Priest River Cold-Water Augmentation Alternatives Analysis

Alternatives Assessment Final Report

McMILLEN JACOBS ASSOCIATES

11/22/2019

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Revision Log

Revision No.	Date	Revision Description
0	02/15/2019	Draft Alternatives Assessment Report
1	03/29/2019	Draft Alternatives Assessment Report
2	05/15/2019	Final Alternatives Assessment Report
3	11/22/2019	Final Alternatives Assessment Report

1.0 Introduction

Section 1 presents a summary of the overall project including authorization, purpose, project understanding, background, as well as the Alternatives Assessment Report organization.

1.1 Authorization

McMillen Jacobs Associates (McMillen Jacobs) was retained by Avista Corporation (Avista), for work being led by request from the Idaho Department of Fish and Game (IDFG), to provide Architect/Engineer (A-E) services and to develop and evaluate conceptual design alternatives for the Priest River cold-water augmentation at Priest Lake, Idaho. The contract was authorized on August 8, 2018, with the contract number R-42062.

1.2 Purpose

The purpose of this Alternatives Assessment Report is to present the basis of analysis, describe the alternatives, present the alternatives evaluation criteria, and assess the feasibility of each alternative with regard to the evaluation criteria.

1.3 Project Understanding

McMillen Jacobs understands the Project to be the development and evaluation of conceptual design alternatives for a cold-water augmentation to cool Priest River for the benefit of fish and aquatic biota using cold water from the lower hypolimnion of Priest Lake in Bonner County, Idaho. Streamflow in the Priest River is regulated by the Outlet Dam, which is operated by the Idaho Water Resource Board (IWRB). Each alternative will need to maintain statutory lake levels, minimum low-flow requirements, aquatic species protection, and existing public uses of the Lake and the River. Therefore, the cold-water augmentation design alternative must be capable of accommodating variable hydraulic head and discharge.

The objective of this Alternatives Assessment is to: (i) develop conceptual designs of alternatives to provide the Priest River with cold water, (ii) assess the various impacts of these designs enumerating their relative advantages and disadvantages, (iii) assess the feasibility of each of these designs, and (iv) develop cost estimates of each alternative for both capital costs and operations and maintenance costs.

1.4 Background

Priest Lake is located in Bonner County, in the northern panhandle of Idaho (Figure 1-1). Priest Lake is approximately 18 miles long running from north to south, and is drained by the Priest River, which is a major tributary of the Pend Oreille River. Streamflow in the Priest River is regulated by the Outlet Dam located at the southwestern end of Priest Lake. Originally built in 1950, the first Priest Lake Outlet Dam was later replaced by a concrete gravity dam with construction completed in 1978. The Outlet Dam is owned and operated by the IWRB. Dam operations in the summer months are manipulated to maintain lake levels for summer recreation and to provide minimum instream flows downstream of the dam.

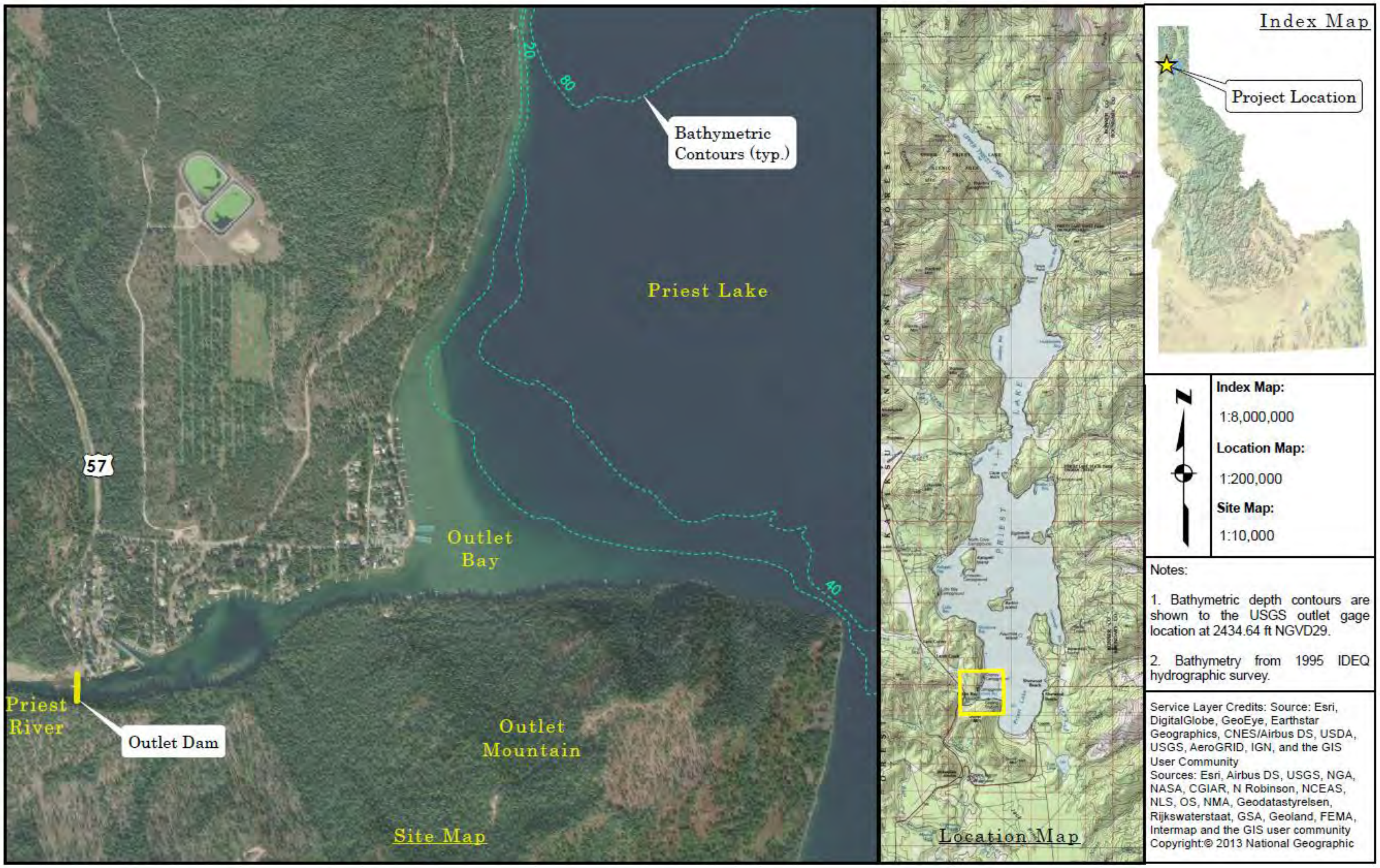

Figure 1-1. Index, Location, and Site Map

Recent stream temperature modeling, contracted by the Kalispel Tribe of Indians and conducted by Portland State University, was conducted in Priest Lake and the Priest River. The modeling confirmed the thermodynamic feasibility of reducing late summer stream temperatures in the Priest River by adding water to Priest River flows that originates in the lower portion (i.e. hypolimnion) of the thermally-stratified Priest Lake. The model considered input flow rates between 30 cfs and 400 cfs and predicted that if 75% of stream inflow came from the hypolimnion, the upper 30 miles of the Priest River could cool between 2 and 10 °C, which would be enough to improve native salmonid habitat conditions and meet the Idaho Department of Environmental Quality (IDEQ) criteria for cold-water aquatic life during August and September (less than 19°C).

1.5 Report Organization

The Alternatives Assessment Report presents the alternatives development and evaluations. Specifically, the report presents a summary of the design elements, basis of analysis, methods and approach, cost estimates, and references used in preparing the analysis and report. The report is organized to provide a logical representation of the alternative development and evaluation for the cold-water augmentation alternatives. The major report sections and intended purpose are presented in Table 1-1.

Table 1-1. Report Organization

Section	Description	Purpose
1	Introduction	Summarizes the project authorization, background, purpose and scope.
2	Basis of Analysis	Presents the pertinent data and design criteria which will be used in the analysis and alternatives development.
3	Alternatives Development	Outlines the approach to developing the conceptual alternatives and presents a detailed description of each alternative. This section includes three different configurations and a summary of the brainstorming meeting.
4	Alternatives Assessment	Presents the evaluation criteria, and the evaluation of each conceptual design alternatives. This section includes the advantages and disadvantages of each alternative, the construction cost estimates (including final design costs), and the feasibility of each alternative.
5	Conclusions and Recommendations	Presents a summary of the conclusions of the conceptual design development and alternatives evaluation along with the recommended alternative for advancement to design development.
6	References	Documents the references used in developing the conceptual design.
Appendices		
A	Conceptual Drawings	Presents drawings illustrating the basic conceptual level details (plan and section) of each of the identified alternatives.
B	Hydrology and Hydraulic Calculations	Presents calculations supporting the conceptual alternatives design development and evaluation.
C	Cost Estimates TM	Summarizes the cost estimates prepared for the alternatives evaluation.

2.0 Design Criteria

2.1 Introduction

Section 2.0 presents the design criteria for the cold-water augmentation alternative analysis.

2.2 Design Criteria

This section presents the basis of analysis in a series of tables. A brief description of the contents of each table is as follows:

- **Table 2-1 Statutory Requirements:** Lists those criteria that the project must meet based on legal/statutory obligations.
- **Table 2-2 Hydrographic and Topographic Data:** Provides a list of all available elevation data that will be used for the project. Note that all elevation data will be converted to the NAVD88 vertical datum, as necessary, assuming a conversion of +3.973 feet when converting from NGVD29 to NAVD88, based on the National Geodetic Society's orthometric height conversion program Vertcon.
- **Table 2-3 Dam Operational Criteria:** Dam operational design criteria are provided by Mott MacDonald (2017) and include those listed in this table that are relevant to the cold-water augmentation project.
- **Table 2-4 Cold-Water Augmentation Operational Criteria:** Provides the operational design criteria specific to the cold-water augmentation that are related to fish passage, fish health, and target temperatures in the Priest River.
- **Table 2-5 Structural Engineering Design Standards:** Provides the codes and standards that will serve as the general structural design criteria for the design of the cold-water augmentation project.
- **Table 2-6 Mechanical Engineering Design Criteria:** Provides the standards that will serve as the general mechanical design criteria for the design of the cold-water augmentation project.
- **Table 2-7 Civil Engineering Design Criteria:** Provides the codes and standards that will serve as the general civil design criteria for the design of the cold-water augmentation project
- **Table 2-8 Electrical Engineering Design Criteria:** Provides the codes and standards that will serve as the general electrical design criteria for the design of the cold-water augmentation project.

Table 2-1. Statutory Requirements

Criteria	Units	Value	Comments
Legal Restrictions	-	Considered	All legal restrictions, including but not limited to water rights and points of diversion, easements, rights of way, and land ownership shall be factored into the alternatives analysis with input from IDFG throughout the design process.

Criteria	Units	Value	Comments
Lake Level	ft	3.0	Maintain a 3-ft lake level at the outlet gage (#12393000) during the recreational season (July 1 through October 8), where the reference datum is at elevation 2434.64 feet NGVD29, or 2438.61 feet NAVD88.
Instream Flow	ft^3/s	60	Maintain current minimum discharge flow requirements downstream of the Outlet Dam. This water use is junior to summer lake level requirement.
Instream Flow Timing	NA	Calendar Year	
Lake Water Quality	NA	Maintained	Maintain existing water quality of Priest Lake.
Idaho Lake Protection Act	NA	Considered	Idaho Statute §58-1301: *It is the express policy of the State of Idaho that the public health, interest, safety and welfare requires that all encroachments upon, in or above the beds or waters of navigable lakes of the state be regulated in order that the protection of property, navigation, fish and wildlife habitat, aquatic life, recreation, aesthetic beauty and water quality be given due consideration and weighed against the navigational or economic necessity or justification for, or benefit to be derived from the proposed encroachment.*
Navigation	ft	3.0	Capable of accommodating boats and canoes year-round (IDAPA 20.03.04). Any new structure would not deteriorate current navigation conditions.
Aesthetics	NA	Considered	Consideration of aesthetic impacts of the project will be made, particularly around the residences surrounding the lake.

Table 2-2. Hydrographic and Topographic Data

Data Type	Collection Method	Year Collected	Author/ Agency	Comments
Topographic	LiDAR	2012	Watershed Sciences	
Hydrographic	Method	1995	IDEQ	Survey of Priest Lake.
Hydrographic	Method	2017	IWRB	Longitudinal profile of Priest River within the expected construction area.

Table 2-3. Dam Operational Criteria

Criteria	Units	Value	Comments
Minimum Non-Recreational Season Lake Level	ft	0.1	As measured at the outlet gage #12393000 between October 9 and June 30.
Maximum Non-Recreational Season Lake Level	ft	3.0	As measured at the outlet gage #12393000 between October 9 and June 30
Maximum Fall Discharge	ft^3/s	2,500	Between July 1 and October 8.

Criteria	Units	Value	Comments
Maximum Change in Outflow	ft^3/s-day	1,200	Between July 1 and October 8.
Earliest Annual Drawdown Begin	-	October 1	
Latest Annual Drawdown End	-	November 1	
Gate Operations	-	See Comment	Operated such that discharge is distributed across all bays due to hydraulic jump/scour. Concerns for hydraulic jump exist primarily during the spring runoff gate operations and not during the fall discharges for lake level reduction.
Flow through 3-Inch Gate Opening	ft^3/s-gate	30	Based on operator rule of thumb; gate opening is determined based on observations of water levels at a marina staff gage and historical flow records.
Radial Gates	-	See Comment	Manually controlled by the Dam operator utilizing an electric powered torque wrench.
Gate Closure	-	See Comment	Subject to opening restrictions to limit effect of hydraulic jump and corresponding risk of downstream apron scouring which could destabilize the dam structure.

Table 2-4. Cold-Water Augmentation Operational Criteria

Criteria	Units	Value	Comments
Target Maximum Daily Average Temperature	°C	19	Per IDEQ (2018) for cold-water aquatic life.
Percent Thermal Mixing	%	25-75	Percent of streamflow contributed by cold-water augmentation; alternative-dependent.
Time of Use	NA	Calendar year	Cold-water augmentation operations are anticipated from July through September. However, operability is extended throughout the calendar year as needed.
Minimum Augmentation Flow	ft^3/s	45 to 105	Minimum augmentation flow is alternative dependent.
Mixing Rate	%	25 to 100	71 cfs is roughly 25% of the average daily flow for the months of August and September (i.e. 284 cfs). 105 cfs is over 100% of the minimum required instream flow of 60 cfs. The value assumed will be alternative-dependent.

Criteria	Units	Value	Comments
Minimum Intake Depth	ft	60.0	Measured from the summer water level. Elevation to bottom of intake will be at most 2378.61 feet NAVD88, based on depth to 8°C thermoclines provided in Figure 27 of Berger et al. (2014). This includes an 11-foot factor of safety to account for water temperature model error, variable hydrodynamic conditions in the lake, and nonstationary climatic conditions.
Cold-Water Augmentation Flow Condition	-	See Comment	Gravity or Pressurized.
Maximum Screen Approach Velocity at Intake	ft/s	0.5	Both the lake and river are federally-designated critical habitat for Bull Trout. Therefore, intake screening and approach velocity limits may be required. 0.4 ft/s is recommended to meet NMFS criteria. However, in section 3.1 – Design Flow Range, NMFS states: *"…screen approach velocity criterion of 0.4 ft/s could be increased to match the smallest life stage expected at the screen site."* The U.S. Fish and Wildlife Service (USFWS) recommends 0.5 ft/s approach velocity at water diversions to avoid impingement or entrainment of juvenile Bull Trout, the most sensitive species and life stage found in the lake. See Reference to USFWS in Avista (2018).
Minimum Screen Sweeping Velocity at Intake	ft/s	0.0	NMFS (2011) does not apply to the project. Furthermore, the screen will be located in quiescent water.
Maximum Screen Mesh Size	in	3/32	USFWS recommends 3/32-inch (2.38 mm) screen mesh. See Avista (2018). NMFS recommends 3/32-in for square screen opening and 1.75 mmm (i.e. 1/16-in) for slotted screen opening.
Effective Screen Area	ft^2	TBD	Calculated by dividing the maximum augmentation flow by the allowable approach velocity.
Augmentation Flow Velocity	ft/s	4.0-15.0	Alternative dependent.
Maximum Release Location	mi	3.0	Maximum allowable reach extents below the Outlet Dam that are not thermally conditioned by the cold-water augmentation. 10% of the distance from the Outlet Dam to the confluence with the Pend Oreille River is 4.5 miles.
Intake Location	-	See Comment	Locate intake in an area with sufficient ambient velocity to minimize sediment accumulation in or around the screen and to facilitate debris removal and fish movement away from the screen face (NMFS 2011).

Criteria	Units	Value	Comments
Re-Entry Velocity (Energy Dissipation)	%	25	Water discharging from the cold-water augmentation shall re-enter the Priest River at a velocity that is within 25% of the velocity at normal depth within the river.
Water Management Study	-	Considered	An on-going analysis of Priest Lake water management is being conducted for IWRB by Mott MacDonald. Alternatives analysis conducted as part of the present study will consider the results from the water management study as they develop.
Screen Cleaning	-	See Comment	Active or Passive.
Outfall Dissolved Oxygen	mg/L	See Comment	Dissolved oxygen is not evaluated for this project.

Table 2-5. Structural Engineering Design Standards

Code	Standard
2015 IBC	2015 International Building Code
SEI/ASCE 7-10	Minimum Design Loads for Buildings and Other Structures, 2010 Edition
ANSI/AISC 360-10	Specification for Structural Steel Buildings, 2010 Edition
AISC 341	Seismic Provisions for Structural Steel Buildings
ACI 318-11	Building Code Requirements for Structural Concrete
ACI 350-06	Code requirements for Environmental Engineering Concrete Structures
ACI 350.4R-04	Design Considerations for Environmental Engineering Concrete Structures
2005 ADM	Aluminum Design Manual, 2005 Edition
AWS D1.1-04	Structural Welding Code - Steel
AWS D1.2-08	Structural Welding Code - Aluminum

Table 2-6. Mechanical Engineering Design Criteria

Standard
American Society of Testing and Materials (ASTM)
American National Standards Institute (ANSI)
Hydraulic Institute (HI)
American Society of Mechanical Engineers (ASME)
American Welding Society (AWS)
National Fire Protection Association International (NFPA)

Table 2-7. Civil Engineering Design Criteria

Design Class	Standard
General Civil	Bonner County Idaho, County Code Titles 1 through 14
Stormwater	Interagency Stormwater Management Plan Criteria and Engineering Standards
Erosion Control	Idaho Department of Transportation Best Management Practices Manual

Design Class	Standard
Air Quality	Idaho Department of Transportation Air Screening Policy
Roads	Idaho Department of Transportation Roadway Design Manual

Table 2-8. Electrical Engineering Design Criteria

Standard	Description
TM 5-811-1	Electric Power Supply and Distribution
TM 5-811-2	Electrical Design, Interior Electrical Systems
EM 1110-2-3105	Mechanical and Electrical Design of Pumping Stations
NFPA 70	National Electrical Code (NEC)
IEEE C2	National Electrical Safety Code (NESC)
IES	Illuminating Engineering Society Lighting Handbook
NFPA-101-HB85	Life Safety Code
ANSI	American National Standards Association
NEMA	National Electrical Manufacturers Association
ISA	Instrument Society of America
UBC	Uniform Building Code
UL	Underwriters Laboratory

3.0 Alternatives Development

3.1 Introduction

Section 3.0 presents the development of the alternatives for the cold-water augmentation alternative analysis. This section presents the initial conceptual design alternatives developed at the brainstorming session, held on August 20, 2018 at the IDFG Regional Office in Coeur D'Alene, Idaho. The following personnel were in attendance, listed by organization: From IDFG was Kiira Siitari (Project Manager), Ken Bouwens (Liaison with Avista, QA/QC), Charles Corsi (Regional Supervisor). From McMillen Jacobs was Vincent Autier (Project Manager) and Kevin Jensen (Project Engineer).

3.2 Approach to Alternatives Development

The intent of the brainstorming session was to identify the full range of potential cold-water augmentation alternatives, evaluate these alternatives based on a range of criteria, then select the three (3) to four (4) alternatives that provide the most feasible approach. Though it is evident that some alternatives are not feasible, the alternatives are documented to demonstrate that a comprehensive evaluation was completed.

Eleven (11) alternatives were prepared as part of the alternatives development process. This section presents a short description of each alternative. The alternatives include:

- Alternative 1 – Gravity system with the intake northeast of Outlet Bay and a microtunnel extending 2.8 miles downstream of the Outlet Dam.
- Alternative 2 – Siphon with the pipe in the road or elsewhere. Intake located closer to Highway 57.
- Alternative 3 - Use groundwater source, either a well field or create some cold-water refuge. (Option 1 – all upstream, Option 2 – distributed).
- Alternative 4 - Pump station.
- Alternative 5 - Ranney well – use groundwater source located in alluvial channel with suction lines extending radially from a central caisson.
- Alternative 6 - Divert high spring flow in the flood plain to recharge.
- Alternative 7 – Tree shading of the Lower Priest River.
- Alternative 8 - Vortex tube/Venturi Effect – flow-induced suction of cold-water through a pipe.
- Alternative 9 - Passively Induced Upwelling – Introduce a structure to create an upwelling of cold-water based on vertically-oriented eddies.
- Alternative 10 - Trap and haul fish from the confluence to the lake.
- Alternative 11 - Mechanical chiller to cool readily-available surface water by pumping it through a heat exchanger.

3.3 Alternatives Considered

A brief description of each alternative is presented in the following paragraphs.

3.3.1 Alternative 1 – Gravity System

Alternative 1 would be a gravity water supply system connecting the deep-water area of Priest Lake with the Lower Priest River downstream of the Outlet Dam. The system would include a screened intake structure located at approximately 60 feet of depth to intercept the 8°C thermocline in the lake. The intake would connect with a small diameter pipe (4 to 6 feet) extending from the lake through Outlet Bay, around the Outlet Dam, and discharging into the Priest River below the Outlet Dam. The outlet into the river would be protected by an energy dissipation structure (e.g. riprap apron).

Advantages of this alternative include the low operation and maintenance (O&M) cost associated with gravity supply systems. Also, no construction through the dam would be needed. Furthermore, control valves and any necessary booster pumps could be located in an underground vault away from view, which may be a mitigating factor for local residents. The system also has the potential to divert large quantities of cold-water, which could help reduce river temperatures during high runoff.

The primary disadvantage of the gravity supply system from the lake is the limited head available to drive flow from the lake into the river. Figure 3-1 indicates the location of the 60-foot depth contour in the lake, which corresponds with the 8°C thermocline. Based on this location, a pipeline would have to extend approximately 8,000 feet from downstream to upstream to access water that is sufficiently cold. Importantly, this length of pipe will have non-trivial friction losses that must be accounted for; therefore, the challenge with this alternative is to provide sufficient hydraulic head to drive the target flow through the pipeline, while considering the friction and minor losses. Reducing head losses may require upsizing the pipe. However, the larger the pipe, the greater the potential or likely impacts to navigation and aesthetics. For this reason, burying the pipe in the shallow part of the outlet channel would both ensure safe navigation and minimize or eliminate concerns about the pipe being exposed and affecting the aesthetics of the area. In addition, the pipeline will likely need to be weighted down to keep it in place in the deeper portions of the route where it is not buried. These constraints will therefore limit the operational range of a gravity system, reducing the factor of safety of the design. Alternatively, achieving a suitable factor of safety for a gravity system may require introducing a booster pump facility to add hydraulic head to the system.

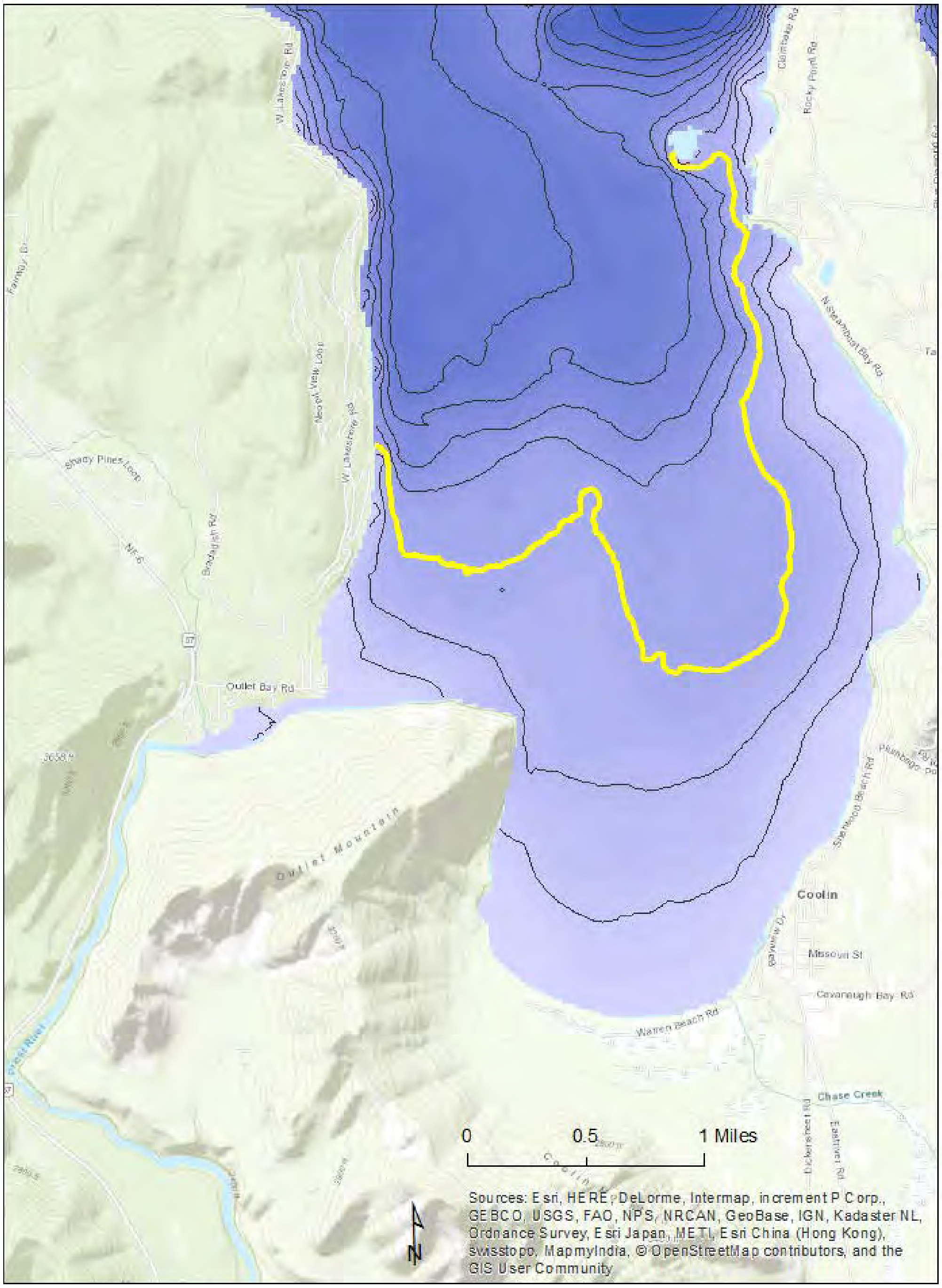

Figure 3-1. Priest Lake Bathymetry Showing 60-Foot Contour (Yellow), Interpolated from IDEQ 1995

3.3.2 Alternative 2 – Siphon System

Alternative 2 would include a siphon system to lift water from the lake and discharge it into the river downstream of the Outlet Dam. A siphon system could be located anywhere along the perimeter of the lake, although the shortest path to acceptable thermoclines will result in the lowest capital cost. A pump system will be required for developing and maintaining a prime on the siphon. Under normal operating conditions, the pump system will not be needed and therefore will not contribute largely to O&M costs of the system. Nevertheless, a dedicated pump house with pump, controls, valves, and other appurtenances will add to the overall capital cost of the Project.

A siphon operates on the principle of suction head developed in a pressure pipeline to generate positive drainage from a reservoir to some downstream location at a lower elevation. Once the siphon is primed, the gravity-driven flow in the descending portion of the siphon generates low pressures at the apex of the pipeline. This, in turn, results in a pressure differential at the intake and a resultant lift is experienced in the ascending portion of the siphon. Siphons have been widely used in reservoirs both for water supply and for reservoir drawdown purposes (PFRA 2000; Morrison Maierle 2012).

There are several limitations or concerns related to siphon intakes:

1. **Intake Air Entrainment:** Air entrainment in the pipe can cause bubbles to rise to the high point and effectively block the flow and slow or stop the siphoning action. To avoid air entrainment, a minimum submergence on the intake must be maintained. Minimum submergence criteria to suppress vortices that cause air entrainment are related to the suction velocity at the intake. Based on the fish screen impingement velocity criterion of 0.5 ft/s, a minimum submergence of 1.0 feet would be required at the intake (Volk 2017). Because the intake will be located 60 feet below the surface of the lake, vortex formation and air entrainment at the intake are not anticipated.

2. **Outlet Air Entrainment:** The outlet represents another location at which air can enter the system and rise to the high point, causing a collapse of the primed water column. This can be avoided by introducing air release valves to the system.

3. **Vapor Pressure:** Water entering the intake pipe will contain some level of dissolved gas. If the pressure experienced in the pipeline approaches the vapor pressure for a given temperature of the water, the water will effectively boil and rapidly release gas, which will lead to a collapse of the primed water column. This puts significant constraints on how much lift is achievable to the apex of the siphon. General "rules of thumb" indicate that the high point can be no more than 20 feet above the reservoir water surface elevation less 1.0 foot for every thousand feet in elevation above sea level (Morrison Maierle 2012):

$$\Delta z \leq 20 - {}^{WSEL}/_{1000} \quad \text{[ft]}$$

   With an assumed water surface elevation in the lake of 2,437.64 feet, the maximum lift is 17.6 feet. The location of the pipe apex can be manipulated to ensure that this maximum lift is not reached.

4. **Saturation Pressure –** Even before vapor pressure is achieved, dissolved gases in the water can devolve at a slower rate. If the pressure reaches the saturation pressure for a given dissolved gas content, the air-water mixture will be in a super-saturated condition and the devolution of the gas will depend on a number of factors (PFRA 2000). For this purpose, siphons need to be designed to maintain a certain discharge rate to ensure that air pockets do not form a blockage at the high point. This relationship is summarized in Figure 3-2.

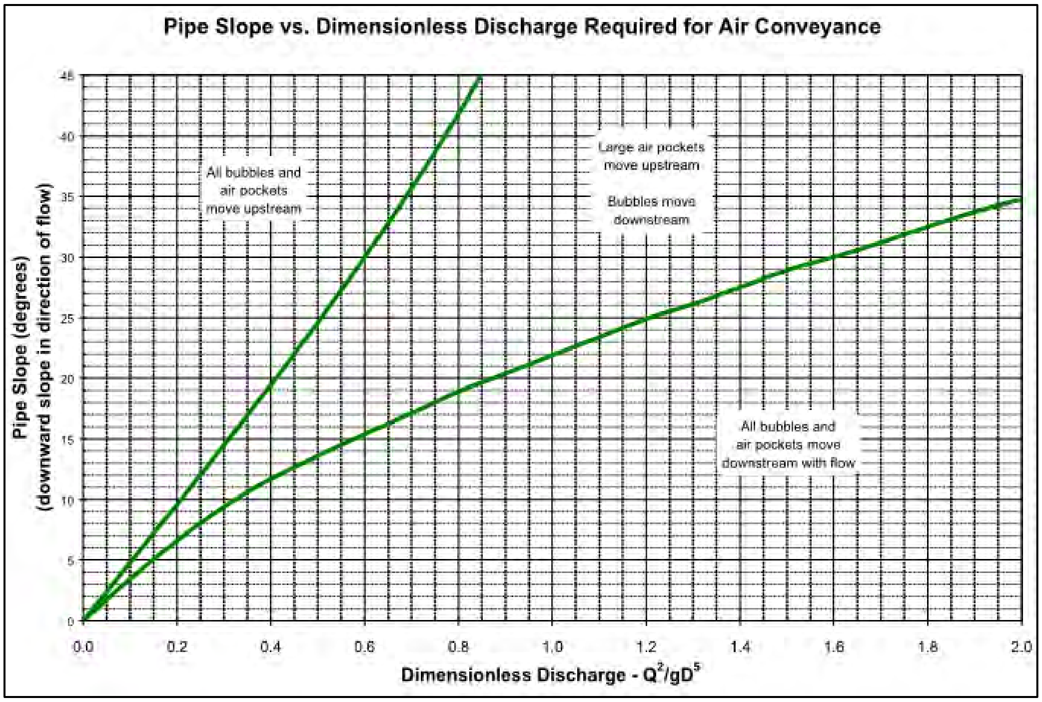

Figure 3-2. Discharge, Slope, and Air Bubble Relationships

Source: PFRA 2000

To ensure that devolved gas moves downstream and exits the siphon at the outfall, the diameter of the discharge line between the pump house and the outfall will be 60 inches and the slope of the discharge line will be 1%.

5. **Joint Air Entrainment:** Air can enter the pipe network at pipe joints, particularly when those joints are ill-suited to vacuum pressure conditions. For this reason, pipe joints should be heat-fused and any flanged connections should be gasketed, air-tight, and tested as such.

6. **Pipe Vacuum Pressure:** In the case that all air entrainment has been eliminated, the siphon would then work properly under vacuum conditions for some distance upstream and downstream of the penstock apex in the right overbank area near the Outlet Dam. However, vacuum pressure must not be so high that the pipe would collapse. The following equation calculates the critical collapse pressure of a steel pipe assuming a wall thickness of 0.18 inches, a modulus of elasticity of 30×10^6 psi, a Poisson's ratio of 0.28, and an outside diameter of 60 inches (Krempetz et al. 1986):

$$P_c = \frac{1}{4}\frac{E}{(1-\nu)^2}\frac{T^3}{R^3}$$

where P_c = collapsing pressure (psi)
E = modulus of elasticity (psi)
T = wall thickness (in)
R = pipe outside radius (in)
ν = Poisson's ratio

Results indicate a collapsing pressure of 3.1 psi, or 7.1 feet of vacuum head. Again, the pipe apex would be manipulated to ensure that this maximum vacuum head is not reached. Furthermore, steel pipe would be required on the discharge side to protect against pipe collapse.

One possible configuration of a siphon system would closely resemble the gravity system described above under Alternative 1. In this configuration, the intake and overall pipe alignment would be the same as that for gravity, and the pipe would likewise be buried through the shallow part of the outlet channel to maintain navigability through this region. The location of the pump house and discharge pipe alignment and outfall would also match that of the gravity system. The primary difference between the two systems is that the siphon would lift water from the reservoir to a point that is higher than the level of the lake using only the vacuum pressure of the fully primed siphon. A pump house located in the overbank area near the Outlet Dam would therefore be above ground, such that siphoned water would rise above the level of the lake.

3.3.3 Alternative 3 – Groundwater Well System

Alternative 3 would forego a surface water source in favor of tempering river water using groundwater. In other words, under this alternative the cooler water of the hypolimnion will not be used. Under this alternative, two different options were conceived. Option 1 would include a single point of diversion using a single groundwater well located near the Outlet Dam and discharging water to the river at one location. Option 2 would include a spatially-distributed well field composed of several wells running the length of the river and discharging groundwater into the river at different locations. The primary advantage of Option 2 over Option 1 is the thermal efficiency of storing groundwater in the aquifer until it is needed, rather than introducing all of the required groundwater at once and forcing it to warm slightly as it travels downstream. Disadvantages of Option 2 over Option 1 include increased capital and O&M costs to procure and run multiple wells and the possibility of not meeting flow requirements at locations far upstream where contributions from only one smaller well pump are realized.

Overall, the advantages of Alternative 3 include the fact that no construction through the upper portion of the river and dam would be needed. Also, depending on the configuration, the system could be located away from the Outlet Dam, which may be a mitigating factor for local residents. Under both options, the system is modular, such that added capacity could be introduced in the future, if needed.

The primary disadvantages of Alternative 3 include the O&M costs, which are expected to be higher than a lake pump station, due to the presumed static groundwater level and required pumping head. A cursory survey of all adjudicated groundwater rights in the immediate vicinity of the Outlet Dam reveals that the distance to the water table from ground surface is approximately 128 feet (Well ID #421893, #333608, #333946, and #333933), which is more than double the pumping height required at the lake. Another important disadvantage of Alternative 3 is the possibility of groundwater temperatures that are higher than the 8°C target temperature for mixing. This might require more pumping of groundwater at a higher required head than was originally conceived. Furthermore, a groundwater right for 45 cfs to 71 cfs may be difficult to obtain.

3.3.4 Alternative 4 – Pump Station

Alternative 4 includes a pump station intake system with both a suction inlet line and a discharge line. The discharge line would extend from the pump station to a discharge location just downstream of the Outlet Dam, and the suction inlet line would extend from the pump house into the lake. A pump station alternative assumes no gravity flow. Therefore, the pipe alignment is free to ascend above the level of the lake. Also, a pump station alternative is distinct from a siphon alternative, such that the suction lift does not have the same limitations as a siphon, and the pump house can be located above the maximum apex of a siphon line.

One possible configuration for the intake facility would include a pump house located in the overbank area away from the lake. The pump house would be situated above a caisson-style wet well that includes an inlet pipe protruding from the caisson horizontally into the lake. The protruding inlet pipe would be capped by a tee-type or other fish screen. In this configuration, screened water would flow through the protruding inlet pipe and into the wet well by gravity. One or more submersible pumps would then pump screened water from the wet well into the discharge line. Screen cleaning could be achieved by air burst using a pressurized line, or by active screen cleaning using mechanical brushes. This wet well configuration has several advantages, including no de-watering, cofferdams, trenching, or substantial shoring, and moderate O&M costs.

3.3.5 Alternative 5 – Ranney Well

Alternative 5 would include a Ranney well collector system located within the Priest River or along the bank of the river downstream of the Outlet Dam. Ranney well collectors consist of a large steel reinforced caisson embedded vertically into the river alluvium and several collector laterals that radiate out horizontally from the caisson. The laterals are screened or slotted ductile iron pipes that are jacked from the caisson into the surrounding alluvium. Groundwater or surface water tempered by the riverbed's hyporheic zone moves through the alluvium by gravity into the central caisson where vertical well pumps then pump the water into a discharge line that conveys the water into the river. Advantages of Ranney well collectors include the affordability of constructing a single large borehole casing and hydraulically jacking the laterals rather than drilling several isolated wells. A Ranney well system would not require a fish screening system, as the lateral collectors are located well below the riverbed, which is another advantage. The major disadvantage of a Ranney well is the uncertainty in water temperatures near the alluvium. Should water quality testing prove that a deeper caisson is needed to collect water that is suitably cold, this would add proportionally to overall O&M costs. Similar to Alternative 3, this alternative will not use the cooler water of the hypolimnion layer, and a groundwater right would need to be obtained. It should be noted that it would be recommended to perform geotechnical survey before advancing this alternative, as the alternative is dependent on geotechnical ground conditions and groundwater temperature.

3.3.6 Alternative 6 – Groundwater Well System with Aquifer Recharge

Alternative 6 would resemble the groundwater well system described under Alternative 3, with the added aquifer recharge. The objective of this alternative is to improve the reliable supply of groundwater by increasing the seasonal infiltration and deep percolation of surface water. Certainly, the river system currently recharges the underlying aquifer, as the two are inter-connected. This alternative would augment that natural recharge by increasing the area of activated floodplain along the river. The river morphology is currently defined by a shallow canyon that confines the channel at most discharges. Increasing the area of activated floodplain will therefore require substantial cuts in the overbank to provide areas of infiltration during seasonal high runoff during the spring and early summer. In some areas this may not be feasible due to the proximity of Highway 57 or the steepness of the terrain and the corresponding cost of excavation. In other areas where excavation may be possible, the location downstream would not be to the advantage of a groundwater well system because the wells may not be able to capture the stored water. Furthermore, there are no real indicators that a reliable supply of groundwater is available, which renders the primary objective of this particular alternative moot. For these reasons, this alternative is dismissed from further consideration.

3.3.7 Alternative 7 – Tree Shading

Alternative 7 would provide tree shading along the Priest River to reduce thermal gains within the reach. The reach below the Outlet Dam is already tree-lined. However, the canopy does not extend well into the channel to provide shade to the water below. Introducing deciduous trees with larger crowns to the river banks would help diminish thermal gains between the Outlet Dam and downstream reaches. The canopy would also be an important source of wood to the river, which can act as refugia and create localized morphologic features amenable to fish spawning and rearing.

There are several important limitations to tree shading. Perhaps the most important of these is the limited effectiveness of shading to reduce temperatures down to the target temperature. In the summer months, when surface water discharging into the river is already at a relatively high temperature, tree shading will help insulate and buffer against thermal gains, rather than noticeably reduce temperatures in the river. This does not meet the objectives of the Project. Furthermore, extending the canopy over the river will require first planting or transplanting trees, which will then require up to a decade to truly extend out over the river. In doing so, it may be necessary to clear out some of the existing timber adjacent to the river to provide sufficient space to promote growth of the new canopy. This may result in an extended period of time in which the shade cover above the river is actually less than it is right now. Finally, the width of the Priest River below the Outlet Dam is approximately 150 feet wide. However, mature cottonwood trees have crown widths up to about 75 feet. Lining both sides of the river would still leave half of the river exposed. This is exacerbated by the fact that the river generally follows a north-south orientation below the Outlet Dam, such that shading is at a minimum during the height of solar insolation around midday in the summer. For all of these reasons, this alternative is dismissed from further consideration.

3.3.8 Alternative 8 – Eductor System

Alternative 8 includes an eductor pump system that uses the Venturi effect to draw water from a lower-elevation thermocline into an inlet pipe located at a higher elevation in the water column. An eductor system utilizes a nozzle to accelerate the motive fluid to create a localized pressure drop that induces suction in a branch line, drawing in deeper water like a pump. The motive fluid is simply lake water located at a higher elevation in the lake. The motive fluid is pumped from the lake using one or multiple eductor pumps located in a pump house. The motive fluid is conveyed back into the lake where the nozzle and suction line connection are located. Cold-water is drawn into the discharge line, which extends to the Outlet Dam, discharging downstream of the structure. See Figure 3-3 for a similar setup used to dredged material from a lakebed.

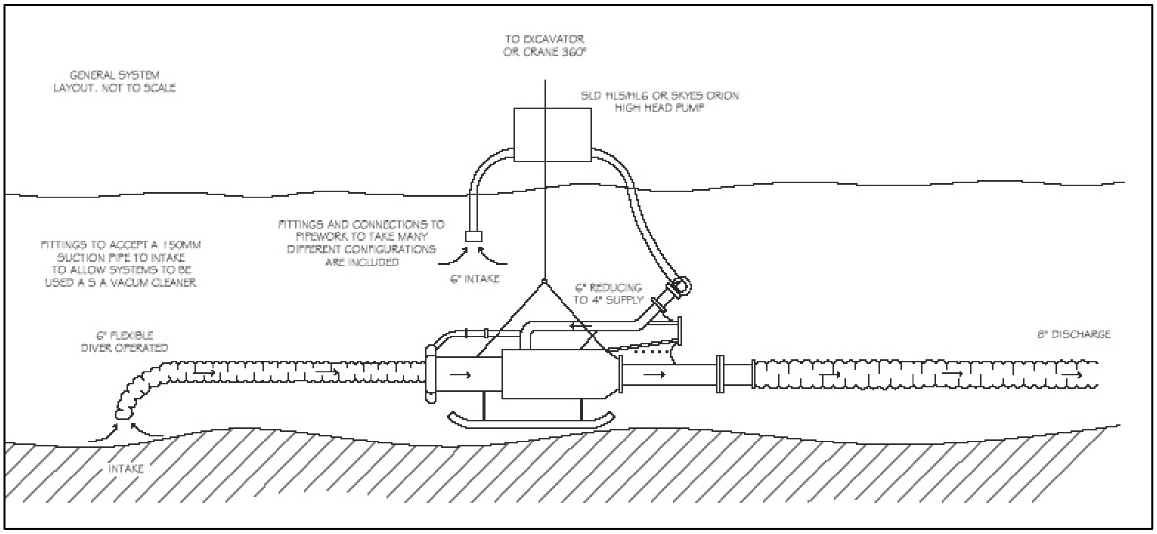

Figure 3-3. Example Eductor System Used for Venturi Dredging

In terms of energy efficiency, eductor systems are superior to conventional pumps, and lead to lower O&M costs. The reason for this efficiency is that the Venturi effect allows the system to overcome a greater head differential than would otherwise be possible using a direct pumping system. Importantly, eductor systems require mixing of the motive fluid with the suction fluid. This is not a fatal flaw for this project, because both fluids are lake water. However, the thermal properties of the two source fluids are different. That is, the motive fluid is sourced from shallower thermoclines, and is therefore warmer. For this reason, the suction line must extend down to lower thermoclines than would otherwise be required. In a thermally stratified lake like Priest Lake, lower thermoclines are located at lower elevations. Therefore, gains in efficiency due to the Venturi effect are offset by the need to access water at lower elevations. In addition, the eductor system requires more appurtenances and more piping than a conventional pump station. For these reasons, the eductor system is dismissed from further investigation.

3.3.9 Alternative 9 – Passive Upwelling

Alternative 9 was conceived during the project brainstorming session. The idea behind this alternative is based on density currents that passively enter and exit lakes. Density currents are defined by heavier, sediment-laden water that enters a lake but, because of its greater density, hugs the bottom of the lake as it approaches the lake outlet. In some cases, these density currents may up-well to the surface near the lake outlet as the water encounters the shoreline topography, or winds induce the upwelling near the shore. Similarly, colder water is denser than warm water and will therefore tend to hug the bottom of a lake. Even in lakes that are not properly thermally stratified, if the water flowing into the lake is sufficiently colder than the surface water of the lake, the inflowing water will dive down into the lake at the delta due to the density gradient, traveling along the lake bottom toward the outlet. There is some indication that tidal fluctuations can assist passive upwelling of cold-water in oceans (Tee and Smith 1993). These actions could be simulated at Priest Lake through operation of the Outlet Dam, in conjunction with the introduction of an appropriate sub-surface barrier. The obvious advantage of this alternative is the passive or near-passive mechanisms by which it would work. However, several major disadvantages preclude this alternative from further investigation, including: unknown temperature of the density current; unproven as an engineered system; unknown need for wind induction; unknown impact of existing lake outlet compared with natural outlet; requires significant and costly numerical and physical modeling; may require costly dewatering or otherwise costly construction.

3.3.10 Alternative 10 – Trap and Haul

Alternative 10 would involve the development and implementation of a trap and haul program for target fish species within the Lower Priest River. Under this alternative, an area of the river would be identified as a haul reach that would establish the extents of the program. Trapping of target species would take place downstream of the haul reach, and the transporting and releasing of fish would take place upstream of the haul reach. Presumably, release of target species would be located somewhere at Priest Lake, e.g. at Kalispell Bay to limit the possibility of fall-back through the Outlet Dam. Trapping fish for the program would require the construction of a seasonal in-river exclusion barrier and trapping facility. The exclusion barrier could be one of several channel-spanning types, including velocity barriers, picket barriers, and vertical drop structures. The trapping facility would include a fishway to allow fish to volitionally enter a trap holding pool. Fish would be sorted in a sorting pool and either bypassed back to the river or loaded into a transport hopper and hauled upstream. A dedicated release site would also be required to safely release fish into the lake. Typical capital costs for a trap and haul facility of the size needed along the Lower Priest River would range from $4M to $6M. Ongoing O&M costs for this alternative would be comparatively high, due to the need for dedicated personnel to operate the fish trap and to haul fish upstream.

As noted in the original project Statement of Work (Avista 2018), the purpose of the project is to “cool Lower Priest River […] for the benefit of fish and aquatic biota.” This alternative would not achieve the stated purpose of the project because it would not lower water temperatures in the river proper. Although this alternative may address some of the needs of fish species, it fails to address the needs of other aquatic biota that would neither be transported upstream and released in the lake, nor necessarily benefit from a lacustrine environment. Furthermore, this alternative does nothing to improve and expand suitable riverine habitat for target species of fish. For these reasons, this alternative is dismissed from further investigation.

3.3.11 Alternative 11 – Mechanical Chiller

Alternative 11 would consist of a mechanical chiller, or a bank of modular chiller units, that would mechanically chill incoming river water and discharge cold-water to the area below the Outlet Dam. Because the chiller units require the inflow to be pressurized, the system would also include a pump and wet well or vault to pump surface water through the chiller. Discharge piping and appurtenances would also be required, as would reliable 3-phase electrical drops to the system. Chiller mechanical and electrical controls would be housed in a utility building, which would include pump controls, monitoring and instrumentation, service equipment and materials.

The assumed incoming water temperature is 22°C (71.6°F), whereas the target water temperature is 19°C (66.2°F). The required temperature differential is therefore 3.0°C (5.4°F). Assuming a minimum tempered flow rate of 45 cfs (20,196 gpm), the required energy to condition the water is:

20,196 gpm x 60 min/hr x 8.33 x 5.4°F = 54,507,388 BTU/hr.

This results in 4,542 tons of chilling capacity. Assuming an air-cooled chiller scaled linearly from a recent hard bid quote for a 160-ton chiller, the resulting chiller capital cost would be approximately $7 to 8 million. This cost does not include freight, installation, or mark-up, nor does it include the cost for ensuring adequate 3-phase power to the site. Water must be pumped through the heat exchangers in the chiller modules, but the cost for pumping, piping, and appurtenances is also not included in this cost. Total capital cost is expected to be in the $15M range.

Assuming the chillers were in operation for two full months per year, assuming a commercial electricity rate of 10¢/kWh, and assuming an excellent part load condition of 0.5 kW/ton of chilling capacity, the total annual operational cost for the chillers would be:

4,542 tons x 0.5 kW/ton x 60 days x 24 hours x 10¢/kWh = $327,024/yr

Over a 20-year period, the life cycle cost of this alternative would be approximately $21.5M in 2018 dollars.

3.4 Initial Screening and Evaluation

As part of the brainstorming meeting, a series of evaluation criteria were used to evaluate the cold-water augmentation alternatives. A brief summary of each alternative evaluation criterion is presented in Table 3-1. Table 3-2 was used to determine which alternatives should be advanced for further evaluation. The alternatives that are recommended for elimination and removal from further consideration are considered infeasible.

Table 3-1. Description of Major Evaluation Criteria

Criterion	Description
Design Feasibility	This criterion means that the alternative has the potential to meet the design criteria presented in the Basis of Analysis.
Advantages	Lists the advantages of the system.
Disadvantages	Lists the disadvantages of the system.
Cost	Ranks the capital and O&M costs according to low, medium, high, and very high. This is a relative measure of the potential cost associated with each alternative. These criteria are intended to provide a relative comparison between alternatives, not a quantitative cost range.
Recommendation	Indicates whether the alternative should be advanced for additional evaluation or removed from further analysis.

Table 3-2. Initial Screening and Evaluation

Alternative Number	Description	Design Feasibility	Advantages	Disadvantages	Cost		Recommendation	Justification
					Capital	O&M		
1	Gravity system with 4 to 5 ft DIA pipeline extending from 8°C thermocline to just downstream of the Outlet Dam.	Yes	• Gravity • Low O&M cost • No construction through Outlet Dam • Potential for high diversion flow rates • Potentially out of sight	• Head losses • Dependent on lake levels and dam operation • No cost sharing opportunity	High	Low	Advance	This alternative has a low OM cost which is enticing.
2	Siphon with 4 to 5 ft DIA pipeline extending from 8°C thermocline to just downstream of the Outlet Dam.	Yes	• Mostly passive operation • Low O&M cost • Potential for high diversion flow rates	• Could lose prime. • Impacts to aesthetics in the upper reach • Temperature mixing • No cost sharing opportunity • Visual obstruction	High	Low; Higher than 1	Advance	This alternative has a low OM cost which is enticing.
3	Use groundwater source, either a well field or create some cold water refuge. (Option 1 – all upstream, Option 2 – distributed)	Yes	• No intake • Short distribution line • Could keep the temperature cooler • Operation flexibility • Fewer impact to property owner • Keep low flow – additive	• Water right • Data gap (water temperature and availability, hydrogeology) • Water quality • May need more flow since we are adding and not replacing – higher flow.	High	High	Advance	Avoids impacts to lake and lake users. Would need new water quantity estimates since cold water would be additive, not replacing existing flow. Distributed wells could create “ladder” of cold water refugia.
4	Pump station.	Yes	• Pump and pump station located along the lakeshore provides accessibility for O&M • Well-proven technology • Variable speed pumps can be adapted depending on required operations	• On-going operations and maintenance costs • High capital and replacement costs	High	Medium	Advance	Proven technology, manipulation options.
5	Ranney well – use groundwater source located in alluvial channel with suction lines extending radially from a central caisson.	Possibly	• Possible saving on O&M cost compared with Alt 3 • May be categorized by agencies as a surface water source, eliminating need for new water rights	• Instream structure • Need to be mitigated against scour, etc. • May not have as low temperature • Data gap (water temperature and availability, hydrogeology)	High	Lower than 3	Do Not advance	This technology is typically used for water treatment. The temperature reduction through the hyporheic zone is unknown.
6	Divert high spring flow in the flood plain to recharge.	Yes, maybe	• Recharge groundwater through active flood plain • Structural habitat addition	• Less control over the temperature and amount of flow • Need to do groundwater investigation • Acquire land • Permitting	Low	none	Do not advance	High uncertainty even after investments are made in land acquisition, floodplain connections, water diversions.
7	Tree shading of the Lower Priest River.	Unlikely	• Low tech • Improves habitat beyond simply temperature	• South facing river • Wide river that is already well-lined with trees may not change temperature profile sufficiently	Low	Low	Do not advance	Warm water will still be the majority of discharge at upstream end of project. Slow ecological response.
8	Eductor system – flow-induced suction of cold water through a pipe.	Possibly	• Could draw cold water at variable depths • Could have efficient application, in theory	• Would require extensive study • New application of technology; not proven • May have significant unforeseen environmental impacts • May have high capital and possibly O&M costs	High	High	Do not advance	Do not want unproven technology.

Alternative Number	Description	Design Feasibility	Advantages	Disadvantages	Cost		Recommendation	Justification
					Capital	O&M		
9	Passively Induced Upwelling – Introduce a structure to create an upwelling of cold water based on vertically-oriented eddies.	Unlikely	• Passive technology, i.e. potential low O&M costs	• Would require extensive study • New technology; not proven • Functionality may vary greatly inter- and intra-annually	High	Low	Do not advance	Don't currently have resources for extensive study but could be option to revisit in the future.
10	Trap and haul fish from the confluence to the lake.	No	• Very low capital costs if a trap and haul facility already exist	• High O&M costs • Non-volitional passage • No improvements to habitat • Maintains habitat dis-connectivity	Low	High	Do not advance	Remove from alternatives, this would not meet project goals of improving river fishery.
11	Mechanical chiller to cool readily available surface water by pumping it through a heat exchanger.	Possibly	• No need to take water from the hypolimnion	• Large mechanical system • Limited by the amount of power to produce a large amount of flow	Very High	Very High	Do not advance	Expense, power needs.

3.5 Alternatives Investigated Further

As a result of the brainstorming session, the team decided to advance four (4) alternatives forward to conceptual design:

- Alternative 1 – Gravity System
- Alternative 2 – Siphon System
- Alternative 3 – Groundwater Well System
- Alternative 4 – Pump Station

Conceptual drawings were developed for each alternative and are presented in Appendix A. The calculations are presented in Appendix B, and the cost estimate is presented in Appendix D. This Section presents a detailed alternative description of each alternative.

Note that each of the alternatives that were investigated further were required to meet a range of flows 45 cfs to 71 cfs (or higher) under normal operating conditions. Alternative 1B is the only alternative that exceeds 71 cfs; it was developed to supply 105 cfs of flow.

3.5.1 Alternative 1 – Gravity System

Alternative 1 would consist of a gravity water supply system connecting the deep-water area of Priest Lake with the Lower Priest River below the Outlet Dam (see Figure 3-4). Upon further consideration a second option was added for the gravity system to allow for additional augmentation flow, leading to Alternative 1A and Alternative 1B. This section describes Alternative 1 generally, and then describes the details individual to Alternative 1A and Alternative 1B. Note that the main difference between Alternative 1A and Alternative 1B is that 1B can accommodate higher flows, up to 105 cfs, through the use of two intake screens, a manifold, and a larger bypass pipe.

The proposed alignment for this alternative is the same as that proposed for Alternative 2 – Siphon System. Therefore, the following notes apply to Alternative 1A, 1B and 2.

- In the area of Outlet Bay where the pipeline approaches the dam, it is not clear what are the exact bathymetry and depths of water throughout the year. During the recreational period, the water level is at least 3.0 feet on the Outlet Dam gage, and during the spring runoff the levels are even higher than this. However, during the winter months, it is not evident how shallow the Outlet Bay becomes, based on available data. For this reason, the proposed pipeline would be buried in the upper portion of the lake near Outlet Bay. The pipeline would be buried with approximately 0 to 2 feet of cover for a distance of approximately 5,600 feet upstream of Outlet Dam. Installing the pipe would require a multi-barge system to excavate the pipe trench, fuse, pick and place the pipe, and backfill over the pipe (if needed).

- Underwater excavation can cause large increases in turbidity and total suspended solids (TSS) by re-suspending fine lake material that has deposited on the lakebed over time. In general, finer material is found closer to dams. The reason for this is that coarser material will drop out in a lake delta where inflowing streams enter the lake and sediment transport velocities drop precipitously. Finer material, on the other hand, will continue to transport toward a lake's outlet, with some fraction depositing upstream of an outlet control (e.g. a dam) and some fraction passing through the outlet. Because finer material is expected near the Outlet Dam, excavating in this area will likely re-suspend fine material. This could pose a risk to aquatic species. Certainly, measures can be employed to minimize these increases in turbidity and TSS. These measures might include silt curtains or bubble curtains, as examples. However, these measures will only reduce the re-suspension of material; complete avoidance of sediment impacts is not possible with underwater excavation.

Working in the downstream direction, the gravity system would include the following components for both Alternative 1A and Alternative 1B:

Note that in the following enumerated notes, there are several references to tee screens. All these references are made as if there is only one tee screen; however, for Alternative 1B, there are two tee screens. An explanation of this difference can be found in Section 3.5.1.1 Alternative 1A and Alternative 1B.

1. The system would include an NMFS-compliant tee screen to keep fish out of the intake pipe. The tee screen and inlet pipe would be located approximately 60 feet below the surface of the lake to convey water from the 8°C thermocline. The approach velocity would be limited to 0.5 ft/s to minimize the possibility of impinging fish and debris against the screen. The screen would be passive, with no automated screen cleaning. Rather, screen cleaning would be part of scheduled maintenance and would involve divers to clear the screen on an annual or semi-annual basis. Pressure transducers on the outlet pipe would indicate the accumulation of debris on the screen, which would allow active, remote monitoring of the screen.

2. The pipe and screen would be supported at intervals by precast reinforced concrete pedestals. Resting the pipe on pedestals would keep the pipe off the lake bottom and would help reduce the chance of debris buildup against the tee screen. The pipe could be affixed to the pedestals using stainless steel pipe straps. The pipe material would be HDPE, which is nearly neutrally buoyant. However, to limit pipe movement vertically and laterally due to any buoyancy and lake currents, pedestals and pipe straps will secure the pipe at intervals not to exceed 20 feet. Both pipe and pedestals could be lowered to the lake bottom from a construction barge, with pipe strap placement and pipe joinery taking place on the barge, and pedestal placement occurring with the help of divers.

3. The pipe would extend approximately 7,500 feet from the intake to the dam, where bends in the pipe would direct the pipe to the right overbank area (looking downstream). The bends would be located approximately mid-stream, roughly 70 feet upstream of the dam near the existing buoys. The pipe in the deeper portions of the lake would be laid on the lake bed and adjusted with the help of divers. Upon entering the Outlet Bay, however, underwater excavation or dewatering is expected to bury the pipeline underground to its outlet on the downstream side of the Outlet Dam,

approximately 5,500 ft. Native material would be backfilled over the pipeline to protect against uplift and to maintain the existing draft depths through the Outlet Bay.

4. Pipe extending underground into the overbank area would then penetrate a booster pump vault in the right overbank. The top of the vault could be flush with the surrounding ground, so that no vertical obstruction would be seen penetrating the field of view. The vault could be a simple pre-cast reinforced concrete vault sunk below ground and large enough to accommodate a booster pump, isolation valves, control panel, pump sump, space for two maintenance personnel, and a butterfly valve to control flow.

   The booster pump could be a low-head, high capacity axial-flow type pump. Sectional drawings of the dam indicate a hydraulic head of 7.35 feet between the dam forebay and the riprap apron downstream of the dam. Total head losses, less than 7.35 feet for both Alternative 1A and 1B (when calculated using the Darcy-Weisbach friction loss equation); therefore, under normal operating scenarios during the recreational season, sufficient net head would exist to convey between 45 and 105 cfs through gravity. However, in the worst case that the dam's radial gates are fully open and the hydraulic head between headwater and tailwater is closer to zero, the booster pump would provide the required pressure to convey 45 to 105 cfs downstream by overcoming the head losses in the pipe. As the pipes age, the head losses in the pipes will increase, which could lead to needing to use the pump more often.

5. Discharge piping would then extend from the booster pump vault to a headwall located along the right bank of the Lower Priest River. The pipe would penetrate the headwall and discharge water onto a riprap apron to dissipate energy. The tailwater below the dam may partially or fully submerge the outlet pipe at certain times of year. This is considered acceptable, provided that the gravity line has enough hydraulic head to convey the target flow into the submerged tailwater.

3.5.1.1 Alternative 1A and Alternative 1B

Alternative 1A has one bypass pipe with one intake screen. Alternative 1B has two fish screens, a manifold, and a bypass pipe. The manifold is formed with two short pipes, which are a combined total of 30 feet in length, connecting the intake screens with the bypass pipe. The manifold allows for two smaller intake screens, instead of one larger one, which offer redundancy and decrease cost. The larger bypass pipe diameter for Alternative 1B (when compared to Alternative 1A) allows for additional augmentation flow. Table 3-3 presents the differences between the two alternatives. In Table 3-3 there is a column for Alternative 1B manifold and another for the bypass pipe. Additionally, the total head losses and the pump horsepower are combined Alternative 1B, instead of breaking it down by pipe (See Appendix B for Calculations).

Note that the tee screen approach velocity for Alternative 1B is 0.33 fps, which is 0.17 ft/s lower than the maximum design criteria for the tee screen approach velocity. The tee screen velocity could be increased by using a tee screen with a smaller diameter. Smaller tee screens, with a smaller diameter pipes leading to the manifold, were considered; however, decreasing pipe diameter, increases frictional head losses in the pipe. The increased head losses in the pipe reduced the factor of safety to less than one foot of available head. To avoid a small factor of safety (less than one foot of available head), larger fish screens and larger pipes to the manifold were chosen.

Table 3-3. Comparison of Alternative 1A and Alternative 1B

	Alternative 1A	Alternative 1B (manifold)	Alternative 1B (bypass pipe)
Total Head Losses (ft) (Using Darcy-Weisbach Equations)	5.28	6.04	
Pump Horsepower (hp)	47	80	
Total Pipe Length (ft)	8000	30	8000
Maximum Augmentation Flow per pipe (cfs)	71	52.5	105
Pipe Diameter (ft)	5	5	6
Pipe Area (ft^2)	19.6	19.6	28.3
Tee Screen Approach Velocity (ft/s)	0.45	0.33	N/A
Tee Screen Area per Tee Screen (ft^2)	157	157	N/A

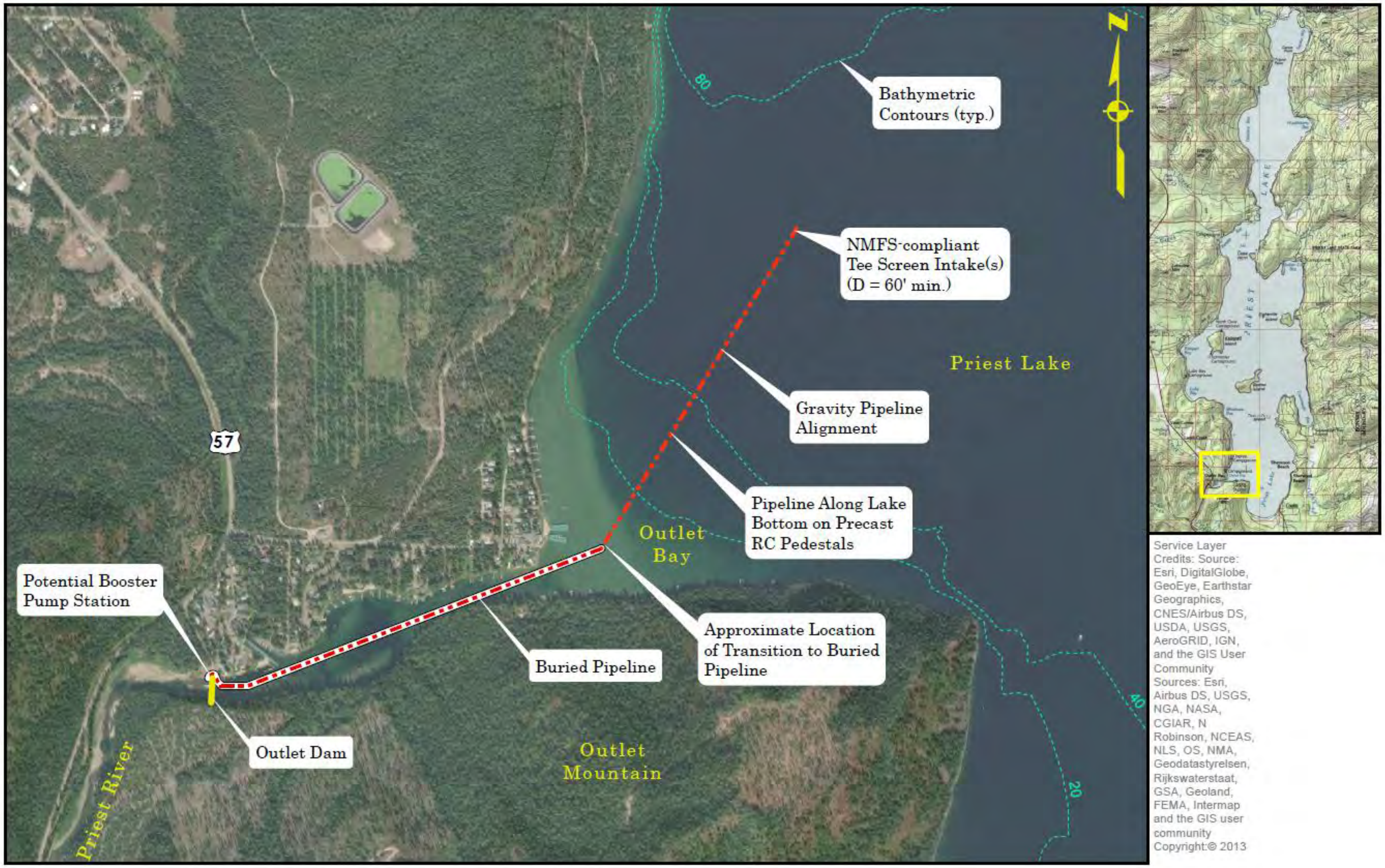

Figure 3-4. Alternative 1: Gravity System Layout

3.5.2 Alternative 2 – Siphon System

Alternative 2 would consist of a siphon system connecting the deep-water area of Priest Lake with the Lower Priest River below the Outlet Dam (see Figure 3-5). The pipe alignment upstream of the dam for this alternative would be identical as that for Alternative 1. See the notes in Section 3.5.1 that apply to the alignment for these two alternatives. Working in the downstream direction, the system would include the following components:

1. The system would include an NMFS-compliant tee screen to keep fish out of the intake pipe. The tee screen and inlet pipe would be located approximately 60 feet below the surface of the lake to convey water from the 8°C thermocline. The tee screen would have a minimum screen area of 157 ft^2 to limit the approach velocity to 0.5 ft/s while conveying a maximum of 71 cfs downstream. These specifications would minimize the possibility of impinging fish and debris against the screen. The screen would be passive, with no automated screen cleaning. Rather, screen cleaning would be part of scheduled maintenance and would involve divers to clear the screens on an annual or semi-annual basis. Pressure transducers on the outlet pipe would indicate the accumulation of debris on the screen, which would allow active, remote monitoring of the screen.

2. The pipe and screen would be supported at intervals by precast reinforced concrete pedestals. Resting the pipe on pedestals would keep the pipe off the lake bottom and would help reduce the chance of debris buildup against the tee screen. The pipe could be affixed to the pedestals using stainless steel pipe straps. The pipe material would be 60-inch diameter HDPE, which is nearly neutrally buoyant. However, to limit pipe movement vertically and laterally due to any buoyancy and lake currents, pedestals and pipe straps will secure the pipe at intervals not to exceed 20 feet. Both pipe and pedestals could be lowered to the lake bottom from a construction barge, with pipe strap placement and pipe joinery taking place on the barge, and pedestal placement occurring with the help of divers.

3. The pipe would extend approximately 7,500 feet from the intake to the dam, where bends in the pipe would direct the pipe to the right overbank area (looking downstream). The bends would be located approximately mid-stream, roughly 70 feet upstream of the dam near the existing buoys. The pipe in the deeper portions of the lake would be laid on the lake bed and adjusted with the help of divers. Upon entering the Outlet Bay, however, underwater excavation or dewatering is expected to bury the pipeline underground to its outlet on the downstream side of the Outlet Dam, approximately 5,500 ft. Native material would be backfilled over the pipeline to protect against uplift and to maintain the existing draft depths through the Outlet Bay.

4. Pipe extending underground into the overbank area would then enter a pump house located in the right overbank. The pump house would be above ground and would include a priming valve and a vacuum pump. The vacuum pump could be a relatively small pump (5-10 hp) used to evacuate air from the siphon line during initial and maintenance priming. As noted above, the apex of the pipe would be no greater than 7.1 feet above the lake water surface in order to meet restrictions related to vapor pressure and critical pipe pressure. The pump house could be a simple pre-fabricated metal

building large enough to accommodate the vacuum pump, isolation valves, control panel, space for two maintenance personnel, and perhaps a throttling valve to control flow.

5. The discharge piping exiting the pump house would fork in two separate directions to satisfy two operational scenarios. During the recreational season, when the hydraulic head across the dam is between 6 and 7 feet, there is sufficient hydraulic drop to discharge siphoned water just below the Outlet Dam, because the drop compensates for the frictional and minor losses in the system. However, when the gates are fully opened and the hydraulic drop across the dam is insufficient, the siphon discharge must be located further downstream, again so that the total losses in the system are compensated. Assuming a river channel slope of 0.2% below the dam, this would require a discharge location approximately 1500 feet downstream of the dam to compensate for 3 feet of head loss. Therefore, piping leaving the pump house could be valved off to isolate the lower head discharge line from the higher head discharge line, depending on dam operations.

6. Both discharge pipes would penetrate concrete headwalls located along the right bank of the Lower Priest River. The pipe would discharge water onto riprap aprons to dissipate energy. The river below the dam may partially or fully submerge the outlet pipes at certain times of year. This is considered acceptable, provided that the siphon can overcome the frictional losses in the system given the head differential.

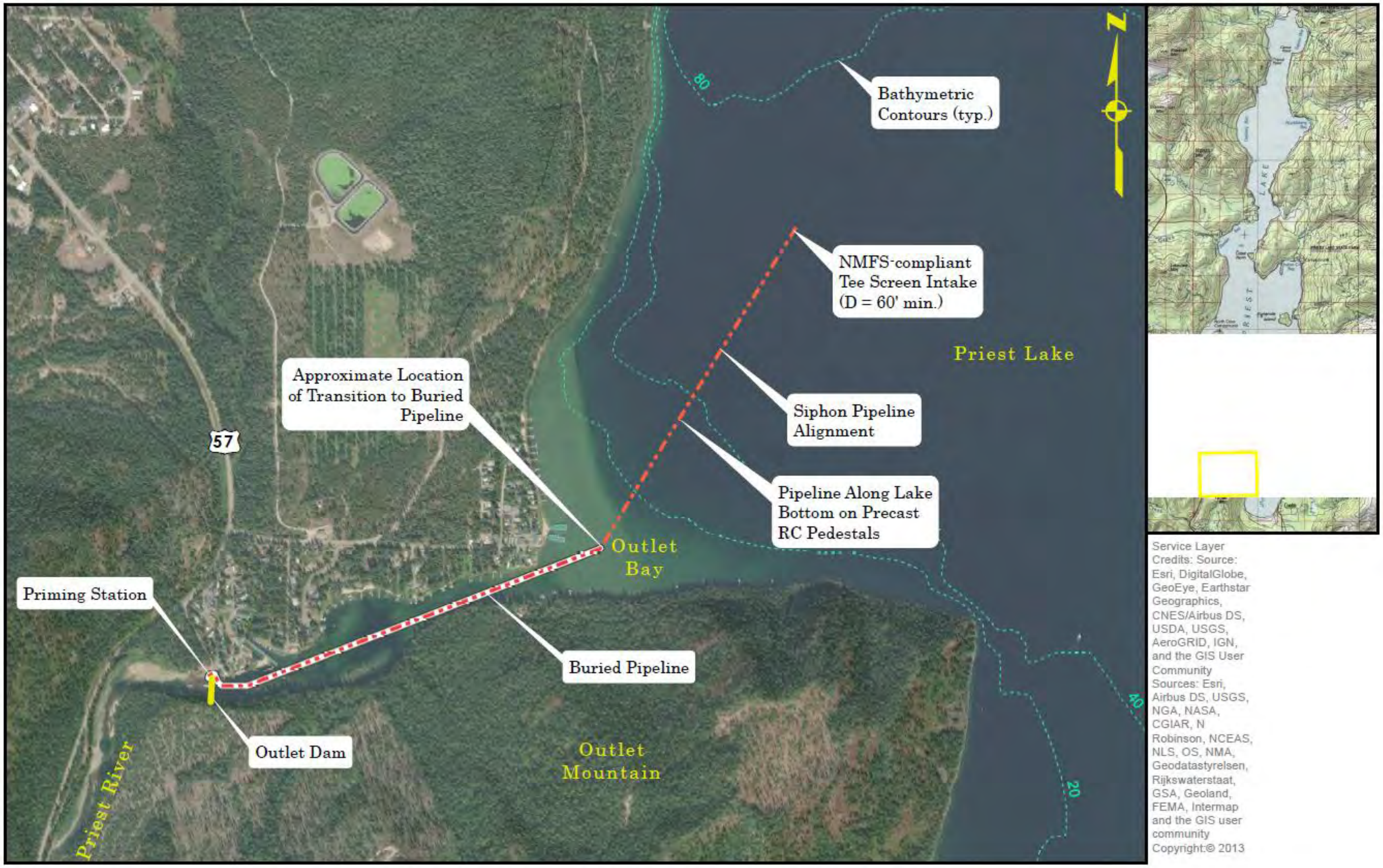

Figure 3-5. Alternative 2: Siphon System Layout

3.5.3 Alternative 3 – Groundwater System

Alternative 3 would include a groundwater well system to supply cool water to the river (see Figure 3-6). The alternative described here is a hybrid of Options 1 and 2 under Alternative 3 that were described earlier, insofar as the groundwater system would neither be distributed along the river from upstream to downstream, nor would it consist of only one well. Instead, a locally-distributed cluster of groundwater wells would be sited near the Outlet Dam that would pump water through a pump house water treatment system and back into the upper reach of the Priest River. The system would include the following components:

1. A groundwater well field would be developed downstream of the Outlet Dam consisting of several submersible groundwater pumps located in deep casings extending below the water table. The depth of each well casing and submersible pump would be dictated by the known low groundwater table, which would be determined by an exploratory groundwater investigation in the vicinity of the Outlet Dam. A cursory survey of several potable water wells drilled in the vicinity of the Outlet Dam reveals that the distance to the water table from ground surface is highly variable depending on location. A total of six well logs were compared (Well ID #333946, #333608, #334231, #421893, #333933, and #358023). In all cases, the wells are not under artesian pressure, such that water levels correspond with the water table of an unconfined aquifer. Static water levels ranged from 188 feet below ground surface (bgs) to 6 ft bgs. In one case (ID #334231), the well is completely dry, with only tightly packed clays observed in the 100+ foot borehole. This well is located in Outlet Bay roughly 1,500 feet upstream of the Outlet Dam. This variability underscores the need for groundwater investigation in the vicinity. For reasons of parsimony, however, the average depth to static water of 128 feet was assumed here. Adding on another 4 feet of head losses due to friction and minor losses and another 8 feet for the degassing towers (see below) results in a total required head of 140 feet. Assuming identical pumps operating at 80% efficiency, which is typical of medium size-class pumps, the total power requirement (brake horsepower) for the well field can be calculated using:

$$BHP = \frac{Q \times H \times SG}{3960 \times \eta}$$

   where Q is the total well field capacity in gallons per minute (gpm), H is the total head in feet (ft), SG is the specific gravity of water (equal to unity), and η is the pump efficiency.

2. The well field pumps would convey cold groundwater from the well through a network of buried pipes to a pump station located near the river. The pump station would consist of a small enclosure/building that would house piping inlet manifold, pump controls and a large discharge pipe. The pump station would also include one or more degassing towers to strip the incoming water of carbon dioxide (CO_2). Groundwater is known to contain higher concentrations of CO_2, and high concentrations of CO_2 (~20 parts per million [ppm] [Piper et al. 1982]) can be harmful to fish. It is not known at this time what the expected concentration of CO_2 is near the Outlet Dam. However, some level of gas stripping would likely be necessary. One alternative to stripping towers would include up-sizing the pumps and reducing the discharge pipe size to discharge water back to

the river at a higher velocity. The higher velocity water could then pass through an energy dissipation system (such as a baffle chute) in order to re-introduce oxygen into, and strip carbon dioxide from, the cooler water. For this alternative, however, a two-tower degassing system is assumed.

3. The water discharge pipe running from the pump station would connect with a small headwall and channel connecting with the river. Discharging water would pass through the headwall and channel and into the river at this point.

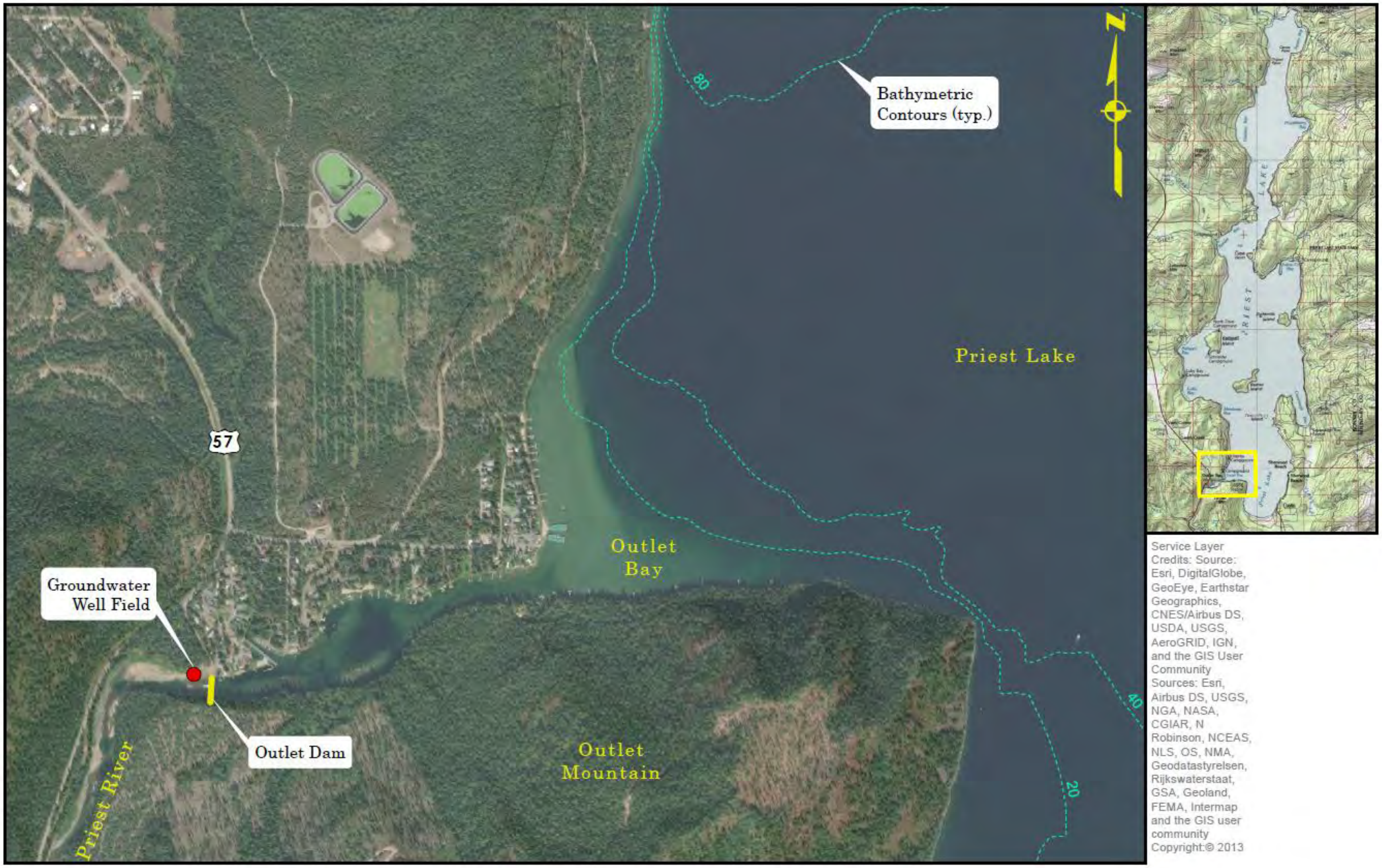

Figure 3-6. Alternative 3: Groundwater System Layout

3.5.4 Alternative 4 – Pump Station

Alternative 4 includes a pump station intake system with both a suction inlet line and a discharge line. The discharge line would extend from the pump station to a discharge location downstream of the Outlet Dam, and the suction inlet line would extend from the pump house into the lake (see Figure 3-7). Working in the downstream direction, the system would include the following components:

1. The system would include a NMFS-compliant tee screen to keep fish out of the intake pipe. The tee screen and inlet pipe would be located approximately 60 feet below the surface of the lake to convey water from the 8°C thermocline. The tee screen would have a minimum screen area of 157 ft^2 to limit the approach velocity to 0.5 ft/s while conveying a maximum of 71 cfs downstream. These specifications would minimize the possibility of impinging fish and debris against the screen. The screen would be passive, with no automated screen cleaning. Rather, screen cleaning would be part of scheduled maintenance and would involve divers to clear the screens on an annual or semi-annual basis. Pressure transducers on the outlet pipe would indicate the accumulation of debris on the screen, which would allow active, remote monitoring of the screen.

2. The pipe and screen would be supported at intervals by precast reinforced concrete pedestals. Resting the pipe on pedestals would keep the pipe off the lake bottom and would help reduce the chance of debris buildup against the tee screen. The pipe could be affixed to the pedestals using stainless steel pipe straps. The pipe material would be 60-inch diameter HDPE, which is nearly neutrally buoyant. However, to limit pipe movement vertically and laterally due to any buoyancy and lake currents, pedestals and pipe straps will secure the pipe at intervals not to exceed 20 feet. Both pipe and pedestals could be lowered to the lake bottom from a construction barge, with pipe strap placement and pipe joinery taking place on the barge, and pedestal placement occurring with the help of divers.

3. The pipe would extend approximately 200 to 300 feet along the lake bed toward the shoreline before running subsurface in the upland toward an intake station and pump house located on Westshore Road. This location is approximately 85 feet above the lake level. Therefore, the duty point of the pump would have to be at 92 feet of head to include losses. Due to the topography along the discharge alignment, a gravity flow option from the pump house is not feasible. Therefore, the discharge pipe will flow under pressure, in which case head losses in the discharge line must be included.

4. The discharge piping exiting the pump house would continue down Westshore Road before converging with Outlet Bay Road and continuing on to the intersection with Highway 57. From here, the pipeline would follow Highway 57 for approximately 1,500 feet and would turn south onto U.S. Forest Service property to the first bend in the Priest River downstream of the Outlet Dam. The pipeline would discharge to the river here, and a riprap energy dissipation apron would be located at the outfall where the pipeline daylights.

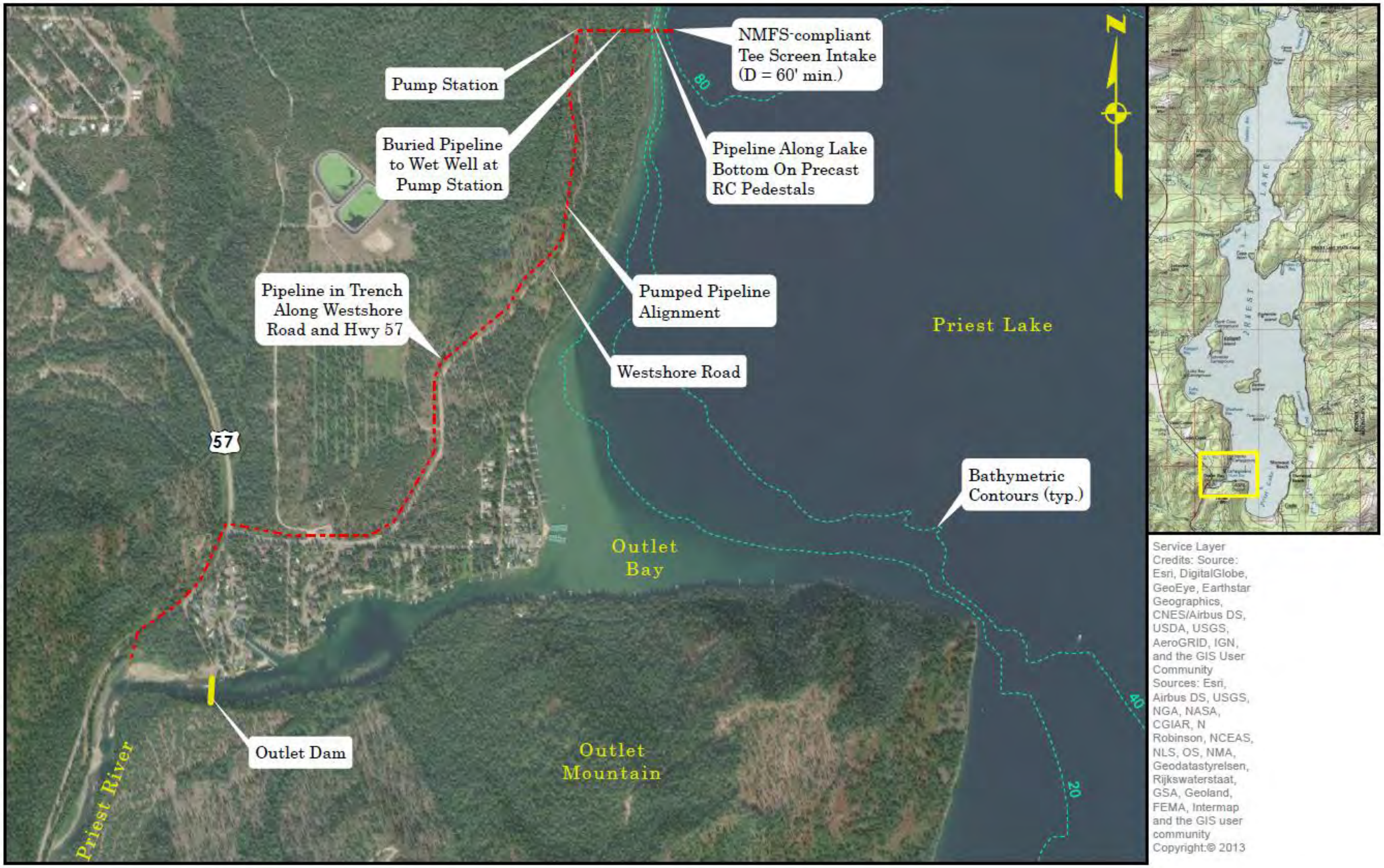

Figure 3-7. Alternative 4: Pump Station System Layout

4.0 Alternatives Assessment

4.1 Introduction

Section 4.0 presents the alternatives evaluation for the Priest River Cold-Water Augmentation alternatives analysis.

4.2 Evaluation Criteria

Conceptual level details were developed for each alternative to gain an understanding of the proposed alternative systems, determine potential operational issues, and develop construction and O&M costs. In order to select a recommended alternative, a comparison was completed between the four alternatives that considers a wide range of selection criteria associated with the cold-water augmentation project. A summary of these evaluation elements is presented in the subsequent sections. As part of the overall evaluation process, it is important to identify any potential "fatal flaws" which could result in a given alternative not meeting the project's goal.

4.3 Criteria Definition

A range of criteria were developed to support the alternatives evaluation. The criteria are grouped as follows:

- ***Potential Public Concern*** is the evaluation criteria used to define the likelihood for the public to express concerns with an alternative. This evaluation criteria looks at public safety, navigability and recreation impact, aesthetics of the alternative, and potential impacts to adjacent property owners.

- ***Biological Efficiency*** which defines the ability to control, in this case reduce, water temperature in Priest River for the benefit of fish. This evaluation criteria considers the seasonal manipulation potential, if the alternative has the potential for year-round manipulation and/or only the potential to lower water temperature during the summer months.

- ***Constructability*** defining the challenges to constructing the alternative features. Special consideration will be given dewatering difficulty. Additional criteria would include staging, space availability, site access, utilities availability, and geotechnical issues.

- ***Environmental Considerations*** quantifying potential impacts to such features as riparian area and wildlife impact. This criteria group should be considered for both the construction period as well as long term impacts following construction. Potential permitting and water rights issues which could impact the implementation feasibility or cost should also be considered within this criteria group.

- ***Operation*** which considers monitoring and automation potential, as well as ease to implement maintenance activities. The ultimate success of the alternative would be one with low operation involvement due to monitoring and automation of the system, and one that would be easily maintained.

- ***Design Approach*** is an evaluation criteria needed to evaluate the potential complexity of the system designed. Preference will be given to an alternative which uses proven technology and will be compatible with proposed Outlet Dam improvements. The criteria will also be used for the sizing of the system, and for the design of the intake screen. This criterion is intended to determine which alternatives are the most successful at developing relatively simple system designs with proven technology.

- ***Cost*** is the final criteria group which encompasses the anticipated project cost including construction, and operation and maintenance costs. The intent of this criteria group is to identify those alternatives that provide the best value considering all cost aspects. Careful consideration of operation and maintenance costs will be required.

4.4 Evaluation

Each alternative is evaluated below using the evaluation criteria defined above. The evaluation follows a qualitative approach, giving assessments of “low”, “medium”, or “high” for each criterion. For some criteria, a ranking of “high” may indicate an advantageous condition (e.g. public safety) where other categories it may represent a disadvantageous condition (e.g. cost). Therefore, each assessment is supplemented with a value judgment indicated by the color, green indicating a favorable condition, red indicating an unfavorable condition, and yellow being intermediate to the two. Note also that the rankings within a given metric are only in relation to each other. Thus, a low public safety ranking does not necessarily signify a public safety hazard. Rather, it only suggests a public safety impact that is greater than the other alternatives with higher rankings. Table 4-1 presents the evaluation matrix.

Table 4-1. Evaluation Matrix

Criteria	Evaluation				
	Alternative 1A Gravity System	Alternative 1B Gravity System	Alternative 2 Siphon System	Alternative 3 Groundwater Well Field	Alternative 4 Pump Station
Potential Public Concerns					
Public Safety	High	High	High	High	High
Navigability and Recreation Impact	Low	Low	Low	Low	Low
Aesthetics	Medium	Medium	Low	Low	High
Potential Impacts to Adjacent Property Owners	Low	Low	Low	Medium	High
Biological Efficiency					
Potential for Lowering Temperature during Summer Months	Medium	High	Medium	Low	Medium
Potential Year-Round Temperature Manipulation	Medium	High	Medium	Low	Medium
Constructability					
Staging	Medium	Medium	Medium	High	Low
Site Access	Low	Low	Low	High	Medium
Utilities Availability	Medium	Medium	High	Low	Low
Dewatering Difficulty	Medium	Medium	Medium	Low	High
Environmental Considerations					
Riparian Impact (Construction/Operation)	High/Low	High/Low	High/Low	Low/Low	Low/Low
Wildlife Impact (Construction/Operation)	Low/Low	Low/Low	Low/Low	Low/Low	Medium/Low
Permitting and Water Rights	Medium	Medium	Medium	High	Low
Operation					
Monitoring	Low	Low	High	Medium	Medium
Automation	High	High	Low	Medium	High
Maintenance	Medium	Medium	Medium	High	Low
Design Approach					
Screening and Intake Velocity	Low	Low	Low	Low	Low
Compatibility with Proposed Outlet Dam Improvements	High	High	High	Medium	Medium
Complexity	Low	Low	Medium	Medium	Low
Cost					
Estimated Capital Cost	Medium	Medium	Medium	Low	High
O&M Cost	Medium	Medium	Low	High	High

4.5 Potential Public Concerns

Public Safety: Each of the five alternatives considered would provide a safe method of cold-water conveyance from Priest Lake to the Lower Priest River. Provided that the alternatives were constructed according to industry-standard construction means and methods, the actual construction of each alternative would not pose any risk to public safety. Because each alternative involves some type of enclosure (e.g. booster pump vault; pump station), adequate signage and site security (e.g. fencing; locked access) would be required to protect each facility. In all cases, no modifications to the Outlet Dam are proposed as part of this study, nor would any impacts to the dam be expected from the construction and operation of any of the alternatives presented here.

Navigability and Recreation Impact: Alternatives 3 and 4 do not suggest any impacts to navigability and recreation. Similarly, because the pipeline is buried in the area of Outlet Bay where water depths are at their lowest, no impacts to navigation and recreation are associated with the pipe alignment under Alternatives 1A, 1B and 2 during regular operations, despite the location of the pipe. During construction, however, Alternatives 1A, 1B and 2 will cause some disruption to normal navigation and recreation in the Outlet Bay region.

Aesthetics: Perhaps the least impactful alternative is Alternative 4 because no part of the pump station system breaks through the horizon when looking out over the lake. Of course, a pump house would be located along Westshore Road, which may impact an otherwise serene and wooded environment. But the pump house is small, with an anticipated footprint no greater than 300 square feet. Second to this alternative is perhaps the gravity systems (Alternative 1A and 1B), which includes a pump vault below the ground surface and does not obstruct the horizon when looking at the Lower Priest River from upstream of the dam. Alternative 3 would have the largest impact on aesthetics, due to the above-ground presence of a pump and water treatment station and several well heads scattered about in the overbank area. All of the system components that are located above ground in these alternatives could include neutral color finishes that help blend the components in with the surrounding environment, helping to mitigate any aesthetic impacts the systems might have.

Potential Impacts to Adjacent Property Owners: Impacts to adjacent property owners could occur during construction or regular operation of the bypass system. During construction, Alternative 4 would likely have the greatest impact to land owners, due simply to the great length of the pipeline that would be laid through town, potentially re-routing traffic, causing noise pollution, and stirring up fugitive dust. Impacts to adjacent landowners during normal operations may come from the noise of pumps and other equipment under Alternatives 3 and 4, although this is unlikely due to the distance to the nearest homes. Other impacts may involve power transmission to the facilities, although power would likely be buried to each site. Maintenance personnel may have to access pumps and equipment under each of these alternatives, although this is expected to be rare under Alternatives 1A and 1B.

4.6 Biological

Potential for Lowering Temperature during Summer Months: Alternatives 1A, 2, and 4 would be able to convey 45 to 71 cfs (and Alternative 1B would be able to convey 45 to 105 cfs) of water from 60 feet below the lake surface to a discharge point on the Lower Priest River during the summer months. Provided that the lake is adequately stratified and a sufficiently cold thermocline is located at 60 feet of depth in the lake, these alternatives should be able to improve the thermal regime of the river during the summer months. Alternative 1B has the potential to provide higher augmentation flows than the rest of the alternatives. This makes Alternative 1B be able to most reliably control the temperature of the outflow even in years were the outflow from the dam is higher than average. The groundwater well alternative, however, may not operate as well. In the absence of groundwater temperature data, it is difficult to determine the extent of river cooling expected from this alternative. Typical groundwater temperatures are in the 45 to 55 °F range (~7 to 13 °C), which bounds the target temperature for this project. However, other issues may hinder this alternative's performance. For example, the stratigraphy underlying the area and the aquifer productivity that would be accessed are not well understood. It might be the case that groundwater in the area is in fact cool enough, but the yield of the aquifer is insufficient or would require so many wells as to be prohibitively expensive.

Potential Year-Round Temperature Manipulation: Alternatives 1A, 2, and 4 would be able to provide 45 to 71 cfs (and Alternative 1B would be able to convey 45 to 105 cfs) of water from 60 feet below the lake surface to discharge points on the Lower Priest River throughout the year. As mentioned above, Alternative 1B provides the most control of water temperature because it can provide the highest augmentation flows. In the case of the siphon system (Alternative 2), the discharge point would be located approximately 1,500 feet downstream due to the lack of hydraulic head across the dam during high flow. In the case of Alternative 1A and 1B, the gravity system would become a pumped system, much like Alternative 4, due to the lack of hydraulic head across the dam. In contrast, Alternative 3 may not be able to provide cold-water throughout the year, due to the reasons mentioned above related to the summer months.

4.7 Constructability

Staging: The ease with which construction materials and equipment are staged will depend on the amount of material and equipment and the speed at which construction progresses. The majority of staged material will consist of piping, regardless of whether it is intended for Alternatives 1A, 1B, 2, or 4, or whether it will serve as well casing under Alternative 3. The lengths of the pipeline alignment are assumed to be much greater than the combined length of well casings, however, such that the overall staging area required for the groundwater well system is probably less than the other alternatives. The total pipeline lengths for the four other alternatives range from about 8,000 feet up to about 9,750 feet. Assuming 20-foot long pipe sticks, this equates to between 400 and 490 pipe sticks. For alternatives 1A, 1B and 2, where partial staging will take place on a barge, auxiliary staging must be provided along the shoreline, perhaps adjacent to the dam. And for Alternative 4, extra staging will be required for pipe bedding material and excavated material from the pipe trench.

Site Access: Vehicular site access to each of the equipment structures will be straightforward and unobstructed. Access to the booster pump vault for the gravity system will require entering a confined space. Access to the intakes will be more or less equally difficult for alternatives 1A, 1B, 2, and 4, as each is located 60 feet below the surface of the lake. Maintenance work on the pipelines would be comparatively easier under Alternative 4, simply because the pipeline is not submerged. However, the fact that the pipeline is buried mitigates this advantage. Overall, access to Alternative 3 is the simplest.

Utilities Availability: Alternatives 2 and 4 would both require access to 3-phase power, due to the large power demand. If the gravity supply systems are to provide up to 71 and 105 cfs year round for Alternative 1A and 1B, respectively, then 3-phase power would be required for those alternatives as well, again due to the large electric power demand. Only the siphon system would not require 3-phase power, because the vacuum pump can be relatively small and serviced with a single-phase power feed. In all cases, power can be delivered from nearby transmission lines. Three-phase power appears to be about ¼ mile away in all cases, while single phase power is even closer.

Dewatering Difficulty: The groundwater system (Alternative 3) would require very little dewatering, located in the immediate vicinity of the discharge into the Lower Priest River. The other four alternatives have about the same level of difficulty in dewatering, except that Alternatives 1A, 1B and 2 may be able to take advantage of concurrent work being done on the Outlet Dam, as proposed by the Water Management Study.

4.8 Environmental

Riparian Impact: During normal operations, riparian impacts from each of the alternatives are expected to be low. During construction, however, some impacts to the riparian environment can be expected from Alternatives 1A, 1B and 2, due to the in-water work through the Outlet Bay. Under these alternatives, some level of re-suspension of settled fines can be expected. This will have localized impacts on the riparian environment around the pipeline during construction.

Wildlife Impact: During normal operations, impacts to wildlife from each of the alternatives are expected to be low. During construction, however, some impacts to wildlife can be expected, particularly under Alternative 4. For example, noise pollution from construction may scare off upland animals and birds. Also, construction within wooded areas may break up continuity in migration corridors. Overall, however, impacts to wildlife are expected to be minor.

Permitting and Water Rights: Alternative 3 is expected to be difficult in terms of permitting and water rights due to the large groundwater right needed. Additional costs may be associated with this alternative if water rights need to be leased from a water bank. All alternatives may require consultation with federal agencies, such as the U.S Army Corps of Engineers and U.S. Fish and Wildlife Service. If any federal permits or work on federal land are required, the National Environmental Policy Act may be triggered.

4.9 Operation

Monitoring: Some level of monitoring will be required for the gravity systems, especially at times of high flow when the booster pump needs to switch on. However, due to the larger flow capacity of Alternative 1B, the booster pump should not need turned on during high flows as much as it would using Alternative 1A. During the recreational period, however, the gravity systems require very little monitoring. A flow meter or pressure transducer in the booster pump vault would allow monitoring of pressure drop in the system, which would help personnel identify debris accumulation at the intake, signaling the need for divers to conduct annual maintenance. Alternatives 3 and 4 would require more frequent monitoring, due to the fact that pumps would be running throughout the entirety of the recreational period. Alternative 4 would also require instrumentation to indicate debris accumulation, much like Alternative 1A and 1B. Alternative 2 will require the highest degree of monitoring, because siphon systems are notoriously problematic and can lose their prime quite easily if air is entrained. Additionally, instrumentation to indicate debris accumulation would be required for this alternative.

Automation: Were the gravity systems to operate only during the recreational period, automation would be nearly absolute. The only possible interruption of operation would be debris accumulation, which takes place over time and would be monitored. During high flow, the Alternative 1A would operate like the pump station system, with manual start of the pumps at a single duty point; however, Alternative 1B would still be capable of operating without the pump. The groundwater well field would require automation to adjust the variable frequency drives on the pumps to withdraw the correct amount of water over time. The pumping rates may change throughout the year depending on the level of the water table and the conductance experienced by each pump. Unlike the other four alternatives, the siphon system would require a near-instantaneous response to impacts in the system because a loss of prime happens very quickly. Automation would allow the vacuum pump to start once a loss of vacuum pressure is observed. If the pump is sized properly, it will be able to evacuate air from the pipe rather quickly and restore the siphon without a major break in supply continuity.

Maintenance: As noted above, the fish tee screens under Alternatives 1A, 1B, 2, and 4 would be passive, with no automated screen cleaning. Screen cleaning would be part of scheduled maintenance and would involve divers to clear the screens on an annual or semi-annual basis. Pressure transducers on the outlet pipe would indicate the accumulation of debris on the screen, which would allow active, remote monitoring of the screen. Alternative 3 would not require screen cleaning because there is no fish screen included as part of the groundwater system. However, due to the sheer number of pump and variety of equipment proposed for Alternative 3, maintenance for this alternative is expected to be more frequent and more varied.

4.10 Design Approach

Screening and Intake Velocity: All of the alternatives scored equally well in terms of screening and intake velocity because they all meet the NMFS fish passage design criteria.

Compatibility with Proposed Outlet Dam Improvements: Alternative 1A, 1B, 2 and 4 could all work in conjunction with the alternatives recommended for Priest Lake as part of the *Priest Lake Water Management Study* (Mott MacDonald 2018). The alternatives proposed in that study include raising the pool level by 6 inches during the months of July and August and providing improvements to both the outlet dam and thoroughfare to achieve this raise in water levels. A higher water surface elevation during the summer months, when the cold-water augmentation is in operation, will improve these three alternatives by increasing the hydraulic head across the dam in the summer months, providing factors of safety and reducing any pumping costs associated with the alternatives. Alternatives 1A, 1B and 2 would benefit further due to the possibility of capital savings in the case that both the cold-water bypass and the proposed Outlet Dam modifications were implemented at the same time, since dewatering would be needed near the dam in both cases. Acknowledging that Alternative 3 would not conflict with the study alternatives, it would not benefit from them.

Complexity: The gravity supply systems are the simplest systems, particularly in the event that no booster pump is needed, and cold-water augmentation is restricted to the warm summer months only. Even if water is needed throughout the year, however, these systems matches that of the pump station alternative in terms of complexity. The siphon system and groundwater well fields, on the other hand, are fairly complex systems. The siphon system requires a specialized priming valve and vacuum pump, and control and automation may be more difficult in the case of losing system prime. In addition, the flow split downstream of the pump house will require added automation or manual control, adding complexity to the operations and maintenance program. The groundwater well system performance must monitor the productivity of several wells at once, requiring one or multiple flow meters and possibly variable speed pumps on each of the wells. Furthermore, water quality restrictions may require a complex manifold that feeds degassing towers, the discharge from which is then pumped back to the river. All of these components require added expertise for personnel operating and maintaining this system.

4.11 Cost

A Technical Memorandum presenting the conceptual cost estimate for each alternative was prepared and is located in Appendix D. The TM presents cost factors, capital cost estimate, present value of operation and maintenance cost, a life cycle cost analysis, and a detailed cost estimate for each alternative. The cost estimate is a Class 5 cost estimate per the American Association of Cost Engineering (AACE). A Class 5 is appropriate for this evaluation and has an expected accuracy range of +100% to -50%.

4.11.1 Capital Cost Estimate

Capital costs include any upfront costs required to construct and fully commission an alternative. Major costs will include piping/casing materials, pumps and well heads, valves, controls, electrical supply, equipment structures, excavation and dewatering, mobilization, installation, specialty labor, skilled labor, and raw materials for bedding and aprons.

Alternative 3 is the most affordable, followed by Alternative 4, Alternative 1A, Alternative 1B, and Alternative 2 (listed in order from most to least affordable). Alternative 3 is substantially less expensive in terms of construction cost. This is due to the fact that this alternative is the only alternative that does not require a large diameter pipe extending out into the lake or running along the road for a significant distance. Capital costs associated with Alternative 3 include specialized drill rigs to drill and drive well casings, several submersible pumps to form a well field, and a piping manifold inside a well field control house to discharge water to the river. Optionally, this alternative may require degassing towers to ensure that CO_2 concentrations are acceptable in the discharge water. The slight increase in price from Alternative 1A to Alternative 1B is mainly attributed to the larger pipe, larger pump and the additional tee screen at the intake.

Table 4-2 provides a summary of the conceptual level costs in 2019 dollars for the Priest River cold-water augmentation alternatives. From the beginning of 2019 to the mid-point of construction, a 3% annual inflation factor should be added to the concept costs below. Note, inflation was not added for this conceptual cost estimate. The capital cost estimate includes additional cost factors, such as, general contract requirements, overhead, profit, bond rate, state sales tax, and design, permitting, and construction support.

Table 4-2. Concept Design Construction Cost Estimates

Line Item	Alternative 1A Gravity System	Alternative 1B Gravity System	Alternative 2 Siphon System	Alternative 3 Groundwater Well System	Alternative 4 Pump Station
Total Conceptual Cost - 2019 Dollars	**$8,276,000**	**$8,601,000**	**$8,715,000**	**$684,000**	**$8,063,000**
+100%	$16,552,000	$17,492,000	$17,430,000	$1,368,000	$16,126,650
-50%	$4,138,000	$4,373,000	$4,358,000	$342,000	$4,032,000

4.11.2 Present Value of Operation and Maintenance

Annual operation and maintenance cost estimates were prepared for the summer operation and for the year-round operation, and they are shown in Table 4-3 and Table 4-4, respectively.

Table 4-3. Summer Operations and Maintenance Costs (Annual)

Line Item	Alternative 1A Gravity System	Alternative 1B Gravity System	Alternative 2 Siphon System	Alternative 3 Groundwater Well System	Alternative 4 Pump Station
Operations[1]	$0	$0	$34	$148,186	$143,740
Maintenance	$2,000	$3,000	$3,000	$8,000	$3,500
Total Annual O&M Cost	$2,000	$3,000	$3,034	$156,186	$147,240

[1] Assumes continuous operation from July 1 through September 30, except for Alternative 2, which assumes 1 hour of operation per day over the same period.

Table 4-4. Year-Round Operations and Maintenance Costs (Annual)

Line Item	Alternative 1A Gravity System	Alternative 1B Gravity System	Alternative 2 Siphon System	Alternative 3 Groundwater Well System	Alternative 4 Pump Station
Operations[1]	$24,918	$39,087	$136	$587,910	$570,273
Maintenance	$2,000	$3,000	$3,000	$8,000	$3,500
Total Annual O&M Cost	$26,918	$42,087	$3,136	$595,910	$573,773

[1] Assumes continuous operation year-round, except for Alternative 2, which assumes 1 hour of operation per day over the same period.

4.11.3 Life Cycle Cost Estimate

In order to correctly compare the costs associated with each alternative, a life cycle cost analysis must be conducted. Life cycle cost analysis requires defining a project useful life, during which operational and maintenance costs are incurred. The useful life of the project was assumed to be 20 years, after which point any pumps and/or mechanical equipment would need replacement. In addition, only the summer operation and maintenance costs were utilized. For this analysis, the salvage value of each alternative was assumed to be zero ($0). The analysis was also conducted assuming a discount rate obtained from the U.S. Office of Management and Budget of 2.875 percent. Table 4-5 provides the results of the life cycle cost analysis. From the table, even though the capital cost of Alternative 3 is appreciably less than the other three alternatives, it nevertheless does not result in the lowest life cycle cost, due to the high cost of pumping. Instead, Alternative 1A results in the lowest life cycle cost. This is the case regardless of whether the system is operated during the summer only or year-round.

Table 4-5. Life Cycle Cost Analysis

Line Item	Alternative 1A Gravity System	Alternative 1B Gravity System	Alternative 2 Siphon System	Alternative 3 Groundwater Well System	Alternative 4 Pump Station
Construction Cost	**$8,276,000**	**$8,601,000**	**$8,715,000**	**$684,000**	**$8,063,000**
Summer O&M Costs, Present Value	$30,355	$45,355	$46,053	$2,370,516	$2,234,744
Life Cycle Cost	**$8,306,355**	**$8,646,355**	**$8,761,053**	**$3,054,516**	**$10,297,744**

4.12 Advantages and Disadvantages

Table 4-6 lists the advantages and disadvantages associated with each alternative.

Table 4-6. Alternatives Advantages and Disadvantages

Alternative	Advantages	Disadvantages
Alternative 1A – Gravity System	• Reliable supply of cold-water during recreational period • Simple system • Ease of operation • Low O&M cost • Comparatively low capital cost • Low visual obstruction • Benefits from proposed dam and lake level modification project	• Challenging access during construction • May require 3-phase power
Alternative 1B – Gravity System	• Reliable supply of cold-water during recreational period • Simple system • Ease of operation • Low O&M cost • Comparatively low capital cost • Low visual obstruction • Benefits from proposed dam and lake level modification project	• Challenging access during construction • May require 3-phase power

Alternative	Advantages	Disadvantages
	• Provides a maximum of 105 cfs of augmentation flow (34 cfs greater than the other alternatives)	
Alternative 2 – Siphon System	• Reliable supply of cold water during recreational period • Low O&M cost • Does not require 3-phase power • Benefits from proposed dam and lake level modification project	• Complex system • Comparatively high capital cost • Requires expertise or special training to operate • Visual obstruction • Challenging access during construction
Alternative 3 – Groundwater Well System	• Does not impact navigability • Ease of constructability • No intake needed • Low impacts to fish and wildlife during construction • Low capital cost	• Feasibility unknown due to lack of hydrogeologic data • Water rights may be difficult to obtain • Water quality needs are unknown (especially temperature) • Requires 3-phase power • High O&M costs
Alternative 4 – Pump Station	• Does not impact navigability • Reliable supply of cold water during recreational period • Simple system • Ease of operation	• Public nuisance during construction • Requires 3-phase power • High O&M costs • Very high capital costs • Difficulty staging and dewatering • May impact wildlife during construction

4.13 Discussion

It should be noted that all the alternatives were evaluated in terms of their abilities to provide between 45 cfs and 105 cfs throughout the entire year, per the design criteria established for the project. However, were the design criteria restricted to the recreational period only, the results of the evaluation would be different, suggesting the gravity systems are superior to Alternatives 2 and 4. The reason for this is related to the hydraulic head across the dam in the recreational period. Because Outlet Dam operators are required to maintain a lake level of 3.0 feet (gage), a hydraulic drop of 6 to 7 feet can be expected across the dam from July to October. This drop means that a gravity system will work just fine without any assistance from a booster pump during those months. If a booster pump is not needed, there could be significant capital savings for these alternatives. For example, a simple valve box would be needed, rather than a larger booster pump vault with personnel access, and 3-phase power would no longer be needed to the vault. Also, operational costs would effectively approach zero, because there would no longer be a pump to turn on during high flow.

A similar effect would be seen with the siphon system, were the design criteria restricted to the recreational period only. In that case, there would no longer be the need for a flow split to discharge water 1,500 feet downstream of the Outlet Dam during high flow. Recall that the need for the flow split stems from the fact that, at high flow when the radial gates are completely open, there is essentially no hydraulic head across the dam that can serve to overcome the head losses in the siphon system. For that reason, the discharge point must be located sufficiently downstream as to recapture an adequate hydraulic head to maintain the siphon. By only operating during the recreational period, when the hydraulic drop across the dam is guaranteed, the siphon can be maintained and discharged just below the Outlet Dam. This would impact the capital cost of the siphon system, because the flow split valving, extra piping, and extra discharge location would no longer be needed. In addition, cold-water would be discharged at the uppermost extent of the river, rather than downstream (even though the benefits of cold-water supply during high flow are questionable due to the low mixing ratio, regardless of where that cold-water is discharged).

Comparing the two gravity systems, there are additional costs associated with Alternative 1B; however, it can provide an additional 34 cfs of flow, making it more capable of controlling the temperature of Priest River downstream, even during high flows.

5.0 Conclusions and Recommendations

5.1 Conclusions

This report has documented the Priest River Cold-Water Augmentation Alternatives Analysis. Recent stream temperature modeling, contracted by the Kalispel Tribe of Indians and conducted by Portland State University, has demonstrated the feasibility of reducing late summer temperatures in the Priest River to target levels for cold-water aquatic biota by introducing 45 to 71 cfs from the colder stratum (hypolimnion; 8°C) of the nearby Priest Lake. Avista retained McMillen Jacobs to evaluate alternatives to supply the upper reach of the Priest River with this cold-water supply. A series of eleven (11) alternatives were developed and screened according to their potential to meet the design criteria, their relative advantages and disadvantages, and a qualitative assessment of their costs, both capital and O&M. This screening process resulted in four (4) alternatives that were subsequently advanced to a further stage of evaluation: (1) a gravity pipeline system with booster pump contingency, (2) a siphon system, (3) a groundwater well system, and (4) a pump station.

For these four alternatives, conceptual-level designs were developed, as well as life-cycle cost estimates, for comparison and evaluation under the following criteria: public concern, biological efficiency, constructability, environmental impact, operation, design approach, and cost. After the development of these four alternatives, an additional gravity system was evaluated that could provide a higher maximum augmentation flow (105 cfs). This became Alternative 1B, and the original (71 cfs capacity) gravity system became Alternative 1A. Table 4-6 summarizes the relative advantages and disadvantages of each of these alternatives.

5.2 Recommendations

Three of the five alternatives recommend themselves for further investigation: Alternative 1A – Gravity System, Alternative 1B – Gravity System, and Alternative 3 – Groundwater System.

Alternative 1A has the second lowest life cycle cost and Alternative 1B has the third lowest life cycle cost. They both have very low O&M costs associated with them. Should the operational strategy include recreation period only cold-water augmentation, the O&M costs for Alternative 1A and 1B would be reduced even further.

Alternative 3 has the lowest overall life cycle cost, with a very low capital cost. However, there is substantial uncertainty associated with this alternative, including:

- Uncertain cost of power over project life, which factors largely in life cycle cost due to large operational cost component;
- Uncertainty related to groundwater availability;
- Uncertainty related to groundwater temperatures;
- Uncertainty related to coordination of lake water releases and amount of water needed from groundwater sources to lower in-stream temperatures (i.e. minimum augmentation flow of 45 cfs may not be adequate under this alternative);

For these reasons, a higher-level project definition, including limited hydrogeological investigation and a Class 4 cost estimate, is recommended for this alternative to better address these uncertainties.

Further investigation into Alternatives 1A, 1B, and 3 will provide a better understanding of project feasibility and an improved basis of comparison for the three alternatives.

6.0 References

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Piper, Robert G., McElwain, Ivan B., Orme, Leo E., Fowler, Laurie G., and Leonard, John R., 1982. *Fish Hatchery Management*, U.S. Department of the Interior, Fish and Wildlife Service.

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Appendix A

Conceptual Drawings

ALT 1A - GRAVITY SYSTEM SITE LAYOUT

SCALE: 1:700

NOTES:

1. SEE FIGURE A1.2 FOR ALIGNMENT PROFILE.

McMILLEN JACOBS ASSOCIATES

ALTERNATIVE 1A
GRAVITY SUPPLY SYSTEM

SITE LAYOUT

AVISTA CORPORATION
SPOKANE, WASHINGTON

1" = 1000' SCALE	DATE	APPROVED	
DESIGN JW	CHECKED		
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DWG NO: FIGURE A1.1

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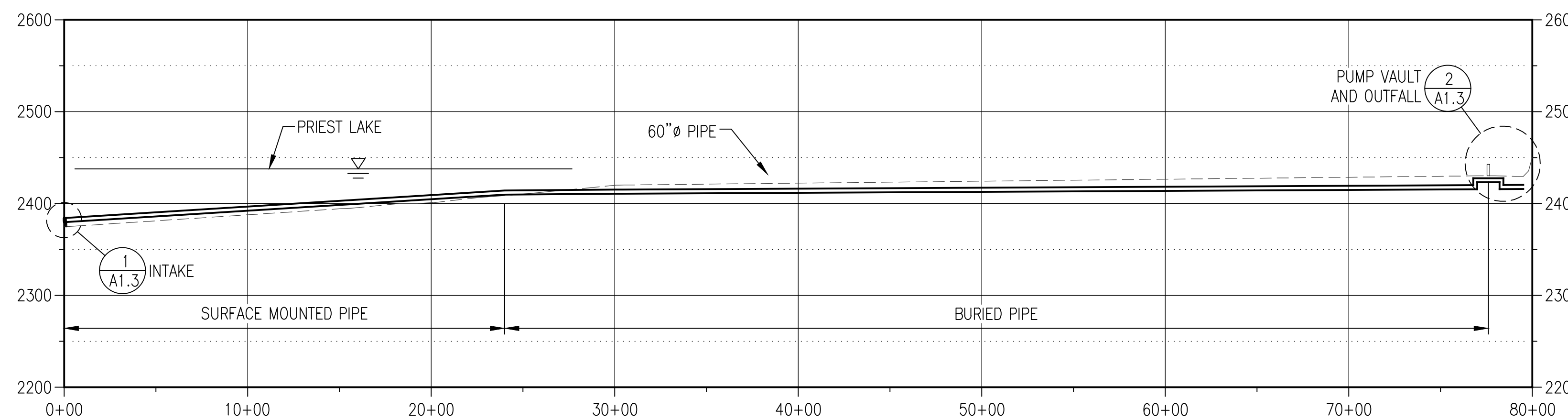
FIGURE A1.2



ALT 1A - GRAVITY BYPASS PROFILE STA 0+00 -80+00

SCALE: 1:500H 1:100V

McMILLEN JACOBS ASSOCIATES

ALTERNATIVE 1A
GRAVITY SUPPLY SYSTEM

PROFILE

AVISTA CORPORATION
SPOKANE, WASHINGTON

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APPROVED
DESIGN JW CHECKED
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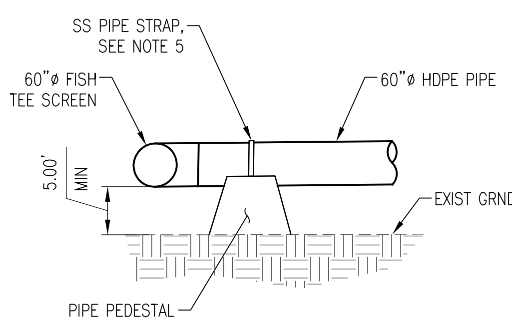
FIGURE A1.3



NOTES:
1. FREE SPAN BETWEEN PEDESTALS NOT TO EXCEED 20-FT.
2. MAX APPROACH VELOCITY 0.5 FT/S.
3. MIN SCREEN AREA 157 SF.
4. SCREEN MESH TO MEET NOAA FISH PASSAGE CRITERIA.
5. SS PIPE STRAPS TO BE ANCHORED TO PEDESTALS WITH SS ANCHOR BOLTS, LOCKWASHERS AND NUTS.

1 / A1.1 DETAIL - INTAKE
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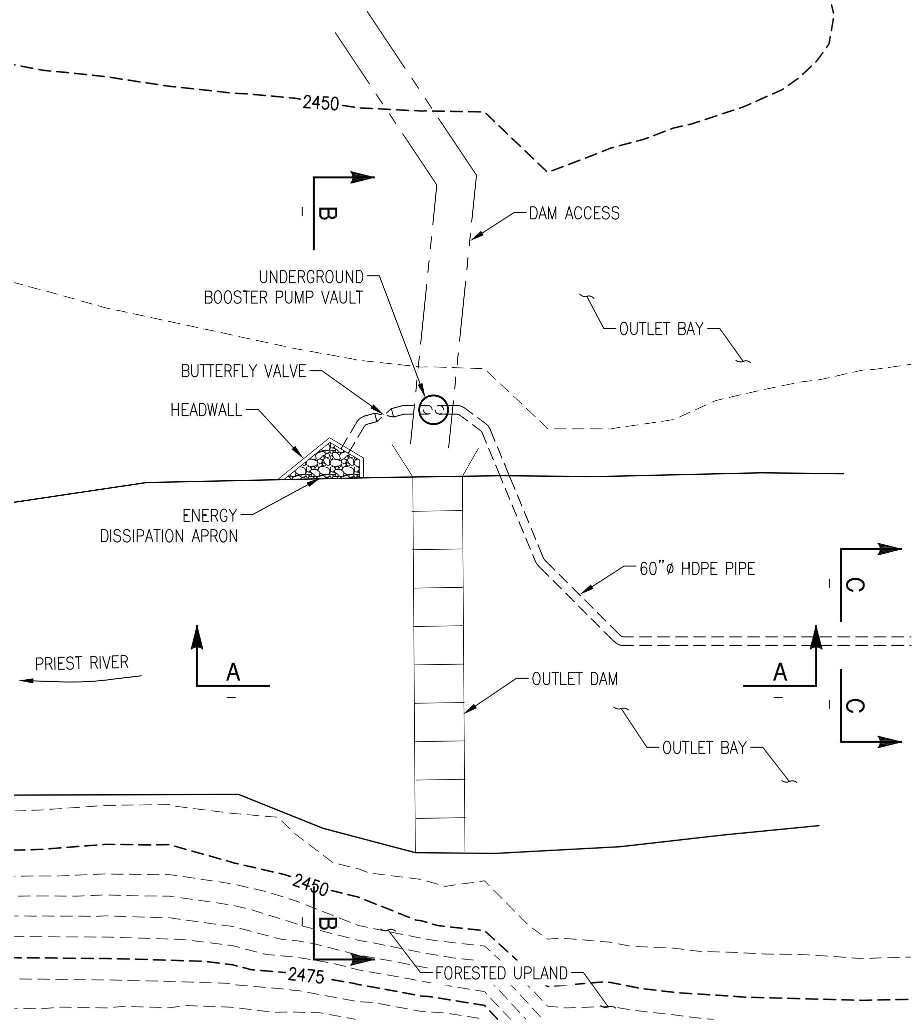
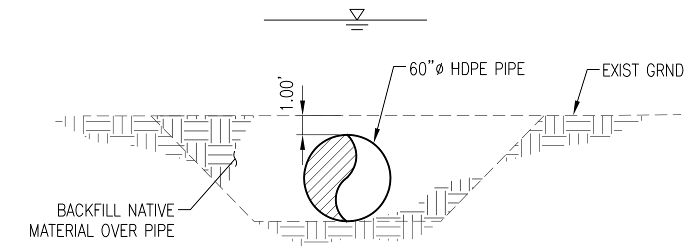


2 / A1.1 DETAIL - PUMP VAULT AND OUTFALL PLAN
SCALE: 1:10



C / A1.3 SECTION
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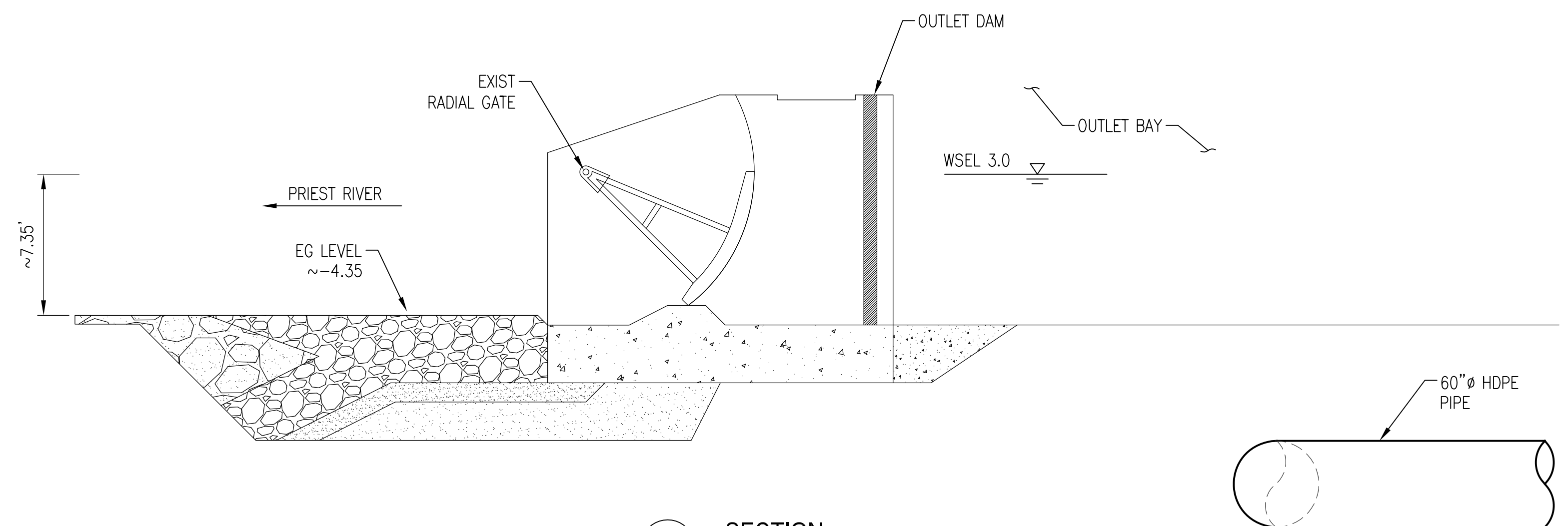
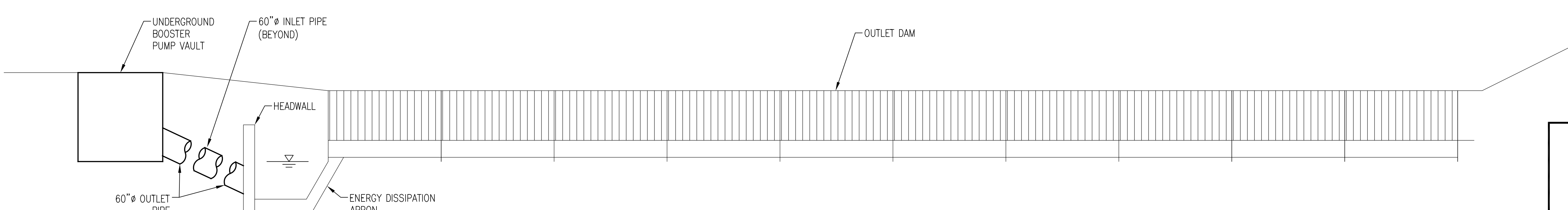


A / A1.3 SECTION
SCALE: 1:5



B / A1.3 SECTION
SCALE: 1:10

McMILLEN JACOBS ASSOCIATES

ALTERNATIVE 1A
GRAVITY SUPPLY SYSTEM

INTAKE, PUMP VAULT, AND OUTFALL
PLAN, SECTIONS, AND DETAILS

AVISTA CORPORATION
SPOKANE, WASHINGTON

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ALT 1B - GRAVITY SYSTEM SITE LAYOUT

SCALE: 1:700

NOTES:

1. SEE FIGURE A2.2 FOR ALIGNMENT PROFILE.

McMILLEN JACOBS ASSOCIATES

ALTERNATIVE 1B
GRAVITY SUPPLY SYSTEM

SITE LAYOUT

AVISTA CORPORATION
SPOKANE, WASHINGTON

1" = 1000'			APPROVED	
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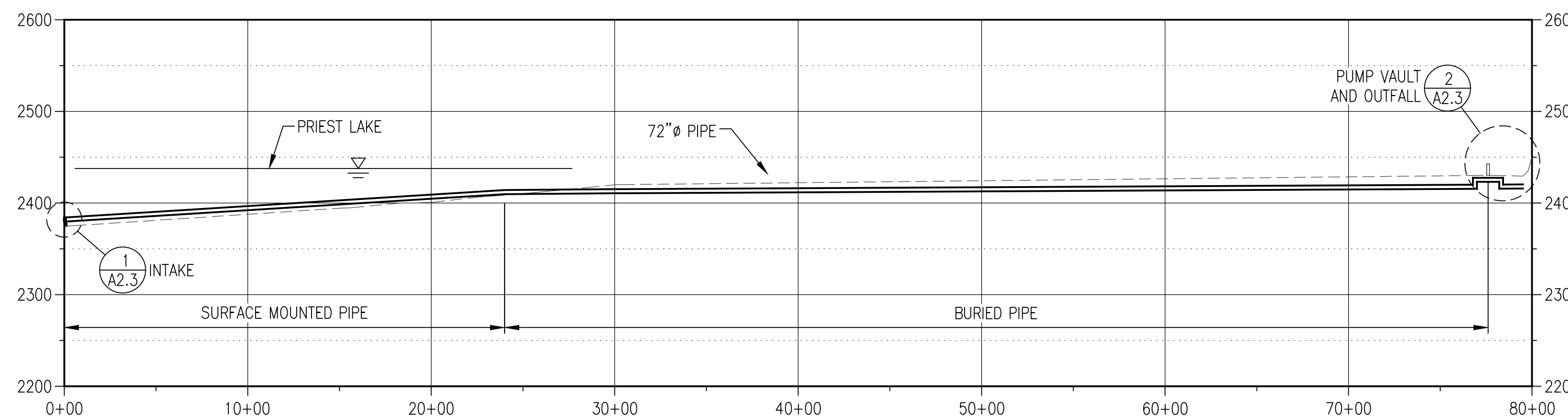

ALT 1B - GRAVITY BYPASS PROFILE STA 0+00 -80+00

SCALE: 1:500H 1:100V

McMILLEN JACOBS ASSOCIATES

ALTERNATIVE 1B
GRAVITY SUPPLY SYSTEM

PROFILE

AVISTA CORPORATION
SPOKANE, WASHINGTON

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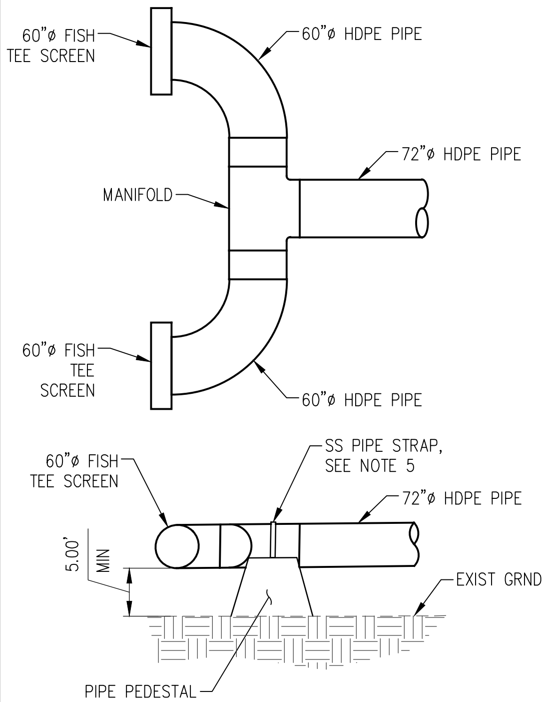

NOTES:
1. FREE SPAN BETWEEN PEDESTALS NOT TO EXCEED 20-FT.
2. MAX APPROACH VELOCITY 0.5 FT/S.
3. MIN SCREEN AREA 157 SF PER SCREEN.
4. SCREEN MESH TO MEET NOAA FISH PASSAGE CRITERIA.
5. SS PIPE STRAPS TO BE ANCHORED TO PEDESTALS WITH SS ANCHOR BOLTS, LOCKWASHERS AND NUTS.

1 A2.1 DETAIL - INTAKE
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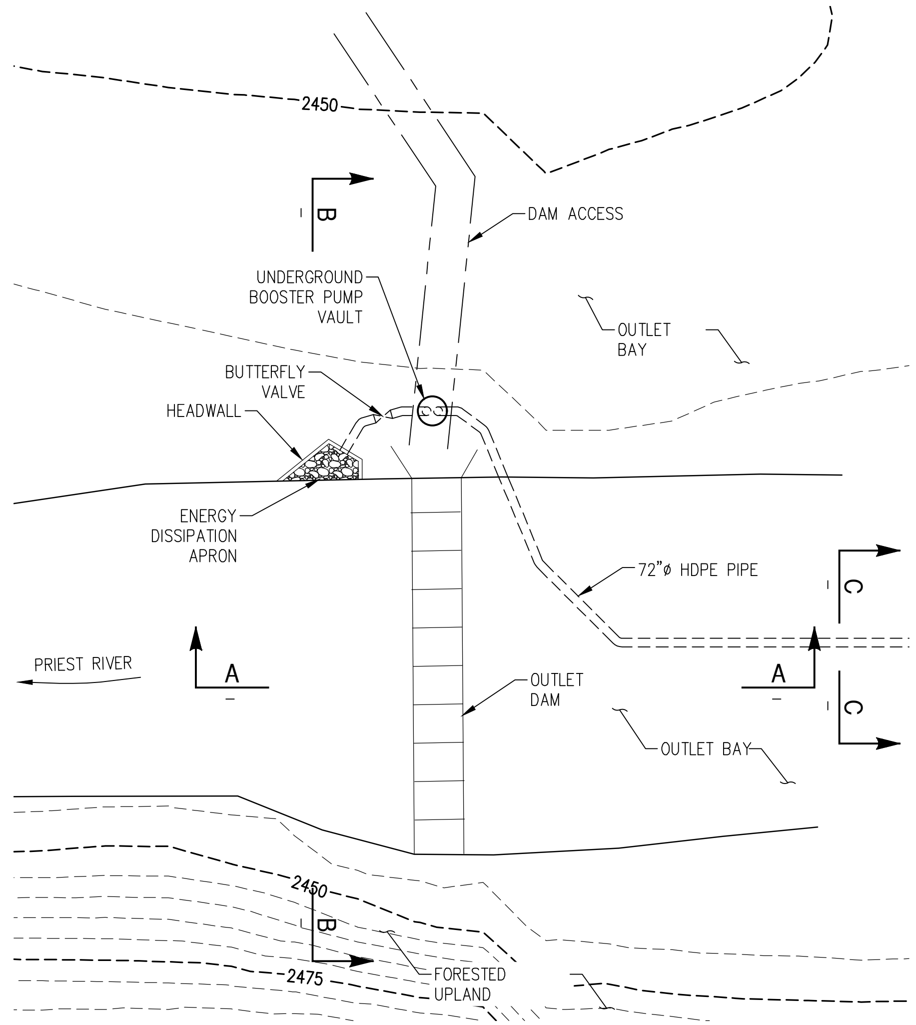
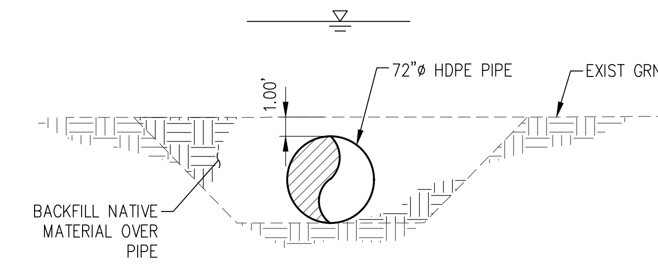


2 A2.1 DETAIL - PUMP VAULT AND OUTFALL PLAN
SCALE: 1:10



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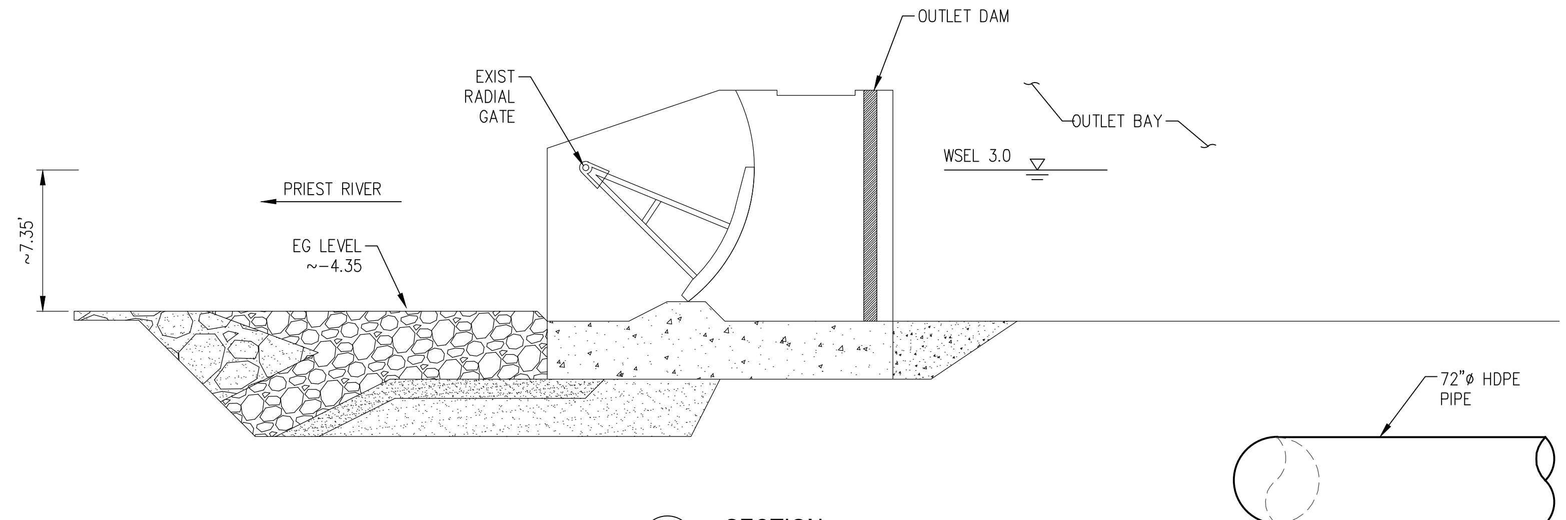
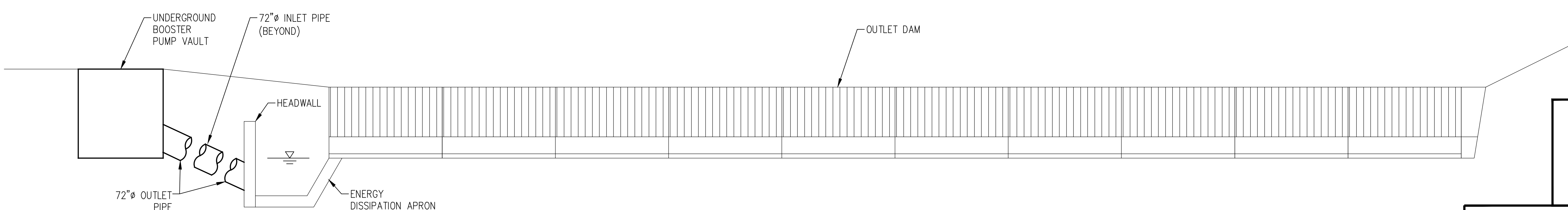


A A2.3 SECTION
SCALE: 1:5



B A2.3 SECTION
SCALE: 1:10

McMILLEN JACOBS ASSOCIATES

ALTERNATIVE 1B
GRAVITY SUPPLY SYSTEM

INTAKE, PUMP VAULT, AND OUTFALL
PLAN, SECTIONS, AND DETAILS

AVISTA CORPORATION
SPOKANE, WASHINGTON

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GEN AUTOCAD DWG

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FIGURE A2.1

ALT 2 - SIPHON SYSTEM SITE LAYOUT
SCALE: 1:700

NOTES:

1. SEE FIGURE A3.2 FOR ALIGNMENT PROFILE.

McMILLEN JACOBS ASSOCIATES

ALTERNATIVE 2
SIPHON SYSTEM

SITE LAYOUT

AVISTA CORPORATION
SPOKANE, WASHINGTON

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1" = 1000' SCALE	DATE	APPROVED
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DWG NO: FIGURE A3.1

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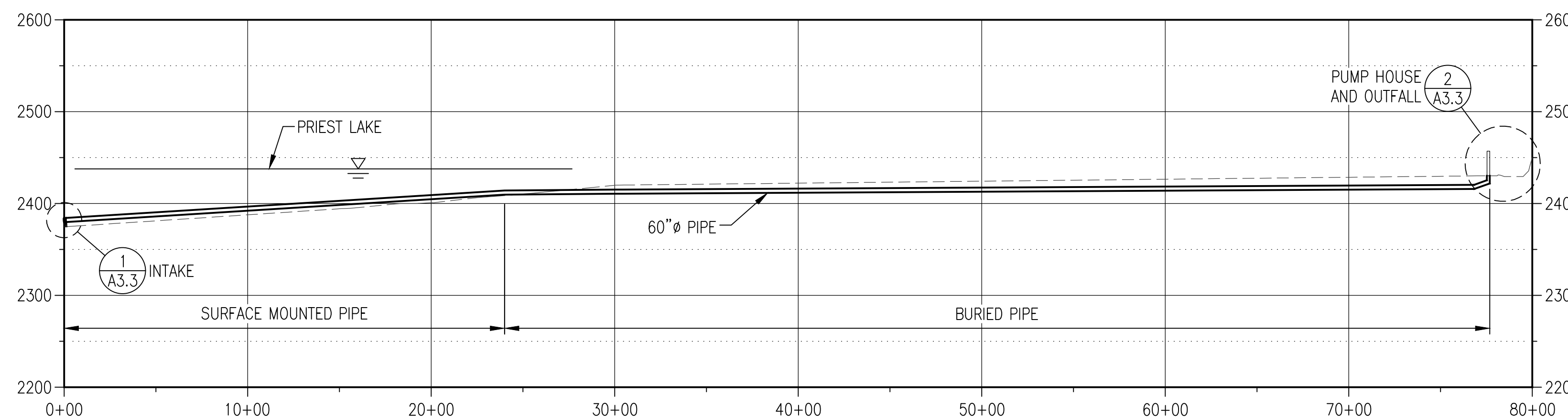

ALT 2 - SIPHON PROFILE STA 0+00 - 80+00

SCALE: 1:500H 1:100V

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0					

McMILLEN JACOBS ASSOCIATES

ALTERNATIVE 2
SIPHON SYSTEM

PROFILE

AVISTA CORPORATION
SPOKANE, WASHINGTON

AS NOTED — SCALE; DATE; APPROVED

DESIGN JW CHECKED
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CHECKED VA NOTED

DATE

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DWG NO: FIGURE A3.2

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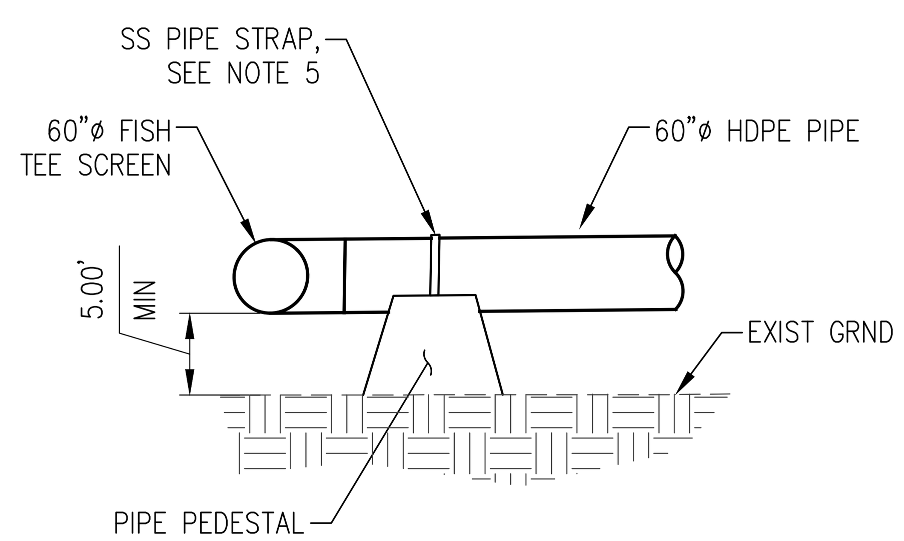
FIGURE A2.2



NOTES:
1. FREE SPAN BETWEEN PEDESTALS NOT TO EXCEED 20-FT.
2. MAX APPROACH VELOCITY 0.5 FT/S.
3. MIN SCREEN AREA 157 SF.
4. SCREEN MESH TO MEET NOAA FISH PASSAGE CRITERIA.
5. SS PIPE STRAPS TO BE ANCHORED TO PEDESTALS WITH SS ANCHOR BOLTS, LOCKWASHERS AND NUTS.

1 / A3.1 DETAIL - INTAKE
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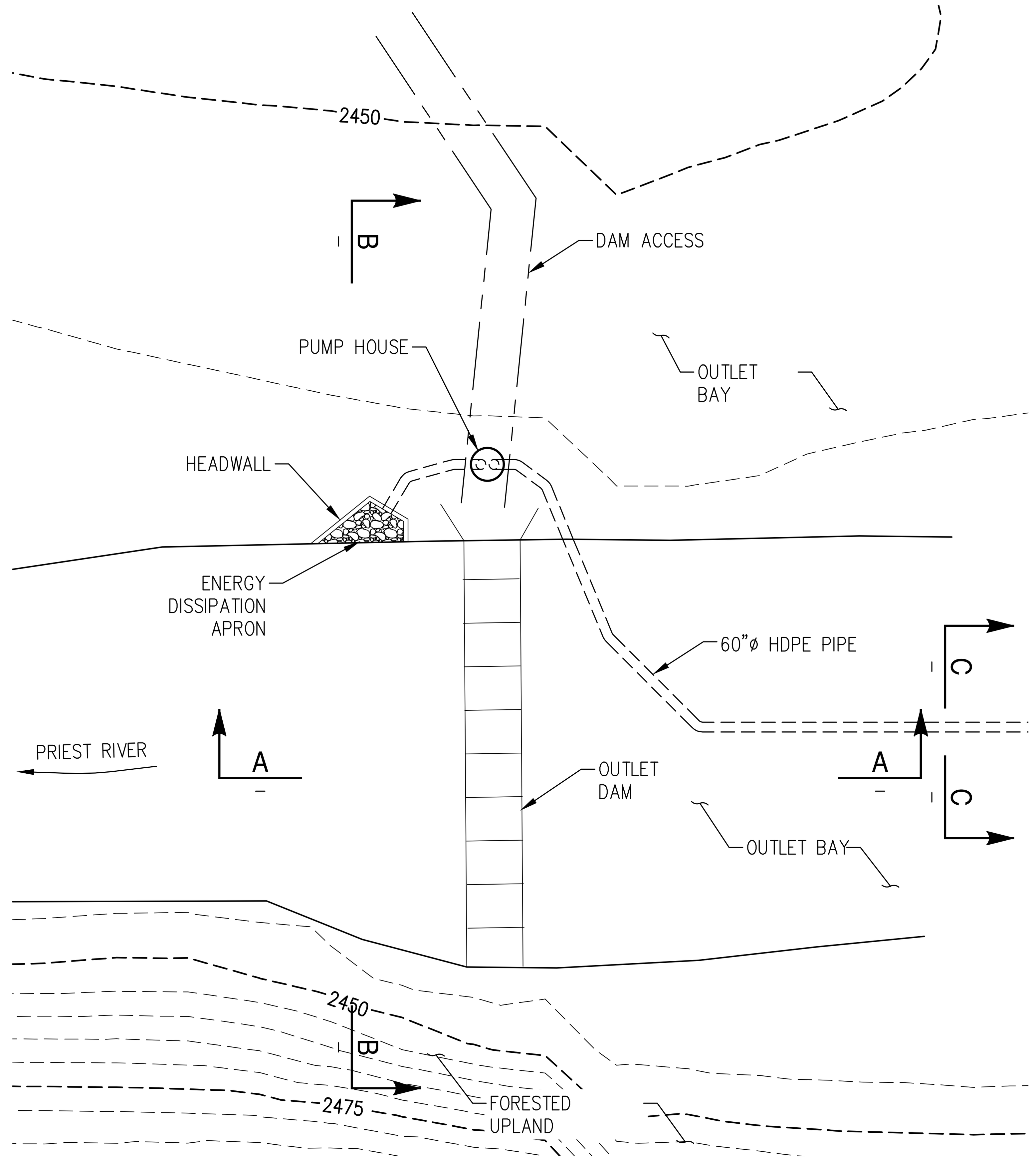

2 / A3.1 DETAIL - PUMP VAULT AND OUTFALL PLAN
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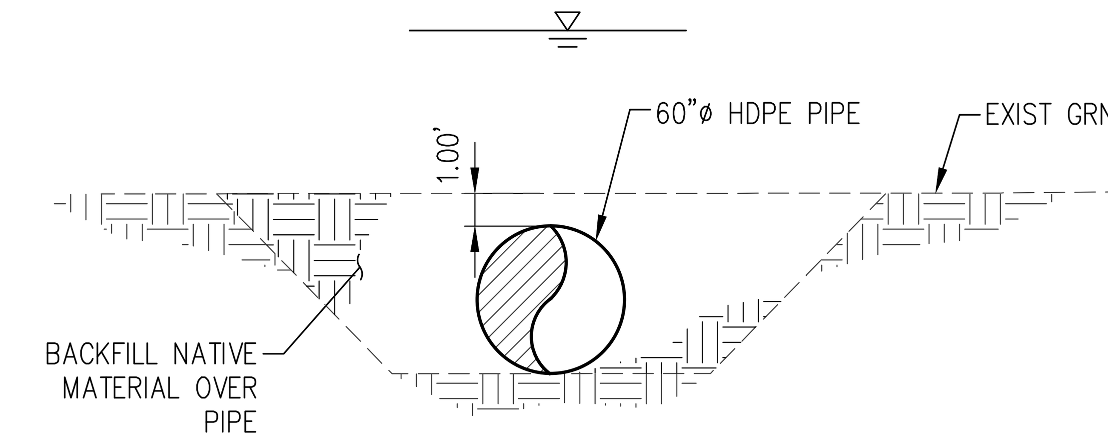

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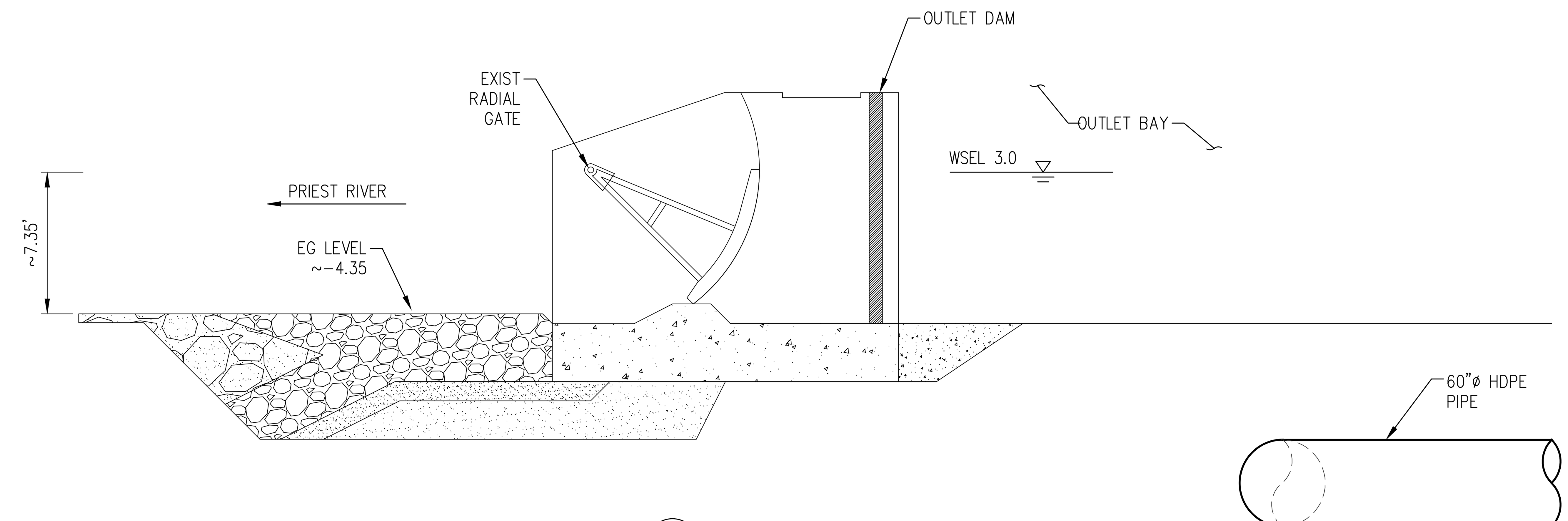

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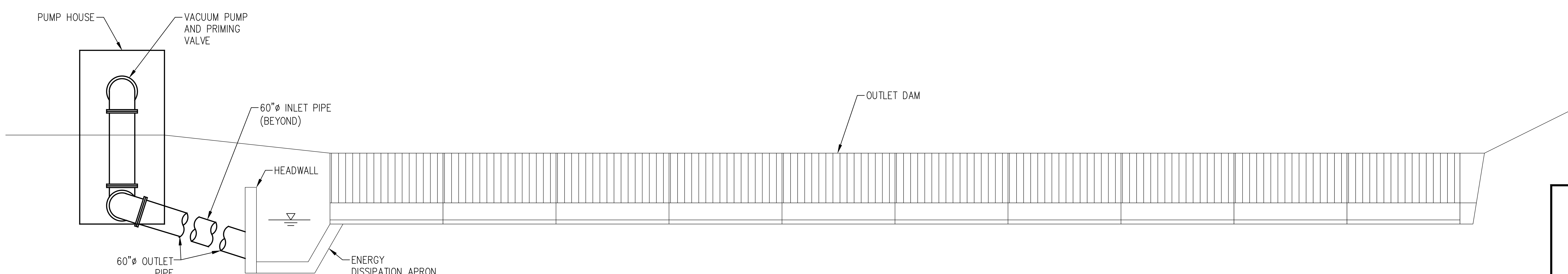

B / A3.3 SECTION
SCALE: 1:10

McMILLEN JACOBS ASSOCIATES

ALTERNATIVE 2
SIPHON SYSTEM

PUMP HOUSE AND OUTFALL
PLAN AND SECTIONS

AVISTA CORPORATION
SPOKANE, WASHINGTON

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DWG NO: FIGURE A3.3

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ALT 3 - GROUNDWATER WELL SYSTEM SITE LAYOUT
SCALE: 1:700

McMILLEN JACOBS ASSOCIATES

ALTERNATIVE 3
GROUNDWATER WELL SYSTEM

SITE LAYOUT

AVISTA CORPORATION
SPOKANE, WASHINGTON

1" = 1000'		APPROVED
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DWG NO: FIGURE A4.1

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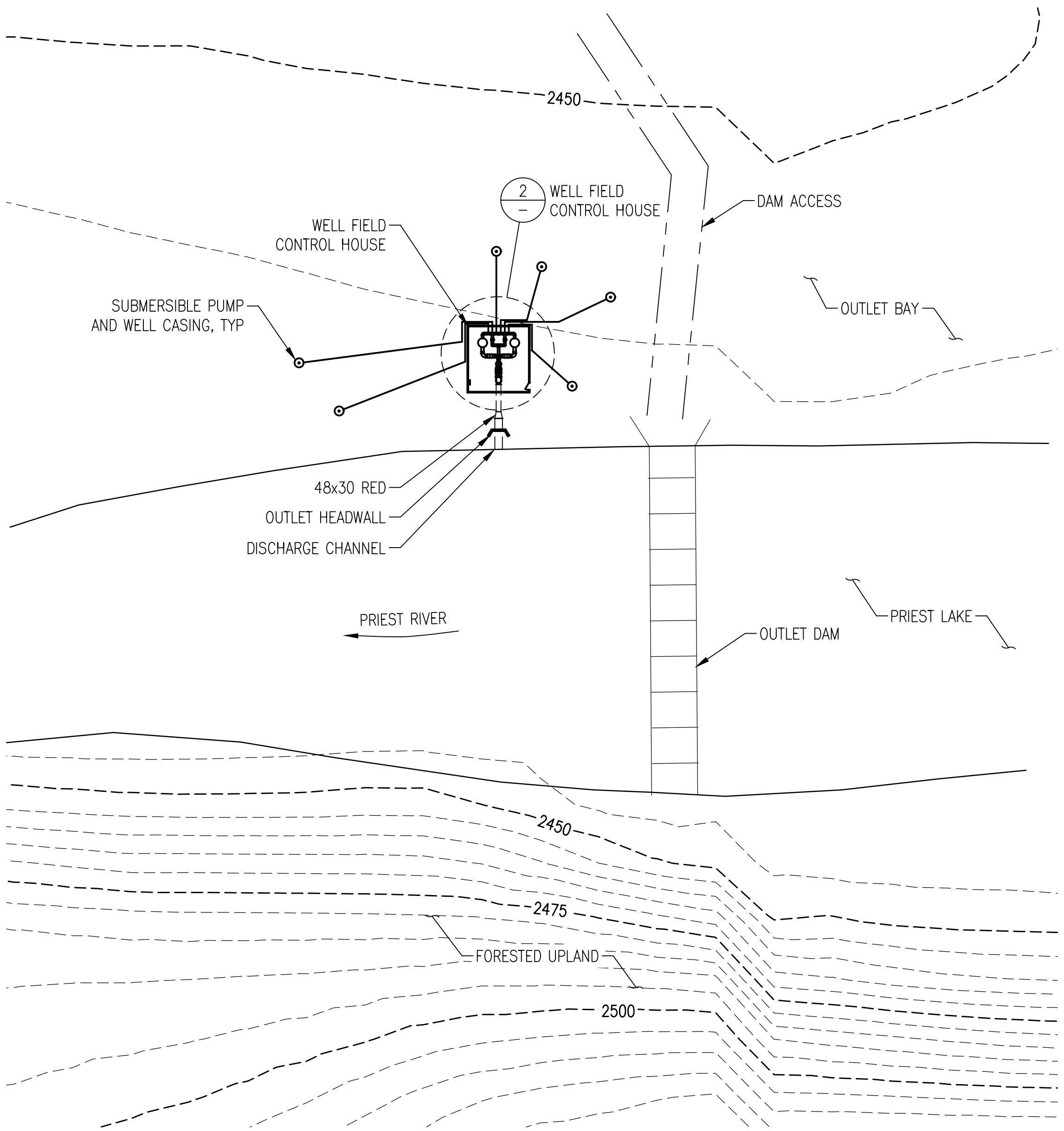
GEN AUTOCAD DWG

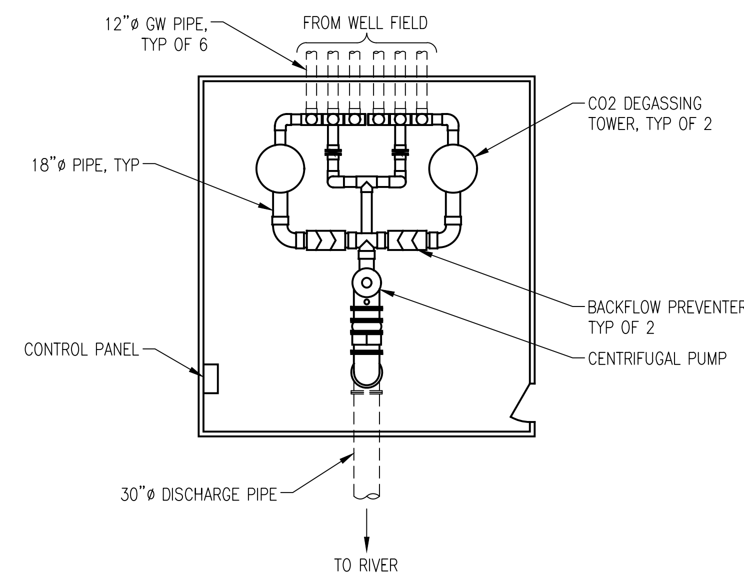
FIGURE A3.2

NOTES:

1. NUMBER AND LOCATION OF GROUNDWATER WELLS TO BE DETERMINED BASED ON GEOPHYSICAL INVESTIGATION AND AQUIFER PUMP TESTING. PUMPS TO BE SIZED ACCORDINGLY.
2. OVERALL LOCATION OF WELL FIELD WILL DEPEND ON WATER AVAILABILITY AND AQUIFER PUMP TESTING. IF RELOCATION IS NEEDED, WELL FIELD WILL SHIFT DOWNSTREAM AND/OR ACROSS THE RIVER.



1 / A4.1 DETAIL - WELL FIELD
SCALE: 1:50



2 / – DETAIL - WELL FIELD CONTROL HOUSE
SCALE: 1:10

McMILLEN JACOBS ASSOCIATES

ALTERNATIVE 3
GROUNDWATER WELL SYSTEM

DETAILS

AVISTA CORPORATION
SPOKANE, WASHINGTON

AS NOTED		APPROVED
SCALE	DATE	
DESIGN KJ	CHECKED	
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DWG NO: FIGURE A4.2

GEN AUTOCAD DWG

11/12/2019 1:43 PM
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ALT 7 - PUMP STATION SITE LAYOUT
SCALE: 1:1000

NOTES:

1. SEE FIGURE A4.2 FOR ALIGNMENT PROFILE.

McMILLEN JACOBS ASSOCIATES

ALTERNATIVE 4
PUMP STATION

SITE LAYOUT

AVISTA CORPORATION
SPOKANE, WASHINGTON

1" = 1000' SCALE	DATE	APPROVED
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DRAWN RG	NOTED	DATE
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FIGURE A4.1

GEN AUTOCAD DWG

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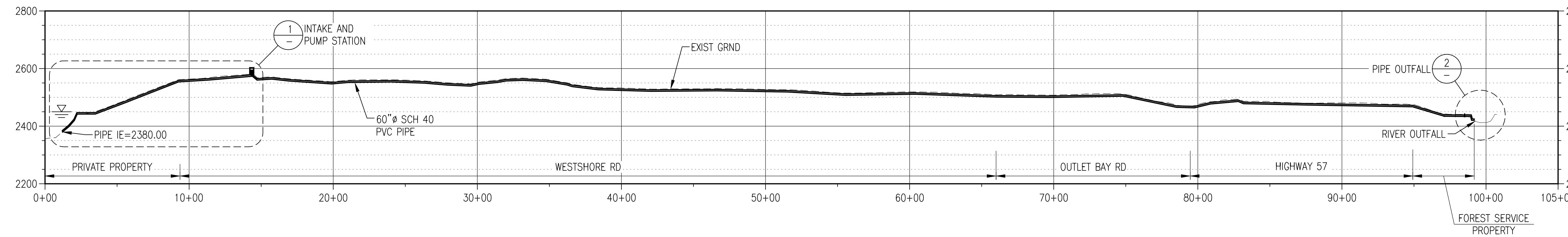
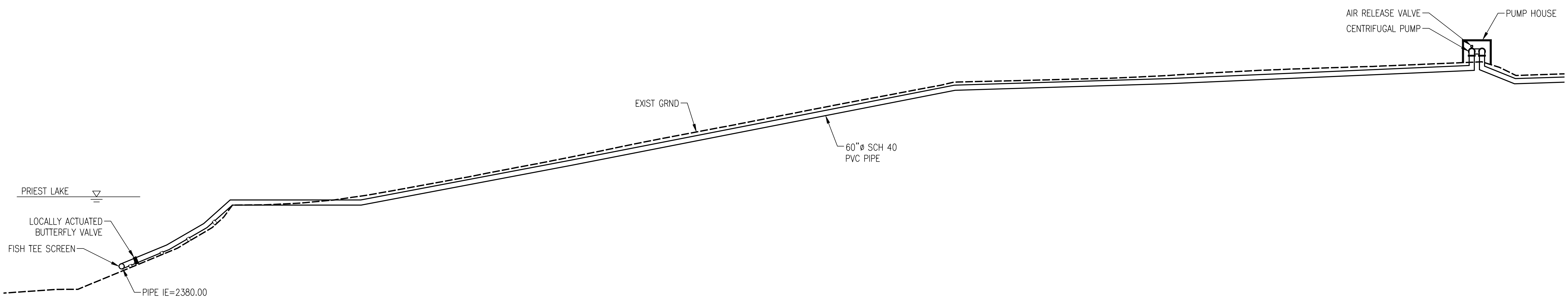
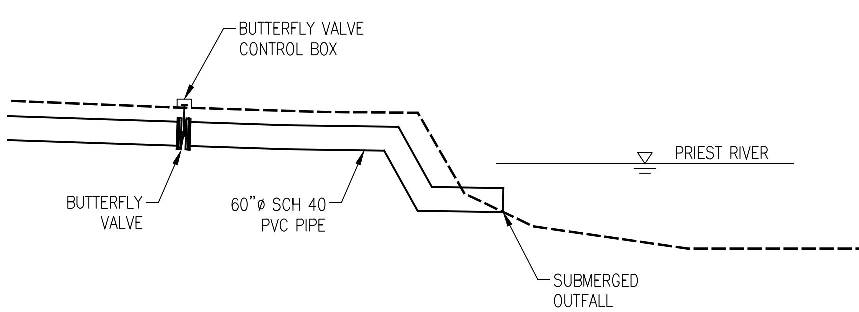


ALT 4 - PIPELINE PROFILE
SCALE: 1:400H 1:200V



1 / A5.1 DETAIL - INTAKE AND PUMP STATION
SCALE: 1" = 50'



2 / A5.1 DETAIL - PIPE OUTFALL
SCALE: 1" = 20'

McMILLEN JACOBS ASSOCIATES

ALTERNATIVE 4
PUMP STATION

PROFILE AND DETAILS

AVISTA CORPORATION
SPOKANE, WASHINGTON

AS NOTED		APPROVED	
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DWG NO: FIGURE A5.2

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GEN AUTOCAD DWG

Appendix B

Hydrology and Hydraulic Calculations

SUBJECT:	Avista Corporation	**BY:** J. Wiegand	**CHK'D BY:** V. Autier
	Priest Lake Cold Water Bypass	**DATE:** 11/22/2019	
	Mechanical Analysis - Pump Sizing	**PROJECT NO.:** 18-090	

Purpose

The purpose of this calculation sheet is to estimate the required brake horsepower for pumps under Alternatives 1A, 1B and 4.

References

- *Volk, Michael, 2017. Pump Characteristics and Applications: Third Edition, CRC Press.*

Equations

Alternative 1A - Gravity System

BHP	$\boxed{47}$	hp	Brake horsepower
Q	31,865	gpm	Target flow rate
H *	5.28	ft	Total head (head losses only)
SG	1	-	Specific gravity
η	0.9	-	Pump efficiency
Q	71	cfs	Target flow rate
h	0	ft	Static head

$$BHP = \frac{Q\ x\ H\ x\ SG}{3960\ x\ \eta}$$

* Assumes that the radial gates on the dam are fully open so that the static head across the dam is roughly zero

Alternative 1B - Gravity System

BHP	$\boxed{80}$	hp	Brake horsepower
Q	47,124	gpm	Target flow rate
H *	6.04	ft	Total head (head losses only)
SG	1	-	Specific gravity
η	0.9	-	Pump efficiency
Q	105	cfs	Target flow rate
h	0	ft	Static head

$$BHP = \frac{Q\ x\ H\ x\ SG}{3960\ x\ \eta}$$

* Assumes that the radial gates on the dam are fully open so that the static head across the dam is roughly zero

Alternative 4 - Intake Pump Station

BHP	$\boxed{780}$	hp	
Q	31,865	gpm	
H	91.06	ft	Static head + head losses
SG	1	-	
η	0.94	-	
h	85	ft	Static head

$$BHP = \frac{Q\ x\ H\ x\ SG}{3960\ x\ \eta}$$

Conclusion

Alternative 1A and 1B would require a 47-hp and 80-hp booster pump, respectively, during the high flow season when insufficient head is available to drive flows with gravity. Alternative 4 would require a 780-hp pump to move 71 cfs over 92 feet of head.

SUBJECT:	Avista Corporation	**BY:** J. Wiegand	**CHK'D BY:** V. Autier
	Priest Lake Cold Water Bypass	**DATE:** 11/22/2019	
	Siphon Hydraulics	**PROJECT NO.:** 18-090	

Purpose

The purpose of this calculation sheet is to estimate the maximum siphon lift based on both vapor pressure and pipe colaps, and to estimate an adequate pipe diameter and outlet slope to minimize air entrainment.

References

- *Morrison Maierle, Inc., 2012. Guidelines for Use of Pumps and Siphons for Emergency Reservoir Drawdown. Accessed at http://madcs.org/files/Guidelines_for_Use_of_Pumps_and _Siphons_for_Emergency_Reservoir_DrawdoDr.pdf, February 1, 2019.*
- *Weitkamp, Don E. and Katz, Max, 1980. "A Review of Dissolved Gas Supersaturation Literature", in Transactions of the American Fisheries Society, Vol. 109, pp. 659-702.*

Equations

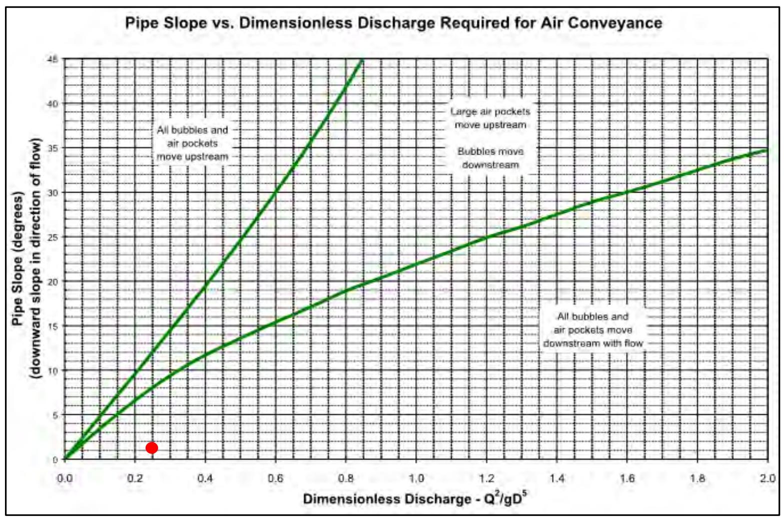
Maximum Lift

dz	17.6	ft	Maximum siphon lift
WSEL	2437.64	ft	Lake water surface elevation

$$\Delta z = 20 - \frac{WSEL}{1000}$$

Bubble Entrainment

Q^2	5041	ft^6/s^2	
g	32.2	ft/s^2	
D^5	3125	ft^5	
Q^2/gD^5	0.05	-	
Outlet Diameter	5	ft	Assumed
Pipe Slope	1%		Assumed



Pipe Collapse Pressure

Pc	3.1	psi	Critical collapse pressure
E	3.0E+07	psi	Modulus of elasticity
T	0.2	in	Pipe wall thickness
nu	0.3	-	Poisson's ratio
R	30.2	in	Pipe outside radius
Hc	7.1	ft	Critical head

$$P_c = \frac{1}{4}\frac{E}{(1-\nu)^2}\frac{T^3}{R^3}$$

Conclusion

The maximum lift is 17.6 feet, whereas the critical head for pipe collapse is 7.1 feet. Therefore, the critical head controls and the maximum lift should not be higher than 7.1 feet for the system. The outlet slope would be 1%, with an outlet pipe diameter of 5 feet.

SUBJECT:	Avista Corporation	**BY:** J. Wiegand	**CHK'D BY:** V. Autier
	Priest Lake Cold Water Bypass	**DATE:** ########	
	Hydraulic Analysis - System Head Losses	**PROJECT NO.:** 18-090	

Purpose

The purpose of this calculation sheet is to estimate the total head losses through the proposed pipelines.

References

- *Houghtalen, Akan & Hwang, 2010. Fundamentals of Hydraulic Engineering Systems: Fourth Edition.*
- *Rennels, Donald and Hudson, Hobart, 2012. Pipe Flow A Practical and Comprehensive Guide, John Wiley and Sons, Inc.*
- *Tent. Stds. Hydr. Inst.*
- *Tullis, J. Paul, 1989. Hydraulics of Pipelines: Pumps, Valves, Cavitation, Transients, John Wiley and Sons, Inc.*
- *Volk, Michael, 2017. Pump Characteristics and Applications: Third Edition, CRC Press.*

Equations

Both the Darcy-Weisbach and Hazen-Williams equations are used to estimate friction losses in pipes.

Darcy-Weisbach Equation

$$h = f \frac{L}{D} \frac{v^2}{2g}$$

where:

f: friction factor (solved for iteratively using Colebrook-White Equation, shown below)

L: length of pipe

D: diameter of pipe

v: velocity of the pipe

g: gravitational acceleration

Hazen-Williams Equation

$$h = 3.022 \frac{v^{1.85} L}{C^{1.85} D^{1.17}}$$

where:

v: velocity (ft/s)

L: length of pipe (ft)

C: Hazen William's Coefficient

D: Diameter (ft)

A range of Cs (100 to 150) will be chosen to compare the head loss for varying C-values. As the pipe ages, the C-value will decrease, increasing the head loss. A C-value of 150 is suggested for High-density polyethylene (HDPE) (Table 3.2, Houghtalen, Akan & Hwang, 2010. Fundamentals of Hydraulic Engineering Systems: Fourth Edition.)

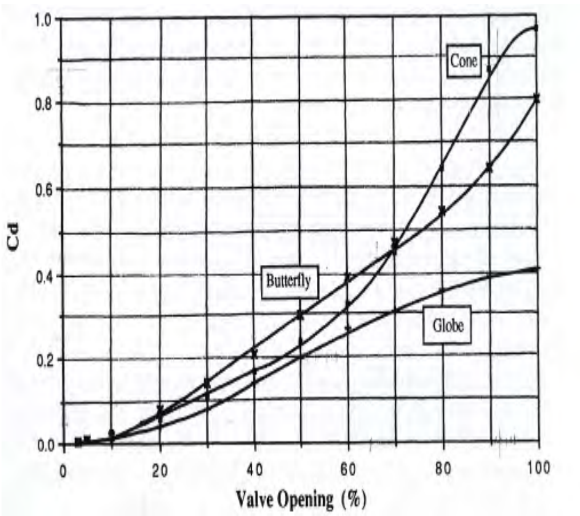
Colebrook-White Equation

The Colebrook-White Equation solves for the friction factor used in the Darcy-Weisbach Equation.

```
Function ColbWhite(D, ee, Q, nu)

Dim sf As Double, sf1 As Double, z As Double

sf = 8
Do
    sf1 = sf
    sf = 1.14 - 2 * Log(ee / D + 7.34347283 * nu * D * sf1 / Q) / Log(10)
    z = Abs(sf - sf1)
Loop While z >= 0.000001

ColbWhite = 1 / sf ^ 2

End Function
```

Butterfly Valve for Alternative 1A, 1B and 4

The valve allows for the flow from the pipes in Alternative 1A, 1B and 4 to be adjusted. The head loss from the butterfly valve is controlled by the % opening of the valve. The head losses from the valve have been calculated assuming that the valve is completely open (100%). This gives the minimum head losses for the valve. The valve can be closed so that there are more head losses, which will restrict the flow when desired. (Tullis, J. Paul, 1989. Hydraulics of Pipelines: Pumps, Valves, Cavitation, Transients, John Wiley and Sons, Inc.)



$$C_d = \left(\frac{1}{K_l + 1}\right)^{0.5}$$

$$h_{l,v} = \mathrm{K}_1 \frac{\mathrm{v}^2}{2g}$$

	Alt. 1A	Alt. 1B	Alt. 4	
$h_{l,v}$ (ft) =	0.11	0.12	0.11	head loss from the valve
v (ft/s) =	3.62	3.71	3.62	velocity through the pipe
K_1 =	0.56	0.56	0.56	
C_d =	0.80	0.80	0.80	
Valve Opening (%) =	an determine from graph abov			

Calculations

Darcy Weisbach Frictional Losses and Minor Losses Alternative 1A

Pipe length	8000	ft	
Gravitational acceleration	32.2	ft/s^2	
Kinematic viscosity	1.30E-05	ft^2/s	@ 55 deg F
Pipe wall roughness	5E-06	ft	(Table 3.1, Houghtalen, Akan & Hwang, 2010. Fundamentals of Hydraulic Engineering Systems: Fourth Edition.)
Pipe diameter	5	ft	
Pipe Area	19.63	ft^2	
Flow	71	cfs	
Velocity	3.62	fps	
Friction factor	0.01105	-	
Velocity head	0.20	ft	
Friction losses	3.59	ft	
K: four 45 deg bends	0.40		45 deg bend (Tullis, J. Paul, 1989. Hydraulics of Pipelines: Pumps, Valves, Cavitation, Transients, John Wiley and Sons, Inc.)
K: exit loss	1.00		
sum of K's	1.40		
minor loss valve	0.11		Butterfly Valve (Figure 4.3, Tullis, J. Paul, 1989. Hydraulics of Pipelines: Pumps, Valves, Cavitation, Transients, John Wiley and Sons, Inc.)
minor losses	0.40	ft	
Total Losses	$\boxed{5.28}$	ft	Includes frictional losses, minor losses and fish tee screen losses

Hazen Williams Frictional Losses for Alternative 1A

C	hf (ft)	min. loss (ft)	Screen Loss (ft)	Total Losses (ft)
100	7.91	0.40	1.29	9.60
110	6.63	0.40	1.29	8.32
120	5.65	0.40	1.29	7.33
130	4.87	0.40	1.29	6.56
140	4.25	0.40	1.29	5.93
150	3.74	0.40	1.29	5.42

Darcy-Weisbach Frictional Losses and Minor Losses Alternatives 1B (Q = 52.5 cfs, there are two of these pipes)

Pipe length	15	ft	
Gravitational acceleration	32.2	ft/s^2	
Kinematic viscosity	1.30E-05	ft^2/s	@ 55 deg F
Pipe wall roughness	5E-06	ft	(Table 3.2, Houghtalen, Akan & Hwang, 2010. Fundamentals of Hydraulic Engineering Systems: Fourth Edition.)
Pipe diameter	5	ft	
Pipe Area	19.63	ft^2	
Flow	52.5	cfs	
Velocity	2.67	fps	
Friction factor	0.01162	-	
Velocity head	0.11	ft	
Friction losses	0.004	ft	
K: 90 deg bend	0.24		90 degree elbow regular flange (Tent. Std. Hydr. Inst.)
sum of K's	0.24		
minor losses	0.03	ft	
Total Losses (1 pipe)	0.03	ft	includes friction losses and minor losses
Total Losses (2 pipes)	$\boxed{2.00}$	ft	this includes the losses for both of the 15 ft, 52.5 cfs pipes and fish screen loss

Darcy-Weisbach Frictional Losses and Minor Losses Alternatives 1B (Q = 105 cfs pipe)

Pipe length	8000	ft	
Gravitational acceleration	32.2	ft/s^2	
Kinematic viscosity	1.30E-05	ft^2/s	@ 55 deg F
Pipe wall roughness	5E-06	ft	(Table 3.2, Houghtalen, Akan & Hwang, 2010. Fundamentals of Hydraulic Engineering Systems: Fourth Edition.)
Pipe diameter	6	ft	
Pipe Area	28.27	ft^2	
Flow	105	cfs	
Velocity	3.71	fps	
Friction factor	0.01067	-	
Velocity head	0.21	ft	
Friction losses	3.05	ft	
K: four 45 deg bends	0.40		45 deg bend (Tullis, J. Paul, 1989. Hydraulics of Pipelines: Pumps, Valves, Cavitation, Transients, John Wiley and Sons, Inc.)
K: exit loss	1.00		
K: tee bend	2.70		Tee bend (Figure 16.15, Volk, Michael, 2017. Pump Characteristics and Applications: Third
sum of K's	4.10		
minor loss valve	0.12		Butterfly Valve (Figure 4.3, Tullis, J. Paul, 1989. Hydraulics of Pipelines: Pumps, Valves, Cavitation, Transients, John Wiley and Sons, Inc.)
sum of minor losses	1.00	ft	Includes from four 45 deg bends, exit loss, tee bend and butterfly valve
Total Losses (105 cfs p	4.05	ft	Includes friction losses and minor losses
Total Losses (all pipes)	$\boxed{6.04}$		Includes total losses from all three pipes

Hazen Williams Frictional Losses for Alternative 1B

C	hf (ft) (Q=52.5 cfs pipe)	hf (ft) (Q=52.5 cfs pipe)	hf (ft) (Q=105 cfs pipe)	min. loss (ft) (Q = 52.5 cfs pipe)	min. loss (ft) (Q = 52.5 cfs pipe)	min. loss (ft) (Q = 105cfs pipe)	Fish Screen Loss (ft)	Total Losses (ft)
100	0.01	0.01	6.72	0.03	0.03	1.00	1.94	9.72
110	0.01	0.01	5.63	0.03	0.03	1.00	1.94	8.63
120	0.01	0.01	4.79	0.03	0.03	1.00	1.94	7.79
130	0.01	0.01	4.13	0.03	0.03	1.00	1.94	7.13
140	0.00	0.00	3.60	0.03	0.03	1.00	1.94	6.60
150	0.00	0.00	3.17	0.03	0.03	1.00	1.94	6.17

Darcy-Weisbach Frictional Losses and Minor Losses Alternative 2

Pipe length	8000	ft	
Gravitational acceleration	32.2	ft/s^2	
Kinematic viscosity	1.30E-05	ft^2/s	@ 55 deg F
Pipe wall roughness	5E-06	ft	(Table 3.2, Houghtalen, Akan & Hwang, 2010. Fundamentals of Hydraulic Engineering Systems: Fourth Edition.)
Pipe diameter	5	ft	
Pipe Area	19.63	ft^2	
Flow	71	cfs	
Velocity	3.6	fps	
Friction factor	0.01105	-	
Velocity head	0.20	ft	
Friction losses	3.59	ft	
K: four 45 deg bends	0.40		45 deg bend (Tullis, J. Paul, 1989. Hydraulics of Pipelines: Pumps, Valves, Cavitation, Transients, John Wiley and Sons, Inc.)
K: exit loss	1.00		
sum of K's	1.40		
minor losses	0.28	ft	Includes friction losses and minor losses
Total Losses	5.16	ft	Includes fish tee screen losses

Hazen Williams Frictional Losses for Alternative 2

C	hf (ft)	min. loss (ft)	Fish Screen Loss (ft)	Total Losses (ft)
100	7.91	0.28	1.29	9.49
110	6.63	0.28	1.29	8.21
120	5.65	0.28	1.29	7.22
130	4.87	0.28	1.29	6.44
140	4.25	0.28	1.29	5.82
150	3.74	0.28	1.29	5.31

Darcy-Weisbach Frictional Losses and Minor Losses Alternative 4

Pipe length	9750	ft	
Gravitational acceleration	32.2	ft/s^2	
Kinematic viscosity	1.30E-05	ft^2/s	@ 55 deg F
Pipe wall roughness	5E-06	ft	(Table 3.2, Houghtalen, Akan & Hwang, 2010. Fundamentals of Hydraulic Engineering Systems: Fourth Edition.)
Pipe diameter	5	ft	
Pipe Area	19.63	ft^2	
Flow	71	cfs	
Velocity	3.62	fps	
Friction factor	0.01105	-	
Velocity head	0.20	ft	
Friction losses	4.37	ft	
K: four 45 deg bends	0.40		45 deg bend (Tullis, J. Paul, 1989. Hydraulics of Pipelines: Pumps, Valves, Cavitation,
K: exit loss	1.00		
sum of K's	1.40		
minor loss valve	0.11		
sum of minor losses	0.40	ft	
Total Losses	6.06	ft	Includes friction losses, minor losses and fish tee screen losses

Hazen Williams Frictional Losses for Alternative 4

C	hf (ft)	min. loss (ft)	Fish Screen Loss (ft)	Total Losses (ft)
100	9.64	0.40	1.29	10.04
110	8.08	0.40	1.29	8.48
120	6.88	0.40	1.29	7.28
130	5.94	0.40	1.29	6.33
140	5.17	0.40	1.29	6.86
150	4.55	0.40	1.29	6.24

Conclusion

The head losses are as follows, when calculated with the Darcy Weisbach equation:
- Alternative 1A: 5.28 ft.
- Alternative 1B: 6.04 ft.
- Alternative 2: 5.16 ft.
- Alternative 4: 6.06 ft.

These head losses were verified using the Hazen Williams equation, which shows a range of head losses that depend on the condition of the pipe.

SUBJECT:	Avista Corporation	**BY:** J. Wiegand	**CHK'D BY:** V. Autier
	Priest Lake Cold Water Bypass	**DATE:** 11/22/2019	
	Hydraulic Analysis - Fish Tee Screen	**PROJECT NO.:** 18-090	

Purpose

The purpose of this calculation sheet is to estimate the total head losses through the proposed fish tee screen.

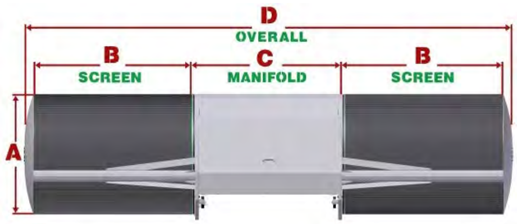
References

- *USBR, "Fish Protection at Water Diversion", April 2006*

Information

Acceleration of Gravity, g = 32.2 ft/s²

Square Entrance Loss, K_e = 2.7 At Manifold of Fish Screen



	Pipe (Q = 71 cfs)	(Q = 52.5 cfs for two pipes)	
Pipe Diameter =	60	60	in
Number of Screens =	1	2	
Fish Screen Diameter, A =	5.0	5	ft
Screen Length, B =	5.0	5	ft
Manifold Length, C =	5.0	5	ft
Overall Length, D =	15.0	15	ft
Screen Area Per Unit =	157.08	157.08	ft²
Total Screen Area =	157	314	ft²
Screen Porosity =	50%	50%	
Suction Pipe Diameter =	3.0	3.0	ft
Suction Pipe Area =	7.1	7.1	ft²
Suction Pipe Porosity =	18%	18%	

Calculation

Fish Screen Loss (Q = 71 cfs)

Flow, Q =	71	ft^3/s	
Profile Bar (Wedgewire)			
Fish Screen Area, A =	157	ft^2	
Velocity, V =	0.45	ft/s	Tee Screen Approach Velocity
Velocity Head =	0.003	ft	Velocity Head, $V^2/2g$
Screen Porosity =	50%	percent	Assumed
Angle to Flow =	90	degrees	
Angle Multiplier =	1.00		(Figure 48, USBR, "Fish Protection at Water Diversion", April 2006)
k =	2.45		(Figure 47, USBR, "Fish Protection at Water Diversion", April 2006)
Adjusted k =	4.90		
Minor Loss, K_l =	4.90		Includes: Fish Screen
Minor Losses, h_l =	0.02	ft	Minor Losses, K_l $(V^2/2g)$
Porosity of Suction Pipe			
Area, A =	94	ft^2	Porosity Area of Suction Pipe
Velocity, V =	0.75	ft/s	
Velocity Head =	0.009	ft	Velocity Head, $V^2/2g$
Porosity =	18%	percent	Calculated based on ratio of open area from Suction Pipe to Screen
Head Loss Coefficient, k =	59.2		(Figure 45, USBR, "Fish Protection at Water Diversion", April 2006)
Minor Loss, K_l =	59.2		Includes: Porosity Plate
Minor Losses, h_l =	0.52	ft	Minor Losses, K_l $(V^2/2g)$
Manifold Connection Area, A =	19.63	ft^2	Manifold Area for 4 Screen Connections
Velocity, V =	3.62	ft/s	
Velocity Head =	0.20	ft	Velocity Head, $V^2/2g$
Minor Loss, K_l =	3.70		Includes: Manifold Intake Loss, Exit Loss
Minor Losses, h_l =	0.75	ft	Minor Losses, K_l $(V^2/2g)$
Total Losses, h_L =	$\boxed{1.29}$	ft	

Fish Screen Loss (Q = 52.5 cfs for two pipes)

Flow, Q =	105.0	ft^3/s	
Profile Bar (Wedgewire)			
Fish Screen Area, A =	314	ft^2	
Velocity, V =	0.33	ft/s	Tee Screen Approach Velocity
Velocity Head =	0.002	ft	Velocity Head, $V^2/2g$
Screen Porosity =	50%	percent	Assumed
Angle to Flow =	90	degrees	
Angle Multiplier =	1.00		(Figure 48, USBR, "Fish Protection at Water Diversion", April 2006)
k =	2.45		(Figure 47, USBR, "Fish Protection at Water Diversion", April 2006)
Adjusted k =	4.90		
Minor Loss, K_l =	4.90		Includes: Fish Screen
Minor Losses, h_l =	0.01	ft	Minor Losses, K_l $(V^2/2g)$
Porosity of Suction Pipe			
Area, A =	188	ft^2	Porosity Area of Suction Pipe
Velocity, V =	0.56	ft/s	
Velocity Head =	0.005	ft	Velocity Head, $V^2/2g$
Porosity =	18%	percent	Calculated based on ratio of open area from Suction Pipe to Screen
Head Loss Coefficient, k =	59.2		(Figure 45, USBR, "Fish Protection at Water Diversion", April 2006)
Minor Loss, K_l =	59.2		Includes: Porosity Plate
Minor Losses, h_l =	0.29	ft	Minor Losses, K_l $(V^2/2g)$
Manifold Connection Area, A =	19.63	ft^2	Manifold Area for 4 Screen Connections
Velocity, V =	5.35	ft/s	
Velocity Head =	0.44	ft	Velocity Head, $V^2/2g$
Minor Loss, K_l =	3.70		Includes: Manifold Intake Loss, Exit Loss
Minor Losses, h_l =	1.64	ft	Minor Losses, K_l $(V^2/2g)$
Total Losses, h_L =	$\boxed{1.94}$	ft	

Conclusion

Total head loss through the fish tee screen (when Q = 71 cfs , D = 60 in) is estimated at 1.29 feet with the assumptions stated above.

Total head loss through two fish tee screens (when Q = 105 cfs, D = 60 in) is estimated at 1.94 feet with the assumptions stated above

Appendix C

Outlet Bay Subsurface Data Review

Technical Memorandum

To:	Kiira Siitari, Idaho Department of Fish and Game Ken Bouwens, Idaho Department of Fish and Game	Project:	Priest River Cold-Water Augmentation Alternatives Analysis
From:	Paul Richards, PE (WA, OR, AK) McMillen Jacobs Vincent Autier, PE McMillen Jacobs	cc:	File
Date:	9/10/19	Job No.:	Avista R-42062
Subject:	Outlet Bay Subsurface Conditions Data Review		

Revision Log

Revision No.	Date	Revision Description
0	August 21, 2019	Draft Outlet Channel Subsurface Data Review
1	September 10, 2019	Draft Updated With Outlet Bay Site Visit Data

1.0 INTRODUCTION

Avista Corporation, in cooperation with the Idaho Department of Fish and Game (IDFG), has contracted with McMillen Jacobs Associates (McMillen Jacobs) to provide design services to develop and evaluate conceptual design alternatives for a cold-water augmentation system for Priest River. The project has been developed to improve the health of aquatic biota by reducing the temperature of Priest River. The project is located near Outlet Bay in Priest Lake, Idaho (Figure 1).

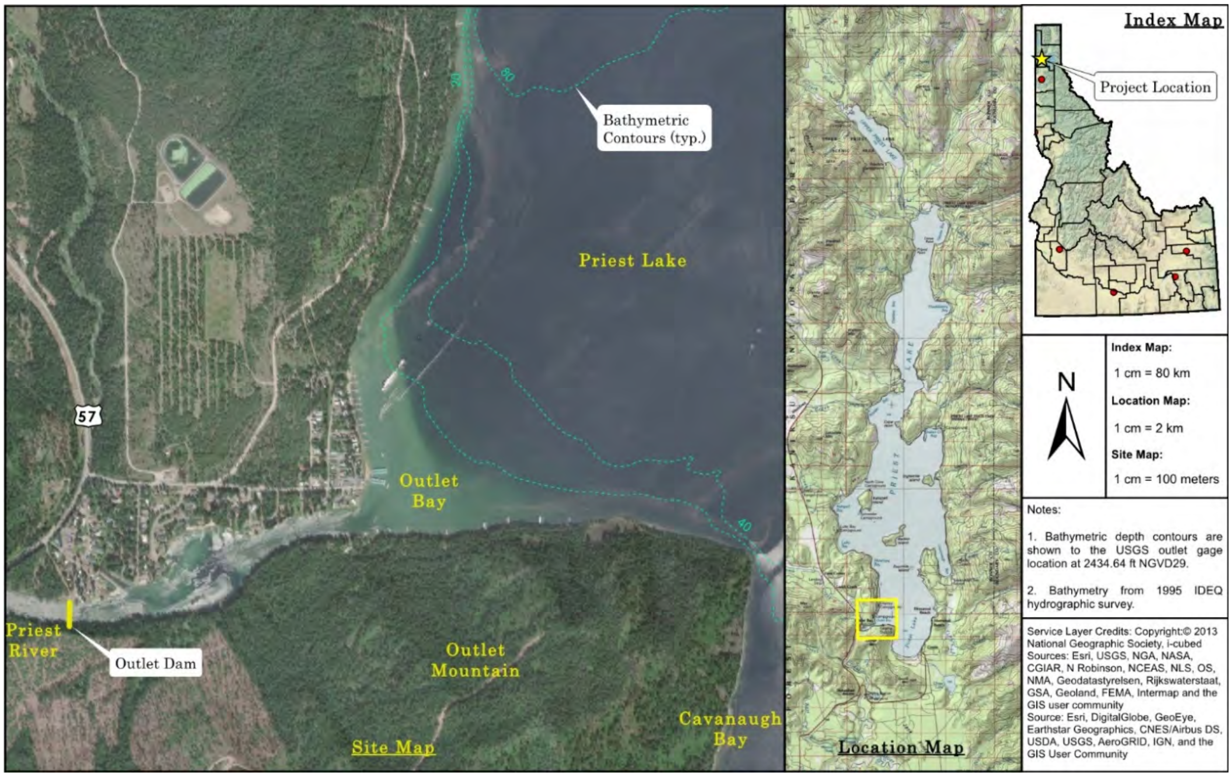


Figure 1. Project Site Location

1.1 Project Background

Priest Lake is located in Bonner County, in the northern panhandle of Idaho. Priest Lake is roughly 18 miles long running from north to south, and is drained by the Priest River, which is a major tributary of the Pend Oreille River. Streamflow in the Priest River is regulated by the Outlet Dam located at the southwestern end of Priest Lake. Originally built in 1950, the first Priest Lake Outlet Dam was later replaced by a concrete gravity dam with construction completed in 1978. The Outlet Dam is owned and operated by the Idaho Water Resources Board (IWRB). Dam operations in the summer months are manipulated to maintain lake levels for summer recreation and to provide minimum instream flows downstream of the dam.

Recent stream temperature modeling, led by the Kalispel Tribe of Indians, was conducted in Priest Lake and the Priest River. The modeling confirmed the thermodynamic feasibility of reducing late summer stream temperatures in the Priest River by adding water to Priest River flows that originates in the lower portion (i.e. hypolimnion) of the thermally stratified Priest Lake. The model considered input flow rates between 30 cfs and 400 cfs, and predicted that, if 75% of stream inflow came from the hypolimnion, the upper 30 miles of the Priest River could cool between 2 and 10 °C, which would be enough to improve native trout habitat conditions and meet the Idaho Department of Environmental Quality (IDEQ) criteria for cold-water aquatic life during August and September (i.e. less than 19°C).

1.2 Purpose

McMillen Jacobs has previously conducted an alternatives analysis assessment for the project (McMillen Jacobs Associates, 2019). A report of the analysis identified two alternatives to develop further, a groundwater system and a gravity system. The groundwater system would involve installing a well field near the Priest Lake Outlet Dam. Water from the well field would then be added to Priest River at this location. This approach is not addressed in this memorandum. The gravity system would include a 60-inch diameter pipeline installed at least 3 feet below the lake surface, requiring trenching and burial at shallower locations to promote boat navigation (Figure 2).

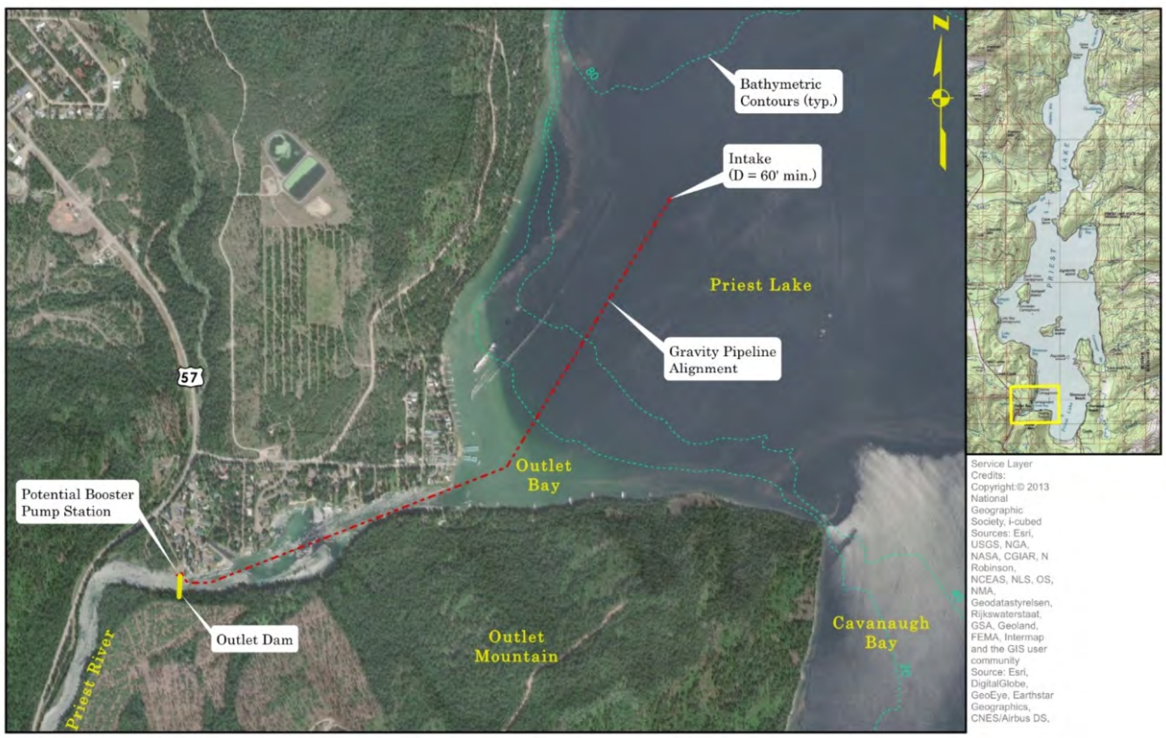


Figure 2. Conceptual Gravity System Layout

Concerns were raised regarding the constructability of the gravity system outlet pipe associated with the outlet system during a public presentation of the alternatives analysis. Specifically, concerns were that rock may be present within the outlet channel, which would result in more difficult construction conditions. Further concerns were that the presence of rock may lead designers to allow for a gravity system outlet pipe that was not deep enough below the water surface to allow for boats to pass without striking the gravity system pipeline.

The purpose of this memorandum is to collect and present available data related to the subsurface conditions at the location of the outlet channel and the Outlet Dam. This information will determine the potential construction impact of the conceptual gravity option. The data presented in this memorandum is intended to address public concerns, guide the cold-water outlet concept evaluation, and provide a basis for recommendations for future site evaluations.

2.0 DOCUMENT REVIEW

McMillen Jacobs has reviewed publicly available regional geologic maps, soil surveys, water well logs, Outlet Dam construction drawings, and photographs to evaluate the conditions of Priest Lake's Outlet Bay and the upper portions of Priest River near the project site. Further, McMillen Jacobs reviewed photographs of these areas from previous site visits.

2.1 Regional Geologic Map

The Idaho Geological Survey (IGS) has published a regional geologic map that includes the project site (Lewis, 2008). The IGS map suggests that the rock of the Belt Super Group is prevalent in the upper portions of Outlet Mountain to the south and to the west of Highway 57. The Belt Supergroup Rocks are intruded by mafic igneous rock in the intermediate slopes of Outlet Mountain. The lower slopes and low-lying areas adjacent to the outlet are mapped as glacial outwash gravels to the south and glaciofluvial deposits to the north (Figure 3).

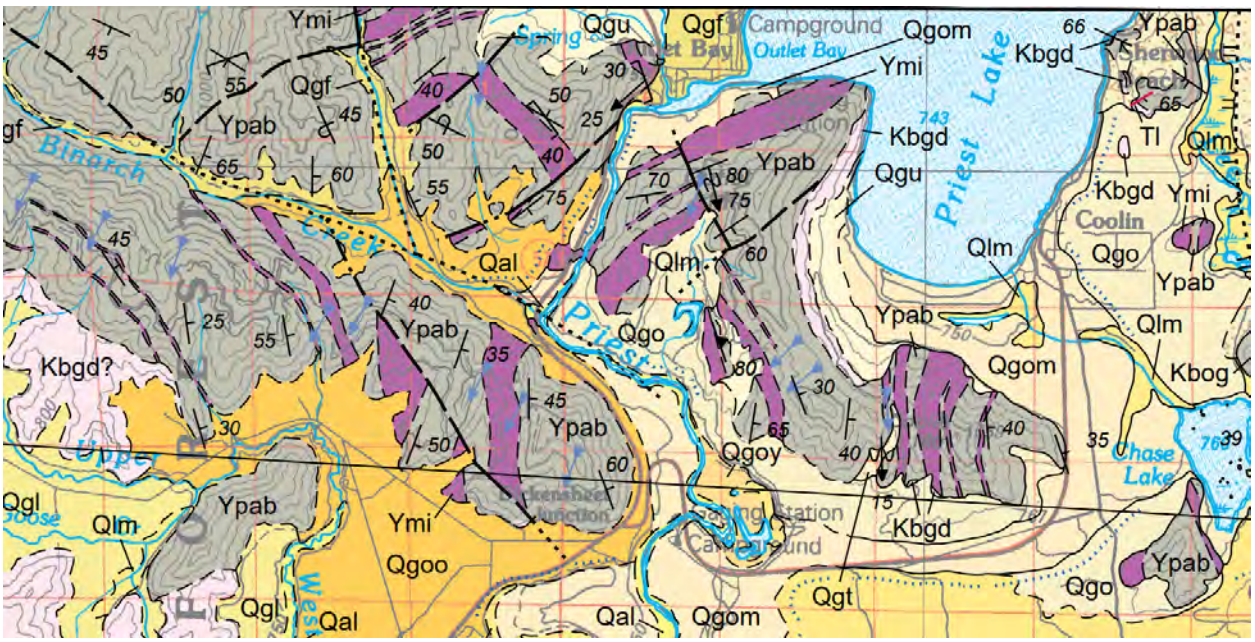


Figure 3. Geologic Map of Project Site (after Lewis et al., 2008)

2.2 Soil Maps

McMillen Jacobs reviewed Natural Resource Conservation Service (NRCS) soil maps to evaluate the presence, characteristics, and anticipated depth of the soils at the project site. Table 1 and Figure 4 present the soils mapped along the outlet shoreline at the project site. Bonner Silt Loam is mapped along the south slope of the outlet, while Caribou Ridge-Stein Families Complex and Glacier Creek soil deposits are mapped to the north of the outlet. In general, each of the soil deposits mapped consist of varying amounts of sand, silt, gravel and cobbles. NRCS mapping indicates that depths to restrictive layers are greater than 80 inches for each of the deposits mapped along the outlet shoreline. These soils are generally consistent with the geologic IGS mapping discussed in Section 2.1.

Figure 4. Map of NRCS Soil Units (after NRCS, 2019)

Table 1. List of NRCS Mapped Soil Units Along Banks of Outlet Bay

NRCS Soil Number	NRCS Soil Name
4	Bonner Silt Loam, cool, 0 to 4 percent slopes
103	Glacier Creek Typic Udifluvents-Marble Creek Families, Complex granitic alluvial substratum, narrow valley bottoms and toe slopes
155	Caribou Ridge-Stien Families, complex outwash plains of mixed geology

2.3 Water Well Logs

McMillen Jacobs reviewed water well logs on file with the Idaho Department of Water Resources (IDWR) to help identify subsurface conditions near the project site. A total of three well logs were identified in close proximity of Outlet Bay and the upper portion of Priest River. Two of the well logs were identified on the northern shoreline (one on Kokanee Park Loop at Lat 48.493, Long -116.903 and one on Match Bay Court at Lat 48.493, Long -116.899). A third water well log was identified near the southern shore of the outlet (Lat 48.489, Long -116.897). The locations of these wells are depicted on Figure 5. Well logs to the north of the channel indicate that bedrock is 30 feet below the ground surface or greater. The well located south of the outlet did not encounter bedrock within 149 feet of the ground surface (IDWR, 2019). The well logs are consistent with the geological and soils data presented above from the IGS and NRCS.

Figure 5. View of Well Locations on File with IDWR

2.4 Outlet Dam Construction Plan

Mott MacDonald recently completed a water management study of Priest Lake that includes scans of selected sheets of the Outlet Dam construction plans as an appendix (Mott MacDonald, 2018). The plans include three geotechnical borings conducted at the site, two of which were advanced within the footprint of the dam (DH#1 and DH#2), and the third boring within Priest River downstream of the dam (DH#4) (Figure 6). Each of the borings were advanced to a depth of 50 feet below grade (Figure 7). Logs indicate that the subsurface conditions within the outlet channel at the location of the Outlet Dam consist of interbedded layers of sand, silt, and clay. The logs show that sand and gravel are present in the upper portions of the soil profile. Rock was not encountered in any of the three borings. Standard penetration test blow counts suggest that the relative density of the sand layers is generally loose and the consistency of silt and clay layers is typically soft to medium stiff.

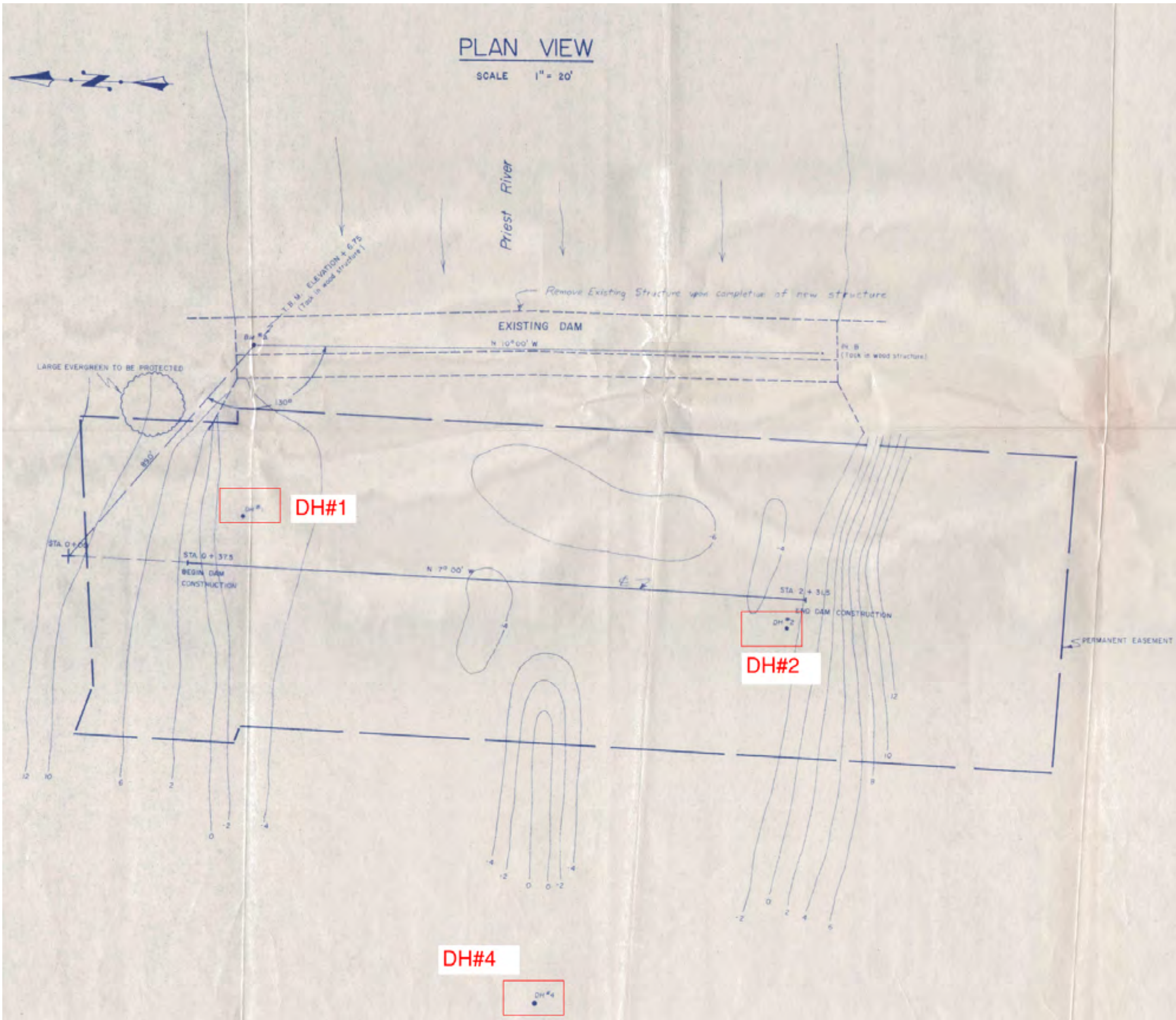


Figure 6. Exploration Plan for Outlet Dam (Emphasis Added for Legibility) (IDWR, 1978)

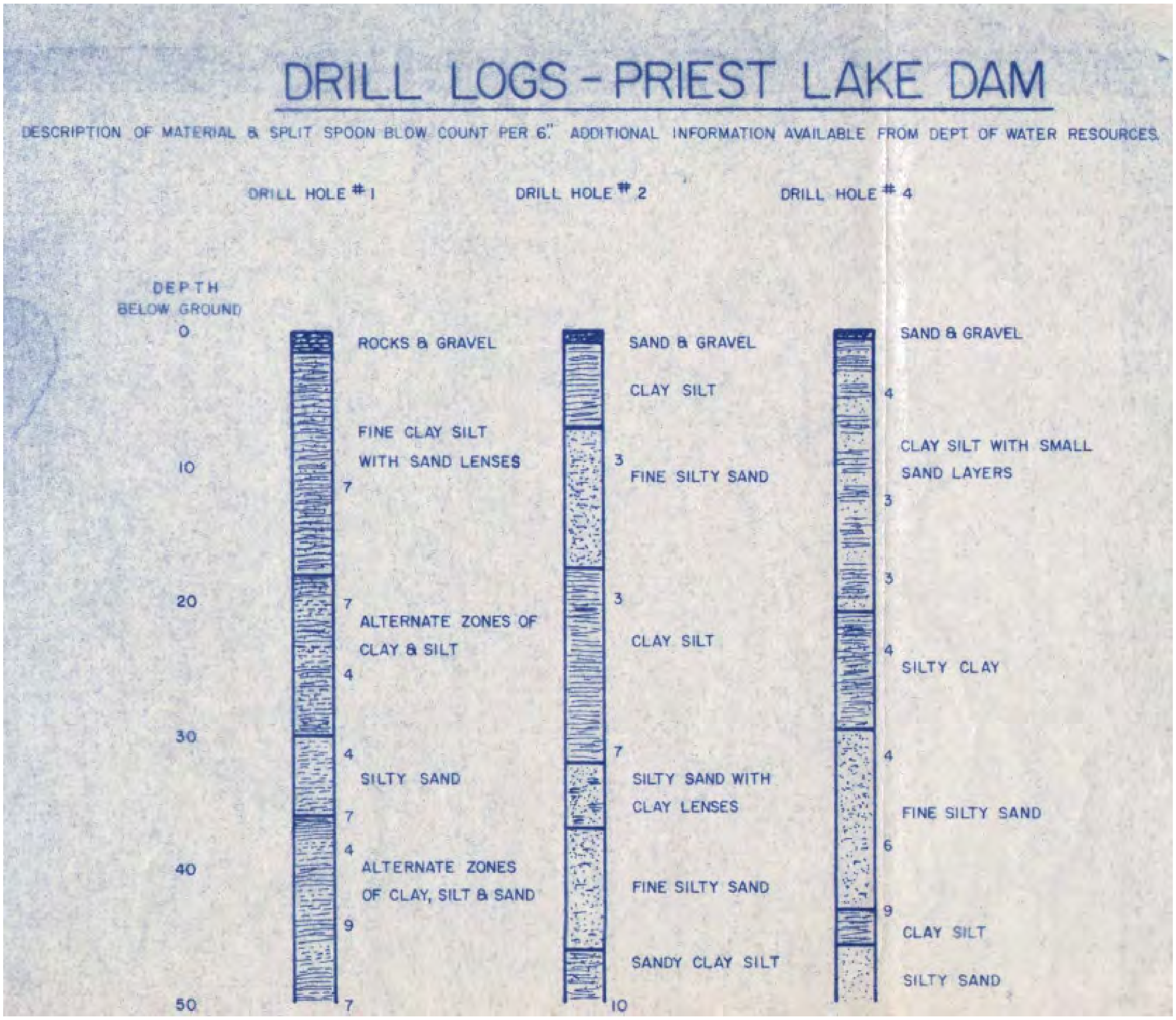


Figure 7. Exploration Logs for Outlet Dam (IDWR 1978)

2.5 Photograph Review

McMillen Jacobs also reviewed available photographs of the site taken during a recent site visit and published in regional newspapers. Figure 8 presents a photograph taken downstream of the outlet gates. In this photo, soil can be seen in the southern shore line (Area A). Soil exposed in the bank appears to be predominately fine grained with cobble to boulder sized talus accumulating at the toe of the slope. Sub-rounded to rounded gravelly and cobbly soil can be observed in the north side of the outlet (Area B). This is similar to a photograph of the outlet dam published by the Spokesman Review (Russel, 2018) (photograph available at link provided in references). These observations are consistent with the geologic mapping, soil survey mapping, well logs, and borings described previously.

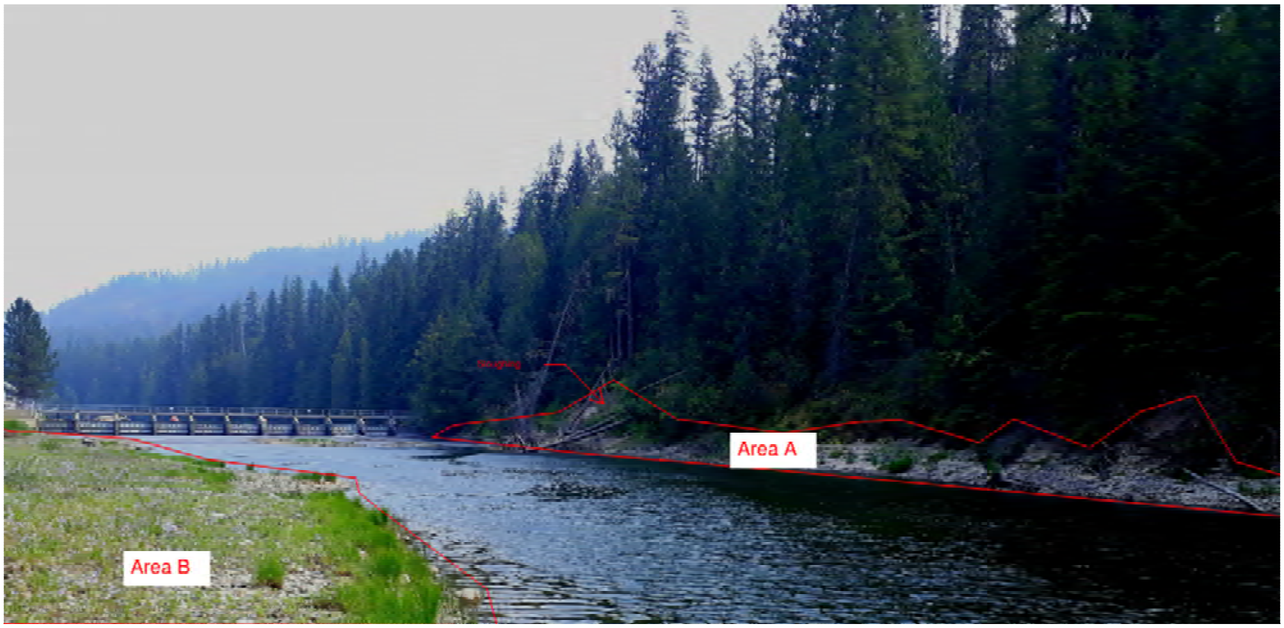


Figure 8. Photograph of Shoreline Slopes Downstream of Outlet Dam

2.6 Site Visit

McMillen Jacobs visited the site on August 26, 2019 to visually observe the conditions of Outlet Bay upstream of the dam. Observations were made from a boat provided and operated by IDFG. McMillen Jacobs and IDFG conducted four passes of the outlet bay to observe the soil and rock conditions along the banks of the outlet and the visible portions of the lake bottom. Where water depths were shallow enough to permit, a 3/8-inch diameter, 8-ft long metal rod was used to probe the bottom of the lake bed at selected locations. Photographs were taken of the lake bed surface were feasible. Representative photographs are presented in Appendix A.

During the site visit, McMillen Jacobs observed that the lake bottom was typically comprised of varying amounts of silt, sand, gravel, cobbles, and occasional boulders. Gravel, cobble and boulder sized particles where typically rounded to sub-rounded, suggesting that they had been transported to their current location rather than weathering in place. Sub-angular to angular boulders were observed in a limited area along the southern shore of Outlet Bay, as depicted in Figure 9. The more angular properties of these boulders would suggest that they originated close to their current location. No surface exposures of bedrock were observed during the site visit. Probing of the subsurface using a 3/8-inch diameter metal rod did not identify rock at any accessible areas encountered during the survey. These observations are consistent with the previous data collected from published sources and site photographs.

In addition, a number of man-made objects were observed below the water surface that may have an impact on the construction of the cold-water augmentation outlet system. A series of timber pilings were observed extending across the central portion of Outlet Bay (Figure 9; Appendix A, Figure A25). Timber pilings are typically driven into soil, which suggest that the presence of rock within the upper ground surface at these locations would be unlikely. A potential slab was observed downstream of the timber pilings in the south-central portion of outlet bay (Figure 9; Appendix A, Figure A15). It is postulated that a rudimentary concrete slab, or other similar structures, could have been constructed prior to being inundated by the installation of the existing outlet structure. Finally, a submerged wooden structure was observed just upstream of the existing outlet structure (Figure 9; Appendix A, Figure A21). McMillen

Jacobs recommends that these man-made objects be further investigated, surveyed, and identified where they correspond with any potential component of the cold-water augmentation system in order to fully inform any bidding contractors and minimize the risk of unforeseen site conditions.

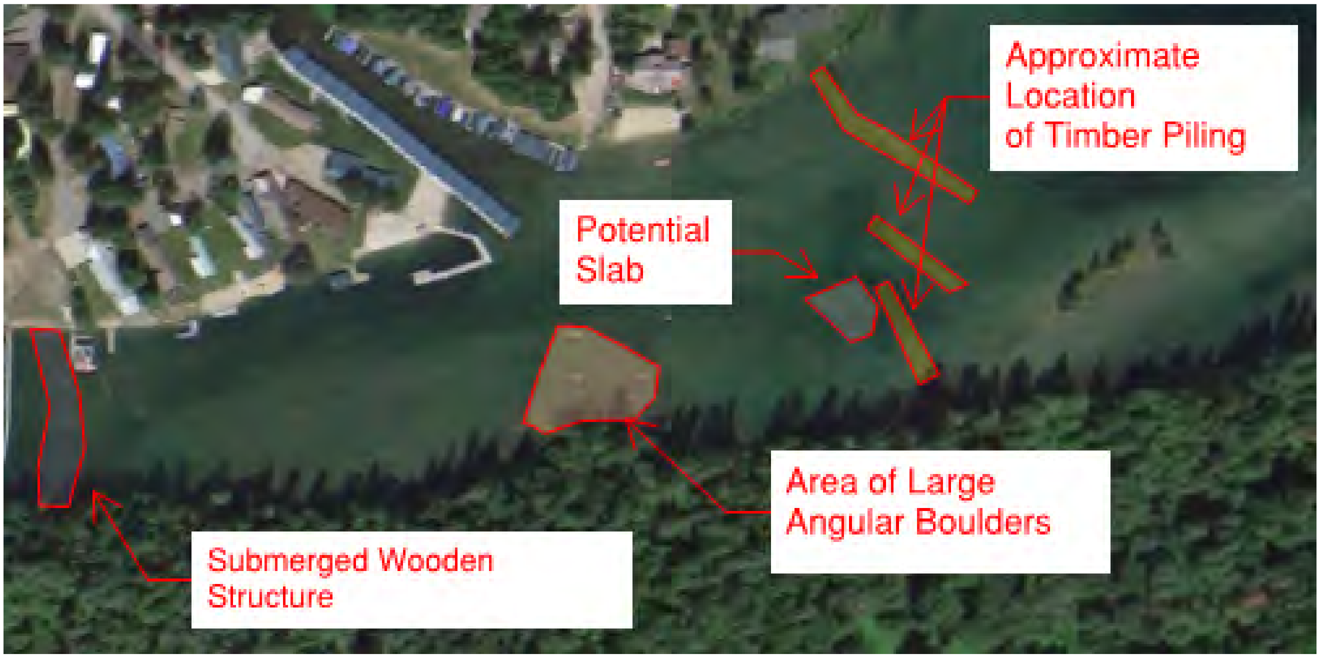


Figure 9. Location of Key Findings During Outlet Bay Site Visit

3.0 CONCLUSIONS AND RECOMMENDATIONS

3.1 Conclusion

Based on our review of data available from the IGS, NRCS, IDWR, construction plans, site photographs, regional newspapers, and site visit observations it is likely that the substrate of the proposed pipeline within Outlet Bay and the upper portion of Priest River consists of soil comprised of mixtures of silt, sand, clay, gravel, cobbles, and boulders. We anticipate that the soil substrate extends beyond the depths of the pipeline based on regional boring logs and NRCS soil surveys. Previous borings conducted at the site of the Outlet Dam and timber piling suggest that the subsurface soils can be excavated using conventional construction equipment.

While McMillen Jacobs was not able to find any evidence of bedrock within the near subsurface of the outlet channel, it is important to note that the anticipated subsurface soils are based on regional data, site-specific data collected at the location of the Outlet Dam, and surface observations. No borings or site-specific subsurface data have been obtained along the upstream portion of the conceptual gravity system pipeline. In addition, subsurface conditions can vary significantly over short distances. As such, the depth of rock along the entire pipeline alignment cannot be known precisely prior to excavation.

3.2 Recommendation

Should additional detail be required to advance the selection of alternative, refine capital cost estimate based on construction methods, further studies would need to be conducted to confirm the anticipated subsurface conditions in an effort to minimize risks due to unforeseen subsurface conditions. Such studies could potentially consist of geophysical surveys, and/or subsurface borings.

4.0 REFERENCES

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Appendix A

Site Visit Photos

Figure A1. Plan of Appendix Photograph Locations

Figure A2. View of Lake Bottom

Figure A2 Discussion

Figure A2was taken near the eastern extent of Outlet Bay. The view shows the presence of fine-grained soil with occasional gravel and cobble sized particles.

Figure A3. View of Lake Bottom and Southern Lake Shore

Figure A3 Discussion

The photograph presented in Figure A3 was taken in the eastern portion of Outlet Bay. The view shows the southern bank of the bank and the lake bottom. Note the lack of visible rock on the lake shore. Fine grained soil with occasional gravel and cobble are visible on the lake bottom.

Figure A4. View of Lake Bottom and Southern Lake Shore

Figure A4 Discussion

Figure A4 is a photograph taken in the eastern portion of Outlet Bay. Fine-grained soil and occasional woody debris (logs and branches) are visible on the lake bottom. No rock is visible along the southern bank of the lake in upper field of view.

Figure A5. View of Lake Bottom and Southern Lake Shore

Figure A5 Discussion

The photograph depicted in Figure A5 was taken in the eastern portion of Outlet Bay. The lake bottom is comprised of silt, sand, and gravel with occasional boulders up to about 18 inches as seen in the left of the field of view. No rock is visible in the south bank of the shoreline.

Figure A6. View of Lake Bottom and Southern Lake Shore

Figure A6 Discussion

Figure A6 is a photograph taken in the eastern portion of Outlet Bay. The lake bottom is comprised of gravel with cobbles and boulders to approximately 16 inches (center of field of view). Cobbles and boulders are rounded to sub-rounded, suggesting they have been transported to their current location. No rock was observed in the south shoreline seen in the upper field of view.

Figure A7. View of Lake Bottom and Southern Lake Shore

Figure A7 Discussion

The photograph in Figure A7 was taken in the eastern portion of Outlet Bay. The lake bottom is comprised of rounded to sub-rounded gravel, cobbles and boulders up to approximately 30 inches in diameter (Lower center in field of view). Rounding suggest that the material is not native to this location. No rock was observed in the southern bank of the shoreline (upper field of view).

Figure A8. View of Lake Bottom and Southern Lake Shore

Figure A8 Discussion

Figure A8 depicts a photograph taken in the eastern portion of Outlet Bay. They lake bottom is comprised primarily of cobbles with gravel and boulders up to approximately 18 inches in diameter (lower-center field of view). Rounding suggest that the material is not native to this location. No rock was observed in the southern bank of the lake (upper left in field of view).

Figure A9. View of Lake Bottom and Southern Lake Shore

Figure A9 Discussion

The photograph in Figure A9 was taken in the eastern portion of Outlet Bay. The lake bottom is comprised primarily of sub-rounded cobbles with occasional boulders up to 12 inches (lower left in field of view). Rounding suggest that the material is not native to this location. No rock was observed in the southern bank of lake (upper field of view).

Figure A10. View of Lake Bottom and Southern Lake Shore

Figure A10 Discussion

Figure A10 is a photograph taken in the eastern portion of Outlet Bay. The lake bottom consists of rounded to sub-rounded gravel, cobbles, boulders (up to approximately 14 inches in lower right field of view) with silt. Rounding suggest that the material is not native to this location. No rock was observed in the southern bank of lake (upper field of view).

Figure A11. View of Lake Bottom and Southern Lake Shore

Figure A11 Discussion

The photograph in Figure A11 was taken in the central portion of Outlet Bay. The lake bottom is comprised of rounded to sub-rounded gravel, cobbles, and boulders up to approximately 36 inches (left field of view). Rounding suggest that the material is not native to this location. No rock was observed in the southern bank of lake (upper field of view).

Figure A12. View of Lake Bottom and Southern Lake Shore

Figure A12 Discussion

The photograph in Figure A12 was taken in the central portion of Outlet Bay. The lake bottom is comprised of rounded to sub-angular gravel, cobbles, and boulders up to approximately 24 inches (left-central field of view). Rounding suggest that the material is not native to this location. No rock was observed in the southern bank of lake (upper field of view).

Figure A13. View of Lake Bottom and Southern Lake Shore

Figure A13 Discussion

Figure A13 is a photograph taken in the central portion of Outlet Bay. The lake bottom is comprised of silt with rounded to sub-rounded gravel, cobbles, and boulders up to approximately 30 inches (lower left in field of view). Rounding suggest that the material is not native to this location. No rock was observed in the southern bank of lake (upper field of view).

Figure A14. View of Lake Bottom and Southern Lake Shore

Figure A14 Discussion

The photograph in Figure A14 was taken in the portion of Outlet Bay. The lake bottom is comprised of silt with rounded to sub-rounded gravel and cobbles. Rounding suggest that the material is not native to this location. No rock was observed in the southern bank of lake (upper field of view).

Figure A15. View of Lake Bottom

Figure A15 Discussion

Figure A15 is a photograph taken in the central portion of Outlet Bay. The lake bottom is comprised of silt with rounded to sub-rounded gravel and cobbles. Rounding suggest that the material is not native to this location. A near-vertical edge is visible in the upper field of view, which may suggest a submerged slab or grouted riprap. A large branch is present in the center of field of view.

Figure A16. View of Lake Bottom and Southern Lake Shore

Figure A16 Discussion

The photograph in Figure A16 was taken in the western portion of Outlet Bay. The lake bottom is comprised of silt with rounded to sub-rounded gravel, cobbles, and boulders (up to approximately 12 inches) in the foreground. Some sub-angular to angular boulders (up to 4 feet) can be seen in the left-center field of view. Rounding suggest that the material is not native to this location. Sub-angular to angular boulders would suggest that they are located near the parent source of the material. No rock was observed in the southern bank of lake (upper field of view).

Figure A17. View of Lake Bottom and Southern Lake Shore

Figure A17 Discussion

Figure A17 is a photograph taken in the western portion of Outlet Bay. The lake bottom is comprised of rounded to sub-rounded gravel, cobbles, and boulders up to approximately 30 inches (left in field of view). Rounding suggest that the material is not native to this location. No rock was observed in the southern bank of lake (upper field of view).

Figure A18. View of Lake Bottom

Figure A18 Discussion

Figure A18 shows a photograph that was taken in the western portion of Outlet Bay. The lake bottom is comprised of rounded to sub-rounded gravel and cobbles. Rounding suggest that the material is not native to this location.

Figure A19. View of Lake Bottom

Figure A19 Discussion

The photograph in Figure A19 was taken in the western portion of Outlet Bay. The lake bottom is comprised of rounded to sub-rounded gravel and cobbles, with occasional sub-rounded to sub-angular boulders up to approximately 18 inches (upper field of view). Rounded to sub-rounded particles suggest that the material is not native to this location. Sub-angular to angular boulders would suggest that they are located relatively closer the parent source of the material.

Figure A20. View of Lake Bottom and Southern Lake Shore

Figure A20 Discussion

Figure A20 is a photograph taken in the western portion of Outlet Bay. The lake bottom is comprised of rounded to sub-rounded gravel, cobbles, and boulders up to approximately 12 inches (lower field of view). Rounding suggest that the material is not native to this location. No rock was observed in the southern bank of lake (upper field of view).

Figure A21. View of Lake Bottom

Figure A21 Discussion

The photograph in Figure A21 was taken in the western portion of Outlet Bay just upstream of the existing outlet structure. A wooden structure is present on the base of the lake bottom. It is postulated that this structure could have been a bridge that was used prior to the construction of the existing outlet structure.

Figure A22. View of Lake Bottom and Norther Lake Shore

Figure A22 Discussion

The photograph in Figure A22 was taken in the western portion of Outlet Bay. View of the lake bottom is obstructed by ripples; however, it is comprised of rounded to sub-rounded gravel and cobbles. No rock is visible in the northern shore of the lake (upper field of view). Some lake front property lots include retaining walls, which presumably are present to support soil at this location.

Figure A23. View of Lake Bottom and Northern Lake Shore

Figure A23 Discussion

Figure A23 is a photograph taken in the western portion of Outlet Bay. Faint outlines of rounded to sub-rounded gravel and cobbles can be seen on the lake bottom. Rounding suggest that the material is not native to this location. No rock is visible along the northern shore of the lake (upper field of view).

Figure A24. View of Northern Lake Shore

Figure A24 Discussion

The photograph in Figure A24 was taken in the central portion of Outlet Bay. The lake bottom is not clearly visible due to the water depth and light reflection. No rock is visible in the northern shore of the lake (upper field of view).

Figure A25. View of Western Extent of Outlet Bay

Figure A25 Discussion

Figure A25 is a photograph that was taken in the central portion of Outlet Bay looking west. No rock can be seen in the southern (upper left and central field of view) or the northern (far right field of view) lake shores. Old timber piling can be seen in the lake in the center of the field of view. The piling suggest that enough soil is present at that location to support the pile at this location. Further, it would suggest that the substrate is soft enough to allow piles to be driven into the ground at this location.

Figure A26. View of Lake Bottom

Figure A26 Discussion

The photograph in Figure A26 was taken in the eastern portion of Outlet Bay. The lake bottom is comprised of rounded to sub-rounded gravel, cobbles, and occasional boulders up to approximately 18 inches (upper right in field of view). Rounded to sub-rounded particles suggest that the material is not native to this location.

Figure A27. View of Lake Bottom and Northern Shore

Figure A27 Discussion

Figure A27 is a photograph taken in the eastern portion of Outlet Bay. The lake bottom is comprised of sand with silt and rounded to sub-rounded gravel, cobbles, and boulders up to approximately 24 inches (center-left in field of view). Rounding suggest that the material is not native to this location. Sub-rounded boulders up to 5 feet can be seen in the northern shoreline, although no rock was observed at this location (upper field of view).

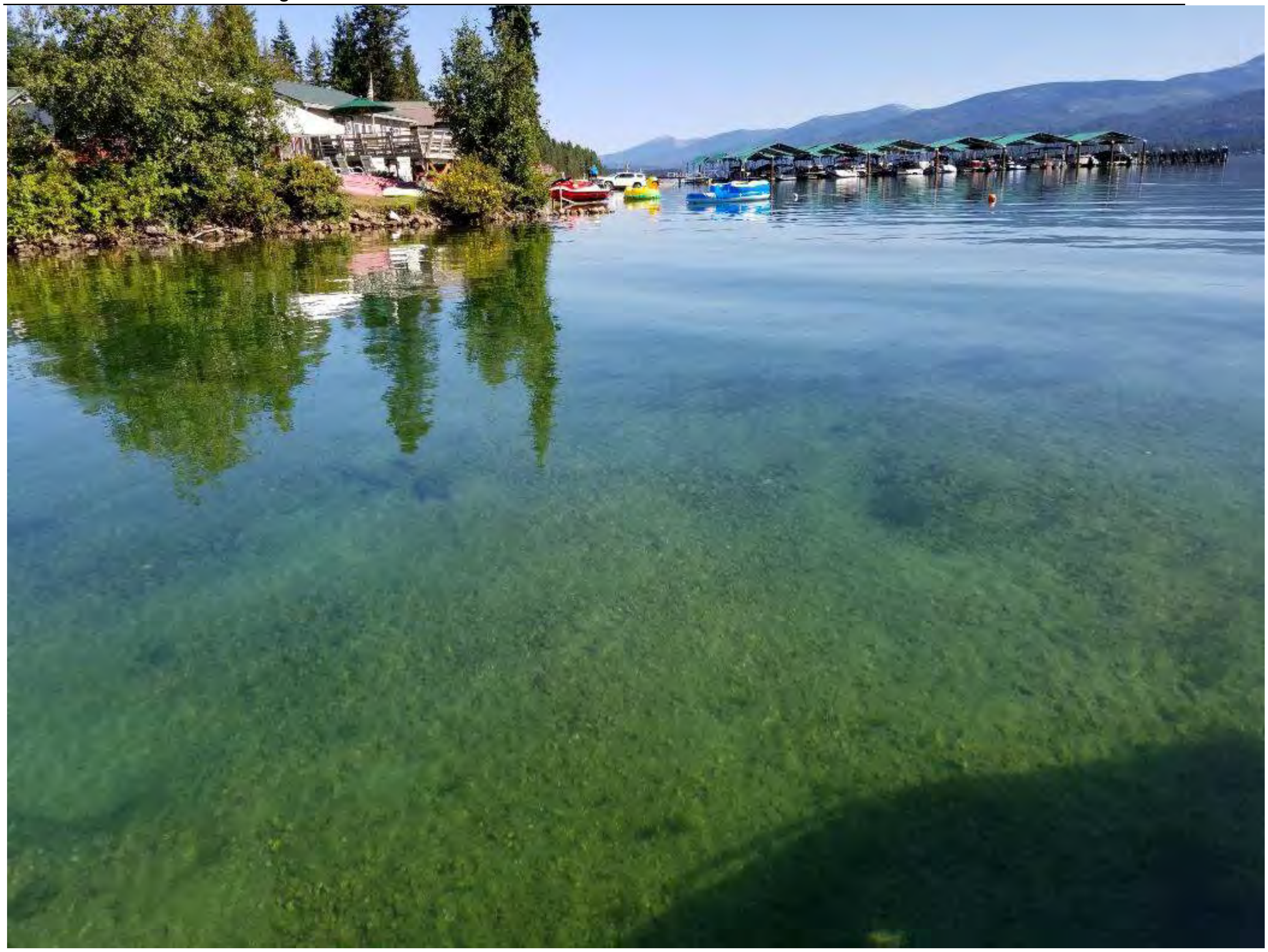

Figure A28. View of Lake Bottom and Northern Shore

Figure A28 Discussion

Figure A28 presents a view of the eastern portion of Outlet Bay. The lake bottom is comprised of sand, silt and gravel. Sub-rounded boulders up to 5 feet can be seen in the northern shoreline, although no rock was observed at this location (upper-left field of view).

Appendix C

Cost Estimates TM

Technical Memorandum

To:	Kiira Siitari	Project:	Priest River Cold-Water Augmentation
From:	Matt McDowell	Cc:	File
Date:	November 22, 2019	Job No.:	18-090
Subject:	Conceptual Cost Estimate		

1.0 INTRODUCTION

1.1 PURPOSE

The purpose of this technical memorandum (TM) is to present the conceptual cost estimate for each alternative developed for the Priest River Cold-Water Augmentation project. The TM presents the additional cost factors, a comparison table of the concept design construction cost estimate, a present value of operation and maintenance cost, a life cycle cost analysis, and a detailed cost estimate for each alternative. The objective is to support the selection of the preferred alternative to take forward.

As part of this study four alternatives were selected through initial screening. This TM presents the life cycle cost of those alternatives, which are:

- Alternative 1A – 45-71 CFS Gravity System
- Alternative 1B – 45-105 CFS Gravity System
- Alternative 2 – Siphon System
- Alternative 3 – Groundwater Well System
- Alternative 4 – Pump Station

1.2 BACKGROUND

Priest Lake is located in Bonner County, in the northern panhandle of Idaho. Priest Lake is approximately 18 miles long running from north to south, and is drained by the Priest River, which is a major tributary of the Pend Oreille River. Streamflow in the Priest River is regulated by the Outlet Dam located at the southwestern end of Priest Lake. Originally built in 1950, the first Priest Lake Outlet Dam was later replaced by a concrete gravity dam with construction completed in 1978. The Outlet Dam is owned and operated by the Idaho Department of Water Resources (IDWR). Dam operations in the summer months are manipulated to maintain lake levels for summer recreation and to provide minimum instream flows downstream of the dam.

Recent stream temperature modeling, led by the Kalispel Tribe of Indians, was conducted in Priest Lake and the Priest River. The modeling confirmed the thermodynamic feasibility of reducing late summer stream temperatures in the Priest River by adding water to Priest River flows that originates in the lower portion (i.e. hypolimnion) of the thermally-stratified Priest Lake. The model considered input flow rates between 30 cfs and 400 cfs and predicted that if 75% of stream inflow came from the hypolimnion, the upper 30 miles of the Priest River could cool between 2 and 10 °C, which would be enough to improve native salmonid habitat conditions and meet the Idaho Department of Environmental Quality (IDEQ) criteria for cold-water aquatic life during August and September (less than 19°C).

2.0 CONCEPTUAL COST ESTIMATE

This Section presents the estimate class selection of the cost estimate for this project, the additional cost factors, construction cost estimate, a present value of operation and maintenance cost, and a life cycle cost analysis.

2.1.1 Estimate Class Selection

The American Association of Cost Engineering (AACE) provides guidelines for development of cost estimates for various levels of project definition (see Table 2-1).

Table 2-1. American Association of Cost Engineering Guidelines

	Primary Characteristic	*Secondary Characteristic*			
ESTIMATE CLASS	LEVEL OF PROJECT DEFINITION Expressed as % of complete definition	END USAGE Typical purpose of estimate	METHODOLOGY Typical estimating method	EXPECTED ACCURACY RANGE Typical variation in low and high ranges (a)	PREPARATION EFFORT Typical degree of effort relative to least cost index of 1 (b)
Class 5	0% to 2%	Concept Screening	Capacity Factored, Parametric Models, Judgment or Analogy	L: -20% to -50% H: +30% to +100%	1
Class 4	1% to 15%	Study of Feasibility	Equipment Factored or Parametric Models	L: -15% to -30% H: +20% to +50%	2 to 4
Class 3	10% to 40%	Budget, Authorization, or Control	Semi-Detailed Unit Costs with Assembly Level Line Items	L: -10% to -20% H: +10% to +30%	3 to 10
Class 2	30% to 70%	Control or Bid/ Tender	Detailed Unit Cost with Forced Detailed Take-Off	L: -5% to -15% H: +5% to +20%	4 to 20
Class 1	50% to 100%	Check Estimate or Bid/Tender	Detailed Unit Cost with Detailed Take-Off	L: -3% to -10% H: +3% to +15%	5 to 100

Notes:
(a) The state of process technology and availability of applicable reference cost data affect the range markedly. The +/- value represents typical percentage variation of actual costs from the dost estimate after application of contingency (typically at a 50% level of confidence) for give scope.
(b) If the range index value of “1” represents 0.005% of project costs, then an index value of 100 represents 0.5%. Estimate preparation effort is highly dependent upon the size of the project and the quality of estimating data and tools.

Source: AACE International Recommended Practice No. 17R-97

For this project, Class 5 cost estimates have been prepared for each alternative; these are also called concept level estimates, as defined by AACE International. This level of estimate is deemed appropriate for the concept screening design level, which corresponds to a range of 0% to 2% level of design development. Class 5 cost estimates are prepared for several purposes, such as strategic planning, business development, project screening, alternative scheme analysis, confirmation of economic or technical feasibility, and preliminary budget approval.

Conceptual cost estimates have been prepared to assist with budget planning for the Priest River cold-water augmentation alternatives. Conceptual cost estimates are conservative estimates based on a conceptual design. Estimated construction costs represent a maximum range and likely cost reductions would be identified in future planning stages through analysis of alternatives and elimination of many uncertainties.

2.1.2 Additional Cost Factors

Additional cost factors are included in the conceptual cost estimates for medium to large capital projects which are complex enough to require bid documents or are likely to require a general contractor and multiple subcontractors. Those additional cost factors are as follow:

General Contract Requirements (15%): This factor includes mobilization/de-mobilization, temporary facilities, erosion control, special testing and other Division 1 contract requirements. It is applied to the construction cost subtotal including contingency. For Alternatives 1 and 2, which will require significant work on the lake, likely from a barge as pipe is laid on the lake bottom, and additional 10% of the construction cost subtotal was added. For Alternative 4, which also includes work from the lake, albeit much less, an additional 3% was added to the construction cost. For Alternative 3, the 15% base was used.

Overhead (6%) and Profit (12%): These factors cover general contractor overhead and profit. They are applied to the construction cost subtotal including contingency.

Bond Rate (0.75%): A bonding rate of 0.75% was assumed based on other projects of similar size and complexity.

State Sales Tax (6.0%): This factor covers the State of Idaho sales tax of 6.0% and a Bonner County sales tax of 0.0% for construction projects. On government projects, retail sales tax is to be paid on materials and equipment and is included in unit costs.

Design, Permitting and Construction Support (15%): This factor includes site surveys, geotechnical investigations, preliminary design, final design, permitting, and construction management, and management costs through project implementation. It is applied to the subtotal of the factors above.

2.1.3 Construction Cost Estimate

Table 2-2 provides a summary of the conceptual level costs in 2019 dollars for the Priest River cold-water augmentation alternatives. From the beginning of 2019 to the mid-point of construction, a 3% annual inflation factor should be added to the concept costs below. Note, inflation was not added for this conceptual cost estimate. An accuracy range of +100% and -50% (i.e. Class 5) is being used for this conceptual level cost estimate, per AACE guidelines. The detailed cost estimate for each alternative is presented in Attachment 1.

The construction cost data provided for this project are not intended to be the lowest cost for completing the work. Instead, the costs represent the median costs that would result from responsible bids received from qualified contractors. The unit costs provided for the Project were derived from several sources. Material costs for much of the work were obtained through budgetary quotes and project experience on projects of similar size and complexity previously performed by McMillen Jacobs. These unit costs were then adjusted to consider the Project location, site logistic challenges and other factors specific to the work on this project.

Table2-2. Concept Design Construction Cost Estimates

Line Item	Alternative 1A 45-71 CFS Gravity System	Alternative 1B 45-105 CFS Gravity System	Alternative 2 Siphon System	Alternative 3 Groundwater Well System	Alternative 4 Pump Station
Construction Subtotal	**$4,805,000**	**$5,078,000**	**$5,060,000**	**$425,000**	**$4,911,000**
Div 1 General Requirements Costs (%)	$1,201,250	$1,269,500	$1,265,000	$63,750 (15%)	$883,980
Overhead - 6%	$288,300	$304,680	$303,600	$25,500	$294,660
Profit - 12%	$576,600	$609,360	$607,200	$51,000	$589,320
Bond Rate – 0.75%	$36,038	$38,085	$37,950	$3,188	$36,833
Sales Tax – 6.0%	$288,300	$304,680	$303,600	$25,500	$294,660
Const. Cost	$7,196,000	$7,605,000	$7,578,000	$594,000	$7,011,000
Design, Permitting and Const Mgmt. - 15%	$1,079,400	$1,140,750	$1,136,700	$89,100	$1,051,650
Total Conceptual Cost - 2019 Dollars	**$8,276,000**	**$8,601,000**	**$8,715,000**	**$684,000**	**$8,063,000**
+100%	$16,552,000	$17,492,000	$17,430,000	$1,368,000	$16,126,650
-50%	$4,138,000	$4,373,000	$4,358,000	$342,000	$4,032,000

2.1.4 Operation and Maintenance Cost

Operations and maintenances (O&M) costs were also estimated for each alternative. O&M costs were estimated for two separate scenarios: 1) assuming continuous operations from July 1 through September 30, and 2) assuming continuous operations year-round.

The primary operations costs for each alternative were the costs of power to run pumps. For alternate 1B, it was assumed that the maximum flow rate of 105 cfs, the rest of the alternates were assumed that the maximum flow was 71 cfs required for cold water augmentation. For Alternative 1A & 1B, there are no pumping costs associated with the summer recreational period, and so the operational costs are zero ($0) for the first scenario. For the second scenario, the booster pump under Alternative 1A & 1B would only begin to operate when the gross head across the dam falls below the combined system head losses. This results in a gradual increase of pumping as discharge from the lake rises during spring runoff. To simplify the analysis, however, continuous operation at full capacity during the non-recreational period was assumed. Under Alternative 2, the siphon pump was assumed to operate for one (1) hour per day for each day of operation.

Maintenance costs were associated with pumps and other mechanical equipment, as well as annual tee-screen cleaning. Annual maintenance costs per pump were assumed to be $500 for the 5hp pump, $1,000 for the 47hp pump, $1,200 for the 80hp pump, $2,000 for the 150hp pump, and $3,000 for the 780hp pump. Maintenance costs for the water quality equipment of Alternative 3 were assumed to be $1,000 per year. Tee-screen cleaning was assumed to cost $1,500 per year for Alternatives 1A, 1B, 2, and 4.

Summer-only operations and maintenance costs for each alternative are presented in Table 2-3. Year-round operations and maintenance costs for each alternative are presented in Table 2-4. The detailed operation and maintenance costs are presented in Attachment 2.

Table 2-3. Summer Operations and Maintenance Costs (Annual)

Line Item	Alternative 1A Gravity System	Alternative 1B Gravity System	Alternative 2 Siphon System	Alternative 3 Groundwater Well System	Alternative 4 Pump Station
Operations[1]	$0	$0	$34	$148,186	$143,740
Maintenance	$2,000	$3,000	$3,000	$8,000	$3,500
Total Annual O&M Cost	$2,000	$3,000	$3,034	$156,186	$147,240

[1] Assumes continuous operation from July 1 through September 30, except for Alternative 2, which assumes 1 hour of operation per day over the same period.

Table 2-4. Year-Round Operations and Maintenance Costs (Annual)

Line Item	Alternative 1A Gravity System	Alternative 1B Gravity System	Alternative 2 Siphon System	Alternative 3 Groundwater Well System	Alternative 4 Pump Station
Operations[1]	$24,918	$39,087	$136	$587,910	$570,273
Maintenance	$2,000	$3,000	$3,000	$8,000	$3,500
Total Annual O&M Cost	$26,918	$42,087	$3,136	$595,910	$573,773

[1] Assumes continuous operation year-round, except for Alternative 2, which assumes 1 hour of operation per day over the same period.

2.1.5 Life Cycle Cost Analysis

In order to correctly compare the costs associated with each alternative, a life cycle cost analysis must be conducted. Life cycle cost analysis requires defining a project useful life, during which operational and maintenance costs are incurred. The useful life of the project was assumed to be 20 years, after which point any pumps and/or mechanical equipment would need replacement. In addition, only the summer operation and maintenance costs were utilized. For this analysis, the salvage value of each alternative was assumed to be zero ($0). The analysis was also conducted assuming a discount rate obtained from the U.S. Office of Management and Budget of 2.875 percent. Table 2-5 provides the results of the life cycle cost analysis. From the table, even though the capital cost of Alternative 3 is appreciably less than the other three alternatives, the operations and maintenance costs associated with this alternative drive the life cycle cost up significantly. Due to the large operating cost component, the actual life cycle cost of Alternative 3 will be closely tied to the cost of power of the life of the project.

Table 2-5. Life Cycle Cost Analysis

Line Item	Alternative 1A Gravity System	Alternative 1B Gravity System	Alternative 2 Siphon System	Alternative 3 Groundwater Well System	Alternative 4 Pump Station
Construction Cost	**$8,276,000**	**$8,601,000**	**$8,715,000**	**$684,000**	**$8,063,000**
Summer O&M Costs, Present Value	$30,355	$45,355	$46,053	$2,370,516	$2,234,744
Life Cycle Cost	**$8,306,355**	**$8,646,355**	**$8,761,053**	**$3,054,516**	**$10,297,744**

Attachment 1

Capital Cost Calculation Spreadsheets

Attachment 2

O&M Cost Calculation Spreadsheet