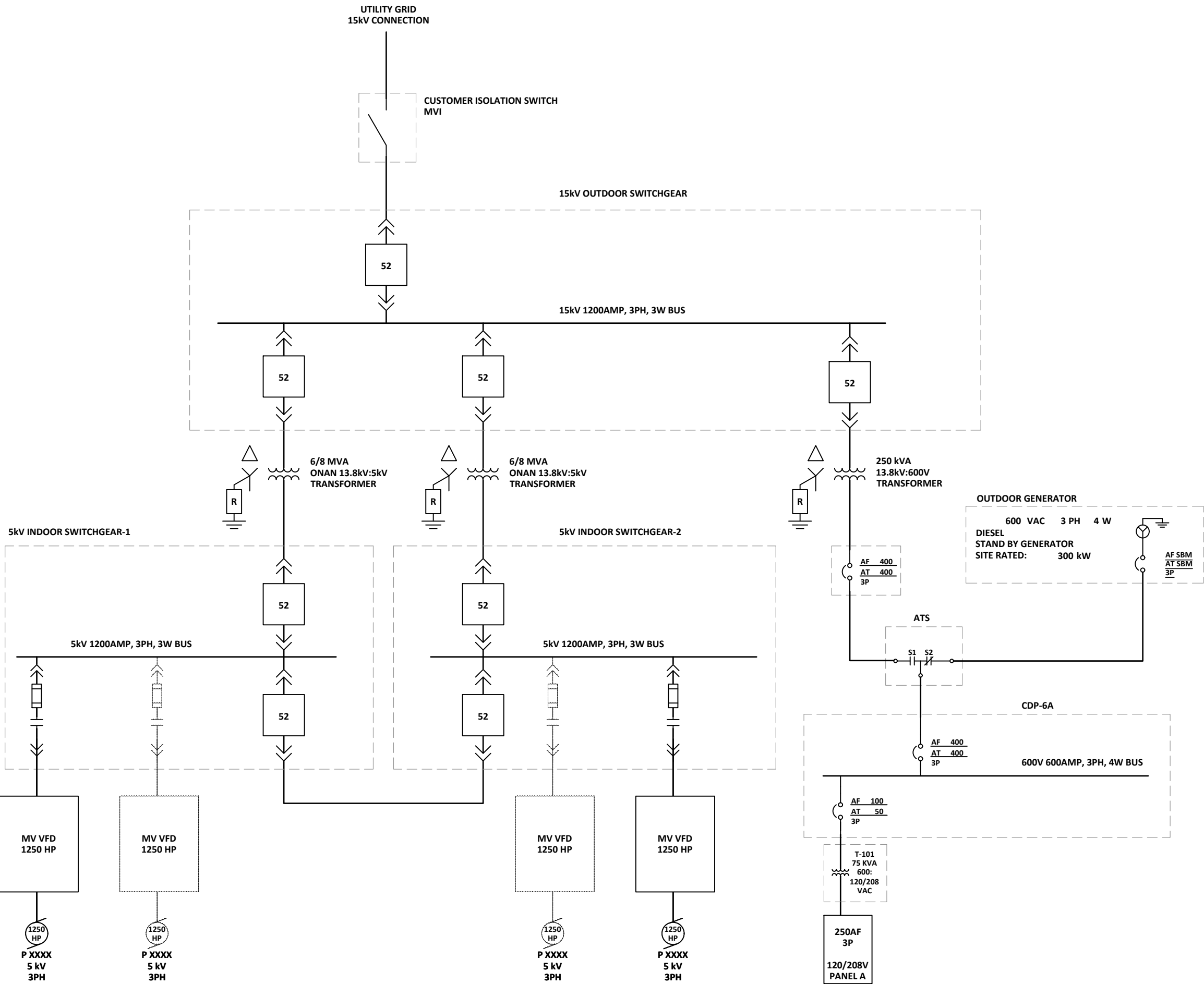
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WATERSMART A HAZEN COMPANY

**GREENVIEW INDUSTRIAL GATEWAY
WATER INTAKE AND RAW WATER TRANSFER
STATION
PROCESS
MAIN FLOOR PLAN**

DESIGNED	I.A.K.	JOB	5946-001-00
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WATERSMART A HAZEN COMPANY		
GREENVIEW INDUSTRIAL GATEWAY WATER INTAKE AND RAW WATER TRANSFER STATION ELECTRICAL SINGLE LINE DIAGRAM		
DESIGNED	N.J.S.	JOB 5946-001-00
DRAWN	N.J.S.	SCALE NTS
DATE	NOVEMBER 2023	DRAWING E2.1

Appendix B

Technical Memorandum #2 Water Intake Concept



a division of Englobe

Technical Memorandum No. 2

MUNICIPAL DISTRICT OF GREENVIEW NO. 16

Greenview Industrial Gateway

Water Intake Concept Evaluation

Date: December 18, 2024

Project No. 5946-001-00

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Appendix C	– Memo – Preliminary Cofferdam Cross Section
Appendix D	– Detailed Cost Estimate

1.0 INTRODUCTION

1.1 General

The following Water Intake Concept Evaluation Technical Memorandum has been prepared for the Municipal District of Greenview (M.D.) on behalf of WaterSMART a Hazen Company (WaterSMART), as part of the Greenview Industrial Gateway (GIG) water intake and eco-industrial zone development. This technical memorandum will discuss the preferred water intake alternative potential construction isolation and its suitability to the proposed GIG project.

1.2 Purpose

The purpose of this memo is to provide a description of the preferred water intake conceptual design and construction of work by utilizing existing data and field investigations that have been completed. This memo will also be used to discuss regulatory approvals, next steps for environmental assessments and constraints, as well as a financial analysis that will include a Class D Cost Estimate.

1.3 Scope of Assessment

The major objectives and goals of the technical memorandum are as follows:

- Provide a description of the preferred water intake alternative including location, benefits, feasibility, and constructability.
- Present results and recommendations of assessments and investigations completed by the consulting team.
- List regulatory approvals required to complete intake construction.
- Provide next steps for environmental assessments and constraints.
- Provide a Class D cost estimate based on the conceptual design.

1.4 Service Area and Location

The service area for the proposed raw water intake will encompass approximately 2,000 acres of the 18,600-acre GIG project area which is located within the following legal land descriptions.

- [10, 11, 14, 15, 16, 21, 22 & 23]-67-5-W6M

This service area is set to include six (6) lots that are expected to host petrochemical facilities as well as nine (9) raw water storage ponds. The intake location is proposed on the west bank of the Smoky River in the reach south of the Canfor Cut-Across Road bridge in NE 30-67-4-W6M.

2.0 DESIGN CRITERIA

2.1 Review of Available Information

2.1.1 Data Collection

Data for the intake concept evaluation has been collected and summarized in reports completed by Associated Engineering, Northwest Hydraulic Consultants (NHC) and Englobe's Geotechnical Team. The data was collected through various site and desktop investigations that included surveys, field assessments, and geotechnical assessments. This section will summarize the key findings and how they apply to the raw water intake concept.

2.1.2 Previous Reports

The following data, plans, reports, and manuals were compiled and reviewed to complete this report:

- Associated Engineering, Greenview Industrial Gateway Intake Environmental Overview, September 2022.
- Associated Engineering, Greenview Industrial Gateway Raw Water Pipeline Alignment Options, January 2023.
- Northwest Hydraulic Consultants, Greenview Industrial Gateway Raw Water Intake Preliminary Hydrotechnical Design, February 2023.
- Associated Engineering, Greenview Industrial Gateway Raw Water Storage Ponds Concept, March 2023.
- Englobe, Greenview Industrial Gateway Geotechnical Investigation, April 2023.
- Englobe, Preliminary Cofferdam Cross Sections – For Planning Purposes Only Rev. 1, June 2023
- Northwest Hydraulic Consultants, Greenview Industrial Gateway Raw Water Intake Preliminary Assessment of Construction Berm Hydraulics, September 2023.
- Northwest Hydraulic Consultants, Greenview Industrial Gateway Raw Water Intake Initial Ice Design - Draft, February 2024.
- Northwest Hydraulic Consultants, Greenview Industrial Gateway Raw Water Intake Preliminary Assessment of Sheet Pile Cofferdam Hydraulics, May 2024.

2.2 Water Demand

The water demands were estimated in a previous report for each lot as 10,000 m³ per day based on similar industrial use in the area. The current design has a total of 6 lots that will be serviced over three phases. It was determined that the storage ponds require 18-weeks (4.5 months) of storage because of the assumption that the raw water withdrawal from the Smoky River may not be possible for up to 18 consecutive weeks due to low flow conditions. The total storage capacity required is 7,560,000 m³, or 2,520,000 m³ per phase, which is 18-weeks of storage. The total estimated annual water demand for all 6 lots is projected to be 21,840,000 m³. The maximum daily withdrawal from the Smoky River is 109,728 m³/d (1270 L/s) with a maximum annual withdrawal of 24,000,000 m³.

2.3 Preliminary Hydrotechnical Design

2.3.1 Preferred Intake Location

NHC completed a preliminary hydrotechnical design and identified two possible intake location options, Option 1, and Option 2. Option 1 is located north of Option 2 between the river bend and south of the bridge. Option 2 is located adjacent to the Macleod Flats Recreation Area on a rock outcropping in a river narrowing at the bend upstream of the bridge.

When comparing the two options, it was determined that Option 2 is the preferred option due to its depth and avoidance of ice buildup during low water flows. Option 2 is deeper along the left bank than Option 1, which is beneficial during winter months and low river flows. Additionally, there appears to be bedrock in the bank at Option 2 which makes the bank less susceptible to erosion while Option 1 is located at the downstream end of a sand bar which showed signs of recent erosion.

Both options were reviewed by the consultancy group and the preferred Option 2 was adopted for the intake design location.

2.3.2 Preferred Intake Geometry

Two intake geometry alternatives to Option 2 (Option 2A and Option 2B) were presented in NHC's report and further reviewed in the "Intake Preliminary Selection Overview" Memorandum prepared by Englobe. The memorandum presented Option 2A as the preferred option because it has a smaller footprint and would have a lower impact on the river and the aquatic habitat than Option 2A. This consideration weighs heavily on regulatory approval requirements.

The proposed geometry for Option 2A would provide a smoother flow pattern in front of the intake ports, and a uniform hydraulic condition along the ports. This would help in clearing any debris as well as discourage fish impingement. The following provides the main characteristics and constituents of the preferred Option 2A geometry presented by NHC:

- A four (4) chamber intake structure sized for a design flow of 355 L/s per chamber. The proposed minimum internal dimensions for each chamber is 3.55 m wide by 3.75 m deep by 8.4 m long.
- A 3.1 m long by 1.3 m deep port opening at each chamber.
- A horizontal trash bar on the outside of each port opening that has an effective minimum opening area of 3.98 m². The bars should be coated with a non-stick coating to reduce adhesion of frazil ice.
- A Tee fish screen installed in each chamber and attached to the intake pipe inlet and oriented perpendicular to the ports to minimize structure size.
- A wedge-shaped intake that consists of a vertical wall containing the intake ports that is positioned at a 20 degree angle relative to the flow direction.
- A nose wall upstream of the intake that extends towards the bank at a 45° angle relative to the flow direction. This nose wall may be truncated to shorten the overall length of the intake.
- Intake Pipe should be designed for a minimum velocity of 0.5 m/s to prevent sedimentation.

A copy of the memo is attached in **Appendix A** and the preferred intake geometry for Option 2A is shown in **Figure 3.1**.

2.4 Aquatic Field Assessment

Associated Engineering, on behalf of WaterSMART, completed an aquatic field assessment for the Option 2 intake location. The assessment extended 500 m upstream and 1,300 m downstream of the proposed intake location. Bank type and fish habitat features were determined during this field assessment. It was identified that there are 23 records of fish species that have been documented in the Smoky River and McMillar Creek. Bull

Trout and Arctic Grayling habitat requirements were the only species reviewed since they are the only two that have status under the Alberta Wildlife Act. At the proposed intake location, it was determined that the habitat is not limited in this area; and does not provide critical function for any resident species that is not otherwise readily available. Therefore, the proposed location of the intake does not remove critical habitat for either species of fish.

2.5 Geotechnical Investigation

Englobe completed a geotechnical investigation to determine the subsurface conditions near the river intake. No boreholes were completed in the river to assess the conditions. The boreholes that were completed and relevant to the intake or intake pipes were located near the bank of the river (BH23-01) and near the approximate location of the proposed pump station (BH23-02). The boreholes were drilled to a depth of 15 m. Samples from the boreholes were taken at regular intervals and transported to the lab for further examination and testing. The tests completed include moisture content, hydrometer, Atterberg limits, and unconfined compressive strength. The Englobe findings were summarized in a draft report along with conclusions and recommendations regarding cofferdam options for the intake construction and soil support options for intake pipe construction. These subsurface conditions along with the conclusions and recommendations will be summarized below.

2.5.1 Land Subsurface Conditions

When analyzing the boreholes, BH23-01 and BH23-02 consisted primarily of sandstone. The sandstone was medium to coarse grained, extremely weak to weak, grey or orange-brown when oxidized. The sandstone near the surface from BH23-01 presented a much greater degree of weathering than BH23-02. Both boreholes also contained a thin layer of surficial soils, with layers of mudstone, bentonite, coal, and siltstone 7 meters below ground surface (mbgs).

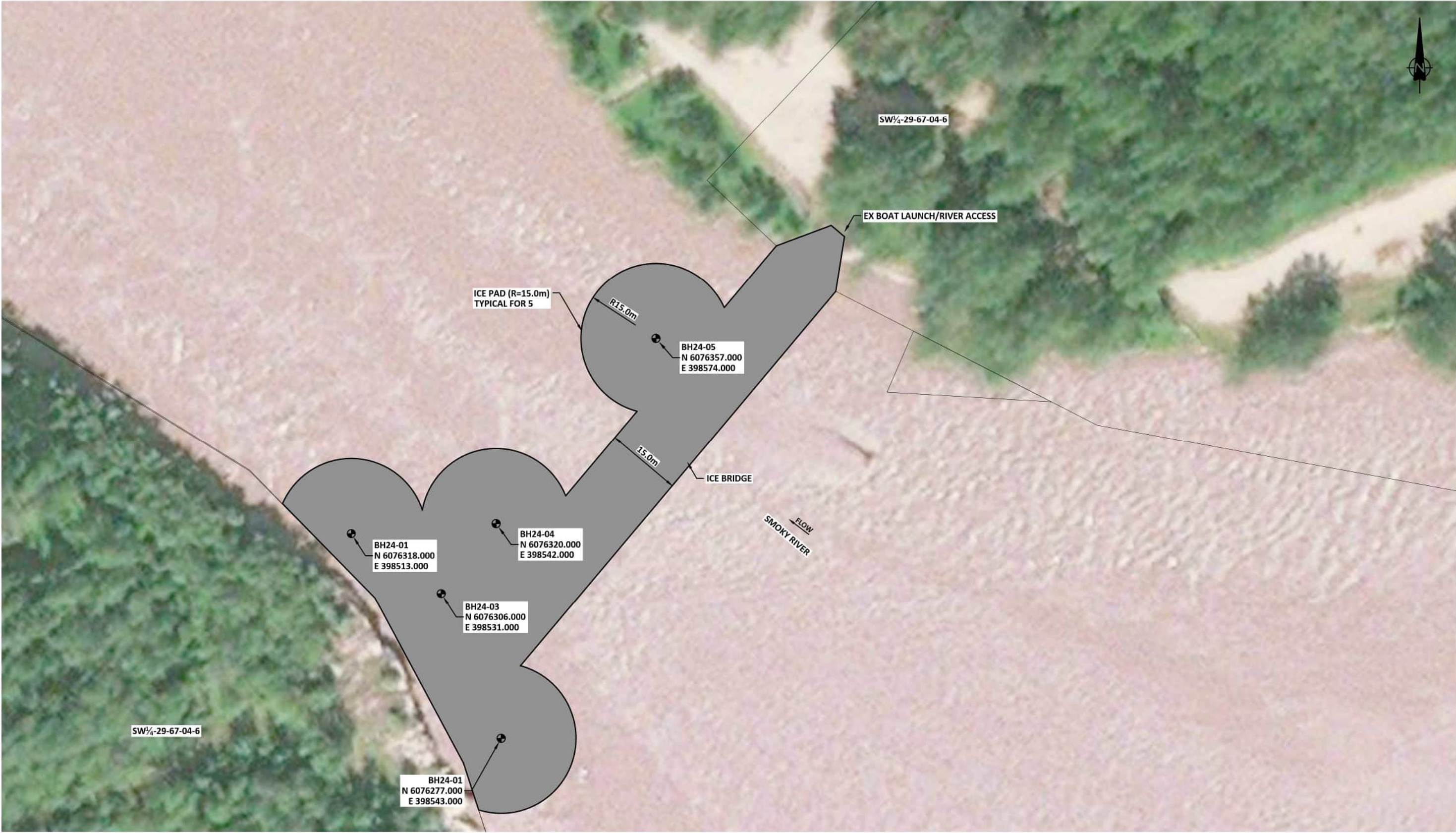
Groundwater conditions were monitored for BH23-02. The groundwater depth 15 days after piezometer installation gave a reading of 5.97 mbgs. Variation in the order of 1 m or more is possible, depending on the season, therefore it will be important to monitor groundwater levels prior to and during construction to ensure adequate support options are used during excavations.

No boreholes were completed in the river to assess the subsurface conditions of the river intake. It was therefore assumed that the riverbed is comprised of shallow alluvial or colluvial sediments overlying sandstone bedrock. Further exploration will be required to determine the actual subsurface conditions at the location of the water intake structure.

2.5.2 Riverbed Investigation

Geotechnical investigation in the Smokey River is recommended to determine the Riverbed characteristics that would facilitate the cofferdam design. Englobe recommends five (5) boreholes be drilled within the riverbed as shown in Figure 2.1 below.

M:\M-DATA\50\46 WATERSMART SOLUTIONS LTD\001 GREENVIEW INDUSTRIAL GATEWAY WATER INTAKE APPROVAL\AUTODESK\3D\004 GEOTECHNICAL\BOREHOLE PLAN\BH PLAN.DWG



NOTES:
1. TOTAL AREA OF DISTURBANCE = 0.41Ha



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WATERSMART SOLUTIONS LTD.
GREENVIEW INDUSTRIAL GATEWAY
RIVER BOREHOLE LOCATIONS AND ICE BRIDGE

SCALE:	1:750	DATE:	FEBRUARY 2024	JOB:	5946-001-00	FIGURE:	2.1
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NHC completed an ice survey of the Smokey River on Feb 8, 2024, and prepared an initial ice platform design as presented in their Initial Ice Design report. Generally, an ice platform construction could have been safely carried out based on the existing ice condition at the time and given that the temperatures remain below 0 degrees Celsius for the duration of the drilling activity.

As the risks associated with maintaining the required ice platform thickness were higher at the time of the initial review which fell in the later winter months (Feb/March 2024), this geotechnical work was differed to the 2024-2025 winter period. It is recommended that the river investigation and all associated ice platform design work be completed early in the winter period when favorable conditions become present.

2.6 Survey

NHC completed a survey of a 3.5 km reach of the Smoky River near the proposed intake location. The survey consisted of a bathymetric survey of the riverbed and a ground-based survey of the riverbanks. The survey data was supplemented with elevation data developed from drone photography. A contour plot of the riverbed and a cross section of the river at the proposed intake location were created. The contour plot helped to identify two scour holes near the left (west) bank of the river where the intake may be deep enough to accommodate the water intake. These two locations were discussed in Section 2.3.1, and as mentioned previously, Option 2 was the location selected for the intake.

3.0 PROPOSED INTAKE DESIGN OVERVIEW

3.1 General

The proposed water intake design concept includes four (4) chambers with four (4) intake screens which will help to provide some redundancy and would include the construction of the entire intake (four chamber) structure however only include mechanical installs and redundancies for each stage.

The design will also include a fish tee-screen to be installed within each chamber of the intake structure. By installing the fish screens within the chamber, the port area can be reduced to about a 1.3 m high by 3.1 m wide per chamber as recommended by NHC. Each chamber will also include a trash bar screen, access hatch, motorized sluice gates, and fish pumps to return any fish that cannot swim back out of the port to the downstream end of the river. The fish pumps would be submersed in each intake chamber. Concept plan and section view drawings of the proposed water intake design along with fish pumping can be found in **Figures 3.1 and 3.2**.

3.2 Design Horizon

An intake is a major facility that may not be easily phased or expanded and is usually designed for the lifetime of the facility. A 50-year design horizon is typical for new intake structures and pipelines.

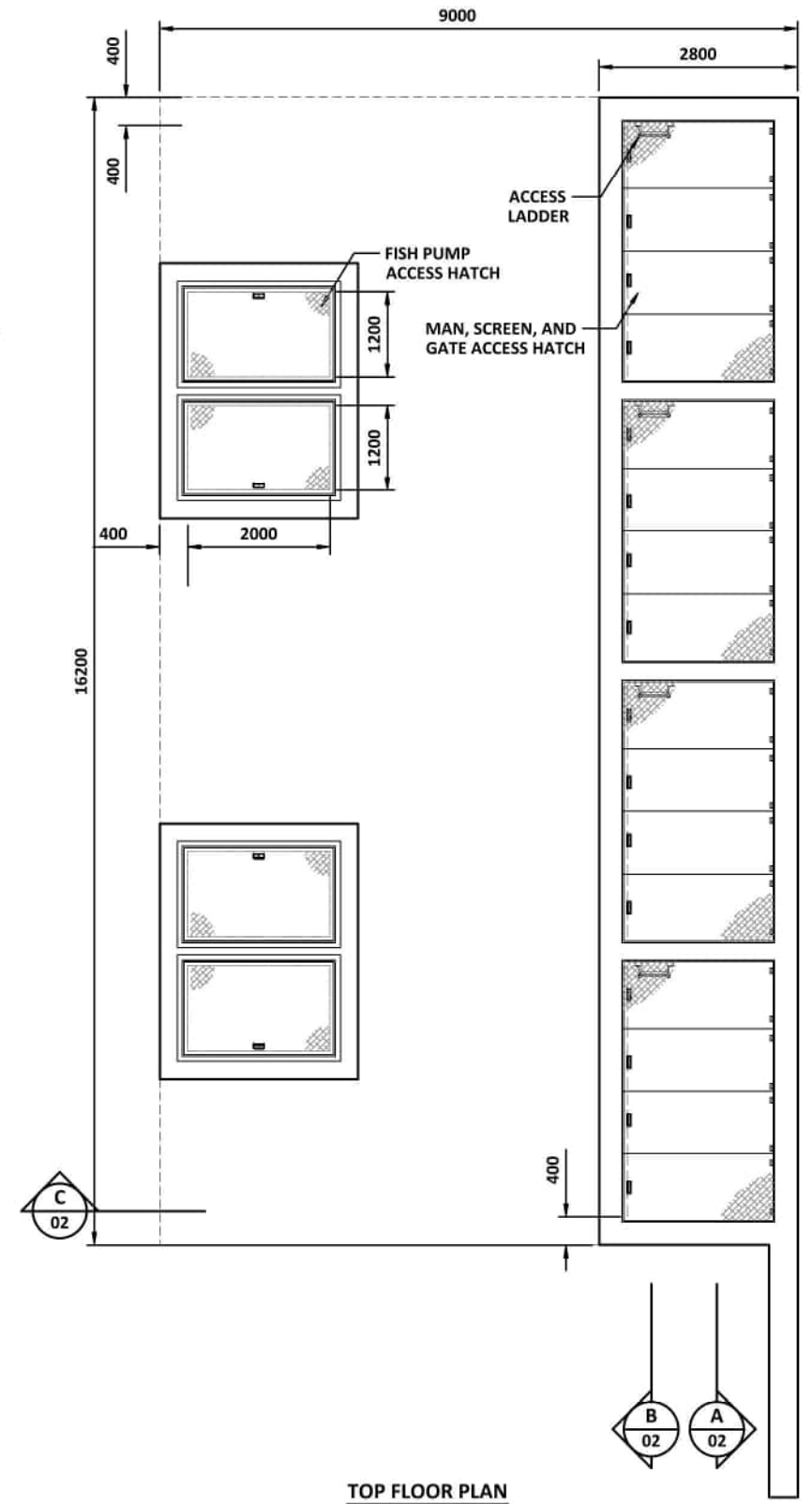
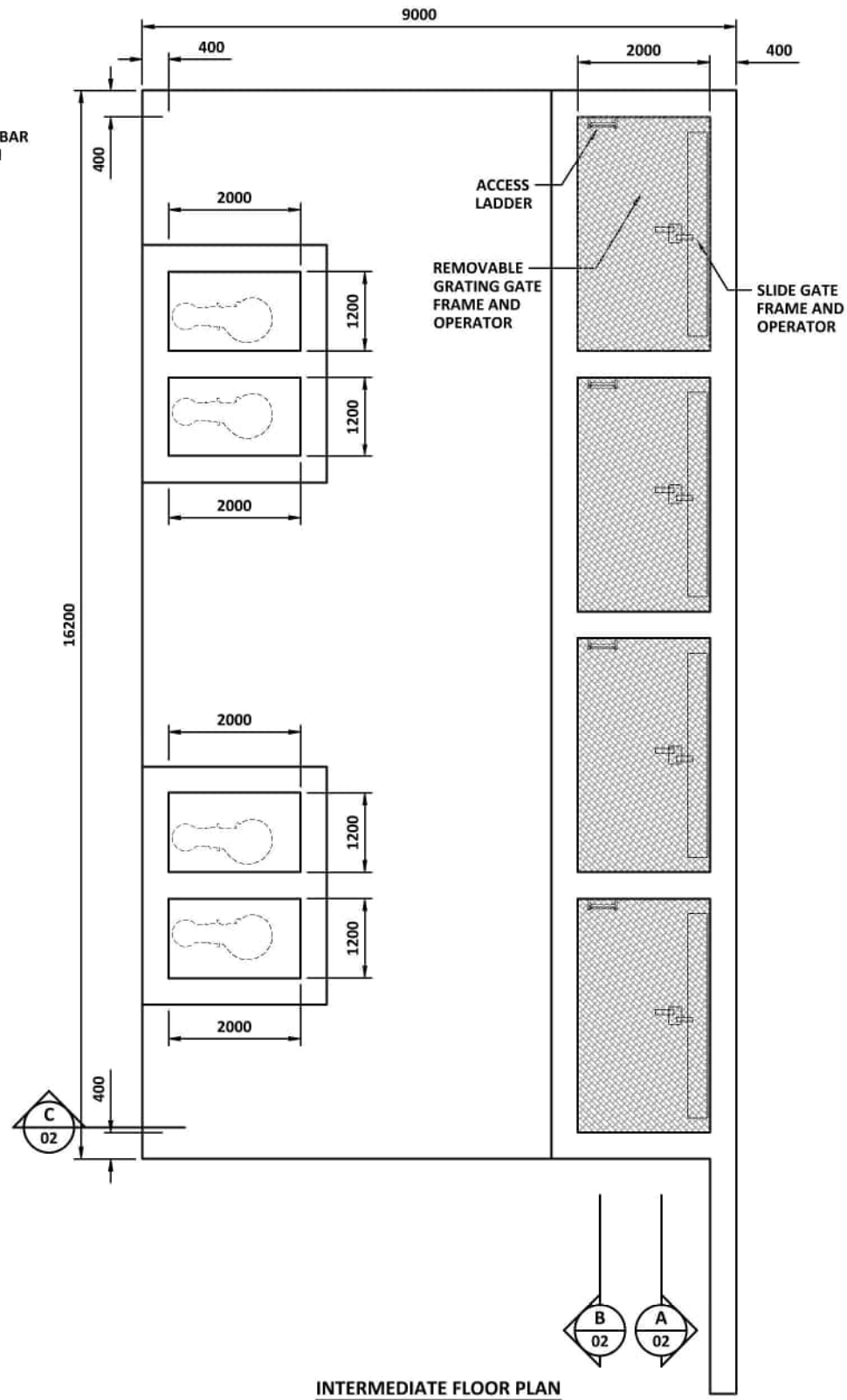
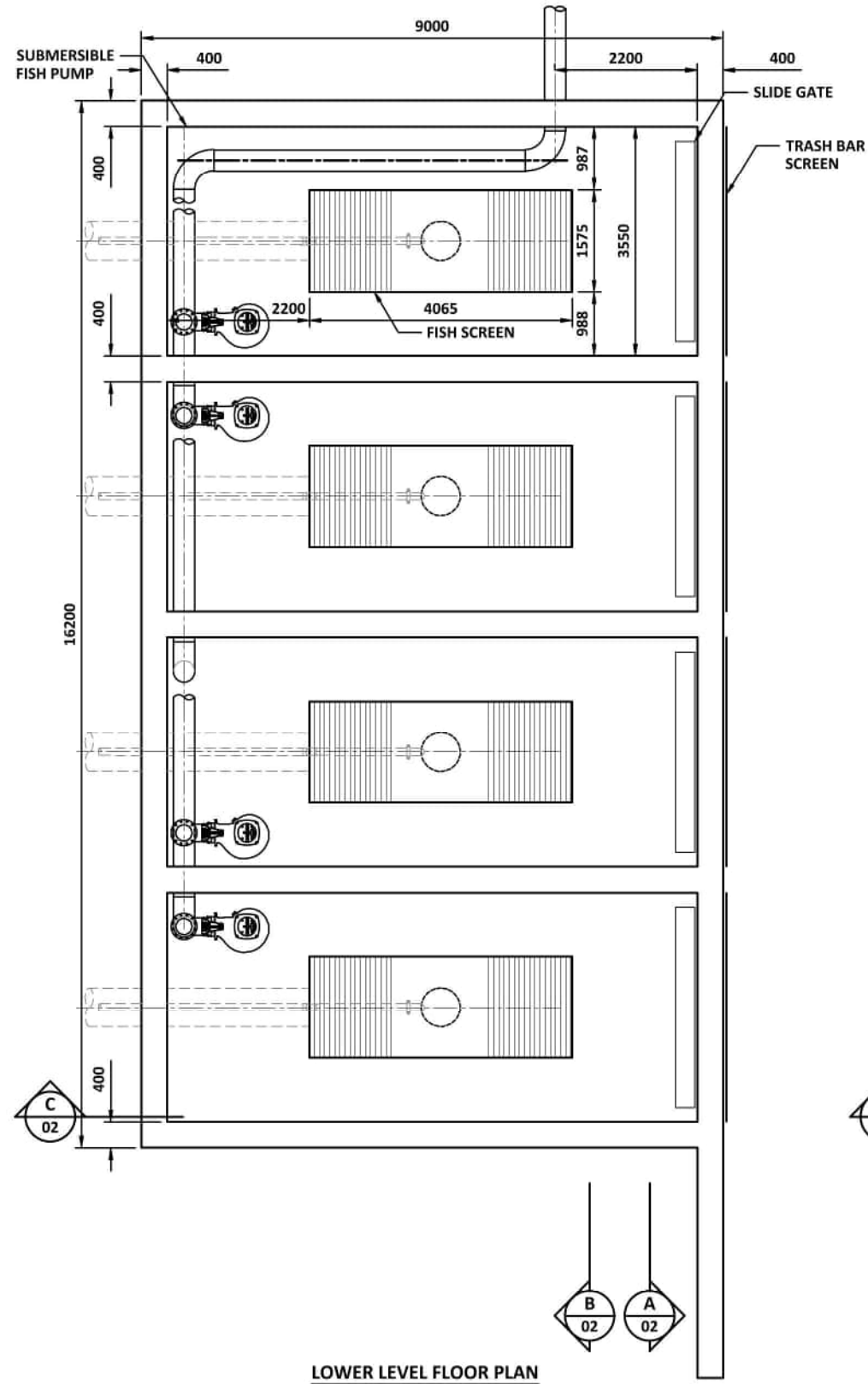
3.3 Design Flows

As noted previously the projected annual demand to service all 6 future lots is estimated at 21,840,000 m³. The current water diversion license that was applied for by the M.D. of Greenview has a maximum annual volume of 24,000,000 m³. The intake pipe design flow rate was based on the projected annual demand while accounting for minimum and maximum instantaneous diversion rates. For the project area, the design parameters are as follows:

• Annual Projected Water Demand	21,840,000 m ³
• Maximum Instantaneous Diversion Rate	1.27 m ³ /s
• Minimum Instantaneous Diversion Rate	0.68 m ³ /s
• Annual Average Diversion Rate	1.065 m ³ /s
• Intake Pipe Flow Rate	0.355 m ³ /s

The intake pipe flow rate of 1.065 m³/s (3 intake pipes at 0.355 m³/s) allows the storage cells to fill during high-flow periods which are from May to August. The 4th pipeline included with the intake provides redundancy in periods where one of the intakes needs to be taken down for maintenance. The individual pipeline capacity of 0.355 m³/s was determined based on no pumping for 18-weeks due to the possibility of low flow conditions. The current pipeline capacity would still be able to provide the annual projected water demand while only operating for 34 weeks of the year.

M:\N-DATA\50\46 WATERSMART SOLUTIONS LTD\001 GREENVIEW INDUSTRIAL GATEWAY WATER INTAKE APPROVAL\AUTODESK\ACAD\FIG.3.1_3.2.DWG



WATERSMART SOLUTIONS LTD.

GREENVIEW INDUSTRIAL GATEWAY - TM-02
WATER INTAKE STRUCTURE
PRELIMINARY PLAN VIEWS

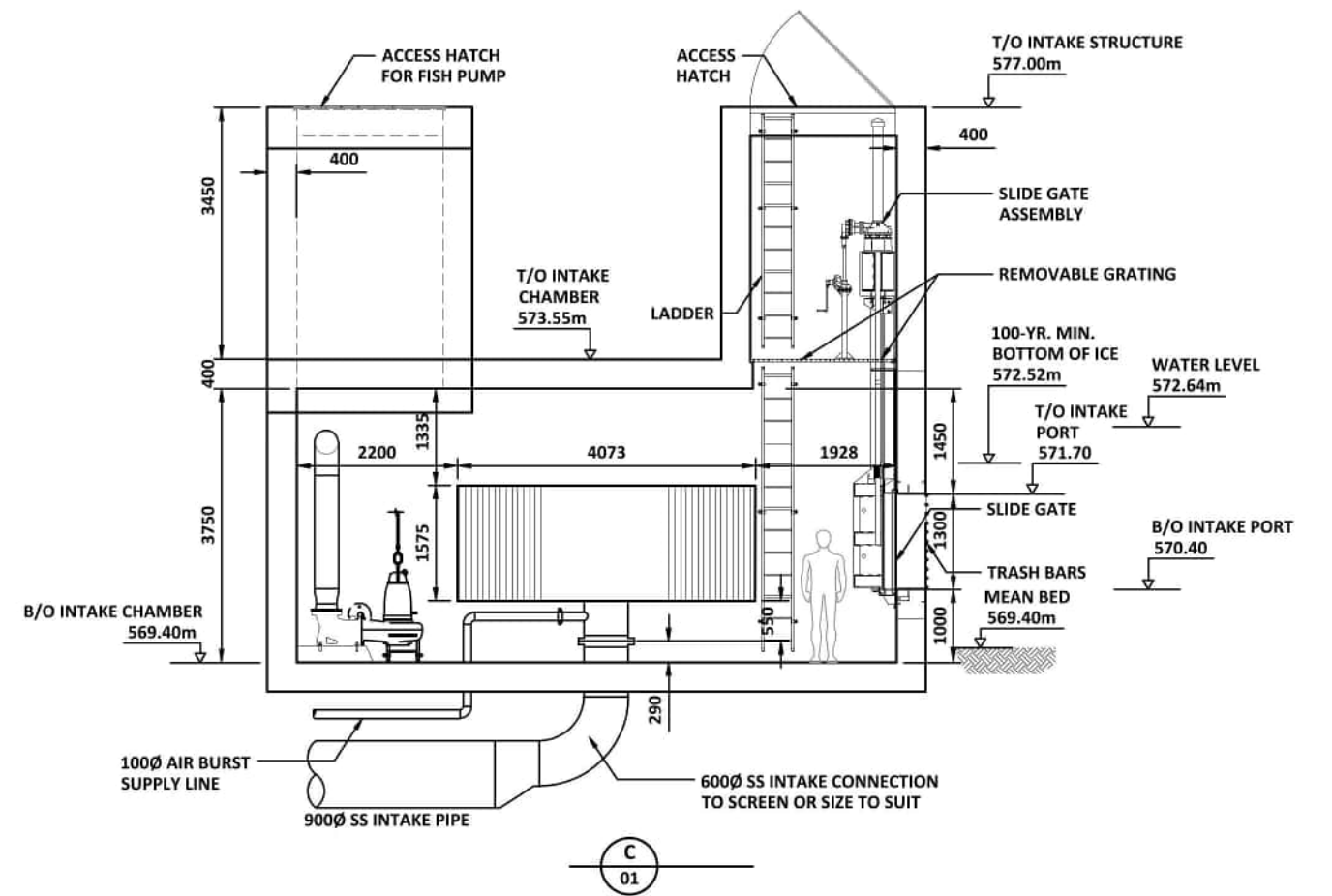
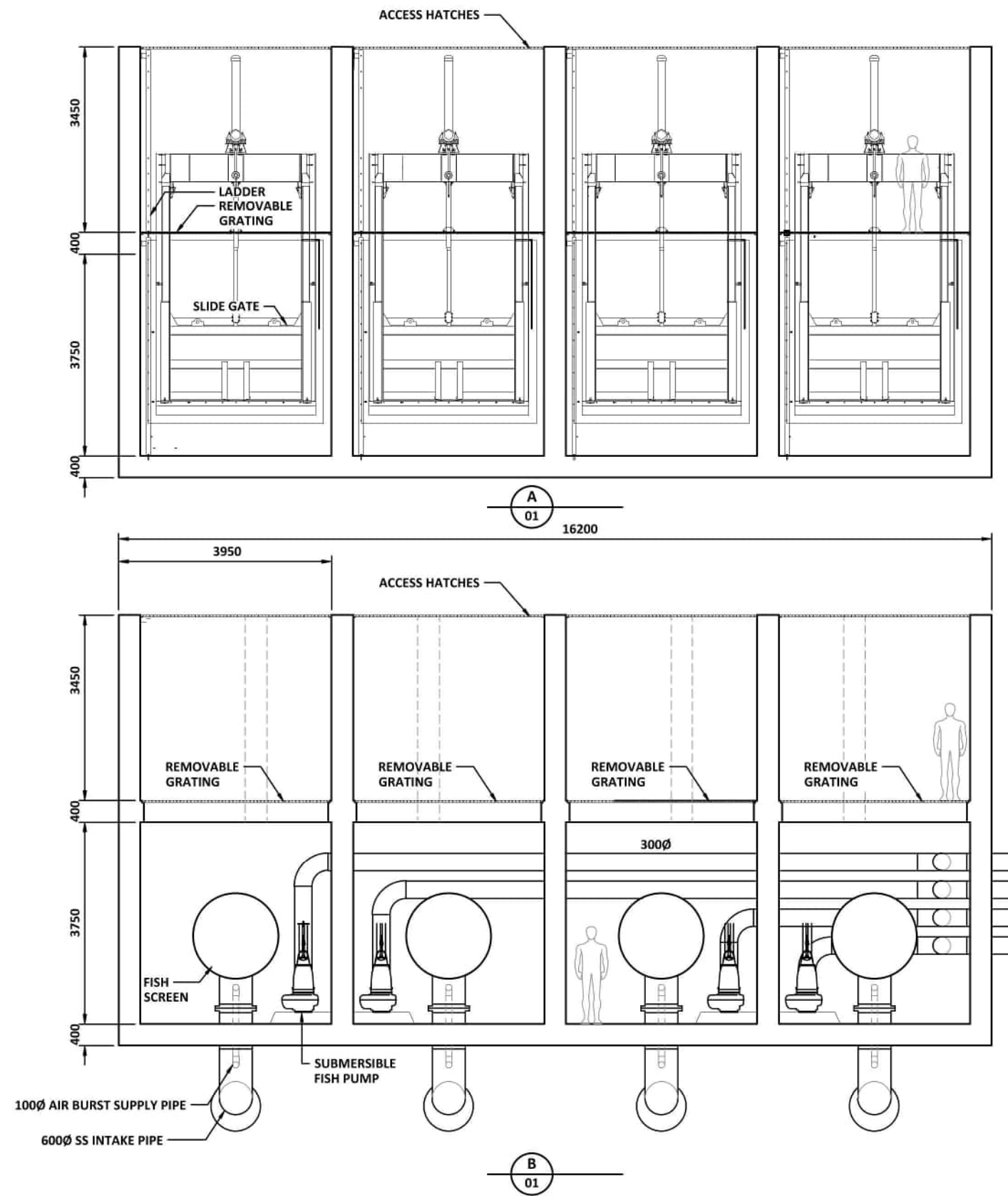
SCALE: 1:100

DATE: MARCH 2024

JOB: 5946-001-00

FIGURE: 3.1

M:\N-DATA\50\46 WATERSMART SOLUTIONS LTD\001 GREENVIEW INDUSTRIAL GATEWAY WATER INTAKE APPROVAL\AUTODESK\ACAD\FIG.3.1_3.2.DWG



WATERSMART SOLUTIONS LTD.

GREENVIEW INDUSTRIAL GATEWAY - TM-02
WATER INTAKE STRUCTURE
PRELIMINARY SECTION VIEWS

SCALE: 1:100

DATE: MARCH 2024

JOB: 5946-001-00

FIGURE: 3.2

3.4 Process Mechanical

3.4.1 Trash Bar Screens

Each individual intake chamber will include a trash bar screen to prevent debris and ice flows from entering the intake. The bars on the trash screen will be a maximum of $\frac{3}{4}$ " thick and installed horizontally within the intake ports. The bars, if made of stainless steel, should be coated with a non-stick coating to reduce adhesion of frazil ice; alternatively, the bars could be made of reinforced fiberglass plastic which has a low thermal conductivity and smooth surface. The bars will have a maximum 3" spacing to restrict larger fish from entering the chamber, and a minimum effective area of 3.98 m². This effective area limits approach velocity to 0.098 m/s which is the maximum velocity recommended for Walleye. Headloss through the trash bars is expected to be negligible.

The top of the screens will be at an elevation of 570.4 m which is 0.8 m below the 100-year minimum bottom of ice level. This will prevent damage or clogging from surface ice or debris; and further help to reduce frazil ice from entering the intake chamber during winter months.

3.4.2 Fish Screens

Fish screens will be installed in each intake chamber to protect fish and fish habitat. The current design includes utilizing a tee-screen within each intake chamber to provide the required minimum fish screen area and approach velocities. The conceptual tee-screen design has a 1.07 m diameter and are 3.8 m in length and provide an effective screen area of 11.57 m². This meets the minimum screen area required which is 10.14 m² and is required to ensure the approach velocity is less than 0.035 m/s - the maximum design velocity to allow free movement of burbot fish. The design also includes 0.254 mm slot openings.

At the design flow of 0.355 L/s, the head loss through each screen is estimated to be approximately 91 mm.

The tee-screens will be made of stainless steel and installed perpendicular to the port face to minimize the intake size.

Screen cleaning will also be required and can be completed with backwash through the intake piping and air bursting using a dedicated air burst (air compressor) supply system. The backwash flow will be provided by the transmission pumps and control valving to divert the water back into the intake piping at high velocities for proper cleaning. This debris may still collect in the chamber and would require inspection and regular cleaning as required.

Placement of the screens within the chamber shall meet the manufacturer's minimum clearance requirements.

Figure 3.3 below as retrieved from Screen Services shows typical clearance requirements for Tee Screens.

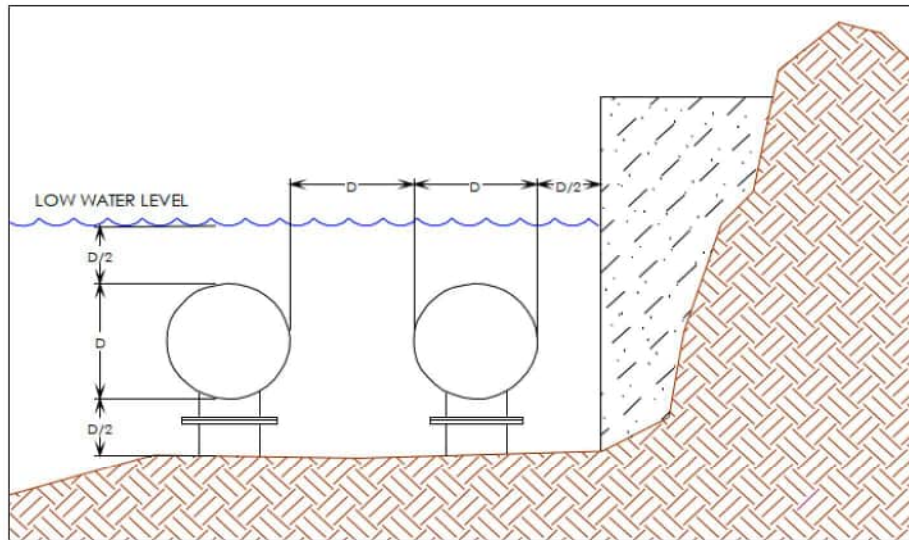


Figure 3.3: Typical Tee Screen Minimum Clearances. (Courtesy of Screen Services)

As each screen has its own chamber, consideration will be placed to have sufficient room around the screen for maintenance or operational personnel to access all sides of the screen. A minimum of 900 mm spacing around the screen sides will be adopted for the detailed design.

3.4.3 Fish Passage

Safe passage of fish is required for the intake design since small fish are still able to enter the intake chambers through the trash bar screens. Based on the conceptual layout of the intake structure with dedicated cylindrical fish screens in each chamber, a submersible fish friendly pump system would be included in the chamber and a perforated crowder barrier to guide the fish to the suction end of the pumps.

Preliminary review determined that fish should be guided towards the bottom and back corner of each chamber where the suction end of the fish pump will collect and transport the fish up through the pump and piping and discharged downstream of the intake.

Fluid dynamic modelling within the intake chamber is needed to determine the best approach and location of the crowder barrier and fish friendly pump. NHC advised that a physical laboratory model could be an easy and accurate approach to determine the layout and systems needed for safe and efficient passage of fish within each chamber.

Various fish friendly pumping systems are available; however, a submersible fish friendly pump system is proposed for the intake chambers that shall be capable of providing 35 L/s flow (10% of the chamber design capacity). Preliminary hydraulics determine that the head requirement is to be about 4 m and a pipe diameter of 200 mm. Fish friendly pumps will be specified to have the following characteristics for the safe passage and transport of fish.

- A minimum 4" (100 mm) free bowl passage which will be well suited for fish that go through the 3" wide trash bars.

- Relatively low impeller peripheral speed
- Screw type physical shape of impeller.
- Hydraulic efficiency that creates smooth flowlines avoiding pressure gradients and turbulence.
- Internal contours that avoid snagging.

During detailed design of the fish friendly pumping system, computation hydraulic analysis should be provided by the pump supplier to demonstrate the hydraulic efficiency of the pump.

3.4.4 Slide Gates

Slide gates would be provided for each intake chamber to allow for water tight isolation of a chamber for maintenance or repair of equipment. Once isolated, each chamber will have access via a hatch and ladder for personnel to complete any task required within the chambers. The slide gate would be stainless steel and is expected to be an approximately 3.0 m by 1.5 m self-contained frame gate with a double rising stem, yoke mounted tandem gearboxes, and handwheel or automated actuator. The decision to utilize a handwheel or automated actuator can be made during the detailed design stage and will depend on the ease of operation.

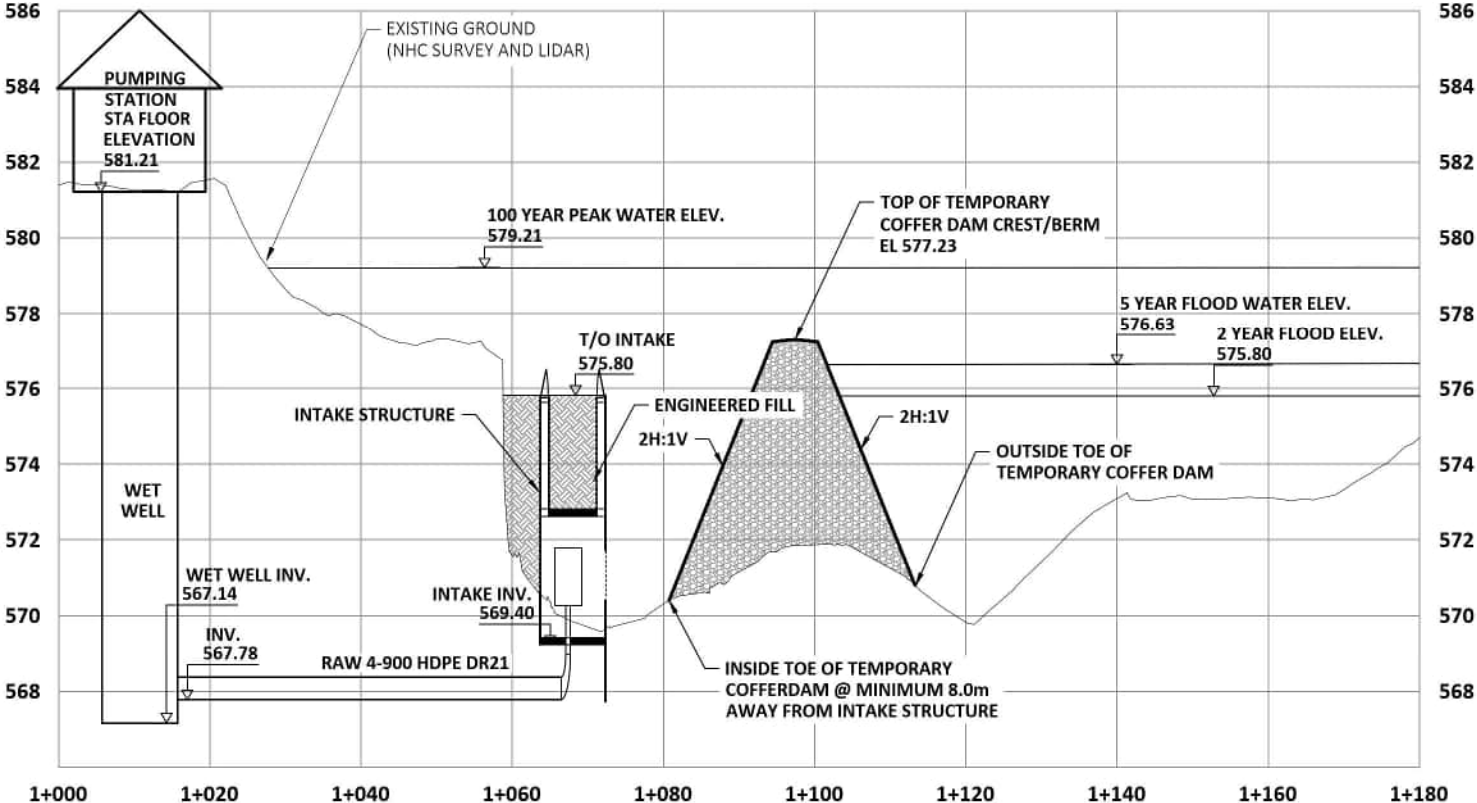
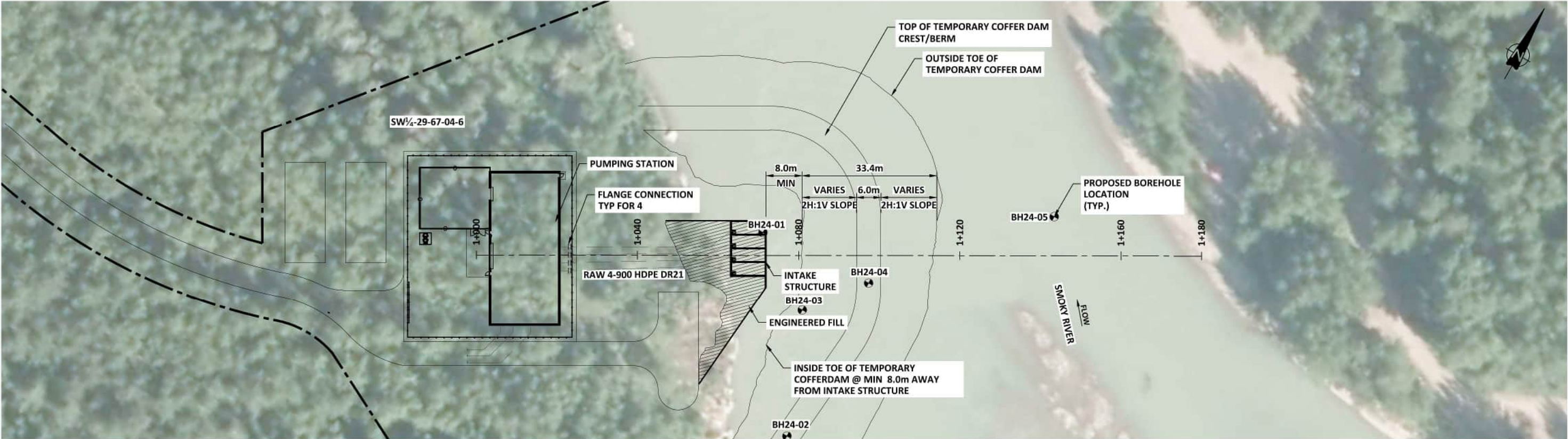
An intermediate platform made of grating would be provided for operations to access the slide gate or to do any other maintenance needed.

3.5 Structural

The proposed intake preliminary structure design will be constructed with cast in place concrete. Shallow foundations overlain by ballast of native material and rip rap are assumed for the structure based on the likelihood of the riverbed being granular soils possibly underlain by bedrock. A geotechnical investigation of the subsurface conditions will be required to determine if shallow foundations will be adequate or if piles will be required to avoid uplift and sliding. The design will assume a wall, and slab thicknesses of approximately 400 mm with a total intake face length of 19.2 m which includes a 3.0 m leading edge. The interior walls between the chambers will allow for isolation of each chamber for maintenance while allowing the other chambers to remain in operation. The exact wall, roof, and slab thicknesses will be determined during detailed design.

The overall footprint of the concrete intake structure is approximately 126 m² which is a length and width of 14.0 m and 9.0 m respectively. Additionally, there will be concrete walls that extend from the south side and north side of the structure towards the bank of the river. **Figure 3.4** illustrates the proposed intake layout along with elevations, dimensions, and the locations of all proposed concrete walls. Fill material will be used to cover the structure up to the access hatch elevation and outward to the concrete walls. This will allow for ease of access to complete maintenance, and it will lessen the steepness of the nearby slope.

The proposed concrete mix design for the Intake structure is to meet the requirement for exposure class of F-1 according to CSA A23.1, Concrete Materials and Methods of Concrete Construction, where concrete is exposed to freezing and thawing in a saturated condition, but not to chlorides. However, if further geotechnical investigation determined that the soil surrounding the intake structure contains excessive sulphate content, concrete mix design to meet the exposure class of S-1 or S-2 will be required. "S" classes are designed to resist sulphate attack of the concrete structure.



PLAN AND PROFILE
1
C1.5 1:1000H
1:200V



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WATERSMART SOLUTIONS LTD.

GREENVIEW INDUSTRIAL GATEWAY - TM02
PLAN AND PROFILE - INTAKE AND PIPELINES
STA 1+000 TO STA 1+180

SCALE: AS SHOWN

DATE: MARCH 2024

JOB: 5946-001-00

FIGURE: 3.4

3.6 Civil Site Works

3.6.1 Preliminary Cofferdam Design

Englobe completed a preliminary analysis of options to implementing an acceptable cofferdam structure for the construction of the intake structure and determined that a stand-alone structural sheet pile cofferdam would mitigate the majority of the risks associated with fish migration, habitat impact, and construction time limitation. However, the feasibility of a structural cofferdam is uncertain without an understanding of the riverbed stratigraphy. A geotechnical investigation of the Smoky River's bed is therefore required to confirm the feasibility of a structural cofferdam as outlined in Section 2.5.2. A copy of the cofferdam option analysis can be found in **Appendix B**.

The other feasible alternative for a cofferdam structure is a rockfill cofferdam comprised of a homogenous zone of rockfill and clay. The rockfill structure is an easier application on the river but would have more negative effects on the environment.

The rockfill cofferdam option will include pushing fill material into the river from the shore to create a berm extending above the water level and extending this along the cofferdam alignment. To limit the loss of soil to erosion, it may be necessary to start with rockfill along the upstream side. For preliminary planning purposes, the cofferdam will have the following characteristics:

- A top of berm elevation of about 577.23 m which is 2 feet above the 5-year flood level, consistent with NHC's recommendations.
- The inside and outside side slopes would be a minimum of 2:1.
- The total width of the cofferdam top of berm would be a minimum of 6.0 m.
- Minimum distance between the inside toe of the cofferdam and the intake structure will be about 8.0 m.
- The wet side of the cofferdam will be armored with riprap that has a median diameter of 0.8 m and a thickness of 1.5 m.
- A 2.0 m thick riprap apron will be placed at the base to the riprap slope extending out 5.6 m to protect against scour.
- A non-woven geotextile will be placed under the rip rap slope.

Figure 3.5 shows the proposed cross section of the rockfill cofferdam prepared by Englobe's geotechnical team.

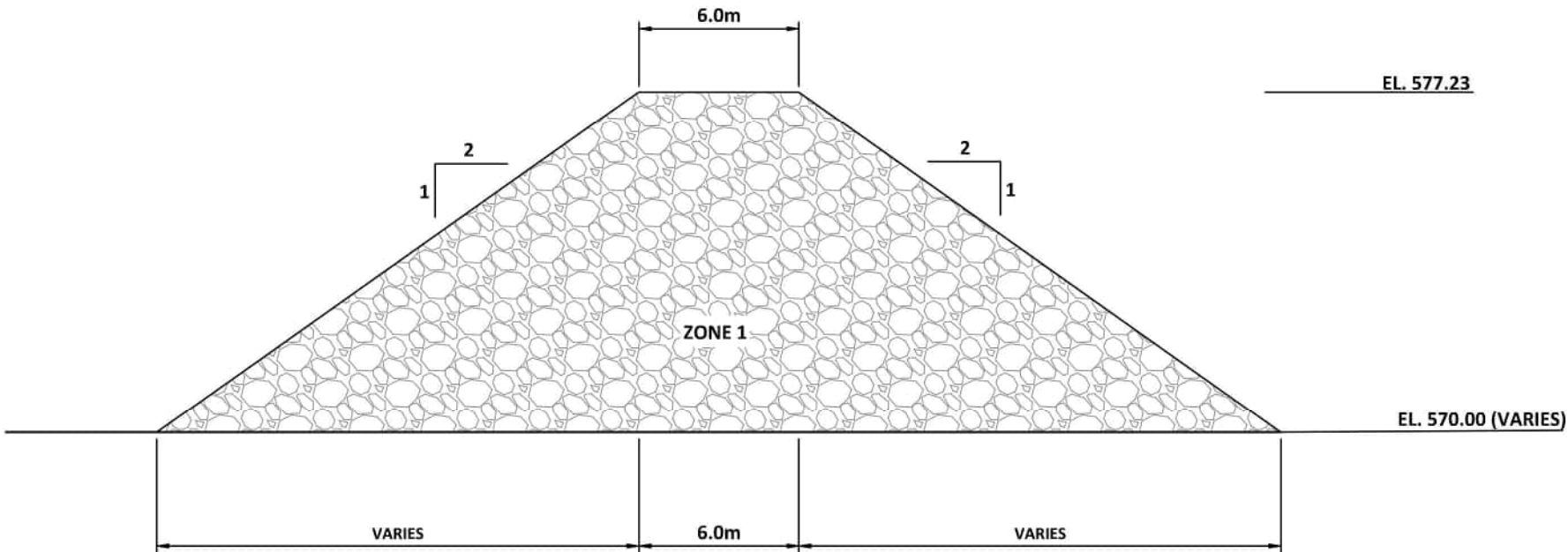
Further review of the preliminary coffer dam details is provided in the Preliminary Cofferdam Cross Section Memorandum included in **Appendix C**. NHC has also completed a preliminary assessment of the earth fill cofferdam hydraulics in a separate report. The findings in this assessment will be discussed further in Section 3.6.4.

3.6.2 Excavation

Once the cofferdam is constructed the water would be pumped out of the inside of the cofferdam to create a dry work area for the construction of the intake. The preliminary geotechnical investigation indicated that the subsurface conditions of the river are likely overlaid with Rock. Therefore, rock excavation would likely be required to embed the intake walls, anchors, and foundation at the appropriate elevations. An excavation plan and safety precautions would be in place prior to rock excavation.

HATCH LEGEND:

 MIXTURE OF RIP RAP &
SMALLER WELL GRADED ROCK



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WATERSMART SOLUTIONS LTD.

GREENVIEW INDUSTRIAL GATEWAY - TM02
TEMPORARY COFFER DAM
CROSS SECTION

SCALE:	NTS	DATE:	OCTOBER 2023	JOB:	5946-001-00	FIGURE:	3.5
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3.6.3 Engineered Fill

Once the construction is complete, imported fill from the raw water pond site would be used to fill the space between the intake and the bank up to the intake access elevations as shown in **Figure 3.4**. The fill will be designed to have a smooth cemented surface sloped towards the river so that debris does not collect during flood periods.

3.6.4 Erosion Protection

During the construction period the cofferdam will restrict flow path and increase the velocity of the flow between the cofferdam and the east bank of the river. NHC has advised that erosion protection will need to be provided for the exterior cofferdam slope and a portion of the east bank across from the cofferdam based on their one-dimensional (1D) numerical hydraulic model. Erosion protection is required on the exposed surface of the cofferdam to ensure high velocities do not cause damage to the berm. It is also recommended for the east bank to ensure that changes in flow patterns due to the presence of the berm do not permanently alter the flow pattern at the intake.

The recommendations for the berm slope protection were discussed in Section 3.6.1 and include armoring the slope with riprap that has a median diameter of 0.8 m. NHC's recommendations for slope protection on the east bank includes a combination of permanent and temporary measures. Figure 4 and 5 in the Preliminary Assessment of Construction Berm Hydraulics completed by NHC shows a plan view and cross sections of the erosion protection extents and recommendations. The permanent erosion protection is located upstream of the cofferdam location and should consist of 0.3 m median diameter riprap with a total thickness of 0.5 m. It should be designed to withstand the 100-year peak flow. The temporary erosion protection is located across from the cofferdam and should consist of 0.5 m diameter riprap. It should be designed to protect against average scour level predicted for a 5-year peak flow for a construction period extending to May 31st.

The completed project would require erosion protection around the intake structure, i.e., scour protection at the base of the structure and bank protection on the banks upstream and downstream of the structure. A preliminary estimate using 100-year peak local velocities at the base of the bank and using a one-dimensional hydraulic model determined that a Class 3 rock riprap with a median diameter about 800 mm would be required. The rock would extend a short distance upstream and downstream of the intake on the banks. Riprap would also be required on the bank above the intake to a buffer height above the 100-year flood level, but smaller rock is likely sufficient at this location.

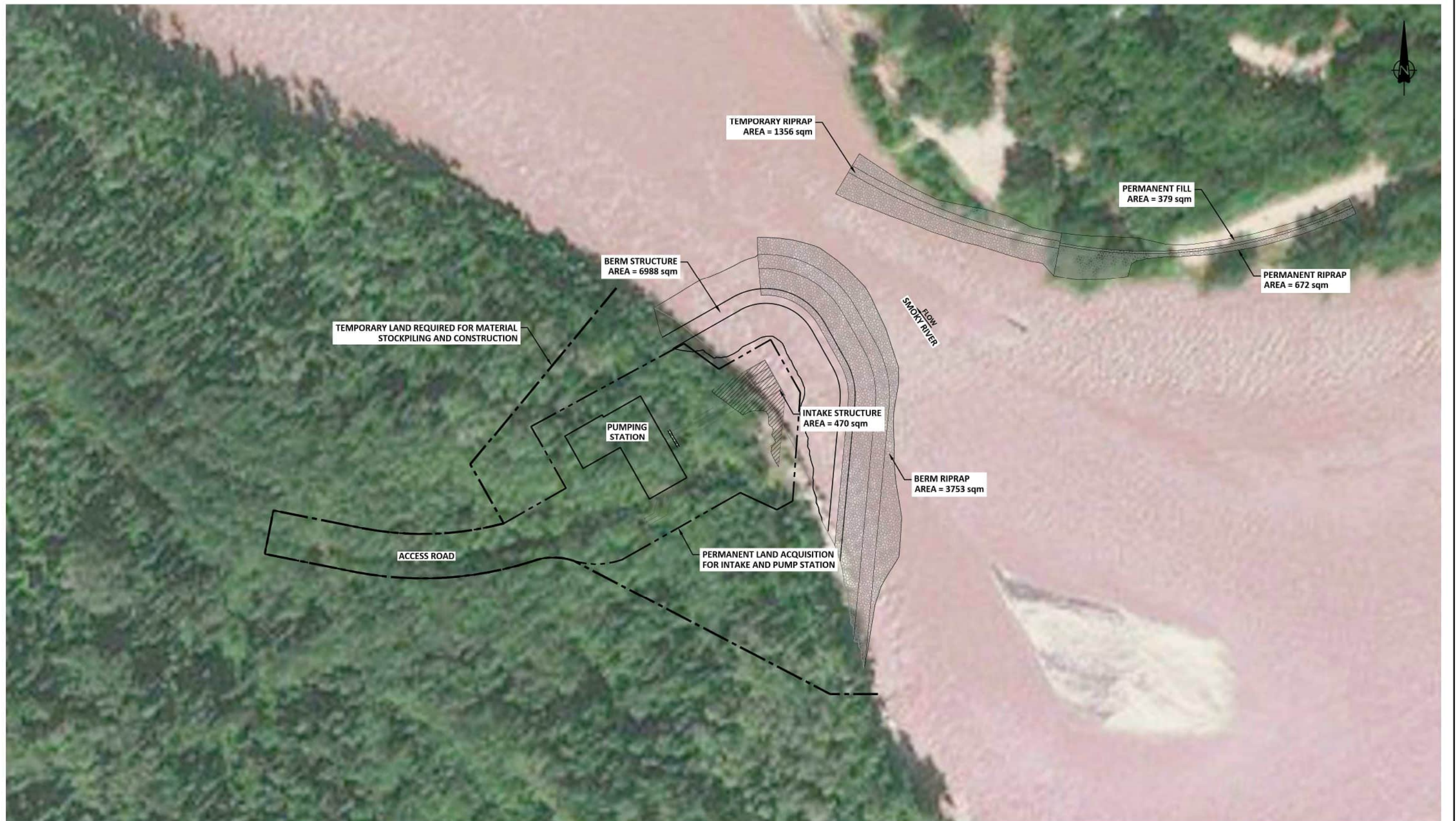
NHC recommends that a two-dimensional hydraulic model be completed during the detailed design phase for this river reach with the intake. This model would confirm the details required for erosion protection around the cofferdam and around the finished intake structure. The 200-year flood elevation should be considered when evaluating the erosion protection on the bank above the intake and before the pump station. This would provide a conservative factor of safety specifically for the pump station.

3.6.5 Site Access

The site access will involve constructing a gravel access road from the pumping station down to the intake structure. This would include some excavation, clearing and grubbing, subgrade prep, and placement of granular material. The road must be constructed with an appropriate grade to allow for safe access to the intake site. The exact locations of the access road will be determined during detailed design.

For the construction and removal of the cofferdam, a large area along the bank will need to be cleared as shown in **Figure 3.6**.

M:\M-DATA\50\46 WATERSMART SOLUTIONS LTD\001 GREENVIEW INDUSTRIAL GATEWAY WATER INTAKE APPROVAL\AUTODESK\CAD\CIVIL 1.1 - RIPRAP.DWG



a division of Englobe

WATERSMART SOLUTIONS LTD.

GREENVIEW INDUSTRIAL GATEWAY
OVERALL SITE PLAN

SCALE: 1:1500

DATE: JULY 2024

JOB: 5946-001-00

FIGURE: 3.6

4.0 INTAKE PIPELINE OVERVIEW

4.1 Pipe Material and Sizing

The intake pipeline will run from the river intake to the pumping station and will be sized at 900 mm nominal diameter. Each chamber will require its own pipeline to transfer the raw water from the river to the pumping station. The current design has four (4) separate chambers which will require 4 separate pipelines. The pipeline will be steel from the intake structure and could transition to HDPE before the connection at the pumping station.

The pipes would be sized to prevent sediment from depositing in pipes by ensuring a minimum velocity of 0.5 m/s through the pipe during minimum flows. Conceptual calculations have determined that a 900 mm HDPE pipe would provide a velocity of 0.67 m/s and headloss of 48 mm. Steel piping will be used immediately outside the structures (i.e., intake chamber and pump station wetwell) for approximately 3.0 m before transitioning to HDPE for the majority of the pipeline length.

4.2 Alignment

The alignment includes running 4 pipelines from the intake to the pumping station. The intake pipes will be located inside each intake chamber and will depart through the bottom of each chamber. A 90-degree bend will be utilized to direct the pipeline towards the pumping station. The intake pipes will then start to converge until minimum spacing requirements between each pipe are met. The pipes will then run parallel at approximately 1.0% slope towards the wet well of the pumping station to allow for drainage and removal of air pockets. Refer to **Figure 3.4** for the preliminary alignment.

4.3 Civil Site Works

The pipelines would be installed by open cut which will involve extensive excavation and backfilling. The installation of the intake pipes would require a sloped excavation to ensure a safe installation. The geotechnical investigation recommends that 2H:1V slopes could be used based on the ground conditions. A steeper slope of 1.5H:1V is also viable if dewatering activities are performed. It is also recommended to apply a thin layer of shotcrete or other sealant to prevent water and air from degrading the exposed soils. Excavation depths are expected to range from 8 m to 14 m in depth which will require a large amount of native material to be excavated and stockpiled away from the edge of the slope. Once the excavating is completed the pipelines can be installed and the native material can be used to backfill the open cut area.

5.0 PRELIMINARY INTAKE HYDRAULICS OVERVIEW

The headloss through each intake from screen to wetwell intakes is based on the following:

- Design flow of 355 L/s
- Headloss through trash bars = 0 mm
- Headloss through screen = 90.8 mm
- Headloss through intake pipeline including minor losses = 48 mm

For the concept evaluation and including a factor of safety, the preliminary headloss from the intake screen to the pump station wetwell shall be 200 mm. Based on the 100-year min water level of 572.52 m the pump station operational level limits should be set such that the wetwell low level and low pump shutoff level are set below 572.32 m. This will ensure the design flow rate is available during extreme low river levels.

To prevent instances of higher than design flow through the screens, the total rate of flow for the pumping system shall be limited to the total capacity of the number of intake screens that are online. If only 2 of the intake screens are online (and the other screens shut off for maintenance), the maximum pump rate of the station should be limited to 710 L/s (2 x 355 L/s).

6.0 CONSTRUCTION LOGISTICS

6.1 General

The current design and location of this project will require some careful consideration when determining the construction logistics and its impacts on schedule. Since the design is still preliminary, the logistics are likely to change during detailed design. It will be important to consider instream restriction periods and how that will affect the overall timeline and feasibility of the project. A high-level construction sequencing of key activities for the intake and intake pipeline construction will be discussed in the following sections.

6.2 Intake

6.2.1 Rockfill Cofferdam

Based on the proposed Rockfill cofferdam design presented earlier, approximately 30,000 m³ of material would be required for the structure, plus an additional amount to replenish washed away material. Grande Cache is anticipated to be the closest source for the temporary rockfill material for the cofferdam and scheduling would bring the material to site ahead of the anticipated start of construction. The Rockfill material from Grande Cache would be the sandstone type which is suitable for the temporary works.

NHC completed one dimensional hydraulic analysis of the river hydraulics for the Rockfill Cofferdam presented in Section 3.6. Results showed that the peak velocities in the river construction from the cofferdam are higher than the natural peak velocities in the river from May to August. Therefore, fish migration enhancement would need to be implemented for this period.

The preferred installation period of the cofferdam would be during the lower flow period that starts during the autumn season, and completion prior to spring thaw.

A biologist would need to be onsite at all times during the cofferdam construction and dewatering of the site to ensure regulatory requirements are met with regards to fish conveyance and protection.

The cofferdam would stay in place until the completion of installation of the intake structure and erosion protection.

6.2.2 Erosion Protection

Temporary erosion protection spurs along the east bank of the river would be needed prior to the rockfill cofferdam construction. The extent of erosion protection of the bank was discussed in Section 3.6.4. Permanent erosion protection measures would be needed around the intake structure and these measures would be installed following completion of the intake structure.

Rock material for erosion protection requires a more durable granite type material which can be sourced from the Town of Rocky Mountain House.

6.2.3 Site Access

The intake will require an access road and laydown area to be constructed prior to any instream activities. Once they are constructed the next activity will be to complete the installation of the rockfill cofferdam. This will involve pushing rockfill into the river from shore to create a berm extending above the water level and extending this rockfill material along the cofferdam alignment. To limit the loss of soil from erosion, rockfill may be necessary to start on the upstream end. After the berm is completed the water inside of the cofferdam will need to be pumped out to create a dry work area. There is only a 2-week period (July 16 to 30) where instream activities are permitted which will not be enough time to install the cofferdam. Therefore, an approval would be required to permit a longer period for in-stream activities to install the cofferdam.

Once the cofferdam is complete and the isolated area is dewatered, construction of the intake can begin. This will include installing structural foundation, excavating, and backfilling for steel intake pipe installation, and constructing the intake concrete structure and walls. It also includes installation of fish screens, fish pumps, slide gates, access hatches, bar screens, and other appurtenances. Once the structure is completed the intake area will be backfilled and the cofferdam will be removed from the river.

6.3 Intake Pipeline

The intake pipeline is assumed to be installed by open cut from the intake to the pumphouse and would likely be constructed at the same time as the water intake. The large excavation depths will require adequate slopes to ensure there are no collapses and large areas for stockpiling of materials. Once excavation is completed the pipes will be installed and the excavation backfilled to grade. Refer to Section 4.3 for additional site works information.

7.0 FINANCIAL ANALYSIS

7.1 Intake and Intake Line Capital Preliminary Cost

A Class D cost estimate was developed to a conceptual level for the construction of the raw water river intake and the intake pipes required to reach the pumphouse from the intake location. Lump sum and unit costs were estimated based on assumed construction methods. The preliminary cost estimate includes contingency and engineering. A detailed cost estimate can be found in **Appendix D**.

The preliminary cost estimate is based on the following assumptions:

- The unit costs are based on quotes and previous projects of similar nature.
- The intake structure will be constructed of cast-in-place concrete and assumed with wall, slab, and roof thicknesses of approximately 400 mm.
- Concrete walls that extend from the intake to the riverbank are assumed to be 400 mm thick.
- A rockfill cofferdam will be used to isolate intake construction area.
- A 40% contingency was included to account for cost uncertainty due to the conceptual nature and the lack of available geotechnical data.
- 900 mm HDPE will be used as intake pipes.
- Intake pipes will likely be installed by open-cut excavation.
- Construction of access road to site was not included in overall cost.
- One submersible fish pump will be included per intake chamber.

The following table, Table 7.1, provides a summary of the Class D cost estimate completed for the construction of the Raw Water Intake Structure and Piping.

Table 7.1: Intake and Pipeline Preliminary Cost Estimate	
Description	Cost
Mobilization/Demobilization	\$4,081,300.00
Cofferdam	\$15,332,000.00
Civil/Site Work	\$4,494,250.00
Structural	\$4,926,000.00
Electrical	\$300,000.00
Equipment	\$2,156,000.00
Total	\$31,289,600.00
Contingency (40%)	\$12,515,900.00
Engineering (15%)	\$6,570,900.00
Grand Total	\$50,376,400.00

7.2 Phasing of Preliminary Capital Costs

The proposed water intake design concept includes four (4) chambers with four (4) intake screens. The GIG project area is scheduled to be constructed in three (3) phases which will require one (1) intake per phase plus redundancy.

Phase I will require the largest capital and include all the underground, civil, and structural components for the full build out. The entire intake structure, the four intake pipelines, the four trash bars and sluices gates would be part of Phase I. Only 2 fish screens and 2 fish friendly pumps would be required for this phase. Phase II will include the installation of the third screen and pump. While the last Phase III would be the final fourth screen and pump.

For costing purposes, it is assumed that Phases II and III will occur in the 10- and 20-year horizon; with a 5% inflation. The following table, Table 7.2, provides the Phase I, II and III breakdowns from the total cost estimate for the construction of the Raw Water Intake and Pipelines.

Table 7.2: Phase I, II, and III Preliminary Cost Breakdown	
Phase	Cost
Phase I	\$49,235,800.00
Phase II	\$929,000.00
Phase III	\$1,513,000.00

8.0 REGULATORY REQUIREMENTS

8.1 General Description

The proposed water supply for the GIG will be regulated by Alberta Environment and Parks (AEP) through the application of the *Water Act*. When applied to municipal waterworks systems, the Water Act is concerned primarily with water quantity allocations. Terms of water allocation are dictated by water diversion licenses that are held by communities and organizations.

8.2 Regulatory Requirements

8.2.1 Design Standards and Guidelines

MPE prepared the assessment in accordance with the following standards and guidelines as a minimum:

- AEP, Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems, 2012
- Freshwater Intake End-of-Pipe Fish Screen Guideline, Department of Fisheries & Oceans, 1995

8.2.2 Water Act Approval

An approval under the province's Water Act must be obtained before taking on any construction activity in a water body. The approval process requires submitting an application with design drawings and reports of the proposed project. The application is reviewed for effects on the aquatic environment, public safety, and nearby approvals. It is also reviewed for hydraulic, hydrological, and hydrogeological effects. The consulting team will compile the required information and provide it during the application phase. Supplemental information will be provided upon request and as required by the regulatory body.

8.2.3 Water Act Diversion License

The Municipal District of Greenview applied for a water diversion license under the Water Act for the GIG project in June of 2021. The source of water is an intake structure located in the Smoky River. The license request is summarized in Table 8.1 below, with a requested term of 20 years.

Table 8.1: Diversion Rate Summary – Water Act License Request

License	Point of Diversion	Min. Diversion Rate	Max. Diversion Rate	License #	License Allocation
Smoky River License for Diversion of Water	NE 30-67-4-W6	680 L/s	1,270 L/s	Pending	24,000,000 m ³

The diversion of water license application is still being processed. Final license details to be determined once application approval is complete.

8.2.4 Fisheries Act

An authorization for construction in the Smoky River would be required for the proposed intake through Fisheries and Oceans Canada (DFO). The Environmental Overview, September 2022 by Associated Engineering indicated that the allowable activity period in the river is 2 weeks (July 16 – July 31). Construction period within the river including installation and removal of the temporary cofferdam would require a DFO approval permitting construction to take place within the restrictive period.

8.2.5 Canadian Navigable Waters Act

The final intake structure top of slab/gravel elevation is conceptually set at 575.80 m which is equivalent to the 2-year flood level return period of the Smoky Lake River. Based on the Environmental Overview the proposed intake does not meet the minor works classification if it does not extend vertically above the bed of the navigable water more than 5% of the water depth.

8.2.6 Public Lands Act

The Crown claims ownership of the bed and shore of all natural water bodies in Canada. And the land on which the pump station and intake piping is proposed lies on Crown land.

A temporary filed authorization will be required for the temporarily occupied areas identified in Figure 3.6.

A disposition is required before performing any construction activities on Crown Lands - activities such as the construction of the intake, intake piping, and pump station. A disposition will be required under the Public Lands Act for the bed and shore of the Smoky River and for the pump station on the Crown owned land. Under the Public Lands Act an adequacy assessment will also be completed to determine the requirement for First Nations consultations. The M.D. has engaged in First Nations consultations with the help of the consulting team.

8.2.7 Historical Resources Act

Water distribution and diversion projects require a formal approval under the Historical Resources Act prior to the onset of activities. Upon completion and review of the Historic Resources Application, project-specific requirements will be issued.

9.0 CONCLUSIONS AND RECOMMENDATIONS

9.1 Conclusions

The following provides the major conclusions of this assessment:

1. Results and recommendations of assessments and investigations completed by the consulting team were summarized and presented in Section 2.0. The key assessments related to the intake include a preliminary hydraulic design, environmental review, and a geotechnical investigation.
2. The preferred preliminary water intake alternative is to be located adjacent to the Macleod Flats Recreation Area on a rock outcropping in a river narrowing at the bend upstream of the bridge. The benefits of this location include adequate water depth to avoid ice buildup during low flows and the riverbank in this area is less susceptible to erosion. Location is identified in **Figure 3.6**.
3. The preferred preliminary intake design includes four (4) pump intakes with four (4) chambers. The total intake face length is 19.2 m which includes a 3.0 m leading edge. Each chamber will include an intake port that is 1.3 m high and 3.1 m wide. The chamber will also include a fish screen, trash bar screen, access hatch, slide gate, and fish pump. The overall footprint of the structure is approximately 126 m². A drawing of the preliminary design can be seen in **Figures 3.1 and 3.2**.
4. The intake pipelines will run from inside each chamber to the pumping station. The pipeline will be 900 mm steel from the intake chamber and transition to 900 mm HDPE before connecting to the pumping station wet well. Installation of the pipes will require deep sloped excavations with a minimum slope of 1.5H:1V.
5. Rockfill is the preliminary cofferdam option that will be required to allow for construction of the water intake. Fill material will need to be pushed out from the shore to create a berm to isolate the construction area. Water from inside the cofferdam will need to be pumped out to create a dry working area. A cross section of the proposed cofferdam can be seen in **Figure 3.4**.
6. Construction logistics for the water intake and intake pipelines are likely to change as the project reaches detailed design. A high-level summary of key activities include:
 - Construct the access road.
 - Construct the cofferdam and dewater.
 - Excavating native material for construction of water intake and installation of intake pipes.
 - Install intake pipes and backfill with native material.
 - Construct intake structure and walls including trash bars and sluice gates.
 - Backfill intake structure area with engineered fill.
7. A Class D cost estimate was completed for the current preliminary design of the water intake structure and conductor piping. A 40% contingency was used due to the conceptual nature of the project. The estimated cost to complete the work is \$50,376,400.00.

9.2 Recommendations

The following recommendations have been made:

- Proceed with a geotechnical investigation in the bed of the river during the next winter season and update the proposed cofferdam accordingly.
- Proceed with the two-dimensional hydraulic modelling of the river and intake location to determine permanent and temporary measures required for the intake and cofferdam construction.
- Proceed with detailed design including design of a Crowder System.

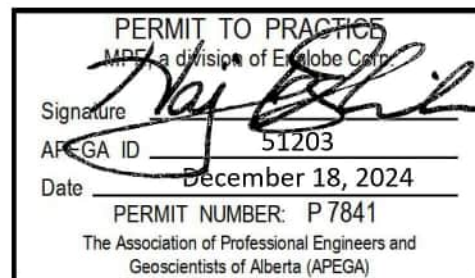
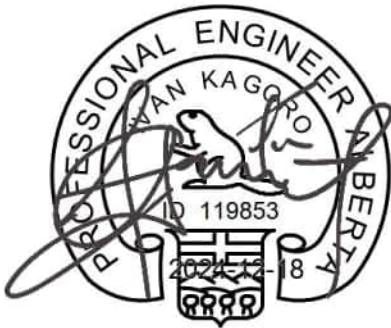
CORPORATE AUTHORIZATION

This report has been prepared by MPE a division of Englobe under authorization of the Municipal District of Greenview (M.D.) on behalf of WaterSMART a Hazen company. The material in this report represents the best judgment of MPE a division of Englobe given the available information. Any use that a third party makes of this report, or reliance on or decisions made based upon it is the responsibility of the third party. MPE a division of Englobe accepts no responsibility for damages, if any, suffered by a third party as a result of decisions made or actions taken based upon this report.

Should any questions arise regarding content of this report, please contact the undersigned.

MPE a division of Englobe

Prepared by:



Ivan Kagoro, P.Eng.

Professional Seal

Corporate Permit

APPENDIX A

Memo - Intake Preliminary Selection Overview

CONSULTANT TEAM REVIEW MEMORANDUM

Intake Preliminary Selection Overview

To: Alison Regan P.Eng., SCMP	From: MPE Engineering Ltd. Team
Director of Projects (WaterSmart Solutions)	cc: Steve Herman
Re: GIG Water Intake Approval	Date: March 1, 2023
File: 5946-001-00 (GIG Water Intake Approval)	Pages: 3
Email: alison.regan@watersmartsolutions.ca	

1.0 INTRODUCTION

Preliminary hydrotechnical design was completed by Northwest Hydraulic Consultants (NHC) for a water intake on the Smokey River and the draft report reviewed by the Owner Group and Project Team. The Option 2 location of the river intake was recommended by NHC and accepted by all. Selection of the river intake structure from the options provided by NHC is the next step in advance of proceeding with further pre-design work.

2.0 PURPOSE

Two intake geometry alternatives to Option 2 (Option 2A and Option 2B) were presented in NHC's report and this memorandum intends to review, summarize discussions, and provide a recommendation of the preferred alternative for Consultant Team consideration. The objective is for the Consultant Team's consensus adoption of the preferred intake structure alternate. The pre-design for the intake and pumping station can then proceed based on the selected alternate.

3.0 REVIEW OF OPTION 2 ALTERNATIVES

The table on the following page briefly compares the two alternatives to Option 2 based on NHC's report.

Option 2 Intake Geometry Comparison		
Geometry Alternative	2A – 13.8m x 8.8m x 3.6m	2B – 29m x 4.7m x 3.4m
Description	Short Stout Configuration i.e., smaller intake with Fish Screens inside each chamber.	Long, Skinny Configuration i.e., larger structure with Fish Screens at the ports. Two screens/ports per chamber.
Screen Type and location	Tee screen installed at the conductor pipe inlet inside the chamber. To be oriented perpendicular to the ports to minimize structure size.	Rectangle screens, installed at the inlet ports behind the bar racks. Intake ports must match the screen size.
Approx. Structure Aquatic Footprint	As shown in Figure 28 of the Prelim Hydrotechnical Report and covers a smaller area relative to Option 2B.	As shown in Figure 30 of the Prelim Hydrotechnical Report. However, estimated to be approximately 1.70 times larger than 2A
Coffer Dam Requirement	Size to suit construction of the structure.	Much larger coffer dam than 2A required.
Fish Protection	Screening with approach velocities < 0.035 m/s for Burbot fish. Fish return system is required.	Screening with approach velocities < 0.035 m/s for Burbot fish. Screen rejects fish at inlet to intake structure. Fish return system not required.
Port size per Chamber	3.98 m ²	10.14 m ²
Screen Security - Frazil Ice	Screens are in the chambers and suspended clear from impact. Potentially more resistant to effects of frazil ice.	Exposed to frazil ice which could block the screens.
River Rock Bank Impact	About 13.8m of the length of rock bank needs to be excavated.	About 29m of the length of rock bank needs to be excavated.
Chamber Maintenance	Chambers will need routine cleaning of sediment.	Chambers will need routine cleaning of sediment.
Screen Maintenance	Backwash and Airburst System (if required).	Backwash and Airburst System (likely required).

Selection of the preferred option for this project will factor heavily on regulatory approval requirements and in particular the option that presents the least impact on the river and the aquatic habitat. Option 2A is estimated to have a footprint or aquatic impact area that is approximately 1.70 times smaller than the aquatic impact area of Option 2B. Option 2B's structure also extends out into the river further than Option 2A does. Based on this, Option 2A would likely be the more preferred option by the regulatory agencies.

The smaller footprint of Option 2A is driven by the screen design being inside the intake chambers instead of at the inlet ports as is the case of Option 2B. The screens being independent of the inlet ports in Option 2A allows for an optimised compact design to reduce the inlet structure faces and overall footprint. For both options, the back faces of the intake structures would be integrated into the river's rock bank, Option 2B has about twice the length of face hence it would require more time and cost to integrate its structure to the riverbank than would Option 2A.

Option 2A requires construction of a smaller coffer dam structure which becomes a significant factor considering the very short non-restrictive activity period (July 16 to July 31) within the river. The smaller footprint of the temporary coffer dam together with the smaller permanent footprint of the final intake structure shall in both cases present less of an obstacle within the river from a navigation standpoint which will be a consideration for Transport Canada (Navigable Waters).

Another advantage of having the screens in the intake chamber for Option 2A is the screens would not be susceptible to debris impact damage and could be less susceptible to frazil ice during river freeze up. Option 2B carries more risk of the exposed screens being damaged by debris impact or blocked by frazil ice.

Option 2A would be a more cost-effective alternative than Option 2B because of the lower footprint. A lower footprint area typically means less construction costs and less offset costs for habitat loss.

One disadvantage of Option 2A is that a fish pumping system would be required to return the fish to the river from the chambers. This is deemed an acceptable method of practise in industry for intakes of this nature. However, this does involve another mechanical process that requires operation and maintenance. Also, debris material that gets through the bar screens at the port may build up within the chamber. However, routine cleaning of the chambers will be required for either intake option.

A significant advantage of Option 2B is that a fish return system is not required due to the screen locations being incorporated at the inlets to the intake structure.

4.0 CLOSING

Of the geometry options presented by NHC for Option 2 River Intake, MPE recommends the Project Team consider Option 2A – the smaller Intake with fish screens on the inside, as the preferred option. We seek consensus from the Consultant Team on the selection.

APPENDIX B

Memo – Cofferdam Option Analysis

CONSULTANT TEAM REVIEW MEMORANDUM

Preliminary Cofferdam Option Risk Analysis

To: Alison Regan P.Eng., SCMP	From: MPE Engineering Ltd. Team
Director of Projects (WaterSmart Solutions)	cc: Steve Herman
Re: GIG Water Intake Approval	Date: December 12, 2023
File: 5946-001-00 (GIG Water Intake Approval)	Pages: 5
Email: alison.regan@watersmartsolutions.ca	

1.0 Background

This memorandum provides a preliminary analysis of options to implementing an acceptable cofferdam structure for the construction of the Greenview Industrial Gateway (GIG) intake structure proposed at Smoky River.

Englobe's geotechnical investigation (June 2023) did not include a physical investigation of the riverbed however provided the following recommendations for a cofferdam based on the overland physical investigations and desktop review.

- The recommended flood return period for a cofferdam is 5 and 10 years for a one- and two-year construction period respectively. The corresponding 5- and 10-year peak river water elevations are 576.63m and 577.18m respectively.
- The smaller footprint option for a cofferdam is rock filled with clay on the upstream faces; this option was preferred by the design team. This cofferdam would have 6 m wide crest and 2:1 slope.
- Installation of a structural cofferdam would be considered a backup in case of excess leakage. However, if the riverbed is composed of boulders or bedrock, installation of driven sheet piles would not be possible. So a more elaborate structural concept would be required.

A conceptual drawing was prepared based on the initial rockfill cofferdam option presented in bullet point 2 above with the crest elevation set at 577.23 Also, it is assumed that the construction period would be in the one-year range so the 5-year return period criteria would be adopted.

Northwest Hydraulic Consultants completed a preliminary assessment of the River hydraulics with the initial rockfill cofferdam; and determined that the constriction created increases the River velocities

above the natural peak velocities typically seen between May to August. So, enhanced fish passage measures would have to be implemented to maintain fish migration during this period.

2.0 Cofferdam Options Overview

In addition to the initial rockfill cofferdam presented in Technical Memorandum 2, four additional options were discussed by the Design Team; and are summarised below.

1. Option 1 – Rockfill Cofferdam. This is the initial option with a 6m crest at 577.23m elevation. Option 1 is not designed for the highest flow period during the month of June so construction would have to be completed before the June high flows (i.e. with a 1 month period).
2. Option 2 –Rockfill Cofferdam c/w Overflow. This option assumes an extended construction period where any flooding into the cofferdam during the peak river flow month of June would be allowed.
3. Option 3 –Rockfill Cofferdam c/w Removal and Replacement. This option is similar to Option 2 however it involves the removal of the Rockfill cofferdam prior to the peak flow month of June and re-installing it after July 15 for a second year of construction.
4. Option 4 –Rockfill & Structural Cofferdam. This option sees a structural cofferdam installed on the downstream face bottom of toe of the rockfill cofferdam - only when it is anticipated that the river water level would exceed the crest elevation in June. The height of the structural cofferdam would be set to allow construction through the high flow month of June.
5. Option 5 – Structural Cofferdam. This option requires the construction of a standalone structural cofferdam that would extend to a height required to allow construction through the high flow month of June.

Table 2.1 provides a more comprehensive review of the cofferdam options.

Table 2.1 - Cofferdam Options Review						
Item	Option	1 - Rockfill Cofferdam, Original	2 - Rockfill Cofferdam + Overflow	3 - Rockfill Cofferdam + Remove and Install	4 - Rockfill Cofferdam + Structural Cofferdam	5 - Structural Cofferdam
1	Description	Rockfill cofferdam as proposed in draft TM#2. It assumes construction would be completed between July 15 and May 31 (11 months).	Rock cofferdam as proposed in draft TM#2. However construction extends into the high flow period of June allowing the berm to overflow. Site would be temporarily shut down during the peak flow periods in June. Dewatering and clean up would be required prior to commencing work in July.	Rock cofferdam as proposed in draft TM#2. However the cofferdam would be removed prior to the month of June and reinstalled in July.	Rock cofferdam as proposed in draft TM#2. Consideration for install of a structural cofferdam on the inside when it is determined that the June river water levels will exceed the berm crest height.	A standalone Structural Cofferdam designed for the conditions and duration needed.
2	Foot Print	Will likely not exceed the 3/4 of river blockage	Will likely not exceed the 3/4 of river blockage	Will likely not exceed the 3/4 of river blockage	With structural cofferdam, an additional 3 - 4 meters would be needed for construction space to allow movement of equipment. e.g. the Counter weight of a backhoe that would normal clear a sloped bank would now need this additional space for a vertical wall. This pushes the rockfill cofferdam further out and potentially occupy more than 3/4 of the river.	The final footprint of the cofferdam would be much smaller than that for the Rockfill options.
3	Fish Migration	High velocities between May and August impede fish migration for majority of species. Migration enhancement would be required.	High velocities between May and August impede fish migration for majority of species. Migration enhancement would be required.	High velocities between May and August impede fish migration for majority of species. Migration enhancement would be required.	River velocities through the constriction would be higher than for Options 1 to 3; making fish migration enhancement more difficult.	The restricted flow area is further increased and velocity further reduced potentially to or below natural river peak levels. Fish passage enhancement features, if needed would be less elaborate than those needed for the Rockfill options
4	Habitat Impact	Due to High Velocities, River bottom is expected to erode between May and August. Erosion protection is proposed for the opposite bank of the river.	Due to High Velocities, River bottom is expected to erode between May and August. Erosion protection is proposed for the opposite bank of the river.	Due to High Velocities, River bottom is expected to erode between May and August. Erosion protection is proposed for the opposite bank of the river.	Because of the higher velocities, negative habitat impact would be worse for this option than Options 1 to 3.	Velocities would be much further reduced so negative impact on the river would be reduced.
5	River Water Quality Impact	Erosion protection would be implemented on the upstream surfaces of the cofferdam.	Overflow into the cofferdam adds risk of carrying debris from the cofferdam interior back into the river.	The removal and reinstall of the cofferdam doubles the chances of river water contamination with sediment and construction material from the unfinished work of the intake.	Additional erosion protection would likely be needed on the upstream surfaces of the coffer dam.	Reduction in river water contamination potential.
6	Availability and Transportation of Construction Material	Approximately 30,000 m3 of rockfill cofferdam and erosion protection material needed. Readily available source of this material is from Grand Cash for the temporary works.	Approximately 30,000 m3 of rockfill cofferdam and erosion protection material needed. Readily available source of this material is from Grand Cash for the temporary works.	Approximately 30,000 m3 of rockfill cofferdam and erosion protection material needed. Readily available source of this material is from Grand Cash for the temporary works..	Approximately 30,000 m3 of rockfill cofferdam and erosion protection material needed. Readily available source of this material is from Grand Cash for the temporary works. Additional structural cofferdam material would be need.	Depends on structural dam concept however materials such as concrete, steel and wood would be readily available in the area.
7	Constructability of Cofferdam	The high velocity river flows make the installation of the Rockfill cofferdam more challenging than your typical normal river flows. There's no way to predict the angle at which fine grained material will settle once placed in flowing water which adds a lot of risk to the footprint and cost.	The high velocity river flows make the installation of the Rockfill cofferdam more challenging than your typical normal river flows. There's no way to predict the angle at which fine grained material will settle once placed in flowing water which adds a lot of risk to the footprint and cost.	The high velocity river flows make the installation of the Rockfill cofferdam more challenging than your typical normal river flows. There's no way to predict the angle at which fine grained material will settle once placed in flowing water which adds a lot of risk to the footprint and cost.	River flow velocities would be higher than those in Options 1-3, so construction will be more difficult. The standby structural cofferdam is expected to be just as challenging.	Installation of structural cofferdams would be challenging however the Rockfill cofferdam in the high velocity water would be just as challenging.
8	Construction duration	11 Months or less however is highly unlikely.	Extends beyond the 11 months and be affected by temporary shutdown and extensive cleanup needed.	Construction period of the cofferdam would likely be doubled to allow for removal and re install.	Extends beyond the 11 months and potentially be affected by structural dam requirement.	Extends beyond the 11 months and be affected by structural dam.
9	Cost Implications	Rockfill cofferdams tend to be cheaper than structural, however due to the high velocity flows in Smoky River, that might not be the case.	Very high cost adder for the dewatering and cleanup work needed. Makes option financially prohibitive.	Very high cost adder for the removal and replacement work needed. Makes option financially prohibitive.	Cost added for the structural cofferdam portion. At a minimum overall Cofferdam costs could double.	Standalone Structural Cofferdam could be just as costly or slightly higher than the Initial Rockfill Option 1
10	Construction Area Maintenance and Safety	Potential for leakage and need for dewatering measures.	Potential for leakage and need for dewatering measures. Overflow could compromise impermeability of structure leading to additional corrective measures needed.	Potential for leakage and need for dewatering measures.	Potential for leakage and need for dewatering measures. Backup structural cofferdams would provide added water tightness.	Potential for leakage and need for dewatering measures but highly dependant on the type of structure and bedding materials.
11	Regulatory	Regulatory concern would be the fish migration enhancement and meeting the construction duration	Regulators will likely not consider this option due to the risks of flooding.	Regulators will likely not consider this option due to the risk to fish habitat from the removal and replacement of the cofferdam.	Regulatory concern would be the footprint and the fish migration enhancement. This option is likely not going to be considered.	Concerns of Fish migration and impact to habitat could be lessened or mitigated.

3.0 Analysis of Options

The cofferdam options will be analysed over a severity score scale from 1 to 5 with 1 being the lowest severity and 5 the highest severity. Below is the criteria used for allocating the severity scores which are based on the parameters in the previous Table 2.1.

1. Exceedance of regulatory requirement for river blockage.
2. Impediment of fish migration requiring passage enhancement measures
3. Negative impact on habitat including riverbed erosion.
4. Negative impact to river water quality including high change of sediment disposition.
5. Availability and transportation of construction material
6. Availability of construction material
7. Ease of construction
8. Negative impact on construction duration
9. Cost Implications
10. Safety and maintenance Concerns
11. Regulatory Acceptance.

Allocation of severity scores is provided in Table 3.1, and is highly subjective at this stage but will allow for preliminary screening of the options for further review.

Item	Description	1 - Rockfill Cofferdam, Original	2 - Rockfill Cofferdam + Overflow	3 - Rockfill Cofferdam + Remove and Install	4 - Rockfill Cofferdam + Structural Cofferdam	5 - Structural Cofferdam
1	Exceedance of regulatory requirement for river blockage.	3	3	3	5	2
2	Impediment of fish migration requiring passage enhancement measures	4	4	4	5	2
3	Negative impact on habitat including riverbed erosion.	4	4	4	5	2
4	Negative impact to river water quality including high chance of sediment disposition.	3	5	5	3	2
5	Availability and transportation of construction material	4	4	4	5	4
6	Ease of construction	3	3	5	5	4
7	Negative impact on construction duration	3	5	5	4	3
8	Cost Implications	3	5	5	4	3
9	Safety and maintenance Concerns	4	5	5	4	3
10	Rejection by Regulatory.	3	5	5	5	2
	TOTAL SCORE	34	43	45	45	27

Options 2- Rockfill Cofferdam + Overflow and Option 3- Rockfill Cofferdam + Remove and Install are both considered to be cost prohibitive due to the added extreme work and are likely to have a higher negative impact on the environment than the other options presented.

Option 4 - Rockfill Cofferdam + Structural Cofferdam, occupies more of the river than any other option so the risks associated with fish migration, habitat destruction, and construction are increased. The increase in river constricted is counter to the intent of reducing the impact on the river habitat so this option will likely be rejected by regulatory.

Option 1 is potentially the most cost-effective option but is contingent on completing all construction work within a 11-month period including installing and removing the cofferdam, installing the conduct pipe to the intake structure, construction of the concrete intake structure and installation of the trash bars and sluice gates. Completion of this work within the timeframe is likely not feasible. Effects of the high velocity river would make construction of the cofferdam challenging – this also applies to Options 2,3 and 4.

A standalone structural cofferdam in Option 5 might be more expensive than the initial Option 1 but would mitigate majority of the risks associated with fish migration, habitat impact, and construction time limitation.

4.0 CONCLUSION

Based on our preliminary review of the Cofferdam Options as presented, Option 5 – Structural Cofferdam seems to be the most acceptable and desirable of the 5 options.

APPENDIX C

Englobe Preliminary Cofferdam Cross Section Memorandum



June 27, 2023

MPE Engineering
10630 172 St NW #101
Edmonton, AB, T5S 1H8
Attention: Mr. Ivan Kagoro, P.Eng.

Subject: **MEMO - Preliminary Cofferdam Cross Section - For Planning Purposes Only Rev.1**
Greenview Industrial Gateway, Municipal District of Greenview, AB
Englobe reference: 5946-001-00

Dear Mr. Kagoro:

Englobe Corp. (Englobe) is pleased to provide MPE Engineering (MPE) with this letter, which includes a discussion of the preliminary cross section proposed for the temporary cofferdam at the water intake along the Smoky River for the Greenview Industrial Gateway (GIG). This memo must be read in conjunction with the report entitled Greenview Industrial Gateway Geotechnical Report (The Report).

Use of this Memo

The cross section discussed below is only to be used for preliminary planning purposes; it is the result of estimations based on previous experience and is not supported by technical analysis or calculations. As such, the proposed section below is subject to change once detailed design has been completed, potentially by a material amount.

Preliminary Cross Section

The proposed cofferdam will consist of a homogenous zone of material, comprised rock fill and clay, with the clay added to the upstream face of the cofferdam after placement of the rock. It is expected that this method of placement will allow 2H:1V slopes to be maintained while also reducing leakage to a minimum. An example of the cross section is shown in Figure 1

Depending on the amount of clay added to the upstream face of the cofferdam the final slope could be flatter than 2H:1V. This may require an adjustment to the slope of the rock fill or other measures to remain within the 2H:1V envelope. The crest width should be a minimum of 6 m to allow adequate working space for equipment.

Depending on the size and type of rock fill used, it may be necessary to provide armoring on the slope to prevent erosion and loss of the clay particles.

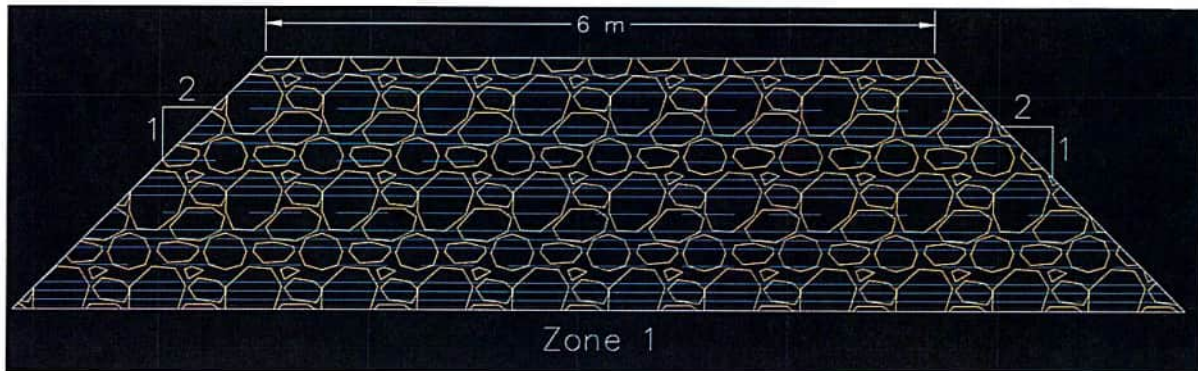


Figure 1 Preliminary Cross Section

Closure

We trust the enclosed is to your satisfaction. If, however, additional information should be required, please communicate with the undersigned.

Yours very truly,

Englobe Corp.



Prepared by:
Caleb Hayduk, M.Eng., P.Eng.
Geotechnical Engineer



Approved by:
Kim Johnston M.Sc., P.Eng.
Senior Geotechnical Engineer

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The findings, recommendations, suggestions, or opinions expressed in this Report reflect the Company's best professional judgment based on observations and/or information reasonably available at the time the work was performed, as appropriate for the scope, work schedule and budgetary constraints established by the Client. No other warranty or representation, expressed or implied, is included in this Report including, but not limited to, that the Report deals with all issues potentially applicable to the site and/or that the Report deals with any and all of the important features of the site, except as expressly provided in the scope of work.

This Report has been prepared for the specific site, development, building, design or building assessment objectives and/or purposes that were described to the Company by the Client. The applicability and reliability of the content of this Report, subject to the limitations provided herein, are only valid to the extent that there has been no material alteration or variation thereto, and the Company expressly disclaims any obligation to update the Report. However, the Company reserves the right to amend or supplement this Report based on additional information, documentation or evidence made available to it.

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Since the passage of time, natural occurrences, and direct or indirect human intervention may affect the views, conclusions and recommendations (if any) provided in this Report, it is intended for immediate use.

The assessment should not be considered a comprehensive audit that covers and eliminates all present, past and future risks. The information presented in this Report is based on data collected during the completion of the monitoring conducted. The overall site conditions were extrapolated based on information collected at specific sampling locations. Professional judgement was exercised in gathering

and analyzing data; however, no monitoring method can completely eliminate the possibility of obtaining partially imprecise or incomplete information; it can only reduce the possibility to an acceptable level. Consequently, the actual site conditions between the sampling points may vary. In addition, analysis has been carried out only for the chemical and physical parameters identified, and it should not be inferred that other chemical species or physical conditions are not present.

It is recommended practice that the Company be retained during subsequent phases of the project, to confirm that the conditions throughout the site do not deviate materially from those encountered throughout the sampling program.

Any description of the site and its physical setting documented in this Report is presented for informational purposes only, to provide the reader a better understanding of the site and scope of work. Any topographic benchmarks and elevations are primarily to establish relative elevation differences between sampling locations and should not be used for other purposes such as grading, excavation, planning, development, or similar purposes.

Any results from laboratory or other subcontractors reported herein have been carried out by others, and the Company cannot warrant their accuracy.

This Report is based on the assumption that the design features relevant to our work will be in accordance with applicable codes, standards and guidelines of practice and constructed substantially in accordance with the Report. If there are any changes to the site development features, or there is any additional information that was not otherwise available at the time the work was performed, the Company should be retained to review the implications thereof to the contents of this Report. The design recommendations expressed in this Report are applicable only to the project described therein.

This Statement of Limitations forms an integral part of the Report.

APPENDIX D

Class D – Cost Estimate



Municipal District of Greenview c/o WaterSMART
Greenview Industrial Park
Raw Water Intake
CLASS D CAPITAL COST ESTIMATE (January 2024)

	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	INSTALL	COST
1	GENERAL ITEMS					
1	Mobilization/Demobilization (15%)	1	L.S.	4,081,300.00		4,081,300.00
	SUBTOTAL					4,081,300.00
2	COFFER DAM					
2.1	Supply and Install Earth / Rock Cofferdam (22,700 m3)	1	L.S.	7,367,000.00		7,367,000.00
2.2	Remove Earth / Rock Cofferdam (22,700 m3)	1	L.S.	3,495,000.00		3,495,000.00
2.3	Supply and Install Erosion Protection (7,310 m3)	1	L.S.	2,597,000.00		2,597,000.00
2.4	Remove Erosion Protection (6,310 m3)	1	L.S.	973,000.00		973,000.00
2.5	Care of Water	1	L.S.	150,000.00		150,000.00
2.6	Erosion and Sedimentation Control	1	L.S.	250,000.00		250,000.00
2.7	Geotechnical Monitoring and Instrumentation of the cofferdam	1	L.S.	500,000.00		500,000.00
	SUBTOTAL					15,332,000.00
3	CIVIL/SITE WORK					
3.1	Gravel Subbase - Access Road & Pad (200mm)	270	m ³	120.00	60.00	48,600.00
3.2	Surface Course - Access Road & Pad (200mm)	270	m ³	120.00	60.00	48,600.00
3.3	Subgrade Prep	1,350	m ²	68.00		91,800.00
3.4	Excavation	12,500	m ³	40.00		500,000.00
3.5	Imported Fill	15,000	m ³	50.00		750,000.00
3.6	Compacted Fill	12,500	m ³	40.00		500,000.00
3.7	Clearing and Grubbing	16,000	m ²	9.00		144,000.00
3.8	Topsoil Stripping & Placement & Seeding	16,000	m ²	15.00		240,000.00
3.9	Engineered Fill Material	1,950	m ³	250.00	125.00	731,250.00
3.10	900 mm Steel Pipe	60	m	5,000.00	2,500.00	450,000.00
3.11	900 mm HDPE DR21	140	m	4,000.00	2,000.00	840,000.00
3.12	100 mm HDPE DR21	200	m	500.00	250.00	150,000.00
	SUBTOTAL					4,494,250.00
4	STRUCTURAL					
4.1	Concrete (Structural, Wetwell, Generator Pad)	540	m ³	2,500.00	2,500.00	2,700,000.00
4.2	Foundation	1	LS	750,000.00	300,000.00	1,050,000.00
4.3	Xypex Coating	1	LS	200,000.00	80,000.00	280,000.00
4.4	Misc Steel	1	LS	40,000.00	16,000.00	56,000.00
4.5	Damp Proofing	1	LS	50,000.00	20,000.00	70,000.00
4.6	Aluminum - Access Hatches - Fish pump	4	ea	15,000.00	6,000.00	84,000.00
4.7	Aluminum - Access Hatches - Screens	4	ea	25,000.00	10,000.00	140,000.00
4.8	Aluminum - Stairs	4	LS	65,000.00	26,000.00	364,000.00
4.9	Aluminum - Grating	1	LS	65,000.00	26,000.00	91,000.00
4.10	Aluminum - Handrails	1	LS	65,000.00	26,000.00	91,000.00
	SUBTOTAL					4,926,000.00
5	ELECTRICAL					
5.10	Miscellaneous	1	L.S.	300,000.00		300,000.00
	SUBTOTAL					300,000.00
6	EQUIPEMENT					
6.1	Trash Bar Screens	4	Ea.	15,000.00	6,000.00	84,000.00
6.2	Fish Screens	4	Ea.	145,000.00	58,000.00	812,000.00
6.3	Fish Pumps	4	Ea.	75,000.00	30,000.00	420,000.00
6.4	Fish Pump Piping	40	m	500.00	200.00	28,000.00
6.5	Sliding Gates	4	Ea.	145,000.00	58,000.00	812,000.00
	SUBTOTAL					2,156,000.00
	TOTAL					\$ 31,289,600.00
	CONTINGENCY (40%)					\$ 12,515,900.00
	ENGINEERING (15%)					\$ 6,570,900.00
	GRAND TOTAL					\$ 50,376,400.00



Municipal District of Greenview c/o WaterSMART
Greenview Industrial Park
Raw Water Intake & Piping
CLASS D PHASING COST ESTIMATE (January 2024)

DESCRIPTION		PHASE 1 (355 L/s)					PHASE 2 (710 L/s)					PHASE 3 (1065 L/s)				
		QUANTITY	UNIT	UNIT PRICE	INSTALL	COST	QUANTITY	UNIT	UNIT PRICE	INSTALL	COST	QUANTITY	UNIT	UNIT PRICE	INSTALL	COST
1	GENERAL ITEMS															
1	Mobilization/Demobilization (15%)	1	L.S.	3,988,900.00		3,988,900.00	1	L.S.	46,200.00		46,200.00	1	L.S.	46,200.00		46,200.00
	SUBTOTAL					3,988,900.00	SUBTOTAL				46,200.00	SUBTOTAL				46,200.00
2	COFFER DAM															
2.1	Supply and Install Earth / Rock Cofferdam (22,700 m3)	1	L.S.	7,367,000.00		7,367,000.00		L.S.	-		-		L.S.	-		-
2.2	Remove Earth / Rock Cofferdam (22,700 m3)	1	L.S.	3,495,000.00		3,495,000.00		L.S.	-		-		L.S.	-		-
2.3	Supply and Install Erosion Protection (7,310 m3)	1	L.S.	2,597,000.00		2,597,000.00		L.S.	-		-		L.S.	-		-
2.4	Remove Erosion Protection (6,310 m3)	1	L.S.	973,000.00		973,000.00		L.S.	-		-		L.S.	-		-
2.5	Care of Water	1	L.S.	150,000.00		150,000.00		L.S.	150,000.00		-		L.S.	150,000.00		-
2.6	Erosion and Sedimentation Control	1	L.S.	250,000.00		250,000.00		L.S.	250,000.00		-		L.S.	250,000.00		-
2.7	Geotechnical Monitoring and Instrumentation of the cofferdam	1	L.S.	500,000.00		500,000.00										
	SUBTOTAL					15,332,000.00	SUBTOTAL				-	SUBTOTAL				-
3	CIVIL/SITE WORK															
3.1	Gravel Subbase - Access Road & Pad (200mm)	270	m ³	120.00	60.00	48,600.00		m ³	120.00	60.00	-		m ³	120.00	60.00	-
3.2	Surface Course - Access Road & Pad (200mm)	270	m ³	120.00	60.00	48,600.00		m ³	120.00	60.00	-		m ³	120.00	60.00	-
3.3	Subgrade Prep	1,350	m2	68.00		91,800.00		m2	68.00		-		m2	68.00		-
3.4	Excavation	12,500	m ³	40.00		500,000.00		m ³	40.00		-		m ³	40.00		-
3.5	Imported Fill	15,000	m ³	50.00		750,000.00		m ³	50.00		-		m ³	50.00		-
3.6	Compacted Fill	12,500	m ³	40.00		500,000.00		m ³	40.00		-		m ³	40.00		-
3.7	Clearing and Grubbing	16,000	m2	9.00		144,000.00		m2	9.00		-		m2	9.00		-
3.8	Topsoil Stripping & Placement & Seeding	16,000	m ²	15.00		240,000.00		m ²	15.00		-		m ²	15.00		-
3.9	Engineered Fill Material	1,950	m3	250.00	125.00	731,250.00		m3	250.00	125.00	-		m3	250.00	125.00	-
3.10	900 mm Steel Pipe	60	m	5,000.00	2,500.00	450,000.00		m	5,000.00	2,500.00	-		m	5,000.00	2,500.00	-
3.11	900 mm HDPE DR21	140	m	4,000.00	2,000.00	840,000.00		m	4,000.00	2,000.00	-		m	4,000.00	2,000.00	-
3.12	100 mm HDPE DR21	200	m	500.00	250.00	150,000.00		m	500.00	250.00	-		m	500.00	250.00	-
	SUBTOTAL					4,494,250.00	SUBTOTAL				-	SUBTOTAL				-
4	STRUCTURAL															
4.1	Concrete (Structural, Wetwell, Generator Pad)	540	m ³	2,500.00	2,500.00	2,700,000.00		m ³	2,500.00	2,500.00	-		m ³	2,500.00	2,500.00	-
4.2	Foundation	1	LS	750,000.00	300,000.00	1,050,000.00		LS	750,000.00	300,000.00	-		LS	750,000.00	300,000.00	-
4.3	Xypex Coating	1	LS	200,000.00	80,000.00	280,000.00		LS	200,000.00	80,000.00	-		LS	200,000.00	80,000.00	-
4.4	Misc Steel	1	LS	40,000.00	16,000.00	56,000.00		LS	40,000.00	16,000.00	-		LS	40,000.00	16,000.00	-
4.5	Damp Proofing	1	LS	50,000.00	20,000.00	70,000.00		LS	50,000.00	20,000.00	-		LS	50,000.00	20,000.00	-
4.6	Aluminum - Access Hatches - Fish pump	4	ea	15,000.00	6,000.00	84,000.00		ea	15,000.00	6,000.00	-		ea	15,000.00	6,000.00	-
4.7	Aluminum - Access Hatches - Screens	4	ea	25,000.00	10,000.00	140,000.00		ea	25,000.00	10,000.00	-		ea	25,000.00	10,000.00	-
4.8	Aluminum - Stairs	4	LS	65,000.00	26,000.00	364,000.00		LS	65,000.00	26,000.00	-		LS	65,000.00	26,000.00	-
4.9	Aluminum - Grating	1	LS	65,000.00	26,000.00	91,000.00		LS	65,000.00	26,000.00	-		LS	65,000.00	26,000.00	-
4.10	Aluminum - Handrails	1	LS	65,000.00	26,000.00	91,000.00		LS	65,000.00	26,000.00	-		LS	65,000.00	26,000.00	-
	SUBTOTAL					4,926,000.00	SUBTOTAL				-	SUBTOTAL				-
5	ELECTRICAL															
5.10	Miscellaneous	1	L.S.	300,000.00		300,000.00		L.S.	300,000.00		-		L.S.	300,000.00		-
	SUBTOTAL					300,000.00	SUBTOTAL				-	SUBTOTAL				-
6	EQUIPEMENT															
6.1	Trash Bar Screens	4	Ea	15,000.00	6,000.00	84,000.00		Ea	15,000.00	6,000.00	-		Ea	15,000.00	6,000.00	-
6.2	Fish Screens	2	Ea	145,000.00	58,000.00	406,000.00	1	Ea	145,000.00	58,000.00	203,000.00	1	Ea	145,000.00	58,000.00	203,000.00
6.3	Fish Pumps	2	Ea	75,000.00	30,000.00	210,000.00	1	Ea	75,000.00	30,000.00	105,000.00	1	Ea	75,000.00	30,000.00	105,000.00
6.4	Fish Pump Piping	40	m	500.00	200.00	28,000.00		m	500.00	200.00	-		m	500.00	200.00	-
6.5	Sliding Gates	4	Ea	145,000.00	58,000.00	812,000.00		Ea	145,000.00	58,000.00	-		Ea	145,000.00	58,000.00	-
	SUBTOTAL					1,540,000.00	SUBTOTAL				308,000.00	SUBTOTAL				308,000.00
	TOTAL					\$ 30,581,200.00	TOTAL				\$ 354,200.00	TOTAL				\$ 354,200.00
	CONTINGENCY (40%)					\$ 12,232,500.00					\$ 141,700.00					\$ 141,700.00
	ENGINEERING (15%)					\$ 6,422,100.00					\$ 74,400.00					\$ 74,400.00
	GRAND TOTAL					\$ 49,235,800.00	GRAND TOTAL				\$ 570,300.00	GRAND TOTAL				\$ 570,300.00

Assume:
Inflation
Period

5%
10

Assume:
Inflation
Period

5%
20

TOTAL	\$ 577,000.00	TOTAL	\$ 939,800.00
	\$ 230,800.00		\$ 376,000.00
	\$ 121,200.00		\$ 197,400.00
GRAND TOTAL	\$ 929,000.00	GRAND TOTAL	\$ 1,513,200.00

Appendix C

Technical Memorandum #3 Pump Station Transfer Concept



a division of Englobe

Technical Memorandum No. 3

MUNICIPAL DISTRICT OF GREENVIEW NO. 16

Greenview Industrial Gateway

Raw Water Transfer Station Concept Design Phase

Date: December 18, 2024

Project No. 5946-001-00

Proud of Our Past... Building the Future

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1.0 INTRODUCTION

1.1 General

This Raw Water Transfer Station Concept Design Phase Technical Memorandum has been prepared for the Municipal District of Greenview No. 16 (M.D.) on behalf of WaterSMART a Hazen company (WaterSMART), as part of the Greenview Industrial Gateway (GIG) water intake and eco-industrial zone development. This technical memorandum will discuss the conceptual Raw Water Transfer Station design for the GIG project.

1.2 Purpose

The purpose of this memo is to provide a conceptual Raw Water Transfer Station design using previously determined design criteria. This memo will also be used to discuss required regulatory approvals and permits, review cursory hydraulics, and provide a Class D cost estimate.

1.3 Scope of Assessment

The major objectives and goals of the technical memorandum are as follows:

- Review of design criteria.
- Review of cursory hydraulics.
- Present conceptual Raw Water Transfer Station design.
- Present conceptual civil design.
- Discuss required regulatory approvals & permits.
- Provide a Class D cost estimate.

1.4 Previous Reports

The following data, plans, reports, and manuals were compiled and reviewed to complete this report:

- Associated Engineering, Greenview Industrial Gateway Intake Environmental Overview, September 2022.
- Associated Engineering, Greenview Industrial Gateway Raw Water Pipeline Alignment Options, January 2023.
- Associated Engineering, Greenview Industrial Gateway Raw Water Storage Ponds Concept, March 2023.
- Englobe, Greenview Industrial Gateway Geotechnical Investigation, April 2023.
- Northwest Hydraulic Consultants, Greenview Industrial Gateway Raw Water Intake Preliminary Hydrotechnical Design, February 2023.

2.0 DESIGN CRITERIA

2.1 General

The proposed Raw Water Transfer Station design concept is based on design criteria established in previous reports and technical memorandums. The projected maximum annual withdrawal from the Smoky River to service all phases and future lots is estimated at 24,000,000 m³. The general design concept is for three (3) phases of raw water pumping and raw water transmission pipeline development to transfer water from the proposed intake structure described in Technical Memorandum No. 2 (TM#2) on the Smoky River to the Raw Water Storage ponds that will service the M.D. of Greenview. Pumping has been sized to satisfy the necessary volume of raw water to be stored outside of a period of low river flows for 18 weeks annually. Design criteria is consistent with previous reports and technical memos and is summarized below:

- Maximum Annual Withdrawal 21,840,000 m³
- Design Flow Rate (individual phase/pipeline) 0.355 m³/s per phase
- Design Flow Rate (Ultimate three phases) 1.065 m³/s

The proposed pumping system design is based on hydraulic requirements established by modelling completed as part of previous reports. The following design criteria were provided by Associated Engineering as detailed for the preliminary raw water transmission pipeline design.

- Flow Rate (Per pipeline) 0.355 m³/s per pump
- Design Head 178 m (253 psi)
- Potential Surge Pressure 352 m (500 psi)

3.0 PROPOSED RAW WATER TRANSFER STATION DESIGN OVERVIEW

3.1 Civil Design

The conceptual Raw Water Transfer Station design will involve various civil site works including tree clearing, access road construction, deep excavations, stockpiling of materials, backfilling, and construction of a gravel parking pad. Once completed the excavation can begin for the wet well portion of the Raw Water Transfer Station. The estimated maximum depth for the excavation is 14 m which will require a soil support option to ensure safety during construction.

Upon completion of the excavation, the wet well building can be constructed. Once constructed the area can be backfilled and the gravel parking pad and fence can be installed.

3.2 Geotechnical Investigation

Englobe completed a geotechnical investigation to determine the subsurface conditions at the anticipated wet well location. One borehole (BH23-02) was drilled near the proposed Raw Water Transfer Station location to a depth of 15 m. Samples from the borehole were taken at regular intervals and transported to the lab for further examination and testing. The tests completed include moisture content, hydrometer, atterberg limits, and unconfined compressive strength. The Englobe findings were summarized in a final report along with conclusions and recommendations regarding soil support options that will allow for a safe excavation and construction of the Raw Water Transfer Station.

When analyzing BH23-02, it consisted primarily of sandstone. The sandstone was medium to coarse grained, extremely weak to weak, grey, or orange-brown when oxidized. The borehole also contained a thin layer of surficial soils with layers of mudstone, bentonite, coal, and siltstone 7 meters below ground surface (mbgs).

Groundwater conditions were monitored for BH23-02. The groundwater depth 15 days after piezometer installation gave a reading of 5.97 mbgs. Variation in the order of 1 m or more is possible, depending on the season, therefore it will be important to monitor prior to and during construction to ensure adequate support options are used during excavations.

A list of suitable soil support options is below:

- Sloped excavation (minimum 2H:1V slope)
- Full-depth secant pile wall
- Anchors with shotcrete

The feasibility of each option will require more in-depth analysis. For this memorandum, it will be assumed that a sloped excavation will be utilized.

Based on the geotechnical investigation completed by Englobe, the wet well building may either be founded on top of the wet well walls, or on an isolated foundation system constructed outside the wet well. An independent foundation would serve to isolate the building from wet well movements and is recommended. If constructed on an isolated deep pile foundation, building settlements would be minimized.

3.3 Building Layout

The Raw Water Transfer Station layout will consist of two levels. The upper level will house all the processing, mechanical, and electrical equipment; the lower level will house the wet well. Concept plan view drawings of the proposed Raw Water Transfer Station building can be found in **Figure 3.1**.

The upper level will sit above the maximum flood elevation and will consist of a main pump and process room, electrical room(s), and a control room. The main pump and process room will require a minimum 20 ft high ceiling to facilitate movement for install, servicing and removal of process equipment including the motors required for the vertical turbine pumps, flanged pump column sections, valves, and piping sections. Movement of this equipment is best achieved through use of a bridge crane with capability for overhead access in all areas of the main process room. The main pump plan includes a large laydown or maintenance bay area in the central part of the room to provide room for on-site servicing of equipment, or flatbed truck entry for loading and unloading of equipment through a roll-up style service door. Hatch access to the lower level will also be located within the main pump room for cleaning and inspection.

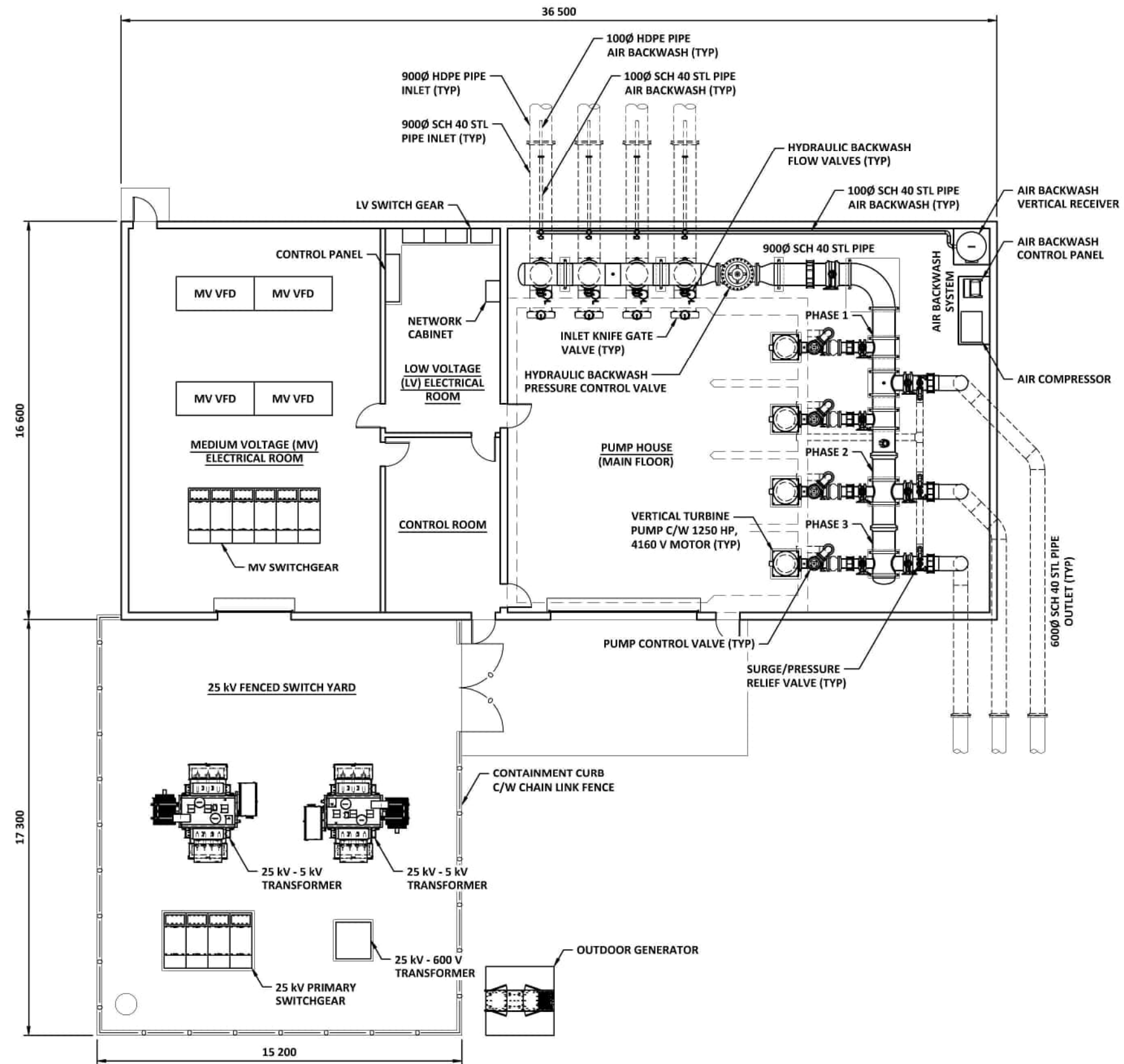
There will be two separate electrical rooms required on the main level to house the higher voltage pump electrical service equipment and the low voltage electrical equipment for smaller process equipment including valves, compressors, and the fish pumps housed in the nearby intake structure as well as building electrical requirements for lighting, heating, and cooling. A separate area has been planned for a control and operations room that will house controls including control panels, SCADA and communications equipment, and a comfort area for operations staff as required. If necessary, the building layout can accommodate the addition of a washroom adjacent to the control room area.

The lower level should be configured with four pump bays each to contain a single pump bowl assembly. A common inlet chamber will allow inflow from each of four bays of the intake structure. The lower level should also include sloped floors to ensure minimal accumulation of solids.

3.4 Site Layout

The Raw Water Transfer Station building will be located approximately 50 m due west of the existing west bank of the Smoky River. The building will require a chain link security fence with an electric gate to facilitate entry from the access road. Within the fence there will be a gravel pad located on the west side of the building with roll-up door maintenance access into the main process area and medium voltage room and man door access into the main process room and control room.

The fenced site area also includes accommodation for a secure high-voltage switch yard that will be required to bring in utility power and house separate transformers for the medium-voltage and low voltage electrical services. The switch yard will be secured separately within the main compound and will include a secondary containment curb and liner to capture runoff and divert through an oil/water separator as required where surface runoff may reach the river. The concept site plan of the proposed Raw Water Transfer Station site can be found in **Figure 3.2**



an Englobe company

WATERSMART SOLUTIONS LTD.

GREENVIEW INDUSTRIAL GATEWAY
PUMPHOUSE
MAIN FLOOR PLAN

SCALE: 1:200

DATE: JUNE 2023

JOB: 5946-001-00

FIGURE: 3.1

4.0 CONCEPTUAL RAW WATER TRANSFER STATION DESIGN

4.1 General

The current Raw Water Transfer Station design includes a four (4) pump configuration that is planned to be completed in three (3) phases. The first phase will include the installation of two (2) pumps to provide the initial capacity with 100% redundancy. Each pump will be sized to provide the required initial flow rate (355 l/s) and storage for the first phase raw water transmission pipeline. The redundant pump can also be used to provide additional capacity for hydraulic backflush of the intake.

4.2 Pump Header

4.2.1 Pump Selection

Preliminary pump selection has been completed based on the design flow rate of 355 L/s at a maximum design head of 194 m. The ultimate pump configuration includes four (4) vertical turbine pumps each with 1250 HP. At least one variable-frequency drive (VFD) is recommended to accommodate capability for control of pump startup surge control, hydraulic backflush of the intake screens, and occasional reduced flow line filling/refilling. To ensure redundancy and for maximum ease of operation all pumps will be fitted with a VFD.

Vertical turbine pumps were selected based on ease of availability, installation, maintenance, and experience in similar applications. Because of the raw water characteristics of the Smoky River water source, the pumps must be suitable for high turbidity in general and specifically for continuous use for water containing silts and abrasives. Both open line shaft (OLS) and enclosed line shaft (ELS) options were reviewed. Consideration was made of the operational inputs, likely requirements for maintenance, expected service life and reliability for each option and ultimately open line shaft pumps are recommended with selection of abrasive resistant materials in contact with water including line shafts, impellers, bearings, and wear-rings.

4.2.2 Piping Selection

Preliminary process piping selection has targeted a flow velocity of less than 3 m/s with pressure rating meeting or exceeding the maximum duty and surge pressures as provided by the hydraulic modelling completed by Associated Engineering for the downstream raw water transmission pipeline. Preliminary pipe selection for all sections of piping in all sizes that may be exposed to maximum surge pressure is Schedule 40 A106B Seamless Carbon Steel. Surges may exceed the pressure rating of standard weight and even extra weight piping in some sizes and therefore it is important to verify surge pressures upon finalization of transmission main models. For all connections to the raw water transmission pipelines a 600 mm pipe diameter will be used while individual pump outlets have been sized to match available pump outlets (400 mm) and all headers that may carry the ultimate maximum Raw Water Transfer Station design flow (1065 l/s) have been sized at 900 mm. ASME B16.5 Class 300 flanges will also be required for all flanges and fittings with potential to be exposed to surges.

4.2.3 Piping Configuration

The raw water pump header conceptual piping configuration includes three separate outlets for ease of phasing as additional pumps and raw water transmission lines are completed as required. Each outlet would include flow metering and an isolation valve. Piping will require engineered thrust blocks at each pump inlet, cross and pipe outlets. The pump discharge header will be connected to the hydraulic backflush header by a pressure reducing valve. It is recommended that computational fluid dynamics (CFD) modelling be completed for the pump header as part of the detailed design.

4.2.4 Valves and Controls

Each of the raw water pump outlets should be fitted with an anti-cavitation pump control valve, silent check, and isolation valve for mitigation of surge generation in the downstream raw water transmission pipeline. Each raw water transmission pipeline connection will be fitted with a separate pressure and surge relief valve as well as isolation butterfly valves. As with the piping, all valves in the raw water pump header will require Class 300 flanges. Additionally, all valves over 400 mm diameter will have a dismantling joint for ease of installation and removal.

4.3 Hydraulic Backflush Header

Based on the recommendations of the preliminary hydrotechnical design (NHC, 2023) the Raw Water Transfer Station will incorporate equipment to facilitate both a hydraulic backflush and air backflush of the raw water intake. The combination of the two backflush methods is intended to reduce the incidence of solids settling within the raw water conveyance and to facilitate the removal of solids from the intake screen itself. It should also provide a benefit in facilitating fish removal from the intake bays. Specific hydraulic performance within the intake bay should be verified by modelling during the detailed design phase.

The preliminary Raw Water Transfer Station design includes accommodation for hydraulic backflush of the raw water intake structure using the proposed raw water pump header. The intent would be to remove sediments by flushing backwards through the raw water conveyance between the inlet structure and Raw Water Transfer Station, out of the fish intake screen and ultimately out of the intake bay. The header will be configured so that all pumps could theoretically be used together to backflush through each individual raw water inlet pipe. However, to avoid exceeding the design maximum inlet flow through the individual raw water intake bays the backflush should be operationally limited to match the intake flow capacity of the remaining three bays.

4.3.1 Piping Configuration

The hydraulic backflush header will allow flow from the main pump header to be redirected to any of the four individual raw water inlet pipes from the raw water intake. Because of the high pressure required for the main pump system the pressure will be reduced prior to connection to hydraulic backflush header, therefore the header can be completed with standard weight steel piping and 150# flanges and fittings.

4.3.2 Valves and Controls

A pressure reducing valve (PRV) with anti-cavitation trim will be installed at the interface between the pump discharge header and the hydraulic backflush interface. The valve will have Class 300 flanges but will reduce pressure downstream to eliminate the risk of damaging downstream fittings and structures. As a result, all downstream flanges and fittings can utilize standard Class 150 flanges. To allow for automation of the hydraulic backflush, flow to each of the individual raw water conveyances will be regulated by actuated butterfly valves located at the outlet of the header and above the Raw Water Transfer Station floor. The inlet conveyance pipes will be fitted with automated actuated knife gate valves that will isolate each pipe from the wetwell during backflush. These knife gates will also allow for isolation of the wetwell from the intake structure as required for inspection and cleaning of the wetwell.

4.4 Air Backflush

The raw water intake screens located within the intake structure have also been chosen with accommodation for air backflush. The equipment required for the air backflush will be housed in the Raw Water Transfer Station. The air backflush system will consist of a compressor, controller, and air receiver to be located within the process area of the building. The proposed 15 HP screw compressor with sound enclosure will charge a large (4000L) air receiver. From the air receiver there will be a 100 mm outlet with a separate automated butterfly valve feeding each of four 100 mm HDPE pipes that extend to the individual raw water intake screens. On a timed interval the controller opens a butterfly valve, and the entire volume of the receiver rapidly backflushes the screen surface. Each screen would be backflushed separately.

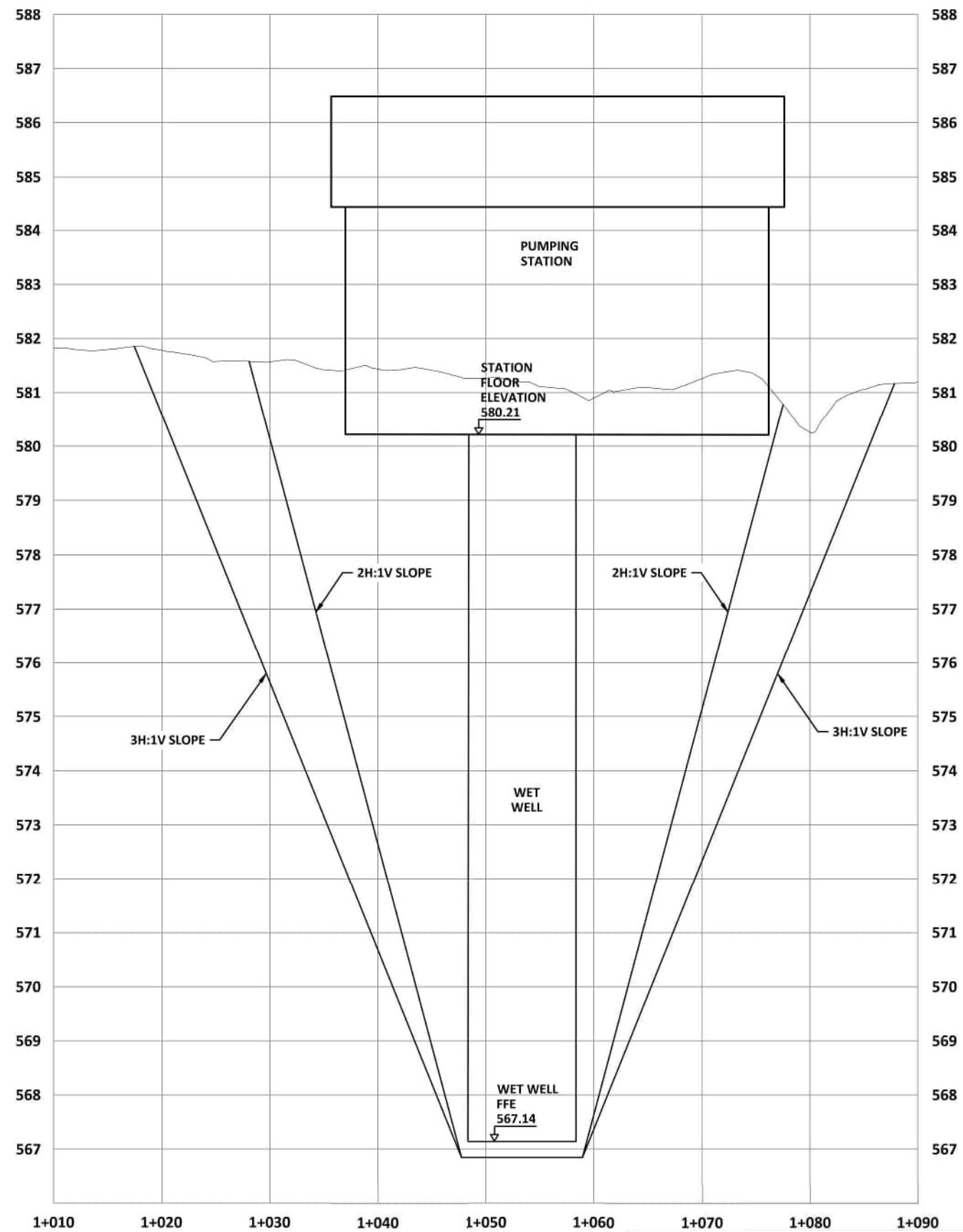
4.5 Wetwell

4.5.1 General

The intent for the wetwell is to have minimum retention time of raw water and a reduced footprint to minimize sedimentation and therefore maintenance requirements within the wet well. The concept section of the wetwell and inlet conveyance/piping can be found in **Figure 4.1** and **Figure 4.2**.

4.5.2 Inlet Piping

The inlet pipes entering the wet well consist of four separate 900 mm raw water inlet pipes each connected to an individual intake bay. Each inlet pipe will have an actuated knife gate valve at the wetwell end for isolation of individual intake bays. The knife gate will allow pressure to be generated in the inlet piping during hydraulic backflush. The inlet is sized to minimize headloss from the intake to the wetwell while still maintaining the minimum recommended velocity of 0.5 m/s in all intake pipes at design inlet flow rates. DR 11 HDPE piping is chosen for the length of the inlet piping with transition to steel for increased strength near the point of connection to the wet well and intake screen.



SECTION
1
C1.3
1:500H
1:100V



SCALE: AS SHOWN

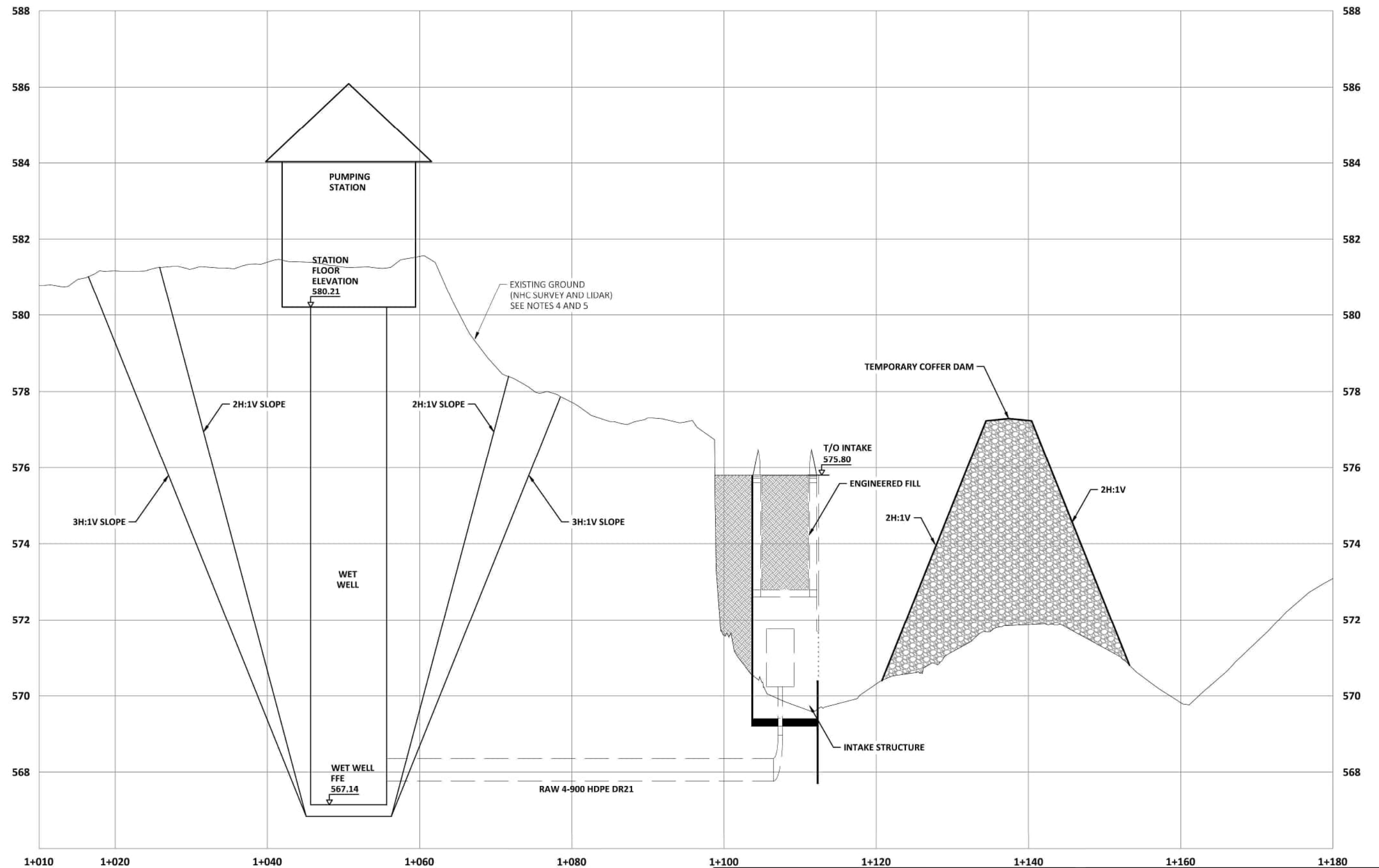
DATE: JUNE 2023

WATERSMART SOLUTIONS LTD.

GREENVIEW INDUSTRIAL GATEWAY - TM03
PUMPING STATION SECTION 1

JOB: 5946-001-00

FIGURE: 4.1



SECTION
2
C1.4 1:500H
1:100V



an Englobe company

WATERSMART SOLUTIONS LTD.

GREENVIEW INDUSTRIAL GATEWAY
PUMPING STATION SECTION

SCALE: AS SHOWN

DATE: AUGUST 2023

JOB: 5946-001-00

FIGURE: 4.2

4.5.3 Depth

The proposed wetwell will have an approximate bottom slab elevation and top slab elevation of 567.14 m and 581.50 m geodetic, respectively. This will give a total approximate wet well depth of slightly more than 14.4 m. The top slab elevation was set above the 100-year peak ice jam water level of 579.2 m with an additional freeboard of 2.3 m. There is a natural plateau on the site above 580 m which aligns well with the top slab of the wetwell. The bottom slab was determined based on the proposed geometry of the intake structure with maintenance of a minimum 2% slope from the intake outlet to the wetwell inlet. The minimum 100-year water level in the Smoky River has an elevation of 572.64 m. When considering the predicted head loss from conveyance and the bottom slab elevation, the water depth will exceed 5 m even when the river is at the 100-year minimum water level elevation. This depth provides ample submergence for the range of vertical turbine pumps reviewed for the preliminary design.

4.5.4 Configuration

The proposed concept and configuration for the wetwell was designed with consideration of the American National Standard Institute (ANSI) for pump intakes and based on the proposed selection of four vertical turbine pumps within a rectangular wetwell. Within the wetwell there are four (4) proposed pumping bays that will be separated by partitions. The bottom slab of the wetwell will also be sloped towards the pumping bays to aid in passing sediment rather than collecting it. Accommodation would also be made for a shallow wash out sump and solids handling pump to accommodate cleanout and maintenance.

The proposed configuration includes a wetwell floorplan area of approximately 130 m², which will provide retention of approximately 650 m³ to 1500 m³ at minimum and maximum water levels respectively. To reduce the sedimentation and subsequent cleaning requirement during Phase 1 of operation, a midline partition wall will halve the wetwell area to approximately 65 m². This will provide water retention volumes of approximately 325 m³ to 750 m³. Based on the proposed Phase 1 pumping rate of 355 L/s and ultimate (Phase 3) flow rate of 1065 L/s the expected retention time range can be estimated at 15-35 min initially and 10-23 min at full buildout.

It is strongly recommended that CFD modelling be completed as part of detailed design of the pump bays and wetwell. The CFD modelling should include phasing scenarios to determine the fluid dynamics as additional pumps are added.

4.6 Hydraulic Overview

4.6.1 Raw Water Transmission Pipeline

Preliminary design of the raw water transmission pipeline from the proposed Raw Water Transfer Station to the proposed raw water reservoirs was completed by Associated Engineering and is understood to be summarized in more detail within Technical Memorandum No. 4 (TM#4). Four options for raw water transmission pipeline alignment with varying quantities of steel and HDPE piping were reviewed in the TM#4. In general, the hydraulic design parameters are similar for all reviewed options and Pipeline Alignment Concept #2 that has a short distance and lowest capital cost was recommended for further analysis. The design head requirement and corresponding potential surge pressure for Pipeline Alignment Concept #2 have been adopted as preliminary design criteria for the Raw Water Transfer Station.

4.6.2 Raw Water Transfer Station Outlet and Service Piping

Piping within the Raw Water Transfer Station should be sized in general for a maximum in-pipe velocity of 3 m/s to reduce in-pipe headloss and local thrust at joints. Raw Water Transfer Station outlet and service piping must be sized to accommodate the maximum design head and potential surge pressure as dictated by the raw water transmission pipeline preliminary design.

4.6.3 Raw Water Transfer Station Inlet Piping

Piping into the Raw Water Transfer Station should be sized to maintain a minimum velocity of 0.5 m/s to prevent sediment deposition per the preliminary hydrotechnical design report completed by Northwest Hydraulic Consultants (NHC). Raw Water Transfer Station inlet piping and fittings should also be sized large enough to minimize head loss between the proposed intake structure and Raw Water Transfer Station wet well. A 900 mm diameter HDPE raw conveyance pipe from each intake bay to the Raw Water Transfer Station wet well will be sized to maintain a minimum velocity of greater than 0.5 m/s under forward flow to prevent sediment deposition, and large enough to ensure an in-pipe velocity of less than 3 m/s during hydraulic backflush using maximum raw water system flow. Estimated head loss in the 900 mm conveyance is less than 50 mm.

4.7 Electrical

The proposed Raw Water Transfer Station will be supplied by a single 25 kV High Voltage Utility feed into a customer owned outdoor switch yard. The switch yard will contain the 25 kV service entrance switchgear, two (2) pad mounted 25 kV:5 kV power distribution transformers (one (1) duty, one (1) redundant) and a 25 kV: 600V step down transformer. An exterior 300 kW (approximate) outdoor standby generator will be located within the fenced compound but outside of the switch yard. The standby generator will be sized to satisfy essential building loads only and will not be sufficient to power the main process equipment. It is assumed that a temporary operational shutdown will be acceptable during utility power loss.

The Medium Voltage (MV) electrical room located inside the Raw Water Transfer Station will contain the 5 kV Main-Tie-Main switchgear and VFDs for powering the distribution pumps. The 5 kV switchgear will be designed to accommodate full future buildout of the station to be fed from either 25 kV transformer with two (2) distribution pumps fed from each of the respective main busses. A Main-Tie-Main electrical service allows for the power for the distribution pumps to be fed from either 25 kV transformer simultaneously, or individually. During maintenance, a Main-Tie-Main electrical service also allows for half of the electrical distribution system to be isolated and shutoff for maintenance, while the other half remains energized and operational. This allows for planned maintenance to be performed without needing to fully shut down the Raw Water Transfer Station.

Each pump will include a motor protection relay for monitoring and safety interlocks ensuring the motor is not run outside of safe operating conditions. Each VFD will be specific to have an “active front end” or 24 pulse configurations, ensuring harmonics are limited to well within the IEEE 519 recommended requirements.

A separate Low Voltage (<750 V) electrical room will house the 600 V and 208 V distribution equipment for Raw Water Transfer Station building loads such as HVAC, smaller pumps, compressors, HVAC, along with network equipment and control panel. The outdoor standby generator will be connected to an automatic transfer switch located in the electrical room to provide standby power for the 600 V and 208 V distribution.

The wiring will be primarily run using RPVC, rigid metal, or armoured teck/tray cable run in cable tray. Where possible, conduit will be run hidden in walls, below grade, or embedded in-slab.

4.7.1 Communications and Controls Overview

The proposed Raw Water Transfer Station will house the control system for the raw water intake and transfer pumping system. A PC in the control room is proposed to act as the primary operator's interface and will house the Supervisory Control and Data Acquisition (SCADA) system. The SCADA system would also be accessible remotely through a secure cellular or satellite-based internet connection.

The Raw Water Transfer Station and Raw Water Intake equipment will be hard wired and controlled via a programmable logic controller (PLC) in the main control panel. Because of the remoteness of the raw water storage ponds, water level feedback from the raw water ponds will need to be via a cellular or satellite internet connection. The system is proposed to be capable of automated operation based on a control narrative including but not limited to, flow setpoint and pressure-based pump control, river and wetwell level monitoring, and automated intake backflush.

4.8 Architectural & Structural

The proposed Raw Water Transfer Station will be designed to meet the requirements in the National Building Code – 2019 Alberta Edition (ABC) and the National Energy Code of Canada for Buildings 2017 (NECB). The building will be classified as a F3 Low Hazard Industrial Occupancy and Post-Disaster structure in accordance with ABC. The following building envelope for exterior walls is proposed for the Raw Water Transfer Station due to its durability, performance and suitability:

- 100 mm thick concrete block veneer
- 38 mm air space
- 100 mm thick rigid insulation
- Peel and stick vapour barrier
- 200 mm thick concrete block

The following roof system is proposed:

- 24 ga. prefinished standing seam metal roofing
- Underlayment
- 16 mm plywood sheathing
- Wood trusses c/w 400 mm thick blown-in insulation

- 6 mil poly vapour barrier
- 10 mm plywood sheathing
- 28 ga. prefinished metal liner

All interior walls will be constructed with 200 mm thick concrete block wall and the building will be sitting on a reinforced concrete slab.

5.0 REGULATORY APPROVAL & PERMIT OVERVIEW

5.1 General Description

The proposed water supply for the GIG will be regulated by Alberta Environment and Parks (AEP) through the application of the *Water Act*. When applied to municipal waterworks systems, the Water Act is concerned primarily with water quantity allocations. Terms of water allocation are dictated by water diversion licenses that are held by communities and organizations.

5.2 Regulatory Requirements

5.2.1 Design Standards and Guidelines

The design shall conform to all applicable local, provincial, and/or federal codes, standards, regulations, and references in effect at time of this document including but not limited to the following:

- AEP, Standards and Guidelines for Municipal Waterworks, Wastewater and Storm Drainage Systems, 2012
- National Building Code; 2019AE
- National Energy Code; 2017
- Illuminating Engineers Society (IES) Standards

The design will comply with the requirements of the following organizations, at a minimum:

- CSA, Canadian Standards Association
- CEC, Canadian Electrical Code C22.1
- IEEE, Institute of Electrical and Electronic Engineers
- EEMAC, Electrical and Electronic Equipment Manufacturers Association of Canada
- NEMA, Standards of National Electrical Manufacturers Association
- ANSI, American National Standards Institute

5.2.2 Water Act and Raw Water Diversion License

The Municipal District of Greenview applied for a water diversion license for the GIG project in June of 2021. The source of water is an intake structure located in the Smoky River. The license request is summarized in Table 5.1 below, with a requested term of 20 years.

Table 5.1: Diversion Rate Summary – Water Act License Request

License	Point of Diversion	Min. Diversion Rate	Max. Diversion Rate	License #	License Allocation
Smoky River License for Diversion of Water	NE 30-67-4-W6	680 L/s	1,270 L/s	Pending	24,000,000 m ³

The diversion of water license application is still currently under review. Final license details to be determined once application approval is complete.

5.2.3 Public Lands Act

The Crown claims ownership of the bed and shore of all natural water bodies in Canada, and the land on which the Raw Water Transfer Station and intake piping is proposed lies on Crown land.

A temporary field authorization will be required for the temporarily occupied areas identified in river intake structure and pipeline as outlined in TM#2.

A disposition is required before performing any construction activities on Crown Lands - activities such as the construction of the intake, intake piping, and lift station. A disposition will be required under the Public Lands Act for the bed and shore of the Smoky River and for the lift station on the Crown-owned land. Under the Public Lands Act an adequacy assessment will also be completed to determine the requirement for First Nation consultations. The M.D. has engaged in First Nation consultations with the help of the consulting team.

5.2.4 Historical Resources Act

Water distribution and diversion projects require formal approval under the Historical Resources Act prior to the onset of activities. Upon completion and review of the Historic Resources Application, project-specific requirements will be issued.

6.0 FINANCIAL ANALYSIS

6.1 Raw Water Transfer Station Capital Cost

A Class D cost estimate was developed to a conceptual level for the construction of the Raw Water Transfer Station. Costs were estimated based on assumed construction methods. The cost estimate includes contingency and engineering fees. The detailed cost estimate can be found in Appendix A.

The cost estimate is based on the following assumptions:

- The unit costs are based on quotes and previous projects of a similar nature.
- A sloped excavation will be used for wetwell excavation.
- A 40% contingency was included to account for cost uncertainty due to the conceptual nature and the lack of available geotechnical data.

The following table, Table 6.1, provides a summary of the Class D cost estimate completed for the construction of the Raw Water Transfer Station.

Table 6.1: Raw Water Pump Station Cost Estimate Summary	
Description	Cost
Mobilization/Demobilization	\$3,786,700.00
Civil/Site Work	\$3,657,000.00
Structural	\$8,707,700.00
HVAC Mechanical	\$561,500.00
Electrical	\$6,630,000.00
Distribution Pumping System	\$5,288,300.00
Air Backflush System	\$400,000.00
Sub Total	\$29,031,200.00
Contingency (40%)	\$11,612,500.00
Engineering (15%)	\$6,096,600.00
Grand Total	\$46,740,300.00

6.2 Phasing of Capital Costs

As stated in Technical Memorandum # 2, the proposed water intake design concept includes four (4) chambers with four (4) intake screens. The GIG project area is scheduled to be constructed in three (3) phases which will require one (1) pump intake per phase plus redundancy.

Phase 1 will require the largest capital and include all the civil, structural, and HVAC components for the full build out. The majority of the electrical including one of the switch yard transformers, the air burst system, and two pumps and their associated piping will be part of Phase I. Phase II will include the installation of the second switch yard transformer and one pump. While the last Phase III will only include the installation of the last fourth pump.

For costing purposes, it is assumed that Phases II and III will occur in the 10- and 20-year horizon; with a 5% inflation. The following table, Table 6.2, provides the Phase I, II and II breakdowns from the total cost estimate for the construction of the Raw Water Transfer Station.

Table 6.2: Phase I, II, and III Preliminary Capital Cost Breakdown	
Phase	Cost
Phase I	\$41,810,400.00
Phase II	\$5,071,400.00
Phase III	\$4,821,800.00

7.0 CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

The following provides the major conclusions of this assessment:

1. Design Criteria based on previous assessments and investigations completed by the consulting team were summarized and presented in Section 2.0. The key assessments related to the Raw Water Transfer Station include a preliminary raw water transmission pipeline design, preliminary hydrotechnical design, and a geotechnical investigation.
2. This Technical Memorandum should be reviewed in combination with TM#2 – Intake Concept Design.
3. The proposed Raw Water Transfer Station lower-level design includes inlet from four (4) 900 mm inlet pipes from the inlet structure. Each inlet pipe will have an isolation gate valve and feed into a common wetwell chamber. The wetwell can be separated into two main chambers to accommodate project phasing with a total of four (4) pump bays each containing the wet end of a vertical turbine pump capable of 0.355 m³/s under the hydraulic conditions dictated by the downstream raw water transmission pipeline requirements.
4. Due to geometry of the site proposed intake design, the proposed wetwell will be required to be approximately 13 m deep. Based on geotechnical review the available construction methods will be a sloped excavation with a minimum 2H:1V side slopes, a full-depth secant pile wall or a steeper excavation requiring anchors with shotcrete. Additional geotechnical review will be required for detailed design. For the purpose of this technical memorandum a sloped excavation with sideslopes of 2H:1V has been assumed.
5. The proposed Raw Water Transfer Station upper-level design includes four (4) planned vertical turbine pumps and a phased high pressure pump header to connect to each of three (3) proposed raw water transmission pipelines that will convey water to the proposed GIG raw water storage. The proposed Raw Water Transfer Station process includes process piping and equipment for hydraulic and air backflush of the nearby intake structure.
6. The proposed Raw Water Transfer Station should include provision for service and removal of all large mechanical and electrical equipment including a flatbed accessible service bay and bridge crane in the main process area.
7. The proposed Raw Water Transfer Station will include provision for medium voltage (5kV) electrical and low voltage (< 750V) service equipment in separate rooms as well as an operations control room and high voltage outdoor switch yard.
8. A Class D cost estimate has been prepared. The total cost of the Raw Water Pump Station has been estimated to be approximately \$46,740,300.

7.2 Recommendations

The following recommendation has been made:

1. Proceed with detailed design including design based on the concepts outlined in the Memo.

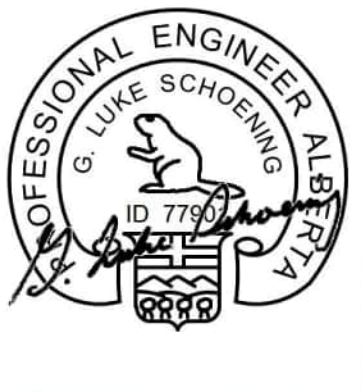
CORPORATE AUTHORIZATION

This report has been prepared by MPE a division of Englobe under authorization of the Municipal District of Greenview (M.D.) on behalf of WaterSMART a Hazen company. The material in this report represents the best judgment of MPE a division of Englobe given the available information. Any use that a third party makes of this report, or reliance on or decisions made based upon it is the responsibility of the third party. MPE a division of Englobe accepts no responsibility for damages, if any, suffered by a third party as a result of decisions made or actions taken based upon this report.

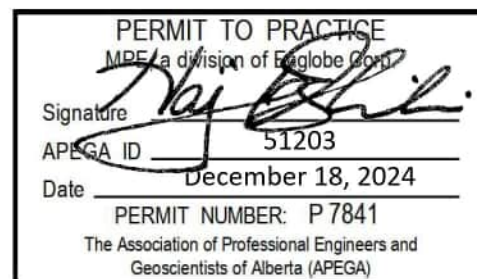
Should any questions arise regarding content of this report, please contact the undersigned.

MPE a division of Englobe

Prepared by:



Luke Schoening, P.Eng.



Corporate Permit

Professional Seal

APPENDIX A

Class D Cost Estimate



Municipal District of Greenview c/o WaterSMART
Greenview Industrial Park
Raw Water Pump Station
CLASS D CAPITAL COST ESTIMATE (January 2024)

	DESCRIPTION	QUANTITY	UNIT	UNIT PRICE	INSTALL	COST
1	GENERAL ITEMS					
1	Mobilization/Demobilization (15%)	1	L.S.	3,786,700.00		3,786,700.00
		SUBTOTAL				3,786,700.00
2	CIVIL/SITE WORK					
2.1	Gravel Subbase - Access Road & Pad (200mm)	3,000	m ³	120.00	60.00	540,000.00
2.2	Surface Course - Access Road & Pad (200mm)	2,500	m ³	120.00	60.00	450,000.00
2.3	Excavation	13,500	m ³	25.00		337,500.00
2.4	Imported Fill	17,550	m ³	50.00		877,500.00
2.5	Compacted Fill	13,500	m ³	40.00		540,000.00
2.7	Topsoil Stripping & Placement & Seeding	4,500	m ²	15.00		67,500.00
2.8	Chain Link Fencing	170	m	400.00	200.00	102,000.00
2.9	Electric Gate	1	ea.	30,000.00	15,000.00	45,000.00
2.10	Man Gate	1	ea.	5,000.00	2,500.00	7,500.00
2.11	Gravel Pad	1,900	m ²	50.00		95,000.00
2.12	600 mm Distribution Lines	150	m	900.00	900.00	270,000.00
2.13	Intake Lines Connections	1	L.S.	100,000.00		100,000.00
2.14	Care of Water	1	L.S.	50,000.00		50,000.00
2.15	Erosion and Sedimentation Control	1	L.S.	25,000.00		25,000.00
2.16	Miscellaneous	1	L.S.	150,000.00		150,000.00
		SUBTOTAL				3,657,000.00
3	STRUCTURAL					
3.1	Masonry Block Walls - 200 x 200 x 400	610	m ²	650.00	260.00	555,100.00
3.2	Masonry Block Veneer - 100 x 200 x 400 Split Face	450	m ²	450.00	180.00	283,500.00
3.3	Concrete (Structural, Wetwell, Generator Pad)	830	m ³	2,500.00	2,500.00	4,150,000.00
3.4	Piles	1	LS	750,000.00	300,000.00	1,050,000.00
3.5	Steel Cladding	1	LS	300,000.00	120,000.00	420,000.00
3.6	SM Insulation Walls	1	LS	100,000.00	40,000.00	140,000.00
3.7	Plywood Sheathing	1	LS	32,000.00	12,800.00	44,800.00
3.8	Xypex Coating	1	LS	200,000.00	80,000.00	280,000.00
3.9	Main Truss	1	LS	120,000.00	48,000.00	168,000.00
3.10	Misc Steel	1	LS	40,000.00	16,000.00	56,000.00
3.11	Windows	1	LS	3,000.00	1,200.00	4,200.00
3.12	Damp Proofing	1	LS	10,000.00	4,000.00	14,000.00
3.13	Epoxy Floor Painting	1	LS	250,000.00	100,000.00	350,000.00
3.14	Aluminum - Access Hatches	1	ea	9,000.00	3,600.00	12,600.00
3.15	Aluminum - Stairs	1	LS	65,000.00	26,000.00	91,000.00
3.16	Aluminum - Grating	1	LS	65,000.00	26,000.00	91,000.00
3.17	Aluminum - Handrails	1	LS	65,000.00	26,000.00	91,000.00
3.18	Aluminum - Pipe Supports	1	LS	60,000.00	24,000.00	84,000.00
3.19	Travelling Crane	1	LS	85,000.00	34,000.00	119,000.00
3.20	Doors and Hardware	10	ea	6,500.00	2,600.00	91,000.00
3.21	Custom Door	1	ea	22,500.00	9,000.00	31,500.00
3.22	Overhead Doors	2	ea	35,000.00	14,000.00	98,000.00
3.23	Wall Paint	1	LS	75,000.00	30,000.00	105,000.00
3.24	Millwork	1	LS	20,000.00	8,000.00	28,000.00
3.25	Miscellaneous	1	L.S.	350,000.00		350,000.00
		SUBTOTAL				8,707,700.00
4	HVAC MECHANICAL					
4.1	Domestic Plumbing	1	LS	12,000.00	12,000.00	24,000.00
4.2	Fire Extinguishers	1	LS	10,000.00	5,000.00	15,000.00
4.3	Indirect Hot Water Heater	1	LS	3,000.00	1,500.00	4,500.00
4.4	Condensing Boilers	1	LS	18,000.00	9,000.00	27,000.00
4.5	Expansion Tanks and Boiler Accessories	1	LS	8,000.00	4,000.00	12,000.00
4.6	Heating Pumps	1	LS	6,000.00	3,000.00	9,000.00
4.7	Unit Heaters	1	LS	50,000.00	25,000.00	75,000.00
4.8	Heating Controls Mechanical Installation Component	1	LS	7,000.00	3,500.00	10,500.00
4.9	Hydronic Heating Piping	1	LS	17,000.00	8,500.00	25,500.00



Municipal District of Greenview c/o WaterSMART
Greenview Industrial Park
Raw Water Pump Station
CLASS D CAPITAL COST ESTIMATE (January 2024)

DESCRIPTION		QUANTITY	UNIT	UNIT PRICE	INSTALL	COST
4.10	Forced Air Furnace	1	LS	5,000.00	2,500.00	7,500.00
4.11	Split Ductless Air Conditioner	1	LS	7,000.00	3,500.00	10,500.00
4.12	Condenser and Fan Coil	1	LS	5,000.00	2,500.00	7,500.00
4.13	Heat Recovery Ventilator	1	LS	3,000.00	1,500.00	4,500.00
4.14	Insulation	1	LS	12,000.00	6,000.00	18,000.00
4.15	Direct Fired Make-up Air Unit	1	LS	65,000.00	32,500.00	97,500.00
4.16	Exhaust Fans	1	LS	25,000.00	12,500.00	37,500.00
4.17	Supply Fans	1	LS	9,000.00	4,500.00	13,500.00
4.18	Motorized Dampers and Louvers	1	LS	35,000.00	17,500.00	52,500.00
4.19	Ductwork and Accessories	1	LS	25,000.00	12,500.00	37,500.00
4.20	Washroom and Laboratory Appliances	1	LS	7,000.00	3,500.00	10,500.00
4.21	Natural Gas Piping	1	LS	8,000.00	4,000.00	12,000.00
4.22	Miscellaneous	1	L.S.	50,000.00		50,000.00
		SUBTOTAL				561,500.00
5	ELECTRICAL					
5.1	Electrical Service Connection	1	L.S.	-		-
5.2	25kV Switchgear / Grounding	1	L.S.	650,000.00	260,000.00	910,000.00
5.3	3/5 MVA Transformers 25 kV to 5 kV	2	ea.	500,000.00	200,000.00	1,400,000.00
5.4	350 kVA Transformer 25 kV to 600 V / Pad / Grounding	1	L.S.	75,000.00	30,000.00	105,000.00
5.5	1250 HP VFDs 5 kV	4	ea.	325,000.00	130,000.00	1,820,000.00
5.6	5kV Switchgear	1	L.S.	450,000.00	180,000.00	630,000.00
5.7	LV Distribution Equipment (600V / 208V MCCs, Panels, Lighting, Recept.	1	L.S.	350,000.00	140,000.00	490,000.00
5.8	PLC SCADA System	1	L.S.	175,000.00	70,000.00	245,000.00
5.9	HV / MV / LV Conduit and wiring etc.	1	L.S.	250,000.00	100,000.00	350,000.00
5.10	Startup and Commissioning	1	L.S.	100,000.00		100,000.00
5.11	Diesel Generator	1	L.S.	200,000.00	80,000.00	280,000.00
5.12	Miscellaneous	1	L.S.	300,000.00		300,000.00
		SUBTOTAL				6,630,000.00
6	DISTRIBUTION PUMPING SYSTEM					
6.1	Vertical Turbine Pumps (1250 HP)	4	ea.	400,000.00	160,000.00	2,240,000.00
6.2	900mm Submerged Knife Gate Valves	4	ea.	150,000.00	60,000.00	840,000.00
6.3	Piping (900 mm)	75	m	2,100.00	840.00	220,500.00
6.4	Piping (600 mm)	25	m	1,800.00	720.00	63,000.00
6.5	Butterfly Valve (900 mm)	1	ea.	115,000.00	46,000.00	161,000.00
6.6	Butterfly Valve (600 mm)	3	ea.	75,000.00	30,000.00	315,000.00
6.7	Butterfly Valve (400 mm)	8	ea.	28,000.00	11,200.00	313,600.00
6.8	Silent Check Valves (400mm)	4	ea.	21,000.00	8,400.00	117,600.00
6.9	Pressure Control Valve	1	ea.	55,000.00	22,000.00	77,000.00
6.10	Pump Control Valve (250 mm)	4	ea.	43,000.00	17,200.00	240,800.00
6.11	900mm Flow Control Valves	4	ea.	60,000.00	24,000.00	336,000.00
6.12	Surge Pressure Relieve Valve	3	ea.	17,500.00	7,000.00	73,500.00
6.13	Magmeter	3	ea.	17,500.00	7,000.00	73,500.00
6.14	Multi Ranger Ultrasonic Level Transmitter	1	LS	6,000.00	2,400.00	8,400.00
6.15	Pressure Gauges with Isolators	4	ea.	1,000.00	400.00	5,600.00
6.16	Level Switches	2	ea.	1,000.00	400.00	2,800.00
6.17	Miscellaneous	1	LS	200,000.00		200,000.00
		SUBTOTAL				5,288,300.00
7	AIR BACKFLUSH SYSTEM					
7.1	Air Backwash c/w Control Panel, Air Compressor and Reciever	1	LS	200,000.00	80,000.00	280,000.00
7.2	Piping & Valving	1	LS	50,000.00	20,000.00	70,000.00
7.3	Miscellaneous	1	LS	50,000.00		50,000.00
		SUBTOTAL				400,000.00
TOTAL						\$ 29,031,200.00
CONTINGENCY (40%)						\$ 11,612,500.00
ENGINEERING (15%)						\$ 6,096,600.00
GRAND TOTAL						\$ 46,740,300.00

Municipal District of Greenview c/o WaterSMART Greenview Industrial Park Raw Water Pump Station CLASS D PHASING ESTIMATE (January 2024)																
		PHASE 1 (355 L/s)					PHASE 2 (710 L/s)					PHASE 3 (1065 L/s)				
DESCRIPTION		QUANTITY	UNIT	UNIT PRICE	INSTALL	COST	QUANTITY	UNIT	UNIT PRICE	INSTALL	COST	QUANTITY	UNIT	UNIT PRICE	INSTALL	COST
1	GENERAL ITEMS															
1	Mobilization/Demobilization (15%)	1	L.S.	3,387,300.00		3,387,300.00	1	L.S.	252,300.00		252,300.00	1	L.S.	147,300.00		147,300.00
		SUBTOTAL				3,387,300.00	SUBTOTAL				252,300.00	SUBTOTAL				147,300.00
2	CIVIL/SITE WORK															
2.1	Gravel Subbase - Access Road & Pad (200mm)	3,000	m ³	120.00	60.00	540,000.00		m ³	120.00	60.00	-		m ³	120.00	60.00	-
2.2	Surface Course - Access Road & Pad (200mm)	2,500	m ²	120.00	60.00	450,000.00		m ²	120.00	60.00	-		m ²	120.00	60.00	-
2.3	Excavation	13,500	m ³	25.00		337,500.00		m ³	25.00		-		m ³	25.00		-
2.4	Imported Fill	17,550	m ³	50.00		877,500.00		m ³	50.00		-		m ³	50.00		-
2.5	Compacted Fill	13,500	m ³	40.00		540,000.00		m ³	40.00		-		m ³	40.00		-
2.7	Topsoil Stripping & Placement & Seeding	4,500	m ²	15.00		67,500.00		m ²	15.00		-		m ²	15.00		-
2.8	Chain Link Fencing	170	m	400.00	200.00	102,000.00		m	400.00	200.00	-		m	400.00	200.00	-
2.9	Electric Gate	1	ea.	30,000.00	15,000.00	45,000.00		ea.	30,000.00	15,000.00	-		ea.	30,000.00	15,000.00	-
2.10	Man Gate	1	ea.	5,000.00	2,500.00	7,500.00		ea.	5,000.00	2,500.00	-		ea.	5,000.00	2,500.00	-
2.11	Gravel Pad	1,900	m ²	50.00		95,000.00		m ²	50.00		-		m ²	50.00		-
2.12	600 mm Distribution Lines	150	m	900.00	900.00	270,000.00		L.S.	100,000.00		-		L.S.	100,000.00		-
2.13	Intake Lines Connections	1	L.S.	100,000.00		100,000.00		L.S.	100,000.00		-		L.S.	100,000.00		-
2.14	Care of Water	1	L.S.	50,000.00		50,000.00		L.S.	50,000.00		-		L.S.	50,000.00		-
2.15	Erosion and Sedimentation Control	1	L.S.	25,000.00		25,000.00		L.S.	25,000.00		-		L.S.	25,000.00		-
2.16	Miscellaneous	1	L.S.	150,000.00		150,000.00		L.S.	150,000.00		-		L.S.	150,000.00		-
		SUBTOTAL				3,657,000.00	SUBTOTAL				-	SUBTOTAL				-
3	STRUCTURAL															
3.1	Masonry Block Walls - 200 x 200 x 400	610	m ²	650.00	260.00	555,100.00		m ²	650.00	260.00	-		m ²	650.00	260.00	-
3.2	Masonry Block Veneer - 100 x 200 x 400 Split Face	450	m ²	450.00	180.00	283,500.00		m ²	450.00	180.00	-		m ²	450.00	180.00	-
3.3	Concrete (Structural, Wetwell, Generator Pad)	830	m ³	2,500.00	2,500.00	4,150,000.00		m ³	2,500.00	2,500.00	-		m ³	2,500.00	2,500.00	-
3.4	Piles	1	L.S.	750,000.00	300,000.00	1,050,000.00		L.S.	750,000.00	300,000.00	-		L.S.	750,000.00	300,000.00	-
3.5	Steel Cladding	1	L.S.	300,000.00	120,000.00	420,000.00		L.S.	300,000.00	120,000.00	-		L.S.	300,000.00	120,000.00	-
3.6	SM Insulation Walls	1	L.S.	100,000.00	40,000.00	140,000.00		L.S.	100,000.00	40,000.00	-		L.S.	100,000.00	40,000.00	-
3.7	Plywood Sheathing	1	L.S.	32,000.00	12,800.00	44,800.00		L.S.	32,000.00	12,800.00	-		L.S.	32,000.00	12,800.00	-
3.8	Xypex Coating	1	L.S.	200,000.00	80,000.00	280,000.00		L.S.	200,000.00	80,000.00	-		L.S.	200,000.00	80,000.00	-
3.9	Main Truss	1	L.S.	120,000.00	48,000.00	168,000.00		L.S.	120,000.00	48,000.00	-		L.S.	120,000.00	48,000.00	-
3.10	Misc Steel	1	L.S.	40,000.00	16,000.00	56,000.00		L.S.	40,000.00	16,000.00	-		L.S.	40,000.00	16,000.00	-
3.11	Windows	1	L.S.	3,000.00	1,200.00	4,200.00		L.S.	3,000.00	1,200.00	-		L.S.	3,000.00	1,200.00	-
3.12	Damp Proofing	1	L.S.	10,000.00	4,000.00	14,000.00		L.S.	10,000.00	4,000.00	-		L.S.	10,000.00	4,000.00	-
3.13	Epoxy Floor Painting	1	L.S.	250,000.00	100,000.00	350,000.00		L.S.	250,000.00	100,000.00	-		L.S.	250,000.00	100,000.00	-
3.14	Aluminum - Access Hatches	1	ea.	9,000.00	3,600.00	12,600.00		ea.	9,000.00	3,600.00	-		ea.	9,000.00	3,600.00	-
3.15	Aluminum - Stairs	1	L.S.	65,000.00	26,000.00	91,000.00		L.S.	65,000.00	26,000.00	-		L.S.	65,000.00	26,000.00	-
3.16	Aluminum - Grating	1	L.S.	65,000.00	26,000.00	91,000.00		L.S.	65,000.00	26,000.00	-		L.S.	65,000.00	26,000.00	-
3.17	Aluminum - Handrails	1	L.S.	65,000.00	26,000.00	91,000.00		L.S.	65,000.00	26,000.00	-		L.S.	65,000.00	26,000.00	-
3.18	Aluminum - Pipe Supports	1	L.S.	40,000.00	16,000.00	56,000.00	1	L.S.	10,000.00	4,000.00	14,000.00	1	L.S.	10,000.00	4,000.00	14,000.00
3.19	Travelling Crane	1	L.S.	85,000.00	34,000.00	119,000.00		L.S.	85,000.00	34,000.00	-		L.S.	85,000.00	34,000.00	-
3.20	Doors and Hardware	10	ea.	6,500.00	2,600.00	91,000.00		ea.	6,500.00	2,600.00	-		ea.	6,500.00	2,600.00	-
3.21	Custom Door	1	ea.	22,500.00	9,000.00	31,500.00		ea.	22,500.00	9,000.00	-		ea.	22,500.00	9,000.00	-
3.22	Overhead Doors	2	ea.	35,000.00	14,000.00	98,000.00		ea.	35,000.00	14,000.00	-		ea.	35,000.00	14,000.00	-
3.23	Wall Paint	1	L.S.	75,000.00	30,000.00	105,000.00		L.S.	75,000.00	30,000.00	-		L.S.	75,000.00	30,000.00	-
3.24	Millwork	1	L.S.	20,000.00	8,000.00	28,000.00		L.S.	20,000.00	8,000.00	-		L.S.	20,000.00	8,000.00	-
3.25	Miscellaneous	1	L.S.	350,000.00		350,000.00		L.S.	350,000.00		-		L.S.	350,000.00		-
		SUBTOTAL				6,679,700.00	SUBTOTAL				14,000.00	SUBTOTAL				14,000.00
4	HVAC MECHANICAL															
4.1	Domestic Plumbing	1	L.S.	12,000.00	12,000.00	24,000.00		L.S.	12,000.00	12,000.00	-		L.S.	12,000.00	12,000.00	-
4.2	Fire Extinguishers	1	L.S.	10,000.00	5,000.00	15,000.00		L.S.	10,000.00	5,000.00	-		L.S.	10,000.00	5,000.00	-
4.3	Indirect Hot Water Heater	1	L.S.	3,000.00	1,500.00	4,500.00		L.S.	3,000.00	1,500.00	-		L.S.	3,000.00	1,500.00	-
4.4	Condensing Boilers	1	L.S.	18,000.00	9,000.00	27,000.00		L.S.	18,000.00	9,000.00	-		L.S.	18,000.00	9,000.00	-
4.5	Expansion Tanks and Boiler Accessories	1	L.S.	8,000.00	4,000.00	12,000.00		L.S.	8,000.00	4,000.00	-		L.S.	8,000.00	4,000.00	-
4.6	Heating Pumps	1	L.S.	6,000.00	3,000.00	9,000.00		L.S.	6,000.00	3,000.00	-		L.S.	6,000.00	3,000.00	-
4.7	Unit Heaters	1	L.S.													

Appendix D

Geotechnical Report (On Land)



June 27, 2023

MPE Engineering
10630 172 St NW #101
Edmonton, AB, T5S 1H8
Attention: Mr. Ivan Kagoro, P.Eng.

Subject: MEMO - Preliminary Cofferdam Cross Section - For Planning Purposes Only Rev.1
Greenview Industrial Gateway, Municipal District of Greenview, AB
Englobe reference: 5946-001-00

Dear Mr. Kagoro:

Englobe Corp. (Englobe) is pleased to provide MPE Engineering (MPE) with this letter, which includes a discussion of the preliminary cross section proposed for the temporary cofferdam at the water intake along the Smoky River for the Greenview Industrial Gateway (GIG). This memo must be read in conjunction with the report entitled Greenview Industrial Gateway Geotechnical Report (The Report).

Use of this Memo

The cross section discussed below is only to be used for preliminary planning purposes; it is the result of estimations based on previous experience and is not supported by technical analysis or calculations. As such, the proposed section below is subject to change once detailed design has been completed, potentially by a material amount.

Preliminary Cross Section

The proposed cofferdam will consist of a homogenous zone of material, comprised rock fill and clay, with the clay added to the upstream face of the cofferdam after placement of the rock. It is expected that this method of placement will allow 2H:1V slopes to be maintained while also reducing leakage to a minimum. An example of the cross section is shown in Figure 1

Depending on the amount of clay added to the upstream face of the cofferdam the final slope could be flatter than 2H:1V. This may require an adjustment to the slope of the rock fill or other measures to remain within the 2H:1V envelope. The crest width should be a minimum of 6 m to allow adequate working space for equipment.

Depending on the size and type of rock fill used, it may be necessary to provide armoring on the slope to prevent erosion and loss of the clay particles.

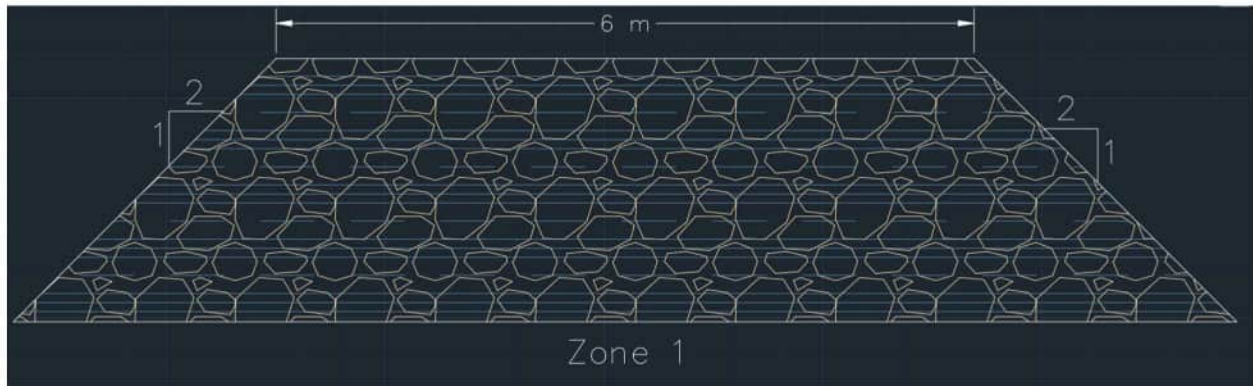


Figure 1 Preliminary Cross Section

Closure

We trust the enclosed is to your satisfaction. If, however, additional information should be required, please communicate with the undersigned.

Yours very truly,

Englobe Corp.



Prepared by:
Caleb Hayduk, M.Eng., P.Eng.
Geotechnical Engineer



Approved by:
Kim Johnston M.Sc., P.Eng.
Senior Geotechnical Engineer

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The findings, recommendations, suggestions, or opinions expressed in this Report reflect the Company's best professional judgment based on observations and/or information reasonably available at the time the work was performed, as appropriate for the scope, work schedule and budgetary constraints established by the Client. No other warranty or representation, expressed or implied, is included in this Report including, but not limited to, that the Report deals with all issues potentially applicable to the site and/or that the Report deals with any and all of the important features of the site, except as expressly provided in the scope of work.

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Since the passage of time, natural occurrences, and direct or indirect human intervention may affect the views, conclusions and recommendations (if any) provided in this Report, it is intended for immediate use.

The assessment should not be considered a comprehensive audit that covers and eliminates all present, past and future risks. The information presented in this Report is based on data collected during the completion of the monitoring conducted. The overall site conditions were extrapolated based on information collected at specific sampling locations. Professional judgement was exercised in gathering

and analyzing data; however, no monitoring method can completely eliminate the possibility of obtaining partially imprecise or incomplete information; it can only reduce the possibility to an acceptable level. Consequently, the actual site conditions between the sampling points may vary. In addition, analysis has been carried out only for the chemical and physical parameters identified, and it should not be inferred that other chemical species or physical conditions are not present.

It is recommended practice that the Company be retained during subsequent phases of the project, to confirm that the conditions throughout the site do not deviate materially from those encountered throughout the sampling program.

Any description of the site and its physical setting documented in this Report is presented for informational purposes only, to provide the reader a better understanding of the site and scope of work. Any topographic benchmarks and elevations are primarily to establish relative elevation differences between sampling locations and should not be used for other purposes such as grading, excavation, planning, development, or similar purposes.

Any results from laboratory or other subcontractors reported herein have been carried out by others, and the Company cannot warrant their accuracy.

This Report is based on the assumption that the design features relevant to our work will be in accordance with applicable codes, standards and guidelines of practice and constructed substantially in accordance with the Report. If there are any changes to the site development features, or there is any additional information that was not otherwise available at the time the work was performed, the Company should be retained to review the implications thereof to the contents of this Report. The design recommendations expressed in this Report are applicable only to the project described therein.

This Statement of Limitations forms an integral part of the Report.

Geotechnical Investigation

Greenview Industrial Gateway

Highway 40, Municipal District of Greenview, AB

Municipal District of Greenview

Final Report | Version 01

Reference no. 02208360.000 (5946-001-00)

June 27, 2023



ENGLOBE

Municipal District of Greenview 02208360.000 (5946-001-00)

Prepared by:



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Revisions and publications log

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0A	April 17, 2023	Draft version published for comments
0B	April 28, 2023	Draft version published for comments
00	May 23, 2023	Final version
01	June 27, 2023	Revision 01

Distribution

1 PDF copy	Haj Oshiki
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Executive Summary

Englobe Corp. (Englobe) was mandated by the Municipal District of Greenview, through its subsidiary MPE Engineering Ltd., to conduct a geotechnical investigation for the proposed river intake and storage structures for the Greenview Industrial Gateway project located south of Grande Prairie, along Highway 40 in the Municipal District of Greenview, Alberta. There are two locations of interest, termed the River Site and the Pond Site.

The general stratigraphy differed at the two sites. At the River Site, sandstone was encountered either at the surface or underlying a thin sandy soil. The sandstone comprised the majority of the investigation depth however there were units of siltstone, mudstone, coal and bentonite identified between 7.4 and 9.4 metres below ground surface (mbgs). Two boreholes, BH23-01 and BH23-02, were advanced to a planned depth of 15 mbgs, while one borehole, BH23-03, was advanced upslope at the top of a hill to ascertain the depth of the soil cover further from the river and was only advanced to 4.0 mbgs until its contact with the sandstone surface.

At the Pond Site, the stratigraphy was comprised of thin topsoil that had been removed at the location of BH23-04 and was underlain by clay to the extent of the boreholes. Both pond boreholes, BH23-04 and BH23-05, were advanced to the planned depth of 15.7 mbgs.

One standpipe piezometer was installed at the River Site, in BH23-02 with a screened interval between 7.6 to 10.7 mbgs with a sand pack and bentonite seal. The 15-day water level in BH23-02 was found to be 5.97 mbgs. Two standpipes were installed at the Pond Site, in BH23-04 and BH23-05, with 10 ft screened intervals extending up from the base of 15.7 mbgs. The 14-day water levels for were 3.4 and 15.3 mbgs, respectively. The groundwater table is expected to fluctuate seasonally, with variation in the order of 1 m or more. Care must be exercised to ensure the groundwater table is considered during construction. Sloughing was not encountered in any of the boreholes.

A preliminary assessment of the feasibility of two wet well excavation methodologies has been undertaken. An open, sloped excavation will likely be feasible with excavation slopes viable somewhere between 1.5:1 and 2:1. The other option deemed feasible, is a vertical cut supported by shotcrete and anchors.

Several methodologies for cofferdam construction are considered, however due to the likely presence of shallow bedrock in the river, sheet pile or cut-off pile methods will be expensive. Therefore, an earth and rock fill cofferdam are considered the most practical.

Investigation at the Pond Site has found a stiff clay throughout the full depth investigated. This material is considered likely suitable for construction of dykes, but more testing is required to better characterize its properties. While the two boreholes advanced did find consistent stratigraphy, it should not be taken as indicative of consistent stratigraphy throughout the Pond Site and additional investigation should be conducted to assess the variability across the ponds and search for geologic units that could present problems for the design of the dykes.

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APPENDICES

Appendix A	Figures
Appendix B	Rock Core Logs and Borehole Logs
Appendix C	Geotechnical Laboratory Results

1 Introduction

Englobe Corp. (Englobe) was mandated by the Municipal District of Greenview, through its subsidiary MPE Engineering Ltd. (MPE), to conduct a geotechnical investigation for the proposed river intake and storage structures for the Greenview Industrial Gateway (GIG) project located south of Grande Prairie, along Highway 40 in the Municipal District of Greenview, Alberta. There are two locations of interest, termed the River Site and the Pond Site.

The objective of the work was to investigate ground conditions at the proposed water intake, assess the excavation stability, investigate cofferdam construction methodology, and conduct a preliminary investigation of the proposed pond locations.

1.1 Scope of Work

The scope of work for this evaluation included three components:

- A field program comprising a total of five boreholes;
- Testing of recovered soil samples in Englobe's laboratory; and,
- Engineering analysis/reporting.

2 Site Description and Methodology

2.1 Proposed Development

A water intake from the Smoky River and an associated wet well is planned for development at the River Site. The wet well will be excavated to an elevation of 566 m, which is 14 m below the current ground surface. At the Pond Site, a series of raw water retaining ponds are planned. The size and location of these ponds has not been finalized. A water transmission line is planned to connect the water intake and the ponds. The route and depth of this transmission line has not been finalized.

2.2 Site Description

The River Site is located east of Highway 40 approximately 45 km south of Grande Prairie, Alberta. The proposed water intake is located along the Smoky River approximately 1.0 km south of the CanFor 200 road and is currently occupied by an abandoned county road, a bedrock outcrop, and undeveloped forest. The Pond Site is located within the designated boundaries of the GIG, east of Highway 40 approximately 50 km south of Grande Prairie, Alberta and spans an area approximately 3 km east to west by 1 km north to south. It is predominantly undeveloped with a few pipeline rights-of-way, access

roads, operating and abandoned pumping stations, forested with recently cleared forest areas. Figures 1 through 4, included in Appendix A, show the location of the Sites and the borehole locations, respectively.

2.3 Field Investigation

The field program was carried out from January 14 to 16, 2023, using a drill rig contracted from Vals Drilling Ltd. The drill rig was equipped with solid stem continuous-flight augers as well as a 50 mm diamond tipped rock coring stem. Soil samples were retrieved at intervals of approximately 0.6 m and all recovered rock core was retained. The soil was classified and logged by Englobe's field representative, Mr. Caleb Hayduk. Standard Penetration Testing was generally performed at regular intervals of 1.5 m where applicable. Water levels were measured in the boreholes 14 days following completion of the drilling.

Subsequent to obtaining public service clearances, five boreholes (BH23-01 to BH23-05) were drilled at the Sites, as shown on Figures 2, 3 and 4. All boreholes were advanced between January 14 and 16, 2023, using a track-mounted rig equipped with continuous flight solid stem augers and 50 mm diameter rock coring equipment supplied and operated by Vals Drilling Ltd., under the continuous supervision of an Englobe representative.

Two boreholes (BH23-01 and BH23-02) at the River Site were planned to a depth of 15 metres below ground surface (mbgs) and were completed as full depth diamond core holes. A 50 mm standpipe piezometer was installed in BH23-02 with the screened interval from 7.6 to 10.7 mbgs and sand was placed to 0.3 m above the filtered length of the standpipe followed by cuttings. The piezometer was sealed with bentonite chips. One borehole, BH23-03, was completed at the River Site as a solid stem auger hole to assess the depth of soil above bedrock at the top of the river valley. Borehole BH23-03 was completed to 3.9 mbgs.

Two boreholes (BH23-04 and BH23-05) at the Pond Site were planned to a depth of 15 mbgs and were completed to the full depth as solid stem soil boreholes. Standpipe piezometers, 25 mm in diameter, were installed to the full depth in both Pond Site boreholes.

Soil samples were recovered at regular intervals of depth using a 50 mm O.D. split barrel sampler driven into the subsoil in accordance with the Standard Penetration Test (SPT) procedure (ASTM D1586). In addition, disturbed soil samples were collected from the auger cuttings. The recovered soil samples were visually examined in the field and then preserved and transported to Englobe's Material Testing laboratory in Edmonton, Alberta, for further examination and testing. Borehole Logs are included in Appendix B.

All recovered rock core was collected, visually examined in the field then preserved and transported to Englobe's Material Testing laboratory in Edmonton, Alberta for further examination and testing. Rock core logs are included in Appendix B.

2.4 Laboratory Testing

In the laboratory, each soil sample was examined as to its visual and textural characteristics. Moisture content determinations (ASTM D2216) were carried out on all recovered samples. Additional testing was carried out on select recovered samples as detailed below:

- Three Hydrometer Analysis (ASTM D422-07);
- Four Atterberg Limits (ASTM D4318); and,
- Seven Unconfined Compressive Strength (UCS) (ASTM D2166/D2166M).

The geotechnical laboratory results are provided in Appendix C.

The number and type of tests conducted are summarized in Table 1.

Table 1: Summary of Laboratory Tests Completed

Name of Test	Number of Tests Completed	Sample Type	Laboratory	ASTMs Completed
Moisture Content	60	Soil	Englobe	ASTM D2216
Hydrometer	3	Soil	Englobe	ASTM D422-07
Atterberg Limits	4	Soil	Englobe	ASTM D4318
UCS	7	Rock	Englobe	ASTM D2166/D2166M

Summaries of testing results are briefly outlined in Tables 2, 3 and 4.

Table 2: Summary of Grain Size Distributions

Borehole No.	Depth (mbgs)	Grain Size Distribution Analyses (%)			
		Gravel	Sand	Fines	
				Silt	Clay
BH23-01	13.39	0.0	87.4	12.6	
BH23-02	4.80	0.0	90.6	9.4	
BH23-04	2.7	0.0	21.4	35.6	43.0
BH23-04	11.9	0.0	16.5	40.1	43.4
BH23-04	14.3	0.2	23.9	35.0	41.0

Table 3: Summary of Atterberg Limits Results

Borehole No.	Depth (mbgs)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Unified Soil Classification
BH23-02	8.8	146	42	103	CH
BH23-04	4.3	47.0	14.3	33	CI
BH23-05	6.7	39.1	13.4	26	CI
BH23-05	8.2	42.9	15.9	27	CI

Table 4: Summary of UCS Results

Borehole No.	Depth (mbgs)	Peak UCS (kPa)
BH23-01	5.68	5560
BH23-01	8.65	2096
BH23-01	12.50	1633
BH23-02	5.30	Damaged
BH23-02	9.82	287
BH23-02	11.67	Damaged
BH23-02	14.61	3943

2.5 Borehole Summary

The borehole coordinates were surveyed using a handheld GPS device by Englobe. Table 5 summarizes the borehole coordinates and borehole depths.

Table 5: Summary of Borehole Coordinates, Elevations and Depths

Borehole No.	Coordinates ¹			Borehole Depth (mbgs)	Instrumentation
	Northing	Easting	Elevation		
BH23-01	6076278	398513.6	578.8	15.2	-
BH23-02	6076280	398488.2	580.2	15.2	Standpipe Piezometer
BH23-03	6076257	398450.6	583.0	3.9	-
BH23-04	6073095	393294.1	725.45	15.2	Standpipe Piezometer
BH23-05	6073374	394148.5	708.37	15.2	Standpipe Piezometer

¹ NAD83 (CSRS)/ UTM 12N

3 Soil Conditions

Detailed descriptions of the soil conditions in each borehole are presented in the Borehole Log Sheets provided in Appendix B. Classification and identification of soils have been based on commonly accepted methods employed in the practice of geotechnical engineering. The stratigraphic boundaries shown on the Borehole Log Sheets represent transitions between soil types rather than distinct lithological boundaries. The subsurface conditions often vary both with depth and laterally between individual borehole locations.

The borehole locations are indicated on the Site Plan (Figure 2), which is included in Appendix A. The general soil conditions are outlined briefly below.

3.1 The River Site

3.1.1 Surficial Geology

Englobe reviewed surface geology mapping published by the Alberta Geological Survey (AGS). According to the Surficial Geology Map of Alberta (Alberta Geological Survey, 2013a), the Site's surficial geology is classified as glaciolacustrine deposits. AGS defines these deposits as follows:

Fluvial Deposits: *Poorly to well sorted, stratified to massive sand, gravel, silt, clay and organic sediments occurring in channel and overbank deposits, in placed includes a significant component of colluvial deposits as these two units are inseparable at 1 million map.*

3.1.2 Bedrock Geology

Englobe reviewed the bedrock geology map (Alberta Geological Survey, 2013b), and the Site's bedrock geology was classified as belonging to the Kaskapau formation. The AGS defines the Kaskapau formation as follows:

Wapiti Formation: *Siltstone and mudstone with subordinate sandstone (thin to thick bedded) coal, and bentonite, fluvial and lacustrine.*

3.1.3 Field Investigation

Based upon the borehole investigation for this project, the soil conditions at the wet well location are generally comprised of a very thin layer of surficial soils underlain by sandstone, with layers of mudstone, bentonite, coal, and siltstone below 7 mbgs. Boreholes were not completed in the river to assess the subsurface conditions of the river intake. At this time, due to the narrowing of the river and the associated water flow rate, it is assumed that the riverbed is comprised of shallow alluvial or colluvial sediments overlying sandstone bedrock.

Assessments of Rock Quality Designation (RQD), Total Core Recovery (TCR), Solid Core Recovery (SCR) and Fracture Index were completed on recovered core samples and are presented on the borehole logs included in Appendix B. Unconfined compression tests (UCS) were performed on samples with enough intact length for the test to be conducted. Two samples were unable to be prepped for testing without damage. As such, the UCS tests should be considered to represent the upper bound of potential rock strength and not representative of the entire unit.

Joint orientations in the bedrock ranged from 0 degrees to 73 degrees from horizontal, however 77 of the 82 identified natural joints dipped less than 30 degrees from the horizontal suggesting the bedding is predominantly flat.

3.1.3.1 Soil - Fill

Road fill soils were encountered in BH23-02 and BH23-03. The road fill varied in thickness from 1.3 to 2.0 m and was encountered on the surface. Fill material was generally well graded coarse to medium gravel and sand at the surface with sandy clay with some gravel and some silt below.

3.1.3.2 Soil - Native

Natural soil was only encountered in BH23-03 underlying the road fill and extended to a depth of approximately 4.0 m. The natural soil was generally clay, with some silt, some sand and trace gravel.

3.1.3.3 Sandstone

Sandstone was encountered in BH23-01 and BH23-02 and is the inferred termination depth of BH23-03. Sandstone was the primary material encountered and for conciseness will be considered to extend the full length of the borehole. In general, the sandstone was highly weathered within the first 3.0 to 4.0 mbgs and slightly weathered to fresh below. Additionally, BH23-01, located nearer to the riverbank, exhibited a greater degree of weathering at the surface than did BH23-02.

The sandstone was medium to coarse grained, extremely weak to weak, grey, or orange-brown when oxidized. Two particle size distributions performed on sandstone samples indicated 87.4% to 90.6% sand, with the remaining proportion comprised of fines.

3.1.3.4 Siltstone

Siltstone was encountered in BH23-02 between 8.42 and 9.40 mbgs. The siltstone was fresh to slightly weathered, fine grained, extremely weak to weak and greyish brown.

3.1.3.5 Mudstone

Mudstone was encountered in BH23-01 and BH23-02 in two intervals that were between 0.2 and 0.3 m in thickness. In BH23-01, the mudstone was encountered at 7.42 and 7.72 mbgs, while in BH23-02, the mudstone was encountered at 7.40 and 8.15 mbgs.

3.1.3.6 Residual Sand

Residual sands were encountered in several intervals in BH23-01 and BH23-02 where weathering had fully degraded the rock structure. In general, these residual sands were medium to coarse, uniformly graded and between 0.1 and 0.3 m in thickness.

3.1.3.7 Coal

Coal was encountered in BH23-02 and is presumed to have occurred in BH23-01 although it was not recovered. Coal was encountered as a unit approximately 0.70 m thick at an elevation of 572.5 m between the two identified mudstone layers. Fine coal seams were also noted within the mudstone and siltstone at a scale of approximately 5 mm.

3.1.3.8 Bentonite

Bentonite was encountered in BH23-02 and is presumed to have occurred in BH23-01 although it was not recovered. An Atterberg test was conducted and found a liquid limit of 146% and a plastic limit of 42%. The bentonite was encountered at 8.83 mbgs and was approximately 0.025 m thick. In general the bentonite was dry, hard and white.

3.1.4 Groundwater Conditions

Sloughing was not noted in any of the boreholes. Water was present upon completion of drilling for BH23-02. Groundwater levels were measured after drilling and 15 days after standpipe installation.

Table 6 summarizes the groundwater measurements. These results are also recorded on the Borehole Logs provided in Appendix B.

Table 6: Groundwater Observations at the River Site

Borehole No.	Instrumentation	Groundwater Depth at Completion (mbgs)	Groundwater Depth after 15 Days (mbgs)	Sloughing or Seepage Depth (mbgs)
BH23-02	Standpipe Piezometer	-	5.97	-

Variation in the order of 1 m or more is possible, depending on the season. As such, groundwater conditions during construction may differ.

Groundwater readings were taken on January 30, 2023, approximately 15 days following standpipe piezometer installation. It is recommended that the standpipe piezometers be left in place to allow further groundwater level measurements as the groundwater table is expected to fluctuate seasonally depending on rainfall events and run-off conditions. The water levels in the monitoring wells must be recorded prior to the start of construction.

3.2 The Pond Site

3.2.1 Surficial Geology

Englobe reviewed surface geology mapping published by the Alberta Geological Survey (AGS). According to the Surficial Geology Map of Alberta (Alberta Geological Survey, 2013a), the Site's surficial geology is classified as glaciolacustrine deposits. AGS defines these deposits as follows:

Glaciolacustrine Deposits: A) *Offshore sediment, rhythmically laminated to massive fine sand, silt and clay, locally debris released from floating ice.* B) *Littoral and nearshore sediments massive to stratified, well-sorted silty sand, pebbly sand and minor gravel.*

3.2.2 Bedrock Geology

Englobe reviewed the bedrock geology map (Alberta Geological survey, 2013b), and the Site's bedrock geology was classified as belonging to the Kaskapau formation. The AGS defines the Kaskapau formation as follows:

Wapiti Formation: Siltstone and mudstone with subordinate sandstone (thin to thick bedded) coal, and bentonite, fluvial and lacustrine.

3.2.3 Field Investigation

3.2.3.1 Topsoil

Topsoil was encountered in BH23-05 at the surface. Borehole BH23-04 did not have topsoil however the forest had recently been cleared and the topsoil was likely removed at that time. In BH23-05, the topsoil was approximately 0.3 m thick and was a sandy organic clay which was amorphous.

3.2.3.2 Clay

Clay was encountered in both boreholes; at the surface in BH23-04 and underlying the topsoil in BH23-05. The clay extended to 15.7 mbgs in both boreholes. Generally, the clay contained 'and silt', was sandy, had trace gravel near the surface, and was moist, medium plastic, was stiff to very stiff with brown and grey mottling.

The SPT 'N' values ranged from 10 to 30 blows per 0.30 m of penetration. The moisture content varied from 14.1% to 29.7%. The moisture content in BH23-05 was generally higher than the moisture contents in BH23-04. Atterberg testing indicated that the clay till was medium plastic. The constituents ranged from 0.0% to 0.2% gravel, 16.5% to 23.9% sand, 35.0% to 40.1% silt, and 41.0% to 43.4% clay. Pockets with higher sand content were observed in BH23-04 below 11.0 mbgs.

3.2.4 Groundwater Conditions

Sloughing was not noted in either borehole. Standpipe piezometers were installed in both boreholes. Water was present upon completion of drilling for BH23-04. Groundwater levels were measured after drilling and 14 days after standpipe installation. Sand was placed to 0.3 m above the filtered length of the standpipe followed by cuttings. The piezometers were sealed with bentonite chips.

Table 7 summarizes the groundwater measurements after completion of drilling and 14 days after standpipe installation. These results are also recorded on the Borehole Logs provided in Appendix B.

Table 7: Groundwater Observations at the Pond Site

Borehole No.	Instrumentation	Groundwater Depth at Completion (mbgs)	Groundwater Depth after 14 Days (mbgs)	Sloughing or Seepage Depth (mbgs)
BH23-04	Standpipe Piezometer	4.8	3.44	Seepage below 11.0
BH23-05	Standpipe Piezometer	Dry	15.34	-

Variation in the order of 1 m or more is possible, depending on the season. As such, groundwater conditions during construction may differ.

Groundwater readings were taken on January 30, 2023, approximately 14 days following standpipe piezometer installation. It is recommended that the standpipe piezometers be left in place to allow further groundwater level measurements as the groundwater table is expected to fluctuate seasonally

depending on rainfall events and run-off conditions. The water levels in the monitoring wells must be recorded prior to the start of construction.

4 Geotechnical Analysis and Recommendations at the River Site

The geotechnical recommendations provided in this section are from the interpretation of the geotechnical data obtained during the investigation and recommendations for geotechnical aspects of the design of foundations and general site development. Where comments are made on construction, they are provided to highlight aspects of construction that could affect the design of the project. It must be noted that information regarding construction must be interpreted to accommodate any change for designs, construction methods, costs, and scheduling.

4.1 Wet Well

The sandstone, mudstone and siltstone are expected to have sufficient bearing capacity for the wet well foundations; however, design of the wet well will be dictated mainly by temporary and permanent lateral support requirements. An adaptive design approach is provided where the stability model can be updated as excavation progresses allowing excessive movement, if observed to be mitigated and/or conservative design relaxed appropriately.

Three suitable wet well support options are presented:

- Sloped excavation;
- Full-depth secant pile wall; and,
- Anchors with shotcrete facing.

A sloped excavation is considered the simplest option of the feasible options; however, it is also likely the most laborious even if relatively steep, 2H:1V, slopes are facilitated by ground strength characteristics. A sloped excavation feasibility is in part governed by the strength of the weakest layer, in this case bentonite, which could pose a risk.

Tangent pile wall shoring is considered marginally feasible and higher risk as the excavation will extend below the groundwater table potentially requiring extensive dewatering. Maintaining pile alignment over 19 m to avoid gaps between piles resulting in soil/rock erosion is less constructable than the secant pile option.

Anchors with shotcrete facing are considered feasible reinforcement from existing grade to the base of the excavation as no surficial soils were identified at the wet well location.

The full depth secant pile wall option has not been discussed with foundation contractors and may be difficult or prohibitively expensive to implement given the depth of the excavation. The depth of the excavation indicates that lateral pile support will be required in the form of anchors and walers in addition to secant piles.

One advantage of a secant pile wall is that degradation of the rock when exposed to air and seepage is limited compared to a shotcrete and anchor wall. Another advantage is the reduced number of rows of

anchors. A disadvantage of the secant pile option is the cost of mobilizing a piling rig to site versus the smaller equipment for anchor and shotcrete installation. Another disadvantage is that the secant pile wall is constructed blind while the rock can be observed and any significant joints, fractures, or changes in the quality (strength) of the rock may be logged with the shotcrete and anchor option.

It is considered advantageous to consider a circular wet well. The lateral earth pressures on a circular shaft forces the concrete piles into compression, taking advantage of the compressive strength of concrete and making it a more efficient design which does not generally require anchors. This configuration would require walers which would likely be reinforced cast-in-place concrete.

Preliminary discussion of the sloped excavation, full depth secant pile and anchor with shotcrete facing options are provided in the following sections. It should be noted that any excavation, whether sloped, or shotcrete and anchor supported, will likely have its stability governed by the weakest unit, in this case bentonite. Any excavation that exposes the bentonite unit will require consideration of additional oxidation, hydration, or weathering in order to preserve its strength.

4.1.1 Sloped Excavation

The intended goals of the slope stability analyses were to assess the in-situ stability of the natural slope, wet well excavation, and river intake construction.

The stability of slopes is typically represented as a Factor of Safety (FS). FS is the ratio of soil or rock shear strength to shear stress along a plane within the slope. An FS of 1.0 is defined as reaching limit equilibrium and in a state of imminent failure or deformation. An FS greater than 1.0 indicates that the slope is currently stable. For this project, an FS of 1.3 is recommended for global stability of the wet well structure, i.e., slope failure surfaces with an FS of 1.3 do not intercept the wet well and its foundations provided that the excavation will only be open temporarily and is not considered a permanent fixture.

The strength properties used were based on Generalized Hoek-Brown Criterion which is an empirical failure criterion which establishes the strength of a jointed rock mass in terms of major and minor principal stresses.

Exposure to the elements could cause significant weathering and wetting to previously unweathered rock units, especially the mudstone, coal, siltstone, and bentonite. Once exposed, these units should be sealed with a thin layer of shotcrete or other sealant to prevent water and air from degrading them.

4.1.1.1 Stability Cross Section

The cross section used for the stability analyses was selected based on the surveyed elevation of the boreholes, LiDAR provided by MPE and a 14 m deep excavation at the proposed location of the wet well.

4.1.1.2 Rock Parameters

Rock strength parameters were based on field RQD estimates, laboratory testing, and laboratory results obtained on samples collected during the investigation. Mohr-Coulomb strength values were derived from Hoek-Brown rock strength values which were calculated from field measurements of GSI, and laboratory UCS tests. The Mohr-Coulomb values were then refined by back analysis of the natural riverbank slope and an assumed factor of safety of 1.0. Values for bentonite, residual soil, and coal were based on field observations, prior experience, and relevant published literature. Strength parameters selected for the analyses contained within this report are shown in Table 8.

Table 8: Soil and Rock Strength Parameters

Property	Bulk Unit Weight (kN/m ³)	Effective Cohesion (kPa)	Effective Internal Friction Angle (°)
Sandstone	23.5	21	35
Siltstone	21	9	19
Mudstone	21	9	19
Coal	19	9	19
Bentonite	16	0	22
Residual Soils	21	0	25

4.1.1.3 Stability Model

The geotechnical modeling computer program, Slope/W, version 2021.4, by GeoStudio, was used to complete the limit equilibrium analyses for this project. The geometry, material properties, and groundwater conditions for the stability model were based on information discussed in the previous sections. The strength parameters (friction angle, cohesion, Hoek-Brown Criterion) were varied to review the impact on the FS. The piezometric surface utilized in the analyses was based upon the groundwater data collected from the standpipe piezometers and assumed dewatering activities.

4.1.1.4 Stability Analysis Results

For the analysis of the slope from upslope of the wet well to the river, several scenarios were analyzed, and two key scenarios are described in further detail below: slope with natural groundwater table, and slope with dewatering activities.

The assumed groundwater level for the stability analyses was based on the ground water reading of 5.9 mbgs taken on January 30, 2023, and no dewatering. The water level measured in the BH23-02 standpipe is indicative of water pressures near the mudstone, coal, and siltstone layers as that is where the piezometer was screened. There is likely groundwater present in fractures nearer the surface as evidenced by oxidation and weathering noted in the borehole logs. The water present in the upper layers could have an impact on slope stability and constructability but cannot be assessed without additional investigation. An additional scenario with dewatering activities occurring was also analyzed.

In all cases, the critical failure surface tended to favor the bentonite and siltstone layers and was relatively shallow on the slope face, rather than a more globalized failure surface passing through the toe or the base of the excavation. Several slope angles, benching configurations, intermediate slope angles, and other geometries were assessed for stability and constructability.

For the scenario of natural groundwater conditions, a 2H:1V slope was found to have an FS of 1.3 which is considered acceptable for temporary excavations. The critical failure surface was relatively shallow, daylighting with failure surface entry point on the face of the slope, with the toe of the surface daylighting through the siltstone layer. The critical slip surface is shown in white on Figure 5.

If dewatering activities are performed to push the groundwater surface away from the slope, a steeper slope is viable. An overall 1.5H:1V slope was assessed as having an FS of 1.3. In this scenario, the critical failure surface is shallower than the one with natural groundwater, with the critical surface entering in the lower half of the upper slope and exiting in the upper half the of the lower slope. The location of the toe of the critical surface still favors the siltstone and bentonite layer. This is shown on Figure 6.

It should be noted that the above analyses are sensitive to the groundwater levels and as such if the no-dewatering option is selected, the groundwater level should be closely monitored during the course of construction. Groundwater levels could fluctuate significantly throughout the year and the water level found during the investigation likely is not at the annual peak. Also, degradation of the rock surface in

the form of swelling, slaking, and ravelling while the excavation is open could block seepage and increase groundwater pressures in the slope.

4.1.1.5 Groundwater Investigation, Seepage into the Excavation, and Dewatering

Water bearing fractures were identified throughout the depth of both boreholes. The hydraulic conductivity of these fractures and therefore feasibility of pumping and dewatering methods for the wet well excavation were not assessed. A 50 mm diameter monitoring well was installed in BH23-02 which could be used to perform slug tests and other tests to assess the risk of water flow through the fractures near the base of the excavation and further refine understanding of the water pressures. Englobe recommends that this further testing be carried out prior to finalization of the wet well design. It could also be beneficial to investigate the water pressures and flow rates in the near surface fractures as these could also have an impact on water ingress to the excavation and stability of the slope.

Dewatering activities would likely be performed with several rows of horizontal drains, drilled at a slight upwards angle, and extending at least 10 m behind the slope. Spacing of the dewatering wells would be dependant on a number of factors, including ground permeability and water table recharge rates. A detailed design of the dewatering efforts should be completed by a qualified contractor. Figure 6 shows the assumed level of the dewatered water table. The cost of dewatering activities, which likely would only be needed uphill or on the side of the excavation furthest from the river, could possibly be offset by the reduction in material that needs to be excavated. Englobe has not assessed the feasibility or costs associated with dewatering at this time.

It is anticipated that dewatering activities will only be required for the duration of the excavation. Once backfill of the excavation begins, dewatering will no longer be required.

4.1.2 Wet Well with Full Depth Secant Piles

If an open cut excavation is not desirable, bored cast-in-place secant piles with approximately 0.2 m interlock will likely be either 914 mm or 1220 mm in diameter and may be implemented. Infill piles may be constructed with a reduced concrete strength of 5 MPa. The reinforced piles will likely be cast with an embedded H or W section. The piles should extend a minimum of 3 m below the finished grade of the bottom of the wet well for stability reasons.

The bending moments on the secant piles will be large so lateral pile analysis will be essential to the detailed design. Soil input parameters vary across software platforms and may be provided by Englobe upon request but the software LPILE is commonly used.

Instrumentation and monitoring during construction is recommended to allow for re-evaluation and updating of the design as excavation progresses. This is common practice for deep excavations. The proposed instrumentation for the full depth secant pile wall option would include:

- Three inclinometers installed within secant piles on the upslope side, with one extending a minimum of 10 m below the finished bottom of excavation elevation and two extending 5 m below the bottom of the excavation;
- Two inclinometers installed in secant piles on the downslope side of the wet well 1.5 m from face extending to a minimum of 5 m below the lowest excavation elevation; and,
- One inclinometer installed in each of the other two sides of the well.

A program for the frequency of reading of the inclinometers, additional monitoring devices if required, and actions to be taken by the engineer and contractor in the event of movement will be developed during detailed design. These recommendations may be modified during construction.

If movement is observed, depending on the rate or change in rate of movement daily readings may be recommended. Inclinometer movement will be used to revise the design as required to maintain stability

of the excavation. If movement is observed the space of anchors may be reduced or their length increased.

A program for the frequency of reading of the inclinometers and actions to be taken by the engineer and contractor in the event of movement will be developed during detailed design. These recommendations may be modified during construction.

4.1.3 Wet Well with Shotcrete and Anchors

Shotcrete and anchors excavation support would allow for top-down construction in tandem with excavation. Ultimate grout to rock bond strengths for anchors were developed from UCS testing and from rock quality designations for the rock logs and published values (Post-Tensioning Institute, 2014). Parameters chosen for the supports, i.e., rock bolts, are shown in Table 9 and Table 10. One unoptimized option for anchor and shotcrete facing is summarized in Table 10 below. The large number of rows of anchors is due to the relatively low grout-rock bond strength. This design includes 10 rows of anchors, all 12 m long. This configuration has a factor of safety of 1.5.

Table 9: Wet Well Anchor Parameters

Material	Ultimate Rock / Soil Grout Bond Strength (kPa)	Elevation at BH23-02
Sandstone	90	Surface to 572.77, Below 570.77
Mudstone	30	572.77 to 571.75
Siltstone	50	571.75 to 570.77

Table 10: Preliminary Design Anchor Properties, full depth 75 mm shotcrete and anchor option, assumes free anchor length of 3.0 m for all rows

Rock Unit	Ultimate Grout-Anchor Bond Stress (kPa)	Drill Hole Diameter (m)	Bond Length (m)	Horizontal Anchor Spacing (m)	Vertical Anchor Spacing (m)
Sandstone	90	0.075	8	1.3	1.0
Mudstone	30	0.075	8	1.3	1.0
Siltstone	50	0.075	8	1.3	1.0

If the planned structure is to be permanent, double corrosion protection will be needed. This could result in larger diameter holes being required for installation. Residual soils were identified during the field investigation. It is possible that these layers will collapse when the anchor drill bit is removed before it can be replaced with the anchor. To mitigate this, casing may be necessary to keep the boreholes open until grouting occurs.

General recommendations of anchors include:

- Staggering of bond lengths for horizontal or vertical spacings of less than 1.2 m;
- Anchors oriented 15 degrees below horizontal;
- Centralizers spaced 3.0 m on center or less;
- Strip drains installed prior to shotcrete placement at 1.2 m to 2.4 m spacing; and,
- Grouting of anchors as soon as they are drilled.

If test anchor capacities are less than anticipated based on pullout testing, design adjustments may be required. Design adjustments may include more anchors or larger diameter anchor boreholes, etc.

The proposed instrumentation for the anchor and shotcrete option would include:

- Two inclinometers located approximately 1.5 m from the excavation face on the upslope side, with one extending to a minimum of 10 m below the finished bottom of the excavation elevation and one extending 5 m below the bottom of the excavation;
- One inclinometer on the downslope side of the wet well 1.5 m from the excavation face extending to a minimum of 5 m below the lowest excavation elevation; and,

As for the secant pile option, a program for the frequency of reading of the inclinometers and actions to be taken by the engineer and contractor in the event of movement will be developed during the detailed design phase. These recommendations may be modified during construction.

If movement is observed, depending on the rate or change in rate, daily readings may be recommended. Inclinometer movement will be used to revise the design as required to maintain stability of the excavation. If movement is observed the spacing of anchors may be reduced or their length increased.

4.1.4 Foundation Movement - Serviceability Limit State

The wet well building may either be founded on top of the wet well walls, or on an isolated shallow foundation constructed outside the wet well. An independent foundation would serve to isolate the building from wet well movements and is recommended. If constructed on an isolated foundation comprised of piles, settlements of 10 mm to 15 mm may be expected.

For foundations constructed on top of (and integrated into) the wet well, settlement estimates become more complex. Settlement of the wet well secant pile wall from construction of the Pump House and Wet Well Building is expected to be less than 10 mm due to the depth of the well and the presence of anchors. Predicting the movement of the walls of the deep excavation is inexact. There may be inward movement of the secant piles over time. The foundation of the building should be designed to accommodate this movement. Such movement is difficult to estimate as it depends on the existing stress state of the rock, groundwater, rate of excavation, the delay between excavation and shotcrete placement for the anchor option, based on other deep excavations it could be up to 50 mm.

If the anchored shotcrete wall option is selected, a thickened shotcrete or cast-in-place footing with adequate reinforcing could be constructed integral with the top of the wet well wall or an independent footing constructed outside the wet well.

4.2 River Intake

The foundation conditions at the river intake were not physically investigated as part of the scope of work. However, it is assumed that the soils at the river intake comprise granular soils with poorly graded sand and gravel predominating, possibly underlain by bedrock. At this time, it is assumed that the river intake will be founded on shallow foundations overlain by ballast of either native fill or riprap.

If investigation in the river is not possible due to cost, it is recommended that investigation be undertaken during construction. It is expected that may be accomplished with an excavator or backhoe. Suitable options should be developed for several likely soil types in advance and the optimal design selected at the time of construction.

4.2.1 General

The cofferdam crest height should be set such that cofferdam flooding is unlikely to occur. The US Federal Emergency Agency (FEMA) provides recommended flood return periods for cofferdams. If the projected length of the construction period is one year, the cofferdam would need to accommodate a flood with a return period of 5 years. If the projected length of the construction period is two years, a

flood with a return period of 10 years would be used (FEMA, 2005). The estimated 5-year and 10-year flood levels at the site are 576.63m and 577.18 m, respectively (Northwest Hydraulic Consultants Ltd., 2023).

Ice jams only occurred in approximately 35%-40% of break ups between 2000 and 2021 at the Watino and Wapiti gauge locations. The 5- and 10-year return peak ice jam elevations are 576.4 and 577.1 respectively, which are in line with the peak water flood elevations. A one-year serviceable cofferdam could be suitable with a crest elevation of 576.63 m plus a suitable freeboard (Northwest Hydraulic Consultants Ltd., 2023).

4.2.2 Dam Safety Considerations

The cofferdam is regulated under the Water Act Alberta *Dam and Canal Safety Directive* published by the Alberta Government in 2018, Section 5.10 Design of temporary structures. This section outlines the regulatory requirements for information to be submitted to the regulatory authorities. This includes:

- a) Design drawing and specifications for cofferdams, weirs conduits, and other temporary structures;
- b) The consequence classification of the structures;
- c) Justification for the selected design criteria for the structures;
- d) Hydraulic analyses of the temporary structures that are used to control or divert water during construction, with supporting hydraulic data;
- e) Stability analyses of the temporary structures under design usual and unusual loading conditions;
- f) Seepage analyses of the temporary structures and details on seepage control during construction, including use of pumping;
- g) An operation, maintenance and surveillance manual for the temporary structure;
- h) An emergency management plan for the temporary structure; and,
- i) The removal of the temporary structures that are used to control or divert water during construction.

When the Consequence Classification is determined, it is then used to inform the level of detail and scrutiny to which the dam is designed. Based on the preliminary information available, the cofferdam will likely have a significant to high consequence classification based on the number of people working within the cofferdam.

In addition to regular dam safety considerations, more hazards need to be assessed and managed due to the cofferdam being constructed in a river. The flow of water applies a force to the dam that will affect the stability balance as well as cause erosion along the exposed surface. If the cofferdam is to be operational during the winter freeze up or the spring thaw, moving ice will have implications for the stability of the dam as well as the required slope armoring. A cofferdam should not be considered impermeable to water. While the majority of water will be prevented from entering the cofferdam, some seepage is to be expected and as such pumps should be on hand to manage it. It is possible that seepage cannot be managed with pumping and sumps, in which case sheet pile cut off walls, watering wells, or internal berms may be required to manage seepage.

The following discussion is to provide feasible options of cofferdam types, but design of the preferred methodology is beyond the scope of this report.

4.2.3 Cofferdam Types

Construction of waterway cofferdams can follow many methodologies such as an earth embankment with:

- Rock fill placed on river side, with impervious clay on the downstream side;
- Earth fill with sheet piles driven along the centerline to reduce seepage;
- Double walled sheet piles backfilled with granular; or,
- Cellular sheet pile walls backfilled with granular fill.

There is a risk of the riverbed being comprised of bedrock with no or minimal overlying riverbed material. If this is the case, cofferdams with sheet piles will likely not be feasible due to the difficulty in seating them adequately in the rock to prevent seepage or stability issues. Because of this, Englobe believes the earth fill cofferdam with rock and clay is the more feasible option.

If there is a significant thickness of cobbles and gravel, dredging may be necessary to avoid significant leakage under the cofferdam. If not, the clay fill will generally tend to infill any flow paths at the interface between the cofferdam and riverbed material. If dredging is not feasible and excessive leakage is observed, sheet piles may need to be driven after the cofferdam is in place.

The earth fill option would include pushing fill into the river from shore to create a berm extending above the water level and extending this along the cofferdam alignment. It may be necessary to start with rock fill along the upstream side of the cofferdam to limit the loss of soil by erosion, then place clay against and upstream of the rock fill to limit seepage through the cofferdam. A filter layer is generally not included in below-water placement; the clay will fill surficial voids in the rock fill. This method is commonly used for cofferdams in lakes and rivers and is likely the preferred method. With this methodology, cofferdam side slopes roughly between 6:1 to 7:1 could be expected; however, some studies have shown cofferdams can have fine grained fill slopes as flat as 20:1. There are many compounding factors that determine the stable slope angle of clay placed into free water, some of which can only be assessed after construction is completed. Because of this complexity, a detailed stability analysis that considers actual selected fill material, the flow rate of the river along with many other factors may need to be performed while the cofferdam is being constructed.

One option to reduce the overall footprint of the cofferdam would be to construct the cofferdam of rockfill with slopes approximately 2:1, and place clay on the upstream face. This would allow the flow of the river to move the clay into the rock fill's void spaces thereby reducing the overall permeability of the cofferdam.

The crest width of the cofferdam would largely be controlled by the working width of the equipment used to construct it. A working area of 6 m has been assumed to allow for access by construction equipment.

The earth fill and sheet pile cofferdam could be considered a backup plan for an earth cofferdam with excessive leakage. The feasibility of this option will be limited if there are cobbles or boulders or shallow bedrock in the riverbed material, which appears likely.

A double walled sheet pile cofferdam could be considered. The risk of sheet pile refusal makes a geotechnical investigation in the river necessary, increasing the cost of this option. The two sheet pile walls would be connected by steel ties / bracing and the space between walls backfilled with granular material. This type of wall relies on friction from the granular material to limit overturning therefore the quality of granular material is essential for a stable wall.

As for the double walled cofferdam option, a barge based geotechnical investigation or similar would be required. The cells would need to be filled with granular material. The advantage of cellular cofferdams over a double walled cofferdam with ties is that less bracing is required for cellular walls. Cellular cofferdams are generally more expensive than other options.

If seepage or boils are observed inside the cofferdam perimeter, they may be covered with sand and gravel to provide stabilization and filtration.

It is understood there is not a nearby borrow for construction materials. Imported granular material for the double wall or cellular fill, low to medium plastic, non-dispersive clay or clay till would be required for the low hydraulic conductivity ("impervious") portion of the earth cofferdam.

Based on the preliminary investigation, the material identified at the Pond Site should be suitable for the cofferdam impermeable core. Granular materials would likely still need to be imported for armoring and stability.

4.2.4 Environmental

As for environmental concerns, regardless of the above construction methodology, prior to construction of the embankment or sheet pile system, the area underneath the structure would have to be dredged, leading to siltation downstream. With the earth embankment option, there will likely be siltation during placement of the granular material, as well as erosion and subsequent siltation if the river scours the placement of granular material, as well as erosion and subsequent siltation if the river scours the granular fill (rip rap on the upstream and river side of the cofferdam should be used to mitigate this).

When sheet piles are employed, there is the concern of excessive noise and vibrations during pile driving that can disturb aquatic life.

With any cofferdam, pumping of water within the cofferdam will be required after the structure is built to expose the river bottom. The stability of the cofferdam should be checked against draw down during dewatering as it may cause sloughing of the downstream side of the cofferdam.

During dewatering, a biologist would likely be involved in fish capture and release, there will also be a continuous dewatering requirement during construction from leakage. Depending on project environment requirements, this water may have to be stored in a settling pond prior to release if it has excessive fines.

Breaching of the cofferdam should be controlled by initially pumping or siphoning water into the dry side of the cofferdam to limit damage to the intake and riverbank erosion. When the major works are complete, and the cofferdam is being decommissioned, there will be some additional siltation during removal of the embankment fill. As well, for sheet piles they are generally cut-off at the base, and the embedded steel is left in the ground.

4.3 Construction Considerations

The design parameters provided in the preceding sections assume that a qualified contractor with relevant design and construction experience completes the work and good construction practices are followed. As ground conditions may vary from the five boreholes completed, it is recommended that a geotechnical engineer be present during construction to confirm that the conditions encountered are similar to those assumed.

Recommendations and design parameters presented herein are provided on the assumption that Englobe will be retained to provide engineering design review and construction supervision. Englobe takes no liability for work performed where Englobe is not retained to provide adequate construction supervision services. Construction inspection should include:

- Inspection of the excavated bearing surface for the wet well and river intake;
- Full time monitoring of pile construction of the river intake;
- Full time monitoring of slopes and any reinforcement options selected for construction and interior excavation during excavation and any slope stability impacting activities;

- Full time instrumentation monitoring during construction including inclinometers and vibrating wire piezometers;
- Full time monitoring and compaction testing for earthworks;
- Strength testing for concrete and grout; and,
- Monitoring and inspection of anchor testing.

Once excavation is complete, monitoring of the slope and reading of the associated instruments could be reduced from daily to weekly. If any activities are performed that could impact slope stability, such as excavation of the base or slope, a change in dewatering effort, placement of weight near the crest of the excavation, slope stability monitoring frequency would be increased. Similarly for the cofferdam, once construction is complete geotechnical monitoring could be reduced from daily to weekly, although the contractor would still be responsible for their own full-time monitoring of the coffer dam during and after construction.

4.3.1 Excavation of Rock

UCS testing of rock core samples has indicated a wide range of strengths, which are summarized in Table 4. It is worth noting that the samples selected for these tests were generally perceived to be on the weaker end of the spectrum in order to conservatively estimate strength values for slope stability. Schmidt hammer tests were conducted on some of the relatively hard samples and found rebound numbers correlating to UCS results between 10 MPa and 30 MPa

There are several ways to estimate the “rippability” or “excavatability” based on a number of rock mass parameters such as UCS, point load strength, GSI, and shear wave velocity. The method proposed by Pettifer and Fookes, is based on UCS and block size, suggests excavation at the River Site could range from hard digging to hard ripping and blasting, Tsiambaos and Saroglou suggest an estimate based on GSI for both rock masses with a high point load strength (>3 MPa) and low point load strength (<3 MPa). Based on the borehole investigation, the rock mass at the River Site is estimated to have a GSI in the range of 30-60 which correlates to ripping or hammering (Tsiambaos and Saroglou 2009).

Caterpillar Inc. (Caterpillar) has some guidance on ground rippability with various versions of their products. Their analysis is based on seismic shear wave velocity, which was not assessed during this investigation. The right equipment can facilitate ripping in rock with seismic velocities as high as 3500 m/s (Caterpillar, 2000) which would correlate to a hard rock (NBCC, 2015). While the specifications for Caterpillar products are referenced here, it is assumed that other similar equipment would be able to facilitate a similar level of ripping effort.

Kirsten proposes a more comprehensive classification methodology in *A Classification System for Excavation in Natural Materials*. Factoring in rock mass, joint spacing, roughness, alteration, dip direction and RQD excavatability is broken down into 8 categories ranging from hand digging to blasting. Based on the results of the field investigation and laboratory testing, the rock strongest rock encountered at the wet well location would be categorized as either 6 or 7, which correlates to very to extremely hard ripping, which is viable with a D9H or D10 sized ripper.

It is expected that these very hard layers do not comprise the majority of the rock mass and it may be feasible to rip through softer materials and hammer through the harder, to avoid blasting. The distinction will require continuous assessment in the field as conditions change.

Selection of equipment will determine the possibility and potential ease with which excavation is carried out. It will be up to the contractor to look at the information provided and determine if the equipment they propose to use will be sufficient to allow for excavation or if blasting will be required.

5 Preliminary findings at the Pond Site

Very stiff clay tills were encountered in both locations, for the full depth of the borehole in the central and east side of the Pond Site. No information can be inferred about the west side of the Pond Site as the distance of the nearest borehole is significant.

The clay tills encountered in both locations are likely to be suitable both for impermeable clay liners and dam fill material due to the high SPT blow counts observed during drilling and the high clay content. Groundwater is potentially an issue in the central region, due to the high-water table encountered, and while the 14-day water levels in BH23-05, did not indicate significant presence of ground water, it should not be considered indicative of the rest of the pond as ground conditions will vary.

A preliminary estimate for a reworked clay liner consisting of the native clay tills encountered at the Pond Site could have a permeability as low as 1×10^{-7} m/s. However, without representative laboratory testing of both undisturbed and remoulded samples from across the pond footprints there is significant uncertainty in this value.

Stiff clays can often support slopes with a steepness on the order of 3H:1V to 5H:1V. However, many factors such as construction methodology, groundwater regime, dam configuration, level of water impounded among others can have a drastic impact on this value. In addition, only a small area was investigated, and material properties can vary significantly over even small distances. The values provided above as simply provided to illustrate a general idea of possible dam size and should not be relied upon for any detailed design.

If, upon further investigation, an insufficient volume of suitable clay is found to reliably construct clay cores or clay liners for the dams, a synthetic liner could be considered. It should be noted that synthetic liners come with their own challenges with regard to slope stability, the buoyancy of the liner, seepage through the dam, and cost among others.

If these ponds are to be greater than 2.5 m in height or contain more than 30 000 m³ of water, they will be dams regulated by the Water Act Alberta *Dam and Canal Safety Directive* published by the Alberta Government in 2018. This directive outlines numerous requirements for the safe construction, maintenance and operation of a dam. Dams must be authorized by the regulator prior to construction and the designs must be approved. Some key components of the process as outlined in the directive include a requirement for a Dam Safety Management Plan, a Consequence Classification that ranges from low to extreme, and an Emergency Management Plan. In addition to the *Dam and Canal Safety Directive*, other documents such as the Canadian Dam Association's *Dam Safety Guidelines 2007 (2013 Edition)* further outline best practices for dam safety management, operation, emergency preparedness, dam safety reviews, and the analysis and assessment of dam design. Dam design is often a multidisciplinary operation that requires a level of thorough detail commensurate with the risk level (Consequence Classification) of that dam.

6 Further Considerations

6.1 The River Site

Access will need to be created to the River Site. For this report, access was obtained with all terrain vehicles, passing along an oil lease road off of the Canfor 2000, across pasture, pipeline rights-of-way, a small creek, and along a decommissioned municipal road. Creating sufficient access for the equipment required at the River Site will likely require construction of an all-weather gravel roadway. Depending on anticipated life span and size of equipment it needs to support, this road may require a geotechnical investigation and design. Englobe can submit a proposal for this scope upon request.

6.2 Pond Site

Further investigation is required to adequately characterize the geological variability in the area. The proposed ponds have a combined perimeter of approximately 8.0 km. Boreholes should be completed to sufficient depth every 100 m along the perimeter of the proposed pond locations, with an additional borehole completed within the pond footprint at a spacing of one per hectare. This will allow for adequate characterization of the soil stratigraphy and should allow for the identification of any smaller scale anomalies that could cause geotechnical issues. Problematic findings could include but are not limited to soft clays, heavily over consolidated clays, silts, low fines content sands or gravels or near surface bedrock of any kind.

Additionally, groundwater monitoring wells should be installed to assess the natural groundwater regime, measure groundwater flow magnitude and direction. These wells will also be used to conduct in situ permeability testing to facilitate modeling of water seepage through and under the dykes.

The drilling program should be accompanied by an extensive laboratory testing program that includes Atterberg, hydrometer, grain size distribution, direct shear, hydraulic conductivity, shear vane and unconfined compressive strength testing. Groundwater monitoring wells should be installed to characterize the existing hydrogeological regime.

Englobe can provide a formal scope of work and cost estimate for the further investigation required for this project.

6.3 Water Transmission Line

The shortest path between the River Site and the Pond Site is approximately 6 km in a straight line. The specific route of the pipeline should be chosen with careful consideration of topography, other utilities in the path, geohazards such as slope instability along creeks, marshes and other environmentally sensitive ecosystems, among others. Once a suitable path is determined, a geotechnical investigation should be completed along the pathway to inform on risks associated with open cut or horizontal directional drilling installation of the water transmission line. Assessment of the water transmission line route was not part of this scope, however, Englobe can submit a proposal for this scope upon request.

7 Closing Remarks

The soil and rock boundaries indicated on the borehole logs are inferred from non-continuous sampling and observations during drilling and must not be interpreted as exact planes of geological change. These boundaries are intended to reflect approximate transition zones for the purpose of geotechnical design. In addition, the subsoil and groundwater conditions have been determined at the borehole locations only.

Any depths expressed as metres below ground surface (mbgs) take into account the original ground elevation at the time of the investigation. If fill material is added to the Site, or if soil is removed, care should be taken during the subsequent design phases to ensure that parameters from the correct elevations are used.

The recommended bearing capacity has been calculated by Englobe from the information obtained from the borehole data.

It is further noted that, depending on the time of year the fieldwork was completed, water levels are expected to vary, perhaps significantly, from those observed at the time of this investigation.

8 References

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- Tsiambaos, G. and Saroglou, H. 2009. Excavatability assessment of rock masses using the Geological Strength Index (GSI).

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Since the passage of time, natural occurrences, and direct or indirect human intervention may affect the views, conclusions and recommendations (if any) provided in this Report, it is intended for immediate use.

The assessment should not be considered a comprehensive audit that covers and eliminates all present, past and future risks. The information presented in this Report is based on data collected during the completion of the monitoring conducted. The overall site conditions were extrapolated based on information collected at specific sampling locations. Professional judgement was exercised in gathering and analyzing data; however, no monitoring method can completely eliminate the possibility of obtaining

partially imprecise or incomplete information; it can only reduce the possibility to an acceptable level. Consequently, the actual site conditions between the sampling points may vary. In addition, analysis has been carried out only for the chemical and physical parameters identified, and it should not be inferred that other chemical species or physical conditions are not present.

It is recommended practice that the Company be retained during subsequent phases of the project, to confirm that the conditions throughout the site do not deviate materially from those encountered throughout the sampling program.

Any description of the site and its physical setting documented in this Report is presented for informational purposes only, to provide the reader a better understanding of the site and scope of work. Any topographic benchmarks and elevations are primarily to establish relative elevation differences between sampling locations and should not be used for other purposes such as grading, excavation, planning, development, or similar purposes.

Any results from laboratory or other subcontractors reported herein have been carried out by others, and the Company cannot warrant their accuracy.

This Report is based on the assumption that the design features relevant to our work will be in accordance with applicable codes, standards and guidelines of practice and constructed substantially in accordance with the Report. If there are any changes to the site development features, or there is any additional information that was not otherwise available at the time the work was performed, the Company should be retained to review the implications thereof to the contents of this Report. The design recommendations expressed in this Report are applicable only to the project described therein.

This Statement of Limitations forms an integral part of the Report.

Appendix A

Figures

Figure 1: Site Location

Figure 2: Aerial Photograph showing Borehole Locations and Access Routes

Figure 3: Detail 'A' Borehole Locations ("The River Site")

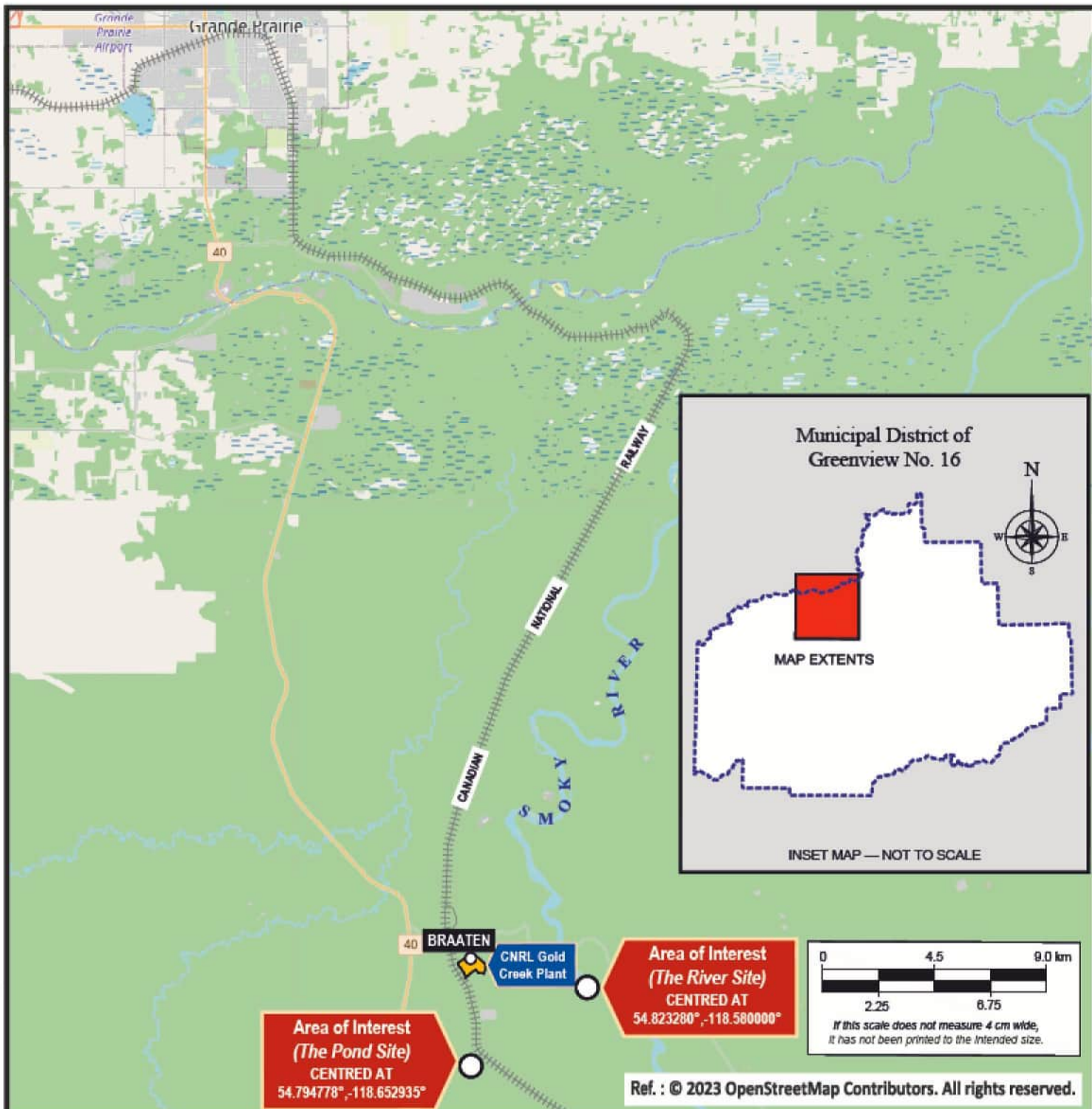
Figure 4: Detail 'B' Borehole Locations ("The Pond Site")

Figure 5: Excavation Stability Section - Natural Groundwater Conditions

Figure 6: Excavation Stability Section - Dewatered Groundwater Conditions



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Municipal District of GREENVIEW

**GEOTECHNICAL INVESTIGATION
GREENVIEW INDUSTRIAL GATEWAY
3-14-67-5 W6M and 4-29-67-4 W6M near Braaten, Alberta**

SITE LOCATION

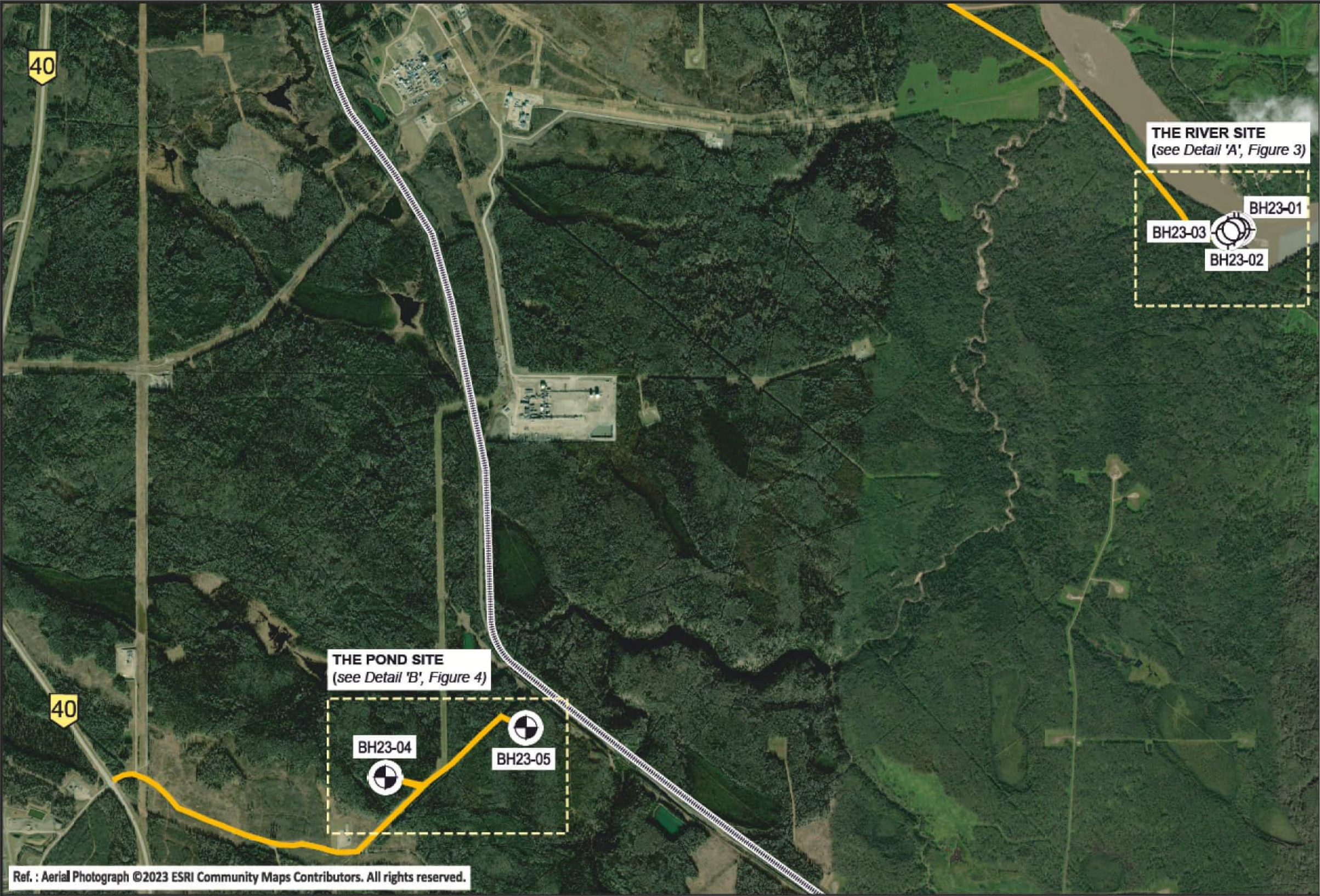


18114, 114 Avenue NW
Edmonton, Alberta
T5M 2Z5
780-481-1418

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Scale : 1:225,000		Drawn by : D. WILSON		Approved by : P. GINGRAS	
Date : June 2023		Figure no. : FIGURE 1			
Layout : LS		Paper size : LETTER (8.5" x 11")		Registration no. : —	

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LEGEND

- Access Route
- Canadian National Railway
- Borehole
- Borehole / Piezometer

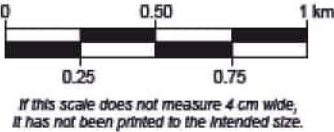
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BH23-03	6076257.31	398450.58	583.81
BH23-04	6073095.26	393294.10	725.46
BH23-05	6073374.18	394148.46	708.37

Datum is NAD83(CSRS) / UTM zone 11N

A	FINAL	2023-06-27	D.W.	R.H.	P.G.	
No.	Version	Date	By	Check	Appr.	
Discipline :		Prepared by :		Checked by :		
Scale :		C. HAYDUK		R. HUCULAK		
Date :		Drawn by :		Approved by :		
June 2023		D. WILSON		P. GINGRAS		
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**GEOTECHNICAL INVESTIGATION
GREENVIEW INDUSTRIAL GATEWAY
3-14-67-5 W6M and 4-29-67-4 W6M near Braaten, Alberta**

**Aerial Photograph showing
Borehole Locations and Access Routes**

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LEGEND

- Access Route
- Borehole

Location	Northing	Easting	Elevation (masl)
BH23-01	6076277.60	398513.58	578.85
BH23-02	6076279.83	398488.17	580.17
BH23-03	6076257.31	398450.58	583.81

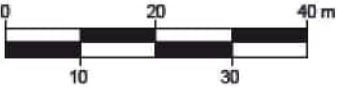
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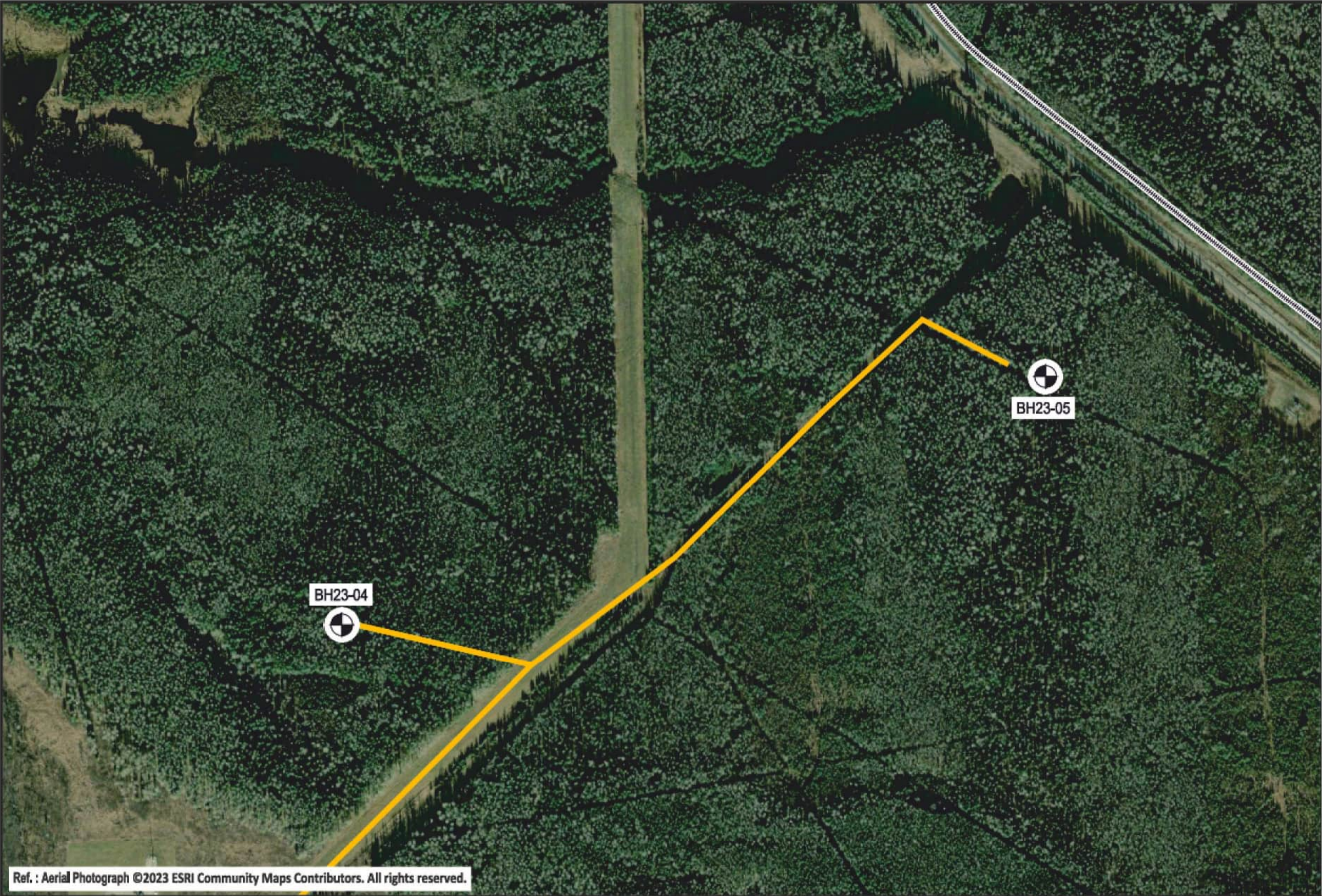
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GEOTECHNICAL INVESTIGATION
GREENVIEW INDUSTRIAL GATEWAY
3-14-67-5 W6M and 4-29-67-4 W6M near Braaten, Alberta

Detail 'A'
Borehole Locations ("The River Site")

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LEGEND

- Access Route
- Canadian National Railway
- Borehole / Piezometer

Location	Northing	Easting	Elevation (masl)
BH23-03	6076257.31	398450.58	583.81
BH23-04	6073095.26	393294.10	725.46
BH23-05	6073374.18	394148.46	708.37

Datum is NAD83(CSRS) / UTM zone 11N

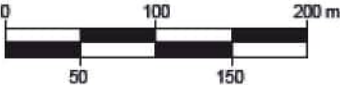
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June 2023		D. WILSON		P. GINGRAS		
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Municipal District of
GREENVIEW



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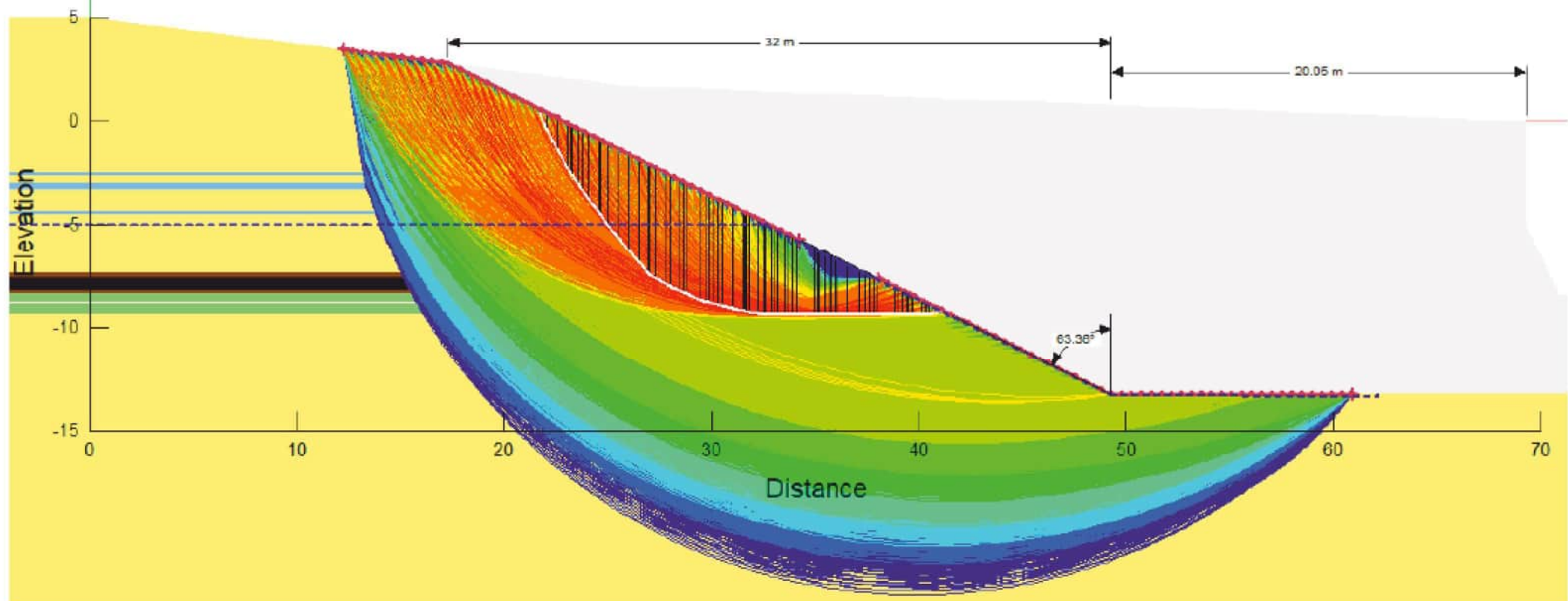
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GEOTECHNICAL INVESTIGATION
GREENVIEW INDUSTRIAL GATEWAY
3-14-67-5 W6M and 4-29-67-4 W6M near Braaten, Alberta

Detail 'B'
Borehole Locations ("The Pond Site")

Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Bentonite	16	0	22
	Coal	19	9.4	18.6
	Mudstone	21	9.4	18.6
	Residual Soils	18	0	25
	River sands	18	0	25
	Sandstone	23.5	21	35
	Siltstone	21	9.4	18.6

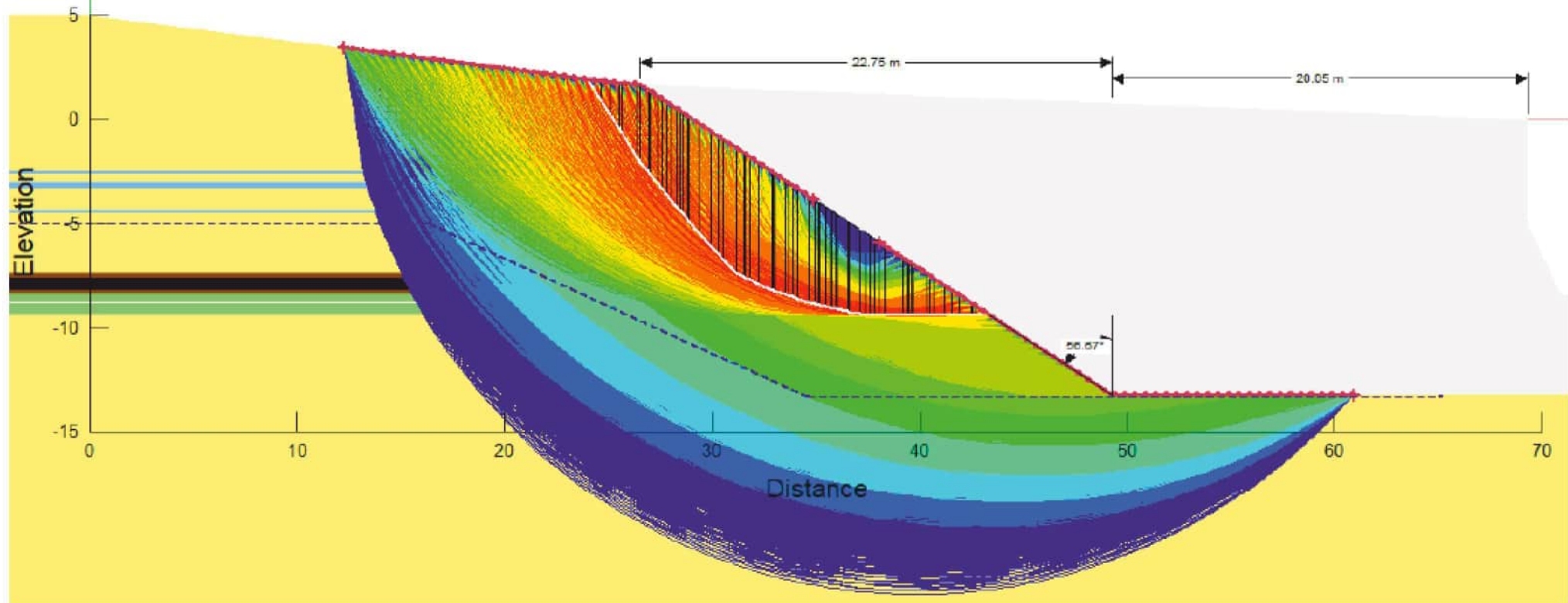
Factor of Safety
 1.350 - 1.400
 1.400 - 1.500
 1.500 - 1.600
 1.600 - 1.700
 1.700 - 1.800
 1.800 - 1.900
 1.900 - 2.000
 2.000 - 2.100
 2.100 - 2.200
 ≥ 2.200



Project:		Greenview Industrial Gateway	
Client:		Municipal District of Greenview	
Analysis Description:		Excavation Stability Section – Natural Groundwater Conditions	
Drawn:	CH	Approver:	KJ
Scale:	1:334	Job:	5946-001-00
		Title:	Figure 5

Color	Name	Unit Weight (kN/m ³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Bentonite	16	0	22
	Coal	19	9.4	18.6
	Mudstone	21	9.4	18.6
	Residual Soils	18	0	25
	River sands	18	0	25
	Sandstone	23.5	21	35
	Siltstone	21	9.4	18.6

Factor of Safety
 1.372-1.472
 1.472-1.572
 1.572-1.672
 1.672-1.772
 1.772-1.872
 1.872-1.972
 1.972-2.072
 2.072-2.172
 2.172-2.272
 ≥ 2.272



Project:		Greenview Industrial Gateway	
Client:		Municipal District of Greenview	
Analysis Description:		Excavation Stability Section – Dewatered Groundwater Conditions	
Drawn:	CH	Approver:	KJ
Job:		5946-001-00	
Scale:	1:334	Title:	Figure 6

Appendix B

Rock Core Logs and Borehole Logs



eNGLOBE

DRILLING DATE: 14/01/2023 - 14/01/2023

COORDINATES: N: 8076278 m E: 398514 m

ORIGINATED BY: CH/KO

PROJECT: 5946-001-00

DATUM:

COMPILED BY: CH

ELEVATION: 578.8 m

DRILL RIG:

CHECKED BY: KJ

INCLINATION: 0° AZIMUTH: 0°

CONTRACTOR: Vals Drilling

REPORT DATE: 30/03/2023

DEPTH - feet	DEPTH - meters	DEPTH - m	DESCRIPTION	SYMBOL	RUN NO.	TOTAL CORE %	SOLID CORE %	RQD %	DISCONTINUITY DATA			STRENGTH				WEATHERING INDEX				NOTES
									FRACT INDEX PER 1.0m	ANGLE wrt CORE AXIS	Jrc									
		0.00	Natural ground surface						0											
			Between 0.0 and 1.52 m: Overburden, loosely cemented residual soil. Not recovered.																	
1																				
5		1.52	SANDSTONE highly weathered, medium to coarse grained, weak, brown to grey, oxidation throughout.																	
2			At 2.15 m: Mudstone layer, approximately 5 mm		01	71	89	81	3											
		2.70	SANDSTONE fresh to slightly weathered, fine to coarse grained, 1mm scale bedding, weak, grey.																	
10		2.95	Residual Soil - SAND																	
		3.04	Between 3.04 and 3.29 m: No recovery																	
		3.29	SANDSTONE slightly weathered to fresh, medium to coarse grained, weak, grey																	
4					02	88	68	71	2											
		4.41	Residual Soil - SAND																	
15		4.56	SANDSTONE fresh, 1mm to 1cm scale beds, fine to coarse grained, weak, grey																	
5					03	97	91	86	4											
20		6.08	Between 6.08 and 7.19 m: No recovery																	
7					04	41	16	15	0											
		7.19	Residual Soil - SAND																	
25		7.23	SANDSTONE slight to moderately weathered, medium to coarse grained, very weak, grey																	
		7.40																		
		7.60	MUDSTONE moderately weathered, 1mm scale bedding, fine grained, extremely weak, dark brown																	
8		7.72	Between 7.6 and 7.72 m: No recovery																	
		8.27	MUDSTONE moderately weathered, 1 cm scale bedding, fine grained, extremely weak, dark brown, clay seems present, highly fractured by drilling		05	97	61	62	7											
9																				
30		9.31	SANDSTONE fresh, 1 cm scale beds, medium to fine grained, weak, grey																	
		9.43	At 9.09 m: 3 cm of Residual Soil - SAND																	
			Between 9.12 and 9.42 m: 1mm scale		06	100	99	96	3											

DRILLING DATE: 14/01/2023 - 14/01/2023

COORDINATES: N: 8076278 m E: 398514 m

ORIGINATED BY: CH/KO

PROJECT: 5946-001-00

DATUM:

COMPILED BY: CH

ELEVATION: 578.8 m

DRILL RIG:

CHECKED BY: KJ

INCLINATION: 0° AZIMUTH: 0°

CONTRACTOR: Vals Drilling

REPORT DATE: 30/03/2023

DEPTH - feet	DEPTH - meters	DEPTH - m	DESCRIPTION	SYMBOL	RUN NO.	TOTAL CORE %	SOLID CORE %	RQD %	DISCONTINUITY DATA				STRENGTH					WEATHERING INDEX						NOTES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
									FRACT INDEX PER 1.0m	ANGLE wrt CORE AXIS	Jrc	UCT (MPa)	R0	R1	R2	R3	R4	R5	R6	W1	W2	W3	W4		W5	W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
			dark brown laminations present, interbedded sand and siltstone						21 17 13 9																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											</

DRILLING DATE: 14/01/2023 - 15/01/2023

COORDINATES: N: 6076280 m E: 398488 m

ORIGINATED BY: CH/KO

PROJECT: 5946-001-00

DATUM:

COMPILED BY: CH

ELEVATION: 580.2 m

DRILL RIG:

CHECKED BY: KJ

INCLINATION: 0° AZIMUTH: 0°

CONTRACTOR: Vals Drilling

REPORT DATE: 11/04/2023

DEPTH - feet	DEPTH - meters	DEPTH - m	DESCRIPTION	SYMBOL	RUN NO.	TOTAL CORE %	SOLID CORE %	RQD %	DISCONTINUITY DATA			STRENGTH				WEATHERING INDEX					NOTES
									FRACT INDEX PER 1.0m	ANGLE wrt CORE AXIS	Jrc										
		0.00	Natural ground surface						0												
			GRAVEL - FILL gravel and sand, coarse to medium grained																		
1																					
5		1.37	Between 1.37 and 1.52 m: No recovery						6												
2		1.52	SANDSTONE slightly weathered, medium to coarse grained, medium strong to weak, grey																		
			Between 1.78 and 1.87 m: Residual Soil - SAND																		
			Below 2.28 m: Becomes highly weathered, extremely weak, brown. Oxides present throughout																		
10		3	Between 2.28 and 2.34 m: Residual soil																		
		3.04	Between 3.04 and 3.24 m: No recovery																		
		3.24	SANDSTONE highly weathered, medium to coarse grained, extremely weak, brown																		
			AT 3.72 m: Soft, permeable																		
15		3.89	SANDSTONE slightly weathered, medium to coarse grained, very weak, grey						3												
20			Between 5.2 and 5.45 m: Sample #1																		
			Below 5.45 m: Becomes more cemented, still weak to very weak																		
25																					
		7.40	MUDSTONE slightly weathered, fine grained, extremely weak, dark brown						2												
8		7.64	COAL highly fractured, blocky, black																		
		8.15	MUDSTONE slightly weathered, fine grained, extremely weak, dark brown																		
		8.42	AT 8.42 m: Coal seam, approximately 5 mm						0												
			SILTSTONE fresh, fine grained, weak, greyish brown																		
30			Between 8.67 and 8.83 m: Becomes extremely weak to soft, coal present																		
		9.40	At 8.83 m: BENTONITE 2.5 cm thick, base dips at 73 degrees						2												
			SANDSTONE fresh, 1-5 mm scale brown																		

DRILLING DATE:14/01/2023 - 15/01/2023

COORDINATES: N: 6076280 m E: 398488 m

ORIGINATED BY: CH/KO

PROJECT: 5946-001-00

DATUM:

COMPILED BY: CH

ELEVATION: 580.2 m

DRILL RIG:

CHECKED BY: KJ

INCLINATION: 0° AZIMUTH: 0°

CONTRACTOR: Vals Drilling

REPORT DATE: 11/04/2023

[illegible]

Greenview Industrial Gateway

N 6076257 E 398450.6

BOREHOLE NO: **BH23-03**

DATES: 01/15/2023 - 01/15/2023

ZONE: 11 N

PROJECT NO: 5946-001-00

ELEVATION: 593.81 m

SAMPLE STATE  Remoulded Intact Core Lost (no recovery)

SAMPLE TYPE AS: Auger Sample

ST: Shelby Tube

SS: Split Spoon

MA: Manual Sample

DC: Diamond Rock Core

DEPTH (m)	WET UNIT WEIGHT (kN/m ³) ▲					COMPRESSION STRENGTH (kPa)					SAMPLE STATE	SAMPLE TYPE and NUMBER	SPT (N)	USC	GROUNDWATER CONDITIONS	ELEVATION - m DEPTH - m	SOIL SYMBOL	SOIL DESCRIPTION	ELEVATION (m)
	17	18	19	20	21	100	200	300	400										
	WATER CONTENT and LIMITS (%)					STANDARD PENETRATION (N) ■													
	PLASTIC M.C. LIQUID																		
	20	40	60	80		20	40	60	80	100									
1																593.8	• •	GRAVEL and SAND - FILL well graded, medium to coarse grained, dry, compact, brown, trace rootlets	593
2																593.3		CLAY - FILL sandy, some silt, trace to some gravel, moist, firm, medium plastic, brown, frozen	592
3																591.8		CLAY some sand, some silt, trace gravel, moist, firm medium plastic, grey to black	591
4																589.9		At 3.0 m: Rocks in SPT spoon	590
5																4.0		END OF BOREHOLE - 3.95 m	589
6																		No standpipe installed, borehole terminated on bedrock contact	588
7																			587
8																			586
9																			585
10																			584
11																			583
12																			582
13																			581
14																			580
15																			579
16																			578
17																			577
																			576

REMARK: Borehole completed to target depth (bedrock contact)

LOGGED BY: CH/KO

COMPILED BY: CH

REVIEWED BY: KJ

REPORT DATE: 03/30/2023



Greenview Industrial Gateway

N 6073095 E 393294.1

BOREHOLE NO: **BH23-04**

DATES: 01/16/2023 - 01/16/2023

ZONE: 11 N

PROJECT NO: 5946-001-00

ELEVATION: 725.46 m

SAMPLE STATE  Remoulded Intact Core Lost (no recovery)

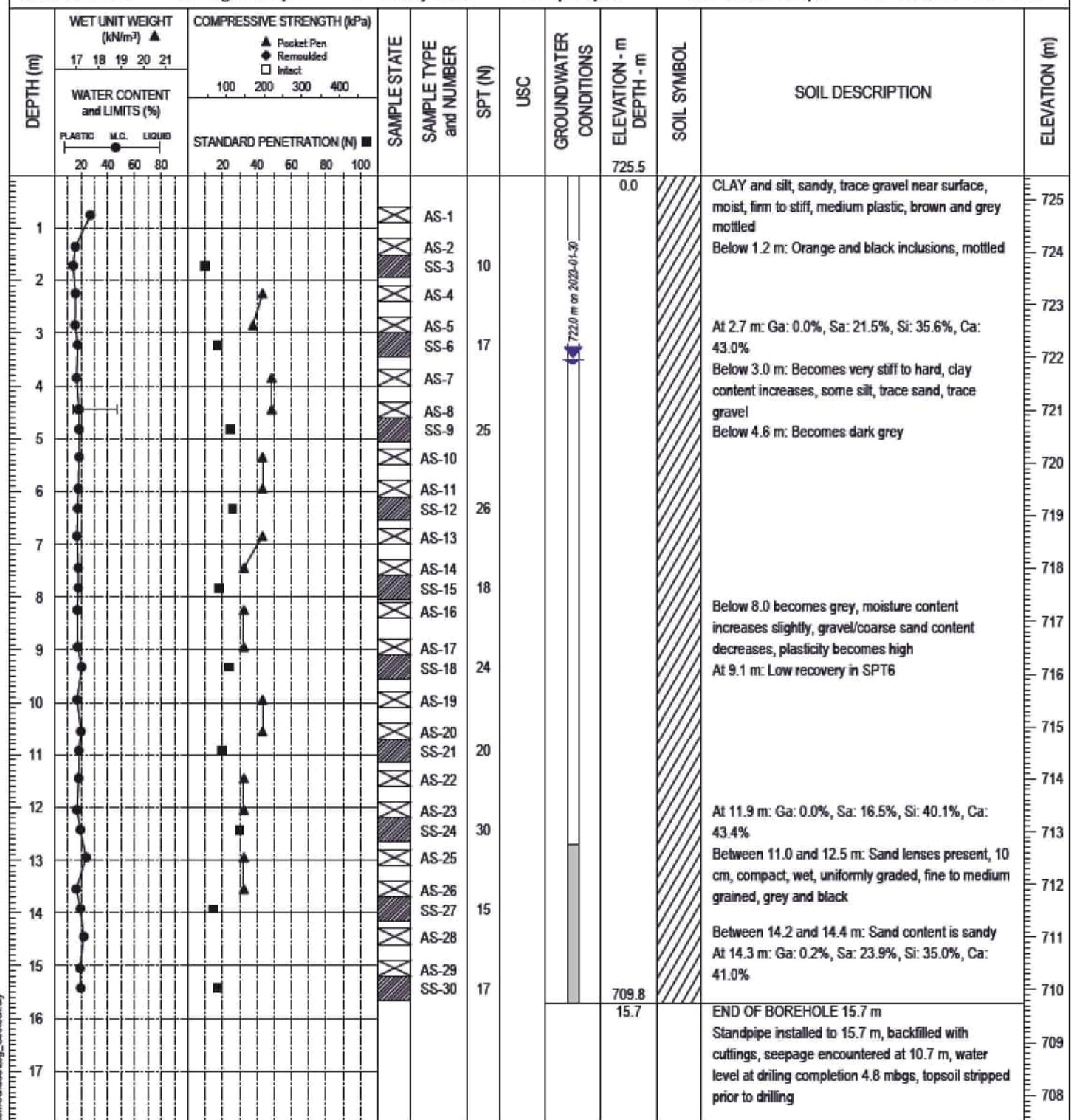
SAMPLE TYPE AS: Auger Sample

ST: Shelby Tube

SS: Split Spoon

MA: Manual Sample

DC: Diamond Rock Core



REMARK: Borehole completed to target depth

LOGGED BY: CH/KO

COMPILED BY: CH

REVIEWED BY: KJ

REPORT DATE: 03/30/2023



Greenview Industrial Gateway

N 6073374 E 394148.5

BOREHOLE NO: **BH23-05**

DATES: 01/16/2023 - 01/16/2023

ZONE: 11 N

PROJECT NO: 5946-001-00

ELEVATION: 708.37 m

SAMPLE STATE  Remoulded Intact Core Lost (no recovery)

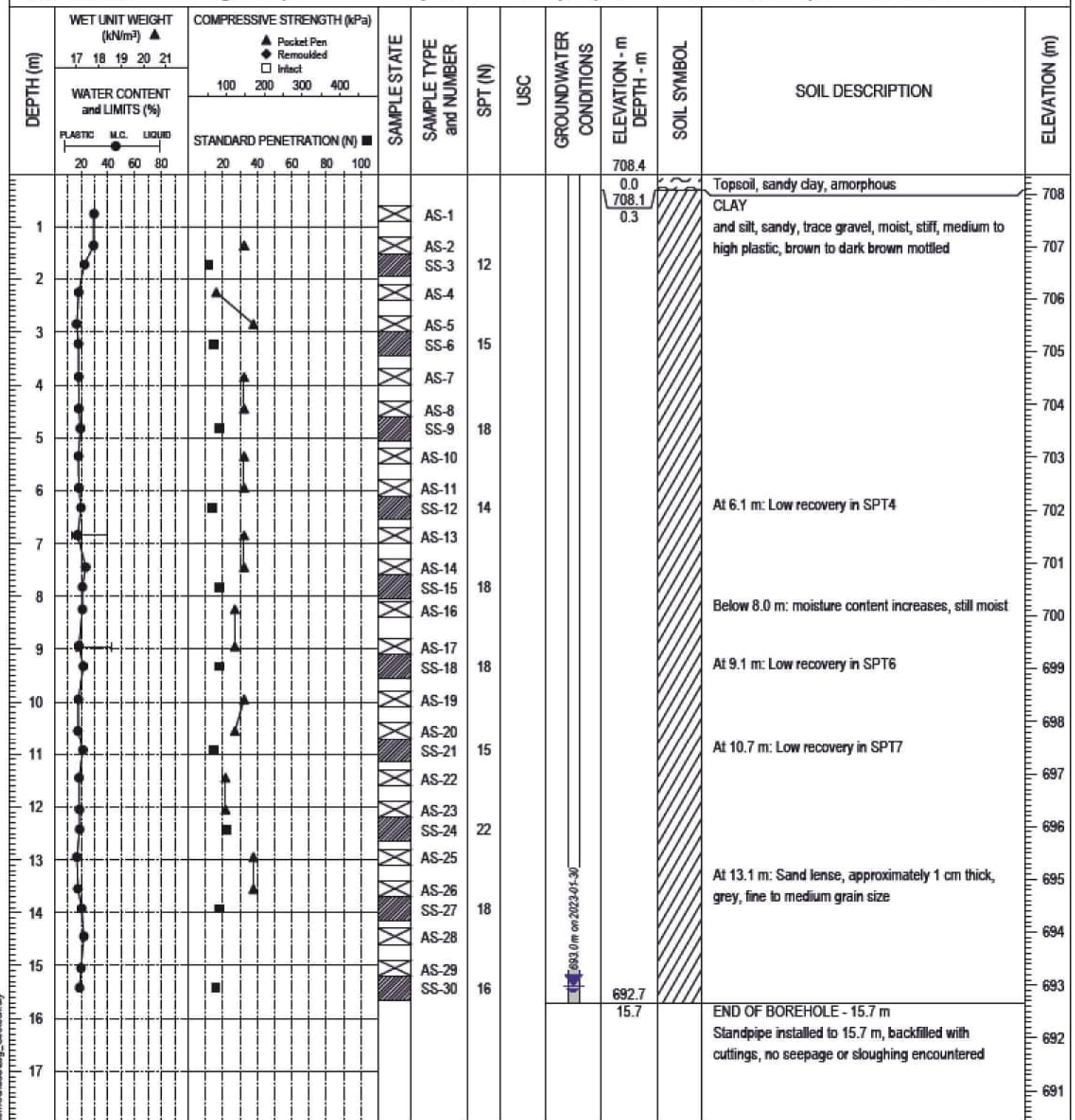
SAMPLE TYPE AS: Auger Sample

ST: Shelby Tube

SS: Split Spoon

MA: Manual Sample

DC: Diamond Rock Core



REMARK: Borehole completed to target depth

LOGGED BY: CH/KO

COMPILED BY: CH

REVIEWED BY: KJ

REPORT DATE: 03/30/2023



Appendix C

Geotechnical Laboratory Results



eNGLOBE



Moisture Content As Per ASTM D2217

Test Hole No.:	BH 23-04	BH 23-04	BH 23-04	BH 23-04	BH 23-04	BH 23-04	BH 23-04	BH 23-04	BH 23-04	BH 23-04
Sample No.:	AU1	AU2	SPT1	AU3	AU4	SPT2	AU5	AU6	SPT3	AU7
Depth:	0.6m	1.2m	1.5m	2.1m	2.7m	3.0m	3.7m	4.3m	4.6m	5.2m
Tare No.:	185	1022	308	275	8A	276	311	310	14A	251
Wet Sample + Tare (g):	241.8	305.2	294.3	281.1	409.3	271.3	256.5	412.3	198.3	223.4
Dry Sample + Tare (g):	191.2	264.3	258.5	243.4	354.7	231.6	220.6	354.2	168.1	189.1
Weight of Tare (g):	4.4	4.3	4.5	4.4	4.4	4.3	4.4	4.2	4.5	4.2
Weight of Water (g):	50.6	40.9	35.8	37.7	54.6	39.7	35.9	58.1	30.2	34.3
Weight of Dry Soil (g):	186.8	260.0	254.0	239.0	350.3	227.3	216.2	350.0	163.6	184.9
Moisture Content (%):	27.1%	15.7%	14.1%	15.8%	15.6%	17.5%	16.6%	16.6%	18.5%	18.6%

Test Hole No.:	BH23-04	BH23-04	BH23-04	BH23-04	BH23-04	BH23-04	BH23-04	BH23-04	BH23-04	BH23-04
Sample No.:	AU8	SPT4	AU9	AU10	SPT5	AU11	AU12	SPT6	AU13	AU14
Depth:	5.8m	6.1m	6.7m	7.3m	7.6m	8.2m	8.8m	9.1m	9.8m	10.4m
Tare No.:	170	239	2048	278	6A	24	181	204	287	280
Wet Sample + Tare (g):	214.3	279.3	202.3	278.2	222.1	234.6	244.4	63.1	255.4	233.7
Dry Sample + Tare (g):	182.4	238.3	173.5	236.6	189.1	200.8	208.9	53.1	219.0	195.6
Weight of Tare (g):	4.2	4.2	4.3	4.4	4.6	4.5	4.4	4.3	4.4	4.4
Weight of Water (g):	31.9	41.0	28.8	41.6	33.0	33.8	35.5	10.0	36.4	38.1
Weight of Dry Soil (g):	178.2	234.1	169.2	232.2	184.5	196.3	204.5	48.8	214.6	191.2
Moisture Content (%):	17.9%	17.5%	17.0%	17.9%	17.9%	17.2%	17.4%	20.5%	17.0%	19.9%

Test Hole No.:	BH23-04	BH23-04	BH23-04	BH23-04	BH23-04	BH23-04	BH23-04	BH23-04	BH23-04	BH23-04
Sample No.:	SPT7	AU15	AU16	SPT8	AU17	AU18	SPT9	AU19	AU20	SPT10
Depth:	10.7m	11.5m	11.9m	12.2m	12.8m	13.1m	13.7m	14.3m	14.9m	15.2m
Tare No.:	226	10A	301	304	137	16A	147	7A	1008	201B
Wet Sample + Tare (g):	238.9	245.8	481.7	221.3	424.7	232.0	273.6	462.1	236.9	294.8
Dry Sample + Tare (g):	202.4	208.7	410.5	185.9	343.7	200.0	229.4	387.1	199.3	246.8
Weight of Tare (g):	4.4	4.4	4.4	4.5	4.4	4.5	4.4	4.5	4.4	4.5
Weight of Water (g):	36.5	37.1	71.2	35.4	81.0	32.0	44.2	75.0	37.6	48.0
Weight of Dry Soil (g):	198.0	204.3	406.1	181.4	339.3	195.5	225.0	382.6	194.9	242.3
Moisture Content (%):	18.4%	18.2%	17.5%	19.5%	23.9%	16.4%	19.6%	19.6%	19.3%	19.8%

Test Hole No.:										
Sample No.:										
Depth:										
Tare No.:										
Wet Sample + Tare (g):										
Dry Sample + Tare (g):										
Weight of Tare (g):										
Weight of Water (g):										
Weight of Dry Soil (g):										
Moisture Content (%):										

Comments: BH23-04 SPT6 contained low sample quantity, less than 100g



Reviewed By: Connor Carlson



Moisture Content As Per ASTM D2217

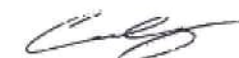
Test Hole No.:	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05
Sample No.:	AU1	AU2	SPT1	AU3	AU4	SPT2	AU5	AU6	SPT3	AU7
Depth:	0.6m	1.2m	1.5m	2.1m	2.7m	3.0m	3.7m	4.3m	4.6m	5.2m
Tare No.:	154	228	282	279	200A	1023	252	169	221	210K
Wet Sample + Tare (g):	230.3	276.9	258.1	281.1	232.8	284.0	267.7	233.4	305.1	267.6
Dry Sample + Tare (g):	178.6	215.1	211.4	238.5	200.1	241.4	227.1	198.0	256.0	218.9
Weight of Tare (g):	4.3	4.2	4.2	4.2	4.3	4.4	4.3	4.3	4.3	4.3
Weight of Water (g):	51.7	61.8	46.7	42.6	32.7	42.6	40.6	35.4	49.1	38.7
Weight of Dry Soil (g):	174.3	210.9	207.2	234.3	195.8	237.0	222.8	193.7	251.7	214.6
Moisture Content (%):	29.7%	29.3%	22.5%	18.2%	16.7%	18.0%	18.2%	18.3%	19.5%	18.0%

Test Hole No.:	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05
Sample No.:	AU8	SPT4	AU9	AU10	SPT5	AU11	AU12	SPT6	AU13	AU14
Depth:	5.8m	6.1m	6.7m	7.3m	7.6m	8.2m	8.8m	9.1m	9.8m	10.4m
Tare No.:	139	20	145	323	19A	12A	289	196	276	1004
Wet Sample + Tare (g):	257.3	112.3	400.2	240.6	227.0	410.0	288.0	39.1	323.9	235.2
Dry Sample + Tare (g):	218.0	94.3	341.6	195.8	188.3	341.3	244.0	32.9	275.1	200.5
Weight of Tare (g):	4.3	4.2	4.2	4.2	4.2	4.2	4.5	4.3	4.3	4.4
Weight of Water (g):	39.3	18.0	58.6	44.8	38.7	68.7	44.0	6.2	48.8	34.7
Weight of Dry Soil (g):	213.7	90.1	337.4	191.6	184.1	337.1	239.5	28.6	270.8	196.1
Moisture Content (%):	18.4%	20.0%	17.4%	23.4%	21.0%	20.4%	18.4%	21.7%	18.0%	17.7%

Test Hole No.:	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05	BH 23-05
Sample No.:	SPT7	AU15	AU16	SPT8	AU17	AU18	SPT9	AU19	AU20	SPT10
Depth:	10.7m	11.5m	11.9m	12.2m	12.8m	13.1m	13.7m	14.3m	14.9m	15.2m
Tare No.:	326	172	21	191	189	155	321	1013	182	186
Wet Sample + Tare (g):	104.7	316.9	248.8	286.8	281.3	244.9	242.0	195.7	200.2	267.8
Dry Sample + Tare (g):	87.0	267.8	210.1	241.9	241.3	208.7	201.7	161.1	167.6	225.9
Weight of Tare (g):	4.3	4.2	4.2	4.2	4.3	4.2	4.3	4.3	4.3	4.4
Weight of Water (g):	17.7	49.1	38.7	44.9	40.0	36.2	40.3	34.6	32.6	41.9
Weight of Dry Soil (g):	82.7	263.6	205.9	237.7	237.0	204.5	197.4	156.8	163.3	221.5
Moisture Content (%):	21.4%	18.6%	18.8%	18.9%	16.9%	17.7%	20.4%	22.1%	20.0%	18.9%

Test Hole No.:										
Sample No.:										
Depth:										
Tare No.:										
Wet Sample + Tare (g):										
Dry Sample + Tare (g):										
Weight of Tare (g):										
Weight of Water (g):										
Weight of Dry Soil (g):										
Moisture Content (%):										

Comments: BH23-05 SPT6 contained low sample quantity, less than 100g



Reviewed By: Connor Carlson



Unconfined Compressive Strength of Cohesive Soil As Per ASTM D2186/D2186M

UCS Specimen	
Initial Diameter of Sample (mm):	59.5
Initial Height of Sample (mm):	135.4
Initial Area of Sample (mm ²):	2782
Initial Vol of Sample (mm ³):	376632
Length to Diameter Ratio:	2.28
Strain Rate (mm/80s):	0.877

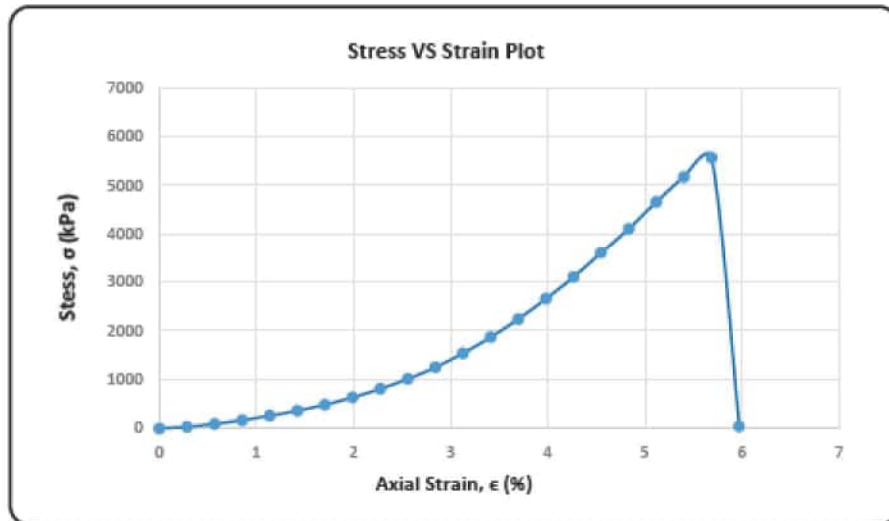
Moisture Content	
Tare No.:	2
Wet Weight + Tare (g):	854.7
Dry Weight + Tare (g):	789.4
Weight of Tare (g):	218.9
Moisture Content of Sample (%):	11.4%

Sample Density	
Wet Density of Sample (kg/m ³):	2251
Dry Density of Sample (kg/m ³):	2020
Wet Unit Wt. of Sample (kN/m ³):	N/A
Dry Unit Wt. of Sample (kN/m ³):	N/A

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
15	0.09	0.169
30	0.28	0.339
45	0.47	0.508
60	0.73	0.677
75	1.02	0.846
90	1.37	1.016
105	1.80	1.185
120	2.30	1.354
135	2.89	1.523
150	3.58	1.693

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
165	4.42	1.862
180	5.38	2.031
195	6.47	2.200
210	7.69	2.370
225	9.01	2.539
240	10.50	2.708
255	12.00	2.877
270	13.65	3.047
285	15.20	3.216
300	16.40	3.385

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
315	0.13	3.554



Peak Unconfined Compressive Strength: 5560.4 kPa

Sample Description: Gray Sandstone

Comments: _____

Connor C

Reviewed By: Connor C



Photographs



Fig No. 1: Sample Prior to Trim



Fig No. 2: Sample Before Break



Fig No. 3: Sample After Break



Fig No. 4: Sample Split

Comments: _____

Connor Carlson

Reviewed By: Connor Carlson



Unconfined Compressive Strength of Cohesive Soil As Per ASTM D2186/D2186M

UCS Specimen	
Initial Diameter of Sample (mm):	59
Initial Height of Sample (mm):	123.8
Initial Area of Sample (mm ²):	2735
Initial Vol of Sample (mm ³):	338602
Length to Diameter Ratio:	2.10
Strain Rate (mm/80s):	0.619

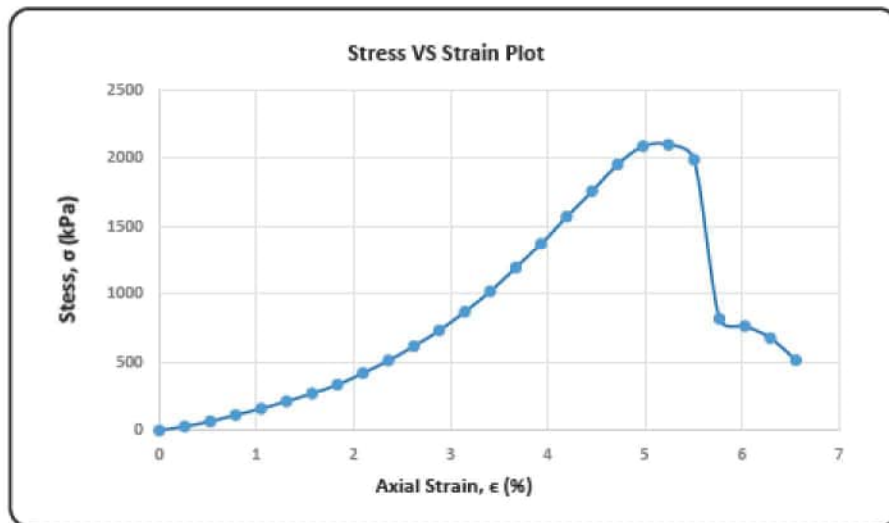
Moisture Content	
Tare No.:	1
Wet Weight + Tare (g):	883
Dry Weight + Tare (g):	811.6
Weight of Tare (g):	218.2
Moisture Content of Sample (%):	12.0%

Sample Density	
Wet Density of Sample (kg/m ³):	2224
Dry Density of Sample (kg/m ³):	1985
Wet Unit Wt. of Sample (kN/m ³):	N/A
Dry Unit Wt. of Sample (kN/m ³):	N/A

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
15	0.08	0.155
30	0.18	0.310
45	0.31	0.464
60	0.44	0.619
75	0.59	0.774
90	0.75	0.929
105	0.93	1.083
120	1.17	1.238
135	1.43	1.393
150	1.73	1.548

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
165	2.06	1.702
180	2.45	1.857
195	2.88	2.012
210	3.38	2.167
225	3.90	2.321
240	4.48	2.476
255	5.03	2.631
270	5.60	2.786
285	6.00	2.940
300	6.05	3.095

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
315	5.75	3.250
330	2.37	3.405
345	2.22	3.559
360	1.97	3.714
375	1.50	3.869



Peak Unconfined Compressive Strength: 2096 kPa

Sample Description: Gray Sandstone, Trace Silt

Comments: Pre-existing crack along upper face (vertical). Cracked along diagonal of pre-existing crack and appearance of vertical cracking.

Reviewed By: Connor C



Photographs



Fig No. 1: Sample Prior to Trim



Fig No. 2: Sample Before Break

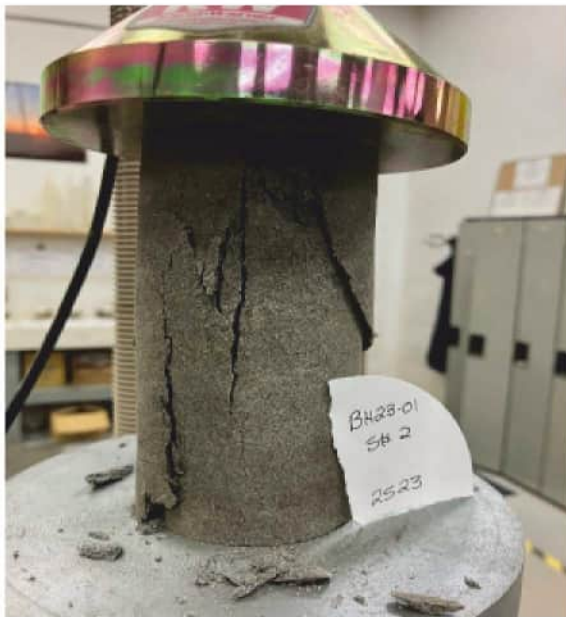


Fig No. 3: Sample After Break



Fig No. 4: Sample Split

Comments: _____

Reviewed By: Connor Carlson



Unconfined Compressive Strength of Cohesive Soil As Per ASTM D2186/D2186M

UCS Specimen	
Initial Diameter of Sample (mm):	58.5
Initial Height of Sample (mm):	135.1
Initial Area of Sample (mm ²):	2689
Initial Vol of Sample (mm ³):	363272
Length to Diameter Ratio:	2.31
Strain Rate (mm/80s):	0.8755

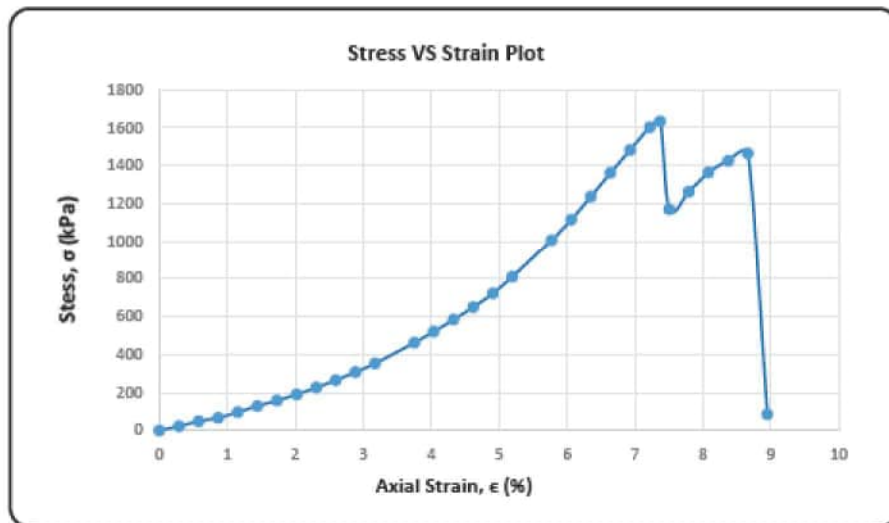
Moisture Content	
Tare No.:	8
Wet Weight + Tare (g):	829.1
Dry Weight + Tare (g):	760.9
Weight of Tare (g):	216.3
Moisture Content of Sample (%):	12.5%

Sample Density	
Wet Density of Sample (kg/m ³):	2210
Dry Density of Sample (kg/m ³):	1964
Wet Unit Wt. of Sample (kN/m ³):	N/A
Dry Unit Wt. of Sample (kN/m ³):	N/A

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
15	0.06	0.169
30	0.13	0.338
45	0.18	0.507
60	0.26	0.676
75	0.35	0.844
90	0.43	1.013
105	0.52	1.182
120	0.62	1.351
135	0.73	1.520
150	0.85	1.689

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
165	0.98	1.858
195	1.29	2.195
210	1.48	2.384
225	1.64	2.533
240	1.83	2.702
255	2.04	2.871
270	2.30	3.040
300	2.87	3.378
315	3.19	3.546
330	3.55	3.715

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
345	3.92	3.884
360	4.28	4.053
375	4.64	4.222
383	4.74	4.312
390	3.40	4.391
405	3.68	4.560
420	3.99	4.729
435	4.18	4.897
450	4.31	5.066
465	0.25	5.235



Peak Unconfined Compressive Strength: 1632.9 kPa

Sample Description: Gray Sandstone, Trace Silt

Comments: _____

Reviewed By: Connor Carlson



Photographs



Fig No. 1: Sample Prior to Trim



Fig No. 2: Sample Before Break



Fig No. 3: Sample After Break



Fig No. 4: Sample Split

Comments: _____

Connor Carlson

Reviewed By: Connor Carlson



Unconfined Compressive Strength of Cohesive Soil As Per ASTM D2186/D2186M

UCS Specimen	
Initial Diameter of Sample (mm):	58
Initial Height of Sample (mm):	118.5
Initial Area of Sample (mm ²):	2643
Initial Vol of Sample (mm ³):	307926
Length to Diameter Ratio:	2.01
Strain Rate (mm/80s):	0.5825

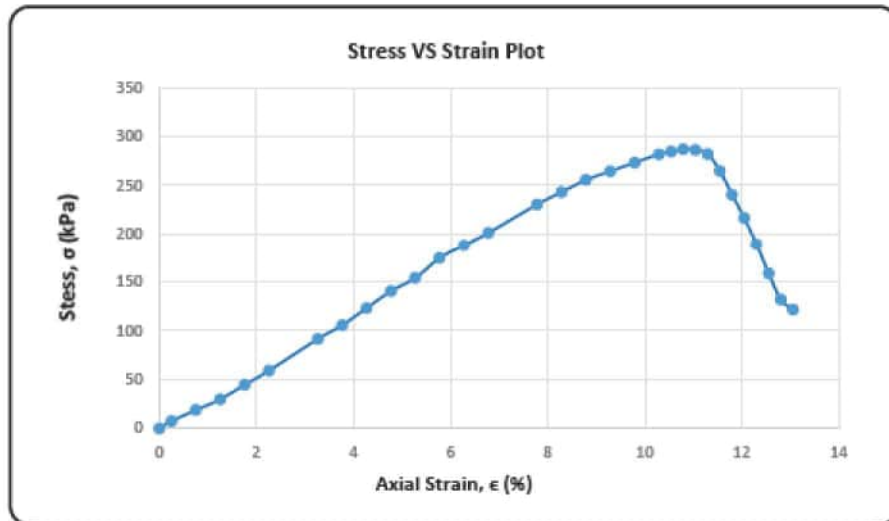
Moisture Content	
Tare No.:	4
Wet Weight + Tare (g):	802.4
Dry Weight + Tare (g):	736.3
Weight of Tare (g):	218.4
Moisture Content of Sample (%):	12.8%

Sample Density	
Wet Density of Sample (kg/m ³):	2021
Dry Density of Sample (kg/m ³):	1792
Wet Unit Wt. of Sample (kN/m ³):	N/A
Dry Unit Wt. of Sample (kN/m ³):	N/A

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
15	0.02	0.146
45	0.05	0.437
75	0.08	0.728
105	0.12	1.019
135	0.16	1.311
195	0.25	1.893
225	0.29	2.184
255	0.34	2.476
285	0.39	2.767
315	0.43	3.058

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
345	0.49	3.349
375	0.53	3.641
405	0.57	3.932
465	0.66	4.514
495	0.70	4.806
525	0.74	5.097
555	0.77	5.388
585	0.80	5.679
615	0.83	5.971
630	0.84	6.116

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
645	0.85	6.262
660	0.85	6.408
675	0.84	6.553
690	0.79	6.699
705	0.72	6.844
720	0.65	6.990
735	0.57	7.136
750	0.48	7.281
765	0.40	7.427
780	0.37	7.573



Peak Unconfined Compressive Strength: 286.9 kPa

Sample Description: Gray Sandstone, Trace Clay, Trace Silt

Comments: Pre-existing crack through center of sample prior to compression.

Reviewed By: Connor Carlson



Photographs



Fig No. 1: Sample Prior to Trim

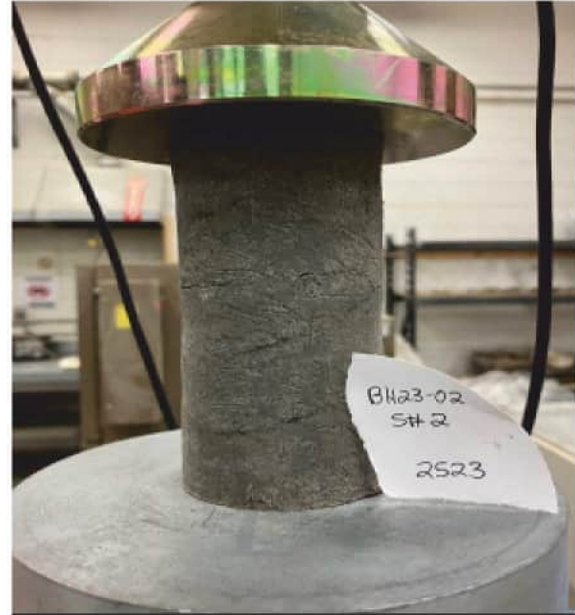


Fig No. 2: Sample Before Break with Crack



Fig No. 3: Sample After Break



Fig No. 4: Sample Split

Comments: _____

Connor Carlson

Reviewed By: Connor Carlson



Unconfined Compressive Strength of Cohesive Soil As Per ASTM D2186/D2186M

UCS Specimen	
Initial Diameter of Sample (mm):	58.5
Initial Height of Sample (mm):	124.2
Initial Area of Sample (mm ²):	2689
Initial Vol of Sample (mm ³):	333983
Length to Diameter Ratio:	2.12
Strain Rate (mm/80s):	0.621

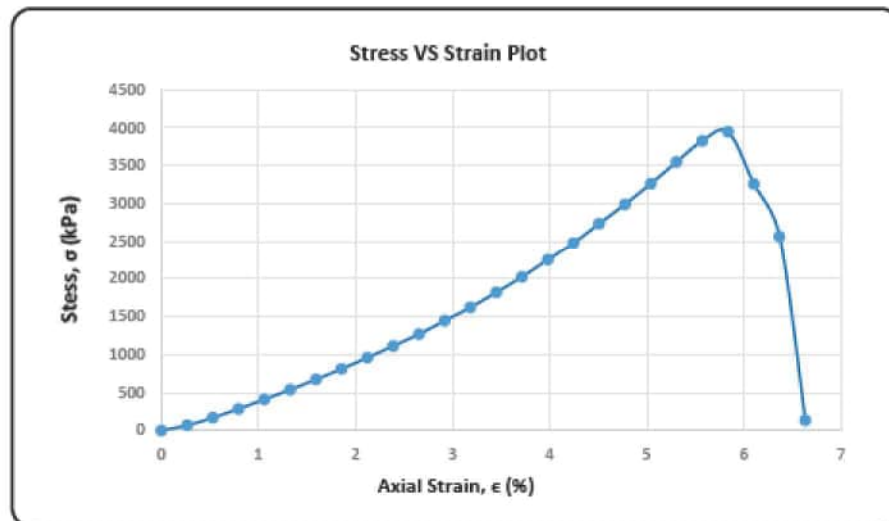
Moisture Content	
Tare No.:	2A
Wet Weight + Tare (g):	883.2
Dry Weight + Tare (g):	819.3
Weight of Tare (g):	218.9
Moisture Content of Sample (%):	10.6%

Sample Density	
Wet Density of Sample (kg/m ³):	2245
Dry Density of Sample (kg/m ³):	2029
Wet Unit Wt. of Sample (kN/m ³):	N/A
Dry Unit Wt. of Sample (kN/m ³):	N/A

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
15	0.18	0.155
30	0.45	0.311
45	0.77	0.466
60	1.11	0.621
75	1.46	0.776
90	1.83	0.932
105	2.22	1.087
120	2.63	1.242
135	3.06	1.397
150	3.50	1.553

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
165	4.00	1.708
180	4.50	1.863
195	5.06	2.018
210	5.65	2.174
225	6.30	2.329
240	6.95	2.484
255	7.70	2.639
270	8.44	2.795
285	9.23	2.950
300	10.07	3.105

Test Data		
Elapsed Time (Sec)	Load Dial (kN)	Strain Dial (mm)
315	10.89	3.260
330	11.26	3.416
345	9.33	3.571
360	7.36	3.726
375	0.38	3.881



Peak Unconfined Compressive Strength: 3943.1 kPa

Sample Description: Gray Sandstone Layered Siltstone

Comments: _____

Reviewed By: Connor Carlson



Photographs



Fig No. 1: Sample Prior to Trim



Fig No. 2: Sample Before Break

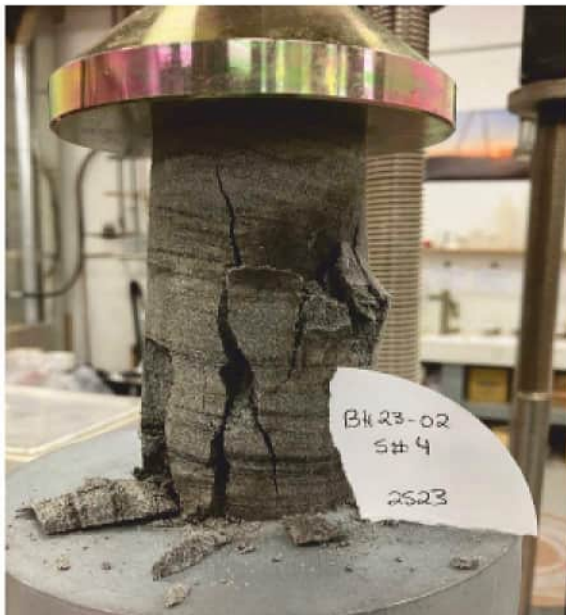


Fig No. 3: Sample After Break



Fig No. 4: Sample Split

Comments: _____

Reviewed By: Connor Carlson



Hydrometer Particle-Size Analysis of Soils As Per ASTM D422

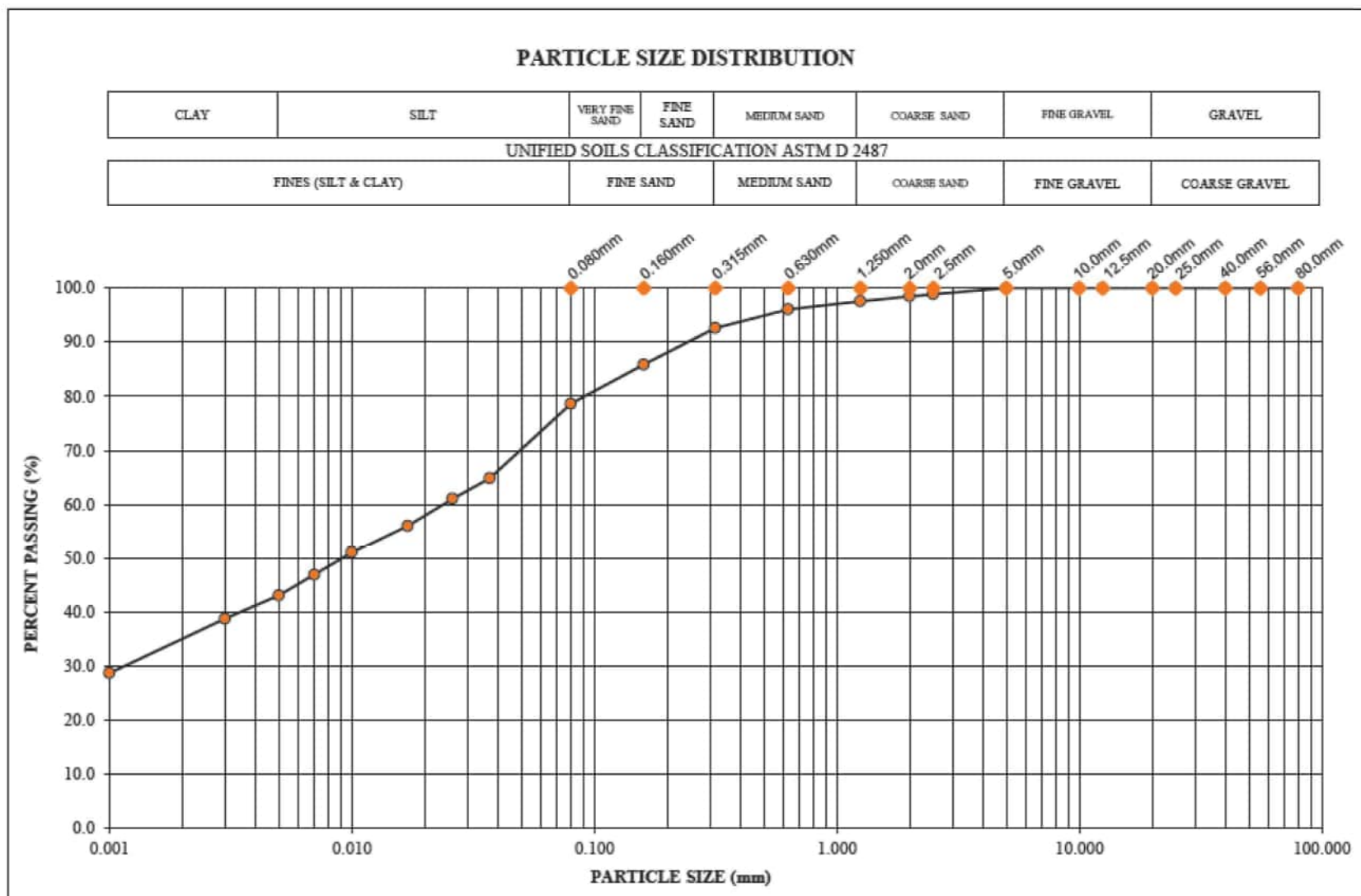
Sample Information	
BH No.:	23-04
Sample No.:	AU4
Depth (m/ft):	2.7m
In-Situ Moisture Cont. (%):	15.6%
Sand & Gravel Shape:	Angular
Dispersion Device:	30oz Mlt Cup
Length of Dispersion (min):	1 min

Grain Size Proportions (%)	
% Gravel (> 5.0mm):	0.0%
% Sand (80µm to 5.0mm):	21.4%
% Silt (5µm to 80µm):	35.6%
% Clay (< 5µm):	43.0%

Grain Size Analysis (Gravel & Sand)	
Sieve Size (µm)	Passing (%)
40 000	100.0%
25 000	100.0%
20 000	100.0%
12 500	100.0%
10 000	100.0%
5 000	100.0%
2 500	98.9%
2 000	98.5%
1 250	97.5%
630	96.1%
315	92.6%
160	85.8%
80	78.6%

Hydrometer Analysis (Silt & Clay)	
Sieve Size (µm)	Passing (%)
37	64.8%
26	61.1%
17	56.0%
10	51.0%
7	46.9%
5	43.0%
3	38.7%
1	28.7%

Comments





Hydrometer Particle-Size Analysis of Soils As Per ASTM D422

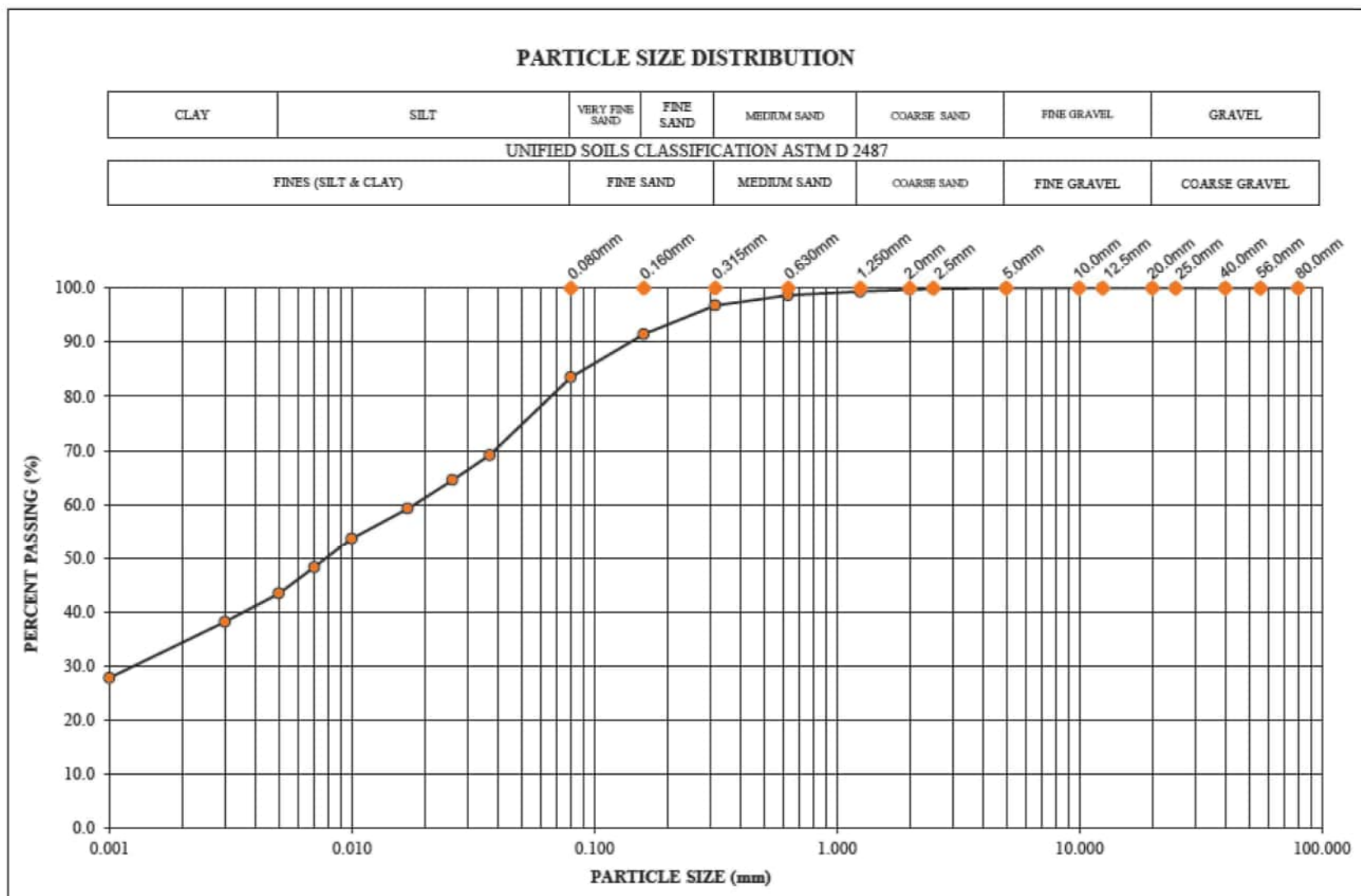
Sample Information	
BH No.:	23-04
Sample No.:	AU16
Depth (m/ft):	11.9m
In-Situ Moisture Cont. (%):	17.5%
Sand & Gravel Shape:	Angular
Dispersion Device:	30oz Mlt Cup
Length of Dispersion (min):	1 min

Grain Size Proportions (%)	
% Gravel (> 5.0mm):	0.0%
% Sand (80µm to 5.0mm):	16.5%
% Silt (5µm to 80µm):	40.1%
% Clay (< 5µm):	43.4%

Grain Size Analysis (Gravel & Sand)	
Sieve Size (µm)	Passing (%)
40 000	100.0%
25 000	100.0%
20 000	100.0%
12 500	100.0%
10 000	100.0%
5 000	100.0%
2 500	99.8%
2 000	99.7%
1 250	99.4%
630	98.7%
315	96.8%
160	91.5%
80	83.5%

Hydrometer Analysis (Silt & Clay)	
Sieve Size (µm)	Passing (%)
37	69.1%
26	64.5%
17	59.2%
10	53.6%
7	48.2%
5	43.4%
3	38.1%
1	27.8%

Comments



Connor Carlson

Reviewed By: Connor Carlson



Hydrometer Particle-Size Analysis of Soils As Per ASTM D422

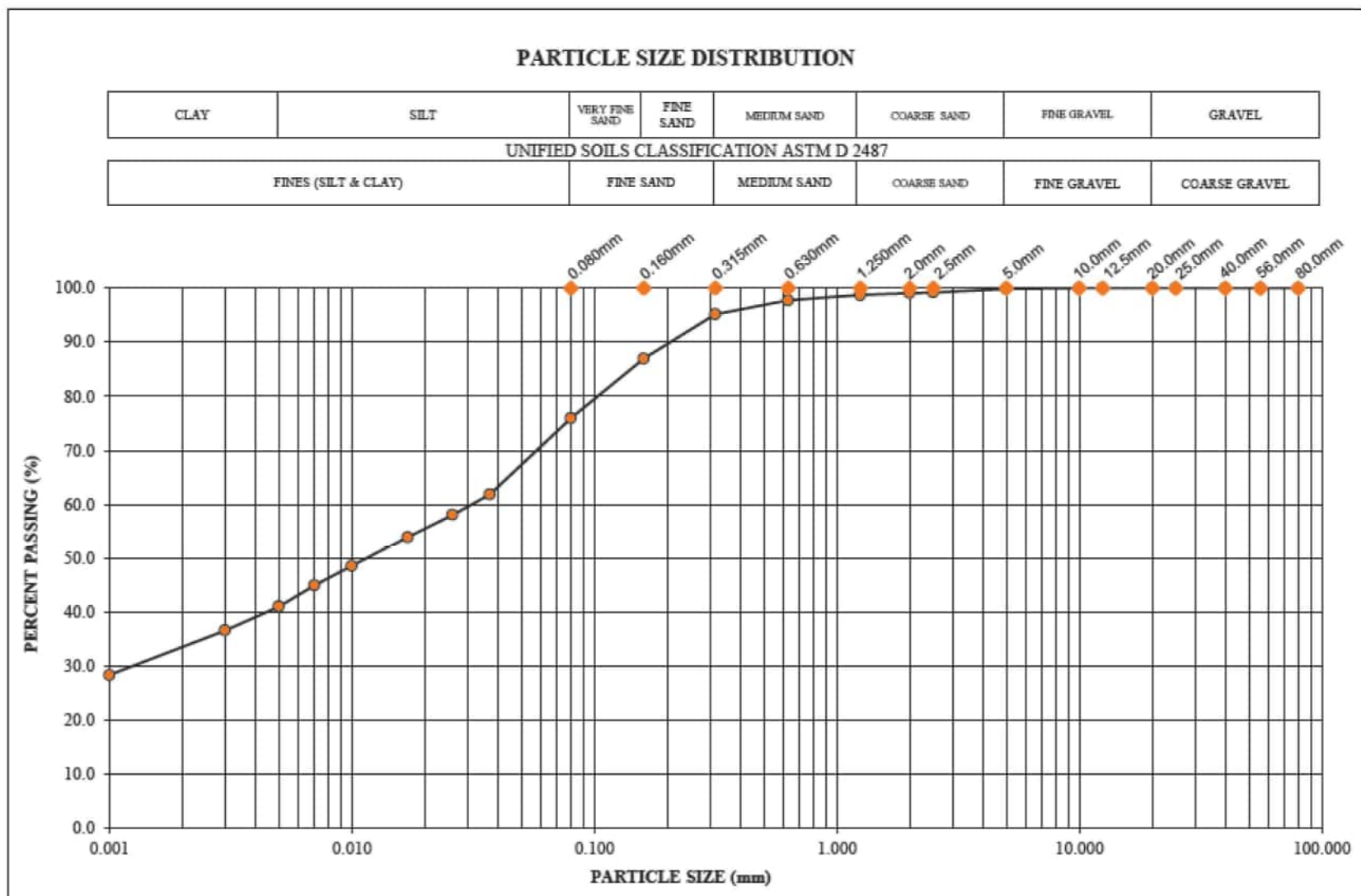
Sample Information	
BH No.:	23-04
Sample No.:	AU19
Depth (m/ft):	14.3m
In-Situ Moisture Cont. (%):	19.6%
Sand & Gravel Shape:	Angular
Dispersion Device:	30oz Mlt Cup
Length of Dispersion (min):	1 min

Grain Size Proportions (%)	
% Gravel (> 5.0mm):	0.2%
% Sand (80µm to 5.0mm):	23.9%
% Silt (5µm to 80µm):	35.0%
% Clay (< 5µm):	41.0%

Grain Size Analysis (Gravel & Sand)	
Sieve Size (µm)	Passing (%)
40 000	100.0%
25 000	100.0%
20 000	100.0%
12 500	100.0%
10 000	100.0%
5 000	99.8%
2 500	99.2%
2 000	99.0%
1 250	98.7%
630	97.7%
315	95.1%
160	87.0%
80	76.0%

Hydrometer Analysis (Silt & Clay)	
Sieve Size (µm)	Passing (%)
37	61.8%
26	58.1%
17	53.9%
10	48.5%
7	44.9%
5	41.0%
3	36.6%
1	28.3%

Comments



Reviewed By: Connor Carlson



Atterberg Limits As Per ASTM D4318

Liquid Limit - Hand Operated Method

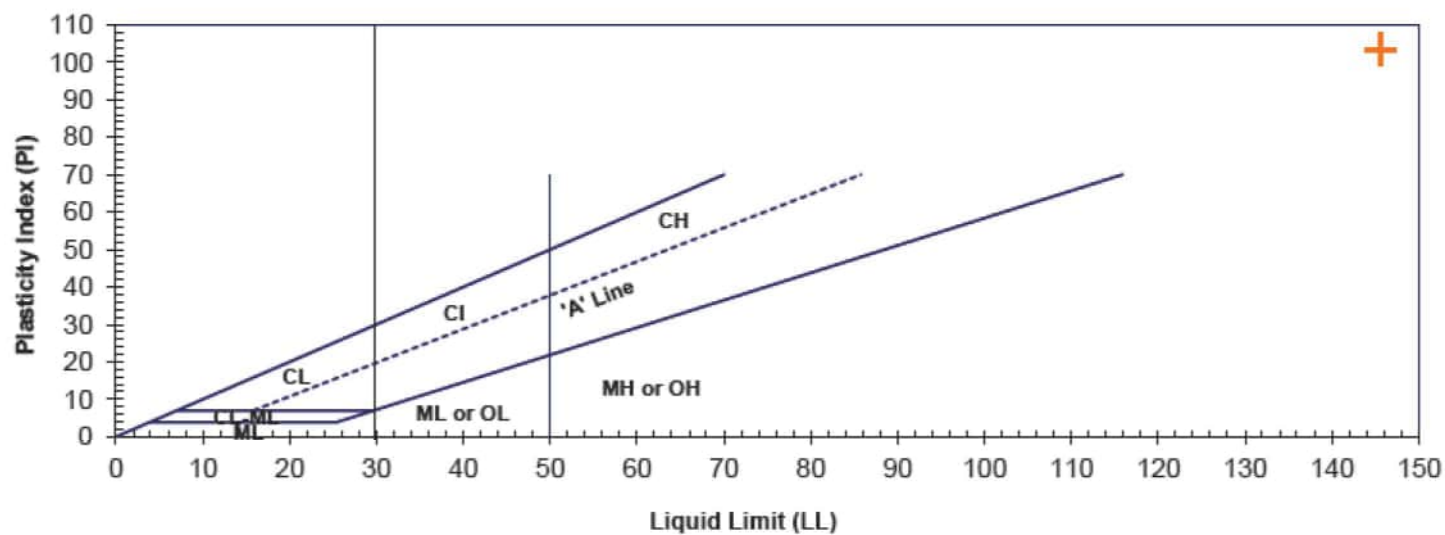
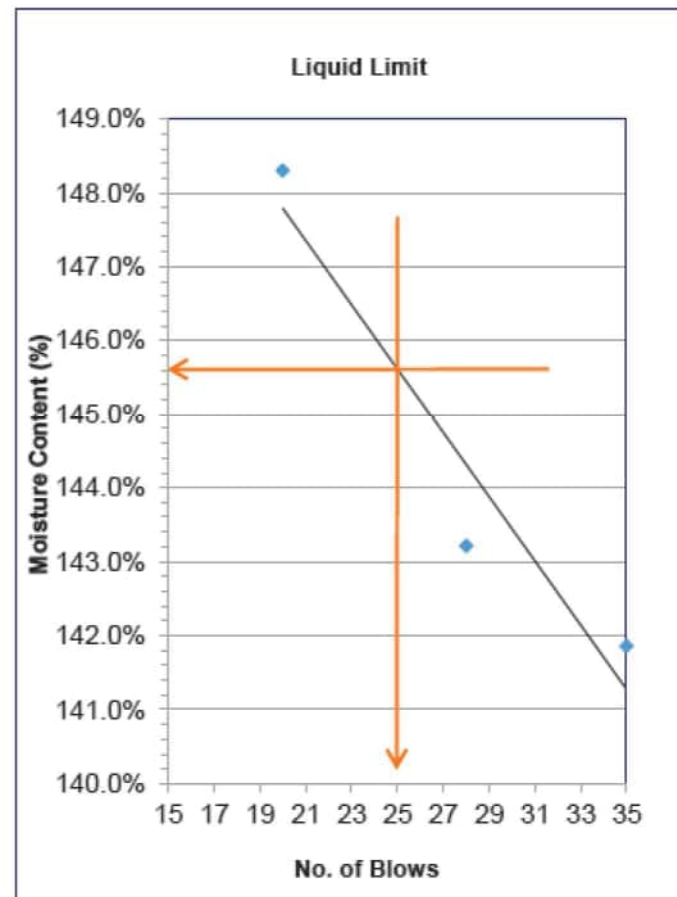
Borehole No.:	22-01	Sample No.:	S1	Depth:	9.0m
Tare No.:		26B	78	ZZ	
Weight of Tare (g):		29.45	28.81	29.43	
Weight of Wet Soil + Tare (g):		54.72	56.14	55.03	
Weight of Dry Soil + Tare (g):		39.84	40.11	39.74	
Weight of Water (g):		14.88	16.03	15.29	
Weight of Dry Soil (g):		10.39	11.30	10.31	
Moisture (%):		143.2%	141.9%	148.3%	
No. of Blows:		28	35	20	
Liquid Limit from Flow Curve				145.6	

Prep Method: Moist ☐ Dry ☒ Liquid Limit Test Method
Oversize: Pestle ☒ Grinder ☐ Method A ☒ Method B ☐

Plastic Limit - Manual Rolling Method

Tare No.:	45	57
Weight of Tare (g):	28.65	28.99
Weight of Wet Soil + Tare (g):	31.98	31.97
Weight of Dry Soil + Tare (g):	30.99	31.09
Weight of Water (g):	0.99	0.88
Weight of Dry Soil (g):	2.34	2.10
Moisture (%):	42.3%	41.9%
Plastic Limit		42.1

Grooving Tool: Plastic ☒ Metal ☐



Summary

Liquid Limit:	146	Material Description:	N/A
Plastic Limit:	42		
Plasticity Index:	103	Material Retained on 425um Sieve (%):	0.3%
Unified Soil Classification:	CH	As Received Moisture (%):	42.0%

Comments: _____

Reviewed By: Connor Carlson



Atterberg Limits As Per ASTM D4318

Liquid Limit - Hand Operated Method

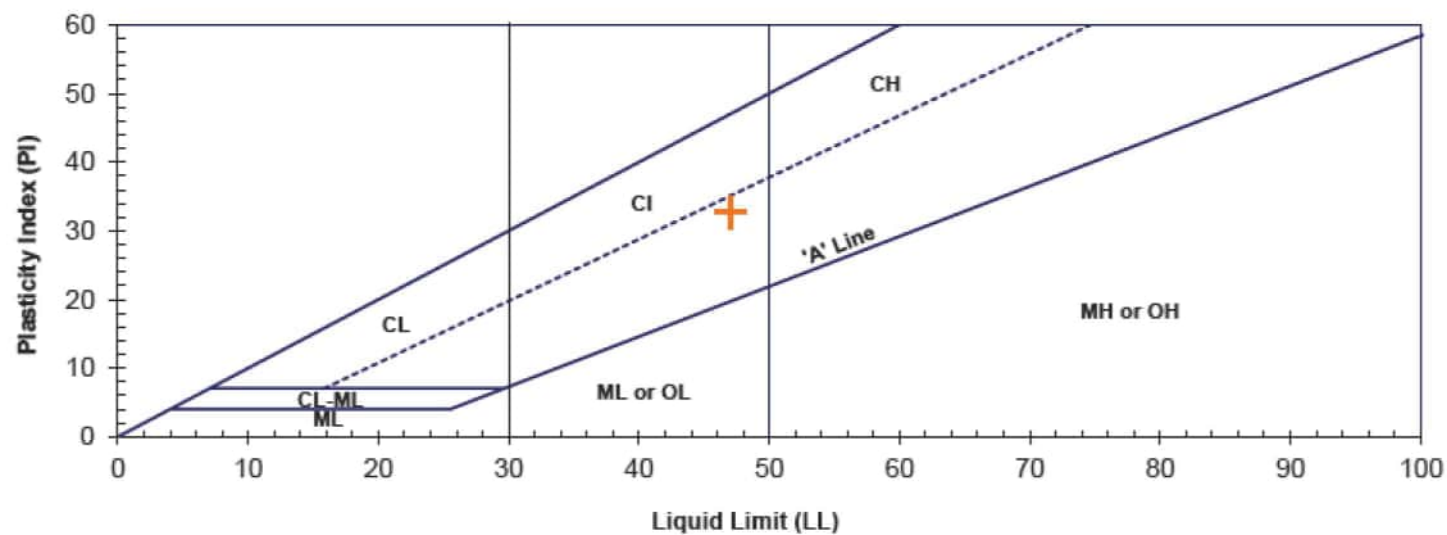
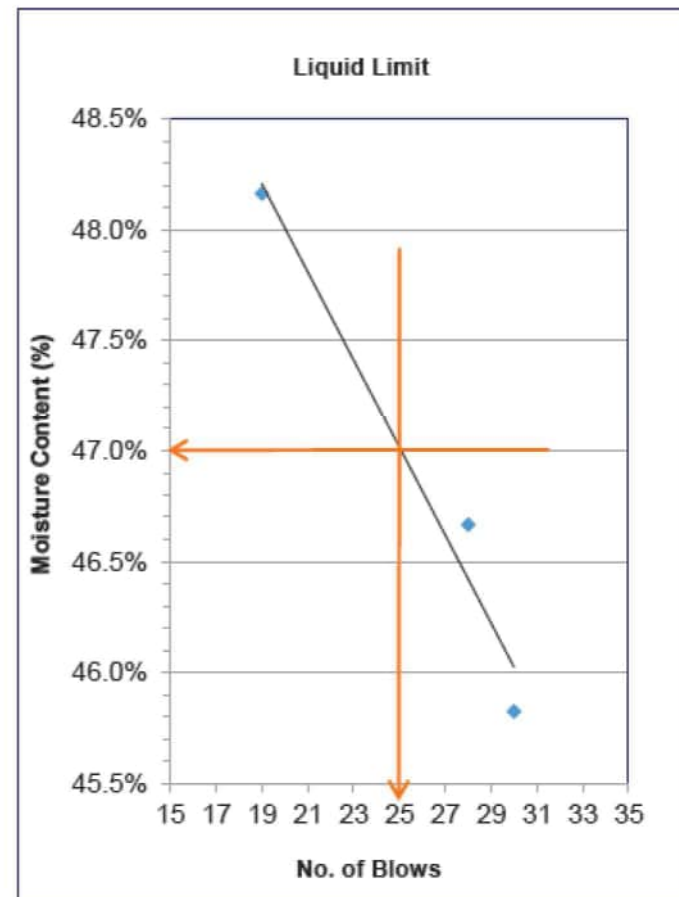
Borehole No.:	23-04	Sample No.:	AU6	Depth:	4.3m
Tare No.:	B11	B19	A27		
Weight of Tare (g):	20.69	20.72	20.79		
Weight of Wet Soil + Tare (g):	39.64	43.82	41.22		
Weight of Dry Soil + Tare (g):	33.48	36.47	34.80		
Weight of Water (g):	6.16	7.35	6.42		
Weight of Dry Soil (g):	12.79	15.75	14.01		
Moisture (%):	48.2%	46.7%	45.8%		
No. of Blows:	19	28	30		
Liquid Limit from Flow Curve	47.0				

Prep Method: Moist ☐ Dry ☒ Liquid Limit Test Method
Oversize: Pestle ☒ Grinder ☐ Method A ☒ Method B ☐

Plastic Limit - Manual Rolling Method

Tare No.:	B23	A33
Weight of Tare (g):	20.62	21.09
Weight of Wet Soil + Tare (g):	27.03	27.97
Weight of Dry Soil + Tare (g):	26.25	27.09
Weight of Water (g):	0.78	0.88
Weight of Dry Soil (g):	5.63	6.00
Moisture (%):	13.9%	14.7%
Plastic Limit	14.3	

Grooving Tool: Plastic ☒ Metal ☐



Summary

Liquid Limit:	47	Material Description: Clay, Some Silt, Trace Sand, Trace Gravel, Dark Brown, Oxidation, Coal Chips
Plastic Limit:	14	
Plasticity Index:	33	
Unified Soil Classification:	CI	
Material Retained on 425um Sieve (%):		4.4%
As Received Moisture (%):		16.6%

Comments:

Reviewed By: Connor Carlson



Atterberg Limits As Per ASTM D4318

Liquid Limit - Hand Operated Method

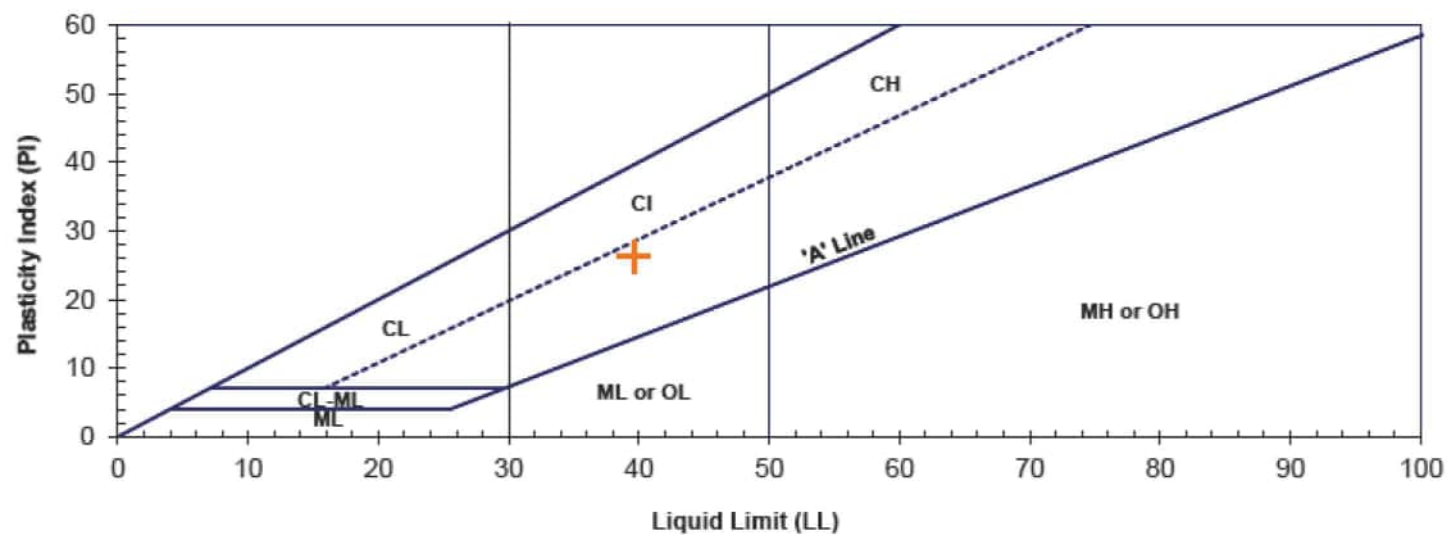
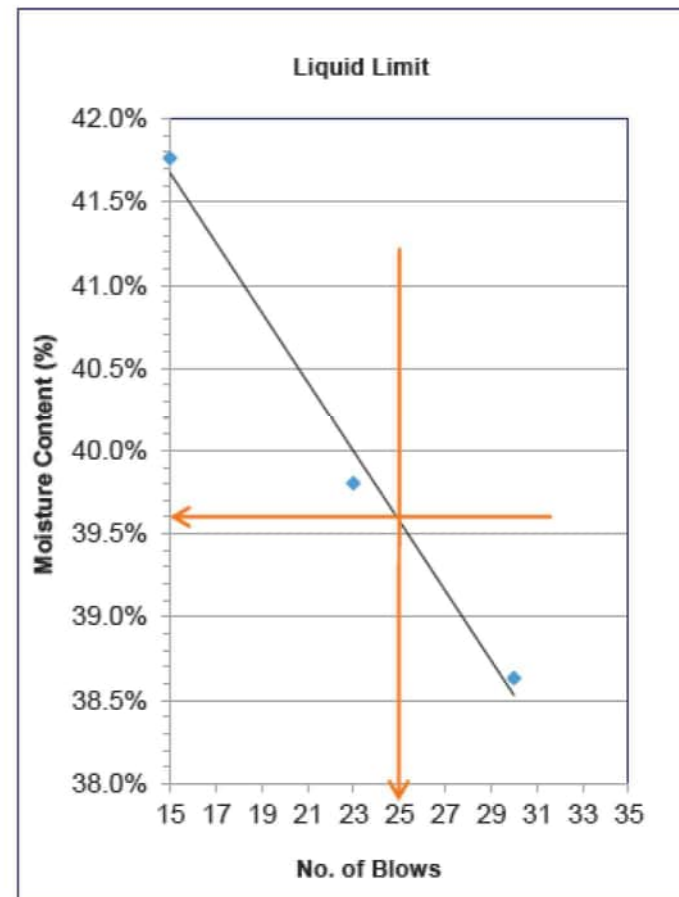
Borehole No.:	23-05	Sample No.:	AU9	Depth:	6.7m
Tare No.:		B24	A23	A29	
Weight of Tare (g):		20.68	20.69	21.00	
Weight of Wet Soil + Tare (g):		37.72	39.27	39.05	
Weight of Dry Soil + Tare (g):		32.70	33.98	34.02	
Weight of Water (g):		5.02	5.29	5.03	
Weight of Dry Soil (g):		12.02	13.29	13.02	
Moisture (%):		41.8%	39.8%	38.6%	
No. of Blows:		15	23	30	
Liquid Limit from Flow Curve				39.6	

Prep Method: Moist ☐ Dry ☒ Liquid Limit Test Method
Oversize: Pestle ☒ Grinder ☐ Method A ☒ Method B ☐

Plastic Limit - Manual Rolling Method

Tare No.:	A25	A13
Weight of Tare (g):	20.60	20.66
Weight of Wet Soil + Tare (g):	32.94	29.48
Weight of Dry Soil + Tare (g):	31.46	28.46
Weight of Water (g):	1.48	1.02
Weight of Dry Soil (g):	10.86	7.80
Moisture (%):	13.6%	13.1%
Plastic Limit		13.4

Grooving Tool: Plastic ☒ Metal ☐



Summary

Liquid Limit:	40	Material Description: Clay, Trace Silt, Trace Sand, Trace Gravel, Dark Brown, Coal Chips
Plastic Limit:	13	
Plasticity Index:	26	
Unified Soil Classification:	CI	
		Material Retained on 425um Sieve (%): 2.9%
		As Received Moisture (%): 17.4%

Comments: _____

Reviewed By: Connor Carlson



Atterberg Limits As Per ASTM D4318

Liquid Limit - Hand Operated Method

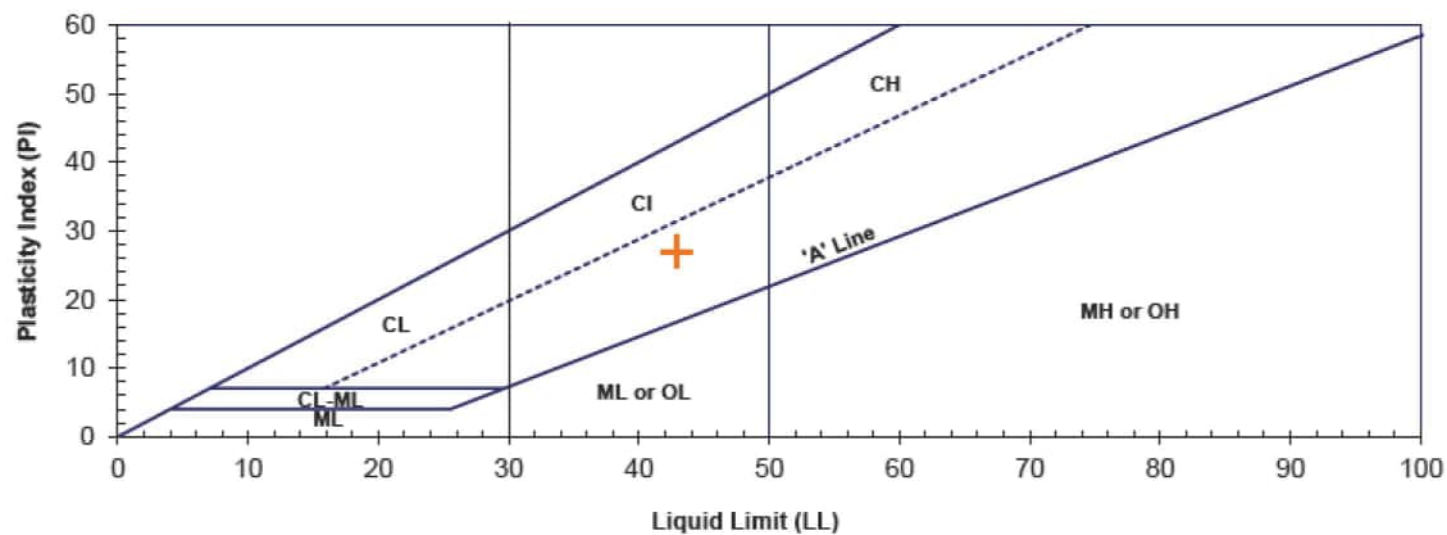
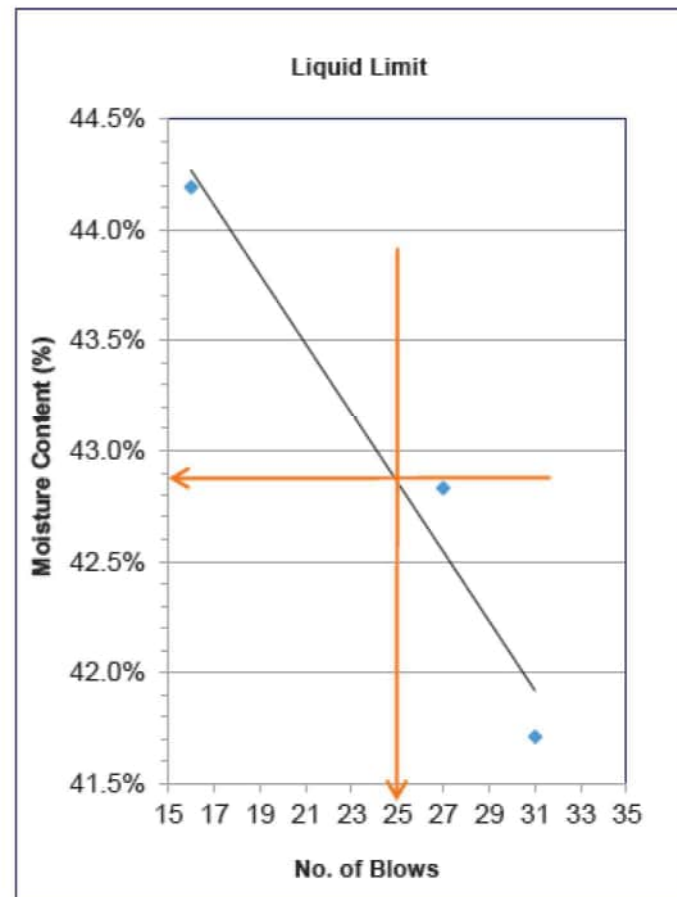
Borehole No.:	23-05	Sample No.:	AU11	Depth:	8.2m
Tare No.:		A30	A31	B18	
Weight of Tare (g):		20.73	20.73	20.80	
Weight of Wet Soil + Tare (g):		44.19	43.84	50.63	
Weight of Dry Soil + Tare (g):		37.00	36.91	41.85	
Weight of Water (g):		7.19	6.93	8.78	
Weight of Dry Soil (g):		16.27	16.18	21.05	
Moisture (%):		44.2%	42.8%	41.7%	
No. of Blows:		16	27	31	
Liquid Limit from Flow Curve				42.9	

Prep Method: Moist ☐ Dry ☒ Liquid Limit Test Method
Oversize: Pestle ☒ Grinder ☐ Method A ☒ Method B ☐

Plastic Limit - Manual Rolling Method

Tare No.:	B30	B17
Weight of Tare (g):	20.66	20.74
Weight of Wet Soil + Tare (g):	32.18	31.42
Weight of Dry Soil + Tare (g):	30.60	29.95
Weight of Water (g):	1.58	1.47
Weight of Dry Soil (g):	9.94	9.21
Moisture (%):	15.9%	16.0%
Plastic Limit		15.9

Grooving Tool: Plastic ☒ Metal ☐



Summary

Liquid Limit:	43	Material Description:	Clay, Trace Silt, Dark Brown
Plastic Limit:	16	Material Retained on 425um Sieve (%):	0.6%
Plasticity Index:	27	As Received Moisture (%):	20.4%
Unified Soil Classification:	CI		

Comments: _____

Reviewed By: Connor Carlson



Sieve Analysis of Fine and Coarse Aggregates As Per ASTM C136/C136M

Moisture Content

Tare No.:	4B
Wet Wt. + Tare (g):	2452.6
Dry Wt. + Tare (g):	2264.6
Tare Weight (g):	627.4
Moisture Content (%):	11.5%

Sieve Analysis Sample

Tare No.:	S-1
Dry Wt. + Tare (g):	616.2
Wt. of Tare (g):	150.7
Dry Wt. of Sample (g):	465.5
Adjusted Sample Wt. (g):	417.6

Aggregate Analysis

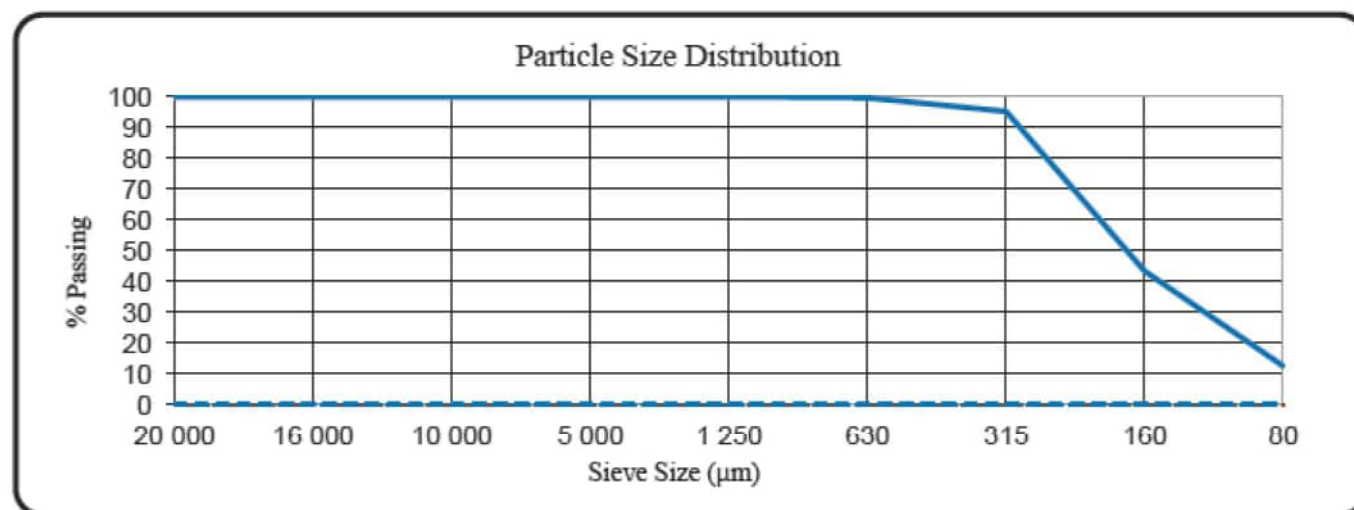
Sieve Size (µm)	Weight Retained (g):	Cumulative Weight Passing (g):	Passing (%)
20 000	0.0	417.6	100.0
16 000	0.0	417.6	100.0
10 000	0.0	417.6	100.0
5 000	0.0	417.6	100.0
1 250	0.0	417.6	100.0
630	1.2	416.4	99.7
315	18.6	397.8	95.3
160	216.3	181.5	43.5
80	128.7	52.8	12.6

Pan:	10.7	Dry Wash + Tare (g):	526.5
Total:	375.5	Tare Wt. (g):	150.7

Specification

No Specification Given	
Min	Max
N/A	N/A
N/A	N/A
N/A	N/A
N/A	N/A
N/A	N/A
N/A	N/A
N/A	N/A
N/A	N/A
N/A	N/A

Sample (g):	375.8
Loss (Max 0.5%):	-0.08%



Fracture Count in Accordance with ASTM D5821

Percentage Fracture: N/A

Comments: Borehole 23-01, S1, 13.39m

Reviewed By: Connor Carlson



Sieve Analysis of Fine and Coarse Aggregates As Per ASTM C136/C136M

Moisture Content

Tare No.:	ML
Wet Wt. + Tare (g):	1883.3
Dry Wt. + Tare (g):	1735.1
Tare Weight (g):	538.0
Moisture Content (%):	12.4%

Sieve Analysis Sample

Tare No.:	1F
Dry Wt. + Tare (g):	510.5
Wt. of Tare (g):	101.8
Dry Wt. of Sample (g):	408.7
Adjusted Sample Wt. (g):	363.7

Aggregate Analysis

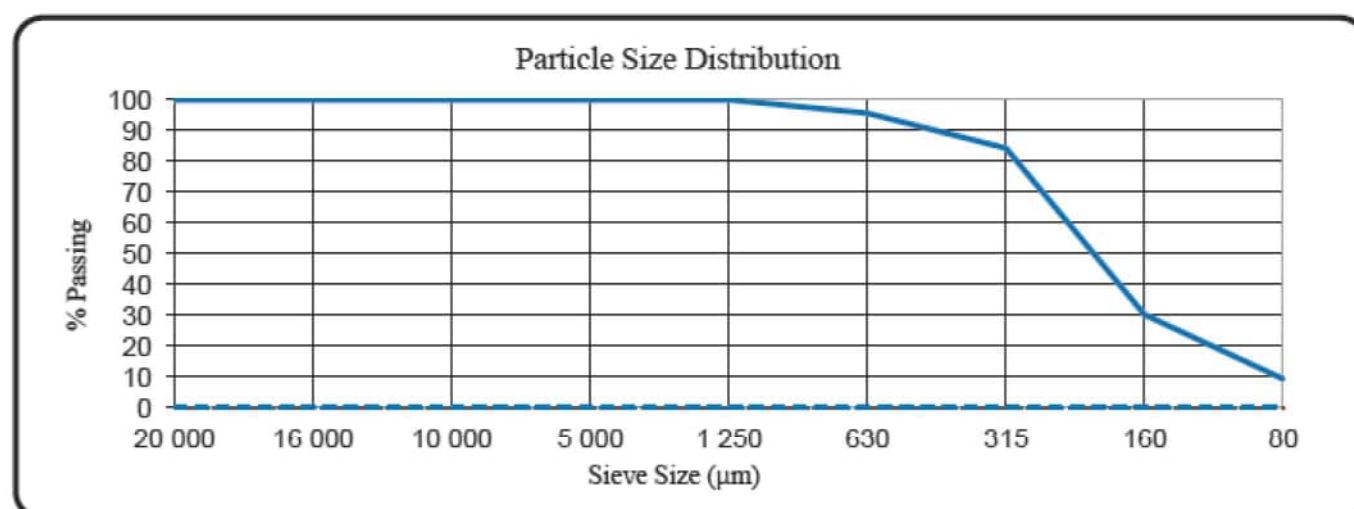
Sieve Size (µm)	Weight Retained (g):	Cumulative Weight Passing (g):	Passing (%)
20 000	0.0	363.7	100.0
16 000	0.0	363.7	100.0
10 000	0.0	363.7	100.0
5 000	0.0	363.7	100.0
1 250	0.0	363.7	100.0
630	15.8	347.9	95.7
315	41.3	306.6	84.3
160	196.3	110.3	30.3
80	76.1	34.2	9.4

Pan:	10.0	Dry Wash + Tare (g):	440.8
Total:	339.5	Tare Wt. (g):	101.8

Specification

No Specification Given	
Min	Max
N/A	N/A
N/A	N/A
N/A	N/A
N/A	N/A
N/A	N/A
N/A	N/A
N/A	N/A
N/A	N/A
N/A	N/A

Sample (g):	339.0
Loss (Max 0.5%):	0.15%



Fracture Count in Accordance with ASTM D5821

Percentage Fracture: N/A

Comments: _____ Borehole 23-02, S1, 4.5m

Reviewed By: Connor Carlson



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Appendix E

Hydrotechnical Report

NHC Ref. No. 1007352

24 February 2023

MPE Engineering
#101, 10630 - 172 Street
Edmonton, AB T5S 1H8

Attention: Haj Oshiki, P.Eng.

Via email: hoshiki@mpe.ca

**Re: Greenview Industrial Gateway Raw Water Intake
Preliminary Hydrotechnical Design – Final Report**

Dear Mr. Oshiki:

1 INTRODUCTION

Northwest Hydraulic Consultants Ltd. (NHC) has been asked by MPE Engineering and WaterSMART Solutions Ltd. to provide preliminary hydrotechnical design for developing a water intake on the Smoky River for M.D. of Greenview. This intake will supply water to the Greenview Industrial Gateway (<https://www.greenviewindustrial.ca/>). The proposed intake will be located on the west bank of the Smoky River in the reach south of the Canfor Cut-Across Road bridge in NE 30-67-4-W6 (**Figure 1**).

2 CHANNEL MORPHOLOGY

The Smoky River in the study reach has an average bankfull width of about 200 m with a low flow width of about 100 m. The channel has an irregular meander pattern, with occasional islands, point bars and mid-channel bars. The morphology in the intake reach is controlled by the bridge crossing downstream, the erosion protection works for the pipeline crossings on the downstream east bank, and the rock outcrop on the upstream west bank. There is a large mid-channel bar directly upstream of the bridge and a large island on the right (east) bank just downstream of the boat launch at the McLeod Flats Recreation Area. The river narrows at the upstream end of the recreation area just downstream of a sharp bend. There is another large mid-channel bar upstream of the bend.

2.1 Site Inspection

NHC carried out a site inspection at the proposed intake location on July 26-27, 2022. A 2500 m reach of the riverbanks were inspected upstream from the bridge. **Figures 2 and 3** show photos of the left and right banks of the reach. The locations of these photos are shown in Figure 1.

Figure 2 shows photographs of the left (west) bank of the river. Photo 2a shows the eroding bank between the bridge and the upstream pipeline right-of-way. Photo 2b shows the fine sand/silt deposit at the mouth of a small tributary and the riprap of the left (north) bank of the tributary. Photo 2c shows part of a large sand bar with small vegetation on it, indicating it was deposited recently. The tree trunks projecting out of the bank indicates that a deposit of large woody debris underlays the sand bar. Photos 2d shows the rock outcrop opposite of the recreation area boat launch and Photo 2e show the bank downstream of the outcrop. Photo 2f shows a line of posts sticking out of the water in the bend upstream of the outcrop nearer to the right bank.

Figure 3 shows photographs of the right (east) bank of the river. Photo 3a shows the eroding bank between the bridge and the upstream pipeline right-of-way while Photo 3b shows the rock spurs at the pipeline right-of-way. Photo 3c shows the entrance to a recently formed side channel downstream of the large island while Photo 3d shows the large gravel bar upstream of the island at the recreation area boat launch. Photo 3e shows the eroding bank which is threatening the access road to the boat launch. Photo 3f shows a rock outcrop at the base of the eroding bank upstream of the recreation area. This rock outcrop is lower than the rock outcrop on the left bank, but they may be connected under the river.

The gradation of the gravel-cobble sized bed material found on the large gravel bar on the left bank upstream of the tributary mouth was measured using a tape sample technique. The gradation is shown in **Figure 4** along with photos of the site. The median size of the bed material was found to be 52 mm.

Ice scars and highwater marks were identified and flagged so that the locations and elevations could be surveyed.

2.2 River Survey

NHC completed a survey of a 3.5 km reach of the Smoky River near the proposed intake from July 25-28, 2022. The survey consisted of both a ground-based survey of the river banks and a boat-based bathymetric survey of the river bed. The ground-based survey points were measured directly with a Real Time Kinematic Global Positioning System (RTK GPS). The RTK GPS has a maximum horizontal and vertical precision of 8 mm and 15 mm, respectively. The bathymetric survey points were measured using a digital echo sounder coupled with the RTK GPS to record both horizontal positioning and bed elevation. A total of 12 cross sections and 12 longitudinal lines were surveyed. The survey was corrected to the UTM Zone 11 coordinate system with a vertical datum of NAD 83 CSRS (2002.0), CGVD28 (HTv2.0).

The survey data was supplemented with elevation data developed from drone photography. This drone data was corrected to the survey datum by surveying and photographing targets placed on the ground. LiDAR data was also obtained to define the overbank areas; however, the LiDAR data was collected in 2007 and significant erosion had occurred after this date, so the survey and LiDAR bank locations are not consistent.

A contour plot of the river in the intake reach is shown in **Figure 5**. The contour plot indicates that there are two scour holes near the left (west) bank of the river in the intake reach which may be deep enough to accommodate a water intake, one about 400 m upstream of the tributary mouth and the other much

deeper area farther upstream at the rock outcrop opposite the boat launch. The downstream site has been labeled Option 1 while the upstream site has been labeled Option 2.

Figure 6 shows photographs the bank at both locations. The Option 1 site is at the downstream end of the sand bar which shows some signs of recent erosion. There appears to be bedrock in the bank at Option 2 which would make the bank less susceptible to erosion.

Figure 7 shows cross sections of the river at the two option locations. The locations of these cross sections are shown in **Figure 5**. At Option 1, the deepest part of the section is at an elevation of 570.81 m about 30 m out from the top of the left bank. Downstream of the section the channel moves farther from the bank and becomes shallower. At Option 2, the deepest part of the section is at an elevation of 569.37 m about 11 m from the top of the left bank. The channel is deeper upstream; however, the additional depth is not necessary and there is a higher risk that the channel could shift away from the bank.

2.3 Air Photos and Satellite Imagery

Historical air photos and satellite images from 1950 to 2022 were evaluated to assess the channel stability in the intake reach. Air photos from 1950, 1966, 1975, and 1983 are shown in **Figure 8** while images from 1997, 2010, and 2022 are shown in **Figure 9**. Traced lines of the banks and bars from these seven images are compared in **Figure 10**. Some of the apparent difference between the bank locations may be due to photo distortion and georeferencing inaccuracies. The air photo series indicates that the bridge was built some time before 1975.

There has been considerable erosion of the right bank within the study reach. The right bank upstream of the bridge has continued to erode significantly since the bridge was built. Rock spurs built along the bank in the pipeline right-of-way since 2010 have reduced bank erosion in the right-of-way, but the erosion has continued to occur just upstream of the bridge. The large island upstream of this has remained in place since it formed after 1975 when an island on the opposite bank was eroded. The right bank upstream of the sharp bend upstream of the recreation area has also eroded significantly; however, the rate of erosion has reduced considerably since 1997. This erosion has resulted in the high flow channel attacking the left bank directly at the narrow section.

The left bank upstream of the bridge has been much more stable over time, especially after the bridge was built. Some minor erosion has occurred recently between the bridge and the pipeline right-of-way. The area of bank from 300 m downstream to 400 m upstream of the tributary mouth has been very stable; however, upstream of this area the left bank has been quite active. The bank in this next 400 m first eroded about 50 m between 1966 and 1997, then re-established the bank near its original location by 2010 and is now eroding again. Upstream of this unstable reach beyond the narrow point at the rock outcrop opposite the boat launch, the left bank has been stable since 1983. Some minor bank erosion has also occurred in the outside of the upstream bend.

The bank line comparison indicates that the left bank of the river at both intake location options has been relatively stable over a period of 72 years. At the Option 1 location, the right bank has been migrating to the east especially downstream of the site so there is some risk that the main channel could shift to the right in the future. The left bank upstream of the Option 1 location has been quite unstable

and may erode again in the near future. At the Option 2 location, the upstream right bank has migrating downstream; however, most of this movement occurred before 1983. There is still some risk that the main channel will migrate north of the intake; however, the right bank migration at the Option 2 location could be controlled with spurs on the right bank which would also protect the access road to the boat launch. Controlling the location of the main channel at Option 1 would require more significant intervention into the channel.

3 HYDROLOGY

The Smoky River originates in the Rocky Mountains southwest of Grand Cache, Alberta. The river flows northeast through foothill and prairie landscapes before entering the Peace River. A number of major tributaries enter the Smoky River, the largest of which are the Wapiti River, the Simonette River, and the Little Smoky River. Most of the annual runoff occurs between May and August and is generated by both snowmelt and large scale intense rain events that occur in the various tributaries along the foothills.

Water Survey of Canada (WSC) operates two streamflow gauges on the Smoky River, one at Watino (07GJ001) downstream of the study reach and one at Hells Creek (07GA001), upstream of the study site. The Watino gauge has a longer record, operating from 1915-1922 and 1955 to present. The Hells Creek gauge has operated since 1968, but winter flows were only reported from 1968 to 1986. The drainage area at Watino of 50,300 km² is much larger than the drainage area of 3874 km² at Hells Creek, so daily flows at the two gauges do not correlate well.

There are six other active gauges on tributaries of the Smoky River with longer term records. The record lengths and drainage areas of these gauges are listed in **Table 1** along with the those from the two Smoky River gauges. Note that the gauge on the Kakwa was moved 8.6 km downstream to its present Hwy 40 location in 1994. Three of the eight gauges provide continuous records of discharge including winter flows. Three other gauges provide winter discharges for only part of their record and two gauges provide no winter records.

The relative locations of the gauges listed in Table 1 are shown in the schematic diagram in **Figure 11**. The Muskeg, Kakwa and Cutbank rivers flow into the Smoky River between the Hells Creek gauge and the study area, while the Wapiti, Simonette and Little Smoky rivers flow into the Smoky River between the study area and the Watino gauge. The Hells Creek gauge is located just downstream of where the Smoky River exits the mountains, so it represents mountain runoff conditions. The Wapiti, Kakwa, Cutbank and Muskeg Rivers have some drainage area in the mountains whereas the Simonette and Little Smoky gauges measure runoff only from the foothills areas.

The drainage area upstream of the local tributary was estimated to be 12,603 km². This was determined from the drainage area 12,675 km² for the Smoky River at the downstream bridge obtained from the Alberta Transportation bridge file and subtracting the drainage area of the local tributary of 72 km².

Table 1 Summary of WSC streamflow gauge records

Gauge Number	Gauge Name	Period of Record	Continuous Record	Drainage Area (km ²)
07GA001	Smoky River above Hells Creek	1968-2020	1968-1986	3,870
07GA002	Muskeg River near Grande Cache	1972-2020	1972-1986	703
07GB003	Kakwa River at Highway No. 40	1975-2020	None	3,370
07GB001	Cutbank River near Grande Prairie	1970-2020	None	842
07GE001	Wapiti River near Grande Prairie	1960-2020	All	11,300
07GF001	Simonette River near Goodwin	1969-2020	1970-1986	5,040
07GH002	Little Smoky River near Guy	1959-2020	All	11,100
07GJ001	Smoky River at Watino	1915-1922, 1955-2020	All	50,300

3.1 Annual Peak Flows

Frequency analysis was carried out on the reported annual peak flows for all eight WSC gauges. The ratios between instantaneous and daily peak flows were used to estimate instantaneous peak flows from the daily peak flows where instantaneous peak flows were not available. Log-Pearson 3 (LP3) distributions were found to fit all the data sets well as shown in **Figure 12**. The frequency distribution for the Smoky River at Hells Creek is much flatter than that of the other distributions, which indicates that the drainage basin upstream behaves differently than the other seven drainage basins.

Figure 13 shows the variation of the 2-year and 100-year annual peak flows with drainage area. The Smoky River at Watino annual peak flows are near the trend line but the Smoky River above Hells Creek falls below the trend line for the 100-year annual peak flows but above the line for the 2-year annual peak flows.

The Hells Creek gauge is on the mainstem of the Smoky River upstream of the intake reach so flows at the gauge directly affect flows at the study reach. Therefore, return period annual peak flows in the study reach were estimated by using regional analysis of the return period inflows between the two Smoky River gauges, prorating these inflows by the drainage area ratio to a power based on the trend for that return period and adding them to the return period annual peak flows for the Hells Creek gauge. This can be expressed as:

$$Q_{intake} = Q_{Hells\ Creek} + (Q_{Watino} - Q_{Hells\ Creek}) * \left[\frac{(A_{Intake} - A_{Hells\ Creek})}{(A_{Watino} - A_{Hells\ Creek})} \right]^m$$

The resulting annual peak flows at the intake site for various return period are listed in Table 2 along with the annual peak flows from the two Smoky River gauges.

3.2 Minimum Flows

Minimum flows produce different water levels in summer and winter due to the effects of the ice cover in winter, so the flow records for the WSC gauges were split into separate ice-affected and open-water periods. WSC notes ice-affected flows with the label “B” in the record, so this label was used to separate the daily flows into open-water (summer) and ice-affected (winter) periods.

Table 2 Summary of annual peak flows for various return periods.

Return Period (years)	Smoky River above Hells Creek Flow (m ³ /s)	Smoky River at Watino Flow (m ³ /s)	Smoky River at Intake Flow (m ³ /s)	Regional Slope, m
100	1364	14,147	4822	0.64
50	1159	11,453	3821	0.66
25	925	9178	2755	0.69
10	771	6704	2105	0.71
5	633	5164	1562	0.74
2	462	3849	958	0.80

3.2.1 Minimum Open-water Flows

The minimum daily flow for each open-water season was extracted from the record from each of the eight gauges listed in Table 1. A significant amount of data was missing from all the gauges in both 2016 and 2017, so no minimum flows were reported for those years.

Frequency analyses of open-water minimum daily flows were carried out on the records of all eight gauges. LP3 distributions were found to fit all the data sets well, as shown in **Figure 14**. Regression analysis was carried out on the gauged tributary inflows for each return period to define the variation of minimum open-water flows with drainage area. The results, an example of which is shown in **Figure 15**, were added to the minimum flow for the Smoky River above Hells Creek to estimate the minimum open-water flow in the intake reach for each return period listed in **Table 3**. The 100-year minimum open-water flow in the intake reach obtained from this method is 12.9 m³/s.

3.2.2 Minimum Ice-affected Flows

The minimum daily ice-affected flows for each contiguous winter season were also extracted from the records for all six gauges with continuous records listed in **Table 1**. A significant amount of data was missing from all the gauges in both 2016 and 2017, so no minimum flows were reported for those years. As well, three of the gauges have only a limited period of continuous record of 15 to 18 years ending in 1986.

Table 3 Summary of minimum flows in the intake reach for various return periods.

Return Period (years)	Open Water Minimum Flow (m ³ /s)	Ice-affected Minimum Flow (m ³ /s)	Breakup Peak Flow (m ³ /s)
100	12.9	9.6	595
50	13.8	10.3	471
25	15.1	11.2	366
10	17.5	12.1	252
5	21.7	13.4	180
2	28.3	14.7	99

A frequency analysis of ice-affected minimum daily flows was carried out on two separate record lengths to assess the effects of reducing the number of gauges from six to three so that a much longer period of record could be evaluated. First, frequency analyses were carried out on all six data sets for the period 1969 to 1986. LP3 distributions were found to fit all the data sets well, as shown in **Figure 16**. The frequency distributions for the Smoky River above Hells Creek and the Wapiti River near Grande Prairie were found to have much flatter distributions than the other tributary gauges.

Frequency analyses were also carried out for the entire continuous records for the Smoky River at Watino, the Wapiti River near Grande Prairie and the Little Smoky River near Guy. The frequency distributions for these long term records were then compared to the short term 1969 to 1986 distributions, as shown in **Figure 17**. While the Smoky River distributions were quite similar, the distributions for the other two gauges were different.

The minimum open-water flow methodology of estimating flow in the intake reach by adding tributary inflows to the Smoky River at Hells Creek flows for each return period using the six available short term gauges was compared to an alternative 3-gauge method for the same 1969 to 1986 record. The 3-gauge method consisted of subtracting tributary inflows between the intake reach and Watino from the discharge at Watino. Tributary inflows were estimated using a weighted average of the Wapiti and Little Smoky distributions. This alternative 3-gauge method was found to estimate flows in the intake reach within about 5% of the 6-gauge method with a weighting of 40% Wapiti and 60% Little Smoky.

The 3-gauge method was used to estimate minimum ice-affected flows using the longer 60+ years of record of the three long term continuous gauges. The results of this analysis are listed in Table 3. The more extreme flows tend to be greater with the longer period of record due to the lower tributary inflows.

3.3 Breakup Peak Flows

Annual breakup peak flows were obtained from the six gauges with continuous records by extracting the maximum flow rate during the breakup period for each year. The breakup records for the three shorter

term continuous gauges (Muskeg, Simonette, Hells Creek) could be extended to 2020 because, even though the records were seasonal, the gauges were started each year before breakup.

Log-Pearson 3 (LP3) distributions were found to fit all the data sets well as shown in **Figure 18**. As with the annual peak flows, the frequency distribution for the Smoky River at Hells Creek is much flatter than that of the other distributions, which indicates that the drainage basin upstream behaves differently than the other drainage basins.

As with the annual peak flows, return period breakup peak flows in the intake reach were estimated by using regional analysis of the return period inflows between the two Smoky River gauges, prorating these inflows by the drainage area ratio to a power based on the trend for that return period and adding them to the return period annual peak flows for the Hells Creek gauge. The results of this analysis are listed in Table 3.

3.4 Daily Flows

Daily flows in the intake reach were estimated by prorating the difference in flow between the two Smoky River gauges by the drainage area ratio to a power and adding it to the flow for the Hells Creek gauge. A power of 0.85 was adopted which is the slope of the mean annual flow trend line for the eight gauges. This value is also between the 0.80 power for the 2-year annual peak flow trend and the 0.93 power for the 2-year minimum open water flow trend. To account for travel time, the flow from one day earlier at Hells Creek was used with the flow from one day later at Watino.

This method estimated a flow rate of 275 m³/s in the intake reach on July 27, 2022. This estimate is within about 5% of the discharge measured in the intake reach on that day of 290 m³/s so the method appears to provide reasonable estimates of daily flows.

Flow duration curves were generated for the intake reach for both open water and ice-affected periods using the above methodology (**Figure 19**). Flow data from 1968 to 2020 was used in the analysis of open water flows and flow data from 1968 to 1986 was used for the ice affected flows. This shorter ice-affected period may not be as representative of longer term conditions, but the method used for extending the minimum ice-affected flows could not be used for average flows due to the increased variability in flows.

Typical ice-affected flows tend to be much lower than typical open water flows in the intake reach. For example, the median ice-affected flow is estimated to be 22.7 m³/s while the median open water flow is estimated to be 182 m³/s.

3.5 Climate Change

Climate change is expected to change the air temperature and precipitation regimes in the Smoky River drainage basin. Climate predictions from the CMIP5 climate model datasets were downscaled to the intake site and bias-adjusted using the BCCAQv2 method (Environment Canada, 2022). According to these datasets, annual air temperature and precipitation are expected to increase but the magnitudes will depend on which future CO₂ reduction scenario occurs. Ten year average values of temperature and

precipitation from 2043 to 2052 were compared to the values from 2013 – 2022 to quantify these changes.

The annual temperature increase predictions range from 0.4°C to 1.4°C with a likely scenario of 1.4°C (RCP4.5). Annual precipitation is also expected to increase, with predictions ranging from 0.9% to 3.8% with a likely scenario of 1.9% (RCP4.5).

Table 4 provides a summary of the expected seasonal changes in temperature and precipitation for the RCP4.5 scenario. Temperature increases will be highest in winter and lowest in spring while precipitation increases will be greatest in the spring and fall with lower precipitation expected in the summer. Individual precipitation events may also become more severe; however, it is more difficult to quantify these changes.

Table 4 Summary of predicted changes in temperature and precipitation for the RCP4.5 emissions scenario.

Period	Temperature °C	Precipitation mm	Precipitation %
Spring	1.1	6.0	7.1%
Summer	1.2	-7.6	-3.5%
Fall	1.4	6.2	6.5%
Winter	1.9	1.5	1.7%
Annual	1.4	9.3	1.9%

These changes in temperature and precipitation will cause some predictable changes in hydrologic conditions. These include a partial shift from snowfall to rainfall in the winter season, the earlier arrival of the snowmelt hydrograph, and the gradual loss of glaciers. It is expected that increased precipitation may lead to higher flood peaks due to increased precipitation intensity, but this will be mitigated by reduced snowpack and drier antecedent moisture conditions due to higher temperatures. Loss of tree cover and soil changes associated with beetle infestation, salvage logging and wildfires can also contribute to higher runoff peaks.

The magnitudes of flow rate changes in the Smoky River are not predictable at present. It is expected that climatic changes may increase peak flows and reduce minimum flows, but there is not a direct correlation between extreme flow rates and temperature or precipitation increases.

To gauge the effects of potential changes in flow due to climate change, the effects of a 10% increase in peak flows and a 10% decrease in minimum flows in the Smoky River were evaluated. A 10% increase in the 100-year open water peak flow would produce a water level increase of about 0.32 m, while a 10% increase in the 100-year breakup peak flow would produce a water level increase of 0.12 m. These changes in peak water levels do not significantly affect the project. A 10% increase in open water peak flow would also increase velocities by about 5%. Erosion protection designs typically have sufficient factors of safety to accommodate this magnitude of velocity increase. A 10% decrease in the 100-year minimum open water flow would produce a decrease in water level of about 0.13 m, and a 10%

decrease in the 100-year winter minimum flow would produce a decrease in the bottom-of-ice level of about 0.03 m. These changes in minimum water levels do not significantly affect the project.

4 HYDRAULIC ANALYSIS

Hydraulic analysis of the study area was carried out to determine design water levels for the intake. The Hydrological Engineering Center's River Analysis System (HEC-RAS) developed by the Army Corps of Engineers was used to develop a one-dimensional (1D) numerical hydraulic model of the reach. The survey data and LiDAR data were combined to develop a model of the intake reach starting about 800 m downstream of the bridge and extending upstream about 3760 m. The model cross section locations are shown in **Figure 20**. Model results at the sections labeled 1716 (Option 1) and 2624 (Option 2) were used to define water levels for various discharges at the two intake option locations.

4.1 Open-water Water Levels

The hydraulic model was calibrated for open water conditions using the discharge and water level data collected during the survey. The simulated water surface profile is shown in **Figure 21** along with the surveyed water level data. A channel roughness of 0.040 was found to best fit the surveyed discharge and water levels. An overbank roughness of 0.120 was adopted due to the dense vegetation on most areas of the floodplain.

The adopted roughness values were checked using the highwater marks identified during the site inspection along with an estimated 2022 peak flow in the intake reach of 1440 m³/s and a 2020 peak flow of 3110 m³/s (**Figure 22**). The simulated water levels for the 2022 peak flow fall between the highwater marks, and the simulated water levels for the 2020 peak flow are slightly above the highwater marks. It is unclear which highwater marks were produced by which flow but the simulations suggest that the bed roughness value may be lower for these higher flow rates. The flow rates for these events and the highwater levels are less accurate than the measured flow rate and water levels for the calibration, so the roughness values were not reduced when modeling high flows.

Open water rating curves for the two intake option locations were generated using the calibrated hydraulic model. The rating curves are shown in **Figure 23 and 24**. Open water peak water levels for both intake locations are listed in **Table 4** for various return periods. The 100-year annual peak water level was found to be 578.66 m at Option 1 and 579.21 at Option 2. Open water minimum water levels for both intake locations are also listed in **Table 4** for various return periods. The 100-year minimum open-water water level was found to be 571.94 m at Option 1 and 572.57 at Option 2.

4.2 Ice Levels

Ice levels are affected by three ice processes: 1) the growth of a solid ice cover 2) the formation of ice jams and 3) the formation of ice runs. Each of these processes causes the water levels to increase relative to the open water level for the same discharge. Bottom-of-ice levels are important for intake port design while ice jam water levels affect the placement of above ground facilities.

Table 5 Summary of open-water water levels for various return periods.

Return Period (years)	Open Water Peak Water Level at Option 1 (m)	Open Water Peak Water Level at Option 2 (m)	Open Water Minimum Water Level at Option 1 (m)	Open Water Minimum Water Level at Option 2 (m)
100	578.66	579.21	571.94	572.57
50	577.83	578.55	571.95	572.59
25	576.89	577.75	571.97	572.61
10	576.21	577.18	572.01	572.65
5	575.67	576.63	572.06	572.71
2	574.83	575.80	572.14	572.80

4.2.1 Bottom-of-ice Levels

Solid ice cover rating curves for the two intake locations were developed using the hydraulic model by adopting a typical under-ice roughness of 0.020 for smooth solid ice. The water level for a particular discharge is also affected by the thickness of the ice. The solid ice water level rating curves shown in **Figures 23 and 24** are for an ice thickness of 0.6 m.

The adopted ice thickness does not affect the bottom-of-ice levels, which are determined by the amount of flow under the ice and the roughness. Bottom-of-ice rating curves for solid ice for the two intake option locations are shown in **Figures 23 and 24**. Minimum bottom-of-ice levels for both intake locations are listed in **Table 5** for various return periods. The 100-year minimum bottom-of-ice level is 571.90 m at Option 1 and 572.52 m at Option 2. Bottom-of-ice rating curves for frazil ice or breakup jams would be higher than for solid ice since the under-ice roughness for these ice conditions would be greater than 0.020. Frazil ice may be distributed unevenly under a solid ice surface so freeboard should be subtracted from the average bottom-of-ice level to account for this variability.

Table 6 Summary of ice-affected water levels for various return periods.

Return Period (years)	Minimum Bottom-of-ice Elevation at Option 1 (m)	Minimum Bottom-of-ice Elevation at Option 2 (m)	100-year Peak Ice Jam Water Level at Option 1 (m)	100-year Peak Ice Jam Water Level at Option 2 (m)
100	571.90	572.52	577.8	579.2
50	571.91	572.54	577.3	578.6
25	571.93	572.55	576.7	578.0
10	571.94	572.57	575.9	577.1
5	571.96	572.60	575.3	576.4
2	571.98	572.62	574.2	575.2

4.2.2 Ice Jam Water Levels

Ice jam rating curves for the two intake locations were also developed using the hydraulic model. No ice jam elevation data was available for calibration; however, ice scar elevations were surveyed along the reach. The ice jam profile simulated for the 25-year breakup flow with an ice jam roughness of 0.080 shown in **Figure 25** is higher than all but the most extreme ice scar elevations.

Ice jam rating curves for the two intake option locations simulated with this adopted ice jam roughness are shown in **Figures 23 and 24**. As expected, they are much higher than the open water or solid ice cover rating curves.

The frequency of ice jam elevations is not the same as the frequency of breakup flows because ice jams may not occur every year in the study reach. In some years the ice may melt in place and in other years an ice run may pass through the reach without jamming. An assessment of the available instantaneous water levels from the Watino gauge and the Wapiti gauge from 2000 to 2021 was carried out to estimate the percentage of time each type of breakup occurred. At the Watino gauge, the ice melted in place about 30% of the breakups and ice jams occurred in about 50% of the remaining breakups. At the Wapiti gauge, the ice melted in place about 40% of the breakups and ice jams occurred in about 60% of the remaining breakups. The percentages from the Wapiti gauge produce slightly higher water levels so they were adopted for the intake reach.

A joint probability analysis of breakup flow and ice jam probability was carried out to estimate the peak ice jam water levels for various return periods. Peak ice jam water levels for both intake locations are listed in **Table 6** for various return periods. The 100-year peak ice jam water level location is 577.8 m at Option 1 and 579.2 m at Option 2. It is recommended that a 2.0 m freeboard be applied to the values in **Table 6** to avoid ice issues because maximum ice levels are higher than the water levels and the most extreme ice scars in the reach are even higher.

4.2.3 Sediment Concentrations

Suspended sediment concentrations are available from WSC for both Watino and Hells Creek gauges on the Smoky River; however, the Hells Creek data are limited. The variation of measured sediment concentrations with discharge at the two sites are shown in **Figure 26**. Sediment concentration varied with the square of discharge at both gauges, but sediment concentrations are much higher at Hells Creek due to the higher flow velocities at this gauge. Based on drainage area, it is estimated that the sediment concentrations in the intake reach would be about halfway between the two gauges for the same discharge. This estimated concentration curve is also shown in **Figure 26**.

Size gradation of the suspended sediment concentrations are not available. Typically, only very fine particles are suspended at low flows and increasingly larger particles are suspended as flow increases.

5 INTAKE DESIGN

Various types of river intakes can be used to withdraw water from rivers, such as gallery intakes, off-stream channel intakes, mid-channel intakes and bank intakes. Each of these types of intakes were considered briefly before the selection of the intake site was finalized.

Gallery intakes are perforated pipes installed in a sand or gravel river bank or bed. These intakes rely on flow through the porous substrate, so they tend to have limited capacity, or require a deep layer of sand or gravel, so are not suitable for this site.

Off-stream channel intakes provide protection for the intake but tend to have channel entrance maintenance issues due to sedimentation in rivers that transport large quantities of sediment, so are not suitable for this site.

Mid-channel intakes are structures located in the middle of the river that are connected to the bank via pipelines. The pipelines are subject to exposure due to bed scour and the intakes are more difficult to construct in the middle of the channel, so they are only used where deep areas in the channel are not located along banks.

Bank intakes are structures that project out a short distance from the bank into a deep area of the river. These intakes are easier to construct because they are connected to the bank. They also tend to promote bed scour due to their projection from the bank. There are locations in the intake reach where there are deep areas near the bank, so a bank intake was selected as the most suitable type of intake for this project.

5.1 Intake Location

The intake structure should be located within an area where natural bed scouring provides the ability for a river to maintain/sustain a natural deeper area, particularly during low flow periods. The outside of stream meanders or reaches having constricted channels are examples.

Analysis of survey data indicates that there are two scour holes near the left (west) bank of the river in the intake reach which may be deep enough to accommodate a water intake. These locations are shown in **Figure 5**. The downstream site has been labeled Option 1 while the upstream site has been labeled Option 2.

Historical air photo analysis indicates that the left bank of the river at both locations has been relatively stable over the last 70 years. At Option 1, the right bank has been migrating to the east, so there is some risk that the main channel could shift to the right from its present location. At Option 2, the upstream right bank has been migrating downstream, so there is some risk that the main channel will migrate north of the intake. The bank migration at Option 2 could be controlled with bank protection on the right bank which would also protect the access road to the campground and boat launch. Controlling the location of the main channel at Option 1 would require more significant intervention into the channel. As well, there appears to be bedrock in the bank at Option 2 which would make this bank less susceptible to erosion.

At Option 1, the deepest part of the section is 1.2 m below the 100-year open water low flow water level and only 1.0 m below the 100-year winter low flow bottom of ice level. This deepest point is about 30 m from the top of the left bank. Downstream of the section, the channel moves farther from the bank and becomes shallower.

At Option 2, the deepest part of the section is 3.3 m below the 100-year low flow open water level and 3.0 m below the 100-year winter low flow bottom of ice level. The deepest part of the channel is 11 m from the top of the left bank. The channel is deeper upstream; however, the additional depth is not necessary and there is a higher risk that the channel could shift away from the bank.

The bottom of the intake port should be at least 0.6 m above the bed, 0.6 m below the bottom if ice and have a minimum port height of 0.3 m; therefore, the depth of flow should be at least 1.5 m during minimum flows. The Option 1 location does not meet these criteria, so this location is not feasible unless these criteria are relaxed. The deeper water at Option 2 allows for more distance between the top of port and the bottom of ice and allows for the option of a shorter, higher port shape to reduce intake size.

Based on the above information, it is recommended that Option 2 be selected for the intake location. The following sections discuss two intake design options at the Option 2 location.

5.2 Intake Geometry

Two proposed intake geometries for the Option 2 location were developed. Option 2A is a smaller intake with fish screens inside the intake chamber while Option 2B is a larger structure with the fish screens at the ports. The following sections provide details of these options. The geometry and layout of Option 2A are provided in **Figures 27 and 28** and the geometry and layout of Option 2B are provided in **Figures 29 and 30**.

5.2.1 Intake Elevation

Two options for top of intake elevation are available. To satisfy navigation requirements, it must be at least 1.0 m below the 2-year low flow water level or at or above the 2-year peak flow water level. It should be noted that both these top of intake elevations will be submerged at times, so the top of the structure should be smooth enough that debris does not snag on it.

For Option 2, the top of the intake must be at or below 571.80 m or at or above 575.80 m. The minimum bed level is 569.4 m, so both elevations are feasible at this site. The higher elevation is recommended since it allows for easy maintenance access at normal flows and is more likely to promote scour at the base of the intake.

5.2.2 Intake Shape

A wedge shaped bank intake is recommended at Option 2. Wedges provide a smoother flow pattern in front of the intake ports and promote scour near the intake face rather than farther out like a rectangular structure would. Bank intakes provide easier access and less piping.

A wedge shaped intake would consist of a vertical wall containing the intake ports which is positioned at a 20° angle relative to the flow direction, which assumed to be perpendicular to the cross section line shown in Figures 28 and 30. Upstream of the intake face a nose wall would extend toward the bank for some distance at an angle of 45° relative to the flow direction. This nose wall may be truncated as shown in Figures 28 and 30 to shorten the length of the intake. These angles may be adjusted after two-dimensional flow modeling is carried out to determine to true angle of the approach flow. Downstream of the intake face, a 90° turn of the wall would extend the intake back into the bank. This shape provides relatively uniform hydraulic conditions along the ports and a sweeping velocity along the face of the intake to clear any debris and discourage fish impingement.

5.2.3 Fish Screens and Port Size

According to the Greenview Industrial Gateway Water Diversion Licence Application, the intake project is needed to ultimately provide a total annual volume of 24,000,000 m³, which requires continuous pumping of 0.761 m³/s. The maximum diversion rate required for the project will be 1.27 m³/s. The project will be completed in three stages so at least three pumps are required. MPE has stated that four pumps, each with a capacity of 0.355 m³/s would provide some redundancy in case of pump failure and have some additional capacity to allow for periods of no pumping. The intake structure should also have four chambers, one to service each pump. The withdrawal capacity of each chamber should therefore be 0.355 m³/s plus any fish diversion flows required. These fish diversion flows are typically about 10% of the withdrawal flow.

The intake should be designed for Burbot which require maximum fish screen approach velocity of 0.035 m/s (Di Rocco and Gervais, 2021). The minimum fish screen area required for each chamber is 10.14 m². Fish screening could be installed directly at the intake or within the intake chamber. Fish pumps and fish diversion flows are required if the screens are located in the intake chambers.

Rectangular fish screens can be installed on the inside of the intake ports behind trash bars. This type of screen has the advantage that backwash will force debris out into the flow where it will be swept downstream. This type of fish screen can also be easily removed for maintenance. Fish pumps are not required with this type of screen because fish are prevented from entering the intake chamber. A disadvantage is that the intake ports must be sized the same as the fish screens, so the intake structure is larger. These screens are also directly exposed to frazil ice in the river and may become blocked if sufficient frazil freezes to the screens. The intake may need to be shut down during freeze-up to avoid frazil freezing to screens; however, it is not known how long this shut down would need to be since no observations of freeze-up are available at the site.

Tee-screens can be installed inside the intake chamber. These tee-screens consist of two drum shaped screens mounted on either side of a central pipe which is connected to the conductor pipe. Tee-screens 1.52 m in diameter and 3.6 m long would provide sufficient screen area for Burbot. The tee-screens should be oriented perpendicular to the port face to minimize the intake size. Another advantage of tee-screens is that the port area can be reduced; however, fish pumps will be necessary to remove the fish which cannot swim back out of the port. To limit the amount and size of fish which require pumping, it is recommended that the ports be sized to limit the approach velocity to 0.098 m/s which is the maximum velocity recommended for Walleye (Di Rocco and Gervais, 2021). The required port area is then reduced to 3.98 m², which accommodates fish diversion flows as well as water diversion flows. Debris can be

cleared from the tee-screen with backwash, but the debris may stay within the intake chamber so additional maintenance may be required. Frazil ice may enter the intake chamber but would tend to accumulate at the top of the intake chamber before blocking the screens.

A water or air backwash system should also be considered to provide frequent cleaning of the screens. A water backwash can be provided by adding piping and valves in the pumphouse to allow the water flow to be reversed. This system can be used with either type of screen. A separate air backwash system could be installed for the rectangular screens but may not be practical for the tee-screens.

5.2.4 Intake Port Geometry

The height of the port is limited to reduce the effects of both fluctuating bed levels from below and variable frazil ice levels from above. The bottom of the port should be at least 0.6 m above the existing local bed level to limit the effects of fluctuating bed levels. The top of the port should also be at least 0.6 m below the bottom of ice level. The depth of flow at the Option 2 location is sufficient to provide these offsets.

Option 2A has tee-screens placed in the intake chambers, so a port 1.3 m high by 3.1 m wide for each chamber would be required to provide sufficient area to satisfy the design velocity for Walleye. The bottom of the ports can be placed at 570.4 m which is 1.0 m above the river bed. The top of ports would then be at elevation 571.7 m which is 0.8 m below the 100-year minimum bottom of ice level. This port geometry would result in a total intake face length of 16.5 m, including a 3.0 m leading edge to allow flow to stabilize along the face.

If screens are placed directly behind the ports as in Option 2B, two ports 1.5 m high and 3.4 m wide for each chamber would provide sufficient area to satisfy the design velocity for Burbot. The bottom of the ports can be placed at 570.4 m which is 1.0 m above the river bed. The top of ports would then be at elevation 571.9 m which is 0.6 m below the 100-year minimum bottom of ice level. This port geometry would result in a total intake face length of 29.6 m, including a 2.0 m leading edge to allow flow to stabilize along the face.

Trash bars should be installed horizontally within the intake ports to prevent debris and ice floes from entering the intake. These trash bars should be coated with a non-stick coating to reduce the adhesion of frazil ice.

5.2.5 Intake Chambers

Intake chambers should be constructed between the intake ports and the conductor pipes to provide space for fish screens or conductor pipe entrances to the pumphouse and provide a working space for maintenance.

For Option 2A, the chambers must be wide enough and high enough to provide at least one-half of the tee-screen diameter all around the tee-screen. Four chambers, each 3.2 m wide by 3.2 m high would provide enough width for the port and sufficient space around the tee-screen. The chamber should be at least 8.4 m long to provide space for the flow to distribute around the screen and for the fish pump. The

fish pump should be located in the back of the chamber so that positive flow is maintained through the chamber to direct fish toward the pump.

For Option 2B, the four chambers must be wide enough to accommodate two ports in each chamber. Chambers 7.0 m wide by 3.0 m high would provide enough width for two ports in each chamber with a 0.2 m space between them. The chambers should be at least 4.0 m long to provide space for the flow to distribute around the entrance of the conductor pipe.

The bottom of the chambers should be below the bottom of the ports and the pipe inverts so that there is some capacity to trap sediment in the chamber. It may be useful to include a slide gate on the intake port so that the chamber can be isolated from the river if removal of sediment is necessary. An access hatch should be provided at the top of the structure to provide access to each chamber.

5.2.6 Conductor Pipes

River water would enter through the intake ports and chambers and be directed into conductor pipes that would convey flow to the pumphouse. The velocity in these pipes should be sufficient to prevent sediment from depositing in the pipes. Pipes with diameters of 457 mm (18 in) or less, each serving one chamber, would keep the velocity above 0.5 m/s which would be sufficient to prevent sediment deposition.

5.3 Intake Material

The intake could be constructed using steel sheet piles or concrete or a combination of the two materials.

5.3.1 Sheet Piles

If the intake structure is constructed with sheet piles, the sheet piles could also be used to isolate the construction area from the river. The sheet piles could also be driven deep enough to protect against bed scour on the outside of the structure; however, the presence of bedrock may limit the depth that the sheet piles can be driven. It would be somewhat more difficult to construct the port in the sheet pile because the surface is corrugated. The sheet pile structure would be filled with granular material and capped with riprap.

5.3.2 Concrete

If the intake structure is constructed with concrete, the construction area will need to be isolated from the river with sheet piles or a berm. A concrete structure has the advantage of having a smoother outer wall and the intake port and chamber can be easily formed. The remaining volume of the structure can be filled with granular material and capped with riprap or even filled with earth and capped with concrete. If a concrete wall structure is selected, it would need to be protected from undermining with a riprap or concrete apron along its outside perimeter.

5.4 Pumphouse

A pumphouse will be located near the bank of the river. The pumphouse floor should be located at an elevation greater than 579.2 m so that it remains dry during the 100-year open water and ice jam events. This elevation is 9.8 m and 10.0 m above the bottom of intake chamber for options 2A and 2B respectively. Given the uncertainties in estimating ice jam flood levels, it is recommended that the pumphouse floor be placed at an elevation of at least 581.2 m to provide 2.0 m of freeboard above the 100-year ice jam water level.

5.5 Erosion Protection

Erosion protection may be required around the intake structure if the bed and banks are not composed of non-erodible rock. This erosion protection may consist of scour protection at the base of the structure and/or bank protection on the banks upstream and downstream of the structure. If required, the scour and bank protection may be constructed using concrete walls or rock riprap.

Detailed design of this erosion protection will be carried out in later stages of the intake design. It is recommended that a two-dimensional hydraulic model be developed to provide more accurate local velocity estimates around the intake. A preliminary estimate of 100-year peak local velocity of 6.1 m/s at the base of the bank with a depth of 10.1 m was obtained from the one-dimensional hydraulic model. Class 3 rock riprap with a median diameter about 800 mm would be required for these hydraulic conditions (**Figure 31**). The rock should extend a short distance upstream and downstream of the intake on the banks. Riprap may also be required on the bank above the intake, but smaller rock is likely sufficient at this location.

Erosion protection is also recommended on the opposite bank upstream of the access road to the boat launch (**Figure 31**). A preliminary estimate of the 100-year peak local velocity obtained from the one-dimensional hydraulic model at this location is 2.0 m/s. Class 1 riprap with a median diameter of 300 mm would be sufficient at this location. The erosion protection could consist of a riprap revetment along the 300 m long bank, or a number of short spurs spaced evenly along the bank.

6 CONCLUSIONS

The results of the hydrotechnical analysis and design resulted in recommendations for the conceptual design of the intake as well recommendations for further detailed hydrotechnical design which was not part of the scope of the conceptual design.

6.1 Recommendations for Conceptual Design

1. The intake should be sited at the Option 2 location.
2. The intake should be a bank intake.
3. The intake should project out from the bank sufficiently so that the intake face is located in the scour hole.
4. Top of new intake should be above the 2-year peak water level to provide better access and local scour and should be smooth enough not to collect debris.

5. The new intake should be wedge shaped to provide better hydraulic conditions at the port.
6. Intake may be constructed with sheet pile or concrete walls depending on cost and constructability.
7. Both Option 2A and 2B port and fish screen geometries are feasible so option selection should depend on cost (size vs complexity). Option 2A may be better for frazil ice but option 2B is better for backflushing.
8. A water or air backflushing system should be designed to clean the screens.
9. Access hatches on the top of the structure should be provided for each intake chamber.
10. Four 457 mm conductor pipes should be used to convey water to the pumphouse
11. Pumphouse floor should be above the 100-year open water and ice jam elevation of 579.2 m.
12. Erosion protection should be provided around the intake structure and on the opposite bank.

6.2 Recommendations for Further Hydrotechnical Design

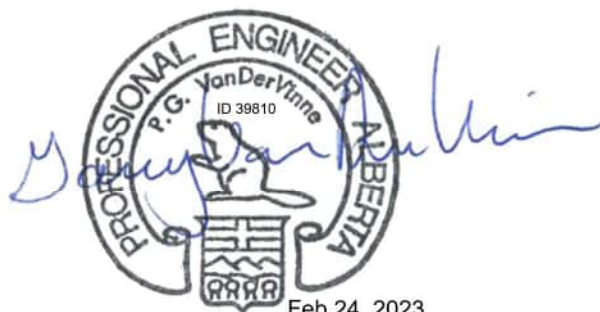
1. Scour protection at the base of the structure should be designed.
2. Erosion protection for the bank should be designed.
3. Erosion protection of opposite bank should be designed.
4. A two-dimensional hydraulic model should be developed to provide local hydraulic conditions for the above designs.
5. More detailed bathymetry should be collected in the vicinity of the intake structure to define the shape and extents of the scour hole.
6. Ice forces on the structure should be determined.
7. Inspections of mid-winter and breakup ice conditions should be carried out.
8. Fish screening detailed design should be developed to establish mesh size, panel size and method of cleaning and replacement.
9. Method for desilting the intake chamber should be determined.
10. Hydraulic effects of construction isolation should be investigated.

7 CLOSURE

Sincerely,

Northwest Hydraulic Consultants Ltd.

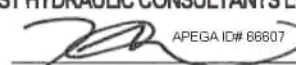
Report prepared by:



Feb 24, 2023

Gary Van Der Vinne, M.Sc., P.Eng.
Principal

Report reviewed by:

PERMIT TO PRACTICE	
NORTHWEST HYDRAULIC CONSULTANTS LTD.	
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Date	24 Feb 2023
PERMIT NUMBER: P 654	
The Association of Professional Engineers and Geoscientists of Alberta	

Robyn Andrishak, M.Sc., P.Eng.
Principal

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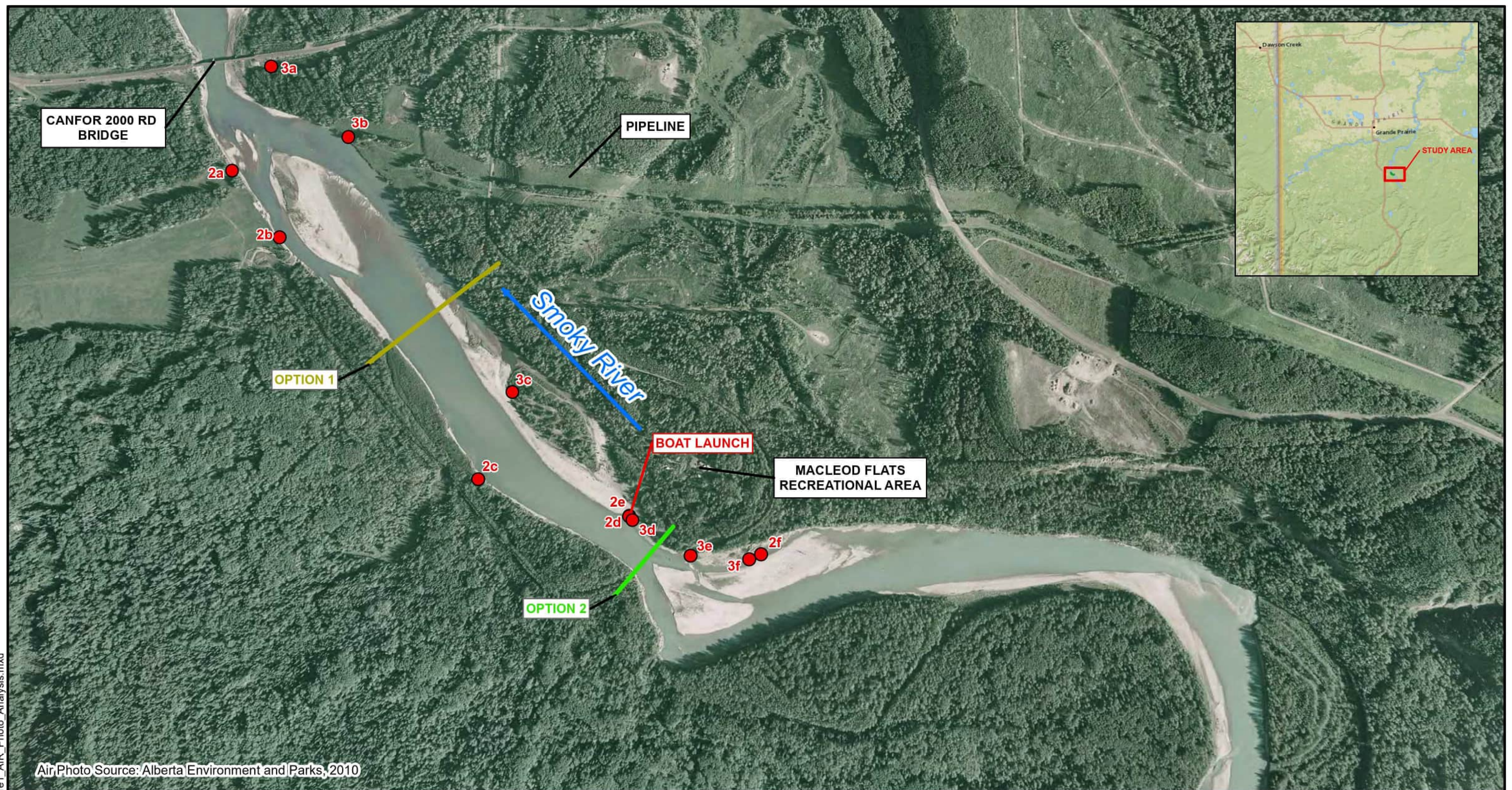
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8 REFERENCES

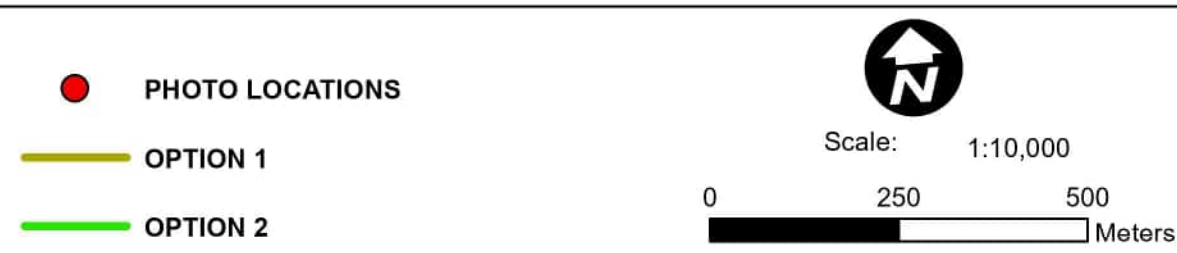
Di Rocco, R, and R. Gervais. 2021. SPOT: Swim Performance Online Tools. Available from <http://www.fishprotectiontools.ca/>

FIGURES

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Air Photo Source: Alberta Environment and Parks, 2010



Projection: WGS_84_UTM_zone_11N

	MUNICIPAL DISTRICT OF GREENVIEW		
	GREENVIEW INDUSTRIAL GATEWAY WATER INTAKE		
	PRELIMINARY HYDROTECHNICAL DESIGN		
	1007352	FEB 2023	Figure 1



a) View looking downstream at eroding bank between bridge and pipeline right-of-way.



b) View of fine sand and small riprap at tributary mouth.



c) View of large sand bar covered with small vegetation and tree trunks projecting out of the bank.



d) View looking at rock outcrop on left bank opposite the boat launch.



e) View of left bank downstream of rock outcrop from boat launch.



f) View of posts sticking out of water looking upstream of rock outcrop from boat launch.



a) View looking upstream at eroding bank between bridge and pipeline right-of-way.



b) View looking upstream at rock spurs at pipeline right-of-way.



c) View of entrance of recently formed side channel downstream of large island.



d) View looking downstream at large gravel bar upstream of large island at boat launch.



e) View looking upstream at eroding threatening access road for boat launch.



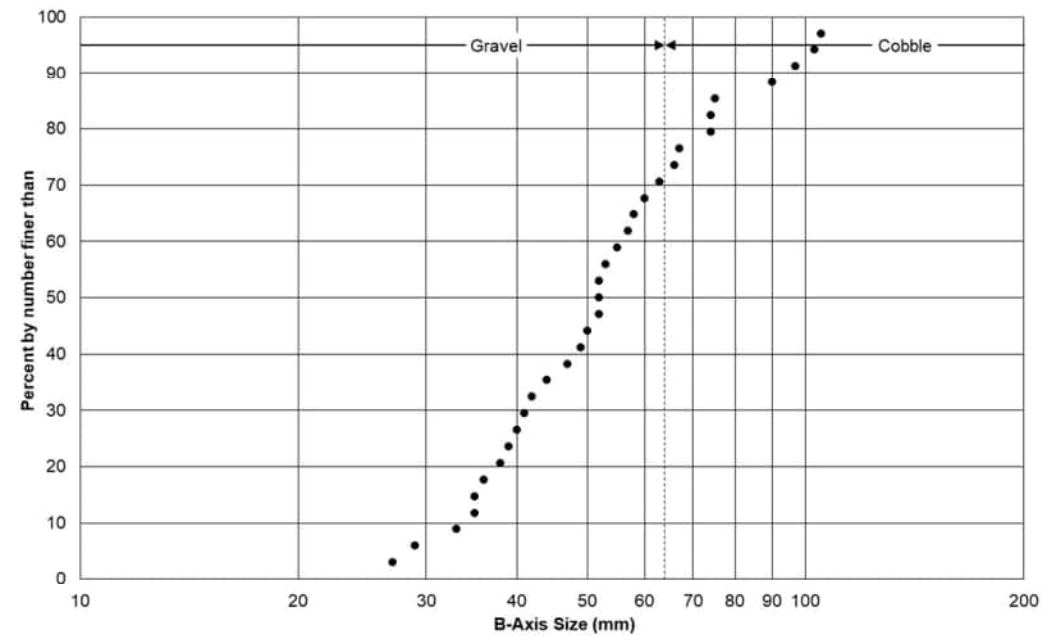
f) View of rock outcrop at the base of the eroding bank upstream of the recreation area

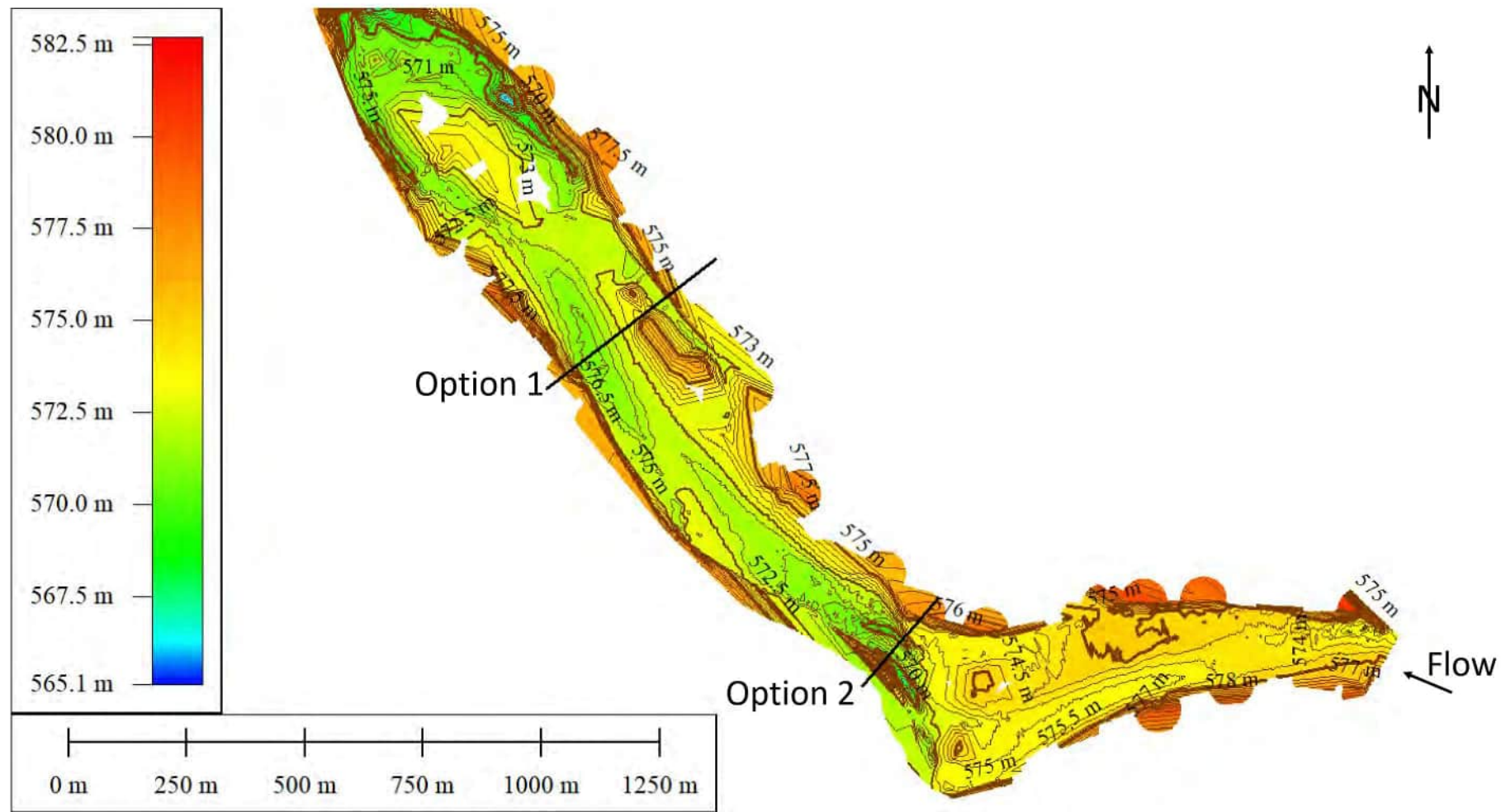


a) View of tape sample site on left bank looking upstream



d) View on bed material at tape sample site.
(Black notebook dimensions are 319 mm x 236 mm)



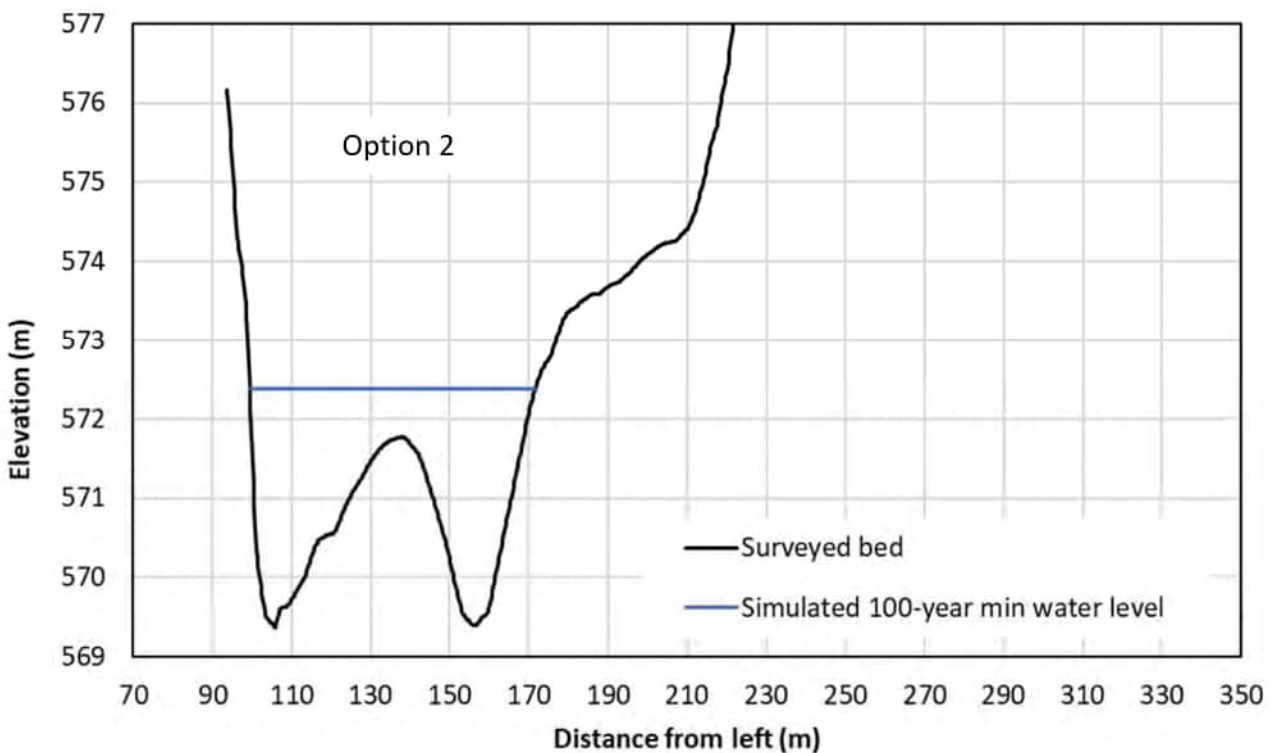
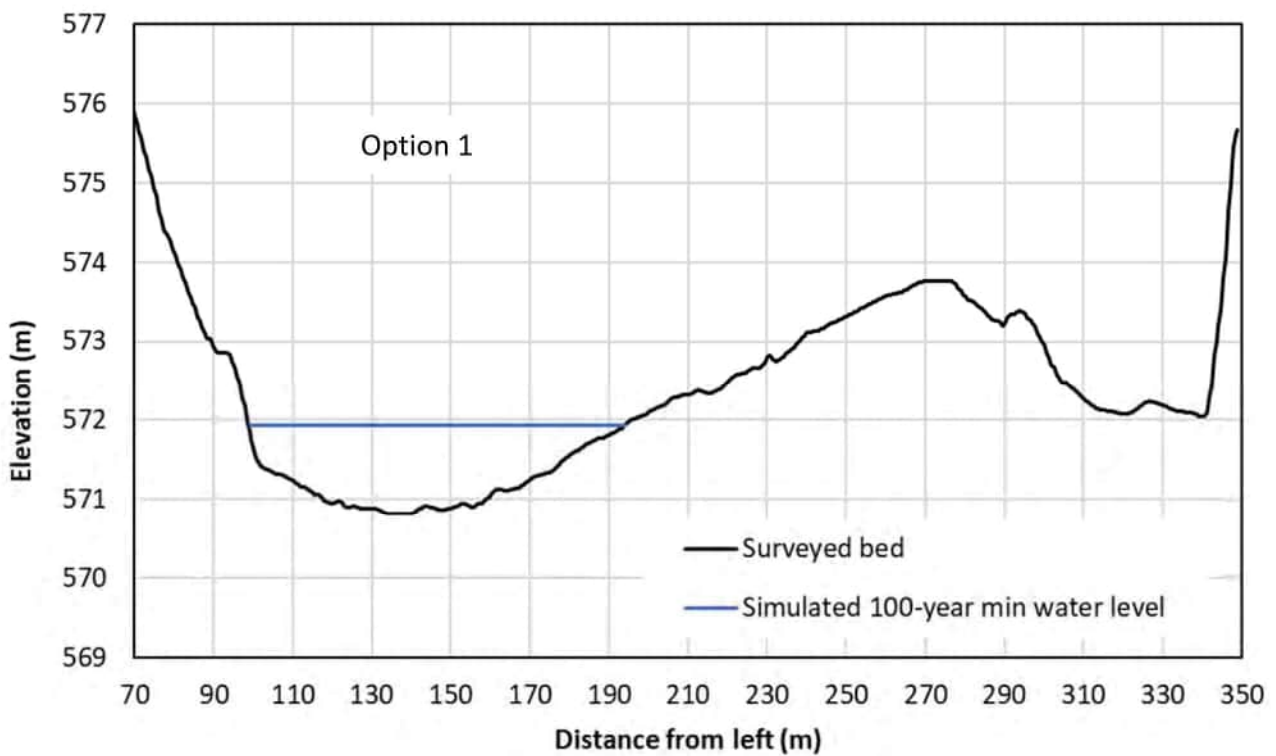




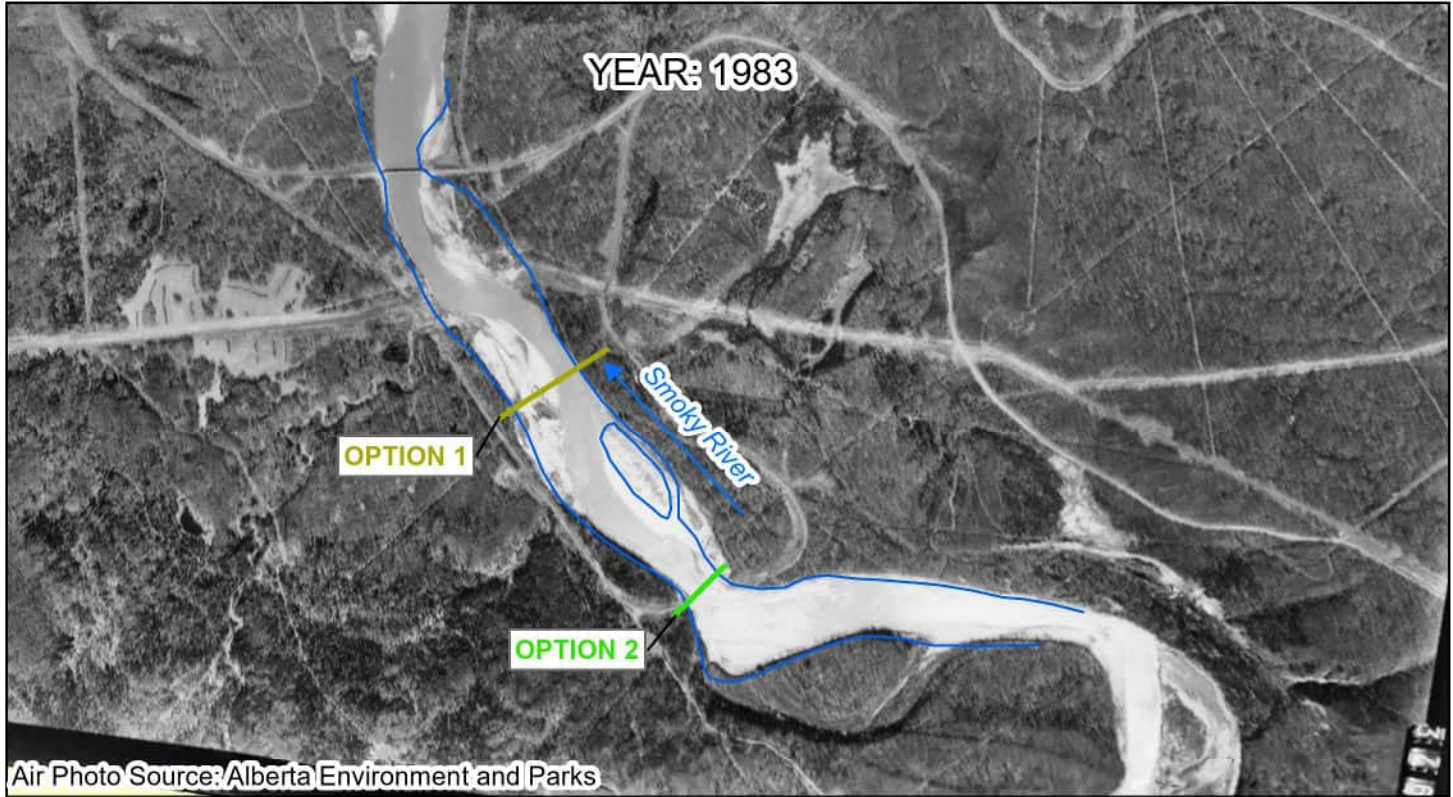
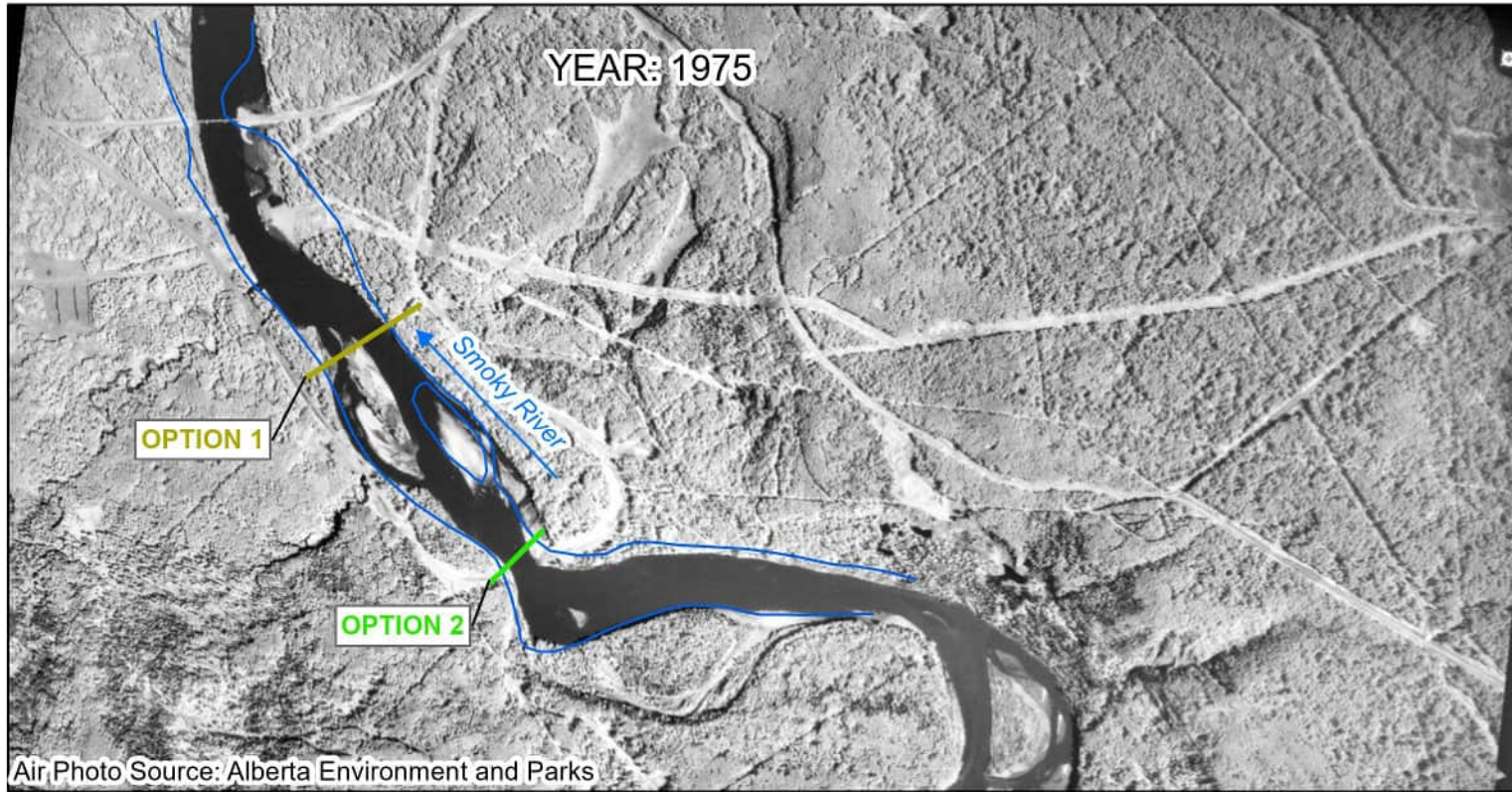
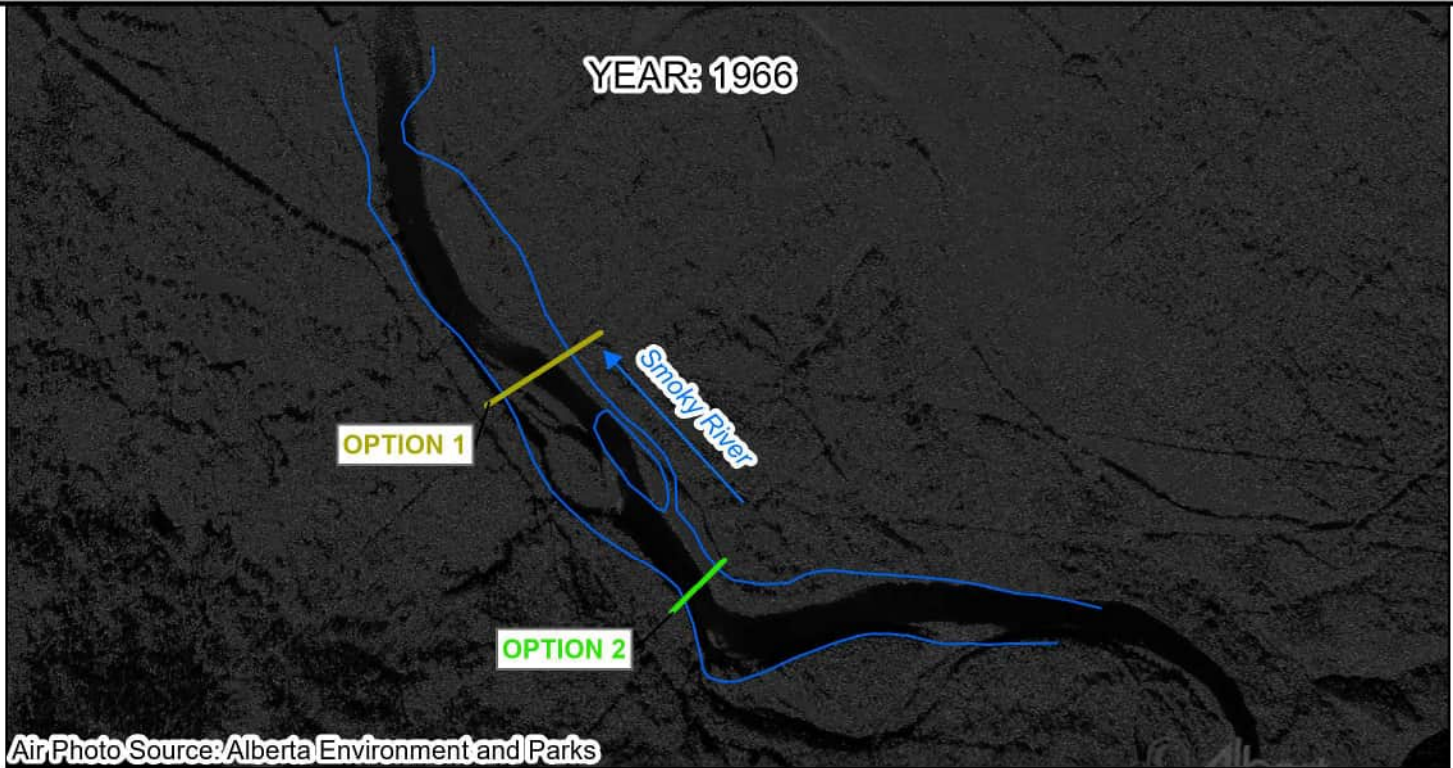
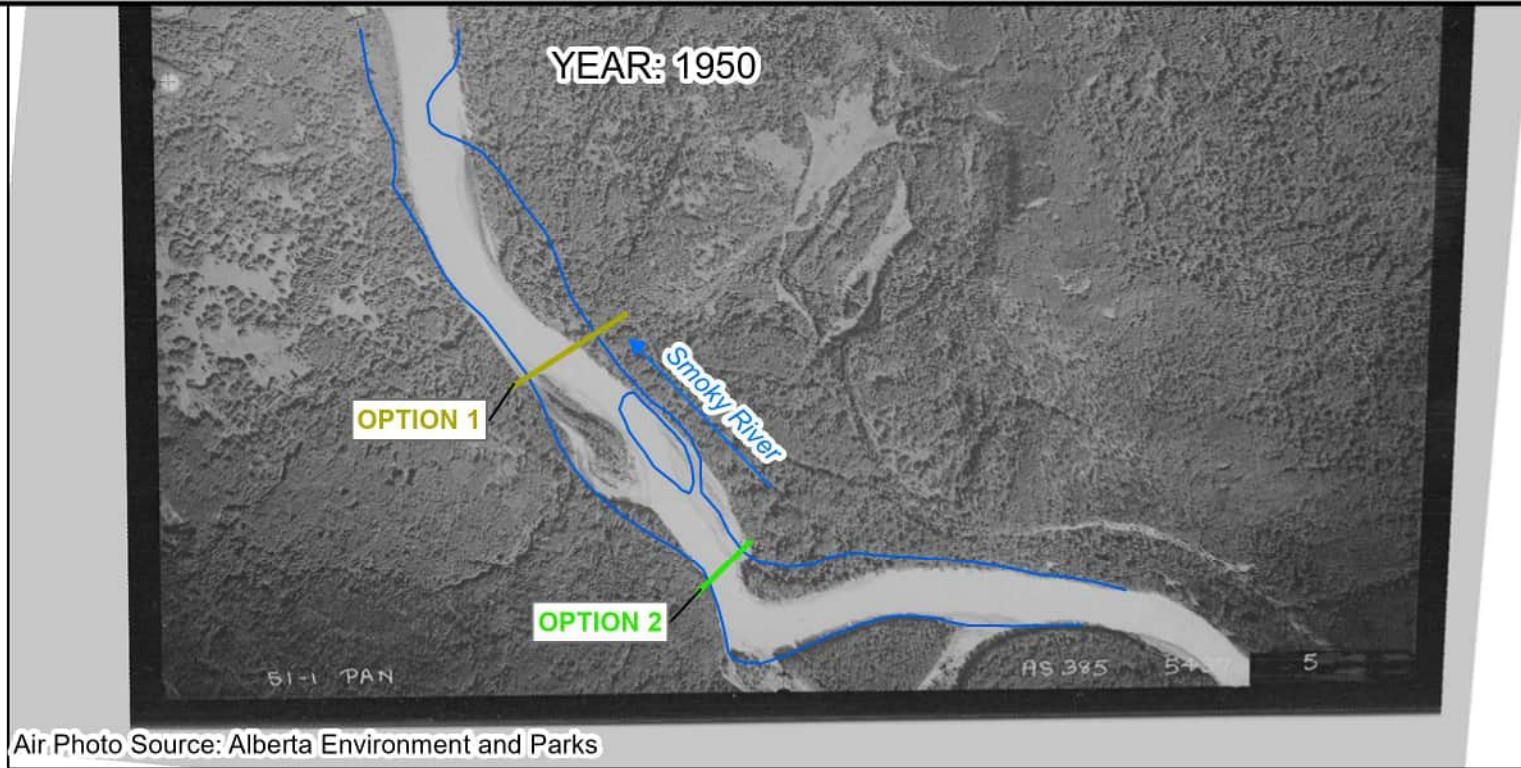
a) View of left bank at Option 1 location



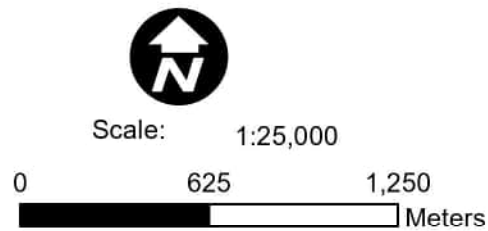
a) View of left bank at Option 2 location



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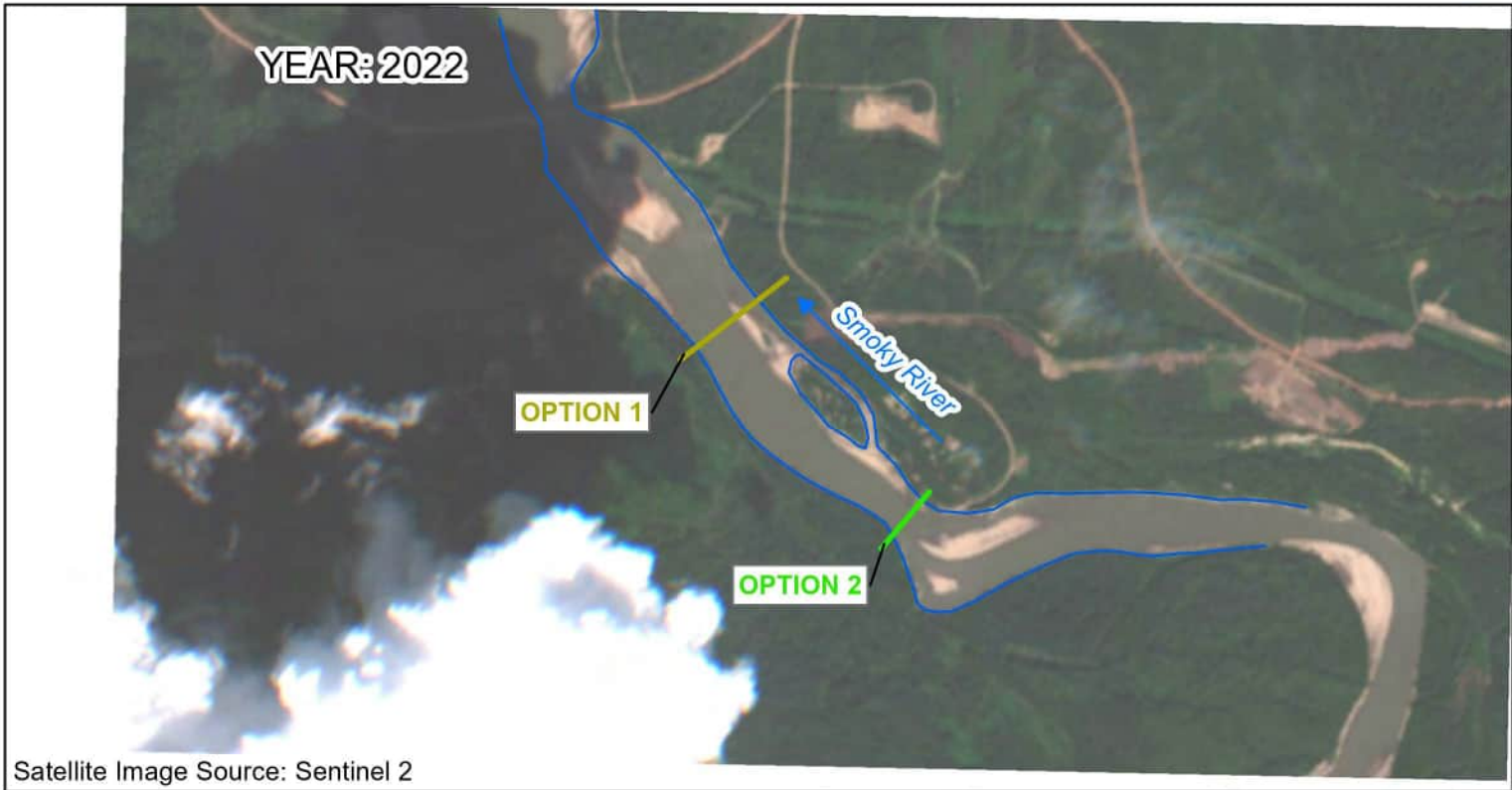
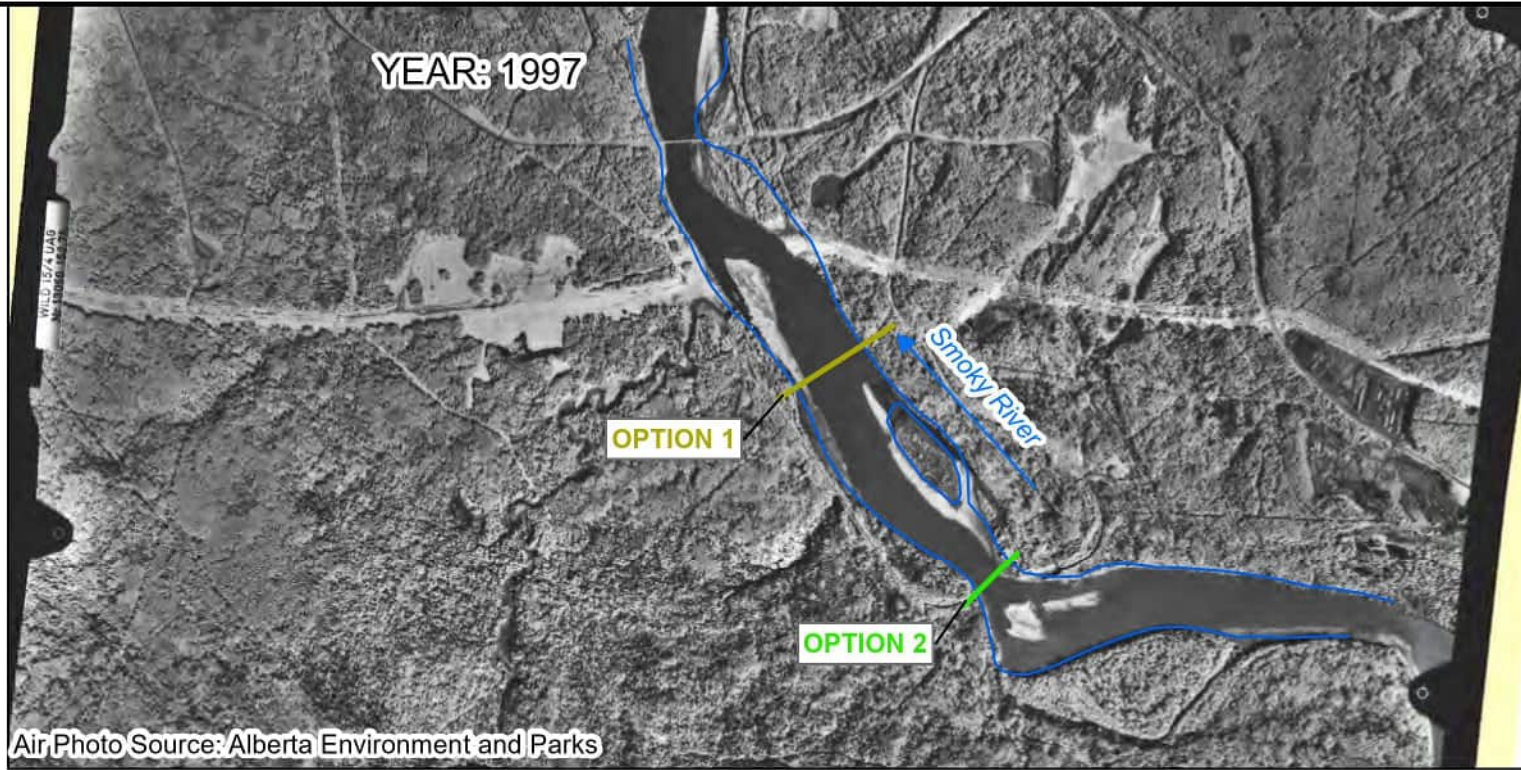
- Option 1
- Option 2
- 2022 Bank Lines



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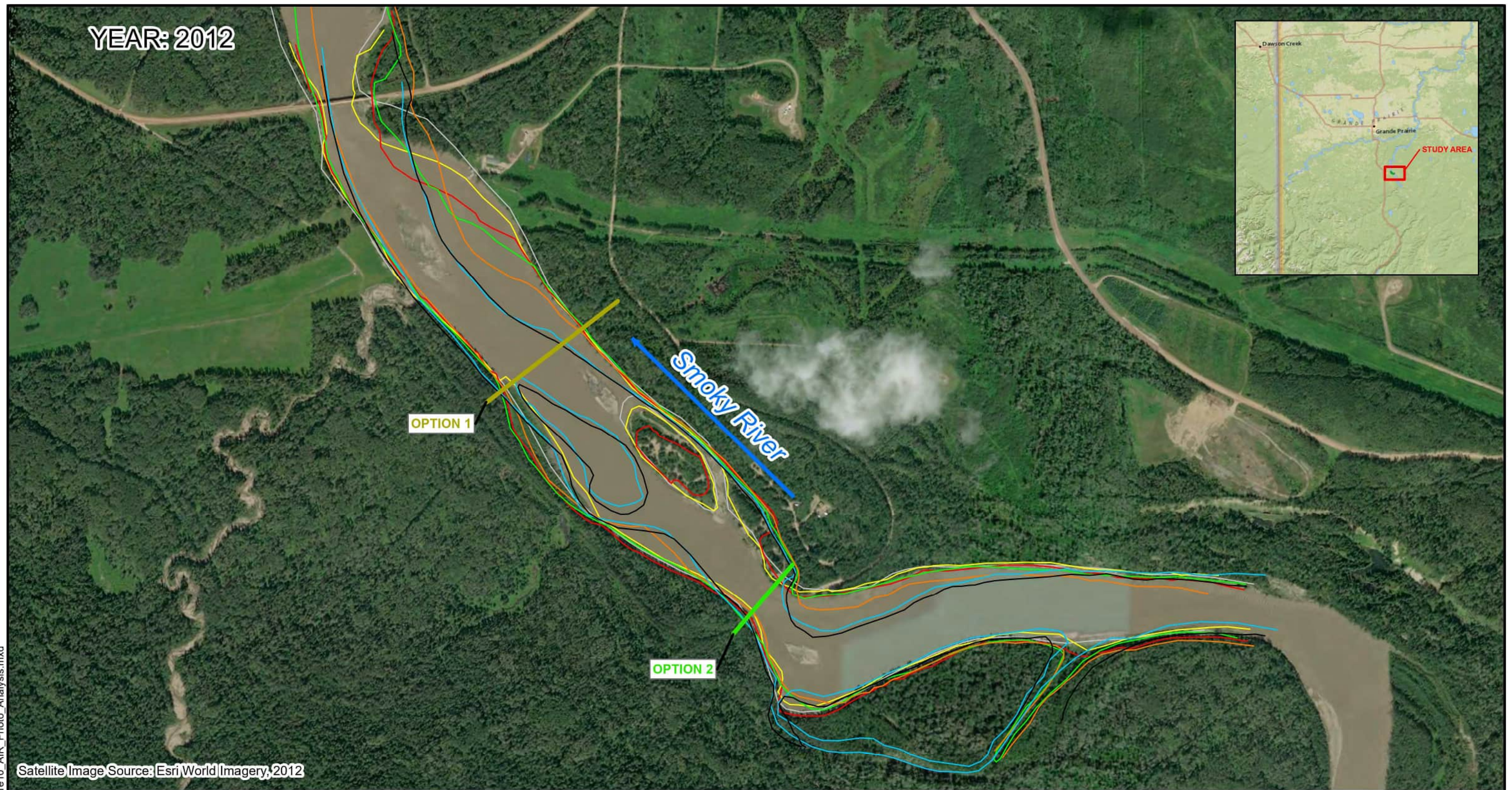
MUNICIPAL DISTRICT OF GREENVIEW		
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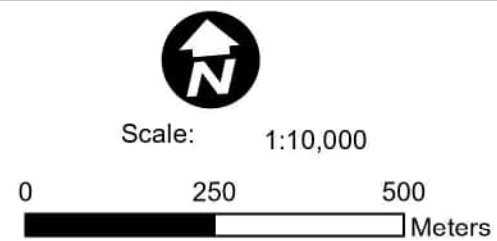
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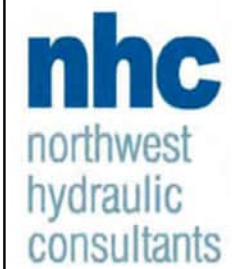


- 1950 Bank Lines
- 1966 Bank Lines
- 1975 Bank Lines
- 1983 Bank Lines
- 1997 Bank Lines
- 2010 Bank Lines
- 2022 Bank Lines

- OPTION 1
- OPTION 2



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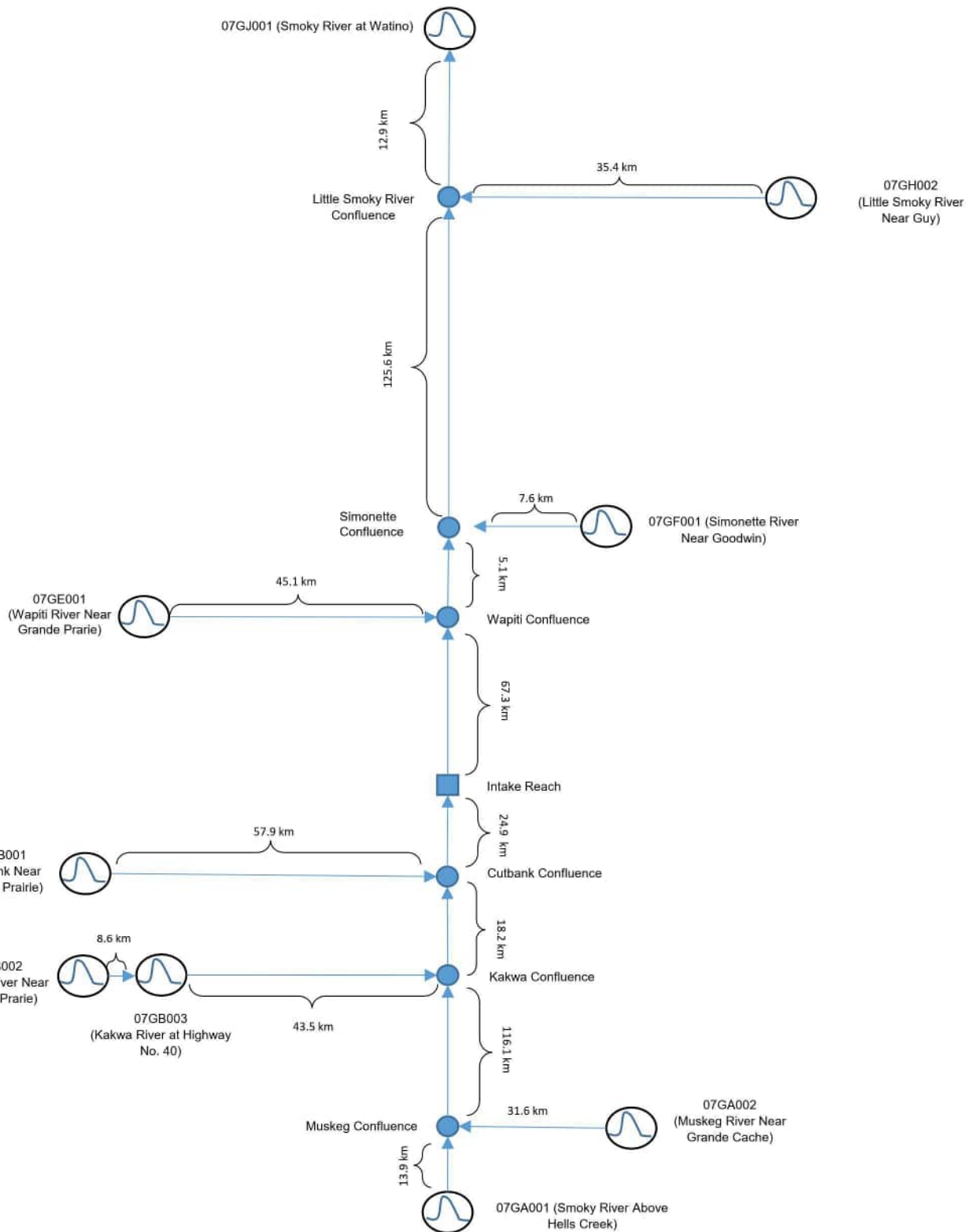
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WATER INTAKE

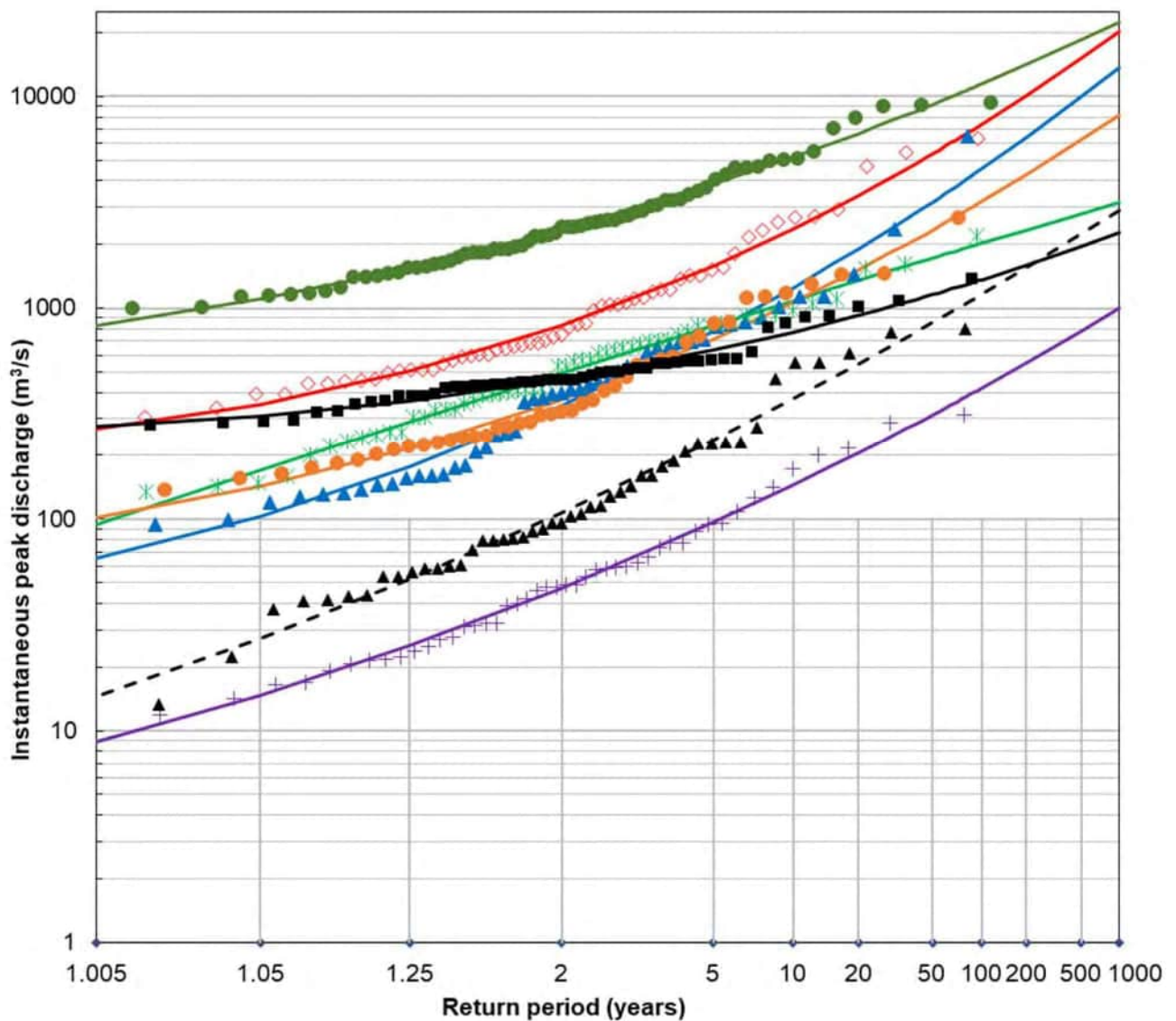
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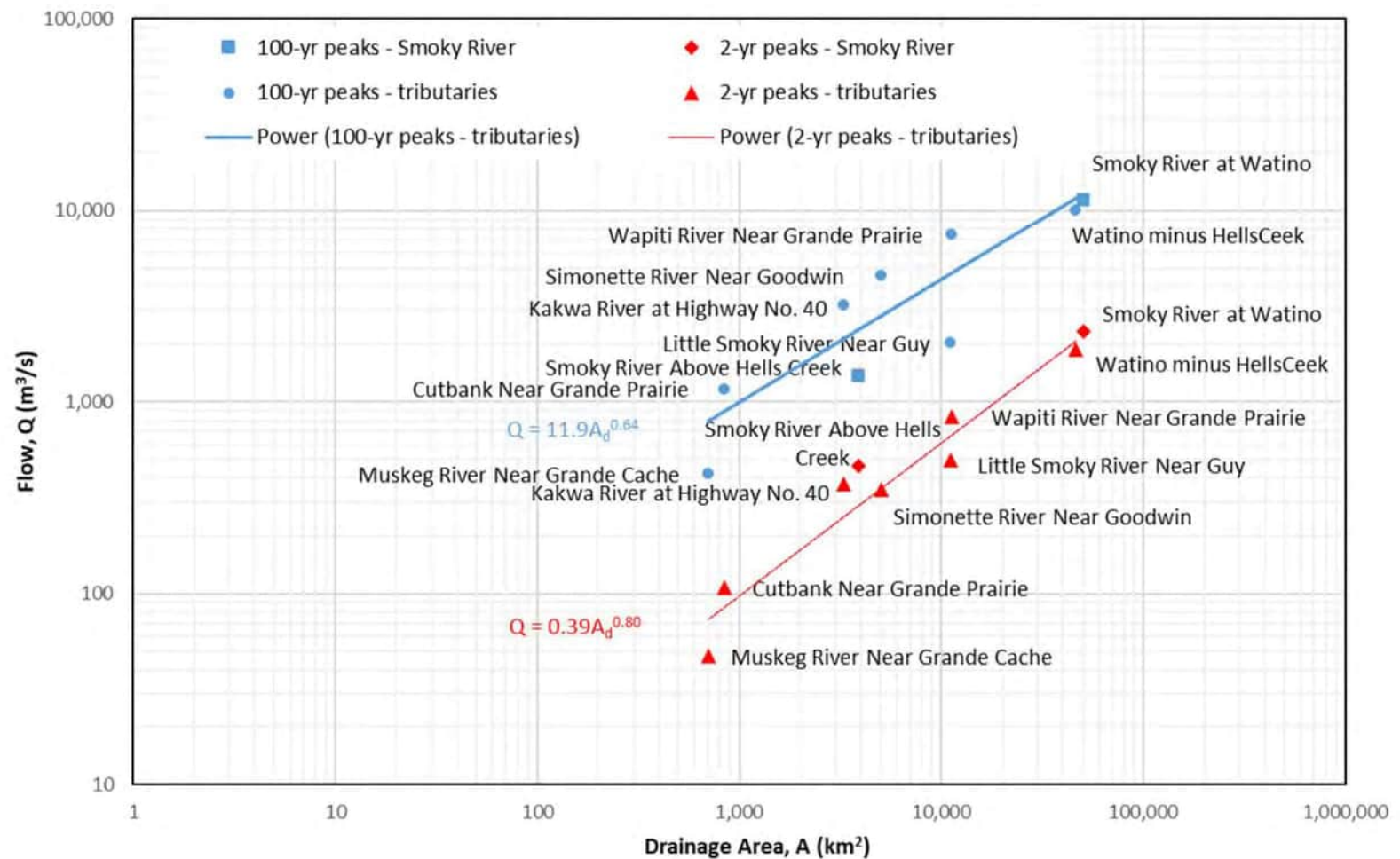
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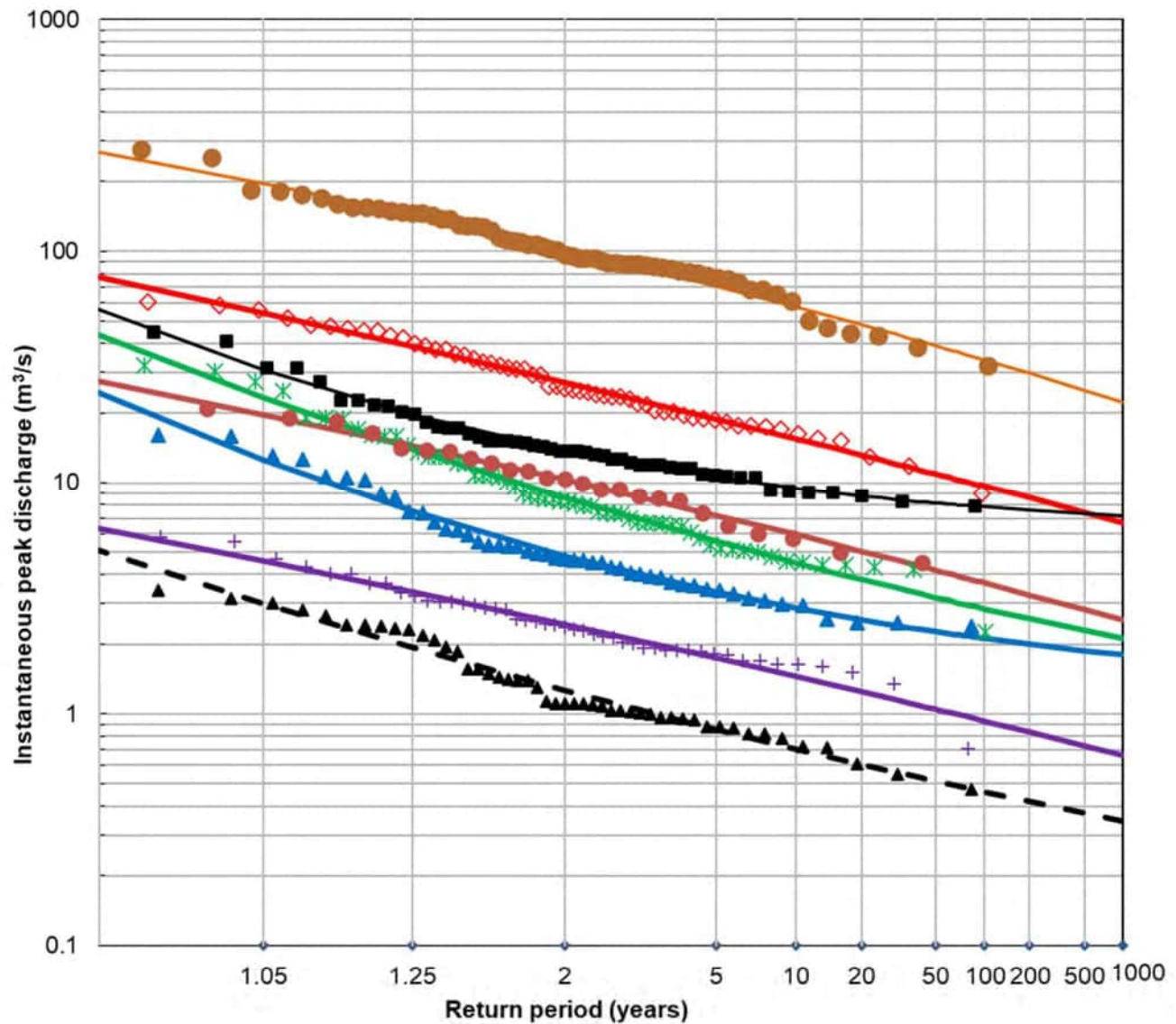
Figure 10



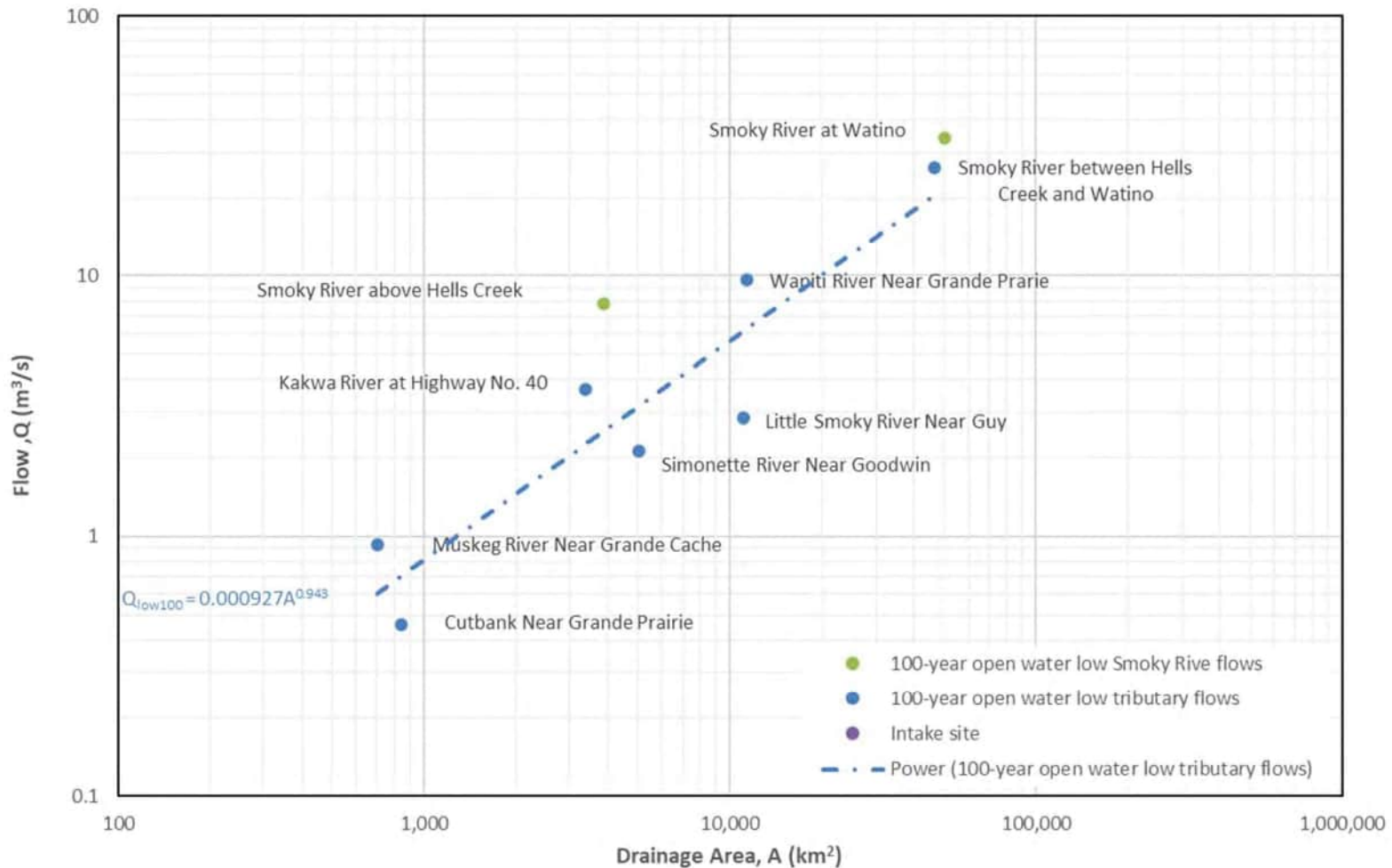


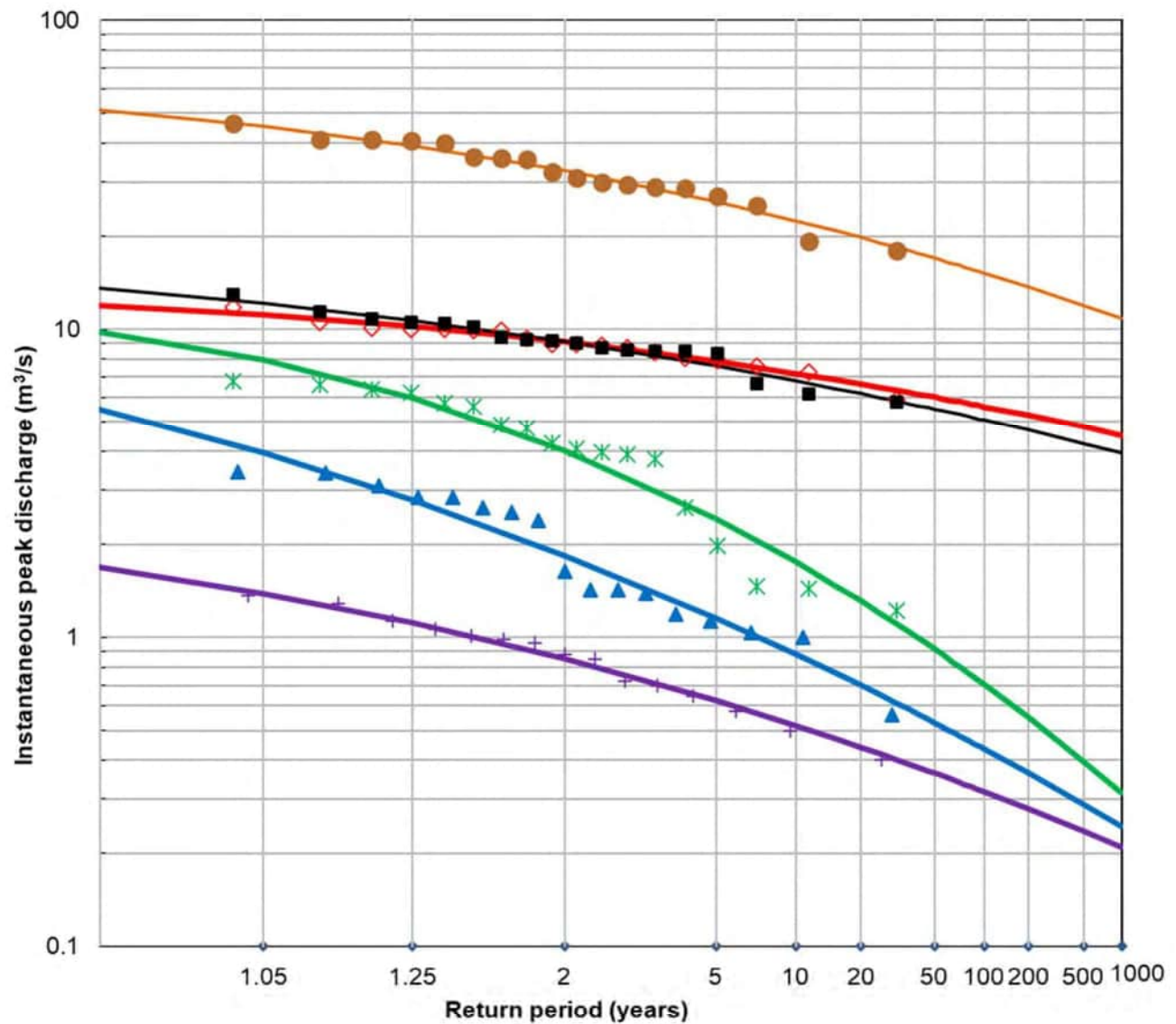
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|------------------------------------|-----------|
| ● Smoky River at Watino | — LP3 |
| ◇ Wapiti River Near Grande Prairie | — LP3 |
| ▲ Simonette River Near Goodwin | — LP3 |
| + Muskeg River Near Grande Cache | — LP3 |
| * Little Smoky River Near Guy | — LP3 |
| ● Kakwa River at Highway No. 40 | — LP3 |
| ■ Smoky River Above Hells Creek | — LP3 |
| ▲ Cutbank Near Grande Prairie | - - - LP3 |



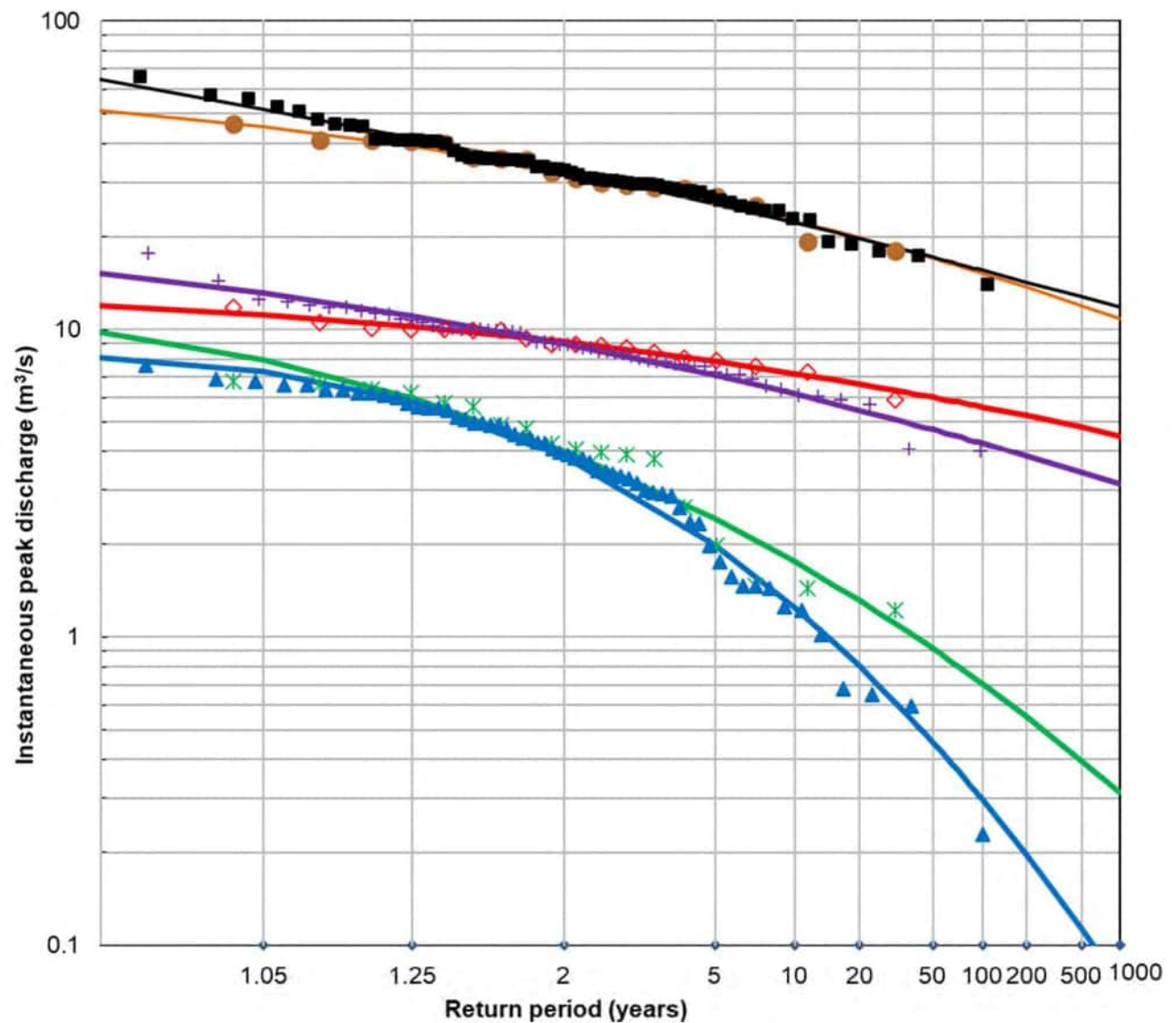


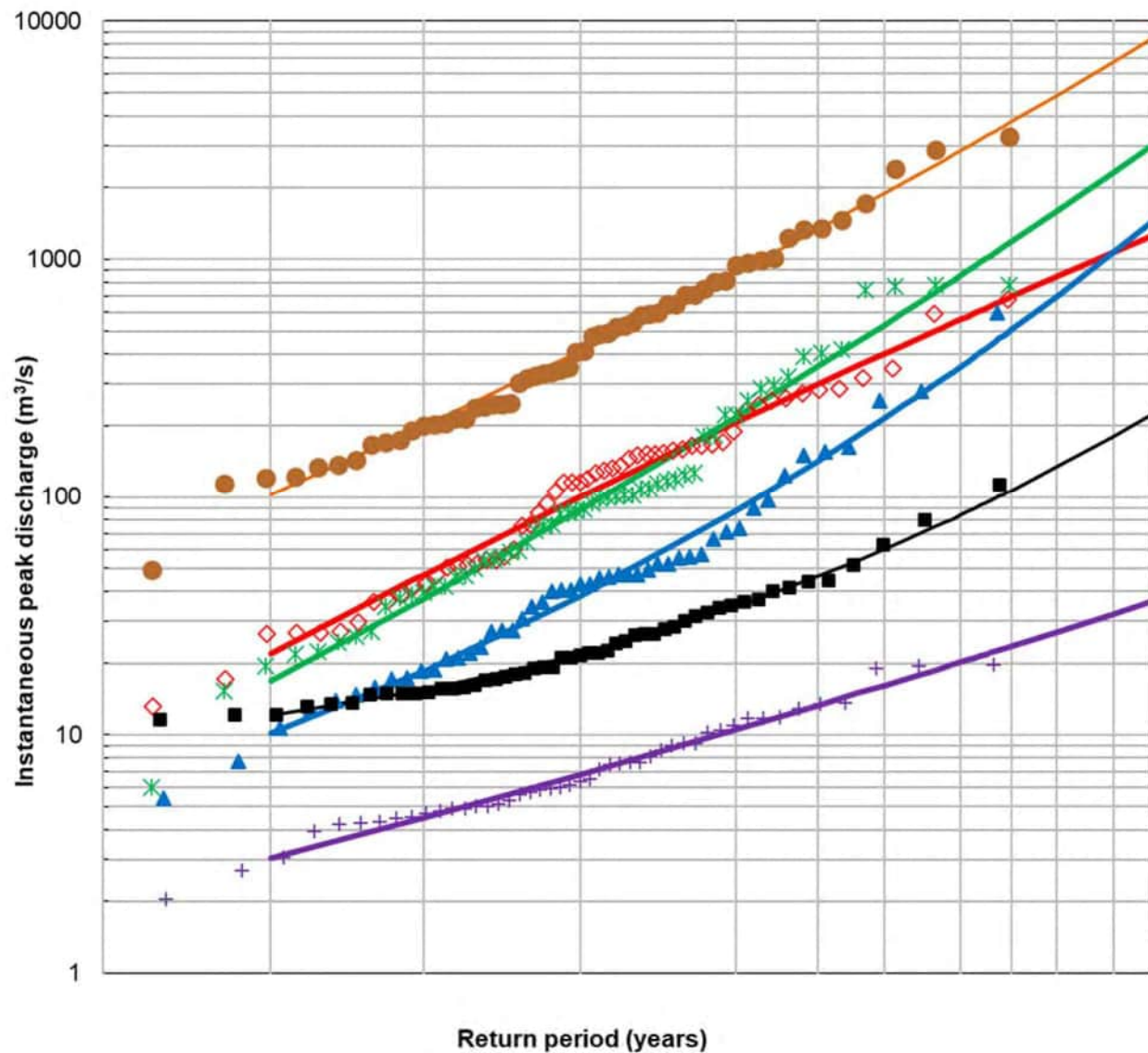
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|-------------------------------------|-------|
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| ◇ Wapiti River near Grande Prairie | — LP3 |
| ▲ Simonette River near Goodwin | — LP3 |
| + Muskeg River near Grande Cache | — LP3 |
| ✕ Little Smoky River near Guy | — LP3 |
| ● Kakwa River at Hwy 40 | — LP3 |
| ▲ Cutbank River near Grande Prairie | — LP3 |
| ■ Smoky River above Hells Creek | — LP3 |

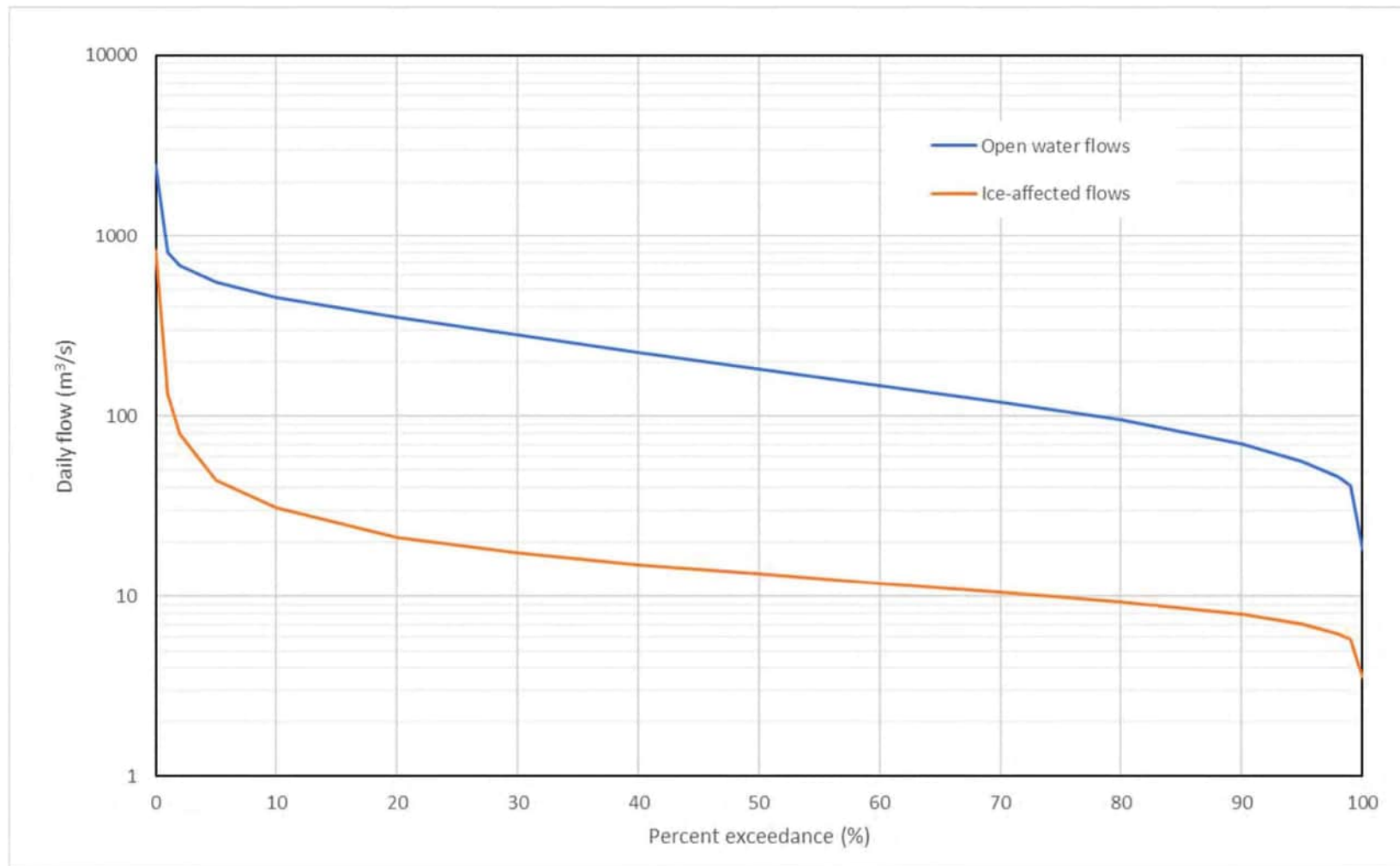




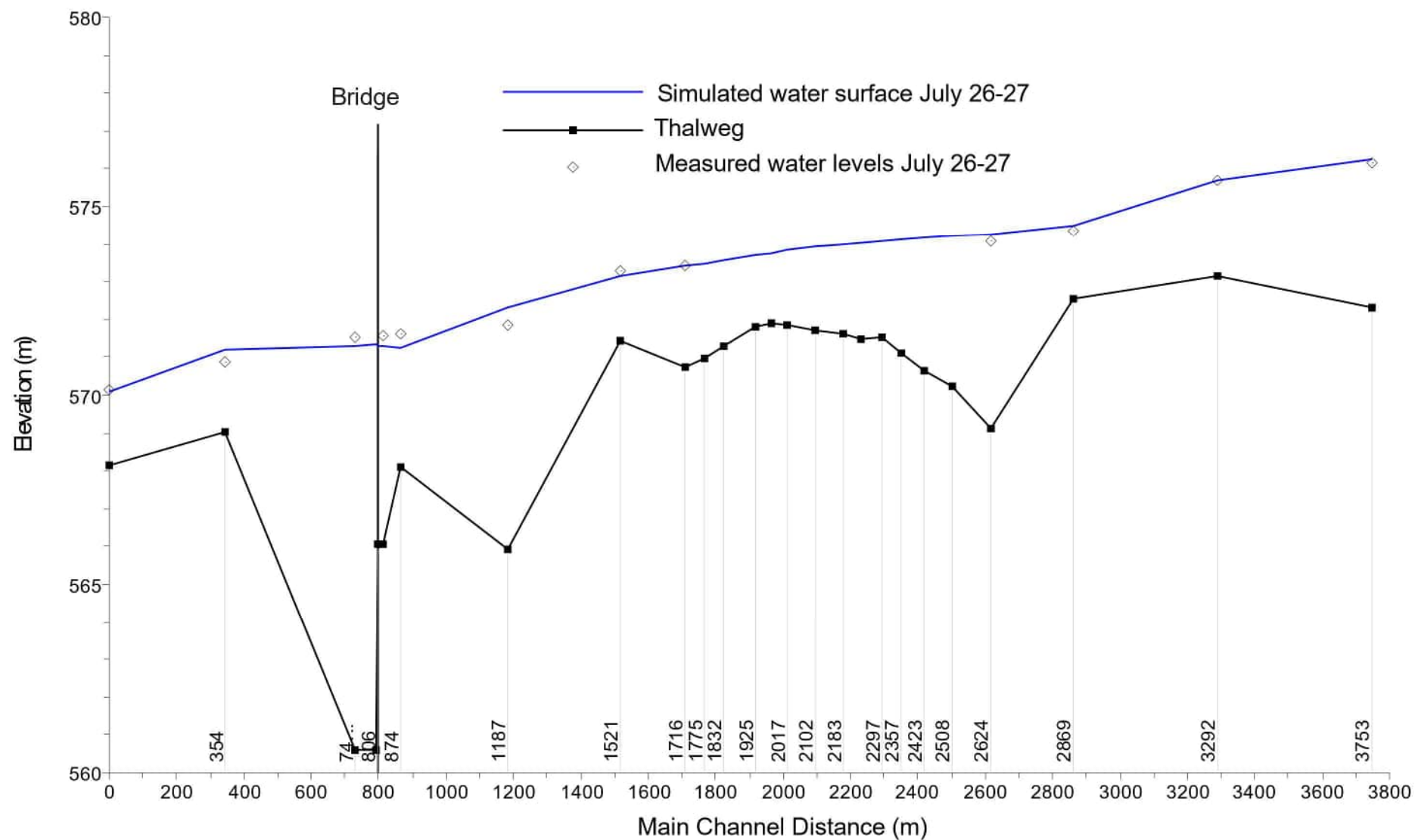
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- * L.Smoky (1986) — LP3
- ▲ Simonette — LP3
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- Hells Creek — LP3

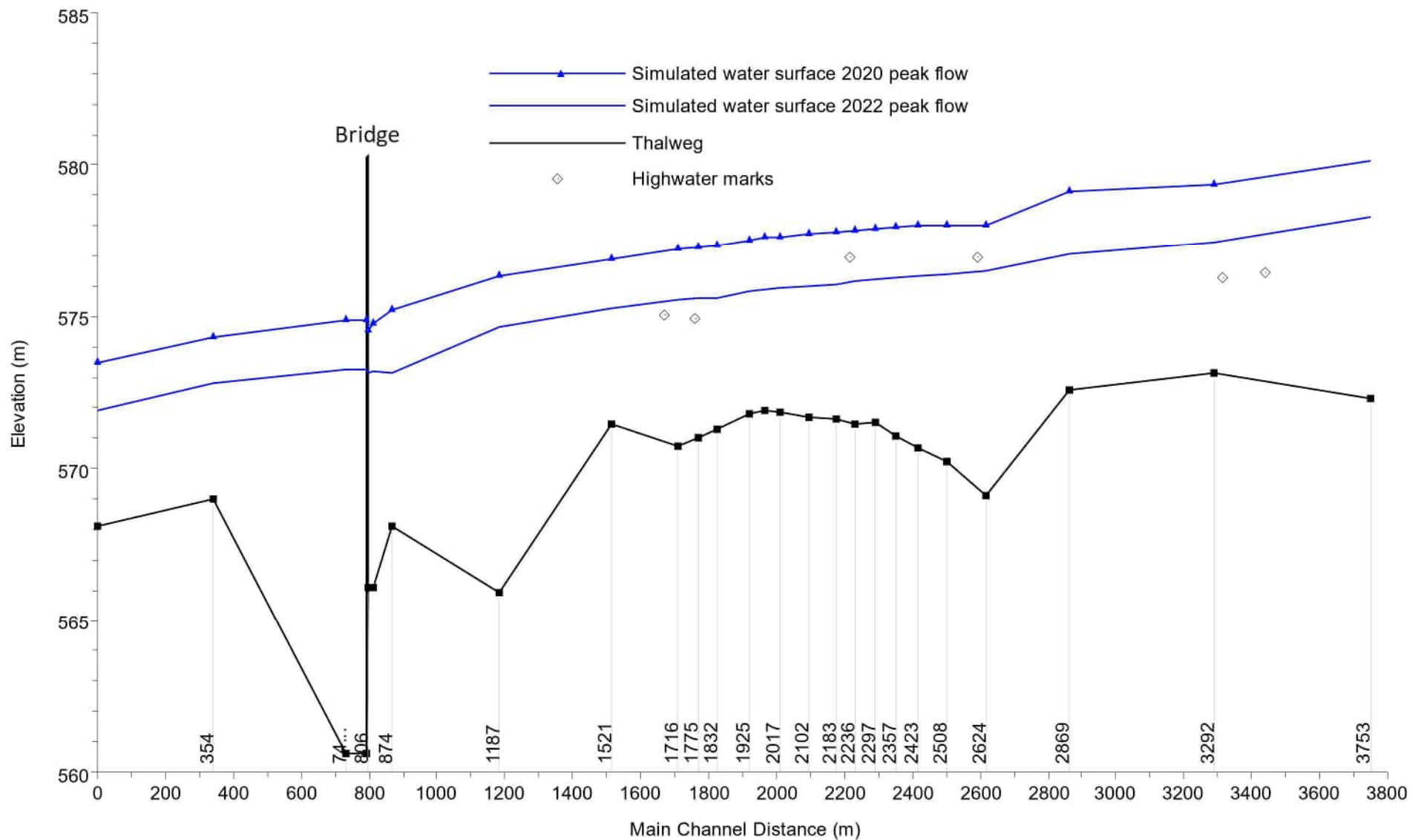


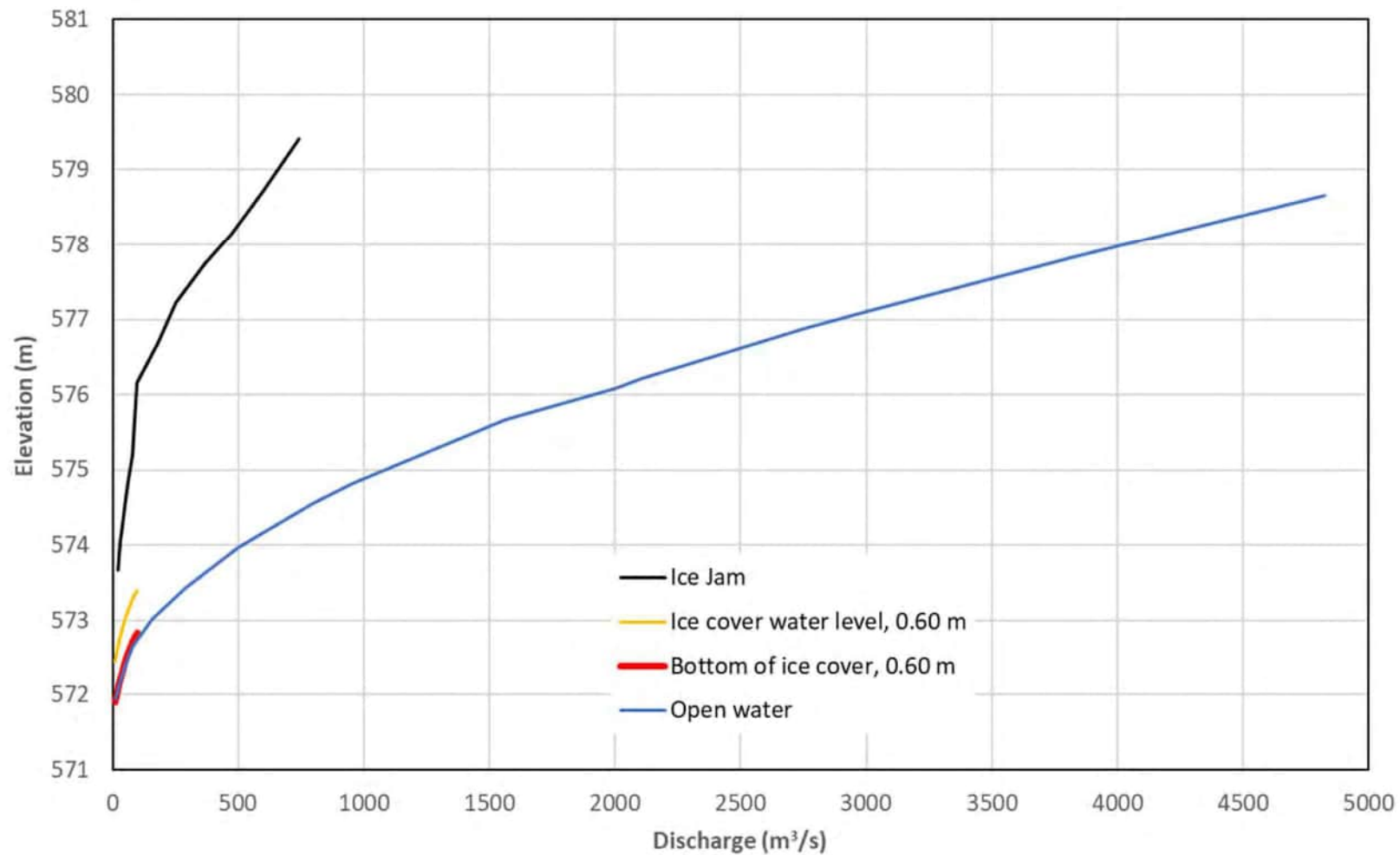


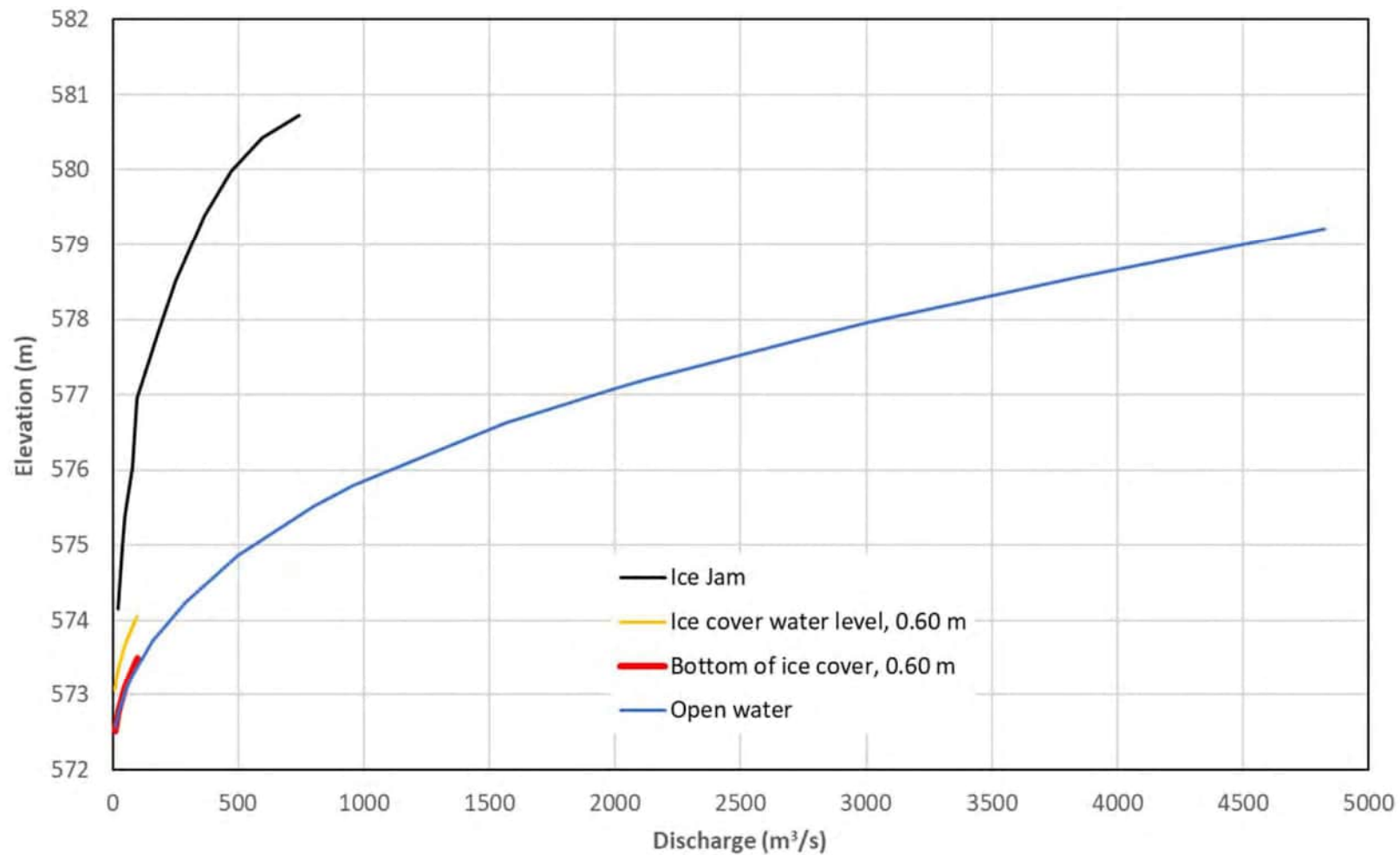


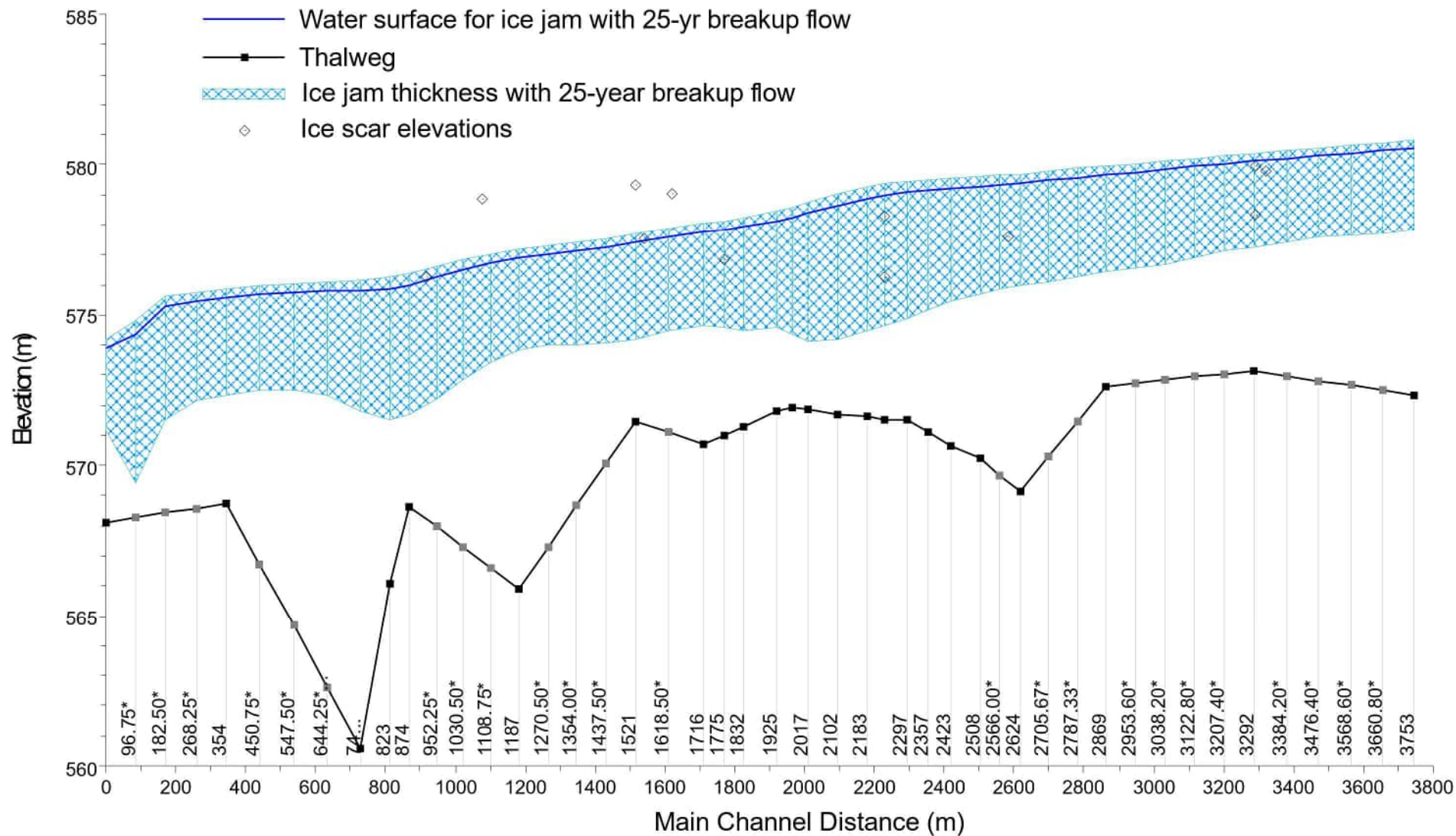


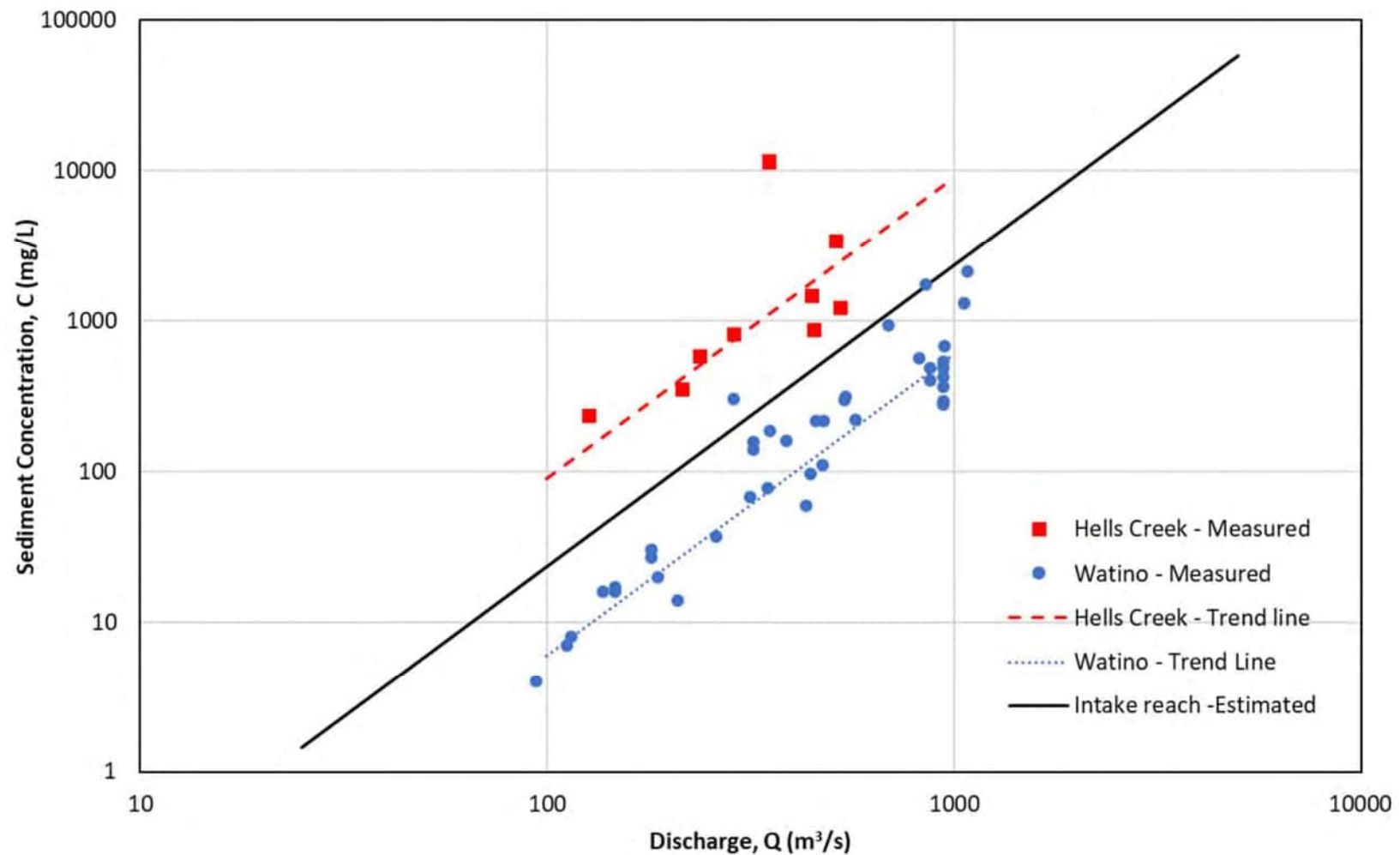




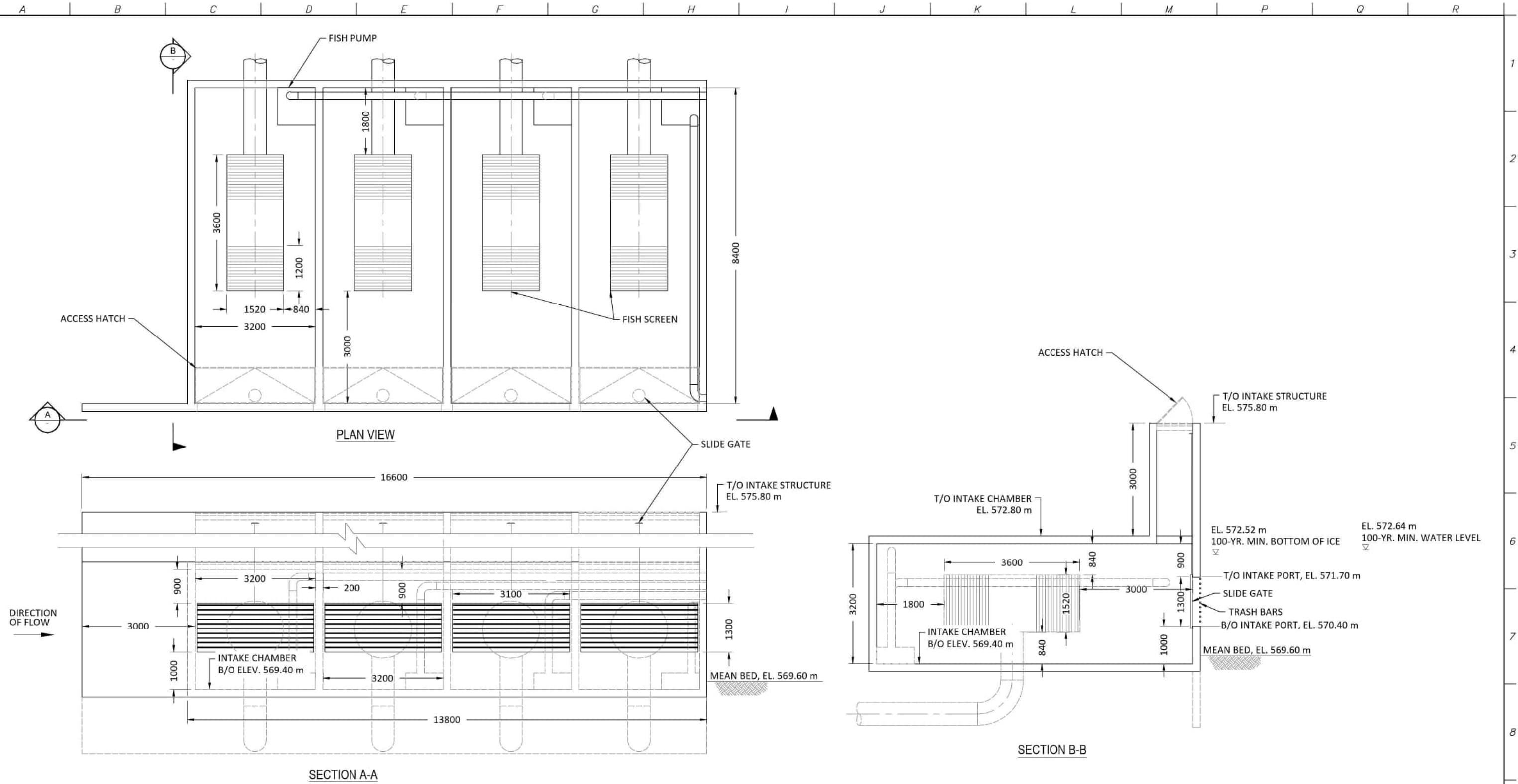








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9819-12 Avenue SW
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REVISIONS		
1	Nov. 9, 2022	ISSUED FOR INTERNAL REVIEW

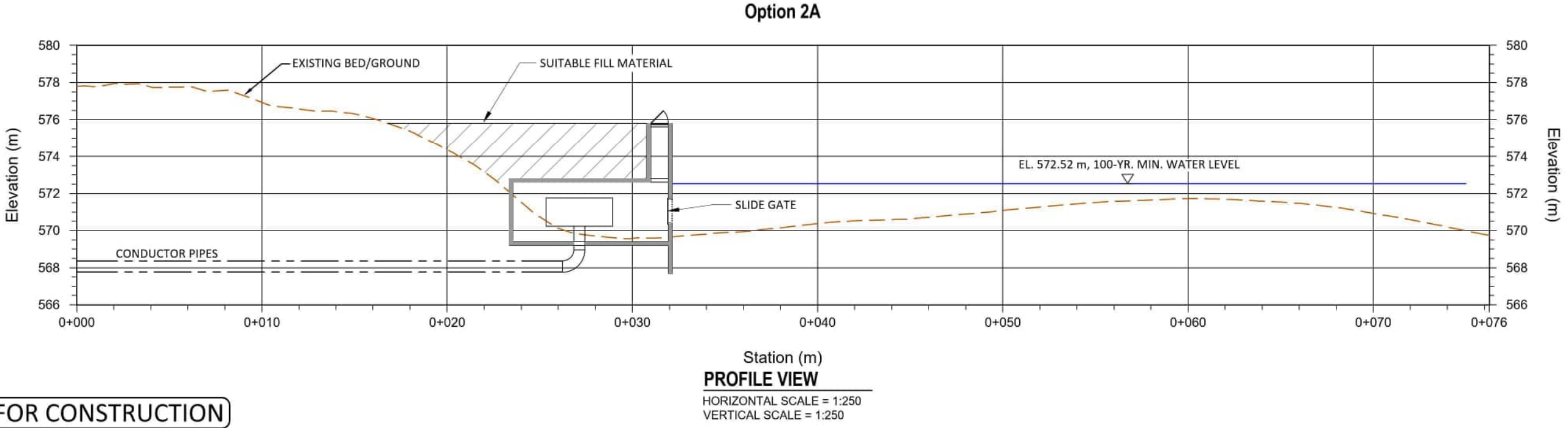
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DATE	23 Feb 2023
DESIGNED BY	MB
DRAWN BY	DL
CHECKED BY	PGV
SHEET SIZE	B (11" x 17")

**GREENVIEW INDUSTRIAL GATEWAY
WATER INTAKE
PRELIMINARY HYDROTECHNICAL DESIGN**

INTAKE GEOMETRY
OPTION 2A

NHC NUMBER	1007352
DRAWING NUMBER	1007352-003
FIGURE NUMBER	27
REVISION	1

Filename: P:_Projects (Active)\1007352 MPE_Greenview Intake\96 CAD\1007352-PP.dwg, 2023-02-23 10:30:00



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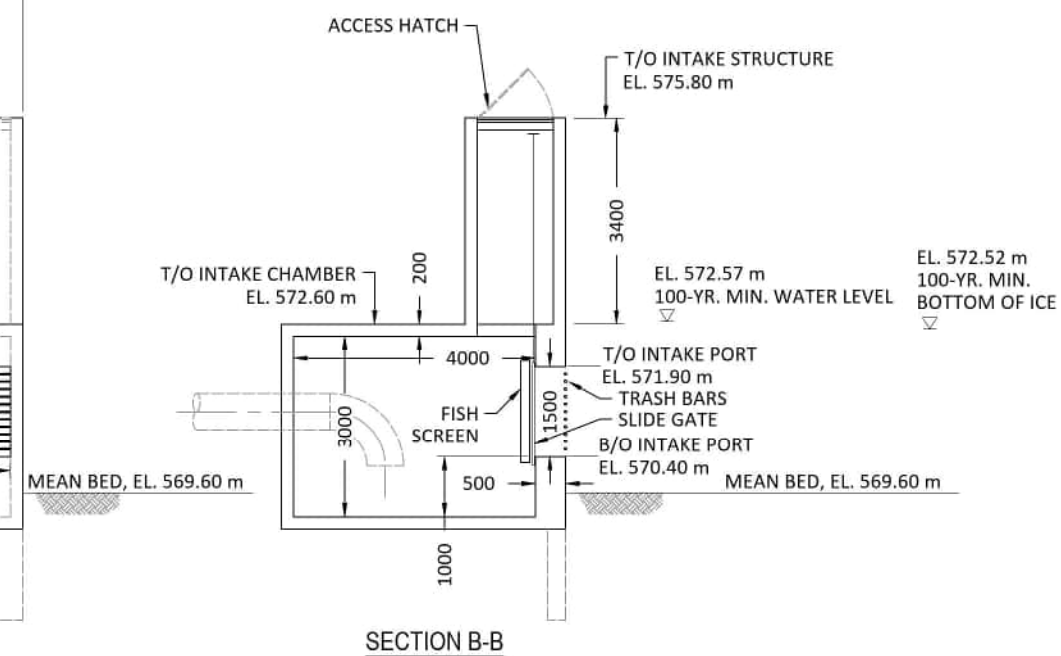
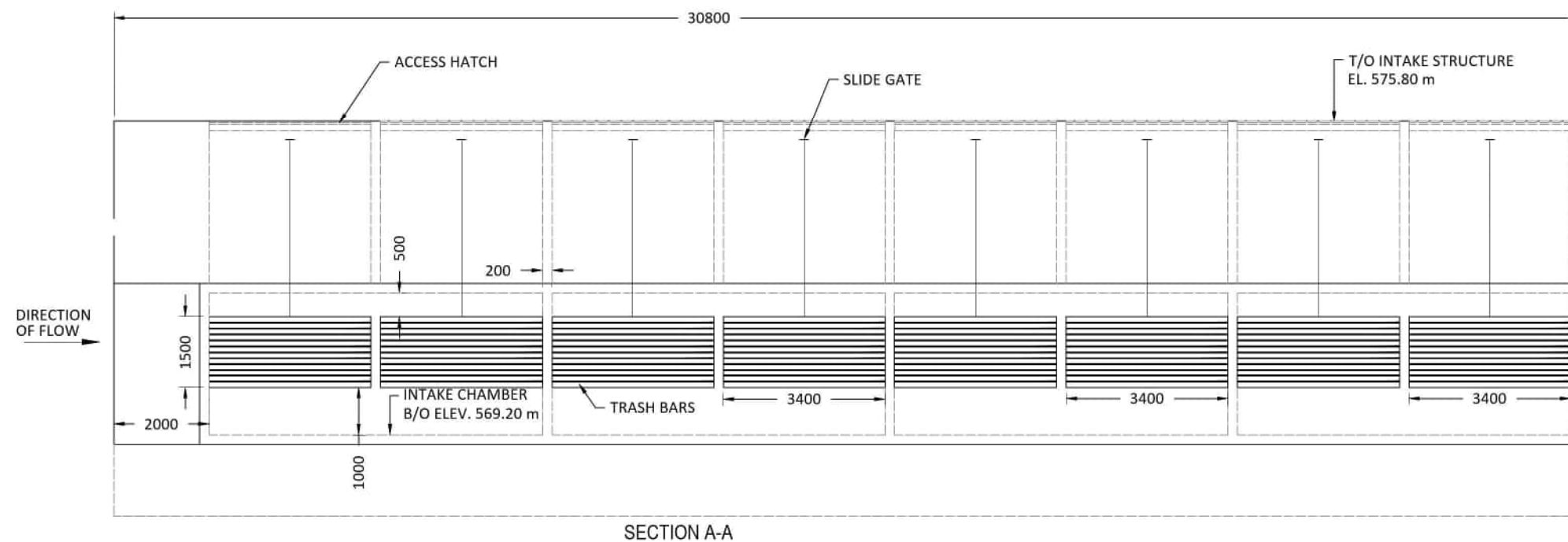
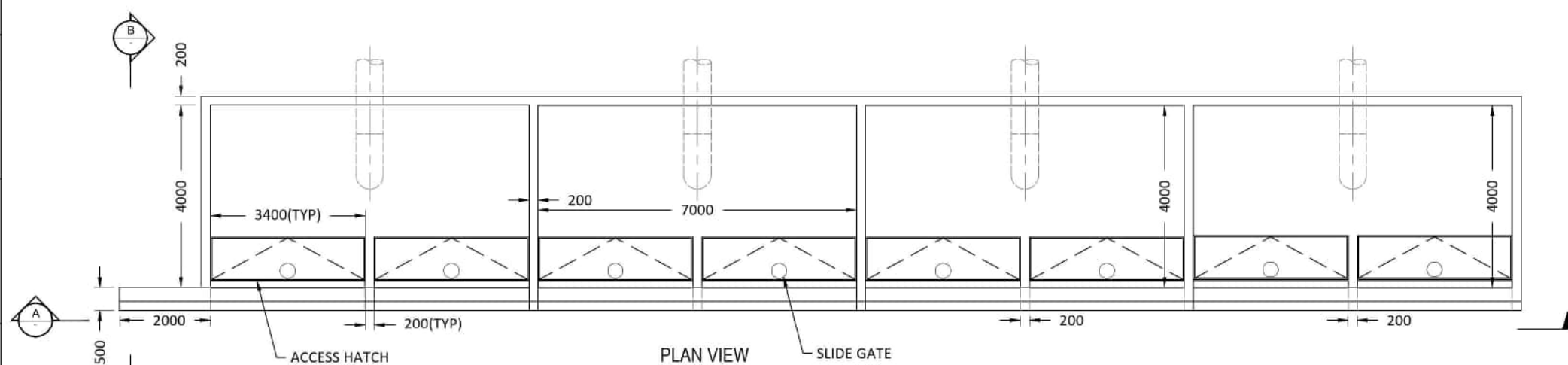
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REVISIONS			DRAWING INFORMATION	
1	Nov. 14, 2022	ISSUED FOR INTERNAL REVIEW	DATE	23 Feb 2023
			DESIGNED BY	MB
			DRAWN BY	DL
			CHECKED BY	PGV
			SHEET SIZE	B (11" x 17")

**GREENVIEW INDUSTRIAL GATEWAY
WATER INTAKE
PRELIMINARY HYDROTECHNICAL DESIGN**

GENERAL PLAN /PROFILE OF WATER INTAKE
OPTION 2A

NHC NUMBER	1007352
DRAWING NUMBER	1007352-003
FIGURE NUMBER	28
REVISION	1



(NOT FOR CONSTRUCTION)



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			DESIGNED BY	MB
			DRAWN BY	DL
			CHECKED BY	PGV
			SHEET SIZE	B (11" x 17")

GREENVIEW INDUSTRIAL GATEWAY
WATER INTAKE
PRELIMINARY HYDROTECHNICAL DESIGN

INTAKE GEOMETRY
OPTION 2B

NHC NUMBER	1007352
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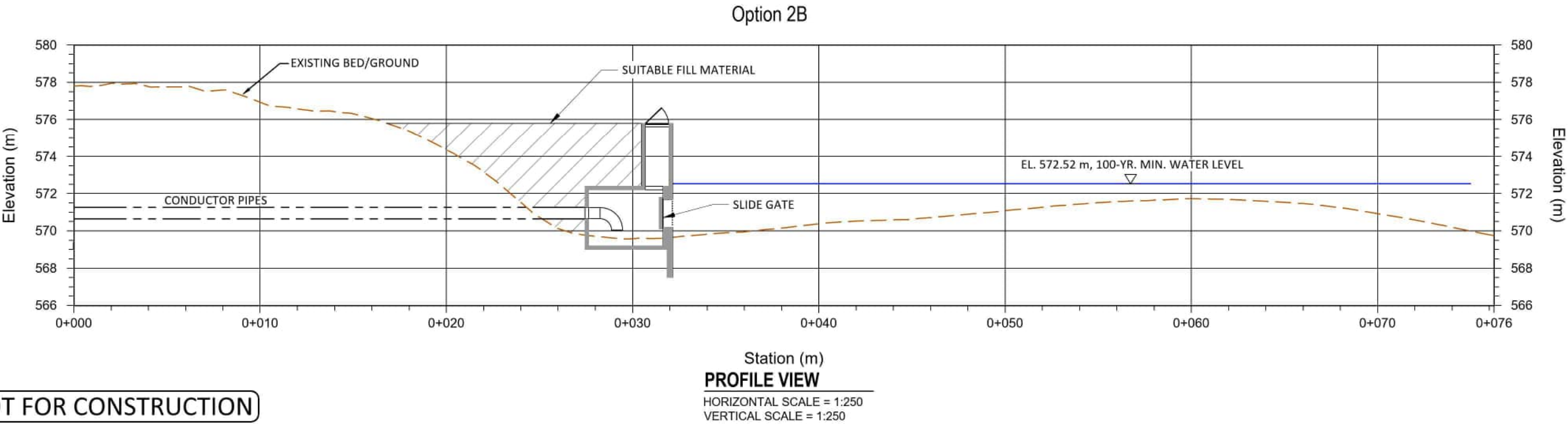
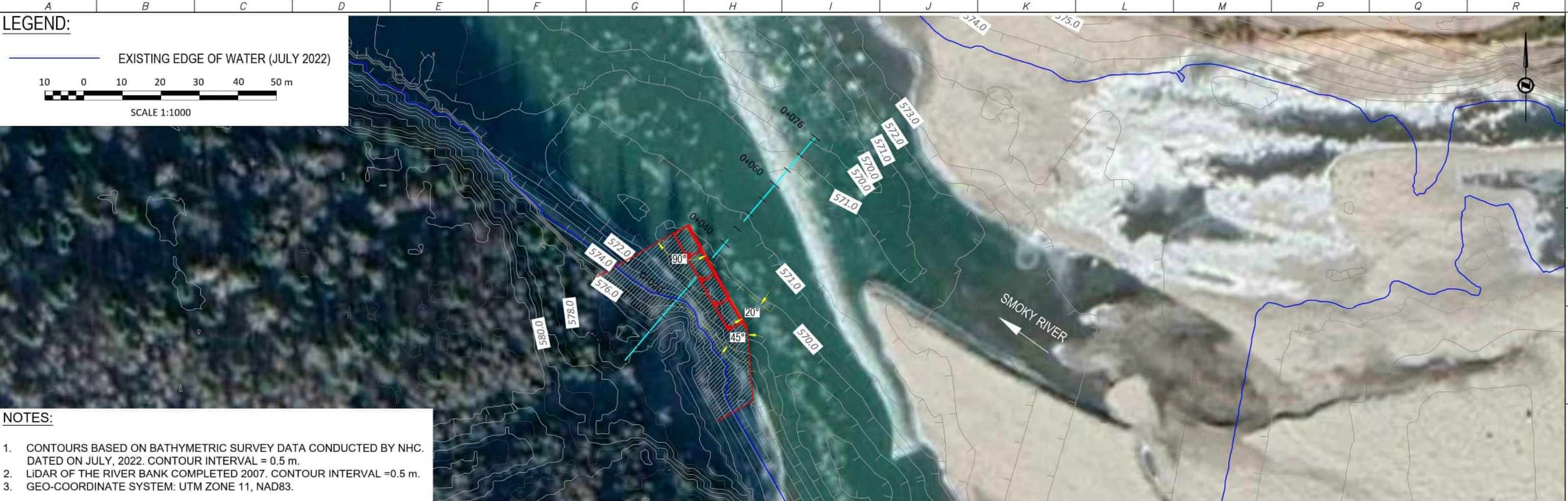
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FIGURE NUMBER

29

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REVISIONS			DRAWING INFORMATION	
1	Nov. 14, 2022	ISSUED FOR INTERNAL REVIEW	DATE	23 Feb 2023
			DESIGNED BY	MB
			DRAWN BY	DL
			CHECKED BY	PGV
			SHEET SIZE	B (11" x 17")

**GREENVIEW INDUSTRIAL GATEWAY
WATER INTAKE
PRELIMINARY HYDROTECHNICAL DESIGN**

GENERAL PLAN /PROFILE OF WATER INTAKE
OPTION 2B

NHC NUMBER	1007352
DRAWING NUMBER	1007352-003
FIGURE NUMBER	30
REVISION	1



		SCALE – AS SHOWN		GREENVIEW INDUSTRIAL GATEWAY WATER INTAKE PRELIMINARY HYDROTECHNICAL DESIGN DRAFT REPORT EROSION PROTECTION LOCATIONS FIGURE 31
		Coordinate System: Units: As Shown		
		Job: 1007352	Date: 2023-02-16	

Appendix F

Rock Cofferdam Hydraulics Report

NHC Ref. No. 1007352-4

July 12, 2024

MPE Engineering
#101, 10630 - 172 Street
Edmonton, AB T5S 1H8

Attention: Haj Oshiki, P.Eng.

Via email: hoshiki@mpe.ca

Re: **Greenview Industrial Gateway Raw Water Intake**
Preliminary Assessment of Construction Berm Hydraulics – Final Report

Dear Mr. Oshiki:

1 INTRODUCTION

Northwest Hydraulic Consultants Ltd. (NHC) has been asked by MPE Engineering and WaterSMART Solutions Ltd. to provide a preliminary assessment of hydraulic conditions for the construction berm required to build a water intake for the M.D. of Greenview. The water intake will be located on the on the west bank of the Smoky River in the reach south of the Canfor Cut-Across Road bridge (**Figure 1**). This preliminary assessment of hydraulic conditions was carried out to provide information for regulatory submissions.

The layout of the construction berm is shown in the drawing provided by MPE in Appendix A. The inside toe of the berm will be offset by 8 m from the eventual intake structure so that scour protection can be placed around the intake. The berm will have a 6 m top width at an elevation of 577.23 m with 2H:1V side slopes.

The construction berm is scheduled to be installed in the river beginning on July 15; however, the date that the berm will be removed may range from Mar 31 to July 31 of the following year, or even later if there are delays in construction. Construction of the berm would take longer than the two weeks instream work period from July 15 to July 31 and would be easier during the lower flow and water levels in the river typically seen in the fall and winter. Ideally construction of the berm would start in September. Removal of the berm could occur after the peak flow period of the following year, so construction could happen through the highest flow periods in June and early July. It would be better to remove the obstruction as early as possible to avoid the high flow period but, if necessary, it may be removed after the restricted activity ends on July 15.

2 HYDROLOGY

Daily flows in the Smoky River at the intake site were estimated using the techniques describe in NHC's Preliminary Hydrotechnical Design Report dated Feb 24, 2023 (NHC, 2023).

2.1 Peak Flows

Yearly values of peak flow were extracted from the daily flows at the intake site for various construction time periods. These time periods all started on July 15, but the end dates were incremented semi-monthly from March 31 to May 31. Annual peak flows (Jul 15 – Jul 14) were also extracted from the daily flows. The yearly peak flows were then multiplied by a peaking factor of 1.104 (NHC, 2023) before frequency analyses were carried out. Log-Pearson 3 (LP3) distributions were found to fit all five flow series reasonably well as shown in **Figure 2**.

A summary of peak flows for various return periods for each of the five construction time periods is given in **Table 1**. Note that the peak flows for the first two time periods, ending Mar 31 and Apr 15 are the same because spring flows do not increase substantially until after Apr 15. Most of the peak flows occur in June, July or April but some peak flows also occur in August and September.

Table 1 Summary of peak flows for various construction time periods and return periods.

Return Period (year)	Peak Flow Jul 15 to Mar 31 (m ³ /s)	Peak Flow Jul 15 to Apr 15 (m ³ /s)	Peak Flow Jul 15 to Apr 30 (m ³ /s)	Peak Flow Jul 15 to May 15 (m ³ /s)	Peak Flow Jul 15 to May 31 (m ³ /s)	Peak Flow Jul 15 to Jul 14 (m ³ /s)
20	1327	1327	1352	1367	1363	1935
10	1009	1009	1049	1074	1112	1510
5	742	742	790	821	891	1154
2	444	444	491	527	626	753

2.2 Breakup Timing

The timing of breakup was estimated by extracting the dates of the last “B” comment in the Water Survey of Canada (WSC) record for the station Wapiti River near Grande Prairie (07GE001). This station was selected since it was the station with the longest record nearby the intake site and was considered to be representative of breakup timing at the intake site. Ice movement and potentially ice jams may occur in the week leading up to the last “B” comment, which is used to indicate that the water levels are affected by the presence of ice.

The average date of last “B” was Apr 17 with the earliest last “B” occurring Apr 2 and the latest occurring Apr 30; therefore, it is very likely that breakup will occur in the time periods which end on April 30 or later. There is roughly about a 50% chance that breakup will occur between Mar 31 and April 15 but very little chance that it will occur before Mar 31. Therefore, ice jam water levels should be considered for all construction time periods except for Jul 15 to Mar 31.

2.3 Fish Passage Flows

Median daily flows and median peak (2-year) daily flows for each month were extracted from the daily flow record for the Smoky River at the intake site. These flows are summarized in **Table 2**. Flows are typically highest in June and lowest in February.

Table 2 Summary of monthly median and median peak flows

Month	Median Daily Flow (m ³ /s)	Median Daily Peak Flow (m ³ /s)
July	285	479
August	161	236
September	107	155
October	76.8	123
November	24.9	50.4
December	15.1	19.9
January	12.0	14.2
February	9.9	11.1
March	10.5	16.4
April	62.8	200
May	251	475
June	392	635

3 HYDRAULIC MODELLING

Hydraulic analysis of the river reach was carried out using the Hydrological Engineering Center's River Analysis System (HEC-RAS) one-dimensional (1D) numerical hydraulic model developed for the project (NHC, 2023). The berm geometry provided by MPE was added to the 1D model so that preliminary estimates of water levels and velocities could be obtained to provide input for regulatory submissions. A two-dimensional (2D) model of the reach should be developed during the detailed design phase to provide more detailed results for detailed design of the construction berm.

3.1 Peak Water Levels

A summary of the peak open water levels for each construction time period for various return periods is given in **Table 3**. A range of flow conditions is provided so that the contractor can make an informed decision on risk when selecting the final top of cofferdam elevation. The water levels reported in **Table 3** are the water levels expected on the upstream side of the berm. The preliminary top of berm elevation of 577.23 m provided by MPE is only exceeded during the 5-year annual (Jul 15 to Jul 14) peak water levels.

Note that for all the time periods except the time period ending Mar 31, peak ice jam water levels will likely occur as well. Peak ice jam levels are unaffected by the presence of the berm. The ice jam levels listed in **Table 3** were obtained from NHC's Preliminary Hydrotechnical Design Report dated Feb 24,

2023 (NHC, 2023). The maximum open water levels which are lower than the ice jam water level are shaded in yellow.

Table 3 Summary of peak water levels

Return Period (year)	Open Water Jul 15 to Mar 31 ¹ (m)	Open Water Jul 15 to Apr 15 (m)	Open Water Jul 15 to Apr 30 (m)	Open Water Jul 15 to May 15 (m)	Open Water Jul 15 to May 31 (m)	Open Water Jul 15 to Jul 14 (m)	Ice Jam (m)
20	577.50	577.50	577.56	577.60	577.59	578.84	578.0
10	576.73	576.73	576.83	576.89	576.98	577.94	577.1
5	576.00	576.00	576.14	576.23	576.42	577.09	576.4
2	575.14	575.14	575.30	575.33	575.65	576.04	575.2

¹ Ice jams do not occur in this period.

The maximum open water levels which are lower than the ice jam water level are shaded in yellow.

3.2 Maximum Velocities

A summary of the maximum velocities for each construction time period for various return periods is given in **Table 4**. The water levels reported in **Table 4** are the cross section average velocities occurring where the channel has been constricted the most by the construction berm. Higher local velocities may occur, but a more detailed 2D model is required to determine the magnitude and location of these local peak velocities.

The velocities listed in **Table 4** are based on the existing bed geometry without allowance for bed scour. Given the magnitudes of these velocities, significant bed scour is likely to occur in the constriction at high flows.

Table 4 Summary of maximum velocities in berm constriction without scour for various construction time periods and return periods.

Return Period (year)	Peak Velocity Jul 15 to Mar 31 (m/s)	Peak Velocity Jul 15 to Apr 15 (m/s)	Peak Velocity Jul 15 to Apr 30 (m/s)	Peak Velocity Jul 15 to May 15 (m/s)	Peak Velocity Jul 15 to May 31 (m/s)	Peak Velocity Jul 15 to Jul 14 (m/s)
20	5.9	5.9	5.9	5.9	5.9	6.2
10	5.6	5.6	5.6	5.6	5.7	6.0
5	4.8	4.8	5.2	5.3	5.4	5.7
2	2.7	2.7	2.9	3.6	4.1	4.9

3.3 Bed Scour

Bed scour in the berm constriction was estimated using the competent mean velocity approach (TAC, 2004). This method determines the velocity at which the bed material is stable and calculates the amount of bed scour needed to reduce the velocity to this value. The median bed material size is 52 mm which, for the range of flow depths at the berm, results in a competent mean velocity of 2.5 m/s. The scour depths required to reduce the average velocities to this value are listed in **Table 5** for each construction time period for various return periods.

Note that the scour depths listed in **Table 5** may be limited by the presence of bedrock. Bedrock outcrops occur on both banks, so it is quite likely the bedrock also exists under the channel. However, the presence of bedrock and its elevation have not been confirmed.

Table 5 Summary of bed scour depths in berm constriction for various construction time periods and return periods.

Return Period (year)	Scour Depth Jul 15 to Mar 31 (m)	Scour Depth Jul 15 to Apr 15 (m)	Scour Depth Jul 15 to Apr 30 (m)	Scour Depth Jul 15 to May 15 (m)	Scour Depth Jul 15 to May 31 (m)	Scour Depth Jul 15 to Jul 14 (m)
20	4.7	4.7	4.7	4.7	4.8	5.1
10	3.8	3.8	3.9	4.0	4.1	4.7
5	2.7	2.7	3.1	3.2	3.4	4.2
2	0.3	0.3	0.5	1.3	1.9	2.8

4 FISH PASSAGE VELOCITIES

Fish passage was assessed by evaluating velocities during the median monthly flows and the median peak (2-year) monthly flows. These velocities were then compared to the swimming speeds for the various fish species found in the river reach.

4.1 Maximum Average Velocities

The maximum cross-section average velocities in the natural river reach are compared to the maximum velocities in the berm constriction and in the bridge waterway for the mean daily flow and median daily peak flow for each month in **Table 6**. The months where the velocity is greater in the berm constriction than any where in the natural reach are highlighted in yellow. This occurs in May, June and July, at the median flows and in May, June, July and August at median peak flows. In the other months, higher velocities occur at other cross sections due to shallow depths at riffle sections. As well, in all cases highlighted in yellow, the maximum average velocity in the bridge waterway is also greater than the maximum average in the berm constriction. This bridge constriction is unlikely to be widened before the cofferdam is removed.

The peak velocity analysis indicates that the berm will only produce a barrier to fish passage in May, June July and August; but, in August, this only occurs during peak flow events. Furthermore, berm construction will occur in late July so the berm will likely not fully constrict the flow until the end of July. Also, if construction could be completed before May 1, fish passage in May and June would not be affected.

4.2 Fish Characteristics

Fish characteristics in the intake reach were provided by MPE and are summarized in **Table 7**. There are nine species listed. The group summary indicated in **Table 7** is based on the categories provided in the Swim Performance Online Tools (SPOT) website (Di Rocco, R, and R. Gervais, 2023) provided by the Department of Fisheries and Oceans (DFO). Three fish groups were identified based on the fish characteristics listed in **Table 6**: Eel, Pike, and Salmon & Walleye.

Table 6 Summary of peak velocities for the median and median peak for each month

Month	Peak Velocity at the Median Daily Flow (m/s)			Peak Velocity at the Median Daily Peak Flow (m/s)		
	Natural	Bridge	at Berm	Natural	Bridge	at Berm
July	1.77	1.78	2.66 ¹	1.80	2.22	3.43 ¹
August	2.14	1.41	1.91	2.09	1.63	2.39
September	2.02	1.20	1.45	2.14	1.39	1.87
October	2.02	1.11	1.14	2.07	1.27	1.60
November	1.58	0.67	0.45	1.78	0.86	0.79
December	1.63	0.56	0.30	1.55	0.63	0.38
January	1.70	0.51	0.25	1.70	0.55	0.28
February	1.64	0.47	0.21	1.67	0.50	0.23
March	1.64	0.49	0.22	1.55	0.58	0.32
April	1.93	1.04	0.98	2.21	1.53	2.18
May	1.94	1.68	2.47	1.79	2.21	3.41 ¹
June	1.73	2.02	3.14 ¹	2.30	3.07	4.48 ¹

¹ These velocities will be limited to 2.5 m/s due to bed scour in the constriction.

Fish may move around in the river at any time of year but migration, when upstream movement is critical, are limited to specific periods of the year. **Table 7** lists the months in which migration is most likely to occur for each species. Of the species listed, only six are likely to migrate in May, June, July, or August when the berm may produce a barrier to fish migration. These six species are all in the Salmon and Walleye group, which are stronger swimmers. Burbot and Northern Pike, the weakest swimmers, do not migrate during this period.

Table 7 Summary of Fish Characteristics in the Intake Reach

Species	DFO Group	Mean Length at First Maturity (mm)	Migration Period
Burbot	Eel	370	Nov-Feb
Northern Pike	Pike	420	Feb-Apr
Arctic Grayling	Salmon & Walleye	326	Mar-Jun
Bull Trout	Salmon & Walleye	385	Aug
Mountain Whitefish	Salmon & Walleye	230	Sep-Nov
Rainbow Trout	Salmon & Walleye	349	May-Jun
Walleye	Salmon & Walleye	440	Mar-May
Longnose Sucker	Salmon & Walleye	240	Apr-May
White Sucker	Salmon & Walleye	253	Jun-Jul

4.3 Fish Swimming Speeds

Fish swim performance was assessed using information obtained from the Fish Swimming Performance Database and Analyses (Katopodis, C, and R. Gervais, 2016) which provides the basis for the SPOT website. This document indicates that the relationship for fish swimming velocity, V and distance, X is described by the following equation:

$$\frac{X}{L} = M \left(\frac{V}{\sqrt{gL}} \right)^a$$

where g is the acceleration due to gravity, L is the length of the fish, and M and a are constants that vary with fish species and the probability of exceedance for a given performance. SPOT groups fish species in six groups and provides M and a values for each of these groups for various probabilities of exceedance. Investigation of the variation of M and a provided for these groups indicated that a was virtually constant for each group while M varied with probability of exceedance according to a log-normal distribution.

An example to the fish swim performance curves generated using the above analyses is shown in **Figure 3**. This figure shows the swim performance curves for a 385 mm long Bull Trout for the standard 2.5%, 12.5%, 50%, 87.5%, and 97.5% probability of exceedances provided by SPOT (these exceedances are the bounds of the 75% and 95% prediction intervals). Also included in **Figure 3** are the velocity-distance curves for the August median peak flow evaluated at the berm constriction. The line labeled 31.2% exceedance passes through the maximum velocity-distance combination for the berm constriction, which indicates that about 30% of the 385 mm Bull Trout would be able to pass through the constriction at this discharge.

Similar evaluations of fish swim performance curves versus velocity-distance curves were carried out for each of the six fish species which may migrate in May through August for the months when each species may be migrating. A summary table of the degree of fish passage affected is provided in **Table 8**. Figures comparing the swim performance curves for each species with velocity-distance curves for the

flow rates are provided in **Appendix B**. The velocities used in this velocity-distance analysis are cross section average velocities adjusted to a 1 m depth because fish will tend to swim in the zone of lower velocity near the bank opposite of the berm. Note that as little as 2% of some species will be able to pass upstream through the constriction without enhancing passage.

As shown in **Table 8**, the length that the fish can swim against the peak velocity is quite limited. However, if refuge is provided at the short intervals, the fish can rest between short bursts of swimming. For example, boulder clusters are commonly used to provide resting areas for fish to improve passage. The short distances needed, however, will require an almost continuous series of boulders rather than widely spaced clusters.

Table 8 Summary of percentage of fish able to pass upstream though the berm constriction at the median peak flow for each month.

Species		Percentage Passing Upstream			Required Boulder Spacing (m)	
	May Median Peak Flow	Jun Median Peak Flow	Jul Median Peak Flow	Aug Median Peak Flow	97.5% Passage	50% Passage
Arctic Grayling	7.0	1.5			0.2	2.3
Bull Trout				31.2	1.4	19.3
Rainbow Trout	8.9	2.1			0.2	2.7
Walleye	17.9				0.8	10.7
Longnose Sucker	2.0				0.2	2.4
White Sucker		0.3	2.6		0.1	1.1

5 EROSION PROTECTION

Erosion protection is required on the exposed surface of the berm to ensure that high velocities do not cause damage to the berm. Erosion protection is also recommended for the opposite bank to ensure that changes in flow patterns due to the presence of the berm do not permanently alter the flow pattern at the intake.

The size of riprap required to protect against erosion depends on its location within the river. The maximum velocities in the existing section (without scour) are listed in **Table 4**. These velocities will occur at the base of the berm if no scour occurs in the cross section. Lower velocities would occur along the opposite bank because depths are lower there.

The size of riprap also depends on the amount for scour that occurs in the constricted section. As scour increases, the average velocity decreases. If bedrock does not limit scour, the average velocity in the constriction will be limited to 2.5 m/s, the velocity required to transport the bed material. Velocities may exceed 2.5 m/s in the short term as bed material transport may occur more slowly than the velocity increase. For this analysis it was assumed that bedrock would limit the scour level in the constriction to

the existing minimum bed elevation but that this would occur across the entire constricted section. Average velocities in the constriction range from 2.9 to 3.5 m/s for the 5-year to 20-year peak flows during the construction time periods. Peak velocities due to the flow acceleration along the berm may be up to 1.5 times these values. Two dimensional modeling should be carried out to confirm these flow conditions before construction is carried out.

5.1 Berm

Given that velocities as high as 5.0 m/s may occur at the berm and that Class 3 riprap with a median diameter of 0.8 m is required for the permanent protection of the intake structure, it is recommended that riprap with a median diameter of 0.8 m also be used for the berm protection. The riprap should extend along the construction berm as shown in **Figure 4**. The length of this riprap is 160 m.

A cross section of the riprap is shown in **Figure 5**. The riprap should be 1.5 m thick extending down to an elevation of 573.0 m at a slope of 2H:1V. A riprap apron should be placed on the bed at the base to the riprap slope extending out 5.6 m with a top elevation of 573.0 m. This apron will protect against the scour predicted for a 5-year peak flow for a constriction time period extending to July 14, or later. A non-woven geotextile should be placed under the riprap slope.

Two-dimensional (2D) flow modelling should be carried out to refine velocities along the berm before the construction berm is placed in the river. If this modeling indicates high velocities may occur in the eddy downstream of the berm, it may be necessary to also place some form of erosion protection along the downstream slope of the berm.

5.2 Opposite Bank

The erosion protection on the bank opposite the construction berm should extend along the bank as shown in **Figure 4**. Part of this extent should be designed as permanent protection as indicated in the Preliminary Hydrotechnical Design Report for the intake (NHC, 2023). Both the permanent and temporary protection extents are shown in **Figure 4**.

5.2.1 Permanent Erosion Protection

The permanent erosion protection should be designed to withstand the 100-year peak flow. As stated in the Preliminary Hydrotechnical Design Report for the intake, the permanent protection should have Class 1 riprap with a median diameter of 0.3 m. The riprap should be 0.5 m thick and extend up to an elevation of 579.7 m which will provide 0.5 m of freeboard above the 100-year water level. As shown in the cross section in **Figure 5**, this will require a small berm to project above the existing bank. Bedrock exists along the base of this bank so the riprap should extend down to the top of bedrock. No scour protection is required due to the presence of the bedrock; however, if the bedrock slopes toward the river, it may be necessary to modify the bedrock surface to improve riprap stability.

5.2.2 Temporary Erosion Protection

The temporary riprap protection was designed to protect against the average scour level predicted for a 5-year peak flow for a constriction time period extending to July 14, which is the same as the annual

peak flow. The bank slope on this side of the river is quite mild, so it is recommended that the entire riprap volume be placed as an apron, which will launch into the correct location if scour occurs. This will limit the disturbance to the bank in this temporary protection area by not requiring bank recontouring. The velocities along this portion of the bank are expected to require the use of Class 2 riprap with a median diameter of 0.5 m to provide protection.

The toe of the riprap should be placed at elevation 573.14 m. Placing the toe of the riprap at this elevation will allow the riprap toe to function as a continuous series of boulder clusters when the water level is above the July median water level as is needed for fish passage. Incorporating the boulder clusters into the riprap will also satisfy navigation requirements by projecting the top of riprap above the 2-year annual water level. The July median water level is 574.14 m so some of the riprap will need to be installed in the water.

The configuration of the temporary riprap is shown in **Figure 5**. The riprap surface should rise from the toe location away from the river at a 2H:1V slope. The top of riprap should be at elevation 577.23 m, the same as the berm elevation, and extend horizontally 3.0 m before sloping back down to ground level, again at a 2H:1V slope.

6 CONCLUSIONS

The preliminary analysis of construction berm hydraulics analysis indicated the following:

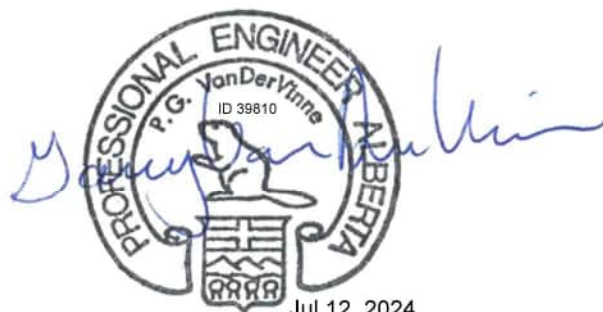
- 1) Peak flows do not change if the construction period is extended from Mar 31 to Apr 15.
- 2) Peak flows increase as the construction period extends from Apr 15 to May 31 but peak flows are greatest if the construction period spans the entire year from Jul 15 to Jul 14.
- 3) Probability of breakup occurring during construction time period increases to about 50% if the construction period is extended from Mar 31 to Apr 15
- 4) Probability of breakup occurring during construction time period increases to about 100% if the construction period is extended to Apr 30 or later.
- 5) Ice jam water levels are higher than peak open water levels in most cases.
- 6) Monthly median and median peak flow are highest in July and lowest in February.
- 7) Peak velocities in the berm constriction increase significantly with return period but are less sensitive to construction time period.
- 8) Monthly median velocities in the berm constriction are only higher than the natural peak velocities in the reach in May, June, and July and monthly median peak velocities at the berm constriction are also higher in August.
- 9) As little as 0.3% of some species will be able to pass upstream through the constriction during monthly median peak flows in their migration periods in May, June, July, and August without enhancing fish passage.
- 10) A nearly continuous series of boulders is required to meet the short spacing needed for passage of some fish species. The continuous series of boulders at the toe of the temporary riprap on the opposite bank will satisfy this requirement.
- 11) it is recommended that riprap with a median diameter of 0.8 m be used for the berm protection and that riprap with a median diameter of 0.5 m be used to provide temporary protection on the opposite bank.
- 12) 2D modelling should be carried out to refine this analysis before the construction berm is placed in the river.

7 CLOSURE

Sincerely,

Northwest Hydraulic Consultants Ltd.

Report prepared by:



Jul 12, 2024

Gary Van Der Vinne, M.Sc., P.Eng.
Principal

Report reviewed by:

PERMIT TO PRACTICE	
NORTHWEST HYDRAULIC CONSULTANTS LTD.	
Signature	 APEGA ID# 68607
Date	12 Jul 2024
PERMIT NUMBER: P 654	
The Association of Professional Engineers and Geoscientists of Alberta	

Robyn Andrishak, M.Sc., P.Eng.
Principal

DISCLAIMER

This report has been prepared by **Northwest Hydraulic Consultants Ltd.** for the benefit of MPE Engineering for specific application to the Preliminary Hydrotechnical Design of the Greenview Industrial Gateway Intake on the Smoky River. The information and data contained herein represent **Northwest Hydraulic Consultants Ltd.** best professional judgment in light of the knowledge and information available to **Northwest Hydraulic Consultants Ltd.** at the time of preparation and was prepared in accordance with generally accepted engineering and geoscience practices.

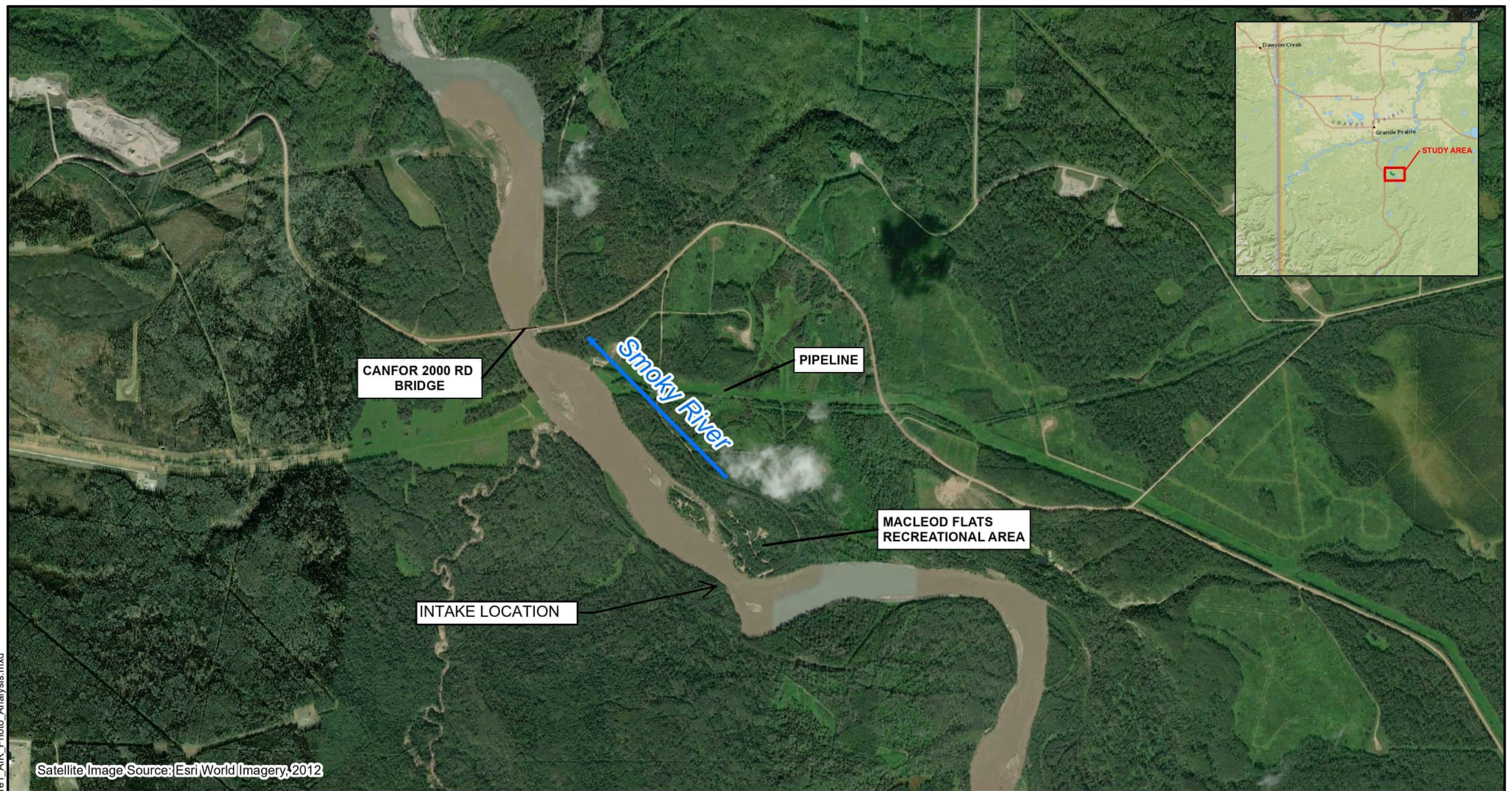
Except as required by law, this report and the information and data contained herein are to be treated as confidential and may be used and relied upon only by MPE Engineering, its officers and employees. **Northwest Hydraulic Consultants Ltd.** denies any liability whatsoever to other parties who may obtain access to this report for any injury, loss or damage suffered by such parties arising from their use of, or reliance upon, this report or any of its contents.

8 REFERENCES

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FIGURES

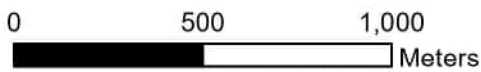
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Satellite Image Source: Esri World Imagery, 2012



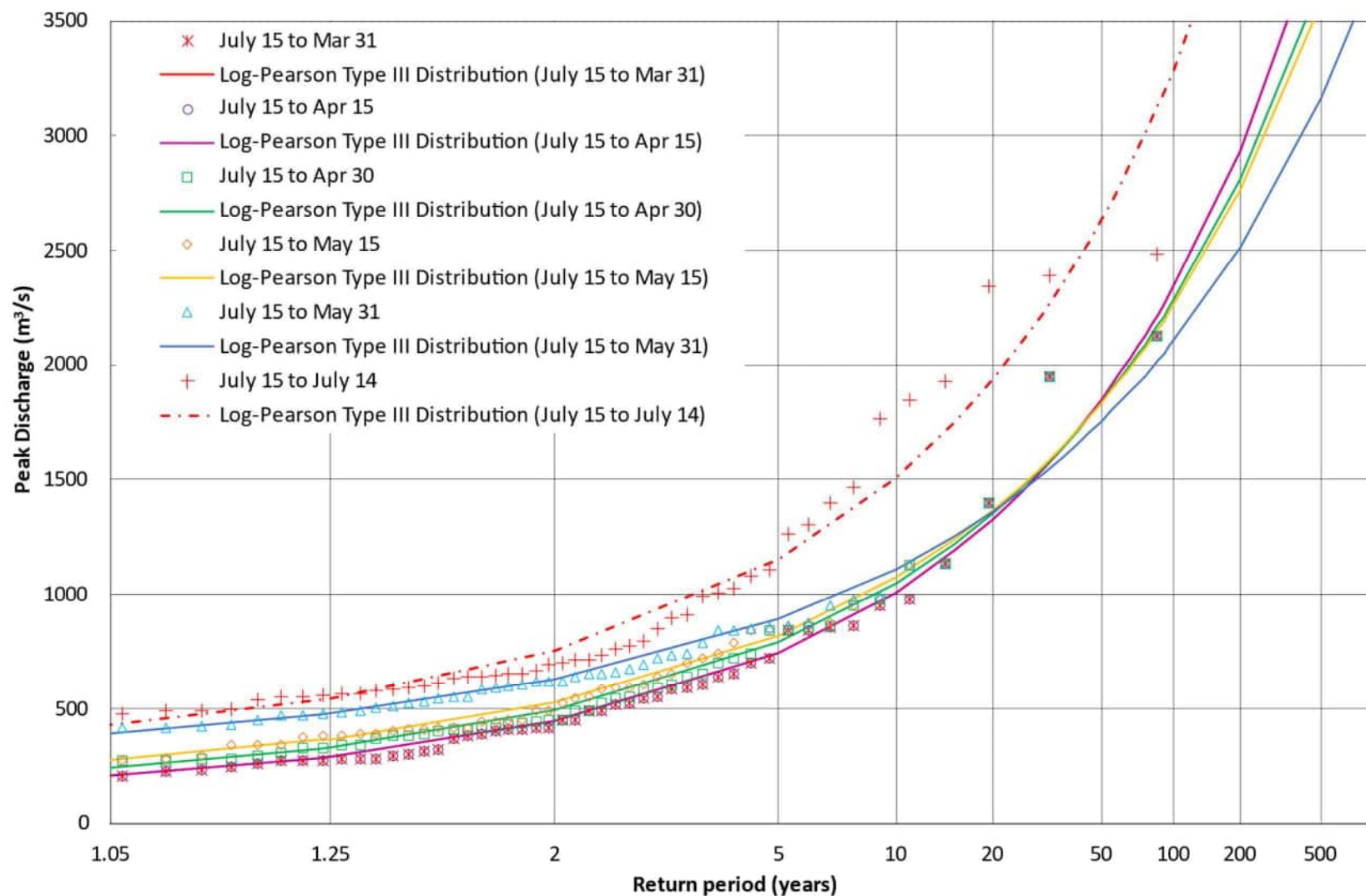
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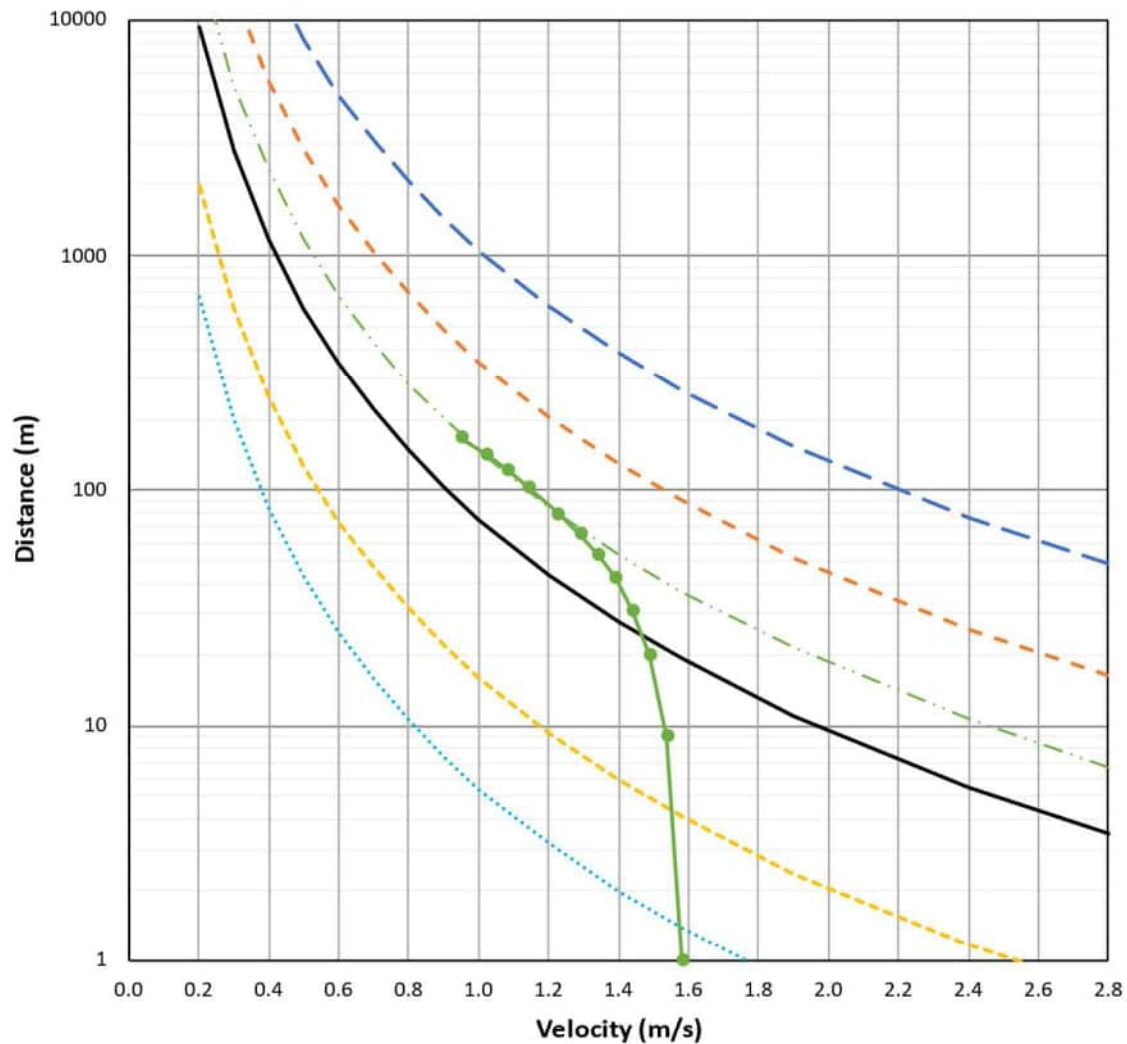
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MPE ENGINEERING		
GREENVIEW INDUSTRIAL GATEWAY WATER INTAKE		
PRELIMINARY ASSESSMENT OF CONSTRUCTION BERM HYDRAULICS		
STUDY REACH AND LOCATION		
1007352-4	JULY 9, 2024	Figure 1



Bull Trout, Length = 385 mm, August Median Peak Flow



- 2.5% Exceedance
- - - 12.5% Exceedance
- 50.% Exceedance
- - - 87.5% Exceedance
- ... 97.5% Exceedance
- Berm Constriction
- · - 31.2% Exceedance

MPE Engineering Ltd.

SCALE – AS SHOWN

GREENVIEW INDUSTRIAL GATEWAY WATER INTAKE
PRELIMINARY ASSESSMENT OF CONSTRUCTION BERM
HYDRAULICS

nhc
northwest hydraulic consultants

Coordinate System:
Units: As Shown

COMPARISON OF FISH PERFORMANCE
CURVES WITH VELOCITIES IN BERM
CONSTRICTION

Job: 1007352-4

Date: 2024-07-09

FIGURE 3

Filename: P:_Projects (Active)\1007352 MPE_Greenview Intake\96 CAD\1007352-Riprap_Layout.dwg, 2024-07-11 08:10:31



NOT FOR CONSTRUCTION



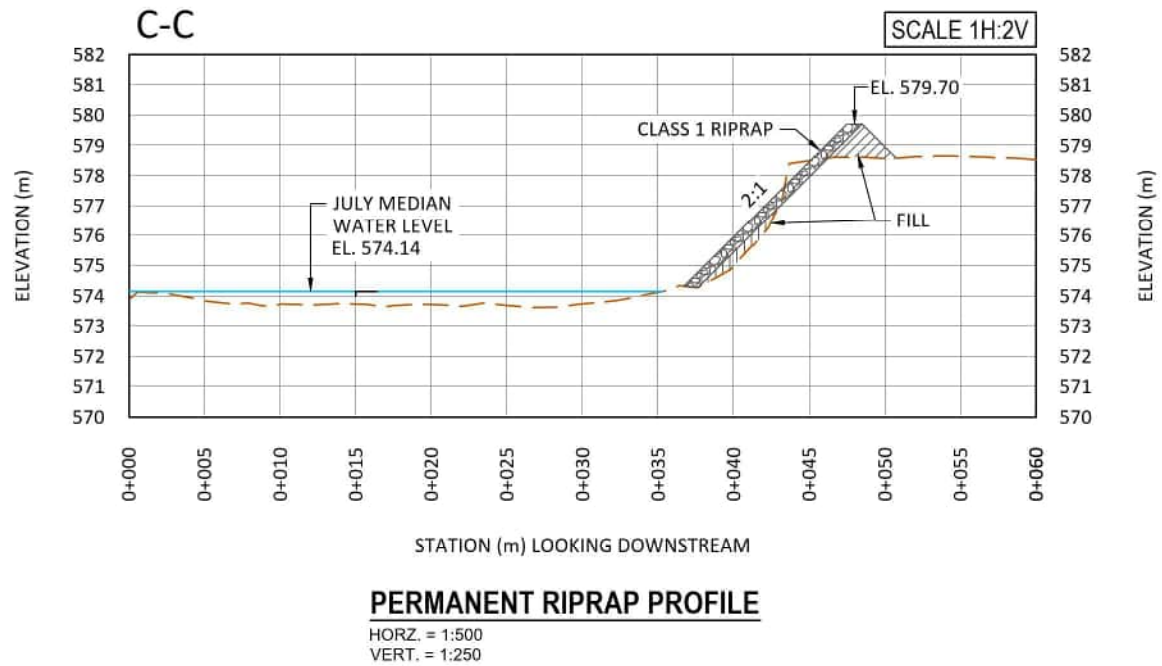
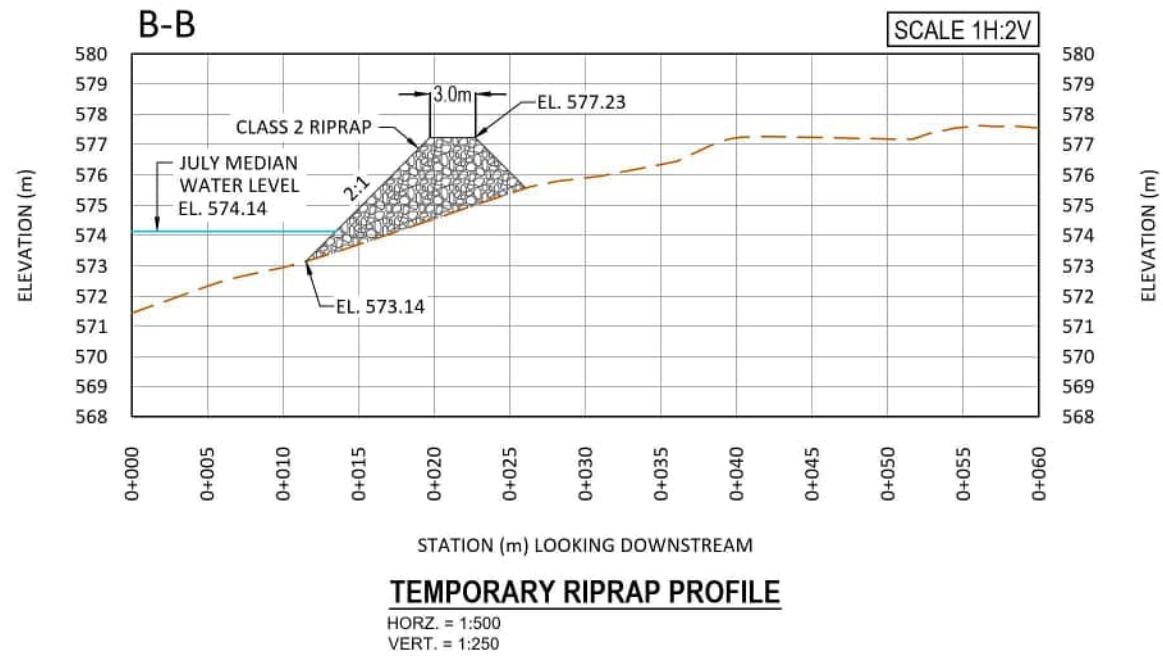
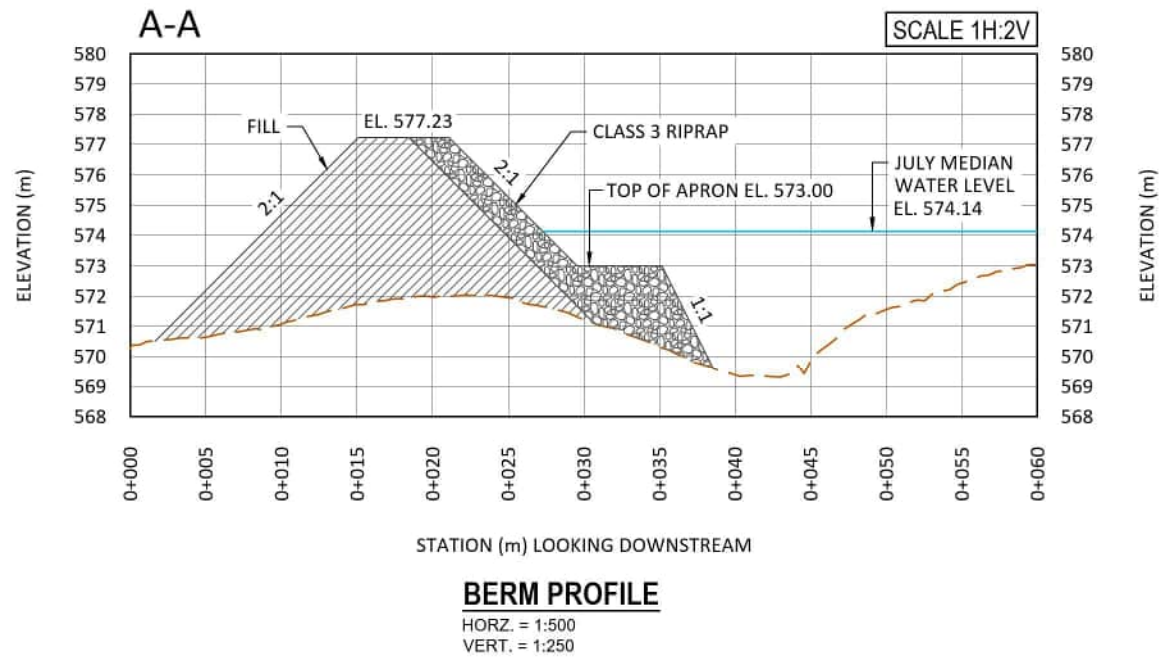
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Edmonton, AB T6X 0E3
Canada
Office: 780.436.5868
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REVISIONS			DRAWING INFORMATION	
			DATE	11 Jul 2024
			DESIGNED BY	MP
			DRAWN BY	DL
			CHECKED BY	PGV
			SHEET SIZE	B (11" x 17")

**GREENVIEW INDUSTRIAL GATEWAY
WATER INTAKE
PRELIMINARY ASSESSMENT OF
CONSTRUCTION BERM HYDRAULICS**
PLAN VIEW WITH RIPRAP LAYOUT

NHC NUMBER	1007352
DRAWING NUMBER	1007352-001
FIGURE NUMBER	4
REVISION	

Filename: P:_Projects (Active)\1007352 MPE_Greenview Intake\96 CAD\1007352-Riprap_Layout.dwg, 2024-07-11 08:10:31



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REVISIONS

NO.	DESCRIPTION	DATE

DRAWING INFORMATION

DATE	11 Jul 2024
DESIGNED BY	MB
DRAWN BY	DL
CHECKED BY	PGV
SHEET SIZE	B (11" x 17")

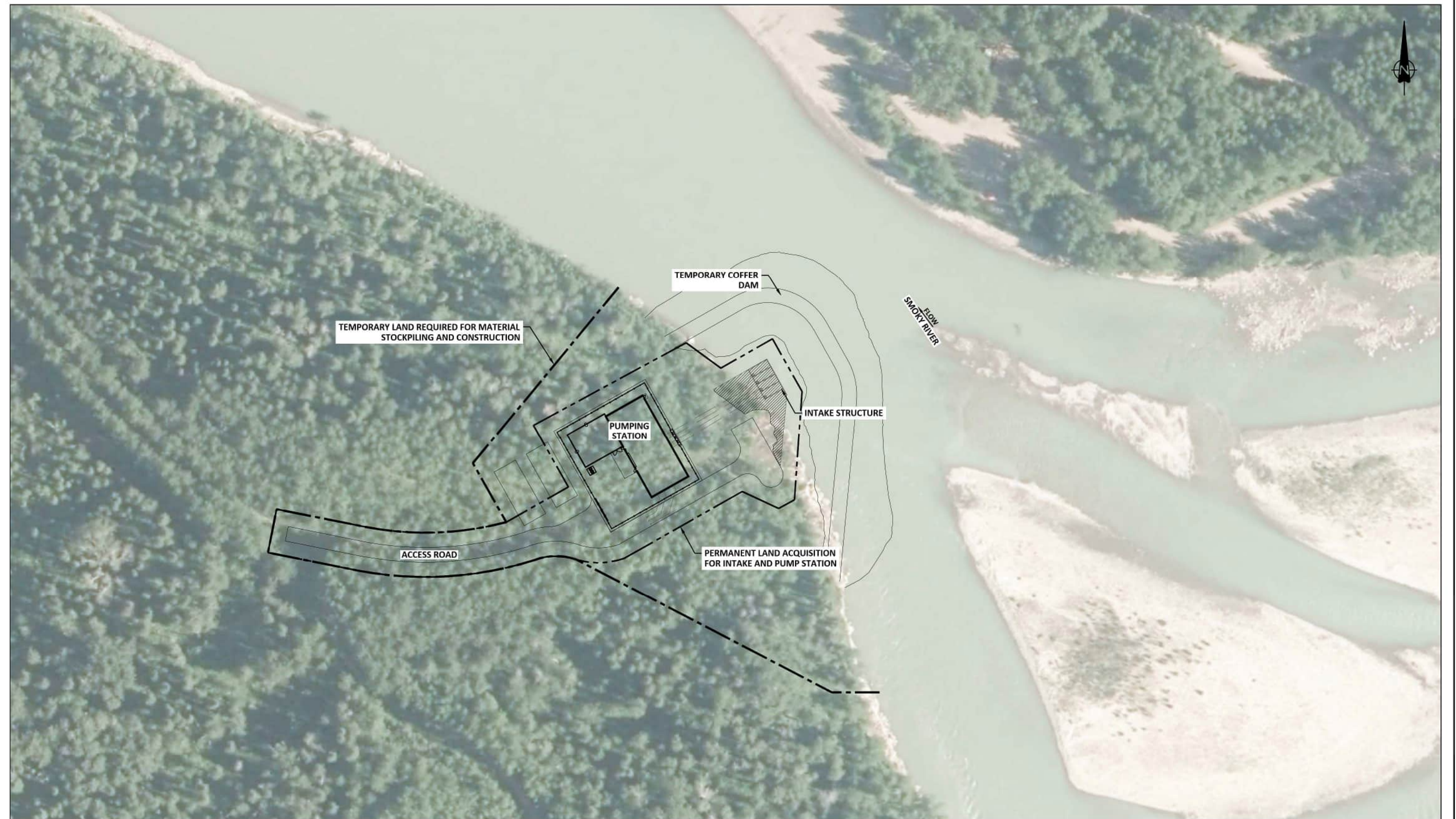
**GREENVIEW INDUSTRIAL GATEWAY
WATER INTAKE
PRELIMINARY ASSESSMENT OF
CONSTRUCTION BERM HYDRAULICS**

RIPRAP PROFILES

NHC NUMBER	1007352
DRAWING NUMBER	1007352-002
FIGURE NUMBER	5
REVISION	1

Appendix A

Construction Berm Drawings from MPE



an Englobe company

WATERSMART SOLUTIONS LTD.

GREENVIEW INDUSTRIAL GATEWAY
OVERALL SITE PLAN

SCALE: 1:1500

DATE: AUGUST 2023

JOB: 5946-001-00

FIGURE: C1.1



an Englobe company

WATERSMART SOLUTIONS LTD.

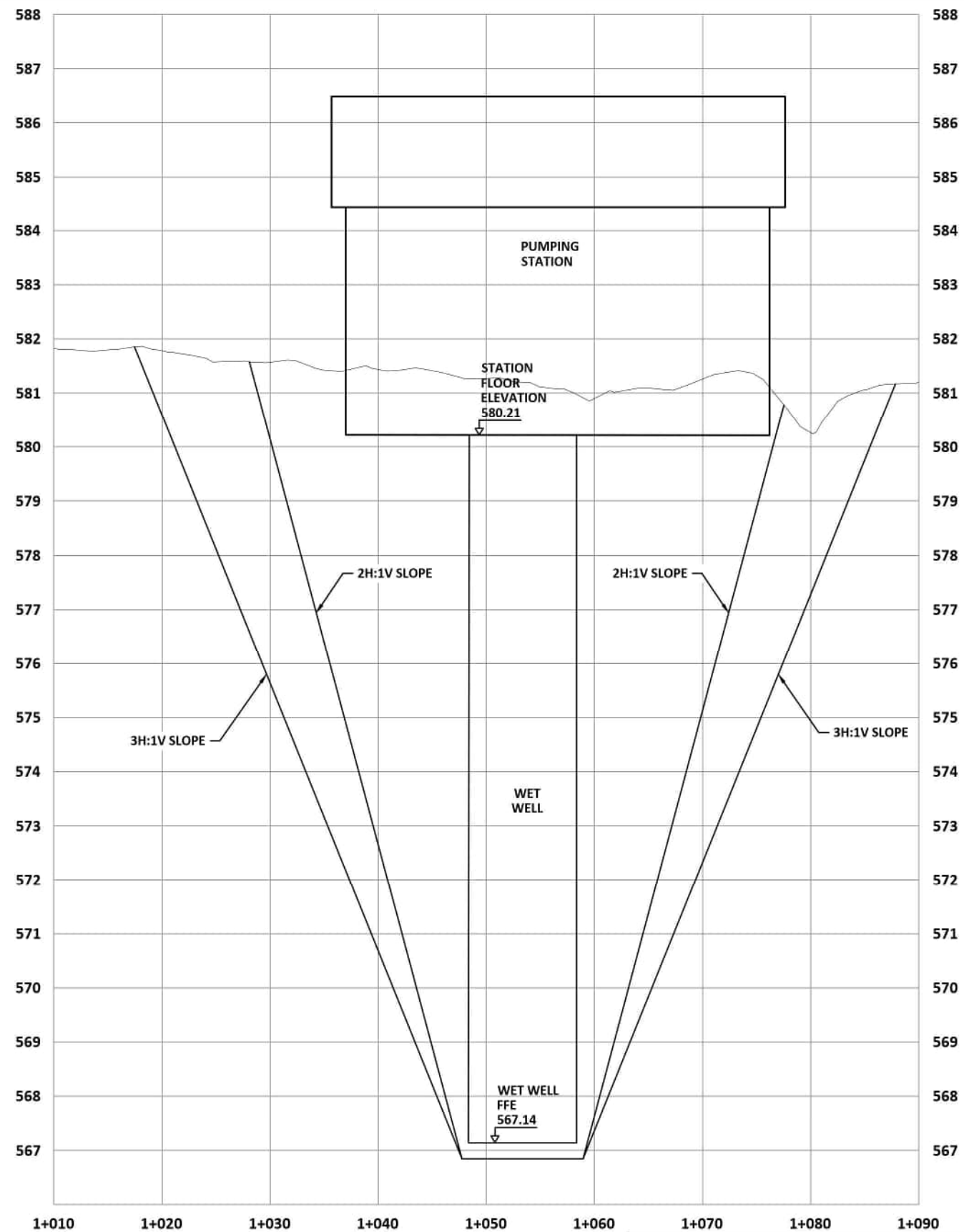
GREENVIEW INDUSTRIAL GATEWAY
PUMPING STATION SITE PLAN

SCALE: 1:750

DATE: AUGUST 2023

JOB: 5946-001-00

FIGURE: C1.2



SECTION
1
C1.3 1:500H
1:100V



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WATERSMART SOLUTIONS LTD.

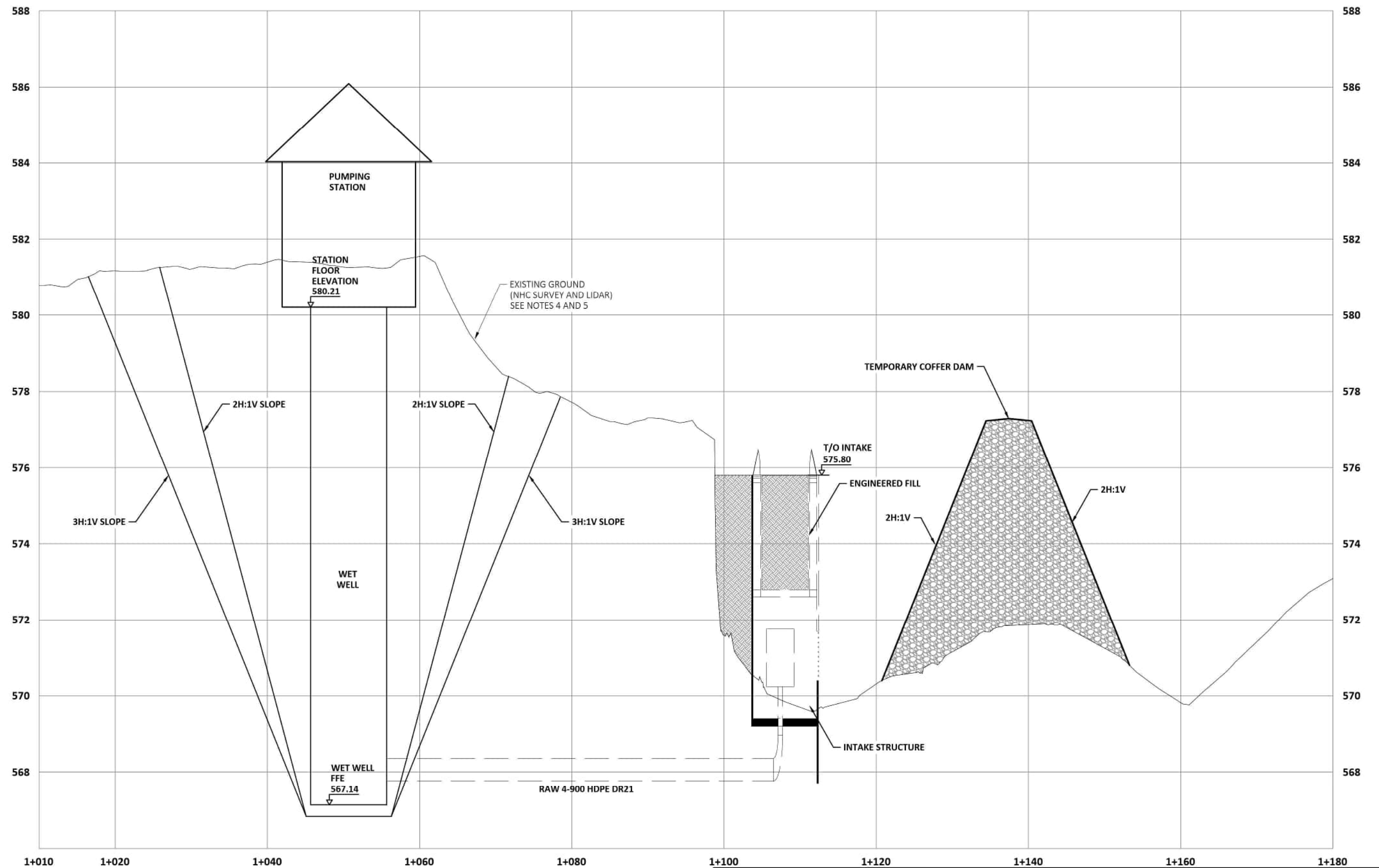
GREENVIEW INDUSTRIAL GATEWAY
PUMPING STATION SECTION

SCALE: AS SHOWN

DATE: AUGUST 2023

JOB: 5946-001-00

FIGURE: C1.3



SECTION
2
C1.4 1:500H
1:100V



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WATERSMART SOLUTIONS LTD.

GREENVIEW INDUSTRIAL GATEWAY
PUMPING STATION SECTION

SCALE: AS SHOWN

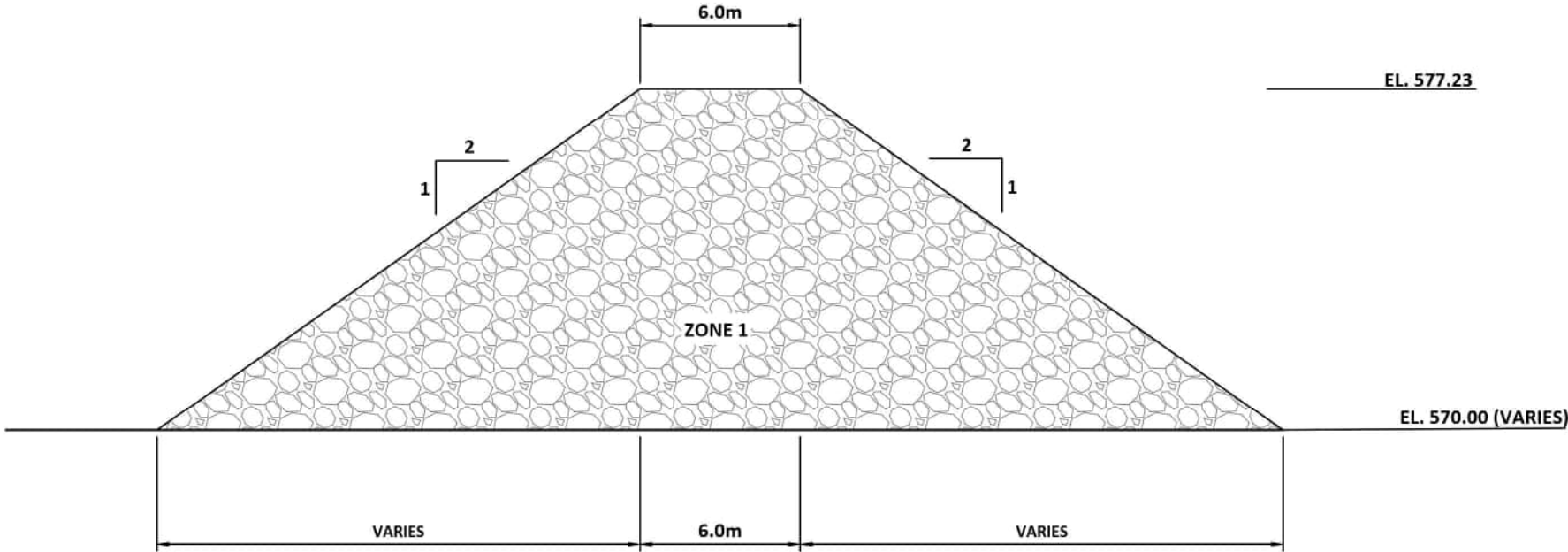
DATE: AUGUST 2023

JOB: 5946-001-00

FIGURE: C1.4

HATCH LEGEND:

 MIXTURE OF RIP RAP &
SMALLER WELL GRADED ROCK



an Englobe company

WATERSMART SOLUTIONS LTD.

GREENVIEW INDUSTRIAL GATEWAY
TEMPORARY COFFER DAM
CROSS SECTION

SCALE:	NTS	DATE:	AUGUST 2023	JOB:	5946-001-00	FIGURE:	C1.6
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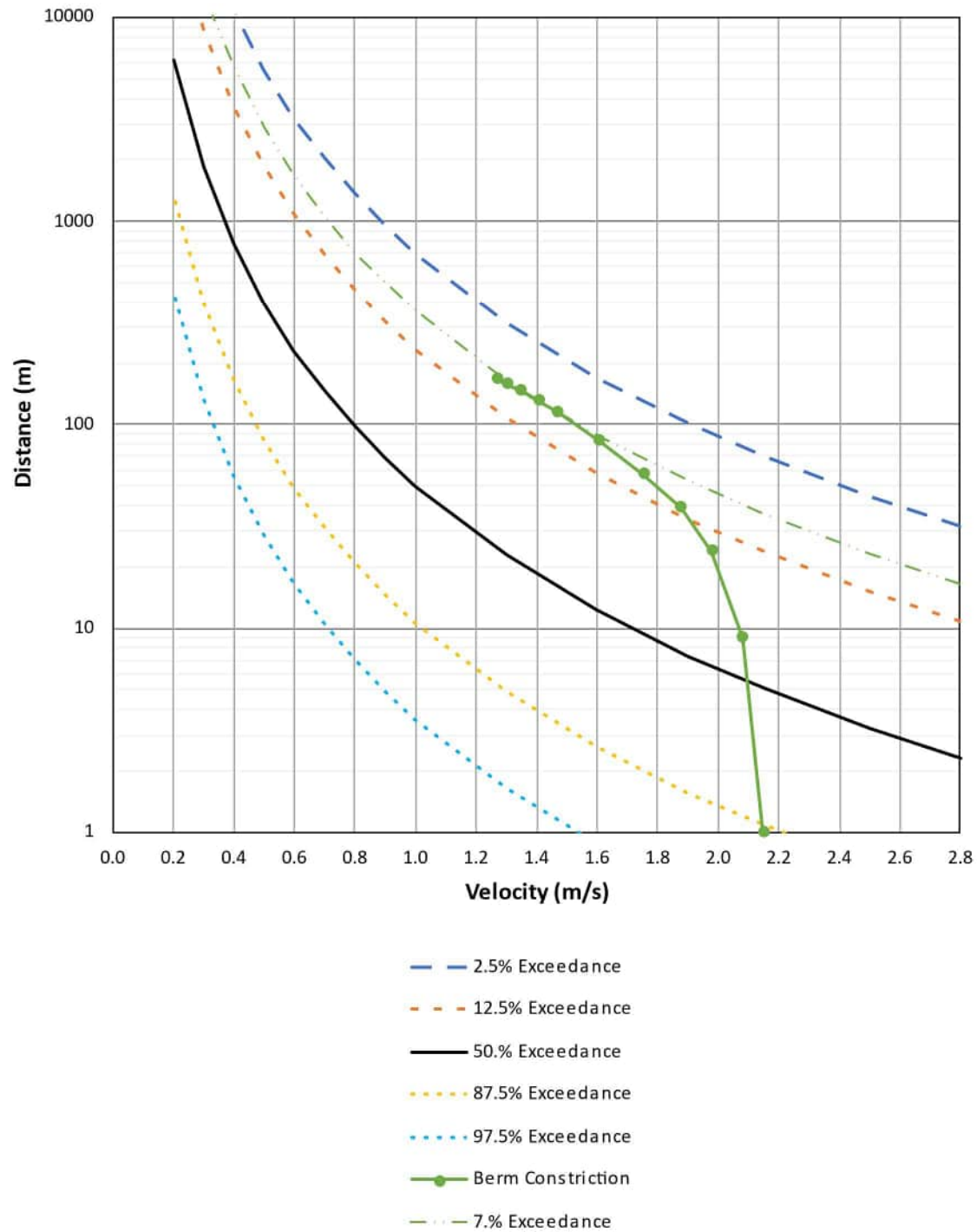
Appendix B

Fish Migration Affected by Berm

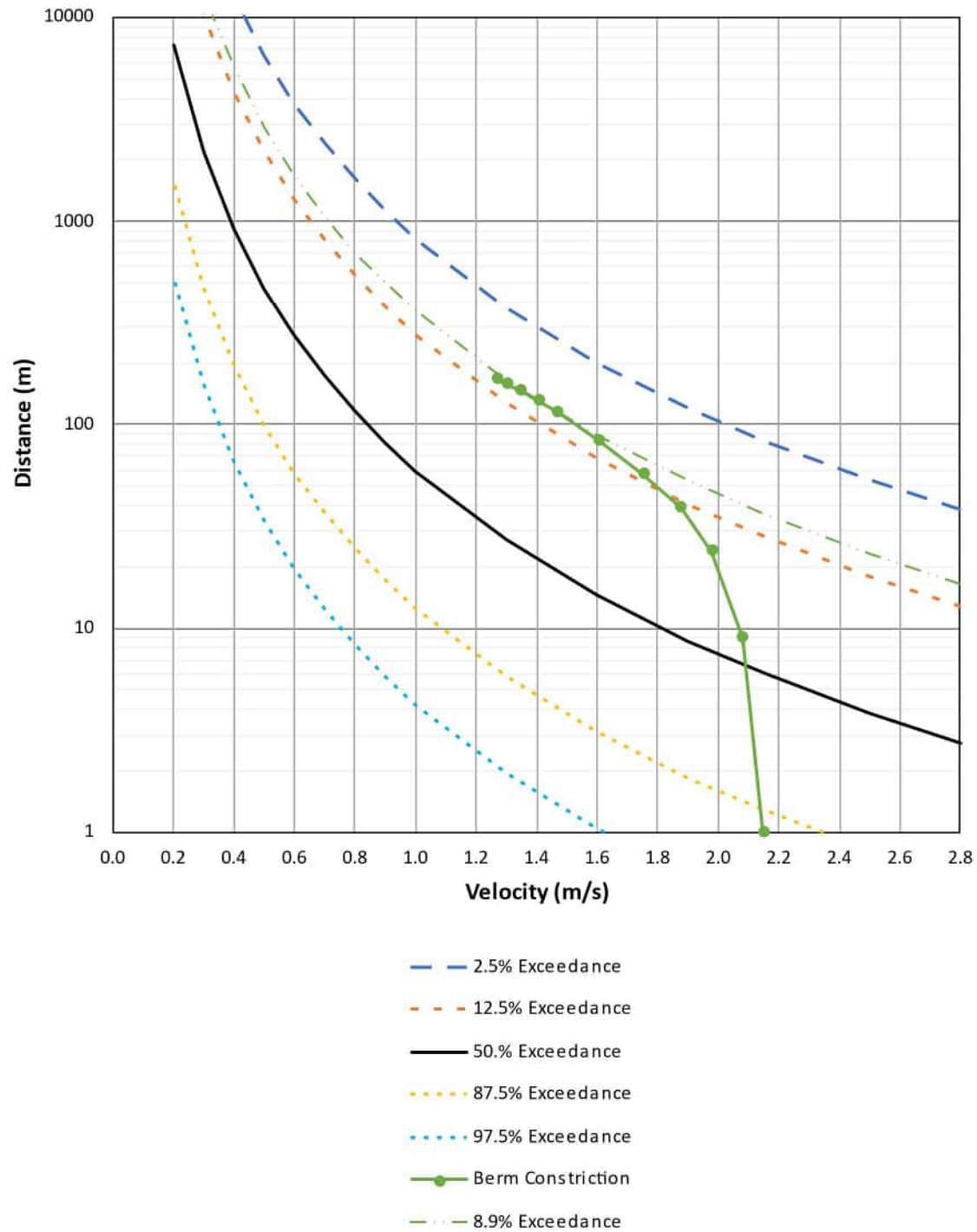
(Velocity-Distance Curves in Berm Constriction
at Median Peak (2-Year) Monthly Flows
compared with Swim Performance Curves)

Month	Fish Species
May	Artic Grayling
May	Rainbow Trout
May	Walleye
May	Longnose Sucker
June	Artic Grayling
June	Rainbow Trout
June	White Sucker
July	White Sucker
August	Bull Trout

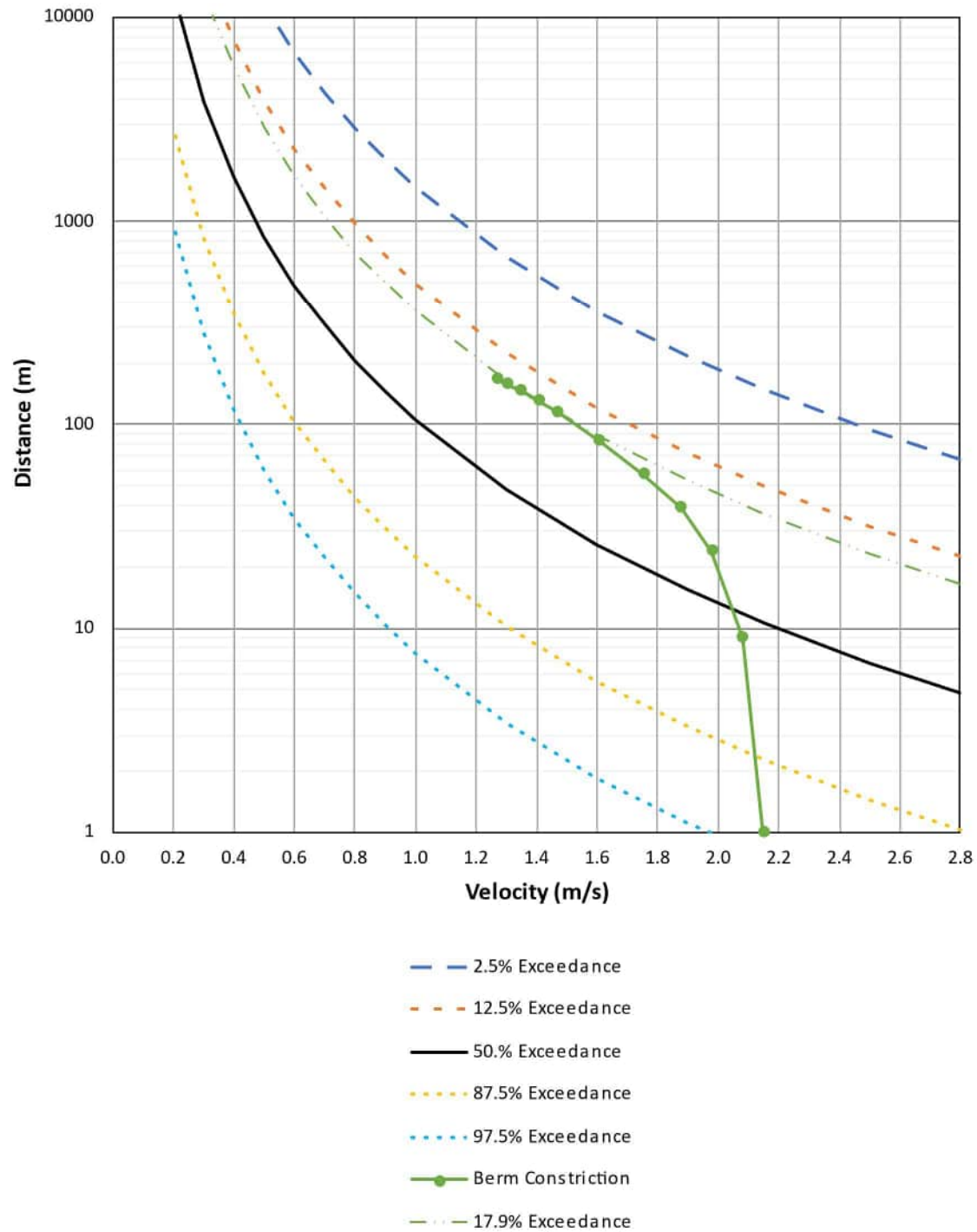
Arctic Grayling, Length = 326 mm, May Median Peak Flow



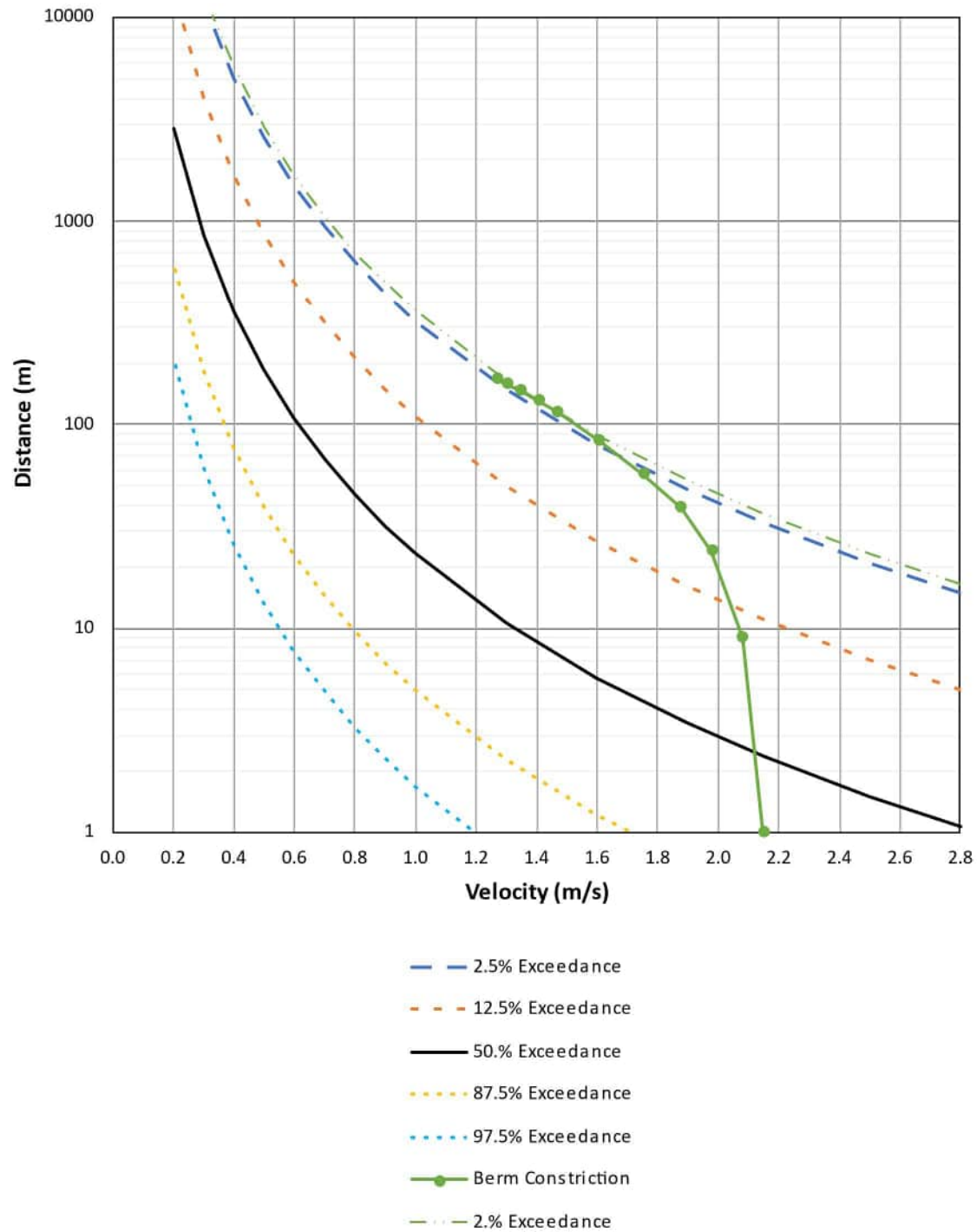
Rainbow Trout, Length = 349 mm, May Median Peak Flow



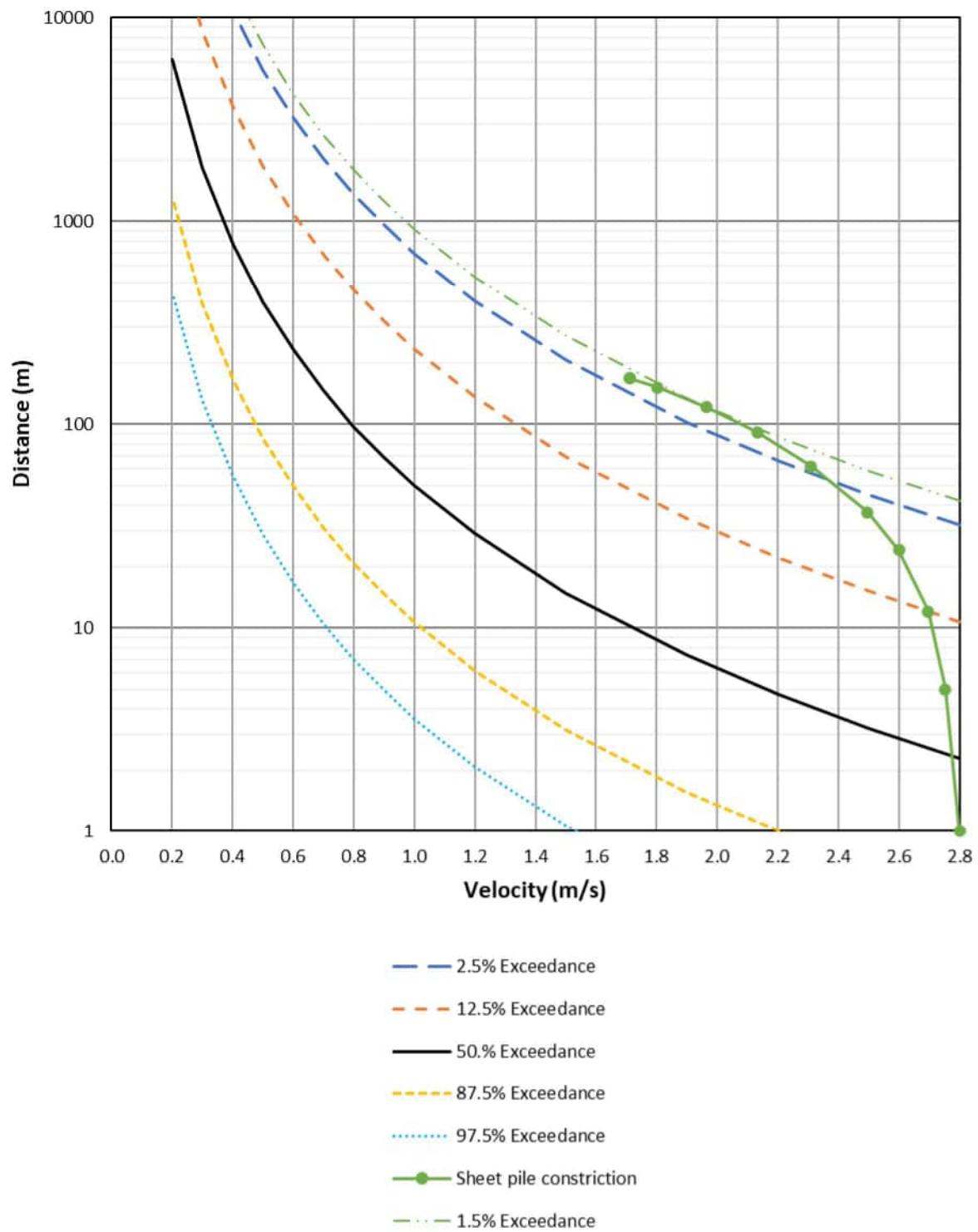
Walleye, Length = 440 mm, May Median Peak Flow



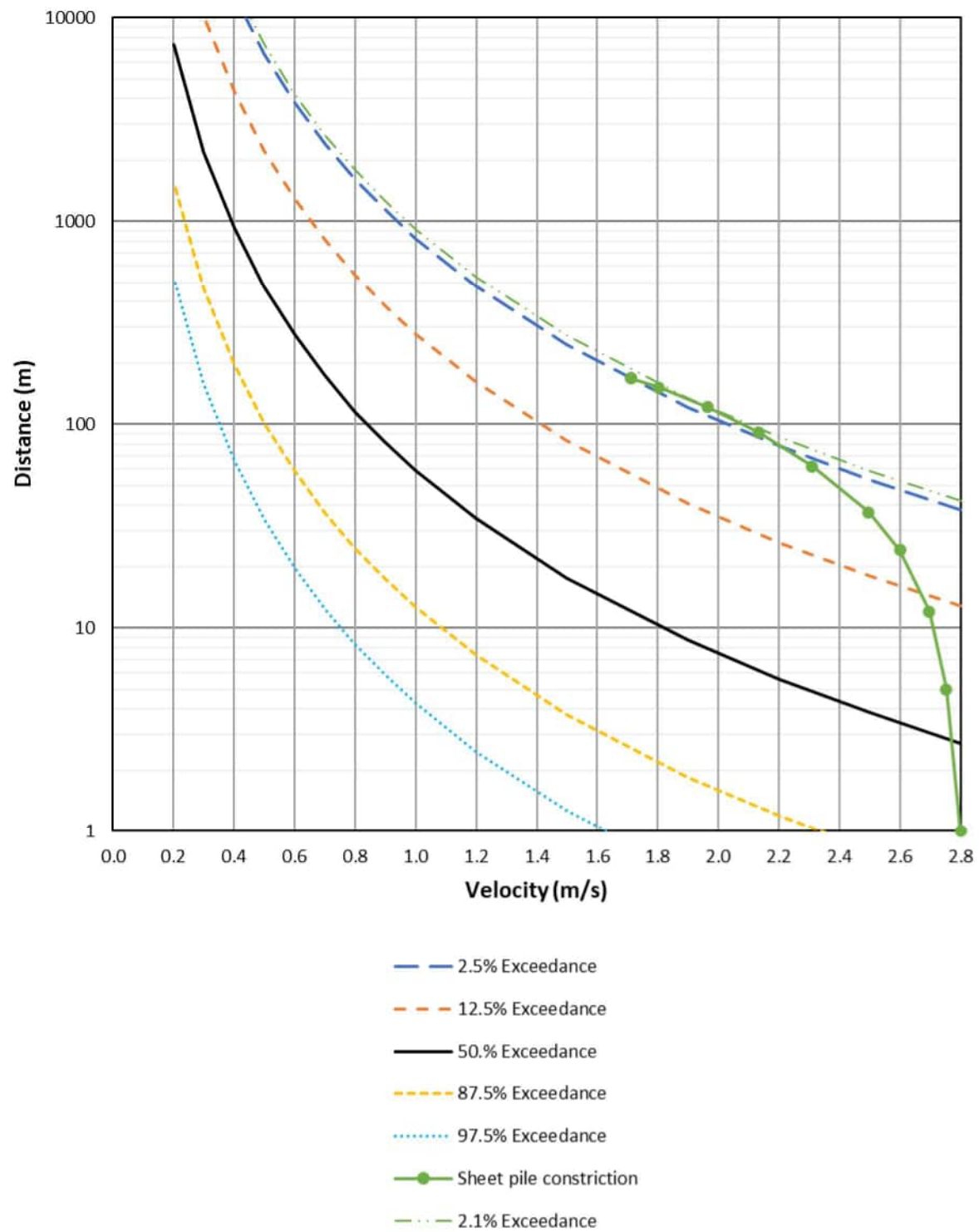
Longnose Sucker, Length = 240 mm, May Median Peak Flow



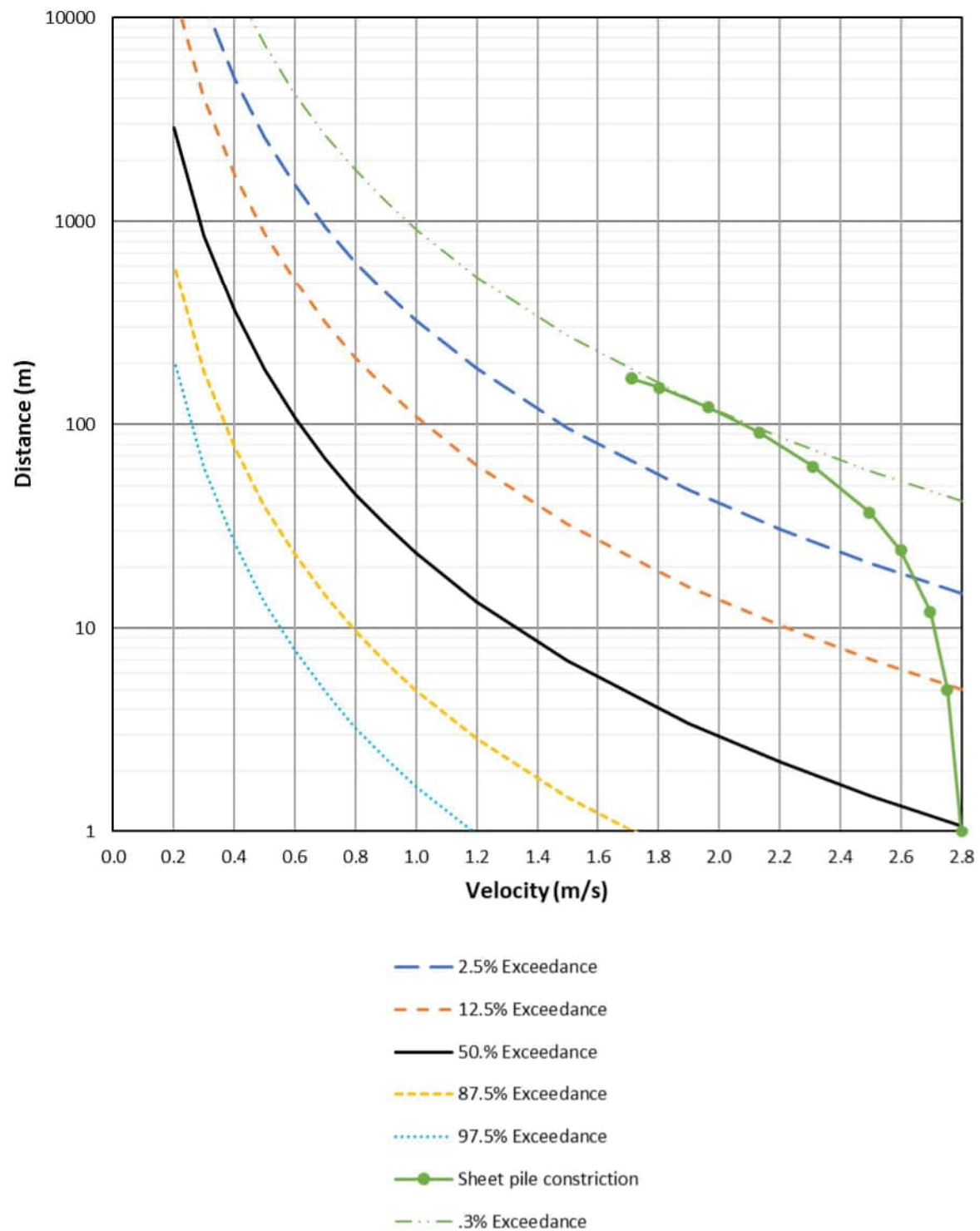
Arctic Grayling, Length = 326 mm, June Median Peak Flow



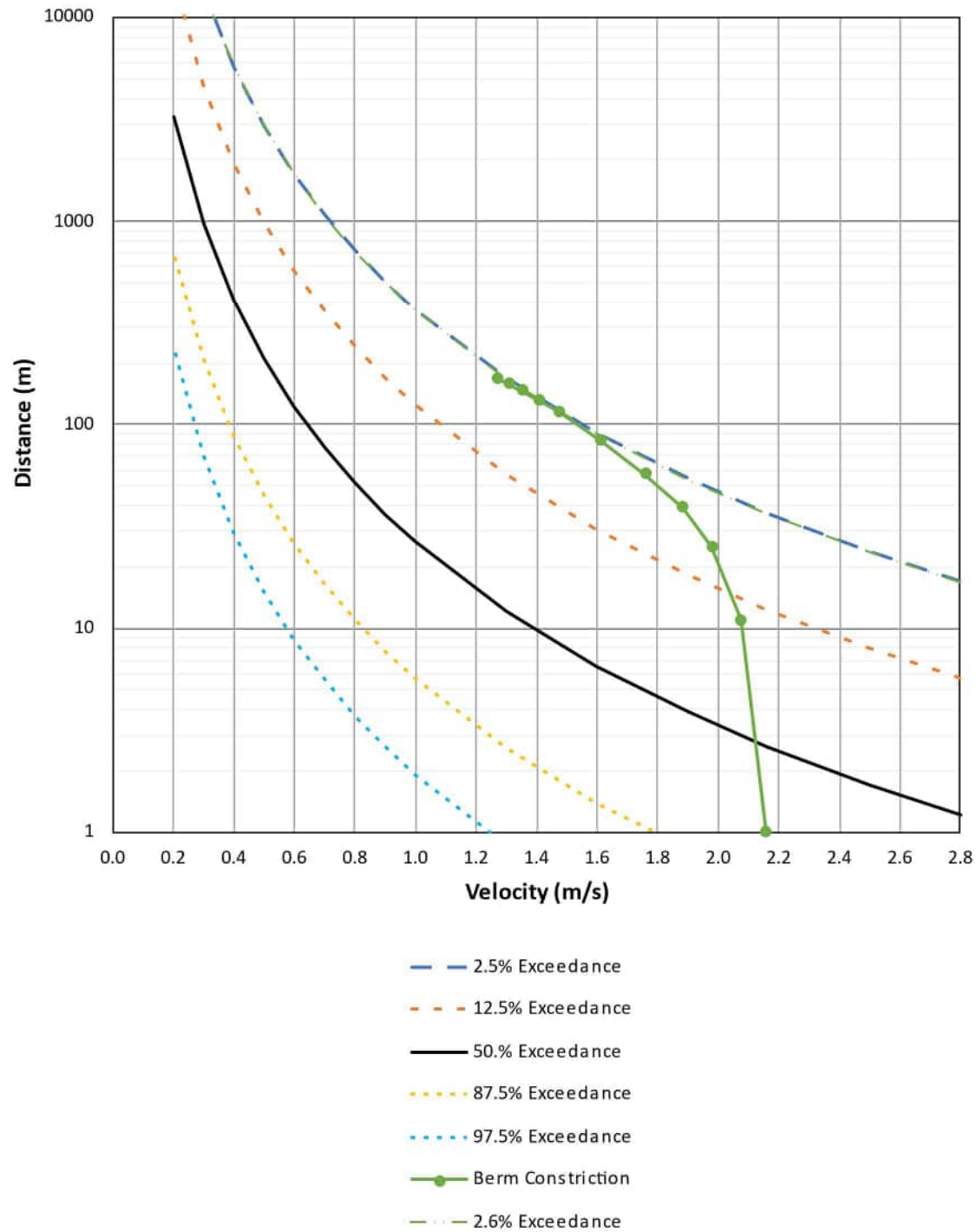
Rainbow Trout, Length = 349 mm, June Median Peak Flow



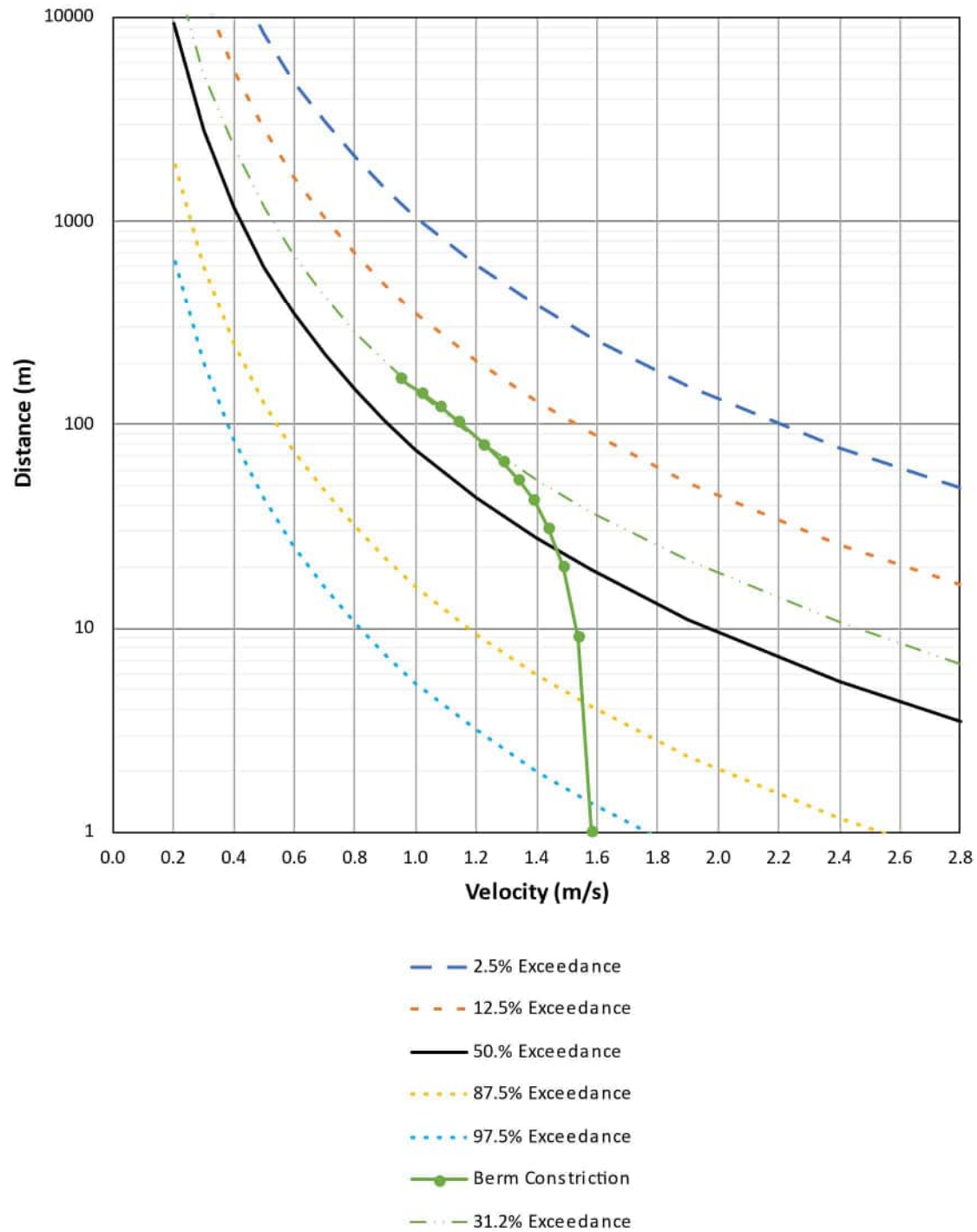
White Sucker, Length = 253 mm, June Median Peak Flow



White Sucker, Length = 253 mm, July Median Peak Flow



Bull Trout, Length = 385 mm, August Median Peak Flow



Appendix G

Steel Pile Cofferdam Hydraulics Report

NHC Ref. No. 1007352-6

July 12, 2024

MPE Engineering
#101, 10630 - 172 Street
Edmonton, AB T5S 1H8

Attention: Haj Oshiki, P.Eng.

Via email: hoshiki@mpe.ca

**Re: Greenview Industrial Gateway Raw Water Intake
Preliminary Assessment of Sheet Pile Cofferdam Hydraulics – Final Report**

Dear Mr. Oshiki:

1 INTRODUCTION

Northwest Hydraulic Consultants Ltd. (NHC) has been asked by MPE Engineering and WaterSMART Solutions Ltd. to provide a preliminary assessment of hydraulic conditions for a sheet pile cofferdam proposed to facilitate the construction of a water intake for the M.D. of Greenview. The water intake will be located on the west bank of the Smoky River in the reach south of the Canfor Cut-Across Road bridge (**Figure 1**). This preliminary assessment of hydraulic conditions was carried out to provide information for regulatory submissions.

The layout of the sheet pile cofferdam is shown in the drawing provided by MPE in Appendix A. The inside toe of the sheet pile will be offset by 10 m from the eventual intake structure so that scour protection can be placed around the intake. The cofferdam will consist of two vertical sheet pile walls spaced 6 m apart with fill material between the two walls. The top of the structure was assumed to be at an elevation of 577.23 m.

The cofferdam is scheduled to be installed in the river beginning on July 15; however, the date that the cofferdam will be removed may range from Mar 31 to July 31 of the following year, or even later if there are delays in construction. Construction of a sheet pile cofferdam would take longer than the two weeks instream work period from July 15 to July 31 and would be easier during the lower flow and water levels in the river typically seen in the fall and winter. Ideally construction of the cofferdam would start in September. Removal of the cofferdam could occur after the peak flow period of the following year, so construction could happen through the highest flow periods in June and early July. It would be better to remove the obstruction as early as possible to avoid the high flow period but, if necessary, it may be removed after the restricted activity ends on July 15.

2 HYDROLOGY

Daily flows in the Smoky River at the intake site were estimated using the techniques describe in NHC's Preliminary Hydrotechnical Design Report dated Feb 24, 2023 (NHC, 2023).

2.1 Peak Flows

Yearly values of peak flow were extracted from the daily flows at the intake site for various construction time periods. These time periods all started on July 15, but the end dates were incremented semi-monthly from March 31 to May 31. Annual peak flows (Jul 15 – Jul 14) were also extracted from the daily flows. The yearly peak flows were then multiplied by a peaking factor of 1.104 (NHC, 2023) before frequency analyses were carried out. Log-Pearson 3 (LP3) distributions were found to fit all six flow series reasonably well as shown in **Figure 2**.

A summary of peak flows for various return periods for each of the six construction time periods is given in **Table 1**. Note that the peak flows for the first two time periods, ending Mar 31 and Apr 15 are the same because spring flows do not increase substantially until after Apr 15. Most of the peak flows occur in June, July or April but some peak flows also occur in August and September.

Table 1 Summary of peak flows for various construction time periods and return periods.

Return Period (years)	Peak Flow Jul 15 to Mar 31 (m ³ /s)	Peak Flow Jul 15 to Apr 15 (m ³ /s)	Peak Flow Jul 15 to Apr 30 (m ³ /s)	Peak Flow Jul 15 to May 15 (m ³ /s)	Peak Flow Jul 15 to May 31 (m ³ /s)	Peak Flow Jul 15 to Jul 14 (m ³ /s)
20	1327	1327	1352	1367	1363	1935
10	1009	1009	1049	1074	1112	1510
5	742	742	790	821	891	1154
2	444	444	491	527	626	753

2.2 Breakup Timing

The timing of breakup was estimated by extracting the dates of the last “B” comment in the Water Survey of Canada (WSC) record for the station Wapiti River near Grande Prairie (07GE001). This station was selected since it was the station with the longest record nearby the intake site and was considered to be representative of breakup timing at the intake site. Ice movement and potentially ice jams may occur in the week leading up to the last “B” comment, which is used to indicate that the water levels are affected by the presence of ice.

The average date of last “B” was Apr 17 with the earliest last “B” occurring Apr 2 and the latest occurring Apr 30; therefore, it is very likely that breakup will occur in the time periods which end on April 30 or later. There is roughly about a 50% chance that breakup will occur between Mar 31 and April 15 but very little chance that it will occur before Mar 31. Therefore, ice jam water levels should be considered for all construction time periods except for Jul 15 to Mar 31.

2.3 Fish Passage Flows

Median daily flows and median peak (2-year) daily flows for each month were extracted from the daily flow record for the Smoky River at the intake site. These flows are summarized in **Table 2**. Flows are typically highest in June and lowest in February.

Table 2 Summary of monthly median and median peak flows

Month	Median Daily Flow (m ³ /s)	Median Daily Peak Flow (m ³ /s)
July	285	479
August	161	236
September	107	155
October	76.8	123
November	24.9	50.4
December	15.1	19.9
January	12.0	14.2
February	9.9	11.1
March	10.5	16.4
April	62.8	200
May	251	475
June	392	635

3 HYDRAULIC MODELLING

Hydraulic analysis of the river reach was carried out using the Hydrological Engineering Center's River Analysis System (HEC-RAS) one-dimensional (1D) numerical hydraulic model developed for the project (NHC, 2023). The sheet pile cofferdam geometry provided by MPE was added to the 1D model so that preliminary estimates of water levels and velocities could be obtained to provide input for regulatory submissions. A two-dimensional (2D) model of the reach should be developed during the detailed design phase to provide more detailed results for detailed design of the sheet pile cofferdam.

3.1 Peak Water Levels

A summary of the peak open water levels for each construction time period for various return periods is given in **Table 3**. A range of flow conditions is provided so that the contractor can make an informed decision on risk when selecting the final top of cofferdam elevation. The water levels reported in **Table 3** are the water levels expected on the upstream side of the cofferdam. The preliminary top of cofferdam elevation of 577.23 m provided by MPE is only exceeded during the 20-year peak water levels.

Note that for all the time periods except the time period ending Mar 31, peak ice jam water levels will likely occur as well. Peak ice jam levels are unaffected by the presence of the cofferdam. The ice jam levels listed in **Table 3** were obtained from NHC's Preliminary Hydrotechnical Design Report dated Feb 24, 2023 (NHC, 2023). The maximum open water levels which are lower than the ice jam water level

are shaded in yellow. The ice jam water levels are higher in every case except for the 2-year event for the Jul 15 to Jul 14 period.

Table 3 Summary of peak water levels

Return Period (years)	Open Water Jul 15 to Mar 31 ¹ (m)	Open Water Jul 15 to Apr 15 (m)	Open Water Jul 15 to Apr 30 (m)	Open Water Jul 15 to May 15 (m)	Open Water Jul 15 to May 31 (m)	Open Water Jul 15 to Jul 14 (m)	Ice Jam (m)
20	576.53	576.53	576.57	576.60	576.59	577.66	578.0
10	575.90	575.90	575.98	576.03	576.11	576.87	577.1
5	575.33	575.33	575.44	575.51	575.66	576.19	576.4
2	574.88	574.88	575.02	574.80	575.06	575.36	575.2

¹ Ice jams do not occur in this period.

The maximum open water levels which are lower than the ice jam water level are shaded in yellow.

3.2 Maximum Velocities

A summary of the maximum velocities for each construction time period for various return periods is given in **Table 4**. The velocities reported in **Table 4** are the cross section average velocities occurring where the channel has been constricted the most by the sheet pile cofferdam. Higher local velocities may occur, but a more detailed 2D model is required to determine the magnitude and location of these local peak velocities.

The velocities listed in **Table 4** are based on the existing bed geometry without allowance for bed scour. Given the magnitudes of these velocities, significant bed scour is likely to occur in the constriction at high flows.

Table 4 Summary of maximum velocities in cofferdam constriction without scour for various construction time periods and return periods.

Return Period (years)	Peak Velocity Jul 15 to Mar 31 (m/s)	Peak Velocity Jul 15 to Apr 15 (m/s)	Peak Velocity Jul 15 to Apr 30 (m/s)	Peak Velocity Jul 15 to May 15 (m/s)	Peak Velocity Jul 15 to May 31 (m/s)	Peak Velocity Jul 15 to Jul 14 (m/s)
20	4.09	4.09	4.15	4.18	4.17	5.57
10	3.38	3.38	3.47	3.54	3.62	4.51
5	2.74	2.74	2.85	2.93	3.09	3.71
2	1.74	1.74	1.86	2.17	2.44	2.76

3.3 Bed Scour

Bed scour in the sheet pile cofferdam constriction was estimated using the competent mean velocity approach (TAC, 2004). This method determines the velocity at which the bed material is stable and

calculates the amount of bed scour needed to reduce the velocity to this value. The median bed material size is 52 mm which, for the range of flow depths at the cofferdam, results in a competent mean velocity of about 2.5 m/s. The scour depths required to reduce the average velocities to the competent mean velocity are listed in **Table 5** for each construction time period for various return periods.

Note that the scour depths listed in **Table 5** may be limited by the presence of bedrock. Bedrock outcrops occur on both banks, so it is quite likely the bedrock also exists under the channel. However, the presence of bedrock and its elevation have not been confirmed.

Table 5 Summary of bed scour depths in sheet pile cofferdam constriction for various construction time periods and return periods.

Return Period (years)	Scour Depth Jul 15 to Mar 31 (m)	Scour Depth Jul 15 to Apr 15 (m)	Scour Depth Jul 15 to Apr 30 (m)	Scour Depth Jul 15 to May 15 (m)	Scour Depth Jul 15 to May 31 (m)	Scour Depth Jul 15 to Jul 14 (m)
20	2.4	2.4	2.5	2.6	2.5	4.8
10	1.3	1.3	1.4	1.5	1.6	3.1
5	0.3	0.3	0.5	0.6	0.8	1.8
2	0.0	0.0	0.0	0.0	0.0	0.4

4 FISH PASSAGE VELOCITIES

Fish passage was assessed by evaluating velocities during the median monthly flows and the median peak (2-year) monthly flows. These velocities were then compared to the swimming speeds for the various fish species found in the river reach.

4.1 Maximum Average Velocities

The maximum cross-section average velocities in the natural river reach and in the downstream bridge waterway are compared to the maximum velocities in the sheet pile cofferdam constriction for the mean daily flow and median daily peak flow for each month in **Table 6**. Months where the maximum average velocity in the sheet pile cofferdam constriction is greater than anywhere in the natural reach are highlighted in yellow. This occurs in May, June and July at the median peak flows and in June at the median flow. In the other months, higher velocities occur at other cross sections due to shallow depths at riffle sections. As well, in all cases highlighted in yellow, the maximum average velocity in the bridge waterway is also greater than the maximum average in the sheet pile cofferdam constriction. This bridge constriction is unlikely to be widened before the cofferdam is removed.

The peak velocity analysis indicates that the cofferdam will only produce a barrier to fish passage in May, June and July. Furthermore, cofferdam construction will occur in late July so the sheet piles will likely not fully constrict the flow until the end of July. Also, if construction could be completed before May 1, fish passage in May and June would not be affected.

Table 6 Summary of peak velocities for the median and median peak for each month

Month	Peak Velocity at the Median Daily Flow (m/s)			Peak Velocity at the Median Daily Peak Flow (m/s)		
	Natural	Bridge	at Cofferdam	Natural	Bridge	at Cofferdam
July	1.77	1.78	1.64	1.80	2.22	2.17
August	2.14	1.41	1.17	2.09	1.63	1.47
September	2.02	1.20	0.89	2.14	1.39	1.14
October	2.02	1.11	0.70	2.07	1.27	0.98
November	1.58	0.67	0.28	1.78	0.86	0.49
December	1.63	0.56	0.19	1.55	0.62	0.24
January	1.70	0.51	0.16	1.70	0.55	0.18
February	1.65	0.47	0.13	1.67	0.49	0.15
March	1.64	0.48	0.14	1.55	0.58	0.20
April	1.93	1.04	0.60	2.21	1.53	1.33
May	1.94	1.68	1.52	1.79	2.21	2.16
June	1.73	2.02	1.96	2.30	3.07 ¹	3.00 ¹

¹ These velocities will be limited to 2.5 m/s due to bed scour in the sheet pile cofferdam constriction.

The maximum velocities in the sheet pile cofferdam constriction which are higher than the maximum natural velocities are shaded in yellow.

4.2 Fish Characteristics

Fish characteristics in the intake reach were provided by MPE and are summarized in **Table 7**. There are nine species listed. The group summary indicated in **Table 7** is based on the categories provided in the Swim Performance Online Tools (SPOT) website (Di Rocco, R, and R. Gervais, 2023) provided by the Department of Fisheries and Oceans (DFO). Three fish groups were identified based on the fish characteristics listed in **Table 7**: Eel, Pike, and Salmon & Walleye.

Fish may move around in the river at any time of year but migration, when upstream movement is critical, are limited to specific periods of the year. **Table 7** lists the months in which migration is most likely to occur for each species. Of the species listed, only five are likely to migrate in May, June, or July when the cofferdam may produce a barrier to fish migration. These five species are all in the Salmon and Walleye group, which are stronger swimmers. Burbot and Northern Pike, which are weaker swimmers, do not migrate during this period.

Table 7 Summary of Fish Characteristics in the Intake Reach

Species	DFO Group	Mean Length at First Maturity (mm)	Migration Period
Burbot	Eel	370	Nov-Feb
Northern Pike	Pike	420	Feb-Apr
Arctic Grayling	Salmon & Walleye	326	Mar-Jun
Bull Trout	Salmon & Walleye	385	Aug
Mountain Whitefish	Salmon & Walleye	230	Sep-Nov
Rainbow Trout	Salmon & Walleye	349	May-Jun
Walleye	Salmon & Walleye	440	Mar-May
Longnose Sucker	Salmon & Walleye	240	Apr-May
White Sucker	Salmon & Walleye	253	Jun-Jul

4.3 Fish Swimming Speeds

Fish swim performance was assessed using information obtained from the Fish Swimming Performance Database and Analyses (Katopodis, C, and R. Gervais, 2016) which provides the basis for the SPOT website. This document indicates that the relationship for fish swimming velocity, V and distance, X is described by the following equation:

$$\frac{X}{L} = M \left(\frac{V}{\sqrt{gL}} \right)^a$$

where g is the acceleration due to gravity, L is the length of the fish, and M and a are constants that vary with fish species and the probability of exceedance for a given performance. SPOT groups fish species in six groups and provides M and a values for each of these groups for various probabilities of exceedance. Investigation of the variation of M and a provided for these groups indicated that a was virtually constant for each group while M varied with probability of exceedance according to a log-normal distribution.

An example of the fish swim performance curves generated using the above analyses is shown in **Figure 3**. This figure shows the swim performance curves for a 349 mm long Rainbow Trout for the standard 2.5%, 12.5%, 50%, 87.5%, and 97.5% probability of exceedances provided by SPOT (these exceedances are the bounds of the 75% and 95% prediction intervals). Also included in **Figure 3** are the velocity-distance curves for the May median peak flow evaluated at the sheet pile cofferdam constriction. The line labeled 39.3% exceedance passes through the maximum velocity-distance combination for the sheet pile cofferdam constriction, which indicates that about 40% of the 349 mm Rainbow Trout would be able to pass through the sheet pile cofferdam constriction at this discharge.

Similar evaluations of fish swim performance curves versus velocity-distance curves were carried out for each of the five fish species which may migrate in May, June and July for the months when each species

may be migrating. A summary table of the degree of fish passage affected is provided in **Table 8**. Figures comparing the swim performance curves for each species with velocity-distance curves for the flow rates are provided in **Appendix B**. The velocities used in this velocity-distance analysis are cross section average velocities adjusted to a 1 m depth because fish will tend to swim in the zone of lower velocity near the bank on the opposite side of the channel from the sheet pile cofferdam. Note that as little as 5% of some species will be able to pass upstream through the constriction without enhancing passage.

As shown in figures in **Appendix B**, the length that the fish can swim against the peak velocity is quite limited. However, if refuge is provided at the short intervals, the fish can rest between short bursts of swimming. For example, boulder clusters are commonly used to provide resting areas for fish to improve passage. The short distances needed, however, will require an almost continuous series of boulders rather than widely spaced clusters. However, any fish migrating upstream will first need to pass through the bridge constriction which has higher velocities than the sheet pile cofferdam constriction. It may not be necessary to provide boulder clusters, since any fish arriving at the sheet pile cofferdam constriction should be able to pass upstream since they had already successfully passed through the bridge constriction.

Table 8 Summary of percentage of fish able to pass upstream through the sheet pile cofferdam constriction at the median peak flow for months when constriction velocities are higher than maximum natural velocities.

Species	Percentage Passing Upstream			Required Boulder Spacing (m)		
	May Median Peak Flow	Jun Median Peak Flow	Jul Median Peak Flow	97.5% Passage	87.5% Passage	50% Passage
Arctic Grayling	34.6	13.7		0.8	2.3	10.8
Rainbow Trout	39.3	16.7		0.9	2.7	12.8
Walleye	56.4			4.6	13.7	64.4
Longnose Sucker	16.7			1.0	3.0	14.2
White Sucker		4.8	19.0	0.4	1.1	5.0

5 EROSION PROTECTION

Erosion protection is not required on the exposed surface of the sheet piles but may be needed at the river bed to protect against bed scour. Erosion protection is recommended for the opposite bank to ensure that changes in flow patterns due to the presence of the sheet pile cofferdam do not permanently alter the flow pattern at the intake.

The size of riprap required to protect against erosion depends on its location within the river. The maximum velocities in the existing section (without scour) are listed in **Table 4**. These velocities will

occur at the base of the sheet pile cofferdam if no scour occurs in the cross section. Lower velocities would occur along the opposite bank because depths are lower there.

The size of riprap also depends on the amount for scour that occurs in the constricted section. As scour increases, the average velocity decreases. If bedrock does not limit scour, the average velocity in the constriction will be limited to 2.5 m/s, the velocity required to transport the bed material. Velocities may exceed 2.5 m/s in the short term as bed material transport may occur more slowly than the velocity increase. For this analysis it was assumed that bedrock would limit the scour level in the constriction to the existing minimum bed elevation but that this would occur across the entire constricted section. Average velocities in the constriction range from 1.8 to 3.6 m/s for the 5-year to 20-year peak flows during the construction time periods. Peak velocities due to the flow acceleration along the cofferdam may be up to 1.5 times these values. Two-dimensional modeling should be carried out to confirm these flow conditions before construction is carried out.

5.1 Sheet Pile Cofferdam

The sheet pile cofferdam provides its own erosion protection; and, if the sheet piles are driven down to the bedrock elevation, no further protection is required. However, if the sheet piles are not driven down to the bedrock elevation, they must be deep enough to extend below the expected scour level. The constriction scour depth predicted for a 5-year peak flow for a constriction time period extending to July 14, or later is about 1.8 m below the existing thalweg but local variations may occur so it is recommended that the local scour depth be taken as twice the average scour level, or 565.6 m so the sheet piles must extend below this level to provide protection against this event.

5.2 Opposite Bank

The erosion protection on the bank opposite the cofferdam should extend along the bank as shown in **Figure 4**. Part of this extent should be designed as permanent protection as indicated in the Preliminary Hydrotechnical Design Report for the intake (NHC, 2023). Both the permanent and temporary protection extents are shown in **Figure 4**.

5.2.1 Permanent Erosion Protection

The permanent erosion protection should be designed to withstand the 100-year peak flow. As stated in the Preliminary Hydrotechnical Design Report for the intake, the permanent protection should be Class 1 riprap with a median diameter of 0.3 m. The riprap should be 0.5 m thick and extend up to an elevation of 579.7 m which will provide 0.5 m of freeboard above the 100-year water level. As shown in the cross section in **Figure 5**, this will require a small berm to project above the existing bank. Bedrock exists along the base of this bank so the riprap should extend down to the top of bedrock. No scour protection is required due to the presence of the bedrock; however, if the bedrock slopes toward the river, it may be necessary to modify the bedrock surface to improve riprap stability.

5.2.2 Temporary Erosion Protection

The temporary riprap protection was designed to protect against the average scour level predicted for a 5-year peak flow for a constriction time period extending to July 14, which is the same as the annual

peak flow. The bank slope on this side of the river is quite mild, so it is recommended that the entire riprap volume be placed as an apron, which will launch into the correct location if scour occurs. This will limit the disturbance to the bank in this temporary protection area by not requiring bank recontouring. The velocities along this portion of the bank are expected to require the use of Class 1 riprap with a median diameter of 0.3 m to provide erosion protection.

The toe of the riprap should be placed at elevation 573.14 m. Placing the toe of the riprap at this elevation will allow the riprap toe to function as a continuous series of boulder clusters when the water level is above the July median water level as is needed for fish passage. Incorporating the boulder clusters into the riprap will also satisfy navigation requirements by projecting the top of riprap above the 2-year annual water level. The July median water level is 574.14 m so some of the riprap will need to be installed in the water.

The configuration of the temporary riprap is shown in **Figure 5**. The riprap surface should rise from the toe location away from the river at a 2H:1V slope. The top of riprap should be at elevation 576.69 m, which will provide 0.5 m of freeboard above the annual 5-year peak flow water level (Jul 15 to Jul 14). The top of riprap should extend horizontally 3.0 m before sloping back down to ground level, again at a 2H:1V slope.

6 CONCLUSIONS

The preliminary analysis of sheet pile cofferdam hydraulics analysis indicated the following:

- 1) Peak flows do not change if the construction period is extended from Mar 31 to Apr 15.
- 2) Peak flows increase as the construction period extends from Apr 15 to May 31 but peak flows are greatest if the construction period spans the entire year from Jul 15 to Jul 14.
- 3) Probability of breakup occurring during construction time period increases to about 50% if the construction period is extended from Mar 31 to Apr 15
- 4) Probability of breakup occurring during construction time period increases to about 100% if the construction period is extended to Apr 30 or later.
- 5) Monthly median and median peak flow are highest in June and lowest in February.
- 6) Ice jam water levels are higher than peak open water levels in most cases.
- 7) Peak velocities in the sheet pile cofferdam constriction increase significantly with return period but are less sensitive to construction time period.
- 8) Monthly median and median peak velocities in the sheet pile cofferdam constriction are only higher than the natural peak velocities in the reach in May, June, and July.
- 9) As little as 5% of some species will be able to pass upstream through the constriction during their migration period in May, June, July, and August without enhancing fish passage.

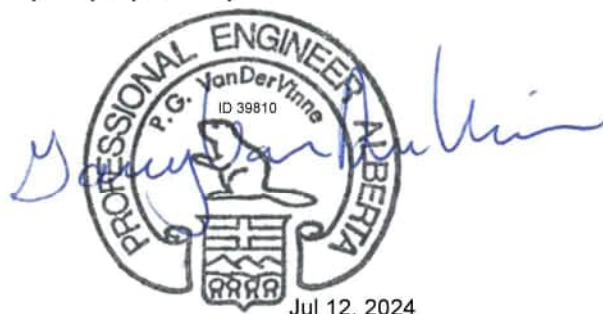
- 10) A nearly continuous series of boulders is needed to meet the short spacing needed for passage of some fish species. The continuous series of boulders at the toe of the temporary riprap on the opposite bank will satisfy this requirement.
- 11) For scour protection, it is recommended that the sheet piles be driven down to bedrock or to elevation 565.6 m, whichever is higher.
- 12) It is recommended that riprap median diameter of 0.3 m be used to provide temporary protection on the opposite bank.
- 13) Two-dimensional modelling should be carried out to refine this analysis before the sheet pile cofferdam is placed in the river.

7 CLOSURE

Sincerely,

Northwest Hydraulic Consultants Ltd.

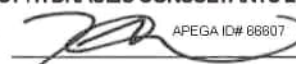
Report prepared by:



Jul 12, 2024

Gary Van Der Vinne, M.Sc., P.Eng.
Principal

Report reviewed by:

PERMIT TO PRACTICE	
NORTHWEST HYDRAULIC CONSULTANTS LTD.	
Signature	 APEGA ID# 68607
Date	12 Jul 2024
PERMIT NUMBER: P 654	
The Association of Professional Engineers and Geoscientists of Alberta	

Robyn Andrishak, M.Sc., P.Eng.
Principal

DISCLAIMER

This report has been prepared by **Northwest Hydraulic Consultants Ltd.** for the benefit of MPE Engineering for specific application to the Preliminary Assessment of Sheet Pile Cofferdam Hydraulics for the Greenview Industrial Gateway Intake on the Smoky River. The information and data contained herein represent **Northwest Hydraulic Consultants Ltd.** best professional judgment in light of the knowledge and information available to **Northwest Hydraulic Consultants Ltd.** at the time of preparation and was prepared in accordance with generally accepted engineering and geoscience practices.

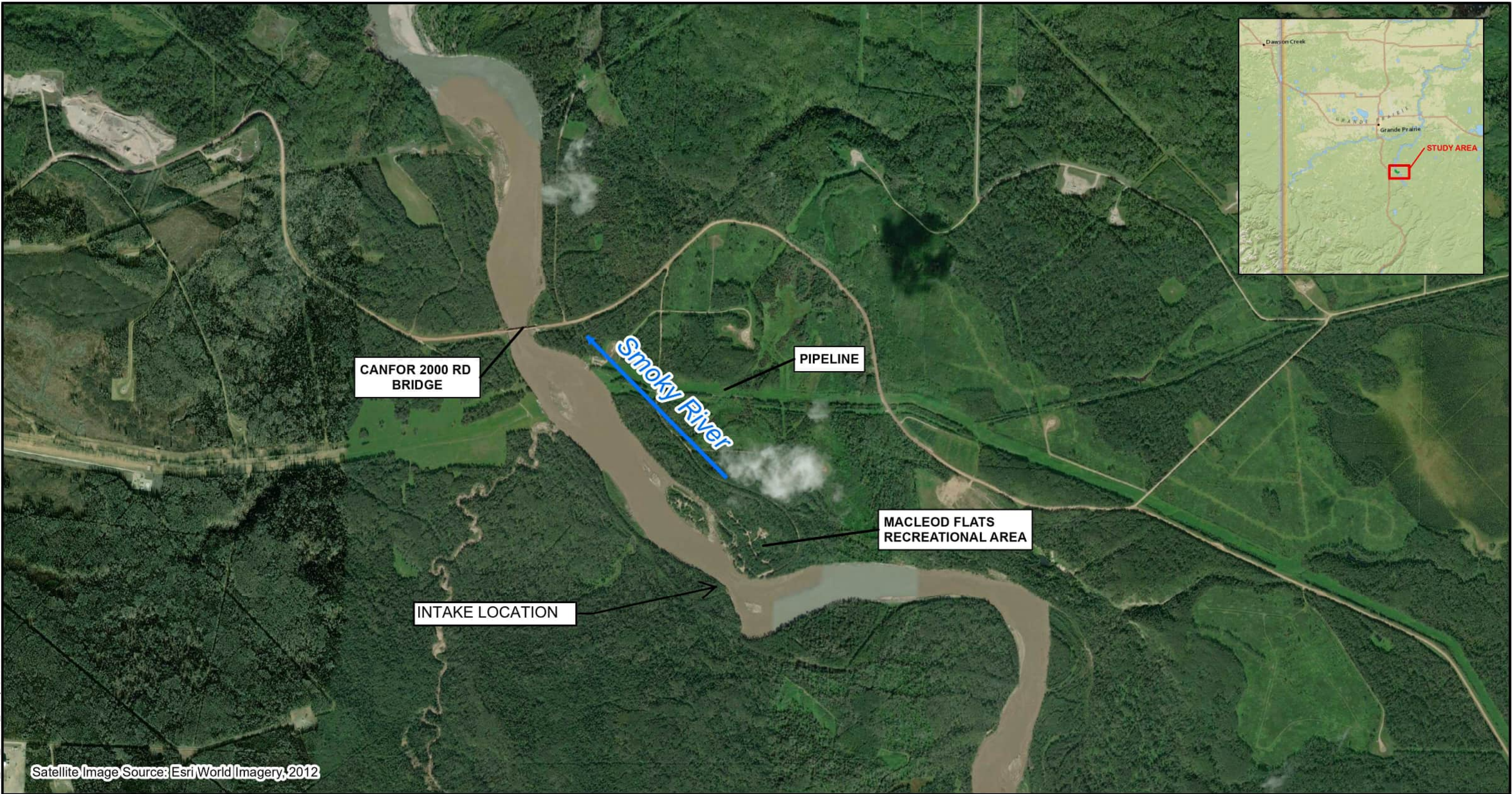
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8 REFERENCES

- Di Rocco, R, and R. Gervais. 2023. SPOT: Swim Performance Online Tools. Available from <https://fishprotectiontools.ca/>.
- Katopodis, C, and R Gervais. 2016. Fish Swimming Performance Database and Analyses. DFO Can. Sci. Advis. Sec. Res. Doc. 2016/002., 550. Available from http://www.dfo-mpo.gc.ca/csas-sccs/Publications/ResDocs-DocRech/2016/2016_002-eng.html.
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FIGURES

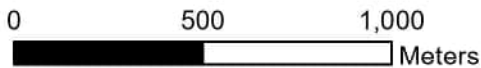
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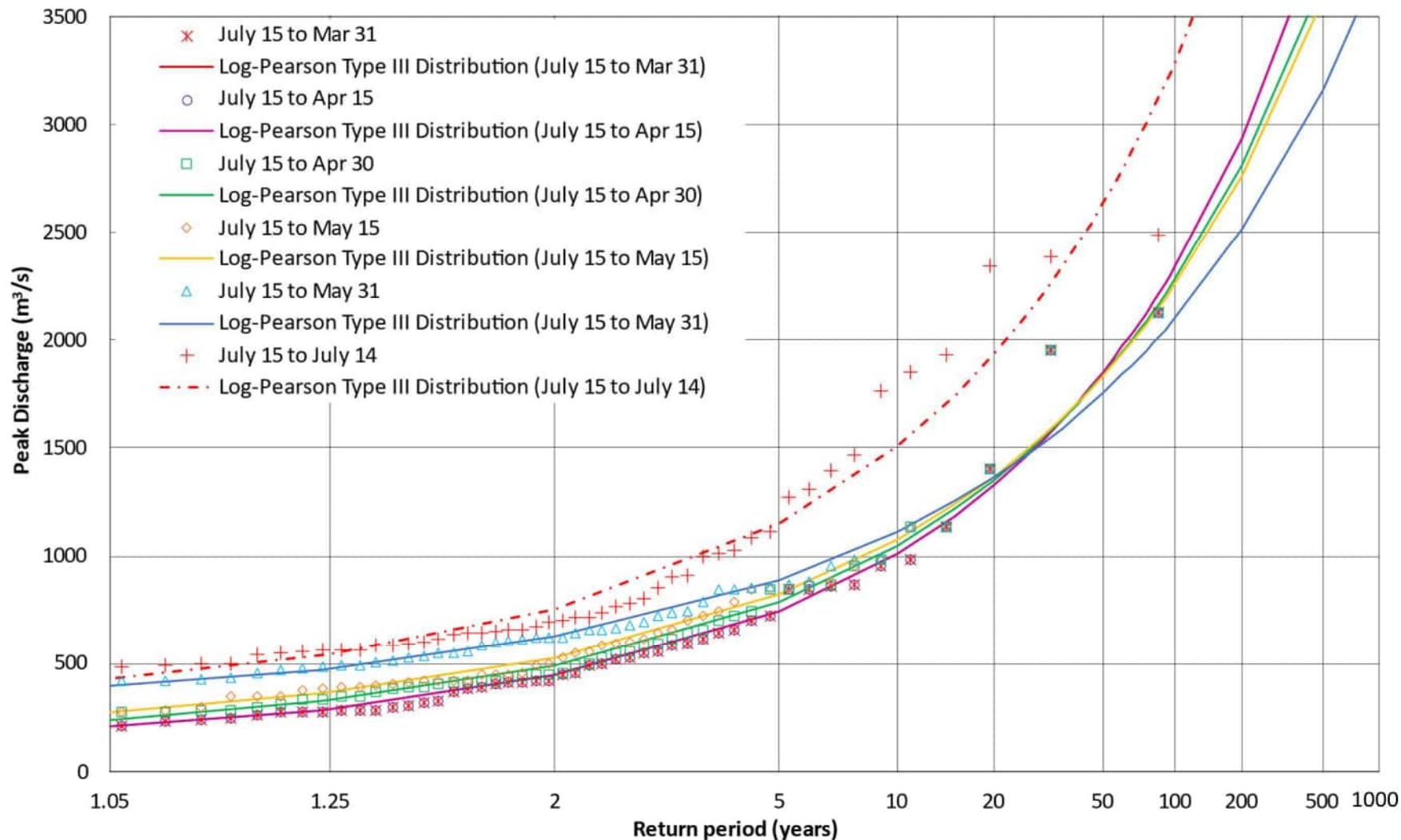
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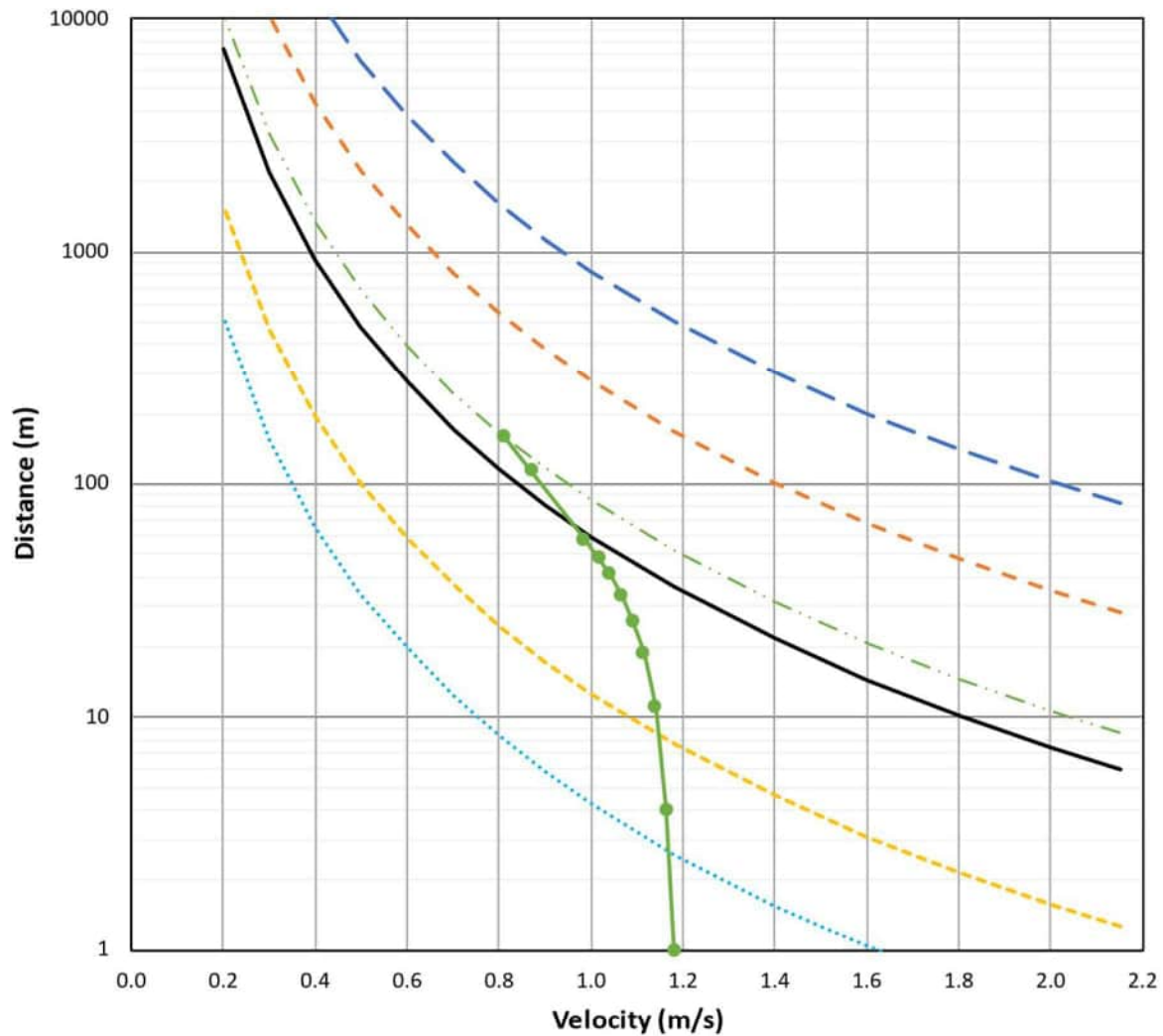
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MPE ENGINEERING		
GREENVIEW INDUSTRIAL GATEWAY WATER INTAKE		
PRELIMINARY ASSESSMENT OF SHEET PILE COFFERDAM HYDRAULICS STUDY REACH AND LOCATION		
1007352-6	2024-07-12	Figure 1

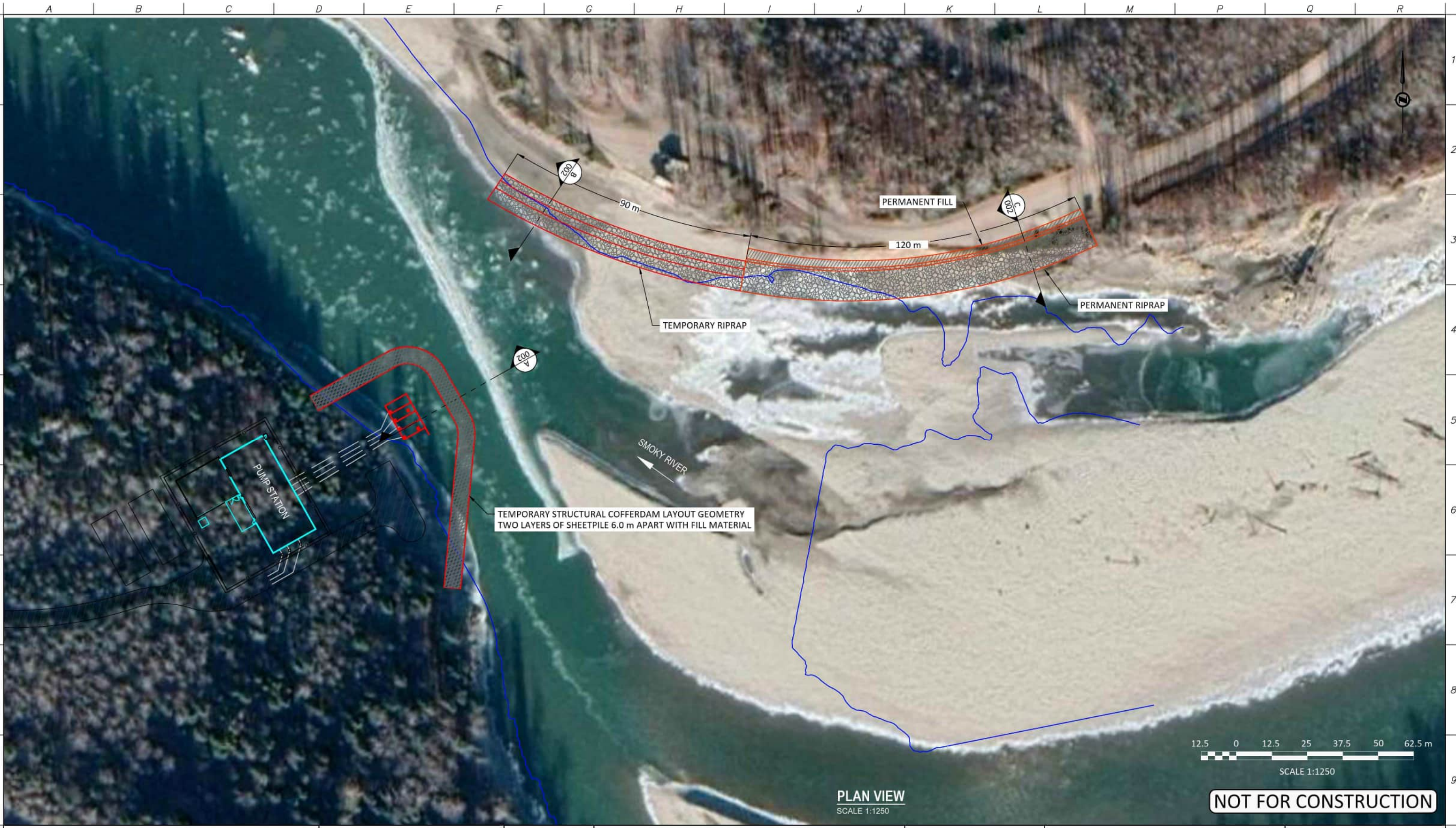


RainbowTrout, Length = 349 mm, May Median Peak Flow



- 2.5% Exceedance
- - - 12.5% Exceedance
- 50% Exceedance
- - - 87.5% Exceedance
- ... 97.5% Exceedance
- Sheet pile constriction
- . - 39.3% Exceedance

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PLAN VIEW
SCALE 1:1250

NOT FOR CONSTRUCTION



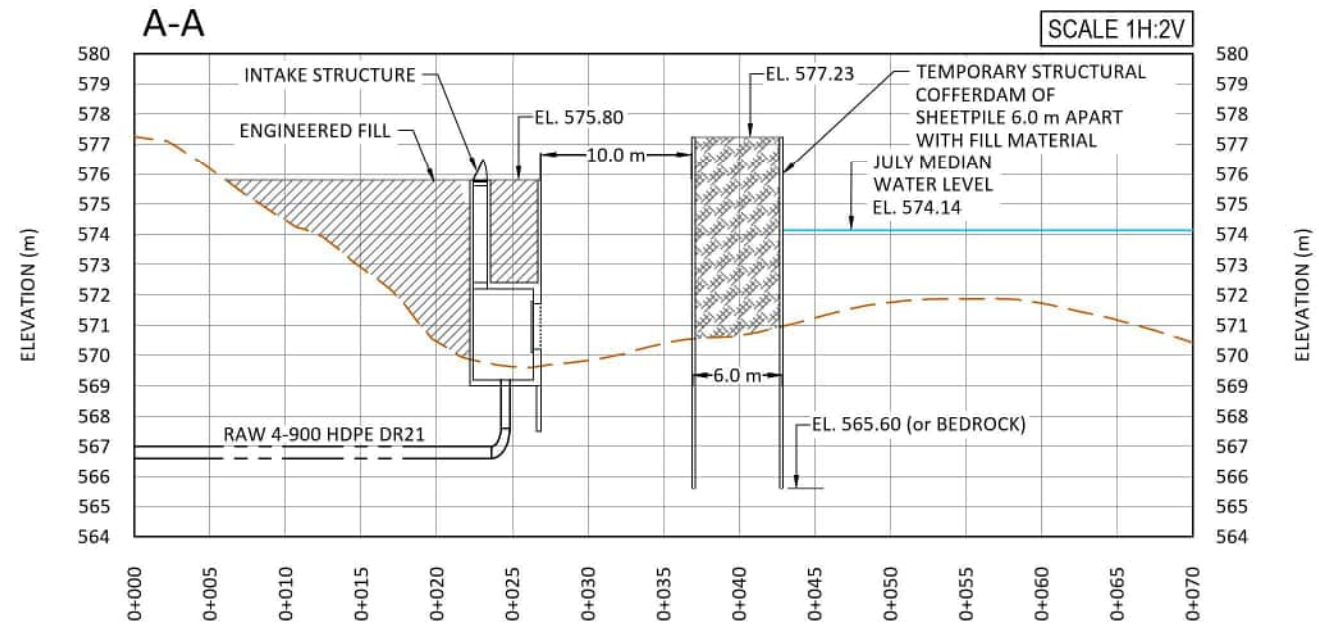
nhc
northwest
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consultants
9819-12 Avenue SW
Edmonton, AB T6X 0E3
Canada
Office: 780.436.5868
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REVISIONS			DRAWING INFORMATION	
			DATE	11 Jul 2024
			DESIGNED BY	MP
			DRAWN BY	DL
			CHECKED BY	PGV
			SHEET SIZE	B (11" x 17")

GREENVIEW INDUSTRIAL GATEWAY
WATER INTAKE
PRELIMINARY ASSESSMENT OF SHEET
PILE COFFERDAM HYDRAULICS
PLAN VIEW WITH RIPRAP LAYOUT

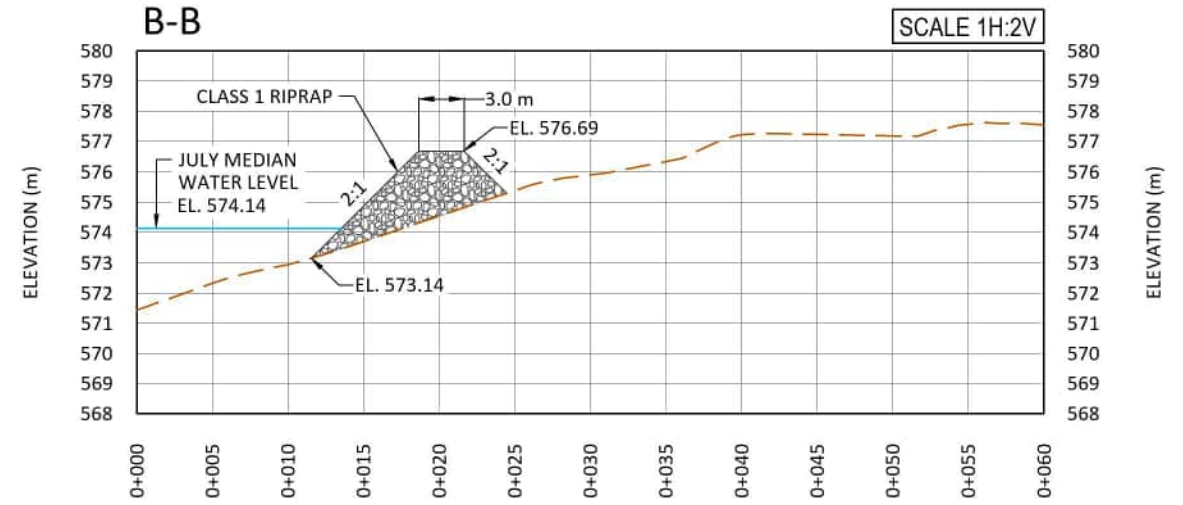
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FIGURE NUMBER	4
REVISION	

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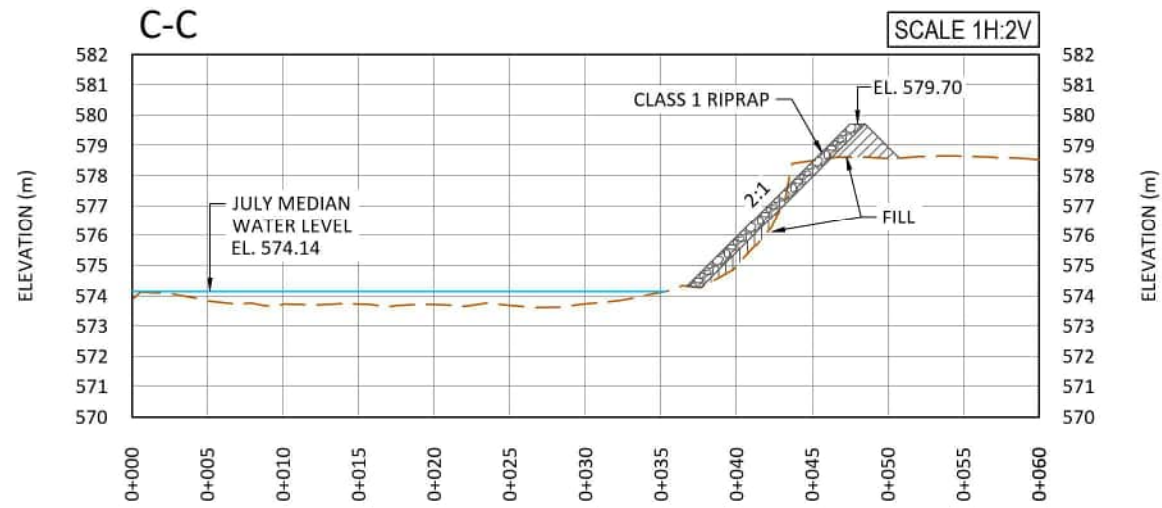
SHEET PILE PROFILE

HORZ. = 1:500
VERT. = 1:250



TEMPORARY RIPRAP PROFILE

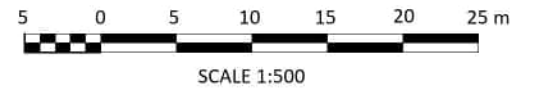
HORZ. = 1:500
VERT. = 1:250



PERMANENT RIPRAP PROFILE

HORZ. = 1:500
VERT. = 1:250

NOT FOR CONSTRUCTION



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REVISIONS		DRAWING INFORMATION	
		DATE	11 Jul 2024
		DESIGNED BY	MB
		DRAWN BY	DL
		CHECKED BY	PGV
		SHEET SIZE	B (11" x 17")

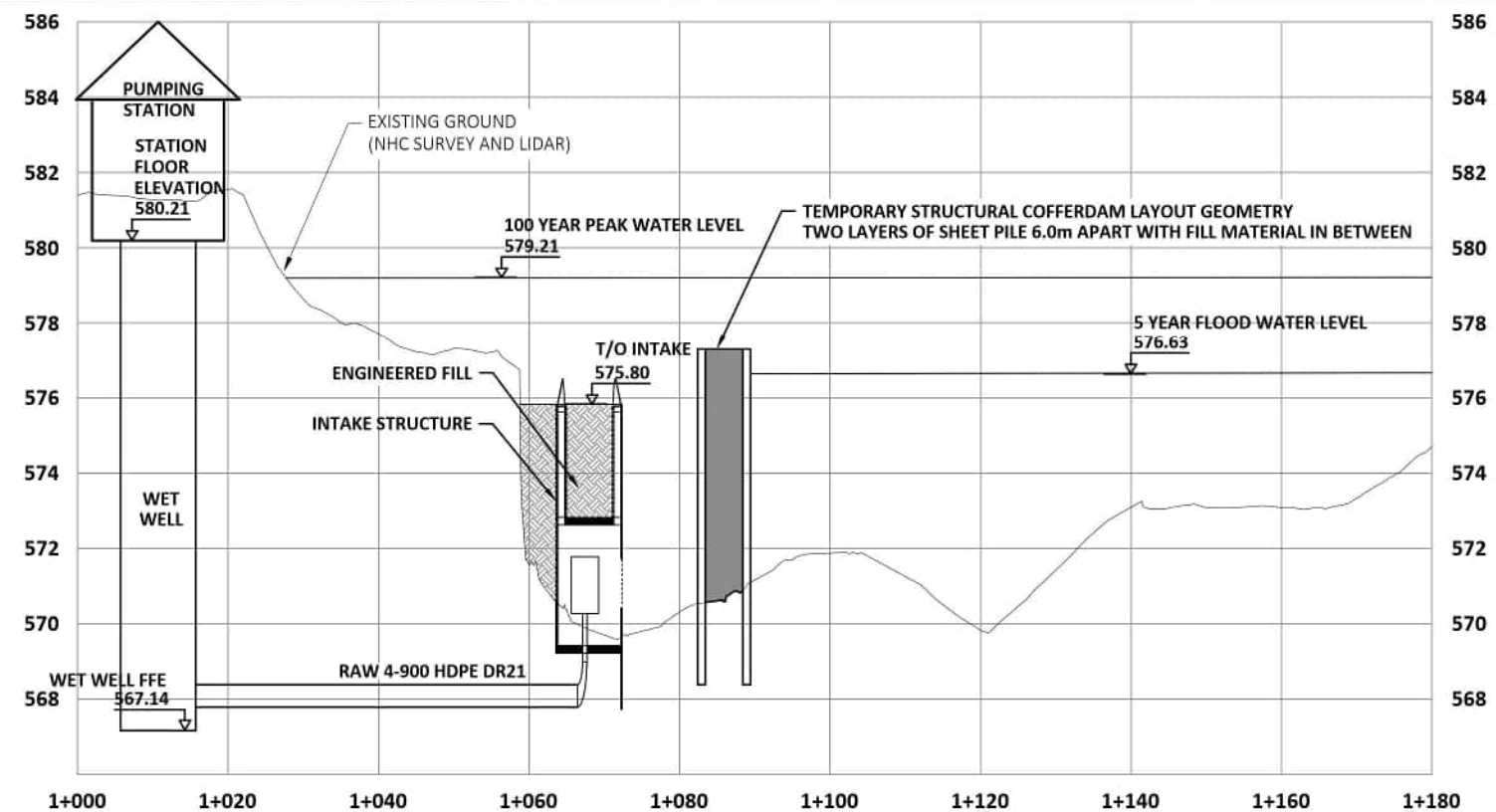
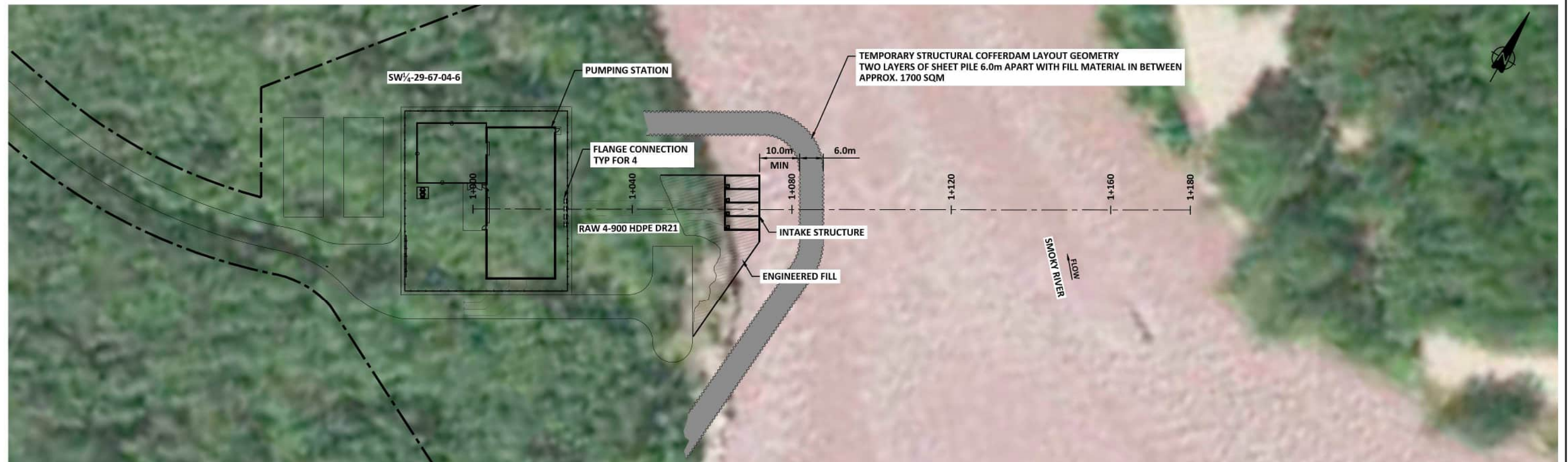
GREENVIEW INDUSTRIAL GATEWAY
WATER INTAKE
PRELIMINARY ASSESSMENT OF SHEET
PILE COFFERDAM HYDRAULICS
PROFILES

NHC NUMBER	1007352
DRAWING NUMBER	1007352-002
FIGURE NUMBER	5
REVISION	1

Appendix A

Sheet Pile Cofferdam Drawing from MPE

M:\M-DATA\50\46 WATERSMART SOLUTIONS LTD\001 GREENVIEW INDUSTRIAL GATEWAY WATER INTAKE APPROVAL\AUTODESK\ACAD\CIVIL 1.5-1.06.DWG



PLAN AND PROFILE
1
1:1000H
1:200V



WATERSMART SOLUTIONS LTD.
GREENVIEW INDUSTRIAL GATEWAY - TM02
PLAN AND PROFILE - INTAKE AND PIPELINES
STATION 1+000 TO STA 226
OPTION 3

SCALE: AS SHOWN	DATE: APRIL 2024	JOB: 5946-001-00	FIGURE: 3.5 - 2
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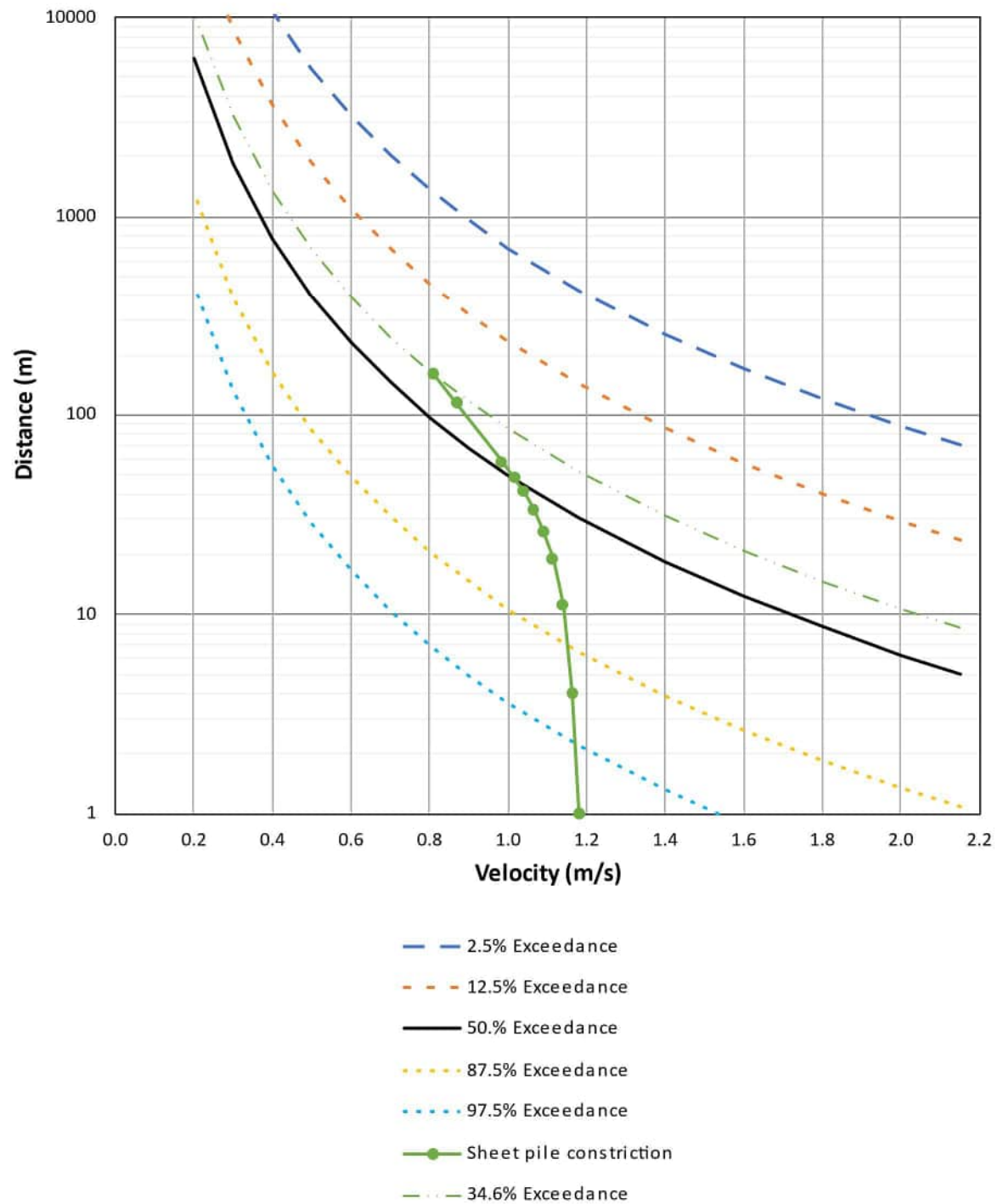
Appendix B

Fish Migration Affected by Sheet Pile Cofferdam

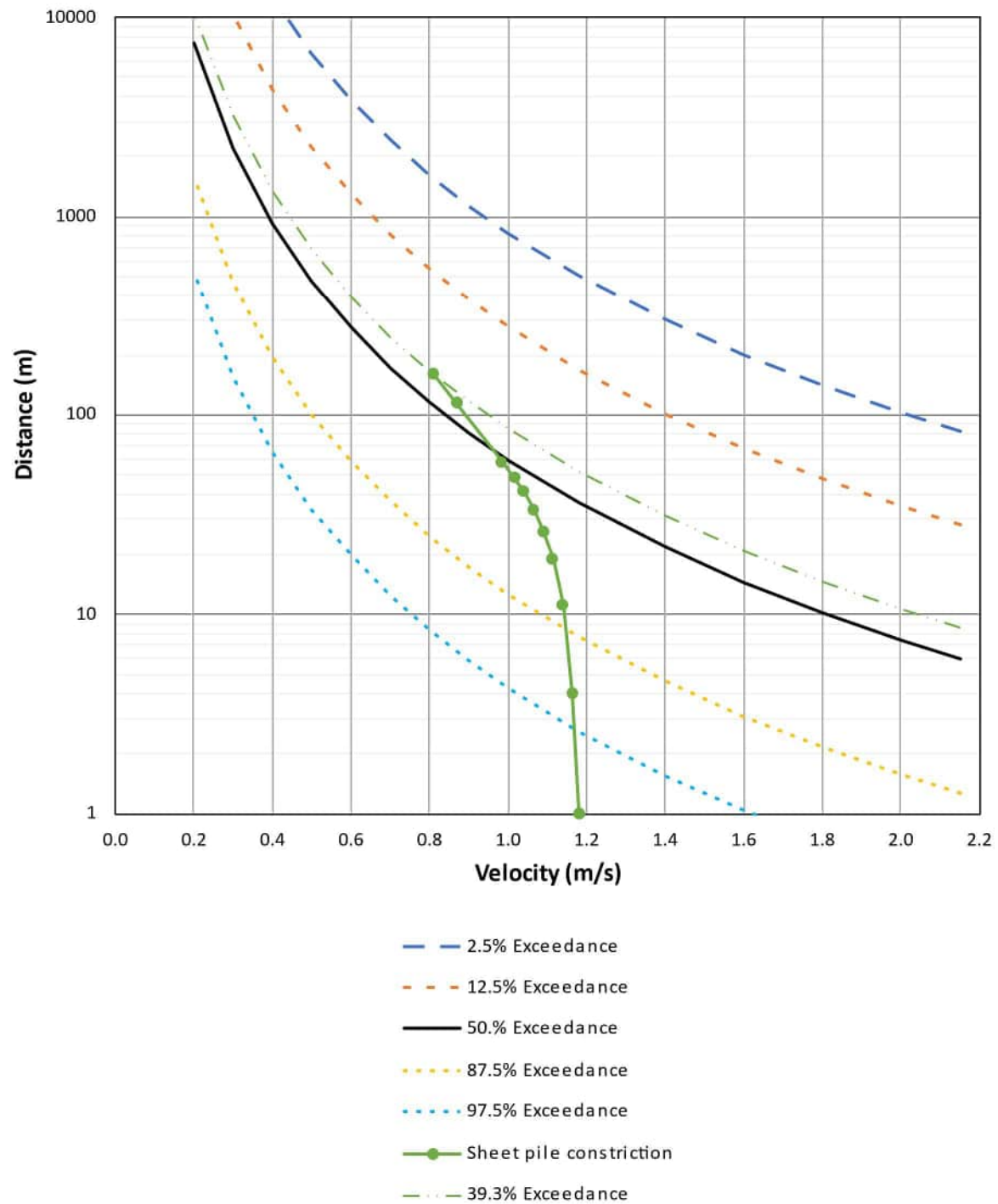
(Velocity-Distance Curves in
Sheet Pile Cofferdam Constriction
at Median Peak (2-Year) Monthly Flows
compared with Swim Performance Curves)

Month	Fish Species
May	Artic Grayling
May	Rainbow Trout
May	Walleye
May	Longnose Sucker
June	Artic Grayling
June	Rainbow Trout
June	White Sucker
July	White Sucker

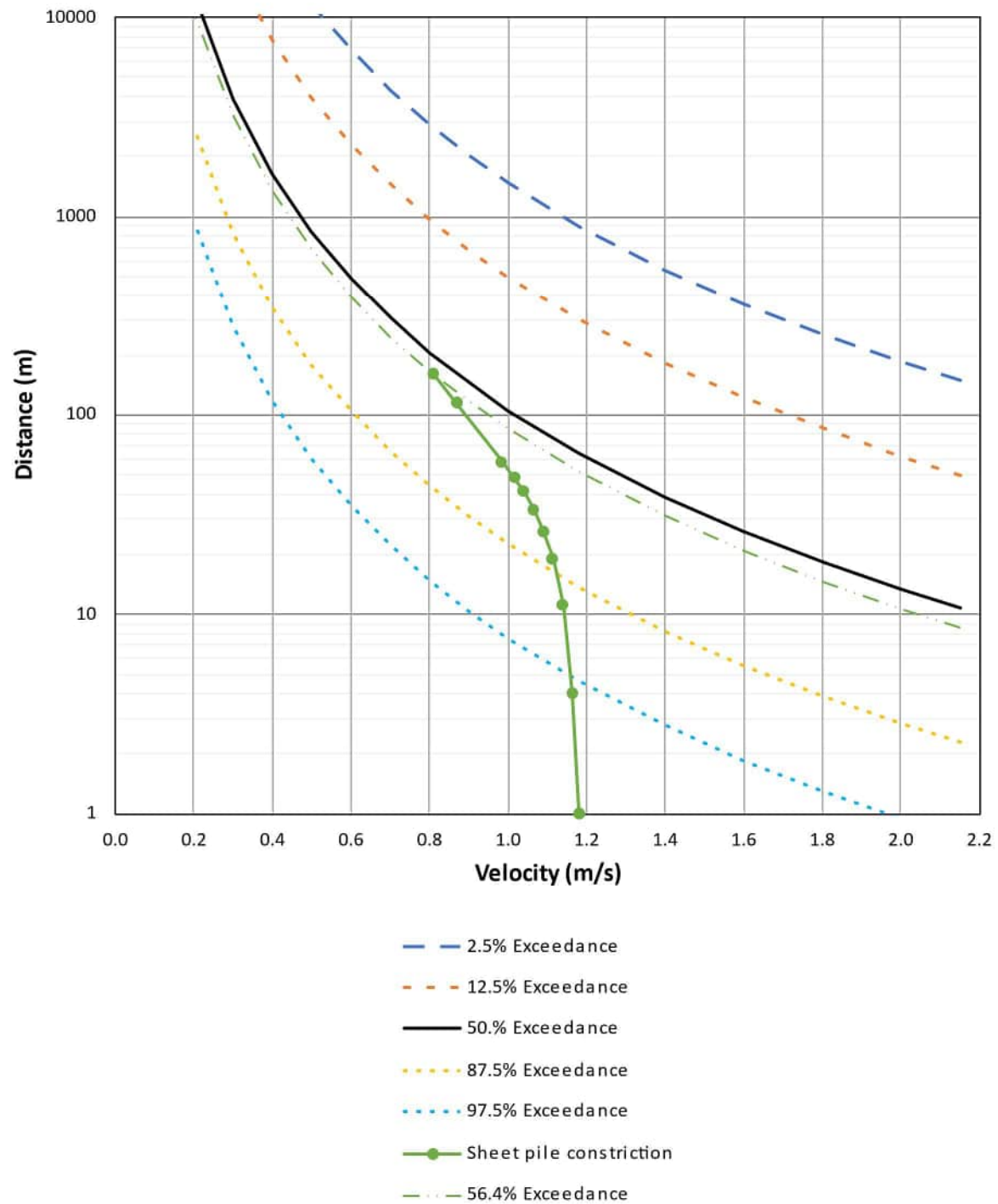
Arctic Grayling, Length = 326 mm, May Median Peak Flow



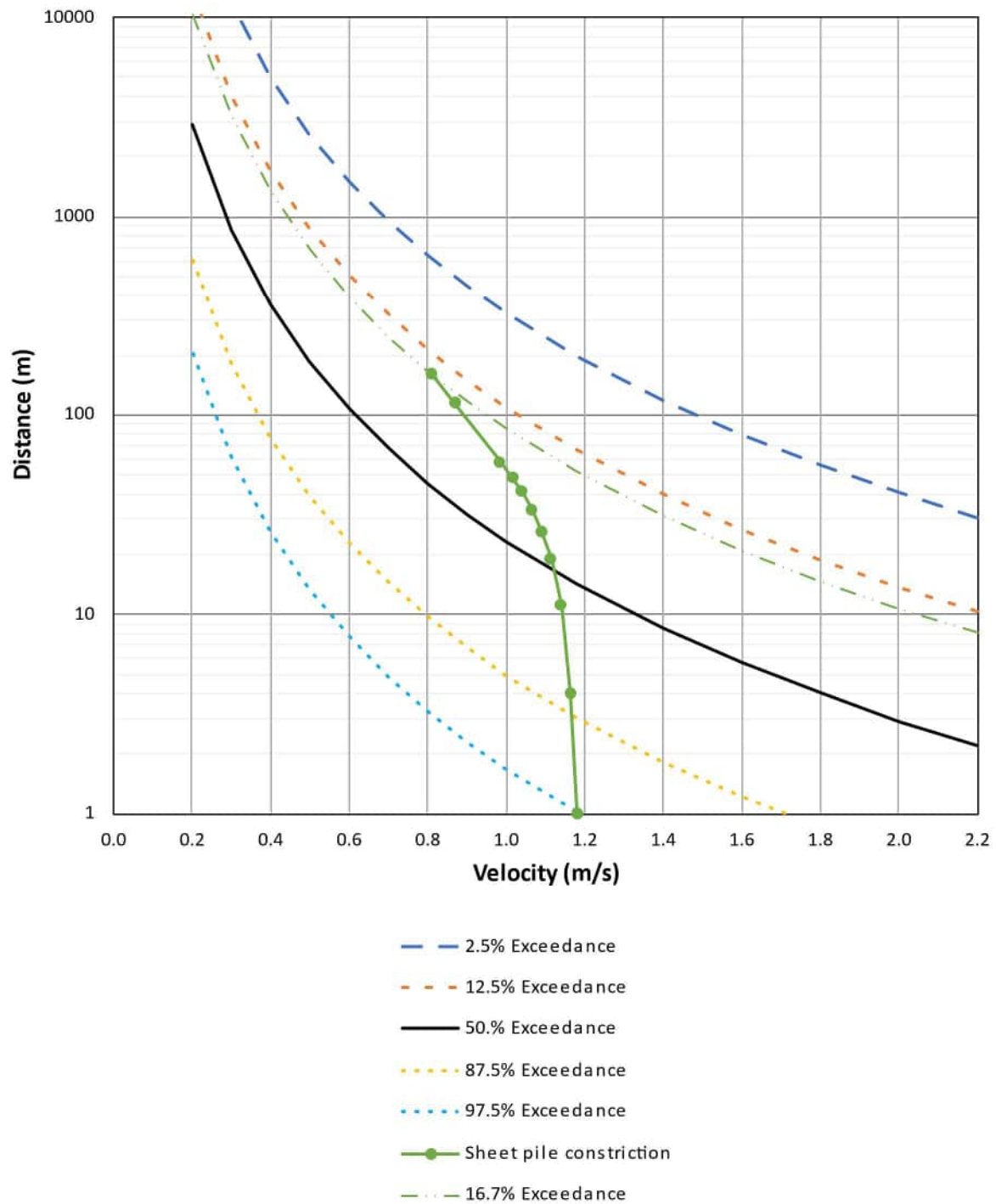
Rainbow Trout, Length = 349 mm, May Median Peak Flow



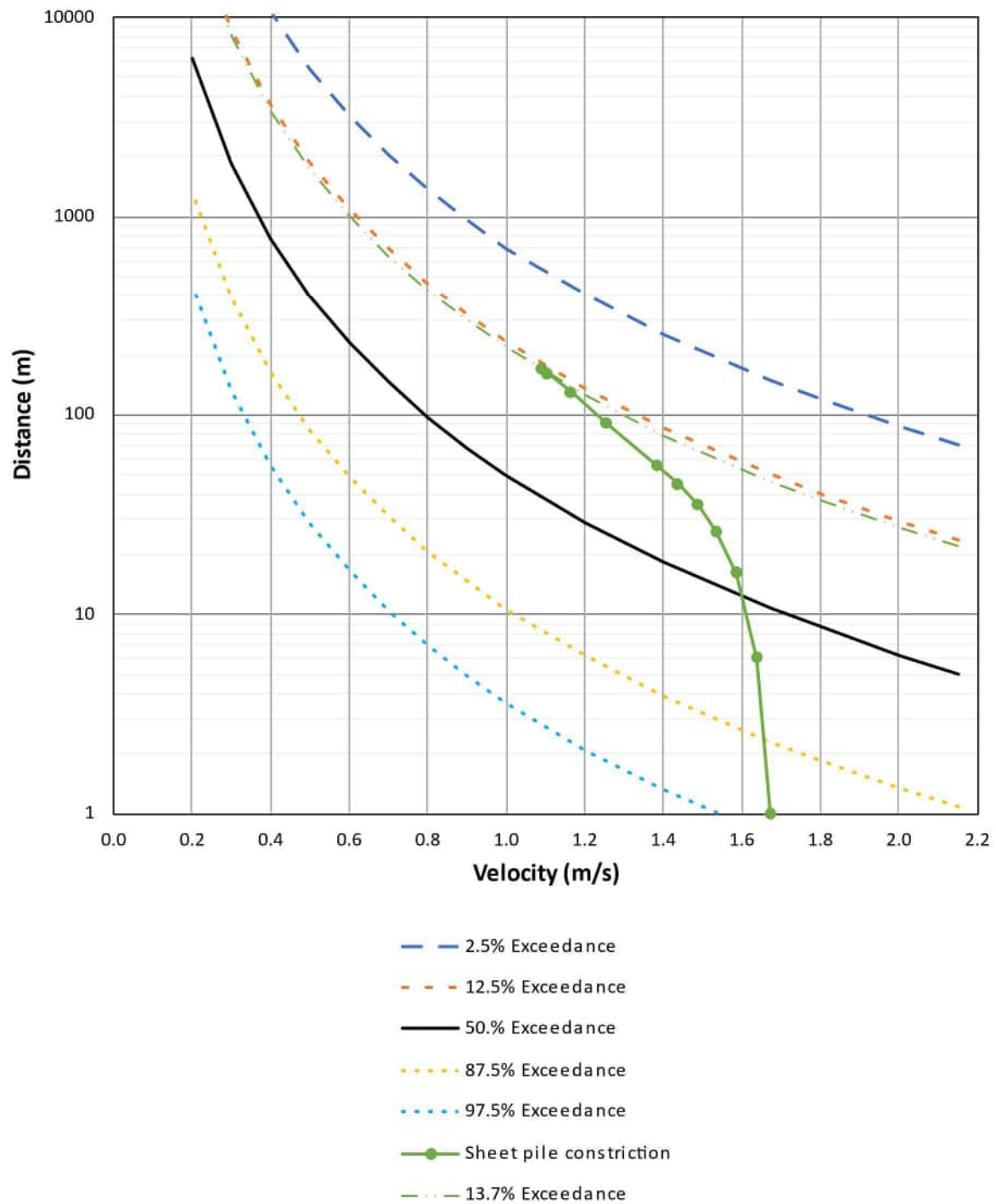
Walleye, Length = 440 mm, May Median Peak Flow



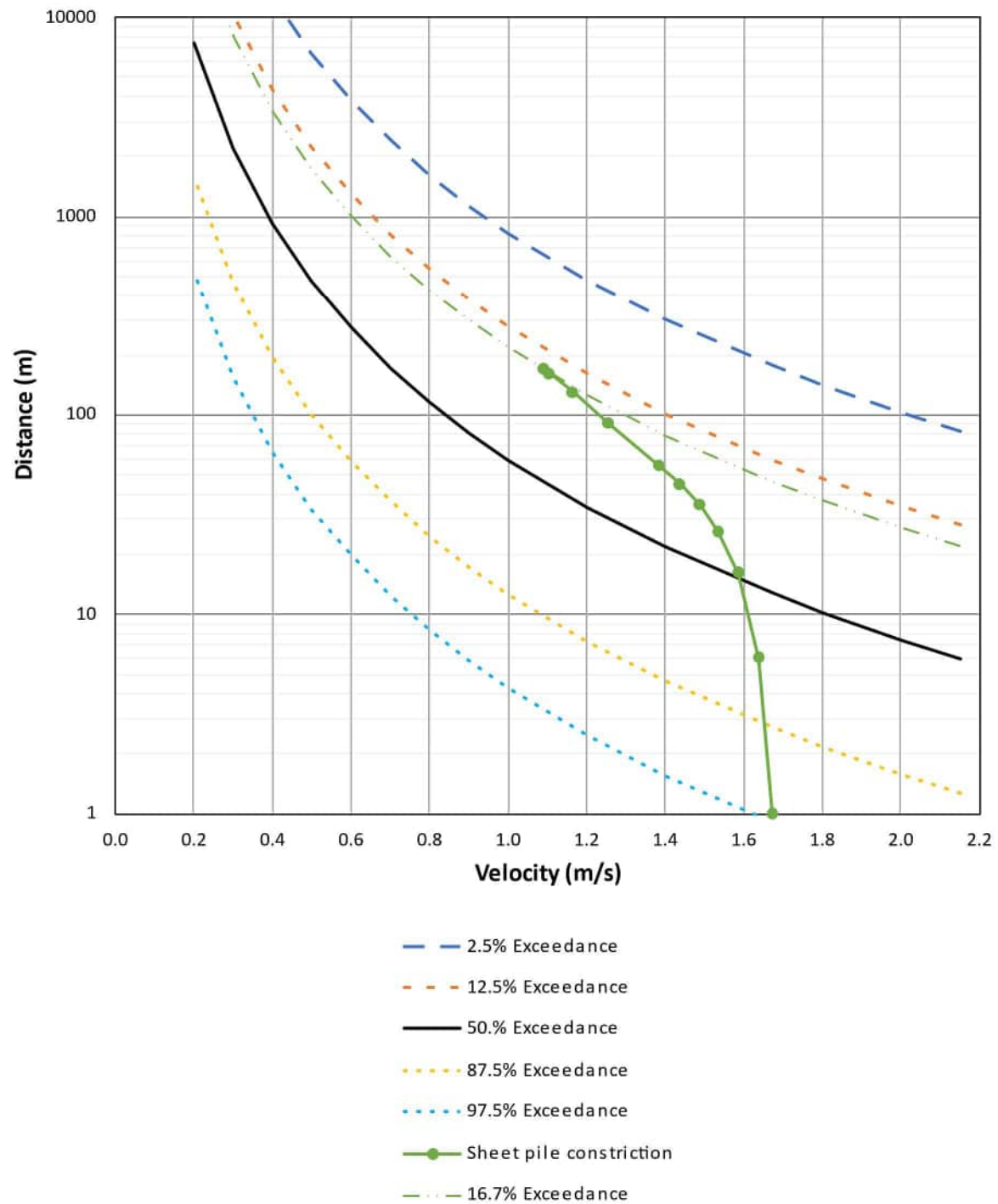
Longnose Sucker, Length = 240 mm, May Median Peak Flow



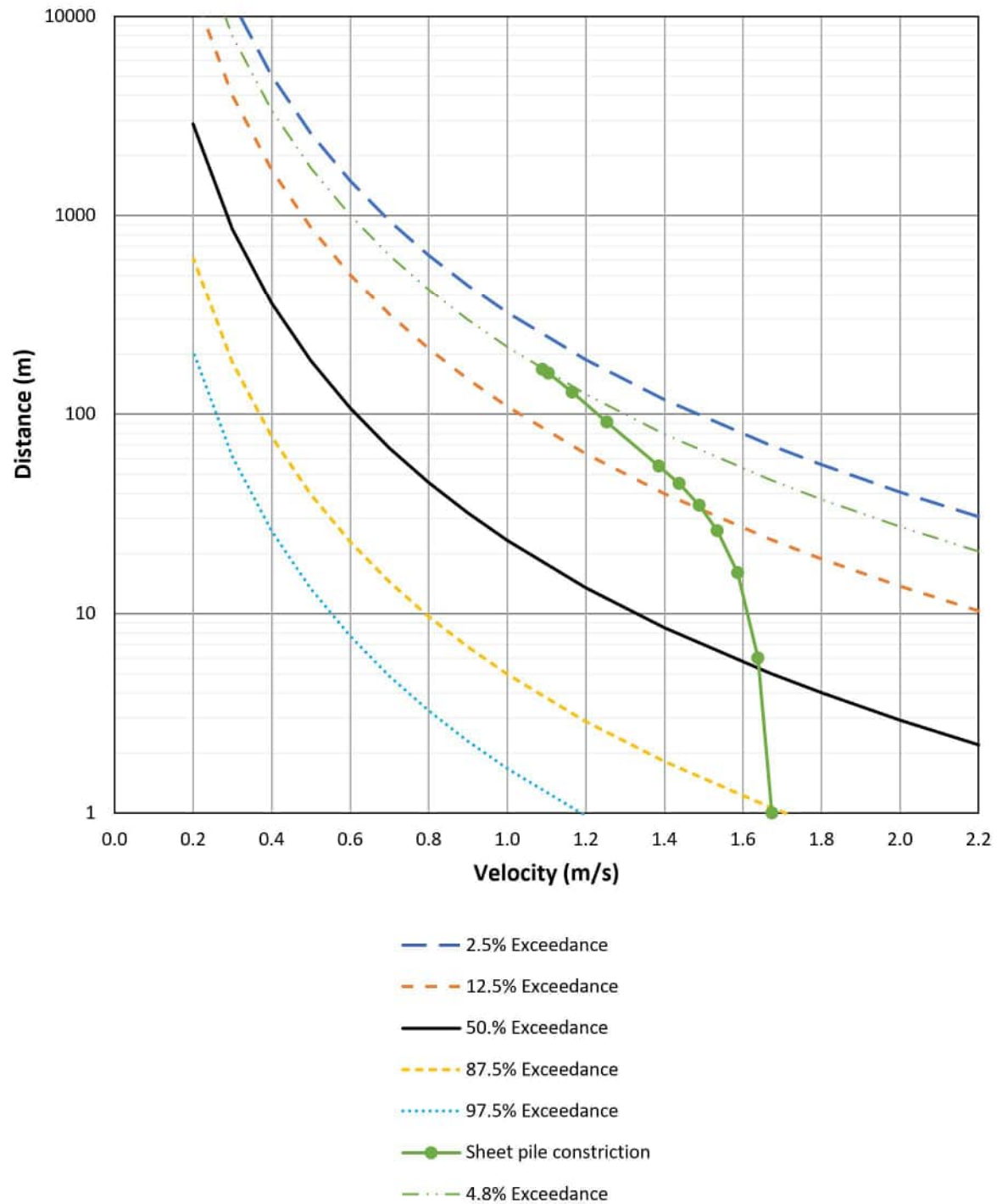
Arctic Grayling, Length = 326 mm, June Median Peak Flow



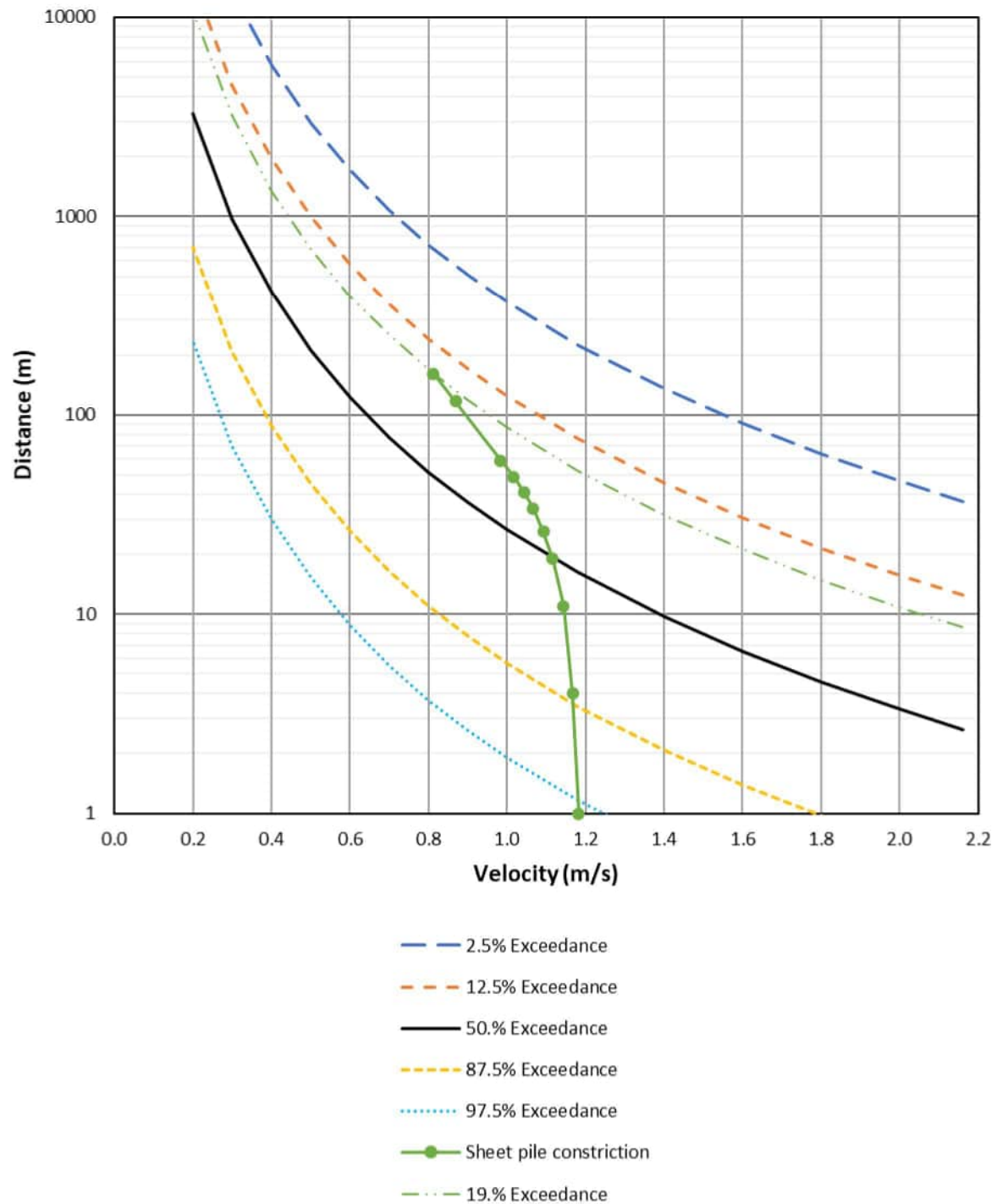
Rainbow Trout, Length = 349 mm, June Median Peak Flow



White Sucker, Length = 253 mm, June Median Peak Flow



White Sucker, Length = 253 mm, July Median Peak Flow



Appendix H

Ice Platform Design

December 11, 2024

MPE Engineering Ltd.
#101, 10630 - 172 Street
Edmonton, AB T5S 1H8

Attention: Haj Oshiki, P.Eng.

Subject: **Greenview Industrial Gateway Raw Water Intake
Ice Platform for Drilling – Initial Ice Design – Issued for Regulatory Approval**

1 INTRODUCTION

The following report provides the initial ice platform design carried out by Northwest Hydraulic Consultants Ltd. (NHC) to support the design of a water intake on the Smoky River for the Greenview Industrial Gateway (GIG). The ice platform will be used for geotechnical drilling at the location of the proposed intake. The initial ice platform design provided herein was based on typical ice characteristics obtained from deflection tests carried out by Northwest Hydraulic Consultants Ltd. (NHC) on other ice platforms and ice bridges, and on work procedures and weights provided by MPE Engineering Ltd (MPE). The ice thicknesses determined in this initial ice design will be used to determine if sufficient ice thickness is attainable to carry out the work plan safely. If the ice thickness is not attainable in the time available, further ice testing and/or adjusted work procedures will be required to potentially reduce the ice thickness requirements.

2 ADOPTED ICE CHARACTERISTICS

The structural characteristics of the ice for the ice platform design were selected based on the average from numerous deflection tests carried out by NHC in previous years on a range of rivers and lakes. The adopted design value for tensile strength of ice was 2000 kPa and adopted design value for the elastic modulus of ice was 1.7 GPa.

Creep, the increasing deflection of the ice under constant loading conditions, becomes important for longer term loading conditions because creep failure is a function of both the load magnitude and the deflection of the ice under the load. A creep time constant of 160 min at a reference load of 20,000 kg with 0.90 m thick ice was adopted based on the results of previous creep measurements. The creep time constant tends to vary with ice stress, so this time constant must be reduced for higher ice stresses. The creep time constant selected is relatively short to be conservative but may lengthen if a deflection test is carried out to determine the actual value for the specific ice sheet.

3 ICE PLATFORM DESIGN OBJECTIVES

The following design objectives were adopted for this initial design for untested ice:

- 1) A safety factor of at least 4.0 should be provided with respect to the stresses in the ice sheet during the worst case, short term loading.
- 2) The short-term deflection of the ice sheet during the worst-case loading scenario should not exceed 8% of the thickness of the ice sheet.
- 3) The specified ice thickness should provide a factor of safety of 9.0 against creep failure of the ice sheet for stationary loads. Creep failure occurs due to the work done to the ice. This work is defined as load times deflection, so the factor of safety on work must be increased to account for the uncertainty in both stress and creep.
- 4) The deflection bowl should be mostly contained within the thickened ice platform. The ice platform should extend at least 2.0 times the characteristic length of the thickened ice out from the center of the load in all directions if natural ice is present at the edges of the ice platform.
- 5) The direct pressure on the ice surface should be less than 500 kPa.

The above factors of safety can be reduced if a deflection test is carried out to determine the actual characteristics of the specific ice sheet.

4 LOADING CONDITIONS

Several ice loading conditions may occur during the drilling program. The loading conditions include short-term individual loads when mobile equipment is travelling between the bank and the borehole locations and longer term, complex loads due to snow ramp construction or when drilling is carried out on the borehole locations. It is assumed that all the vehicles will be removed from the ice each night.

4.1 Short-term Individual Loads

Table 1 provides a summary of the weights and dimensions of loads generated by individual vehicles traveling between the bank and the drill locations and the ice thicknesses required to safely support them. The load radius listed in **Table 1** is defined as the equivalent radius for the contact area for each individual load which is needed to determine the maximum stress under the load. The short-term loads are either moving or stationary for less than two hours. The ice thicknesses required to safely support these short-term loads listed in **Table 1** are assuming no boundary effects.

Table 1 Summary of short-term loads (<2 hr) and ice thicknesses required to safely support them on an infinite ice sheet.

Equipment	Weight (kg)	Length (m)	Width (m)	Load Radius (m)	Minimum Ice Thickness (m)
TSCC-06 Sonic Drill Rig	11,800	2.5	2.4	1.4	0.55
Skidsteer with 200 kg payload	1,250	1.8	2.0	1.1	0.19
Support Truck	5,500	4.0	2.0	1.6	0.34
Skidsteer with 1000 kg of snow	4,400	2.0	2.0	1.1	0.37

4.2 Drilling

Drilling will be carried out with a TSCC-06 Short Stroke Compact Sonic Drill Rig with the support of a Skidsteer and Support Truck. It was assumed that the Skidsteer would be at least 1.0 m from the Drill Rig and that the Support Truck would be at least 3 m from both the Drill Rig and the Skidsteer. The weights and dimensions of these vehicles are listed in **Table 1**. Drilling will be carried out at 5 borehole sites which are spaced about 20 m or more apart. Each bore hole will take up to two 12 hour days to complete. The complex longer-term load scenario of this drilling group requires thicker ice than those for the individual vehicles. In addition, a pullup load of up to 7,620 kg may be generated at the completion of each borehole as the drill stem is removed from the hole. It is assumed that no other vehicles will be on the ice platform during the drilling activity.

The ice thicknesses required to safely support to proposed drilling group loads are summarised in **Table 2**. An ice thickness of 0.89 m is required to safely support this load scenario the Drill Rig is removed from the ice each night. If the Drill Rig is left on the ice overnight during the drilling process an ice thickness of 0.94 m would be required.

Table 2 Summary of load scenarios and ice thicknesses required to safely support the loads.

Load Scenarios	Maximum Load (kg)	Maximum Duration (hrs)	Minimum Ice Thickness (m)
Drilling group with pullup (removed overnight)	28,200	2*12	0.89
Drilling group with pullup (remaining overnight)	28,200	36	0.94

5 ICE PLATFORM DIMENSIONS AND SPECIFICATIONS

The ice platform must be large enough to accommodate the drilling activity and travel to the borehole locations of the ramp. It must also extend far enough from these locations so that the deflection bowls are mostly contained within the thickened ice.

The ice platform will be accessed from the MacLeod Flats Recreation Area on the east side of the river at the ice platform location. The borehole locations are close together and the ice platform will extend

across the river from the ice access location, so there is no need for a travel zone between the ice access location and the ice platform.

The maximum thickness required to safely support the ice platform activities is 0.89 m if the drill rig is removed overnight. It is recommended that the entire ice platform be constructed to this thickness since the required thickness must extend out at least 20 m from each borehole location. **Table 3** summarizes the ice characteristics and required dimensions for the ice platform. The layout of the ice platform relative to the river bank and borehole locations is provided in **Figure 1** and the dimension details are provided in **Figure 2**. Note that Borehole 5 is located on the edge of the grounded ice and that ice thicknesses on grounded ice may be less than 0.89 m due to the support of the river bank. However, the floating ice near this location should still be built up to 0.89 m if any portion of the drill group will be on the floating ice.

Table 3 Summary of ice platform dimensions

Characteristic	Value
Required ice thickness (m)	0.89
Characteristic length of ice sheet (m)	10
Minimum width of ice platform (m)	40
Minimum distance of ice platform edge from boreholes (m)	20

The use of the ice platform is limited by the following general specifications:

- 1) Speeds of vehicles should be limited to 5 km/hr.
- 2) When travelling, the Drill Rig should remain at least 10 m from the floating edges of the flooded ice.
- 3) The Support Truck should stay at least 3 m from the Skidsteer and Drill Rig when parked.
- 4) Alternative loading conditions with increased loads, reduced spacing or longer durations will require further analysis before they can be used on the ice platform.
- 5) All vehicles must be removed from the ice each night.
- 6) If water begins to flow onto the ice, work activities should be stopped, and vehicles removed from the area.

6 ICE PLATFORM CONSTRUCTION

Water for ice platform construction will be pumped from the river. Ice thickness measurements taken by NHC on Feb 7 and 8, 2024 indicated the average ice thickness over the ice platform extents was 0.71 m. The ice platform will require up to 1400 m³ of water to flood the ice to build it to the required thickness. This volume includes a contingency for seepage losses of 50%.

The natural ice must reach certain thicknesses before various types of work can be safely carried out on the ice. The Best Practices for Building and Working Safely on Ice Covers in Alberta (2013) indicates that a minimum ice thickness of 0.10 m is required to walk safely on the ice and 0.18 m is required for

operating a snowmobile. A Skidsteer weighing 4,400 kg while pushing 1,000 kg of snow can be safely used on the minimum measured thickness of 0.37 m.

There is currently about 0.14 m of snow on the ice. This snow should be removed from the ice cover or packed down before flooding begins. After this, ice can be built up by flooding the ice surface in layers.

The minimum ice thickness measured by NHC in the ice platform area was 0.37 m, so 0.52 m of ice will need to be constructed in this area. Ice should be built up in layers no greater than 0.025 m. The rate of ice building may be as high as 0.05 m/day if two layers are flooded in cold weather, but care must be taken to ensure that each layer is completely frozen before the next flood. Assuming 0.025 m per day, it would take 21 days to build up the thinnest ice to the required thickness.

7 ICE PLATFORM MONITORING

Once the design ice thickness has been attained over the entire ice platform, the ice platform can be used for the procedures described above. Note that the presence of frazil ice under the ice cover and the highly variable ice thickness are indications that an open lead occurred upstream of the site after initial freeze-up of the river. The Sentinel-2 satellite image from Dec 23, 2023, shown in **Figure 3** shows open leads both upstream and downstream of the ice platform. These open leads were frozen over in the latest satellite image from Feb 11, 2024; however, there is a possibility that these open leads will reform during warm weather.

When the ice platform is being used, ice thicknesses should be monitored to ensure that the required ice thickness is maintained. As a minimum, this should include:

1. Ice thickness measurements will be carried before the work begins and once per week until the work is completed.
2. Ice thickness measurements at critical locations will be carried out **twice per week** if daily **maximum** air temperatures are above zero for more than one day in a row.
3. Ice thickness measurements at critical locations will be carried out **daily** if daily **average** air temperatures are above zero for more than one day in a row.
4. Observations for cracks should be carried out daily at the start of the day. Wet cracks must be refrozen before the ice platform can be used. Dry cracks should be sealed by flooding after the work is done for that day.
5. Ice conditions upstream of the ice platform should be visually monitored at the beginning of each day. If an open lead reaches within 200 m upstream of the work area, ice thicknesses at the upstream edge of the work area must be measured at the beginning of each day to ensure that they still meet the design requirements.
6. During drilling, water levels must be monitored to ensure that water does not overtop the hole in the ice. Frozen snow may be built up around the edge of the hole to contain the water. If water begins to pond on the ice, all activities near the hole must be stopped as soon as safely possible and the equipment removed from the area. The ice thickness should be increased, or other measures put in place to stop water from spilling onto the ice before work is resumed.

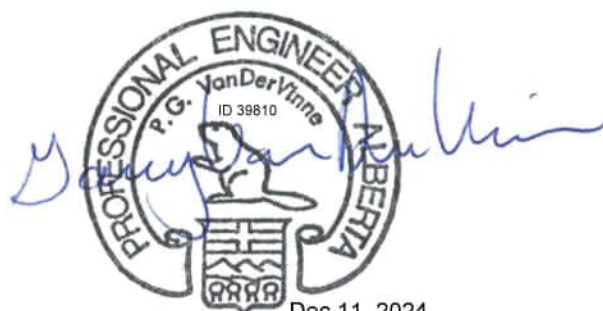
The latest date for safely working on the ice is expected to be about Mar 15, since significant deterioration of the surface ice due to solar radiation can occur after this date. If work on the ice is required after this date, an assessment of the ice quality by NHC staff will be needed to ensure that the ice can be used safely before work begins each day. Note that the required ice thickness is similar to the maximum ice thickness expected under natural so ice platform removal should not be required.

8 CLOSURE

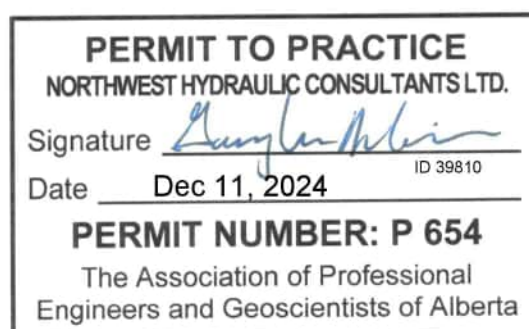
If you have any questions or require analysis of alternate work procedures and ice loads, please contact me by email at gvinne@nhcweb.com or by phone at 780-920-2978.

Sincerely,

Northwest Hydraulic Consultants Ltd.

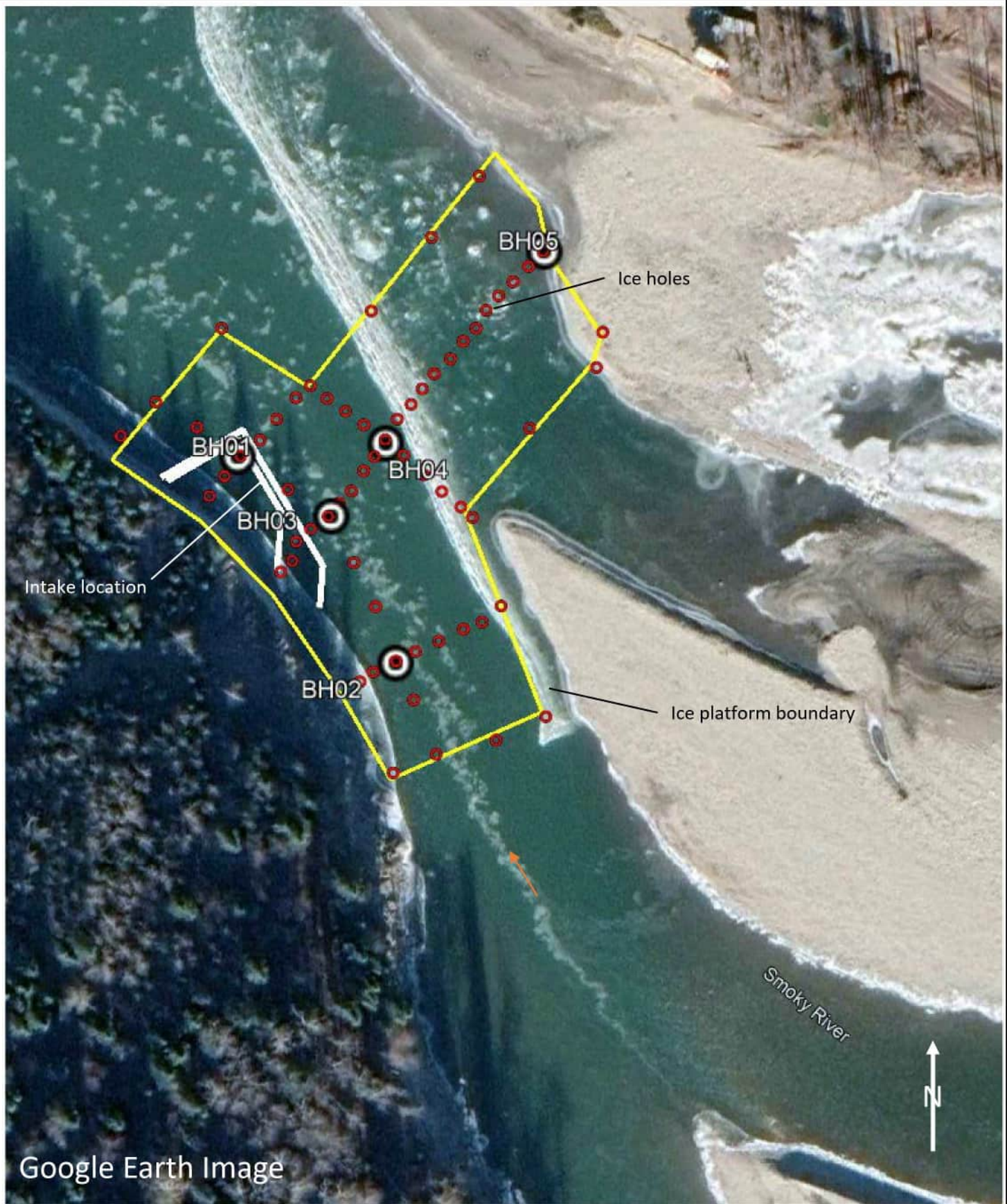


Dec 11, 2024
Gary Van Der Vinne, M.Sc., P.Eng.
Principal

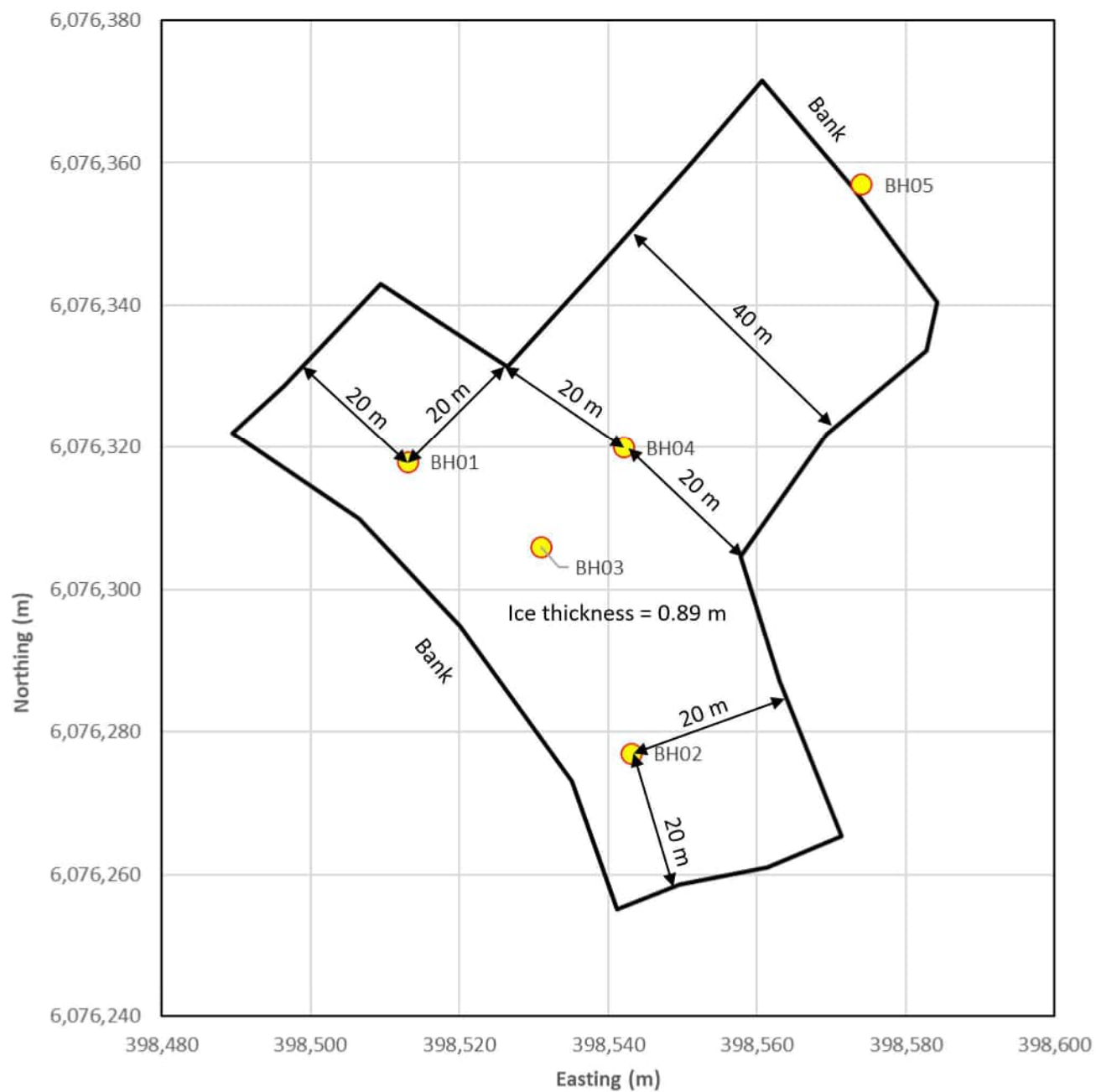


This report has been prepared by Northwest Hydraulic Consultants Ltd. for the benefit of MPE Engineering for specific application to the Preliminary Hydrotechnical Design of the Greenview Industrial Gateway Intake on the Smoky River. The information and data contained herein represent Northwest Hydraulic Consultants Ltd. best professional judgment in light of the knowledge and information available to Northwest Hydraulic Consultants Ltd. at the time of preparation and was prepared in accordance with generally accepted engineering and geoscience practices.

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	SCALE – 1000:1		GREENVIEW INDUSTRIAL GATEWAY WATER INTAKE INITIAL ICE DESIGN	
	Coordinate System: Units: As Shown			ICE PLATFORM LOCATION PLAN
	Job: 1007352	Date: 2024-02-16		



— Ice platform boundary

● Borehole locations

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Coordinate System:
Units: As Shown

Job: 1007352

Date: 2024-02-16

GREENVIEW INDUSTRIAL GATEWAY WATER INTAKE
INITIAL ICE DESIGN

ICE PLATFORM DIMENSIONS

FIGURE 2



			GREENVIEW INDUSTRIAL GATEWAY WATER INTAKE INITIAL ICE DESIGN SENTINEL-2 IMAGE FROM DEC 23, 2023	
	Coordinate System: Units: As Shown	Job: 1007352		FIGURE 3
	Date: 2024-02-16			