MOTT MACDONALD

Priest Lake Water Management Study Report

Priest Lake, Idaho

February 20, 2018

Idaho Water Resource Board

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Acronyms and Abbreviations

ADCP	Acoustic Doppler Current Profiler (current meter)
ASCE	American Society of Civil Engineers
cfs	Cubic feet per second
FEED	Front End Engineering Design
ft	Feet
D_{50}	Median Sediment Diameter
IDL	Idaho Department of Lands
IDFG	Idaho Department of Fish & Game
IDEQ	Idaho Department of Environmental Quality
IDWR	Idaho Department of Water Resources
IWRB	Idaho Water Resources Board
LIDAR	Light Detection and Ranging (laser altimetry and survey)
MCE	Maximum Credible Earthquake
USACE	United States Army Corps of Engineers
USFS	United States Forest Service
USGS	United States Geological Service

Executive Summary

The Priest Lake Water Management Study was initiated by the Idaho Water Resource Board (IWRB) to evaluate opportunities for improving operation of the Priest Lake and Priest River system in order to meet long-term management objectives. The purpose of the study was to evaluate the Lower Priest Lake (Lake) system and develop alternatives that preserve Idaho Code required lake levels, and provide continued navigable access from the Lake into the Thorofare during the recreational season. The study included the following requirements to be assessed:

- Evaluate alternatives for maintaining required lake levels and for maintaining current minimum discharge requirements of 60 cfs downstream from the outlet dam.
- Assess potential structural and operational modifications to the Priest Lake Outlet Dam.
- Analyze options to improve access and navigable conditions for the Priest Lake Thorofare.
- Adjustment to summer recreation season lake levels to be a contingency plan for forecasted dry years.

An extensive review of available data, reports, engineering documents and mapping was conducted throughout the execution of the study and then supplemented with limited new data collection and site assessments to provide the baseline of information for the study. Criteria for the evaluation were developed in coordination with the IWRB and stakeholders. Criteria included identification of the recreational season duration, water level management scenarios, Thorofare navigation requirements, minimum discharge requirements for the outlet dam, and minimum recreational season water level.

Stakeholder and Public outreach was conducted throughout the duration of the work. Outreach consisted of consulting with, and including interested and affected individuals, organizations, agencies, and governmental entities in the decision-making process. Stakeholder workshops, outreach meetings and interviews, public open houses, IWRB briefing meetings, social media briefing updates, press releases, mailers, fact sheets, and IDWR website briefing materials were conducted. The following major topics were identified in the comments received during the outreach:

- Water Levels: Changes in water level operations as a dry year contingency for less than a 6-inch water level raise are reasonable and supported to meet the study objectives.
- Thorofare Improvements: Improvements at the Thorofare, including a combination of structural improvements and dredging to provide sustainable access is supported to meet the study objectives.
- Outlet Dam Structure Modifications: Improve automation, redundancy, and flexibility in operations of outlet dam tainter gates to reduce risk and improve efficiency of management of water resource to meet study objectives. Conduct structural modifications to improve safety.

Engineering analysis and assessments were conducted for pool raise scenarios, outlet dam structure stability and hydraulics, and Thorofare hydrodynamic processes indicated the following:

- Pool Raise Assessment: A 6-inch raise in pool level as a dry year contingency for the summer recreational period is feasible to meet the project objectives and criteria without major impacts to shoreline beaches, habitat or infrastructure.
- Outlet Dam Structure Stability: A 6-inch pool raise would result in additional forces on the outlet dam and increased erosion potential for the downstream scour apron during critical flow conditions, thereby requiring some improvements to the structure.

- The proposed upgrades would allow the water surface level of Priest Lake to be raised by a few inches in dry years to ensure there is sufficient water storage in the lake to maintain flows into Priest River at the outlet dam. Lake operations would not change in normal water years.
- Thorofare Hydrodynamic Processes: The location of the Thorofare at the interface with the lake is complex with respect to the processes effecting the sedimentation and reduced navigability. Sedimentation was determined to be dominated by lake shore sediments transported through and around the porous (open) timber breakwater, combined with Thorofare hydraulic flow splitting due to the open breakwater structure, thereby reducing sediment transport capacity. Reconstructing a new solid (non-porous) breakwater would reduce sediment transported into the Thorofare and confine Thorofare flows to increase velocities and improve sediment transport capability.

Conceptual engineering design was conducted for the Thorofare and Outlet Dam Structure project areas based on the results of the analysis and assessment work. Recommended improvements are the following:

- Thorofare: Installation of a longer, solid type breakwater along a slightly rotated alignment and extended further into the lake. The breakwater would be constructed of armor rock, boulder/cobble, sheet pile or a combination thereof. Dredging of the Thorofare would be conducted to provide a 5-ft depth along the reach adjacent to the breakwater. Construction would need to be conducted with equipment mobilized to the site using barges.
- Outlet Dam Structure: Installation of a new concrete scour apron to replace the existing riprap, raising and strengthening of the tainter gate, and new gate operators and electrical controls. Improvements would increase operational efficiency, reduce risk of scour, and improve outlet dam structure stability.

The estimated cost of the improvements is $5 million for the combined outlet dam structure and Thorofare improvements. Additional data collection, engineering analysis and design, and regulatory permitting would be required during the next phase of the project.

1 Introduction

Mott MacDonald prepared this report for the Idaho Water Resource Board (IWRB) to summarize Priest Lake Water Management Study results. Priest Lake is in North Idaho and is a water body system comprised of the Upper Priest Lake, Lower Priest Lake, and Thorofare connecting the two lakes (see Figure 1-1). The study focuses on the Lower Priest Lake and Thorofare at the entrance to the lower lake.

Priest Lake water levels and discharges into Priest River are managed by an outlet dam structure located near Coolin, Idaho. Upper Priest Lake is a natural lake system unaffected by the operations of the outlet dam.

The purpose of the study is to evaluate the Lower Priest Lake (Lake) system and develop alternatives that preserve the lake levels required by Idaho regulations and provide continued navigable access from the Lake into the Thorofare during the recreational season.

This report provides a high-level summary of the study methodology, engineering assessments (Lake “Pool” Raise, Outlet Dam Structure Modifications, and Thorofare Improvements), conceptual engineering design and implementation strategy. The Appendices provide details on the analysis, evaluations, assessments of improvements and site conditions, as well as conceptual-level graphics and technical memorandums which are referenced throughout the body of this report.

Lower Priest Lake (hereafter referred to as Priest Lake) is approximately 18 miles long, has a maximum depth greater than 300 feet, and has active storage volume of approximately 120,000 acre-feet. It is connected to Upper Priest Lake (which is approximately 3.3 miles long) by a 3-mile long channel known as the "Thorofare" which has long been used by the public for recreation and access to the upper lake. A 1,400-foot-long timber breakwater at the north end of Priest Lake is intended to manage sediment transported through the Thorofare into Priest Lake, while providing wave and erosion protection to landowners at the north end of Priest Lake. This study addresses the outlet dam, Thorofare, and breakwater. The location and photographs of these features are shown in Figures 1-2 to 1-5.

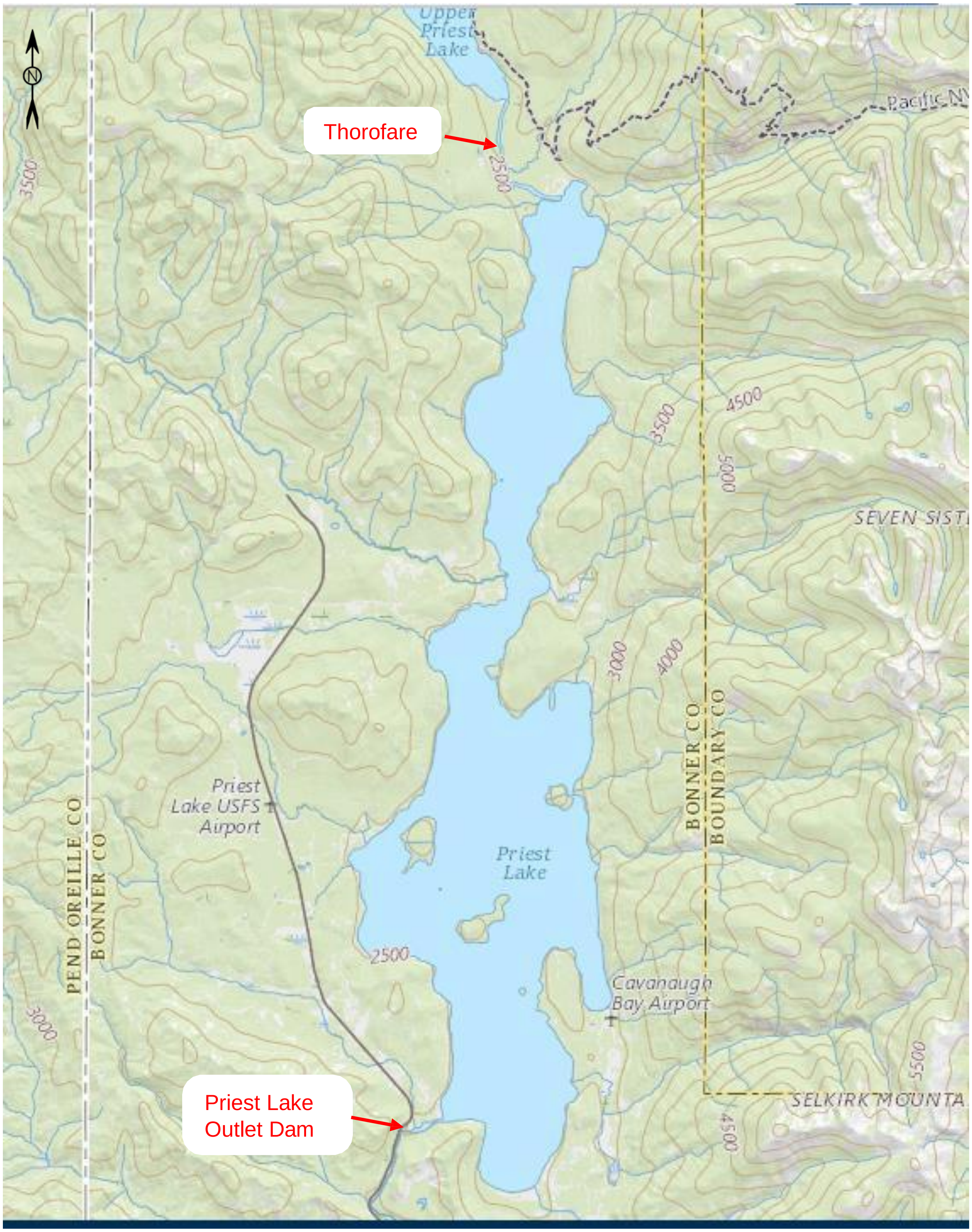
1.1 Study Background

The Priest Lake Water Management Study was initiated by the Idaho Water Resource Board (IWRB) to evaluate opportunities for improving operation of the Priest Lake and Priest River system in order to meet long-term management objectives. The study includes several action items:

- Evaluate alternatives for maintaining required lake levels and for maintaining current minimum discharge requirements of 60 cfs downstream from the outlet dam.
- Assess potential structural and operational modifications to the Priest Lake Outlet Dam.
- Analyze options to improve access and navigable conditions for the Priest Lake Thorofare.

The goal of the study is to develop a range of feasible alternatives in consultation with stakeholders and the public for improving Priest Lake Water Management.

Figure 1-1 – Area Map



Source: USGS Topo Map

1.2 Project Study Area Description

The outlet dam maintains lake levels in Priest Lake and manages downstream flow into Priest River. The dam is owned by the Idaho Department of Water Resources (IDWR) and operated by IDWR. The dam is approximately 12 feet high with gates that regulate discharge and does not have an emergency spillway.

Water levels in the lake are measured at the USGS outlet gage (#12393000) located in Outlet Bay near the dam. Lake level begins to rise in April and May during the spring runoff, reaching a maximum level of up to 5 feet (outlet gage level) in early June. The lake level recedes to roughly 3 feet in July and this level is maintained through the summer recreational season. Storage releases commonly start during the second week of October, but have started as early as October 4th and as late as October 16th. Storage releases normally end sometime in November. The gates remain open through the fall, winter, and early spring season to allow natural passage of flows through Priest Lake.

1.3 Brief History

Priest Lake is located on the northern Idaho Panhandle. Priest Lake Outlet Dam was first constructed in 1951. The existing dam was constructed in 1978 to replace the deteriorating original dam. The lake is a significant draw for tourism and recreation, both of which add economic benefit to Bonner County. The area is known for the pristine variety of wildlife, clear and clean water, and recreational opportunities. In 2015, limited water supply and drought conditions in northern Idaho made maintaining the required summer lake levels and downstream flow in the river challenging. This situation, coupled with concerns about the breakwater structure and reduced access through the Thorofare, increased interest in developing both operational and engineered improvements to the entire system.

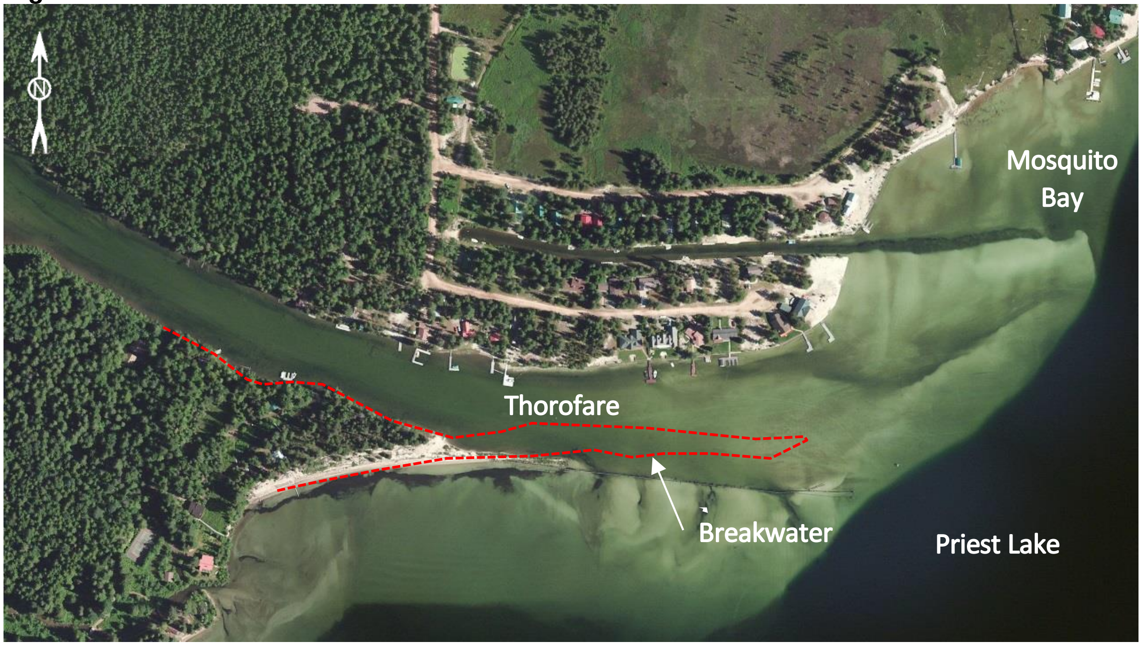
In response to area stakeholders' concerns, the IWRB authorized funding to perform an evaluation of strategies and options that could meet the long-term water management objectives for the Priest Lake system.

1.4 Study Goals & Objectives

The goal of the study is to develop a range of feasible alternatives in consultation with stakeholders and the public for improving Priest Lake water management. The goals and objectives for improvements at Priest Lake include the following:

- Preserving lake levels through the summer recreation season. This goal supports the local economy and meets current lake level requirements. It also minimizes negative impacts to downstream river flows.
- Maintaining vessel access through the Thorofare channel between Upper Priest Lake and Priest Lake.

Figure 1-2 – Thorofare Aerial Photo



Source: Google Earth

Figure 1-3 – Thorofare Oblique Aerial Photo (looking upstream)

Source: Photograph credit Sara K. Williams

Figure 1-4 – Outlet Bay

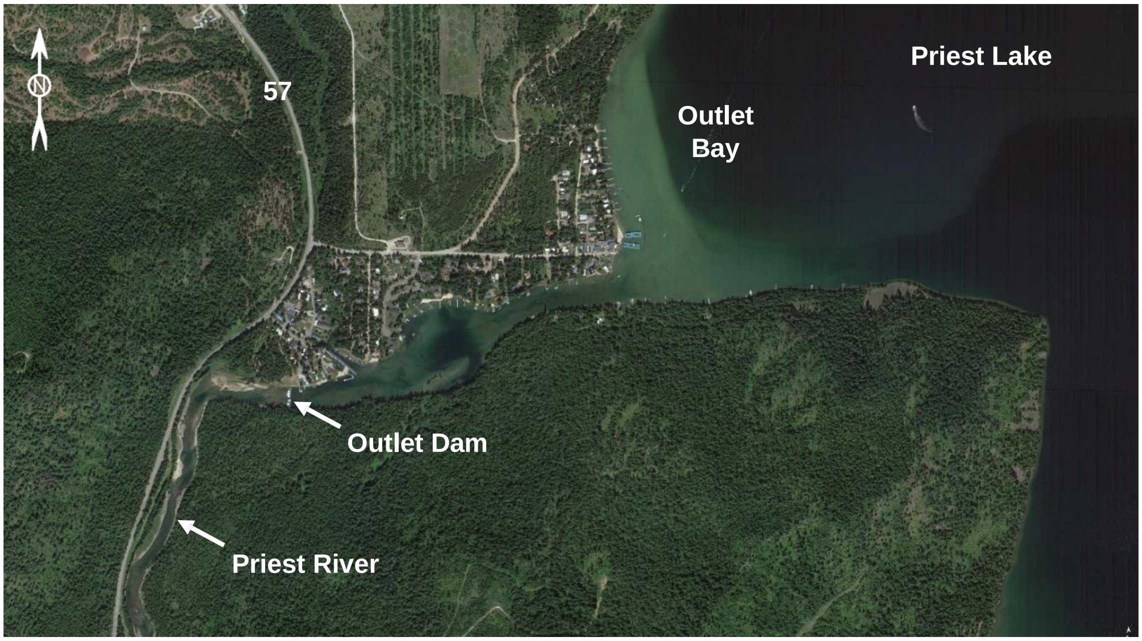


Figure 1-5 – Outlet Dam (looking upstream)

2 Study Methodology

2.1 Introduction

The study was completed in steps, beginning first with a review of existing information, compilation of a project database, and identification of data gaps. Critical data gaps were filled through limited new field data collection, desktop research, field observations, and information provided by stakeholders. Assessment of fish resources and habitat were conducted to inform the study criteria development and evaluation of alternatives. A public and stakeholder outreach program was conducted to acquire input throughout execution of the study. Criteria were then developed to guide the approach for conducting the assessment and the corresponding development and evaluation of improvement alternatives.

2.2 Existing Data Review

A review of available data, reports, engineering documents and mapping was conducted throughout the execution of the study. Outreach for data collection included stakeholders and the public during outreach, state and federal agencies, internet databases, museums, and the consultant team prior project databases. The following summarizes the type of data collected for conducting the study:

- Outlet Dam Operations
- Priest Lake Water Levels
- Mapping, GIS Data, and Lidar
- Outlet Dam Condition Assessments & Inspections
- Lake Sediments
- Erosion Assessments
- Land Use
- Engineering Analysis & Design Documents
- Hydrologic Data & Analysis
- Water Quality Data & Assessments
- Fish Resources & Habitat

A summary of these data types and a corresponding bibliography of sources are provided in Appendix A.1.

2.3 Data Gaps Assessment

The following were identified as data gaps in our assessment:

- Thorofare Hydrographic Conditions: Very limited to no historical hydrographic survey data was available for the Thorofare. A new hydrographic survey was recommended and conducted to develop a base map for hydrodynamic analysis and conceptual engineering design. New data collection was focused within the Thorofare area adjacent to and near the breakwater at the interface between the Thorofare and Priest Lake.

- Thorofare Water Level & Discharge: Limited water level and discharge information was available for the Thorofare. Water level and discharge data for the Thorofare was recommended to be collected during the hydrographic survey work. A correlation of water level and discharge in the Thorofare relative to lake level and outlet dam discharge was then conducted utilizing available data from common time periods. Hydrologic data was provided by IDWR staff for the watershed draining to Priest Lake and the Thorofare.

- Dredging Records: Anecdotal descriptions of historical dredging actions were provided regarding era (decade), frequency and method of dredging. Documentation, details, drawings, or surveys for historical dredging work were not available.

2.4 New Data Collection

Collection of new data was conducted to supplement the compiled existing data to provide a combined data set for conducting the study and included the following:

- Field Reconnaissance: A field reconnaissance to review the existing project site conditions along the Lake shoreline, Outlet Dam structure, and Thorofare was conducted on May 10, 2017. The field reconnaissance included observations of infrastructure, vegetation, existing habitat, substrates, and geomorphologic conditions (sediment processes and shoreline and channel substrate). The field reconnaissance was conducted in coordination with the new data collection work to assist in finalizing the areas for new data collection, and identify potential opportunities and risks prior to conducting detailed analysis.
- Topographic/Bathymetric Surveying: A combination of survey grade equipment were employed to collect the new survey data within the Thorofare entrance vicinity to develop a comprehensive base map of data to aid the hydrodynamic analysis, evaluation of alternatives, and conceptual design work. The survey area included the Thorofare, adjacent lakeshore beaches, and breakwater structure. Depths were determined to be as shallow as 18 inches (summer pool 3.0-foot gage level) at many areas of the entrance to the Thorofare. The survey also showed the presence of a number of distributary channels through the breakwater structure rather than a single deeper outlet channel.
- Thorofare Discharge & Water Levels: During the hydrographic survey work, a recording water level gage was installed within the Thorofare to provide a temporal record of the lake and Thorofare water surface elevation during the execution of the survey work. Thorofare discharges were estimated from collection of acoustic doppler profiler equipment (ADCP) conducted for the day of the survey.
- Sediment Grab Samples: Collection of seven bulk sediment samples were conducted within the Thorofare vicinity and sieve analysis was performed to obtain a physical characterization of sediments to aid in the geomorphologic and habitat assessment work. Samples were collected throughout the Thorofare study area. Sediments were determined to be of uniform size throughout the Thorofare study area and characterized as clean fine to medium grained sand.

A detailed reporting of the new data collection is provided in Appendix A.2.

2.5 Basis of Analysis & Criteria Development

The following criteria guided the approach for development of improvement alternative concepts and provided a framework for evaluation and prioritization of alternatives. Additional information on the basis of analysis and criteria are provided in Appendix A.3.

- Basis of Analysis
    - Lake level management (maintaining a 3-foot level at the outlet gage #1249000) for the recreational season and developing operational strategies that improve habitat and minimize shoreline impacts in accordance with Idaho Statute 70-507.

  - Maintain current minimum discharge flow requirements downstream of dam (60cfs).
  - Provide sustainability for the Thorofare (promoting self-sustaining improvements to improve Thorofare access, navigability, and water quality).

- Study Criteria Summary
  - Water Management Code Requirements (Idaho 70-507): No change in outlet dam operations outside of the study period to maintain pool levels in accordance with current code and historical operational procedures.
  - Engineering Standards: Utilize industry standards for the type of work being conducted such as U.S. Army Corps of Engineers, U.S. Bureau of Reclamation, and Idaho Department of Water Resources.
  - Property Ownership: Improvements limited to public lands.
  - Water Levels: Water information was obtained from available sources. The summer recreational pool level is 3.0 feet on the USGS gage #12393000 under existing and historical conditions. Consider water level changes (temporary pool raise) up to 6 inches higher as part of the study. Pool raise considerations for dry year contingency plan only; not considered for regular, normal, or wet-year operations.
  - Time Period for Pool Raise: The time period of pool raise assessment for outlet dam operational changes is June through early October.
  - Water Level Management Evaluation Scenarios: Water years of 2001 and 2005 were selected as representative dry years to evaluate pool raise scenarios for improved water management schemes to meet the study objectives. The 2015 drought year was not selected as a complete dataset was not available for that year.
  - Pool Raise Assessment: Consideration for basement flooding of existing properties, beach erosion, recreational beach use, boat moorage structures, ecosystem and boat launch facilities will be provided in the evaluation of pool raise effects.
  - Recreational Season: The time period is July 1 to the first weekend in October (October 8) for purposes of this study. Actual end date varies year by year, but is typically in the first 10 days.
  - Thorofare Navigation
    - Vessel Size: Less than 26-feet trailerable type vessel.
    - Navigable Depth: Desired to be minimum of 3 feet gage datum, (at summer recreational pool level) with 4-foot and 5-foot depths to be considered.
    - Width: Two-way vessel traffic with navigation aids.
  - Outlet Dam
    - Size classification is large per IDAPA 37.03.06 and hazard classification is significant per Idaho Dam Safety code I.C.42-1711(q) requirements.
    - Minimum discharge of 60 cfs from outlet dam.
  - Regulatory Permitting
    - Work Window: Likely to be allowed year-round for the Thorofare and Outlet Dam locations.

2.6 Public Outreach Plan

Public outreach was identified as a critical element of the study process to gain input throughout the duration of the work. The public outreach included interested and affected individuals, organizations, agencies, and governmental entities in the decision-making process. In addition to providing information, the Study Team solicited responses regarding the stakeholders and public's needs, values, and evaluations of alternatives.

The following outreach groups were identified for the development of the public outreach plan:

- Policy Group – Idaho Water Resources Board, Idaho Department of Water Resources & Bonner County.
- Study Team – Mott MacDonald and Policy Group.
- Steering Committee – Local advisory group assembled by County and IWRB.
- Stakeholder Groups – Local resident and government agency representatives.
- General Public – Local landowners, residents, and broader public.

Representatives for each group were identified and included in the outreach planning and execution process. The Study Team identified the following objectives for the public outreach as part of the study process:

- Inform the public regarding the process for and progress of study development.
- Provide opportunities for stakeholders and the general public to provide input during the study process.
- Incorporate stakeholder and public input regarding issues of concern.
- Document stakeholder recommendations.
- Prepare and distribute background and technical information.
- Develop and demonstrate stakeholder support and understanding regarding the study and outcomes.
- Monitor outreach activities for effectiveness and adjust the approach as necessary during implementation.
- Focus on quality outreach activities.

Steering committee workshops, stakeholder outreach meetings and interviews, public open houses, IWRB briefing meetings, social media briefing updates, press releases, mailers, fact sheets, and IDWR website briefing materials were activities and tools used to conduct the public outreach. A summary of the public outreach plan is provided in Appendix A.5 and public open house comments in Appendix A.6.

The following were identified as major topics reflected in the comments received during the stakeholder and public outreach:

- Water Level Operations: Modified lake level (pool raise of less than 6 inches) for a dry year contingency plan is reasonable and supported to meet the study objectives. Change in operations to provide a yearly pool raise or a larger pool raise (greater than 6 inches) of operations could be a concern for recreational use of properties and potential for impacts.

- Thorofare Improvements: Improvements at the Thorofare including a combination of structural improvements and dredging to provide sustainable access is supported to meet the study objectives.

- Outlet Dam Structure Modifications: Improved automation, redundancy, and flexibility in operations of outlet dam gates to reduce risk and improve efficiency of management of water resource and outlet dam stability improvements is supported to meet study objectives.

3 Pool Raise Assessment

3.1 Introduction

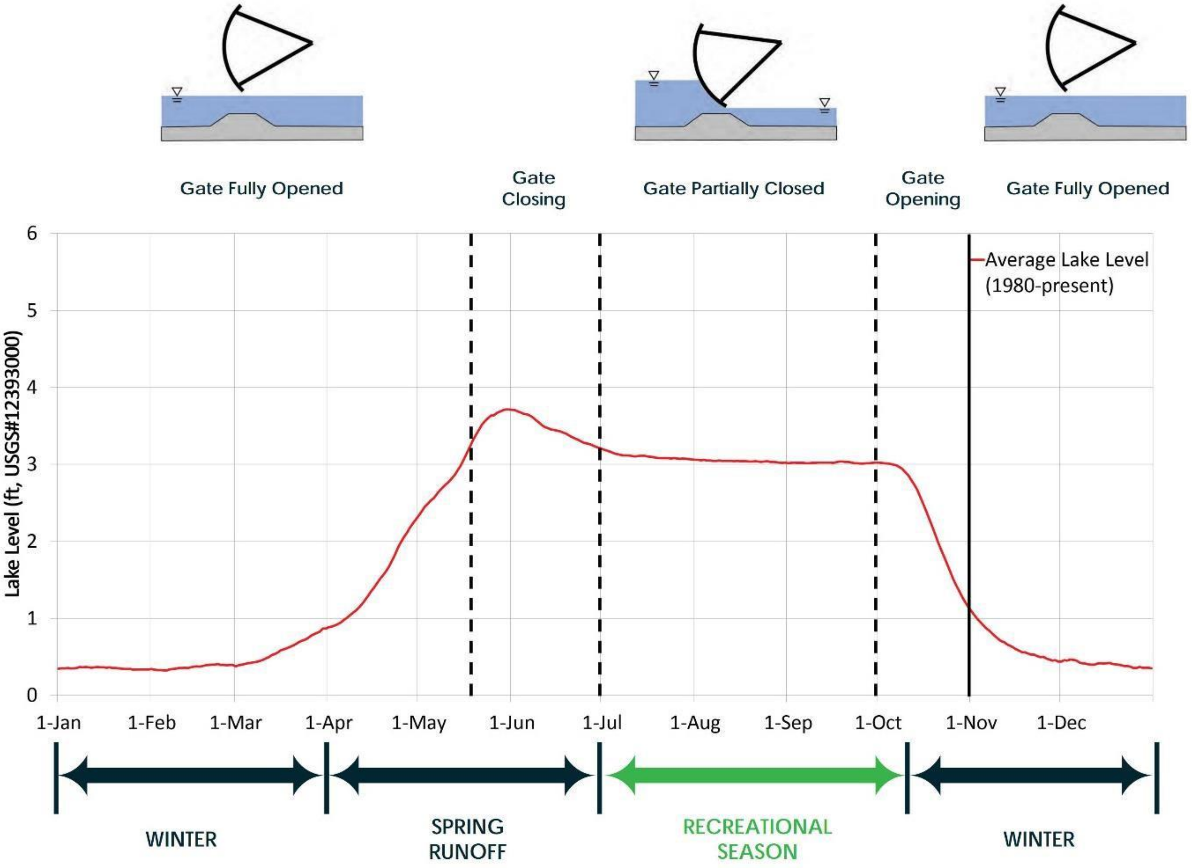
A goal of the study was to evaluate alternatives for maintaining required Priest Lake water levels (3.0 feet on USGS lake gage) during the summer recreational period and to maintain discharge requirements of 60 cfs downstream from the dam. These conditions were of particular interest during the summer and early fall low flow time period during dry years. In recent years, meeting these requirements were problematic, and in some cases for short periods of time were not achieved. These included calendar years 2015 and 2016. The assessment included an evaluation of existing dam and lake operations, hydrologic analysis, and pool raise assessment.

3.2 Existing Outlet Dam Structure & Lake Management Operations

Priest Lake water levels are managed by the outlet dam structure (see Section 4 for a detailed description) located in Outlet Bay near Coolin (see Figure 1-4). Eleven tainter gates are used to manage the lake levels throughout the summer recreational season to ensure the required 3.0-foot gage water level is maintained. Operation of the outlet dam varies throughout the year, as described in Figure 3-1, and as follows:

- Mid-Fall, Winter, Mid-Spring – Gates are fully open and lake level fluctuates unimpeded by outlet dam.
- Mid-Spring to Early October – Gates are operated as required to manage water to achieve a water level of 3.0 feet on the lake gage between July 1 and October 8.
- Early October to Mid-Fall – Gates are operated as required to effectively and safely discharge Priest Lake reservoir storage.

Figure 3-1 – Gate operation relative to seasonal water levels



The existing outlet dam was constructed in 1978 to replace the original deteriorating dam built in 1950-1951. The dam was replaced to improve operations with tainter gates in lieu of timber stoplogs to better manage the water resource through improved control over discharges from the dam. Historical lake levels were reviewed to better understand yearly operation of the dam and lake water levels. The analysis focused on years starting in 1980 which represented the water management schemes put in place since the time of dam reconstruction in 1978. A summary of the water level and discharge analysis is described in Figure 3-2.

3.3 Pool Raise Concept Assessment

An assessment of historical water level management, review of previous water management hydrologic studies, and outreach to stakeholders was conducted to aid in the development of pool raise concepts for further evaluation in the hydrologic analysis work. The following is a summary of the assessment results:

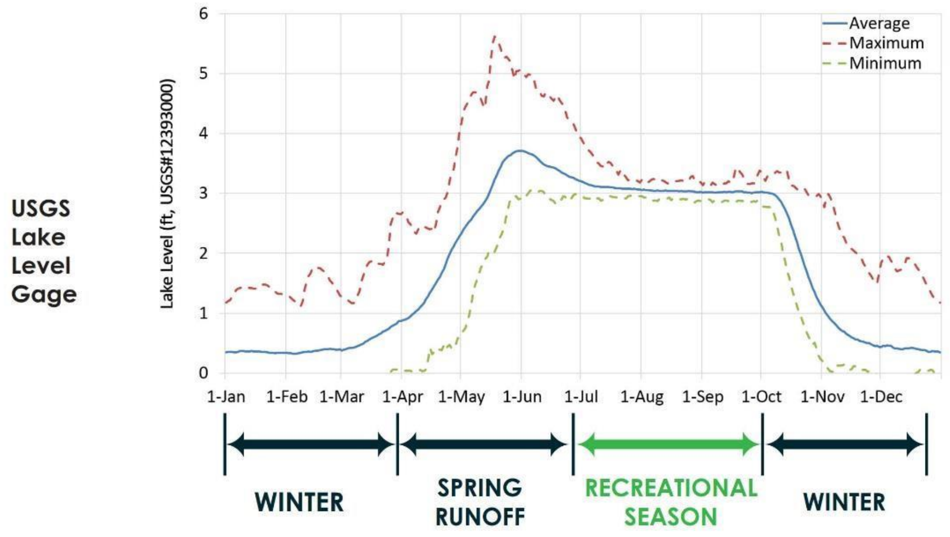
- Historical Water Level Data Review – Review of Priest Lake water levels indicated maximum water levels during the recreational season could routinely be at 3.25 feet (3 inches higher than the summer recreational pool level). A 6-inch higher water level (above recreational pool level) is not uncommon during early July. A summary of the assessment is described in Figure 3-2.

- Prior Hydrologic Study Review – Prior studies conducted by IDWR (1977) and the U.S. Army Corps of Engineers (USACOE, 1992) analyzed the feasibility of raising the Priest Lake level by up to 6 inches for the purpose of increasing storage to provide the minimum discharge of 60 cfs from the dam during dry years. Although these studies were not conducted for the same goal and had different criteria, the results provided a baseline to assist in developing the parameters for pool raise level concepts. A 3- and 6-inch pool raise during the recreational season was determined to be a reasonable range of alternatives to consider for further evaluation in the hydrologic analysis for this study.

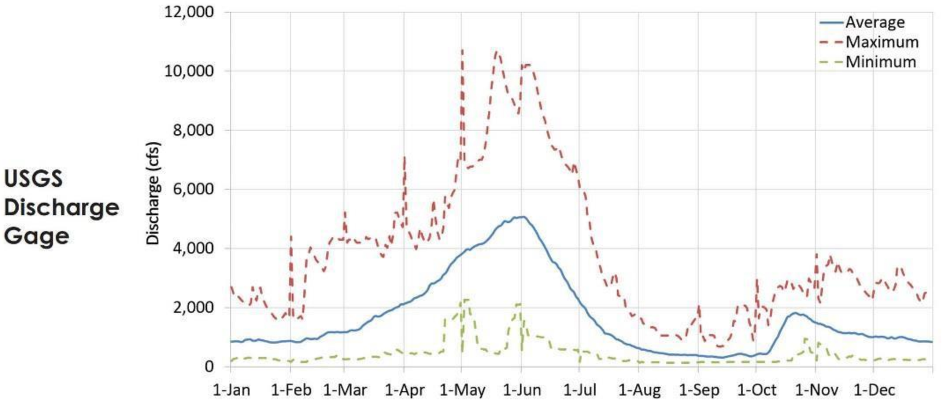
- Stakeholder Outreach – Information obtained during the outreach efforts indicates pool raise should not be more than 6 inches to not have effects on shoreline properties (beaches, infrastructure, etc.). Preference was a 6-inch increase early in the recreational season with drawdown to a 3-inch higher level during late summer.

Figure 3-2 – Historical Priest Lake Water Level and Discharge Charts

ANNUAL LAKE LEVEL VARIATION (1980 - PRESENT)

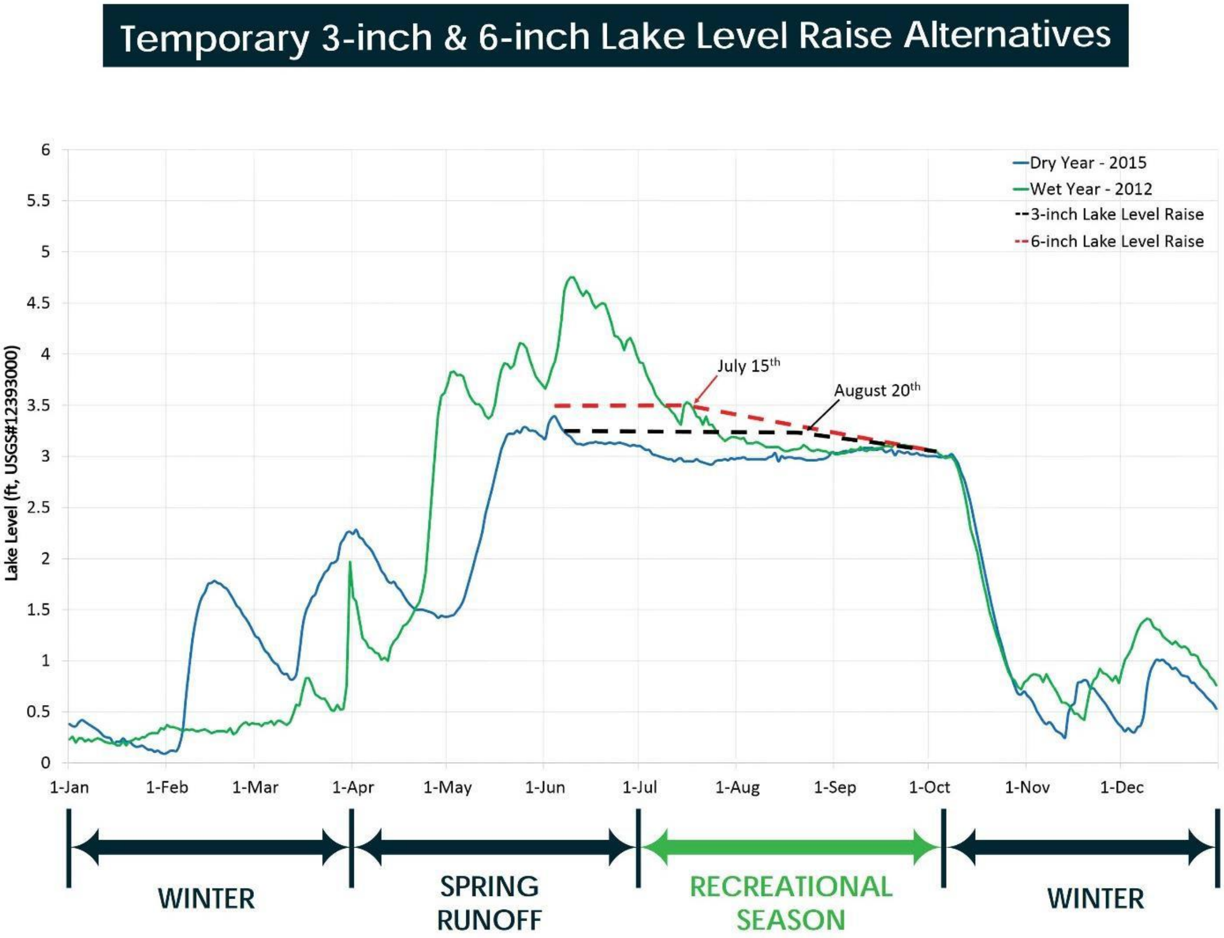


ANNUAL DISCHARGE PATTERN (1980 - PRESENT)



Based on the results of the assessment, it was determined the hydrologic analysis would focus on pool raise scenarios of 3 and 6 inches higher than the summer recreational season water level (3.0-foot gage level). A graphical representation of the two scenarios is described in Figure 3-3.

Figure 3-3 Conceptual Graphic Representing Pool Raise Concepts for Evaluation



3.4 Hydrologic Analysis

3.4.1 Introduction

A hydrologic analysis was conducted to develop a water management analysis tool for Priest Lake and the outlet dam. A detailed summary of the analysis and results are described in Appendix B.3 – Water Level Management Hydrologic Analysis. Existing data was compiled from IDWR and other publicly-available data sources to develop a lake simulation model using the HEC-ResSim software (USACE, 2013). The model was used to evaluate different components of the Priest Lake water balance (inflows, lake level, discharge) for identified water management scenarios.

3.4.2 Analysis

Development of the model required establishment of a lake-volume rating curve, outlet dam stage-discharge rating curve, Priest Lake basin hydrologic conditions for inflows from major tributaries, and the development of water management simulation scenarios. The Mott MacDonald Team relied on existing data and prior analysis conducted by IDWR hydrology staff, USGS, and USFS for establishing the hydrologic input parameters. Modeling scenarios and criteria were developed in coordination with IDWR and are described in Appendix B.1 – Pool Raise Scenarios Memorandum. Criteria used for conducting the analysis are outlined in Table 3-1.

		Recreation Lake Level			Priest River Discharge (Q)			
Alternative	Water-year Type	Stage (ft)	Start Date	End Date	Q (cfs)	Start Date	End Date	Inflow Calendar Year
Exist-D1	Dry	3.0	July 1	Oct 8	60 (min)	Jan 1	Dec 31	2001
Exist-D2	Dry	3.0	July 1	Oct 8	60 (min)	Jan 1	Dec 31	2005
Exist-N	Normal	3.0	July 1	Oct 8	60 (min)	Jan 1	Dec 31	2002
Alt 1	Dry	3.5	July 1	Aug 30	60 (min)	Jan 1	Dec 31	2001
		3.5 to 3.0	Sept 1	Oct 8	2,000 (max)	Oct 9	Oct 31	
Alt 2	Dry	3.5	July 1	Aug 30	60 (min)	Jan 1	Dec 31	2005
		3.5 to 3.0	Sept 1	Oct 8	2,000 (max)	Oct 9	Oct 31	
Alt 2N	Normal	3.5	July 1	Aug 30	60 (min)	Jan 1	Dec 31	2002
		3.5 to 3.0	Sept 1	Oct 8	2,000 (max)	Oct 9	Oct 31	
Alt 3	Dry	3.25	July 1	Aug 30	60 (min)	Jan 1	Dec 31	2001
		3.25 to 2.75	Sept 1	Sept 30	2,000 (max)	Oct 1	Oct 31	
Alt 4	Dry	3.5	July 1	Aug 15	60 (min)	Jan 1	Dec 31	2001
		3.5 to 3.0	Aug 16	Sep 15	2,000 (max)	Oct 1	Oct 31	
		3 to 2.75	Sept 16	Oct 8				
Alt 5	Dry	3.25	July 1	Aug 30	60 (min)	Jan 1	Dec 31	2001
		3.25 to 3.0	Sept 1	Oct 8	2,000 (max)	Oct 9	Oct 31	

Table 3-1 Priest Lake Water Management for Existing Operations (Exist) and Alternatives (Alt) for Dry-Year Types and a Normal Type Year Check

The simulation of alternative water management scenarios (Alternatives 1 through 5) indicate that increased lake levels during the recreational season can be maintained during the dry water years modeled, and within the constraints set on minimum and maximum Priest Lake outlet dam discharge.

3.5 Increased Reservoir Pool

Increasing the lake level by 6 inches would allow for an additional 11,500 acre-feet of water to be stored in Priest Lake during dry years to aid in meeting the minimum discharge requirements from the dam and maintaining a minimum 3-ft water level throughout the duration of the summer recreational period. The additional 6-inch storage provides an additional 55 cfs for the summer recreational period without factoring losses from evaporation, infiltration, and other considerations. Raising the pool by 6 inches would store the additional water during the spring runoff (June) when high flows from snowmelt fill the reservoir for the month of July. IDWR would release the additional stored water during the mid- to later summer recreational season to meet the minimum 60 cfs outlet dam discharge.

Hydrologic analysis indicates increasing the lake level by 6 inches during the early summer recreation period would provide the necessary additional storage needed to meet project goals and criteria. A summary of the hydrologic analysis is outlined in Appendix B.3 – Water Level Management Hydrologic Analysis. Increasing the lake level would be considered an alternative water management scheme that would be implemented during predicted dry or drought years only. No change to current lake level operations would occur during non-dry or non-drought years. An adaptive management approach for discharge of the additional stored water would be employed in order to adjust operations to the unique climate and snowpack conditions for dry year conditions that exceed an agreed threshold. Based on review of historical water level and outlet dam discharge data, the frequency of occurrence could be on the order of once every 5 to 7 years, not accounting for future climate change. Additionally, a 6-inch higher water level could be used as an operating tolerance to allow more efficient management of the water resource during the summer recreational period.

The compilation of operating criteria, guidelines, and specifications that govern the storage and release function of a reservoir is referred to as its rule curve. The next phase would include more detailed analysis and the development of lake level management rule curve alternatives. Development of a rule curve for both regular and dry water years would be recommended to improve seasonal management of the water resources.

3.6 Pool Raise Assessment

3.6.1 Introduction

Based on the results of the hydrologic analysis, a pool raise of up to 6 inches during dry or drought years would be capable of meeting the project goals and criteria. The additional storage would occur during the spring runoff period, and would result in maintaining the higher pool through the month of July and then gradually drawing down the additional storage capacity throughout the months of August and September.

The pool raise assessment was conducted and included the following tasks:

- Review of site conditions.
- Collect and analyze wind and water level data.
- Evaluate wind-generated and vessel-generated wave climates.
- Assess potential impacts of pool raise on shoreline features.
- Evaluate additional considerations such as basement flooding and natural shoreline (as outlined in Figure 3-4).

A detailed description of the pool raise assessment is outlined in Appendix B.2 – Pool Raise Assessment Summary.

Figure 3-4 Pool Raise Assessment Considerations

3.6.2 Assessment Summary

Site assessments were conducted on May 11, 2017 when the lake level was at 3.5 feet on the USGS gage. Shoreline areas were observed both by foot at locations along the west and east side of the lake, as well as by boat. The majority of the lake shoreline could be classified as natural (vegetated with boulders and cobbles) sandy beach, or modified with shoreline structures. Bonner County LIDAR data and other nearshore topographic survey data were also reviewed and analyzed.

Wind data analysis and two-dimensional (2-D) numerical wind-wave modeling was conducted for the lake. Winds dominate from the south during the daytime and from the north during the evening and night. A comparison of wind data for the summer months relative to May/June indicated minor differences. Spring runoff high water levels (above 3.5-foot gage level) combined with wind storms occur during the months of May and June, and represent similar or more extreme conditions than what would be expected during a temporary pool raise in the month of July.

Water levels are at or above the 3.5-ft pool raise during the months of May and June, based on the water level record (see Figure 3-3). An increase in pool level to 3.5 feet during July would be similar to or less than the conditions that occur annually during the months of May and June.

Wave analysis was conducted for both the existing lake level and a 6-inch water level increase. Vessel wakes were evaluated utilizing empirical analysis of recreational vessels traveling at an assumed speed and distance from the lake shoreline.

Priest Lake beaches are predominantly sand. Beaches were observed to be fine to medium grain sand with localized areas of small gravel. Typical beach slopes were observed to generally vary from 10H:1V to 15H:1V. A 6-inch raise in water level would shift the waterline approximately 5 feet to 8 feet further landward.

Shoreline structures such as boat lifts are located throughout the entire lake shoreline. The majority of boat lifts are removed during the fall for protection from winter waves and ice and are reinstalled during the late spring. A pool raise may require some non-adjustable boat lifts to be installed at a slightly higher level to remain operational during a 6-inch higher lake level.

Boating facility usability and access was reviewed for public boat ramps and private marinas. A pool raise would have minimal effect on these facilities, and could improve accessibility at many facilities that already have depth constraints during the summer recreational season.

Review of shoreline habitat features were conducted based on a site assessment, review of aerial photographs, and Bonner County GIS data.

Bull trout and Kokanee were determined to be the species of concern for this study, based on consultation with state, federal, and tribal agencies. Increased lake levels during the summer period were determined to not result in any restricted access to tributaries for the bull trout that occur during May to November, and would not impact Kokanee spawning which occurs in the fall.

Basement flooding was evaluated utilizing information provided by community members through the stakeholder outreach work. Flooding was determined to be problematic for low elevation basements at houses in close proximity to the lake during peak spring runoff time periods, when water levels are in excess of the 3.5-foot gage level. A 6-inch pool raise would likely not have an effect on basement flooding.

Thorofare navigation was evaluated for effects from increased summer lake levels and was determined to have no direct impact. Both alternatives (3” and 6” pool raise) could improve navigation by providing a slightly greater water depth.

3.6.3 Results and Conclusions

The following summarizes the results and conclusions from the pool raise impact assessment, based on the available information and conceptual-level study:

- Wind data indicates wind speed and frequency of winds are similar for the time period of higher pool, as during the time period when the lake historically is at and above the 3.5-foot gage level.
- The wave climate along the lake shoreline was determined to have a minimal difference at the shoreline and beaches for the pool raise.
- Vessel wakes are currently a concern in areas where wake board boats operate close to the shoreline. Enforcement of minimum offshore operating distances to minimize wake energy at the shoreline and localized confined areas (small distance between an island

and the mainland shoreline) of the lake potentially as no wake zones should be considered under existing and any proposed pool raise scenario. Additional investigation within localized areas may be needed during the next phase of the project.

- Lakeshore wind-wave induced beach sediment processes are anticipated to have minimal change as a result of a 6-inch pool raise during the recreational season.
- A temporary pool raise will not pose a major impact at the majority of recreational beaches. Localized areas could experience loss of usable beach for short time periods during the month of July.
- Habitat and wetlands on the Lake shoreline should have minimal effect from the short-term increase in pool levels. Additional site assessment work in the next phase should be conducted to verify habitats of concern and their elevations for final confirmation prior to regulatory permitting.
- Shoreline conditions are complex and variable throughout the Lake. An assessment was conducted for a range of conditions utilizing the available information. Additional refinement of the analysis utilizing more data and information will be needed in the next phase for localized areas.
- Basement flooding typically occurs during lake levels of 5 feet (USGS gage level) and higher; therefore, a short duration increase of 6 inches will not have any effect on basement flooding.

3.7 Climate Change Considerations

Water management flexibility for the Priest Lake system will become increasingly important as water supply uncertainty increases in a changing regional climate. Under some projected climate change scenarios, the Priest Lake watershed is forecasted to experience increased annual precipitation and increased winter precipitation (USFS 2017). However, the snow-water equivalent on April 1 is projected to decrease by 20 to 200 percent, while snow residence time is projected to decrease 20 to 80 percent (USFS, 2017). These changes in project water supply suggest the need for flexibility in the Priest Lake system to manage changes in the timing and volume of water supply.

4 Outlet Dam Structure Assessment

4.1 Introduction

In 1950, a timber and stoplog dam was constructed at the outlet of Priest Lake to manage lake levels at a higher stage during the summer recreational period. The water stored in the summer months was discharged in the fall. In 1978, the original outlet dam structure was replaced, due to severe deterioration, by a new concrete outlet dam structure with adjustable gates and located further downstream. Figures 4-1 and 4-2 show the current outlet dam structure during high and low flow conditions.

Figure 4-1 – Outlet Dam Structure during May Spring Flow Condition

Figure 4-2 – Outlet Dam Structure during Summer Low Flow Condition

The dam is classified by Idaho Administrative Code 37.03.06 as Large Size and Significant Hazard per I.C.42-1711(8) for dam safety regulations. The current outlet dam structure consists of eleven equally sized 16'-wide bays, with ten piers, including one at each abutment. Each bay houses a tainter gate operated by a manual hoist on the adjacent pier. The handwheels have been removed and replaced with a nut to allow the use of a handheld powered drive to raise and lower the gates when required. There is a 1-foot high concrete sill at the base of the structure, and the tainter gates are 7 feet high, giving a total static hydraulic head of 8 feet. There is a concrete deck walkway running along the structure, allowing access to operate each of the tainter gates individually.

4.2 Background

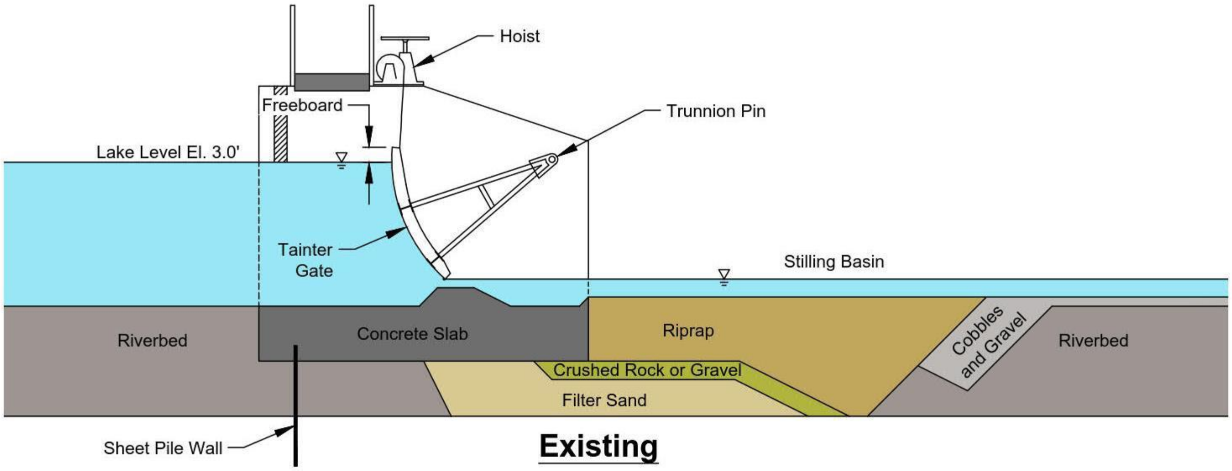
The purpose of the outlet dam structure assessment was to review the existing structure to determine what modifications may be required if the pool level is raised or if additional operational control of discharges is required. A cross-section of the existing outlet dam structure is shown in Figure 4-3. The following summarizes the work conducted in the outlet dam structure assessment:

- Hydraulics Assessment
  - Review design documentation and existing data, including available design documentation and operation data records;
  - Review existing data and develop an opinion of current gate capacity;
  - Evaluate hydraulic conditions for gate operations to determine susceptibility to hydraulic jump and downstream scour;

  - Evaluate potential for modifications to reduce risk of downstream scour for increased 6-inch water level;
  - Describe a range of alternatives that reduce risk of scour and minimize formation of hydraulic jump; and
  - Develop recommendations regarding improvements for gate operations to improve control of discharges.
- Gate Assessment
  - Review design documentation and existing data;
  - Evaluate existing tainter gate (Waterman Model T-1 gate) ability to resist additional 6 inches of water level; and
  - Develop opinion regarding capability of the gate to accept a 6-inch pool raise.
- Stability Assessment
  - Review design documentation and existing data;
  - Evaluate outlet dam structure stability (sliding, bearing pressure, overturning and seismic) for a 6-inch raise in water level; and
  - Develop a range of alternatives to improve structure stability.

A detailed description of the existing outlet dam structure components included in the assessment is shown in Figure 4-3. Assessments were based on a combination of site visit observations (May 12, 2017), interviews with IDWR staff, review of available documentation (see Appendix A.1), and application of industry standards. An overall summary of the outlet dam structure is provided in Appendix C.3 Outlet Dam Structure Assessment Summary.

Figure 4-3 – Existing Outlet Dam Structure



4.3 Hydraulic Assessment

An assessment of the existing outlet dam structure with respect to hydraulic flow regime, scour potential, and operations was conducted for existing conditions and a potential water level and gate height increase of 6 inches. Operational scenarios were developed for the assessment to represent a range of seasonal operating conditions. Spring, summer and fall discharge, and water levels were identified based on a review of historical information and potential pool raise conditions. Hydraulic computations for flows discharging through the spillway for the identified operational scenarios were conducted to assist in conducting the assessment work. A detailed summary of the hydraulic and gate assessment is provided in Appendix C.1 – Outlet Dam Structure Hydraulic & Gate Assessment.

4.3.1 Hydraulic Assessment

An assessment of the hydraulic conditions for the outlet dam structure was conducted relative to flow regime, operations, and scour potential. Assessment work was based on interview of outlet dam structure operators, site observations, review of available documents, and hydraulic computations. A review of historical data was conducted which included documentation outlined in Appendix A.1 – Existing Data & Bibliography Summary. The following were determined from a review of historical documents that include design review reports and memorandums describing major repair work:

- No end sill or concrete apron to control location and formation of a hydraulic jump was provided (Ch2mHill, 1978).
- Riprap is undersized and susceptible to erosion and therefore destabilization of the outlet dam structure (Ch2mHill, 1978).
- Damage to the scour apron occurred during the first year of operation, resulting in the need to conduct extensive repairs to the riprap scour apron (IDWR, 1979).

The following is a summary of the hydraulic assessment:

- Gate operations are conducted based on extensive prior experience of the site operators, but limited documentation was available for gate operational procedures.
- High velocities are a concern for high pool, low tailwater conditions; in particular, when a small number of gates are utilized to discharge water through the structure. Velocities will be expected to increase for any pool raise condition.
- Hydraulic jump forms beyond the existing concrete slab and in some operational scenarios beyond the riprap scour apron.
- Operation of single gates and along the edge of the channel should be limited to reduce the potential for scour development. Operation of more than a single gate and within the center of the channel would be preferred.
- Erosion potential of the existing riprap scour apron is increased by 9 percent for a 6-inch water level increase discharge condition.
- Existing riprap stone is likely undersized for certain discharge flow, lake level, and gate operational conditions.
- Modification to the scour apron is recommended to reduce scour risk, improve operational flexibility, and improve structure stability for the 6-inch pool raise condition.

- Gates are operated to meet the 3.0-foot pool level and maintain that level throughout the recreational season with minimal operating tolerance. An operating tolerance is recommended to provide greater flexibility in capturing summer rainfall events runoff during dry years.
- Improved control over gate operation and corresponding discharge should be considered. This would include retrofit of the gates from manual operations to having electronic controls and a control panel board.
- Calibration of gate opening operation to discharge measurements downstream of the dam is recommended.
- Formalized operational procedures be established and documented to reduce risk of operational error and mitigate risk through design. Design improvements would be improved flow control (calibration of discharge to gate opening), electric operation, and written procedures on operation of gates under the various seasonal operational conditions.

4.3.2 Outlet Dam Structure Modifications

The riprap scour apron functions to protect the concrete structure from erosion of foundation soils and thereby maintaining dam stability. Alternatives were investigated to mitigate the potential increased risk of erosion of the downstream riprap scour apron, and to increase structure stability. Improvements considered included a concrete stilling basin, concrete apron, baffle blocks at the downstream edge of the existing concrete slab, gate modifications to improve discharge hydraulic flow regime, a larger size riprap apron, grouting the existing riprap apron, and installation of an end sill (see Figure 4-4).

4.4 Gate Assessment & Modification

4.4.1 Assessment

The existing tainter gate capability to accommodate an increase in lake water levels of 6 inches was reviewed. Tainter gates transfer hydrostatic loads on the gate through the gate arms to the trunnion bearings mounted on the intermediate piers downstream of the gate. The effect of extending the gate to accommodate a 6-inch raise in water level will be to slightly alter the angle of the resultant force and to increase the loading on the trunnions. The gate hoist is sized to lift the dead weight of the gate plus the friction of the trunnion bearing and side seals. An increase in gate height would increase the weight of the gate. It is estimated the additional hoist load would increase by less than 15 percent of the existing. Load increases on the gate and hoist system were determined to result in small reductions in the factor of safety. Additional analysis in a subsequent design phase will need to be conducted to further quantify the reduction in the safety factor. A detailed summary of the Lake assessment is described in Appendix C.1. – Outlet Dam Structure Hydraulic & Gate Assessment.

4.4.2 Gate Modification

The current gate provides 2 inches of freeboard above the summer regulated water level of 3.0 feet. Freeboard is typically recommended to be a minimum of 4 inches for the conditions present at the outlet dam structure. Gate modifications would be needed to accommodate both an increase in the static water levels and the minimum required freeboard. A vertical extension would need to be conducted to allow the gates to be operated at a higher-level water of 6 inches and maintain the minimum freeboard. It is estimated the spillway gate modifications would require extending the steel gate higher by 8 inches above the existing, localized strengthening of the gate structure (beams and trunnion arms), recoating of steel structures, installation of new electric gate operators, electrical tie-in and control board, security and environmental enclosure, and a backup generator system for extended periods of power outages. A retrofit would be conducted to all 11 gates (see Figure 4-4).

4.5 Dam stability Assessment

As part of concept development and preliminary studies, a preliminary geotechnical stability assessment of the existing outlet dam was conducted. A detailed summary of the stability analysis is provided in Appendix C.2 – Outlet Dam Structure Stability Analysis. No new data was collected and the analysis relied upon original engineering design, geotechnical borings, and periodic dam safety inspection reports as the basis for the assessment. Outlet dam structure stability criteria for resistance to sliding and foundation bearing pressures were developed utilizing the Army Corps of Engineers (USACE) manual EM 1110-2-2200 Gravity Dam Design guidelines.

A review of historical data was conducted which included documentation outlined in Appendix A.1. The following were determined from the historical document review:

- Sheet Pile Cutoff Wall: 1978 dam design considered the contribution of the sheet pile for stability in design, and a peer review of the computations indicated a need to neglect the contribution of the sheet pile wall on sliding resistance.
- Stilling Basin Scour Risk: No end sill or concrete apron to control the location and formation of a hydraulic jump and the riprap scour protection in the stilling basin may be undersized and susceptible to erosion and thereby destabilization of the dam. Sliding resistance of outlet dam structure is dependent on the filter sand and gravel layer and downstream riprap scour apron to achieve the required minimum sliding resistance.
- Stilling Basin Scour: Scour of the stilling basin riprap occurred during the first year of operation and were replaced with a greater thickness and size of riprap.

The focus of the assessment was to evaluate potential pool raise scenarios between zero and six inches. Tailwater level was determined for low discharge conditions based on historical observations. Seismic analysis for this phase of the study utilized the maximum considered earthquake (MCE) as outlined in ASCE 7-10. The analysis was conducted assuming the foundation soils do not liquefy during the seismic event or the impact from liquefaction on stability of the dam is acceptable.

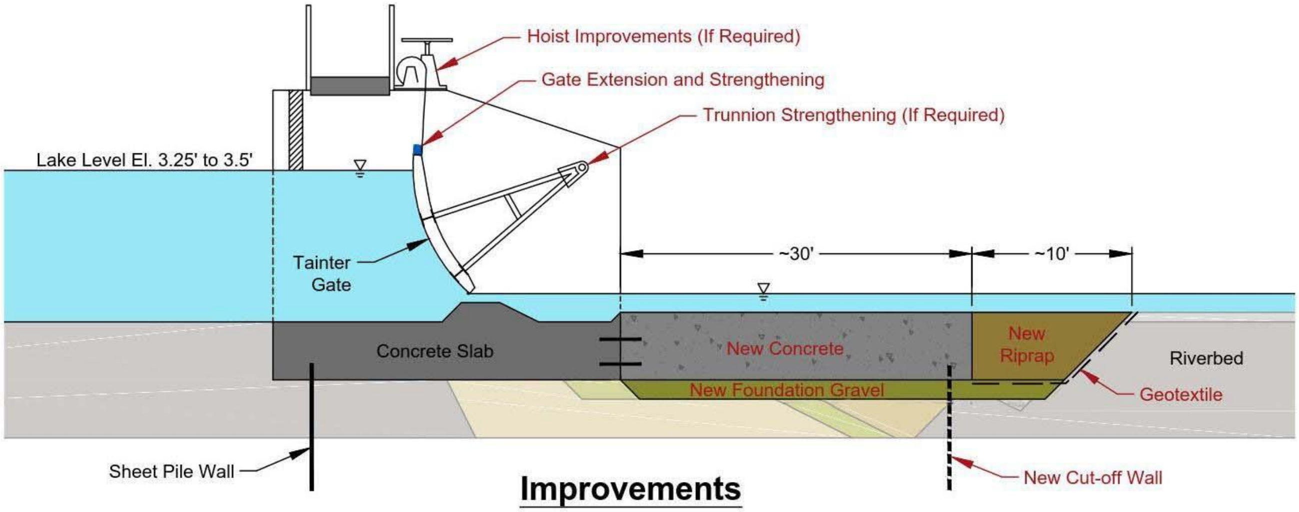
Critical assumptions made for the assessment included the structural effect for the sheet pile wall on stability is neglected and a three-foot-thick sand and gravel filter below the sill would provide pressure relief.

Sliding resistance was determined to require improvement to meet the currently required factors of safety based on the critical assumptions.

4.6 Outlet Dam Structure Stability Improvements

To improve dam stability and improve scour protection downstream of the dam, a new scour apron was determined to be required to implement any change in gates to increase the pool level and the corresponding static head. Figure 4-4 outlines the approximate geometry and features for the scour protection improvements. The new scour apron would be in direct contact with the existing sill and therefore would also improve the sliding resistance of the existing outlet structure. Installation of the scour apron would increase the pseudo static lateral thrust resistance during a seismic event and result in a safety factor at or greater than the minimum required by the Corps’ guidelines. A detailed summary of the outlet dam structure improvement conceptual design is shown in Appendix E.3.

Figure 4-4 – Outlet Dam Structure Improvements

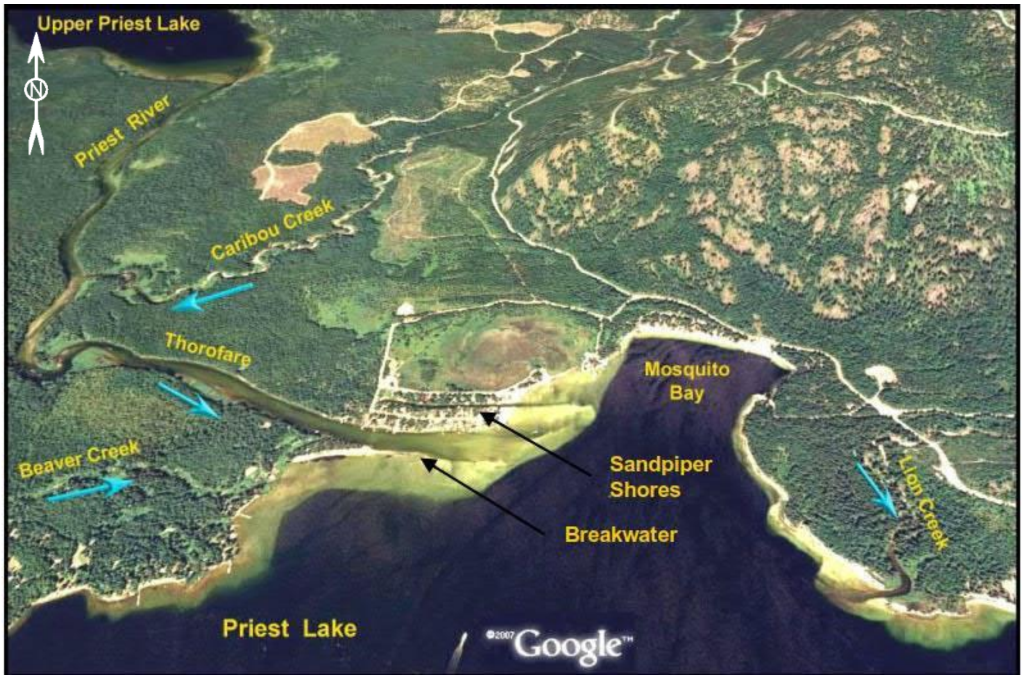


5 Thorofare Assessment

5.1 Introduction

Upper Priest Lake drains and connects to Priest Lake through the 2.7-mile natural channel referred to as the Thorofare (Figure 5-1). A geomorphic and hydraulic analysis was conducted to develop a baseline understanding of the project site physical processes to aid in the development and evaluation of improvements to maintain navigation into the Thorofare. A detailed description of the Hydraulic and Geomorphic processes assessment for the Thorofare are summarized in Appendices D.1 and D.2.

Figure 5-1 – Thorofare Project Vicinity Features



Vessel access to the Thorofare and navigation within the lower Thorofare providing access into and from the Upper Priest Lake has become problematic. Figure 5-2 shows the project area features and location of concern for improving navigation between Priest Lake and the Thorofare. This study evaluates the existing site physical processes to assist in developing improvements for the Thorofare to maintain sustainable navigation for the recreational boating community.

Figure 5-2 – Thorofare Study Area

5.2 Geomorphic Assessment

The purpose of the geomorphic assessment work was to evaluate sedimentation processes at the Thorofare mouth to aid in the evaluation of Thorofare improvement alternatives. The following summarizes an assessment of factors which were determined to be critical to conducting the geomorphic analysis work:

- History: Historical aerial photographs indicate a breakwater type structure or berm present at a location further north than the present-day breakwater (see Figure 5-3). This provided a narrower, more confined channel, likely resulting in higher current velocities.
- Breakwater: The breakwater is constructed of timber piles with vertical planks connected to timber beams spanning the timber piles (see Figure 5-4). The breakwater is classified as a porous structure (~20% to 35%) due to the gaps in the vertical planks and at the base of the beam. A timber breakwater has been at the Thorofare since the 1920’s and was last replaced in 1990. A review of historical records indicates the service life of a breakwater at the site to be approximately 30 to 40 years. In recent years the breakwater has required increasingly more maintenance and repair and is nearing the end of its service life.
- Navigation: Concerns regarding navigable access have existed in the record since 1994. Minimum desired navigable depth was identified to be 4 feet. Bonner County has been responsible for marking the entrance channel with a buoy to aid vessel operators in finding the deepest part of the channel through the entrance to the Thorofare.
- Dredging: Dredging records for the Thorofare were sparse and limited to descriptions from local residents and a few photographs. Regular maintenance dredging occurred up until the 1990’s and at which time it was curtailed.
- Thorofare Hydraulic Conditions: Measurement of flows in the Thorofare was not conducted on a regular basis, but limited historical data is available for 1994 and 1995. Additionally, flow velocity measurements were taken by Mott MacDonald on May 11, 2017 during the hydrographic survey work. Annual mean daily flow was estimated to be 400 to 500 cfs, and a peak mean daily flow during spring runoff of approximately 2,500 cfs.

- Thorofare flows at the interface with the lake become complex as a result of flow spreading through distributary channels throughout the mouth and breakwater structure (see Figure 5-5). A review of aerial photographs from 2004 to 2015 indicates a large variation in location, size, and number of distributary channels exiting out through the breakwater (see Figure 5-6).
- Thorofare Hydrography: Hydrographic survey data collected in May 2017 indicates a very large shoal (shallow area) within the mouth of the Thorofare at the Lake (see Figure 5-7). Current depths are less than 18 inches in a large area of the entrance to the Thorofare.
- Thorofare Sediment Processes: A review of potential sources of sediment entering the lower Thorofare at the breakwater was conducted. The sediments are primarily clean medium-grained sand. The sediment is a combination of bed and suspended sediments that enter the Lake from the Thorofare.
- Lake Wave Conditions: The Thorofare and breakwater location are primarily affected by up lake (southerly) winds that generate waves which interact with the lake shore and breakwater. South winds are the dominate wind direction and typically occur during the day in the afternoon.
- Lake Shore Sediment Processes: Sediment along the lake shore are fine to medium-grained sand. Sediments are delivered to the Thorofare from longshore sediment transport as a result of the south-dominated wind-waves. Figure 5-8 provides a summary of the assessment of lake sediment processes.

Figure 5-3 – Historical Extent of Breakwater (1935 position shown in red)

Figure 5-4 – Existing Breakwater Structure (porous or open structure)

Figure 5-5 – Distributary Flow Regime at location of Thorofare flow entering Lake

Figure 5-6 – Location of recent historical distributary channels through the breakwater

Figure 5-7 – Depths in Thorofare and Breakwater Project Area

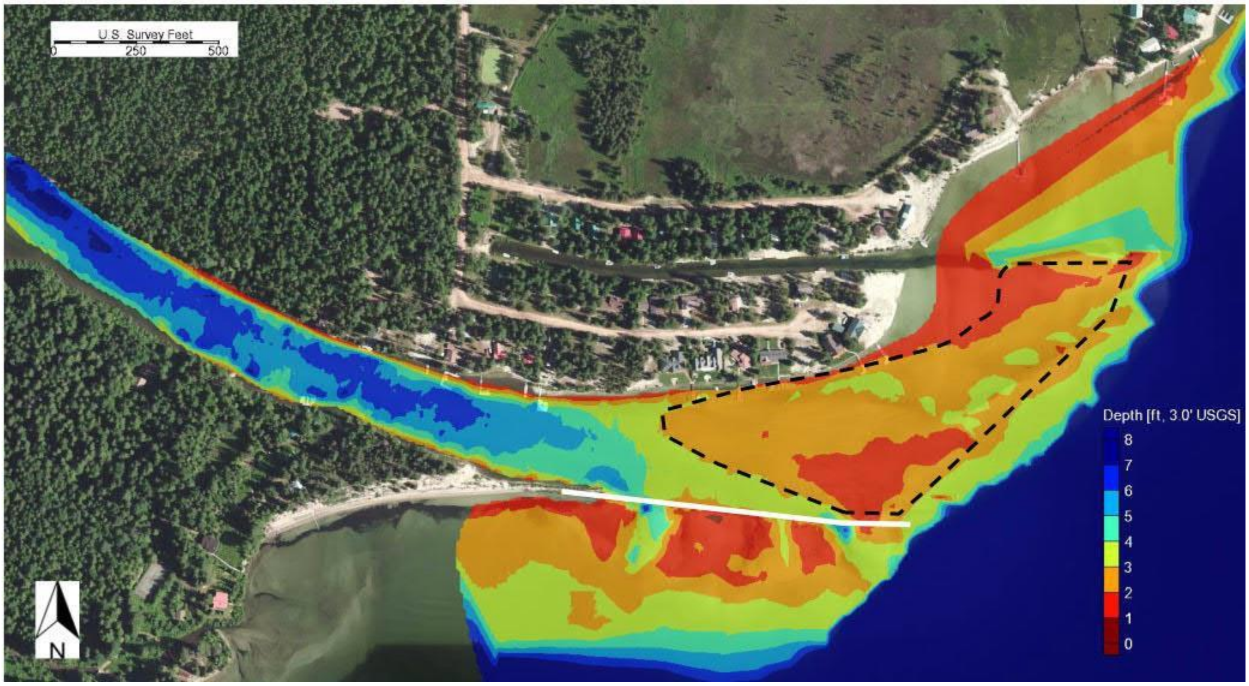
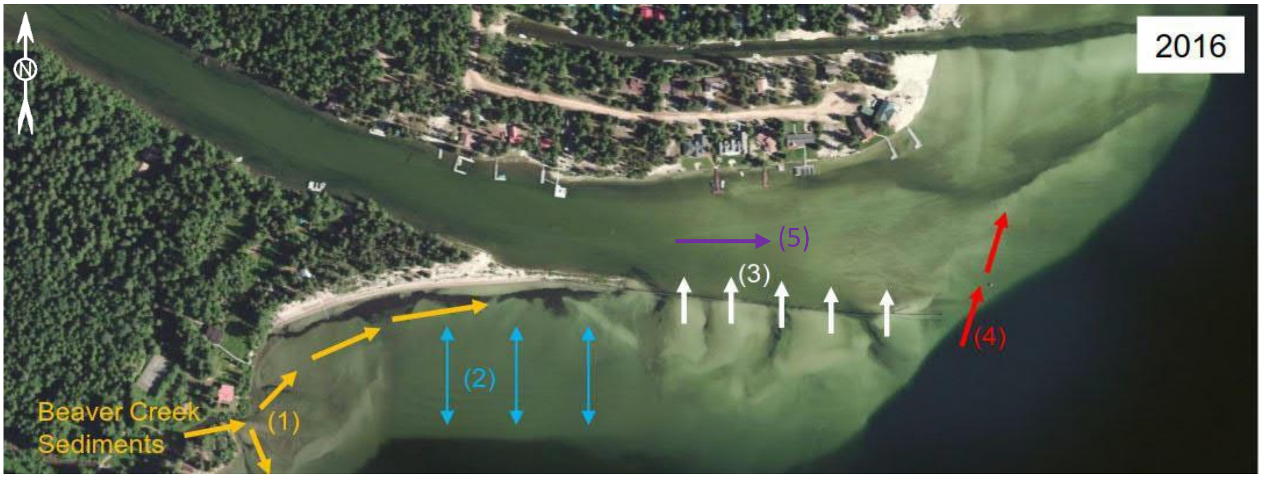


Figure 5-8 – Interpretation of Lake Geomorphic Processes: (1) longshore sediment transport; (2) cross shore sediment transport; (3) wave driven sediment transport through breakwater during low Thorofare flow conditions; (4) wave driven sediment transport around end of breakwater; (5) river bed and suspended sediments

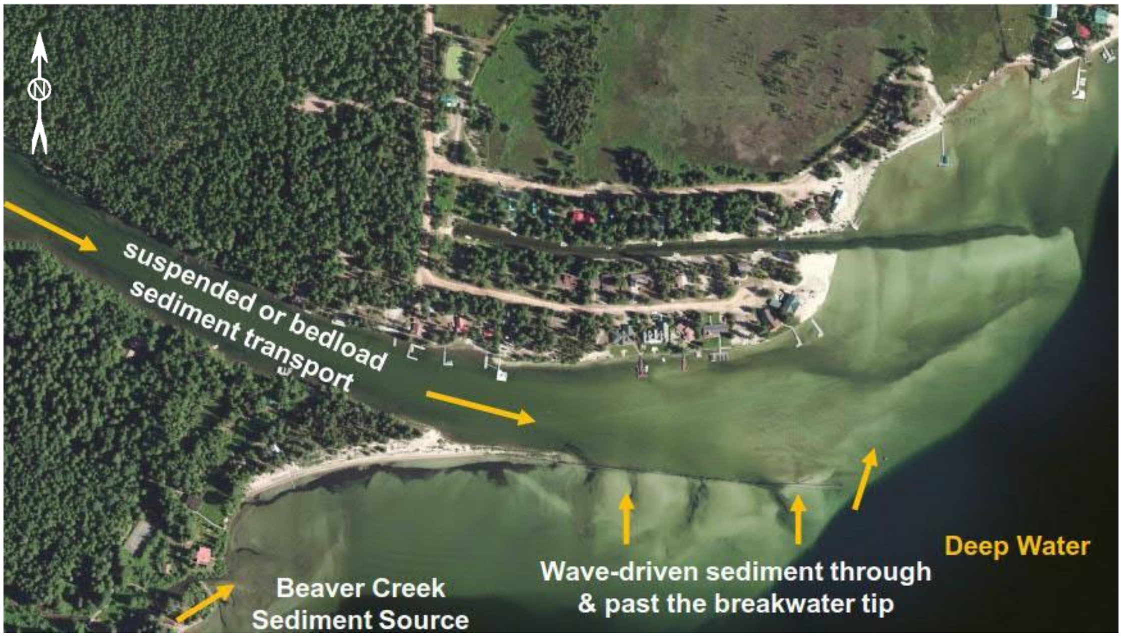


5.2.1 Conclusions

The following summarizes the conclusions from the geomorphic processes assessment work and is outlined in Figure 5-9:

- Sediment deposition from Thorofare sources is a result of decreased transport capacity as the low-gradient Thorofare flow meets the zero-gradient lake and flow spreading occurs through the porous breakwater.
- The Thorofare flow passes through the breakwater and has episodically scoured the bed underneath the breakwater, leading to significant spreading of the flow and reduced sediment transport capacity at the outlet of the Thorofare into Priest Lake.
- Lake shore sediment transport is a major contributor to shoaling (sedimentation) of the entrance to the Thorofare as a result of wind-driven sediment transport through and around the breakwater structure.
- Deposition has been accentuated by three factors at the mouth:
  - Widening of the Thorofare channel along the breakwater;
  - Reduced Thorofare discharge and velocity as water passes through or under the existing timber breakwater; and
  - Wind-driven sediments are pushed through and around the breakwater’s eastern end.

Figure 5-9 – Interpretation of Project Area Geomorphic Processes (Lake and Thorofare)

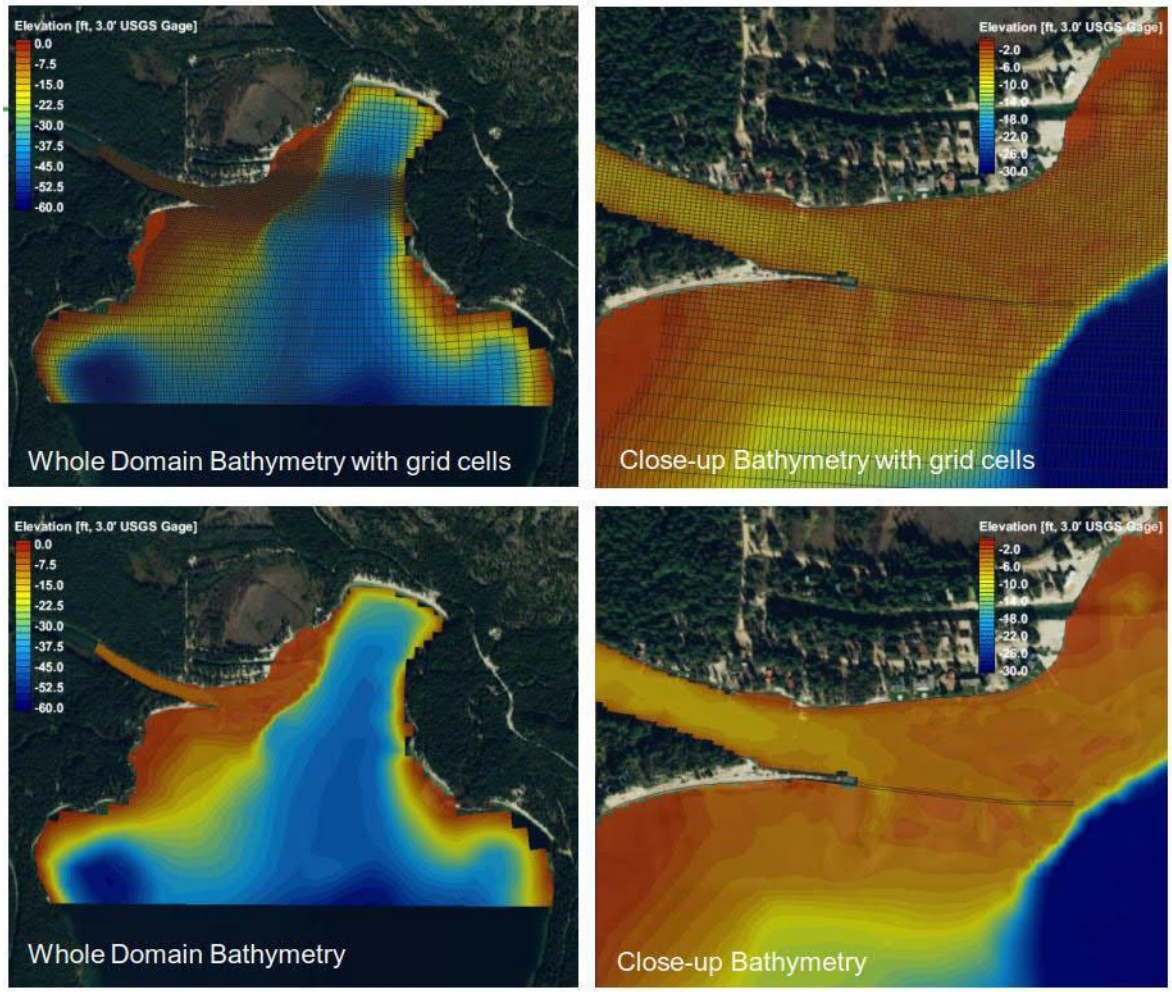


5.3 Hydraulic & Sediment Transport Analysis

5.3.1 Introduction

The purpose of the hydrodynamic (hydraulic and sediment transport) analysis work was to simulate existing hydraulic conditions to establish a baseline for evaluation of alternatives for improving Thorofare navigation and sustainability. The approach was to utilize available discharge, hydrographic survey, water level, and sediment size data to conduct a conceptual-level hydrodynamic analysis using numerical modeling tools such as DELFT3D-FLOW. DELFT3D is a three-dimensional (3-D) hydrodynamic simulation program which calculates non-steady flow and sediment transport phenomena that result from water level, meteorological and river forcing on a curvilinear, boundary fitted model grid.

Figure 5-10 – Numerical Modeling Domain

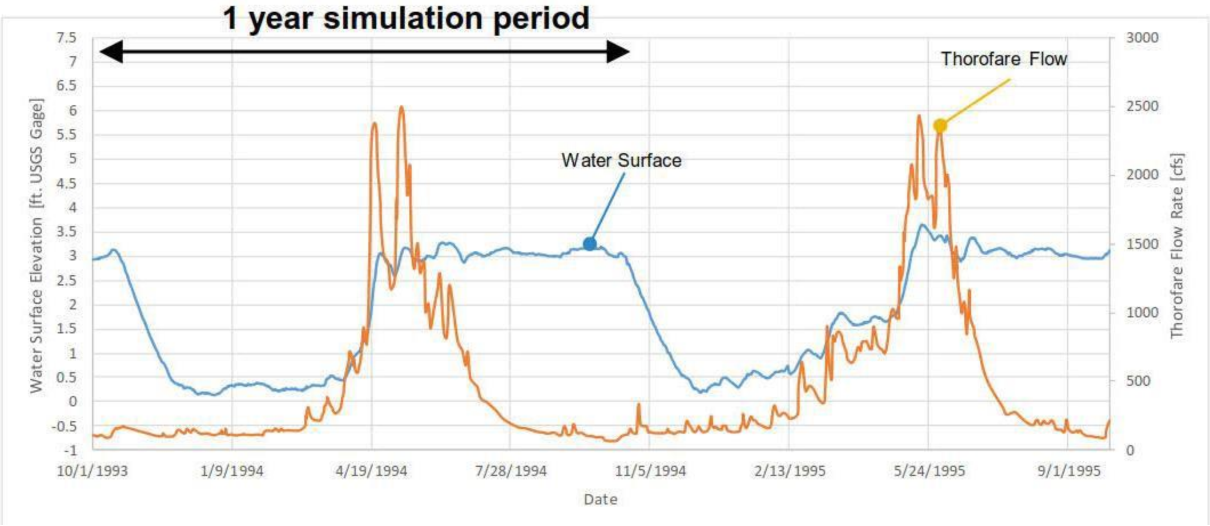


5.3.2 Model Input

The following were model input parameters developed from the project database and prior similar project experience using DELFT3D:

- Discharge: Time series 1994 water year (see Figure 5-11).
- Sediment Size: Uniformly graded sand with a D_{50} of 0.50 mm as determined from sediment sample sieve analysis.
- Water Surface: Time series 1994 water year (see Figure 5-11).
- Simulation Period: Compressed a 370-day (calendar year) simulation into 37-day modeling period.

Figure 5-11 – Model Input (Water Level & Discharge Time Series)



5.3.3 Results

The DELFT3D numerical model was determined to reasonably represent hydraulic and sediment transport processes for existing conditions. The following summarizes the results of the hydraulic modeling of existing conditions:

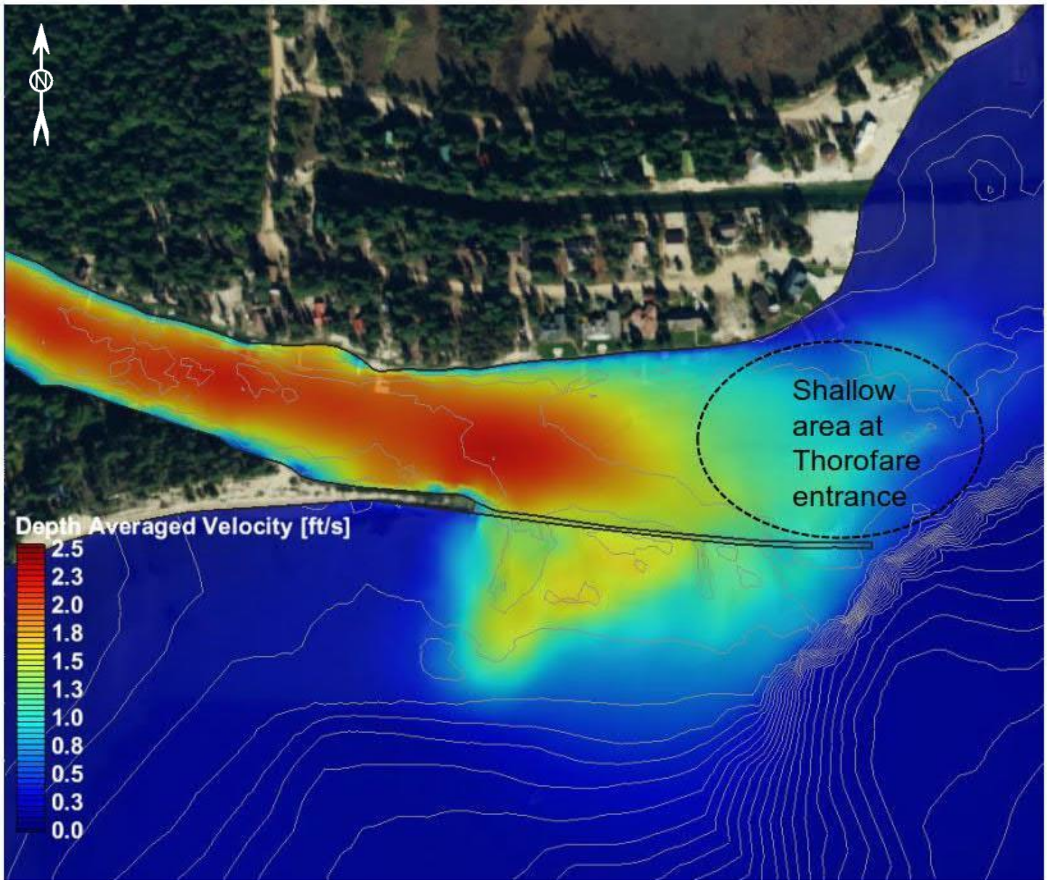
- Flow spreading observed along the breakwater is represented in the modeling results (Figure 5-12).
- Significant drop in velocity at approximately ½ length of the breakwater. Velocity reduction zone corresponds with the area of reduced depth (shoal) at the Thorofare entrance (Figure 5-12).
- Maximum velocity in the Thorofare coincides with maximum spring runoff.
- Numerical modeling simulation can be used to evaluate alternatives on a qualitative comparative analysis basis.

5.3.4 Prescreening Analysis

Evaluation of existing geomorphic and hydraulic processes indicated the porous breakwater structure is a contributor to the sedimentation of the Thorofare entrance. A prescreening analysis of breakwater concepts was conducted to focus further alternatives evaluation on an appropriate type of improvement for the Thorofare. Considerations which were used in the development of alternatives for evaluation included the following:

- Motorized boat access (recreational and emergency services) and navigation is challenging at the Thorofare mouth; improvements need to result in sustainable increase in depths.
- Deteriorated breakwater structure with periodic maintenance and repair history; continued reliance on the existing structure should not be assumed as a component of any alternative.
- Thorofare flow splitting (flow passing through porous breakwater) is less effective for sediment transport through the outer Thorofare into the Lake.

Figure 5-12 – Velocity Plot for Spring Discharge



- Repeated failures of breakwater increasing extent of Thorofare flow splitting.
- Sediment migration from lake shoreline into Thorofare is contributing to the sedimentation of the Thorofare and blockage of that sediment transport should be a consideration in the improvement alternative.

The following range of potential alternatives were developed for Thorofare improvements:

- No Action (maintain existing).
- Removal of Breakwater.
- Rehabilitate Existing Porous Breakwater.
- Replace Existing Porous Breakwater with Solid Sediment Protection Feature.
- In Channel Flow Diversion.

Hydraulic and sediment transport analysis was conducted for a porous breakwater (similar to existing) and a new solid breakwater structure. A detailed summary of the screening analysis is described in Appendix D.3 – Thorofare Alternatives Screening Summary. The results of the modeling and analysis work are show in Figures 5-13 (Hydraulics) and 5-14 (Sediment Transport).

Figure 5-13 – Hydraulic Analysis Results for Porous vs. Solid Breakwater

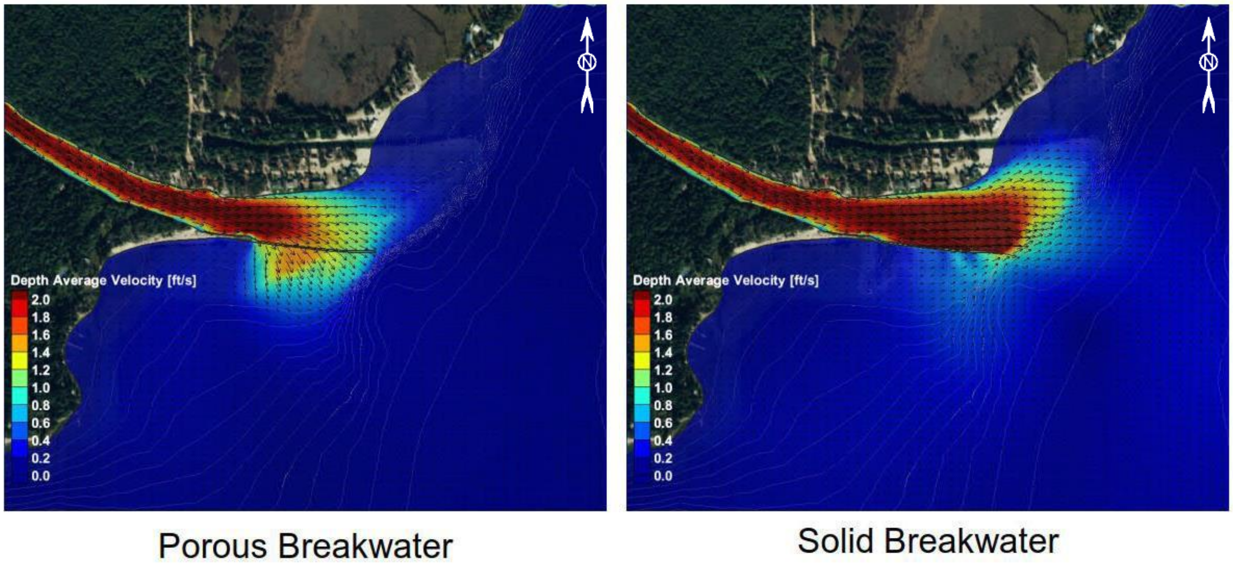
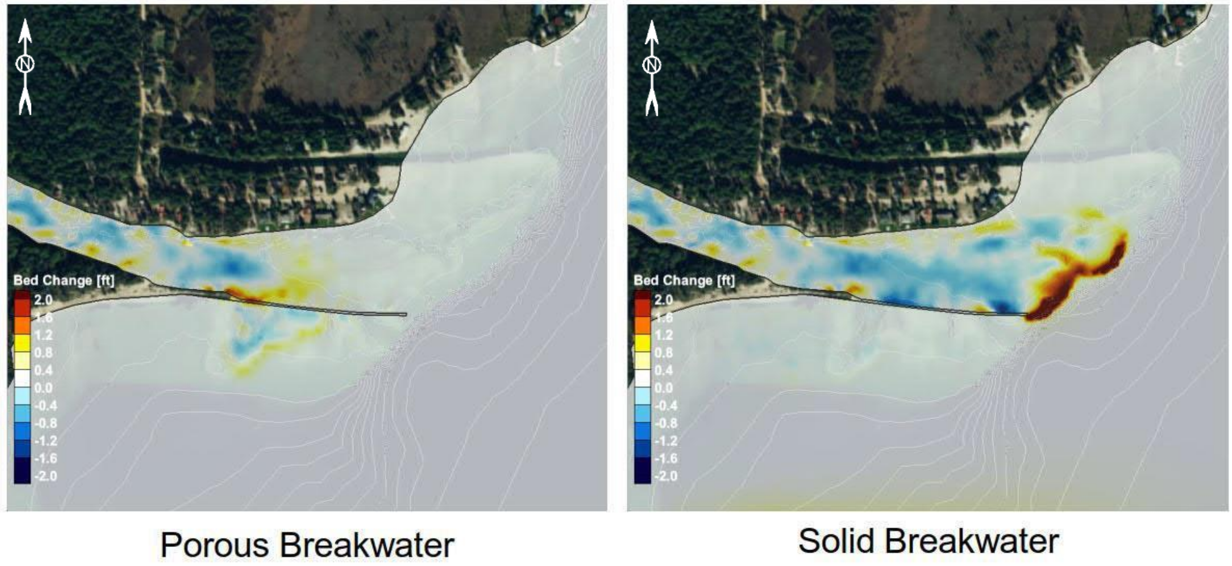


Figure 5-14 – Sediment Transport Analysis results for porous vs. solid breakwater



5.3.5 Results

Hydraulic and sediment transport analysis indicated the following:

- Sensitivity testing showed that the Solid Breakwater better confines the flow to the Thorofare when compared to the Porous Breakwater, and as a result, higher velocities will occur in the Thorofare mouth.
- Sensitivity testing showed that the Solid Breakwater transports more sediments along the Thorofare and deposits them in deeper water when compared to the Porous Breakwater.
- Flow diversion in the Thorofare does not occur within a solid (non-porous) breakwater feature.

5.3.6 Conclusions

The following summarizes conclusions developed from the hydraulic and geomorphic assessment work:

- The Thorofare ranks low in suspended sediment transport; the majority of sediment is bedload.
- Flow confinement in the Thorofare is important for improvement of navigation sustainability.
- A Solid Breakwater is preferred versus a Porous Breakwater.
- Redirecting lakeshore sediment movement from entering the Thorofare and bypassing further east toward the Thorofare mouth will reduce that sediment source, and will reduce future maintenance dredging needs.
- Based on results of hydrodynamic analysis, Thorofare improvement concepts to be considered may include:
  1. Replace porous breakwater with a solid breakwater at current location;
  2. Replace breakwater with new solid breakwater and change location (rotation of alignment); and
  3. Replace breakwater with new solid breakwater and extend eastward.

6 Thorofare Conceptual Engineering Analysis/Design

6.1 Introduction

Analysis of Thorofare improvements to the existing breakwater was conducted. As outlined in Section 5, replacement of the breakwater with a new solid breakwater type structure, providing increased sedimentation protection from longshore sediment transport from Priest Lake, is a requirement to meet the project goals for conducting improvements to the Thorofare. Pre-screening analysis indicated a solid sediment protection structure with either an extension and/or a rotation is preferred. Analysis of the breakwater material type, alignment and geometry were then conducted to aid in the development of a Thorofare improvement plan and to estimate the corresponding construction costs.

Evaluation of the breakwater structure alignment and material type was conducted with consideration of the following factors:

- Impervious sediment protection feature.
- Eliminate diversion flows from Thorofare at entrance to Lake.
- Eliminate lake shore sediment from migrating into Thorofare during low Thorofare flow conditions.
- Potential for non-structural feature (in lieu of breakwater); soft stabilization.
- Evaluate performance of potential alternatives with conceptual Thorofare dredging concept.

6.2 Breakwater Alignment Alternatives Assessment

6.2.1 Introduction

Based on the results of the Thorofare improvement assessment work, it was determined that a rotated and lengthened breakwater structure is a promising alternative and should be considered for further evaluation. Three concepts were evaluated: A.) Current Alignment with Extension; B.) 15-degree Rotation with Extension (Figure 6-1); and C.) 30-degree Rotation with Extension (Figure 6-2). A detailed summary of the assessment is described in Appendix D.3 Thorofare Improvement Alternatives Screening Summary.

Figure 6-1 – (15-degree Rotation with Extension)

Figure 6-2 – (30-degree Rotation with Extension)

6.2.2 Breakwater Concept Assessment

Engineering analysis of the alignment and extension concepts were conducted based on a review of the hydrodynamics, numerical modeling results, required geometry and associated additional construction costs, and compatibility with placement of dredged materials. The assessment concluded an extension of the breakwater would be a required feature of any breakwater alternative. The 30-degree rotated alternative was eliminated from consideration due to the substantially higher construction cost from a significantly longer structure.

Numerical analysis for an existing porous structure, a new solid structure along the existing alignment, and new solid structure with 15-degree rotation was conducted utilizing the DELFT3D model to aid in the selection of a preferred concept. Existing conditions were evaluated to provide a baseline structure condition for comparison of alternatives to aid in reviewing performance. Figures 6-3, 6-4, 6-5, and 6-7 describe the results of the existing porous, new solid structure with existing alignment, and the new solid structure 15-degrees scenarios. Improvements to the breakwater are intended to improve the performance of any dredging work by reducing sedimentation during post-dredge conditions. Numerical modeling of concepts included consideration for dredging of the Thorofare.

Figure 6-3 – Existing porous breakwater (current velocity [left] and sedimentation [right])

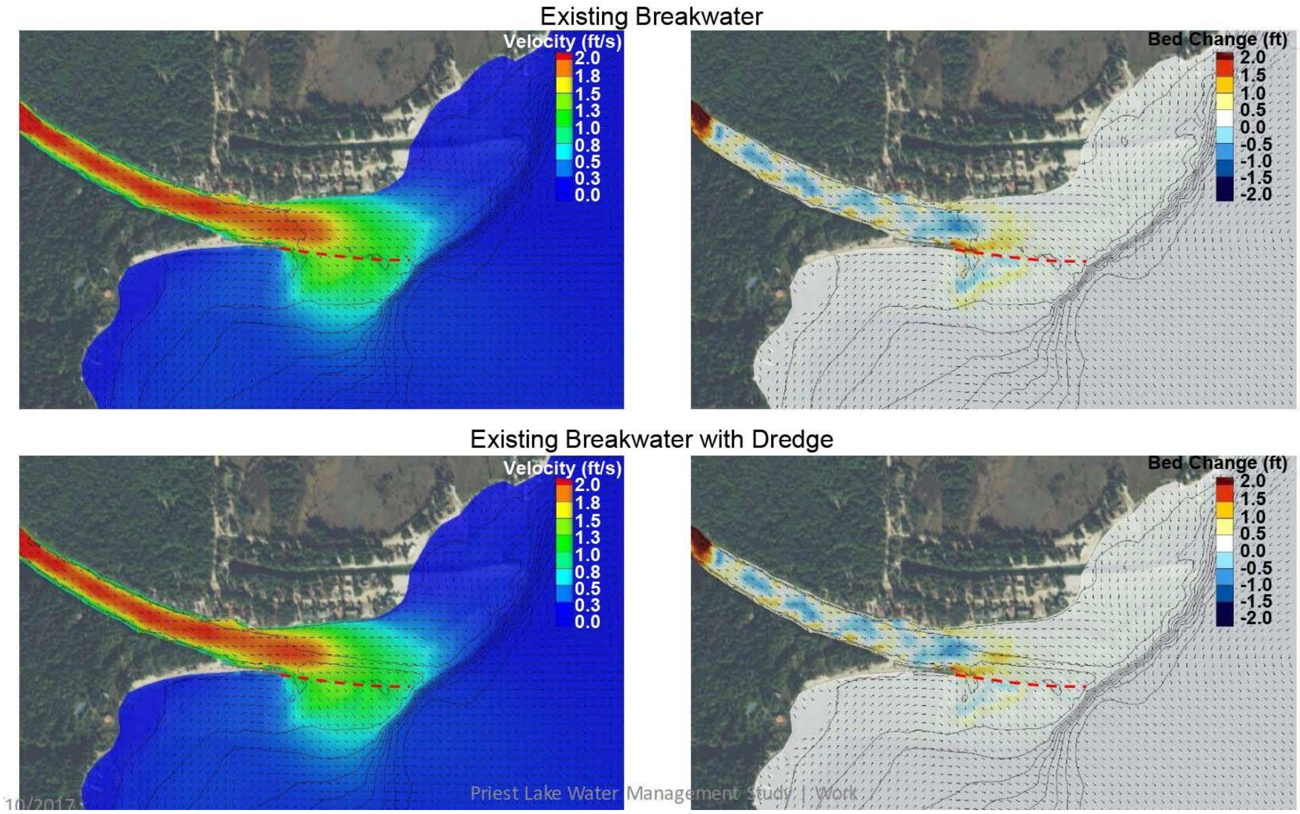


Figure 6-4 – Solid breakwater, existing alignment (current velocity [left] and sedimentation [right])

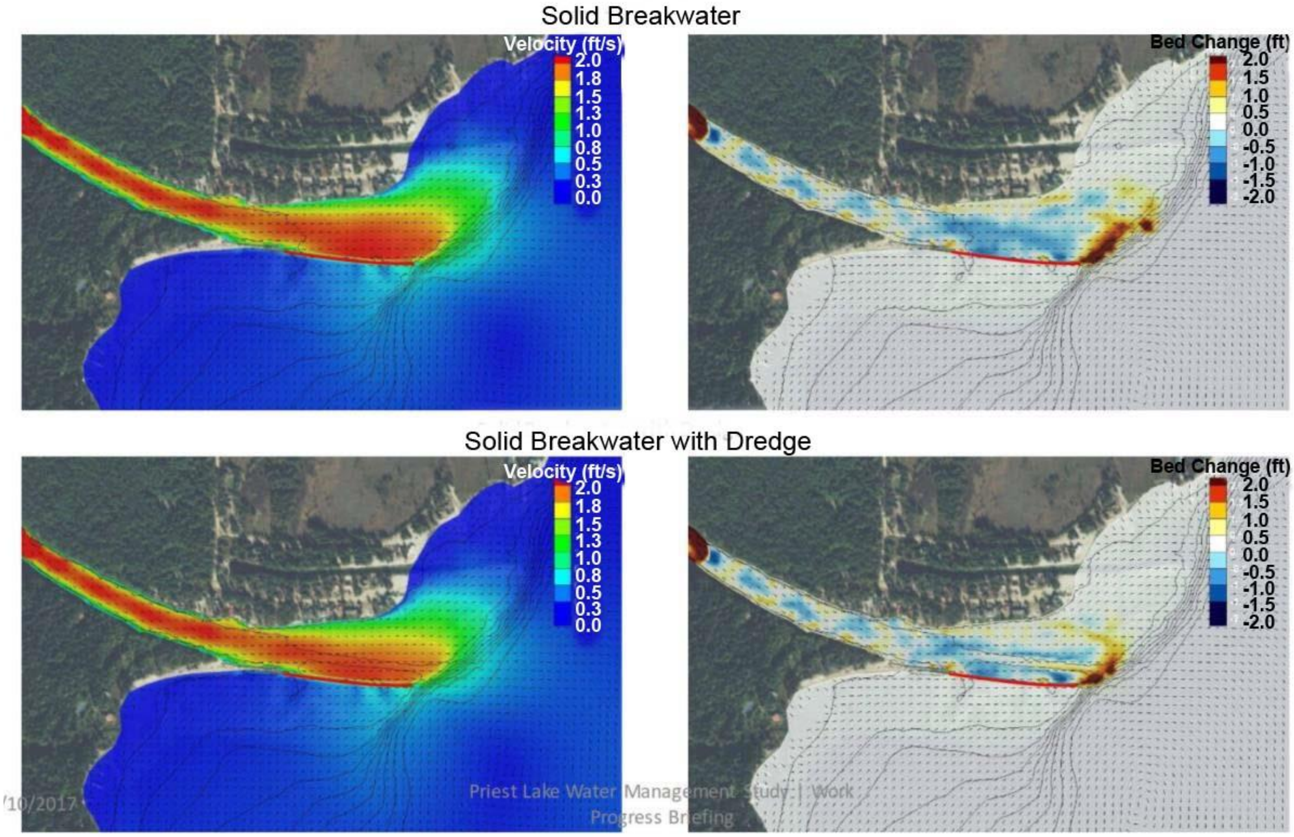
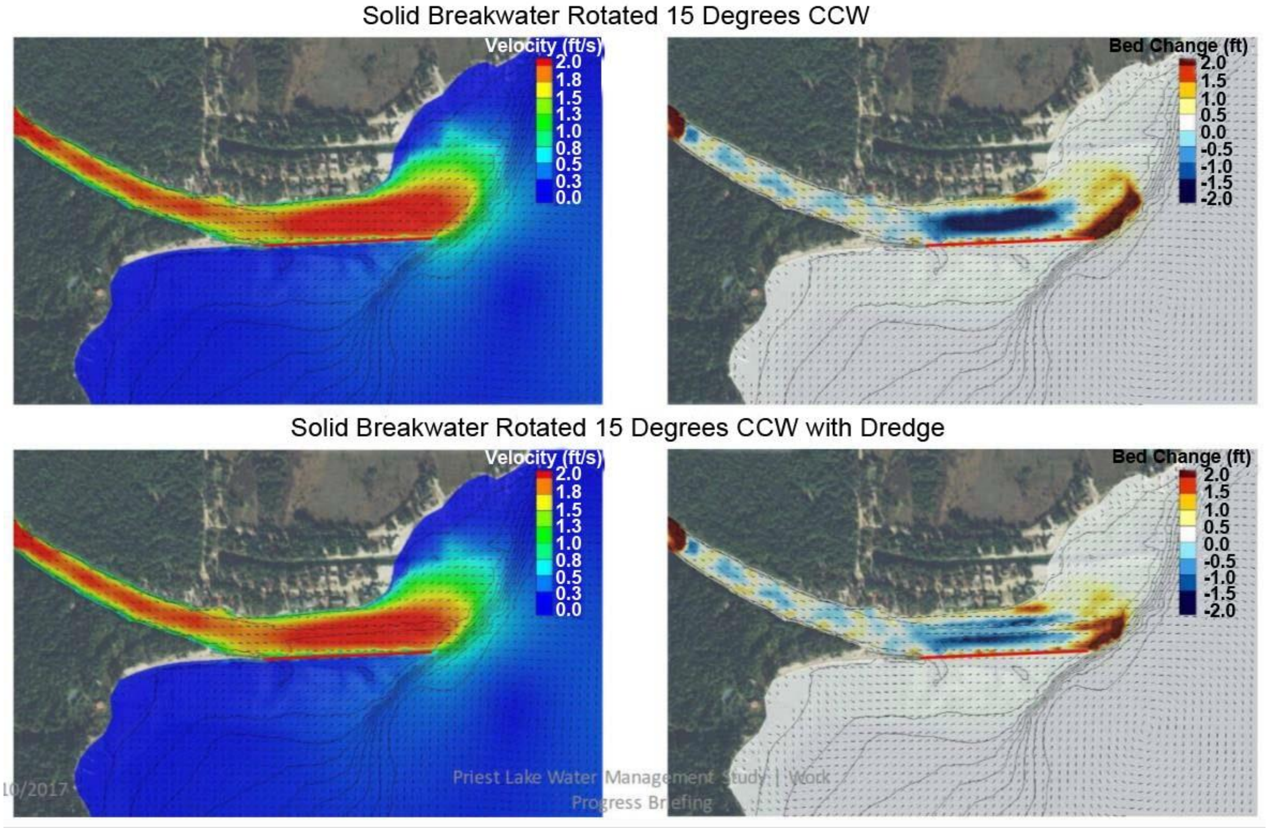


Figure 6-5 – Solid breakwater at 15-degree rotation (current velocity [left] and sedimentation [right])



Solid Breakwater Rotated 15 Degrees CCW

Solid Breakwater Rotated 15 Degrees CCW with Dredge

The 15-degree rotation with an extension was determined to be the preferred concept for meeting the project goals. Higher velocities occur through the entire channel length and extend out into the Lake at the location of larger depths, thereby reducing the sedimentation rate within the Thorofare.

6.2.3 Assessment Results

An extension should be considered in final alternative development to aid in diverting longshore lake shore sediment transport toward deeper water away from the Thorofare, and to provide an opportunity to place dredged material on the lake side of the breakwater. A rotation of 15 degrees appears to be the most efficient alternative relative to benefit and cost. Increased velocities in the Thorofare are anticipated for all rotated alignments. Increased velocity will require additional considerations for structure design along the Thorofare to mitigate potential for localized scour.

6.3 Breakwater Structure Type Alternatives Evaluation

A review of breakwater structure types was conducted to aid in the development of a feasible, preferred concept. Additional details are outlined in Appendix E.2 – Thorofare Improvements Breakwater Alternatives Evaluation. Key characteristics considered were the following:

- Compatibility with new rotated alignment and extension.
- Compatibility with Thorofare dredging and dredged material placement.
- Construction Cost.
- Future Maintenance.
- Constructability.

The alternatives to be considered would be required to be solid (low porosity) to prevent sediment from passing through the structure, durable to resist hydrodynamic forces of flood flows from the Thorofare and storm waves from the south, provide an option to place dredged material along the lake side of structure, and a long service life with reduced level of maintenance and repair relative to the current breakwater structure.

Based on the results of the Thorofare screening analysis results, prior similar project experience, and outreach to stakeholders, the following three alternatives were developed for assessment:

- Alt A - Bio-engineered: This alternative consists of materials natural to the Thorofare and Lake shoreline areas such as gravel, cobble, boulders, sand, geotextile fabric, vegetative plantings and in localized areas large wood debris (Figure 6-6). This alternative would require much flatter slopes for stability of installed materials.
- Alt B – Rubblemound: This alternative consists of imported large angular stone, bedding stone and geotextile fabric (Figure 6-7). This alternative would require a smaller footprint than Alt A.
- Alt C – Sheet Pile: This alternative consists of installing steel sheet pile and a cap (Figure 6-8).

Figure 6-6 – Bio-Engineered Alt A

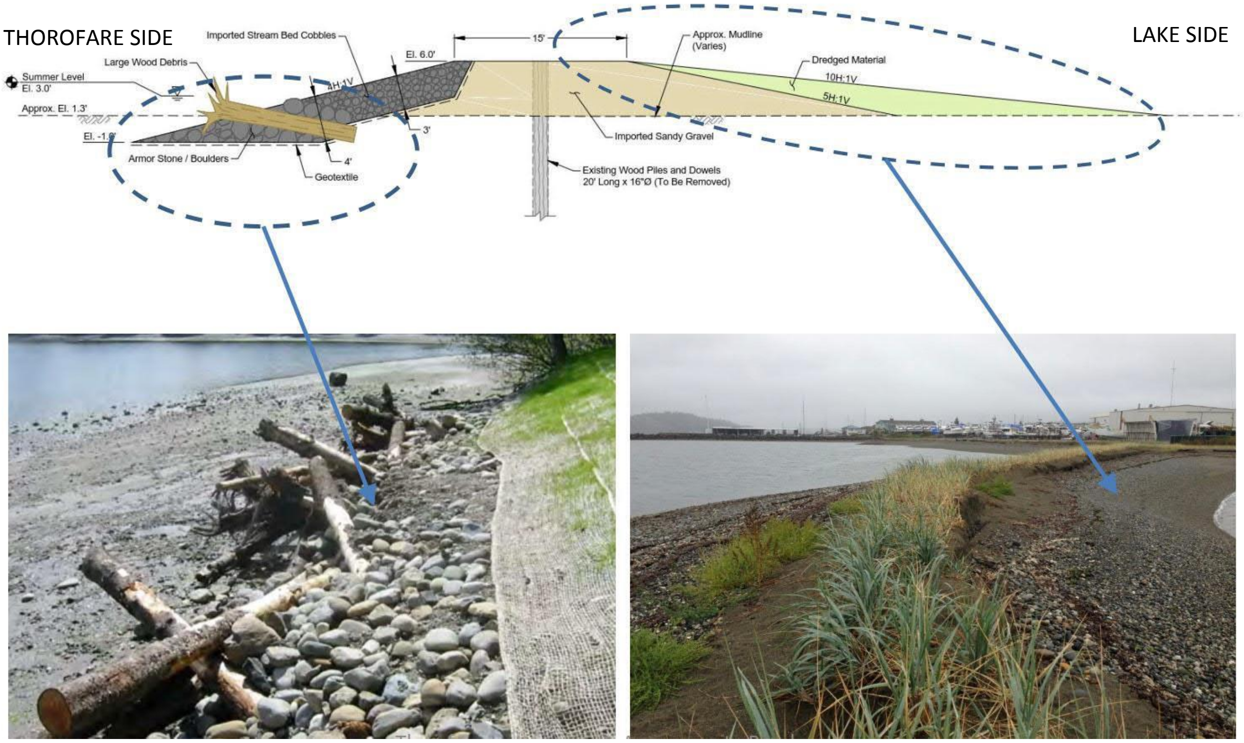


Figure 6-7 – Rubblemound Structure Alt B

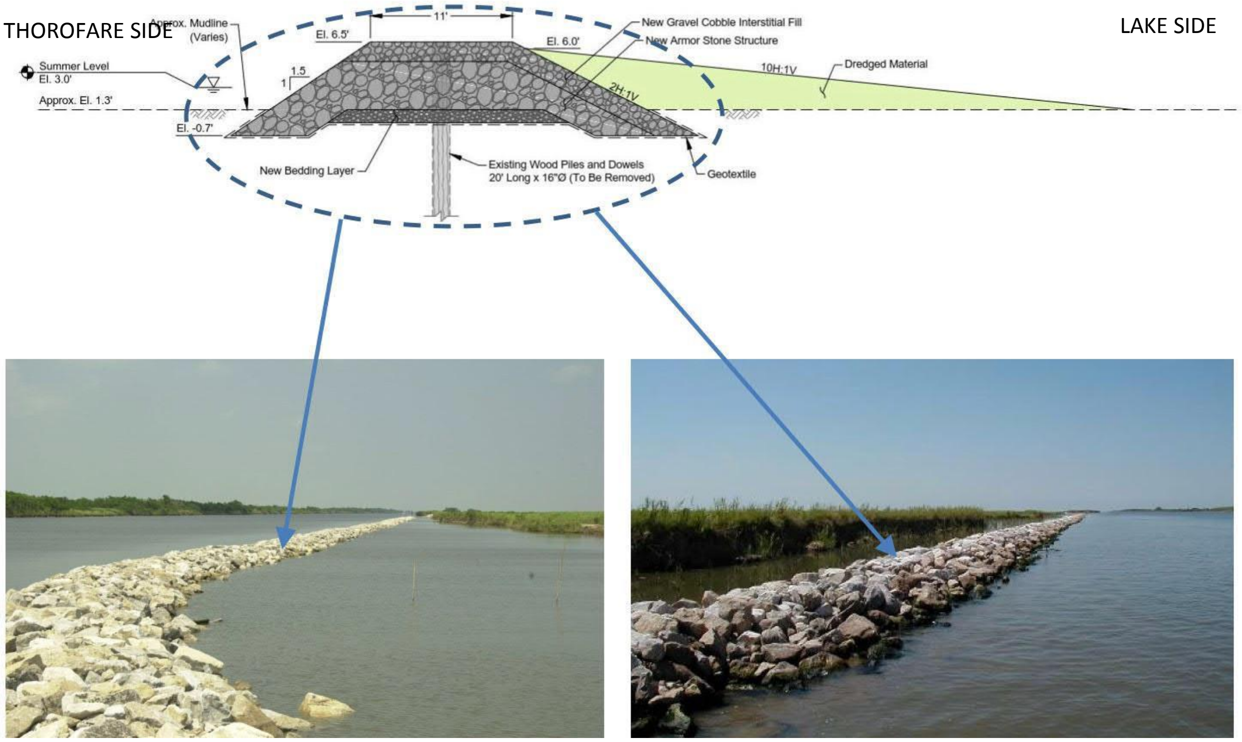
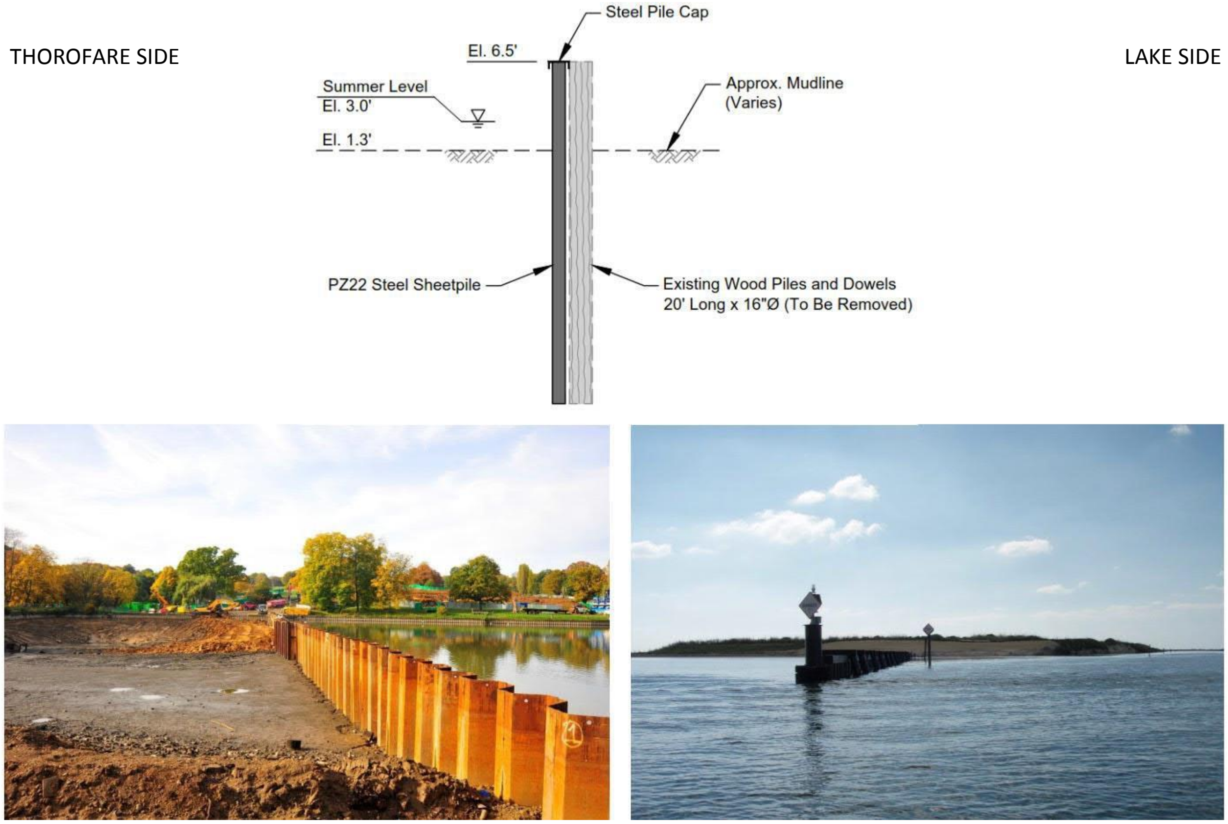


Figure 6-8 – Sheet Pile Structure Alt C



A summary of the alternatives evaluation is outlined in Figure 6-9. All of the alternatives were determined to be feasible for the project site. The following summarizes the results of the structure type alternatives evaluation:

- Dredging Considerations: Combine dredging concept with sediment protection feature (breakwater replacement) to ensure long-term sustainability of navigation through the Thorofare, enhance the lake shore side of the structure, and aid in reducing construction costs for dredging.
- Breakwater Orientation: Extension and rotation are beneficial and recommended for the breakwater replacement. Final orientation to be determined during the next phase of design.
- Type: Low porosity (sediment tight) to keep hydrodynamics separated between the lake and Thorofare, and to prevent sediment from passing through the structure. A combination of structure types could also be considered (for example, Alt A along west segment at connection to existing upland, then transitioning to Alt B for eastern segment). Alt B and C were determined to be the best suited for the outer portions of the rotated and extension alignment.
- Cost: The cost of the alternatives are similar. Project funding can be developed based on the preferred alignment concept. Development of the final material type could be done during the next phase based on more detailed discussion with regulatory agencies.

Figure 6-9 – Evaluation Summary Table

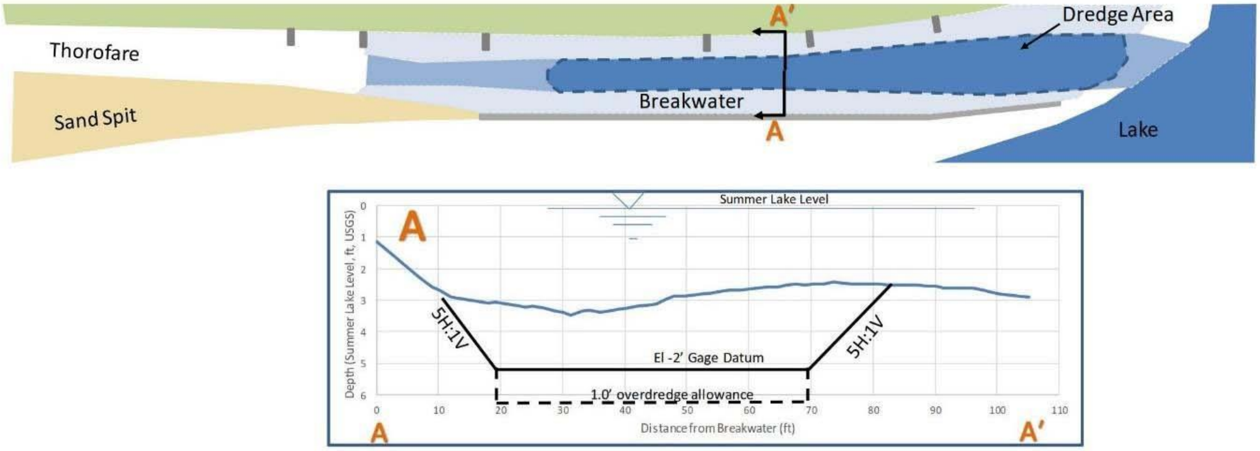
Alt Type	Constructability	Compatibility w/ Site Conditions	Cost/FT	Maintenance
A Bio-Engineered	Poor to Good (near entrance) Good elsewhere	High	$1,200 to $1,500	Med
B Stone	Good to Excellent	High-Medium	$1,100 to $1,400	Low
C Sheet Pile	Good to Excellent	Medium	$1,300 to $1,600	Low-Med

6.4 Thorofare Dredging Assessment

The purpose of the assessment was to review the dredge location, volumes, dredging equipment, and disposal/beneficial reuse options and concepts to aid in the development of a recommended Thorofare Navigation Improvement concept and corresponding construction costs. The assessment included a review of dredging methods (hydraulic, bucket dredge from a barge, and bucket dredge from a temporary earth berm) and dredged material placement and disposal (in-water beneficial reuse, in-water deep disposal, beach nourishment, upland near the project site, and upland offsite). Results of the assessment are summarized in Appendix E.1 Dredging Assessment & Constructability.

The dredging area to improve navigation was developed based on the established design criteria (size of vessel less than 26-foot length, minimum required depth of 4 to 5 feet, navigation aid marking and minimize future maintenance). A navigable width of 50 feet was developed based on the type of vessel use and industry standards. A schematic plan of the dredging requirements for improving navigation in the Thorofare are described in Figure 6-10.

Figure 6-10 – Schematic plan of dredging requirements to improve navigation in Thorofare



Note: Dredging width to be finalized; 50' is estimate for assessment phase only. Likely range is 40 to 75' width.

Final selection of the optimal dimensions and location will be conducted in the next phase of engineering. The dredging work is estimated to not require more than 10,000 cubic yards for a 5-foot depth (at 3.0-foot gage water level). A constructability review was also conducted and the results are described in the Appendix E.1 and in Section 7 of this report.

6.5 Thorofare Conceptual Design

Conceptual-level engineering design was conducted for each of the three breakwater alternatives and for the preferred dredging concept. Details for the conceptual design are shown in Appendix E.3.

7 Implementation

7.1 Constructability

Constructability is an important feature of a design, affecting the ability to efficiently build the planned improvements. A review of constructability was conducted to aid the feasibility evaluation of the improvement concepts and to assist in the development and refinement of the estimated construction costs. The important factors affecting the constructability were identified for both the outlet dam structure and the Thorofare project areas.

7.1.1 Outlet Dam Structure

Evaluation of construction requirements for the outlet dam structure were reviewed during the development of construction cost estimates. The following were determined to be important aspects of constructing the outlet dam improvements:

- Work area is easily accessible via an existing, gated access road from Highway 57.
- Work would need to be conducted in late summer and early fall during the time period of low base flow.
- Diversion of the river and dewatering of the in-stream work area through the installation of cofferdams would be required to conduct improvements to the scour apron and any improvement to the existing outlet dam structure concrete slab.
- Summer pool stored water would likely need to be removed in advance of the normal early October time period during the year of construction to ensure adequate time is available to conduct the work. Having an early fall work window would reduce risk to the Contractor during execution of the in-stream work.

7.1.2 Thorofare

Evaluation of construction requirements for the Thorofare improvement were reviewed during the development of construction cost estimates. Figure 7-1 outlines the Thorofare improvement features reviewed during the assessment. The following were determined to be important aspects of constructing the Thorofare Improvements.

7.1.2.1 General

- The project area is located in a remote area that is likely not reachable directly by land for mobilization of equipment. Mobilization utilizing floating equipment is assumed to be required from an existing public launch facility.
- Construction season depends on type of equipment and methods employed to conduct the work. Floating equipment will require a minimum of 2 to 3 feet draft to operate and therefore will require the lake to be at the summer recreation season 3.0-foot gage level to conduct the work. A construction season of at least 2 to 3 months would likely be required; thereby requiring construction to start earlier in the recreation season or the lake being held at the summer recreational level longer into the fall past the normal drawdown time period.
- Secure easement for access to the lake for staging and mobilization of equipment to the project site since land access at the site is not presently available.

Figure 7-1 – Thorofare improvement features reviewed during assessment

7.1.2.2 Dredging

- Use of a hydraulic pipeline dredge would be the most efficient and effective dredging method if a nearby dredged material placement site can be secured. Mechanical bucket dredging from a flexi-float system is a feasible alternative and would be required if a nearby placement site could not be secured. An offsite sediment handling area (such as a boat launch) would be needed to transfer materials from bin barges to an upland re-handling area.
- Refinement of the dredge prism (width, depth, length, and location) to be optimized during the next phase of more detailed engineering and to work collaboratively with the breakwater replacement system. Analysis of the navigation channel requirements and evaluation of geomorphologic processes indicated dredging requirements would be approximately 10,000 cubic yards.
- Pursuit of multiple dredging methods and placement sites during the permit approval process to provide maximum flexibility for construction to minimize the cost and time period for construction.

7.1.2.3 Breakwater

- Use of flexi-float for access to the site for pile driving crane or excavator for stone placement will be required.
- Temporary fill for equipment staging and operation may be required within the vicinity of the breakwater.
- Rubblemound breakwater may require construction during the lower pool level and thereby a fall construction period.
- Rubblemound (rock) and sheet pile breakwaters are the highest-ranking alternatives relative to performance, longevity, and lower level of maintenance.

- Placement of dredged material along lake side (south side) should be an integral part of the breakwater design.
- Orientation of the breakwater alignment, length and end geometry are important to the sustainability of the navigation channel dredging improvements.

7.2 Fisheries Assessment

An assessment of fisheries resources in the Priest Lake system were reviewed to aid the study in evaluation of improvement alternatives. The native fish assemblages in Priest Lake and its tributaries include: bull trout, west slope cutthroat trout, mountain whitefish, northern pikeminnow, suckers, redside shiner, and sculpin. Brook trout were introduced prior to 1920, lake trout were introduced in 1925, and kokanee salmon were introduced beginning in 1942. Additional information regarding the Fisheries Assessment work are provided in Appendix A.4.

7.3 Conceptual Design

Conceptual engineering design was conducted for the Thorofare and outlet dam structure improvements and are described in Appendix E.3.

7.4 Construction Costs

Based upon the evaluation of alternatives described herein, it is estimated the cost for the improvements for the outlet dam structure and Thorofare would be approximately $5M in 2018 dollars. Additional refinement of construction costs would be conducted during the next phase of engineering based on a more refined design concept. A 35-percent contingency is included to account for uncertainties related to this phase of planning and design. Major changes to the project criteria could also have an impact to the estimated construction costs. These are further outlined in the subsequent Risks and Unknowns (7.6) and Recommendations for Future Work (7.7) sections of the report. These could include outlet dam foundation improvements to address liquefaction, environmental impacts mitigation, and offsite dredged material disposal a the Thorofare.

The development of construction costs is affected by the bidding environment and pool of available experienced contractors at the time of project bidding. It is assumed multiple local qualified contractors would be interested and available to conduct the work. Should economic conditions change, the bidding environment could become less competitive. Additionally, if project elements are phased and broken up into multiple elements, additional costs could apply.

7.5 Environmental Permitting Assessment

Modifications to the Priest Lake Thorofare (Breakwater and Dredging) and Outlet Dam Structure will require an extensive and thorough permit review process. Regulatory authorization will be required from Bonner County, the State of Idaho, and the federal government. The following summarizes the permits and authorizations that may likely be needed for implementing the project:

- Bonner County – Conditional Use permit.
- Idaho Department of Lands (IDL) – Lake Encroachment permit under the Idaho Lake Protection Act.
- Idaho Department of Environmental Quality (IDEQ) – Section 401 water quality permit.
- Idaho Department of Water Resources (IDWR) – Stream channel alternation and dam safety authorization for modifications to the outlet dam structure.
- Idaho Department of Fish & Game (IDFG) – Recommendations for permit conditions issued by other agencies.

- U.S. Army Corps of Engineers (USACE) – Section 404 permit for dredging and filling work and potentially Section 10 permits.

The timeline for regulatory permitting varies but can be 6 to 9 months, depending on the type of proposed work and considerations for processing and issuing the permits.

7.6 Risks and Unknowns

The study was conducted at a conceptual level engineering and planning and is based upon available existing data, observations and information provided by stakeholders. The following summarizes potential risks and unknowns associated with the study work and conclusions that would need to be addressed in the next phase of the project:

7.6.1 Outlet Dam Structure

- A detailed condition assessment of the gates will need to be conducted to aid a more detailed structural assessment.
- Confirm capacity of reinforced concrete piers, hoist gear, trunnion and bearing mounts, and gate structural capacity in more detail in the next phase to finalize the scope of improvements and upgrades required.
- Confirm existing gate operators can be retrofitted or require replacement with new system for electric power drive.
- Additional soils explorations should be performed to identify ground conditions at the outlet dam site. Additional data collection costs will likely be needed depending on the level of dam improvements required to mitigate earthquake liquefaction.

7.6.2 Thorofare

- Geotechnical subsurface conditions at the breakwater location for pile design or rock structure mound settlement analysis. Borings would need to be conducted during the next phase of design. Soils for building new breakwater were assumed to be granular. If finer grained soils or soft soils are encountered, additional cost for the structure could be incurred.
- Sediment characterization for confirmation of sediment quality for reuse in an aquatic environment. This would require grab sampling and laboratory testing in accordance with the Pacific Northwest Sediment Evaluation framework.
- Dredged material disposal assumed to be either nearshore upland or nearby aquatic reuse (adjacent to breakwater). Further investigation thereof will be required in the next phase of the project. If a nearby disposal site is not available thereby requiring transport of dredged material a further distance, additional costs will be incurred.

7.6.3 Regulatory Permitting

- Environmental permitting is assumed to require an individual permit and some level of NEPA compliance. It is assumed that an Environmental Impact Statement will not be required for the range of preferred improvements developed in this phase.
- Mitigation costs for in-water improvements (Breakwater, Dredging, Dam) were assumed to be minimal and primarily self-mitigating. Mitigation requirements will need to be investigated in more detail in the next phase to determine the need to offset any impacts and the costs associated with that required mitigation.

7.7 Recommendations for Future Work

Implementation of the project would require the following next steps to proceed toward implementation:

- Data Collection: Collection of additional data would be required for the Thorofare and Outlet Dam Structure locations. The Outlet Dam Structure would require conducting additional borings and piezometer testing, topographic and hydrographic surveying, and water level measurements (correlate water levels between lake gage and dam locations). The Thorofare would require supplemental hydrographic surveying, stage-discharge measurements, and sediment samples for testing of sediment quality.
- Engineering Analysis: There may be risk of liquefaction of the ground below the existing dam and proposed apron during a seismic event. The risk of liquefaction needs to be quantified in subsequent phases of the project, and should entail a geotechnical site investigation to determine the properties of the foundation soils at the site. Ground improvement of the soils may be required to mitigate the risk of liquefaction. Additional engineering analysis of the Thorofare hydrodynamics utilizing new data collection to aid in the optimization of the breakwater and dredging improvements.
- Engineering Design: Proceed with Front End Engineering Design (FEED) and final engineering design for the preferred concept utilizing newly collected data. Conduct additional hydraulic engineering analysis to aid in the refinement of design concepts.
- Public Outreach: Conduct outreach to stakeholders and general public regarding the final concepts and details for changes in lake operations and improvements to the Thorofare and Outlet Dam Structure.
- Environmental Assessment: Conduct additional field assessments to review the conditions of the shoreline areas, wetlands, and habitat relative to the anticipated higher pool level in the months of July and August to document for quantifying any impacts prior to finalizing regulatory permit applications.
- Regulatory Permitting: Initiation of regulatory agency consultation during preliminary design phase prior to submission of permit application documents. Prepare environmental assessments and permit application documents for submission to local, state, and federal regulatory agencies.
- Bidding and Contracting: Upon receipt of regulatory approvals, advertise project for public bid process, conduct bid evaluations, and award contract to selected construction company.
- Construction: Conduct construction management plan to administer construction contract and monitor progress of construction. Complete the project within the allotted construction time period.

8 Findings and Conclusions

This study summary report describes alternatives for improvements needed for meeting summer recreation period minimum water levels, outlet dam minimum discharge (60 cfs), and for improved navigation in the Thorofare. Raising the pool level by 6 inches during the months of July and August to provide the necessary storage needed to meet the minimum outlet dam discharge of 60 cfs is feasible. The preferred alternative consists of improvements to both the outlet dam and Thorofare to provide up to a 6-inch raise in water levels during the summer recreational season for predicted dry years.

Outlet dam improvements would include gate strengthening, gate extension, electrical operator upgrade and a new concrete scour apron. Improvements to the outlet dam are required to improve operations, stability, and improve the efficiency of utilizing the stored water resource. Additional investigation of dam stability for earthquake performance and need for improvements to mitigate liquefaction are needed and were beyond the scope of this study.

Thorofare improvements would include replacement of the existing porous timber breakwater with a new solid breakwater (rubblemound, bio-engineered, sheet pile, or a combination thereof), and dredging. Improvements are needed to maintain sustainability of navigation from Priest Lake into the Thorofare and upper Priest Lake.

9 References

References are listed in Appendix A.1.

10 Appendices

A. Study Methodology

A.1 Existing Data and Bibliography Summary

A.2 New Data Collection

A.3 Basis of Analysis and Criteria

A.4 Fisheries Assessment Memorandum

A.5 Public Outreach Plan

A.6 Public Outreach Comments

A.1 Existing Data and Bibliography Summary

Priest Lake Water Management Study

Existing Data & Bibliography Summary

				Lake or Streams		Dam/Thorofare								
Year	**Agency**	**Title**	**Dam Operation**	**Water Levels**	**Map/GIS data**	**Condition Assessments**	**Sediments**	**Erosion**	**Elevation**	**Land Use**	**Design Documents**	**Hydrologic Data/Analysis**	**Water Quality**	**Fish**
2010	BCPD	Zoning District Map			X									
2010	BCPD	Projected Land Use Map			X									
2016	Bonner County Daily Bee	Judge Orders Removal of Fill at Priest Lake						X						
2003	Bonner County Planning Dep	Natural Resources Component of BC Comprehensive Plan		X								X	X	
	Bonner County Planning Dep	Priest Lake Snowmobile Trails			X									
1978	CH2MHill	Review of Priest Lake Dam Design				D						X		
2009	CHE	Priest Lake Thoroughfare Breakwater Analysis SOW				T								
2009	CORPS - ERDC	Updated Pend Oreille River model Development, Calibration, and Application										X	X	
1995	DEQ	Priest Lake Management Plan - Priest Lake Project						X					X	
1997	DEQ	Phase 1 Diagnostic Analysis Priest Lake		S	X		X					X	X	
2012	DEQ	Priest River Subbasin Assessment and Total Maximum Daily Load		X	X								X	
2015	DEQ	Priest River Subbasin Assessment and Total Maximum Daily Load		X	X								X	
2003	IDFG	Amendments to Priest River Basin Plan	X											X
2014	IDFG	Management Planning for Priest and Upper Priest Lakes												X
2015	IDFG	IDFG in Comprehensive Effort to learn more about Lake Trout/Kokannee in Priest Lake												X
1977	IDWR	Phase 1 Construction of Sheet Pile Cutoff Wall - Contract Documents									X			
1978	IDWR	Replacement of Control Structure - Contract Documents									X			
1978	IDWR	Replacement of Control Structure - Plans									X			
1995	IWRB	Priest River Basin Comprehensive State Water Plan	X	X									X	X
2010	IWRB	State of Idaho Water Resource Inventory												
2012	IWRB	State Water Plan												
2016	IWRB	IWRB Work Session for Meeting July 21, 2016 - Priest lake Water Management Study	X	X								X		
1974	IWWRI	Analysis of Alternative Water Release Operations for Priest Lake	X				X						X	X
2017	Lakes Commission	Lake Pend Oreille and Priest Lake River Commission - Issues		X										X
2017	Lakes Commission	Lake Pend Oreille and Priest Lake River Commission - Home		X										
2006	NRCS	8-Digit Hydrologic Unit Profile			X							X		
2016	NRCS	Washington Water Supply Outlook Report		X								X		
2017	NRCS	Idaho Water Supply Outlook Report												
2004	NWCC (Northwest Conserva	Intermountain Province Subbasin Plans								X				
2004	PWCC													
2013	USGS	Report and Affidavit Relative to Operation of Dam	X	X								X	X	
2015	USGS	Idaho Hydrologic Update		X								X		
2016	USGS	Idaho Hydrologic Update		X								X		
1992	CORPS	Priest Lake Outlet Structure Study	X											
2015	IDWR	Status of Water Surface Elevation in Priest Lake Memorandum												
2002	IDWR ??	Priest Lake Agreement Changes - Producer Price Index												
1992	IDWR ??	Affidavit of William Ondrechen												
2002	IDWR ??	Operations and Maintenance Agreement between IDWR and Avista Corporation -Final	X											
2001	IDWR ??	Operations and Maintenance Agreement between IDWR and Avista Corporation - DRAFT	X											
1984	IDWR ??	Amendment to Contract between IDWR and Washington Water Power Company												
2003	IDWR ??	Amendment to Outlet Structure Operations	X											
1979	IDWR ??	Proposed Modified Operation of Priest Lake Memorandum and Data	X	X										
1983	IDWR ??	Priest Lake Storage Contract Update Review												
1984	IDWR ??	Discussion Paper for Meeting IDWR and Wash Water Power Co												
1984	IDWR	Water Rights Comments on Huckleberry Project												
1979	IDFG	Lake and Reservoir Investigations												X
2003	IDWR	Priest Lake Outlet Structure Operation Plan 2003	X	X										
1991	IDWR & IDFG	Study of Fishery Resources needed on Lower Priest River												
2016	IDWR	Priest Lake Water Levels -Low Years		X										
2017	IDWR	April 1st Water Supply Forecast		X										
1997	IDWR	Water Right License		X										
1999	IDWR	Water Right License - Backfile												
2009	IDWR	Beneficial Use Field Report												
2009	IDWR	Issuance of License												
1932	Museum	Dredging the Thorofare River (connects Priest Lake and Upper Priest Lake), Idaho												
1950	Idaho State Legislature	Idaho Statutues Title 70 Watercourses and Port Districts												
1989	IDL	Thorofare Breakwater Replacement				T								
2015	IDWR	Dam Safety Report	X									X		

A.2 New Data Collection

Point	UTM-11 meters		NGVD29 (meters)	Observations	Description
	East	North			
RTK Base	**511140.816**	**5398543.615**	743.54	10,686	Iron pipe base of pile
RTK1	511112.164	5398546.701	744.17	30	End of Breakwater - on spit
RTK2	511126.160	5398545.101	744.25	31	Breakwater gap - on pile
RTK3	511131.038	5398544.576	744.26	32	Breakwater gap - on pile
RTK4	511164.600	5398539.257	744.09	29	East end of Breakwater - Upper Waler
RTK5	511251.560	5398528.103	744.21	35	East end of re-enforcement - pile
RTK6	511287.456	5398524.338	744.10	31	West end of re-enforcement - pile
RTK7	511441.026	5398508.004	744.14	28	Angle change - West end
RTK8	511514.445	5398503.314	744.36	30	Signage Board? End of old piles
RTK9	511514.904	5398509.152	744.41	29	Western end of Breakwater
RTK10	511557.602	5398535.288	745.36	35	Entrance Light - Deck
RTK11	511319.986	5398617.242	743.62	34	Corner of dock-Across Thorofare

NOTES:

1. Survey related to Temporary RTK Benchmark:
   E 511140.816 m
   N 5398543.615 m
   Height above NGVD29: +743.544m (+2,439.441')

   Corrected to USGS Gage 12393000 Datum. Gage 3' is +2,437.64' NGVD29.
   Contours shown as depth referenced to maintained lake level +3'.
2. UTM - Zone 11 Grid shown (meters). Tickmarks at 100-m x 100-m.
3. Water Level +2,438.946 ft (+3.51' USGS gage) average during survey.
4. Orthomosaic imagery collected on May 10, 2017. (GSD=2.54cm / Resolution=2.54cm/pixel).
5. The Breakwater is not a solid barrier and is elevated off the bottom in many areas. (photo)
6. Location of sediment samples shown for reference.

0 30 60
Scale in Meters

Delphis
Technical Support and Solutions, LLC

13332 69th DR SE
Snohomish, WA 98296
Tel: 425-773-0722

PRIEST_LAKE_THOROFARE
ORTHOMOSAIC-ANCILLARY_POINTS
MAY_10TH,_2017

JOB NO. 201703	FIGURE NO.6
HORIZONTAL DATUM: UTM	SURVEYED BY: KJS
ZONE: ZONE_11	DRAWING DATE: 6/16/2017
UNITS: METERS	DRAWN BY: KJS
VERTICAL DATUM: DEPTH_IN_FEET_AT_MAINTAINED_LEVEL	SCALE: 1"=30M
SURVEY DATE: MAY_11,_2017	DRAWING SIZE: 11"X17"

2017-THOROFARE-ORTHODETAILED-USGSPLUS3.DWG

⌖ RTK Base (Benchmark)

NOTES:

1. Survey related to Temporary RTK Benchmark:
   E 511140.816 m
   N 5398543.615 m
   Height above NGVD29: +743.544m (+2,439.441')
2. UTM - Zone 11 Grid shown (meters). Tickmarks at 100-m x 100-m.
3. Water Level +2,438.946 ft average during survey.
4. Background Image: Google Earth (7/11/2014).

0 100 200
Scale in Meters

Delphis
Technical Support and Solutions, LLC

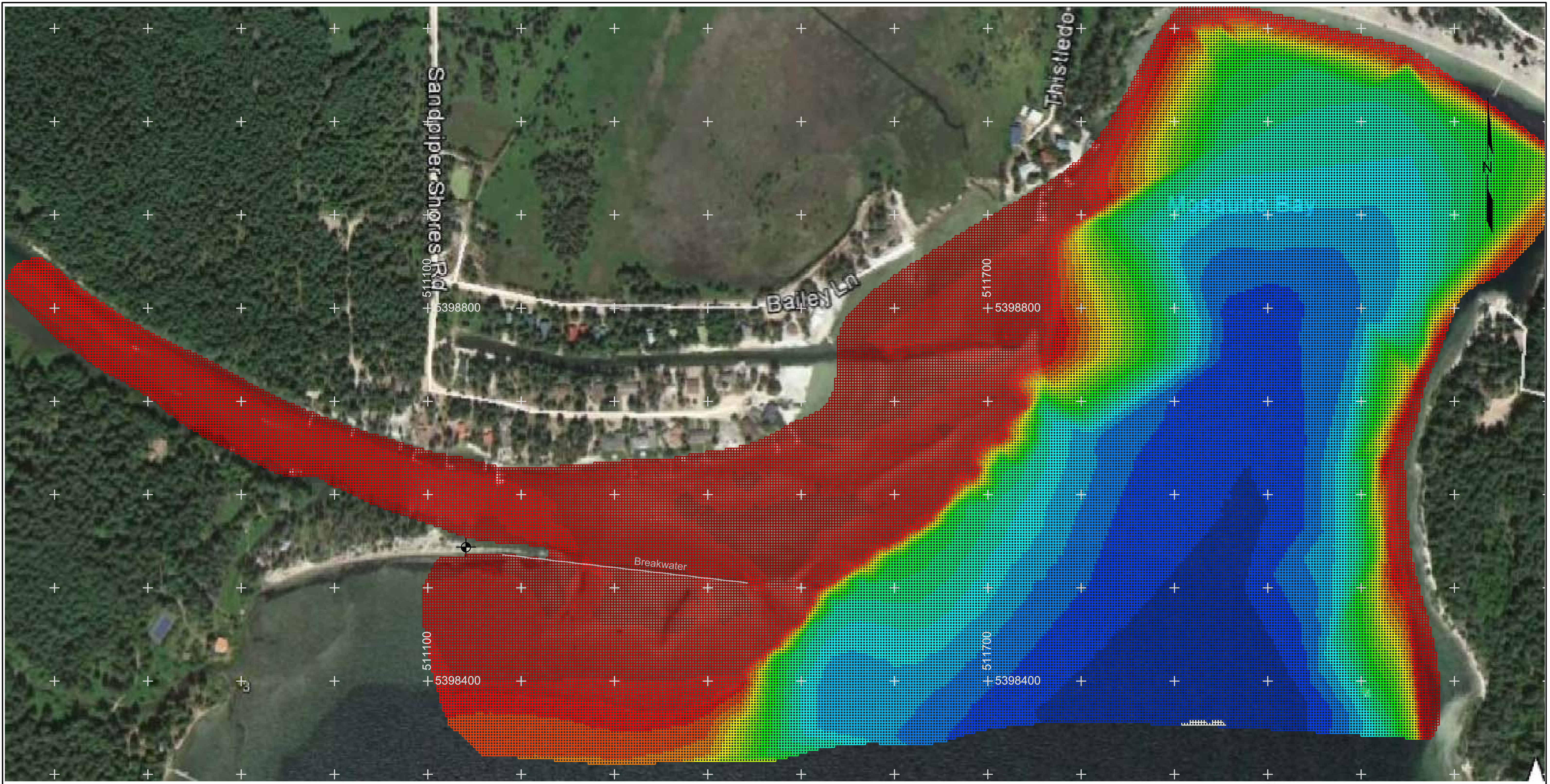
13332 69th DR SE
Snohomish, WA 98296
Tel: 425-773-0722

PRIEST_LAKE_THOROFARE
TRACKLINE_SOUNDINGS
MAY_10TH,_2017

JOB NO. 201703 **FIGURE NO.7**

HORIZONTAL DATUM: UTM
ZONE: ZONE_11
UNITS: METERS
VERTICAL DATUM:_
SURVEY DATE: MAY_11,-2017
SURVEYED BY: KJS
DRAWING DATE: 5/21/2017
DRAWN BY: KJS
SCALE: 1"=100M
DRAWING SIZE: 11"X17"

2017-THOROFARE-OTS.DWG



2017-THOROFARE-AND-BAY-100XP-USGSPLUS3

NOTES:

1. Survey related to Temporary RTK Benchmark:
   E 511140.816 m
   N 5398543.615 m
   Height above NGVD29: +743.544m (+2,439.441')

   USGS Gage 12393000 Datum. Gage 0' is +2,434.64' NGVD29.

   Corrected to Summer Water Level at +3.0' USGS Gage. Shown as Depth.
2. UTM - Zone 11 Grid shown (meters). Tickmarks at 100-m x 100-m.
3. Water Level +3.51ft (USGS) average during survey.
4. 3-meter x 3 meter Grid shown.
5. Background Image: Google Earth (7/11/2014).

0 100 200
Scale in Meters

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Technical Support and Solutions, LLC

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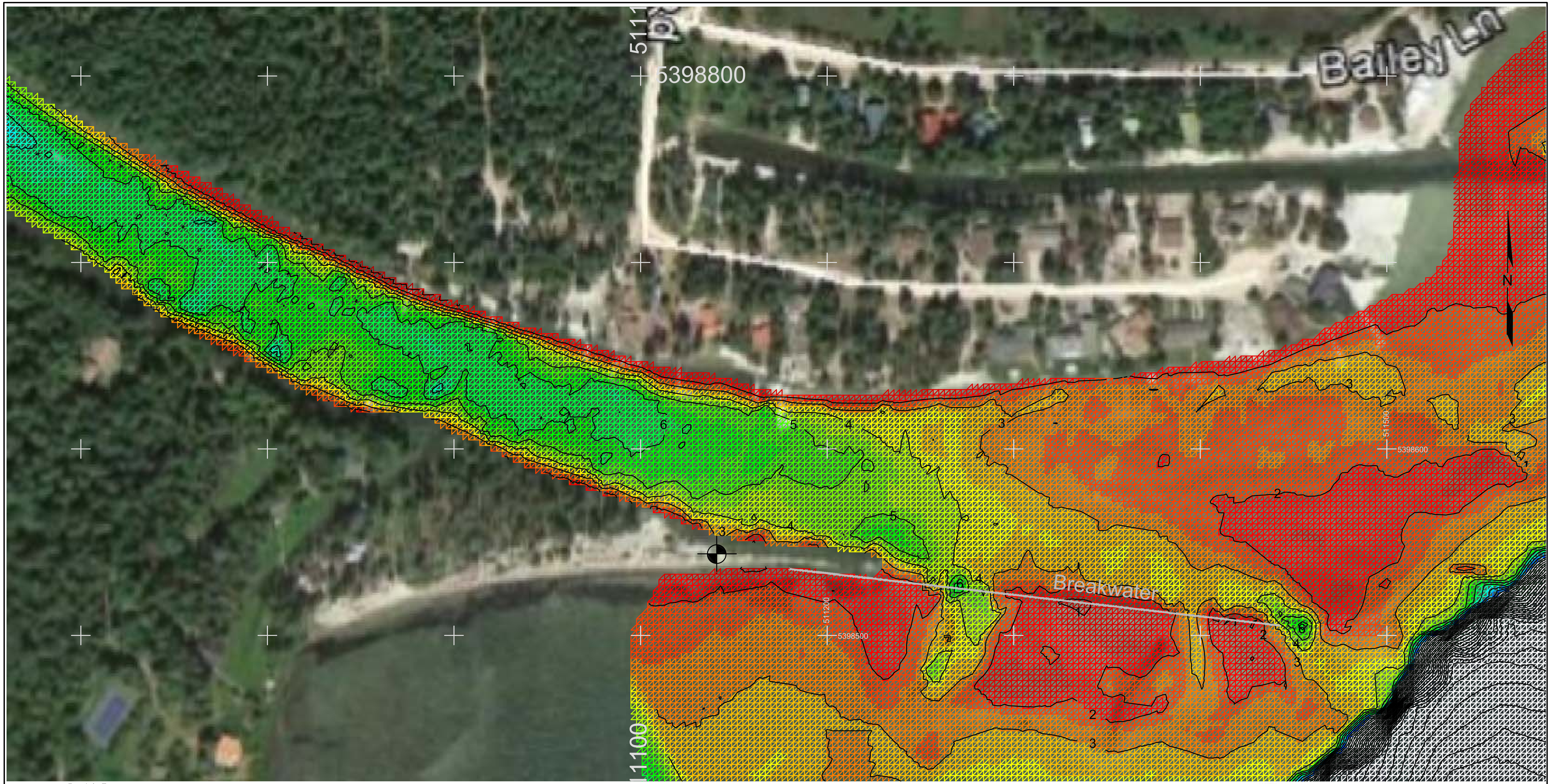
Snohomish, WA 98296
Tel: 425-773-0722

PRIEST_LAKE_THOROFARE

DEPTH_AT_MAINTAINED_+3.0FT
MAY_10TH,_2017

	JOB NO. 201703	FIGURE NO.8

HORIZONTAL DATUM: UTM	SURVEYED BY: KJS
ZONE: ZONE_11	DRAWING DATE: 6/12/2017
UNITS: METERS	DRAWN BY: KJS
VERTICAL DATUM: USGS_REFERENCED+3FT	SCALE: 1"=100M
SURVEY DATE: MAY_11,-2017	DRAWING SIZE: 11"X17"



Depth in Feet

0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0
5.5
6.0
6.5
7.0
7.5
8.0
8.5
9.0
9.5
10
Below 10

⌖ RTK Base (Benchmark)

2017-THOROFARE-OTS.DWG

NOTES:

1. Survey related to Temporary RTK Benchmark:
   E 511140.816 m
   N 5398543.615 m
   Height above NGVD29: +743.544m (+2,439.441')

   USGS Gage 12393000 Datum. Gage 0' is +2,434.64' NGVD29.

   Corrected to Summer Water Level at +3.0' USGS Gage. Shown as Depth.
2. UTM - Zone 11 Grid shown (meters). Tickmarks at 100-m x 100-m.
3. Water Level +3.51ft (USGS) average during survey.
4. Background Image: Google Earth (7/11/2014).
5. The Breakwater is not a solid structure and is elevated off the bottom in many areas. (photo)
6. Contour Interval - 1 foot.

0 50 100
Scale in Meters

Delphis
Technical Support and Solutions, LLC

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Snohomish, WA 98296
Tel: 425-773-0722

PRIEST_LAKE_THOROFARE

DEPTH_AT_MAINTAINED_+3.0FT
MAY_10TH,_2017

JOB NO. 201703
FIGURE NO.9

HORIZONTAL DATUM: UTM	SURVEYED BY: KJS
ZONE: ZONE_11	DRAWING DATE: 6/12/2017
UNITS: METERS	DRAWN BY: KJS
VERTICAL DATUM: USGS_REFERENCED+3FT	SCALE: 1"=50M
SURVEY DATE: MAY_11,_2017	DRAWING SIZE: 11"X17"

Point	UTM-11 meters		NGVD29 (meters)	Observations	Description
	East	North			
RTK Base	**511140.816**	**5398543.615**	743.54	10,686	Iron pipe base of pile
RTK1	511112.164	5398546.701	744.17	30	End of Breakwater - on spit
RTK2	511126.160	5398545.101	744.25	31	Breakwater gap - on pile
RTK3	511131.038	5398544.576	744.26	32	Breakwater gap - on pile
RTK4	511164.600	5398539.257	744.09	29	East end of Breakwater - Upper Waler
RTK5	511251.560	5398528.103	744.21	35	East end of re-enforcement - pile
RTK6	511287.456	5398524.338	744.10	31	West end of re-enforcement - pile
RTK7	511441.026	5398508.004	744.14	28	Angle change - West end
RTK8	511514.445	5398503.314	744.36	30	Signage Board? End of old piles
RTK9	511514.904	5398509.152	744.41	29	Western end of Breakwater
RTK10	511557.602	5398535.288	745.36	35	Entrance Light - Deck
RTK11	511319.986	5398617.242	743.62	34	Corner of dock-Across Thorofare

NOTES:

1. Survey related to Temporary RTK Benchmark:
   E 511140.816 m
   N 5398543.615 m
   Height above NGVD29: +743.544m (+2,439.441')
2. UTM - Zone 11 Grid shown (meters). Tickmarks at 100-m x 100-m.
3. Water Level +2,438.946 ft average during survey.
4. Orthomosaic imagery collected on May 10, 2017. (GSD=2.54cm / Resolution=2.54cm/pixel).
5. The Breakwater is not a solid structure and is elevated off the bottom in many areas. (photo)

0 30 60
Scale in Meters

Delphis
Technical Support
and Solutions, LLC

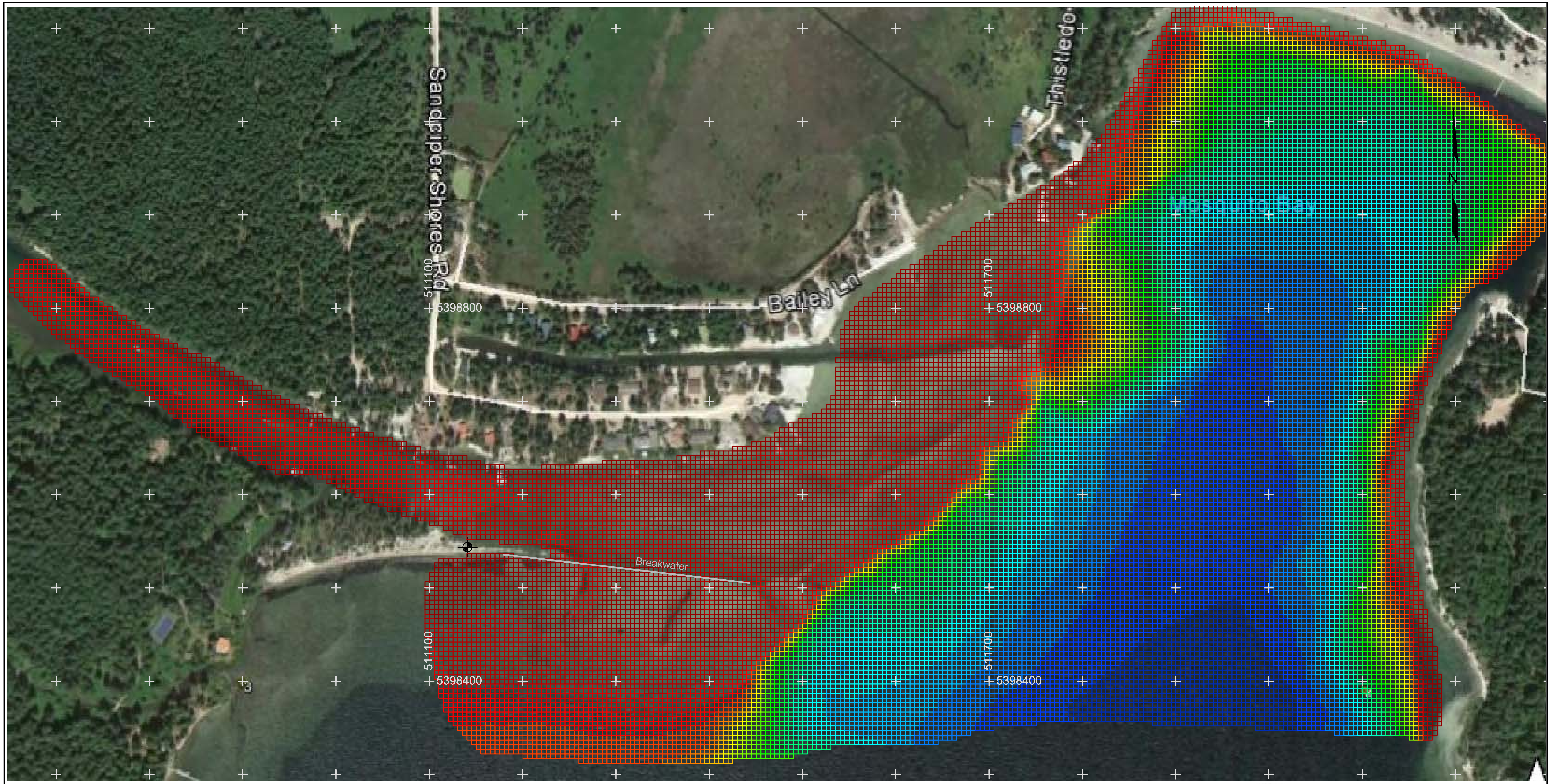
13332 69th DR SE
Snohomish, WA 98296
Tel: 425-773-0722

PRIEST_LAKE_THOROFARE
ORTHOMOSAIC-ANCILLARY_POINTS
MAY_10TH,_2017

JOB NO. 201703
FIGURE NO.6

HORIZONTAL DATUM: UTM
ZONE: ZONE_11
UNITS: METERS
VERTICAL DATUM: NGVD29
SURVEY DATE: MAY_11,_2017
SURVEYED BY: KJS
DRAWING DATE: 5/21/2017
DRAWN BY: KJS
SCALE: 1"=30M
DRAWING SIZE: 11"X17"

2017-THOROFARE-ORTHODETSAILED.DWG



Z Value Legend (Feet-NGVD29)
- 2437.0
- 2434.8
- 2432.5
- 2430.3
- 2428.0
- 2425.8
- 2423.5
- 2421.3
- 2419.0
- 2416.8
- 2414.5
- 2412.3
- 2410.0
- 2407.8
- 2405.5
- 2403.3
- 2401.0
- 2398.8
- 2396.5
- 2394.3

⌖ RTK Base (Benchmark)

2017-THOROFARE-OTS.DWG

NOTES:

1. Survey related to Temporary RTK Benchmark:
   E 511140.816 m
   N 5398543.615 m
   Height above NGVD29: +743.544m (+2,439.441')
2. UTM - Zone 11 Grid shown (meters). Tickmarks at 100-m x 100-m.
3. Water Level +2,438.946 ft average during survey.
4. 5-meter x 5 meter Grid shown.
5. Background Image: Google Earth (7/11/2014).

0 100 200
Scale in Meters

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Technical Support and Solutions, LLC

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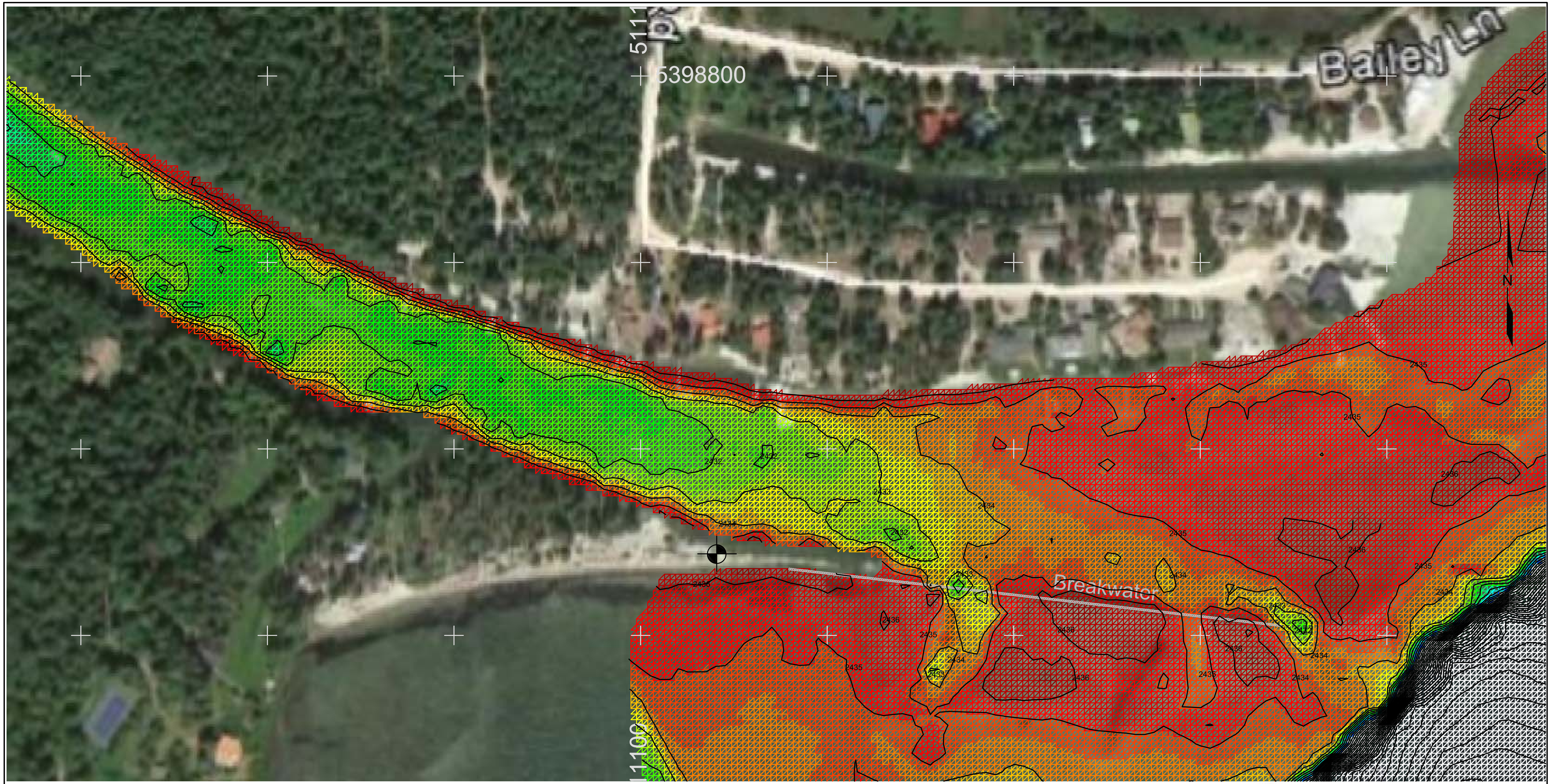
Snohomish, WA 98296
Tel: 425-773-0722

PRIEST_LAKE_THOROFARE
BATHYMETRY
MAY_10TH,_2017

JOB NO. 201703	FIGURE NO.8
HORIZONTAL DATUM: UTM	SURVEYED BY: KJS
ZONE: ZONE_11	DRAWING DATE: 5/21/2017
UNITS: METERS	DRAWN BY: KJS
VERTICAL DATUM: NGVD29	SCALE: 1"=100M
SURVEY DATE: MAY_11,_2017	DRAWING SIZE: 11"X17"



Z Value Legend (Feet NGVD29)

🟥	2437
🟥	2436.5
🟥	2436
🟥	2435.5
🟧	2435
🟧	2434.5
🟧	2434
🟨	2433.5
🟨	2433
🟩	2432.5
🟩	2432
🟩	2431.5
🟩	2431
🟩	2430.5
🟩	2430
🟦	2429.5
🟦	2429
🟦	2428.5
🟦	2428
🟦	2427.5
🟦	2427
⬜	Below - 2427
⌖	RTK Base (Benchmark)

NOTES:

1. Survey related to Temporary RTK Benchmark:
   E 511140.816 m
   N 5398543.615 m
   Height above NGVD29: +743.544m (+2,439.441')
2. UTM - Zone 11 Grid shown (meters). Tickmarks at 100-m x 100-m.
3. Water Level +2,438.946 ft average during survey.
4. 5-meter x 5 meter Grid shown.
5. Background Image: Google Earth (7/11/2014).
6. The Breakwater is not a solid structure and is elevated off the bottom in many areas. (photo)
7. Contour Interval - 1 foot.

0 50 100

Scale in Meters

Delphis
Technical Support and Solutions, LLC

13332 69th DR SE
Snohomish, WA 98296
Tel: 425-773-0722

PRIEST_LAKE_THOROFARE

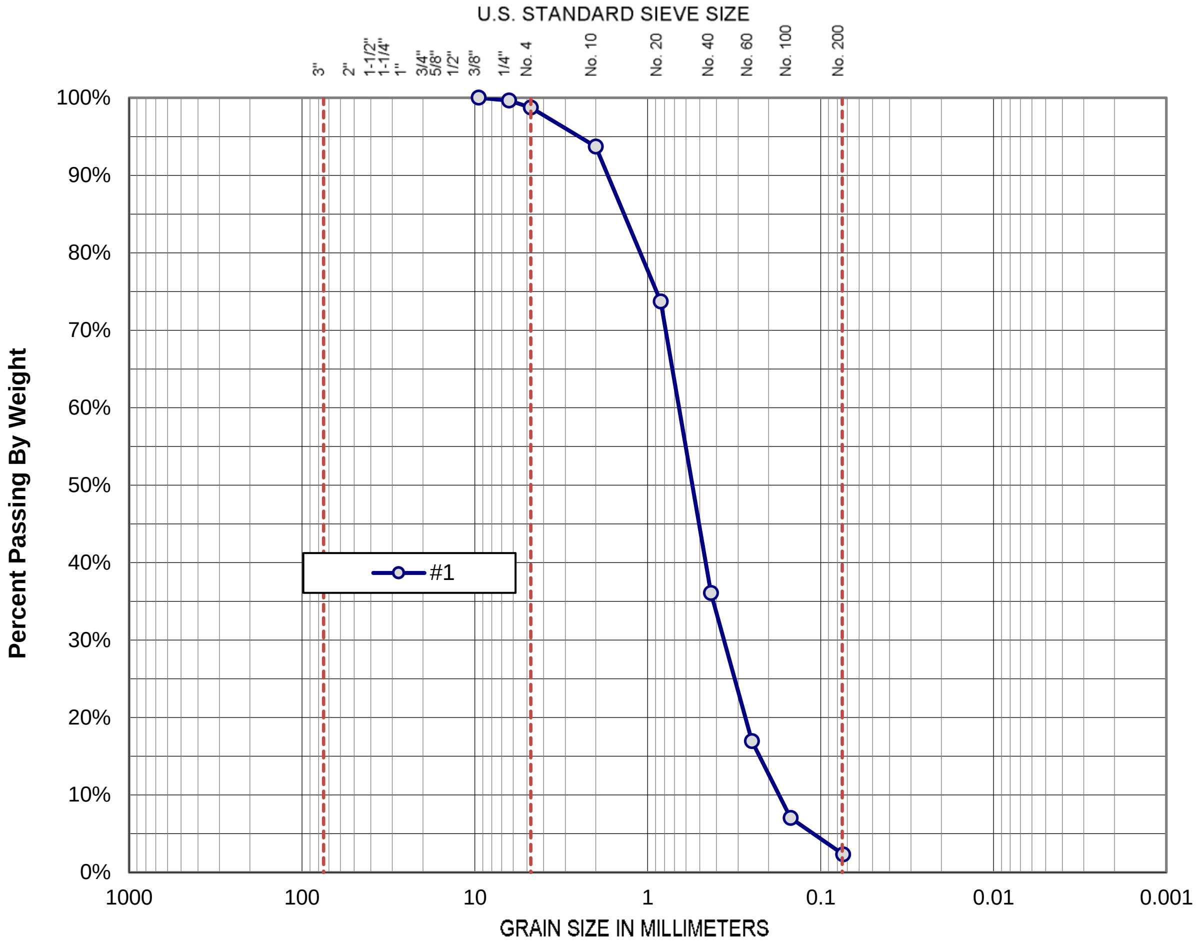
BREAKWATER-BATHYMETRY

MAY_10TH,_2017

JOB NO. 201703
FIGURE NO.9

HORIZONTAL DATUM: UTM	SURVEYED BY: KJS
ZONE: ZONE_11	DRAWING DATE: 5/21/2017
UNITS: METERS	DRAWN BY: KJS
VERTICAL DATUM: NGVD29	SCALE: 1"=50M
SURVEY DATE: MAY_11,_2017	DRAWING SIZE: 11"X17"

2017-THOROFARE-OTS.DWG



COBBLES	GRAVEL		SAND			FINES - SILT and CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

SUMMARY

PERCENT PASSING
BY WEIGHT

Screen	%
3"	100
2"	100
1.5"	100
1"	100
3/4"	100
5/8"	100
1/2"	100
3/8"	100
1/4"	100
No. 4	99
No. 10	94
No. 20	74
No. 40	36
No. 60	17
No. 100	7
No. 200	2.3

Sample ID	WC %	C_U	C_C	USC	D2487 Classification
#1	28.3	3.8	1.1	SP	fine to medium sand with trace silt
Estimated values for C_u C_c calculation	D_{60} 0.7		D_{30} 0.36		D_{10} 0.175

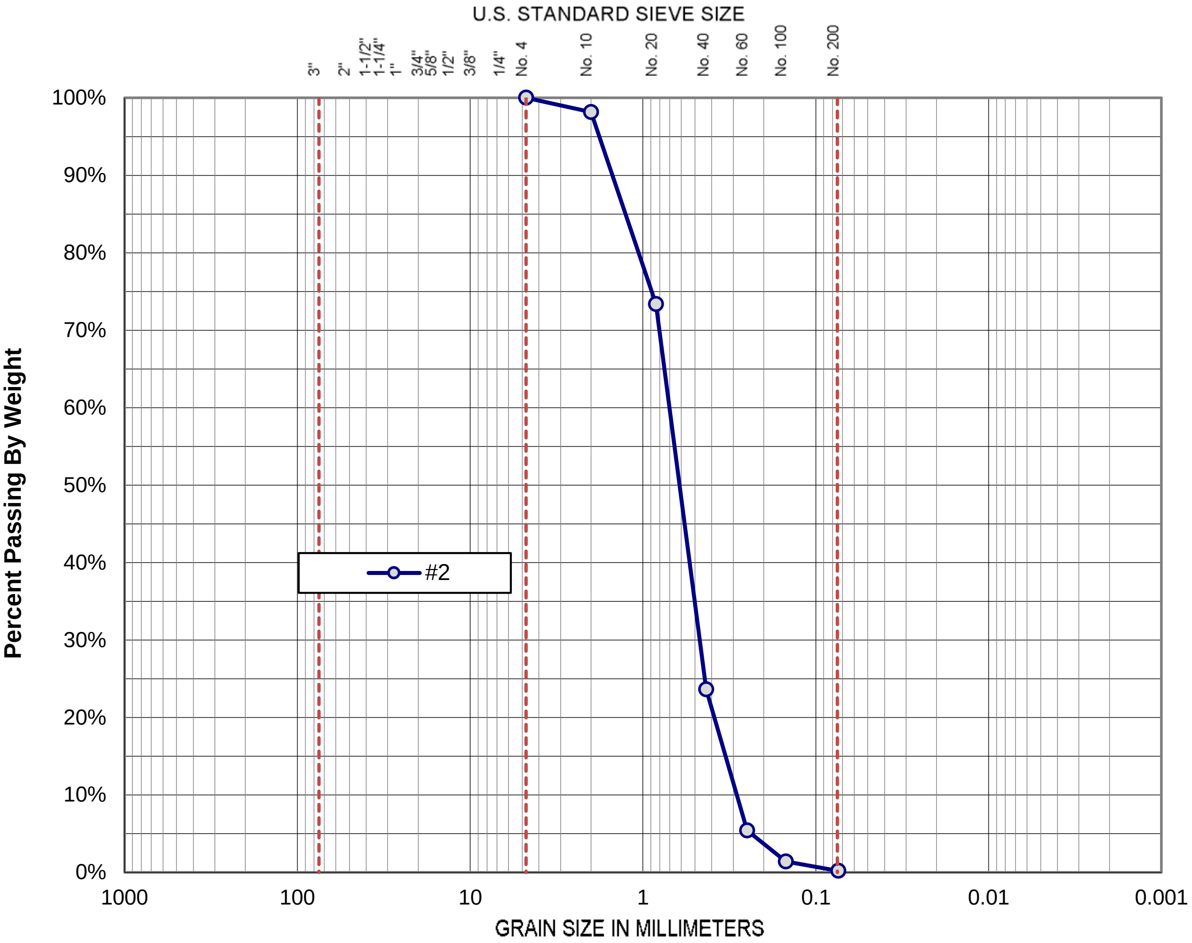
Percent Passing No. 200 by D1140 Wash Method, total sample basis:	**2.0**

Test Method: ASTM C136/D6913

Project	Priest Lake Water Management Study	**Date Submitted**	5/16/17
File No.	22593-001-00 (0200)	**Date Tested**	5/16/17
GEI ID No.	n/a	**Tested By**	GT
Shaker ID	SL1296 **Sieve Set ID** A	**Checked By**	
Description in Percent	Gravel 1	Sand 96	Fines 2.3

NOTE: Test results are applicable only to the specific sample on which they were performed, and should not be interpreted as representative of any other samples obtained at other times, depths or locations or generated by separate operations or processes. This report may not be reproduced, except in full, without written approval of GeoEngineers, Inc.

ASTM C136/6913 Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis



COBBLES	GRAVEL		SAND			FINES - SILT and CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

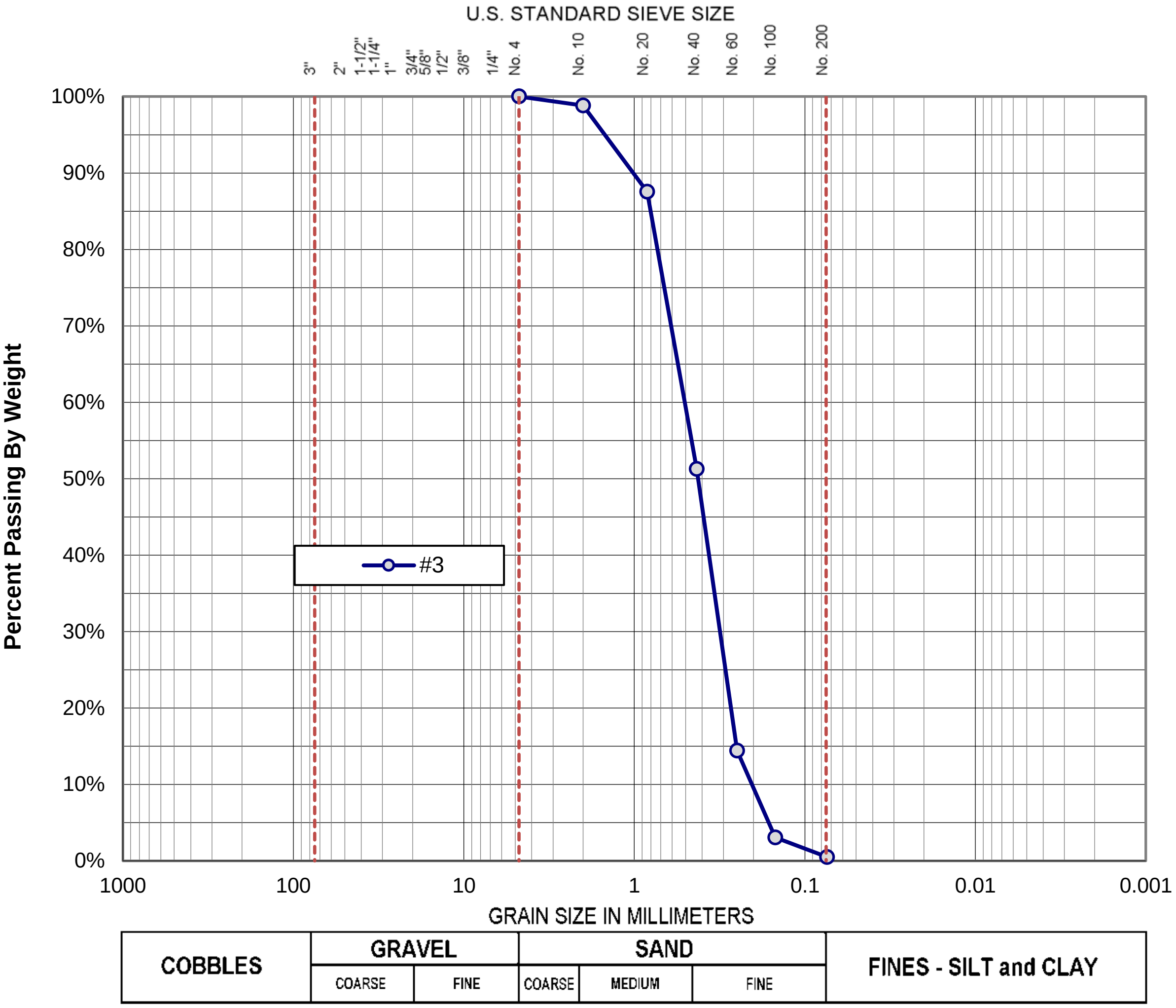
SUMMARY	
PERCENT PASSING BY WEIGHT	
Screen	%
3"	100
2"	100
1.5"	100
1"	100
3/4"	100
5/8"	100
1/2"	100
3/8"	100
1/4"	100
No. 4	100
No. 10	98
No. 20	73
No. 40	24
No. 60	5
No. 100	1
No. 200	0.2

Sample ID	WC %	C_U	C_C	USC	D2487 Classification
#2	24.0	2.5	1.1	SP	fine to medium sand
Estimated values for C_u C_c calculation	D_{60} 0.7		D_{30} 0.47		D_{10} 0.285

Percent Passing No. 200 by D1140 Wash Method, total sample basis:	0.3

Test Method: ASTM C136/D6913

Project	Priest Lake Water Management Study	**Date Submitted**	5/16/17
File No.	22593-001-00 (0200)	**Date Tested**	5/16/17
GEI ID No.	n/a	**Tested By**	GT
Shaker ID	SL1296 **Sieve Set ID** A	**Checked By**	
Description in Percent	Gravel 0	Sand 100	Fines 0.2



SUMMARY	
PERCENT PASSING BY WEIGHT	
Screen	%
3"	100
2"	100
1.5"	100
1"	100
3/4"	100
5/8"	100
1/2"	100
3/8"	100
1/4"	100
No. 4	100
No. 10	99
No. 20	88
No. 40	51
No. 60	14
No. 100	3
No. 200	0.5

Sample ID	WC %	C_U	C_C	USC	D2487 Classification
#3	25.4	2.5	1.0	SP	fine to medium sand
Estimated values for C_u C_c calculation	D_{60} 0.5		D_{30} 0.31		D_{10} 0.2

Percent Passing No. 200 by D1140 Wash Method, total sample basis:	0.4

Test Method: ASTM C136/D6913

Project	Priest Lake Water Management Study	**Date Submitted**	5/16/17
File No.	22593-001-00 (0200)	**Date Tested**	5/16/17
GEI ID No.	n/a	**Tested By**	GT
Shaker ID	SL1296 **Sieve Set ID** A	**Checked By**	
Description in Percent	Gravel 0	Sand 99	Fines 0.5

NOTE: Test results are applicable only to the specific sample on which they were performed, and should not be interpreted as representative of any other samples obtained at other times, depths or locations or generated by separate operations or processes. This report may not be reproduced, except in full, without written approval of GeoEngineers, Inc.

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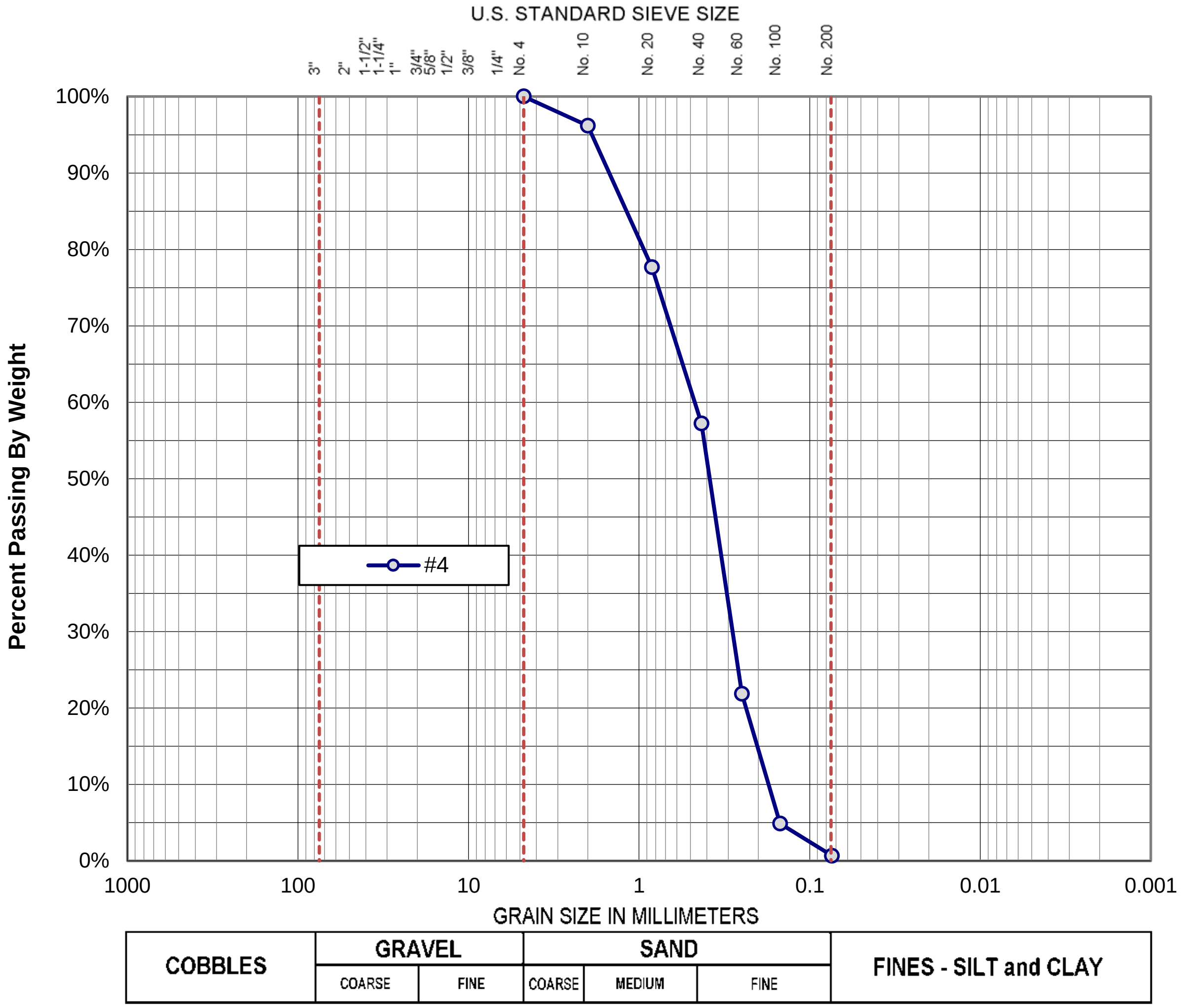
523 East Second Avenue, Spokane, WA 99202

ASTM C136/6913 Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis

Version date 9/9/2013

Fig. # of #



SUMMARY	
PERCENT PASSING BY WEIGHT	
Screen	%
3"	100
2"	100
1.5"	100
1"	100
3/4"	100
5/8"	100
1/2"	100
3/8"	100
1/4"	100
No. 4	100
No. 10	96
No. 20	78
No. 40	57
No. 60	22
No. 100	5
No. 200	0.7

Sample ID	WC %	C_U	C_C	USC	D2487 Classification
#4	25.6	2.8	1.0	SP	fine to medium sand
Estimated values for C_u C_c calculation	**D_{60}** 0.5		**D_{30}** 0.28		**D_{10}** 0.17

Percent Passing No. 200 by D1140 Wash Method, total sample basis:	**0.5**

Test Method: ASTM C136/D6913

Project	Priest Lake Water Management Study	**Date Submitted**	5/16/17
File No.	22593-001-00 (0200)	**Date Tested**	5/16/17
GEI ID No.	n/a	**Tested By**	GT
Shaker ID	SL1296 **Sieve Set ID** A	**Checked By**	
Description in Percent	Gravel 0	Sand 99	Fines 0.7

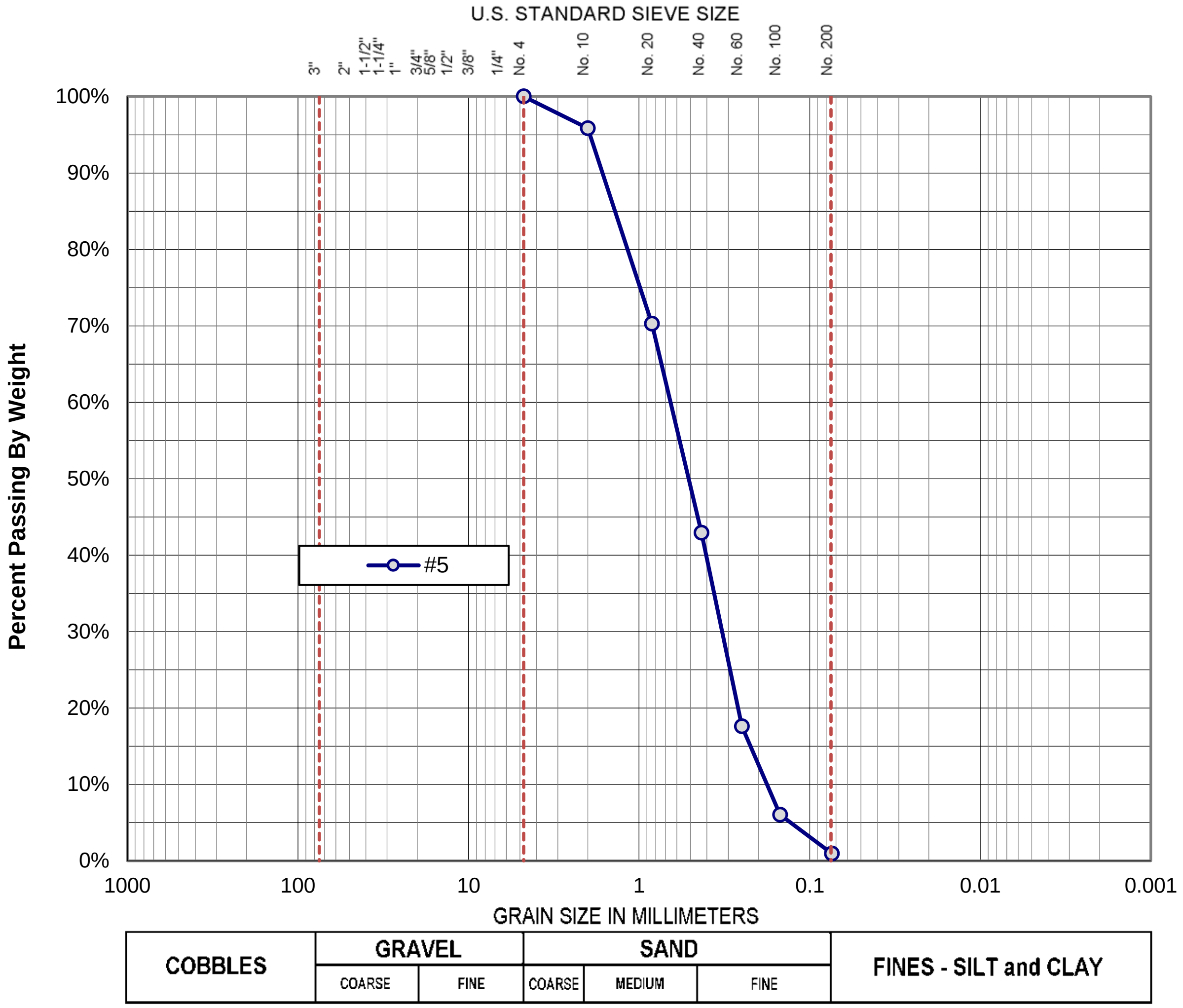
NOTE: Test results are applicable only to the specific sample on which they were performed, and should not be interpreted as representative of any other samples obtained at other times, depths or locations or generated by separate operations or processes. This report may not be reproduced, except in full, without written approval of GeoEngineers, Inc.

GeoEngineers

523 East Second Avenue, Spokane, WA 99202

ASTM C136/6913 Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis

Version date 9/9/2013



SUMMARY	
PERCENT PASSING BY WEIGHT	
Screen	%
3"	100
2"	100
1.5"	100
1"	100
3/4"	100
5/8"	100
1/2"	100
3/8"	100
1/4"	100
No. 4	100
No. 10	96
No. 20	70
No. 40	43
No. 60	18
No. 100	6
No. 200	1.0

Sample ID	WC %	C_U	C_C	USC	D2487 Classification
#5	21.8	3.6	0.9	SP	fine to medium sand
Estimated values for C_u C_c calculation	D_{60} 0.7		D_{30} 0.32		D_{10} 0.18

Percent Passing No. 200 by D1140 Wash Method, total sample basis:	**0.8**

Test Method: ASTM C136/D6913

Project	Priest Lake Water Management Study	**Date Submitted**	5/16/17
File No.	22593-001-00 (0200)	**Date Tested**	5/16/17
GEI ID No.	n/a	**Tested By**	GT
Shaker ID	SL1296 **Sieve Set ID** A	**Checked By**	
Description in Percent	Gravel 0	Sand 99	Fines 1.0

NOTE: Test results are applicable only to the specific sample on which they were performed, and should not be interpreted as representative of any other samples obtained at other times, depths or locations or generated by separate operations or processes. This report may not be reproduced, except in full, without written approval of GeoEngineers, Inc.

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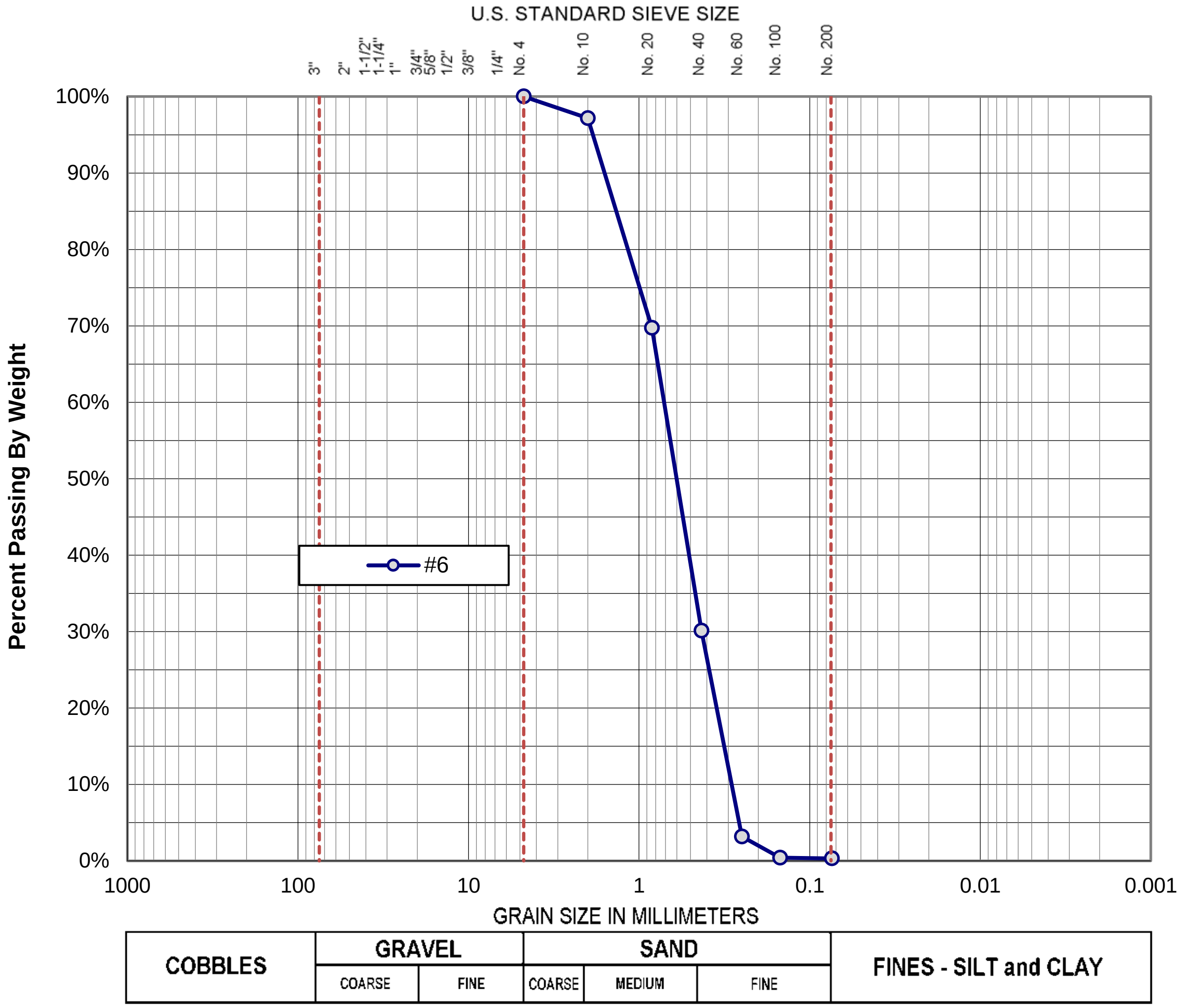
523 East Second Avenue, Spokane, WA 99202

ASTM C136/6913 Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis

Version date 9/9/2013

Fig. # of #



SUMMARY	
PERCENT PASSING BY WEIGHT	
Screen	%
3"	100
2"	100
1.5"	100
1"	100
3/4"	100
5/8"	100
1/2"	100
3/8"	100
1/4"	100
No. 4	100
No. 10	97
No. 20	70
No. 40	30
No. 60	3
No. 100	0
No. 200	0.3

Sample ID	WC %	C_U	C_C	USC	D2487 Classification
#6	29.2	2.5	0.9	SP	fine to medium sand
Estimated values for C_u C_c calculation	D_{60} 0.7		D_{30} 0.42		D_{10} 0.28

Percent Passing No. 200 by D1140 Wash Method, total sample basis:	0.3

Test Method: ASTM C136/D6913

Project	Priest Lake Water Management Study	**Date Submitted**	5/16/17
File No.	22593-001-00 (0200)	**Date Tested**	5/16/17
GEI ID No.	n/a	**Tested By**	GT
Shaker ID	SL1296 **Sieve Set ID** A	**Checked By**	
Description in Percent	Gravel 0	Sand 100	Fines 0.3

NOTE: Test results are applicable only to the specific sample on which they were performed, and should not be interpreted as representative of any other samples obtained at other times, depths or locations or generated by separate operations or processes. This report may not be reproduced, except in full, without written approval of GeoEngineers, Inc.

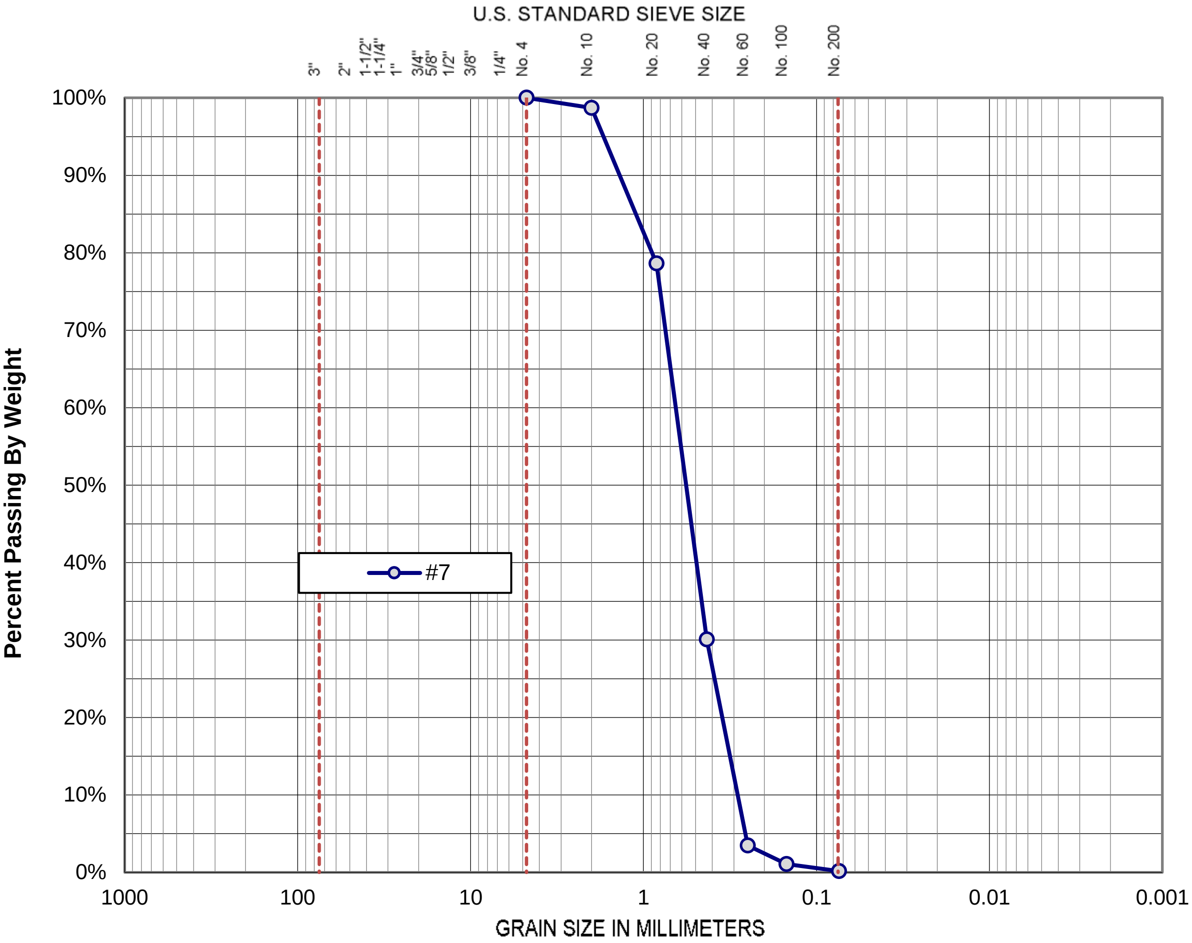
GEOENGINEERS

523 East Second Avenue, Spokane, WA 99202

ASTM C136/6913 Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis

Version date 9/9/2013

Fig. # of #



COBBLES	GRAVEL		SAND			FINES - SILT and CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

SUMMARY	
PERCENT PASSING BY WEIGHT	
Screen	%
3"	100
2"	100
1.5"	100
1"	100
3/4"	100
5/8"	100
1/2"	100
3/8"	100
1/4"	100
No. 4	100
No. 10	99
No. 20	79
No. 40	30
No. 60	3
No. 100	1
No. 200	0.2

Sample ID	WC %	C_U	C_C	USC	D2487 Classification
#7	26.7	2.3	1.0	SP	fine to medium sand
Estimated values for C_u C_c calculation	D_{60} 0.7		D_{30} 0.42		D_{10} 0.28

Percent Passing No. 200 by D1140 Wash Method, total sample basis:	0.1

Test Method: ASTM C136/D6913

Project	Priest Lake Water Management Study	**Date Submitted**	5/16/17
File No.	22593-001-00 (0200)	**Date Tested**	5/16/17
GEI ID No.	n/a	**Tested By**	GT
Shaker ID	SL1296 **Sieve Set ID** A	**Checked By**	
Description in Percent	Gravel 0	Sand 100	Fines 0.2

NOTE: Test results are applicable only to the specific sample on which they were performed, and should not be interpreted as representative of any other samples obtained at other times, depths or locations or generated by separate operations or processes. This report may not be reproduced, except in full, without written approval of GeoEngineers, Inc.

ASTM C136/6913 Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis

A.3 Basis of Analysis and Criteria

BASIS OF ANALYSIS

PRIEST LAKE WATER MANAGEMENT STUDY

September 29, 2017-Rev C

IDAHO WATER RESOURCE BOARD

M M
MOTT
MACDONALD

BASIS OF ANALYSIS

Priest Lake Water Management Study, Priest Lake, Idaho

1. Study Background

The Priest Lake Water Management Study was initiated by the Idaho Water Resource Board (IWRB) to evaluate opportunities for improving operation of the Priest Lake and Priest River system in order to meet long-term management objectives. The study includes several action items:

- Evaluate alternatives for maintaining required lake levels and for maintaining current minimum discharge requirements of 60 cfs downstream from the dam.
- Assess potential structural and operational modifications to the Priest Lake Outlet Dam.
- Analyze options to improve access and navigable conditions for the Priest Lake Thorofare.

The goal of the study is to develop a range of feasible alternatives in consultation with stakeholders and the public for improving Priest Lake Water Management.

2. Project Study Area Description

Lower Priest Lake (hereafter referred to as Priest Lake) is approximately 18 miles long, has a maximum depth greater than 300 feet, and has active storage space of approximately 760,000 acre-feet. It is connected to Upper Priest Lake (which is approximately 3.3 miles long) by a three-mile long channel known as the "Thorofare" which has long been used by the public for recreation and access to the upper lake. A 1,400-foot-long timber breakwater at the north end of Priest Lake is intended to manage sediment from the upper lake, while providing wave and erosion protection to landowners at the north end of the Priest Lake.

The original Priest Lake Outlet Dam was constructed in 1951. The current dam was constructed in 1978 to replace the deteriorating original dam. The dam was constructed to maintain lake level in Priest Lake and manage downstream flows into Priest River. The dam is owned by IDWR and operated by a contractor on behalf of IDWR. The dam is approximately 12 feet high with gates that regulate discharge. It does not have an emergency spillway.

Water levels in the lake are measured at the USGS outlet gage (#12393000). Lake level begins to rise in April and May during the spring runoff, reaching a maximum level of 3 to 5 ft (outlet gage level) in early June. The level recedes to roughly 3 ft in July and this level is maintained through the summer recreational season. Storage releases commonly start during the second week of October, but have started as early as October 4th and as late as October 16th. Storage releases normally end sometime in November. The gates remain open through the fall, winter, and early spring season to allow natural passage of flows through Priest Lake.

3. Brief History

Priest Lake is located in the northern Idaho Panhandle. It is a significant draw for tourism and recreation, both of which add to the economic benefit to Bonner County. The area is known for the pristine variety of wildlife, clear and clean water, and recreational opportunities. In 2015, limited water supply and drought conditions in northern Idaho made maintaining the required summer lake levels and downstream flow in the river very difficult. This situation, coupled with concerns about the breakwater structure and Thorofare access issues, increased interest in developing both operational and engineered improvements to the entire system.

In response to area stakeholders' concerns, the IWRB authorized funding to perform an evaluation of strategies and options that could meet the long-term water management solutions for the Priest Lake system.

4. Study Criteria

The purpose of the study is to conduct a feasibility assessment to evaluate operational improvements for the Priest Lake system with respect to various criteria. These criteria include the following:

- Lake Level Management (maintaining a 3-ft level at the outlet gage (#12393000) for recreational season and developing operational strategies that improve habitat and minimize shoreline impacts) in accordance with Idaho Statutes § 70-507;
- Maintain current minimum discharge flow requirements downstream of dam (60 cfs); and,
- Provide sustainability for the Thorofare (promoting self-sustaining improvements to improve Thorofare access, navigability, and water quality).

5. Engineering Standards

The following standards and guidelines will be used to conduct the study and corresponding assessment work.

- Outlet Structure
  - U.S. Bureau of Reclamation. Design of Small Dams, 1987.
  - Idaho Administrative Procedures Act (IDAPA). 37.03.06; Safety of Dams Rules.
  - U.S. Army Corps of Engineers (USACE). Gravity Dam Design; EM 1110-2-2200; 30 June 1995.
  - U.S. Army Corps of Engineers (USACE). Stability Analysis of Concrete Structures; EM 1110-2-2100, 1 December 2005.
- Thorofare
  - U.S. Army Corps of Engineers. (USACE). Coastal Engineering Manual, EM 1110-2-1100, 30 April 2006.
  - U.S. Army Corps of Engineers (USACE). Design of Breakwaters & Jetties, EM 1110-2-2904; 8 August 1986.

6. Property Ownership

The study will focus on an assessment of water management change benefits and impacts to the entire Priest Lake. Improvement alternatives (modification of structures, dredging, new structures, etc.) will be focused on government-owned property within the following areas:

- Outlet Dam: The outlet dam is located on Idaho Department of Water Resources property.
- Thorofare: Breakwater and Thorofare is located on Idaho Department of Lands property.

Parcel information for the project study areas will be obtained from the Bonner County GIS database.

7. Hydrographic/Topographic Data & Datums

Topographic and hydrographic data for the project is summarized below:

Data Set Name	Topographic/Hydrographic	Source	Year
Light Detection and Ranging (LiDAR)	Topographic	Watershed Sciences collected for University of Idaho	2012
Priest Lake Survey	Hydrographic	Idaho DEQ	1995
Thorofare Survey	Hydrographic	Delphis for Mott MacDonald	2017

The vertical datum will be: Lake Datum 0.0 ft = 2434.64 ft NGVD29 at USGS outlet gage (#12393000).

8. Climate Change Considerations

It is anticipated that frequency of low summer flow and drought conditions or intensity thereof could increase as a result of climate change. Considerations for climate change should be part of the improvement alternative evaluation. This would include evaluation of drought conditions similar to 2001 and 2005 and a backcheck of water management concepts relative to 1977 and 2015 low water years.

9. Water Levels & Discharge

Water level and discharge data for the project is measured at the gages summarized below:

Gage Name	Measurement	USGS Number	Record Available	Measurement Increment	Source
Priest Lake at outlet NR Coolin, ID	Lake height	12393000	1928-present	daily	USGS website
Priest River NR Coolin, ID	Discharge	12394000	1948-2006	daily	USGS website
Priest River at Falk Ranch NR Priest River, ID	Discharge	12394500	1911-1912	daily	USGS website
Priest River NR Priest River, ID	Discharge Gage height	12395000	1903-present 1910-1911	daily	USGS website
Priest River at Outlet of Priest Lake NR Coolin ID	Discharge	12393500	1912-1948	daily	USGS website
Priest R Outflow NR Coolin, ID	Discharge	12393501	2016-present	15 minutes	USGS website
Upper West Branch Near Dickensheet Junction, ID	N/A	12394100	N/A	N/A	N/A
Staff gage at Marina	N/A	N/A	N/A	N/A	N/A
Staff gage at Dam	N/A	N/A	N/A	N/A	N/A
Thorofare	Discharge	N/A	1994-1995	27 measurements	IDEQ (IDEQ 1997)

- Lake levels are managed at the outlet structure and measured at the outlet gage (#12393000). Levels are required to be no less than 0.1 ft on the gage during the winter and at 3.0 ft during the summer recreational season.
- Average daily water levels are given in Figure 1 below based on data from the outlet gage (#12393000).

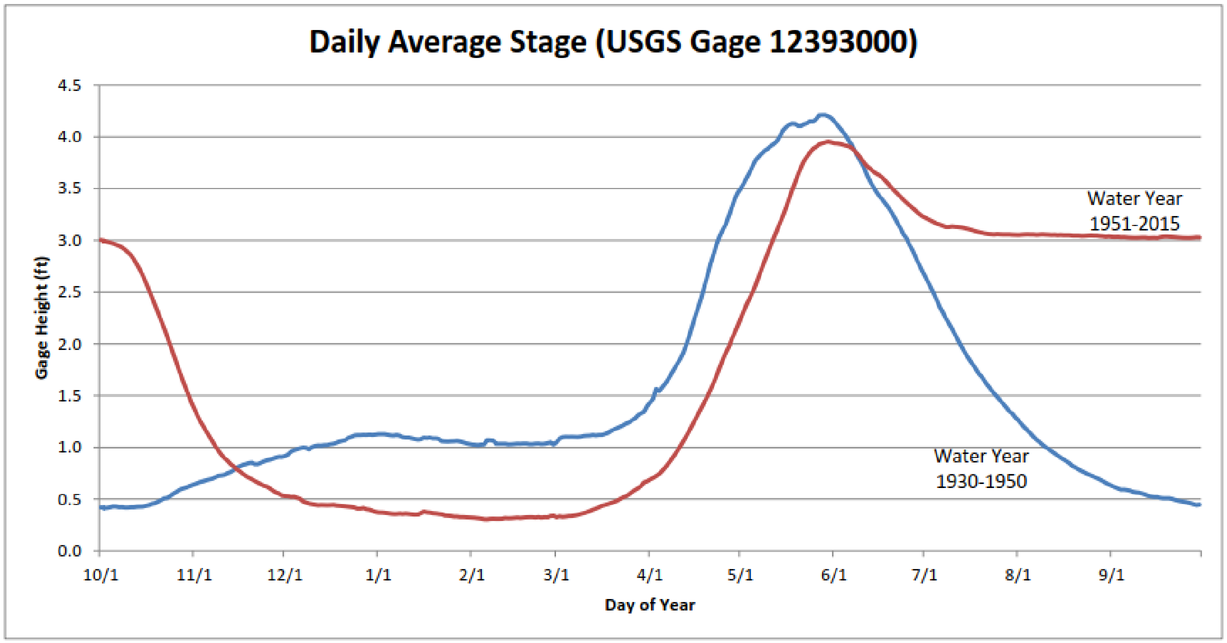


Figure 1: Daily Water Level at USGS Outlet Gage (#12393000).

10. Wind

- Data Sources: Wind data for purposes of wind-wave growth analysis will be obtained from the Western Regional Climate Center Priest Lake Remote Automated Weather Station (RAWS) station. The wind data is collected on an hourly basis with a data record from October 2001 to present.
- Background: The dominant daytime wind direction is from the south shifting to a dominant wind direction from the north at night.
- Wind Conditions: Evaluation of wind-waves for the purpose of conducting impact assessments will utilize a 2-year return period wind speed. Wind-waves for evaluating structure design requirements will utilize a minimum 50-year return period wind speed.

11. Water Management State Code Requirements (Idaho Statutes 70-507)

- Legislature authorized the Priest Lake Outlet dam facility to stabilize summer lake levels of Priest Lake for recreation.
- Code provides explicit requirements for the water surface levels in Priest Lake.
  - Lake level is allowed to exceed 3.0 ft on the outlet gage (#12393000) during spring runoff.
  - After the spring runoff period, the lake level must be at least 3.0 ft until the close of the recreational season as determined by the director of IDWR.
  - At other times of the year, the lake level must be maintained between 0.1 and 3.0 ft on the outlet gage (#12393000).

12. Recreational Season

The recreational season for Priest Lake for the purpose of this study was defined to be the following:

- Definition
  - As determined by the Director of IDWR.

- Timeframe
    - Historical: Typically July 1 to first weekend after 1st full week of October.
    - Study Definition: July 1 to October 8.

12.1. Vessel Type & Size Class

Vessel types and sizes that utilize Priest Lake and those that will be utilized for the study and assessment of Thorofare navigation sustainability are outlined herein.

- Recreational Vessels.
    - Thorofare - Recreational. Vessels that will utilize the Thorofare and their characteristics were assumed to be the following:

Vessel Type	Example	Beam (ft)	Draft (ft)	Length (ft)
Water Sports Boats (includes wake boards & ski boats w inboard/outboard motors)	Chaparral SSX Surf	8.5	3	25
Fishing Boats with outboards	North River Seahawk	8.5	2.5	25
Pontoon Boats with outboards	Crestliner Rally DX Pontoon	8.5	2	25

- Lake - Recreational. Vessels utilizing the lake but not the Thorofare were assumed to be the following:

Vessel Type	Example	Beam (ft)	Draft (ft)	Length (ft)
Cabin Cruisers	Carver Cabin Cruiser	14	4	38
Sail Boats	Hunter 26'/SL	8.5	2 to 6	26

- Non-Recreational.
    - Work Barge. U.S. Forest Service work barge and landing craft. These vessels may utilize the Thorofare under special circumstances but will not be the basis for the analysis.

13. Thorofare Navigation

It is desired to provide sustainable navigable access in the Thorofare from Priest Lake into upper Priest Lake for the specified class of vessel (Thorofare – Recreational). The following will be criteria for the navigation and Thorofare improvements evaluation:

- Channel. Navigation in the Thorofare will be through the naturally-formed channel alignment (marked with navigation buoys) and will not be a straight channel to promote sustainability and reduce maintenance dredging needs.
- Navigation Depth. The minimum water depth at summer recreational period (when lake level is at 3.0 ft outlet gage (#12393000)) is desired to be 4 ft. An alternative for 5 ft of water depth will be evaluated.
- Width. A minimum width at the navigable depth is needed for vessels to transit the Thorofare and is a function of the width (beam) of the vessel. The minimum width depends on one-way

versus two-way traffic for the size of vessels being evaluated. Minimum width requirements will be evaluated as a part of conducting the study.

- Navigation Aids. Historically, the County has installed navigation buoys along the entrance to the Thorofare. The practice of navigation aid buoys will continue for any alternative being evaluated. The need for additional buoys will be assessed during the evaluation of alternatives.

14. Species Considerations

Based on conversations with the state and federal regulatory agencies, the following is a summary of considerations for species of concern within Priest Lake and the Thorofare.

- Bull Trout:
    - Migration period mid-May through November. Typically, in spawning streams by November.
    - Adjusted lake levels do not result in restricted access to tributary streams.
- Kokanee:
    - Spawning in November, December, in nearshore gravels. No presence within sand substrate for spawning.
    - Lake level to be returned to natural, non-regulated conditions by November 1; start of potential Kokanee spawning.

15. Regulatory Considerations

Based on conversations with the state and federal regulatory agencies, the following is a summary of requirements that will likely apply to any proposed in-water construction work:

- Work Window
    - Year around will likely be allowed for the Thorofare and Outlet Dam, assuming no full blockage to fish and fish can escape from work area. Additional restrictions or limitations will likely apply for protection of summer recreational season and winter ice conditions.
- Water Quality
    - Protection of water quality during construction in accordance with Section 401 Permit issued by Idaho Department of Environmental Quality (DEQ).
- Construction Materials
    - Pressure treated timber materials above or in contact with waters of Priest Lake are not allowed per Priest Lake Management Plan, IDL (1990).
    - In-water disposal of dredged materials will require further investigation from regulatory agencies on requirements and limitations if some form of in-water disposal or beneficial reuse were proposed.

16. Dam Operations

The following is a summary of existing and historical dam operations as determined from the IDWR records, water rights and discussions with the dam operator.

- Discharge & Operational Requirements
    - Gage Level:
        - 3.0 ft outlet gage (#12393000) water level during summer recreational period.
    - Minimum Discharge:

        - Flow: 60 cfs min.
        - Timing: Entire calendar year.
    - Fall Discharge:
        - Flows
            - Not greater than 2,500 cfs during fall discharge.
            - No greater than 1,200 cfs increase in 24-hour period (Avista 2002 operating scheme).
        - Timing
            - Start drawdown not prior to October 1.
            - Historically started on the first Sunday of October.
            - Complete drawdown prior to November 1.
    - Spring Discharge:
        - Gate Closure.
            - Gates operated to capture runoff in June to achieve a 3.0' outlet gage (#12393000) level by July 1.
            - Gate closure subject to opening restrictions to limit effect of hydraulic jump and corresponding risk of downstream apron scouring which could destabilize the dam structure.
    - Notification made 2 weeks prior to the start of drawdown Maximum Discharge: Not regulated by outlet dam (gates fully open during spring runoff). Peak discharge during fall discharge is limited to minimize erosional effects from hydraulic jump.
- Manually Controlled Gates
    - The radial gates are manually controlled by the Dam operator utilizing an electric powered torque wrench.
    - Gate opening is determined based on observations of water levels at a marina staff gage and historical flow records.
    - Dam operator rules of thumb – 3" opening = 30 cfs/gate.
    - Discharge Rating Curve relative to marina staff gage was developed.
    - Discharge 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.
- Water Levels
    - Primarily monitored at the marina staff gage in the morning before gate adjustment.
    - Variations from local atmospheric pressure and wind events (seiche) effect the water levels at the dam and are taken into consideration during gate operations. Up to 6 to 8-inches of variation within 8 to 10 hours' time has been observed due to this phenomenon.
    - There is a water level difference between the marina staff gage and the staff gage at the dam.
- Historical Records
    - Records of each day gate opening and lake level are available.

17. Lake Water Management Evaluation Scenarios

Water management operational alternatives for evaluation will be developed in coordination with IDWR. Appendix A provides a detailed summary of the scenarios to be evaluated. The following will be considered in the water management alternatives:

1. Timeframe
    a. July 1 to October 8.
2. Water Levels (outlet gage (#12393000) datum)
    a. Variations from 2.75 ft to 3.5 ft.
3. Outlet Dam Discharges
    a. Current Minimum Requirement of 60 cfs.
4. Water Years
    a. Drought years
        i. 2001 and 2005 as the basis for analysis.
        ii. 1977 and 2015 as backcheck years.
    b. Wet Years
        i. Not Evaluated
    c. Typical Average Year
        i. Use as a backcheck for a drought year condition for comparison.
5. Gate Operational Procedures
    a. Evaluate timing for gate operations to capture runoff during drought or near drought years to ensure water level and discharge criteria are met throughout the summer recreational period.
    b. Determine earliest time period in year to capture storage to meet study criteria.

Additional details on water management alternatives will be provided under a separate technical memorandum (see Appendix A).

18. Pool Raise Considerations

For evaluation of pool raise alternatives, an assessment of potential for impacts will be conducted on the following shoreline features and infrastructure:

- Basement Flooding
- Beach Erosion
- Recreational Beach
    - Loss of Use.
    - Fire Pits.
    - Benches.
    - Beach Width.
- Boat Cover
- Ecosystem (wetlands, riparian)
- Piers

- Boat Launch

19. Outlet Dam Stability Assessment

19.1. General Criteria

The Priest Lake Outlet Dam is classified as a large dam under IDAPA 37.03.06 Safety of Dams Rules due to its storage capacity exceeding 4,000 acre-ft and it is in the Significant Hazard Category of IDAPA 37.03.06. Any proposed modifications to the outlet Dam will be in conformance with IDAPA 37.03.06 requirements, supplemented with Federal design guidelines for dams and hydraulic structures— including applicable technical guidelines and engineering manuals from the Bureau of Reclamation and the U.S. Army Corps of Engineers. The following category and classification applies to the Priest Lake Outlet Dam:

- Dam size classification per IDAPA 37.03.06: Large
- Dam hazard category per IDAPA 37.03-06: Significant

19.2. Geotechnical Criteria

The stability against sliding and foundation stability shall satisfy the safety levels indicated in Table 1, per USACE EM 1110-2-2200 *Gravity Dam Design.*

Table 1. Dam stability criteria per EM 1110-2-2200 Gravity Dam Design

Load Condition	Resultant location at base	Minimum Sliding FS	Foundation Bearing Pressure
Usual	Middle 1/3	2.0	≤ allowable
Unusual	Middle 1/2	1.7	≤ allowable
Extreme	Within base	1.3	≤ 1.33 x allowable

19.3. Structural

The structural assessment will provide evaluation in accordance with ACI 318-14 and AISC Steel Construction Manual for concrete and steel components of the dam, respectively. Supplement with other structural codes and design guides to determine loads and load combinations, listed under *Applicable Codes and References.*

19.4. Seismic Criteria

If the dam stability analysis for the pool raise indicates a factor of safety of less than 2.0 thereby necessitating an upgrade/modification, seismic analysis and upgrades to meet current code would be required.

IDWR requires seismic evaluation since Priest Lake Dam is classified as a large dam per IDAPA 37.03.06 Section 040. The horizontal acceleration (seismic coefficient) of 0.156 is what has been applied for this area for pseudo-static analysis.

19.5. Hydraulic Criteria

Hydraulic criteria for use in evaluating dam stability will include the following:

- Summer Recreation Period (Static Load on Radial Gates):
  - Headwater Priest Lake level = 3.5 ft outlet gage (#12393000).

- Tailwater level based on discharge Flows = 60 cfs.

20. References

American Concrete Institute (ACI). Building Code Requirements for Structure Concrete. 318-14. 2014.

American Institute of Steel Construction (AISC). Steel Construction Manual. 2011.

Idaho Administrative Procedures Act 37.03.06; Safety of Dams Rules.

Idaho Administrative Procedures Act 37-03-06.

Idaho Department of Water Resources (IDWR). Water Right Number 97-07380.

Idaho Department of Lands (IDL). Response to permit for replacement of Thorofare Breakwater. Permit No. L-97-S-891.

Idaho Division of Environmental Quality (IDEQ). Phase 1 Diagnostic Analysis Priest Lake. 1997.

Idaho Statues 70-507.

U.S. Army Corps of Engineers (USACE). Gravity Dam Design; EM 1110-2-2200; 30 June 1995.

U.S. Army Corps of Engineers (USACE). Stability Analysis of Concrete Structures; EM 1110-2-2100, 1 December 2005.

U.S. Army Corps of Engineers (USACE). Coastal Engineering Manual, EM 1110-2-1100, 30 April 2002.

U.S. Army Corps of Engineers (USACE). Design of Breakwaters & Jetties, EM 1110-2-2904; 8 August 1986.

U.S. Bureau of Reclamation. (USBR). Design of Small Dams, 1977.

U.S. Geological Survey (USGS). Water Data for the Nation. Priest Lake at Outlet NR Coolin ID, 12393000. <https://nwis.waterdata.usgs.gov/nwis/inventory/?site_no=12393000&agency_cd=USGS>

APPENDIX A

MOTT MACDONALD

Memorandum

Subject: Priest Lake Water Management Project, Water Management Operational Alternatives Discussion

To: IDWR & Bonner County

From: Mott MacDonald & GeoEngineers

Date: Rev C September 29, 2017

Table 1 was assembled as a starting point for the analysis of potential water management operational alternatives to be considered for meeting the criteria and objectives of the study (as outlined in the Basis of Analysis Memorandum).

Table 1. Priest Lake Water Management for Existing Operations (Exist) and Alternatives (Alt) for Dry-Year Types and a Normal Type Year Check

Alternative	Water-year Type	Recreation Lake Level			Priest River Discharge (Q)			Inflow Calendar Year	Commentary
		Stage (ft)	Start Date	End Date	Q (cfs)	Start Date	End Date		
Exist-D1	Dry	3.0	July 1	Oct 8	60 (min)	Jan 1	Dec 31	2001	Base, run 1st to understand time periods for lack of water to better refine alternatives for timing of increased water level and drawdowns of the additional storage.
Exist-D2	Dry	3.0	July 1	Oct 8	60 (min)	Jan 1	Dec 31	2005	Base
Exist-N	Normal	3.0	July 1	Oct 8	60 (min)	Jan 1	Dec 31	2002	Base
Alt 1	Dry	3.5	July 1	Aug 30	60 (min)	Jan 1	Dec 31	2001	Basic Low Flow
		3.5 to 3.0	Sept 1	Oct 8	2,000 (max)	Oct 9	Oct 31		
Alt 2	Dry	3.5	July 1	Aug 30	60 (min)	Jan 1	Dec 31	2005	Basic Low Flow
		3.5 to 3.0	Sept 1	Oct 8	2,000 (max)	Oct 9	Oct 31		
Alt 2N	Normal	3.5	July 1	Aug 30	60 (min)	Jan 1	Dec 31	2002	Run normal year after completing dry condition alternatives; so water level range could be different than that described.
		3.5 to 3.0	Sept 1	Oct 8	2,000 (max)	Oct 9	Oct 31		

Alternative	Water-year Type	Recreation Lake Level			Priest River Discharge (Q)			Inflow Calendar Year	Commentary
		Stage (ft)	Start Date	End Date	Q (cfs)	Start Date	End Date		
Alt 3	Dry	3.25	July 1	Aug 30	60 (min)	Jan 1	Dec 31	2001	Lower water level in Sept.
		3.25 to 2.75	Sept 1	Sept 30	2,000 (max)	Oct 1	Oct 31		
Alt 4	Dry	3.5	July 1	Aug 15	60 (min)	Jan 1	Dec 31	2001	Variation in timing of water levels.
		3.5 to 3.0	Aug 16	Sep 15	2,000 (max)	Oct 1	Oct 31		
		3 to 2.75	Sept 16	Oct 8					
Alt 5	Dry	3.25	July 1	Aug 30	60 (min)	Jan 1	Dec 31	2001	Variation in timing of water levels.
		3.25 to 3.0	Sept 1	Oct 8	2,000 (max)	Oct 9	Oct 31		
Alt 6	Dry	TBD	July 1	TBD	60 (min)	Jan 1	Dec 31	TBD	To be determined after running dry alternatives.

Footnote:

1. TBD = To be determined

Footnotes on Scenarios, Assumptions & Criteria for Water Management Alternatives:

- Additional detail on data and approach for conducting the analysis is outlined in the attached June 15, 2017 GeoEngineers Approach to Water Management Modeling Memorandum.
- In accordance with IDWR approved criteria, the water management analysis is focused on dry and drought year lake system water management operational changes. Therefore, wet year simulations will not be conducted as those operations will remain unchanged from historical conditions. No modeling of wet water-year types included.
- Flows (Q) are minimums and maximums for the stated time periods.
- August 15 date to be adjusted based on refinement of low water availability and time period for storage required to meet the minimum 60 cfs stream flow.
- Maximum discharge in October of 2,000 cfs (no more than 1,200 cfs increase in 24-hour period) assumed to be the limited flow condition in October based on review of historical operations. Discharge curve to mimic a prior year discharge and gate operation pattern (such as 1979, per IDWR documentation).
- Water Right 97-07380 as measured at the confluence with the East River is converted to outlet dam discharge values in accordance with information provided by IDWR (Personnel communication, Matt Anders, June 2017). Those conversion values are as follows:
    - 700 cfs at East River = 550 cfs at outlet dam
    - 300 cfs at East River = 250 cfs at outlet dam
- Starting time for gate operation to achieve revised higher pool level by July 1 (or summer recreational period) will be evaluated. It is assumed that gate operations can be modified to meet any new operational needs and requirements. As part of the outlet dam assessment work, an evaluation of the gate operations will be conducted.
- Drought and dry years of 2001 and 2005 were selected due to their full data record and being representative of current dam operations. Other dry/drought years (such as 1977 and 2015) will be backchecked once the analysis is completed for 2001 and 2005.
- Priest Lake inflows calculated by IDWR will be used for the calendar years identified in Table 1.

A.4 Fisheries Assessment Memorandum

GeoEngineers

523 East Second Avenue, Spokane, Washington 99202, Telephone: 509.363.3125 www.geoengineers.com

Memorandum

To:	Shane Phillips, PE and Younes Nouri, PE; Mott MacDonald
From:	Jason Scott and Tim Hanrahan; GeoEngineers, Inc.
Date:	December 12, 2017
File:	22593-001-00
Subject:	Priest Lake Water Management Study Fisheries Assessment

INTRODUCTION

The overall study is intended to achieve three main goals. First, preserve lake levels through the recreation season during dry and marginally dry years. Second, maintain a minimum discharge of 60 cubic feet per second (cfs) from the dam during the recreation season. Third, provide sustainable modifications to improve thorofare access, navigability and water quality. The purpose of the fisheries assessment is to evaluate potential effects water management changes and thorofare improvements might have on focus fish species. More specifically, our assessment focused on spawning migration, spawning success, species interactions and access to critical habitats.

The native fish assemblage in Priest Lake and its tributaries include; bull trout (*Salvelinus confluentus*), westslope cutthroat trout (*Oncorhynchus clarki lewisi*), mountain whitefish (*Prosopium williamsoni*), pygmy whitefish (*P. coulteri*), northern pikeminnow (*Ptychocheilus oregonensis*), suckers (*Catastomus* sp), redside shiner (*Richardsonius baltetus*), dace (*Rhinichthys* sp.), and sculpin (*Cottus* sp.) (Bjornn 1957). Brook trout (*S. fontinalis*) were introduced prior to 1920, lake trout (*S. namaycush*) were introduced in 1925, and kokanee salmon (*O. nerka*) were introduced beginning in 1942 (Bjornn 1957). Through the 1960's and early 1970's, kokanee harvest dominated the creel and supported a thriving trophy lake trout fishery and rapid population growth (Rieman et al. 1979). In the late 1960's, opossum shrimp (*Mysis diluviana*) were introduced to the lake and by 1976 the kokanee population collapsed, likely due to the bottom-up and top-down pressure the opossum shrimp and lake trout (respectively) were putting on the kokanee population (Rieman et al. 1979 and Fredericks et al. 2009).

Concurrent with the rapid kokanee and lake trout population expansion was a decline in the native fish population (Fredericks 1999). Bjornn (1957) reported that westslope cutthroat trout population and size had been declining since at least 1948. Rieman et al. (1979) noted that the Priest Lake system supported a successful bull trout fishery prior to the late 1970s when a rapid decline was noticed, and by the late 1990s, Fredericks (1999) suggested that bull trout were only present in the upper lake in very low abundance.

Due to the complexity of fish populations throughout the system, managing the fishery in Priest Lake and Upper Priest Lake is challenging. Currently, the majority of the angler effort in Priest Lake is spent targeting lake trout. However, the Idaho Department of Fish and Game (IDFG) is in the process of working with stakeholder groups to define a management direction that could change the fish population dynamics and/or angler effort. Results of that process and subsequent management actions will not be made until at least late 2017, after this study is complete. It is unlikely that recommendations associated with this project will have consequences on fish management in the lake (Andy Dux and Kirra Sitari personal communication).

POTENTIAL EFFECTS OF WATER MANAGEMENT CHANGES AND THOROFARE IMPROVEMENT ON PRIEST LAKE FISH POPULATIONS AND HABITAT

This study consists of two primary elements:

- **Lake Level Management:** In accordance with Idaho Code 70-507, the outlet dam is operated to maintain the lake levels at 3.0 feet on the outlet gage (USGS No. 12393000) for recreation purposes. While maintaining the lake level is a statutory requirement, there is a policy to attempt to meet a minimum outflow discharge of 60 cfs. In dry years, such as 2015, maintaining the statutory lake level and desired outflow discharge is difficult and sometimes unachievable. This study is looking at water management strategies that could meet both objectives during dry and moderately dry years.
- **Thorofare Improvement:** The study is analyzing options to provide sustainable navigation access through the thorofare. The thorofare channel has long been used by the public for recreation and access to the upper lake. However, sedimentation at the downstream end of the thorofare is increasingly restricting access, especially for motorized vessels.

Fish management within Priest Lake and Upper Priest Lake are largely focused on bull trout, lake trout, westslope cutthroat trout and kokanee salmon. Because fish management is focused mainly on those species, we focused our assessment on the same species. A brief description of each of those species is outlined below.

Bull trout

Bull trout provided a successful fishery in Priest Lake and Upper Priest Lake until approximately the late 1970s when a rapid decline was noticed (Rieman et al. 1979). By the late 1990s Fredericks (1999) suggested that bull trout were only present in the upper lake in very low abundance. The population decline is likely attributable to competition with lake trout and the collapse of the kokanee population (Fredericks 1999).

Bull trout were listed as threatened under the Endangered Species Act (ESA) in 1998. The Priest Lake (including Upper Priest) population is one of 35 core areas within the Columbia Headwaters Recovery Unit (USFWS 2015). Bull trout have specific requirements for spawning and rearing in tributaries, which include clean gravel, cold water, high dissolved-oxygen, and complex channels with cover and pools. Some populations in the Priest Lake Core Area exhibit a resident life-history strategy, which means they spend their entire life in tributary streams. However, it is likely that most of the Priest Lake Core Area population exhibit an adfluvial life-history strategy, which means they spend a significant part of their adult life in the lakes and migrate to tributaries to spawn.

Lake Level Management

The Priest Lake water management study is evaluating lake level management for dry and moderately dry years so the resident (tributary) populations will not be effected. However, because adfluvial bull trout are known to inhabit the lake environments, we focused our assessment on estimating potential impacts different water management strategies might have on that population, if any.

Because spawning and early life-stage rearing occur in tributaries to the lakes, a potential impact could be access to tributary streams. Water level changes are proposed to temporarily increase slightly (3 to 6 inches) during the early summer and be at current levels during the bull trout migration periods. Therefore, access to tributary habitats will not be effected. It appears that the threats to the bull trout population in Priest Lake and Upper Priest Lake are primarily due to competition with Lake trout and low forage base and proposed water management adjustments are not likely to influence those interactions. It is our opinion that bull trout populations will neither benefit or be impacted by the proposed water management adjustments.

Thorofare Improvement

Potential bull trout migration through the thorofare are generally expected to occur when water temperatures are relatively cool (May and September/October are the most probable months). At this point it is unclear what alternatives will be for access improvement. However, if construction (e.g. dredging) is a preferred alternative, it could coincide with migrating bull trout. If construction is necessary, it will be conducted during daylight hours. Bull trout are known to primarily migrate at night so it is not likely that construction activities will stop migration into the thorofare. Additionally, regardless of the alternative selected for improvement, the channel will not be fully blocked and migration routes will be maintained as mandated by permit conditions. Fish exclusion measures such as block nets or bubble curtains, will be used to prevent impacts if and when disruptive activities are being conducted. Therefore, it is our opinion that thorofare improvement is not likely to negatively impact bull trout populations.

Westslope cutthroat trout

Historically, westslope cutthroat trout was the most popular fishery in the system (IDFG 2013). However, Bjornn (1957) reported that westslope cutthroat trout population and size had been declining since at least 1948. While it is unclear the exact causes of the cutthroat decline; overharvest, egg taking efforts and competition with non-native species likely contributed (Bjornn 1957, Beach 1971, IDFG 2013).

Adfluvial populations of westslope cutthroat trout in the Priest Lake system typically spawn in tributaries between approximately April and mid-June (Bjornn 1957, IDFG 2013). Fry emergence is generally complete by mid-August and juvenile fish rear in nursery streams for 2 to 3 years prior to moving into the lakes (Bjornn 1957). Fish generally mature at ages between 4 and 6, with most fish maturing at age 5 (Bjornn 1957 and IDFG 2013).

Lake Level Management

As it applies to lake management, it appears the only potential effect to the westslope cutthroat trout population could be impeding access to tributary streams. However, results of this study and subsequent management operations will not change considerably and passage into and out of tributaries will not change. Therefore, we conclude that westslope cutthroat trout populations will not be effected as a result of this project.

Thorofare Improvement

The extent of cutthroat migration through the thorofare is unknown. Because they spawn in the spring months in tributary streams, if migration in the thorofare is occurring, it is likely in the late winter/early spring, which will be outside the period when improvements would be made. Therefore, it is unlikely that cutthroat populations will be negatively affected as a result of thorofare improvement.

Lake trout

Lake trout were introduced into Priest Lake in 1925 (Bjornn 1957). From the time of their introduction through approximately the 1970s, their population density remained relatively low (Reiman et al. 1979, Venard and Scarnecchia 2005, IDFG 2013). After the introduction of kokanee in the early 1940s, their prey base increased dramatically, which made way for a trophy size fishery for lake trout until the late 1970s/early 1980s. The introduction of opossum shrimp in 1965, and their subsequent population increase, improved the juvenile lake trout forage abundance, which resulted in more successful adult recruitment and a dramatic increase in the lake trout population (Reiman et al. 1979, Venard and Scarnecchia 2005, IDFG 2013). The rapidly increasing lake trout population, coupled with the collapse of the kokanee population, essentially shifted the diverse yield and trophy fishery to one dominated by lake trout (IDFG 2013). In the 1970s about 200 lake trout per year were harvested with an average weight of 20 pounds (IDFG 2013). In contrast, the harvest rate has decreased dramatically and was up to 30,000 annually by 2003 with an average weight of 2.1 pounds (IDFG 2013, Ng et al. 2016).

Since the 1980s, when the lake trout population became more abundant, IDFG's policy has been to manage Upper Priest Lake for native fish conservation (westslope cutthroat trout and bull trout). However, lake trout immigration to the upper lake, through the thorofare, led to a population in the upper lake of 1,000 fish by the 1990s and has continued to increase (Mauser 1986, Fredericks 1999, Venard and Scarnecchia 2005, IDFG 2013, Ng et al. 2016). Gillnetting to remove lake trout have removed between 2,000 and 5,000 fish annually but those efforts have, at best, maintained the population (Liter and Maiolie 2003, IDFG 2013). Conceptually, blocking lake trout access through the thorofare seems to be a way to control the population in the upper lake but those efforts have largely been unsuccessful due to numerous logistical and social constraints (IDFG 2013).

Lake Level Management

Lake trout spend most of their life in the lake environment and already are experiencing lake level fluctuation. Therefore, it is unlikely that changes to lake level management will benefit or negatively impact lake trout populations.

Thorofare Improvement

As it applies to this project, improving navigation through the downstream end of the thorofare might provide better access for lake trout to the upper lake. However, as evidenced by the continued high immigration to the upper lake, it is apparent that passage through the thorofare isn't impeding migration currently. Therefore, we conclude that results of this study and subsequent actions are not likely to result in increased lake trout immigration to the upper lake.

Kokanee salmon

Kokanee were introduced into the Priest Lake system in the early 1940s (Bjornn 1957). The population grew rapidly in the lake and through 1971 averaged a harvest of nearly 100,000 fish annually (Bjornn 1957, Irizarry 1975, Reiman et al. 1979, IDFG 2013). Reiman et al. (1979) noted a collapse in the population in 1976 and by 1983 the kokanee harvest was less than 100 fish.

Lake drawdown, the introduction of opossum shrimp (*Mysis diluviana)* in 1965*,* and predation by a thriving lake trout population are the three factors that likely contributed to the kokanee collapse (Irizarry 1975, Rieman et al. 1979, Venard and Scarnecchia 2005, IDFG 2013). Each of these factors impacted the population in different ways but since lake level management is the focus of this study, drawdown and potential redd dewatering is the emphasis of our analysis.

Lake Level Management

In Priest Lake, kokanee spawn along the shoreline of the lake, generally beginning in mid-October and extending through mid-December with peak timing between mid-November through early December (Bjornn 1957 and Irizarry 1975). Irizarry (1975) found that the majority of kokanee observed spawned in water 10 inches deep and less and within 20 feet from the shoreline. The same study estimated that in a 1-mile stretch of beach, where spawning was occurring (near the mouth of Granite Creek), 45 percent of observed redds were dewatered by mid-December, which accounted for an estimated loss of over 1.2 million eggs. While redd dewatering undoubtedly contributed to the population decline, it is important to note that similar observations were being made as early as the 1950s (Bjornn 1957) and the population continued to thrive through the early 1970s. What's more, in the 1980's millions of kokanee fry were stocked in Priest Lake but that effort was unsuccessful. Additionally, Whitlock et al. (2015) found that spawning success in nearby Lake Pend Oreille was not related to depth or from higher winter water levels. Still, the threat and potential impacts of redd dewatering can't be ignored and it appears the Priest Lake kokanee population could benefit from stabilized lake levels during the spawning and incubation periods.

This study is evaluating water management strategies to maintain the lake level during the summer recreation period and timing of modifications will not coincide with kokanee spawning. By the time spawning begins, the dam gates will be open and lake levels will fluctuate as they currently do. It is unlikely that any modifications proposed during the summer recreation period will have an effect on kokanee spawning success but subsequent studies and water management strategies could be evaluated to improve spawning/incubation success (e.g. stabilize lake levels by November 1).

Thorofare Improvement

It appears that kokanee migration between the lakes is minimal, if at all. Therefore, it is unlikely that thorofare improvement will negatively affect kokanee populations.

SUMMARY AND CONCLUSIONS

Based on the study objectives and preliminary conclusions, relative to dry year lake management level strategies and thorofare improvements, it is our opinion that the focus fish populations in the lake are not likely to be negatively impacted (Table 1). However, our opinion is based on past research that was conducted for management purposes and did not assess lake level management and thorofare improvements specifically. As such, our conclusions should be considered as an opinion, based on available science, and not as a documented fact.

TABLE 1: SUMMARY OF THE FOCUS FISH SPECIES AND POTENTIAL EFFECTS ASSOCIATED WITH MODIFIED RECREATION SEASON LAKE LEVELS AND THOROFARE IMPROVEMENT CONSTRUCTION

Fish Species	Spawning location	Spawn timing	Effects of Recreation Season Lake Level Management	Potential Effects of Thorofare Improvement Construction
Bull Trout	Tributaries	September through October	None Anticipated	Possible Short-Term Delay in Migration during construction but Unlikely Because Passage will be Maintained
Westslope Cutthroat Trout	Tributaries	April through Mid-June	None Anticipated	None Anticipated
Lake Trout	Lakes up to 40 feet deep	September through October	None Anticipated	None Anticipated
Kokanee	Lake Shoreline	Mid-October through early-December Peak Mid-November through Mid-December	None Anticipated	None Anticipated

PERSONAL COMMUNICATIONS

In conducting the Priest Lake fisheries investigation for this project, we contacted the following people with the intent of using their experience and expertise to help us better understand fish populations and their associated habitats.

- Jason Connor – Kalispel Tribe of Indians
- Jason Olson – Kalispel Tribe of Indians
- Joe Maroney – Kalispel Tribe of Indians
- Kiira Siitari – Idaho Department of Fish and Game
- Andy Dux – Idaho Department of Fish and Game
- Jason Flory – US Fish and Wildlife Service
- Jill Cobb – US Forest Service

LITERATURE CITED AND REPORTS ASSEMBLED

In addition to the scientists we contacted about Priest Lake fisheries, we also collected and reviewed the following reports.

Beach, D.R. 1971. Lake and Reservoir Investigations: Survival and growth of resident and stocked cutthroat trout in Priest and Upper Priest Lake. Job Completion Report. Idaho Department of Fish and Game Project F-53-R-6.

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Irizarry, R.A. 1972. Survival and growth of resident and stocked cutthroat trout in Priest and Upper Priest lakes (survey). Job Completion Report Project F-53-R-7. Boise, Idaho.

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U.S. Fish and Wildlife Service (USFWS). 2015. Recovery plan for the coterminous United States population of bull trout (Salvelinus confluentus). Portland, Oregon. xii+179 pages.

Venard, J.A. and D.L. Scarnecchia. 2005. Seasonally dependent movment of lake trout between two northern Idaho lakes. North American Lournal of Fisheries Management 25:635-639.

Whitlock, S.L., M.C. Quist, and A.M. Dux. 2015. Incubation success and habitat selection of shore-spawning kokanee Oncorhynchus nerka: effects of water-level regulation and habitat characteristics. Ecology of Freshwater Fish 24:412-423.

A.5 Public Outreach Plan

PUBLIC OUTREACH PLAN

PRIEST LAKE WATER MANAGEMENT STUDY

February 20, 2018 – Rev C

IDAHO WATER RESOURCE BOARD

MOTT MACDONALD

PUBLIC OUTREACH PLAN

Priest Lake Water Management Study, Priest Lake, Idaho

March 2017

1. Project Background

The purpose of the Priest Lake Water Management Study is to conduct a feasibly assessment to evaluate operational improvements for the Priest Lake system with respect to the following primary criteria:

- Lake Level Management (maintain 3-ft level at USGS outlet gauge for recreation season and developing operational strategies that improve habitat and minimize shoreline impacts).
- Maintain current minimum discharge flows downstream of the dam (60 cfs).
- Provide sustainability for the Thorofare by promoting self-sustaining modifications to improve Thorofare access, navigability, and water quality.

2. Study Goals

The goal of the study is to develop a range of feasible alternative in consultation with stakeholders and the public for improving Priest Lake Water Management.

3. Study Organization & Identification

This study will be a collaborative effort including participants from local (Bonner County) and state government (Idaho Water Resources Board (IWRB), Idaho Department of Water Resources (IDWR)) organizations and the selected consultant team lead by Mott MacDonald. The following is a summary of study group organizations and their respective roles and responsibilities:

Consultant Team (CT). The study Consultant Team consists of Mott MacDonald (MM), GeoEngineers (Geo), and SMK Communication (SMK). The MM Project Manager will be the main point of contact for the Consultant Team for the duration of the study work. A technical lead may also be identified as the study progresses. The Mott MacDonald Project Manager and primary point-of-contact (POC) is:

Shane Phillips, P.E.
Phone: 425.778.6042
Cell: 425.417.6016
Email: shane.philips@mottmac.com

Study Team. Study team consists of Idaho Water Resources Board (IWRB), Bonner County (County) and the Consultant Team. The Study team is responsible for conducting and overseeing the study and development of recommendations to the IWRB. The Study Team is also responsible for development of criteria for the study, development of alternatives and development of a public outreach plan. IWRB and the County (Study Partners) are responsible for decision making during the course of executing the study. The members of the Study Team are listed in Appendix A. The Study Team Project Manager and primary point of contact is:

Neeley Miller
Phone: 208.287.4831
Email: Neeley.Miller@idwr.idaho.gov

Steering Committee. A steering committee composed of local stakeholders as an advisory group was developed by the County and IWRB. The steering committee was formed to assist the Study Team with history and local knowledge of the Priest Lake System. The members of the Steering Committee are listed in Appendix B. The primary point of contact for the Steering Committee is:

Steve Klatt
Phone: (208) 255-5681, ext 4.
Email: sklatt@bonnercountyid.gov

Stakeholders. The Steering Committee (with input from the Study Team) will be responsible for identifying other local, state, federal agency, Tribal, property owner, business groups, homeowner's associations, recreational user groups and other interested stakeholders who will have interest in the project and will need to be part of the outreach plan. Stakeholders may include those identified in Appendix C.

Public. The public includes landowners, lessees, local businesses, anglers, recreational users, businesses, and a broader group of public having an interest in the study and Priest Lake system. A communication plan for outreach to a diverse group of public will be developed to inform the public regarding status of the study.

Public Outreach Points of Contact. The following individuals are the primary points of contact for public outreach.

Neeley Miller
Phone: 208.287.4831
Email: Neeley.Miller@idwr.idaho.gov

Susan Keibert, MM CT
Mobile: (208) 597-4219
Email: skiebert@lhtac.org

Sample outreach groups and their level of engagement

Audience	Who they are	Spectrum of engagement
Policy group – IWRB/County	Individuals with decision-making authority (internal and external);	Inform/consult *Final decision
Steering Committee	Representatives assigned to represent a cross section of stakeholders and effected groups	Collaborate
Stakeholder group	Agency representatives directly involved in management of Priest Lake, fishery, lake area dependent businesses, representatives of affected landowners and economic interests, others if overlooked	Involve
Interested public	Landowners and lessees, local businesses, anglers, agency staff, recreational users, etc.	Consult
Unsurprised apathetic public	Broader public who does not have a direct interest in actively participating in process (local and regional)	Inform

A graphic depicting the outreach organizations A summary the outreach plan is outlined in Appendix D.

4. Public Outreach Objectives

The study team has identified the following objectives for public outreach as part of the study process:

- Inform the public regarding the process for and progress of study development
- Provide meaningful opportunities for stakeholders and the general public to provide input during the study process.
- Incorporate stakeholder and public input regarding issues of concerns
- Document stakeholder recommendations in a clear, complete manner for consideration by the Study Team.
- Prepare and distribute background and technical information to public, stakeholders
- Develop and demonstrate stakeholder support and understanding regarding the study and outcomes
- Monitor outreach activities for effectiveness and adjust the approach as necessary during implementation.
- Focus on quality outreach activities. It is better to perform limited stakeholder input efforts well than to do large-scale efforts poorly.

To pursue these objectives, it is recognized a need to match appropriate involvement opportunities and outreach methods with groups targeted for inclusion in the process.

5. Assumptions

The following assumptions were developed as a part of this public outreach plan:

- The decision/decisions will be clearly defined by IWRB/County.
- The final decision-maker will commit to considering the recommendations generated with public input in good-faith.
- Steering Committee, IWRB and County wants to actively engage with a subset of stakeholders to build consensus around.
- County/IWRB will take the lead on identifying a subset of stakeholders with input from MM team.
- IWRB wants to hear input from a broader public and is comfortable with a relatively transparent process.
- Sufficient time and resources are available to support meaningful public engagement.
- Public engagement is fully integrated into the SOW.

6. Steps/Methods for Public Outreach

The following are key steps in development and implementation of a public outreach plan for this project:

- Preliminary Planning & Design
    - Situational analysis, decision process, informational exchange, stakeholder identification, planning team development, available resources, approval process.
    - Organize participation. Identify and get to know stakeholders, pick appropriate level of public participation, integrate public participation into decision process, match public participation tools to objectives throughout process.
    - Timeline: Early March to mid-April.

- Stakeholder Engagement Plan Development
    - Identify objectives, desired outcomes, major issues, key stakeholder development, what information is desired from stakeholders, methods, set up system for preparation and receipt of information, critical path schedule items, resources, roles, and responsibilities.
    - Timeline: March to early April.
- Plan Implementation
    - Apply public participation method, provide/receive information, monitor process and adapt.
    - Timeline: Early March to mid-April.
- Feedback
    - Reporting to decision makers, participants, evaluate overall process.
    - Timeline: Ongoing April through end of Study.

7. Decision Planning

Decision making will need to be developed and described. Define and explain in public process at public meetings and on website.

- Study Scope (already by IWRB/County), outreach plan, evaluation criteria, alternatives to be evaluated, etc. - Study Team.
- Members of Steering Committee, stakeholders group contacts – IWRB/County.
- Location of meetings – MM Team and Study Team.
- Content for distribution materials – MM Team & Study Team.
- Project Schedule and key steps – MM Team & Study Team.
- Key dates for public involvement selected – MM Team & Study Team.

8. Outreach Methods and Tools

The following outreach methods and tools will be utilized as needed through the public involvement process. Outreach can make information available, share information in a one-way direction (inform or be two-way direction through interacting with public representatives (consult) or with steering committee (involve and consult). The following tools and methods will be employed to gain the needed outreach for the study process.

- IWRB Board Meetings – 3 total.
- Steering Committee Meetings – 5 total.
- Stakeholder Outreach Meetings - # to be determined.
- Public Meetings – 2 in person.
- Interviews – targeted stakeholders throughout study process. Follow up after meetings.
- Website – ongoing throughout study.
- Social Media – Periodic Announcements as needed.
- Virtual Open House – Use of fact sheets and notifications; anticipated maybe 1.
- Press Releases – Notifications periodically through study duration if needed.
- Mailers – Notifications periodically through study duration if needed.
- Fact Sheets – periodically as needed.
- Newsletters – if determined to be needed.
- Others.

9. Summary Major Outreach Meetings

The following is a summary of major outreach activities currently outlined in the MM scope. Additional outreach activities will be needed which are assumed to be conducted by other Study Team members.

- Three steering committee workshops.
- Stakeholder outreach meetings.
- Two public meetings.
- IWRB meeting progress briefings (as requested, minimum three); 1 in-person; others materials to be provided.
- Five public progress updates (email or other); provide materials, no CT attendance.

10. Steering Committee Workshops

- Steering Committee #1 (3.20.17)
  - Work Scope Discussion, Alignment of study purpose/goals, alignment of public outreach purpose/goals, public outreach outline – Inform/Consult
  - Planning – Consult
  - Decision Planning – Consult
  - Lead – MM
- Steering Committee #2 (5.11.17)
  - Work Update (Analysis Summary) - Inform
  - Criteria Development – Consult
  - Range of Alternatives Discussion – Consult
  - Stakeholder Input – Consult
  - Information Sharing – Consult
  - Lead – MM
- Steering Committee #3 & Stakeholder (6.8.17)
  - Work Update (Analysis Update) – Inform
  - Alt. Evaluation – Inform
  - Additional Alternatives Discussion – Consult
  - Stakeholder Input – Consult
  - Information Sharing – Consult
  - Lead - MM
- Stakeholder Committee #4 (7.14.17)
  - Work Update (Alt Analysis) – Inform
  - Stakeholder Input – Consult
  - Preparation for public meeting – Consult
  - Lead – MM

11. Stakeholder Outreach Meetings & Informational Briefings

- State/Federal Agency Kickoff Meeting (4.18.17)
  - Work Scope Discussion, Alignment of study purpose/goals, alignment of public outreach purpose/goals, public outreach outline – Inform.
  - Planning – Consult.

  - Lead – MM Team.
  - Invitees.
    - IWRB/IDWR
    - Idaho Dept. of Lands
    - Idaho Fish & Game
    - Idaho Parks
    - Idaho DEQ
    - US Forest Service
    - US Fish & Wildlife Service
    - Kalispell Tribe
    - U.S. Army Corps of Engineers
- State/Federal Agency Progress Meeting (6.27.17)
  - Work status discussion.
  - Lead – TBD.
- Property/Landowner Groups (Dates TBD).
  - Work Scope Discussion, Alignment of study purpose/goals, alignment of public outreach purpose/goals.
  - Lead – TBD (Shared Responsibility).
  - Invitees. As outlined in Appendix C.
- Lakes Commission (May 2 and Oct 27).
  - Work Scope Discussion, Alignment of study purpose/goals, alignment of public outreach purpose/goals.
  - Lead – County.
- State/Federal Agency Progress Meeting (10.10.17)
  - Work status discussion.
  - Lead – TBD.

12. Public Meetings

- Number: 2 in person public meetings.
  - July 20 – Introduce project scope, objectives, public outreach overview, analysis overview, potential types of alternatives, next steps and next meeting.
  - September 22 – Update on analysis, alternatives evaluation results, next steps.
- Location. Priest Lake area, meeting space TBD.
- Format. TBD. Susan to provide input.
- Lead – MM coordination, content. Meeting execution shared responsibility (MM, IWRB, County).

13. IWRB Board Meetings

IWRB staff to provide quarterly updates to the Water Resources Board. Anticipated meeting dates include the following:

- Board Meeting #1 (5.18.17)
  - Work Scope Discussion, Alignment of study purpose/goals, alignment of public outreach purpose/goals, public outreach outline – Inform/Consult
  - Planning – Consult
  - Decision Planning – Consult

  - Lead – IWRB
- Board Meeting #2 (7.28.17)
  - Work Update (Analysis Summary) - Inform
  - Criteria Development – Inform
  - Range of Alternatives – Inform/Consult
  - Stakeholder Input – Inform
  - Information Sharing – Inform/Consult
  - Lead – IWRB
- Board Meeting #2 (9.11.17)
  - Work Update (Analysis Summary) – Inform
  - Range of Alternatives – Inform/Consult
  - Information Sharing – Inform/Consult
- Board Meeting #3 (10.24.17)
  - Work Update (Analysis Summary) - Inform
  - Analysis Results – Inform
  - Alternatives Evaluation Results – Inform/Consult
  - Stakeholder Input – Inform
  - Information Sharing – Inform/Consult
  - Lead – IWRB

14. Interviews

- Audience. Targeted Stakeholders. Identified throughout study process to acquire additional technical information or regarding status of the outreach and study efforts to ascertain if dissemination of information is effective.
  - Examples: Ken Hagman (local conditions in Thorofare), Duncan (Outlet Structure Operations), etc.
- Frequency. As needed to follow up on information and details needed to feed into study and ensure message is being disseminated. Follow up after key meetings to get input from stakeholders to feed back into the study process.
- Responsibility. Entire Study Team; coordination of who to make contact and then follow up with written notes to entire study team afterwards.
- Lead. Shared Responsibility depending on entities to be contacted. Split up amongst the Study Team (IWRB, County, MM Team).

15. Website

- Hosted & Managed By IWRB
  - Setup, periodic updates, processing of questions and routing to study team.
  - Monitor number of times posted files are accessed, survey web visitors regarding usefulness and ease of use, conduct brief telephone survey with residents to identify their sources of information and level of understanding.
- Content Goals
  - Project Background, goals, study team description, schedule, upcoming meetings, status, new information, maps, fact sheets and Question and Comments sections.
- Content input from MM Team

  - Assist with concepts for graphic, content and layout.

16. Social Media

- Managed By Bonner County
  - Will send periodic notifications to local social media to direct interested public to the IWRB managed website to review new information and for notification of upcoming meetings.
  - As the Lake Churns and other Priest Lake area social media sites.

17. Virtual Open House

- Managed by IWRB on website.
- Content. Use of fact sheets and notifications to inform and solicit input through online Question and comment forms.

18. Press Releases

- Managed by IWRB/County
- Content assistance from MM Team.
- Frequency. Notifications periodically through study duration.

19. Mailers

- Managed by IWRB/County
- Content assistance from MM Team.
- Frequency. Notifications periodically through study duration

20. Fact Sheets

- Managed by IWRB/County
- Content assistance from MM Team.
- Frequency. Notifications periodically through study duration

Timeline, activities, and milestones

Timeframe	What	Who	Activity/Tool	Purpose and Desires Outcome	Spectrum
March 2017	Decision analysis for public participating component	IWRB and consultant team	In-person meeting as part of project kick-off meeting	• Identify decisions being made and framework for those decisions • Confirm final decision maker • Confirm decision stages and overall schedule • Identify any institutional constraints and unique circumstances • Identify level of public participation needed and desired outcomes of public participation	N/A
March 2017	Public participation process planning	IWRB and consultant team plus others as identified	In-person meeting and one-on-one outreach (interviews, etc.)	• Stakeholder and issue identification • Identify potential areas of controversy and approaches to mitigate • Identify desired outcome of public participation at each stage (who, what, why) • Identify appropriate public participation techniques to support above • Complete public participation plan	N/A
March 2017	Steering Committee #1	Steering Committee	Telecon	• Team Introduction • Overview of Scope of Work • Stakeholder and issue identification • Identify potential areas of controversy and approaches to mitigate • Identify desired outcome of public participation at each stage (who, what, why)	Involve
April 2017	Announce planning process, timeframe and opportunities for participation	Policy group, publics and identified stakeholders	Press release, emails, IWRB posting, paid advertisement, etc.	• Provide updated information about the process, timeline and desired outcome to publics, stakeholders and policy makers • Provide opportunity for interested publics to identify themselves and participate	Inform
April/May 2017	Progress update	IWRB	IWRB meeting	• Update on progress, opportunity for IWRB Q&A prior to outreach (no surprises)	Consult
May 2017	Steering Committee workshop (Meeting #2)	Steering Committee	Facilitated workshop	• Provide overview of project framework, timeline, milestones and roles and responsibilities of stakeholder group and other decision-makers • Present proposed management objectives • Present potential evaluation criteria	Involve

Timeframe	What	Who	Activity/Tool	Purpose and Desires Outcome	Spectrum
				• Discuss any materials presented and document response and recommendations from stakeholder group (workshop processes TBD)	
June 2017	Progress update	Policy group, IWRB, publics and identified stakeholders	Email newsletter, web site posting, mailing as needed	• Maintain momentum and sense of progress and transparency • Communicate results of initial stakeholder workshop to larger group • Provide opportunity for comment and Q&A, post answers on web site • Review schedule and upcoming opportunities for participation (keep door open)	Inform
July 2017	Steering Committee Meeting #3 & Stakeholder Meeting	Steering Committee and Stakeholder	Facilitated workshop (maybe 2 days depending on group and complexity of information)	• Recap results of initial stakeholder workshop and how their input was or was not incorporated into management objectives and evaluation criteria – and why if not. • Present alternatives and impact analysis alternatives for pool raise • Present alternatives and analysis of dam operations • Present potential range of alternatives for Breakwater and Thorofare improvements • Build stakeholder understanding of analysis, identify and address questions, discuss and document any additional recommendations, input on preferred alternatives, and rationales for that input	Involve & Consult
July 2017	Progress update	IWRB	IWRB meeting	• Update on progress, opportunity for IWRB Q&A prior to outreach (no surprises)	Consult
July 2017	Steering Committee Meeting #4	Steering Committee	Telecon	• Update status of study work and analysis results. • Preparation for Public Meeting #1	Consult
July 2017	Progress update	Policy group, IWRB, publics and identified stakeholders	Email newsletter, web site posting, mailing as needed	• Maintain momentum and sense of progress and transparency • Announce public meeting • Summarize results of second stakeholder workshop • Review schedule and upcoming opportunities for participation (keep door open)	Inform

Timeframe	What	Who	Activity/Tool	Purpose and Desires Outcome	Spectrum
July 2017	Public meeting #1	Interested publics	Public meeting	• Provide overview of project framework, timeline, milestones and roles and responsibilities of stakeholder group and other decision-makers • Present summary of alternatives analysis, stakeholder group input, and preferred alternatives • Answer questions and hear and document additional input from public	Consult
September 2017	Progress update	IWRB	IWRB meeting	• Update on progress, opportunity for IWRB Q&A prior to outreach (no surprises)	Consult
September 2017	Steering Committee Meeting #5	Steering Committee	Facilitated workshop	• Recap project framework, timeline, etc. • Recap results of initial stakeholder workshop and how their input was or was not incorporated into management objectives and evaluation criteria – and why if not. • Present preferred concepts for pool raise and Thorofare (refined) • Present any additional information on analysis of dam operations and maintenance (next steps, etc.) • Address questions, identify and document any additional concerns and/or recommendations, and level of buy-in among stakeholder group	Involve
September 2017	Public meeting #2	Interested publics	Public meeting	• Review project framework, timeline, milestones and what's happened to date • Present refined concepts for alternatives evaluated • Review stakeholder and public input and how that was incorporated or not and why • Answer questions and hear and document additional input from public • Explain what happens next	Consult
October 2017	Board meeting #3	IWRB	IWRB meeting	• Present final study results	Inform
October 24, 2017	IWRB meeting	IWRB	Presentation	• Overview of study results • Respond to questions • Gain input to finalize report	Inform

Timeframe	What	Who	Activity/Tool	Purpose and Desires Outcome	Spectrum
December 2017 or January 2017	Progress update	Policy group, IWRB, publics and identified stakeholders	Email newsletter, web site posting, mailing as needed	• Notice of completion of final report. Link to report if public. • Thank you to participants • Explanation of what happens next and timeline if known	Inform
				•	

APPENDIX A

MEMBERS OF STUDY TEAM

Contact Name	Organization	Role	Phone	Email	Address
Neeley Miller	Idaho Water Resource Board	Project Manager	Ph: 208.287.4831 Mobile:	Neeley.Miller@idwr.idaho.gov	322 East Front St PO Box 83720 Boise, ID 83720-0098
Rick Collingwood	Idaho Water Resource Board	Contract Manager	Ph: 208.287.4835 Fax: 208.287.6700	rick.collingwood@idwr.idaho.gov	322 East Front St PO Box 83720 Boise, ID 83720-0098
Cynthia Bridge Clark	Idaho Water Resource Board	Program Lead	Ph: 208.287.4817 Fax: 208.287.6700	cynthia.clark@idwr.idaho.gov	322 East Front St PO Box 83720 Boise, ID 83720-0098
Steve Klatt	Bonner County Parks and Waterways	Study Sponsor Lead	Ph: 208.255.5681; Ext #4	sklatt@bonnercountyid.gov	1500 Highway 2 Sandpoint, ID 83864
Shane Phillips	Mott MacDonald	CT Project Manager	Ph: 425.778.6042 Mobile: 425.417.6016	shane.phillips@mottmac.com	110 James St, Ste 101 Edmonds, WA 98020
Vladimir Shepsis	Mott MacDonald	Technical Advisor	Ph: 425.778.6733 Mobile: 425.231.2488	vladimir.shepsis@mottmac.com	110 James St, Ste 101 Edmonds, WA 98020
Younes Nouri	Mott MacDonald	Project Engineer	Ph: 425.778.4687 Mobile: 410.905.5119	younes.nouri@mottmac.com	110 James St, Ste 101 Edmonds, WA 98020
Christoffer Brodbaek	Mott MacDonald	Geotechnical/Structural Engineer	Ph: 925.398.7260 Mobile: 415.350.8187	christoffer.brodbaek@mottmac.com	4301 Hacienda Drive Ste 300 Pleasanton, CA 94588
Jason Scott	GeoEngineers	Regulatory/Biology	Ph: 509.209.2816 Mobile: 509.953.4462	jscott@geoengineers.com	523 East Second Ave Spokane, WA 99202
Tim Hanrahan	GeoEngineers	Hydrology	Ph: 509.209.2821	thanrahan@geoengineers.com	523 East Second Ave Spokane, WA 99202
Susan Kiebert	SMK Communication	Public Involvement	Ph: 208.264.6401 Mobile: 208.597.4219	skiebert@lhtac.org	231 N Third-St. 108 Sandpoint, ID 83864

APPENDIX B

MEMBERS OF STEERING COMMITTEE

Priest Lake for the 21st Century

IDAHO WATER RESOURCE BOARD's WATER MANAGEMENT STUDY

Steering Committee & Prime Contact List

Priest Lake Residential Representative

Tom Weitz, ph. = 406 431-1219, e-mail = weitzinhel@msn.com

Priest Lake Commercial Representative

Ken Hagman, ph. = 208 443-2193, e-mail = ken@copperbayconstruction.com

Priest Lake Environmental Representative

Bruce Yocum, ph. = 208 443-0703, e-mail = ryocum@hughes.net

Idaho Water Resource Board

Dale Van Stone, Water Board Member: ph. = 208 264-5225, e-mail = dvanstone19@hotmail.com

Idaho Department of Fish & Game

Kiira Siitari, ph. = 208 769-1414, e-mail = kiira.siitari@idfg.idaho.gov

Lakes Commission

Craig Hill, ph. = 208 443-2551, e-mail = craighill@hillsresort.com

Bonner County

Steve Klatt, Director, Bonner Country Parks & Waterways: ph. = 208 255-5681 ext. 4, e-mail = sklatt@bonnercountyid.gov

Idaho Department of Water Resources Staff

Cynthia Bridge Clark, Water Projects Section Manager: ph. = 208 287-4817, e-mail = cynthia.clark@idwr.idaho.gov

Rick Collingwood, Staff Engineer: ph. = 208-287-4835, e-mail = rick.collingwood@idwr.idaho.gov

Neeley Miller, Senior Water Resource Planner: ph. = 208-287-4831,
e-mail = Neeley.Miller@idwr.idaho.gov

Morgan Case, Northern Region Manager: ph. = 208-762-2800,
e-mail = morgan.case@idwr.idaho.gov

Public Involvement

Julie Halliday, Bonner County Waterways: ph. = 208 290-8570, e-mail = jhalliday@bonnercountyid.gov

Molly McCahon, BS&WD Lakes Commission: ph. = 208 263-5310 ext 107, e-mail = lakescommission@gmail.com

Mott MacDonald - Susan Kiebert

APPENDIX C

STAKEHOLDERS

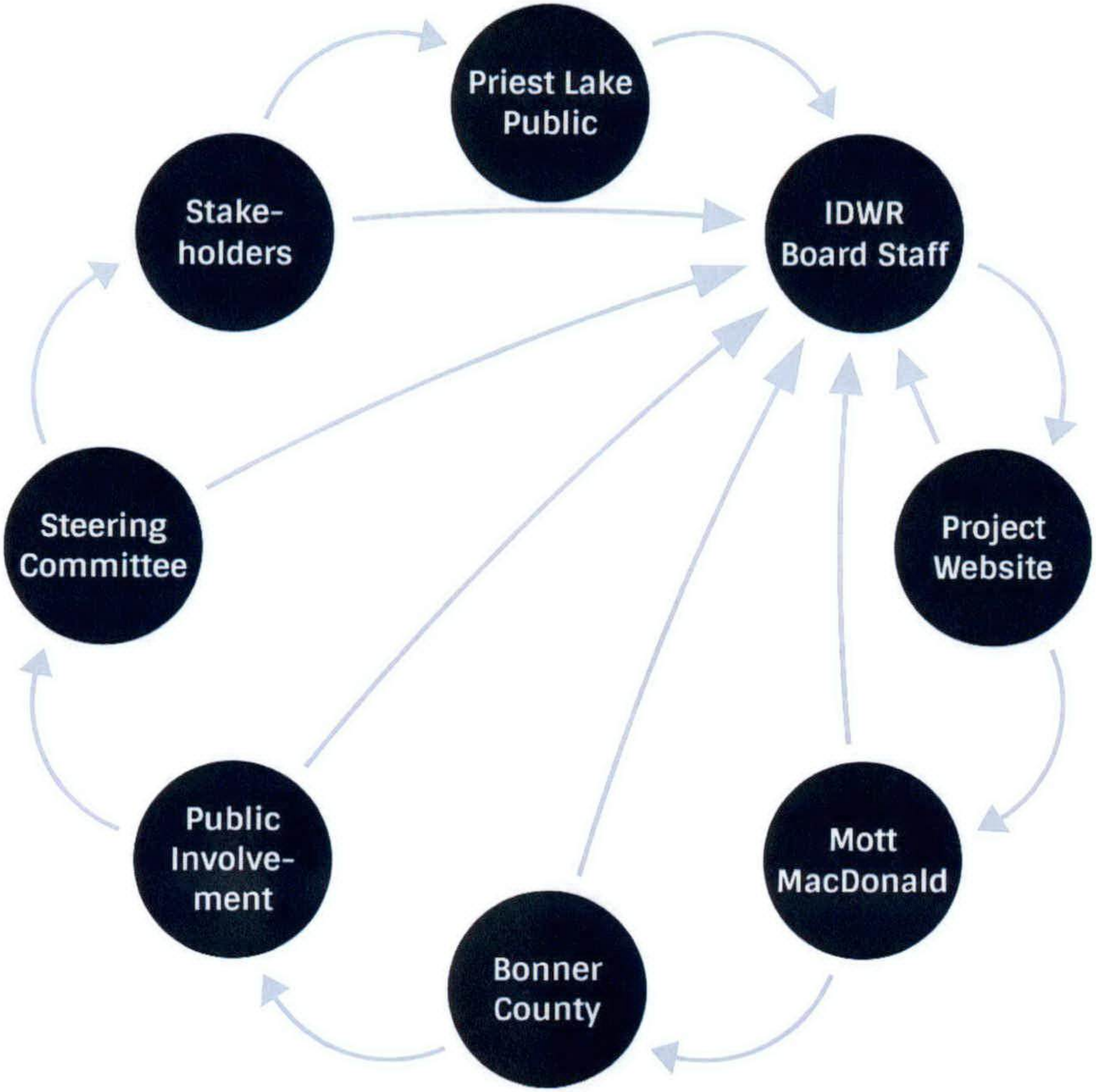
PRIEST LAKE STAKEHOLDERS COLLABORATIVE

AREA	NAME	PHONE	E-MAIL	respond	
Beaver Creek	Mary Rutherford	509 448-5405			
Granite Creek	Bob Mansfield	509 990-8121			
Reeder Bay	Pam Martin	443-2001/946-3374	pam@entreegallery.com		
Kalispell Bay	Rob Ward	443-3279/610-6806	sevenbreezes@gmail.com		
USFS Lessees - 121	Eric Johnson		ejpluskj@comcast.net		
Shoshone Bay	Chris Martinson	509 378-2620			
Outlet Bay					
Coolin	Bob & Geo'ne Bonds	801 272-0104			
Sherwood Beach					
Steamboat Bay					
Cavannaugh Bay					
State Lease Assoc	George Nethercutt		george.nethercutt@gmail.com		
Eight Mile Island					
Cape Horn					
Bear Creek					
Diamond Park					
Huckleberry Bay					
Sandy Shores					
Sandpiper Shores	Stephanie Coy	509 270-6514	Stephanie.coy1@yahoo.com		
Resorts/Marinas					
Condos/HOAs					
Selkirk Conservat'n	Cheryl Moody	208 443-0760			

APPENDIX D

OUTREACH ORGANIZATION GRAPHIC

Priest Lake Study Information Flow



Priest Lake Public

IDWR Board Staff

Project Website

Mott MacDonald

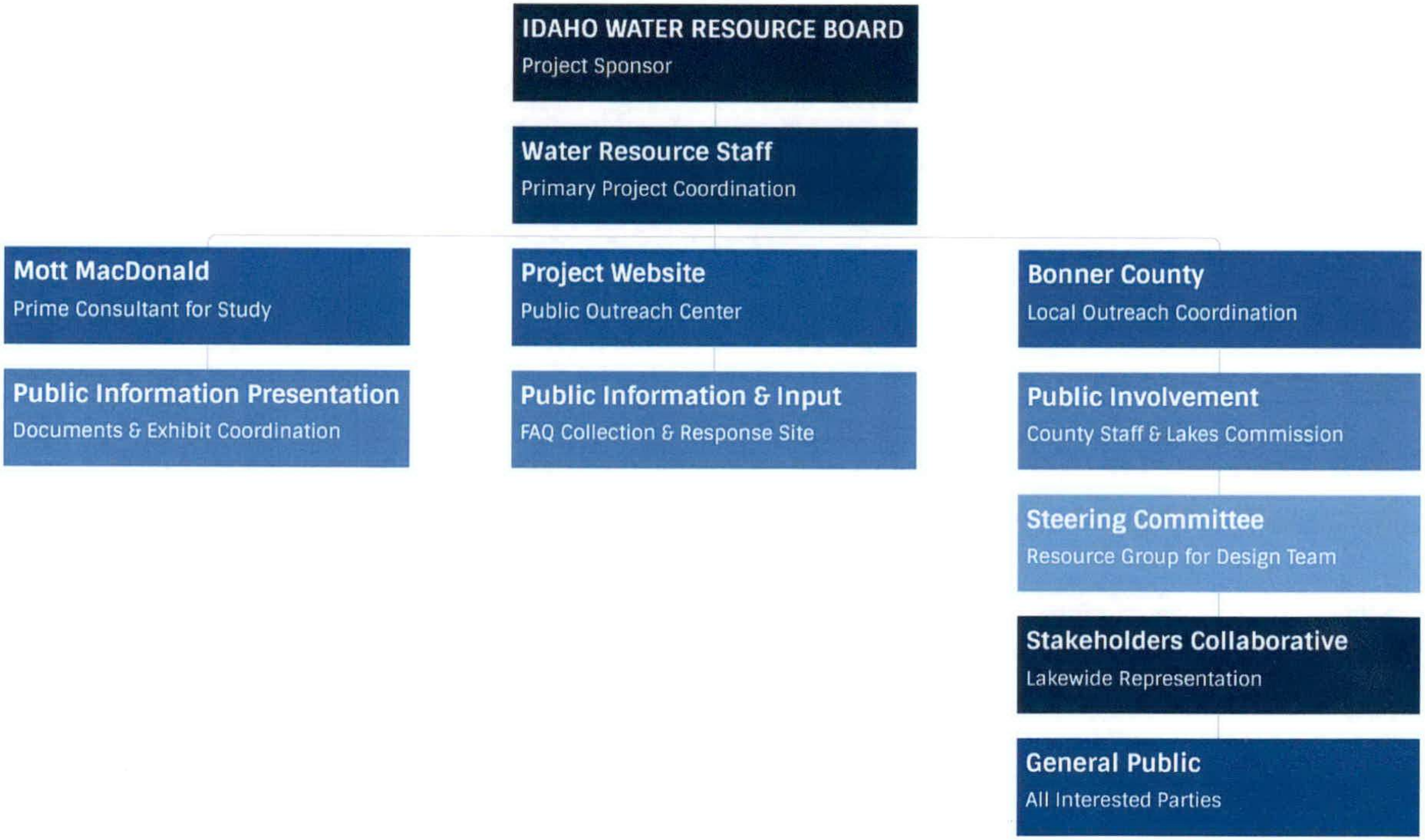
Bonner County

Public Involvement

Steering Committee

Stakeholders

PRIEST LAKE STUDY - 2017



IDAHO WATER RESOURCE BOARD
Project Sponsor

Water Resource Staff
Primary Project Coordination

Mott MacDonald
Prime Consultant for Study

Public Information Presentation
Documents & Exhibit Coordination

Project Website
Public Outreach Center

Public Information & Input
FAQ Collection & Response Site

Bonner County
Local Outreach Coordination

Public Involvement
County Staff & Lakes Commission

Steering Committee
Resource Group for Design Team

Stakeholders Collaborative
Lakewide Representation

General Public
All Interested Parties

created with VIZZLO

A.6 Public Outreach Comments

HISTORY & PURPOSE OF STUDY

BRIEF HISTORY

In 2015, drought conditions made maintaining the required summer lake levels & minimum discharge from the dam very difficult. In addition, there are concerns about the breakwater structure & Thorofare access.

PURPOSE OF STUDY

Evaluate operation improvements to achieve these three goals:

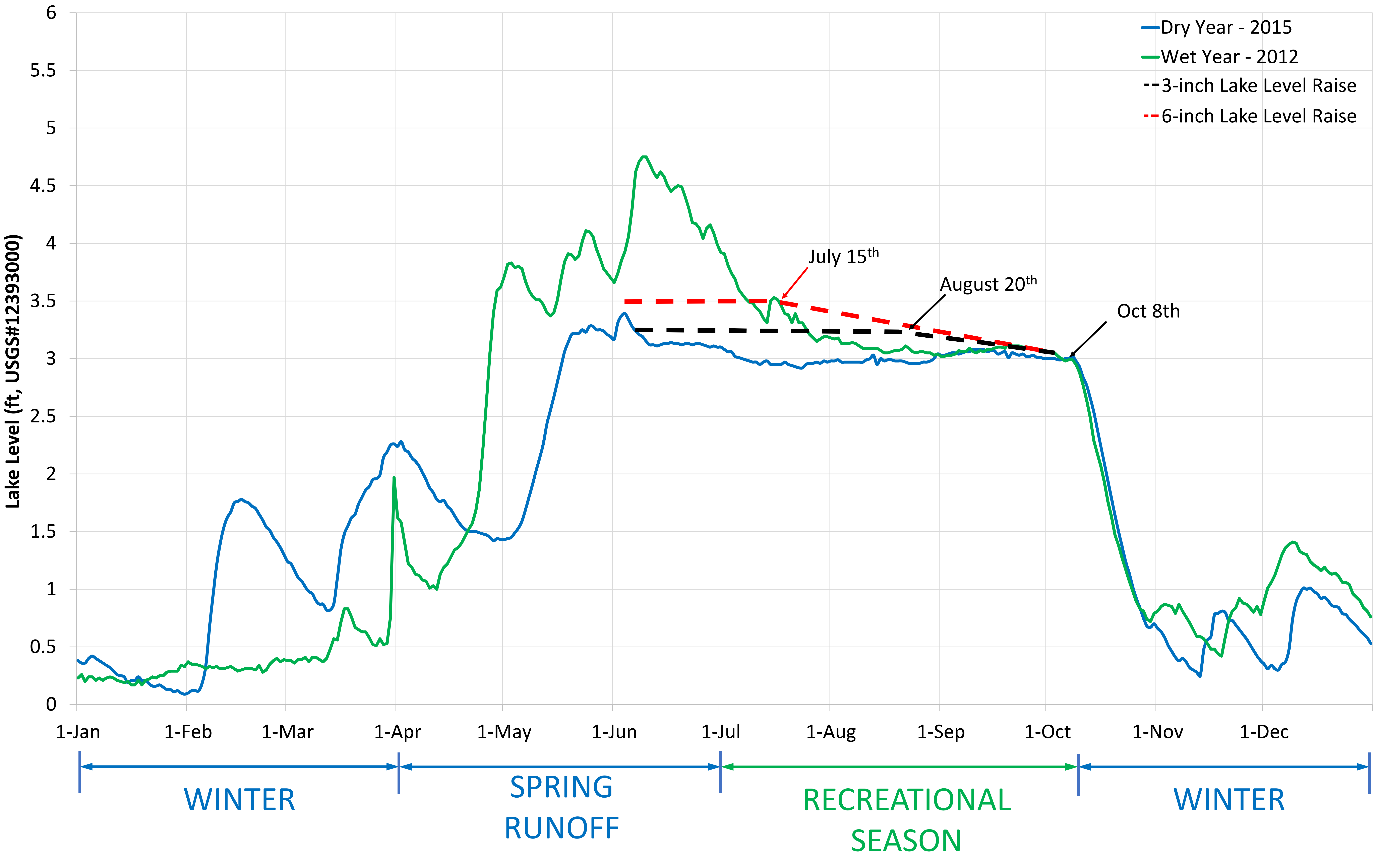
- Preserving lake levels through the recreational season during dry & marginally dry years. This goal supports the local economy and meets statutory lake level requirements.

- Maintaining a minimum discharge of 60 cfs from the dam during recreational season.

- Providing sustainable modifications to improve Thorofare access, navigability, & water quality.

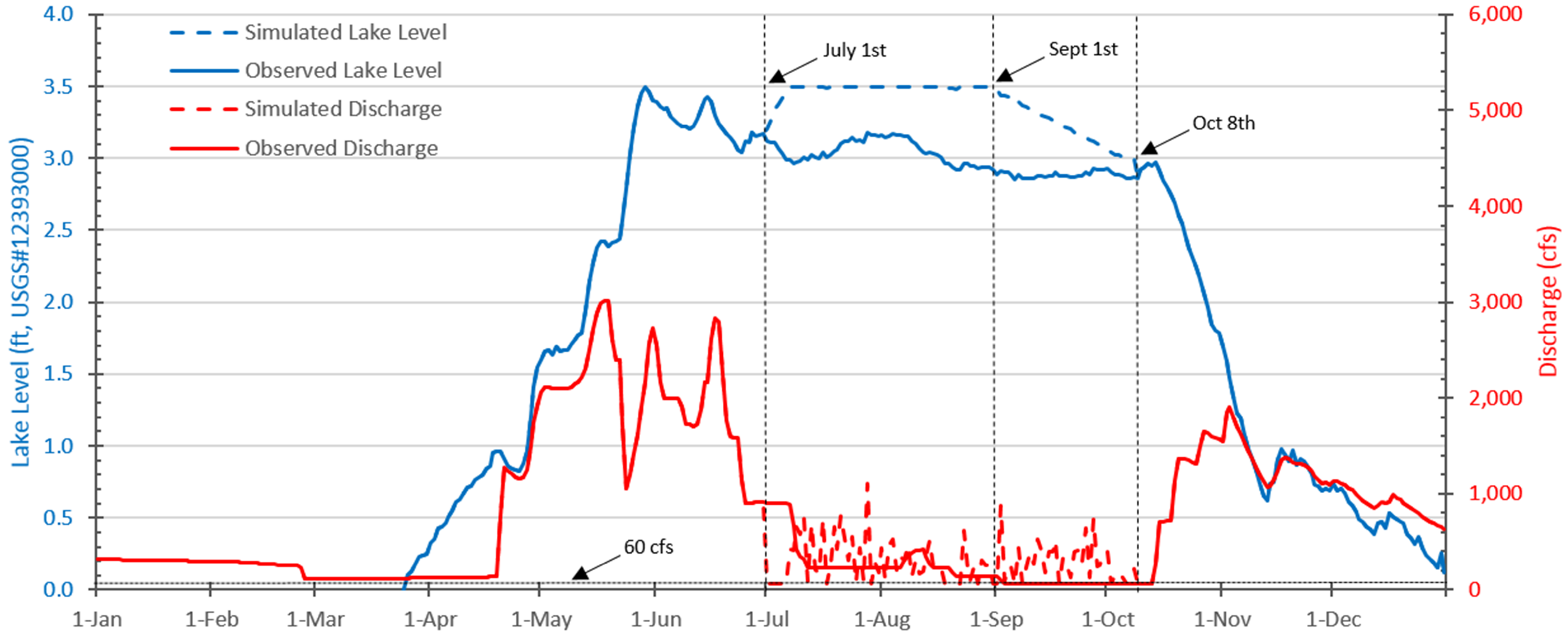
WATER LEVEL MANAGEMENT ALTERNATIVES FOR DRY YEARS

Temporary 3-inch & 6-inch Lake Level Raise Alternatives



WATER LEVEL MANAGEMENT SCENARIOS FOR DRY YEARS

Dry Year 2001



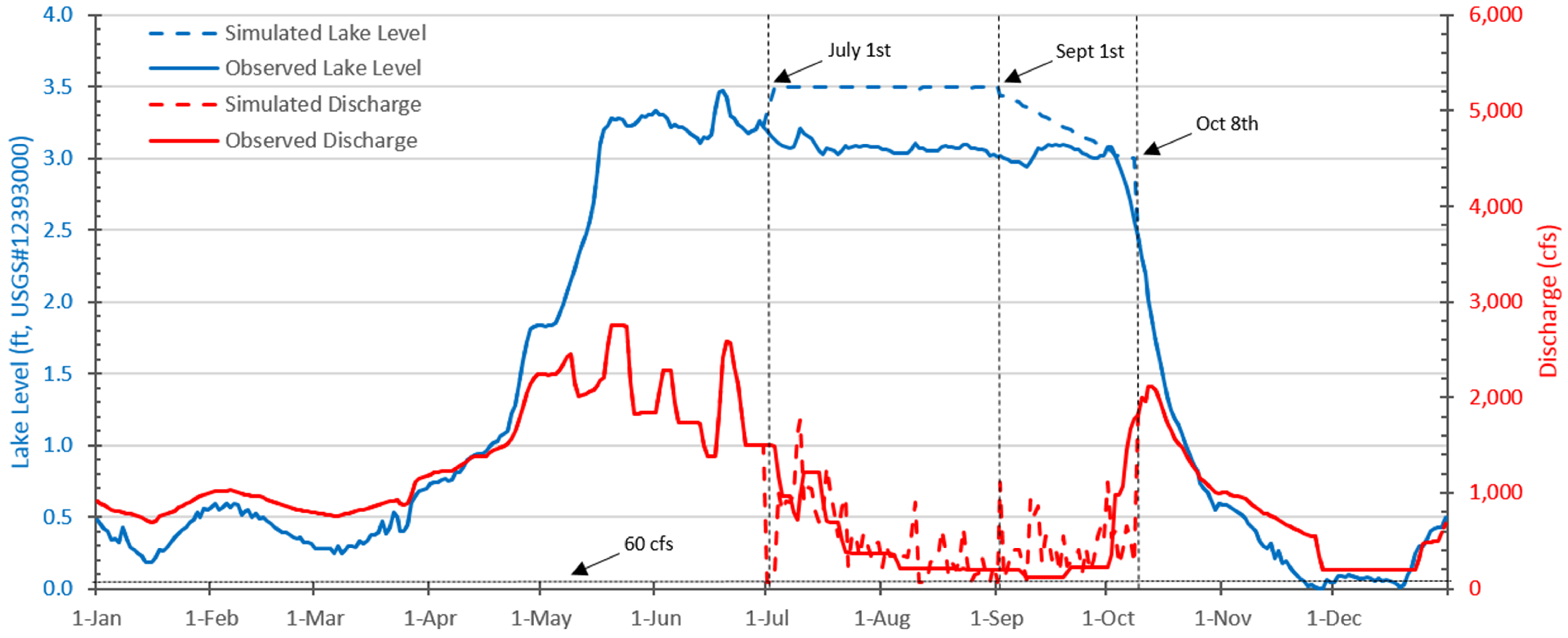
WINTER

SPRING RUNOFF

RECREATIONAL SEASON

WINTER

Dry Year 2005



WINTER

SPRING RUNOFF

RECREATIONAL SEASON

WINTER

ASSESSMENT OF HIGHER WATER LEVEL

Shoreline Features

Recreational Beaches

- Pier Access
- Benches
- Beaches

- Stairs
- Fire Pit

Public/Private Boarding Access

- Pier Access
- Boat Ramps

Boat Covers

- Fixed Height Structures
- Height of Vessel Adjustments

Natural Shoreline

- Natural Shoreline Features
- Rocks, Vegetation
- Large % of Overall Lake Shoreline

Additional Considerations

Vessel Wakes

Basement Flooding

Wetlands

Dry Year Pool Raise Assessment Summary

Temporary pool raise is being considered as an improvement measure **only** for dry and marginally dry years. Therefore, any possible impact will be limited to these years.

Alternative	Recreational Beach Use	Lake Shoreline Erosion	Access to Fixed Structures	Navigation Access to Marinas	Boat Launch Facilities	Fish Habitat	Thorofare Navigation	Wetland & Riparian Vegetation	Basement Flooding
3-inch Pool Raise	▨	▨	▨	🟩	▨	▨	🟩	▨	▨
6-inch Pool Raise	(1) 🟨▨	▨	(2) 🟨▨	🟩	▨	▨	🟩	▨	▨

Change with respect to a typical or a wet year:

- 🟩 Positive Change
- ▨ No or Negligible Change
- 🟨 Low Impact
- 🟥 High Impact

Footnotes:

(1): There will be no impact on majority of the beaches. Localized areas will see loss of usable dry beach.

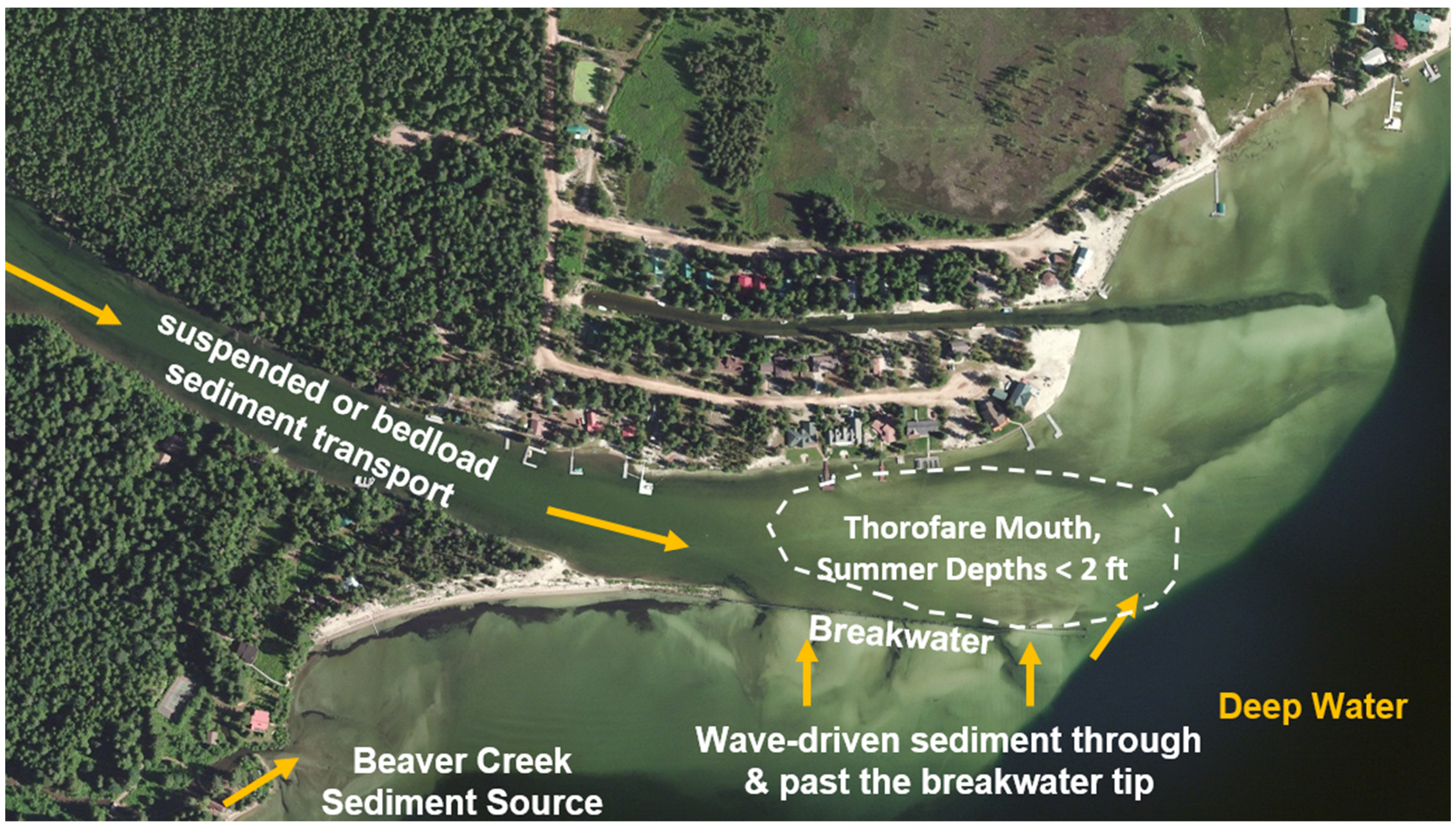
(2): There will be no impact on majority of fixed structures. A low percentage of structures will see low impacts.

Priest Lake Water Management Study

THOROFARE HYDRAULICS & SEDIMENTATION

Flow Spreading

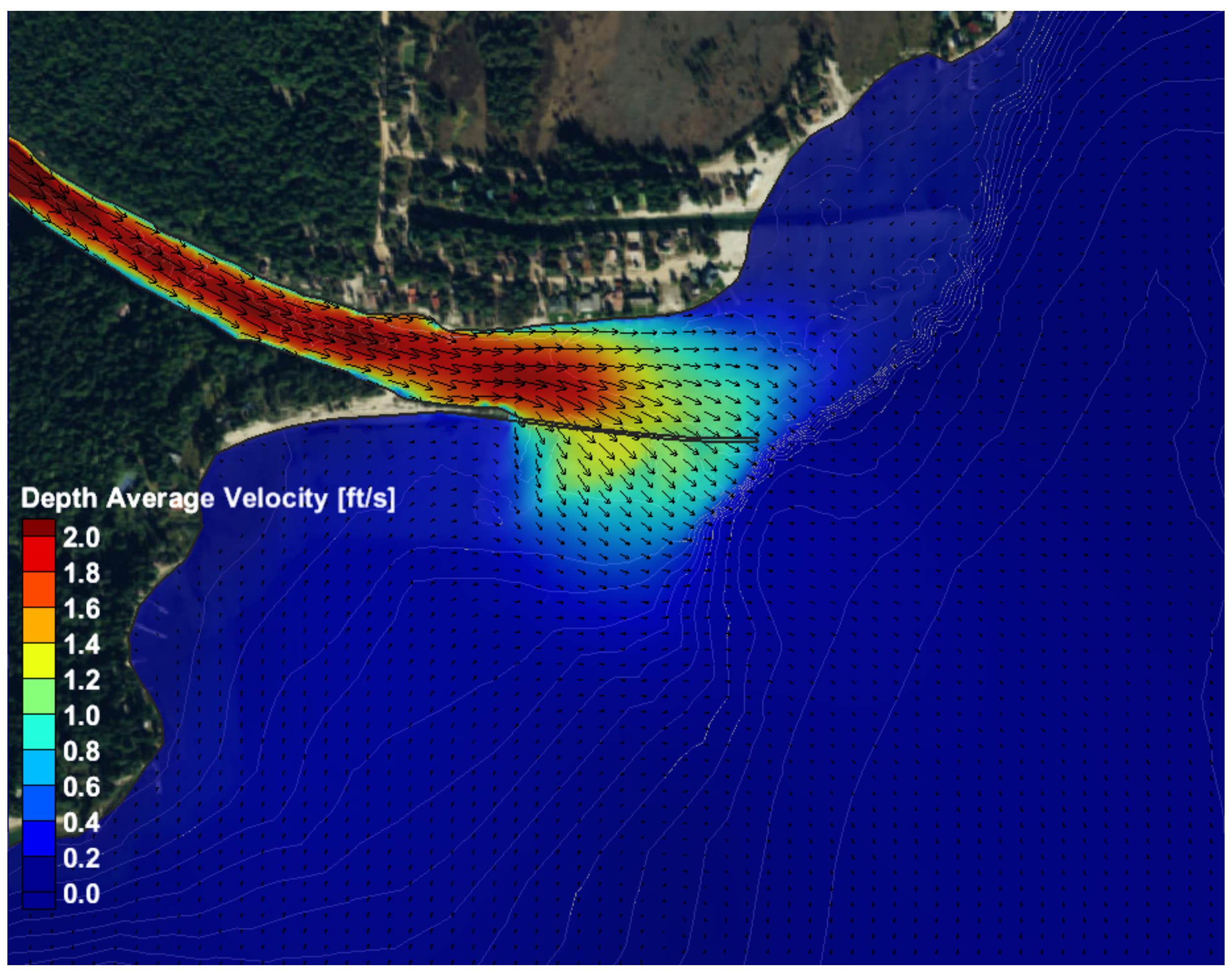
Sediment Sources

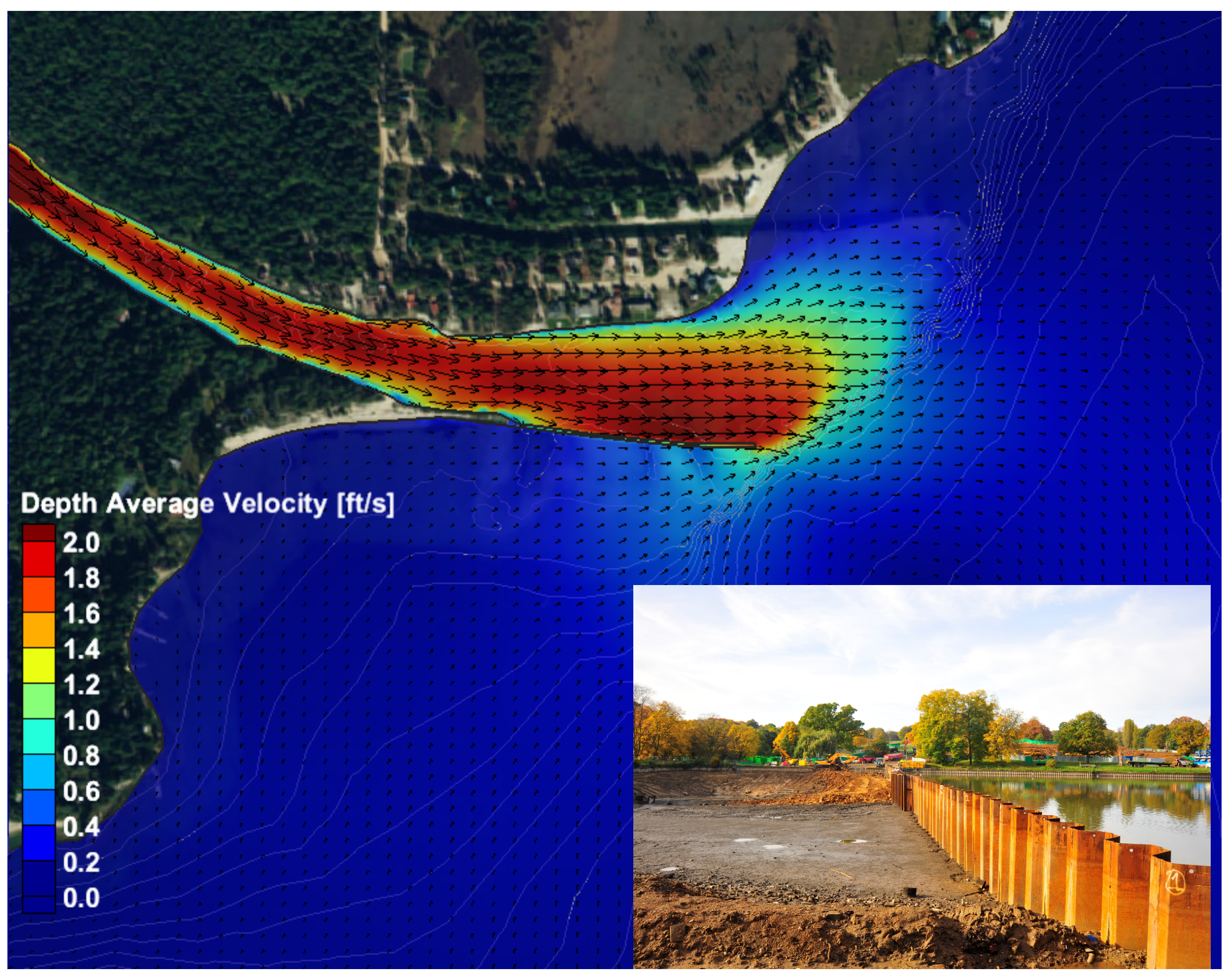


Porous Breakwater

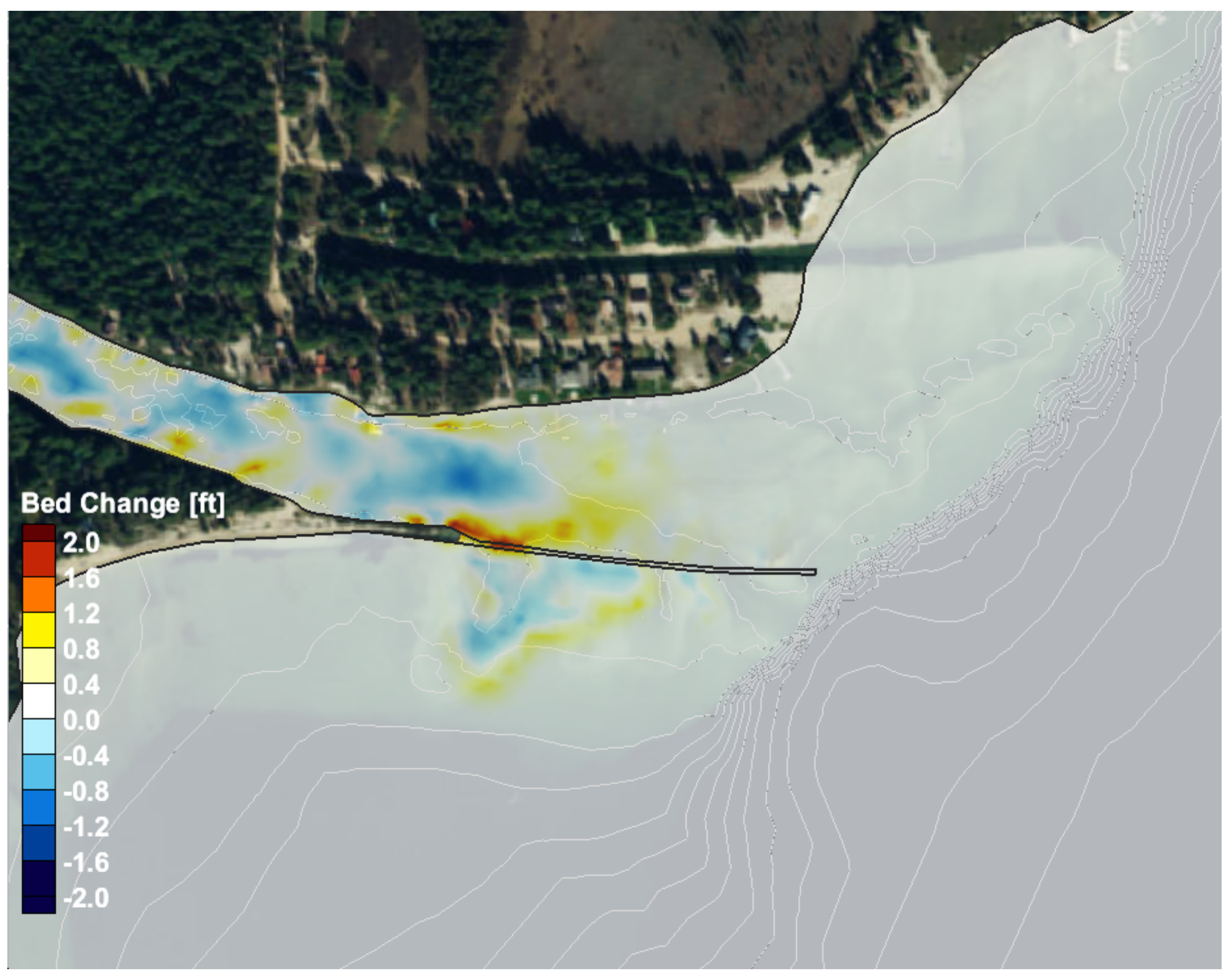
Solid Breakwater

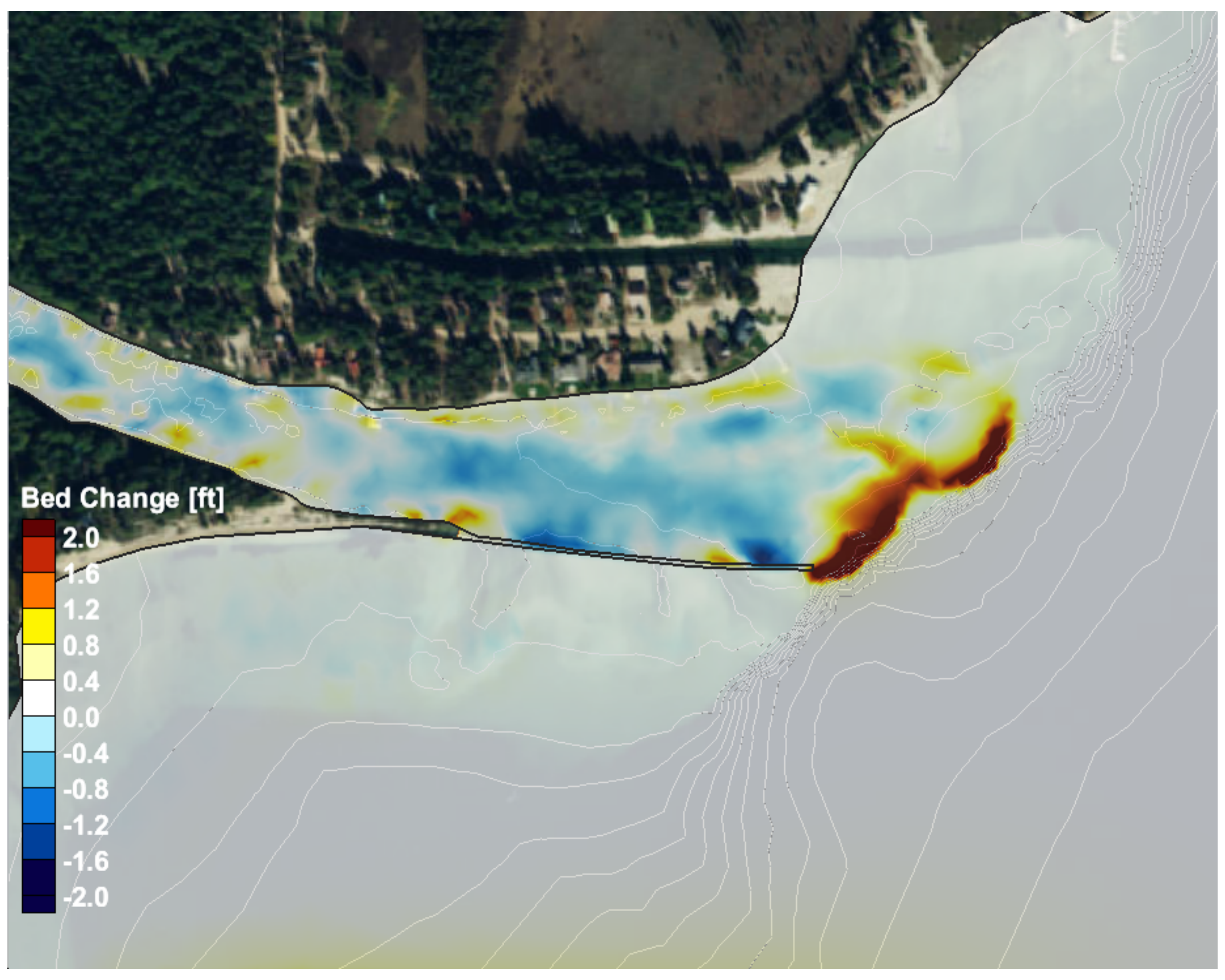
Velocity





Sedimentation

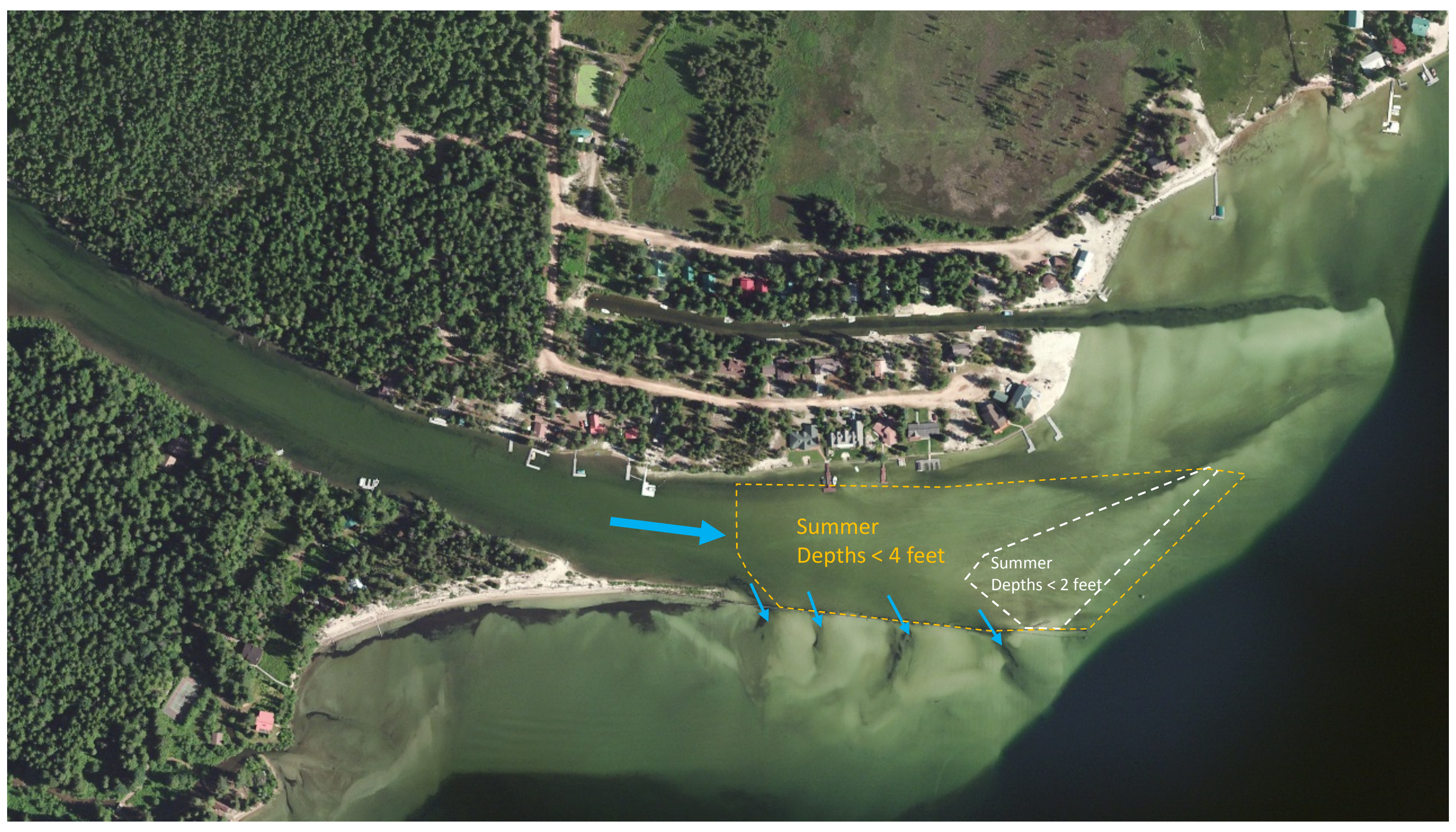




Priest Lake Water Management Study

THOROFARE IMPROVEMENT ALTERNATIVES

No Action



Remove Breakwater

Rehabilitate Existing Porous Breakwater

New Sediment Retention Feature

New In-channel Flow Diversion

New Partial In-channel Flow Diversion

Priest Lake Water Management Study

THOROFARE IMPROVEMENT CONCEPTS

Stone


Bio-engineered


Large Woody Debris

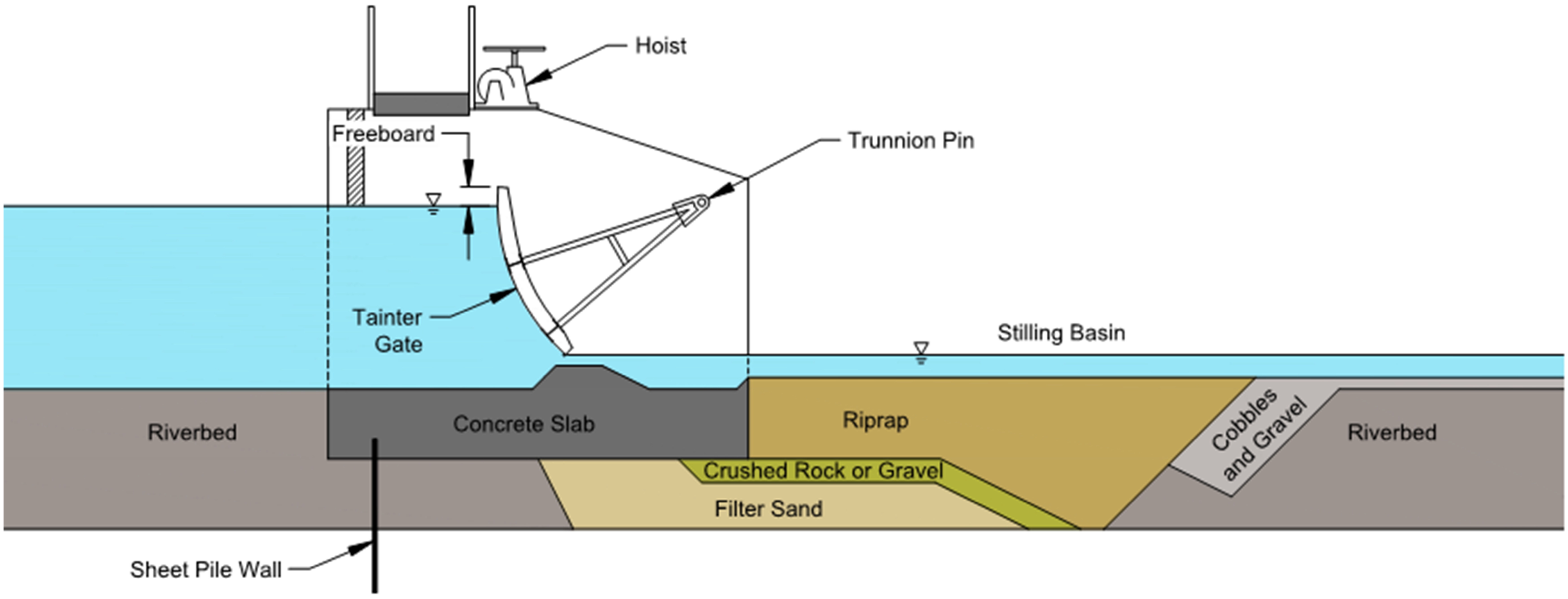
Sheet Pile Wall


Flow Diversion Features

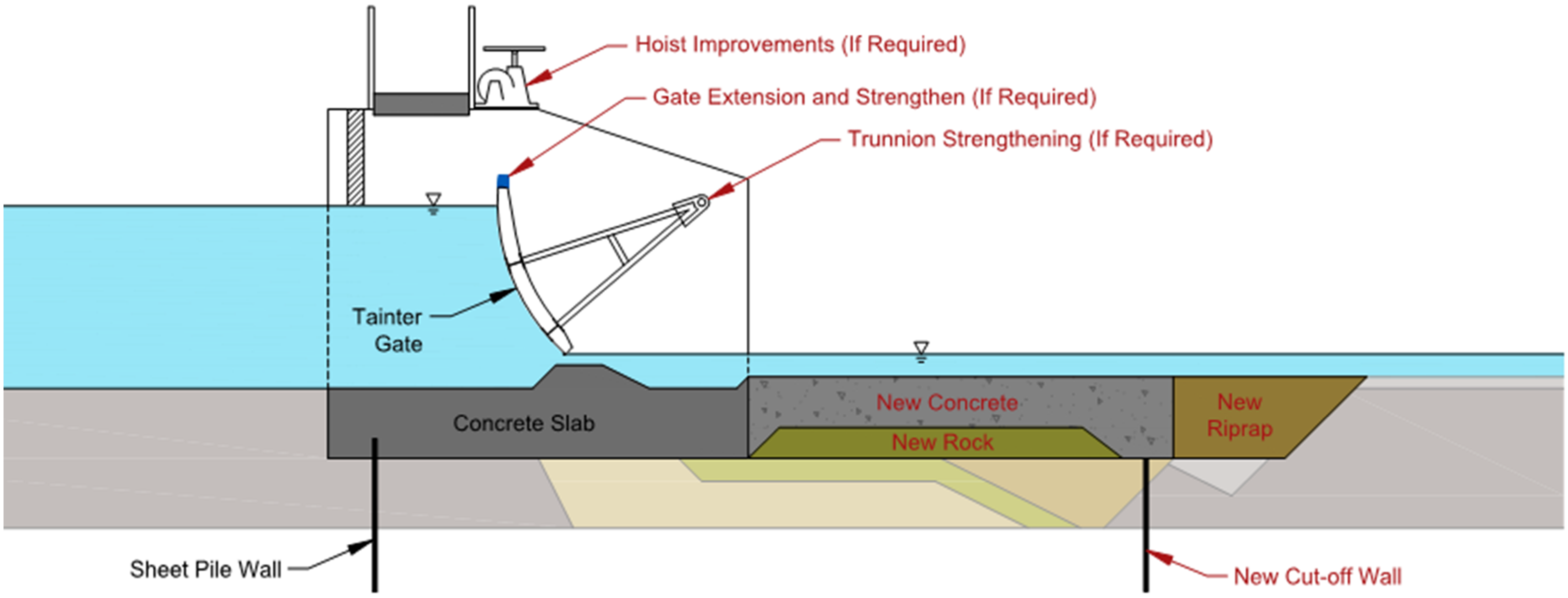

Priest Lake Water Management Study

OUTLET DAM STRUCTURE

Existing Conditions



Improvement Measures



Priest Lake Water Management Study
Open House

COMMENT FORM

Thursday, July 20 2017, from 2:00-5:00p.m.

Priest Lake Groomers & Ambulance Building

4677 Dickensheet Road
Coolin, Idaho 83821

LEAVE COMMENTS, MAIL OR E-MAIL BY AUGUST 20, 2017, TO:

Bonner County Waterways &Parks
Attn: Steve Klatt
1500 Hwy. 2- Ste 101
Sandpoint, ID 83864

Email: Neeley.Miller@idwr.idaho.gov

Thank you for attending today's open house. Your input is important.

Please share any suggestions/comments you may have. a couple values will be important to consider. • with the increased lake level There will be a cost to have the finite and coordinated control between the Lake gage and the River discharge. It would be important to know what the cost would be. • with the increased Lake level how much does a slower inundated thoroughfare loose its transport of sediment capability (competency) to increase deposition at the mouth of the thoroughfare. will this increased deposition impact navigability, or accumulate more sediment adjacent to the new breakwater. • Are there areas of lake-shore that will be more eroding at this new elevation, and will this increased sedimentation impact the effectiveness of the breakwater when longshore currents transport an increasing sediment load by longshore currents. • Has there been any quantification of the increased nutrient load to the lake and the potential impact to ambient water quality. • Lastly, an estimate of the overall cost to benefit ratio to identify How many Dollars for the overall monitary value of the benefit to the River. Eventually a population study to track the benefit to the River and Native species.

Thank you for the opportunity to pose a couple basic questions. We look forward the meeting this fall that may have some data and answers to questions.

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Open House

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1500 Hwy. 2- Ste 101
Sandpoint, ID 83864

Email: Neeley.Miller@idwr.idaho.gov

Thank you for attending today's open house. Your input is important.

Please share any suggestions/comments you may have.

We are owners of beachfront lot in Diamond Park (East side mile 12). We have permanent dock and boathouse and very small beach (10' deep & 15' wide). Increase of lake level by 3'-6' in Rec Season would render dock, boathouse and Beach, unusable! We are concerned that whatever plan is adopted, take into consideration government "taking" of our deeded property and would expect/hope for compensation so that we could afford our own mitigation. Thank you for your efforts and consideration. Bill and Sandra Dodge

Priest Lake Water Management Study
Open House

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Email: Neeley.Miller@idwr.idaho.gov

Thank you for attending today's open house. Your input is important.

Please share any suggestions/comments you may have.

I have found the dam to be a good benefit. It is my opinion that a 2-3 inch increase in lake level would be acceptable but more would cause too much erosion particularly in the Thorofare.

Dredging would be bad because it would encourage larger boats with larger wakes with more erosion.

Frank [illegible]

Priest Lake Water Management Study
Open House

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Email: Neeley.Miller@idwr.idaho.gov

Thank you for attending today's open house. Your input is important.

Please share any suggestions/comments you may have.

1) During the summer season the gates at the dam is not designed to hold ~~wat~~. the water level any higher than 3 ft level. So the gates would need to be redesign to accomplish a higher level.

2) I think you raise the water level to 6in above 3ft the beaches would suffer a lot.

Priest Lake Water Management Study
Open House

COMMENT FORM

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Email: Neeley.Miller@idwr.idaho.gov

Thank you for attending today's open house. Your input is important.

Please share any suggestions/comments you may have.

I THINK THE THOROUGHFARE SHOULD be
KEPT OPEN BY DREDGING & A NEW BREAKWATER

I'M INFAVOR OF HOLDING BACK MORE WATER
IN DRY YEARS BUT NEED TO MAKE SURE
IT DOES MINIMAL HARM People AND ~~CAUSE~~
SHORE EROSION) PERHAPS ONLY 3" ABOVE

Priest Lake Water Management Study
Open House

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1500 Hwy. 2- Ste 101
Sandpoint, ID 83864

Email: Neeley.Miller@idwr.idaho.gov

Thank you for attending today's open house. Your input is important.

Please share any suggestions/comments you may have.

(1) Managing to 3.0 ft is not practical or best for the watershed. Loosen the statute. Manage to 3.0 ± (1.0' or 2')

(2) Let thoroughfare go to natural state. Make upper lake only for non-motorized access.

(3) Maintain the river flow. That should have -<u>priority</u>- over the 3.0' statute. Reducing discharge levels will increase the concentration levels of the discharge from the sewer district etc. Plus we need to keep the fish + wildlife (and river recreation) priorities at at ~~the~~ least the same level of priority as lake shoreline. Lake shoreline changes many time over the year. The impact on the lake and lake recreation of a few inches is negligible. Impact on the river is significant. Let the lakelevel vary – <u>even during the summer</u>!

Priest Lake Water Management Study
Open House

COMMENT FORM

Thursday, July 20 2017, from 2:00-5:00p.m.

Priest Lake Groomers & Ambulance Building

4677 Dickensheet Road
Coolin, Idaho 83821

LEAVE COMMENTS, MAIL OR E-MAIL
BY AUGUST 20, 2017, TO:

Bonner County Waterways &Parks
Attn: Steve Klatt
1500 Hwy. 2- Ste 101
Sandpoint, ID 83864

Email: Neeley.Miller@idwr.idaho.gov

Thank you for attending today's open house. Your input is important.

Please share any suggestions/comments you may have.

The underlying problem is the statutory requirement to maintain the lake level at +3 ft. The politicians should change this and make the primary requirement be a minimum flow level through the outlet river. Increasing lake level to build up more reservoir capacity will hurt beach and other recreation. Lets put the people above the resort owners.

Let the thorofare return to its natural state. We need to quit trying to fight it through engineering. The upper lake will be more special without motorized vessels.

Priest Lake Water Management Study
Open House

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Please share any suggestions/comments you may have.

WE HAVE A FIXED ELEVATION DOCK. A 3" INCREASE IN LAKE LEVEL WOULD BE UP TO THE LOWER FRAME WORK AND 6" WOULD HALF SUBMERGE THE STRUCTURE. EITHER WOULD HAVE A VERY ADVERSE EFFECT ON THE DOCK.

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Please share any suggestions/comments you may have.

I live near outlet. It will be interesting to see how ~~this~~ increasing the lake level will change the river portion above the dam.

Very informative information.

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Please share any suggestions/comments you may have.

I would not object to Summer lake level
going up by 2-3" if the study suggests
this - even 6" would work for us - Sherwood Bay.
I suggest making dam Re-construction - easiest +
least expensive if necessary

Keep thoroughfare open to all "No Wake" Vessels
+ motorized - but No Wake.
Better Control of - No Water skiing or fast
Jet skiis on upper lake.

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Thank you for attending today's open house. Your input is important.

Please share any suggestions/comments you may have.

- Concerns with fixed dock structure if water levels go to high.

- We take a 26 foot boat up thorofare 3-4 x's during the summer. It is too shallow at the entrance & needs to be dredged. The thorofare also needs the logs removed.

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Thank you for attending today's open house. Your input is important.

Please share any suggestions/comments you may have.

We are for having the lake 3" higher during the
summer. Also the brakewater needs to be repaired
on the ~~f~~ thorfare. We have a 19' inboard-out board boat.
Last year we didn't go up to the upper lake because of the
low water getting into the thorefare. Thanks for
looking into all of this!

Priest Lake Water Management Study
Open House

COMMENT FORM

Friday, September 22, 2017
4:00-7:00p.m.

Priest Lake Elementary School
27732 SH,57
Priest Lake, Idaho 83856

LEAVE COMMENTS, MAIL OR E-MAIL BY OCTOBER 6, 2017, TO:

Bonner County Waterways & Parks
Attn: Steve Klatt
1500 Hwy. 2- Ste 101
Sandpoint, ID 83864

Email: Neeley.Miller@idwr.idaho.gov

Thank you for attending today's open house. Your input is important.

Please share any suggestions/comments you may have.

The Thorofare's dredging is a concern.
- Does it need to accomodate the biggest boats?
- Does it need to accomodate motorized boats?

* Your presentations were informative & expertly presented.
If I had to vote on Thorofare solution, I would prefer #5 with diversion & natural rock.
White signage is important, too many signs & buoys take away from pristine value.

Priest Lake Water Management Study
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Email: Neeley.Miller@idwr.idaho.gov

Thank you for attending today's open house. Your input is important.

Please share any suggestions/comments you may have.

Our concern was a rumor about lowering the lake 12ft. We were concerned we would lose our lake access. This presentation was very informative and we were happy to hear they are going to maintain lake level good.

We were also interested in the thorough fare break water and its replacement. Looks like access to the upper lake will be maintained.

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Email: Neeley.Miller@idwr.idaho.gov

Thank you for attending today's open house. Your input is important.

Please share any suggestions/comments you may have.

I am very concerned about raising the lake level and what that will do to the shoreline. Especially with all the very large boats and wakeboard boats that are now on this lake. I grew up @ Linger longer. We use to have sand clear to the water. Now a large Rock retaining wall has been built to help hold back the shore. The errosion from wakes has destroyed the shore and caused so many beautiful P-Pines to become danger trees due to roots - tress have been removed & the sand bar is now gone. And 3'-6" to ~~what~~ the lake level we have now - and I am concerned it is a formula for more & more errosion.

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Thank you for attending today's open house. Your input is important.

Please share any suggestions/comments you may have.

The problem seems to be sediment.
have you looked at Creating
More wet lands upstream in
Caribou & beaver creek;
Can sediment be reduced by
different Logging - or Road building
practices.

natural looking break water
improvements look good

Bob Anderson
53 N Ryan Rd
P.L.

Priest Lake Water Management Study
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Thank you for attending today's open house. Your input is important.

Please share any suggestions/comments you may have.

Thank you for offering this nice presentation. It is appreciated.

It appears a lot of thoughtful planning has taken place.

Prefer a natural looking breakwater or flow ~~traffic~~ directing structures.

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Thank you for attending today's open house. Your input is important.

Please share any suggestions/comments you may have.

Dreg & replace Breakwater
with Rocks &

Jeri Hill

leave the Lake & River
the same as
alway

Priest Lake Water Management Study
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Thank you for attending today's open house. Your input is important.

Please share any suggestions/comments you may have.

I would like to see Idaho Department of Water Resources) who owns the Dam at Priest lake sit down and determine the minimum flow out of the dam that is necessary to maintain the 19C - 22C° to meet and be in Compliance with State + Federal laws for Clean water.

I would like information from IDWR on how the number 60 CFS for was established for Priest River. (Scientific instream flow analysis is needed)

IDWR is aware that the Priest River is impaired - what is it doing to meet Clean Water Act standards on the Priest River?

Betty Gardner bettygardnerid@gmail.com

Priest Lake Water Management Study
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Thank you for attending today's open house. Your input is important.

Please share any suggestions/comments you may have.

1) A scientific instream flow analysis of P.R. is needed to ascertain what the flow should be before the lake level is manipulated.

year "dry year"
2) A mechanism for funding the drought analysis needs to be in place, science based, not political.

3) The thorofare needs to be profiled using hydro-geomorphic transects/Rosgen classification before any changes are made.

4) Is FERC responsible for dam safety reviews & permits? Fish passage should be included if modifications are made

5) Dredge volumes & fill volumes are needed for each thorofare structure alternative

6) If 3" of raise is approved, a 3" draw below 3' summer pool should also be approved in recognized drought years.

7) The lake draw down date seems to vary and has been pushed later & later — Who makes these decisions and what data are used?

8) Criteria used to make the no impact / no negative impacts needs to be made public, data provided.

9) Who will monitor potential impacts to shorelines, water quality, sedimentation rates in the changes are approved. What is the consultants liability if their modeling is wrong? E&O limits?

Cheryl Moody

cam99645@hotmail.com

644 Hagman Rd
Nordman, ID 83848

Priest Lake Water Management Study
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Thank you for attending today's open house. Your input is important.

Please share any suggestions/comments you may have.

① Don't be too concerned about "large" boats going through the Thorofare. Let people use small boats or ~~[illegible]~~, canoes and kayaks.

② Rather than raise lake level, get authorization to lower the lake to provide the water necessary for lower Priest River flow.

③ With higher lake levels, we would have much more erosion of shoreline and wetlands than we do now.

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Email: Neeley.Miller@idwr.idaho.gov

Thank you for attending today's open house. Your input is important.

Please share any suggestions/comments you may have.

Great presentation. I like the idea of improving ~~the~~ the access to the upper lake and holding back water during the summer in case its needed later in the year if the flow down stream is needed. I have been on Priest Lake in Diamond Park for 53 years and recall many high and low water events. Happy to share my knowledge of past lake level events.

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Email: Neeley.Miller@idwr.idaho.gov

Thank you for attending today's open house. Your input is important.

Please share any suggestions/comments you may have.

- Six inch increase in the water level in drought years, in my opinion would cause too much bad effects to the docks around the lake and the beaches.

Priest Lake Water Management Study
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Email: Neeley.Miller@idwr.idaho.gov

Thank you for attending today's open house. Your input is important.

Please share any suggestions/comments you may have.

This was Very helpful in understanding the True Concerns of regulating Water levels.

Thank you

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Thank you for attending today's open house. Your input is important.

Please share any suggestions/comments you may have.

Nice presentation
Well-explained and informative

We agree with 3-inch rise
We like the rock wall or bio engineered
for the thoroughfare.

All our questions were answered.

thank you very much
Peckhams - Three Waters
Outlet Bay

Priest Lake Water Management Study
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Thank you for attending today's open house. Your input is important.

Please share any suggestions/comments you may have.

Thorofare: it looks like the public will want to have input on what form of breakwater is chosen — those options with the lowest first cost (and possibly least attractive) will need to be weighed against higher first cost and possibly higher maintenance. Since Bonner County taxes will help pay for whatever choice is made, it would be helpful to have the first and ongoing costs available to guide the decisions.

Also - not in the scope but... enforcement of "no wake" speeds in the thorofare is needed.

There will be elements of the study that the public can really make meaningful comments on and those they can't — It would be helpful to point out where comments will make a difference.

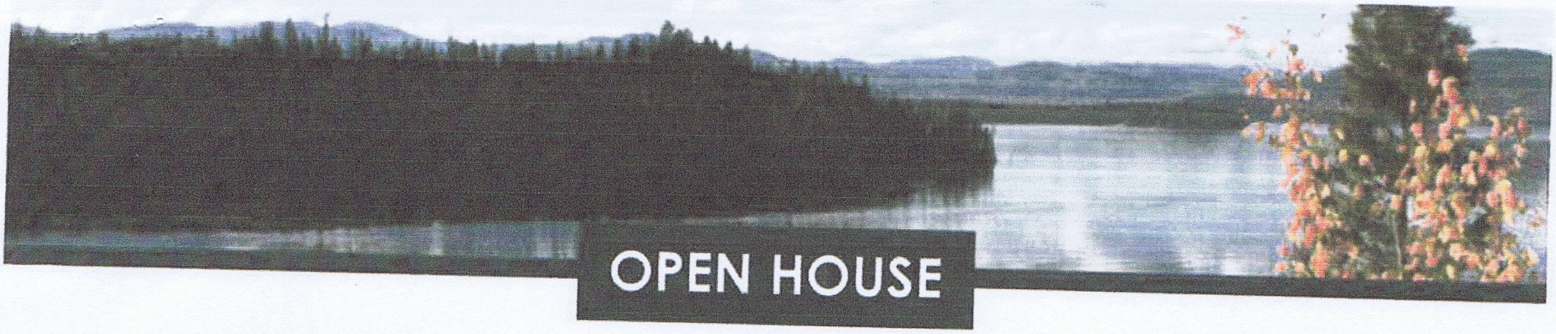
OPEN HOUSE

Thursday, July 20, 2017

2-5pm – Priest Lake Groomers & Ambulance Building

Name	PL Neighborhood	E-Mail
Dave & Cheryl Spencer	Murray Acres	Dsloan40@comcast.net
Russ Brown	Sherwood Beach	russbrown7@att.net
FRED COX	Kalispel Bay	fredcox@SISNA.com
Nick Ivie	River Valley Beacon	news@rivervalleybeacon.com
Ed Jochum	Bonner Co Sheriff	ed.jochum@bonnercounty id.gov
[illegible]	Priest River	
Steve Booth Booth	Nordman	208-443-2938
Lori Burns	Neopit	~~tom~~ burnslh@gmail.com
Bob Burns	〃	reburns@me.com
Sandra Dodge	Diamond park.	
MICHAEL KAIN	KALISPELL CREEK (BAY)	jameson1780mwk@YAHOO
Anne Hungate Ashburn	Mosquito Bay	—
Eleanor Hungate Burns	Mosquito Bay	ejjones3@gmail.com
Mick Schanilec	IDL - Cavanagh Bay	mschanilec@idl.idaho.gov
Mike Courteau	Coolin	courteaumike@yahoo.com
Janet Conlin	Cocolalla	mooseandsquirel@gmail.com

Schanilec



OPEN HOUSE

Thursday, July 20, 2017

2-5pm – Priest Lake Groomers & Ambulance Building

Name	PL Neighborhood	E-Mail
Kathy Marfil	Coolin	gkmarfil@hotmail.com
GARY MARFIL	"	" "
Carolyn Walchak	"	Freew@aol.com
Bob Mansfield	Mt View - Copper Bay	robertmansfield@ comcast.net
DALE VAN STONE	Water Resource Board	dvanstone19@hotmail.com
Karl Duncan	Dam care	priest lake Id 83856
Bob Stutz	Lamb Cr.	bobandbarb83@gmail.com
Bill & Mabel McInerney	Outlet	BILLMCPRIESTLAKE@sisna.com
Bill & Sandra Dodge	Diamond Park	billdodge@yahoo.com
Lydia Hungate	Mosquito Bay	jell60@hotmail.com
MICHAEL BOGE	518 HAGMAN Rd.	MOUNTAINFEVER1@FRONTIER.COM
ANNWEL BOGE	518 HAGMAN Rd.	
Bob Steed	Idaho DEQ 83814 2110 Ironwood Pkwy CDA	robert.steed@deq.idaho.gov.
BILL PARISH	1458 Sherwood Beach Rd Coolin ID 83821	
Thomas HERRON	915 E Front Ave Coeur d'Alene, ID	Thomas.HERRON@deq.idaho.gov
Janet and Dave Stephenson	330 Beaver Creek Lane	jdsteph@povn.com

OPEN HOUSE

Thursday, July 20, 2017

2-5pm – Priest Lake Groomers & Ambulance Building

Name	PL Neighborhood	E-Mail
[illegible]	N. End	[illegible]
Andy McLeod	S. End	amac936@yahoo.com
Roy Peckham	Outlet	peckham-rs@g.com
[illegible] Slocum	Squaw Valley	—
Stan Miller	Cougar Creek	samillerh2o@comcast.net
Sage Dixon	Samuels	Sage@Sagedixon.com
Jeff Connolly	Priest River Id	jeff.Connolly Bonner.gov
Kiira Siitari		kiira.siitari@idfg.idaho.gov

10 + Sum

OPEN HOUSE

Friday, September 22, 2017

4-7pm – Priest Lake Elementary School

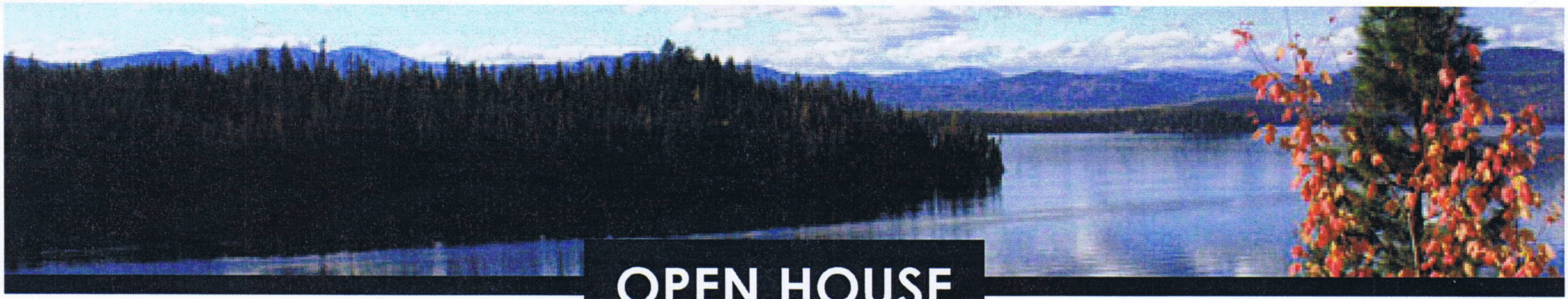
Name	PL Neighborhood	E-Mail
Douglas Jones		doug.jones@idwr.idaho.gov
Karl Duncan	outlet Bay.	41 Turtle Lane
Marge Anderson	Ruby BAy + outlet	marge @ COMMERCIALLendingNOW.c
Sandy Mansfield	Hagman Rd	sandramansfield@comcas .net
BRIAN MILLER	KALISPELL BAY	bdmiller777@hotmail.c
Mark Kiessling	Beaver Creek	markkithpokotha@yahoo
Betty Watts	Kalispell Bay	fredjwatts@yahoo .com
Roy Peckham	Three Waters OutletBay	peckham_rs@q.com
Sue Peckham	Three Waters OutletBay	
DALE VAN STONE	Water Resource Board	d van stone 19 @ Hotmail.com
JoAnn Van Stone		
Susan Castleberry	BEAVER creek	
James F. Short	BEAVER CREEK	
BOB DAVIS	REEDER BAY	bsdavis@me.com
Ken Hagman	Granite BAY	Ken@copperBayconstruc .com
Jane & Corlin		mooseandsquirrel@ gmail.co

OPEN HOUSE

Friday, September 22, 2017

4-7pm – Priest Lake Elementary School

Name	PL Neighborhood	E-Mail
Bruce Yocum	Squaw Valley	
Annet Tom Weitz	Priest Lake	
Bob Mansfield	Copper Bay	
Bob + Fran Airey	Reeder Bay Rd	franni1a@gmail.co
Bob Stutz	Priest Lake	bobandbarb83@gmai co
MaryAnn Rutherford	Beaver Creek	
JACK NYMAN	BEAVER CREEK	
Kathleen & Lowell Kuen	Grandview	
Dani/Brent Guyer	Kalispel Bay	
Bryce Glen	Hess Point	
Pat and Dee Bartleson	Elkins - Reeder Bay Rd	padee205.comcast.co
Ed & Tammy SPRENKEL	6575 W LAKESHORE Rd.	moose ylmarina@wbates .ne
Jeff Connolly	#770 - 4th PR	jkconnolly@frontier.Com
Teri Hill	4777 W Lakeshore Rd	



OPEN HOUSE

Friday, September 22, 2017

4-7pm – Priest Lake Elementary School

Name	PL Neighborhood	E-Mail
Victoria Smith	Coolin, ID	nannavic@msn.com
Donald Chappell	Steamboat Bay	208-880-3267
Walt Anderson	Luby Bay - Outlet Bay	509-220-0696
Betty Gardner	Priest River	208- 448- 2700
Fred Watts	Kalispel Bay	fredjwatts@yahoo.co
Cheryl Mundy	Mt. View	cam99645@hotmail.com
FRED COX	Kalispel Bay	fredcox@SiSNA.Co
Kiira Siitari	IDFG	kiira.siitari@idfg.idaho.gov
Erich Prahl	Diamond Park	ericalanski@gmail.com
Michael Short	Beaver Creek	jmichael.photo@gmail.co
Mick Schanilec	IDL - Priest Lake	mschanilec@idl.idaho.gov
Steve Castleberry	Thorofare	shoutberry@charter.net
THOMAS HERRON	Coeur d'Alene DEQ	Thomas.HERRON@deq.idaho.g
BILL & KAREN BELDINK	PRIEST RIVER	WCBELDINK@GMAIL.co
RON & MARY TAYLOR	CANOE POINT	rtaylor@tilster.com
Alice + Bob Anderson	53 W. Ryan Rd Priest Lake 83856	bcalpd@aol.com

WELCOME

Today's Open House is an opportunity for the public to learn more about the Priest Lake Water Management Study. Each display has information about the different study elements. Project representatives are on hand to answer any of your questions. No formal presentation is scheduled.

Please take your time and visit each display and discuss any questions or concerns you may have with staff.

We encourage you to leave written comments using the comment forms at the sign-in desk. Staff will be happy to help you with this.

There will be future opportunities for public input into the process. If you would like to receive future fliers or notices of meetings, please leave your contact information at the sign-in desk.

THANK YOU FOR COMING!

HISTORY & PURPOSE OF STUDY

BRIEF HISTORY

In 2015, drought conditions made maintaining the required summer lake level & minimum discharge from the dam very difficult. In addition, there are concerns about the breakwater structure & Thorofare access.

PURPOSE OF STUDY

Evaluate operation improvements to achieve these three goals:

- Preserving lake levels through the recreational season during dry & marginally dry years. This goal supports the local economy and meets statutory lake level requirements.

Source: Wikipedia

Source: IDWR

- Maintaining a minimum discharge of 60 cfs from the dam during recreational season.

- Providing sustainable modifications to improve Thorofare access, navigability, & water quality.

Source: Tom Weitz

Priest Lake Water Management Study

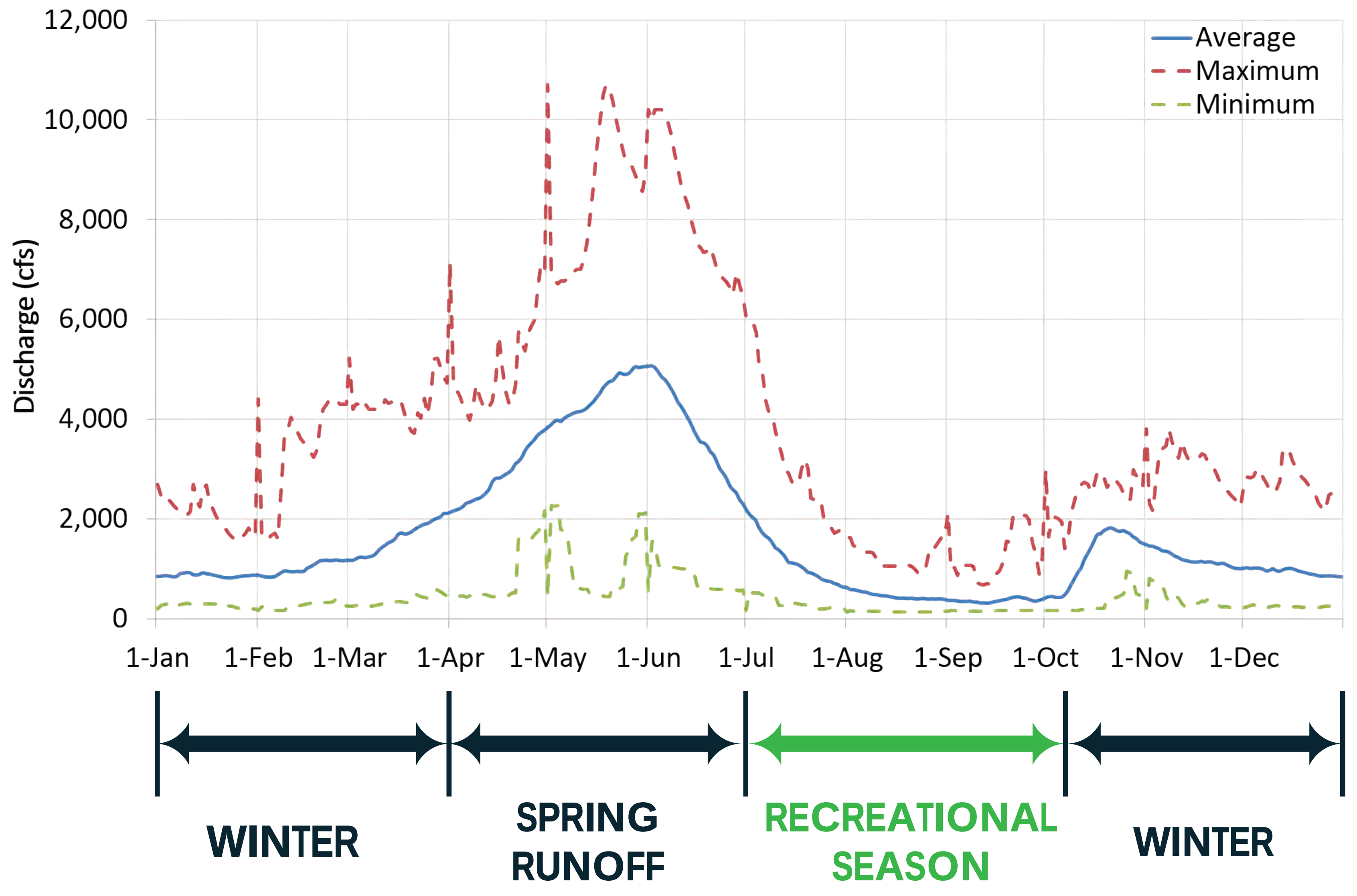
STUDY BACKGROUND

Study Goals & Objectives:

Maintain Lake Level at 3.0’ during Recreational Season.
Maintain 60 cfs Discharge at the Dam

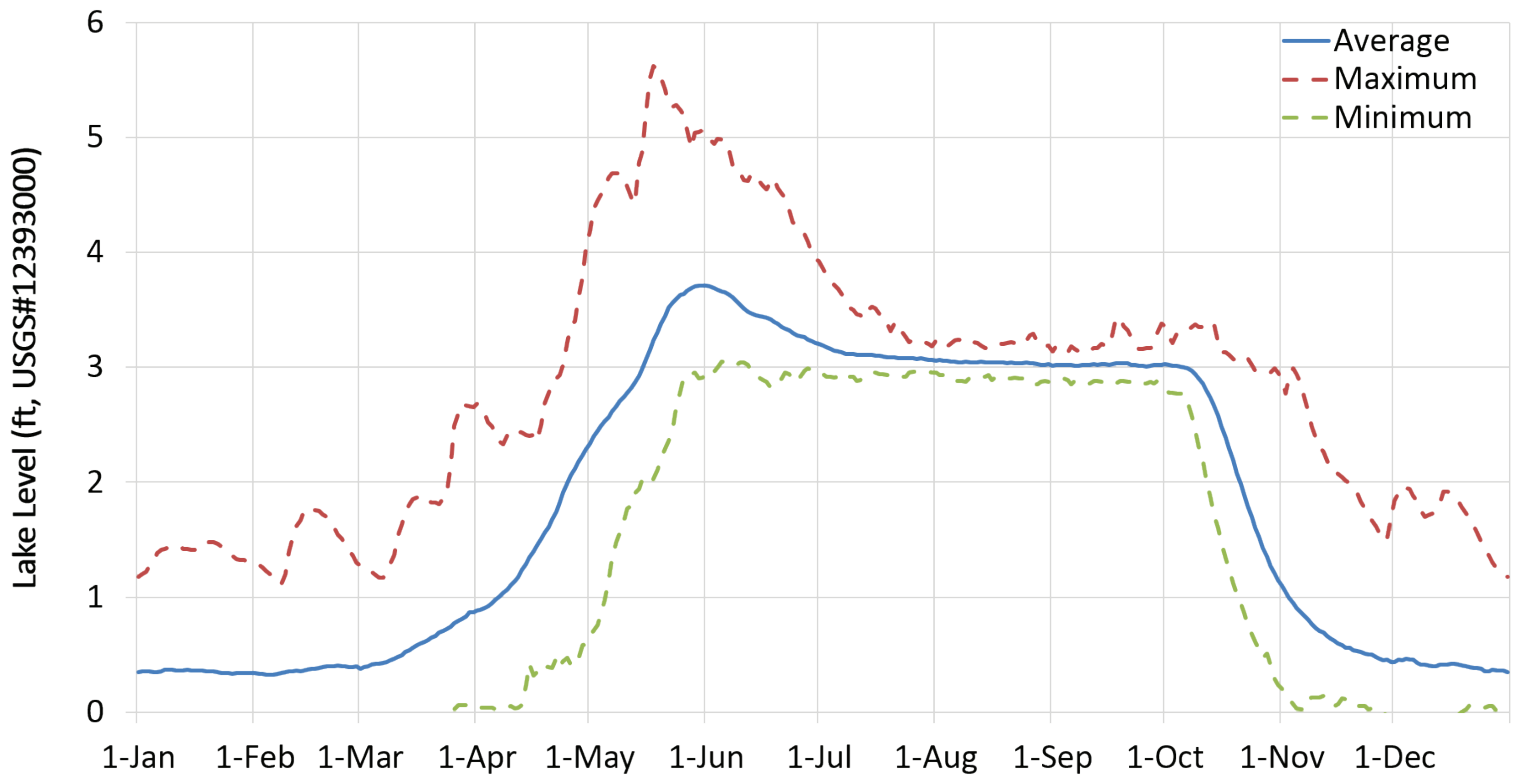
ANNUAL LAKE LEVEL VARIATION (1980 - PRESENT)

USGS Lake Level Gage

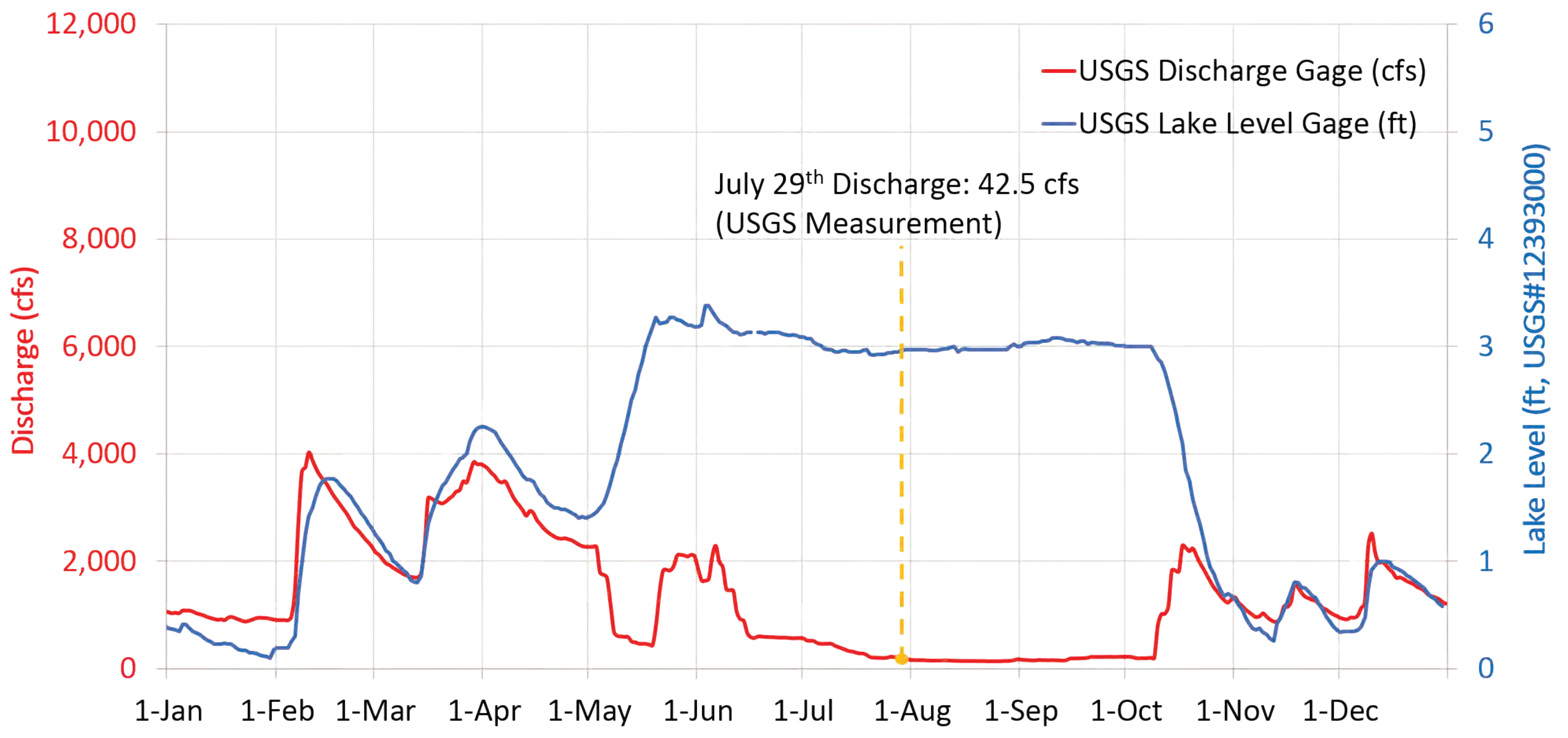


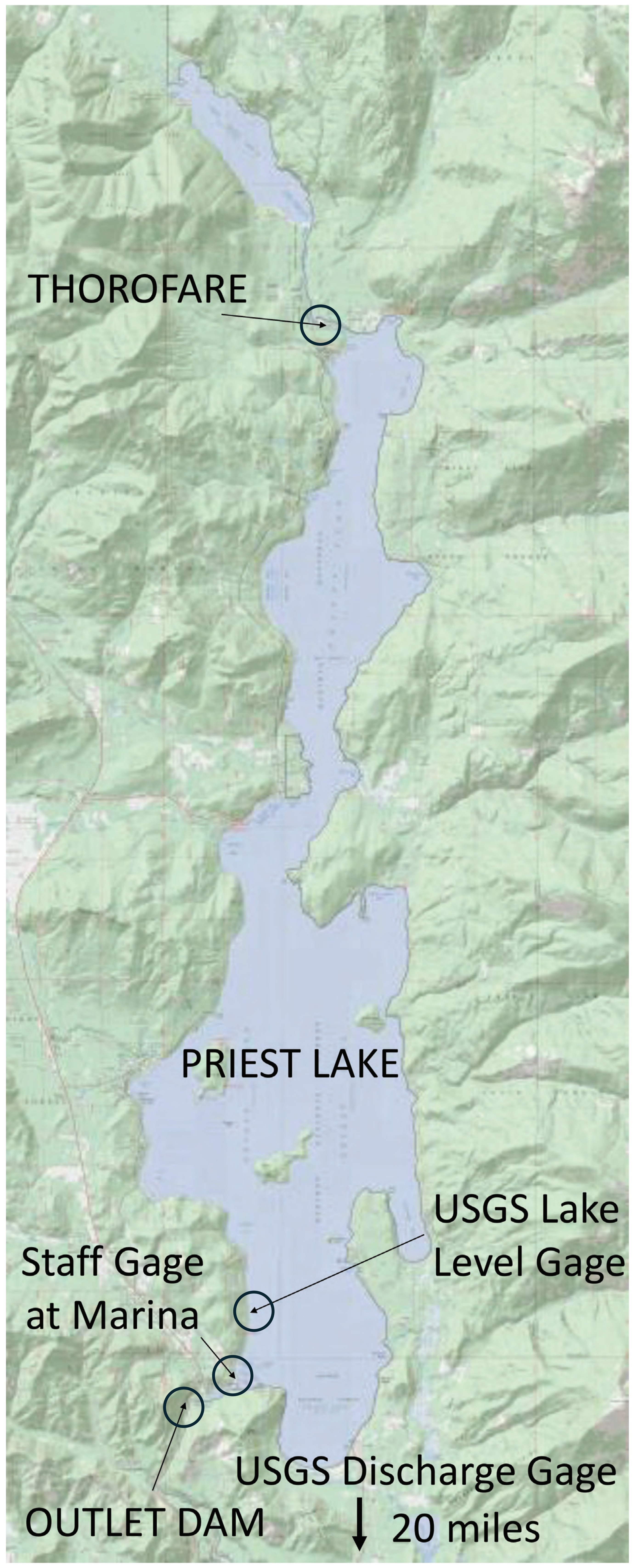
ANNUAL DISCHARGE PATTERN (1980 - PRESENT)

USGS Discharge Gage



2015 LAKE LEVEL & DISCHARGE PATTERN

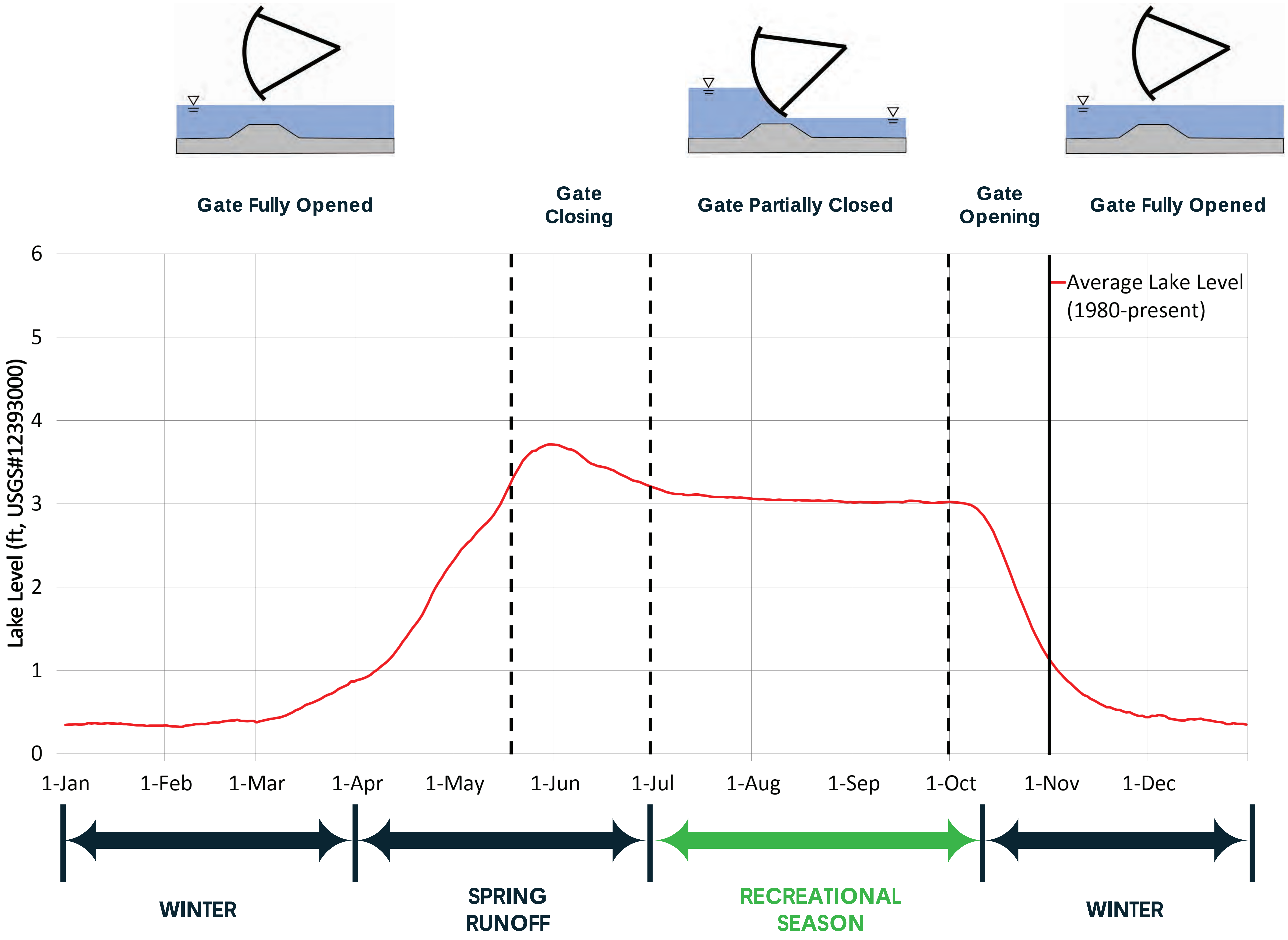




Priest Lake Water Management Study

EXISTING WATER LEVEL MANAGEMENT

Seasonal Variation of Lake Level & Outlet Dam Operation



Lake freeze can occur during winter

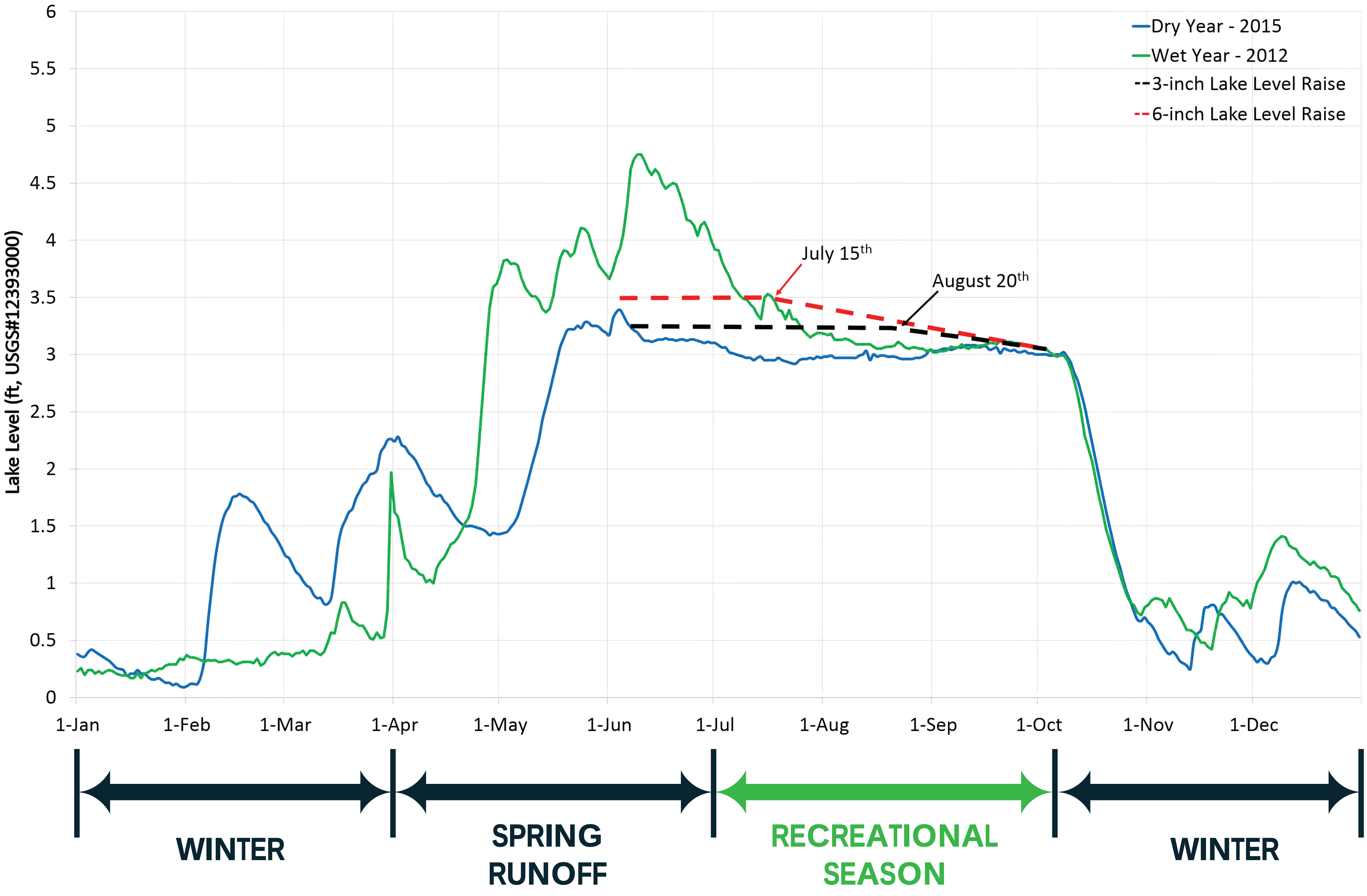
Maintaining lake level through recreational season is crucial for the community

Lake level must be drawn down by Nov 1st for Kokanee spawning

Priest Lake Water Management Study

WATER LEVEL MANAGEMENT ALTERNATIVES FOR DRY YEARS

Temporary 3-inch & 6-inch Lake Level Raise Alternatives



Priest Lake Water Management Study

ASSESSMENT OF HIGHER WATER LEVEL

SHORELINE FEATURES

Recreational Beaches

- Pier Access
- Benches
- Beaches
- Stairs
- Fire Pit

Boat Covers

- Fixed Height Structures
- Height of Vessel Adjustments

Natural Shoreline

- Natural Shoreline Features
- Rocks, Vegetation
- Large % of Overall Lake Shoreline

Public/Private Boarding Access

- Beach
- Boat Ramps

ADDITIONAL CONSIDERATIONS

Basement Flooding

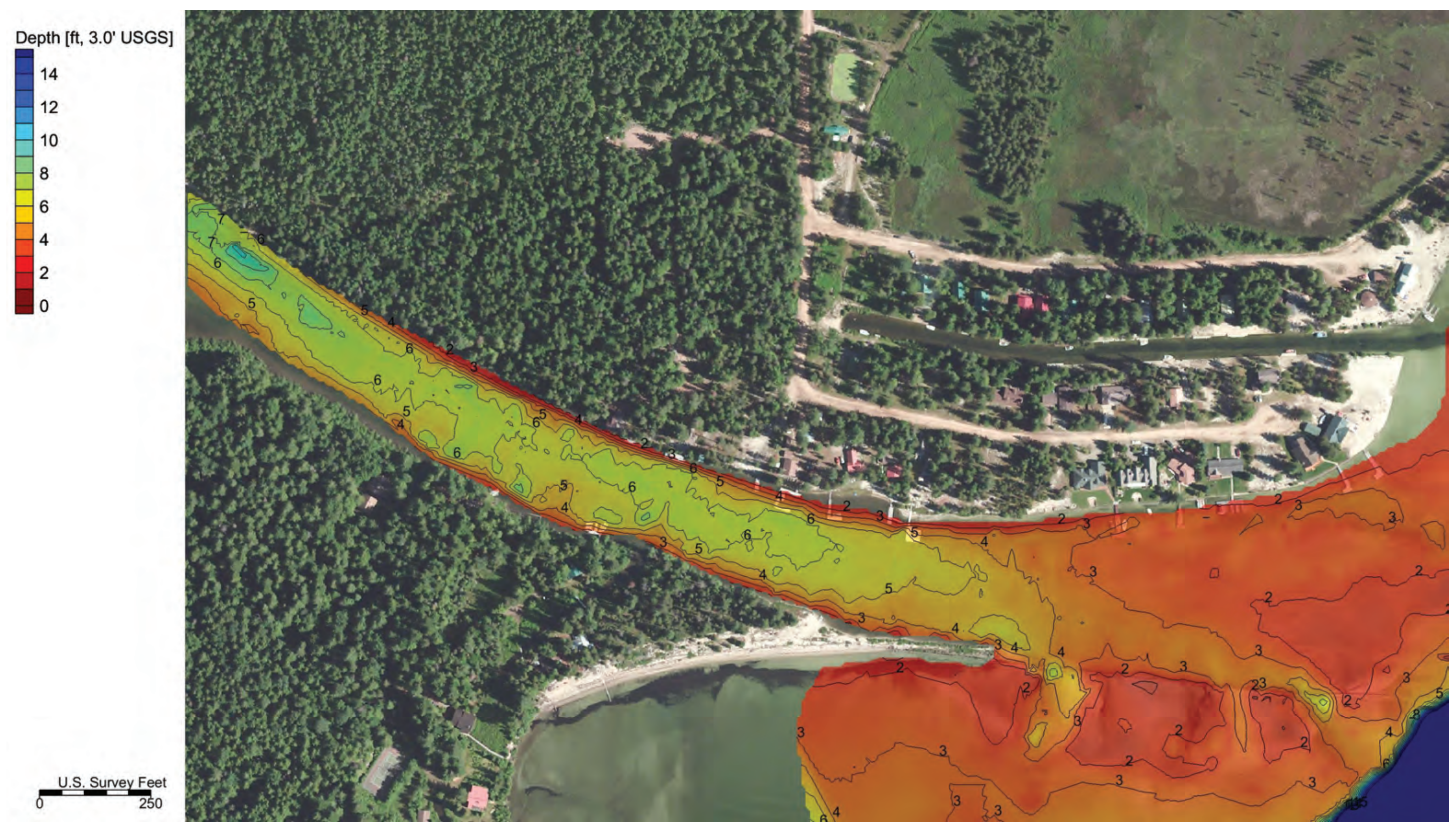
Vessel Wakes

Wetlands

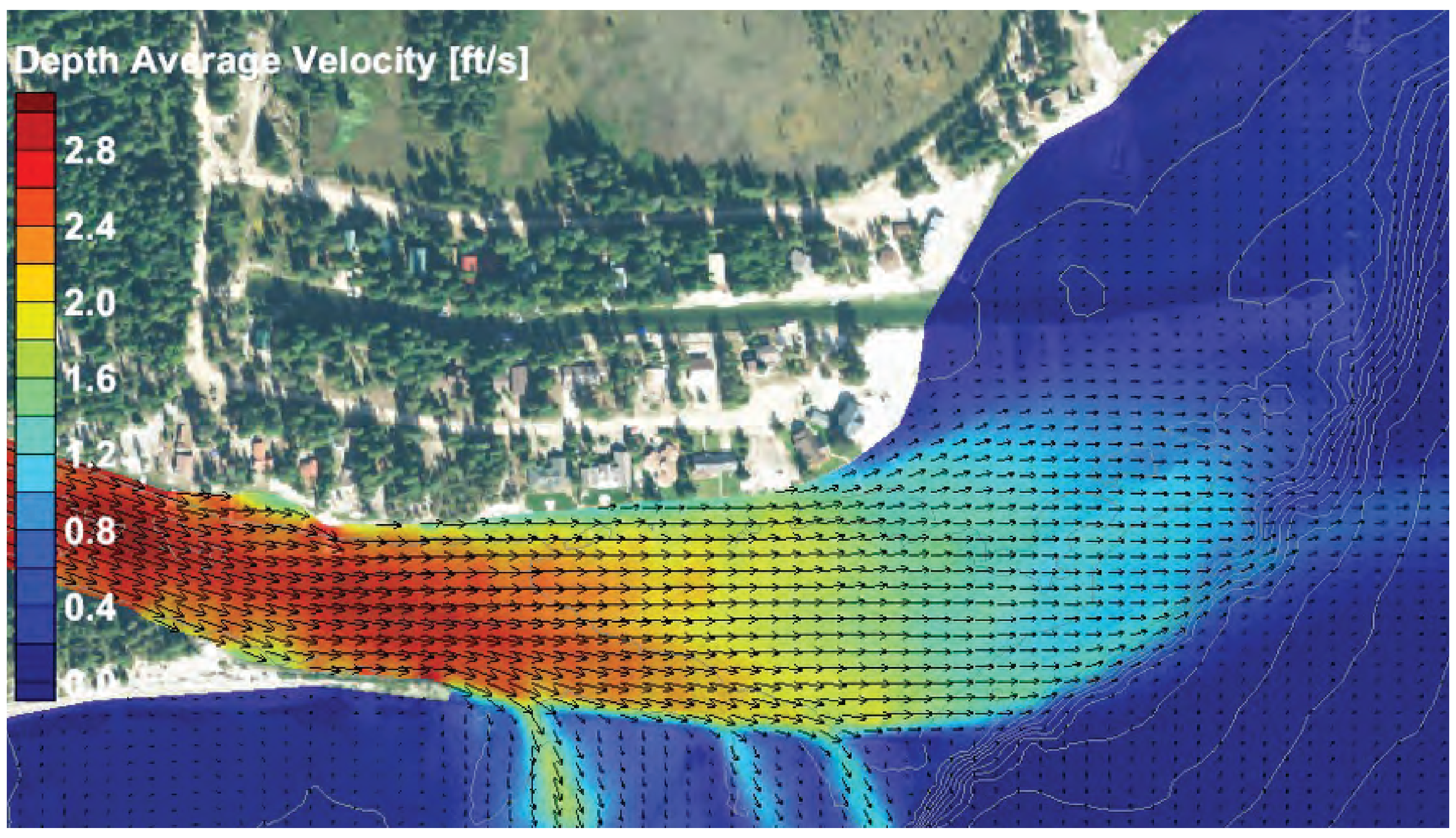
THOROFARE ACCESS & SUSTAINABILITY

Thorofare Sustainability:
Promote self-sustaining improvements to Thorofare access, navigability and water quality

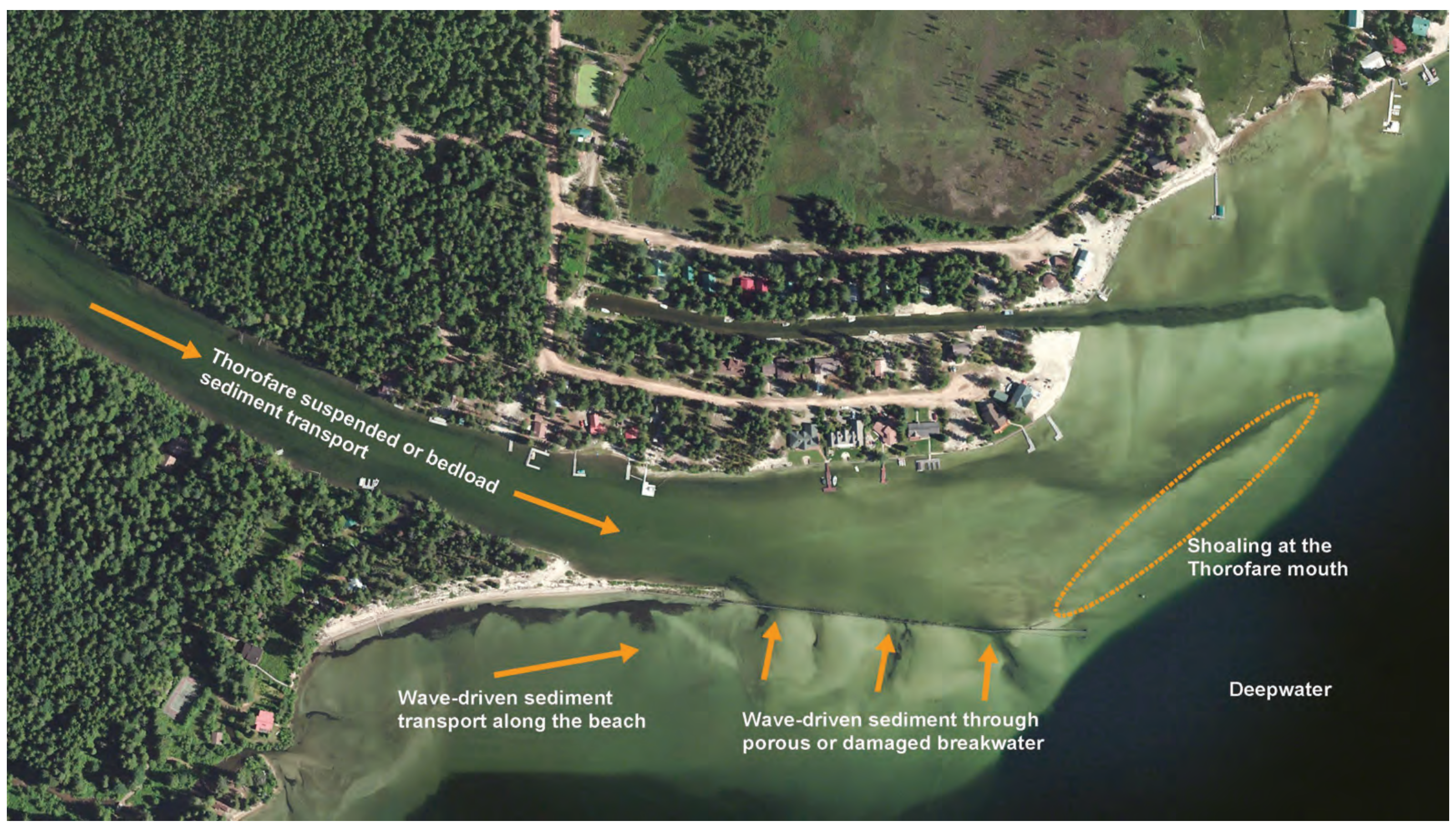
Thorofare Hydrography (2017)



Thorofare Hydrodynamics



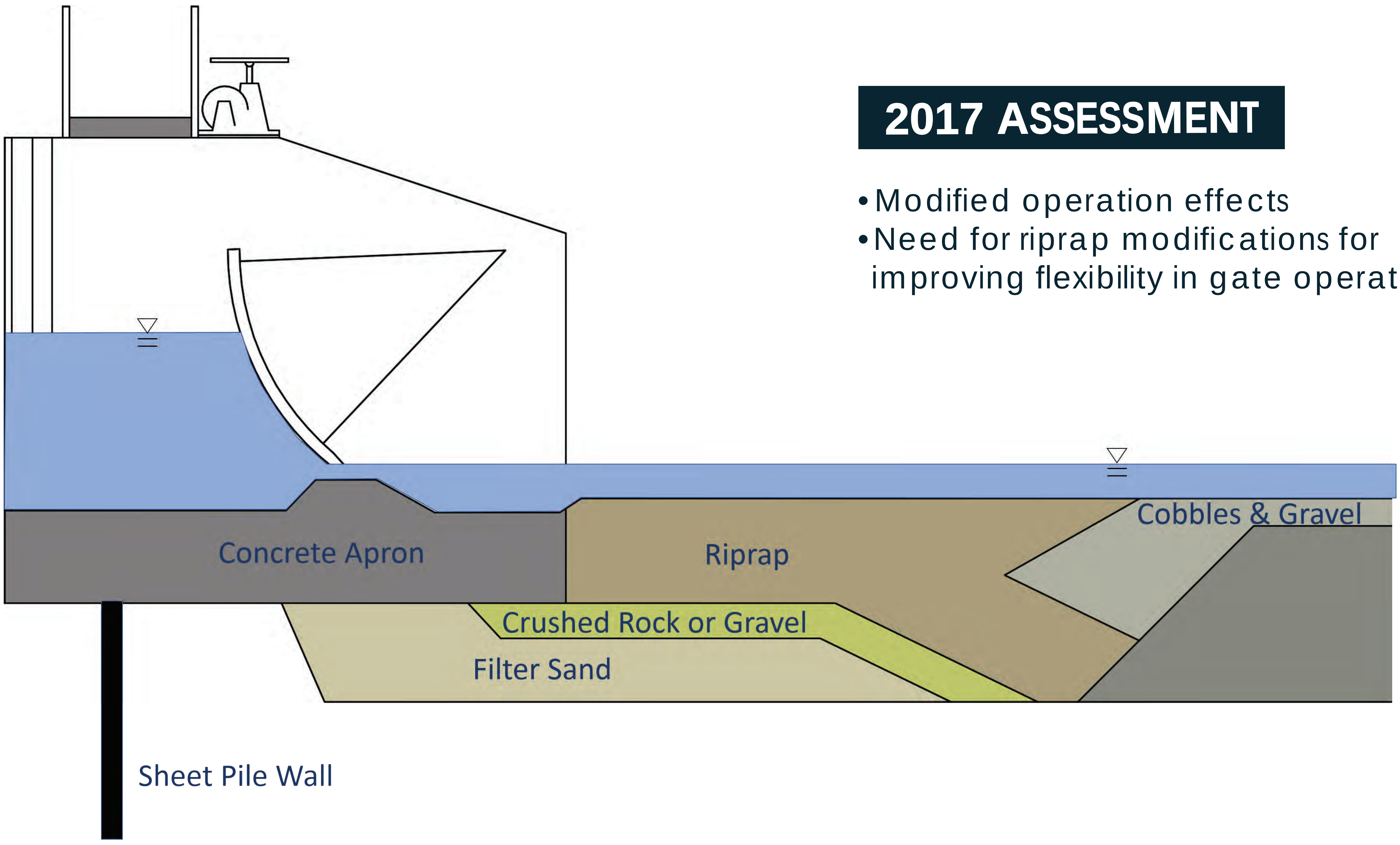
Thorofare Geomorphic Processes



Breakwater Structure

Priest Lake Water Management Study

OUTLET DAM



2017 ASSESSMENT

- Modified operation effects
- Need for riprap modifications for improving flexibility in gate operations

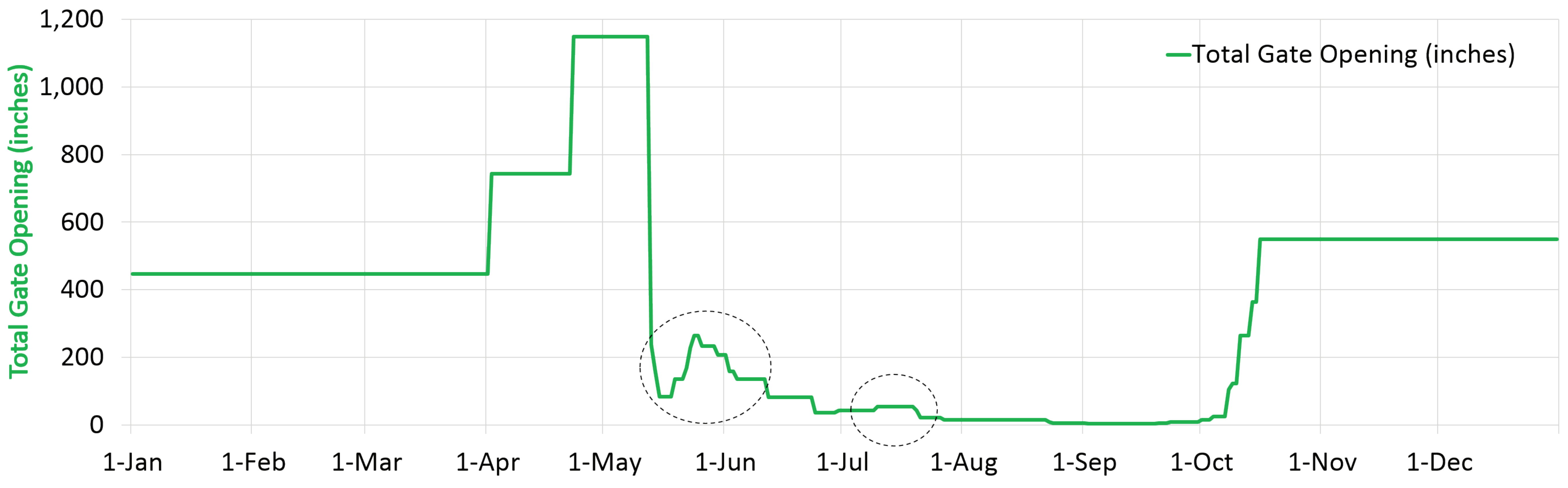
Outlet Dam and Gates
Dam operated based on marina gage readings, weather, and estimates of discharge based on gate openings

Dam Operation
- Manual operation
- Seasonal adjustments

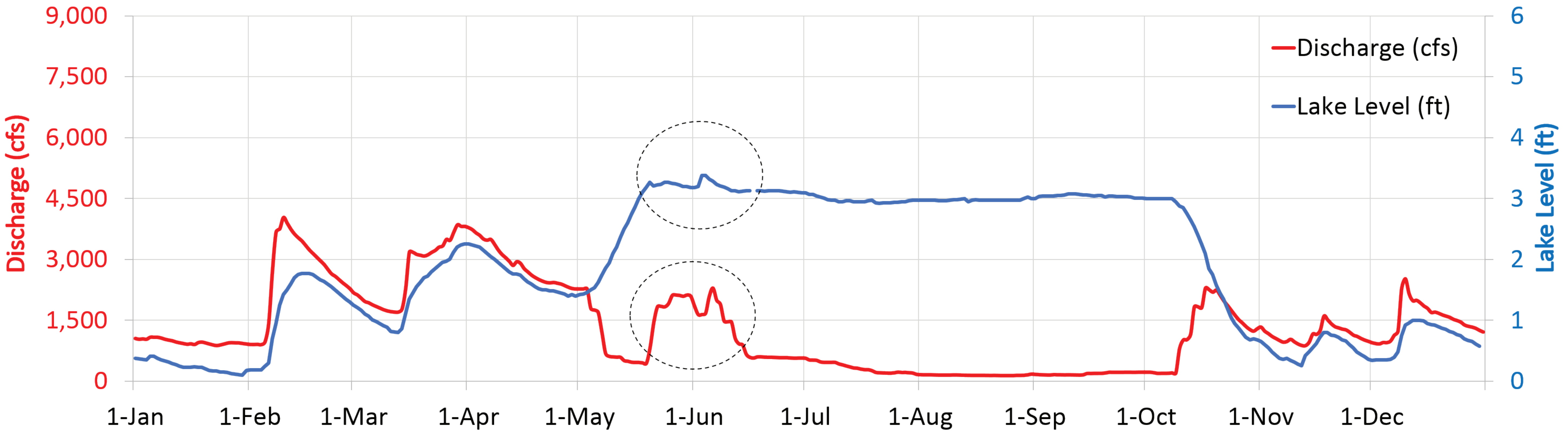
OUTLET DAM OPERATION

2015 DRY YEAR

Dam Operation

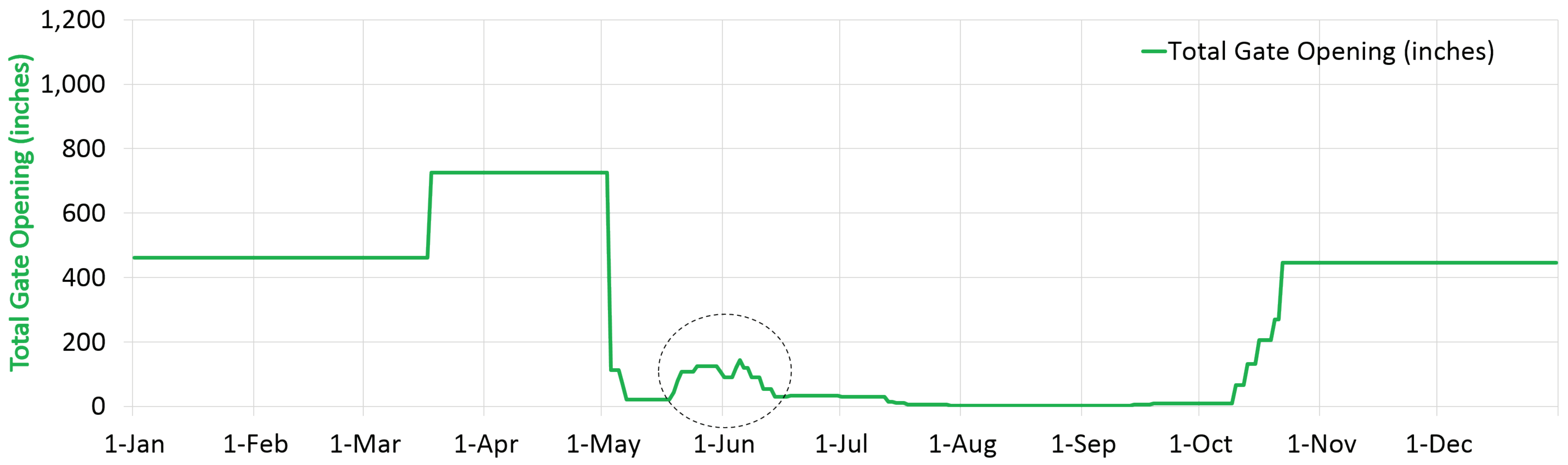


Lake Level & Discharge

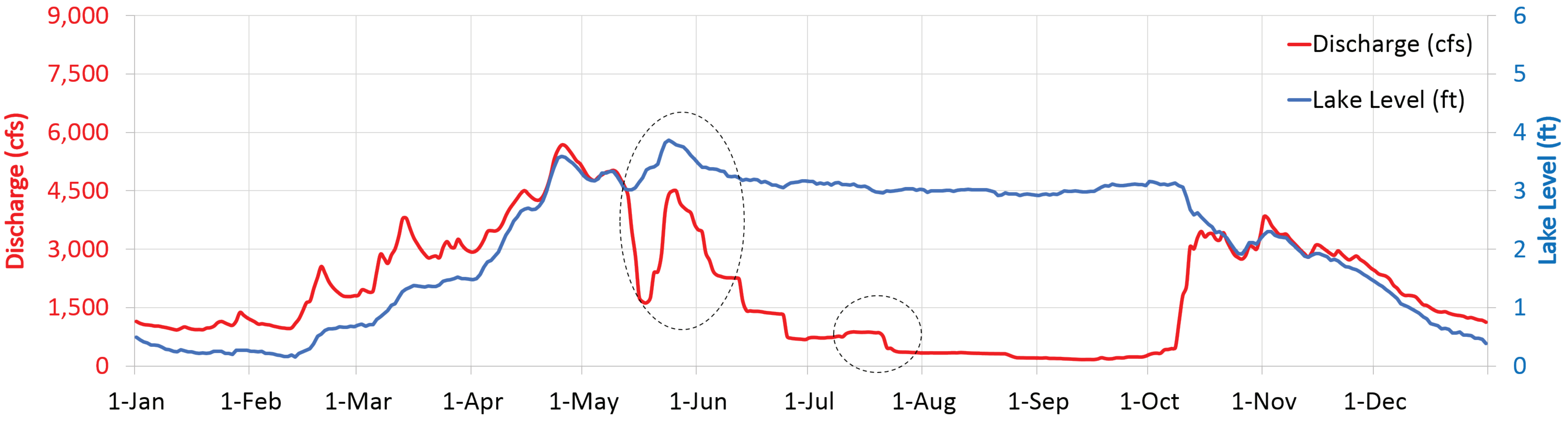


2016 MARGINALLY DRY YEAR

Dam Operation



Lake Level & Discharge



B. Pool Raise Assessment

B.1 Pool Raise Scenarios Memorandum

B.2 Pool Raise Assessment Summary

B.3 Water Level Management Hydrologic Analysis

B.1 Pool Raise Scenarios Memorandum

MOTT MACDONALD

Memorandum

Subject: Priest Lake Water Management Project, Water Management Operational Alternatives Discussion

To: IDWR & Bonner County

From: Mott MacDonald & GeoEngineers

Date: June 15, 2017

Table 1 was assembled as a starting point for the analysis of potential water management operational alternatives to be considered for meeting the criteria and objectives of the study (as outlined in the Basis of Analysis Memorandum).

Table 1. Priest Lake Water Management for Existing Operations (Exist) and Alternatives (Alt) for Dry-Year Types and a Normal Type Year Check

Alternative	Water-year Type	Recreation Lake Level			Priest River Discharge (Q)			Inflow Calendar Year	Commentary
		Stage (ft)	Start Date	End Date	Q (cfs)	Start Date	End Date		
Exist-D1	Dry	3.0	July 1	Oct 8	60 (min)	July 1	Oct 8	2001	Base, run 1st to understand time periods for lack of water to better refine alternatives for timing of increased water level and drawdowns of the additional storage.
Exist-D2	Dry	3.0	July 1	Oct 8	60 (min)	July 1	Oct 8	2005	Base
Exist-N	Normal	3.0	July 1	Oct 8	60 (min)	July 1	Oct 8	2002	Base
Alt 1	Dry	3.5	July 1	Aug 30	60 (min)	July 1	Oct 8	2001	Basic Low Flow
		3.5 to 3.0	Sept 1	Oct 8	2,000 (max)	Oct 9	Oct 31		
Alt 2	Dry	3.5	July 1	Aug 30	60 (min)	July 1	Oct 8	2005	Basic Low Flow
		3.5 to 3.0	Sept 1	Oct 8	2,000 (max)	Oct 9	Oct 31		
Alt 2N	Normal	3.5	July 1	Aug 30	60 (min)	July 1	Oct 8	2002	Run normal year after completing dry condition alternatives; so water level range could be different than that described.
		3.5 to 3.0	Sept 1	Oct 8	2,000 (max)	Oct 9	Oct 31		

Alternative	Water-year Type	Recreation Lake Level			Priest River Discharge (Q)			Inflow Calendar Year	Commentary
		Stage (ft)	Start Date	End Date	Q (cfs)	Start Date	End Date		
Alt 3	Dry	3.25	July 1	Aug 30	60 (min)	July 1	Sept 30	2001	Lower water level in Sept.
		3.25 to 2.75	Sept 1	Sept 30	2,000 (max)	Oct 1	Oct 31		
Alt 4	Dry	3.5	July 1	Aug 15	60 (min)	July 1	Sept 30	2001	Variation in timing of water levels.
		3.5 to 3.0	Aug 16	Sep 15	2,000 (max)	Oct 1	Oct 31		
		3 to 2.75	Sept 16	Oct 8					
Alt 5	Dry	3.25	July 1	Aug 30	60 (min)	July 1	Oct 8	2001	Variation in timing of water levels.
		3.25 to 3.0	Sept 1	Oct 8	2,000 (max)	Oct 9	Oct 31		

Footnote:

1. TBD = To be determined

Footnotes on Scenarios, Assumptions & Criteria for Water Management Alternatives:

- Additional detail on data and approach for conducting the analysis is outlined in the attached June 15, 2017 GeoEngineers Approach to Water Management Modeling Memorandum.
- In accordance with IDWR approved criteria, the water management analysis is focused on dry and drought year lake system water management operational changes. Therefore, wet year simulations will not be conducted as those operations will remain unchanged from historical conditions. No modeling of wet water-year types included.
- Flows (Q) are minimums and maximums for the stated time periods.
- August 15 date to be adjusted based on refinement of low water availability and time period for storage required to meet the minimum 60 cfs stream flow.
- Maximum discharge in October of 2,000 cfs (no more than 1,200 cfs increase in 24-hour period) assumed to be the limited flow condition in October based on review of historical operations. Discharge curve to mimic a prior year discharge and gate operation pattern (such as 1979, per IDWR documentation).
- Water Right 97-07380 as measured at the confluence with the East River is converted to outlet dam discharge values in accordance with information provided by IDWR (Personnel communication, Matt Anders, June 2017). Those conversion values are as follows:
    - 700 cfs at East River = 550 cfs at outlet dam
    - 300 cfs at East River = 250 cfs at outlet dam
- Starting time for gate operation to achieve revised higher pool level by July 1 (or summer recreational period) will be evaluated. It is assumed that gate operations can be modified to meet any new operational needs and requirements. As part of the outlet dam assessment work, an evaluation of the gate operations will be conducted.
- Drought and dry years of 2001 and 2005 were selected due to their full data record and being representative of current dam operations. Other dry/drought years (such as 1977 and 2015) will be backchecked once the analysis is completed for 2001 and 2005.
- Priest Lake inflows calculated by IDWR will be used for the calendar years identified in Table 1.

GEOENGINEERS

523 East Second Avenue, Spokane, WA 99202, Telephone: 509.209.2821 www.geoengineers.com

Memorandum

To: Younes Nouri and Shane Phillips (Mott MacDonald)

From: Tim Hanrahan

Date: 15 June 2017

File: 22593-001-00, Priest Lake Water Management Study

Subject: Approach to Water Management Modeling

I am writing this memorandum in response to the discussion between Younes and myself today concerning:

1. Our approach to water management modeling
2. Identifying any outstanding data issues that need to be addressed before the modeling can proceed further

The modeling approach is summarized in the following narrative. There remains one outstanding issue that needs to be finalized by the project team and IWRB/IDWR. This issue concerns the Priest Lake outlet dam stage-discharge curve to be used for modeling. A description of this issue is described in the following section "Outlet Dam Physical Data."

Approach to Water Management Modeling

Water management alternatives for Priest Lake will be modeled with the reservoir simulation model HEC-ResSim (USACE, 2013). The software is commonly used as a decision support tool for evaluating reservoir operations for a variety of operational goals and constraints. For the Priest Lake system, HEC-ResSim will be used to model the operational goal of a prescribed lake level (e.g., recreation season level of 3.0 feet from July 1 to October 8) under corresponding constraints of prescribed discharges at the Priest Lake outlet dam, and total inflow to Priest Lake. Each water management alternative to be modeled will include a unique combination of, 1) lake level goal during the calendar year, 2) discharge constraints during the calendar year, and 3) total calculated inflow to Priest Lake during the calendar year. Each alternative will be modeled at a daily time step over the duration of selected calendar years.

Completion of the water management simulations in HEC-ResSim requires data for the physical characteristics of Priest Lake, the physical characteristics of the Priest Lake outlet dam, and the inflow hydrology to Priest Lake.

Priest Lake Physical Data

The relationship between lake level and lake volume (level-volume rating curve) is required for the simulation modeling. These data are available in the form of a rating table for USGS gage 12393000, which was provided by IDWR (Matt Anders, personal communication) in the spreadsheet "12393000_2015.xlsx." This spreadsheet lists the gage height (feet) and corresponding lake volume capacity (acre-feet) for gage heights ranging from 0.0 feet to 6.69 feet. The elevation datum of the gage is 2,434.64 feet (NGVD29), and this elevation will be used in the modeling to simulate lake levels relative to the geodetic datum.

The lake-volume rating curve was extrapolated at the upper and lower bounds. USGS water year summary reports for gage 12393000 indicate a minimum lake level of -0.46 feet occurred in 1977 and 2001, with a corresponding lake volume of 37,500 acre-feet. Accordingly, the rating curve was extended to -0.46 feet by using the linear fit between lake level and lake volume. Similarly, the rating curve was extended to 7.5 feet by using a linear fit equation. Extending the data to 7.5 feet of lake level was done to use these data with the available stage-discharge rating curve for the outlet dam.

Outlet Dam Physical Data

The relationship between lake level and Priest Lake discharge (stage-discharge rating curve) is required for the simulation modeling. The lake level must be in the same vertical datum as USGS gage 12393000. These data are available in the form of a rating curve developed by IDWR (1977, Figure 1). Data points from this curve were entered into a spreadsheet and used to develop a polynomial fit between lake stage and discharge. The upper and lower bounds of this rating curve were extended by using the polynomial fit. Extending the rating curve was done to use these data with the available level-volume rating curve for Priest Lake.

A second stage-discharge rating curve is also available from IDWR (date unknown, Figure 2). Data points from this curve were entered into a spreadsheet and used to develop a polynomial fit between lake stage and discharge. Neither the date nor source of this curve is known, and thus the provenance of the data described by the curve are unknown.

The stage-discharge curves of Figure 1 and Figure 2 differ slightly (Figure 3). However, small differences in lake level can be manifested into large differences in lake volume and discharge. For example, in the Priest Lake system, a one-day 0.1 feet of lake level change corresponds to approximately 2,360 acre-feet of lake volume or 1,190 cubic feet per second (cfs) discharge.

The stage-discharge curve to be used for modeling remains to be finalized by the project team and IWRB/IDWR.

Priest Lake Inflow Hydrology

Total daily inflow to Priest Lake is required for the simulation modeling. These data are available from IDWR (Matt Anders, personal communication) in the spreadsheet “Calc Inflow.xlsx.” Because the tributary streams to Priest Lake are ungauged, the total daily inflow was calculated from a water balance based on measured change in lake volume, discharge from Priest Lake into Priest River measured at USGS gage 1234000, and calculated lake evaporation.

The calculated daily inflow is available for the selected calendar years to be modeled (2001, 2002, 2005).

References

Idaho Department of Water Resources (IDWR). 1977. Priest Lake Outlet, Phase 1, Construction of Sheet Pile Cutoff Wall. Plans, Specifications, and Contract Documents. Section 5, Plans and Drawings. Idaho Department of Water Resources, Boise, Idaho.

U.S. Army Corps of Engineers (USACE). 2013. HEC-ResSim Reservoir System Simulation User’s Manual, version 3.1. USACE Institute for Water Resources, Hydrologic Engineering Center, Davis, California.

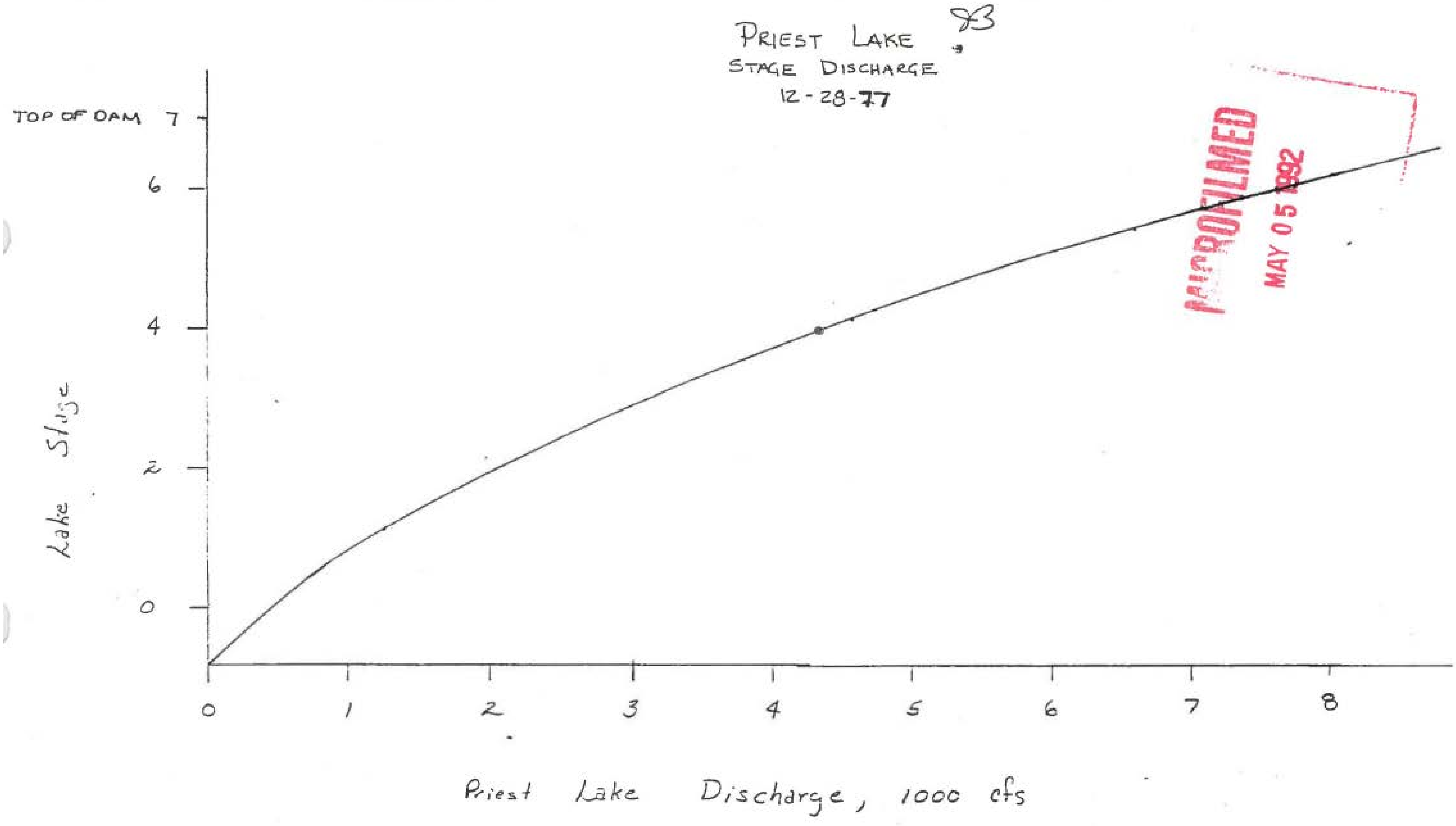


Figure 1. Priest Lake outlet stage-discharge curve (IDWR, 1977).

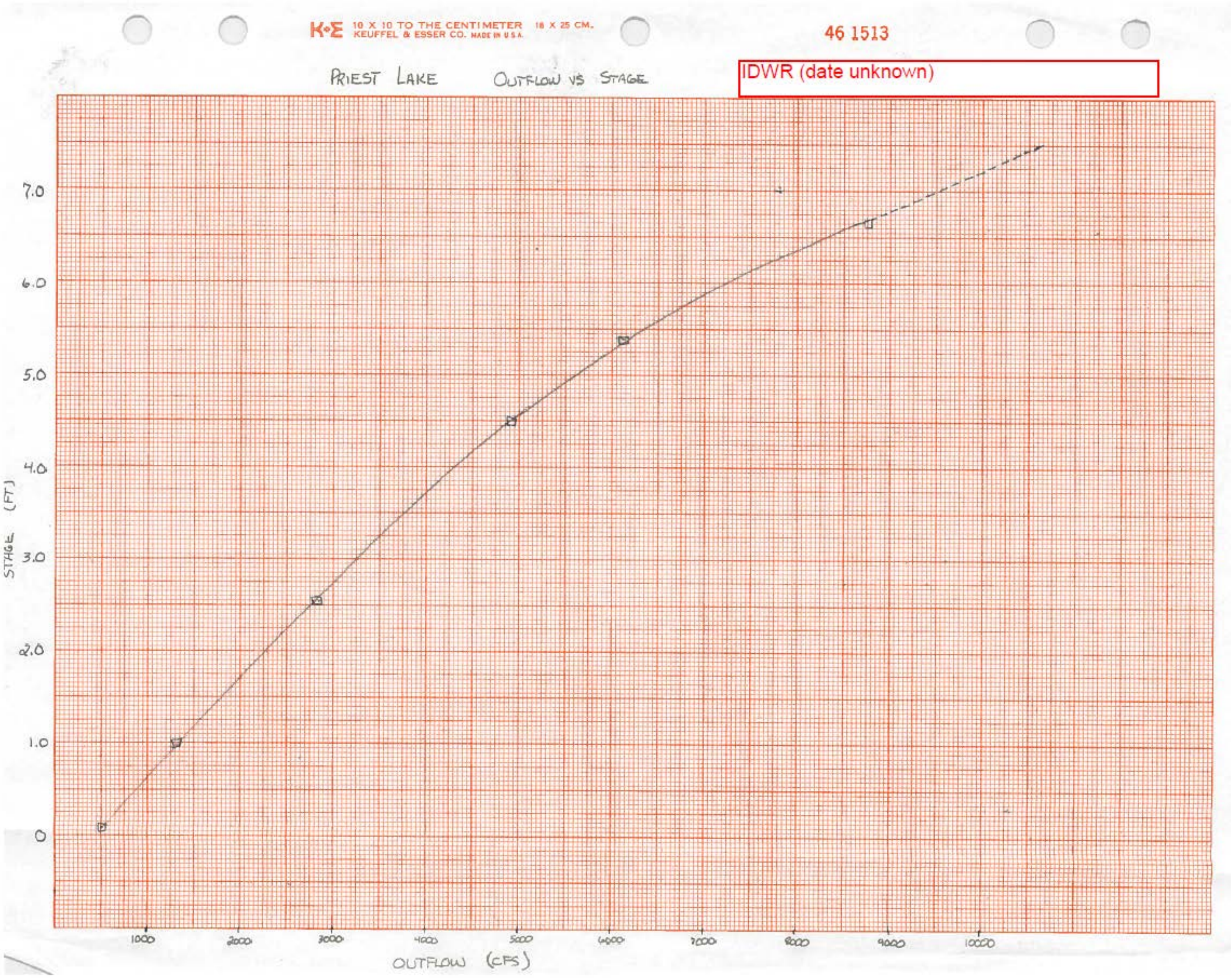


Figure 2. Priest Lake outlet stage-discharge curve (IDWR, Date Unknown).

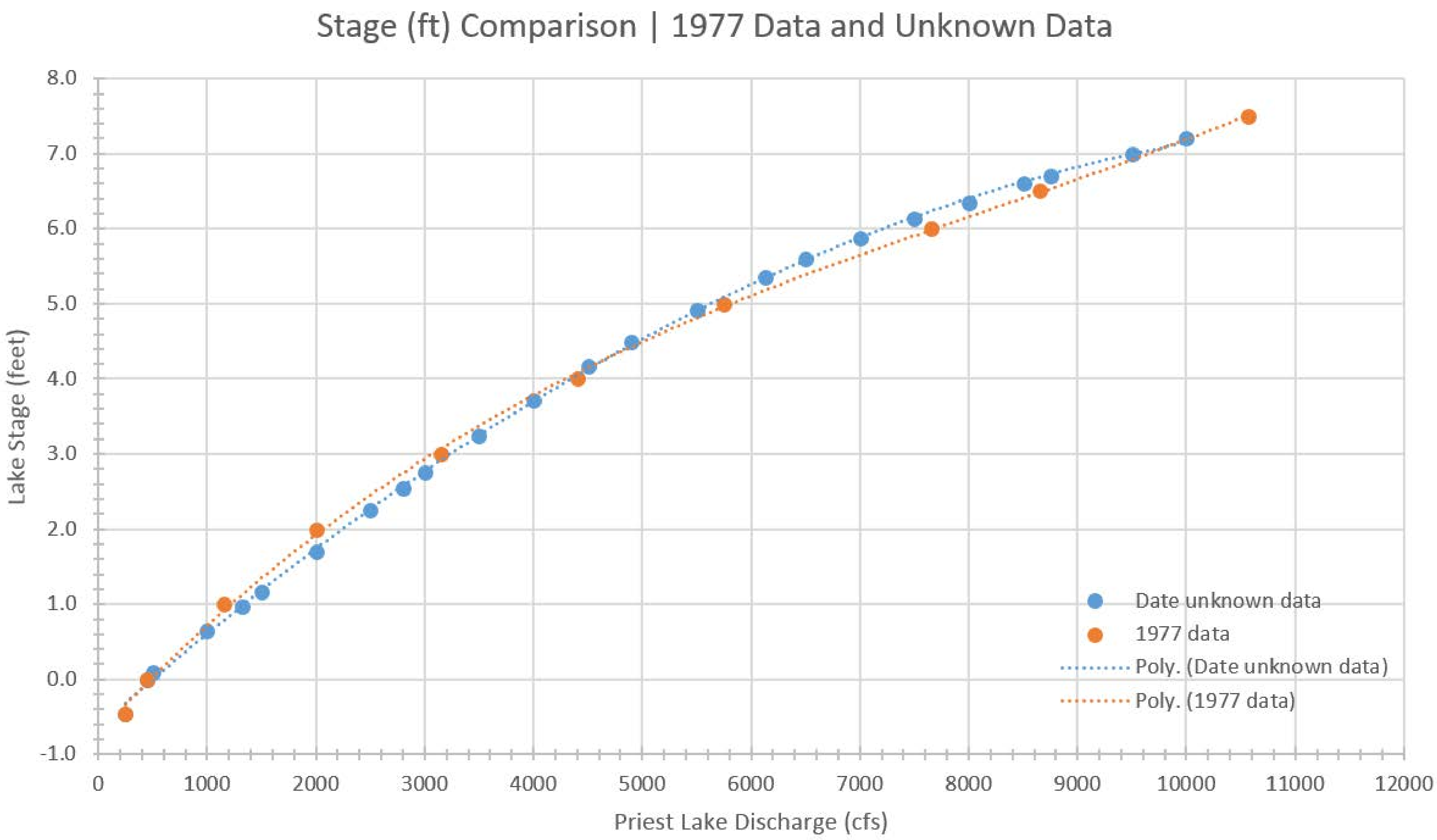


Figure 3. Comparison of available Priest Lake outlet stage-discharge curves.

B.2 Pool Raise Assessment Summary

MOTT MACDONALD

Priest Lake Water Management Study

Dry Year Pool Raise Assessment – 08/10/2017

IDAHO WATER RESOURCE BOARD

Water Level Management – Background & Purpose

- In 2015, drought conditions made maintaining the required summer lake levels & minimum discharge from the dam very difficult. The discharge from the dam was reduced below the current policy of minimum 60 cfs to maintain the lake level and meet statutory requirements.

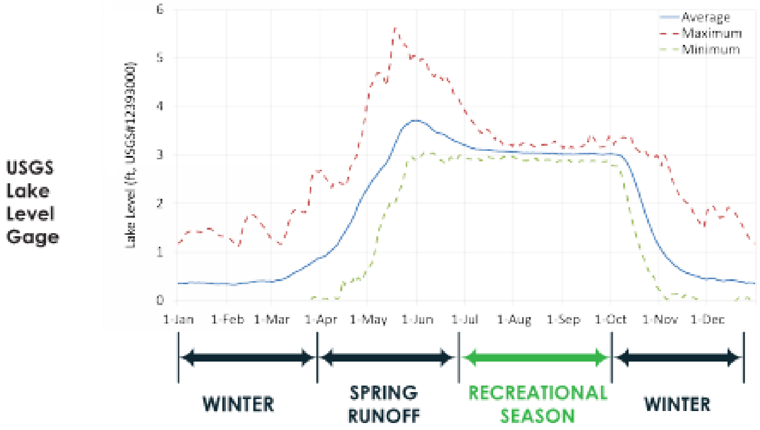
- In 2016, which had seemed to be a typical year, the same issue occurred and a crisis was nearly averted.

- These incidents highlighted the need for improvements to lake level management and measures that may need to be taken during dry or marginally dry years.

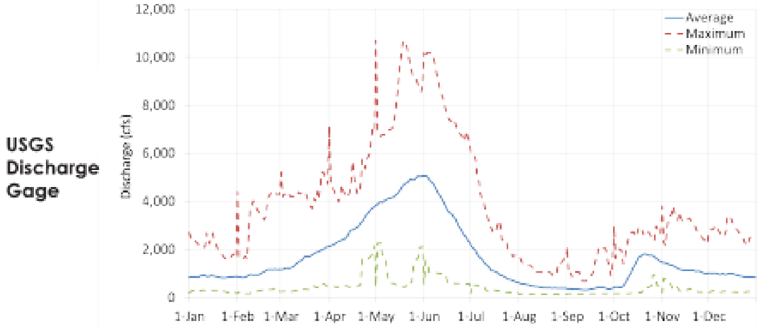
Study Purpose:

- Evaluate possible changes for a dry year water management scheme consisting of either a 3-inch or 6-inch higher lake level during part of the summer recreational season

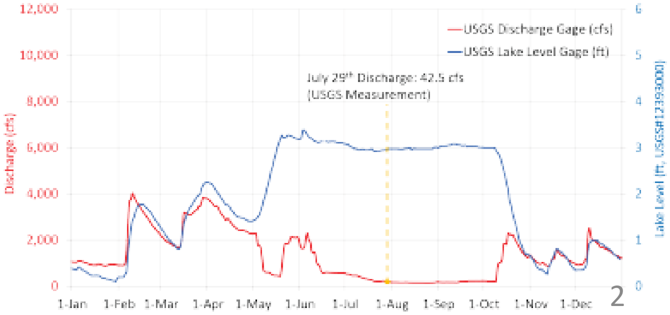
ANNUAL LAKE LEVEL VARIATION (1980 - PRESENT)



ANNUAL DISCHARGE PATTERN (1980 - PRESENT)



2015 LAKE LEVEL & DISCHARGE PATTERN

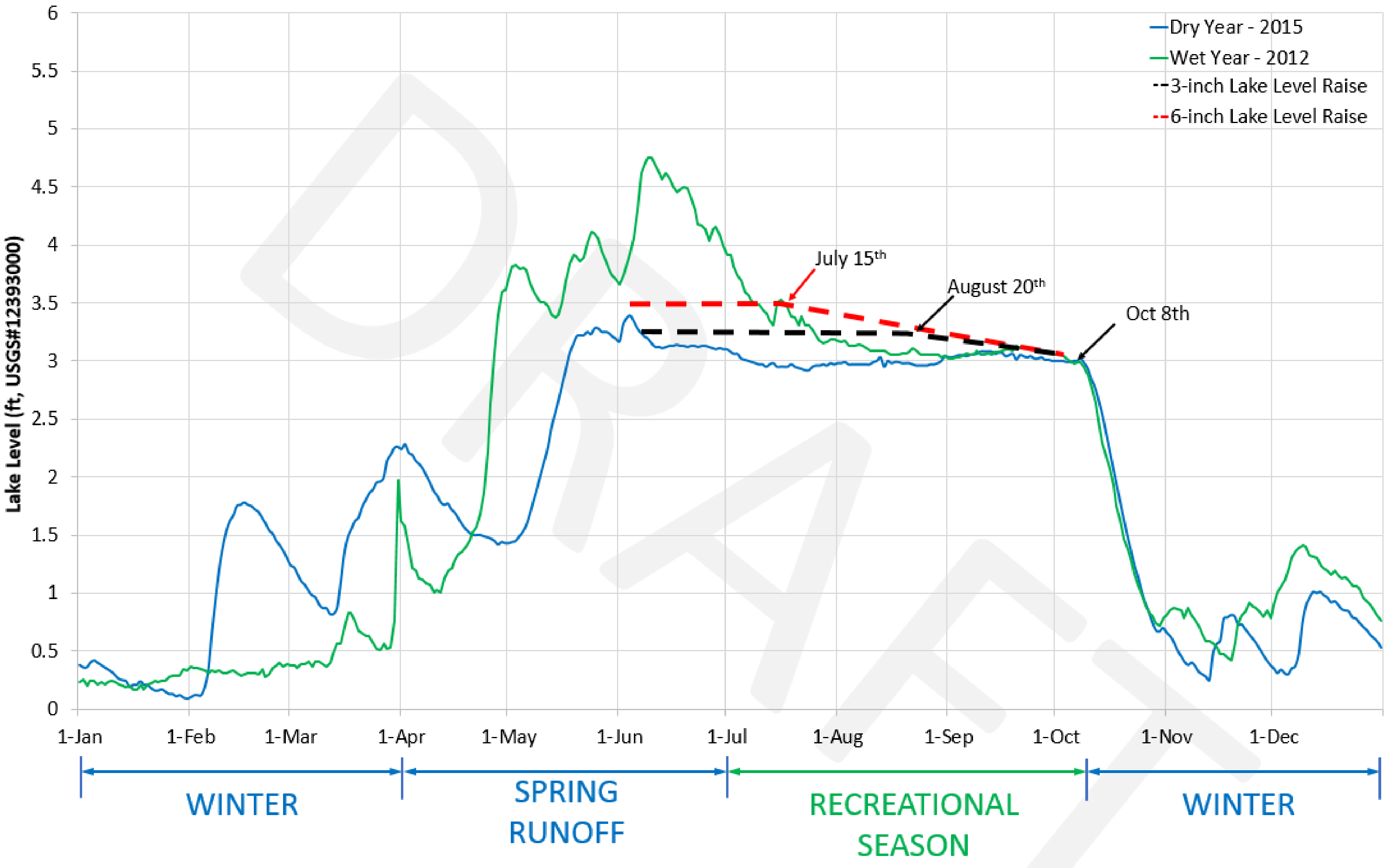


Pool Raise - Assessment Methodology

- Existing conditions will be reviewed
- Proposed conditions for a possible 3-in or 6-in pool raise will be investigated
- Assessment will highlight change in existing conditions due to dry year pool raise and potential for adverse impacts

Priest Lake Water Level - Background

- It is typical for the lake level to be higher than required 3.0’ during the recreational season.
- Water level chart shows that in 2012, lake level was 6-in higher than 3.0’ required level till July 15th.
- Water level chart also shows that in 2012, lake level was 3-in higher than the 3.0’ required level till July 20th.
- Temporary pool raise during dry years can be thought of as managing lake level similar to natural lake level in wet years with a slight increase during month of August



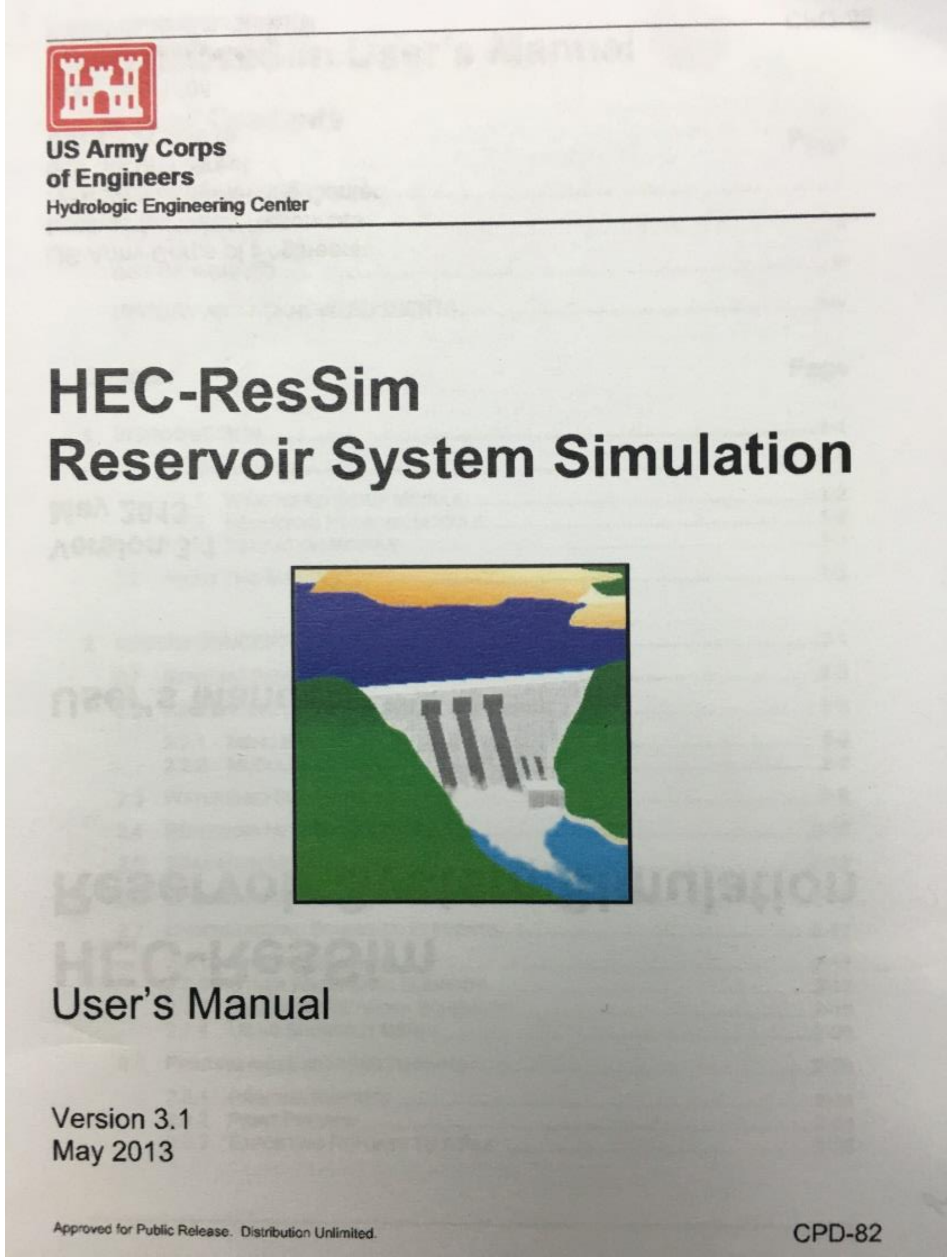
LAKE WATER LEVEL MANAGEMENT OPTION ANALYSIS

- Water Management Analysis – See Appendix B.3
- Evaluation of Outlet Dam operations
- Input Data
- Simulations
- Preliminary Conclusions

- 3” Pool Raise likely will work relative to historical dry years to meet the defined criteria.

- Integration of real time streamflow data into dam operations

- Provide larger tolerance in operations to allow more flexibility (currently operated to maintain as close to 3.0’ as possible). Allow variation of 3 to 4”.



US Army Corps
of Engineers
Hydrologic Engineering Center

HEC-ResSim
Reservoir System Simulation

User’s Manual

Version 3.1
May 2013

Approved for Public Release. Distribution Unlimited.

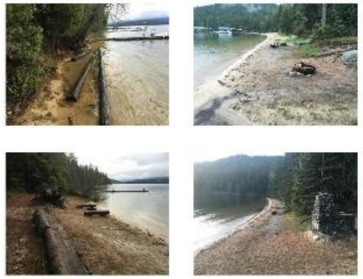
CPD-82

Dry Year Pool Raise Assessment

Purpose: Evaluate changes from existing conditions & potential for impacts on the following elements due to pool raise (3” or 6”).

SHORELINE FEATURES

Recreational Beaches



- Pier Access
- Benches
- Beaches
- Stairs
- Fire Pit

Boat Covers

- Fixed Height Structures
- Height of Vessel Adjustments

Natural Shoreline

- Natural Shoreline Features
- Rocks, Vegetation
- Large % of Overall Lake Shoreline

Public/Private Boarding Access

- Beach
- Boat Ramps

ADDITIONAL CONSIDERATIONS

Basement Flooding

Vessel Wakes

Wetlands

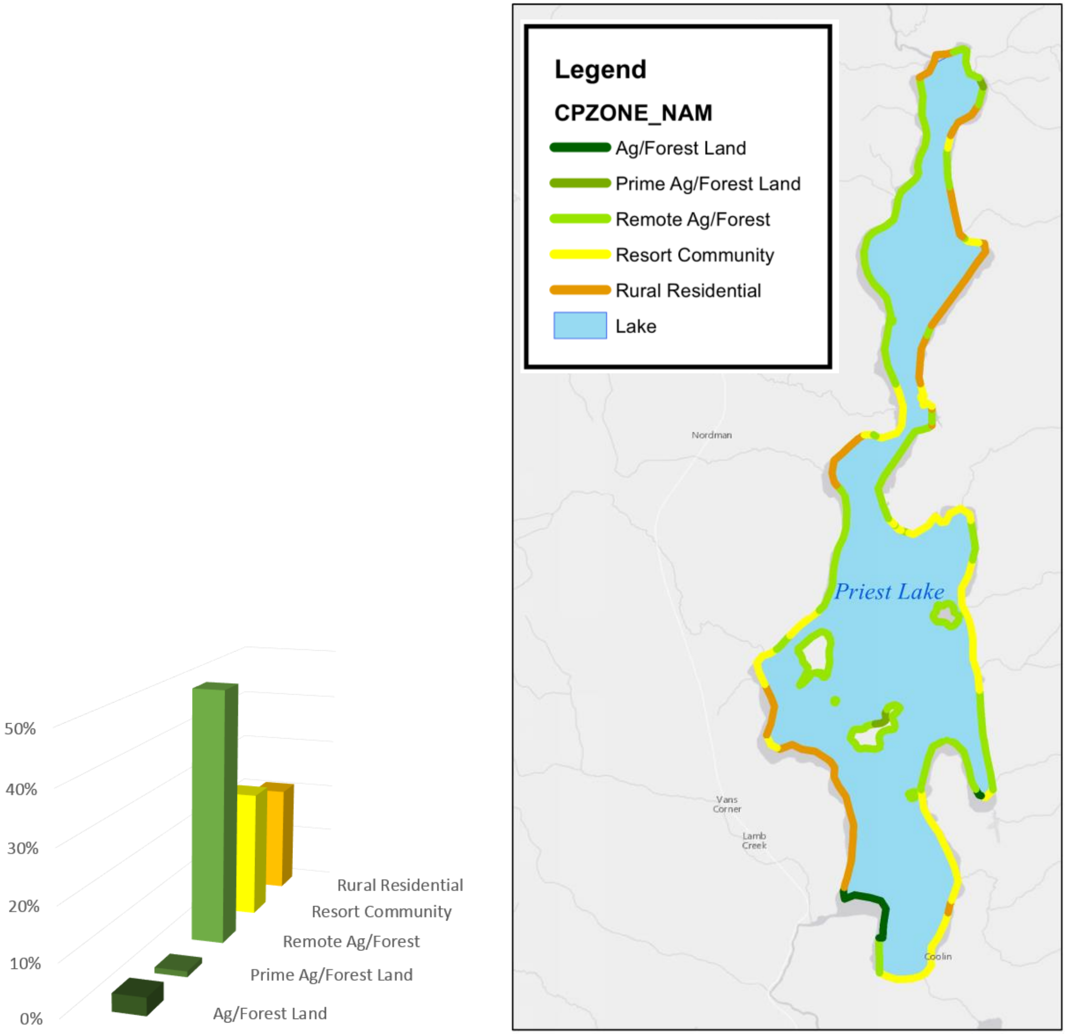
Shoreline Type - Background

1) Natural Vegetated

2) Beach

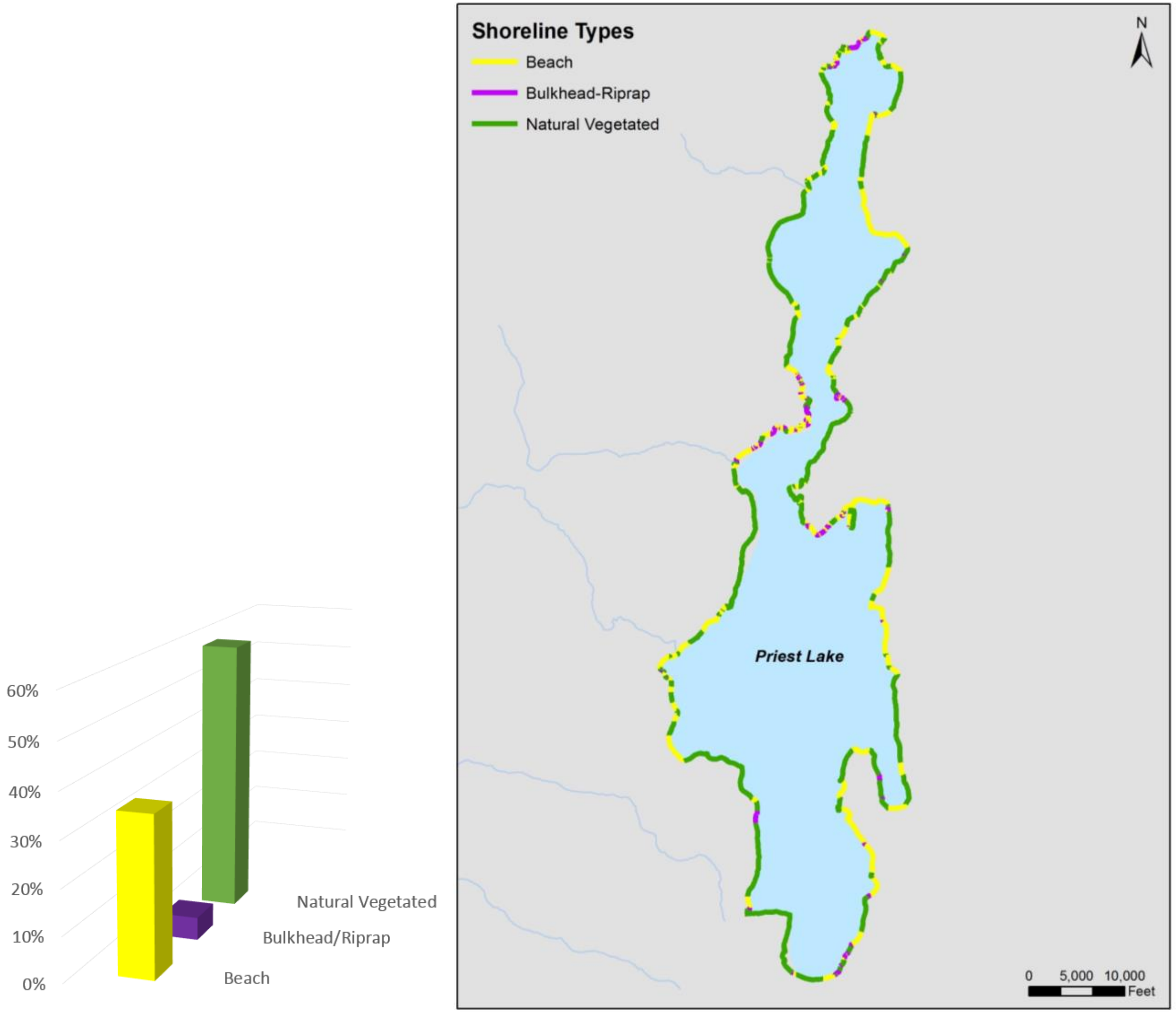
3) Bulkhead/Riprap

Land Use & Shoreline Type - Background



Land Use

(using Bonner County GIS Data)



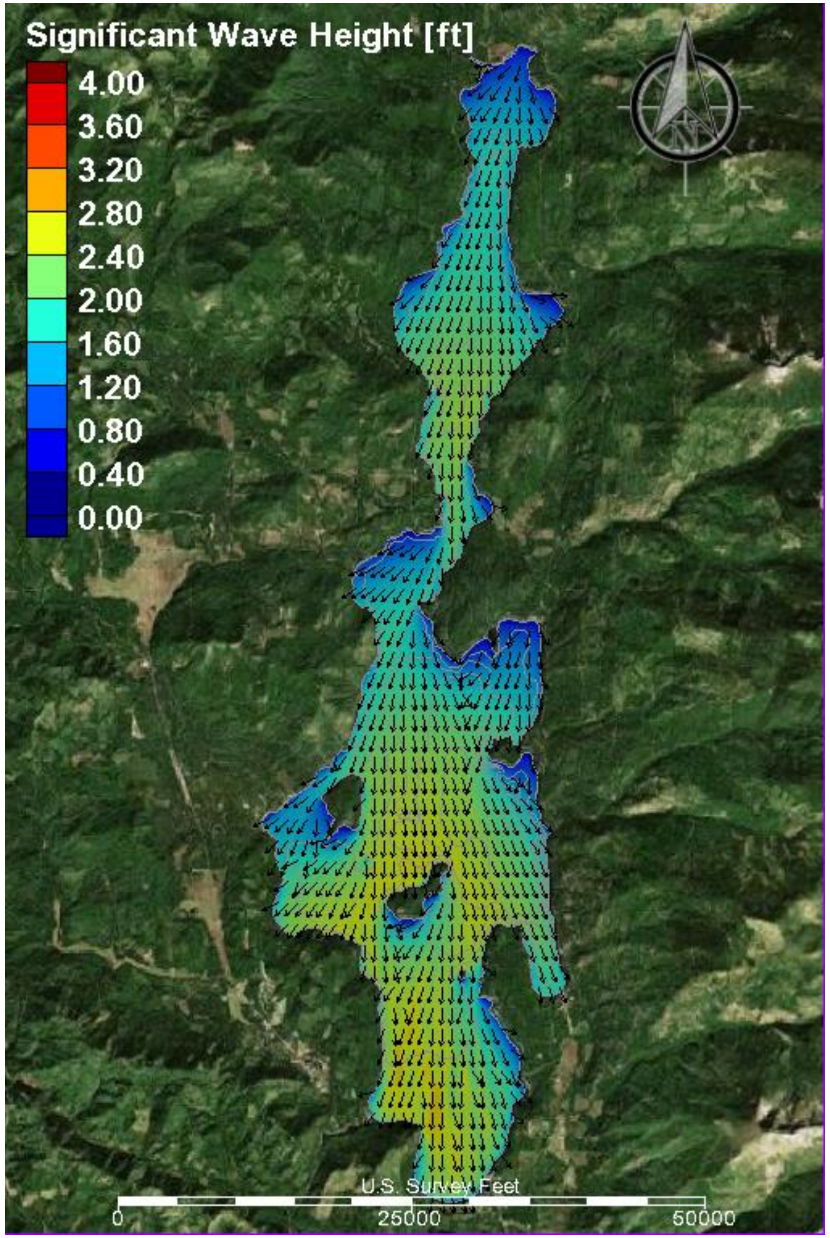
Shoreline Types

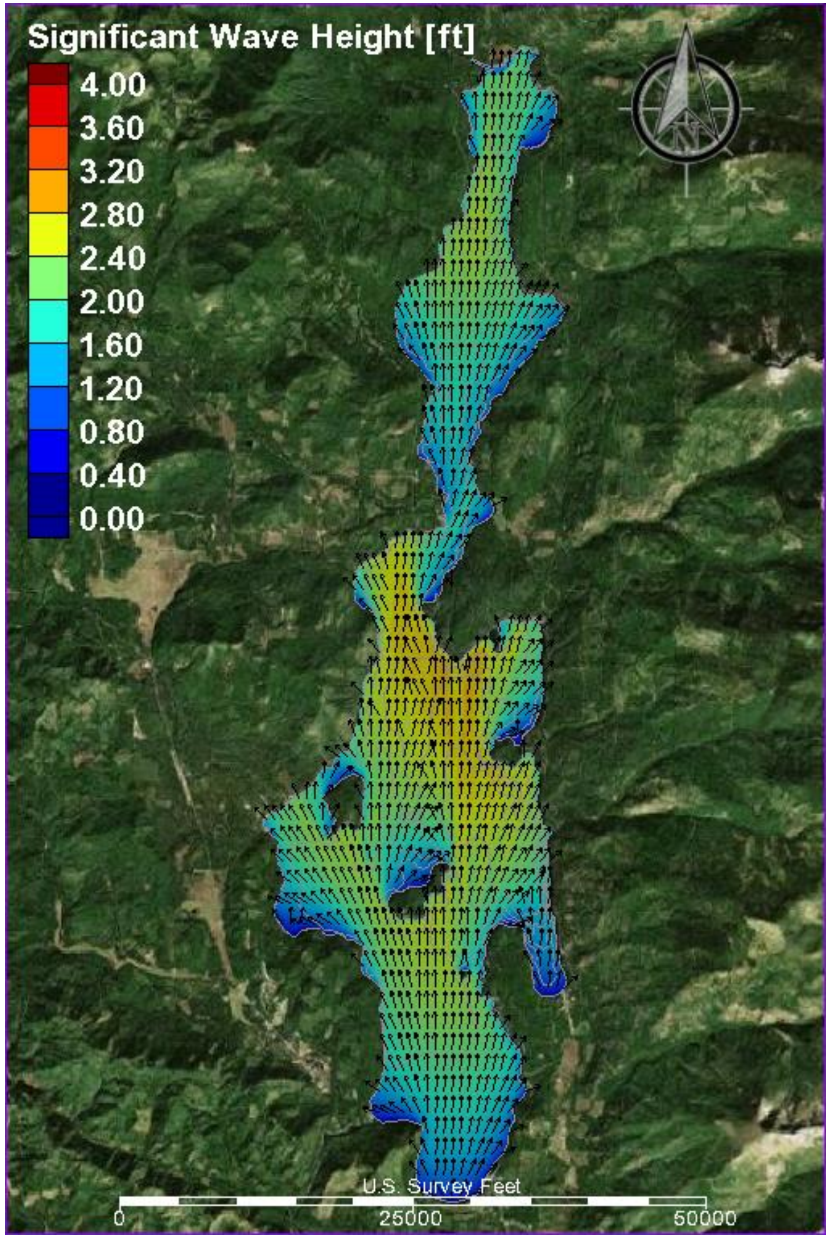
(digitized based on aerial imagery)

Wind Wave Climate - Background

- Dominant southerly (blowing from South) winds during daytime and northerly winds during night time generate largest waves on the lake

- South-facing sandy beaches are exposed to largest waves on the lake





Dry Year Pool Raise Assessment

Changes from existing conditions & possible impacts on the following elements due to pool raise will be investigated:

- Recreational Beach Use
- Shoreline Erosion
- Access to Fixed Structures (Docks, Boat Lifts)
- Navigation Access to Marinas
- Boat Launch Facilities
- Fish Habitat
- Thorofare Navigation
- Wetland & Riparian Vegetation
- Basement Flooding

Recreational Beach Use

- Typical slopes for sandy beaches around the lake were found to be approximately 10:1 to 15:1.
- Approximately additional 2.5 ft of beach will be underwater during temporary 3-in pool riase
- Approximately 5 ft of beach will be underwater during temporary 6-in pool raise
- MM's site visit in May 2017 with Lake Level @ 3.51' (6-in higher than required 3.0') showed that most benches/fire pits were out of water.

Assessments:

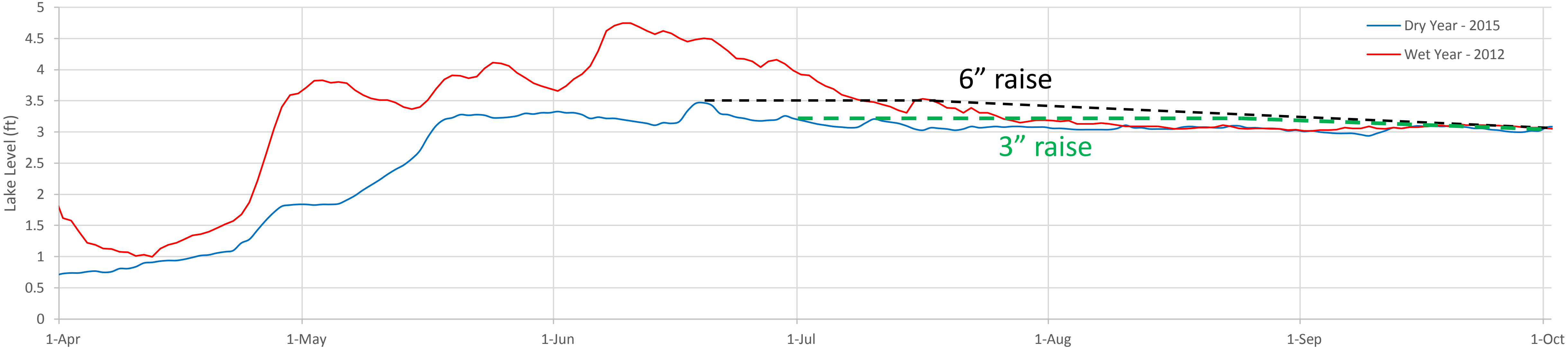
- Temporary 3-in pool raise will not pose significant impact on recreational beach use at majority of existing beaches
- Temporary 6-in pool riase may have a noticeable impact on properties with small pocket beaches

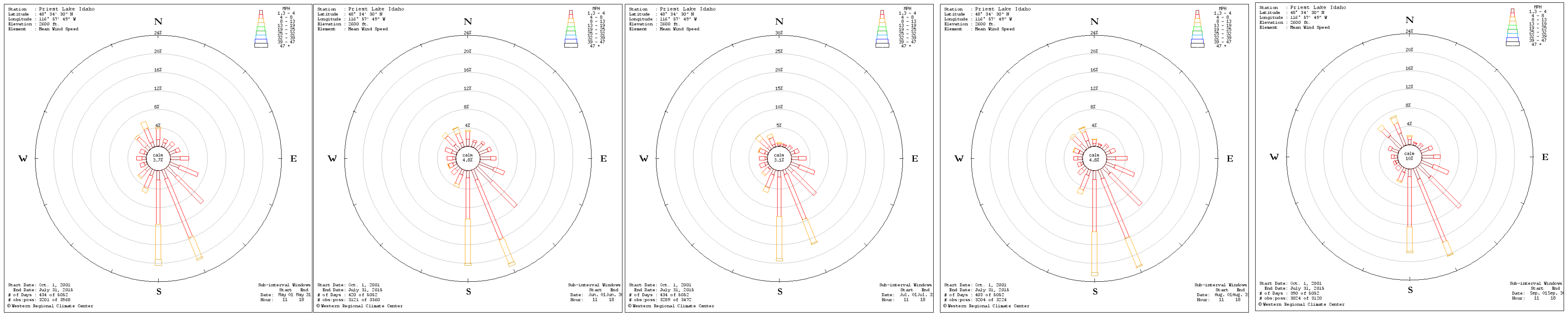

Photo taken with Lake Level @ 3.51'


Photo taken with Lake Level @ 3.51'

Shoreline Erosion – Wind Waves

Seasonal Variation of Day-Time Wind





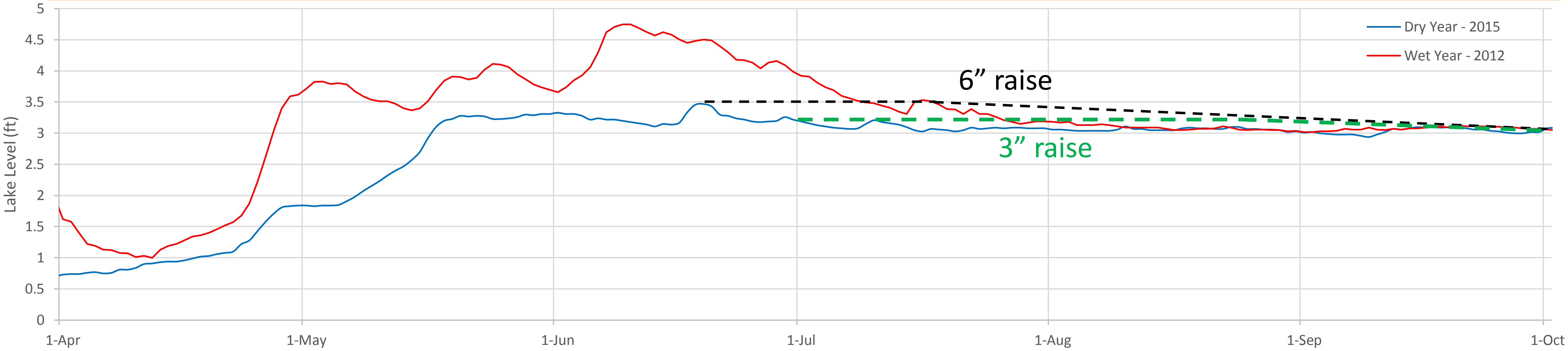
Observation:

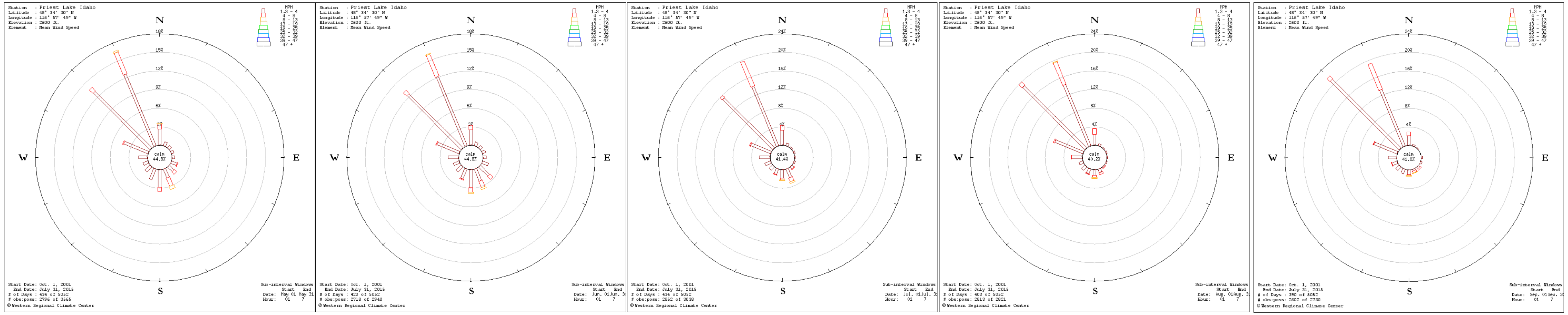
- No significant variation in wind speed, direction & frequency during summer season (with high lake levels)
- High water levels (due to spring runoff) & Wind Storms occurring simultaneously are probable in May & June.

Assessments: Expected to be minimal

Shoreline Erosion – Wind Waves

Seasonal Variation of Night-Time Wind





Observation:

- No significant variation in wind speed, direction & frequency during summer season (with high lake levels)
- High water levels (due to spring runoff) & Wind Storms occurring simultaneously are probable in May & June.

Assessments: Expected to be minimal

Shoreline Erosion – Vessel Wakes

Vessel wake energy attenuates with distance from the vessel track

Vessel wake erosion is important in areas where boats travel quite close to shoreline or narrow waterways.

An example for shoreline prone to erosion due to vessel wakes is the Northwestern shoreline of Kalipsel Island

Pontoon Boat - Avalon

Beam = 8.5’
Length = 25’ ~ 27’

North River – Seahawk Outboard 21’

Beam = 8.5’
Length = 25’

Wake Board & Surfing Boats

Beam = 8.5’
Length = 25’

Assessments: Impacts expected to be minimal if vessels operated at a reasonable distance from shoreline. Additional investigation in next phase to review vessel operations relative to areas of concern.

Access to Fixed Structures

- Engineered fixed structures (docks and boat lifts) are typically designed with a 12-in freeboard during summer recreational period.
- A few non-engineered boat lifts and docks were observed that could be affected with a 6-in pool raise.

Assessments:

- Negligible/no impact due to a 3-in pool raise
- Minor impact on non-engineered structures due to a 6-in pool raise such as water on sloped end of gangway
- Many boat lifts are adjusted annually and could be adjusted to accommodate the anticipated drought year pool raise.

Photo taken with Lake Level @ 3.51’

Photo taken with Lake Level @ 3.51’

Navigation Access to Marinas

- A number of docks and marinas have limited depths for access and moorage areas. As an example, Elkins Resort marina has shallow depth due to Reeder Creek outlet being inside the marina.

- Pool raise will benefit those facilities with current depth constraints.

- Water management schemes reducing water level during recreational season will impact access and use of those docks/marinas.

Assessments:

- Positive change expected with a 3-in or 6-in pool raise

Boat Launch Facilities

- Boat launch ramps typically have a slope of 12% to 15%. Position of vessel/trailer on launch with a 3-in and 6-in pool varies 20-in to 40-in.

- Boat ramps observed were generally designed for higher lake level (above normal lake level at 3.0’ USGS gage) and experience higher lake levels during spring run off.

Assessments:

- Ramp usability will not be impeded. Slight change in location of interface of floating handling floats expected. Impacts expected to be minimal.

Photo taken with Lake Level @ 3.51’

Photo taken with Lake Level @ 3.51’

Fish Habitat

Bull Trout:

- Migration period mid-May through November. Typically, in spawning streams by November.
    - Adjusted lake levels don't result in restricted access to tributary streams

Kokanee:

- Spawning in November, December, in nearshore gravels.
- Lake level to be stabilized by November 1; start of potential kokanee spawning.

Assessments:

Minimal/no impact on Bull trout and Kokanee

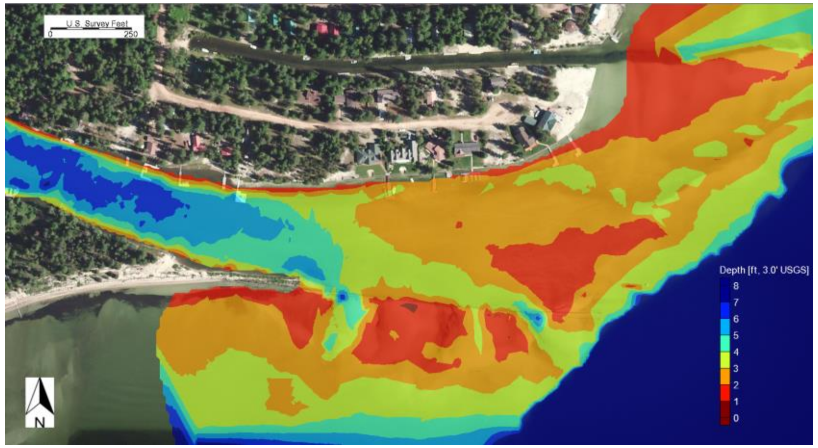
Thorofare Navigation

- Current navigation access to Thorofare is challenging
- Red areas show water depth shallower than 2 ft
- The figures show that areas with shallow depth shrink in size with increase in water level

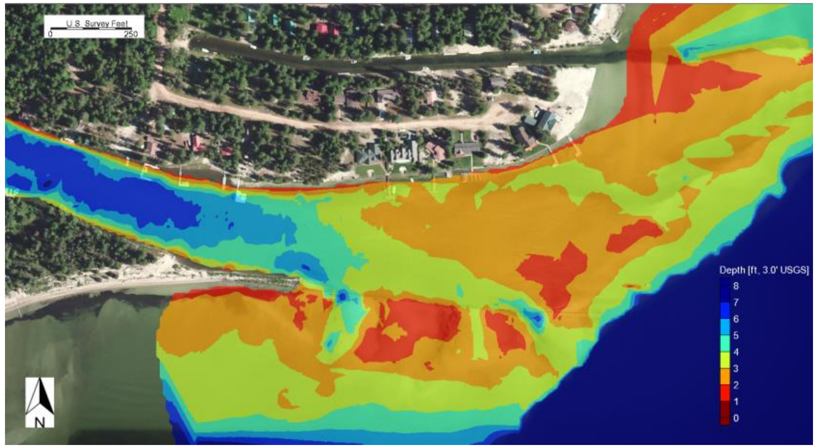
Assessments:

No direct impact to Thorofare navigation. Both alternatives would improve navigation access to Thorofare.

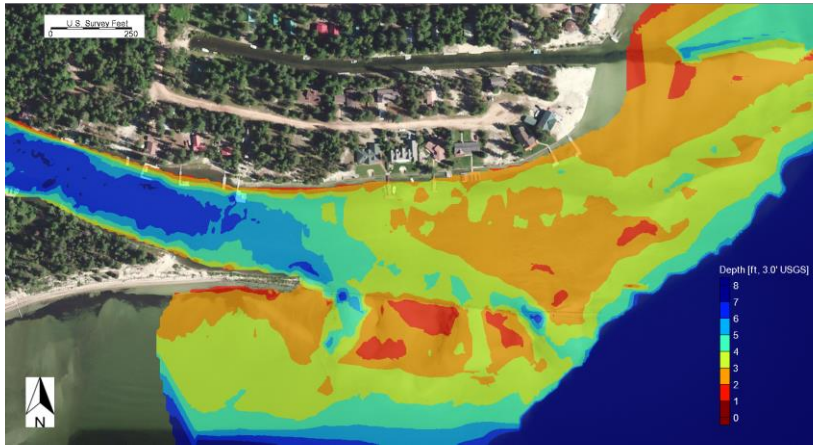
Existing Conditions
WL= 3.0’ ft USGS



3-in Pool Raise
WL= 3.25’ ft USGS



6-in Pool Raise
WL= 3.50’ ft USGS



Wetland & Riparian Vegetation

Oblique aerial photo of 5/28/2018 shows inundation of wetlands to the Northwest of Sandpiper's shore

Water level on 5/28/2017 = 3.88'

Wetlands are frequently inundated during spring runoff

Primarily locations for review include:

- Adjacent to Thorofare

Assessments:

Negligible impact to wetlands & riparian vegetation due to temporary 3-in or 6-in pool raise. Additional investigation to be conducted in next phase.

Basement Flooding

Based on input from lake residents/Steering Committee, basement flooding only occurs during extreme periods of spring run-off when Lake Level exceeds 5.0 or 6.0 ft.

Assessments:
No impact due to a 3-in or 6-in temporary pool raise.

Dry Year Pool Raise Assessment - Summary

Temporary pool raise is being considered as an improvement measure **only** for dry and marginally dry years. Therefore, any possible impact will be limited to these years.

Alternative	Recreational Beach Use	Lake Shoreline Erosion	Access to Fixed Structures	Navigation Access to Marinas	Boat Launch Facilities	Fish Habitat	Thorofare Navigation	Wetland & Riparian Vegetation	Basement Flooding
3-inch Pool Raise	⬜	⬜	⬜	🟩	⬜	⬜	🟩	⬜	⬜
6-inch Pool Raise	(1) 🟨 ⬜	(3) ⬜	(2) 🟨 ⬜	🟩	⬜	⬜	🟩	⬜	⬜

Change with respect to a typical or a wet year:

- 🟩 Positive Change
- ⬜ No or Negligible Change
- 🟨 Low Impact
- 🟥 High Impact

Footnotes:

(1): There will be no impact on majority of the beaches. Localized areas will see loss of usable dry beach.

(2): There will be no impact on majority of fixed structures. A low percentage of structures will see low impacts.

(3): Majority of beaches will have no sustained impact. Some localized areas need further investigation for vessel wake effects.

Pool Raise Assessment – Next Steps

- Additional investigation is needed during the next phase of the project. Those would include the following:
  - Detailed review of the wetlands by site investigation, review of available wetland data, and review of vegetation type relative to existing elevation and proposed pool level change.
  - Detailed review of vessel wake impacts as a result of higher pool.
    - Review of existing vessel wake areas of concern and need for reduced speed or no wake zones.

MOTT
MACDONALD

Priest Lake Water Management Study

Dry Year Pool Raise Assessment – 08/10/2017

IDAHO WATER RESOURCE BOARD

Draft v0

B.3 Water Level Management Hydrologic Analysis

GEOENGINEERS

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Memorandum

To: Shane Phillips, PE and Younes Nouri, PE; Mott MacDonald
From: Tim Hanrahan and Jason Scott; GeoEngineers, Inc.
Date: December 12, 2017
File: 22593-001-00
Subject: DRAFT - Priest Lake Water Management Study Hydrologic Analysis

INTRODUCTION

The Priest Lake Water Management Study (Study) was completed to identify water management options that would achieve three main goals. First, preserve lake levels through the recreation season during dry and marginally dry years. Second, maintain a minimum discharge of 60 cubic feet per second (cfs) from the dam throughout the year. Third, provide sustainable modifications to improve thorofare access, navigability and water quality.

The objective of the hydrologic analysis task was to develop a water management analysis tool for Priest Lake and the outlet dam. This objective was addressed by using the data compiled from other Study tasks to develop a lake simulation model using the HEC-ResSim software (USACE 2013). The model was used to evaluate different components of the Priest Lake water balance (inflows, lake level, discharge) for different water management scenarios.

METHODS

Water management alternatives for Priest Lake were modeled with the reservoir simulation model HEC-ResSim (USACE 2013). The software is commonly used as a decision support tool for evaluating reservoir operations for a variety of operational goals and constraints. For the Priest Lake system, HEC-ResSim was used to model the operational goal of a prescribed lake level (e.g., recreation season level of 3.0 feet from July 1 to October 8) under corresponding constraints of prescribed discharges at the Priest Lake outlet dam, and total inflow to Priest Lake. Each water management alternative modeled (Table 1) included a unique combination of: (1) lake level goal during the calendar year; (2) discharge constraints during the calendar year; and (3) total calculated inflow to Priest Lake during the calendar year. Each alternative was modeled at a daily time step over the duration of selected calendar years to represent dry (2001, 2005) and normal (2002) water-year types.

Completion of the water management simulations in HEC-ResSim required data for the physical characteristics of Priest Lake, the physical characteristics of the Priest Lake outlet dam, and the inflow hydrology to Priest Lake.

Priest Lake Physical Data

The relationship between lake level and lake volume (level-volume rating curve) is required for the simulation modeling. These data were available in the form of a rating table for U.S. Geological Survey (USGS) gage 12393000, which was provided by Idaho Department of Water Resources (Matt Anders, IDWR, personal communication). The rating table from IDWR provided the gage height (feet) and corresponding lake volume capacity (acre-feet) for gage heights ranging from 0.0 feet to 6.69 feet. The elevation datum of the gage is

2,434.64 feet (NGVD29), and this elevation was used in the modeling to simulate lake levels relative to the geodetic datum.

The lake-volume rating curve was extrapolated at the upper and lower bounds. USGS water year summary reports for gage 12393000 indicate a minimum lake level of -0.46 feet occurred in 1977 and 2001, with a corresponding lake volume of 37,500 acre-feet. Accordingly, the rating curve was extended to -0.46 feet by using the linear fit between lake level and lake volume. Similarly, the rating curve was extended to 7.5 feet by using a linear fit equation. Extending the data to 7.5 feet of lake level was done to use these data with the available stage-discharge rating curve for the outlet dam.

The HEC-ResSim reservoir network setup process was used to define Priest Lake in the model schematic. The model was setup as one pool with the lake-volume rating curve described above, one “virtual” inflow location at the upper end of the pool, and one outlet at the Priest Lake Dam.

Outlet Dam Physical Data

The relationship between lake level and Priest Lake discharge (stage-discharge rating curve) is required for the simulation modeling. The lake level must be in the same vertical datum as USGS gage 12393000. These data were available in the form of a rating curve developed by IDWR (1977). Data points from this curve were entered into a spreadsheet and used to develop a polynomial fit between lake stage and discharge. The upper and lower bounds of this rating curve were extended by using the polynomial fit. Extending the rating curve was done to use these data with the available level-volume rating curve for Priest Lake. These outlet dam physical data were used to setup the Priest Lake outlet in the HEC-ResSim model schematic.

Priest Lake Inflow Hydrology

Total daily inflow to Priest Lake is required for the simulation modeling. There are twenty-one tributary streams flowing into Priest Lake. Because the tributary streams are ungauged, IDWR calculated the total daily inflow from a water balance based on measured change in lake volume at USGS gage 12393000, discharge from Priest Lake into Priest River measured at USGS gage 12394000, and calculated lake evaporation (Matt Anders, IDWR, personal communication). These data were provided by IDWR and used in the HEC-ResSim model and HEC-DSS data management software.

Hydrology data from three different calendar years were used to simulate water management scenarios with HEC-ResSim. The available data were based on the calendar years 1979 – 2006, because this represents the time period since the existing Priest Lake Dam was rebuilt in 1979, and before the removal of USGS gage 12394000 in 2006. Based on the total annual discharge at USGS gage 12394000, calendar year 2002 was selected to represent a normal water year because the runoff for 2002 was 682,000 acre-feet, which is slightly larger than the 1951-2015 average of 638,000 acre-feet. Calendar years 2001 and 2005 were selected to represent dry water years, because they are the two most recent years with the lowest annual discharge at USGS gage 12394000.

Water Management Simulations

HEC-ResSim was used to simulate each of the water management scenarios (Table 1). The scenarios included existing lake operations of a 3.0 feet lake stage during the recreation season, and an accompanying year-round minimum discharge of 60 cfs from Priest Lake Dam. For each of the existing conditions simulations, empirical lake level data were used in the model as the target/goal elevation for each day of the calendar year. Priest River discharge constraints (Table 1) were applied to the model as flow rules that limited the model-determined discharge from Priest Lake Dam. Using the daily virtual inflow hydrology, the HEC-ResSim model determined the Priest Lake Dam discharge necessary to maintain the prescribed lake level goal within the applied discharge constraints. The inflow hydrology included calculated evaporation losses (Matt Anders, IDWR, personal communication) for each day of the simulation, while groundwater inflows and seepage losses were not included. Existing lake operations were simulated to compare the model setup with empirical data for each of the three calendar years considered.

Alternative lake operations were simulated with a similar approach, but with modified lake level goals and Priest River discharge constraints (Table 1). For each calendar year, empirical lake level data were modified to reflect the lake level elevation goals during the recreation season (Table 1), while outside of the recreation season the empirical lake level data were used as the lake elevation goal for each calendar year simulation. Priest River discharge constraints included a 60 cfs minimum flow year-round, 2000 cfs maximum flow during October, and 1,200 cfs maximum increase per day during October (Table 1). Using the daily virtual inflow hydrology for each calendar year simulated, the HEC-ResSim model determined the Priest Lake Dam discharge necessary to maintain the prescribed lake level goal within the applied discharge constraints.

CONCLUSIONS

The simulation of existing conditions indicates good agreement between model results and empirical observations (Attachment A). For the 2001 calendar year, the model performed well at matching observed lake elevation and discharge. Similarly, good model performance was observed for the 2002 and 2005 calendar years. For all of the existing conditions simulations, there are some periods where the modeled lake elevation is slightly higher than the observed lake elevation. These differences are contributed to several factors:

- The model algorithms are set to manage the lake level at or above the elevation goal, given the calculated inflow and discharge constraints (if any).
- Large, rapid changes in calculated inflow over a daily time-step.
- Differences in dam operations, whereby the model discharge is adjusted to maintain a constant lake elevation that does not exist in the observed lake elevation data.
- Uncertainty in the model input data, including calculated “virtual” inflow and evaporation at a daily time step, lake elevation from USGS gage 12393000 at a daily time step, the Priest Lake elevation-volume rating curve for USGS gage 12393000.

The simulation of alternative water management scenarios (Alternatives 1 through 5) indicate that increased lake levels during the recreation season can be maintained during the typical water years modeled, within the constraints set on minimum and maximum Priest Lake Dam discharge (Attachment A). For example, Alternative 1D results suggest that during a dry water year like 2001, a 6-inch higher lake level during the recreation season can be achieved while meeting the discharge constraints of a 60 cfs minimum year-round flow. In order to achieve these higher summer lake levels, the minimum monthly discharge for many alternatives is 60 cfs.

Water management flexibility for the Priest Lake system will become increasingly important as water supply uncertainty increases in the future. Under projected climate change scenarios, the Priest Lake watershed is forecasted to experience increased annual precipitation and increased winter precipitation (USFS 2017). However, the snow-water equivalent on April 1 is projected to decrease by 20 to 100 percent, while the snow residence time is projected to decrease by 20 to 80 percent (USFS, 2017). These changes in projected water supply, relative to historic conditions, suggest the need for flexibility in the Priest Lake system to manage changes in the timing and volume of water supply.

REFERENCES

Idaho Department of Water Resources (IDWR). 1977. Priest Lake Outlet, Phase 1, Construction of Sheet Pile Cutoff Wall. Plans, Specifications, and Contract Documents. Section 5, Plans and Drawings. Idaho Department of Water Resources, Boise, Idaho.

U.S. Army Corps of Engineers (USACE). 2013. HEC-ResSim Reservoir System Simulation User's Manual, version 3.1. USACE Institute for Water Resources, Hydrologic Engineering Center, Davis, California.

U.S. Forest Service (USFS). 2017. National Forest Climate Change Maps. Available at https://www.fs.fed.us/rm/boise/AWAE/projects/national-forest-climate-change-maps.html. Accessed 25 September 2017.

Attachments:

Table 1. Priest Lake Water Management for Existing Operations and Alternatives for dry and normal water-year types
Attachment A. Priest Lake Water Management Simulation Results

TABLE 1. PRIEST LAKE WATER MANAGEMENT FOR EXISTING OPERATIONS (EXIST) AND ALTERNATIVES (ALT) FOR DRY AND NORMAL WATER-YEAR TYPES.

		Recreation Lake Level			Priest River Discharge (Q)			
Management Scenario	**Water-year Type**	**Stage (ft)**	**Start Date**	**End Date**	**Q (cfs)**	**Start Date**	**End Date**	**Inflow Calendar Year**
Exist D1	Dry	3.0	Jul 01	Oct 08	60 min	Jan 01	Dec 31	2001
Exist D2	Dry	3.0	Jul 01	Oct 08	60 min	Jan 01	Dec 31	2005
Exist N	Normal	3.0	Jul 01	Oct 08	60 min	Jan 01	Dec 31	2002
Alt 1D	Dry	3.5	Jul 01	Aug 31	60 min	Jan 01	Dec 31	2001
		3.5 to 3.0	Sept 1	Oct 8	2,000 max	Oct 1	Oct 31	
					1,200 max increase per day	Oct 1	Oct 31	
Alt 2D	Dry	3.5	Jul 01	Aug 31	60 min	Jan 01	Dec 31	2005
		3.5 to 3.0	Sept 1	Oct 08	2,000 max	Oct 1	Oct 31	
					1,200 max increase per day	Oct 1	Oct 31	
Alt 2N	Normal	3.5	Jul 01	Aug 31	60 min	Jan 01	Dec 31	2002
		3.5 to 3.0	Sept 1	Oct 08	2,000 max	Oct 1	Oct 31	
					1,200 max increase per day	Oct 1	Oct 31	
Alt 3D	Dry	3.25	Jul 01	Aug 31	60 min	Jan 01	Dec 31	2001
		3.25 to 2.75	Sept 1	Sept 30	2,000 max	Oct 1	Oct 31	
					1,200 max increase per day	Oct 1	Oct 31	
Alt 4D	Dry	3.5	Jul 01	Aug 15	60 min	Jan 01	Dec 31	2001
		3.5 to 3.0	Aug 16	Sep 15	2,000 max	Oct 1	Oct 31	
		3 to 2.75	Sept 16	Oct 08	1,200 max increase per day	Oct 1	Oct 31	
Alt 5D	Dry	3.25	Jul 01	Aug 31	60 min	Jan 01	Dec 31	2001
		3.25 to 3.0	Sept 1	Oct 8	2,000 max	Oct 1	Oct 31	
					1,200 max increase per day	Oct 1	Oct 31	

ATTACHMENT A
Priest Lake Water Management Simulation Results

Simulation Results for Priest Lake Water Management Alternatives

- HEC-ResSim used to model operational goal of prescribed lake level under corresponding constraints of prescribed outlet dam discharge and total inflow to Priest Lake

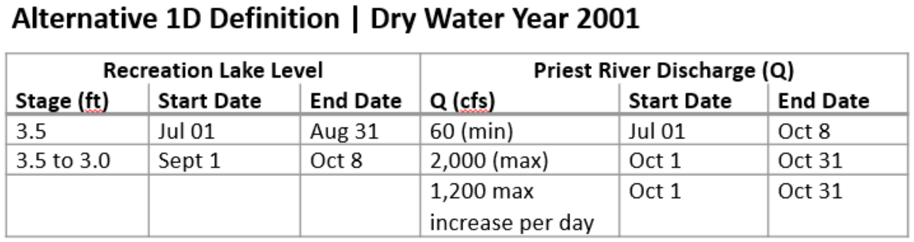
- Three existing conditions scenarios and six water management alternatives were modeled, each including a different combination of:
  - Lake level goal during the calendar year
  - Discharge constraints during the calendar year
  - Total calculated inflow to Priest Lake during the calendar year

- Simulations were completed at a daily time step using data from dry years (2001, 2005) and a normal year (2002)

Key to Simulation Results for Each Alternative

- There are three slides summarizing the results of each alternative
  1. Alternative definition and tables of results

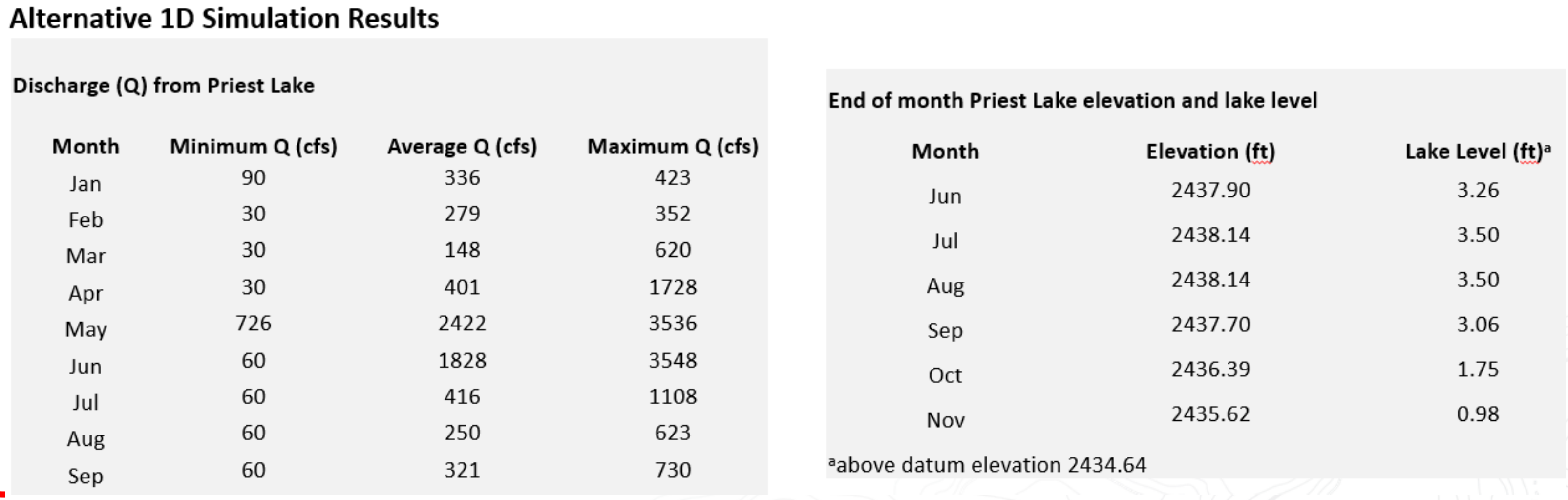
Water management alternative definition



Alternative 1D Definition | Dry Water Year 2001

Recreation Lake Level			Priest River Discharge (Q)		
Stage (ft)	**Start Date**	**End Date**	**Q (cfs)**	**Start Date**	**End Date**
3.5	Jul 01	Aug 31	60 (min)	Jul 01	Oct 8
3.5 to 3.0	Sept 1	Oct 8	2,000 (max)	Oct 1	Oct 31
			1,200 max increase per day	Oct 1	Oct 31

Discharge and lake level simulation results



Alternative 1D Simulation Results

Discharge (Q) from Priest Lake

Month	Minimum Q (cfs)	Average Q (cfs)	Maximum Q (cfs)
Jan	90	336	423
Feb	30	279	352
Mar	30	148	620
Apr	30	401	1728
May	726	2422	3536
Jun	60	1828	3548
Jul	60	416	1108
Aug	60	250	623
Sep	60	321	730

End of month Priest Lake elevation and lake level

Month	Elevation (ft)	Lake Level (ft)[a]
Jun	2437.90	3.26
Jul	2438.14	3.50
Aug	2438.14	3.50
Sep	2437.70	3.06
Oct	2436.39	1.75
Nov	2435.62	0.98

[a] above datum elevation 2434.64

2. Time series plot of lake level and discharge results
3. Time series plot of lake level results and inflow

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Existing Condition D1 Definition | Dry Water Year 2001

Recreation Lake Level			Priest River Discharge (Q)		
Stage (ft)	**Start Date**	**End Date**	**Q (cfs)**	**Start Date**	**End Date**
3.0	Jul 01	Oct 08	60 (min)	Jan 01	Dec 31

Existing Condition D1 Simulation Results

Discharge (Q) from Priest Lake

Month	Minimum Q (cfs)	Average Q (cfs)	Maximum Q (cfs)
Jan	295	316	426
Feb	60	262	345
Mar	60	115	239
Apr	60	510	1660
May	1420	2265	2901
Jun	901	1778	2830
Jul	60	468	2345
Aug	60	149	475
Sep	60	66	229
Oct	60	759	1650
Nov	1010	1253	1699
Dec	704	922	1119

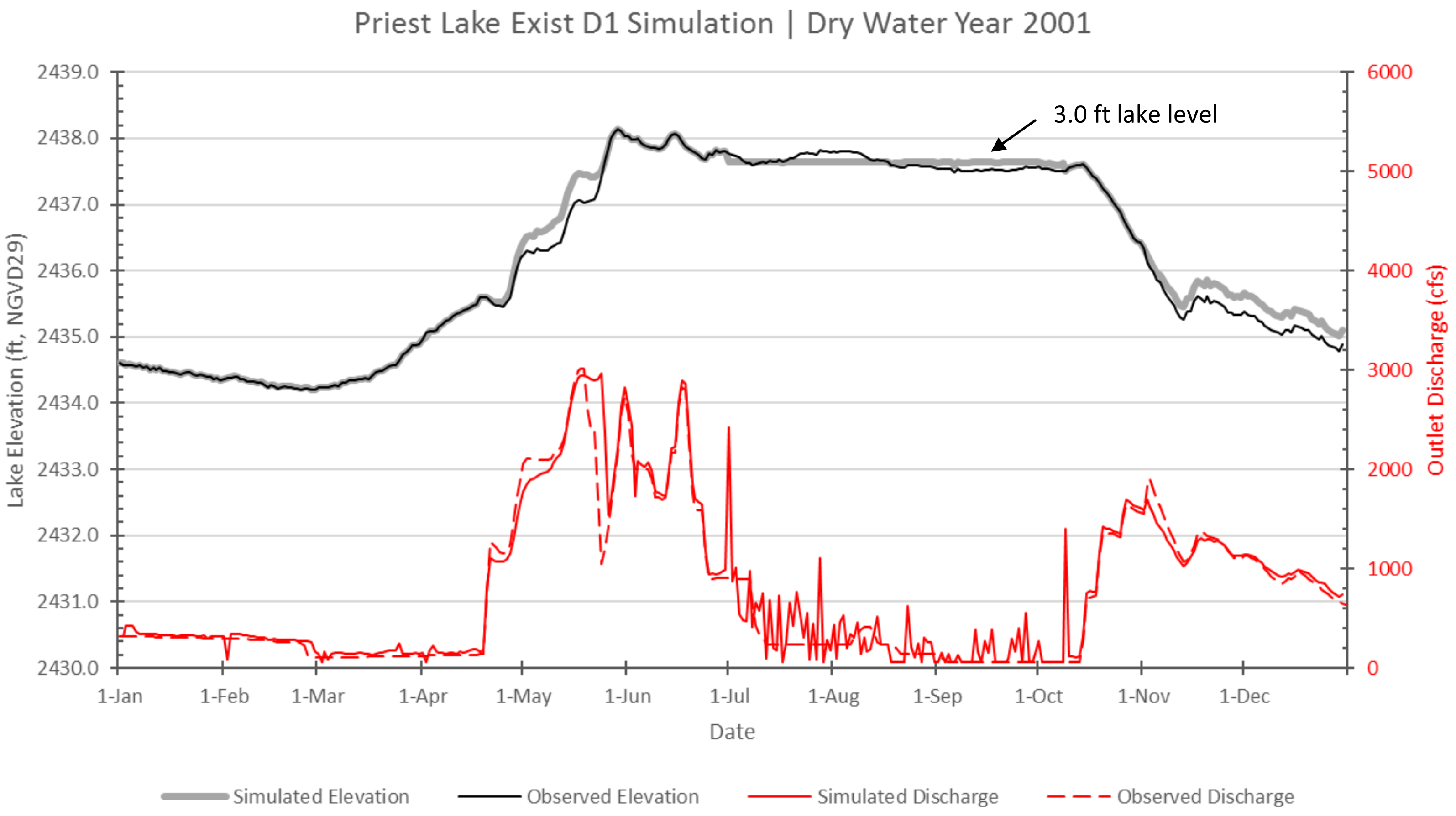
End of month Priest Lake elevation and lake level

Month	Elevation (ft)	Lake Level (ft)[a]
Jun	2437.81	3.17
Jul	2437.64	3.00
Aug	2437.63	2.99
Sep	2437.63	2.99
Oct	2436.43	1.79
Nov	2435.57	0.93

[a] above datum elevation 2434.64

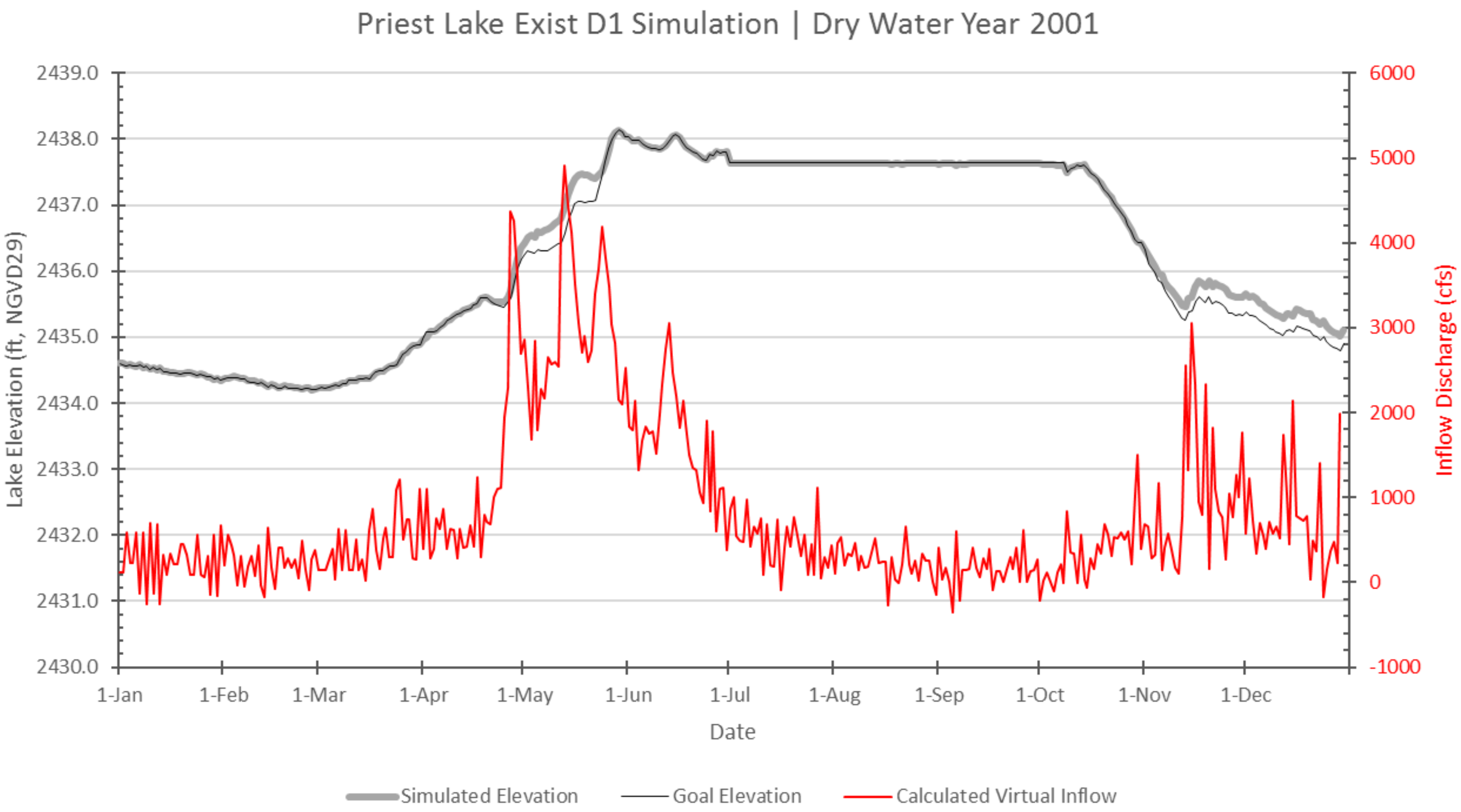
Existing Condition D1 Simulation Key Findings

- Recreation season lake level goal elevation can be achieved
- Discharge constraints are met
- May and Nov – Dec simulated elevation is higher than observed elevation because of differences in dam operations (simulated vs. observed), increasing lake inflow and constraints on outlet discharge



Existing Condition D1 Simulation Key Findings

- Differences in dam operations (simulated vs. observed), and large, rapid changes in calculated inflow result in short-term disparity between simulated elevation and observed elevation



Existing Condition D2 Definition | Dry Water Year 2005

Recreation Lake Level			Priest River Discharge (Q)		
Stage (ft)	**Start Date**	**End Date**	**Q (cfs)**	**Start Date**	**End Date**
3.0	Jul 01	Oct 08	60 (min)	Jan 01	Dec 31

Existing Condition D2 Simulation Results

Discharge (Q) from Priest Lake

Month	Minimum Q (cfs)	Average Q (cfs)	Maximum Q (cfs)
Jan	658	790	956
Feb	808	919	996
Mar	763	863	1095
Apr	1114	1453	2197
May	1830	2358	2798
Jun	1380	1839	2590
Jul	135	849	3310
Aug	60	207	807
Sep	60	168	715
Oct	145	1322	3022
Nov	536	771	1020
Dec	134	281	591

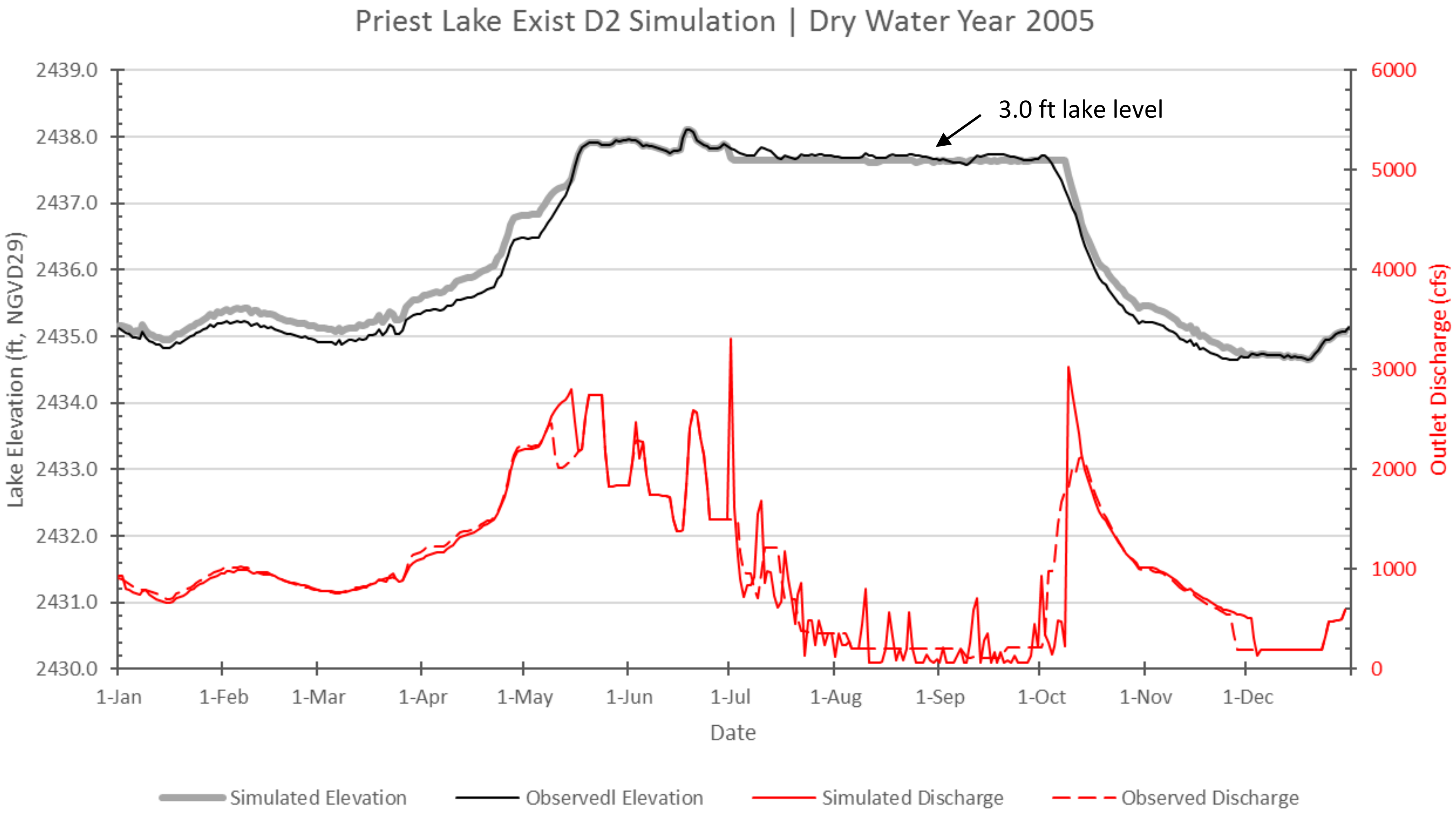
End of month Priest Lake elevation and lake level

Month	Elevation (ft)	Lake Level (ft)[a]
Jun	2437.86	3.22
Jul	2437.64	3.00
Aug	2437.64	3.00
Sep	2437.64	3.00
Oct	2435.46	0.82
Nov	2434.74	0.10

[a] above datum elevation 2434.64

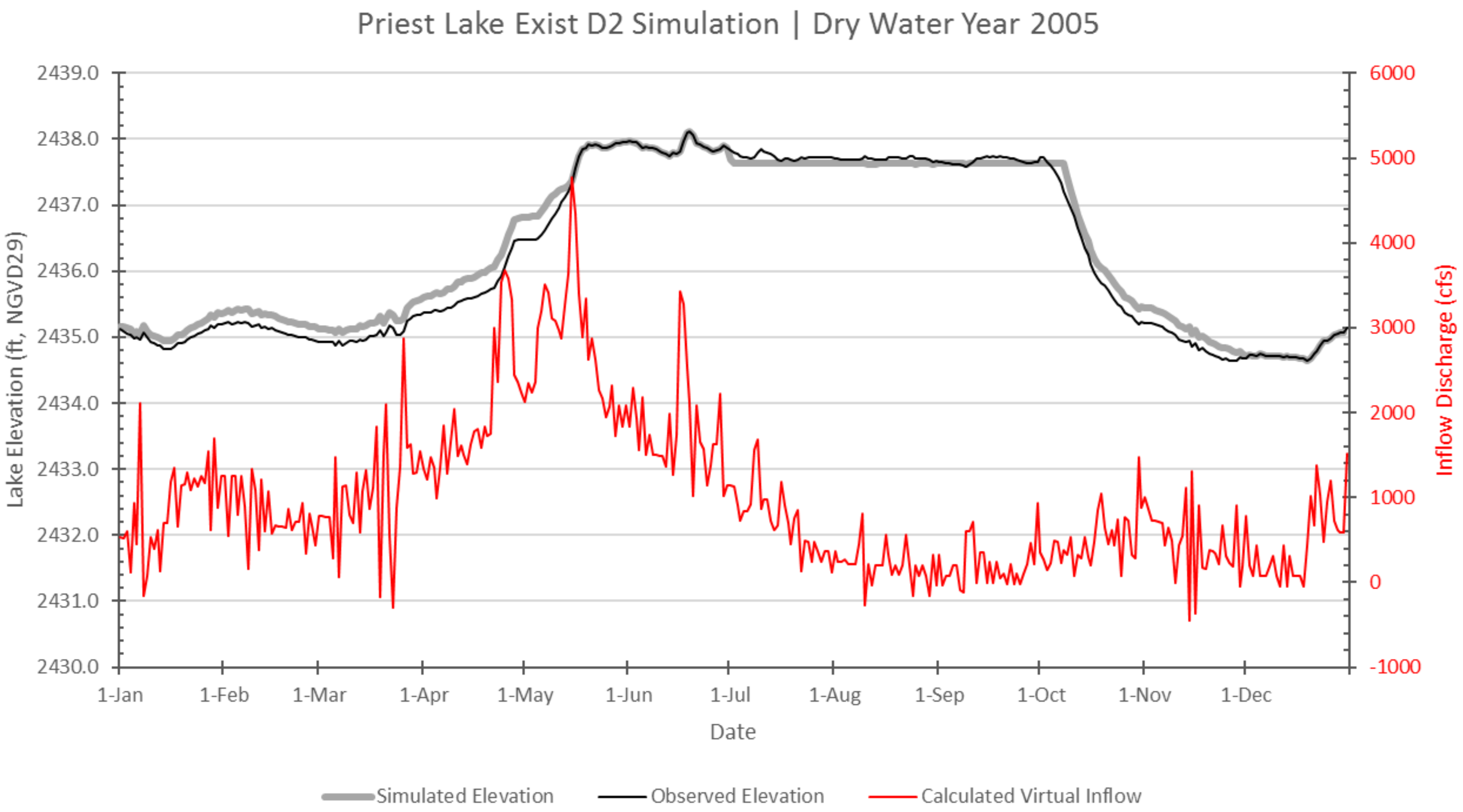
Existing Condition D2 Simulation Key Findings

- Recreation season lake level goal elevation can be achieved
- Discharge constraints are met
- Simulated elevation is sometimes slightly higher than observed elevation because of differences in dam operations (simulated vs. observed), increasing lake inflow and constraints on outlet discharge, and large, rapid changes in calculated inflow



Existing Condition D2 Simulation Key Findings

- Simulated elevation is sometimes slightly higher than observed elevation because of differences in dam operations (simulated vs. observed), increasing lake inflow and constraints on outlet discharge, and large, rapid changes in calculated inflow



Existing Condition N Definition | Normal Water Year 2002

Recreation Lake Level			Priest River Discharge (Q)		
Stage (ft)	**Start Date**	**End Date**	**Q (cfs)**	**Start Date**	**End Date**
3.0	Jul 01	Oct 08	60 (min)	Jan 01	Dec 31

Existing Condition N Simulation Results

Discharge (Q) from Priest Lake

Month	Minimum Q (cfs)	Average Q (cfs)	Maximum Q (cfs)
Jan	237	700	1180
Feb	763	892	1112
Mar	711	778	859
Apr	720	1756	2693
May	2730	3707	6193
Jun	2040	4637	6456
Jul	60	972	3339
Aug	60	195	466
Sep	60	91	442
Oct	60	1180	2991
Nov	201	385	736
Dec	89	555	886

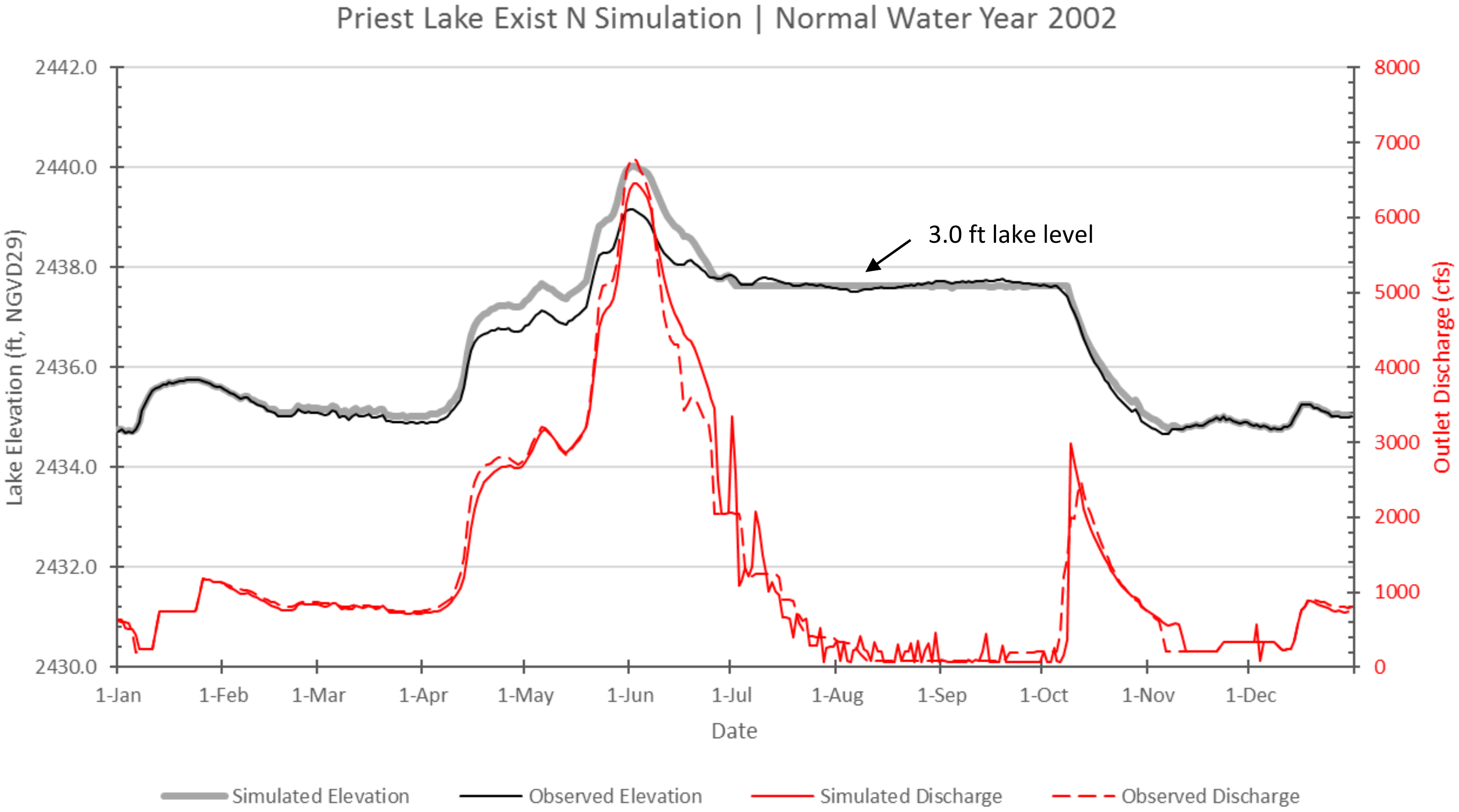
End of month Priest Lake elevation and lake level

Month	Elevation (ft)	Lake Level (ft)[a]
Jun	2437.85	3.21
Jul	2437.64	3.00
Aug	2437.63	2.99
Sep	2437.63	2.99
Oct	2435.07	0.43
Nov	2434.89	0.25

[a] above datum elevation 2434.64

Existing Condition N Simulation Key Findings

- Recreation season lake level goal elevation can be achieved
- Discharge constraints are met
- Simulated elevation is sometimes slightly higher than observed elevation because of differences in dam operations (simulated vs. observed), increasing lake inflow and constraints on outlet discharge, and large, rapid changes in calculated inflow



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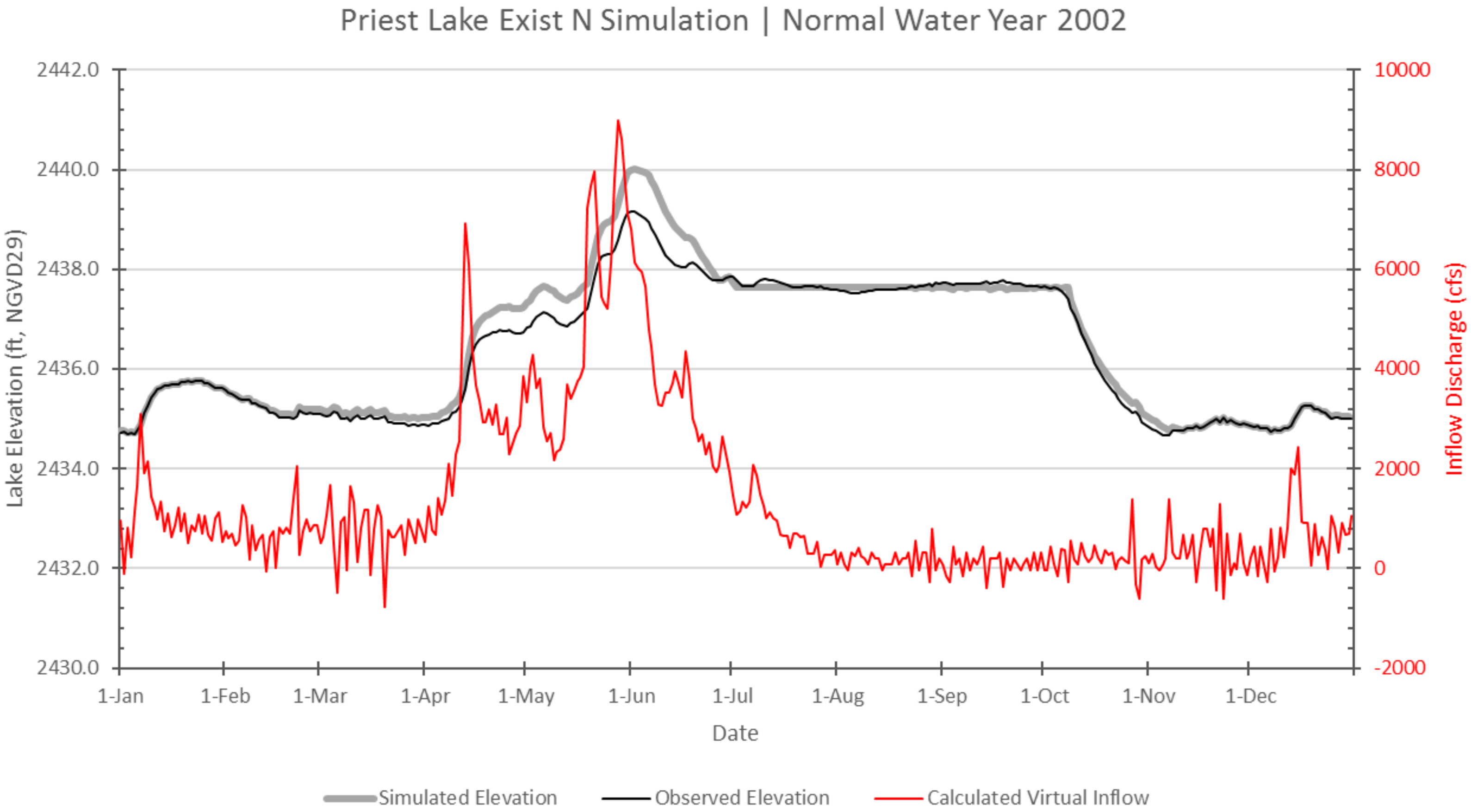
Existing Condition N Simulation Key Findings

- Simulated elevation is sometimes slightly higher than observed elevation because of differences in dam operations (simulated vs. observed), increasing lake inflow and constraints on outlet discharge, and large, rapid changes in calculated inflow



Alternative 1D Definition | Dry Water Year 2001

Recreation Lake Level			Priest River Discharge (Q)		
Stage (ft)	**Start Date**	**End Date**	**Q (cfs)**	**Start Date**	**End Date**
3.5	Jul 01	Aug 31	60 (min)	Jan 01	Dec 31
3.5 to 3.0	Sept 1	Oct 8	2,000 (max)	Oct 1	Oct 31
			1,200 max increase per day	Oct 1	Oct 31

Alternative 1D Simulation Results

Discharge (Q) from Priest Lake

Month	Minimum Q (cfs)	Average Q (cfs)	Maximum Q (cfs)
Jan	295	316	426
Feb	60	262	345
Mar	60	115	239
Apr	60	510	1660
May	1420	2265	2901
Jun	901	1778	2830
Jul	60	275	1006
Aug	60	149	475
Sep	60	234	662
Oct	60	789	1650
Nov	1010	1253	1699
Dec	704	922	1119

End of month Priest Lake elevation and lake level

Month	Elevation (ft)	Lake Level (ft)[a]
Jun	2437.81	3.17
Jul	2438.14	3.50
Aug	2438.13	3.49
Sep	2437.71	3.07
Oct	2436.43	1.79
Nov	2435.57	0.93

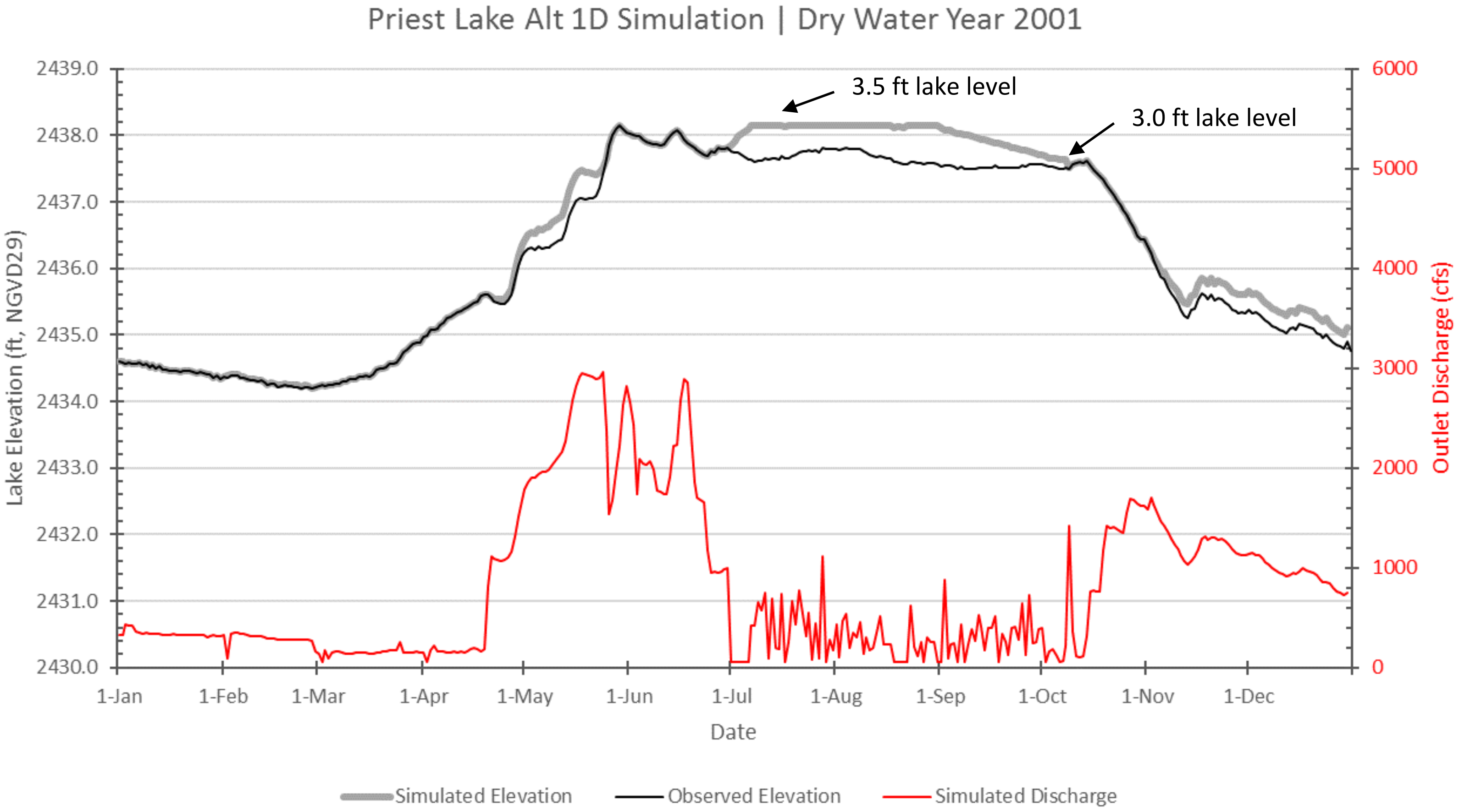
[a] above datum elevation 2434.64

Alternative 1D Simulation Key Findings

- Recreation season lake level goal elevation can be achieved

- Discharge constraints are met

- May and Nov – Dec simulated elevation is higher than observed elevation because of differences in dam operations (simulated vs. observed), increasing lake inflow and constraints on outlet discharge



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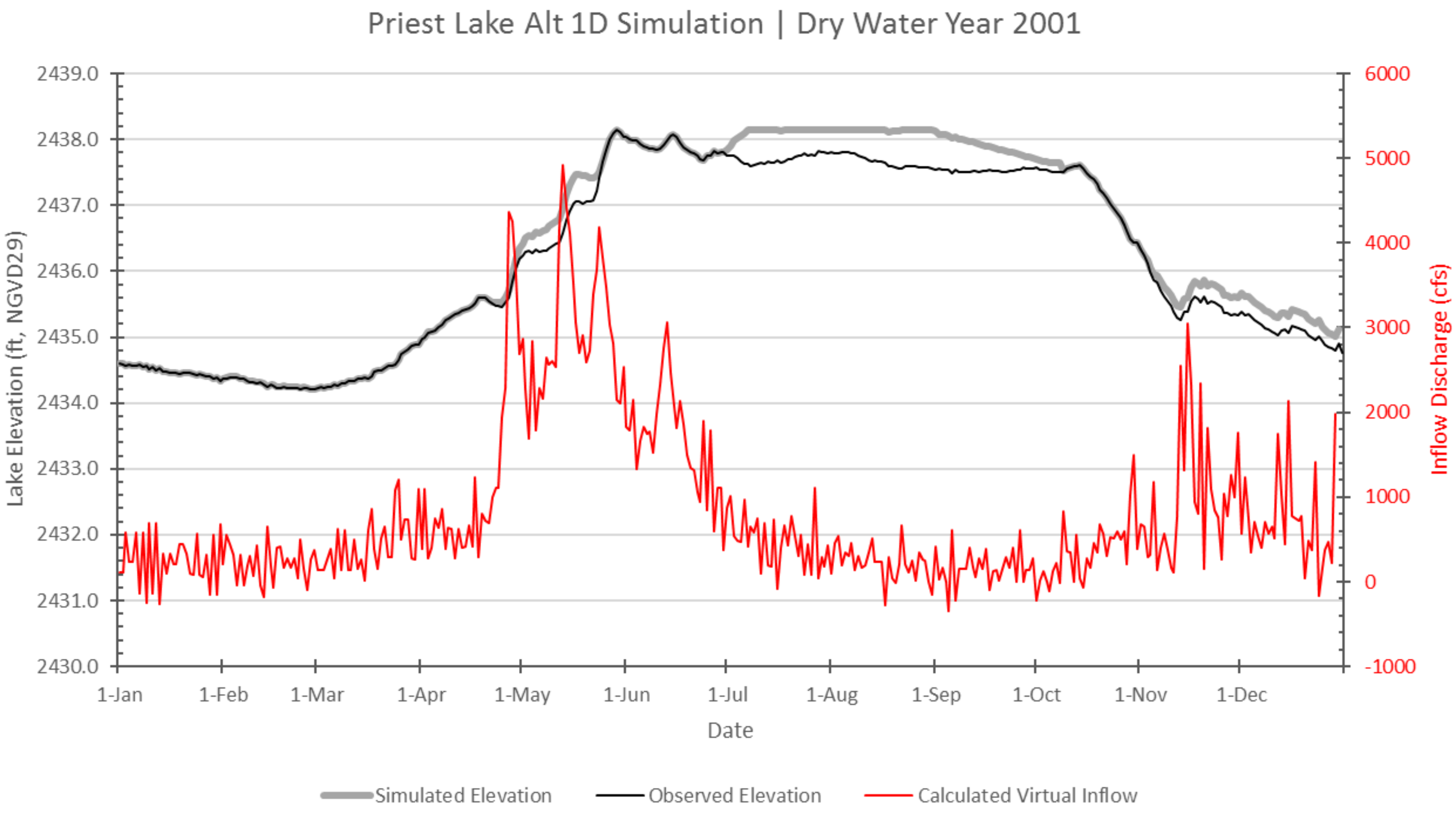
Alternative 1D Simulation Key Findings

- Differences in dam operations (simulated vs. observed), and large, rapid changes in calculated inflow result in short-term disparity between simulated elevation and observed elevation



Alternative 2D Definition | Dry Water Year 2005

Recreation Lake Level			Priest River Discharge (Q)		
Stage (ft)	**Start Date**	**End Date**	**Q (cfs)**	**Start Date**	**End Date**
3.5	Jul 01	Aug 31	60 (min)	Jan 01	Dec 31
3.5 to 3.0	Sept 1	Oct 8	2,000 (max)	Oct 1	Oct 31
			1,200 max increase per day	Oct 1	Oct 31

Alternative 2D Simulation Results

Discharge (Q) from Priest Lake

Month	Minimum Q (cfs)	Average Q (cfs)	Maximum Q (cfs)
Jan	658	790	956
Feb	808	919	996
Mar	763	863	1095
Apr	1114	1453	2197
May	1830	2358	2798
Jun	1380	1839	2590
Jul	60	656	1681
Aug	60	207	807
Sep	60	340	939
Oct	205	1313	2000
Nov	548	802	1084
Dec	134	288	591

End of month Priest Lake elevation and lake level

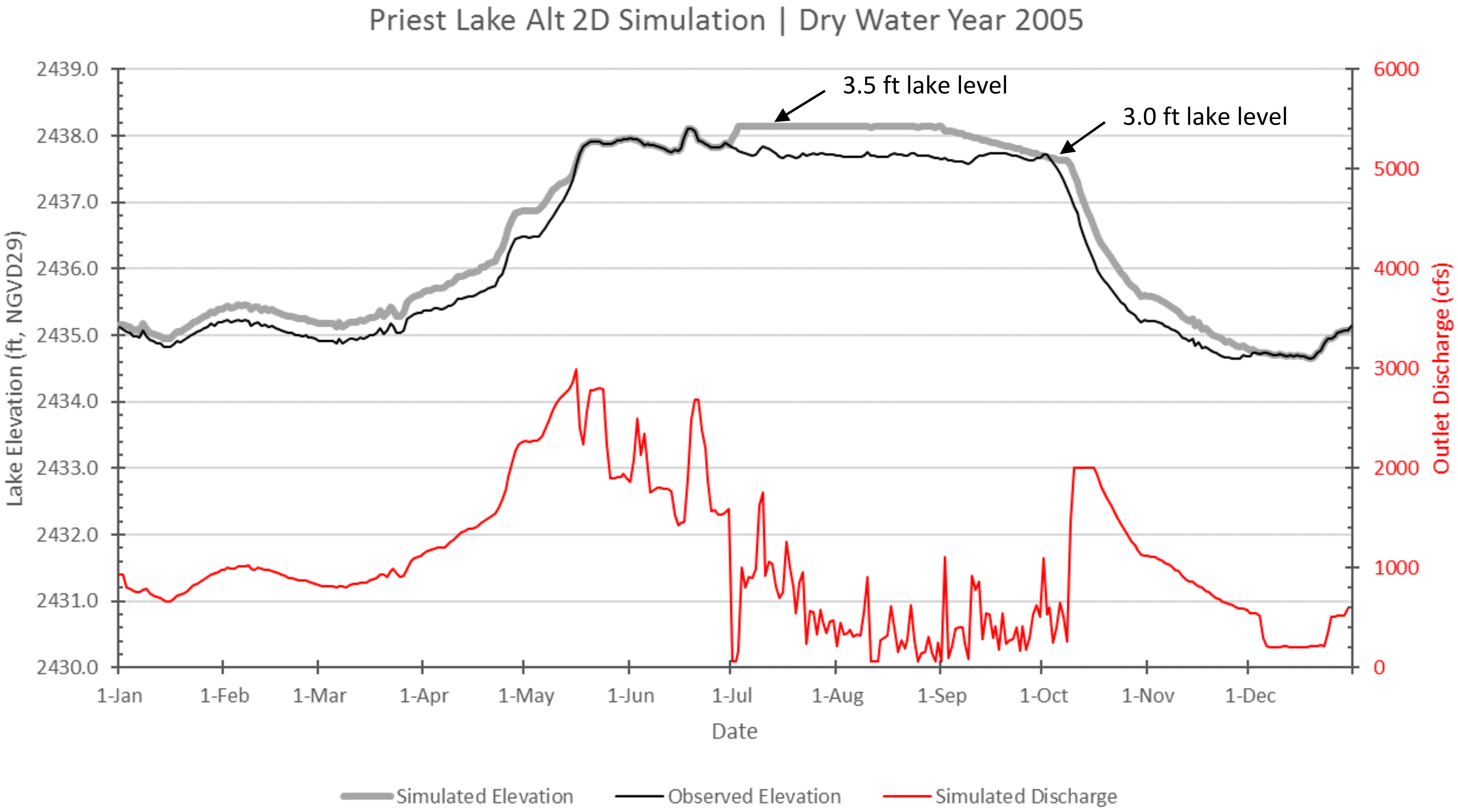
Month	Elevation (ft)	Lake Level (ft)[a]
Jun	2437.86	3.22
Jul	2438.14	3.50
Aug	2438.14	3.50
Sep	2437.71	3.07
Oct	2435.55	0.91
Nov	2434.75	0.11

[a] above datum elevation 2434.64

GeoEngineers

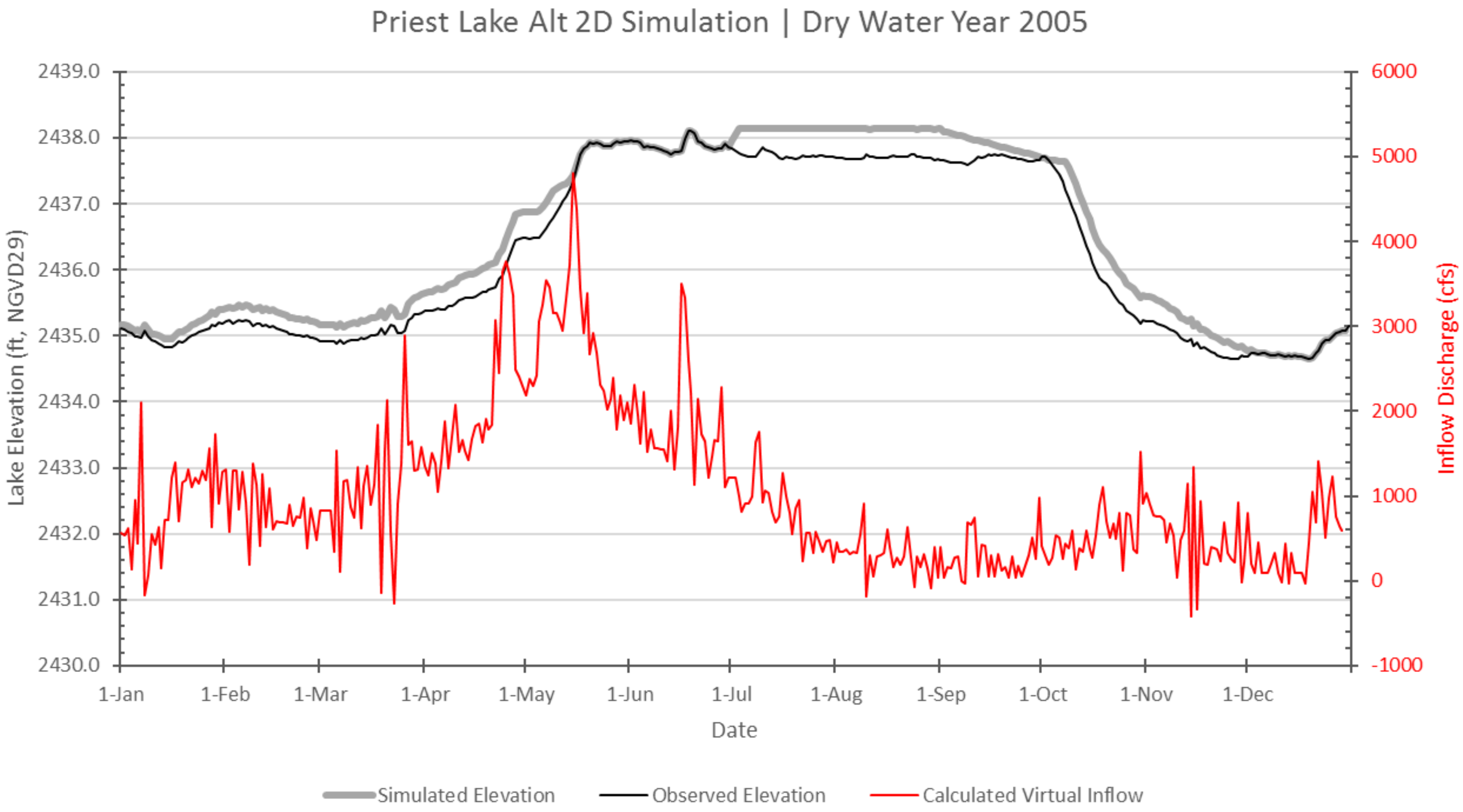
Alternative 2D Simulation Key Findings

- Recreation season lake level goal elevation can be achieved
- Discharge constraints are met
- Feb – May and Nov – Dec simulated elevation is higher than observed elevation because of differences in dam operations (simulated vs. observed), increasing lake inflow and constraints on outlet discharge



Alternative 2D Simulation Key Findings

- Differences in dam operations (simulated vs. observed), and large, rapid changes in calculated inflow result in short-term disparity between simulated elevation and observed elevation



Alternative 2N Definition | Normal Water Year 2002

Recreation Lake Level			Priest River Discharge (Q)		
Stage (ft)	**Start Date**	**End Date**	**Q (cfs)**	**Start Date**	**End Date**
3.5	Jul 01	Aug 31	60 (min)	Jan 01	Dec 31
3.5 to 3.0	Sept 1	Oct 8	2,000 (max)	Oct 1	Oct 31
			1,200 max increase per day	Oct 1	Oct 31

Alternative 2N Simulation Results

Discharge (Q) from Priest Lake

Month	Minimum Q (cfs)	Average Q (cfs)	Maximum Q (cfs)
Jan	237	700	1180
Feb	763	892	1112
Mar	711	778	859
Apr	720	1756	2693
May	2730	3707	6193
Jun	2040	4637	6456
Jul	60	778	2080
Aug	60	195	466
Sep	60	259	802
Oct	60	1178	2000
Nov	201	419	795
Dec	89	555	886

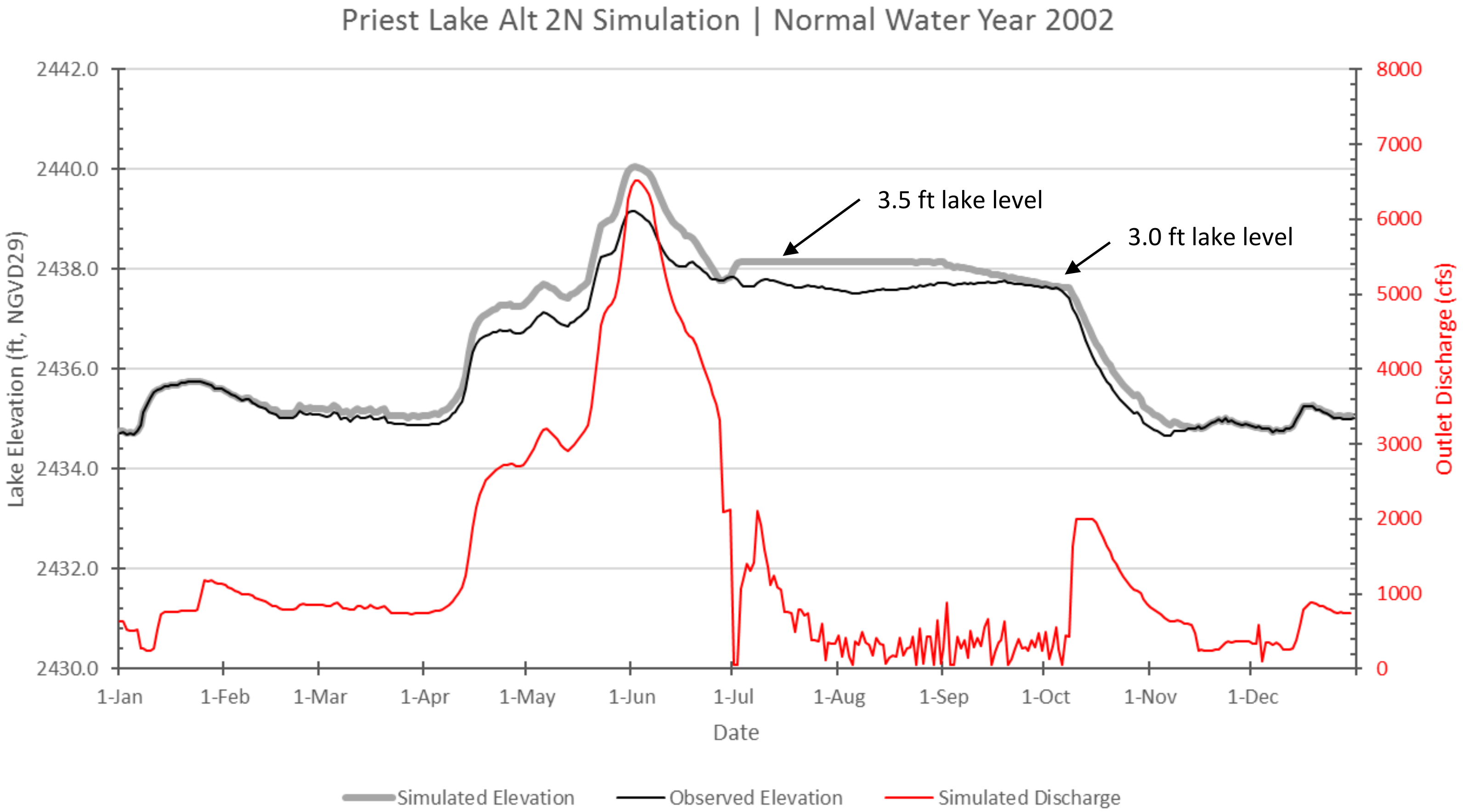
End of month Priest Lake elevation and lake level

Month	Elevation (ft)	Lake Level (ft)[a]
Jun	2437.85	3.21
Jul	2438.14	3.50
Aug	2438.13	3.49
Sep	2437.71	3.07
Oct	2435.15	0.51
Nov	2434.89	0.25

[a] above datum elevation 2434.64

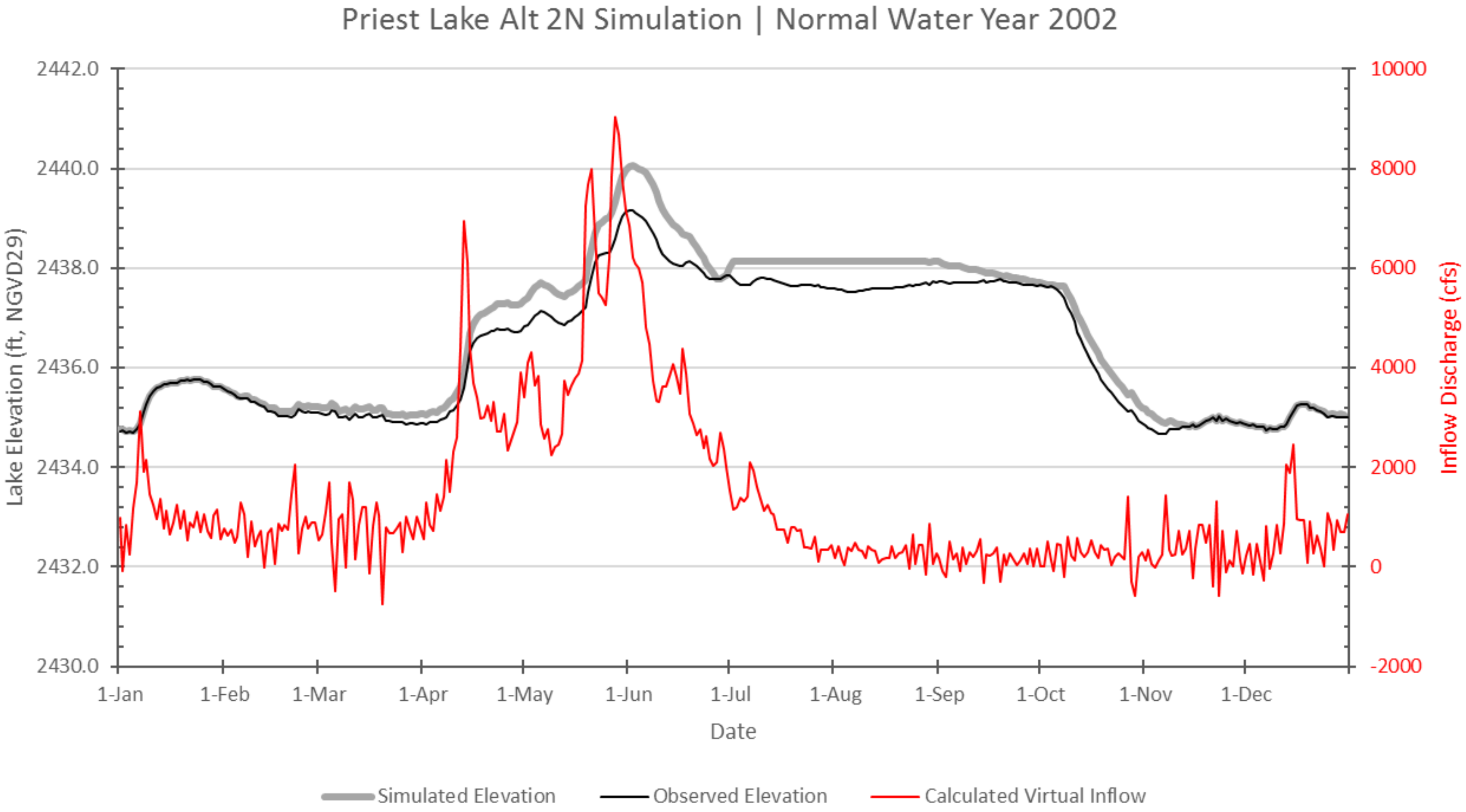
Alternative 2N Simulation Key Findings

- Recreation season lake level goal elevation can be achieved
- Discharge constraints are met
- Feb – May and Nov – Dec simulated elevation is higher than observed elevation because of differences in dam operations (simulated vs. observed), increasing lake inflow and constraints on outlet discharge



Alternative 2N Simulation Key Findings

- Differences in dam operations (simulated vs. observed), and large, rapid changes in calculated inflow result in short-term disparity between simulated elevation and observed elevation



Alternative 3D Definition | Dry Water Year 2001

Recreation Lake Level			Priest River Discharge (Q)		
Stage (ft)	**Start Date**	**End Date**	**Q (cfs)**	**Start Date**	**End Date**
3.25	Jul 01	Aug 31	60 (min)	Jan 01	Dec 31
3.25 to 2.75	Sept 1	Sept 30	2,000 (max)	Oct 1	Oct 31
			1,200 max increase per day	Oct 1	Oct 31

Alternative 3D Simulation Results

Discharge (Q) from Priest Lake

Month	Minimum Q (cfs)	Average Q (cfs)	Maximum Q (cfs)
Jan	295	316	426
Feb	60	262	345
Mar	60	115	239
Apr	60	510	1660
May	1420	2265	2901
Jun	901	1778	2830
Jul	60	371	1006
Aug	60	149	475
Sep	60	250	662
Oct	60	653	1940
Nov	1042	1272	1781
Dec	706	926	1128

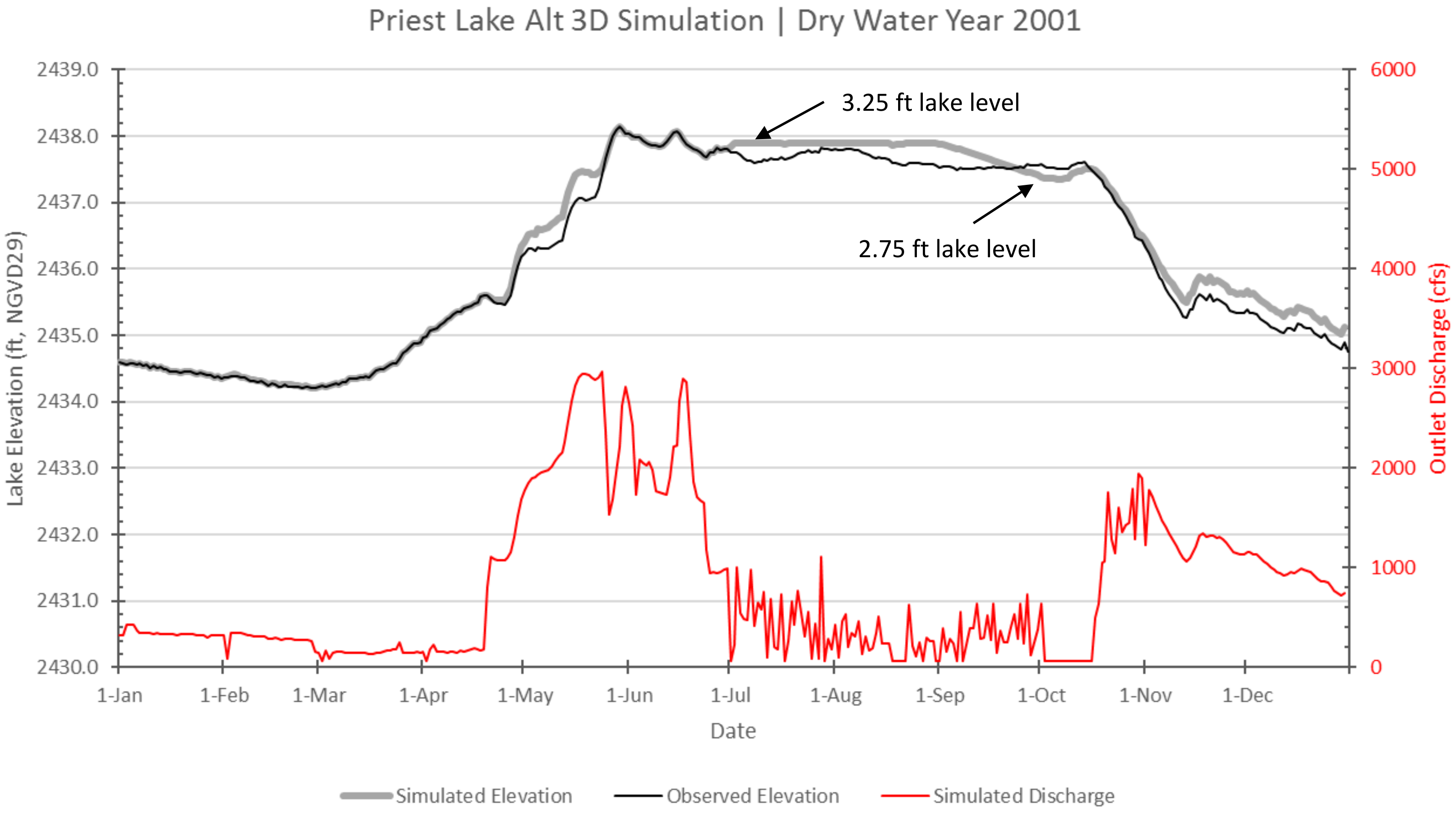
End of month Priest Lake elevation and lake level

Month	Elevation (ft)	Lake Level (ft)[a]
Jun	2437.81	3.17
Jul	2437.89	3.25
Aug	2437.88	3.24
Sep	2437.42	2.78
Oct	2436.49	1.85
Nov	2435.58	0.94

[a] above datum elevation 2434.64

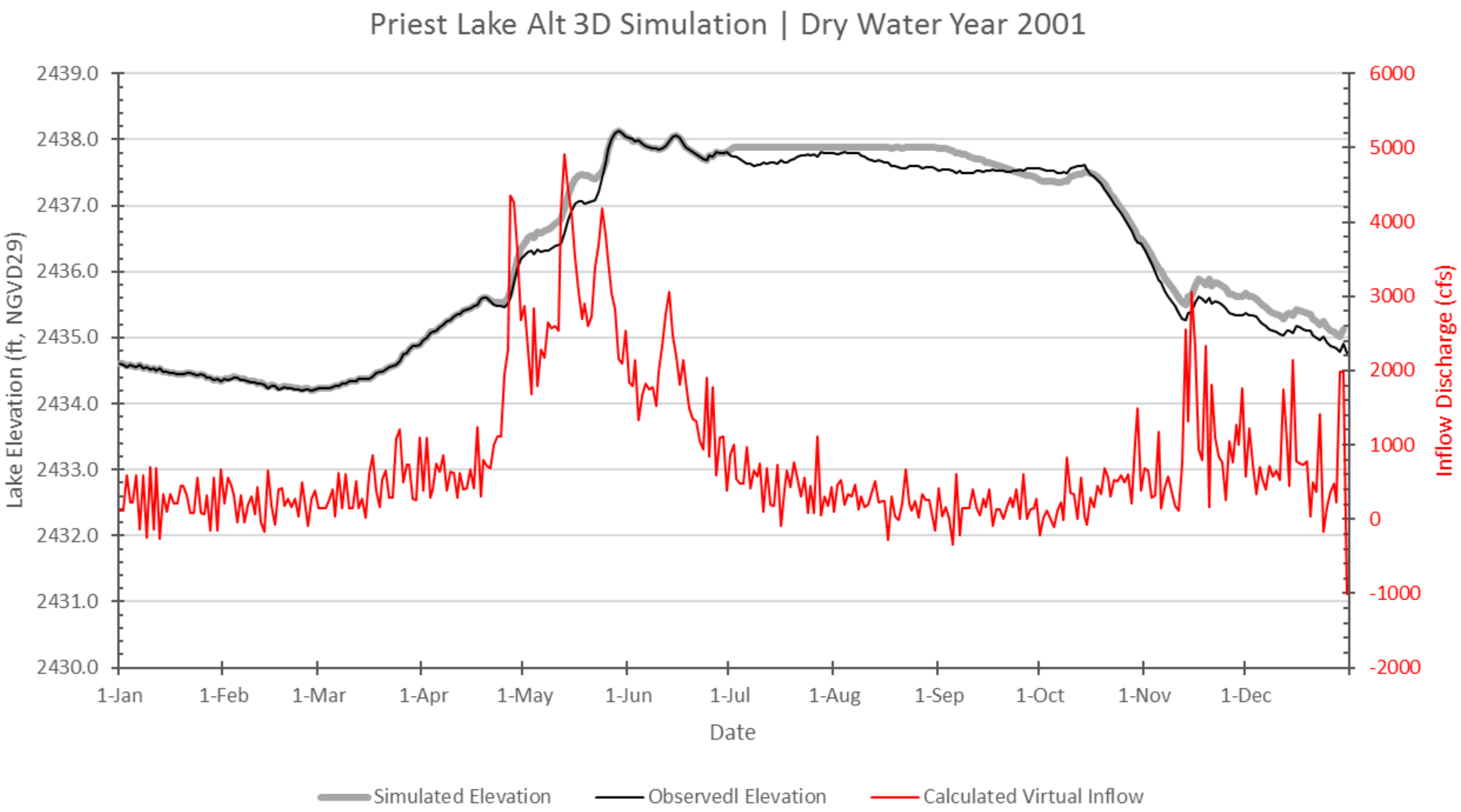
Alternative 3D Simulation Key Findings

- Recreation season lake level goal elevation can be achieved
- Discharge constraints are met
- May and Nov – Dec simulated elevation is higher than observed elevation because of differences in dam operations (simulated vs. observed), increasing lake inflow and constraints on outlet discharge



Alternative 3D Simulation Key Findings

- Differences in dam operations (simulated vs. observed), and large, rapid changes in calculated inflow result in short-term disparity between simulated elevation and observed elevation



GeoEngineers

Alternative 4D Definition | Dry Water Year 2001

Recreation Lake Level			Priest River Discharge (Q)		
Stage (ft)	**Start Date**	**End Date**	**Q (cfs)**	**Start Date**	**End Date**
3.5	Jul 01	Aug 15	60 (min)	Jan 01	Dec 31
3.5 to 3.0	Aug 16	Sept 15	2,000 (max)	Oct 1	Oct 31
3.0 to 2.75	Sept 16	Oct 8	1,200 max increase per day	Oct 1	Oct 31

Alternative 4D Simulation Results

Discharge (Q) from Priest Lake

Month	Minimum Q (cfs)	Average Q (cfs)	Maximum Q (cfs)
Jan	295	316	426
Feb	60	262	345
Mar	60	115	239
Apr	60	510	1660
May	1420	2265	2901
Jun	901	1778	2830
Jul	60	275	1006
Aug	60	261	745
Sep	60	235	783
Oct	60	677	1650
Nov	1010	1253	1699
Dec	704	922	1119

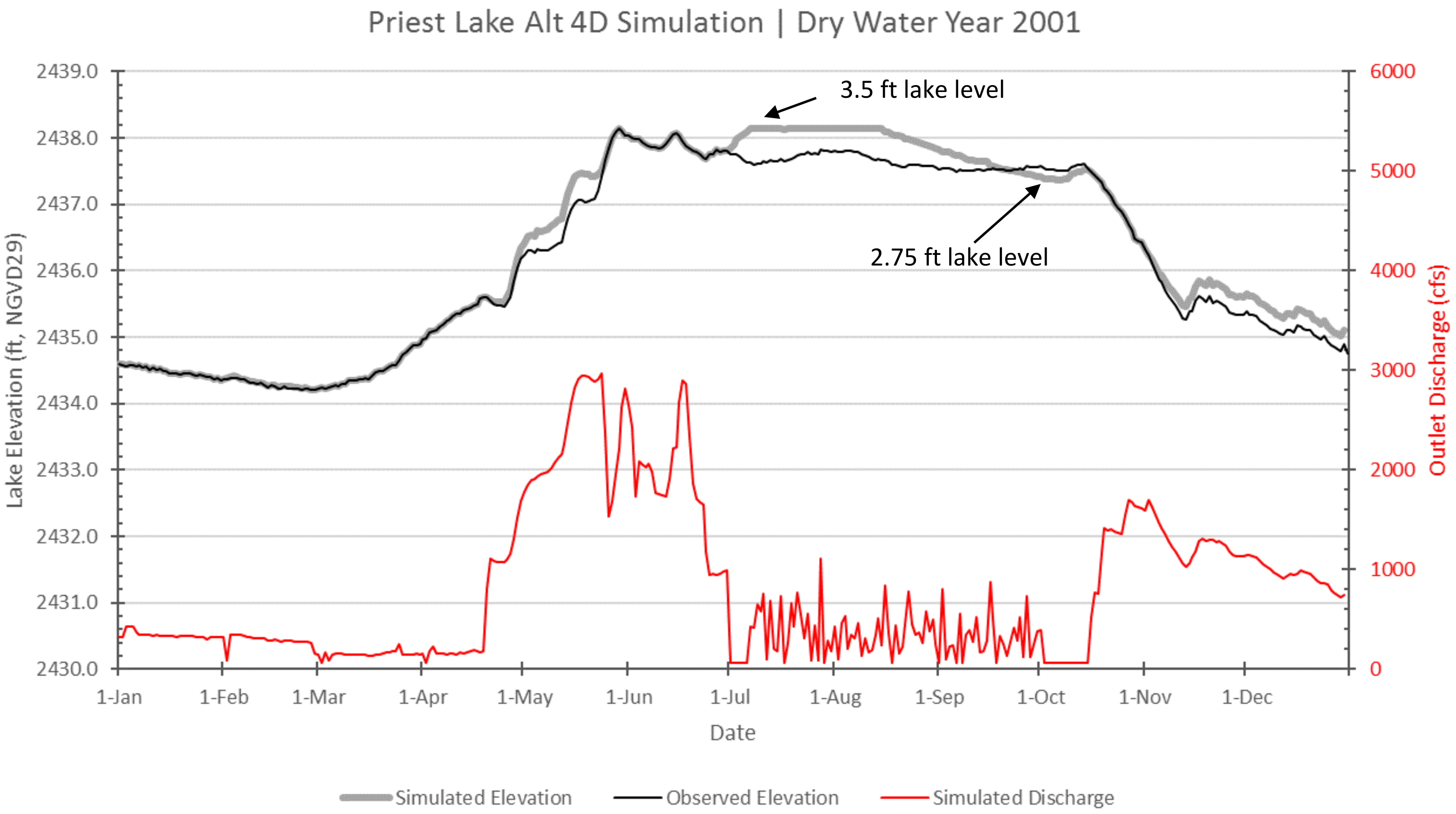
End of month Priest Lake elevation and lake level

Month	Elevation (ft)	Lake Level (ft)[a]
Jun	2437.81	3.17
Jul	2438.14	3.50
Aug	2437.84	3.20
Sep	2437.42	2.78
Oct	2436.43	1.79
Nov	2435.57	0.93

[a]above datum elevation 2434.64

Alternative 4D Simulation Key Findings

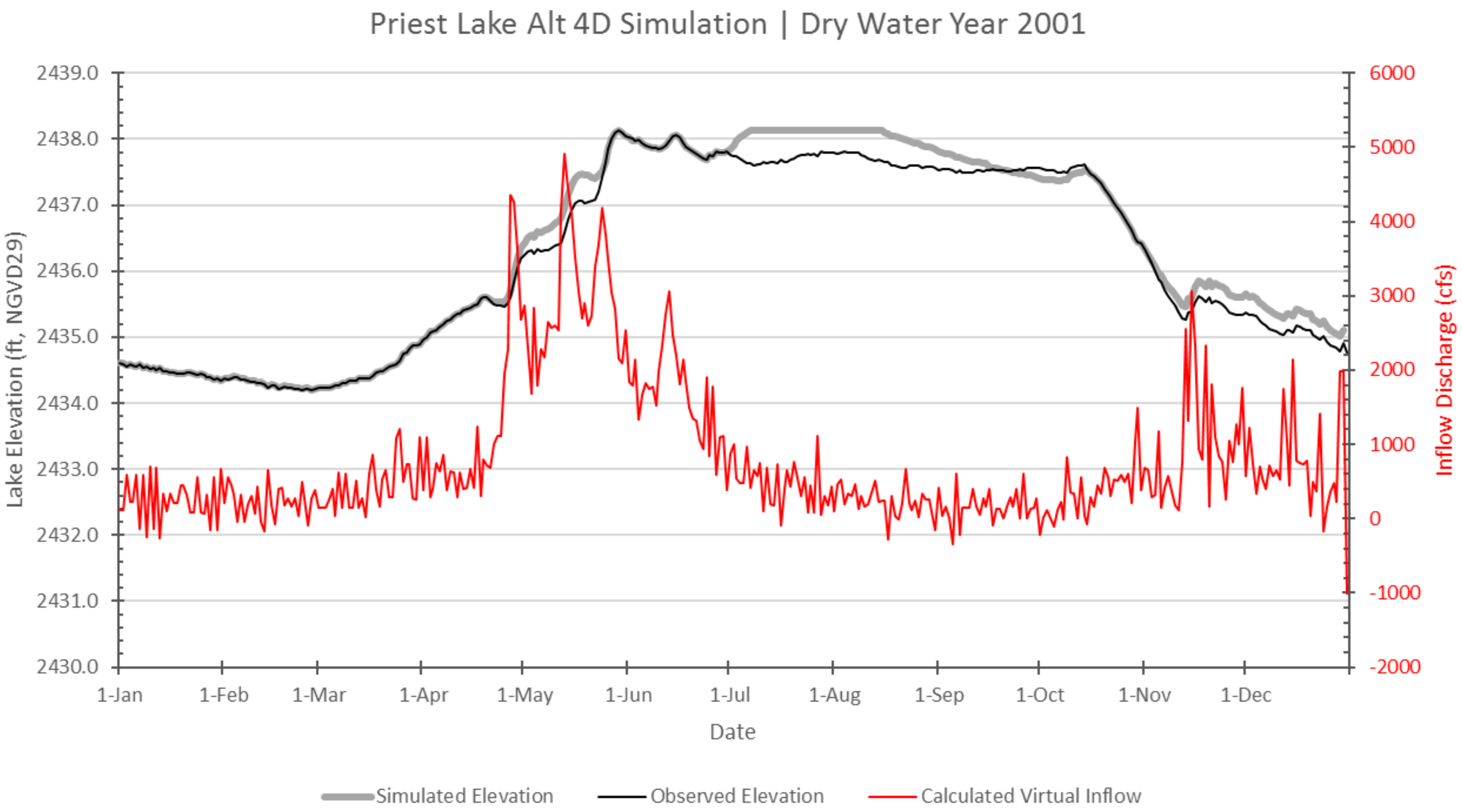
- Recreation season lake level goal elevation can be achieved
- Discharge constraints are met
- May and Nov – Dec simulated elevation is higher than observed elevation because of differences in dam operations (simulated vs. observed), increasing lake inflow and constraints on outlet discharge



GeoEngineers

Alternative 4D Simulation Key Findings

- Differences in dam operations (simulated vs. observed), and large, rapid changes in calculated inflow result in short-term disparity between simulated elevation and observed elevation



GeoEngineers

Alternative 5D Definition | Dry Water Year 2001

Recreation Lake Level			Priest River Discharge (Q)		
Stage (ft)	**Start Date**	**End Date**	**Q (cfs)**	**Start Date**	**End Date**
3.25	Jul 01	Aug 31	60 (min)	Jan 01	Dec 31
3.25 to 3.0	Sept 1	Oct 8	2,000 (max)	Oct 1	Oct 31
			1,200 max increase per day	Oct 1	Oct 31

Alternative 5D Simulation Results

Discharge (Q) from Priest Lake

Month	Minimum Q (cfs)	Average Q (cfs)	Maximum Q (cfs)
Jan	295	316	426
Feb	60	262	345
Mar	60	115	239
Apr	60	510	1660
May	1420	2265	2901
Jun	901	1778	2830
Jul	60	371	1006
Aug	60	149	475
Sep	60	154	628
Oct	60	770	1650
Nov	1010	1253	1699
Dec	704	922	1119

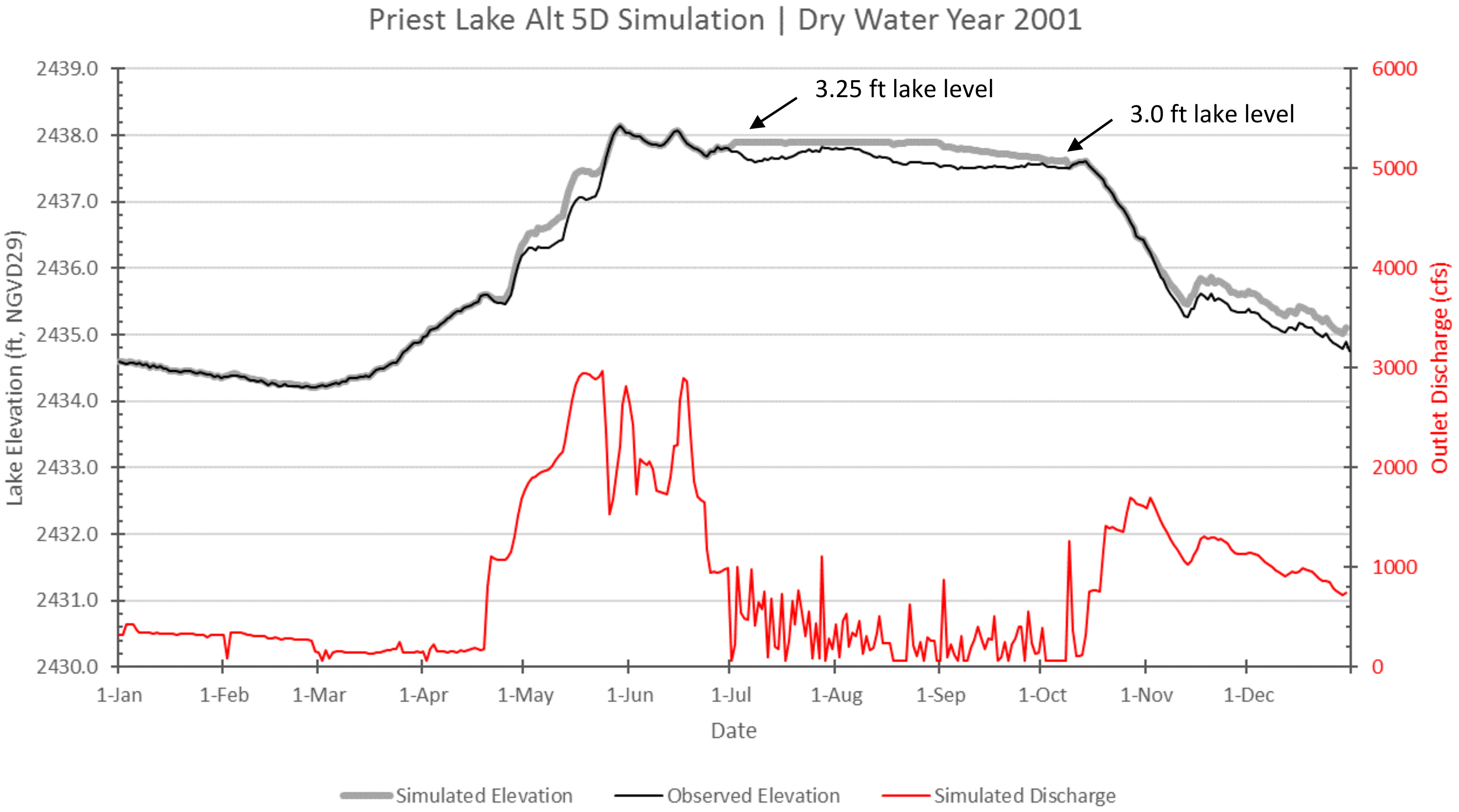
End of month Priest Lake elevation and lake level

Month	Elevation (ft)	Lake Level (ft)[a]
Jun	2437.81	3.17
Jul	2437.89	3.25
Aug	2437.88	3.24
Sep	2437.66	3.02
Oct	2436.43	1.79
Nov	2435.57	0.93

[a] above datum elevation 2434.64

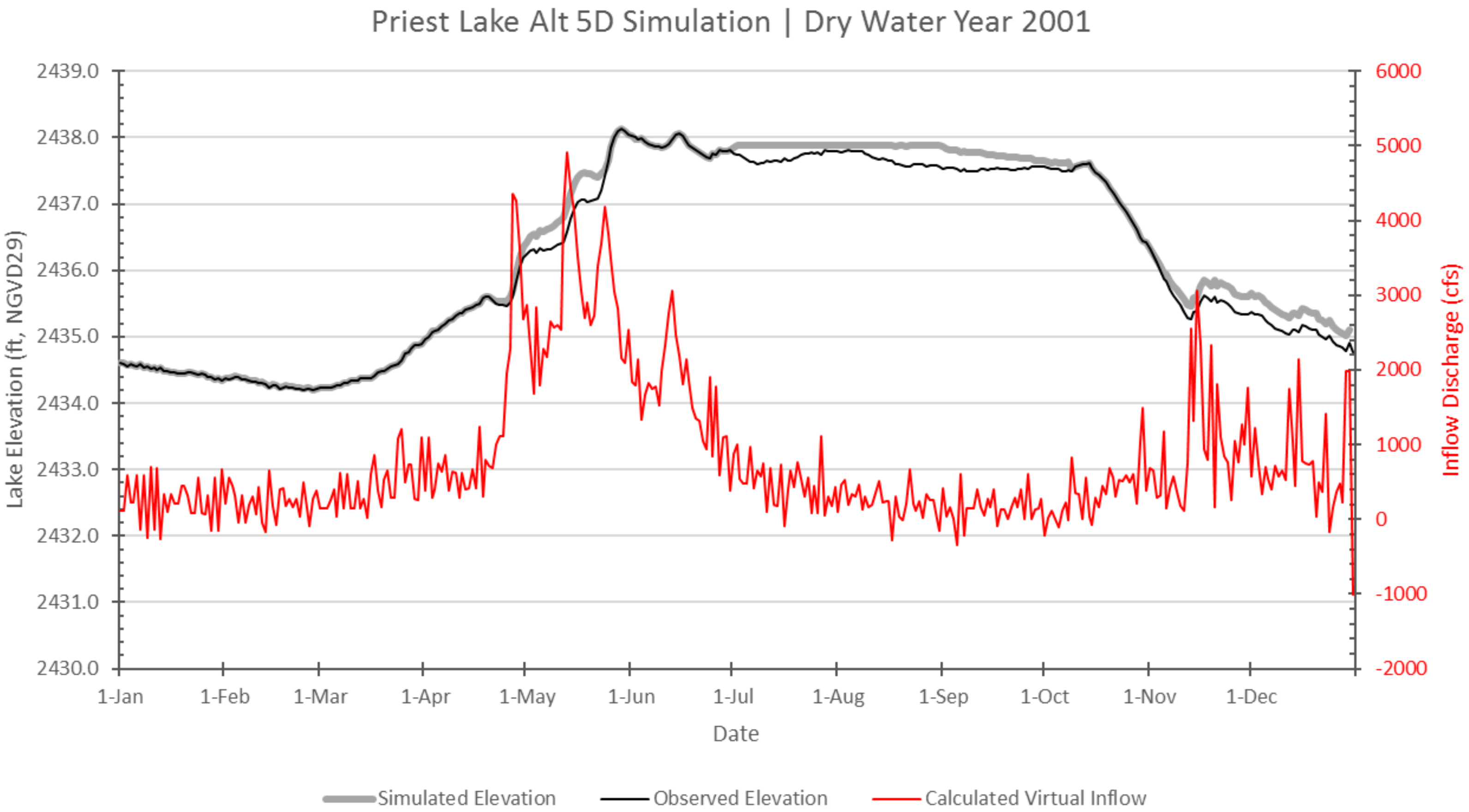
Alternative 5D Simulation Key Findings

- Recreation season lake level goal elevation can be achieved
- Discharge constraints are met
- May and Nov – Dec simulated elevation is higher than observed elevation because of differences in dam operations (simulated vs. observed), increasing lake inflow and constraints on outlet discharge



Alternative 5D Simulation Key Findings

- Differences in dam operations (simulated vs. observed), and large, rapid changes in calculated inflow result in short-term disparity between simulated elevation and observed elevation



C. Outlet Dam Assessment

C.1 Outlet Dam Structure Hydraulic and Gate Assessment

C.2 Outlet Dam Structure Stability Analysis

C.3 Outlet Dam Structure Assessment Summary

C.1 Outlet Dam Structure Hydraulic and Gate Assessment

Priest Lake Water Management Study

Hydraulic and Gate Assessment

February 20, 2018

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Priest Lake Water Management Study

Hydraulic and Gate Assessment

February 20, 2018

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A. Hydraulic Calculations 19

1 Introduction

Priest Lake is located in the northern panhandle of Idaho State in the United States of America. The need for a recreational lake was identified in the 1940's and a barrage control structure was built in 1951 to provide water level control during the summer months. In 1978, the original barrage structure was replaced due to severe deterioration with a new outlet structure located further downstream. Figure 1 shows the current outlet structure, looking upstream.

Figure 1: Photo of Priest Lake Outlet Dam looking Upstream

Source: Mott MacDonald, taken in May 2017

In March 2017, the Idaho Water Resource Board appointed Mott MacDonald to undertake a water resources assessment of Priest Lake to consider the following elements:

1. Evaluation of alternatives for maintaining required recreational lake levels and maintaining current minimum discharge downstream of the Priest Lake Outlet Dam (60 cfs).
2. Assessing potential structural and operational modifications to the dam.
3. Analysing options to improve access and navigable conditions for the Priest Lake Thorofare.
   Following this appointment, the Mott MacDonald team based in Cambridge, UK was commissioned to investigate the following and to include conclusions and recommendations with respect to potential future works to mitigate any identified risks:
   a. Review the impact of raising the water level in the lake from +3.0 ft on USGS gage 12393000 as per State Statute 70-507 by 6 inches in the summer months for recreational use of the lake including several water sports and fishing;

b. Review the current and future operational regime and potential improvements to reduce risk of scour downstream from the dam.

1.1 Scope of this Report

As described above the scope of this report is an outline level engineering assessment of items a and b. As such this report considers the following elements:

- Outlet Structure Hydraulics Assessment
  - Review design documentation and existing data, including available design documentation and operation data records;
  - Review existing data and develop an opinion of current gate discharge capacity;
  - Evaluate hydraulic conditions for gate operations to determine susceptibility to hydraulic jump and downstream scour;
  - Evaluate potential for modifications to reduce risk of downstream scour.
  - Describe a range of alternatives that reduce risk of scour and minimize formation of hydraulic jump.
- Outlet Structure – Gate Assessment
  - Review design documentation and existing data;
  - Evaluate existing gate ability to resist additional 6” of water level;

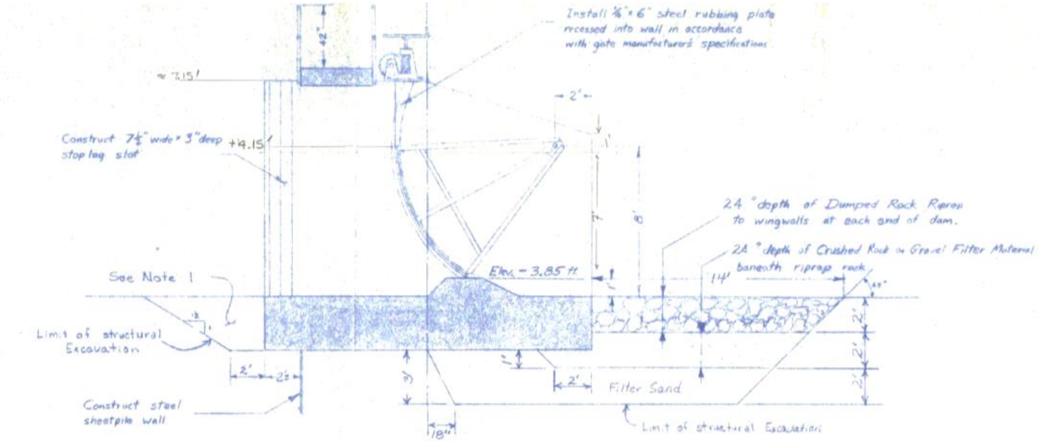
In particular, the potential risk for significant erosion immediately downstream of the outlet structure is considered for both the current and future operational regime of the gates in conjunction with the current provision of downstream channel protection. No consideration of the structure’s existing condition has been included in this study.

1.2 Background

The Priest Lake Dam was originally built in 1951, but following severe deterioration was rebuilt further downstream in 1978. The new barrage structure consists of eleven equally sized bays, with ten piers, including an additional one at each abutment. Each bay houses a tainter (radial) gate operated by a manual hoist on the adjacent pier. The handwheels have been removed and replaced with a nut to allow the use of a handheld electric powered driver to raise and lower the gates when required.

There is a 1 ft high concrete sill at the base of the structure, and the tainter gates are 7 ft high, giving a total static hydraulic head of 8 ft. There is a concrete walkway across the top of ~~running along~~ the structure, allowing pedestrian access to operate each of the tainter gates individually. This is done by using a handheld powered drive.

Figure 2: Section of Priest Lake Outlet Dam

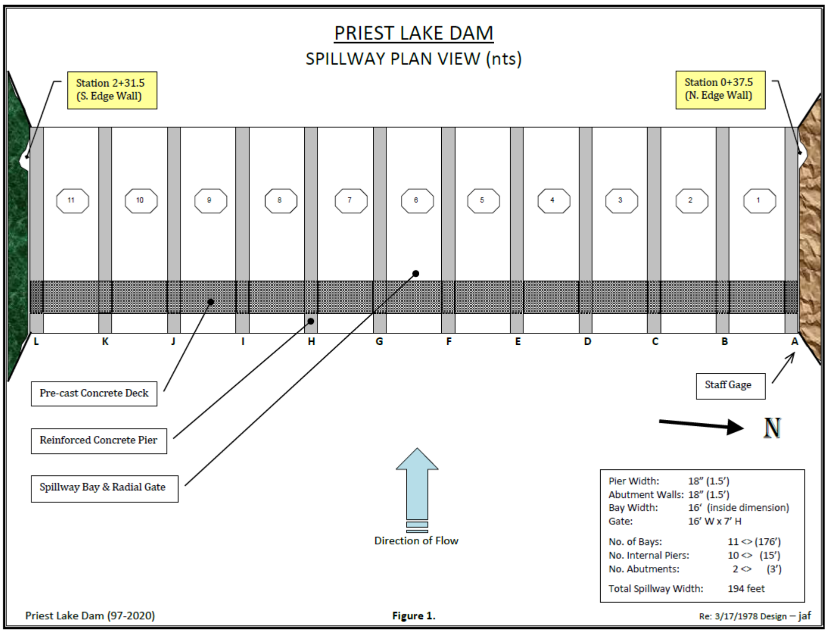


Source: Taken from original construction drawings (YEAR of drawings?)

Following the construction of the barrage in 1978, severe erosion of the concrete apron occurred during initial releases in 1979. This was attributed to poor design and improper gate operation. Following the incident, the thickness of the downstream stone blanket was increased from 2 ft to 3 ft. Additional riprap was also placed downstream from both abutments for 10 or 20 ft.

Following a state dam safety ~~an~~ inspection in 2015, it was noted that minor scour/erosion has occurred in the riprap downstream from the concrete apron. In particular, areas below bay numbers 5-7 in Figure 3 appeared to exhibit the deepest scour holes, approximately 3.5 ft below the concrete surface, and are likely to have progressed with time due to the way the dam is operated.

Figure 3: Plan diagram of Priest Lake Outlet Dam (not to scale)



Pier Width: 18" (1.5')
Abutment Walls: 18" (1.5')
Bay Width: 16' (inside dimension)
Gate: 16' W x 7' H

No. of Bays: 11 <> (176')
No. Internal Piers: 10 <> (15')
No. Abutments: 2 <> (3')

Total Spillway Width: 194 feet

Source: Dam Safety Inspection Report Dated 08/25/2015

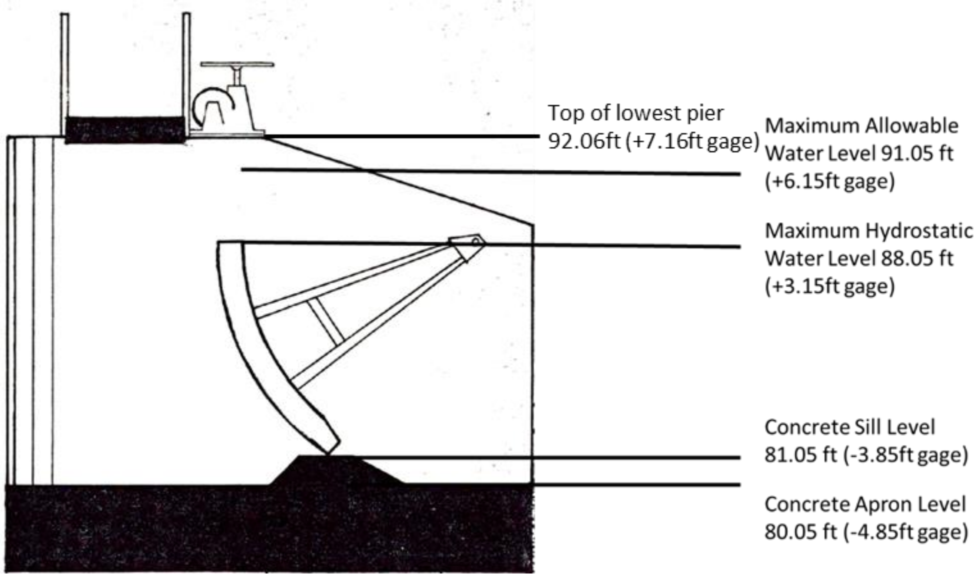
1.3 Documentation Reviewed

The following documents were reviewed as part of this study:

- Selection of photos and videos – taken by Mott MacDonald staff in May 2017
- Calculations
  - Original calculations by Idaho Department of Water Resources (IDWR), 11/22/1977
  - Comments on dam stability by Corps of Engineers, Walla Walla District, 12/05/1977
  - Revised hydraulic calculations, 03/14/1978
  - CH2M Hill review of design, 03/17/1978
  - Design Modifications Memorandums, 03/17/2017
- Construction Drawings
  - Set of three drawings for the Waterman Model T-1 Overflow Type Tainter Gate
  - Set of five drawings showing the location, plan and profile, structural detail, edge walls and miscellaneous detail
  - A document including 'Standard Drawings', showing fence, gates, rubbing plate for side seal, sill, frost control modifications and sheet pile cut-off wall
- Documentation from Scour Apron Repairs in 1979
  - Outlet Structure Cross Section
  - Riprap Sizing – Note from Mike Stubblefield, (IDWR), dated 05/05/1978
  - Letter from Dave LePard, (IDWR), following supervision of the repairs
- IDWR Dam Safety Inspection Reports
  - Gate Inspection 04/20/1979 – following the dropping of Gate 3 during construction
  - Report Dated 02/13/2013
  - Form Dated 08/25/2015
- Dam Operation Logs, kept by the dam operator (Karl Duncan)
  - 2011
  - 2012
  - 2015
  - 2016
  - 2017 (until 05/20/2017)

IDWR performed a level survey for the structure was carried out during the 2015 inspection. The levels taken from this have been used as part of this assessment and are summarised in Figure 4, with absolute and USGS gage levels given. The maximum allowable water level is from the original calculations and is taken as the design level for the structure.

Figure 4: Levels of structure



Source: Outlet Structure Cross Section, with information from IDWR 97-2020-(8-25-2015)-Insp Report.

2 Documentation Review

2.1 Design

Based on inspection of the design calculations, the structure appears to have been designed for an event with a 0.2% annual probability of exceedance (i.e., 1 in 500-year return period, Q_{500}) with an assumed outflow of 11,200 cfs. The original design was reviewed by both the US Army Corps, Walla Walla District and CH2M Hill. Both reviews concluded with a range of concerns which included:

- Inconsistencies within hydrology
- Reliance on sheet pile cutoff to resist sliding as well as inadequate stability factors such as overturning and bearing capacity
- Potential issue with piping as possibility for short-circuiting of seepage around the ends of the dam
- Lack of stilling basin and inadequate erosion protection below (downstream) of the dam
- Further investigation of scour potential under various operating conditions and procedures

In general, the comments made are thought to be reasonable. In particular, it is highly unusual to rely on the sheet piling for sliding resistance, particularly as little modeling appears to have been done to consider the sheet induced shear stresses within the sheet pile in such a case. Furthermore, short-circuiting of seepage around walls can often be a problem if adequate cut-off is not provided.

It is believed that these concerns were addressed by IDWR prior to construction as seen in the Design Modifications Memorandums dates 17/03/1978, although it should be noted in particular that the whole structure was founded 0.15 ft higher than the original calculations.

2.2 Overall Construction

An initial assessment of the cross section of the barrage found that no formal stilling basin was included as part of the design. This implies that scour potential could be high due to lack of tail water level under certain operational scenarios. Furthermore, the lack of a formal stilling basin could result in the formation of a hydraulic jump on the riprap rather than on top of the concrete apron, thereby exacerbating the potential for significant scour.

It is expected that any such scour could then result in either decreasing the overturning resistance at the downstream toe leading to structure movements or, more likely, cause cracking and failure of the downstream concrete apron. This aligns with the comment in the Inspection Report (08/25/2015) noting that bays 5-7 exhibit the deepest scour holes, approximately 3.5 ft below the concrete surface.

Finally, a sheet pile wall has been installed as a cutoff, below the concrete sill for the purposes of limiting seepage under the structure. It is more common for low head structures such as this to install a sheet pile wall along the toe of the structure to provide additional protection against undermining, and with a further upstream line of sheet piles to alleviate uplift forces on the structure and limit seepage.

2.3 Operation of Structure

Complete records for operation of the structure were available from 2015 until May 2017. All other records from 2011 and 2012 were found to be incomplete and therefore not able to provide useful information for the purposes of this study. It should be noted that the scale used to measure gate opening is hand-drawn onto the concrete piers, marked with three-inch increments. It is understood that this is unconfirmed by survey, and that the operational data can therefore only be taken as indicative.

These records were interrogated to establish the operational regime and are summarised in Table 1 and in Figure 5. It can be see that for approximately 63% of the time, all eleven gates are operational, primarily in the winter months and between one and three gates are operational for approximately 23% of the time during the summer months.

Table 1: Gate Operation Statistics

Number of Gates	Average Gate Opening (inches)			Number of Days	% of Days
	Maximum	Minimum	Average		
1	0.82	0.27	0.38	85	9.76%
2	2.18	0.55	0.93	24	2.76%
3	2.73	0.82	1.62	78	8.96%
4	1.09	1.09	1.09	3	0.34%
5	3.27	1.36	2.58	30	3.44%
6	3.82	3.00	3.36	23	2.64%
7	4.91	3.82	4.75	12	1.38%
8	-	-	-	0	0.00%
9	24.00	4.91	11.53	71	8.15%
10	-	-	-	0	0.00%
11	104.36	6.00	51.18	545	62.57%

Source: Dam Operation Logs 2015-2017

Indicative flow rates were calculated based upon lake level and average gate opening. The results of this exercise are summarised in Table 2. It is noted that during the summer releases of less than 60 cfs appear to occur 0% of the time and releases of less than 200 cfs appear to occur for approximately 10% of the time.

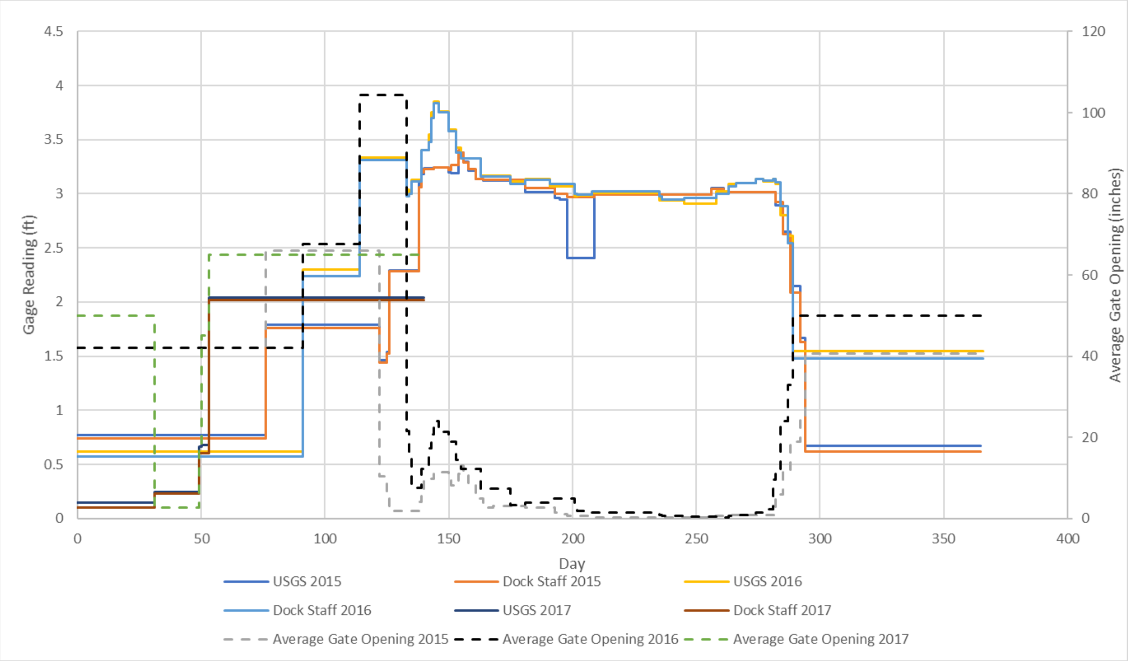
Table 2: Summary of Calculated Flow Rates

Number of Gates	Flow Rate (cfs)		
	Maximum	Minimum	Average
1	170	56	78
2	448	112	192
3	443	168	296
4	223	223	223
5	669	279	526
6	779	612	684
7	991	769	957
8	-	-	-
9	4517	992	2228
10	-	-	-
11	7961	520	3666

Source: MM calculated based upon Dam Operation Logs 2015-2017

Furthermore, from Figure 5, a clear seasonal trend can be seen based upon gage levels and average gate opening. During the winter months, gates are typically open, with gage readings between 1.5-2 ft. In the summer, the gage reading is commonly at 3 ft or above, with a much smaller gate opening to allow the lake level to be maintained. The transition between these two periods appears to occur within a regular 2-4-week interval, at the end of April and in mid-October. During this period water levels appear to be around 2.3 ft – 2.5 ft. Water levels 3.2 ft appear to have been recorded during 2017, however have been assumed to be unusual. The readings from May 2017 are much higher than seen previously given that all gates are fully open (66 inches) and have been assumed to be anomalous.

Figure 5: Plot of Gage Readings and Average Gate Opening with time



Source: Dam Operation Logs 2015-2017

3 Calculations

Please note that calculations have been included in Appendix A and were calculated using Microsoft Excel 2016.

3.1 Operational Scenarios

Four operational scenarios were considered for the hydraulic assessment. The scenarios selected were the original design flow (Q_{500}), the existing summer case with 1 gate open to the maximum recorded opening in that scenario (and therefore the worst case) as well as a new summer case with the pool raised by 6 inches. In addition, a winter scenario was considered, using all 11 gates with a typical gate opening and pool level as based upon the operational data. The four cases are summarised in Table 3.

Table 3: Operational scenarios considered

Scenario	Gage level	Upstream Water Level	Number of Gates Open	Gate Opening	Assumed Q
Q_{500} Design Flow	6.15 ft	90.9 ft	11	84 inches	11,200 cfs
Summer	3 ft	87.9 ft	1	9 inches	250 cfs
Winter	2.3 ft	87.2 ft	11	60 inches	7,570 cfs
Summer - New	3.5 ft	88.4 ft	1	9 inches	260 cfs

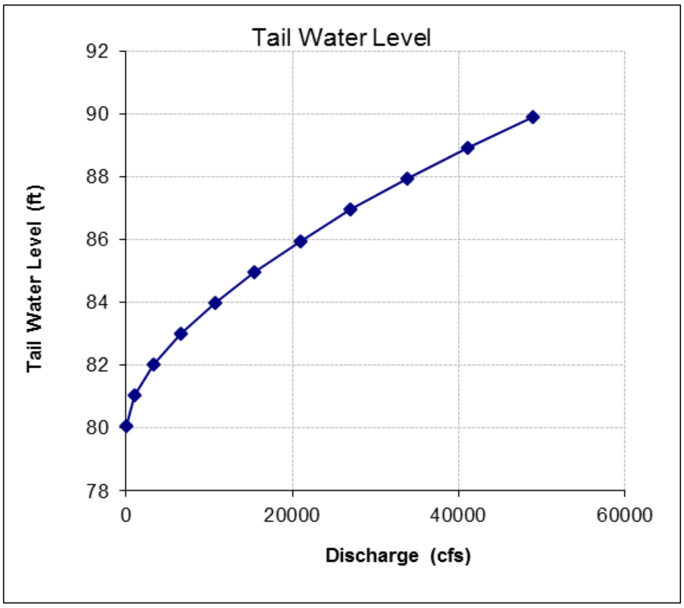
Source: Dam Operation Logs 2015-2017

3.2 Methodology – Hydraulic Assessment

A downstream rating curve was produced to estimate the tail water level versus discharge relationship for the structure. The rating curve developed was based on that used in the design calculations (Revised hydraulic calculations, IDWR, 03/14/1978) which assumed a trapezoidal channel with a side slope of 1:1, bed slope of 0.01 and Manning's coefficient of 0.03. It should be noted that at low flows the trapezoidal channel could underestimate tailwater levels compared to that of the natural channel shape as it is likely low flow channels exist immediately downstream of the channel. Furthermore, this method assumes free flow within the channel downstream of the outlet structure, and does not consider any scour or channel erosion that may occur at higher energy flows. Limited information with respect to the downstream watercourse was available at the time of the study, with a full hydraulic assessment beyond the scope of the current assignment. With this in mind, the assumed design rating has been taken for the purposes of this assessment. However, validation of the downstream rating curve has not been undertaken; and therefore, may not be an accurate reflection of the downstream hydraulic conditions of the watercourse.

Using basic geometry for various depths of flows, the flow area and hydraulic radius were calculated. These were then used to calculate the discharge, and thus the rating curve shown in the figure below (Figure.6).

Figure 6: Downstream Rating Curve



Source: MM

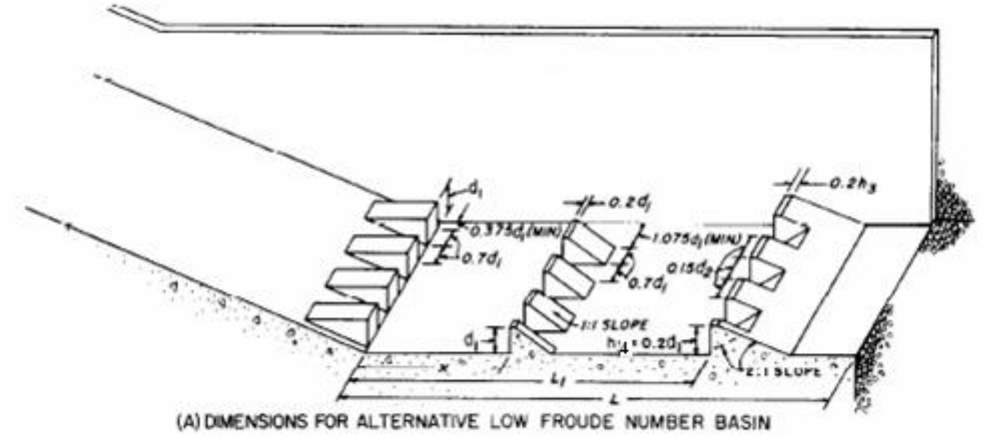
To evaluate the hydraulic conditions of the proposed operational scenarios listed in Table 3, the flow rate associated with the gate conditions described was required to be estimated. For this purpose, the gate width (16 ft), sill level (81.05 ft) and gate height (7 ft) were taken per original construction drawings and flow rate was estimated assuming standard tainter gate head to flow relationships.

A stilling basin assessment was undertaken for each of the cases considered. A stilling basin safety factor of 1.1 was assumed. This factor of safety is usually considered only for analyzing newly built structures to allow for unknowns including downstream water levels and gate discharge parameters, it is included here to give an indication to what a consistent design level for each condition would equate. Typically, a greater value is used during actual design.

Downstream conditions for the various scenarios were determined from the downstream rating curve as described above. Approach velocities were estimated using the inflow and upstream depth of flow.

The length of the jump was evaluated by considering design curves for a free forming hydraulic jump with no chute or baffle blocks. Further, an indicative design for the Q_{500} design flow was also undertaken for comparison to understand what such an arrangement would look like. In particular, it is common for low head structures such as that at Priest Lake to use a type IVA stilling basin to control the hydraulic jump, as shown in Figure 7 below. The United States Bureau of Reclamation specifically commissioned the development of this basin type for such low energy hydraulic conditions. As such, the figure below gives design parameters for a Type IVA basin.

Figure 7: Type IVA Stilling Basin



Source: USBR Design of Small Dams (1987)

Table 4: Summary of Stilling Basin Investigation

Scenario	Flow Rate (cfs)	Stilling Basin Invert Level Required (ft)	Basin Length Required, L (ft)	Difference in Invert Level from Existing (ft)	Difference in Length from Existing (ft)
Q_{500} Design Flow	11,200	71.12	55.94	8.93	51.72
Summer	250	74.61	31.07	5.44	26.82
Winter	7,570	76.76	37.73	3.29	33.48
Summer - New	260	74.43	32.25	5.62	28
Q_{500} Design Flow (Type IVA basin)	11,200	71.12	31.82	8.93	27.57

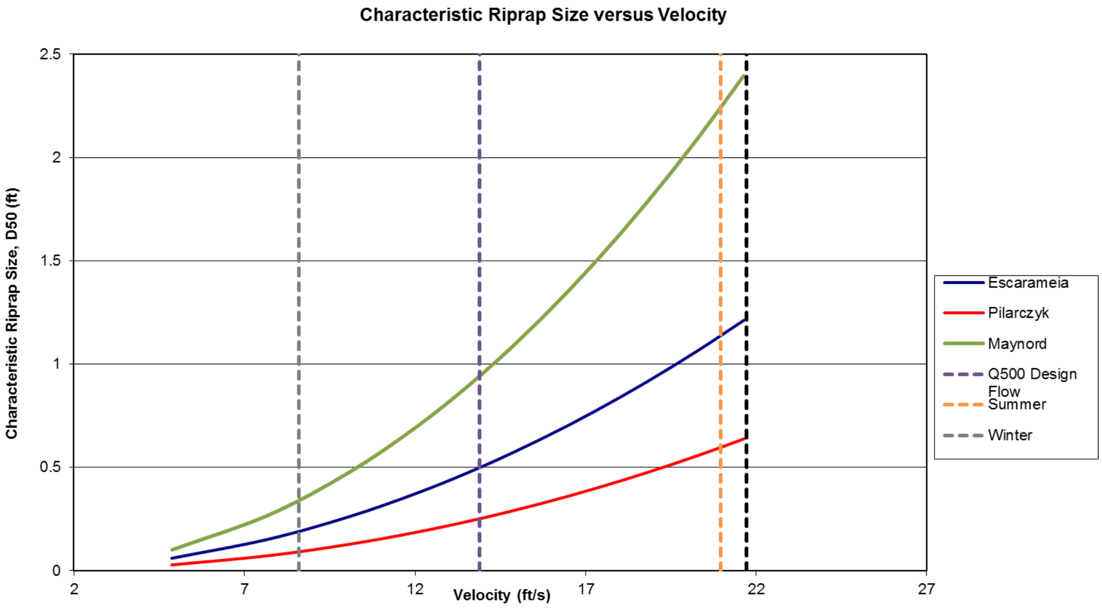
Source: MM NB: The basin length and difference in basin length is taken for the required stilling basin level, rather than the existing

Erosion stone sizing was undertaken using The Rock Manual with the Escarameia, Pilarczyk and Maynord methods. The downstream rating curve was used as input for this, rather than localised parameters. For the Escarameia method, the turbulence intensity was taken as 60% as recommended for areas of high turbulence such as that expected at Priest Lake.

The existing riprap size is believed to be 1 ft, based upon the note from Mike Stubblefield dated 05/05/1978. Figure 8 shows the characteristic stone sizes plotted against the range of velocities that would be expected within the rip rap area. It can be seen that for the winter and design cases design stone sizes are within the range of 0.1 and 0.3 ft for the former and 0.25 ft and 0.9 ft for the latter. In the case of the summer a range of 0.6 to 2.4 ft.

From this it is clear that the current stone size provided is at the limit of acceptable sizing, and explains why more scour has been seen below gates 5-7. Data taken from the structure suggests that the central gates (5-7) are operated more frequently than the rest, particularly during summer. As all the other gates are closed there is no tailwater level; therefore, this is why there are more signs of scour at these locations.

Figure 8: Graph showing characteristic riprap size against velocity



Source: MM

4 Discussion

4.1 Hydraulic

In the summer scenario, where only one gate is utilised, it was found that the unit discharge was higher than that of the Q_{500} design flow. This suggests that the higher flow rate, coupled with the low tailwater levels could lead to higher scour potential and movement of riprap downstream. That said, the assumed tailwater could be underestimated, as discussed in Section 3, and as such could result in an overly conservative assessment under the summer scenario.

The existing arrangement for the Winter scenario still does not provide adequate length for the hydraulic jump to form on the concrete sill and therefore is likely to result in downstream scour due to turbulence if not operated in the correct fashion.

For the Q_{500} design scenario, as the existing scenario, a stilling basin invert of 80.05ft would be required, with a free forming hydraulic jump length of 39.2 ft. If a new type IVA stilling basin were provided, meeting modern design standards, an invert of approximately 75.1 ft with a basin length of 31.82 ft and end still height of 2.0 ft would be required.

Riprap sizing using the Escarameia, Pilarczyk and Maynord methods give a wide range of d_{50} sizes that are appropriate for the design case, ranging from 1 ft to 3 ft. A d_{50} of 2.5 ft is considered to be better suited to the structure and the expected range of velocities compared to 1 ft currently in place. It is recognised that this method is necessarily conservative and it is assumed that the current arrangement is at the upper end of the acceptable hydraulic conditions.

The energy associated with the different scenarios was quantified to give an indication of the erosive potential of each and when scour effects are likely to become an issue. The critical stream power was calculated, along with the stream power for each of the scenarios, which are summarised in the Table 5. It is evident that for existing conditions, the stream power is approximately twice the critical stream power, therefore there is increased potential for scour. Increasing the pool level by 6 inches results in an increase in stream power of approximately 9%.

Table 5: Summary of Stream Power

Scenario	Stream Power (W/ft^2)
Critical	0.0226
Q_{500} design	0.1007
Winter	0.0694
Summer – existing	0.0481
Summer – with 6-inch pool raise	0.0525

Source: MM

4.2 Mechanical

Tainter gates by virtue of their geometry transfer the hydrostatic loads on the gate through the gate arms to the trunnion bearings mounted on the intermediate piers downstream of the gate. The effect of raising the gate height will be to slightly alter the angle of the resultant force and to increase the loading on the trunnions. Overall, we anticipate that the resultant load will increase by about two metric tonnes per gate with an increase of about 3800 lbf (17kN) in the horizontal load and 1580 lbf (7kN) in the vertical loading. Although it is likely that such a change would only result in a slight reduction in factors of safety within the wall, a more detailed analysis would be required to fully quantify the effect.

The hoist is sized to lift the dead weight of the gate plus the friction in the trunnion bearing and side seals. The addition to the height of the gate will add weight to the gate and we estimate this increase to be about 660lbs. In addition, the additional loading on the gate due to the additional head of water will result in an increased friction load of about 340 lbf (1.5kN). We expect these changes have the effect of raising the hoisting load from 7830 lbf (34.8kN) currently to about 8815lbf (39.2kN).

It should be relatively simple to raise the gate to facilitate the increased pool level. According to the drawings the gate leaf is made of cast iron so the best approach is likely to be to bolt an extension piece, probably a stiffened plate, onto the top of the existing gate. The joint will need to include measures, for example non-conducting sealant and isolation washers, to ensure the dissimilar materials do not promote corrosion as the extension is most likely to be a steel fabrication. The height of the gate will provide both the required increase in pool level and ensure the required amount of freeboard. The amount of freeboard will depend on the site conditions – normally 4” would be considered sufficient but this will depend on the expected wave height and amount and type of floating debris and/or ice likely to be encountered.

5 Recommendations

5.1 Modification to Operation Strategy

As previously discussed, the stilling arrangements of the structure are felt to be unusual and is likely to result in the difficulty in operating the structure. It was found that during the winter and design conditions the scour protection was at the limit of the rip rap performance. Furthermore, it was found that during the summer period the operation of a single gate at a high unit discharge and low tail water levels could also result in significant risk of scour if not operated with care.

As such, where possible, it is recommended that all eleven, or at least the central nine gates are operated at the same time with the similar gate openings and slightly larger gate openings in central bays allowed to prevent any possible rotational flows from forming in the downstream channel. Where the operation of fewer gates is necessitated by the required discharge rates, the operation should be undertaken with extreme care and due consideration of the tailwater levels and downstream turbulence.

If further assessment of the performance of the basin area is required, it is recommended that either further topographic survey is undertaken or flow – stage monitoring is undertaken to determine a more accurate downstream rating curve.

It would be beneficial if an accurate level gage could be surveyed into each pier to allow recording of the gate opening more precisely than the current markings on the pier.

5.2 Modifications to Structure

Further to the recommendations above the following modifications to the structure could be considered that would reduce the risk of downstream scour due to gate operation:

1. The most robust solution would be to construct new a USBR type IVA stilling basin, which would meet modern design standards. However, it is recognised that such a solution would be expensive with limited benefit to the client.
2. Alternatively, the concrete slab could be extended to provide a suitable length of protection for the hydraulic jump to form. However, similar to the above, the cost of the diversion work associated with such a solution would result in a similarly expensive solution.
3. The riprap could be replaced with a d_{50} of 3 ft rather than the existing 1 ft. This is a significantly more cost-effective solution. However, sourcing this standard of material is likely to be problematic, and the thickness of the required filters underlying such rip rap would make this solution impractical;
4. The riprap could be grouted to act as an extension of the concrete slab, however this is likely to require regular maintenance to ensure adequate performance for extreme events and is likely to crack over time due to ground movement;
5. A notched ramp could be provided at the end of the concrete slab to allow sufficient tailwater level to encourage a hydraulic jump on the slab, while allowing standing water to drain when the gates are closed. Such a design would require detailed hydraulic analysis, to potentially include computational fluid dynamics (CFD) or physical modeling. Furthermore, such a solution is likely to lead to turbulence downstream of the concrete slab causing further erosion and thus would not eliminate the problem;

6. The operator of the structure has suggested the possibility of ‘fingers’ installed to the bottom of the gates to break up the flow passing under the structure. It is likely that this would only be beneficial in the summer scenario and would require complex analysis, such as computational fluid dynamics (CFD) to confirm its effectiveness.

5.2.1 Feasibility of structure modifications

The most robust solutions are 1-3, however in the case of the stilling basin this would be very expensive relative to the reduction in the risk of erosion due to poor gate operation. In the case of the rip rap option, this is likely to be unachievable due to the large size required and relative size of filters necessary for such an arrangement.

In light of this, the preferred solution would be to either extend the concrete sill and ensure adequate rip rap at the end of the sill or provide automation and monitor the erosion downstream providing regular maintenance where necessary to maintain the protection.

In either case erosion would still be expected with maintenance works likely to be reduced in the case of the concrete sill. As such a regular survey (every 2-3 years) should be conducted in either case to evaluate the scour and to fill in any holes that have formed with rip rap. Such a maintenance regime could be maintained until such a time that it is decided the structure should be replaced.

5.3 Further Gate Investigation

In our opinion, it is likely that the existing gates will be suitable to accommodate the proposed maximum 6-inch increase in pool level although this will inevitably erode the existing safety margins of the gate structure. To confirm the gates suitability, we advise the following work is required to be undertaken:

1. Condition assessment of the existing gates and gather details of hoist gear dimensions etc.,
2. Full calculation of proposed loading to ensure stress and deflection remain within acceptable limits,
3. Confirm trunnions and bearing mounts (cast in to pier walls) are sufficient for increased loading,
4. Check hoist gear for increased loading,
5. Assuming above is satisfactory, provide design for gate extension.
6. Confirm existing reinforced concrete piers can accommodate added stress

5.4 Powered Operation & Automation

The gates are currently manually operated, albeit by using a portable electric powered drive in place of the original handwheels. It may be possible to improve this arrangement if desired by providing powered operation to the gates either operated locally (adjacent to the gate) or remotely (at a location, or locations, not immediately adjacent to the gate). A logical development of powered operation is “automation” where the gate is automatically operated by the control system without direct human involvement based on some control philosophy often based on river level.

It would be relatively straightforward to provide powered operation to the existing gates and there are two common approaches:

a) Retro-fit a motor and gearbox to the existing, or a modified, drive with a local starter panel including the required protection (overload, emergency stop, power available etc.), controls (start, stop, fault reset, etc.) and indication (running, stopped, heathy, fault indications, position, fully open / closed, etc.). A position indication often driven by a rotary encoder (absolute type to ensure zero position is maintained) can be provided scaled in appropriate units (% or distance open etc.). Limit switches are used to ensure the motor is stopped when the gate reaches the extent of its travel and to provide fully open/closed signals. If required the local panel will also include connections to allow remote operation in which case a local/remote selector switch is also provided. A second gearbox will also need to be provided with a handwheel to allow operation in the event of a power failure.

b) An alternative, which can be more economic, is to use a valve actuator such as those manufactured by Obermeyer, Rotork or Auma. This approach has the advantage that the controls, position indication and limit switches are all contained within the actuator. Connections are invariably available for remote operation and all that is required is a power supply. Most actuators also include a geared hand wheel to allow operation in the event of a power failure. Each gate would need a mounting bracket or pedestal to support the actuator. Actuators are normally supplied with a blank drive nut – this can then be machined to suit the application, in this case the connection to the existing drive mechanism.

In both cases, in public areas it is common to provide enclosures or guarding to provide security for the controls.

Remote operation can be from a control building or enclosure adjacent to the river or from a remote desk at a water company office and it is also possible to provide remote monitoring only which can be useful, allowing a remote company control room to monitor levels and gate status in order to dispatch a technician to operate the gates as and when required. Sometimes if the operation is from a point where the gates are not visible CCTV is provided to allow the gate operator to see the gates during operation.

If the gates have been powered, the logical “next step” is automation but this is not always economic or beneficial depending on the complexity of the control required. The simplest form of control is to monitor upstream (or downstream if downstream flow is the control parameter) river levels and to open and close the gates to maintain the upstream level within a control band. Various algorithms can be used to share the flow between multiple gates and to distribute the flows if required. Seasonal variations can be built into the control philosophy to provide summer and winter settings for example. Limitations in the power supply, common in rural areas, can be accommodating by limiting the number of motors which can run simultaneously.

In the event the gates are automated precautions may need to be taken to ensure the gates can safely operate automatically without having an operator present and able to check that the gates can be safely operated.

A further factor in choosing whether to power or automate the gates is to consider the skills of company maintenance staff. The use of actuators usually only needs an electrician with support for the actuators, usually very reliable, being provided by the manufacturer’s service organisation. The other options require both electricians and ICA (Instrumentation, control and automation) technicians.

A common approach is to initially provide powered operation, often with remote monitoring, with a view to adding automation in the future.

Appendices

A. Hydraulic Calculations 19

A. Hydraulic Calculations

Project **Priest Lake Dam**					
Calculations for 000 Cover Sheet.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

Operational Scenario

Number of Operational Gates	11		
Flow	317.19	m^3s^{-1}	**Q500=11,200cfs**
Gage Level	1.87452	m	**6.15 ft**
River Level	27.75	mAD	**91.05ft (6.15ft gage)**
Gate Opening	2.134	m	**7ft**
Stilling Basin	As existing		

Downstream Rating Curve

A downstream rating curve was produced to enable us identify the tail water level versus discharge assuming a trapezoidal channel.
A bed width of 220 ft was taken using a side slope of 1:1, bed slope of 0.01 and Manning's coefficient of 0.03 as per the original design calculation. The bed level was taken as 80.05 ft, the level of the concrete apron.
Using basic geometry for various depths of flows, the flow area and hydraulic radius were calculated. These could then be used to calculate the discharge.

Radial Gate with Raised Cill

The gate width (16 ft), cill level (81.05 ft) and gate height (7 ft) were taken as per original construction drawings. The safety factor was assumed to be 1.
Assuming unsubmerged conditions and a maximum opening of the 7ft high radial gate, the maximum unit discharge can then be calculated.

Stilling Basin Level Design

Three worst-case scenarios were considered for the stilling basin design. These are:

- 100% flow both upstream and downstream of the structure, to simulate design flow conditions
- 10% upstream flow with 0% flow downstream to simulate sudden gate opening
- 30% upstream flow with 30% downstream flow

A stilling basin safety factor of 1.1 was used for this calculation. Downstream conditions for the various scenarios can be determined from the downstream rating curve. Approach velocities were estimated using the inflow and height of the gates.
Head differences across the gate could be calculated by using the known qate levels and velocities upstream and downstream of the structure. This can be used to determine various parameters about the flow at the glasis and conjugate depth of the given scenario. Based on the Froude number, the performance of the stilling basin can be determined. For the scenarios the minimum basin level, maximum Froude number, maximum conjugate depth and maximum free jump length are selected as the parameters required for the critical basin.

USBR Stilling Basins

This was undertaken using the USBR Design of Small Dams for a type I stilling basin with no chute or baffle blocks. Along with the parameters determined from the level design, the required basin and end sill dimensions were determined.

Erosion Stone Sizing

Erosion stone sizing was undertaken using The Rock Manual with the Escarameia, Pilarczyk and Maynord methods. The relative density of riprap was taken as 2.65 and the angle of the bank to the horizontal was taken as 45 degrees. All other required inputs were taken from geometry of the structure.

Project

Priest Lake Dam

MOTT MACDONALD

Calculations for

001 EVT-HYES-HYD-300 Trapizoidal Rating Curve.

Division	WCD	File No.		
Calc by	JM	Date	31/07/2017	Number of sheets
Checked by	PJH	Date	31/07/2017	of

Manning's Rating Curve

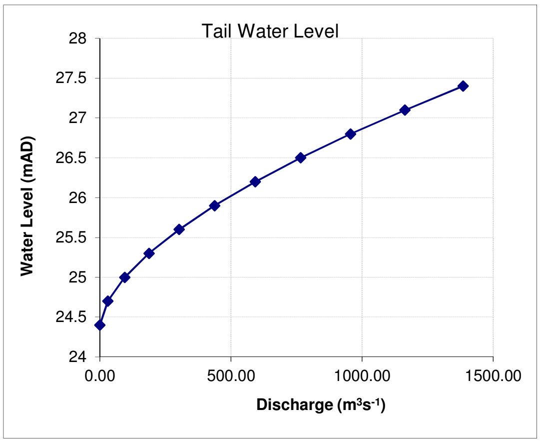
Open Channel Hydraulics, 1959, Ven Te Chow

1. INPUTS

Bed Width	B	67.056	m	**Comments:**	**220 ft as per original calculation**
Side Slope	s	1	H:1V	**Comments:**	**As per original calc**
Bed Slope	i	0.01	m/m	**Comments:**	**As per original calc**
Manning's Coefficient	n	0.03	sm$^{-1/3}$	**Comments:**	**As per original calc**
Bed Level	Zb	24.39924	mAD	**Comments:**	**80.05 ft**
Max Depth	Dmax	3	m	**Comments:**	

2. OUTPUTS

#	Depth of Flow	Flow Area	Hydraulic Radius	Discharge	Water Level	Average Velocity	Froude
n	D	A	P	Q	Zwl	v	Fr
-	m	m^2	m	m^{3}s^{-1}	mAD	ms-1	-
-	-	$D_n*(B+D_n*s)$	$B+2*D_n*\sqrt{(1+s^2)}$	$A(A/P)^{2/3}*\sqrt{(i)}$ / n	D_n+Z_b	Q/A	$v/\sqrt{(g*D_n)}$
1	0	0	67.06	0.00	24.39924	0	0
2	0.3	20.2068	67.90	30.02	24.69924	1.49	0.87
3	0.6	40.5936	68.75	95.23	24.99924	2.35	0.97
4	0.9	61.1604	69.60	187.03	25.29924	3.06	1.03
5	1.2	81.9072	70.45	301.88	25.59924	3.69	1.07
6	1.5	102.834	71.30	437.57	25.89924	4.26	1.11
7	1.8	123.9408	72.15	592.59	26.19924	4.78	1.14
8	2.1	145.2276	73.00	765.77	26.49924	5.27	1.16
9	2.4	166.6944	73.84	956.17	26.79924	5.74	1.18
10	2.7	188.3412	74.69	1163.06	27.09924	6.18	1.20
11	3	210.168	75.54	1385.80	27.39924	6.59	1.22



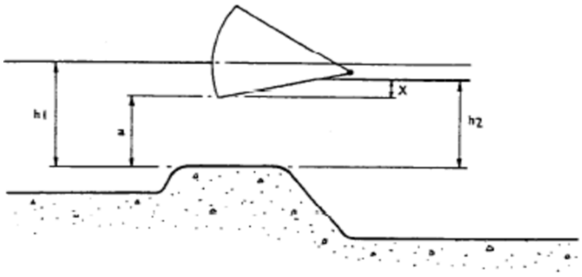
Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 002 EVT-HYES-HYD-561 Radial Gate with Raised Cill.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

Radial Gate With Cill

Hydraulic Design of Canal Structures - MMP August 1985

USBR - Design of Small Small Dams

1. INPUTS



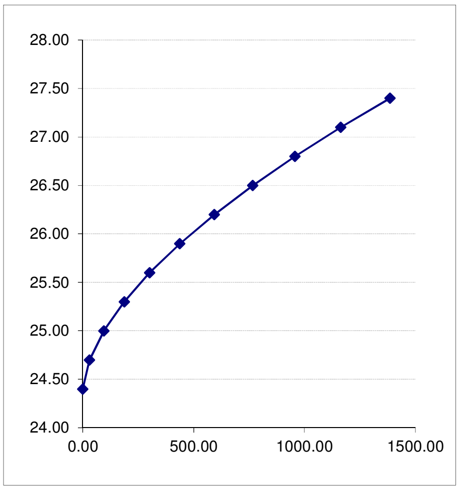
1. Hydraulic Design of Canal Structures, Sept 1986, MM & Partners p. 1 - p. 11

2. Design of Small Dams - USBR

1.1 GENERAL

Design Discharge	Q	317.19	m^3s^{-1}	**Comments:**	**Q500=11,200cfs**
Number of Gates	N_g	11	-	**Comments:**	
Gate Width	w	4.88	m	**Comments:**	**16 ft**
Cill Level	Z_c	24.70	mAD	**Comments:**	**81.05ft**
Max Upstream Water Level	Z_{uswl}	27.75	mAD	**Comments:**	**91.05ft (6.15ft gage)**
Structure Flow Saftey Factor	S_f	1	-	**Comments:**	
Maximum gate opening (as percentage of max head)	μ	44%	-	**Comments:**	**Maximum opening for design dischage**
Design Gate Height	a	2.134	-	**Comments:**	**7ft**
Design Downstream Water Level	Z_{dswl}	25.87	mAD	**Comments:**	

	Level	Flow	Velocity
1	mAD	m3/s	m/s
2	24.40	0.00	0.00
3	24.70	30.02	1.49
4	25.00	95.23	2.35
5	25.30	187.03	3.06
6	25.60	301.88	3.69
7	25.90	437.57	4.26
8	26.20	592.59	4.78
9	26.50	765.77	5.27
10	26.80	956.17	5.74
11	27.10	1163.06	6.18
12	27.40	1385.80	6.59
13			
14			



1.2 Effective Crest Coefficients

Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 002 EVT-HYES-HYD-561 Radial Gate with Raised Cill.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

Pier Contraction Coefficient	k_p	0.020	-	**Comments:**	**USBR Design of Small Dams, square nosed piers**
Abutment Contraction Coefficient	k_a	0.2	-	**Comments:**	**USBR Design of Small Dams, square abutments**

2. OUTPUTS

2.1 GENERAL

Number of Piers	N	10		N_g-1	**Comments:**
Sum of Gates Width	b'	53.65	m	N_g*w	**Comments:**
Design Factored Discharge	Q_f	317.19	m^3s^{-1}	S_f*Q	**Comments:**
Upstream Head Over Cill	h_1	3.05	m	$Z_{uswl} - Z_c$	**Comments:**
Downstream Head Over Cill	h_2	1.165	m	$Z_{dswl} - Z_c$	**Comments:**
Effective Crest Length	b	51.21	m	$b'-2*(N*K_p+k_a)*h_1$	**Comments:**

2.2 FREE FLOW CALCULATIONS

Max Height of Gate Opening	$a_{max\ 1}$	1.34	m	$\mu*h_1$	**Comments:**
Opening - DS Water Level Difference	x	0.79	m	$h_2 - a$	**Comments:**
Max Unit Discharge	q_{max}	6.223	m^2s^{-1}	$0.6*a_{max\ 1}*\surd(2*g*h_1)$	**Comments:**
Design Unit Discharge	q	6.194	m^2s^{-1}	Q_f/b	**Comments:**
Flow	Q_{max}	333.817	m^3s^{-1}	$q*w*N$	**Comments:**
CHECK		OK			**Comments:**
Gate Radius	R	1.788	m	4/3*amax 1	**Comments:**
Depth Above Gate	a'	1.71	m	h1-amax 1	**Comments:**
Angle	θ	17.34	Degrees	$cos^{-1}(a/R)$	**Comments:**
Vena Contrata Factor	δ	0.65	-		**Comments:**
Vena Contrata	α	0.87	m	$a*\delta$	**Comments:**
Velocity	v	7.14	ms^{-1}	q/α	**Comments:**
Froude Number	Fr	2.44	-		**Comments:**
Conjugate Depth	y_2	3.01	m	$y_1/2*\surd((1+8Fr_1^{2})-1)$	**Comments:**
Water Level	Z_c	27.71	mAD	y_2+Z_c	**Comments:**
Downstream Water Level	Z_{ds}	25.67	mAD		**Comments:**
Condition		UNSUBMERGED			**Comments:**

2.3 SUBMERGED CONDITIONS

Max Height of Gate Opening	$a_{max\ 2}$	2.03	m	$2/3*h_1$	**Comments:**
Opening - DS Water Level Difference	x	-0.87	m	$h_2 - a$	**Comments:**
Head Drop	H	1.88	m	h_1-h_2	
Max Unit Discharge	q_{max}	8.276	m^2s^{-1}	$0.67*a_{max\ 2}*\surd(2*g*(h_1-h_2))$	

Project **Priest Lake Dam**	MOTT MACDONALD				
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Design Unit Discharge	q	6.194 m^2s^{-1}	Q_f/b'	**Comments:**
CHECK		OK	$q < q_{max}$	**Comments:**
Gate Radius	R	2.709 m	$4/3*a$	**Comments:**
Flow Through One Gate @ 100% a	$Q_{n=1}$	38.525 m^3s^{-1}	$q_{max}*w$	**Comments:**
Design Flow	Q	423.778 m^3s^{-1}		

2.4 DESIGN CONDITIONS

Condition		UNSUBMERGED		**Comments:**
Design Flow	Q_d	333.817 m^3s^{-1}	$q*w*N$	**Comments:**

Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 003 EVT-HYES-HYD-001 - Stilling Basin Level Design.xls.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
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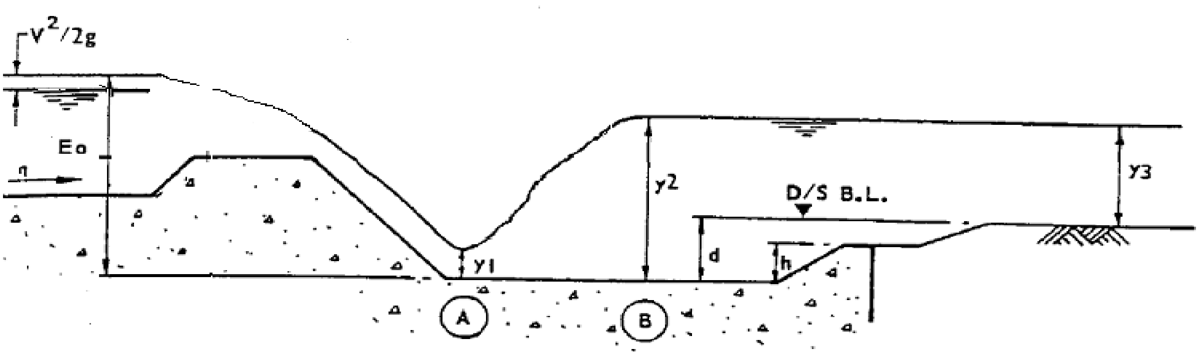
Stilling Basin Level Design

1. MMP Hydraulic Design of Canal Structures, 1985, 1-13 - 1-14

1. INPUTS

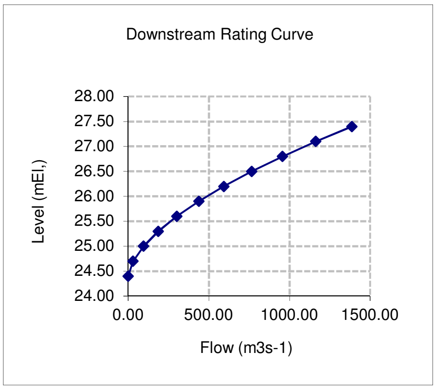


1.1 GENERAL

Design Flow	Q_d	333.8	m^3s^{-1}	**Comments:**	
Flow Safety Factor	S_f	1.00	-	**Comments:**	
Width of Stilling Basin	B	53.64	m	**Comments:**	**16ft*11 no. = 176 ft**
Number of Basins	N	1	-	**Comments:**	
Basin Safety Factor	S_f	1.1	-	**Comments:**	
Bed Level US	Z_{usbl}	24.40	mAD	**Comments:**	**80.05 ft**
Bed Level DS	Z_{dsbl}	24.40	mAD	**Comments:**	**80.05 ft**
Allowance For Retrogression (Percentage of D/S Depth)	ρ	0%	%	**Comments:**	
Gate Width	w_g	53.64	m	**Comments:**	**16 ft * 11 no.**

Downstream Rating Curve (Table 1.1)

#	Level	Flow	Velocity
n	Z_{dswl}	Q	v
-	mAD	m^3s^{-1}	ms^{-1}
1	24.40	0.00	0.00
2	24.70	30.02	1.49
3	25.00	95.23	2.35
4	25.30	187.03	3.06
5	25.60	301.88	3.69
6	25.90	437.57	4.26
7	26.20	592.59	4.78
8	26.50	765.77	5.27
9	26.80	956.17	5.74
10	27.10	1163.06	6.18
11	27.40	1385.80	6.59



1.2 CONDITION 1

1.2.1 General

Percentage of Flow US	$m_{us,1}$	100%		**Comments:**	
Percentage of Flow DS	$m_{ds,1}$	100%		**Comments:**	
Additional flow downstream	$Q_{add,1}$	0.0	m^3s^{-1}	**Comments:**	

1.3.2 Upstream Conditions

Water Level	$Z_{us,1}$	27.8	mAD	**Comments:**	**91.05ft (6.15ft gage)**
Velocity	$v_{us,1}$	0.270	ms^{-1}	**Comments:**	
Maximum Unit Discharge	$q_{max,1}$	6.223	m^2s^{-1}	**Comments:**	

1.3 CONDITION 2

1.3.1 General

Percentage of Flow US	$m_{us,2}$	10%		**Comments:**	
Percentage of Flow DS	$m_{ds,2}$	0%		**Comments:**	
Additional flow downstream	$Q_{add,2}$	0.0	m^3s^{-1}	**Comments:**	

1.3.2 Upstream Conditions

Water Level	$Z_{us,2}$	27.8	mAD	**Comments:**	
Velocity	$v_{us,1}$	0.28	ms^{-1}	**Comments:**	**q/7ft**
Maximum Unit Discharge	$q_{max,1}$	0.594	m^2s^{-1}	**Comments:**	

1.4 CONDITION 3

1.4.1 General

Percentage of Flow US	$m_{us,3}$	30%		**Comments:**	
Percentage of Flow DS	$m_{ds,3}$	30%		**Comments:**	
Additional flow downstream	$Q_{add,3}$	0.0	m^3s^{-1}	**Comments:**	

1.4.2 Upstream Conditions

Water Level	$Z_{us,3}$	27.8	mAD	**Comments:**	
Velocity	$v_{us,3}$	0.84	ms^{-1}	**Comments:**	**q/7ft**
Maximum Unit Discharge	$q_{max,3}$	1.782	m^2s^{-1}	**Comments:**	

Project **Priest Lake Dam**	MOTT MACDONALD					
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2. OUTPUTS

Condition 1 - 100% Design Flow with 100% Design Flow Downstream

Condition 2 - 10% Design Flow with 0% Design Flow Downstream

Condition 3 - 30% Design Flow with 30% Design Flow Downstream

2.1 GENERAL

Factored Design Flow	Q_f	333.8	m^3s^{-1}	Q_d*S_f	**Comments:**
2.1.1 Condition 1					**Comments:**
Factored Discharge US	$Q_{us,1}$	333.8	m^3s^{-1}	$Q_f*m_{us,1}$	**Comments:**
Discharge DS	$Q_{ds,1}$	333.8	m^3s^{-1}	$Q_f*m_{ds,1}$	**Comments:**
DS Water Level	$Z_{ds,1}$	25.7	mAD	*Interpolate (Table 1.1)*	**Comments:**
Velocity @ Q DS	$v_{ds,1}$	3.82	ms^{-1}	*Interpolate (Table 1.1)*	**Comments:**
2.1.2 Condition 2					**Comments:**
Factored Discharge US	$Q_{us,2}$	33.4	m^3s^{-1}	$Q_f*m_{us,2}$	**Comments:**
Discharge DS	$Q_{ds,2}$	0.0	m^3s^{-1}	$Q_f*m_{ds,2}$	**Comments:**
DS Water Level	$Z_{ds,2}$	24.399	mAD	*Interpolate (Table 1.1)*	**Comments:**
Velocity @ Q DS	$v_{ds,2}$	0.00	ms^{-1}	*Interpolate (Table 1.1)*	**Comments:**
2.1.3 Condition 3					**Comments:**
Factored Discharge US	$Q_{us,3}$	100.1	m^3s^{-1}	$Q_f*m_{us,3}$	**Comments:**
Discharge DS	$Q_{us,3}$	100.1	m^3s^{-1}	$Q_f*m_{ds,3}$	**Comments:**
DS Water Level	$Z_{ds,3}$	25.015	mAD	*Interpolate (Table 1.1)*	**Comments:**
Velocity @ Q DS	$v_{ds,3}$	2.38	ms^{-1}	*Interpolate (Table 1.1)*	**Comments:**

2.2 DOWNSTREAM CONDITION

Water Depth DS Condition 1	$D_{ds,1}$	1.271	m	$Z_{dswl}-Z_{bl}$	**Comments:**
Water Depth DS Condition 2	$D_{ds,2}$	0.000	m	$Z_{dswl}-Z_{bl}$	**Comments:**
Water Depth DS Condition 3	$D_{ds,3}$	0.616	m	$Z_{dswl,70}-Z_{bl}$	**Comments:**
Retrogression @ Condition 1	r_1	0.000	m	$\rho*D_{ds,1}$	**Comments:**
Retrogression @ Condition 2	r_2	0.000	m	$\rho*D_{ds,2}$	**Comments:**
Retrogression @ Condition 3	r_3	0.000	m	$\rho*D_{ds,3}$	**Comments:**
Water Level Condition 1 with Retrogression	$Z_{dsl,r}$	25.670	mAD	$Z_{ds,1}-r_1$	**Comments:**
Water Level Condition 2 with Retrogression	$Z_{dsl,r}$	24.399	mAD	$Z_{ds,2}-r_2$	**Comments:**
Water Level Condition 3 with Retrogression	$Z_{dsl,70,r}$	25.015	mAD	$Z_{ds,3}-r_3$	**Comments:**

Project	
Priest Lake Dam	MOTT MACDONALD

Calculations for	Division	WCD	File No.		
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Stilling Basin General Design - Condition 1

1. INPUTS

Design Flow	$Q_{us,1}$	333.82	m^3s^{-1}
Width of Stilling Basin	B	53.64	m
Upstream Water Level	$Z_{us,1}$	27.75	mAD
Approach velocity	$v_{us,1}$	0.270	ms^{-1}
Downstream Water Level	$Z_{ds,1}$	25.7	mAD
Downstream Velocity	$v_{ds,1}$	3.82	ms^{-1}
Basin Safety Factor	S_f	1.1	-

2.2 OUTPUTS - CONDITION 1

Flow Intensity	q_1	6.22	m^2s^{-1}	$Q_{us,1}/B/N$	**Comments:**
Critical Depth	$d_{c,1}$	1.58	m	$(q_1{}^2/g)^{1/3}$	**Comments:**
Upstream Velocity Head	$h_{v,1}$	0.00	m	$v_{us,1}{}^2/2g$	**Comments:**
Downstream Velocity Head	$h_{v2,1}$	0.74	m	$v_{ds,1}{}^2/2g$	**Comments:**
Upstream Total Head	$E_{o,1}$	27.76	m	$Z_{us,1} + h_{v,1}$	**Comments:**
Downstream Total Head	$E_{2,1}$	26.41	m	$Z_{ds,1} + h_{2v,1}$	**Comments:**
Head Difference Across Gate	$H_{L,1}$	1.34	m	$E_{0,1}-E_{2,1}$	**Comments:**
	H_L/d_c	0.85	m	H_L/d_c	**Comments:**
	a_1	0.24	-	*GOAL SEEK*	**Comments:**
	$v_{1,1}$	0.85	-	$(1-a_1)^3/(2a_1)*((1+2a_1)^2-1)^{-1/3})$	
Congugate Depth	$y_{2,1}$	2.97	m	$4*H_{L,1}*a_1/(1-a_1)^3$	
Velocity @ Congugate Depth	$v_{2,1}$	2.09	ms^{-1}	$q_1/y_{2,1}$	
Depth of Flow at Bottom of Glasis	$y_{1,1}$	0.72	m	$a_1*y_{2,1}$	
Velocity @ Toe of Glasis	$v_{1,1}$	8.66	ms^{-1}	$q_1/y_{1,1}$	**Comments:**
Froude Number	Fr_1	3.26	-	$v_{1,1}/\sqrt{(gy_{1,1})}$	**Comments:**
Conjugate Depth Check Calc	$y_{2,1}$	2.97	m	$y_{1,1}/2*\sqrt{((1+8Fr_1{}^2)-1)}$	**Comments:**
Total Energy at Congugate Depth	$E_{1,1}$	3.20	m	$y_{2,1}+d_{c,1}{}^3/(2*y_{2,1}{}^2)$	
Basin Level	$\mathbf{Z_{b,1}}$	**22.90**	**mAD**	$E_{2,1}-S_f*E_{1,1}$	**Comments:**
Downstream Jump Level	Z_{dsjl}	25.87	m	$Z_{dsbs} + y_2$	**Comments:**
	L/y_2	5.43	m	*INTERPOLATE*	**Comments:**
	L/y_2	5.43	m	*INTERPOLATE*	**Comments:**
Jump Length	L	16.15	m	L/y_2*y_2	**Comments:**
Free Jump Performance		Wavy			
Basin Type		Type IVA			**Comments:**
Basin Jump Length Factor	L_B/y_2	3.16	m		
Basin Jump Length	L_B	9.40	m		

Project **Priest Lake Dam**	MOTT MACDONALD				
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2.2.1 Design Check (Set Outputs First)

Total Energy	$E_{0,1d}$	3.36	m	$Z_{us,1}+h_{v,1}-Z_b$	**Comments:**
Velocity	$v_{1,1d}$	6.95	ms^{-1}	$\sqrt{}(2g*(E_{0,1d}-y_{1,1d}))$	**Comments:**
Depth of Flow	$y_{1,1d}$	0.896	m	$q_1/v_{1,1d}$	**Comments:**
Upstream Froude Number	$\mathbf{Fr_{1,1d}}$	**2.344**	-	$v_{1,1d}/\sqrt{}(gy_{1,1d})$	**Comments:**
Conjugate Depth	$y_{2,1d}$	2.55	-	$y_{1,1d}/2*(\sqrt{}(1+8Fr_{1d}^2)-1)$	
Conjugate Level	$y_{2,1d}$	26.95	-	$y_{1,1d}/2*(\sqrt{}(1+8Fr_{1d}^2)-1)$	

Project **Priest Lake Dam**					MOTT MACDONALD
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Stilling Basin General Design - Condition 2

1. INPUTS

Design Flow	$Q_{us,2}$	33.38	m^3s^{-1}
Width of Stilling Basin	B	53.64	m
Upstream Water Level	$Z_{us,2}$	27.75	mAD
Approach velcoity	$v_{us,2}$	0.278	ms^{-1}
Downstream Water Level	$Z_{ds,2}$	24.4	mAD
Downstream Velocity	$v_{ds,2}$	0.00	ms^{-1}
Basin Safety Factor	S_f	1.1	-

2.3 OUTPUTS - CONDITION 2

Flow Intensity	q_2	0.62	m^2s^{-1}	$Q_{us,2}/B/N$	**Comments:**
Critical Depth	$d_{c,2}$	0.34	m	$(q_2^{2}/g)^{1/3}$	**Comments:**
Upstream Velocity Head	$h_{v,2}$	0.00	m	$v_{us,2}^{2}/2g$	**Comments:**
Downstream Velocity Head	$h_{v2,2}$	0.00	m	$v_{ds,2}^{2}/2g$	**Comments:**
Upstream Total Head	$E_{o,2}$	27.76	m	$Z_{us,2} + h_{v,2}$	**Comments:**
Downstream Total Head	$E_{2,2}$	24.40	m	$Z_{ds,2} + h_{2v,2}$	**Comments:**
Head Difference Across Gate	$H_{L,2}$	3.36	m	$E_{0,2}-E_{2,2}$	**Comments:**
	H_L/d_c	9.86	m	H_L/d_c	**Comments:**
	a_2	0.06	-	*GOAL SEEK*	**Comments:**
	$v_{1,2}$	9.86	-	$(1-a_2)^{3}/(2a_2)*((1+2a_2)^{2}-1)^{-1/3})$	
Congugate Depth	$y_{2,2}$	1.05	m	$4*H_{L,2}*a_2/(1-a_2)^{3}$	
Velocity @ Congugate Depth	$v_{2,2}$	0.59	ms^{-1}	$q_2/y_{2,2}$	
Depth of Flow at Bottom of Glacis	$y_{1,2}$	0.07	m	$a_2*y_{2,2}$	
Velocity @ Toe of Glacis	$v_{1,2}$	9.25	ms^{-1}	$q_2/y_{1,2}$	**Comments:**
Froude Number	Fr_2	11.38	-	$v_{1,2}/\surd(gy_{1,2})$	**Comments:**
Conjugate Depth Check Calc	$y_{2,2}$	1.05	m	$y_{1,2}/2*\surd((1+8Fr_2^{2}$	**Comments:**
Total Energy at Congugate Depth	$E_{1a,2}$	1.07	m	$y_{2,2}+d_{c,2}^{3}/(2*y_{2,2}$	**Comments:**
Basin Level	$\mathbf{Z_{dsbs}}$	**23.22**	**mAD**	$E_{2,2}-S_f*E_{1,2}$	**Comments:**
Downstream Jump Level	Z_{dsjl}	24.27	m	$Z_{dsbs} + y_2$	**Comments:**
	L/y_2	6.05	m	*INTERPOLATE*	**Comments:**
	L/y_2	6.05	m	*INTERPOLATE*	**Comments:**
Jump Length	L	6.35	m	L/y_2*y_2	**Comments:**
Free Jump Performance		Acceptable Performance			
Basin Type		Type III			**Comments:**
Basin Jump Length Factor	L_B/y_2	2.76	m		
Basin Jump Length	L_B	2.90	m		

Project **Priest Lake Dam**					
Calculations for 003 EVT-HYES-HYD-001 - Stilling Basin Level Design.xls.	Division	WCD	File No.		
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2.3.1 Design Check (Set Outputs First)

Total Energy	E_0	3.36	m	$Z_{uswl}+h_v-Z_b$	**Comments:**
Velocity	v_1	8.02	ms^{-1}	$\sqrt{}(2g*(E_0-y_1))$	**Comments:**
Depth of Flow	y_1	0.078	m	q/v	**Comments:**
Upstream Froude Number	**Fr_1**	**9.194**	-	$v_1/\sqrt{}(gy_1)$	**Comments:**
Conjugate Depth	y_2	0.97	-	$y_1/2*(\sqrt{}(1+8Fr_1^2)-1)$	**Comments:**
Conjugate Level	y_2	25.37	-	$y_1/2*(\sqrt{}(1+8Fr_1^2)-1)$	**Comments:**

Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 003 EVT-HYES-HYD-001 - Stilling Basin Level Design.xls.	Division	WCD	File No.		
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Stilling Basin General Design - Condition 3

MM Hydraulic Structures, 1985

1. INPUTS

Design Flow	$Q_{us,3}$	100.15	m^3s^{-1}
Width of Stilling Basin	B	53.64	m
Upstream Water Level	$Z_{us,3}$	27.75	mAD
Approach velcoity	$v_{us,3}$	0.835	ms^{-1}
Downstream Water Level	$Z_{ds,3}$	25.0	mAD
Downstream Velocity	$v_{ds,3}$	2.38	ms^{-1}
Basin Safety Factor	S_f	1.1	-

2.4 OUTPUTS - CONDITION 3

Flow Intensity	q_3	1.87	m^2s^{-1}	$Q_{us,3}/B/N$	**Comments:**
Critical Depth	$d_{c,3}$	0.71	m	$(q_3{}^2/g)^{1/3}$	**Comments:**
Upstream Velocity Head	$h_{v,3}$	0.04	m	$v_{us,3}{}^2/2g$	**Comments:**
Downstream Velocity Head	$h_{v2,3}$	0.29	m	$v_{ds,3}{}^2/2g$	**Comments:**
Upstream Total Head	$E_{o,3}$	27.79	mAD	$Z_{us,3} + h_{v,3}$	**Comments:**
Downstream Total Head	$E_{2,3}$	25.30	mAD	$Z_{ds,3} + h_{2v,3}$	**Comments:**
Head Difference Across Gate	$H_{L,3}$	2.48	m	$E_{0,3}-E_{2,3}$	**Comments:**
	H_L/d_c	3.51	m	H_L/d_c	**Comments:**
	a_3	0.12	-	*GOAL SEEK*	**Comments:**
	$v_{1,3}$	3.51	-	$(1-a_3)^3/(2a_3)*((1+2a_3)^2-1)^{-1/3})$	
Congugate Depth	$y_{2,3}$	1.74	m	$4*H_{L,3}*a_3/(1-a_3)^3$	
Velcoity @ Congugate Depth	$v_{2,3}$	1.07	ms^{-1}	$q_3/y_{2,3}$	
Depth of Flow at Bottom of Glasis	$y_{1,3}$	0.21	m	$a_3*y_{2,3}$	
Velocity @ Toe of Glasis	$v_{1,3}$	8.94	ms^{-1}	$q_3/y_{1,3}$	**Comments:**
Froude Number	Fr_3	6.25	-	$v_{1,3}/\sqrt{}(gy_{1,3})$	**Comments:**
Conjugate Depth Check Calc	$y_{2,3}$	1.74	m	$y_{1,3}/2*\sqrt{}((1+8Fr_3{}^2)-1)$	**Comments:**
Total Energy at Congugate Depth	$E_{1a,3}$	1.80	m	$y_{2,3}+d_{c,3}{}^3/(2*y_{2,3}{}^2)$	
Basin Level	$\mathbf{Z_{dsbs}}$	**23.32**	**mAD**	$E_{2,3}-S_f*E_{1,3}$	**Comments:**
Downstream Jump Level	Z_{dsjl}	25.07	m	$Z_{dsbs} + y_2$	**Comments:**
	L/y_2	6.08	m	*INTERPOLATE*	**Comments:**
	L/y_2	6.08	m	*INTERPOLATE*	**Comments:**
Jump Length	L	10.60	m	L/y_2*y_2	**Comments:**
Free Jump Performance		Best Performance			
Basin Type		Type III			**Comments:**
Basin Jump Length Factor	L_B/y_2	2.48	m		
Basin Jump Length	L_B	4.33	m		

Project **Priest Lake Dam**	MOTT MACDONALD				
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2.4.1 Design Check (Set Outputs First)

Total Energy	E_0	3.39	m	$Z_{uswl}+h_v-Z_b$	**Comments:**
Velocity	v_1	7.86	ms^{-1}	$\sqrt{(2g*(E_0-y_1))}$	**Comments:**
Depth of Flow	y_1	0.237	m	q/v	**Comments:**
Upstream Froude Number	$\mathbf{Fr_1}$	**5.152**	-	$v_1/\sqrt{(gy_1)}$	**Comments:**
Conjugate Depth	y_2	1.62	-	$y_1/2*(\sqrt{(1+8Fr_1^2)}-1)$	**Comments:**
Conjugate Level	y_2	26.01	-	$y_1/2*(\sqrt{(1+8Fr_1^2)}-1)$	**Comments:**

Project **Priest Lake Dam**					
Calculations for 003 EVT-HYES-HYD-001 - Stilling Basin Level Design.xls.	Division	WCD	File No.		
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Stilling Basin Check - Outputs

MM Hydraulic Structures, 1985

2. OUTPUTS

Condition 1 - 100% Design Flow with 100% Design Flow Downstream

Condition 2 - 10% Design Flow with 0% Design Flow Downstream

Condition 3 - 30% Design Flow with 30% Design Flow Downstream

2.1 SUMMARY OF CRITICAL BASIN

2.2.1 Basin Level					
Basin Level	Z_{bs}	22.896	mAD	*MIN(C1,C2,C3)*	**Comments:**
Critical Condition		Condition 1	-		**Comments:**
Basin Level Selected	Z_b	**24.400**	mAD	***USER DEFINED***	**Comments:**
2.2.2 Froude Number					
Froude		11.379	-	*MAX(C1,C2,C3)*	**Comments:**
Critical Froude Condition		Condition 2	-		**Comments:**
2.2.3 Conjugate Depth					
Conjugate Depth		2.974	m	*MAX(C1,C2,C3)*	**Comments:**
Max Conjugate Depth Condition		Condition 1	-		**Comments:**
2.2.4 Designed Basin Length					
Max Basin Length		9.397	-	*MAX(C1,C2,C3)*	**Comments:**
Max Basin Length Condition		Condition 1	-		**Comments:**
2.2.5 Free Jump Length					
Max Free Jump Length		16.151	m	*MAX(C1,C2,C3)*	**Comments:**
2.2.6 General					
Minimum Basin Length		17.845	m	6*y2,crit	**Comments:**

Project

Priest Lake Dam

Calculations for

003 EVT-HYES-HYD-001 - Stilling Basin Level Design.xls.

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Stilling Basin Check - Gates Open Check

MM Hydraulic Structures, 1985

2. Outputs

2.1 100% MAX GATE DISCHARGE

Maximum Unit Discharge Per Gate	q_{max}	6.22	m^2s^{-1}		**Comments:**	
Maximum Water Level - Basin Level Difference	E_0	3.35	m	$Z_{uswl,c1}-Z_{bs}$	**Comments:**	
Depth of Flow at Toe of Glasis	y_1	0.896	m	$q/(\sqrt{}(2*g*(E_0-y_1))$	**Comments:**	
Velocity @ Toe	v_1	6.94	ms^{-1}	q/y_1	**Comments:**	
Froude @ Toe	Fr	2.34	-	$v_1/\sqrt{}(g*y_1)$	**Comments:**	
Conjugate Depth	y_2	2.55	m	$y_1/2*(\sqrt{}(1+8*Fr^2)-1)$	**Comments:**	
Water Level	Z_{cg}	25.45	mAD	$Z_{bs}+y_2$	**Comments:**	
Required Flow in Downstream Channel	q_{req}	244.53	m^3s^{-1}	*INTERPOLATE(Table 1.1)*	**Comments:**	
Number of Gates Required to Aquire DS Level	N_g	0.7	-	$Q_{req}/(q_{max}*w_g)$	**Comments:**	Minimum number of gates = 1

2.2 50% Max Gate Discharge

Maximum Unit Discharge Per Gate	q_{max}	3.11	m^2s^{-1}		**Comments:**	
Maximum Water Level - Basin Level Difference	E_0	3.35	m	$Z_{uswl,c1}-Z_{bs}$	**Comments:**	
Depth of Flow at Toe of Glasis	y_1	0.41	m	$q/(\sqrt{}(2*g*(E_0-y_1))$	**Comments:**	
Velocity @ Toe	v_1	7.60	ms^{-1}	q/y_1	**Comments:**	
Froude @ Toe	Fr	3.79	-	$v_1/\sqrt{}(g*y_1)$	**Comments:**	
Conjugate Depth	y_2	2.00	m	$y_1/2*(\sqrt{}(1+8*Fr^2)-1)$	**Comments:**	
Water Level	Z_{cg}	24.90	mAD	$Z_{bs}+y_2$	**Comments:**	
Required Flow in Downstream Channel	q_{req}	72.94	m^3s^{-1}	*INTERPOLATE(Table 1.1)*	**Comments:**	
Number of Gates Required to Aquire DS Level	N_g	0.4	-	$q_{req}/(q_{max}*w_g)$	**Comments:**	Minimum number of gates = 1

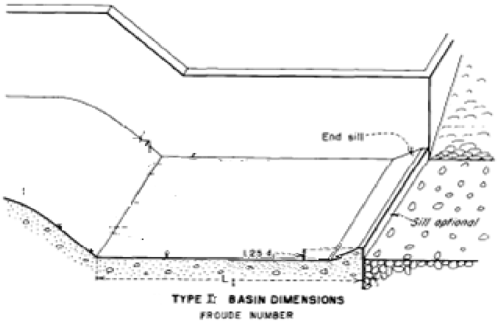
Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 004 EVT-HYES-HYD-002 - USBR Stilling Basins.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

USBR Small Dams Stilling Basin Design

1. USBR Small Dams p.387 - 395

2. USBR Low Froude Stilling Basins

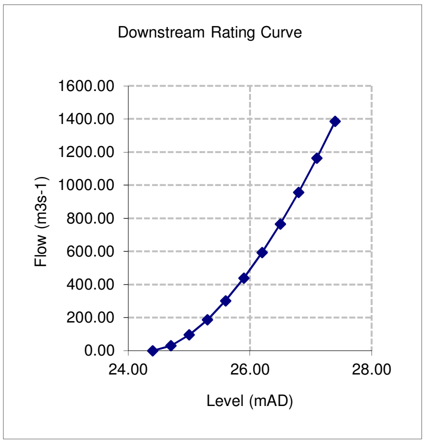
1. INPUTS



Upstream Water Level	Z_{uswl}	27.75	mAD	**Comments:**	**91.05ft (6.15ft gage)**
Upstream Velocity	v_0	0.27	ms^{-1}	**Comments:**	
Basin Level	Z_b	24.40	mAD	**Comments:**	**Downstream level**
Total Flow	Q	333.8	m^3s^{-1}	**Comments:**	
Stilling Basin Width	B	53.645	m	**Comments:**	
Number of Stilling Basins	N	1	-	**Comments:**	
Number of Chute Blocks	Nc	0	-	**Comments:**	
Number of Baffle Blocks	Nb	0	-	**Comments:**	
Basin Type		Type I		**Comments:**	
End Sill Width Multiple	α	0.150		**Comments:**	
Glasis Slope	s	2	H:1V	**Comments:**	

Downstream Rating Curve (Table 1.1)

#	Level	Flow	Velcocity
n	Z_{dswl}	Q	v
-	mAD	m^3s^{-1}	ms^{-1}
1	24.40	0.00	0.00
2	24.70	30.02	1.49
3	25.00	95.23	2.35
4	25.30	187.03	3.06
5	25.60	301.88	3.69
6	25.90	437.57	4.26
7	26.20	592.59	4.78
8	26.50	765.77	5.27
9	26.80	956.17	5.74
10	27.10	1163.06	6.18
11	27.40	1385.80	6.59



2. OUTPUTS

Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 004 EVT-HYES-HYD-002 - USBR Stilling Basins.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

2.1 GENERAL

Unit Flow	q	6.22	m^2s^{-1}	Q/(B*N)	**Comments:**	
Upstream Velocity Head	h_v	0.00	m	$v_0^2/(2g)$	**Comments:**	
Total Energy	E_0	3.36	m	$Z_{uswl}+h_v-Z_b$	**Comments:**	
Velocity	v_1	6.95	ms^{-1}	$\sqrt{(2g*(E_0-y_1))}$	**Comments:**	
Depth of Flow	y_1	0.896	m	q/v	**Comments:**	
Upstream Froude Number	$\mathbf{Fr_1}$	**2.344**	-	$v_1/\sqrt{(gy_1)}$	**Comments:**	
Conjugate Depth	y_2	2.55	-	$y_1/2*(\sqrt{(1+8Fr_1^2)}-1)$	**Comments:**	
Conjugate Level	Zc	26.95	-	*y2+Zb*	**Comments:**	
Initial Design Flow	Qi	1063.00	m^3s^{-1}	*INTERPOLATE*	**Comments:**	
Free Jump Length Factor		4.68	m	*INTERPOLATE*	**Comments:**	
Free Jump Length		11.95	m	L/y_2*y_2	**Comments:**	
MM Basin Length		15.33	m	$6*y_{2,design}$	**Comments:**	
Rough Downstream Water Level		25.67	mEL	*INTERPOLATE*	**Comments:**	

2.2 BASIN DIMENSIONS

2.2.1 Basin

Basin Jump Length Factor	F_L	4.68		*INTERPOLATE*	**Comments:**	
Minimum Basin Length	L_B	11.95	m	F_L*y_2	**Comments:**	
Design Length	**L**	**2.80**	**m**	***User Defined***	**Comments:**	**existing case**

2.3 END SILL

Toe Baffle Distance From Chute Toe Factor	L_1/y_2	N/A	-	*INTERPOLATE*	**Comments:**	
Minimum Toe Baffle Distance From Chute Toe	$L_{1,min}$	N/A	m	L_1/y_2*y_2	**Comments:**	
Toe Baffle Distance From Chute Toe	$\mathbf{L_1}$	**0.00**	**-**	***User Defined***	**Comments:**	**existing case**
Height Ratio	η_1	1.25	-			
Minimum End Sill Height	$H_{s,min}$	1.12	-	η_1*y_1	**Comments:**	
End Sill Height	$\mathbf{H_s}$	**0.00**	**-**	***User Defined***	**Comments:**	**existing case**
Minimum End Sill Thickness	$T_{es,min}$	0.00	m	$0.2*H_s$	**Comments:**	
End Sill Thickness	$\mathbf{T_{es}}$	**0.00**	**-**	***User Defined***	**Comments:**	**existing case**
Minimum End Sill Block Width	$W_{es,min}$	N/A	m	$\alpha*y_2$	**Comments:**	
End Sill Block Width	$\mathbf{W_{es}}$	**0.00**	**-**	***User Defined***	**Comments:**	**existing case**
Minimum End Sill Block Spacing	$S_{es,min}$	N/A	-	$\alpha*y_2$	**Comments:**	
End Sill Block Spacing	$\mathbf{S_{es}}$	**0.00**	**-**	***User Defined***	**Comments:**	**existing case**
Number of Blocks	N_{es}	N/A	-	$(B+S_{es})/(S_{es}+W_{es})$		
Spacing @ End	s_e	0.00	-	$(B+S_{es})/(S_{es}+W_{es})$		

Maynord (1990)

The Rock Manual - CIRIA C683 (2007), 2nd Edition. Page 651.

Recommended for the design of riprap

$$D_{30} = S_f C_s C_v C_T y \left[\left(\frac{1}{s-1} \right)^{0.5} \frac{U_d}{\sqrt{(K_1 g y)}} \right]^{2.5}$$

where: D30 is the characteristic riprap size of which 30% is finer by weight
Sf is a safety factor
Cs is a stability coefficient
Cv is a velocity distribution coefficient
Ct is a blanket thickness coefficient
s is the relative density of stone
Ud is the depth averaged flow velocity
K1 is a side slope correction factor
alpha is the angle of the bank to the horizontal
g is the acceleration due to gravity

Enter Data

Safety factor	Sf	1.5	(1.5 suggested)
Rock shape	A or R	R	(Angular or Rounded)
Location	S or D	D	(Straight channel or D/S of structure)
Thickness coeff	Ct	1	(1.0 suggested)
Rel. density of stone	s	2.65	
Angle of bank to horiz.	alpha	45	
Accel due to gravity	g	9.807	

Depth averaged velocity	Ud1	1.49	m/s
(use 3 to test sensitivity)	Ud2	2.35	m/s
	Ud3	3.06	m/s
	Ud4	3.69	m/s
	Ud5	4.26	m/s
	Ud6	4.78	m/s
	Ud7	5.27	m/s
	Ud8	5.74	m/s
	Ud9	6.18	m/s
	Ud10	6.59	m/s

Water depths	y1	0.30	m
	y2	0.60	m
	y3	0.90	m
	y4	1.20	m
	y5	1.50	m
	y6	1.80	m
	y7	2.10	m
	y8	2.40	m
	y9	2.70	m
	y10	3.00	m

Results

Stability coefficient	Cs	0.375
Vel. distribution coeff	Cv	1.25
COT alpha		1.000
Side slope correction	K1	0.416

Water depth, y (m)	Velocity, Ud (m/s)	D30 (m)	D50 (m)	Dn50 (min)	Dn50 (max)
0.3	1.485746161	0.24	0.34	0.28	0.31
0.6	2.345969986	0.62	0.89	0.75	0.81
0.9	3.058061436	1.09	1.56	1.31	1.42
1.2	3.685578544	1.62	2.31	1.94	2.10
1.5	4.255158398	2.19	3.13	2.63	2.85
1.8	4.781256906	2.80	4.00	3.36	3.64
2.1	5.272865326	3.44	4.91	4.13	4.47
2.4	5.73607816	4.11	5.87	4.93	5.34
2.7	6.175280015	4.79	6.85	5.75	6.23
3	6.593766666	5.50	7.86	6.60	7.15

Pilarczyk (1990)

The Rock Manual - CIRIA C683 (2007), 2nd Edition. Page 649.

Recommended for the design of riprap, cabled concrete blocks, box gabions and asphalt mattresses

$$\boxed{D_{n50} = \frac{\mu}{(s-1)}\frac{0.035}{\Psi_{CR}}\frac{K_T K_Y}{K_S}\frac{U^2}{2g}}$$

where:

μ is the stability correction factor

s is the relative density of the revetment

$\boxed{\Psi \quad CR}$ is the stability factor

D is the characteristic size of the protection
Kt is a turbulence factor
Kh is a depth factor
Ks is the slope factor
Ud is the depth averaged flow velocity
g is the acceleration due to gravity

Enter Data

Location	C or E	C	Continuous protection or Edges and transitions
Relative density of stone	s	2.65	
Porosity of stone	n	0.4	(suggest 0.4 for stone and sand)
Type of revetment	R or G	R	Riprap or Gabions
Bank slope	alpha	0	
Internal friction angle	sigma	45	
Longitudinal channel slope	beta	0.00176	
Turbulence factor	N or H	H	Normal or High
Accel due to gravity	g	9.807	m/s2
Flow decvelopment			

Depth averaged velocity	Ud1	1.49	m/s
	Ud2	2.35	m/s
	Ud3	3.06	m/s
	Ud4	3.69	m/s
	Ud5	4.26	m/s
	Ud6	4.78	m/s
	Ud7	5.27	m/s
	Ud8	5.74	m/s
	Ud9	6.18	m/s
	Ud10	6.59	m/s

Water depths	y1	0.30	m
	y2	0.60	m
	y3	0.90	m
	y4	1.20	m
	y5	1.50	m
	y6	1.80	m
	y7	2.10	m
	y8	2.40	m
	y9	2.70	m
	y10	3.00	m

Results

Stability correction factor		1
Stability factor		0.035
Relative density of revetment		1.65
Side slope term	Kd	1.000
Longitudinal slope	k1	1.000
Slope factor	Ks	1.000
Turbulence factor	Kt	1.500

tan alpha	0.000
cos alpha	1.000
tan sigma	1.000
sin sigma - beta	0.707
sin sigma	0.707

Estimate Dn50	Depth averaged velocity Ud (m/s)	Depth y (m)	Depth factor Kh	Dn50 (m)	D50 (min)	D50 (max)	D50 (ave)	D30 (min)	D30 (max)
0.08	1.485746161	0.3	0.764	0.078	0.09	0.09	0.09	0.06	0.07
0.21	2.345969986	0.6	0.808	0.206	0.23	0.25	0.24	0.16	0.17
0.36	3.058061436	0.9	0.833	0.361	0.40	0.43	0.41	0.28	0.30
0.54	3.685578544	1.2	0.851	0.536	0.59	0.64	0.61	0.41	0.45
0.73	4.255158398	1.5	0.865	0.726	0.80	0.86	0.83	0.56	0.60
0.93	4.781256906	1.8	0.876	0.928	1.02	1.11	1.06	0.71	0.77
1.14	5.272865326	2.1	0.885	1.141	1.25	1.36	1.31	0.88	0.95
1.36	5.73607816	2.4	0.893	1.362	1.50	1.62	1.56	1.05	1.13
1.59	6.175280015	2.7	0.899	1.590	1.75	1.89	1.82	1.22	1.32
1.82	6.593766666	3	0.905	1.824	2.00	2.17	2.09	1.40	1.52

Escarameia and May (1992) - HR Wallingford

The Rock Manual - CIRIA C683 (2007), 2nd Edition. Page 650.

Recommended for the design of riprap, loose or interlocking concrete blocks and gabion mattresses.

$$D_{n50} = C\frac{U_b{}^2}{2g(s-1)}$$

where: Dn50 is the characteristic size of stone

Dn50 is the size of the equivalent cube

$$D_{n50} = \left(\frac{W_{50}}{\rho_s}\right)^{\frac{1}{3}}$$

W50 is the weight of particle, Ps is the density of stone

C is a coefficient that takes account of the turbulence intensity TI

g is acceleration due to gravity

s is the relative density of the revetment material

Ub is the velocity near the bend (at 10% of the water depth above the bed)

Turbulence Levels

Situation	Turbulence Level	
	Qualitative	TI
Straight river or channel reaches and wide natural bends (R/W>26)	Normal (low)	0.12
Edge of revetments in straight reaches	Normal (higher)	0.20
Bridge piers, caissons and groynes; transitions	Medium to high	0.35-0.50
Downstream of hydraulic structures (weirs, culverts, stilling basins)	Very high	0.60

where: R is the centreline radius of bend

W is the water surface width at the upstream end of the bend

Values of C

Type of revetment	Value of C	Observations
Riprap	12.3TI-0.20	Valid for TI>=0.05
Concrete blocks	9.22TI-0.15	Valid for TI>=0.05
Gabion mattresses	12.3TI-1.65	Valid for TI>=0.12

Enter Data

Turbulence Intensity	TI	0.6	
Type of revetment	R,C or G	R	(Riprap, Concrete or Gabion)
Relative density of riprap	s	2.65	
Accel due to gravity	g	9.81	m/s2
Depth averaged velocity	Ud1	1.49	m/s
	Ud2	2.35	m/s
	Ud3	3.06	m/s
	Ud4	3.69	m/s
	Ud5	4.26	m/s
	Ud6	4.78	m/s
	Ud7	5.27	m/s
	Ud8	5.74	m/s
	Ud9	6.18	m/s
	Ud10	6.59	m/s

Suggested factor for Ub	47%
Actual	60%

Results

Coefficient C 7.18

Depth averaged velocity	Ub	Ub2	Dn50	D50 (min)	D50 (max)	D50 (ave)	D30 (min)	D30 (max)
1.486	0.891	0.795	0.18	0.19	0.21	0.20	0.14	0.15
2.346	1.408	1.981	0.44	0.48	0.52	0.50	0.34	0.37
3.058	1.835	3.367	0.75	0.82	0.89	0.85	0.57	0.62
3.686	2.211	4.890	1.08	1.19	1.29	1.24	0.83	0.90
4.255	2.553	6.518	1.45	1.59	1.72	1.65	1.11	1.20
4.781	2.869	8.230	1.83	2.01	2.17	2.09	1.40	1.52
5.273	3.164	10.009	2.22	2.44	2.64	2.54	1.71	1.85
5.736	3.442	11.845	2.63	2.89	3.13	3.01	2.02	2.19
6.175	3.705	13.728	3.04	3.35	3.62	3.49	2.34	2.54
6.594	3.956	15.652	3.47	3.81	4.13	3.97	2.67	2.89

Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 999 Summary Sheet.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

Operational Scenario

Number of Operational Gates	11.00		
Flow	317.19	m^3s^{-1}	**Q500=11,200cfs**
Gage Level	1.87	m	**6.15 ft**
River Level	27.75	mAD	**91.05ft (6.15ft gage)**
Gate Opening	2.13	m	**7ft**
Stilling Basin	As existing		

Downstream Rating Curve

Depth of Flow	Water Level	Discharge	Average Velocity	Froude
D	Zwl	Q	v	Fr
m	mAD	m^3s^{-1}	ms-1	-
0.00	24.40	0.00	0.00	0.00
0.30	24.70	30.02	1.49	0.87
0.60	25.00	95.23	2.35	0.97
0.90	25.30	187.03	3.06	1.03
1.20	25.60	301.88	3.69	1.07
1.50	25.90	437.57	4.26	1.11
1.80	26.20	592.59	4.78	1.14
2.10	26.50	765.77	5.27	1.16
2.40	26.80	956.17	5.74	1.18
2.70	27.10	1163.06	6.18	1.20
3.00	27.40	1385.80	6.59	1.22

Radial Gate with Raised Cill

Max Unit Discharge	qmax	6.22	m^2s^{-1}
Design Unit Discharge	q	6.19	m^2s^{-1}
Flow	Qmax	333.82	m^3s^{-1}

Stilling Basin Level Design

Condition 1 - 100% Design Flow with 100% Design Flow Downstream

Condition 2 - 10% Design Flow with 0% Design Flow Downstream

Condition 3 - 30% Design Flow with 30% Design Flow Downstream

Summary of Critical Basin

Basin Level	22.90	mAD
Critical Condition	Condition 1	
Froude	11.38	
Critical Condition	Condition 2	

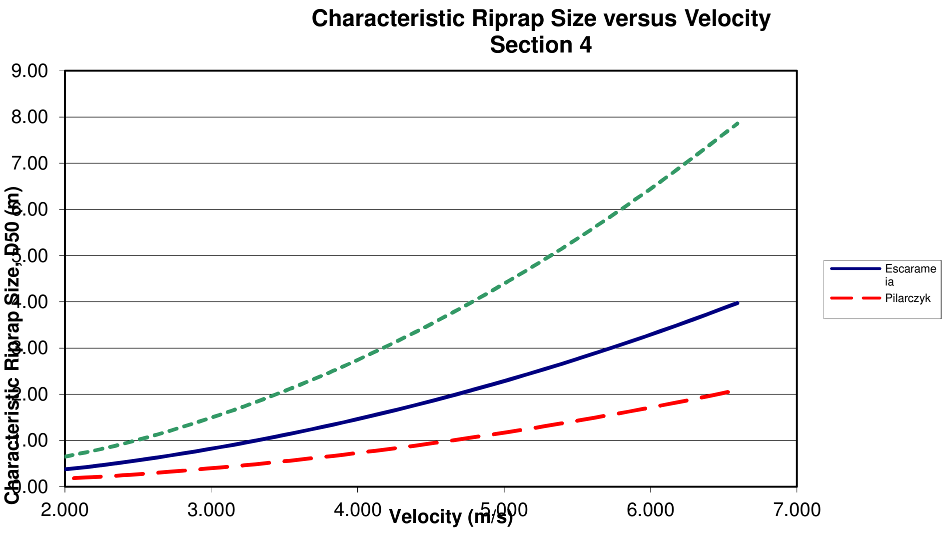
Project **Priest Lake Dam**					
Calculations for 999 Summary Sheet.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

Conjugate Depth	2.97	mAD
Critical Condition	Condition 1	
Max Basin Length	9.40	mAD
Critical Condition	Condition 1	

USBR Stilling Basins

Minimum Basin Length	11.95	m
Minimum End Sill Height	1.12	m
Minimum End Sill Thickness	0.00	m

Erosion Stone Sizing



Project **Priest Lake Dam**					
Calculations for 000A Cover Sheet.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

Operational Scenario

Number of Operational Gates	11		
Flow	317.19	m^3s^{-1}	**Q500=11,200cfs**
Gage Level	1.87452	m	**6.15 ft**
River Level	27.75	mAD	**91.05ft (6.15ft gage)**
Gate Opening	2.134	m	**7ft**
Stilling Basin	Type IVA		

Downstream Rating Curve

A downstream rating curve was produced to enable us identify the tail water level versus discharge assuming a trapezoidal channel.
A bed width of 220 ft was taken using a side slope of 1:1, bed slope of 0.01 and Manning's coefficient of 0.03 as per the original design calculation. The bed level was taken as 80.05 ft, the level of the concrete apron.
Using basic geometry for various depths of flows, the flow area and hydraulic radius were calculated. These could then be used to calculate the discharge.

Radial Gate with Raised Cill

The gate width (16 ft), cill level (81.05 ft) and gate height (7 ft) were taken as per original construction drawings. The safety factor was assumed to be 1.
Assuming unsubmerged conditions and a maximum opening of the 7ft high radial gate, the maximum unit discharge can then be calculated.

Stilling Basin Level Design

Three worst-case scenarios were considered for the stilling basin design. These are:
- 100% flow both upstream and downstream of the structure, to simulate design flow conditions
- 10% upstream flow with 0% flow downstream to simulate sudden gate opening
- 30% upstream flow with 30% downstream flow

A stilling basin safety factor of 1.1 was used for this calculation. Downstream conditions for the various scenarios can be determined from the downstream rating curve. Approach velocities were estimated using the inflow and height of the gates.
Head differences across the gate could be calculated by using the known qate levels and velocities upstream and downstream of the structure. This can be used to determine various parameters about the flow at the glasis and conjugate depth of the given scenario. Based on the Froude number, the performance of the stilling basin can be determined.
For the scenarios the minimum basin level, maximum Froude number, maximum conjugate depth and maximum free jump length are selected as the parameters required for the critical basin.

USBR Stilling Basins

This was undertaken using the USBR Design of Small Dams for a type IVA stilling basin. Along with the parameters determined from the level design, the required basin and end sill dimensions were determined.

Erosion Stone Sizing

Erosion stone sizing was undertaken using The Rock Manual with the Escarameia, Pilarczyk and Maynord methods. The relative density of riprap was taken as 2.65 and the angle of the bank to the horizontal was taken as 45 degrees. All other required inputs were taken from geometry of the structure.

Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 001 EVT-HYES-HYD-300 Trapizoidal Rating Curve.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

Manning's Rating Curve

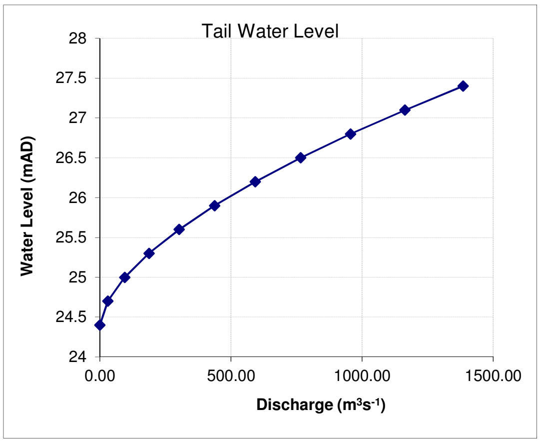
Open Channel Hydraulics, 1959, Ven Te Chow

1. INPUTS

Bed Width	B	67.056	m	**Comments:**	**220 ft as per original calculation**
Side Slope	s	1	H:1V	**Comments:**	**As per original calc**
Bed Slope	i	0.01	m/m	**Comments:**	**As per original calc**
Manning's Coefficient	n	0.03	$sm^{-1/3}$	**Comments:**	**As per original calc**
Bed Level	Zb	24.39924	mAD	**Comments:**	**80.05 ft**
Max Depth	Dmax	3	m	**Comments:**	

2. OUTPUTS

#	Depth of Flow	Flow Area	Hydraulic Radius	Discharge	Water Level	Average Velocity	Froude
n	D	A	P	Q	Zwl	v	Fr
-	m	m^2	m	m^3s^{-1}	mAD	ms-1	-
-	-	$D_n*(B+D_n*s)$	$B+2*D_n*\sqrt{(1+s^2)}$	$A(A/P)^{2/3}*\sqrt{(i)}$ / n	D_n+Z_b	Q/A	$v/\sqrt{(g*D_n)}$
1	0	0	67.06	0.00	24.39924	0	0
2	0.3	20.2068	67.90	30.02	24.69924	1.49	0.87
3	0.6	40.5936	68.75	95.23	24.99924	2.35	0.97
4	0.9	61.1604	69.60	187.03	25.29924	3.06	1.03
5	1.2	81.9072	70.45	301.88	25.59924	3.69	1.07
6	1.5	102.834	71.30	437.57	25.89924	4.26	1.11
7	1.8	123.9408	72.15	592.59	26.19924	4.78	1.14
8	2.1	145.2276	73.00	765.77	26.49924	5.27	1.16
9	2.4	166.6944	73.84	956.17	26.79924	5.74	1.18
10	2.7	188.3412	74.69	1163.06	27.09924	6.18	1.20
11	3	210.168	75.54	1385.80	27.39924	6.59	1.22



Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 002 EVT-HYES-HYD-561 Radial Gate with Raised Cill.	Division	WCD	File No.		
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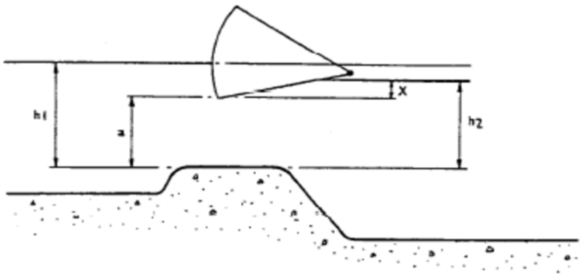
Radial Gate With Cill

Hydraulic Design of Canal Structures - MMP August 1985

USBR - Design of Small Small Dams

1. INPUTS



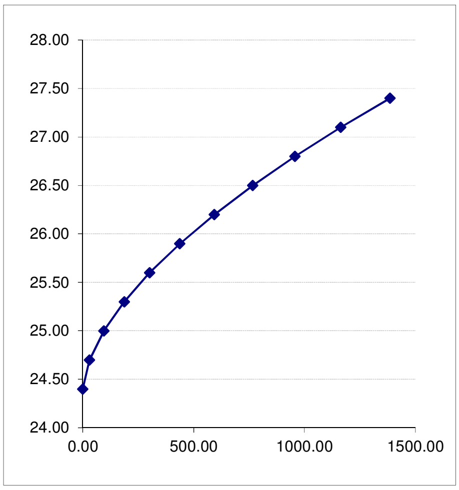
1. Hydraulic Design of Canal Structures, Sept 1986, MM & Partners p. 1 - p. 11

2. Design of Small Dams - USBR

1.1 GENERAL

Design Discharge	Q	317.19	m^3s^{-1}	**Comments:**	**Q500=11,200cfs**
Number of Gates	N_g	11	-	**Comments:**	
Gate Width	w	4.88	m	**Comments:**	**16 ft**
Cill Level	Z_c	24.70	mAD	**Comments:**	**81.05ft**
Max Upstream Water Level	Z_{uswl}	27.75	mAD	**Comments:**	**91.05ft (6.15ft gage)**
Structure Flow Saftey Factor	S_f	1	-	**Comments:**	
Maximum gate opening (as percentage of max head)	μ	44%	-	**Comments:**	**Maximum opening for design dischage**
Design Gate Height	a	2.134	-	**Comments:**	**7ft**
Design Downstream Water Level	Z_{dswl}	25.87	mAD	**Comments:**	

	Level	Flow	Velocity
1	mAD	m3/s	m/s
2	24.40	0.00	0.00
3	24.70	30.02	1.49
4	25.00	95.23	2.35
5	25.30	187.03	3.06
6	25.60	301.88	3.69
7	25.90	437.57	4.26
8	26.20	592.59	4.78
9	26.50	765.77	5.27
10	26.80	956.17	5.74
11	27.10	1163.06	6.18
12	27.40	1385.80	6.59
13			
14			



1.2 Effective Crest Coefficients

Project **Priest Lake Dam**				MOTT MACDONALD	
Calculations for 002 EVT-HYES-HYD-561 Radial Gate with Raised Cill.	Division	WCD	File No.		
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Pier Contraction Coefficient	k_p	0.020	-	**Comments:**	**USBR Design of Small Dams, square nosed piers**
Abutment Contraction Coefficient	k_a	0.2	-	**Comments:**	**USBR Design of Small Dams, square abutments**

2. OUTPUTS

2.1 GENERAL

Number of Piers	N	10		N_g-1	**Comments:**
Sum of Gates Width	b'	53.65	m	N_g*w	**Comments:**
Design Factored Discharge	Q_f	317.19	m^3s^{-1}	S_f*Q	**Comments:**
Upstream Head Over Cill	h_1	3.05	m	$Z_{uswl} - Z_c$	**Comments:**
Downstream Head Over Cill	h_2	1.165	m	$Z_{dswl} - Z_c$	**Comments:**
Effective Crest Length	b	51.21	m	$b'-2*(N*K_p+k_a)*h_1$	**Comments:**

2.2 FREE FLOW CALCULATIONS

Max Height of Gate Opening	$a_{max\ 1}$	1.34	m	$\mu*h_1$	**Comments:**
Opening - DS Water Level Difference	x	0.79	m	$h_2 - a$	**Comments:**
Max Unit Discharge	q_{max}	6.223	m^2s^{-1}	$0.6*a_{max\ 1}*\sqrt{(2*g*h_1)}$	**Comments:**
Design Unit Discharge	q	6.194	m^2s^{-1}	Q_f/b	**Comments:**
Flow	Q_{max}	333.817	m^3s^{-1}	$q*w*N$	**Comments:**
CHECK		OK			**Comments:**
Gate Radius	R	1.788	m	4/3*amax 1	**Comments:**
Depth Above Gate	a'	1.71	m	h1-amax 1	**Comments:**
Angle	θ	17.34	Degrees	$\cos^{-1}(a/R)$	**Comments:**
Vena Contrata Factor	δ	0.65	-		**Comments:**
Vena Contrata	α	0.87	m	$a*\delta$	**Comments:**
Velocity	v	7.14	ms^{-1}	q/α	**Comments:**
Froude Number	Fr	2.44	-		**Comments:**
Conjugate Depth	y_2	3.01	m	$y_1/2*\sqrt{((1+8Fr_1^2)-1)}$	**Comments:**
Water Level	Z_c	27.71	mAD	y_2+Z_c	**Comments:**
Downstream Water Level	Z_{ds}	25.67	mAD		**Comments:**
Condition		UNSUBMERGED			**Comments:**

2.3 SUBMERGED CONDITIONS

Max Height of Gate Opening	$a_{max\ 2}$	2.03	m	$2/3*h_1$	**Comments:**
Opening - DS Water Level Difference	x	-0.87	m	$h_2 - a$	**Comments:**
Head Drop	H	1.88	m	h_1-h_2	
Max Unit Discharge	q_{max}	8.276	m^2s^{-1}	$0.67*a_{max\ 2}*\sqrt{(2*g*(h_1-h_2))}$	

Project **Priest Lake Dam**	MOTT MACDONALD				
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Design Unit Discharge	q	6.194	m^2s^{-1}	Q_f/b'	**Comments:**
CHECK		OK		$q < q_{max}$	**Comments:**
Gate Radius	R	2.709	m	$4/3*a$	**Comments:**
Flow Through One Gate @ 100% a	$Q_{n=1}$	38.525	m^3s^{-1}	$q_{max}*w$	**Comments:**
Design Flow	Q	423.778	m^3s^{-1}		

2.4 DESIGN CONDITIONS

Condition		UNSUBMERGED			**Comments:**
Design Flow	Q_d	333.817	m^3s^{-1}	$q*w*N$	**Comments:**

Project **Priest Lake Dam**			MOTT MACDONALD		
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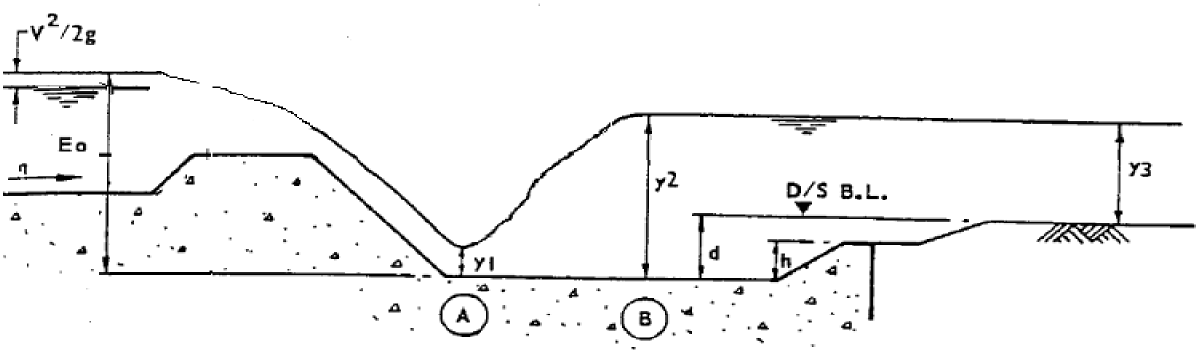
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

Stilling Basin Level Design

1. MMP Hydraulic Design of Canal Structures, 1985, 1-13 - 1-14

1. INPUTS

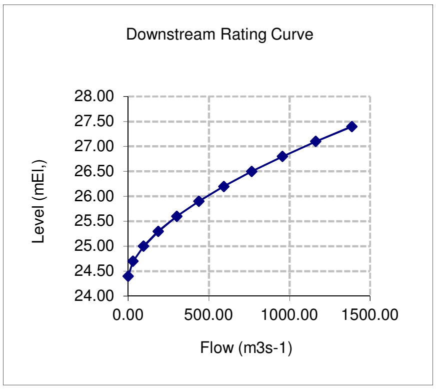


1.1 GENERAL

Design Flow	Q_d	333.8	m^3s^{-1}	**Comments:**	
Flow Safety Factor	S_f	1.00	-	**Comments:**	
Width of Stilling Basin	B	53.64	m	**Comments:**	**16ft*11 no. = 176 ft**
Number of Basins	N	1	-	**Comments:**	
Basin Safety Factor	S_f	1.1	-	**Comments:**	
Bed Level US	Z_{usbl}	24.40	mAD	**Comments:**	**80.05 ft**
Bed Level DS	Z_{dsbl}	24.40	mAD	**Comments:**	**80.05 ft**
Allowance For Retrogression (Percentage of D/S Depth)	ρ	0%	%	**Comments:**	
Gate Width	w_g	53.64	m	**Comments:**	**16 ft * 11 no.**

Downstream Rating Curve (Table 1.1)

#	Level	Flow	Velocity
n	Z_{dswl}	Q	v
-	mAD	m^3s^{-1}	ms^{-1}
1	24.40	0.00	0.00
2	24.70	30.02	1.49
3	25.00	95.23	2.35
4	25.30	187.03	3.06
5	25.60	301.88	3.69
6	25.90	437.57	4.26
7	26.20	592.59	4.78
8	26.50	765.77	5.27
9	26.80	956.17	5.74
10	27.10	1163.06	6.18
11	27.40	1385.80	6.59



Project **Priest Lake Dam**	MOTT MACDONALD				
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1.2 CONDITION 1

1.2.1 General

Percentage of Flow US	$m_{us,1}$	100%		**Comments:**	
Percentage of Flow DS	$m_{ds,1}$	100%		**Comments:**	
Additional flow downstream	$Q_{add,1}$	0.0	m^3s^{-1}	**Comments:**	

1.3.2 Upstream Conditions

Water Level	$Z_{us,1}$	27.8	mAD	**Comments:**	**91.05ft (6.15ft gage)**
Velocity	$v_{us,1}$	0.270	ms^{-1}	**Comments:**	
Maximum Unit Discharge	$q_{max,1}$	6.223	m^2s^{-1}	**Comments:**	

1.3 CONDITION 2

1.3.1 General

Percentage of Flow US	$m_{us,2}$	10%		**Comments:**	
Percentage of Flow DS	$m_{ds,2}$	0%		**Comments:**	
Additional flow downstream	$Q_{add,2}$	0.0	m^3s^{-1}	**Comments:**	

1.3.2 Upstream Conditions

Water Level	$Z_{us,2}$	27.8	mAD	**Comments:**	
Velocity	$v_{us,1}$	0.28	ms^{-1}	**Comments:**	**q/7ft**
Maximum Unit Discharge	$q_{max,1}$	0.594	m^2s^{-1}	**Comments:**	

1.4 CONDITION 3

1.4.1 General

Percentage of Flow US	$m_{us,3}$	30%		**Comments:**	
Percentage of Flow DS	$m_{ds,3}$	30%		**Comments:**	
Additional flow downstream	$Q_{add,3}$	0.0	m^3s^{-1}	**Comments:**	

1.4.2 Upstream Conditions

Water Level	$Z_{us,3}$	27.8	mAD	**Comments:**	
Velocity	$v_{us,3}$	0.84	ms^{-1}	**Comments:**	**q/7ft**
Maximum Unit Discharge	$q_{max,3}$	1.782	m^2s^{-1}	**Comments:**	

Project **Priest Lake Dam**	MOTT MACDONALD				
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2. OUTPUTS

Condition 1 - 100% Design Flow with 100% Design Flow Downstream

Condition 2 - 10% Design Flow with 0% Design Flow Downstream

Condition 3 - 30% Design Flow with 30% Design Flow Downstream

2.1 GENERAL

Factored Design Flow	Q_f	333.8	m^3s^{-1}	Q_d*S_f	**Comments:**
2.1.1 Condition 1					**Comments:**
Factored Discharge US	$Q_{us,1}$	333.8	m^3s^{-1}	$Q_f*m_{us,1}$	**Comments:**
Discharge DS	$Q_{ds,1}$	333.8	m^3s^{-1}	$Q_f*m_{ds,1}$	**Comments:**
DS Water Level	$Z_{ds,1}$	25.7	mAD	*Interpolate (Table 1.1)*	**Comments:**
Velocity @ Q DS	$v_{ds,1}$	3.82	ms^{-1}	*Interpolate (Table 1.1)*	**Comments:**
2.1.2 Condition 2					**Comments:**
Factored Discharge US	$Q_{us,2}$	33.4	m^3s^{-1}	$Q_f*m_{us,2}$	**Comments:**
Discharge DS	$Q_{ds,2}$	0.0	m^3s^{-1}	$Q_f*m_{ds,2}$	**Comments:**
DS Water Level	$Z_{ds,2}$	24.399	mAD	*Interpolate (Table 1.1)*	**Comments:**
Velocity @ Q DS	$v_{ds,2}$	0.00	ms^{-1}	*Interpolate (Table 1.1)*	**Comments:**
2.1.3 Condition 3					**Comments:**
Factored Discharge US	$Q_{us,3}$	100.1	m^3s^{-1}	$Q_f*m_{us,3}$	**Comments:**
Discharge DS	$Q_{us,3}$	100.1	m^3s^{-1}	$Q_f*m_{ds,3}$	**Comments:**
DS Water Level	$Z_{ds,3}$	25.015	mAD	*Interpolate (Table 1.1)*	**Comments:**
Velocity @ Q DS	$v_{ds,3}$	2.38	ms^{-1}	*Interpolate (Table 1.1)*	**Comments:**

2.2 DOWNSTREAM CONDITION

Water Depth DS Condition 1	$D_{ds,1}$	1.271	m	$Z_{dswl}-Z_{bl}$	**Comments:**
Water Depth DS Condition 2	$D_{ds,2}$	0.000	m	$Z_{dswl}-Z_{bl}$	**Comments:**
Water Depth DS Condition 3	$D_{ds,3}$	0.616	m	$Z_{dswl,70}-Z_{bl}$	**Comments:**
Retrogression @ Condition 1	r_1	0.000	m	$\rho*D_{ds,1}$	**Comments:**
Retrogression @ Condition 2	r_2	0.000	m	$\rho*D_{ds,2}$	**Comments:**
Retrogression @ Condition 3	r_3	0.000	m	$\rho*D_{ds,3}$	**Comments:**
Water Level Condition 1 with Retrogression	$Z_{dsl,r}$	25.670	mAD	$Z_{ds,1}-r_1$	**Comments:**
Water Level Condition 2 with Retrogression	$Z_{dsl,r}$	24.399	mAD	$Z_{ds,2}-r_2$	**Comments:**
Water Level Condition 3 with Retrogression	$Z_{dsl,70,r}$	25.015	mAD	$Z_{ds,3}-r_3$	**Comments:**

Project **Priest Lake Dam**	M M MOTT MACDONALD				
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Stilling Basin General Design - Condition 1

1. INPUTS

Design Flow	$Q_{us,1}$	333.82	m^{3}s^{-1}
Width of Stilling Basin	B	53.64	m
Upstream Water Level	$Z_{us,1}$	27.75	mAD
Approach velocity	$v_{us,1}$	0.270	ms^{-1}
Downstream Water Level	$Z_{ds,1}$	25.7	mAD
Downstream Velocity	$v_{ds,1}$	3.82	ms^{-1}
Basin Safety Factor	S_f	1.1	-

2.2 OUTPUTS - CONDITION 1

Flow Intensity	q_1	6.22	m^{2}s^{-1}	$Q_{us,1}/B/N$	**Comments:**
Critical Depth	$d_{c,1}$	1.58	m	$(q_1^2/g)^{1/3}$	**Comments:**
Upstream Velocity Head	$h_{v,1}$	0.00	m	$v_{us,1}^2/2g$	**Comments:**
Downstream Velocity Head	$h_{v2,1}$	0.74	m	$v_{ds,1}^2/2g$	**Comments:**
Upstream Total Head	$E_{o,1}$	27.76	m	$Z_{us,1} + h_{v,1}$	**Comments:**
Downstream Total Head	$E_{2,1}$	26.41	m	$Z_{ds,1} + h_{2v,1}$	**Comments:**
Head Difference Across Gate	$H_{L,1}$	1.34	m	$E_{0,1} - E_{2,1}$	**Comments:**
	H_L/d_c	0.85	m	H_L/d_c	**Comments:**
	a_1	0.24	-	*GOAL SEEK*	**Comments:**
	$v_{1,1}$	0.85	-	$(1-a_1)^3/(2a_1)*((1+2a_1)^2-1)^{-1/3})$	
Congugate Depth	$y_{2,1}$	2.97	m	$4*H_{L,1}*a_1/(1-a_1)^3$	
Velocity @ Congugate Depth	$v_{2,1}$	2.09	ms^{-1}	$q_1/y_{2,1}$	
Depth of Flow at Bottom of Glasis	$y_{1,1}$	0.72	m	$a_1*y_{2,1}$	
Velocity @ Toe of Glasis	$v_{1,1}$	8.66	ms^{-1}	$q_1/y_{1,1}$	**Comments:**
Froude Number	Fr_1	3.26	-	$v_{1,1}/\surd(gy_{1,1})$	**Comments:**
Conjugate Depth Check Calc	$y_{2,1}$	2.97	m	$y_{1,1}/2*\surd((1+8Fr_1^2)-1)$	**Comments:**
Total Energy at Congugate Depth	$E_{1,1}$	3.20	m	$y_{2,1}+d_{c,1}^3/(2*y_{2,1}^2)$	
Basin Level	$\mathbf{Z_{b,1}}$	**22.90**	**mAD**	$E_{2,1}-S_f*E_{1,1}$	**Comments:**
Downstream Jump Level	Z_{dsjl}	25.87	m	$Z_{dsbs} + y_2$	**Comments:**
	L/y_2	5.43	m	*INTERPOLATE*	**Comments:**
	L/y_2	5.43	m	*INTERPOLATE*	**Comments:**
Jump Length	L	16.15	m	L/y_2*y_2	**Comments:**
Free Jump Performance		Wavy			
Basin Type		Type IVA			**Comments:**
Basin Jump Length Factor	L_B/y_2	3.16	m		
Basin Jump Length	L_B	9.40	m		

Project **Priest Lake Dam**	MOTT MACDONALD				
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2.2.1 Design Check (Set Outputs First)

Total Energy	$E_{0,1d}$	3.36	m	$Z_{us,1}+h_{v,1}-Z_b$	**Comments:**
Velocity	$v_{1,1d}$	6.95	ms^{-1}	$\sqrt{(2g^*(E_{0,1d}-y_{1,1d}))}$	**Comments:**
Depth of Flow	$y_{1,1d}$	0.896	m	$q_1/v_{1,1d}$	**Comments:**
Upstream Froude Number	$\mathbf{Fr_{1,1d}}$	**2.344**	-	$v_{1,1d}/\sqrt{(gy_{1,1d})}$	**Comments:**
Conjugate Depth	$y_{2,1d}$	2.55	-	$y_{1,1d}/2^*(\sqrt{(1+8Fr_{1d}^2)}-1)$	
Conjugate Level	$y_{2,1d}$	26.95	-	$y_{1,1d}/2^*(\sqrt{(1+8Fr_{1d}^2)}-1)$	

Project **Priest Lake Dam**	MOTT MACDONALD				
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Stilling Basin General Design - Condition 2

1. INPUTS

Design Flow	$Q_{us,2}$	33.38	m^3s^{-1}
Width of Stilling Basin	B	53.64	m
Upstream Water Level	$Z_{us,2}$	27.75	mAD
Approach velcoity	$v_{us,2}$	0.278	ms^{-1}
Downstream Water Level	$Z_{ds,2}$	24.4	mAD
Downstream Velocity	$v_{ds,2}$	0.00	ms^{-1}
Basin Safety Factor	S_f	1.1	-

2.3 OUTPUTS - CONDITION 2

Flow Intensity	q_2	0.62	m^2s^{-1}	$Q_{us,2}/B/N$	**Comments:**
Critical Depth	$d_{c,2}$	0.34	m	$(q_2{}^2/g)^{1/3}$	**Comments:**
Upstream Velocity Head	$h_{v,2}$	0.00	m	$v_{us,2}{}^2/2g$	**Comments:**
Downstream Velocity Head	$h_{v2,2}$	0.00	m	$v_{ds,2}{}^2/2g$	**Comments:**
Upstream Total Head	$E_{o,2}$	27.76	m	$Z_{us,2} + h_{v,2}$	**Comments:**
Downstream Total Head	$E_{2,2}$	24.40	m	$Z_{ds,2} + h_{2v,2}$	**Comments:**
Head Difference Across Gate	$H_{L,2}$	3.36	m	$E_{0,2}-E_{2,2}$	**Comments:**
	H_L/d_c	9.86	m	H_L/d_c	**Comments:**
	a_2	0.06	-	GOAL SEEK	**Comments:**
	$v_{1,2}$	9.86	-	$(1-a_2)^3/(2a_2)*((1+2a_2)^2-1)^{-1/3})$	
Congugate Depth	$y_{2,2}$	1.05	m	$4*H_{L,2}*a_2/(1-a_2)^3$	
Velocity @ Congugate Depth	$v_{2,2}$	0.59	ms^{-1}	$q_2/y_{2,2}$	
Depth of Flow at Bottom of Glacis	$y_{1,2}$	0.07	m	$a_2*y_{2,2}$	
Velocity @ Toe of Glacis	$v_{1,2}$	9.25	ms^{-1}	$q_2/y_{1,2}$	**Comments:**
Froude Number	Fr_2	11.38	-	$v_{1,2}/\surd(gy_{1,2})$	**Comments:**
Conjugate Depth Check Calc	$y_{2,2}$	1.05	m	$y_{1,2}/2*\surd((1+8Fr_2{}^2$	**Comments:**
Total Energy at Congugate Depth	$E_{1a,2}$	1.07	m	$y_{2,2}+d_{c,2}{}^3/(2*y_{2,2}{}^2$	**Comments:**
Basin Level	$\mathbf{Z_{dsbs}}$	**23.22**	**mAD**	$E_{2,2}-S_f*E_{1,2}$	**Comments:**
Downstream Jump Level	Z_{dsjl}	24.27	m	$Z_{dsbs} + y_2$	**Comments:**
	L/y_2	6.05	m	INTERPOLATE	**Comments:**
	L/y_2	6.05	m	INTERPOLATE	**Comments:**
Jump Length	L	6.35	m	L/y_2*y_2	**Comments:**
Free Jump Performance		Acceptable Performance			
Basin Type		Type III			**Comments:**
Basin Jump Length Factor	L_B/y_2	2.76	m		
Basin Jump Length	L_B	2.90	m		

Project **Priest Lake Dam**	MOTT MACDONALD				
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2.3.1 Design Check (Set Outputs First)

Total Energy	E_0	3.36	m	$Z_{uswl}+h_v-Z_b$	**Comments:**
Velocity	v_1	8.02	ms^{-1}	$\sqrt{(2g^*(E_0-y_1))}$	**Comments:**
Depth of Flow	y_1	0.078	m	q/v	**Comments:**
Upstream Froude Number	$\mathbf{Fr_1}$	**9.194**	-	$v_1/\sqrt{(gy_1)}$	**Comments:**
Conjugate Depth	y_2	0.97	-	$y_1/2^*(\sqrt{(1+8Fr_1^2)}-1)$	**Comments:**
Conjugate Level	y_2	25.37	-	$y_1/2^*(\sqrt{(1+8Fr_1^2)}-1)$	**Comments:**

Project

Priest Lake Dam

MOTT MACDONALD

Calculations for	Division	WCD	File No.		
003 EVT-HYES-HYD-001 - Stilling Basin Level Design.xls.	Calc by	JM	Date	31/07/2017	Number of sheets
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Stilling Basin General Design - Condition 3

MM Hydraulic Structures, 1985

1. INPUTS

Design Flow	$Q_{us,3}$	100.15	m^3s^{-1}
Width of Stilling Basin	B	53.64	m
Upstream Water Level	$Z_{us,3}$	27.75	mAD
Approach velcoity	$v_{us,3}$	0.835	ms^{-1}
Downstream Water Level	$Z_{ds,3}$	25.0	mAD
Downstream Velocity	$v_{ds,3}$	2.38	ms^{-1}
Basin Safety Factor	S_f	1.1	-

2.4 OUTPUTS - CONDITION 3

Flow Intensity	q_3	1.87	m^2s^{-1}	$Q_{us,3}/B/N$	**Comments:**
Critical Depth	$d_{c,3}$	0.71	m	$(q_3{}^2/g)^{1/3}$	**Comments:**
Upstream Velocity Head	$h_{v,3}$	0.04	m	$v_{us,3}{}^2/2g$	**Comments:**
Downstream Velocity Head	$h_{v2,3}$	0.29	m	$v_{ds,3}{}^2/2g$	**Comments:**
Upstream Total Head	$E_{o,3}$	27.79	mAD	$Z_{us,3} + h_{v,3}$	**Comments:**
Downstream Total Head	$E_{2,3}$	25.30	mAD	$Z_{ds,3} + h_{2v,3}$	**Comments:**
Head Difference Across Gate	$H_{L,3}$	2.48	m	$E_{0,3}-E_{2,3}$	**Comments:**
	H_L/d_c	3.51	m	H_L/d_c	**Comments:**
	a_3	0.12	-	*GOAL SEEK*	**Comments:**
	$v_{1,3}$	3.51	-	$(1-a_3)^3/(2a_3)*((1+2a_3)^2-1)^{-1/3})$	
Congugate Depth	$y_{2,3}$	1.74	m	$4*H_{L,3}*a_3/(1-a_3)^3$	
Velcoity @ Congugate Depth	$v_{2,3}$	1.07	ms^{-1}	$q_3/y_{2,3}$	
Depth of Flow at Bottom of Glasis	$y_{1,3}$	0.21	m	$a_3*y_{2,3}$	
Velocity @ Toe of Glasis	$v_{1,3}$	8.94	ms^{-1}	$q_3/y_{1,3}$	**Comments:**
Froude Number	Fr_3	6.25	-	$v_{1,3}/\sqrt{}(gy_{1,3})$	**Comments:**
Conjugate Depth Check Calc	$y_{2,3}$	1.74	m	$y_{1,3}/2*\sqrt{}((1+8Fr_3{}^2)-1)$	**Comments:**
Total Energy at Congugate Depth	$E_{1a,3}$	1.80	m	$y_{2,3}+d_{c,3}{}^3/(2*y_{2,3}{}^2)$	
Basin Level	$\mathbf{Z_{dsbs}}$	**23.32**	**mAD**	$E_{2,3}-S_f*E_{1,3}$	**Comments:**
Downstream Jump Level	Z_{dsjl}	25.07	m	$Z_{dsbs} + y_2$	**Comments:**
	L/y_2	6.08	m	*INTERPOLATE*	**Comments:**
	L/y_2	6.08	m	*INTERPOLATE*	**Comments:**
Jump Length	L	10.60	m	L/y_2*y_2	**Comments:**
Free Jump Performance		Best Performance			
Basin Type		Type III			**Comments:**
Basin Jump Length Factor	L_B/y_2	2.48	m		
Basin Jump Length	L_B	4.33	m		

Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 003 EVT-HYES-HYD-001 - Stilling Basin Level Design.xls.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

2.4.1 Design Check (Set Outputs First)

Total Energy	E_0	3.39	m	$Z_{uswl}+h_v-Z_b$	**Comments:**
Velocity	v_1	7.86	ms^{-1}	$\sqrt{(2g*(E_0-y_1))}$	**Comments:**
Depth of Flow	y_1	0.237	m	q/v	**Comments:**
Upstream Froude Number	Fr_1	**5.152**	-	$v_1/\sqrt{(gy_1)}$	**Comments:**
Conjugate Depth	y_2	1.62	-	$y_1/2*(\sqrt{(1+8Fr_1^2)}-1)$	**Comments:**
Conjugate Level	y_2	26.01	-	$y_1/2*(\sqrt{(1+8Fr_1^2)}-1)$	**Comments:**

Project **Priest Lake Dam**					
Calculations for 003 EVT-HYES-HYD-001 - Stilling Basin Level Design.xls.	Division	WCD	File No.		
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Stilling Basin Check - Outputs

MM Hydraulic Structures, 1985

2. OUTPUTS

Condition 1 - 100% Design Flow with 100% Design Flow Downstream

Condition 2 - 10% Design Flow with 0% Design Flow Downstream

Condition 3 - 30% Design Flow with 30% Design Flow Downstream

2.1 SUMMARY OF CRITICAL BASIN

2.2.1 Basin Level					
Basin Level	Z_{bs}	22.896	mAD	*MIN(C1,C2,C3)*	**Comments:**
Critical Condition		Condition 1	-		**Comments:**
Basin Level Selected	Z_b	**24.400**	mAD	***USER DEFINED***	**Comments:**
2.2.2 Froude Number					
Froude		11.379	-	*MAX(C1,C2,C3)*	**Comments:**
Critical Froude Condition		Condition 2	-		**Comments:**
2.2.3 Conjugate Depth					
Conjugate Depth		2.974	m	*MAX(C1,C2,C3)*	**Comments:**
Max Conjugate Depth Condition		Condition 1	-		**Comments:**
2.2.4 Designed Basin Length					
Max Basin Length		9.397	-	*MAX(C1,C2,C3)*	**Comments:**
Max Basin Length Condition		Condition 1	-		**Comments:**
2.2.5 Free Jump Length					
Max Free Jump Length		16.151	m	*MAX(C1,C2,C3)*	**Comments:**
2.2.6 General					
Minimum Basin Length		17.845	m	6*y2,crit	**Comments:**

Project **Priest Lake Dam**	MOTT MACDONALD				
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	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

Stilling Basin Check - Gates Open Check

MM Hydraulic Structures, 1985

2. Outputs

2.1 100% MAX GATE DISCHARGE

Maximum Unit Discharge Per Gate	q_{max}	6.22	m^2s^{-1}		**Comments:**	
Maximum Water Level - Basin Level Difference	E_0	3.35	m	$Z_{uswl,c1}-Z_{bs}$	**Comments:**	
Depth of Flow at Toe of Glasis	y_1	0.896	m	$q/(\sqrt{(2*g*(E_0-y_1))})$	**Comments:**	
Velocity @ Toe	v_1	6.94	ms^{-1}	q/y_1	**Comments:**	
Froude @ Toe	Fr	2.34	-	$v_1/\sqrt{(g*y_1)}$	**Comments:**	
Conjugate Depth	y_2	2.55	m	$y_1/2*(\sqrt{(1+8*Fr^2)}-1)$	**Comments:**	
Water Level	Z_{cg}	25.45	mAD	$Z_{bs}+y_2$	**Comments:**	
Required Flow in Downstream Channel	q_{req}	244.53	m^3s^{-1}	*INTERPOLATE(Table 1.1)*	**Comments:**	
Number of Gates Required to Aquire DS Level	N_g	0.7	-	$Q_{req}/(q_{max}*w_g)$	**Comments:**	Minimum number of gates = 1

2.2 50% Max Gate Discharge

Maximum Unit Discharge Per Gate	q_{max}	3.11	m^2s^{-1}		**Comments:**	
Maximum Water Level - Basin Level Difference	E_0	3.35	m	$Z_{uswl,c1}-Z_{bs}$	**Comments:**	
Depth of Flow at Toe of Glasis	y_1	0.41	m	$q/(\sqrt{(2*g*(E_0-y_1))})$	**Comments:**	
Velocity @ Toe	v_1	7.60	ms^{-1}	q/y_1	**Comments:**	
Froude @ Toe	Fr	3.79	-	$v_1/\sqrt{(g*y_1)}$	**Comments:**	
Conjugate Depth	y_2	2.00	m	$y_1/2*(\sqrt{(1+8*Fr^2)}-1)$	**Comments:**	
Water Level	Z_{cg}	24.90	mAD	$Z_{bs}+y_2$	**Comments:**	
Required Flow in Downstream Channel	q_{req}	72.94	m^3s^{-1}	*INTERPOLATE(Table 1.1)*	**Comments:**	
Number of Gates Required to Aquire DS Level	N_g	0.4	-	$q_{req}/(q_{max}*w_g)$	**Comments:**	Minimum number of gates = 1

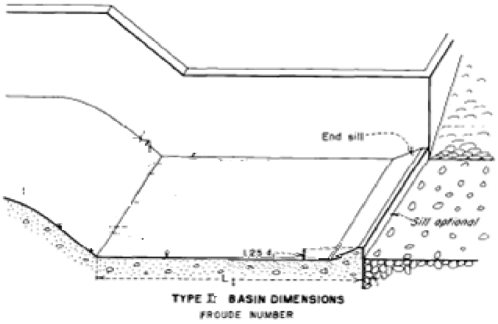
Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 004A EVT-HYES-HYD-002 - USBR Stilling Basins.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

USBR Small Dams Stilling Basin Design

1. USBR Small Dams p.387 - 395

2. USBR Low Froude Stilling Basins

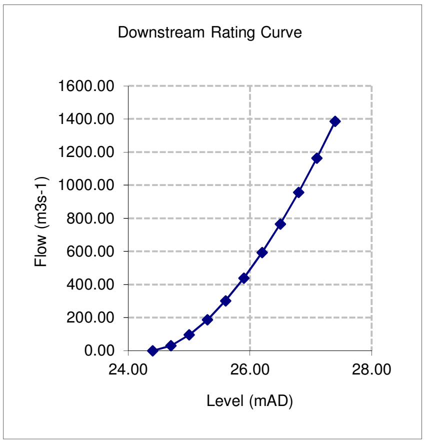
1. INPUTS



Upstream Water Level	Z_{uswl}	27.75	mAD	**Comments:**	**91.05ft (6.15ft gage)**
Upstream Velocity	v_0	0.27	ms^{-1}	**Comments:**	
Basin Level	Z_b	22.90	mAD	**Comments:**	**New level**
Total Flow	Q	333.8	m^3s^{-1}	**Comments:**	
Stilling Basin Width	B	53.645	m	**Comments:**	
Number of Stilling Basins	N	1	-	**Comments:**	
Number of Chute Blocks	Nc	0	-	**Comments:**	
Number of Baffle Blocks	Nb	0	-	**Comments:**	
Basin Type		Type IVA		**Comments:**	
End Sill Width Multiple	α	0.150		**Comments:**	
Glasis Slope	s	2	H:1V	**Comments:**	

Downstream Rating Curve (Table 1.1)

#	Level	Flow	Velcocity
n	Z_{dswl}	Q	v
-	mAD	m^3s^{-1}	ms^{-1}
1	24.40	0.00	0.00
2	24.70	30.02	1.49
3	25.00	95.23	2.35
4	25.30	187.03	3.06
5	25.60	301.88	3.69
6	25.90	437.57	4.26
7	26.20	592.59	4.78
8	26.50	765.77	5.27
9	26.80	956.17	5.74
10	27.10	1163.06	6.18
11	27.40	1385.80	6.59



2. OUTPUTS

Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for	Division	WCD	File No.		
004A EVT-HYES-HYD-002 - USBR Stilling Basins.	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

2.1 GENERAL

Unit Flow	q	6.22	m^2s^{-1}	Q/(B*N)	**Comments:**	
Upstream Velocity Head	h_v	0.00	m	$v_0^2/(2g)$	**Comments:**	
Total Energy	E_0	4.86	m	$Z_{uswl}+h_v-Z_b$	**Comments:**	
Velocity	v_1	9.05	ms^{-1}	$\sqrt{(2g*(E_0-y_1))}$	**Comments:**	
Depth of Flow	y_1	0.688	m	q/v	**Comments:**	
Upstream Froude Number	Fr_1	**3.483**	-	$v_1/\sqrt{(gy_1)}$	**Comments:**	
Conjugate Depth	y_2	3.06	-	$y_1/2*(\sqrt{(1+8Fr_1^2)}-1)$	**Comments:**	
Conjugate Level	Zc	25.96	-	*y2+Zb*	**Comments:**	
Initial Design Flow	Qi	467.79	m^3s^{-1}	*INTERPOLATE*	**Comments:**	
Free Jump Length Factor		5.57	m	*INTERPOLATE*	**Comments:**	
Free Jump Length		17.05	m	L/y_2*y_2	**Comments:**	
MM Basin Length		18.37	m	$6*y_{2,design}$	**Comments:**	
Rough Downstream Water Level		25.67	mEL	*INTERPOLATE*	**Comments:**	

2.2 BASIN DIMENSIONS

2.2.1 Basin

Basin Jump Length Factor	F_L	3.17		*INTERPOLATE*	**Comments:**	
Minimum Basin Length	L_B	9.70	m	F_L*y_2	**Comments:**	
Design Length	**L**	**2.80**	**m**	***User Defined***	**Comments:**	**existing case**

2.3 END SILL

Toe Baffle Distance From Chute Toe Factor	L_1/y_2	#NUM!	-	*INTERPOLATE*	**Comments:**	
Minimum Toe Baffle Distance From Chute Toe	$L_{1,min}$	#NUM!	m	L_1/y_2*y_2	**Comments:**	
Toe Baffle Distance From Chute Toe	L_1	**0.00**	**-**	***User Defined***	**Comments:**	**existing case**
Height Ratio	η_1	0.20	-	*0.2*		
Minimum End Sill Height	$H_{s,min}$	0.61	-	η_1*y_1	**Comments:**	
End Sill Height	H_s	**0.00**	**-**	***User Defined***	**Comments:**	**existing case**
Minimum End Sill Thickness	$T_{es,min}$	0.00	m	$0.2*H_s$	**Comments:**	
End Sill Thickness	T_{es}	**0.00**	**-**	***User Defined***	**Comments:**	**existing case**
Minimum End Sill Block Width	$W_{es,min}$	0.46	m	$\alpha*y_2$	**Comments:**	
End Sill Block Width	W_{es}	**0.00**	**-**	***User Defined***	**Comments:**	**existing case**
Minimum End Sill Block Spacing	$S_{es,min}$	0.46	-	$\alpha*y_2$	**Comments:**	
End Sill Block Spacing	S_{es}	**0.00**	**-**	***User Defined***	**Comments:**	**existing case**
Number of Blocks	N_{es}	#DIV/0!	-	$(B+S_{es})/(S_{es}+W_{es})$		
Spacing @ End	s_e	0.00	-	$(B+S_{es})/(S_{es}+W_{es})$		

Maynord (1990)

The Rock Manual - CIRIA C683 (2007), 2nd Edition. Page 651.

Recommended for the design of riprap

$$D_{30} = S_f C_s C_v C_T y \left[\left(\frac{1}{s-1} \right)^{0.5} \frac{U_d}{\sqrt{(K_1 g y)}} \right]^{2.5}$$

where:
D30 is the characteristic riprap size of which 30% is finer by weight
Sf is a safety factor
Cs is a stability coefficient
Cv is a velocity distribution coefficient
Ct is a blanket thickness coefficient
s is the relative density of stone
Ud is the depth averaged flow velocity
K1 is a side slope correction factor
alpha is the angle of the bank to the horizontal
g is the acceleration due to gravity

Enter Data

Safety factor	Sf	1.5	(1.5 suggested)
Rock shape	A or R	R	(Angular or Rounded)
Location	S or D	D	(Straight channel or D/S of structure)
Thickness coeff	Ct	1	(1.0 suggested)
Rel. density of stone	s	2.65	
Angle of bank to horiz.	alpha	45	
Accel due to gravity	g	9.807	

Depth averaged velocity	Ud1	1.49	m/s
(use 3 to test sensitivity)	Ud2	2.35	m/s
	Ud3	3.06	m/s
	Ud4	3.69	m/s
	Ud5	4.26	m/s
	Ud6	4.78	m/s
	Ud7	5.27	m/s
	Ud8	5.74	m/s
	Ud9	6.18	m/s
	Ud10	6.59	m/s

Water depths	y1	0.30	m
	y2	0.60	m
	y3	0.90	m
	y4	1.20	m
	y5	1.50	m
	y6	1.80	m
	y7	2.10	m
	y8	2.40	m
	y9	2.70	m
	y10	3.00	m

Results

Stability coefficient	Cs	0.375
Vel. distribution coeff	Cv	1.25
COT alpha		1.000
Side slope correction	K1	0.416

Water depth, y (m)	Velocity, Ud (m/s)	D30 (m)	D50 (m)	Dn50 (min)	Dn50 (max)
0.3	1.485746161	0.24	0.34	0.28	0.31
0.6	2.345969986	0.62	0.89	0.75	0.81
0.9	3.058061436	1.09	1.56	1.31	1.42
1.2	3.685578544	1.62	2.31	1.94	2.10
1.5	4.255158398	2.19	3.13	2.63	2.85
1.8	4.781256906	2.80	4.00	3.36	3.64
2.1	5.272865326	3.44	4.91	4.13	4.47
2.4	5.73607816	4.11	5.87	4.93	5.34
2.7	6.175280015	4.79	6.85	5.75	6.23
3	6.593766666	5.50	7.86	6.60	7.15

Pilarczyk (1990)

The Rock Manual - CIRIA C683 (2007), 2nd Edition. Page 649.

Recommended for the design of riprap, cabled concrete blocks, box gabions and asphalt mattresses

$$\boxed{D_{n50} = \frac{\mu}{(s-1)}\frac{0.035}{\Psi_{CR}}\frac{K_T K_Y}{K_S}\frac{U^2}{2g}}$$

where:

μ is the stability correction factor

s is the relative density of the revetment

$\boxed{\Psi\ CR}$ is the stability factor

D is the characteristic size of the protection
Kt is a turbulence factor
Kh is a depth factor
Ks is the slope factor
Ud is the depth averaged flow velocity
g is the acceleration due to gravity

Enter Data

Location	C or E	C	Continuous protection or Edges and transitions
Relative density of stone	s	2.65	
Porosity of stone	n	0.4	(suggest 0.4 for stone and sand)
Type of revetment	R or G	R	Riprap or Gabions
Bank slope	alpha	0	
Internal friction angle	sigma	45	
Longitudinal channel slope	beta	0.00176	
Turbulence factor	N or H	H	Normal or High
Accel due to gravity	g	9.807	m/s2
Flow decvelopment			

Depth averaged velocity	Ud1	1.49	m/s
	Ud2	2.35	m/s
	Ud3	3.06	m/s
	Ud4	3.69	m/s
	Ud5	4.26	m/s
	Ud6	4.78	m/s
	Ud7	5.27	m/s
	Ud8	5.74	m/s
	Ud9	6.18	m/s
	Ud10	6.59	m/s

Water depths	y1	0.30	m
	y2	0.60	m
	y3	0.90	m
	y4	1.20	m
	y5	1.50	m
	y6	1.80	m
	y7	2.10	m
	y8	2.40	m
	y9	2.70	m
	y10	3.00	m

Results

Stability correction factor		1
Stability factor		0.035
Relative density of revetment		1.65
Side slope term	Kd	1.000
Longitudinal slope	k1	1.000
Slope factor	Ks	1.000
Turbulence factor	Kt	1.500

tan alpha	0.000
cos alpha	1.000
tan sigma	1.000
sin sigma - beta	0.707
sin sigma	0.707

Estimate Dn50	Depth averaged velocity Ud (m/s)	Depth y (m)	Depth factor Kh	Dn50 (m)	D50 (min)	D50 (max)	D50 (ave)	D30 (min)	D30 (max)
0.08	1.485746161	0.3	0.764	0.078	0.09	0.09	0.09	0.06	0.07
0.21	2.345969986	0.6	0.808	0.206	0.23	0.25	0.24	0.16	0.17
0.36	3.058061436	0.9	0.833	0.361	0.40	0.43	0.41	0.28	0.30
0.54	3.685578544	1.2	0.851	0.536	0.59	0.64	0.61	0.41	0.45
0.73	4.255158398	1.5	0.865	0.726	0.80	0.86	0.83	0.56	0.60
0.93	4.781256906	1.8	0.876	0.928	1.02	1.11	1.06	0.71	0.77
1.14	5.272865326	2.1	0.885	1.141	1.25	1.36	1.31	0.88	0.95
1.36	5.73607816	2.4	0.893	1.362	1.50	1.62	1.56	1.05	1.13
1.59	6.175280015	2.7	0.899	1.590	1.75	1.89	1.82	1.22	1.32
1.82	6.593766666	3	0.905	1.824	2.00	2.17	2.09	1.40	1.52

Escarameia and May (1992) - HR Wallingford

The Rock Manual - CIRIA C683 (2007), 2nd Edition. Page 650.

Recommended for the design of riprap, loose or interlocking concrete blocks and gabion mattresses.

$$D_{n50} = C\frac{U_b{}^2}{2g(s-1)}$$

where: Dn50 is the characteristic size of stone
Dn50 is the size of the equivalent cube

$$D_{n50} = \left(\frac{W_{50}}{\rho_s}\right)^{\frac{1}{3}}$$

W50 is the weight of particle, Ps is the density of stone
C is a coefficient that takes account of the turbulence intensity TI
g is acceleration due to gravity
s is the relative density of the revetment material
Ub is the velocity near the bend (at 10% of the water depth above the bed)

Turbulence Levels

Situation	Turbulence Level	
	Qualitative	TI
Straight river or channel reaches and wide natural bends (R/W>26)	Normal (low)	0.12
Edge of revetments in straight reaches	Normal (higher)	0.20
Bridge piers, caissons and groynes; transitions	Medium to high	0.35-0.50
Downstream of hydraulic structures (weirs, culverts, stilling basins)	Very high	0.60

where: R is the centreline radius of bend
W is the water surface width at the upstream end of the bend

Values of C

Type of revetment	Value of C	Observations
Riprap	12.3TI-0.20	Valid for TI>=0.05
Concrete blocks	9.22TI-0.15	Valid for TI>=0.05
Gabion mattresses	12.3TI-1.65	Valid for TI>=0.12

Enter Data

Turbulence Intensity	TI	0.6	
Type of revetment	R,C or G	R	(Riprap, Concrete or Gabion)
Relative density of riprap	s	2.65	
Accel due to gravity	g	9.81	m/s2
Depth averaged velocity	Ud1	1.49	m/s
	Ud2	2.35	m/s
	Ud3	3.06	m/s
	Ud4	3.69	m/s
	Ud5	4.26	m/s
	Ud6	4.78	m/s
	Ud7	5.27	m/s
	Ud8	5.74	m/s
	Ud9	6.18	m/s
	Ud10	6.59	m/s
Suggested factor for Ub		47%	
Actual		60%	

Results

Coefficient C 7.18

Depth averaged velocity	Ub	Ub2	Dn50	D50 (min)	D50 (max)	D50 (ave)	D30 (min)	D30 (max)
1.486	0.891	0.795	0.18	0.19	0.21	0.20	0.14	0.15
2.346	1.408	1.981	0.44	0.48	0.52	0.50	0.34	0.37
3.058	1.835	3.367	0.75	0.82	0.89	0.85	0.57	0.62
3.686	2.211	4.890	1.08	1.19	1.29	1.24	0.83	0.90
4.255	2.553	6.518	1.45	1.59	1.72	1.65	1.11	1.20
4.781	2.869	8.230	1.83	2.01	2.17	2.09	1.40	1.52
5.273	3.164	10.009	2.22	2.44	2.64	2.54	1.71	1.85
5.736	3.442	11.845	2.63	2.89	3.13	3.01	2.02	2.19
6.175	3.705	13.728	3.04	3.35	3.62	3.49	2.34	2.54
6.594	3.956	15.652	3.47	3.81	4.13	3.97	2.67	2.89

Project

Priest Lake Dam

Calculations for	Division	WCD	File No.		
999A Summary Sheet.	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

Operational Scenario

Number of Operational Gates	11.00		
Flow	317.19	m^3s^{-1}	**Q500=11,200cfs**
Gage Level	1.87	m	**6.15 ft**
River Level	27.75	mAD	**91.05ft (6.15ft gage)**
Gate Opening	2.13	m	**7ft**
Stilling Basin	Type IVA		

Downstream Rating Curve

Depth of Flow	Water Level	Discharge	Average Velocity	Froude
D	Zwl	Q	v	Fr
m	mAD	m^3s^{-1}	ms-1	-
0.00	24.40	0.00	0.00	0.00
0.30	24.70	30.02	1.49	0.87
0.60	25.00	95.23	2.35	0.97
0.90	25.30	187.03	3.06	1.03
1.20	25.60	301.88	3.69	1.07
1.50	25.90	437.57	4.26	1.11
1.80	26.20	592.59	4.78	1.14
2.10	26.50	765.77	5.27	1.16
2.40	26.80	956.17	5.74	1.18
2.70	27.10	1163.06	6.18	1.20
3.00	27.40	1385.80	6.59	1.22

Radial Gate with Raised Cill

Max Unit Discharge	qmax	6.22	m^2s^{-1}
Design Unit Discharge	q	6.19	m^2s^{-1}
Flow	Qmax	333.82	m^3s^{-1}

Stilling Basin Level Design

Condition 1 - 100% Design Flow with 100% Design Flow Downstream

Condition 2 - 10% Design Flow with 0% Design Flow Downstream

Condition 3 - 30% Design Flow with 30% Design Flow Downstream

Summary of Critical Basin

Basin Level	22.90	mAD
Critical Condition	Condition 1	
Froude	11.38	
Critical Condition	Condition 2	

Project **Priest Lake Dam**	MOTT MACDONALD					
Calculations for 999A Summary Sheet.	Division	WCD	File No.			
	Calc by	JM	Date	31/07/2017	Number of sheets	
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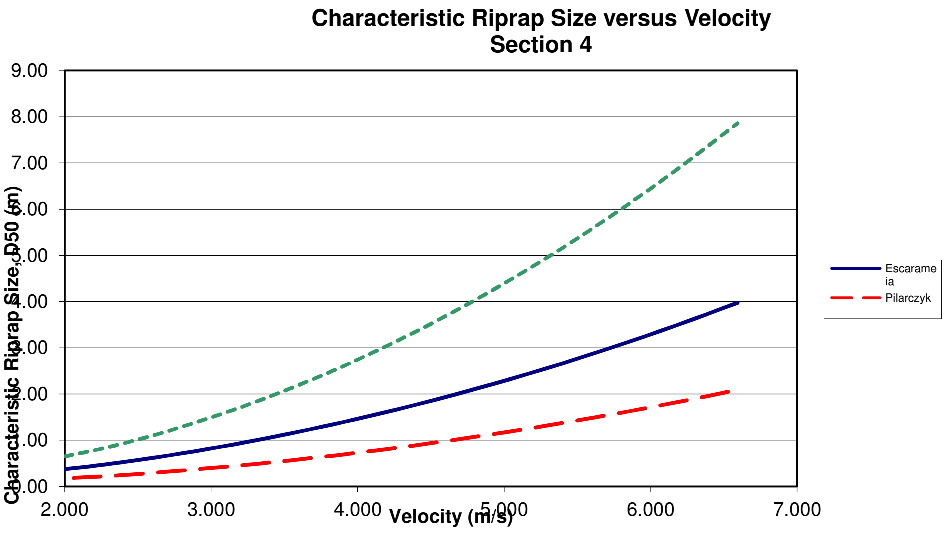
Conjugate Depth	2.97	mAD
Critical Condition	Condition 1	
Max Basin Length	9.40	mAD
Critical Condition	Condition 1	

USBR Stilling Basins

Minimum Basin Length	9.70	m
Minimum End Sill Height	0.61	m
Minimum End Sill Thickness	0.00	m

Erosion Stone Sizing



Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 000 Cover Sheet.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

Operational Scenario

Number of Operational Gates	1		
Flow	7.08	m3s-1	**250 cfs**
Gage Level	0.9144	m	**3 ft**
River Level	26.79	mAD	**87.9 ft**
Gate Opening	0.229	m	**9 inches/0.75 ft**
Stilling Basin	As existing		

Downstream Rating Curve

A downstream rating curve was produced to enable us identify the tail water level versus discharge assuming a trapezoidal channel.
A bed width of 220 ft was taken using a side slope of 1:1, bed slope of 0.01 and Manning's coefficient of 0.03 as per the original design calculation. The bed level was taken as 80.05 ft, the level of the concrete apron.
Using basic geometry for various depths of flows, the flow area and hydraulic radius were calculated. These could then be used to calculate the discharge.

Radial Gate with Raised Cill

The gate width (16 ft), cill level (81.05 ft) and gate height (7 ft) were taken as per original construction drawings. The safety factor was assumed to be 1.
Assuming unsubmerged conditions and a maximum opening of the 7ft high radial gate, the maximum unit discharge can then be calculated.

Stilling Basin Level Design

Three worst-case scenarios were considered for the stilling basin design. These are:

- 100% flow both upstream and downstream of the structure, to simulate design flow conditions
- 10% upstream flow with 0% flow downstream to simulate sudden gate opening
- 30% upstream flow with 30% downstream flow

A stilling basin safety factor of 1.1 was used for this calculation. Downstream conditions for the various scenarios can be determined from the downstream rating curve. Approach velocities were estimated using the inflow and height of the gates.
Head differences across the gate could be calculated by using the known qate levels and velocities upstream and downstream of the structure. This can be used to determine various parameters about the flow at the glasis and conjugate depth of the given scenario. Based on the Froude number, the performance of the stilling basin can be determined.
For the scenarios the minimum basin level, maximum Froude number, maximum conjugate depth and maximum free jump length are selected as the parameters required for the critical basin.

USBR Stilling Basins

This was undertaken using the USBR Design of Small Dams for a type I stilling basin with no chute or baffle blocks. Along with the parameters determined from the level design, the required basin and end sill dimensions were determined.

Erosion Stone Sizing

Erosion stone sizing was undertaken using The Rock Manual with the Escarameia, Pilarczyk and Maynord methods. The relative density of riprap was taken as 2.65 and the angle of the bank to the horizontal was taken as 45 degrees. All other required inputs were taken from geometry of the structure.

Project

Priest Lake Dam

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Calculations for

001 EVT-HYES-HYD-300 Trapizoidal Rating Curve.

Division	WCD	File No.		
Calc by	JM	Date	31/07/2017	Number of sheets
Checked by	PJH	Date	31/07/2017	of

Manning's Rating Curve

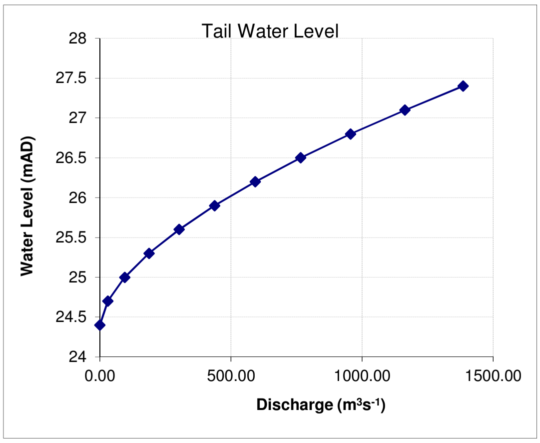
Open Channel Hydraulics, 1959, Ven Te Chow

1. INPUTS

Bed Width	B	67.056	m	**Comments:**	**220 ft as per original calculation**
Side Slope	s	1	H:1V	**Comments:**	**As per original calc**
Bed Slope	i	0.01	m/m	**Comments:**	**As per original calc**
Manning's Coefficient	n	0.03	$sm^{-1/3}$	**Comments:**	**As per original calc**
Bed Level	Zb	24.39924	mAD	**Comments:**	**80.05 ft**
Max Depth	Dmax	3	m	**Comments:**	

2. OUTPUTS

#	Depth of Flow	Flow Area	Hydraulic Radius	Discharge	Water Level	Average Velocity	Froude
n	D	A	P	Q	Zwl	v	Fr
-	m	m^2	m	m^3s^{-1}	mAD	ms-1	-
-	-	D_n*$(B+D_n$*s)	B+2*D_n*√$(1+s^2)$	$A(A/P)^{2/3}$*√(i) / n	D_n+Z_b	Q/A	v/√(g*D_n)
1	0	0	67.06	0.00	24.39924	0	0
2	0.3	20.2068	67.90	30.02	24.69924	1.49	0.87
3	0.6	40.5936	68.75	95.23	24.99924	2.35	0.97
4	0.9	61.1604	69.60	187.03	25.29924	3.06	1.03
5	1.2	81.9072	70.45	301.88	25.59924	3.69	1.07
6	1.5	102.834	71.30	437.57	25.89924	4.26	1.11
7	1.8	123.9408	72.15	592.59	26.19924	4.78	1.14
8	2.1	145.2276	73.00	765.77	26.49924	5.27	1.16
9	2.4	166.6944	73.84	956.17	26.79924	5.74	1.18
10	2.7	188.3412	74.69	1163.06	27.09924	6.18	1.20
11	3	210.168	75.54	1385.80	27.39924	6.59	1.22



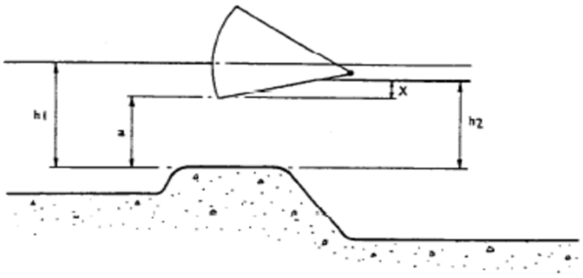
Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
002 EVT-HYES-HYD-561 Radial Gate with Raised Cill.	Checked by	PJH	Date	31/07/2017	of

Radial Gate With Cill

Hydraulic Design of Canal Structures - MMP August 1985

USBR - Design of Small Small Dams

1. INPUTS



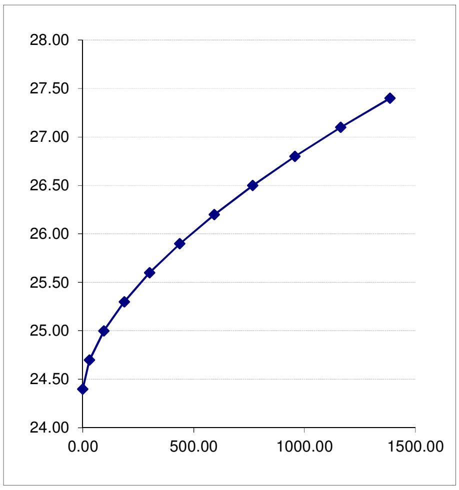
1. Hydraulic Design of Canal Structures, Sept 1986, MM & Partners p. 1 - p. 11

2. Design of Small Dams - USBR

1.1 GENERAL

Design Discharge	Q	7.08	m^3s^{-1}	**Comments:**	**250fps**
Number of Gates	N_g	1	-	**Comments:**	
Gate Width	w	4.88	m	**Comments:**	**16 ft**
Cill Level	Z_c	24.70	mAD	**Comments:**	**81.05ft**
Max Upstream Water Level	Z_{uswl}	26.79	mAD	**Comments:**	**87.9ft (3ft gage)**
Structure Flow Saftey Factor	S_f	1	-	**Comments:**	
Maximum gate opening (as percentage of max head)	μ	22%	-	**Comments:**	**Maximum opening for design dischage**
Design Gate Height	a	0.229	-	**Comments:**	**9 inches/0.75 ft**
Design Downstream Water Level	Z_{dswl}	24.64	mAD	**Comments:**	

	Level	Flow	Velocity
1	mAD	m3/s	m/s
2	24.40	0.00	0.00
3	24.70	30.02	1.49
4	25.00	95.23	2.35
5	25.30	187.03	3.06
6	25.60	301.88	3.69
7	25.90	437.57	4.26
8	26.20	592.59	4.78
9	26.50	765.77	5.27
10	26.80	956.17	5.74
11	27.10	1163.06	6.18
12	27.40	1385.80	6.59
13			
14			



1.2 Effective Crest Coefficients

Project

Priest Lake Dam

MOTT MACDONALD

Calculations for	Division	WCD	File No.		
002 EVT-HYES-HYD-561 Radial Gate with Raised Cill.	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

Pier Contraction Coefficient	k_p	0.020	-	**Comments:**	**USBR Design of Small Dams, square nosed piers**
Abutment Contraction Coefficient	k_a	0.2	-	**Comments:**	**USBR Design of Small Dams, square abutments**

2. OUTPUTS

2.1 GENERAL

Number of Piers	N	0		N_g-1	**Comments:**
Sum of Gates Width	b'	4.88	m	N_g*w	**Comments:**
Design Factored Discharge	Q_f	7.08	m^3s^{-1}	S_f*Q	**Comments:**
Upstream Head Over Cill	h_1	2.09	m	Z_{uswl} - Z_c	**Comments:**
Downstream Head Over Cill	h_2	-0.060	m	Z_{dswl} - Z_c	**Comments:**
Effective Crest Length	b	4.04	m	$b'-2*(N*K_p+k_a)*h_1$	**Comments:**

2.2 FREE FLOW CALCULATIONS

Max Height of Gate Opening	$a_{max\ 1}$	0.46	m	$\mu*h_1$	**Comments:**
Opening - DS Water Level Difference	x	-0.23	m	h_2 - a	**Comments:**
Max Unit Discharge	q_{max}	1.764	m^2s^{-1}	$0.6*a_{max\ 1}*\sqrt{(2*g*h_1)}$	**Comments:**
Design Unit Discharge	q	1.752	m^2s^{-1}	Q_f/b	**Comments:**
Flow	Q_{max}	8.602	m^3s^{-1}	q*w*N	**Comments:**
CHECK		OK			**Comments:**
Gate Radius	R	0.612	m	4/3*amax 1	**Comments:**
Depth Above Gate	a'	1.63	m	h1-amax 1	**Comments:**
Angle	θ	#NUM!	Degrees	$cos^{-1}(a/R)$	**Comments:**
Vena Contrata Factor	δ	0.65	-		**Comments:**
Vena Contrata	α	0.30	m	$a*\delta$	**Comments:**
Velocity	v	5.91	ms^{-1}	q/α	**Comments:**
Froude Number	Fr	3.45	-		**Comments:**
Conjugate Depth	y_2	1.46	m	$y_1/2*\sqrt{((1+8Fr_1^2)-1)}$	**Comments:**
Water Level	Zc	26.16	mAD	y2+Zc	**Comments:**
Downstream Water Level	Zds	24.49	mAD		**Comments:**

2.3 SUBMERGED CONDITIONS

Max Height of Gate Opening	$a_{max\ 2}$	1.39	m	$2/3*h_1$	**Comments:**
Opening - DS Water Level Difference	x	-1.45	m	h_2 - a	**Comments:**
Head Drop	H	2.15	m	h_1-h_2	
Max Unit Discharge	q_{max}	6.054	m^2s^{-1}	$0.67*a_{max\ 2}*\sqrt{(2*g*(h_1-h_2)}$	
Design Unit Discharge	q	1.752	m^2s^{-1}	Q_f/b'	**Comments:**
CHECK		OK		$q < q_{max}$	**Comments:**

Project **Priest Lake Dam**	MOTT MACDONALD			
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	Checked by	PJH	Date	31/07/2017	of

Gate Radius	R	1.856	m	*4/3*a*	**Comments:**
Flow Through One Gate @ 100% a	$Q_{n=1}$	24.470	m^3s^{-1}	q_{max} **w*	**Comments:**
Design Flow	Q	24.470	m^3s^{-1}		

2.4 DESIGN CONDITIONS

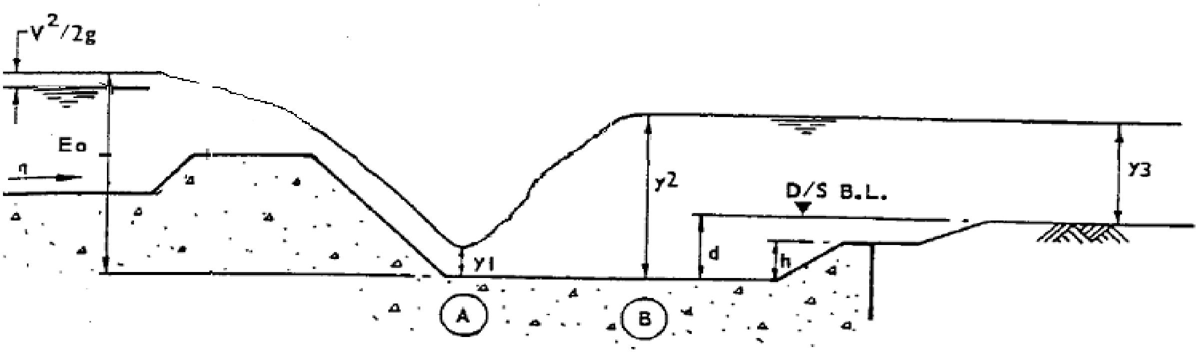
Condition		UNSUBMERGED			**Comments:**
Design Flow	Q_d	8.602	m^3s^{-1}	*q*w*N*	**Comments:**

Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 003 EVT-HYES-HYD-001 - Stilling Basin Level Design.xls.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

Stilling Basin Level Design

1. MMP Hydraulic Design of Canal Structures, 1985, 1-13 - 1-14

1. INPUTS

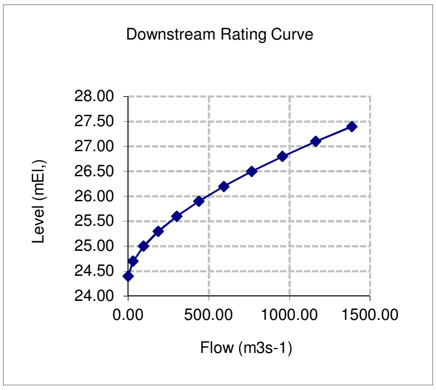


1.1 GENERAL

Design Flow	Q_d	7.1	m^3s^{-1}	**Comments:**	
Flow Safety Factor	S_f	1.00	-	**Comments:**	
Width of Stilling Basin	B	4.88	m	**Comments:**	**16ft*1 no.**
Number of Basins	N	1	-	**Comments:**	
Basin Safety Factor	S_f	1.1	-	**Comments:**	
Bed Level US	Z_{usbl}	24.40	mAD	**Comments:**	**80.05 ft**
Bed Level DS	Z_{dsbl}	24.40	mAD	**Comments:**	**80.05 ft**
Allowance For Retrogression (Percentage of D/S Depth)	ρ	0%	%	**Comments:**	
Gate Width	w_g	4.88	m	**Comments:**	**16ft*1 no.**

Downstream Rating Curve (Table 1.1)

#	Level	Flow	Velocity
n	Z_{dswl}	Q	v
-	mAD	m^3s^{-1}	ms^{-1}
1	24.40	0.00	0.00
2	24.70	30.02	1.49
3	25.00	95.23	2.35
4	25.30	187.03	3.06
5	25.60	301.88	3.69
6	25.90	437.57	4.26
7	26.20	592.59	4.78
8	26.50	765.77	5.27
9	26.80	956.17	5.74
10	27.10	1163.06	6.18
11	27.40	1385.80	6.59



Project

Priest Lake Dam

MOTT MACDONALD

Calculations for	Division	WCD	File No.		
003 EVT-HYES-HYD-001 - Stilling Basin Level Design.xls.	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

1.2 CONDITION 1

1.2.1 General

Percentage of Flow US	$m_{us,1}$	100%		**Comments:**	
Percentage of Flow DS	$m_{ds,1}$	100%		**Comments:**	
Additional flow downstream	$Q_{add,1}$	0.0	m^3s^{-1}	**Comments:**	

1.3.2 Upstream Conditions

Water Level	$Z_{us,1}$	26.8	mAD	**Comments:**	**87.9 ft (3ft gage)**
Velocity	$v_{us,1}$	1.941	ms^{-1}	**Comments:**	
Maximum Unit Discharge	$q_{max,1}$	1.764	m^2s^{-1}	**Comments:**	

1.3 CONDITION 2

1.3.1 General

Percentage of Flow US	$m_{us,2}$	10%		**Comments:**
Percentage of Flow DS	$m_{ds,2}$	0%		**Comments:**
Additional flow downstream	$Q_{add,2}$	0.0	m^3s^{-1}	**Comments:**

1.3.2 Upstream Conditions

Water Level	$Z_{us,2}$	26.8	mAD	**Comments:**
Velocity	$v_{us,1}$	0.77	ms^{-1}	**Comments:**
Maximum Unit Discharge	$q_{max,1}$	0.176	m^2s^{-1}	**Comments:**

1.4 CONDITION 3

1.4.1 General

Percentage of Flow US	$m_{us,3}$	30%		**Comments:**
Percentage of Flow DS	$m_{ds,3}$	30%		**Comments:**
Additional flow downstream	$Q_{add,3}$	0.0	m^3s^{-1}	**Comments:**

1.4.2 Upstream Conditions

Water Level	$Z_{us,3}$	26.8	mAD	**Comments:**
Velocity	$v_{us,3}$	2.31	ms^{-1}	**Comments:**
Maximum Unit Discharge	$q_{max,3}$	0.529	m^2s^{-1}	**Comments:**

Project **Priest Lake Dam**	MOTT MACDONALD					
Calculations for 003 EVT-HYES-HYD-001 - Stilling Basin Level Design.xls.	Division	WCD	File No.			
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2. OUTPUTS

Condition 1 - 100% Design Flow with 100% Design Flow Downstream

Condition 2 - 10% Design Flow with 0% Design Flow Downstream

Condition 3 - 30% Design Flow with 30% Design Flow Downstream

2.1 GENERAL

Factored Design Flow	Q_f	7.1	m^3s^{-1}	Q_d*S_f	**Comments:**
2.1.1 Condition 1					**Comments:**
Factored Discharge US	$Q_{us,1}$	7.1	m^3s^{-1}	$Q_f*m_{us,1}$	**Comments:**
Discharge DS	$Q_{ds,1}$	7.1	m^3s^{-1}	$Q_f*m_{ds,1}$	**Comments:**
DS Water Level	$Z_{ds,1}$	24.5	mAD	*Interpolate (Table 1.1)*	**Comments:**
Velocity @ Q DS	$v_{ds,1}$	0.35	ms^{-1}	*Interpolate (Table 1.1)*	**Comments:**
2.1.2 Condition 2					**Comments:**
Factored Discharge US	$Q_{us,2}$	0.7	m^3s^{-1}	$Q_f*m_{us,2}$	**Comments:**
Discharge DS	$Q_{ds,2}$	0.0	m^3s^{-1}	$Q_f*m_{ds,2}$	**Comments:**
DS Water Level	$Z_{ds,2}$	24.399	mAD	*Interpolate (Table 1.1)*	**Comments:**
Velocity @ Q DS	$v_{ds,2}$	0.00	ms^{-1}	*Interpolate (Table 1.1)*	**Comments:**
2.1.3 Condition 3					**Comments:**
Factored Discharge US	$Q_{us,3}$	2.1	m^3s^{-1}	$Q_f*m_{us,3}$	**Comments:**
Discharge DS	$Q_{us,3}$	2.1	m^3s^{-1}	$Q_f*m_{ds,3}$	**Comments:**
DS Water Level	$Z_{ds,3}$	24.420	mAD	*Interpolate (Table 1.1)*	**Comments:**
Velocity @ Q DS	$v_{ds,3}$	0.11	ms^{-1}	*Interpolate (Table 1.1)*	**Comments:**

2.2 DOWNSTREAM CONDITION

Water Depth DS Condition 1	$D_{ds,1}$	0.071	m	$Z_{dswl}-Z_{bl}$	**Comments:**
Water Depth DS Condition 2	$D_{ds,2}$	0.000	m	$Z_{dswl}-Z_{bl}$	**Comments:**
Water Depth DS Condition 3	$D_{ds,3}$	0.021	m	$Z_{dswl,70}-Z_{bl}$	**Comments:**
Retrogression @ Condition 1	r_1	0.000	m	$\rho*D_{ds,1}$	**Comments:**
Retrogression @ Condition 2	r_2	0.000	m	$\rho*D_{ds,2}$	**Comments:**
Retrogression @ Condition 3	r_3	0.000	m	$\rho*D_{ds,3}$	**Comments:**
Water Level Condition 1 with Retrogression	$Z_{dsl,r}$	24.470	mAD	$Z_{ds,1}-r_1$	**Comments:**
Water Level Condition 2 with Retrogression	$Z_{dsl,r}$	24.399	mAD	$Z_{ds,2}-r_2$	**Comments:**
Water Level Condition 3 with Retrogression	$Z_{dsl,70,r}$	24.420	mAD	$Z_{ds,3}-r_3$	**Comments:**

Project **Priest Lake Dam**					
Calculations for 003 EVT-HYES-HYD-001 - Stilling Basin Level Design.xls.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
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Stilling Basin General Design - Condition 1

1. INPUTS

Design Flow	$Q_{us,1}$	7.08	m^3s^{-1}
Width of Stilling Basin	B	4.88	m
Upstream Water Level	$Z_{us,1}$	26.79	mAD
Approach velocity	$v_{us,1}$	1.941	ms^{-1}
Downstream Water Level	$Z_{ds,1}$	24.5	mAD
Downstream Velocity	$v_{ds,1}$	0.35	ms^{-1}
Basin Safety Factor	S_f	1.1	-

2.2 OUTPUTS - CONDITION 1

Flow Intensity	q_1	1.45	m^2s^{-1}	$Q_{us,1}/B/N$	**Comments:**
Critical Depth	$d_{c,1}$	0.60	m	$(q_1{}^2/g)^{1/3}$	**Comments:**
Upstream Velocity Head	$h_{v,1}$	0.19	m	$v_{us,1}{}^2/2g$	**Comments:**
Downstream Velocity Head	$h_{v2,1}$	0.01	m	$v_{ds,1}{}^2/2g$	**Comments:**
Upstream Total Head	$E_{o,1}$	26.98	m	$Z_{us,1} + h_{v,1}$	**Comments:**
Downstream Total Head	$E_{2,1}$	24.48	m	$Z_{ds,1} + h_{2v,1}$	**Comments:**
Head Difference Across Gate	$H_{L,1}$	2.51	m	$E_{0,1}-E_{2,1}$	**Comments:**
	H_L/d_c	4.19	m	H_L/d_c	**Comments:**
	a_1	0.11	-	*GOAL SEEK*	**Comments:**
	$v_{1,1}$	4.19	-	$(1-a_1)^3/(2a_1)*((1+2a_1)^2-1)^{-1/3})$	
Congugate Depth	$y_{2,1}$	1.53	m	$4*H_{L,1}*a_1/(1-a_1)^3$	
Velocity @ Congugate Depth	$v_{2,1}$	0.95	ms^{-1}	$q_1/y_{2,1}$	
Depth of Flow at Bottom of Glasis	$y_{1,1}$	0.17	m	$a_1*y_{2,1}$	
Velocity @ Toe of Glasis	$v_{1,1}$	8.77	ms^{-1}	$q_1/y_{1,1}$	**Comments:**
Froude Number	Fr_1	6.88	-	$v_{1,1}/\sqrt{}(gy_{1,1})$	**Comments:**
Conjugate Depth Check Calc	$y_{2,1}$	1.53	m	$y_{1,1}/2*\sqrt{}((1+8Fr_1{}^2)-1)$	**Comments:**
Total Energy at Congugate Depth	$E_{1,1}$	1.58	m	$y_{2,1}+d_{c,1}{}^3/(2*y_{2,1}{}^2)$	
Basin Level	$\mathbf{Z_{b,1}}$	**22.74**	**mAD**	$E_{2,1}-S_f*E_{1,1}$	**Comments:**
Downstream Jump Level	Z_{dsjl}	24.27	m	$Z_{dsbs} + y_2$	**Comments:**
	L/y_2	6.10	m	*INTERPOLATE*	**Comments:**
	L/y_2	6.10	m	*INTERPOLATE*	**Comments:**
Jump Length	L	9.34	m	L/y_2*y_2	**Comments:**
Free Jump Performance		Best Performance			
Basin Type		Type III			**Comments:**
Basin Jump Length Factor	L_B/y_2	2.56	m		
Basin Jump Length	L_B	3.92	m		

Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 003 EVT-HYES-HYD-001 - Stilling Basin Level Design.xls.	Division	WCD	File No.		
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	Checked by	PJH	Date	31/07/2017	of

2.2.1 Design Check (Set Outputs First)

Total Energy	$E_{0,1d}$	2.58	m	$Z_{us,1}+h_{v,1}-Z_b$	**Comments:**
Velocity	$v_{1,1d}$	6.82	ms^{-1}	$\sqrt{(2g^*(E_{0,1d}-y_{1,1d}))}$	**Comments:**
Depth of Flow	$y_{1,1d}$	0.213	m	$q_1/v_{1,1d}$	**Comments:**
Upstream Froude Number	$\mathbf{Fr_{1,1d}}$	**4.720**	-	$v_{1,1d}/\sqrt{(gy_{1,1d})}$	**Comments:**
Conjugate Depth	$y_{2,1d}$	1.32	-	$y_{1,1d}/2^*(\sqrt{(1+8Fr_{1d}^2)}-1)$	
Conjugate Level	$y_{2,1d}$	25.72	-	$y_{1,1d}/2^*(\sqrt{(1+8Fr_{1d}^2)}-1)$	

Project **Priest Lake Dam**	MOTT MACDONALD				
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Stilling Basin General Design - Condition 2

1. INPUTS

Design Flow	$Q_{us,2}$	0.71	m^3s^{-1}
Width of Stilling Basin	B	4.88	m
Upstream Water Level	$Z_{us,2}$	26.79	mAD
Approach velcoity	$v_{us,2}$	0.772	ms^{-1}
Downstream Water Level	$Z_{ds,2}$	24.4	mAD
Downstream Velocity	$v_{ds,2}$	0.00	ms^{-1}
Basin Safety Factor	S_f	1.1	-

2.3 OUTPUTS - CONDITION 2

Flow Intensity	q_2	0.15	m^2s^{-1}	$Q_{us,2}/B/N$	**Comments:**
Critical Depth	$d_{c,2}$	0.13	m	$(q_2^2/g)^{1/3}$	**Comments:**
Upstream Velocity Head	$h_{v,2}$	0.03	m	$v_{us,2}^2/2g$	**Comments:**
Downstream Velocity Head	$h_{v2,2}$	0.00	m	$v_{ds,2}^2/2g$	**Comments:**
Upstream Total Head	$E_{o,2}$	26.82	m	$Z_{us,2} + h_{v,2}$	**Comments:**
Downstream Total Head	$E_{2,2}$	24.40	m	$Z_{ds,2} + h_{2v,2}$	**Comments:**
Head Difference Across Gate	$H_{L,2}$	2.42	m	$E_{0,2}-E_{2,2}$	**Comments:**
	H_L/d_c	18.78	m	H_L/d_c	**Comments:**
	a_2	0.04	-	*GOAL SEEK*	**Comments:**
	$v_{1,2}$	18.78	-	$(1-a_2)^3/(2a_2)*((1+2a_2)^2-1)^{-1/3})$	
Congugate Depth	$y_{2,2}$	0.46	m	$4*H_{L,2}*a_2/(1-a_2)^3$	
Velocity @ Congugate Depth	$v_{2,2}$	0.31	ms^{-1}	$q_2/y_{2,2}$	
Depth of Flow at Bottom of Glacis	$y_{1,2}$	0.02	m	$a_2*y_{2,2}$	
Velocity @ Toe of Glacis	$v_{1,2}$	7.50	ms^{-1}	$q_2/y_{1,2}$	**Comments:**
Froude Number	Fr_2	17.23	-	$v_{1,2}/\surd(gy_{1,2})$	**Comments:**
Conjugate Depth Check Calc	$y_{2,2}$	0.46	m	$y_{1,2}/2*\surd((1+8Fr_2^2$	**Comments:**
Total Energy at Congugate Depth	$E_{1a,2}$	0.47	m	$y_{2,2}+d_{c,2}^3/(2*y_{2,2}$	**Comments:**
Basin Level	$\mathbf{Z_{dsbs}}$	**23.89**	**mAD**	$E_{2,2}-S_f*E_{1,2}$	**Comments:**
Downstream Jump Level	Z_{dsjl}	24.35	m	$Z_{dsbs} + y_2$	**Comments:**
	L/y_2	5.78	m	*INTERPOLATE*	**Comments:**
	L/y_2	5.78	m	*INTERPOLATE*	**Comments:**
Jump Length	L	2.67	m	L/y_2*y_2	**Comments:**
Free Jump Performance		Expensive Stilling Basin and Rough Surface Conditions			
Basin Type		Type III			**Comments:**
Basin Jump Length Factor	L_B/y_2	2.73	m		
Basin Jump Length	L_B	1.26	m		

Project **Priest Lake Dam**	MOTT MACDONALD				
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	Checked by	PJH	Date	31/07/2017	of

2.3.1 Design Check (Set Outputs First)

Total Energy	E_0	2.42	m	$Z_{uswl}+h_v-Z_b$	**Comments:**
Velocity	v_1	6.86	ms^{-1}	$\sqrt{}(2g^*(E_0-y_1))$	**Comments:**
Depth of Flow	y_1	0.021	m	q/v	**Comments:**
Upstream Froude Number	$\mathbf{Fr_1}$	**15.068**	-	$v_1/\sqrt{}(gy_1)$	**Comments:**
Conjugate Depth	y_2	0.44	-	$y_1/2^*(\sqrt{}(1+8Fr_1^2)-1)$	**Comments:**
Conjugate Level	y_2	24.84	-	$y_1/2^*(\sqrt{}(1+8Fr_1^2)-1)$	**Comments:**

Project **Priest Lake Dam**					
Calculations for 003 EVT-HYES-HYD-001 - Stilling Basin Level Design.xls.	Division	WCD	File No.		
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	Checked by	PJH	Date	31/07/2017	of

Stilling Basin General Design - Condition 3

MM Hydraulic Structures, 1985

1. INPUTS

Design Flow	$Q_{us,3}$	2.12	m^3s^{-1}
Width of Stilling Basin	B	4.88	m
Upstream Water Level	$Z_{us,3}$	26.79	mAD
Approach velcoity	$v_{us,3}$	2.315	ms^{-1}
Downstream Water Level	$Z_{ds,3}$	24.4	mAD
Downstream Velocity	$v_{ds,3}$	0.11	ms^{-1}
Basin Safety Factor	S_f	1.1	-

2.4 OUTPUTS - CONDITION 3

Flow Intensity	q_3	0.44	m^2s^{-1}	$Q_{us,3}/B/N$	**Comments:**
Critical Depth	$d_{c,3}$	0.27	m	$(q_3^{2}/g)^{1/3}$	**Comments:**
Upstream Velocity Head	$h_{v,3}$	0.27	m	$v_{us,3}^{2}/2g$	**Comments:**
Downstream Velocity Head	$h_{v2,3}$	0.00	m	$v_{ds,3}^{2}/2g$	**Comments:**
Upstream Total Head	$E_{o,3}$	27.07	mAD	$Z_{us,3} + h_{v,3}$	**Comments:**
Downstream Total Head	$E_{2,3}$	24.42	mAD	$Z_{ds,3} + h_{2v,3}$	**Comments:**
Head Difference Across Gate	$H_{L,3}$	2.64	m	$E_{0,3}-E_{2,3}$	**Comments:**
	H_L/d_c	9.85	m	H_L/d_c	**Comments:**
	a_3	0.06	-	*GOAL SEEK*	**Comments:**
	$v_{1,3}$	9.85	-	$(1-a_3)^3/(2a_3)*((1+2a_3)^2-1)^{-1/3})$	
Congugate Depth	$y_{2,3}$	0.83	m	$4*H_{L,3}*a_3/(1-a_3)^3$	
Velcoity @ Congugate Depth	$v_{2,3}$	0.53	ms^{-1}	$q_3/y_{2,3}$	
Depth of Flow at Bottom of Glasis	$y_{1,3}$	0.05	m	$a_3*y_{2,3}$	
Velocity @ Toe of Glasis	$v_{1,3}$	8.21	ms^{-1}	$q_3/y_{1,3}$	**Comments:**
Froude Number	Fr_3	11.37	-	$v_{1,3}/\surd(gy_{1,3})$	**Comments:**
Conjugate Depth Check Calc	$y_{2,3}$	0.83	m	$y_{1,3}/2*\surd((1+8Fr_3^{2})-1)$	**Comments:**
Total Energy at Congugate Depth	$E_{1a,3}$	0.84	m	$y_{2,3}+d_{c,3}^{3}/(2*y_{2,3}^{2})$	
Basin Level	$\mathbf{Z_{dsbs}}$	**23.50**	**mAD**	$E_{2,3}-S_f*E_{1,3}$	**Comments:**
Downstream Jump Level	Z_{dsjl}	24.32	m	$Z_{dsbs} + y_2$	**Comments:**
	L/y_2	6.05	m	*INTERPOLATE*	**Comments:**
	L/y_2	6.05	m	*INTERPOLATE*	**Comments:**
Jump Length	L	5.01	m	L/y_2*y_2	**Comments:**
Free Jump Performance		Acceptable Performance			
Basin Type		Type III			**Comments:**
Basin Jump Length Factor	L_B/y_2	2.76	m		
Basin Jump Length	L_B	2.28	m		

Project **Priest Lake Dam**	MOTT MACDONALD				
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2.4.1 Design Check (Set Outputs First)

Total Energy	E_0	2.67	m	$Z_{uswl}+h_v-Z_b$	**Comments:**
Velocity	v_1	7.15	ms^{-1}	$\sqrt{}(2g*(E_0-y_1))$	**Comments:**
Depth of Flow	y_1	0.061	m	q/v	**Comments:**
Upstream Froude Number	$\mathbf{Fr_1}$	**9.245**	-	$v_1/\sqrt{}(gy_1)$	**Comments:**
Conjugate Depth	y_2	0.77	-	$y_1/2*(\sqrt{}(1+8Fr_1^2)-1)$	**Comments:**
Conjugate Level	y_2	25.17	-	$y_1/2*(\sqrt{}(1+8Fr_1^2)-1)$	**Comments:**

Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for	Division	WCD	File No.		
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Stilling Basin Check - Outputs

MM Hydraulic Structures, 1985

2. OUTPUTS

Condition 1 - 100% Design Flow with 100% Design Flow Downstream

Condition 2 - 10% Design Flow with 0% Design Flow Downstream

Condition 3 - 30% Design Flow with 30% Design Flow Downstream

2.1 SUMMARY OF CRITICAL BASIN

2.2.1 Basin Level

Basin Level	Z_{bs}	22.743	mAD	*MIN(C1,C2,C3)*	**Comments:**
Critical Condition		Condition 1	-		**Comments:**
Basin Level Selected	Z_b	**24.400**	mAD	***USER DEFINED***	**Comments:**

2.2.2 Froude Number

Froude	17.227	-	*MAX(C1,C2,C3)*	**Comments:**
Critical Froude Condition	Condition 2	-		**Comments:**

2.2.3 Conjugate Depth

Conjugate Depth	1.530	m	*MAX(C1,C2,C3)*	**Comments:**
Max Conjugate Depth Condition	Condition 1	-		**Comments:**

2.2.4 Designed Basin Length

Max Basin Length	3.924	-	*MAX(C1,C2,C3)*	**Comments:**
Max Basin Length Condition	Condition 1	-		**Comments:**

2.2.5 Free Jump Length

Max Free Jump Length	9.339	m	*MAX(C1,C2,C3)*	**Comments:**

2.2.6 General

Minimum Basin Length	9.181	m	6*y2,crit	**Comments:**

Project WSIP	MOTT MACDONALD				
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Stilling Basin Check - Gates Open Check

MM Hydraulic Structures, 1985

2. Outputs

2.1 100% MAX GATE DISCHARGE

Maximum Unit Discharge Per Gate	q_{max}	1.76	m^2s^{-1}		**Comments:**	
Maximum Water Level - Basin Level Difference	E_0	2.39	m	$Z_{uswl,c1}-Z_{bs}$	**Comments:**	
Depth of Flow at Toe of Glasis	y_1	0.274	m	$q/(\surd(2*g*(E_0-y_1))$	**Comments:**	
Velocity @ Toe	v_1	6.45	ms^{-1}	q/y_1	**Comments:**	
Froude @ Toe	Fr	3.93	-	$v_1/\surd(g*y_1)$	**Comments:**	
Conjugate Depth	y_2	1.39	m	$y_1/2*(\surd(1+8*Fr^2)-1)$	**Comments:**	
Water Level	Z_{cg}	24.13	mAD	$Z_{bs}+y_2$	**Comments:**	
Required Flow in Downstream Channel	q_{req}	-99.00	m^3s^{-1}	*INTERPOLATE(Table 1.1)*	**Comments:**	
Number of Gates Required to Aquire DS Level	N_g	-11.5	-	$Q_{req}/(q_{max}*w_g)$	**Comments:**	Minimum number of gates = 1

2.2 50% Max Gate Discharge

Maximum Unit Discharge Per Gate	q_{max}	0.88	m^2s^{-1}		**Comments:**	
Maximum Water Level - Basin Level Difference	E_0	2.39	m	$Z_{uswl,c1}-Z_{bs}$	**Comments:**	
Depth of Flow at Toe of Glasis	y_1	0.13	m	$q/(\surd(2*g*(E_0-y_1))$	**Comments:**	
Velocity @ Toe	v_1	6.66	ms^{-1}	q/y_1	**Comments:**	
Froude @ Toe	Fr	5.84	-	$v_1/\surd(g*y_1)$	**Comments:**	
Conjugate Depth	y_2	1.03	m	$y_1/2*(\surd(1+8*Fr^2)-1)$	**Comments:**	
Water Level	Z_{cg}	23.77	mAD	$Z_{bs}+y_2$	**Comments:**	
Required Flow in Downstream Channel	q_{req}	-99.00	m^3s^{-1}	*INTERPOLATE(Table 1.1)*	**Comments:**	
Number of Gates Required to Aquire DS Level	N_g	-23.0	-	$q_{req}/(q_{max}*w_g)$	**Comments:**	Minimum number of gates = 1

Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 004 EVT-HYES-HYD-002 - USBR Stilling Basins.	Division	WCD	File No.		
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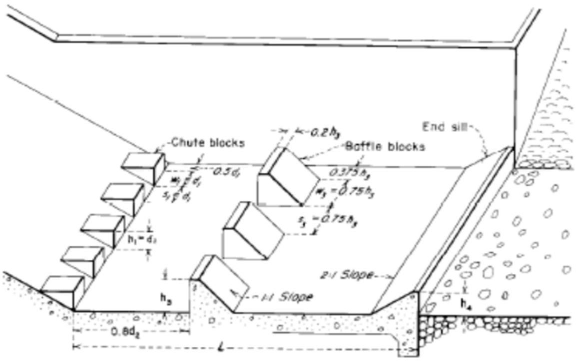
	Checked by	PJH	Date	31/07/2017	of

USBR Small Dams Stilling Basin Design

1. USBR Small Dams p.387 - 395

2. USBR Low Froude Stilling Basins

1. INPUTS

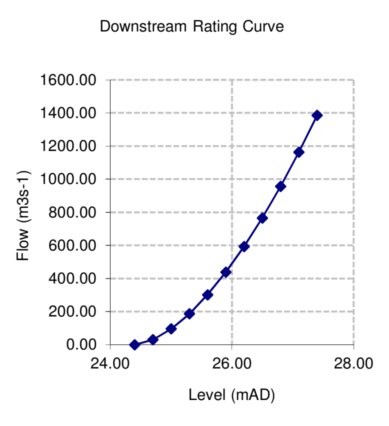


(A) TYPE III BASIN DIMENSIONS

Upstream Water Level	Z_{uswl}	26.79	mAD	**Comments:**	**87.9 ft (3ft gage)**
Upstream Velocity	v_0	1.94	ms^{-1}	**Comments:**	
Basin Level	Z_b	24.40	mAD	**Comments:**	**Downstream level**
Total Flow	Q	7.1	m^3s^{-1}	**Comments:**	
Stilling Basin Width	B	4.877	m	**Comments:**	
Number of Stilling Basins	N	1	-	**Comments:**	
Number of Chute Blocks	Nc	0	-	**Comments:**	
Number of Baffle Blocks	Nb	0	-	**Comments:**	
Basin Type		Type I		**Comments:**	
End Sill Width Multiple	α	0.150		**Comments:**	
Glasis Slope	s	2	H:1V	**Comments:**	

Downstream Rating Curve (Table 1.1)

#	Level	Flow	Velcocity
n	Z_{dswl}	Q	v
-	mAD	m^3s^{-1}	ms^{-1}
1	24.40	0.00	0.00
2	24.70	30.02	1.49
3	25.00	95.23	2.35
4	25.30	187.03	3.06
5	25.60	301.88	3.69
6	25.90	437.57	4.26
7	26.20	592.59	4.78
8	26.50	765.77	5.27
9	26.80	956.17	5.74
10	27.10	1163.06	6.18
11	27.40	1385.80	6.59



2. OUTPUTS

Project

Priest Lake Dam

MOTT MACDONALD

Calculations for	Division	WCD	File No.		
004 EVT-HYES-HYD-002 - USBR Stilling Basins.	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

2.1 GENERAL

Unit Flow	q	1.45	m^2s^{-1}	Q/(B*N)	**Comments:**	
Upstream Velocity Head	h_v	0.19	m	$v_0^2/(2g)$	**Comments:**	
Total Energy	E_0	2.58	m	$Z_{uswl}+h_v-Z_b$	**Comments:**	
Velocity	v_1	6.82	ms^{-1}	$\surd(2g*(E_0-y_1))$	**Comments:**	
Depth of Flow	y_1	0.213	m	q/v	**Comments:**	
Upstream Froude Number	$\mathbf{Fr_1}$	**4.721**	-	$v_1/\surd(gy_1)$	**Comments:**	
Conjugate Depth	y_2	1.32	-	$y_1/2*(\surd(1+8Fr_1^2)-1)$	**Comments:**	
Conjugate Level	Zc	25.72	-	*y2+Zb*	**Comments:**	
Initial Design Flow	Qi	355.49	m^3s^{-1}	*INTERPOLATE*	**Comments:**	
Free Jump Length Factor		5.94	m	*INTERPOLATE*	**Comments:**	
Free Jump Length		7.84	m	L/y_2*y_2	**Comments:**	
MM Basin Length		7.91	m	$6*y_{2,design}$	**Comments:**	
Rough Downstream Water Level		24.47	mEL	*INTERPOLATE*	**Comments:**	

2.2 BASIN DIMENSIONS

2.2.1 Basin

Basin Jump Length Factor	F_L	5.94		*INTERPOLATE*	**Comments:**	
Minimum Basin Length	L_B	7.84	m	F_L*y_2	**Comments:**	
Design Length	**L**	**2.80**	**m**	***User Defined***	**Comments:**	**existing case**

2.3 END SILL

Toe Baffle Distance From Chute Toe Factor	L_1/y_2	N/A	-	*INTERPOLATE*	**Comments:**	
Minimum Toe Baffle Distance From Chute Toe	$L_{1,min}$	N/A	m	L_1/y_2*y_2	**Comments:**	
Toe Baffle Distance From Chute Toe	$\mathbf{L_1}$	**0.00**	**-**	***User Defined***	**Comments:**	**existing case**
Height Ratio	η_1	1.25	-			
Minimum End Sill Height	$H_{s,min}$	0.27	-	η_1*y_1	**Comments:**	
End Sill Height	$\mathbf{H_s}$	**0.00**	**-**	***User Defined***	**Comments:**	**existing case**
Minimum End Sill Thickness	$T_{es,min}$	0.00	m	$0.2*H_s$	**Comments:**	
End Sill Thickness	$\mathbf{T_{es}}$	**0.00**	**-**	***User Defined***	**Comments:**	**existing case**
Minimum End Sill Block Width	$W_{es,min}$	N/A	m	$\alpha*y_2$	**Comments:**	
End Sill Block Width	$\mathbf{W_{es}}$	**0.00**	**-**	***User Defined***	**Comments:**	**existing case**
Minimum End Sill Block Spacing	$S_{es,min}$	N/A	-	$\alpha*y_2$	**Comments:**	
End Sill Block Spacing	$\mathbf{S_{es}}$	**0.00**	**-**	***User Defined***	**Comments:**	**existing case**
Number of Blocks	N_{es}	N/A	-	$(B+S_{es})/(S_{es}+W_{es})$		
Spacing @ End	s_e	0.00	-	$(B+S_{es})/(S_{es}+W_{es})$		

Maynord (1990)
The Rock Manual - CIRIA C683 (2007), 2nd Edition. Page 651.
Recommended for the design of riprap

$$D_{30} = S_f C_s C_v C_T y \left[\left(\frac{1}{s-1} \right)^{0.5} \frac{U_d}{\sqrt{(K_1 g y)}} \right]^{2.5}$$

where: D30 is the characteristic riprap size of which 30% is finer by weight
Sf is a safety factor
Cs is a stability coefficient
Cv is a velocity distribution coefficient
Ct is a blanket thickness coefficient
s is the relative density of stone
Ud is the depth averaged flow velocity
K1 is a side slope correction factor
alpha is the angle of the bank to the horizontal
g is the acceleration due to gravity

Enter Data

Safety factor	Sf	1.5	(1.5 suggested)
Rock shape	A or R	R	(Angular or Rounded)
Location	S or D	D	(Straight channel or D/S of structure)
Thickness coeff	Ct	1	(1.0 suggested)
Rel. density of stone	s	2.65	
Angle of bank to horiz.	alpha	45	
Accel due to gravity	g	9.807	

Depth averaged velocity	Ud1	1.49	m/s
(use 3 to test sensitivity)	Ud2	2.35	m/s
	Ud3	3.06	m/s
	Ud4	3.69	m/s
	Ud5	4.26	m/s
	Ud6	4.78	m/s
	Ud7	5.27	m/s
	Ud8	5.74	m/s
	Ud9	6.18	m/s
	Ud10	6.59	m/s

Water depths	y1	0.30	m
	y2	0.60	m
	y3	0.90	m
	y4	1.20	m
	y5	1.50	m
	y6	1.80	m
	y7	2.10	m
	y8	2.40	m
	y9	2.70	m
	y10	3.00	m

Results

Stability coefficient	Cs	0.375
Vel. distribution coeff	Cv	1.25
COT alpha		1.000
Side slope correction	K1	0.416

Water depth, y (m)	Velocity, Ud (m/s)	D30 (m)	D50 (m)	Dn50 (min)	Dn50 (max)
0.3	1.485746161	0.24	0.34	0.28	0.31
0.6	2.345969986	0.62	0.89	0.75	0.81
0.9	3.058061436	1.09	1.56	1.31	1.42
1.2	3.685578544	1.62	2.31	1.94	2.10
1.5	4.255158398	2.19	3.13	2.63	2.85
1.8	4.781256906	2.80	4.00	3.36	3.64
2.1	5.272865326	3.44	4.91	4.13	4.47
2.4	5.73607816	4.11	5.87	4.93	5.34
2.7	6.175280015	4.79	6.85	5.75	6.23
3	6.593766666	5.50	7.86	6.60	7.15

Pilarczyk (1990)

The Rock Manual - CIRIA C683 (2007), 2nd Edition. Page 649.

Recommended for the design of riprap, cabled concrete blocks, box gabions and asphalt mattresses

$$\boxed{D_{n50} = \frac{\mu}{(s-1)} \frac{0.035}{\Psi_{CR}} \frac{K_T K_Y}{K_S} \frac{U^2}{2g}}$$

where:

μ is the stability correction factor

s is the relative density of the revetment

$\boxed{\Psi \quad CR}$ is the stability factor

D is the characteristic size of the protection
Kt is a turbulence factor
Kh is a depth factor
Ks is the slope factor
Ud is the depth averaged flow velocity
g is the acceleration due to gravity

Enter Data

Location	C or E	C	Continuous protection or Edges and transitions
Relative density of stone	s	2.65	
Porosity of stone	n	0.4	(suggest 0.4 for stone and sand)
Type of revetment	R or G	R	Riprap or Gabions
Bank slope	alpha	0	
Internal friction angle	sigma	45	
Longitudinal channel slope	beta	0.00176	
Turbulence factor	N or H	H	Normal or High
Accel due to gravity	g	9.807	m/s2
Flow decvelopment			

Depth averaged velocity	Ud1	1.49	m/s
	Ud2	2.35	m/s
	Ud3	3.06	m/s
	Ud4	3.69	m/s
	Ud5	4.26	m/s
	Ud6	4.78	m/s
	Ud7	5.27	m/s
	Ud8	5.74	m/s
	Ud9	6.18	m/s
	Ud10	6.59	m/s

Water depths	y1	0.30	m
	y2	0.60	m
	y3	0.90	m
	y4	1.20	m
	y5	1.50	m
	y6	1.80	m
	y7	2.10	m
	y8	2.40	m
	y9	2.70	m
	y10	3.00	m

Results

Stability correction factor		1
Stability factor		0.035
Relative density of revetment		1.65
Side slope term	Kd	1.000
Longitudinal slope	k1	1.000
Slope factor	Ks	1.000
Turbulence factor	Kt	1.500

tan alpha	0.000
cos alpha	1.000
tan sigma	1.000
sin sigma - beta	0.707
sin sigma	0.707

Estimate Dn50	Depth averaged velocity Ud (m/s)	Depth y (m)	Depth factor Kh	Dn50 (m)	D50 (min)	D50 (max)	D50 (ave)	D30 (min)	D30 (max)
0.08	1.485746161	0.3	0.764	0.078	0.09	0.09	0.09	0.06	0.07
0.21	2.345969986	0.6	0.808	0.206	0.23	0.25	0.24	0.16	0.17
0.36	3.058061436	0.9	0.833	0.361	0.40	0.43	0.41	0.28	0.30
0.54	3.685578544	1.2	0.851	0.536	0.59	0.64	0.61	0.41	0.45
0.73	4.255158398	1.5	0.865	0.726	0.80	0.86	0.83	0.56	0.60
0.93	4.781256906	1.8	0.876	0.928	1.02	1.11	1.06	0.71	0.77
1.14	5.272865326	2.1	0.885	1.141	1.25	1.36	1.31	0.88	0.95
1.36	5.73607816	2.4	0.893	1.362	1.50	1.62	1.56	1.05	1.13
1.59	6.175280015	2.7	0.899	1.590	1.75	1.89	1.82	1.22	1.32
1.82	6.593766666	3	0.905	1.824	2.00	2.17	2.09	1.40	1.52

Escarameia and May (1992) - HR Wallingford

The Rock Manual - CIRIA C683 (2007), 2nd Edition. Page 650.

Recommended for the design of riprap, loose or interlocking concrete blocks and gabion mattresses.

$$D_{n50} = C\frac{U_b{}^2}{2g(s-1)}$$

where: Dn50 is the characteristic size of stone
Dn50 is the size of the equivalent cube

$$D_{n50} = \left(\frac{W_{50}}{\rho_s}\right)^{\frac{1}{3}}$$

W50 is the weight of particle, Ps is the density of stone
C is a coefficient that takes account of the turbulence intensity TI
g is acceleration due to gravity
s is the relative density of the revetment material
Ub is the velocity near the bend (at 10% of the water depth above the bed)

Turbulence Levels

Situation	Turbulence Level	
	Qualitative	TI
Straight river or channel reaches and wide natural bends (R/W>26)	Normal (low)	0.12
Edge of revetments in straight reaches	Normal (higher)	0.20
Bridge piers, caissons and groynes; transitions	Medium to high	0.35-0.50
Downstream of hydraulic structures (weirs, culverts, stilling basins)	Very high	0.60

where: R is the centreline radius of bend
W is the water surface width at the upstream end of the bend

Values of C

Type of revetment	Value of C	Observations
Riprap	12.3TI-0.20	Valid for TI>=0.05
Concrete blocks	9.22TI-0.15	Valid for TI>=0.05
Gabion mattresses	12.3TI-1.65	Valid for TI>=0.12

Enter Data

Turbulence Intensity	TI	0.6	
Type of revetment	R,C or G	R	(Riprap, Concrete or Gabion)
Relative density of riprap	s	2.65	
Accel due to gravity	g	9.81	m/s2
Depth averaged velocity	Ud1	1.49	m/s
	Ud2	2.35	m/s
	Ud3	3.06	m/s
	Ud4	3.69	m/s
	Ud5	4.26	m/s
	Ud6	4.78	m/s
	Ud7	5.27	m/s
	Ud8	5.74	m/s
	Ud9	6.18	m/s
	Ud10	6.59	m/s

Suggested factor for Ub	47%
Actual	60%

Results

Coefficient C 7.18

Depth averaged velocity	Ub	Ub2	Dn50	D50 (min)	D50 (max)	D50 (ave)	D30 (min)	D30 (max)
1.486	0.891	0.795	0.18	0.19	0.21	0.20	0.14	0.15
2.346	1.408	1.981	0.44	0.48	0.52	0.50	0.34	0.37
3.058	1.835	3.367	0.75	0.82	0.89	0.85	0.57	0.62
3.686	2.211	4.890	1.08	1.19	1.29	1.24	0.83	0.90
4.255	2.553	6.518	1.45	1.59	1.72	1.65	1.11	1.20
4.781	2.869	8.230	1.83	2.01	2.17	2.09	1.40	1.52
5.273	3.164	10.009	2.22	2.44	2.64	2.54	1.71	1.85
5.736	3.442	11.845	2.63	2.89	3.13	3.01	2.02	2.19
6.175	3.705	13.728	3.04	3.35	3.62	3.49	2.34	2.54
6.594	3.956	15.652	3.47	3.81	4.13	3.97	2.67	2.89

Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 999 Summary Sheet.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

Operational Scenario

Number of Operational Gates	1.00		
Flow	7.08	m^3s^{-1}	**250 cfs**
Gage Level	0.91	m	**3 ft**
River Level	26.79	mAD	**87.9 ft**
Gate Opening	0.23	m	**9 inches/0.75 ft**
Stilling Basin	As existing		

Downstream Rating Curve

Depth of Flow	Water Level	Discharge	Average Velocity	Froude
D	Zwl	Q	v	Fr
m	mAD	m^3s^{-1}	ms-1	-
0.00	24.40	0.00	0.00	0.00
0.30	24.70	30.02	1.49	0.87
0.60	25.00	95.23	2.35	0.97
0.90	25.30	187.03	3.06	1.03
1.20	25.60	301.88	3.69	1.07
1.50	25.90	437.57	4.26	1.11
1.80	26.20	592.59	4.78	1.14
2.10	26.50	765.77	5.27	1.16
2.40	26.80	956.17	5.74	1.18
2.70	27.10	1163.06	6.18	1.20
3.00	27.40	1385.80	6.59	1.22

Radial Gate with Raised Cill

Max Unit Discharge	qmax	1.76	m^2s^{-1}
Design Unit Discharge	q	1.75	m^2s^{-1}
Flow	Qmax	8.60	m^3s^{-1}

Stilling Basin Level Design

Condition 1 - 100% Design Flow with 100% Design Flow Downstream

Condition 2 - 10% Design Flow with 0% Design Flow Downstream

Condition 3 - 30% Design Flow with 30% Design Flow Downstream

Summary of Critical Basin

Basin Level	22.74	mAD
Critical Condition	Condition 1	
Froude	17.23	
Critical Condition	Condition 2	

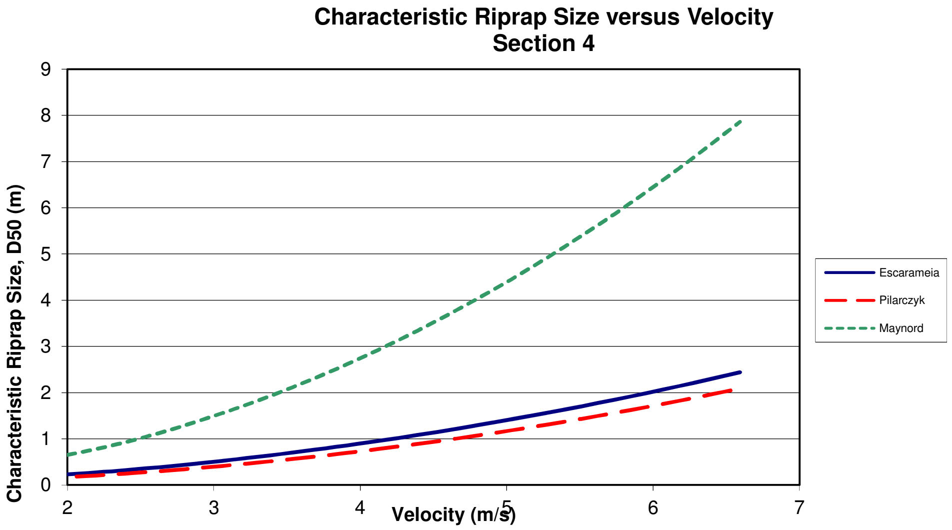
Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 999 Summary Sheet.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

Conjugate Depth	1.53	mAD
Critical Condition	Condition 1	
Max Basin Length	3.92	mAD
Critical Condition	Condition 1	

USBR Stilling Basins

Minimum Basin Length	7.84	m
Minimum End Sill Height	0.27	m
Minimum End Sill Thickness	0.00	m

Erosion Stone Sizing



Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 000 Cover Sheet.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

Operational Scenario

Number of Operational Gates	1		
Flow	7.36	m^3s^{-1}	**260 cfs**
Gage Level	1.0668	m	**3.5 ft**
River Level	26.94	mAD	**88.4 ft**
Gate Opening	0.229	m	**9 inches/0.75 ft**
Stilling Basin	As existing		

Downstream Rating Curve

A downstream rating curve was produced to enable us identify the tail water level versus discharge assuming a trapezoidal channel.
A bed width of 220 ft was taken using a side slope of 1:1, bed slope of 0.01 and Manning's coefficient of 0.03 as per the original design calculation. The bed level was taken as 80.05 ft, the level of the concrete apron.
Using basic geometry for various depths of flows, the flow area and hydraulic radius were calculated. These could then be used to calculate the discharge.

Radial Gate with Raised Cill

The gate width (16 ft), cill level (81.05 ft) and gate height (7 ft) were taken as per original construction drawings. The safety factor was assumed to be 1.
Assuming unsubmerged conditions and a maximum opening of the 7ft high radial gate, the maximum unit discharge can then be calculated.

Stilling Basin Level Design

Three worst-case scenarios were considered for the stilling basin design. These are:
- 100% flow both upstream and downstream of the structure, to simulate design flow conditions
- 10% upstream flow with 0% flow downstream to simulate sudden gate opening
- 30% upstream flow with 30% downstream flow

A stilling basin safety factor of 1.1 was used for this calculation. Downstream conditions for the various scenarios can be determined from the downstream rating curve. Approach velocities were estimated using the inflow and height of the gates.
Head differences across the gate could be calculated by using the known qate levels and velocities upstream and downstream of the structure. This can be used to determine various parameters about the flow at the glasis and conjugate depth of the given scenario. Based on the Froude number, the performance of the stilling basin can be determined. For the scenarios the minimum basin level, maximum Froude number, maximum conjugate depth and maximum free jump length are selected as the parameters required for the critical basin.

USBR Stilling Basins

This was undertaken using the USBR Design of Small Dams for a type I stilling basin with no chute or baffle blocks. Along with the parameters determined from the level design, the required basin and end sill dimensions were determined.

Erosion Stone Sizing

Erosion stone sizing was undertaken using The Rock Manual with the Escarameia, Pilarczyk and Maynord methods. The relative density of riprap was taken as 2.65 and the angle of the bank to the horizontal was taken as 45 degrees. All other required inputs were taken from geometry of the structure.

Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 001 EVT-HYES-HYD-300 Trapizoidal Rating Curve.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

Manning's Rating Curve

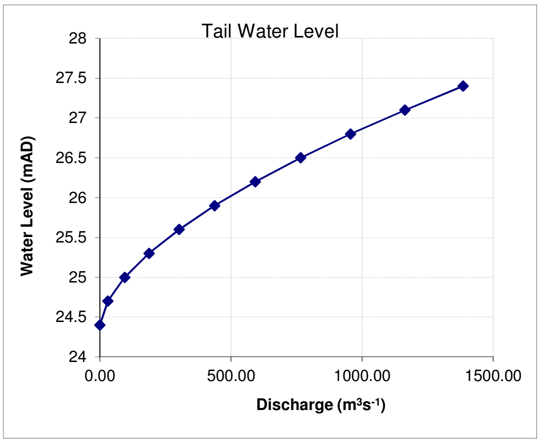
Open Channel Hydraulics, 1959, Ven Te Chow

1. INPUTS

Bed Width	B	67.056	m	**Comments:**	**220 ft as per original calculation**
Side Slope	s	1	H:1V	**Comments:**	**As per original calc**
Bed Slope	i	0.01	m/m	**Comments:**	**As per original calc**
Manning's Coefficient	n	0.03	$sm^{-1/3}$	**Comments:**	**As per original calc**
Bed Level	Zb	24.39924	mAD	**Comments:**	**80.05 ft**
Max Depth	Dmax	3	m	**Comments:**	

2. OUTPUTS

#	Depth of Flow	Flow Area	Hydraulic Radius	Discharge	Water Level	Average Velocity	Froude
n	D	A	P	Q	Zwl	v	Fr
-	m	m^2	m	m^3s^{-1}	mAD	ms-1	-
-	-	D_n*$(B+D_n$*s)	B+2*D_n*√$(1+s^2)$	$A(A/P)^{2/3}$*√(i) / n	D_n+Z_b	Q/A	v/√(g*D_n)
1	0	0	67.06	0.00	24.39924	0	0
2	0.3	20.2068	67.90	30.02	24.69924	1.49	0.87
3	0.6	40.5936	68.75	95.23	24.99924	2.35	0.97
4	0.9	61.1604	69.60	187.03	25.29924	3.06	1.03
5	1.2	81.9072	70.45	301.88	25.59924	3.69	1.07
6	1.5	102.834	71.30	437.57	25.89924	4.26	1.11
7	1.8	123.9408	72.15	592.59	26.19924	4.78	1.14
8	2.1	145.2276	73.00	765.77	26.49924	5.27	1.16
9	2.4	166.6944	73.84	956.17	26.79924	5.74	1.18
10	2.7	188.3412	74.69	1163.06	27.09924	6.18	1.20
11	3	210.168	75.54	1385.80	27.39924	6.59	1.22



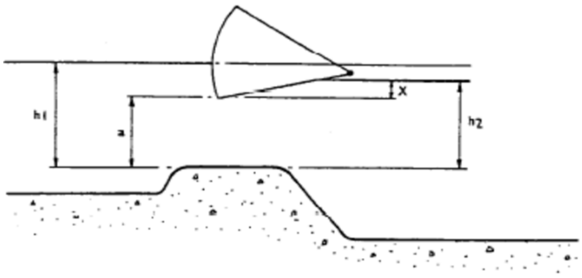
Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 002 EVT-HYES-HYD-561 Radial Gate with Raised Cill.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

Radial Gate With Cill

Hydraulic Design of Canal Structures - MMP August 1985

USBR - Design of Small Small Dams

1. INPUTS



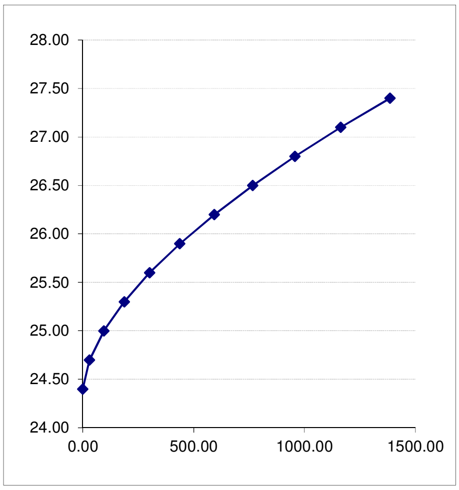
1. Hydraulic Design of Canal Structures, Sept 1986, MM & Partners p. 1 - p. 11

2. Design of Small Dams - USBR

1.1 GENERAL

Design Discharge	Q	7.36	m^3s^{-1}	**Comments:**	**260fps**
Number of Gates	N_g	1	-	**Comments:**	
Gate Width	w	4.88	m	**Comments:**	**16 ft**
Cill Level	Z_c	24.70	mAD	**Comments:**	**81.05ft**
Max Upstream Water Level	Z_{uswl}	26.94	mAD	**Comments:**	**88.4ft (3.5ft gage)**
Structure Flow Saftey Factor	S_f	1	-	**Comments:**	
Maximum gate opening (as percentage of max head)	μ	21%	-	**Comments:**	**Maximum opening for design dischage**
Design Gate Height	a	0.229	-	**Comments:**	**9 inches/0.75 ft**
Design Downstream Water Level	Z_{dswl}	24.66	mAD	**Comments:**	

	Level	Flow	Velocity
1	mAD	m3/s	m/s
2	24.40	0.00	0.00
3	24.70	30.02	1.49
4	25.00	95.23	2.35
5	25.30	187.03	3.06
6	25.60	301.88	3.69
7	25.90	437.57	4.26
8	26.20	592.59	4.78
9	26.50	765.77	5.27
10	26.80	956.17	5.74
11	27.10	1163.06	6.18
12	27.40	1385.80	6.59
13			
14			



Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 002 EVT-HYES-HYD-561 Radial Gate with Raised Cill.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

1.2 Effective Crest Coefficients

Pier Contraction Coefficient	k_p	0.020	-		**Comments:**	**USBR Design of Small Dams, square nosed piers**
Abutment Contraction Coefficient	k_a	0.2	-		**Comments:**	**USBR Design of Small Dams, square abutments**

2. OUTPUTS

2.1 GENERAL

Number of Piers	N	0		N_g-1	**Comments:**
Sum of Gates Width	b'	4.88	m	N_g*w	**Comments:**
Design Factored Discharge	Q_f	7.36	m^3s^{-1}	S_f*Q	**Comments:**
Upstream Head Over Cill	h_1	2.24	m	$Z_{uswl} - Z_c$	**Comments:**
Downstream Head Over Cill	h_2	-0.039	m	$Z_{dswl} - Z_c$	**Comments:**
Effective Crest Length	b	3.98	m	$b'-2*(N*K_p+k_a)*h_1$	**Comments:**

2.2 FREE FLOW CALCULATIONS

Max Height of Gate Opening	$a_{max\ 1}$	0.47	m	$\mu*h_1$	**Comments:**
Opening - DS Water Level Difference	x	-0.24	m	$h_2 - a$	**Comments:**
Max Unit Discharge	q_{max}	1.871	m^2s^{-1}	$0.6*a_{max\ 1}*\sqrt{(2*g*h_1)}$	**Comments:**
Design Unit Discharge	q	1.850	m^2s^{-1}	Q_f/b	**Comments:**
Flow	Q_{max}	9.127	m^3s^{-1}	q*w*N	**Comments:**
CHECK		OK			**Comments:**
Gate Radius	R	0.627	m	4/3*amax 1	**Comments:**
Depth Above Gate	a'	1.77	m	h1-amax 1	**Comments:**
Angle	θ	#NUM!	Degrees	$\cos^{-1}(a/R)$	**Comments:**
Vena Contrata Factor	δ	0.65	-		**Comments:**
Vena Contrata	α	0.31	m	$a*\delta$	**Comments:**
Velocity	v	6.12	ms^{-1}	q/α	**Comments:**
Froude Number	Fr	3.53	-		**Comments:**
Conjugate Depth	y_2	1.53	m	$y_1/2*\sqrt{((1+8Fr_1^2)-1)}$	**Comments:**
Water Level	Zc	26.23	mAD	y2+Zc	**Comments:**
Downstream Water Level	Zds	24.49	mAD		**Comments:**

2.3 SUBMERGED CONDITIONS

Max Height of Gate Opening	$a_{max\ 2}$	1.49	m	$2/3*h_1$	**Comments:**
Opening - DS Water Level Difference	x	-1.53	m	$h_2 - a$	**Comments:**
Head Drop	H	2.28	m	h_1-h_2	
Max Unit Discharge	q_{max}	6.692	m^2s^{-1}	$0.67*a_{max\ 2}*\sqrt{(2*g*(h_1-h_2))}$	
Design Unit Discharge	q	1.850	m^2s^{-1}	Q_f/b'	**Comments:**

Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 002 EVT-HYES-HYD-561 Radial Gate with Raised Cill.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

CHECK		OK		$q < q_{max}$	**Comments:**
Gate Radius	R	1.991	m	*4/3*a*	**Comments:**
Flow Through One Gate @ 100% a	$Q_{n=1}$	26.638	m^3s^{-1}	q_{max} **w*	**Comments:**
Design Flow	Q	26.638	m^3s^{-1}		

2.4 DESIGN CONDITIONS

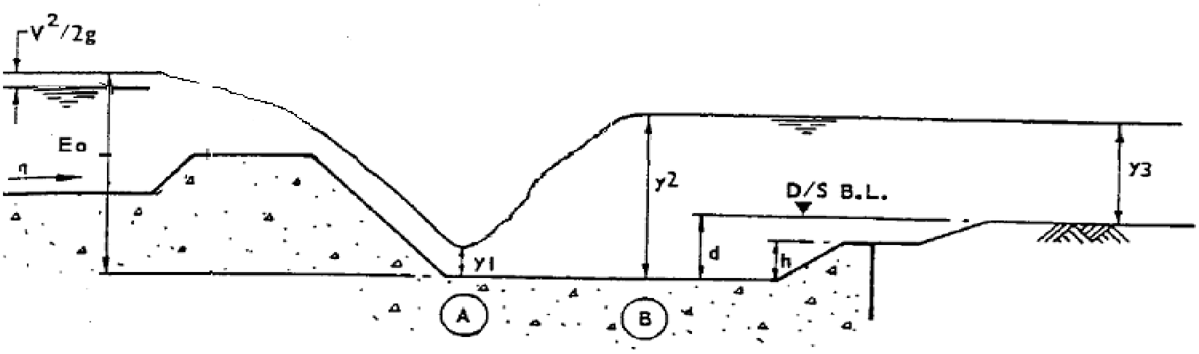
Condition		UNSUBMERGED			**Comments:**
Design Flow	Q_d	9.127	m^3s^{-1}	*q*w*N*	**Comments:**

Project **Priest Lake Dam**		MOTT MACDONALD			
Calculations for 003 EVT-HYES-HYD-001 - Stilling Basin Level Design.xls.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

Stilling Basin Level Design

1. MMP Hydraulic Design of Canal Structures, 1985, 1-13 - 1-14

1. INPUTS

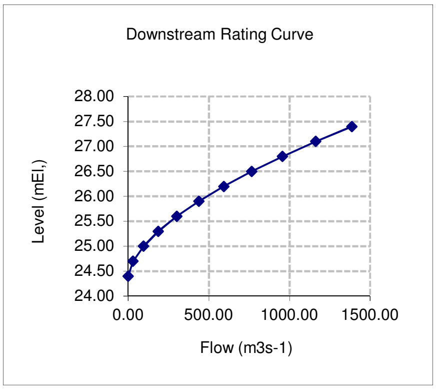


1.1 GENERAL

Design Flow	Q_d	7.4	m^3s^{-1}	**Comments:**	
Flow Safety Factor	S_f	1.00	-	**Comments:**	
Width of Stilling Basin	B	4.88	m	**Comments:**	**16ft*1 no.**
Number of Basins	N	1	-	**Comments:**	
Basin Safety Factor	S_f	1.1	-	**Comments:**	
Bed Level US	Z_{usbl}	24.40	mAD	**Comments:**	**80.05 ft**
Bed Level DS	Z_{dsbl}	24.40	mAD	**Comments:**	**80.05 ft**
Allowance For Retrogression (Percentage of D/S Depth)	ρ	0%	%	**Comments:**	
Gate Width	w_g	4.88	m	**Comments:**	**16ft*1 no.**

Downstream Rating Curve (Table 1.1)

#	Level	Flow	Velocity
n	Z_{dswl}	Q	v
-	mAD	m^3s^{-1}	ms^{-1}
1	24.40	0.00	0.00
2	24.70	30.02	1.49
3	25.00	95.23	2.35
4	25.30	187.03	3.06
5	25.60	301.88	3.69
6	25.90	437.57	4.26
7	26.20	592.59	4.78
8	26.50	765.77	5.27
9	26.80	956.17	5.74
10	27.10	1163.06	6.18
11	27.40	1385.80	6.59



Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 003 EVT-HYES-HYD-001 - Stilling Basin Level Design.xls.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

1.2 CONDITION 1

1.2.1 General

Percentage of Flow US	$m_{us,1}$	100%		**Comments:**	
Percentage of Flow DS	$m_{ds,1}$	100%		**Comments:**	
Additional flow downstream	$Q_{add,1}$	0.0	m^3s^{-1}	**Comments:**	

1.3.2 Upstream Conditions

Water Level	$Z_{us,1}$	26.9	mAD	**Comments:**	**88.4ft (3.5ft gage)**
Velocity	$v_{us,1}$	2.023	ms^{-1}	**Comments:**	
Maximum Unit Discharge	$q_{max,1}$	1.871	m^2s^{-1}	**Comments:**	

1.3 CONDITION 2

1.3.1 General

Percentage of Flow US	$m_{us,2}$	10%		**Comments:**	
Percentage of Flow DS	$m_{ds,2}$	0%		**Comments:**	
Additional flow downstream	$Q_{add,2}$	0.0	m^3s^{-1}	**Comments:**	

1.3.2 Upstream Conditions

Water Level	$Z_{us,2}$	26.9	mAD	**Comments:**	
Velocity	$v_{us,1}$	0.82	ms^{-1}	**Comments:**	
Maximum Unit Discharge	$q_{max,1}$	0.187	m^2s^{-1}	**Comments:**	

1.4 CONDITION 3

1.4.1 General

Percentage of Flow US	$m_{us,3}$	30%		**Comments:**	
Percentage of Flow DS	$m_{ds,3}$	30%		**Comments:**	
Additional flow downstream	$Q_{add,3}$	0.0	m^3s^{-1}	**Comments:**	

1.4.2 Upstream Conditions

Water Level	$Z_{us,3}$	26.9	mAD	**Comments:**	
Velocity	$v_{us,3}$	2.46	ms^{-1}	**Comments:**	
Maximum Unit Discharge	$q_{max,3}$	0.561	m^2s^{-1}	**Comments:**	

Project

Priest Lake Dam

MOTT MACDONALD

Calculations for	Division	WCD	File No.		
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2. OUTPUTS

Condition 1 - 100% Design Flow with 100% Design Flow Downstream

Condition 2 - 10% Design Flow with 0% Design Flow Downstream

Condition 3 - 30% Design Flow with 30% Design Flow Downstream

2.1 GENERAL

Factored Design Flow	Q_f	7.4	m^3s^{-1}	Q_d*S_f	**Comments:**
2.1.1 Condition 1					**Comments:**
Factored Discharge US	$Q_{us,1}$	7.4	m^3s^{-1}	$Q_f*m_{us,1}$	**Comments:**
Discharge DS	$Q_{ds,1}$	7.4	m^3s^{-1}	$Q_f*m_{ds,1}$	**Comments:**
DS Water Level	$Z_{ds,1}$	24.5	mAD	*Interpolate (Table 1.1)*	**Comments:**
Velocity @ Q DS	$v_{ds,1}$	0.36	ms^{-1}	*Interpolate (Table 1.1)*	**Comments:**
2.1.2 Condition 2					**Comments:**
Factored Discharge US	$Q_{us,2}$	0.7	m^3s^{-1}	$Q_f*m_{us,2}$	**Comments:**
Discharge DS	$Q_{ds,2}$	0.0	m^3s^{-1}	$Q_f*m_{ds,2}$	**Comments:**
DS Water Level	$Z_{ds,2}$	24.399	mAD	*Interpolate (Table 1.1)*	**Comments:**
Velocity @ Q DS	$v_{ds,2}$	0.00	ms^{-1}	*Interpolate (Table 1.1)*	**Comments:**
2.1.3 Condition 3					**Comments:**
Factored Discharge US	$Q_{us,3}$	2.2	m^3s^{-1}	$Q_f*m_{us,3}$	**Comments:**
Discharge DS	$Q_{us,3}$	2.2	m^3s^{-1}	$Q_f*m_{ds,3}$	**Comments:**
DS Water Level	$Z_{ds,3}$	24.421	mAD	*Interpolate (Table 1.1)*	**Comments:**
Velocity @ Q DS	$v_{ds,3}$	0.11	ms^{-1}	*Interpolate (Table 1.1)*	**Comments:**

2.2 DOWNSTREAM CONDITION

Water Depth DS Condition 1	$D_{ds,1}$	0.074	m	$Z_{dswl}-Z_{bl}$	**Comments:**
Water Depth DS Condition 2	$D_{ds,2}$	0.000	m	$Z_{dswl}-Z_{bl}$	**Comments:**
Water Depth DS Condition 3	$D_{ds,3}$	0.022	m	$Z_{dswl,70}-Z_{bl}$	**Comments:**
Retrogression @ Condition 1	r_1	0.000	m	$\rho*D_{ds,1}$	**Comments:**
Retrogression @ Condition 2	r_2	0.000	m	$\rho*D_{ds,2}$	**Comments:**
Retrogression @ Condition 3	r_3	0.000	m	$\rho*D_{ds,3}$	**Comments:**
Water Level Condition 1 with Retrogression	$Z_{dsl,r}$	24.473	mAD	$Z_{ds,1}-r_1$	**Comments:**
Water Level Condition 2 with Retrogression	$Z_{dsl,r}$	24.399	mAD	$Z_{ds,2}-r_2$	**Comments:**
Water Level Condition 3 with Retrogression	$Z_{dsl,70,r}$	24.421	mAD	$Z_{ds,3}-r_3$	**Comments:**

Project **Priest Lake Dam**	MOTT MACDONALD				
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Stilling Basin General Design - Condition 1

1. INPUTS

Design Flow	$Q_{us,1}$	7.36	m^3s^{-1}
Width of Stilling Basin	B	4.88	m
Upstream Water Level	$Z_{us,1}$	26.94	mAD
Approach velocity	$v_{us,1}$	2.023	ms^{-1}
Downstream Water Level	$Z_{ds,1}$	24.5	mAD
Downstream Velocity	$v_{ds,1}$	0.36	ms^{-1}
Basin Safety Factor	S_f	1.1	-

2.2 OUTPUTS - CONDITION 1

Flow Intensity	q_1	1.51	m^2s^{-1}	$Q_{us,1}/B/N$	**Comments:**
Critical Depth	$d_{c,1}$	0.61	m	$(q_1^2/g)^{1/3}$	**Comments:**
Upstream Velocity Head	$h_{v,1}$	0.21	m	$v_{us,1}^2/2g$	**Comments:**
Downstream Velocity Head	$h_{v2,1}$	0.01	m	$v_{ds,1}^2/2g$	**Comments:**
Upstream Total Head	$E_{o,1}$	27.15	m	$Z_{us,1} + h_{v,1}$	**Comments:**
Downstream Total Head	$E_{2,1}$	24.48	m	$Z_{ds,1} + h_{2v,1}$	**Comments:**
Head Difference Across Gate	$H_{L,1}$	2.67	m	$E_{0,1}-E_{2,1}$	**Comments:**
	H_L/d_c	4.35	m	H_L/d_c	**Comments:**
	a_1	0.11	-	*GOAL SEEK*	**Comments:**
	$v_{1,1}$	4.35	-	$(1-a_1)^3/(2a_1)*((1+2a_1)^2-1)^{-1/3})$	
Congugate Depth	$y_{2,1}$	1.58	m	$4*H_{L,1}*a_1/(1-a_1)^3$	
Velocity @ Congugate Depth	$v_{2,1}$	0.95	ms^{-1}	$q_1/y_{2,1}$	
Depth of Flow at Bottom of Glasis	$y_{1,1}$	0.17	m	$a_1*y_{2,1}$	
Velocity @ Toe of Glasis	$v_{1,1}$	9.01	ms^{-1}	$q_1/y_{1,1}$	**Comments:**
Froude Number	Fr_1	7.02	-	$v_{1,1}/\sqrt{(gy_{1,1})}$	**Comments:**
Conjugate Depth Check Calc	$y_{2,1}$	1.58	m	$y_{1,1}/2*\sqrt{((1+8Fr_1^2)-1)}$	**Comments:**
Total Energy at Congugate Depth	$E_{1,1}$	1.63	m	$y_{2,1}+d_{c,1}^3/(2*y_{2,1}^2)$	
Basin Level	$\mathbf{Z_{b,1}}$	**22.69**	**mAD**	$E_{2,1}-S_f*E_{1,1}$	**Comments:**
Downstream Jump Level	Z_{dsjl}	24.27	m	$Z_{dsbs}+y_2$	**Comments:**
	L/y_2	6.11	m	*INTERPOLATE*	**Comments:**
	L/y_2	6.11	m	*INTERPOLATE*	**Comments:**
Jump Length	L	9.67	m	L/y_2*y_2	**Comments:**
Free Jump Performance		Best Performance			
Basin Type		Type III			**Comments:**
Basin Jump Length Factor	L_B/y_2	2.58	m		
Basin Jump Length	L_B	4.09	m		

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2.2.1 Design Check (Set Outputs First)

Total Energy	$E_{0,1d}$	2.75	m	$Z_{us,1}+h_{v,1}-Z_b$	**Comments:**
Velocity	$v_{1,1d}$	7.06	ms^{-1}	$\surd(2g*(E_{0,1d}-y_{1,1d}))$	**Comments:**
Depth of Flow	$y_{1,1d}$	0.214	m	$q_1/v_{1,1d}$	**Comments:**
Upstream Froude Number	$\mathbf{Fr_{1,1d}}$	**4.872**	-	$v_{1,1d}/\surd(gy_{1,1d})$	**Comments:**
Conjugate Depth	$y_{2,1d}$	1.37	-	$y_{1,1d}/2*(\surd(1+8Fr_{1d}^{2})-1)$	
Conjugate Level	$y_{2,1d}$	25.77	-	$y_{1,1d}/2*(\surd(1+8Fr_{1d}^{2})-1)$	

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Stilling Basin General Design - Condition 2

1. INPUTS

Design Flow	$Q_{us,2}$	0.74	m^3s^{-1}
Width of Stilling Basin	B	4.88	m
Upstream Water Level	$Z_{us,2}$	26.94	mAD
Approach velcoity	$v_{us,2}$	0.819	ms^{-1}
Downstream Water Level	$Z_{ds,2}$	24.4	mAD
Downstream Velocity	$v_{ds,2}$	0.00	ms^{-1}
Basin Safety Factor	S_f	1.1	-

2.3 OUTPUTS - CONDITION 2

Flow Intensity	q_2	0.15	m^2s^{-1}	$Q_{us,2}/B/N$	**Comments:**
Critical Depth	$d_{c,2}$	0.13	m	$(q_2{}^2/g)^{1/3}$	**Comments:**
Upstream Velocity Head	$h_{v,2}$	0.03	m	$v_{us,2}{}^2/2g$	**Comments:**
Downstream Velocity Head	$h_{v2,2}$	0.00	m	$v_{ds,2}{}^2/2g$	**Comments:**
Upstream Total Head	$E_{o,2}$	26.98	m	$Z_{us,2} + h_{v,2}$	**Comments:**
Downstream Total Head	$E_{2,2}$	24.40	m	$Z_{ds,2} + h_{2v,2}$	**Comments:**
Head Difference Across Gate	$H_{L,2}$	2.58	m	$E_{0,2}-E_{2,2}$	**Comments:**
	H_L/d_c	19.47	m	H_L/d_c	**Comments:**
	a_2	0.04	-	*GOAL SEEK*	**Comments:**
	$v_{1,2}$	19.47	-	$(1-a_2)^3/(2a_2)^*((1+2a_2)^2-1)^{-1/3})$	
Congugate Depth	$y_{2,2}$	0.48	m	$4^*H_{L,2}{}^*a_2/(1-a_2)^3$	
Velocity @ Congugate Depth	$v_{2,2}$	0.32	ms^{-1}	$q_2/y_{2,2}$	
Depth of Flow at Bottom of Glacis	$y_{1,2}$	0.02	m	$a_2{}^*y_{2,2}$	
Velocity @ Toe of Glacis	$v_{1,2}$	7.73	ms^{-1}	$q_2/y_{1,2}$	**Comments:**
Froude Number	Fr_2	17.65	-	$v_{1,2}/\surd(gy_{1,2})$	**Comments:**
Conjugate Depth Check Calc	$y_{2,2}$	0.48	m	$y_{1,2}/2^*\surd((1+8Fr_2{}^2$	**Comments:**
Total Energy at Congugate Depth	$E_{1a,2}$	0.48	m	$y_{2,2}+d_{c,2}{}^3/(2^*y_{2,2}$	**Comments:**
Basin Level	$\mathbf{Z_{dsbs}}$	**23.87**	**mAD**	$E_{2,2}-S_f{}^*E_{1,2}$	**Comments:**
Downstream Jump Level	Z_{dsjl}	24.35	m	$Z_{dsbs} + y_2$	**Comments:**
	L/y_2	5.74	m	*INTERPOLATE*	**Comments:**
	L/y_2	5.74	m	*INTERPOLATE*	**Comments:**
Jump Length	L	2.74	m	$L/y_2{}^*y_2$	**Comments:**
Free Jump Performance		Expensive Stilling Basin and Rough Surface Conditions			
Basin Type		Type III			**Comments:**
Basin Jump Length Factor	L_B/y_2	2.73	m		
Basin Jump Length	L_B	1.30	m		

Project **Priest Lake Dam**	MOTT MACDONALD				
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2.3.1 Design Check (Set Outputs First)

Total Energy	E_0	2.58	m	$Z_{uswl}+h_v-Z_b$	**Comments:**
Velocity	v_1	7.08	ms^{-1}	$\sqrt{(2g^*(E_0-y_1))}$	**Comments:**
Depth of Flow	y_1	0.021	m	q/v	**Comments:**
Upstream Froude Number	$\mathbf{Fr_1}$	**15.490**	-	$v_1/\sqrt{(gy_1)}$	**Comments:**
Conjugate Depth	y_2	0.46	-	$y_1/2^*(\sqrt{(1+8Fr_1^2)}-1)$	**Comments:**
Conjugate Level	y_2	24.86	-	$y_1/2^*(\sqrt{(1+8Fr_1^2)}-1)$	**Comments:**

Project **Priest Lake Dam**	MOTT MACDONALD				
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Stilling Basin General Design - Condition 3

MM Hydraulic Structures, 1985

1. INPUTS

Design Flow	$Q_{us,3}$	2.21	m^3s^{-1}
Width of Stilling Basin	B	4.88	m
Upstream Water Level	$Z_{us,3}$	26.94	mAD
Approach velcoity	$v_{us,3}$	2.456	ms^{-1}
Downstream Water Level	$Z_{ds,3}$	24.4	mAD
Downstream Velocity	$v_{ds,3}$	0.11	ms^{-1}
Basin Safety Factor	S_f	1.1	-

2.4 OUTPUTS - CONDITION 3

Flow Intensity	q_3	0.45	m^2s^{-1}	$Q_{us,3}/B/N$	**Comments:**
Critical Depth	$d_{c,3}$	0.28	m	$(q_3^{\,2}/g)^{1/3}$	**Comments:**
Upstream Velocity Head	$h_{v,3}$	0.31	m	$v_{us,3}^{\,2}/2g$	**Comments:**
Downstream Velocity Head	$h_{v2,3}$	0.00	m	$v_{ds,3}^{\,2}/2g$	**Comments:**
Upstream Total Head	$E_{o,3}$	27.25	mAD	$Z_{us,3} + h_{v,3}$	**Comments:**
Downstream Total Head	$E_{2,3}$	24.42	mAD	$Z_{ds,3} + h_{2v,3}$	**Comments:**
Head Difference Across Gate	$H_{L,3}$	2.83	m	$E_{0,3}-E_{2,3}$	**Comments:**
	H_L/d_c	10.27	m	H_L/d_c	**Comments:**
	a_3	0.06	-	*GOAL SEEK*	**Comments:**
	$v_{1,3}$	10.27	-	$(1-a_3)^3/(2a_3)*((1+2a_3)^2-1)^{-1/3})$	
Congugate Depth	$y_{2,3}$	0.86	m	$4*H_{L,3}*a_3/(1-a_3)^3$	
Velcoity @ Congugate Depth	$v_{2,3}$	0.53	ms^{-1}	$q_3/y_{2,3}$	
Depth of Flow at Bottom of Glasis	$y_{1,3}$	0.05	m	$a_3*y_{2,3}$	
Velocity @ Toe of Glasis	$v_{1,3}$	8.46	ms^{-1}	$q_3/y_{1,3}$	**Comments:**
Froude Number	Fr_3	11.67	-	$v_{1,3}/\sqrt{(gy_{1,3})}$	**Comments:**
Conjugate Depth Check Calc	$y_{2,3}$	0.86	m	$y_{1,3}/2*\sqrt{((1+8Fr_3^{\,2})-1)}$	**Comments:**
Total Energy at Congugate Depth	$E_{1a,3}$	0.87	m	$y_{2,3}+d_{c,3}^{\,3}/(2*y_{2,3}^{\,2})$	
Basin Level	$\mathbf{Z_{dsbs}}$	**23.46**	**mAD**	$E_{2,3}-S_f*E_{1,3}$	**Comments:**
Downstream Jump Level	Z_{dsjl}	24.32	m	$Z_{dsbs} + y_2$	**Comments:**
	L/y_2	6.05	m	*INTERPOLATE*	**Comments:**
	L/y_2	6.05	m	*INTERPOLATE*	**Comments:**
Jump Length	L	5.19	m	L/y_2*y_2	**Comments:**
Free Jump Performance		Acceptable Performance			
Basin Type		Type III			**Comments:**
Basin Jump Length Factor	L_B/y_2	2.77	m		
Basin Jump Length	L_B	2.38	m		

Project **Priest Lake Dam**					
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2.4.1 Design Check (Set Outputs First)

Total Energy	E_0	2.85	m	$Z_{uswl}+h_v-Z_b$	**Comments:**
Velocity	v_1	7.40	ms^{-1}	$\sqrt{}(2g^*(E_0-y_1))$	**Comments:**
Depth of Flow	y_1	0.061	m	q/v	**Comments:**
Upstream Froude Number	$\mathbf{Fr_1}$	**9.548**	-	$v_1/\sqrt{}(gy_1)$	**Comments:**
Conjugate Depth	y_2	0.80	-	$y_1/2^*(\sqrt{}(1+8Fr_1^2)-1)$	**Comments:**
Conjugate Level	y_2	25.20	-	$y_1/2^*(\sqrt{}(1+8Fr_1^2)-1)$	**Comments:**

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Stilling Basin Check - Outputs

MM Hydraulic Structures, 1985

2. OUTPUTS

Condition 1 - 100% Design Flow with 100% Design Flow Downstream

Condition 2 - 10% Design Flow with 0% Design Flow Downstream

Condition 3 - 30% Design Flow with 30% Design Flow Downstream

2.1 SUMMARY OF CRITICAL BASIN

2.2.1 Basin Level					
Basin Level	Z_{bs}	22.687	mAD	*MIN(C1,C2,C3)*	**Comments:**
Critical Condition		Condition 1	-		**Comments:**
Basin Level Selected	Z_b	**24.400**	mAD	***USER DEFINED***	**Comments:**
2.2.2 Froude Number					
Froude		17.647	-	*MAX(C1,C2,C3)*	**Comments:**
Critical Froude Condition		Condition 2	-		**Comments:**
2.2.3 Conjugate Depth					
Conjugate Depth		1.583	m	*MAX(C1,C2,C3)*	**Comments:**
Max Conjugate Depth Condition		Condition 1	-		**Comments:**
2.2.4 Designed Basin Length					
Max Basin Length		4.088	-	*MAX(C1,C2,C3)*	**Comments:**
Max Basin Length Condition		Condition 1	-		**Comments:**
2.2.5 Free Jump Length					
Max Free Jump Length		9.675	m	*MAX(C1,C2,C3)*	**Comments:**
2.2.6 General					
Minimum Basin Length		9.501	m	6*y2,crit	**Comments:**

Project Priest Lake Dam	MOTT MACDONALD				
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Stilling Basin Check - Gates Open Check

MM Hydraulic Structures, 1985

2. Outputs

2.1 100% MAX GATE DISCHARGE

Maximum Unit Discharge Per Gate	q_{max}	1.87	m^2s^{-1}		**Comments:**	
Maximum Water Level - Basin Level Difference	E_0	2.54	m	$Z_{uswl,c1}-Z_{bs}$	**Comments:**	
Depth of Flow at Toe of Glasis	y_1	0.281	m	$q/(\surd(2*g*(E_0-y_1))$	**Comments:**	
Velocity @ Toe	v_1	6.66	ms^{-1}	q/y_1	**Comments:**	
Froude @ Toe	Fr	4.02	-	$v_1/\surd(g*y_1)$	**Comments:**	
Conjugate Depth	y_2	1.46	m	$y_1/2*(\surd(1+8*Fr^2)-1)$	**Comments:**	
Water Level	Z_{cg}	24.15	mAD	$Z_{bs}+y_2$	**Comments:**	
Required Flow in Downstream Channel	q_{req}	#VALUE!	m^3s^{-1}	INTERPOLATE(Table 1.1)	**Comments:**	
Number of Gates Required to Aquire DS Level	N_g	#VALUE!	-	$Q_{req}/(q_{max}*w_g)$	**Comments:**	Minimum number of gates = 1

2.2 50% Max Gate Discharge

Maximum Unit Discharge Per Gate	q_{max}	0.94	m^2s^{-1}		**Comments:**	
Maximum Water Level - Basin Level Difference	E_0	2.54	m	$Z_{uswl,c1}-Z_{bs}$	**Comments:**	
Depth of Flow at Toe of Glasis	y_1	0.14	m	$q/(\surd(2*g*(E_0-y_1))$	**Comments:**	
Velocity @ Toe	v_1	6.87	ms^{-1}	q/y_1	**Comments:**	
Froude @ Toe	Fr	5.95	-	$v_1/\surd(g*y_1)$	**Comments:**	
Conjugate Depth	y_2	1.08	m	$y_1/2*(\surd(1+8*Fr^2)-1)$	**Comments:**	
Water Level	Z_{cg}	23.77	mAD	$Z_{bs}+y_2$	**Comments:**	
Required Flow in Downstream Channel	q_{req}	#NUM!	m^3s^{-1}	INTERPOLATE(Table 1.1)	**Comments:**	
Number of Gates Required to Aquire DS Level	N_g	#NUM!	-	$q_{req}/(q_{max}*w_g)$	**Comments:**	Minimum number of gates = 1

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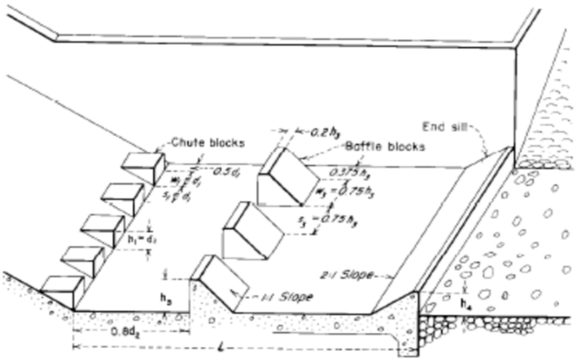
USBR Small Dams Stilling Basin Design

1. USBR Small Dams p.387 - 395

2. USBR Low Froude Stilling Basins

1. INPUTS

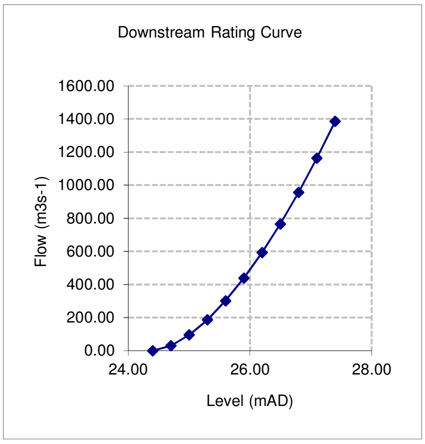


(A) TYPE III BASIN DIMENSIONS

Upstream Water Level	Z_{uswl}	26.94	mAD	**Comments:**	**88.4ft (3.5ft gage)**
Upstream Velocity	v_0	2.02	ms^{-1}	**Comments:**	
Basin Level	Z_b	24.40	mAD	**Comments:**	**Downstream level**
Total Flow	Q	7.4	m^3s^{-1}	**Comments:**	
Stilling Basin Width	B	4.877	m	**Comments:**	
Number of Stilling Basins	N	1	-	**Comments:**	
Number of Chute Blocks	Nc	0	-	**Comments:**	
Number of Baffle Blocks	Nb	0	-	**Comments:**	
Basin Type		Type I		**Comments:**	
End Sill Width Multiple	α	0.150		**Comments:**	
Glasis Slope	s	2	H:1V	**Comments:**	

Downstream Rating Curve (Table 1.1)

#	Level	Flow	Velcocity
n	Z_{dswl}	Q	v
-	mAD	m^3s^{-1}	ms^{-1}
1	24.40	0.00	0.00
2	24.70	30.02	1.49
3	25.00	95.23	2.35
4	25.30	187.03	3.06
5	25.60	301.88	3.69
6	25.90	437.57	4.26
7	26.20	592.59	4.78
8	26.50	765.77	5.27
9	26.80	956.17	5.74
10	27.10	1163.06	6.18
11	27.40	1385.80	6.59



Project **Priest Lake Dam**	MOTT MACDONALD					
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2. OUTPUTS

2.1 GENERAL

Unit Flow	q	1.51	m^2s^{-1}	Q/(B*N)	**Comments:**	
Upstream Velocity Head	h_v	0.21	m	$v_0^2/(2g)$	**Comments:**	
Total Energy	E_0	2.75	m	$Z_{uswl}+h_v-Z_b$	**Comments:**	
Velocity	v_1	7.06	ms^{-1}	$\sqrt{(2g*(E_0-y_1))}$	**Comments:**	
Depth of Flow	y_1	0.214	m	q/v	**Comments:**	
Upstream Froude Number	$\mathbf{Fr_1}$	**4.873**	-	$v_1/\sqrt{(gy_1)}$	**Comments:**	
Conjugate Depth	y_2	1.37	-	$y_1/2*(\sqrt{(1+8Fr_1^2)}-1)$	**Comments:**	
Conjugate Level	Zc	25.77	-	*y2+Zb*	**Comments:**	
Initial Design Flow	Qi	379.24	m^3s^{-1}	*INTERPOLATE*	**Comments:**	
Free Jump Length Factor		5.97	m	*INTERPOLATE*	**Comments:**	
Free Jump Length		8.19	m	L/y_2*y_2	**Comments:**	
MM Basin Length		8.23	m	$6*y_{2,design}$	**Comments:**	
Rough Downstream Water Level		24.47	mEL	*INTERPOLATE*	**Comments:**	

2.2 BASIN DIMENSIONS

2.2.1 Basin

Basin Jump Length Factor	F_L	5.97		*INTERPOLATE*	**Comments:**	
Minimum Basin Length	L_B	8.19	m	F_L*y_2	**Comments:**	
Design Length	**L**	**2.80**	**m**	***User Defined***	**Comments:**	**existing case**

2.3 END SILL

Toe Baffle Distance From Chute Toe Factor	L_1/y_2	N/A	-	*INTERPOLATE*	**Comments:**	
Minimum Toe Baffle Distance From Chute Toe	$L_{1,min}$	N/A	m	L_1/y_2*y_2	**Comments:**	
Toe Baffle Distance From Chute Toe	$\mathbf{L_1}$	**0.00**	**-**	***User Defined***	**Comments:**	**existing case**
Height Ratio	η_1	1.25	-			
Minimum End Sill Height	$H_{s,min}$	0.27	-	η_1*y_1	**Comments:**	
End Sill Height	$\mathbf{H_s}$	**0.00**	**-**	***User Defined***	**Comments:**	**existing case**
Minimum End Sill Thickness	$T_{es,min}$	0.00	m	$0.2*H_s$	**Comments:**	
End Sill Thickness	$\mathbf{T_{es}}$	**0.00**	**-**	***User Defined***	**Comments:**	**existing case**
Minimum End Sill Block Width	$W_{es,min}$	N/A	m	$\alpha*y_2$	**Comments:**	
End Sill Block Width	$\mathbf{W_{es}}$	**0.00**	**-**	***User Defined***	**Comments:**	**existing case**
Minimum End Sill Block Spacing	$S_{es,min}$	N/A	-	$\alpha*y_2$	**Comments:**	
End Sill Block Spacing	$\mathbf{S_{es}}$	**0.00**	**-**	***User Defined***	**Comments:**	**existing case**
Number of Blocks	N_{es}	N/A	-	$(B+S_{es})/(S_{es}+W_{es})$		
Spacing @ End	s_e	0.00	-	$(B+S_{es})/(S_{es}+W_{es})$		

Maynord (1990)

The Rock Manual - CIRIA C683 (2007), 2nd Edition. Page 651.

Recommended for the design of riprap

$$D_{30} = S_f C_s C_v C_T y \left[\left(\frac{1}{s-1} \right)^{0.5} \frac{U_d}{\sqrt{(K_1 g y)}} \right]^{2.5}$$

where:
D30 is the characteristic riprap size of which 30% is finer by weight
Sf is a safety factor
Cs is a stability coefficient
Cv is a velocity distribution coefficient
Ct is a blanket thickness coefficient
s is the relative density of stone
Ud is the depth averaged flow velocity
K1 is a side slope correction factor
alpha is the angle of the bank to the horizontal
g is the acceleration due to gravity

Enter Data

Safety factor	Sf	1.5	(1.5 suggested)
Rock shape	A or R	R	(Angular or Rounded)
Location	S or D	D	(Straight channel or D/S of structure)
Thickness coeff	Ct	1	(1.0 suggested)
Rel. density of stone	s	2.65	
Angle of bank to horiz.	alpha	45	
Accel due to gravity	g	9.807	

Depth averaged velocity	Ud1	1.49	m/s
(use 3 to test sensitivity)	Ud2	2.35	m/s
	Ud3	3.06	m/s
	Ud4	3.69	m/s
	Ud5	4.26	m/s
	Ud6	4.78	m/s
	Ud7	5.27	m/s
	Ud8	5.74	m/s
	Ud9	6.18	m/s
	Ud10	6.59	m/s

Water depths	y1	0.30	m
	y2	0.60	m
	y3	0.90	m
	y4	1.20	m
	y5	1.50	m
	y6	1.80	m
	y7	2.10	m
	y8	2.40	m
	y9	2.70	m
	y10	3.00	m

Results

Stability coefficient	Cs	0.375
Vel. distribution coeff	Cv	1.25
COT alpha		1.000
Side slope correction	K1	0.416

Water depth, y (m)	Velocity, Ud (m/s)	D30 (m)	D50 (m)	Dn50 (min)	Dn50 (max)
0.3	1.485746161	0.24	0.34	0.28	0.31
0.6	2.345969986	0.62	0.89	0.75	0.81
0.9	3.058061436	1.09	1.56	1.31	1.42
1.2	3.685578544	1.62	2.31	1.94	2.10
1.5	4.255158398	2.19	3.13	2.63	2.85
1.8	4.781256906	2.80	4.00	3.36	3.64
2.1	5.272865326	3.44	4.91	4.13	4.47
2.4	5.73607816	4.11	5.87	4.93	5.34
2.7	6.175280015	4.79	6.85	5.75	6.23
3	6.593766666	5.50	7.86	6.60	7.15

Pilarczyk (1990)

The Rock Manual - CIRIA C683 (2007), 2nd Edition. Page 649.

Recommended for the design of riprap, cabled concrete blocks, box gabions and asphalt mattresses

$$\boxed{D_{n50} = \frac{\mu}{(s-1)} \frac{0.035}{\Psi_{CR}} \frac{K_T K_Y}{K_S} \frac{U^2}{2g}}$$

where:

μ is the stability correction factor

s is the relative density of the revetment

$\boxed{\Psi \quad CR}$ is the stability factor

D is the characteristic size of the protection
Kt is a turbulence factor
Kh is a depth factor
Ks is the slope factor
Ud is the depth averaged flow velocity
g is the acceleration due to gravity

Enter Data

Location	C or E	C	Continuous protection or Edges and transitions
Relative density of stone	s	2.65	
Porosity of stone	n	0.4	(suggest 0.4 for stone and sand)
Type of revetment	R or G	R	Riprap or Gabions
Bank slope	alpha	0	
Internal friction angle	sigma	45	
Longitudinal channel slope	beta	0.00176	
Turbulence factor	N or H	H	Normal or High
Accel due to gravity	g	9.807	m/s2
Flow decvelopment			

Depth averaged velocity	Ud1	1.49	m/s
	Ud2	2.35	m/s
	Ud3	3.06	m/s
	Ud4	3.69	m/s
	Ud5	4.26	m/s
	Ud6	4.78	m/s
	Ud7	5.27	m/s
	Ud8	5.74	m/s
	Ud9	6.18	m/s
	Ud10	6.59	m/s

Water depths	y1	0.30	m
	y2	0.60	m
	y3	0.90	m
	y4	1.20	m
	y5	1.50	m
	y6	1.80	m
	y7	2.10	m
	y8	2.40	m
	y9	2.70	m
	y10	3.00	m

Results

Stability correction factor		1		tan alpha	0.000
Stability factor		0.035		cos alpha	1.000
Relative density of revetment		1.65		tan sigma	1.000
				sin sigma - beta	0.707
Side slope term	Kd	1.000		sin sigma	0.707
Longitudinal slope	k1	1.000			
Slope factor	Ks	1.000			
Turbulence factor	Kt	1.500			

Estimate Dn50	Depth averaged velocity Ud (m/s)	Depth y (m)	Depth factor Kh	Dn50 (m)	D50 (min)	D50 (max)	D50 (ave)	D30 (min)	D30 (max)
0.08	1.485746161	0.3	0.764	0.078	0.09	0.09	0.09	0.06	0.07
0.21	2.345969986	0.6	0.808	0.206	0.23	0.25	0.24	0.16	0.17
0.36	3.058061436	0.9	0.833	0.361	0.40	0.43	0.41	0.28	0.30
0.54	3.685578544	1.2	0.851	0.536	0.59	0.64	0.61	0.41	0.45
0.73	4.255158398	1.5	0.865	0.726	0.80	0.86	0.83	0.56	0.60
0.93	4.781256906	1.8	0.876	0.928	1.02	1.11	1.06	0.71	0.77
1.14	5.272865326	2.1	0.885	1.141	1.25	1.36	1.31	0.88	0.95
1.36	5.73607816	2.4	0.893	1.362	1.50	1.62	1.56	1.05	1.13
1.59	6.175280015	2.7	0.899	1.590	1.75	1.89	1.82	1.22	1.32
1.82	6.593766666	3	0.905	1.824	2.00	2.17	2.09	1.40	1.52

Escarameia and May (1992) - HR Wallingford

The Rock Manual - CIRIA C683 (2007), 2nd Edition. Page 650.

Recommended for the design of riprap, loose or interlocking concrete blocks and gabion mattresses.

$$D_{n50} = C\frac{U_b{}^2}{2g(s-1)}$$

where: Dn50 is the characteristic size of stone
Dn50 is the size of the equivalent cube

$$D_{n50} = \left(\frac{W_{50}}{\rho_s}\right)^{\frac{1}{3}}$$

W50 is the weight of particle, Ps is the density of stone
C is a coefficient that takes account of the turbulence intensity TI
g is acceleration due to gravity
s is the relative density of the revetment material
Ub is the velocity near the bend (at 10% of the water depth above the bed)

Turbulence Levels

Situation	Turbulence Level	
	Qualitative	TI
Straight river or channel reaches and wide natural bends (R/W>26)	Normal (low)	0.12
Edge of revetments in straight reaches	Normal (higher)	0.20
Bridge piers, caissons and groynes; transitions	Medium to high	0.35-0.50
Downstream of hydraulic structures (weirs, culverts, stilling basins)	Very high	0.60

where: R is the centreline radius of bend
W is the water surface width at the upstream end of the bend

Values of C

Type of revetment	Value of C	Observations
Riprap	12.3TI-0.20	Valid for TI>=0.05
Concrete blocks	9.22TI-0.15	Valid for TI>=0.05
Gabion mattresses	12.3TI-1.65	Valid for TI>=0.12

Enter Data

Turbulence Intensity	TI	0.6	
Type of revetment	R,C or G	R	(Riprap, Concrete or Gabion)
Relative density of riprap	s	2.65	
Accel due to gravity	g	9.81	m/s2
Depth averaged velocity	Ud1	1.49	m/s
	Ud2	2.35	m/s
	Ud3	3.06	m/s
	Ud4	3.69	m/s
	Ud5	4.26	m/s
	Ud6	4.78	m/s
	Ud7	5.27	m/s
	Ud8	5.74	m/s
	Ud9	6.18	m/s
	Ud10	6.59	m/s

Suggested factor for Ub	47%
Actual	60%

Results

Coefficient C 7.18

Depth averaged velocity	Ub	Ub2	Dn50	D50 (min)	D50 (max)	D50 (ave)	D30 (min)	D30 (max)
1.486	0.891	0.795	0.18	0.19	0.21	0.20	0.14	0.15
2.346	1.408	1.981	0.44	0.48	0.52	0.50	0.34	0.37
3.058	1.835	3.367	0.75	0.82	0.89	0.85	0.57	0.62
3.686	2.211	4.890	1.08	1.19	1.29	1.24	0.83	0.90
4.255	2.553	6.518	1.45	1.59	1.72	1.65	1.11	1.20
4.781	2.869	8.230	1.83	2.01	2.17	2.09	1.40	1.52
5.273	3.164	10.009	2.22	2.44	2.64	2.54	1.71	1.85
5.736	3.442	11.845	2.63	2.89	3.13	3.01	2.02	2.19
6.175	3.705	13.728	3.04	3.35	3.62	3.49	2.34	2.54
6.594	3.956	15.652	3.47	3.81	4.13	3.97	2.67	2.89

Project

Priest Lake Dam

MOTT MACDONALD

Calculations for

999 Summary Sheet.

Division	WCD	File No.		
Calc by	JM	Date	31/07/2017	Number of sheets
Checked by	PJH	Date	31/07/2017	of

Operational Scenario

Number of Operational Gates	1.00		
Flow	7.36	m^3s^{-1}	**260 cfs**
Gage Level	1.07	m	**3.5 ft**
River Level	26.94	mAD	**88.4 ft**
Gate Opening	0.23	m	**9 inches/0.75 ft**
Stilling Basin	As existing		

Downstream Rating Curve

Depth of Flow	Water Level	Discharge	Average Velocity	Froude
D	Zwl	Q	v	Fr
m	mAD	m^3s^{-1}	ms-1	-
0.00	24.40	0.00	0.00	0.00
0.30	24.70	30.02	1.49	0.87
0.60	25.00	95.23	2.35	0.97
0.90	25.30	187.03	3.06	1.03
1.20	25.60	301.88	3.69	1.07
1.50	25.90	437.57	4.26	1.11
1.80	26.20	592.59	4.78	1.14
2.10	26.50	765.77	5.27	1.16
2.40	26.80	956.17	5.74	1.18
2.70	27.10	1163.06	6.18	1.20
3.00	27.40	1385.80	6.59	1.22

Radial Gate with Raised Cill

Max Unit Discharge	qmax	-0.24	m^2s^{-1}
Design Unit Discharge	q	1.87	m^2s^{-1}
Flow	Qmax	1.85	m^3s^{-1}

Stilling Basin Level Design

Condition 1 - 100% Design Flow with 100% Design Flow Downstream

Condition 2 - 10% Design Flow with 0% Design Flow Downstream

Condition 3 - 30% Design Flow with 30% Design Flow Downstream

Summary of Critical Basin

Basin Level	22.69	mAD
Critical Condition	Condition 1	
Froude	17.65	
Critical Condition	Condition 2	

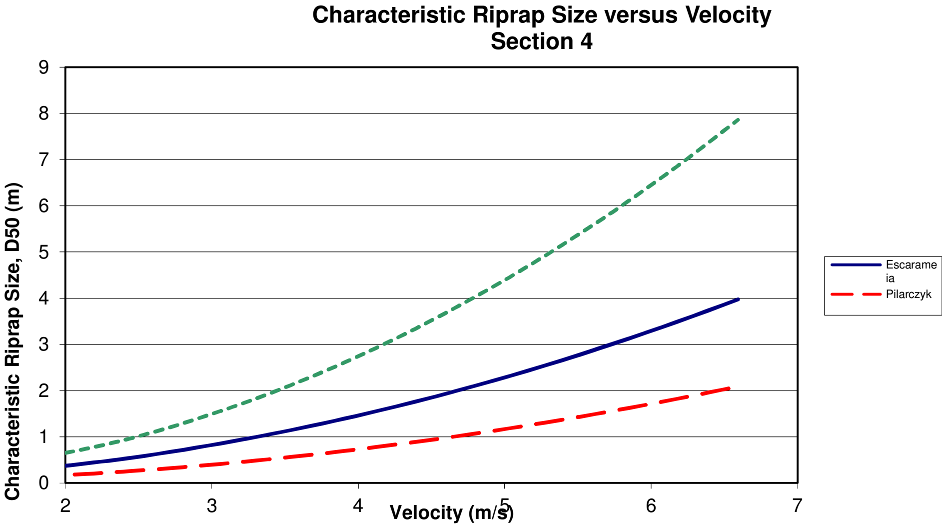
Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 999 Summary Sheet.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

Conjugate Depth	1.58	mAD
Critical Condition	Condition 1	
Max Basin Length	4.09	mAD
Critical Condition	Condition 1	

USBR Stilling Basins

Minimum Basin Length	5.97	m
Minimum End Sill Height	1.25	m
Minimum End Sill Thickness	0.00	m

Erosion Stone Sizing



Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for	Division	WCD	File No.		
000 Cover Sheet.	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

Operational Scenario

Number of Operational Gates	11		
Flow	214.39	m^3s^{-1}	**7,570cfs**
Gage Level	0.70104	m	**2.3 ft**
River Level	26.58	mAD	**87.2 ft**
Gate Opening	1.524	m	**60 inches/5 ft**
Stilling Basin	As existing		

Downstream Rating Curve

A downstream rating curve was produced to enable us identify the tail water level versus discharge assuming a trapezoidal channel.
A bed width of 220 ft was taken using a side slope of 1:1, bed slope of 0.01 and Manning's coefficient of 0.03 as per the original design calculation. The bed level was taken as 80.05 ft, the level of the concrete apron.
Using basic geometry for various depths of flows, the flow area and hydraulic radius were calculated. These could then be used to calculate the discharge.

Radial Gate with Raised Cill

The gate width (16 ft), cill level (81.05 ft) and gate height (7 ft) were taken as per original construction drawings. The safety factor was assumed to be 1.
Assuming unsubmerged conditions and a maximum opening of the 7ft high radial gate, the maximum unit discharge can then be calculated.

Stilling Basin Level Design

Three worst-case scenarios were considered for the stilling basin design. These are:
- 100% flow both upstream and downstream of the structure, to simulate design flow conditions
- 10% upstream flow with 0% flow downstream to simulate sudden gate opening
- 30% upstream flow with 30% downstream flow

A stilling basin safety factor of 1.1 was used for this calculation. Downstream conditions for the various scenarios can be determined from the downstream rating curve. Approach velocities were estimated using the inflow and height of the gates.
Head differences across the gate could be calculated by using the known qate levels and velocities upstream and downstream of the structure. This can be used to determine various parameters about the flow at the glasis and conjugate depth of the given scenario. Based on the Froude number, the performance of the stilling basin can be determined.
For the scenarios the minimum basin level, maximum Froude number, maximum conjugate depth and maximum free jump length are selected as the parameters required for the critical basin.

USBR Stilling Basins

This was undertaken using the USBR Design of Small Dams for a type I stilling basin with no chute or baffle blocks. Along with the parameters determined from the level design, the required basin and end sill dimensions were determined.

Erosion Stone Sizing

Erosion stone sizing was undertaken using The Rock Manual with the Escarameia, Pilarczyk and Maynord methods. The relative density of riprap was taken as 2.65 and the angle of the bank to the horizontal was taken as 45 degrees. All other required inputs were taken from geometry of the structure.

Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 001 EVT-HYES-HYD-300 Trapizoidal Rating Curve.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

Manning's Rating Curve

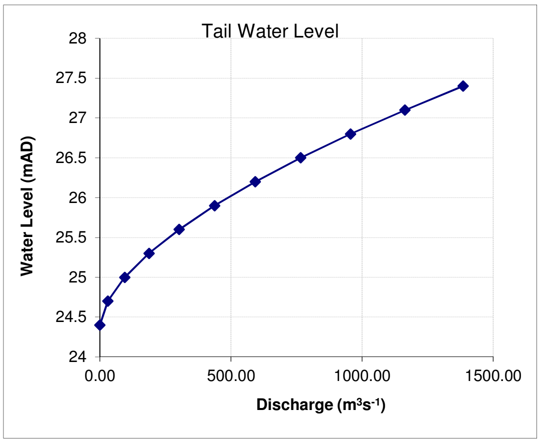
Open Channel Hydraulics, 1959, Ven Te Chow

1. INPUTS

Bed Width	B	67.056	m	**Comments:**	**220 ft as per original calculation**
Side Slope	s	1	H:1V	**Comments:**	**As per original calc**
Bed Slope	i	0.01	m/m	**Comments:**	**As per original calc**
Manning's Coefficient	n	0.03	$sm^{-1/3}$	**Comments:**	**As per original calc**
Bed Level	Zb	24.39924	mAD	**Comments:**	**80.05 ft**
Max Depth	Dmax	3	m	**Comments:**	

2. OUTPUTS

#	Depth of Flow	Flow Area	Hydraulic Radius	Discharge	Water Level	Average Velocity	Froude
n	D	A	P	Q	Zwl	v	Fr
-	m	m^2	m	m^3s^{-1}	mAD	ms-1	-
-	-	$D_n*(B+D_n*s)$	$B+2*D_n*\sqrt{(1+s^2)}$	$A(A/P)^{2/3}*\sqrt{(i)} / n$	D_n+Z_b	Q/A	$v/\sqrt{(g*D_n)}$
1	0	0	67.06	0.00	24.39924	0	0
2	0.3	20.2068	67.90	30.02	24.69924	1.49	0.87
3	0.6	40.5936	68.75	95.23	24.99924	2.35	0.97
4	0.9	61.1604	69.60	187.03	25.29924	3.06	1.03
5	1.2	81.9072	70.45	301.88	25.59924	3.69	1.07
6	1.5	102.834	71.30	437.57	25.89924	4.26	1.11
7	1.8	123.9408	72.15	592.59	26.19924	4.78	1.14
8	2.1	145.2276	73.00	765.77	26.49924	5.27	1.16
9	2.4	166.6944	73.84	956.17	26.79924	5.74	1.18
10	2.7	188.3412	74.69	1163.06	27.09924	6.18	1.20
11	3	210.168	75.54	1385.80	27.39924	6.59	1.22



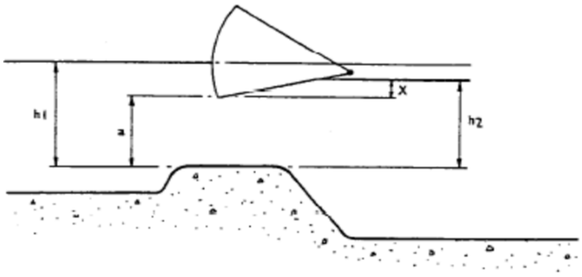
Project **Priest Lake Dam**					
Calculations for 002 EVT-HYES-HYD-561 Radial Gate with Raised Cill.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

Radial Gate With Cill

Hydraulic Design of Canal Structures - MMP August 1985

USBR - Design of Small Small Dams

1. INPUTS



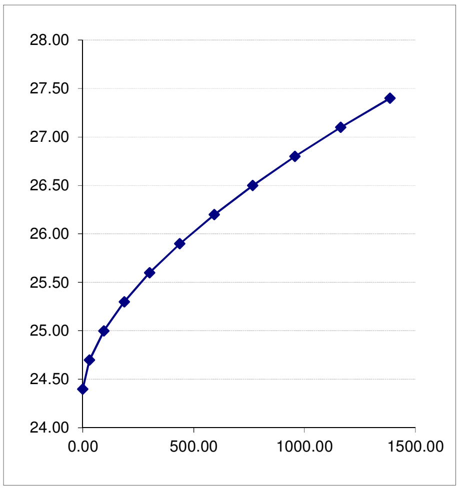
1. Hydraulic Design of Canal Structures, Sept 1986, MM & Partners p. 1 - p. 11

2. Design of Small Dams - USBR

1.1 GENERAL

Design Discharge	Q	214.39	m^3s^{-1}	**Comments:**	**7,570cfs**
Number of Gates	N_g	11	-	**Comments:**	
Gate Width	w	4.88	m	**Comments:**	**16 ft**
Cill Level	Z_c	24.70	mAD	**Comments:**	**81.05ft**
Max Upstream Water Level	Z_{uswl}	26.58	mAD	**Comments:**	**87.2ft(2.3ft gage)**
Structure Flow Saftey Factor	S_f	1	-	**Comments:**	
Maximum gate opening (as percentage of max head)	μ	61%	-	**Comments:**	**Maximum opening for design dischage**
Design Gate Height	a	1.524	-	**Comments:**	**60 inches/5 ft**
Design Downstream Water Level	Z_{dswl}	25.37	mAD	**Comments:**	

	Level	Flow	Velocity
1	mAD	m3/s	m/s
2	24.40	0.00	0.00
3	24.70	30.02	1.49
4	25.00	95.23	2.35
5	25.30	187.03	3.06
6	25.60	301.88	3.69
7	25.90	437.57	4.26
8	26.20	592.59	4.78
9	26.50	765.77	5.27
10	26.80	956.17	5.74
11	27.10	1163.06	6.18
12	27.40	1385.80	6.59
13			
14			



Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for	Division	WCD	File No.		
002 EVT-HYES-HYD-561 Radial Gate with Raised Cill.	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

1.2 Effective Crest Coefficients

Pier Contraction Coefficient	k_p	0.020	-	**Comments:**	**USBR Design of Small Dams, square nosed piers**
Abutment Contraction Coefficient	k_a	0.2	-	**Comments:**	**USBR Design of Small Dams, square abutments**

2. OUTPUTS

2.1 GENERAL

Number of Piers	N	10		N_g-1	**Comments:**
Sum of Gates Width	b'	53.65	m	N_g*w	**Comments:**
Design Factored Discharge	Q_f	214.39	m^3s^{-1}	S_f*Q	**Comments:**
Upstream Head Over Cill	h_1	1.87	m	$Z_{uswl} - Z_c$	**Comments:**
Downstream Head Over Cill	h_2	0.663	m	$Z_{dswl} - Z_c$	**Comments:**
Effective Crest Length	b	52.15	m	$b'-2*(N*K_p+k_a)*h_1$	**Comments:**

2.2 FREE FLOW CALCULATIONS

Max Height of Gate Opening	$a_{max\,1}$	1.14	m	$\mu*h_1$	**Comments:**
Opening - DS Water Level Difference	x	0.38	m	$h_2 - a$	**Comments:**
Max Unit Discharge	q_{max}	4.161	m^2s^{-1}	$0.6*a_{max\,1}*\sqrt{(2*g*h_1)}$	**Comments:**
Design Unit Discharge	q	4.111	m^2s^{-1}	Q_f/b	**Comments:**
Flow	Q_{max}	223.202	m^3s^{-1}	$q*w*N$	**Comments:**
CHECK		OK			**Comments:**
Gate Radius	R	1.525	m	4/3*amax 1	**Comments:**
Depth Above Gate	a'	0.73	m	h1-amax 1	**Comments:**
Angle	θ	61.35	Degrees	$\cos^{-1}(a/R)$	**Comments:**
Vena Contrata Factor	δ	0.65	-		**Comments:**
Vena Contrata	α	0.74	m	$a*\delta$	**Comments:**
Velocity	v	5.60	ms^{-1}	q/α	**Comments:**
Froude Number	Fr	2.07	-		**Comments:**
Conjugate Depth	y_2	2.18	m	$y_1/2*\sqrt{((1+8Fr_1^2)-1)}$	**Comments:**
Water Level	Zc	26.88	mAD	y2+Zc	**Comments:**
Downstream Water Level	Zds	25.39	mAD		**Comments:**

2.3 SUBMERGED CONDITIONS

Max Height of Gate Opening	$a_{max\,2}$	1.25	m	$2/3*h_1$	**Comments:**
Opening - DS Water Level Difference	x	-0.59	m	$h_2 - a$	**Comments:**
Head Drop	H	1.21	m	h_1-h_2	
Max Unit Discharge	q_{max}	4.083	m^2s^{-1}	$0.67*a_{max\,2}*\sqrt{(2*g*(h_1-h_2))}$	
Design Unit Discharge	q	4.111	m^2s^{-1}	Q_f/b'	**Comments:**

Project **Priest Lake Dam**	MOTT MACDONALD				
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CHECK		NOT OK		$q < q_{max}$	**Comments:**
Gate Radius	R	1.666	m	$4/3*a$	**Comments:**
Flow Through One Gate @ 100% a	$Q_{n=1}$	19.354	m^3s^{-1}	$q_{max}*w$	**Comments:**
Design Flow	Q	212.889	m^3s^{-1}		

2.4 DESIGN CONDITIONS

Condition		UNSUBMERGED			**Comments:**
Design Flow	Q_d	223.202	m^3s^{-1}	$q*w*N$	**Comments:**

Project

Priest Lake Dam

Calculations for

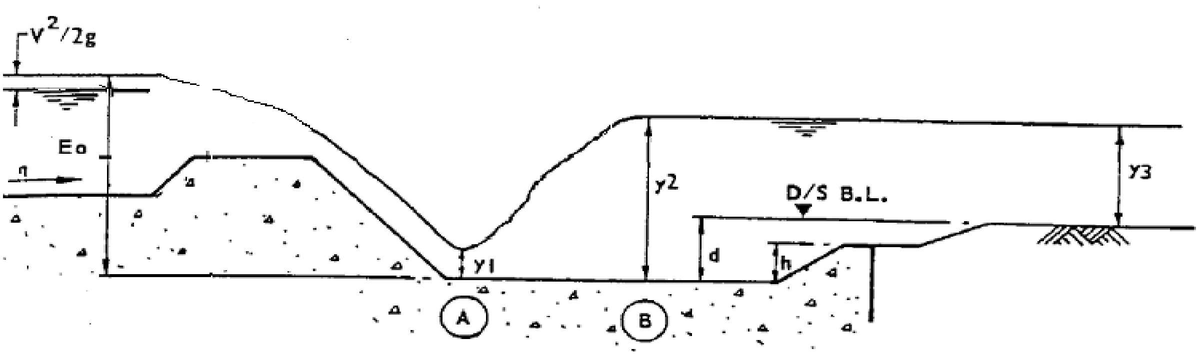
003 EVT-HYES-HYD-001 - Stilling Basin Level Design.xls.

Division	WCD	File No.		
Calc by	JM	Date	31/07/2017	Number of sheets
Checked by	PJH	Date	31/07/2017	of

Stilling Basin Level Design

1. MMP Hydraulic Design of Canal Structures, 1985, 1-13 - 1-14

1. INPUTS

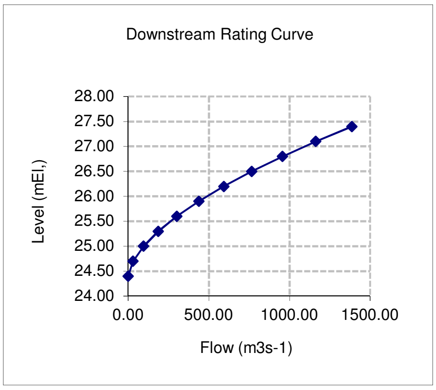


1.1 GENERAL

Design Flow	Q_d	223.2	m^3s^{-1}	**Comments:**	
Flow Safety Factor	S_f	1.00	-	**Comments:**	
Width of Stilling Basin	B	53.64	m	**Comments:**	**16ft*11 no. = 176 ft**
Number of Basins	N	1	-	**Comments:**	
Basin Safety Factor	S_f	1.1	-	**Comments:**	
Bed Level US	Z_{usbl}	24.40	mAD	**Comments:**	**80.05 ft**
Bed Level DS	Z_{dsbl}	24.40	mAD	**Comments:**	**80.05 ft**
Allowance For Retrogression (Percentage of D/S Depth)	ρ	0%	%	**Comments:**	
Gate Width	w_g	53.64	m	**Comments:**	**16 ft * 11 no.**

Downstream Rating Curve (Table 1.1)

#	Level	Flow	Velocity
n	Z_{dswl}	Q	v
-	mAD	m^3s^{-1}	ms^{-1}
1	24.40	0.00	0.00
2	24.70	30.02	1.49
3	25.00	95.23	2.35
4	25.30	187.03	3.06
5	25.60	301.88	3.69
6	25.90	437.57	4.26
7	26.20	592.59	4.78
8	26.50	765.77	5.27
9	26.80	956.17	5.74
10	27.10	1163.06	6.18
11	27.40	1385.80	6.59



Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 003 EVT-HYES-HYD-001 - Stilling Basin Level Design.xls.	Division	WCD	File No.		
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1.2 CONDITION 1

1.2.1 General

Percentage of Flow US	$m_{us,1}$	100%		**Comments:**	
Percentage of Flow DS	$m_{ds,1}$	100%		**Comments:**	
Additional flow downstream	$Q_{add,1}$	0.0	m^3s^{-1}	**Comments:**	

1.3.2 Upstream Conditions

Water Level	$Z_{us,1}$	26.6	mAD	**Comments:**	**87.2ft(2.3ft gage)**
Velocity	$v_{us,1}$	0.181	ms^{-1}	**Comments:**	
Maximum Unit Discharge	$q_{max,1}$	4.161	m^2s^{-1}	**Comments:**	

1.3 CONDITION 2

1.3.1 General

Percentage of Flow US	$m_{us,2}$	10%		**Comments:**
Percentage of Flow DS	$m_{ds,2}$	0%		**Comments:**
Additional flow downstream	$Q_{add,2}$	0.0	m^3s^{-1}	**Comments:**

1.3.2 Upstream Conditions

Water Level	$Z_{us,2}$	26.6	mAD	**Comments:**
Velocity	$v_{us,1}$	0.27	ms^{-1}	**Comments:**
Maximum Unit Discharge	$q_{max,1}$	0.416	m^2s^{-1}	**Comments:**

1.4 CONDITION 3

1.4.1 General

Percentage of Flow US	$m_{us,3}$	30%		**Comments:**
Percentage of Flow DS	$m_{ds,3}$	30%		**Comments:**
Additional flow downstream	$Q_{add,3}$	0.0	m^3s^{-1}	**Comments:**

1.4.2 Upstream Conditions

Water Level	$Z_{us,3}$	26.6	mAD	**Comments:**
Velocity	$v_{us,3}$	0.82	ms^{-1}	**Comments:**
Maximum Unit Discharge	$q_{max,3}$	1.248	m^2s^{-1}	**Comments:**

Project **Priest Lake Dam**	MOTT MACDONALD				
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2. OUTPUTS

Condition 1 - 100% Design Flow with 100% Design Flow Downstream

Condition 2 - 10% Design Flow with 0% Design Flow Downstream

Condition 3 - 30% Design Flow with 30% Design Flow Downstream

2.1 GENERAL

Factored Design Flow	Q_f	223.2	m^3s^{-1}	Q_d*S_f	**Comments:**
2.1.1 Condition 1					**Comments:**
Factored Discharge US	$Q_{us,1}$	223.2	m^3s^{-1}	$Q_f*m_{us,1}$	**Comments:**
Discharge DS	$Q_{ds,1}$	223.2	m^3s^{-1}	$Q_f*m_{ds,1}$	**Comments:**
DS Water Level	$Z_{ds,1}$	25.4	mAD	*Interpolate (Table 1.1)*	**Comments:**
Velocity @ Q DS	$v_{ds,1}$	3.26	ms^{-1}	*Interpolate (Table 1.1)*	**Comments:**
2.1.2 Condition 2					**Comments:**
Factored Discharge US	$Q_{us,2}$	22.3	m^3s^{-1}	$Q_f*m_{us,2}$	**Comments:**
Discharge DS	$Q_{ds,2}$	0.0	m^3s^{-1}	$Q_f*m_{ds,2}$	**Comments:**
DS Water Level	$Z_{ds,2}$	24.399	mAD	*Interpolate (Table 1.1)*	**Comments:**
Velocity @ Q DS	$v_{ds,2}$	0.00	ms^{-1}	*Interpolate (Table 1.1)*	**Comments:**
2.1.3 Condition 3					**Comments:**
Factored Discharge US	$Q_{us,3}$	67.0	m^3s^{-1}	$Q_f*m_{us,3}$	**Comments:**
Discharge DS	$Q_{us,3}$	67.0	m^3s^{-1}	$Q_f*m_{ds,3}$	**Comments:**
DS Water Level	$Z_{ds,3}$	24.869	mAD	*Interpolate (Table 1.1)*	**Comments:**
Velocity @ Q DS	$v_{ds,3}$	1.97	ms^{-1}	*Interpolate (Table 1.1)*	**Comments:**

2.2 DOWNSTREAM CONDITION

Water Depth DS Condition 1	$D_{ds,1}$	0.994	m	$Z_{dswl}-Z_{bl}$	**Comments:**
Water Depth DS Condition 2	$D_{ds,2}$	0.000	m	$Z_{dswl}-Z_{bl}$	**Comments:**
Water Depth DS Condition 3	$D_{ds,3}$	0.470	m	$Z_{dswl,70}-Z_{bl}$	**Comments:**
Retrogression @ Condition 1	r_1	0.000	m	$\rho*D_{ds,1}$	**Comments:**
Retrogression @ Condition 2	r_2	0.000	m	$\rho*D_{ds,2}$	**Comments:**
Retrogression @ Condition 3	r_3	0.000	m	$\rho*D_{ds,3}$	**Comments:**
Water Level Condition 1 with Retrogression	$Z_{dsl,r}$	25.394	mAD	$Z_{ds,1}-r_1$	**Comments:**
Water Level Condition 2 with Retrogression	$Z_{dsl,r}$	24.399	mAD	$Z_{ds,2}-r_2$	**Comments:**
Water Level Condition 3 with Retrogression	$Z_{dsl,70,r}$	24.869	mAD	$Z_{ds,3}-r_3$	**Comments:**

Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 003 EVT-HYES-HYD-001 - Stilling Basin Level Design.xls.	Division	WCD	File No.		
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Stilling Basin General Design - Condition 1

1. INPUTS

Design Flow	$Q_{us,1}$	223.20	m^3s^{-1}
Width of Stilling Basin	B	53.64	m
Upstream Water Level	$Z_{us,1}$	26.58	mAD
Approach velocity	$v_{us,1}$	0.181	ms^{-1}
Downstream Water Level	$Z_{ds,1}$	25.4	mAD
Downstream Velocity	$v_{ds,1}$	3.26	ms^{-1}
Basin Safety Factor	S_f	1.1	-

2.2 OUTPUTS - CONDITION 1

Flow Intensity	q_1	4.16	m^2s^{-1}	$Q_{us,1}/B/N$	**Comments:**
Critical Depth	$d_{c,1}$	1.21	m	$(q_1^2/g)^{1/3}$	**Comments:**
Upstream Velocity Head	$h_{v,1}$	0.00	m	$v_{us,1}^2/2g$	**Comments:**
Downstream Velocity Head	$h_{v2,1}$	0.54	m	$v_{ds,1}^2/2g$	**Comments:**
Upstream Total Head	$E_{o,1}$	26.58	m	$Z_{us,1} + h_{v,1}$	**Comments:**
Downstream Total Head	$E_{2,1}$	25.93	m	$Z_{ds,1} + h_{2v,1}$	**Comments:**
Head Difference Across Gate	$H_{L,1}$	0.65	m	$E_{0,1} - E_{2,1}$	**Comments:**
	H_L/d_c	0.53	m	H_L/d_c	**Comments:**
	a_1	0.29	-	*GOAL SEEK*	**Comments:**
	$v_{1,1}$	0.54	-	$(1-a_1)^3/(2a_1)*((1+2a_1)^2-1)^{-1/3})$	
Congugate Depth	$y_{2,1}$	2.11	m	$4*H_{L,1}*a_1/(1-a_1)^3$	
Velocity @ Congugate Depth	$v_{2,1}$	1.97	ms^{-1}	$q_1/y_{2,1}$	
Depth of Flow at Bottom of Glasis	$y_{1,1}$	0.61	m	$a_1*y_{2,1}$	
Velocity @ Toe of Glasis	$v_{1,1}$	6.78	ms^{-1}	$q_1/y_{1,1}$	**Comments:**
Froude Number	Fr_1	2.76	-	$v_{1,1}/\sqrt{(gy_{1,1})}$	**Comments:**
Conjugate Depth Check Calc	$y_{2,1}$	2.11	m	$y_{1,1}/2*\sqrt{((1+8Fr_1^2)}-1)$	**Comments:**
Total Energy at Congugate Depth	$E_{1,1}$	2.31	m	$y_{2,1}+d_{c,1}^3/(2*y_{2,1}^2)$	
Basin Level	$\mathbf{Z_{b,1}}$	**23.40**	**mAD**	$E_{2,1}-S_f*E_{1,1}$	**Comments:**
Downstream Jump Level	Z_{dsjl}	25.50	m	$Z_{dsbs} + y_2$	**Comments:**
	L/y_2	5.07	m	*INTERPOLATE*	**Comments:**
	L/y_2	5.07	m	*INTERPOLATE*	**Comments:**
Jump Length	L	10.69	m	L/y_2*y_2	**Comments:**
Free Jump Performance		Wavy			
Basin Type		Type IVA			**Comments:**
Basin Jump Length Factor	L_B/y_2	3.18	m		
Basin Jump Length	L_B	6.72	m		

Project **Priest Lake Dam**					
Calculations for 003 EVT-HYES-HYD-001 - Stilling Basin Level Design.xls.	Division	WCD	File No.		
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2.2.1 Design Check (Set Outputs First)

Total Energy	$E_{0,1d}$	2.18	m	$Z_{us,1}+h_{v,1}-Z_b$	**Comments:**
Velocity	$v_{1,1d}$	5.21	ms^{-1}	$\sqrt{}(2g*(E_{0,1d}-y_{1,1d}))$	**Comments:**
Depth of Flow	$y_{1,1d}$	0.799	m	$q_1/v_{1,1d}$	**Comments:**
Upstream Froude Number	$\mathbf{Fr_{1,1d}}$	**1.859**	-	$v_{1,1d}/\sqrt{}(gy_{1,1d})$	**Comments:**
Conjugate Depth	$y_{2,1d}$	1.74	-	$y_{1,1d}/2*(\sqrt{}(1+8Fr_{1d}^2)-1)$	
Conjugate Level	$y_{2,1d}$	26.14	-	$y_{1,1d}/2*(\sqrt{}(1+8Fr_{1d}^2)-1)$	

Project **Priest Lake Dam**	MOTT MACDONALD				
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Stilling Basin General Design - Condition 2

1. INPUTS

Design Flow	$Q_{us,2}$	22.32	m^3s^{-1}
Width of Stilling Basin	B	53.64	m
Upstream Water Level	$Z_{us,2}$	26.58	mAD
Approach velcoity	$v_{us,2}$	0.273	ms^{-1}
Downstream Water Level	$Z_{ds,2}$	24.4	mAD
Downstream Velocity	$v_{ds,2}$	0.00	ms^{-1}
Basin Safety Factor	S_f	1.1	-

2.3 OUTPUTS - CONDITION 2

Flow Intensity	q_2	0.42	m^2s^{-1}	$Q_{us,2}/B/N$	**Comments:**
Critical Depth	$d_{c,2}$	0.26	m	$(q_2^2/g)^{1/3}$	**Comments:**
Upstream Velocity Head	$h_{v,2}$	0.00	m	$v_{us,2}^2/2g$	**Comments:**
Downstream Velocity Head	$h_{v2,2}$	0.00	m	$v_{ds,2}^2/2g$	**Comments:**
Upstream Total Head	$E_{o,2}$	26.58	m	$Z_{us,2} + h_{v,2}$	**Comments:**
Downstream Total Head	$E_{2,2}$	24.40	m	$Z_{ds,2} + h_{2v,2}$	**Comments:**
Head Difference Across Gate	$H_{L,2}$	2.18	m	$E_{0,2}-E_{2,2}$	**Comments:**
	H_L/d_c	8.39	m	H_L/d_c	**Comments:**
	a_2	0.07	-	*GOAL SEEK*	**Comments:**
	$v_{1,2}$	8.39	-	$(1-a_2)^3/(2a_2)*((1+2a_2)^2-1)^{-1/3})$	
Congugate Depth	$y_{2,2}$	0.77	m	$4*H_{L,2}*a_2/(1-a_2)^3$	
Velocity @ Congugate Depth	$v_{2,2}$	0.54	ms^{-1}	$q_2/y_{2,2}$	
Depth of Flow at Bottom of Glacis	$y_{1,2}$	0.05	m	$a_2*y_{2,2}$	
Velocity @ Toe of Glacis	$v_{1,2}$	7.57	ms^{-1}	$q_2/y_{1,2}$	**Comments:**
Froude Number	Fr_2	10.30	-	$v_{1,2}/\sqrt{}(gy_{1,2})$	**Comments:**
Conjugate Depth Check Calc	$y_{2,2}$	0.77	m	$y_{1,2}/2*\sqrt{}((1+8Fr_2^2$	**Comments:**
Total Energy at Congugate Depth	$E_{1a,2}$	0.79	m	$y_{2,2}+d_{c,2}^3/(2*y_{2,2}$	**Comments:**
Basin Level	$\mathbf{Z_{dsbs}}$	**23.53**	**mAD**	$E_{2,2}-S_f*E_{1,2}$	**Comments:**
Downstream Jump Level	Z_{dsjl}	24.31	m	$Z_{dsbs} + y_2$	**Comments:**
	L/y_2	6.07	m	*INTERPOLATE*	**Comments:**
	L/y_2	6.07	m	*INTERPOLATE*	**Comments:**
Jump Length	L	4.70	m	L/y_2*y_2	**Comments:**
Free Jump Performance		Acceptable Performance			
Basin Type		Type III			**Comments:**
Basin Jump Length Factor	L_B/y_2	2.73	m		
Basin Jump Length	L_B	2.11	m		

Project **Priest Lake Dam**	MOTT MACDONALD				
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2.3.1 Design Check (Set Outputs First)

Total Energy	E_0	2.18	m	$Z_{uswl}+h_v-Z_b$	**Comments:**
Velocity	v_1	6.45	ms^{-1}	$\sqrt{(2g*(E_0-y_1))}$	**Comments:**
Depth of Flow	y_1	0.065	m	q/v	**Comments:**
Upstream Froude Number	**Fr_1**	**8.102**	-	$v_1/\sqrt{(gy_1)}$	**Comments:**
Conjugate Depth	y_2	0.71	-	$y_1/2*(\sqrt{(1+8Fr_1^2)}-1)$	**Comments:**
Conjugate Level	y_2	25.11	-	$y_1/2*(\sqrt{(1+8Fr_1^2)}-1)$	**Comments:**

Project **Priest Lake Dam**	MOTT MACDONALD				
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Stilling Basin General Design - Condition 3

MM Hydraulic Structures, 1985

1. INPUTS

Design Flow	$Q_{us,3}$	66.96	m^3s^{-1}
Width of Stilling Basin	B	53.64	m
Upstream Water Level	$Z_{us,3}$	26.58	mAD
Approach velcoity	$v_{us,3}$	0.819	ms^{-1}
Downstream Water Level	$Z_{ds,3}$	24.9	mAD
Downstream Velocity	$v_{ds,3}$	1.97	ms^{-1}
Basin Safety Factor	S_f	1.1	-

2.4 OUTPUTS - CONDITION 3

Flow Intensity	q_3	1.25	m^2s^{-1}	$Q_{us,3}/B/N$	**Comments:**
Critical Depth	$d_{c,3}$	0.54	m	$(q_3{}^2/g)^{1/3}$	**Comments:**
Upstream Velocity Head	$h_{v,3}$	0.03	m	$v_{us,3}{}^2/2g$	**Comments:**
Downstream Velocity Head	$h_{v2,3}$	0.20	m	$v_{ds,3}{}^2/2g$	**Comments:**
Upstream Total Head	$E_{o,3}$	26.61	mAD	$Z_{us,3} + h_{v,3}$	**Comments:**
Downstream Total Head	$E_{2,3}$	25.07	mAD	$Z_{ds,3} + h_{2v,3}$	**Comments:**
Head Difference Across Gate	$H_{L,3}$	1.55	m	$E_{0,3}-E_{2,3}$	**Comments:**
	H_L/d_c	2.85	m	H_L/d_c	**Comments:**
	a_3	0.13	-	*GOAL SEEK*	**Comments:**
	$v_{1,3}$	2.85	-	$(1-a_3)^3/(2a_3)*((1+2a_3)^2-1)^{-1/3})$	
Congugate Depth	$y_{2,3}$	1.28	m	$4*H_{L,3}*a_3/(1-a_3)^3$	
Velcoity @ Congugate Depth	$v_{2,3}$	0.98	ms^{-1}	$q_3/y_{2,3}$	
Depth of Flow at Bottom of Glasis	$y_{1,3}$	0.17	m	$a_3*y_{2,3}$	
Velocity @ Toe of Glasis	$v_{1,3}$	7.28	ms^{-1}	$q_3/y_{1,3}$	**Comments:**
Froude Number	Fr_3	5.62	-	$v_{1,3}/\sqrt{}(gy_{1,3})$	**Comments:**
Conjugate Depth Check Calc	$y_{2,3}$	1.28	m	$y_{1,3}/2*\sqrt{}((1+8Fr_3{}^2)-1)$	**Comments:**
Total Energy at Congugate Depth	$E_{1a,3}$	1.33	m	$y_{2,3}+d_{c,3}{}^3/(2*y_{2,3}{}^2)$	
Basin Level	$\mathbf{Z_{dsbs}}$	**23.61**	**mAD**	$E_{2,3}-S_f*E_{1,3}$	**Comments:**
Downstream Jump Level	Z_{dsjl}	24.89	m	$Z_{dsbs} + y_2$	**Comments:**
	L/y_2	6.05	m	*INTERPOLATE*	**Comments:**
	L/y_2	6.05	m	*INTERPOLATE*	**Comments:**
Jump Length	L	7.72	m	L/y_2*y_2	**Comments:**
Free Jump Performance		Best Performance			
Basin Type		Type III			**Comments:**
Basin Jump Length Factor	L_B/y_2	2.42	m		
Basin Jump Length	L_B	3.10	m		

Project **Priest Lake Dam**	**M M** **MOTT MACDONALD**				
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2.4.1 Design Check (Set Outputs First)

Total Energy	E_0	2.21	m	$Z_{uswl}+h_v-Z_b$	**Comments:**
Velocity	v_1	6.29	ms^{-1}	$\sqrt{}(2g*(E_0-y_1))$	**Comments:**
Depth of Flow	y_1	0.199	m	q/v	**Comments:**
Upstream Froude Number	$\mathbf{Fr_1}$	**4.505**	-	$v_1/\sqrt{}(gy_1)$	**Comments:**
Conjugate Depth	y_2	1.17	-	$y_1/2*(\sqrt{}(1+8Fr_1^2)-1)$	**Comments:**
Conjugate Level	y_2	25.57	-	$y_1/2*(\sqrt{}(1+8Fr_1^2)-1)$	**Comments:**

Project **Priest Lake Dam**	MOTT MACDONALD				
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Stilling Basin Check - Outputs

MM Hydraulic Structures, 1985

2. OUTPUTS

Condition 1 - 100% Design Flow with 100% Design Flow Downstream

Condition 2 - 10% Design Flow with 0% Design Flow Downstream

Condition 3 - 30% Design Flow with 30% Design Flow Downstream

2.1 SUMMARY OF CRITICAL BASIN

2.2.1 Basin Level					
Basin Level	Z_{bs}	23.396	mAD	*MIN(C1,C2,C3)*	**Comments:**
Critical Condition		Condition 1	-		**Comments:**
Basin Level Selected	Z_b	**24.400**	mAD	***USER DEFINED***	**Comments:**
2.2.2 Froude Number					
Froude		10.301	-	*MAX(C1,C2,C3)*	**Comments:**
Critical Froude Condition		Condition 2	-		**Comments:**
2.2.3 Conjugate Depth					
Conjugate Depth		2.109	m	*MAX(C1,C2,C3)*	**Comments:**
Max Conjugate Depth Condition		Condition 1	-		**Comments:**
2.2.4 Designed Basin Length					
Max Basin Length		6.717	-	*MAX(C1,C2,C3)*	**Comments:**
Max Basin Length Condition		Condition 1	-		**Comments:**
2.2.5 Free Jump Length					
Max Free Jump Length		10.694	m	*MAX(C1,C2,C3)*	**Comments:**
2.2.6 General					
Minimum Basin Length		12.656	m	6*y2,crit	**Comments:**

Project

Priest Lake Dam

MOTT MACDONALD

Calculations for

003 EVT-HYES-HYD-001 - Stilling Basin Level Design.xls.

Division	WCD	File No.		
Calc by	JM	Date	31/07/2017	Number of sheets
Checked by	PJH	Date	31/07/2017	of

Stilling Basin Check - Gates Open Check

MM Hydraulic Structures, 1985

2. Outputs

2.1 100% MAX GATE DISCHARGE

Maximum Unit Discharge Per Gate	q_{max}	4.16	m^2s^{-1}		**Comments:**	
Maximum Water Level - Basin Level Difference	E_0	2.18	m	$Z_{uswl,c1}-Z_{bs}$	**Comments:**	
Depth of Flow at Toe of Glasis	y_1	0.800	m	$q/(\surd(2*g*(E_0-y_1))$	**Comments:**	
Velocity @ Toe	v_1	5.20	ms^{-1}	q/y_1	**Comments:**	
Froude @ Toe	Fr	1.86	-	$v_1/\surd(g*y_1)$	**Comments:**	
Conjugate Depth	y_2	1.74	m	$y_1/2*(\surd(1+8*Fr^2)-1)$	**Comments:**	
Water Level	Z_{cg}	25.13	mAD	$Z_{bs}+y_2$	**Comments:**	
Required Flow in Downstream Channel	q_{req}	136.45	m^3s^{-1}	INTERPOLATE(Table 1.1)	**Comments:**	
Number of Gates Required to Aquire DS Level	N_g	0.6	-	$Q_{req}/(q_{max}*w_g)$	**Comments:**	Minimum number of gates = 1

2.2 50% Max Gate Discharge

Maximum Unit Discharge Per Gate	q_{max}	2.08	m^2s^{-1}		**Comments:**	
Maximum Water Level - Basin Level Difference	E_0	2.18	m	$Z_{uswl,c1}-Z_{bs}$	**Comments:**	
Depth of Flow at Toe of Glasis	y_1	0.35	m	$q/(\surd(2*g*(E_0-y_1))$	**Comments:**	
Velocity @ Toe	v_1	6.00	ms^{-1}	q/y_1	**Comments:**	
Froude @ Toe	Fr	3.25	-	$v_1/\surd(g*y_1)$	**Comments:**	
Conjugate Depth	y_2	1.43	m	$y_1/2*(\surd(1+8*Fr^2)-1)$	**Comments:**	
Water Level	Z_{cg}	24.83	mAD	$Z_{bs}+y_2$	**Comments:**	
Required Flow in Downstream Channel	q_{req}	57.59	m^3s^{-1}	INTERPOLATE(Table 1.1)	**Comments:**	
Number of Gates Required to Aquire DS Level	N_g	0.5	-	$q_{req}/(q_{max}*w_g)$	**Comments:**	Minimum number of gates = 1

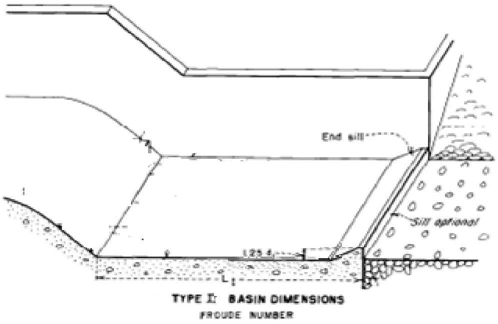
Project **Priest Lake Dam**					
Calculations for 004 EVT-HYES-HYD-002 - USBR Stilling Basins.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

USBR Small Dams Stilling Basin Design

1. USBR Small Dams p.387 - 395

2. USBR Low Froude Stilling Basins

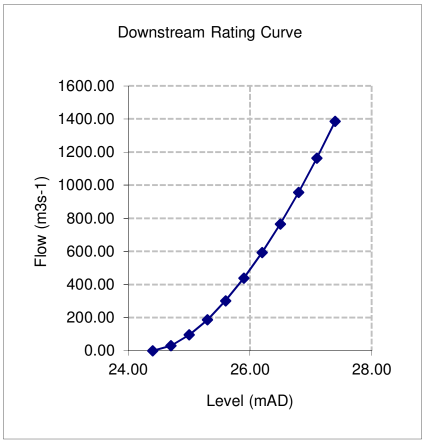
1. INPUTS



Upstream Water Level	Z_{uswl}	26.58	mAD	**Comments:**	**87.2ft(2.3ft gage)**
Upstream Velocity	v_0	0.18	ms^{-1}	**Comments:**	
Basin Level	Z_b	24.40	mAD	**Comments:**	**Downstream level**
Total Flow	Q	223.2	m^3s^{-1}	**Comments:**	
Stilling Basin Width	B	53.645	m	**Comments:**	
Number of Stilling Basins	N	1	-	**Comments:**	
Number of Chute Blocks	Nc	0	-	**Comments:**	
Number of Baffle Blocks	Nb	0	-	**Comments:**	
Basin Type		Type I		**Comments:**	
End Sill Width Multiple	α	0.150		**Comments:**	
Glasis Slope	s	2	H:1V	**Comments:**	

Downstream Rating Curve (Table 1.1)

#	Level	Flow	Velcocity
n	Z_{dswl}	Q	v
-	mAD	m^3s^{-1}	ms^{-1}
1	24.40	0.00	0.00
2	24.70	30.02	1.49
3	25.00	95.23	2.35
4	25.30	187.03	3.06
5	25.60	301.88	3.69
6	25.90	437.57	4.26
7	26.20	592.59	4.78
8	26.50	765.77	5.27
9	26.80	956.17	5.74
10	27.10	1163.06	6.18
11	27.40	1385.80	6.59



Project

Priest Lake Dam

Calculations for	Division	WCD	File No.		
004 EVT-HYES-HYD-002 - USBR Stilling Basins.	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

2. OUTPUTS

2.1 GENERAL

Unit Flow	q	4.16	m^{2}s^{-1}	Q/(B*N)	**Comments:**	
Upstream Velocity Head	h_v	0.00	m	$v_0^2/(2g)$	**Comments:**	
Total Energy	E_0	2.18	m	$Z_{uswl}+h_v-Z_b$	**Comments:**	
Velocity	v_1	5.21	ms^{-1}	$\sqrt{}(2g*(E_0-y_1))$	**Comments:**	
Depth of Flow	y_1	0.799	m	q/v	**Comments:**	
Upstream Froude Number	$\mathbf{Fr_1}$	**1.860**	-	$v_1/\sqrt{}(gy_1)$	**Comments:**	
Conjugate Depth	y_2	1.74	-	$y_1/2*(\sqrt{}(1+8Fr_1^2)-1)$	**Comments:**	
Conjugate Level	Zc	26.14	-	*y2+Zb*	**Comments:**	
Initial Design Flow	Qi	561.47	m^{3}s^{-1}	*INTERPOLATE*	**Comments:**	
Free Jump Length Factor		3.80	m	*INTERPOLATE*	**Comments:**	
Free Jump Length		6.60	m	L/y_2*y_2	**Comments:**	
MM Basin Length		10.44	m	$6*y_{2,design}$	**Comments:**	
Rough Downstream Water Level		25.39	mEL	*INTERPOLATE*	**Comments:**	

2.2 BASIN DIMENSIONS

2.2.1 Basin

Basin Jump Length Factor	F_L	3.80		*INTERPOLATE*	**Comments:**	
Minimum Basin Length	L_B	6.60	m	F_L*y_2	**Comments:**	
Design Length	**L**	**2.80**	**m**	***User Defined***	**Comments:**	**existing case**

2.3 END SILL

Toe Baffle Distance From Chute Toe Factor	L_1/y_2	N/A	-	*INTERPOLATE*	**Comments:**	
Minimum Toe Baffle Distance From Chute Toe	$L_{1,min}$	N/A	m	L_1/y_2*y_2	**Comments:**	
Toe Baffle Distance From Chute Toe	$\mathbf{L_1}$	**0.00**	**-**	***User Defined***	**Comments:**	**existing case**
Height Ratio	η_1	1.25	-			
Minimum End Sill Height	$H_{s,min}$	1.00	-	η_1*y_1	**Comments:**	
End Sill Height	$\mathbf{H_s}$	**0.00**	**-**	***User Defined***	**Comments:**	**existing case**
Minimum End Sill Thickness	$T_{es,min}$	0.00	m	$0.2*H_s$	**Comments:**	
End Sill Thickness	$\mathbf{T_{es}}$	**0.00**	**-**	***User Defined***	**Comments:**	**existing case**
Minimum End Sill Block Width	$W_{es,min}$	N/A	m	$\alpha*y_2$	**Comments:**	
End Sill Block Width	$\mathbf{W_{es}}$	**0.00**	**-**	***User Defined***	**Comments:**	**existing case**
Minimum End Sill Block Spacing	$S_{es,min}$	N/A	-	$\alpha*y_2$	**Comments:**	
End Sill Block Spacing	$\mathbf{S_{es}}$	**0.00**	**-**	***User Defined***	**Comments:**	**existing case**
Number of Blocks	N_{es}	N/A	-	$(B+S_{es})/(S_{es}+W_{es})$		
Spacing @ End	s_e	0.00	-	$(B+S_{es})/(S_{es}+W_{es})$		

Maynord (1990)

The Rock Manual - CIRIA C683 (2007), 2nd Edition. Page 651.

Recommended for the design of riprap

$$D_{30} = S_f C_s C_v C_T y \left[\left(\frac{1}{s-1} \right)^{0.5} \frac{U_d}{\sqrt{(K_1 g y)}} \right]^{2.5}$$

where: D30 is the characteristic riprap size of which 30% is finer by weight
Sf is a safety factor
Cs is a stability coefficient
Cv is a velocity distribution coefficient
Ct is a blanket thickness coefficient
s is the relative density of stone
Ud is the depth averaged flow velocity
K1 is a side slope correction factor
alpha is the angle of the bank to the horizontal
g is the acceleration due to gravity

Enter Data

Safety factor	Sf	1.5	(1.5 suggested)
Rock shape	A or R	R	(Angular or Rounded)
Location	S or D	D	(Straight channel or D/S of structure)
Thickness coeff	Ct	1	(1.0 suggested)
Rel. density of stone	s	2.65	
Angle of bank to horiz.	alpha	45	
Accel due to gravity	g	9.807	

Depth averaged velocity	Ud1	1.49	m/s
(use 3 to test sensitivity)	Ud2	2.35	m/s
	Ud3	3.06	m/s
	Ud4	3.69	m/s
	Ud5	4.26	m/s
	Ud6	4.78	m/s
	Ud7	5.27	m/s
	Ud8	5.74	m/s
	Ud9	6.18	m/s
	Ud10	6.59	m/s

Water depths	y1	0.30	m
	y2	0.60	m
	y3	0.90	m
	y4	1.20	m
	y5	1.50	m
	y6	1.80	m
	y7	2.10	m
	y8	2.40	m
	y9	2.70	m
	y10	3.00	m

Results

Stability coefficient	Cs	0.375
Vel. distribution coeff	Cv	1.25
COT alpha		1.000
Side slope correction	K1	0.416

Water depth, y (m)	Velocity, Ud (m/s)	D30 (m)	D50 (m)	Dn50 (min)	Dn50 (max)
0.3	1.485746161	0.24	0.34	0.28	0.31
0.6	2.345969986	0.62	0.89	0.75	0.81
0.9	3.058061436	1.09	1.56	1.31	1.42
1.2	3.685578544	1.62	2.31	1.94	2.10
1.5	4.255158398	2.19	3.13	2.63	2.85
1.8	4.781256906	2.80	4.00	3.36	3.64
2.1	5.272865326	3.44	4.91	4.13	4.47
2.4	5.73607816	4.11	5.87	4.93	5.34
2.7	6.175280015	4.79	6.85	5.75	6.23
3	6.593766666	5.50	7.86	6.60	7.15

Pilarczyk (1990)

The Rock Manual - CIRIA C683 (2007), 2nd Edition. Page 649.

Recommended for the design of riprap, cabled concrete blocks, box gabions and asphalt mattresses

$$\boxed{D_{n50} = \frac{\mu}{(s-1)}\frac{0.035}{\Psi_{CR}}\frac{K_T K_Y}{K_S}\frac{U^2}{2g}}$$

where:

μ is the stability correction factor

s is the relative density of the revetment

$\boxed{\Psi \quad CR}$ is the stability factor

D is the characteristic size of the protection
Kt is a turbulence factor
Kh is a depth factor
Ks is the slope factor
Ud is the depth averaged flow velocity
g is the acceleration due to gravity

Enter Data

Location	C or E	C	Continuous protection or Edges and transitions
Relative density of stone	s	2.65	
Porosity of stone	n	0.4	(suggest 0.4 for stone and sand)
Type of revetment	R or G	R	Riprap or Gabions
Bank slope	alpha	0	
Internal friction angle	sigma	45	
Longitudinal channel slope	beta	0.00176	
Turbulence factor	N or H	H	Normal or High
Accel due to gravity	g	9.807	m/s2
Flow decvelopment			

Depth averaged velocity	Ud1	1.49	m/s
	Ud2	2.35	m/s
	Ud3	3.06	m/s
	Ud4	3.69	m/s
	Ud5	4.26	m/s
	Ud6	4.78	m/s
	Ud7	5.27	m/s
	Ud8	5.74	m/s
	Ud9	6.18	m/s
	Ud10	6.59	m/s

Water depths	y1	0.30	m
	y2	0.60	m
	y3	0.90	m
	y4	1.20	m
	y5	1.50	m
	y6	1.80	m
	y7	2.10	m
	y8	2.40	m
	y9	2.70	m
	y10	3.00	m

Results

Stability correction factor		1
Stability factor		0.035
Relative density of revetment		1.65
Side slope term	Kd	1.000
Longitudinal slope	k1	1.000
Slope factor	Ks	1.000
Turbulence factor	Kt	1.500

tan alpha	0.000
cos alpha	1.000
tan sigma	1.000
sin sigma - beta	0.707
sin sigma	0.707

Estimate Dn50	Depth averaged velocity Ud (m/s)	Depth y (m)	Depth factor Kh	Dn50 (m)	D50 (min)	D50 (max)	D50 (ave)	D30 (min)	D30 (max)
0.08	1.485746161	0.3	0.764	0.078	0.09	0.09	0.09	0.06	0.07
0.21	2.345969986	0.6	0.808	0.206	0.23	0.25	0.24	0.16	0.17
0.36	3.058061436	0.9	0.833	0.361	0.40	0.43	0.41	0.28	0.30
0.54	3.685578544	1.2	0.851	0.536	0.59	0.64	0.61	0.41	0.45
0.73	4.255158398	1.5	0.865	0.726	0.80	0.86	0.83	0.56	0.60
0.93	4.781256906	1.8	0.876	0.928	1.02	1.11	1.06	0.71	0.77
1.14	5.272865326	2.1	0.885	1.141	1.25	1.36	1.31	0.88	0.95
1.36	5.73607816	2.4	0.893	1.362	1.50	1.62	1.56	1.05	1.13
1.59	6.175280015	2.7	0.899	1.590	1.75	1.89	1.82	1.22	1.32
1.82	6.593766666	3	0.905	1.824	2.00	2.17	2.09	1.40	1.52

Escarameia and May (1992) - HR Wallingford

The Rock Manual - CIRIA C683 (2007), 2nd Edition. Page 650.

Recommended for the design of riprap, loose or interlocking concrete blocks and gabion mattresses.

$$D_{n50} = C\frac{U_b{}^2}{2g(s-1)}$$

where: Dn50 is the characteristic size of stone

Dn50 is the size of the equivalent cube

$$D_{n50} = \left(\frac{W_{50}}{\rho_s}\right)^{\frac{1}{3}}$$

W50 is the weight of particle, Ps is the density of stone

C is a coefficient that takes account of the turbulence intensity TI

g is acceleration due to gravity

s is the relative density of the revetment material

Ub is the velocity near the bend (at 10% of the water depth above the bed)

Turbulence Levels

Situation	Turbulence Level	
	Qualitative	TI
Straight river or channel reaches and wide natural bends (R/W>26)	Normal (low)	0.12
Edge of revetments in straight reaches	Normal (higher)	0.20
Bridge piers, caissons and groynes; transitions	Medium to high	0.35-0.50
Downstream of hydraulic structures (weirs, culverts, stilling basins)	Very high	0.60

where: R is the centreline radius of bend

W is the water surface width at the upstream end of the bend

Values of C

Type of revetment	Value of C	Observations
Riprap	12.3TI-0.20	Valid for TI>=0.05
Concrete blocks	9.22TI-0.15	Valid for TI>=0.05
Gabion mattresses	12.3TI-1.65	Valid for TI>=0.12

Enter Data

Turbulence Intensity	TI	0.6	
Type of revetment	R,C or G	R	(Riprap, Concrete or Gabion)
Relative density of riprap	s	2.65	
Accel due to gravity	g	9.81	m/s2
Depth averaged velocity	Ud1	1.49	m/s
	Ud2	2.35	m/s
	Ud3	3.06	m/s
	Ud4	3.69	m/s
	Ud5	4.26	m/s
	Ud6	4.78	m/s
	Ud7	5.27	m/s
	Ud8	5.74	m/s
	Ud9	6.18	m/s
	Ud10	6.59	m/s
Suggested factor for Ub		47%	
Actual		60%	

Results

Coefficient C 7.18

Depth averaged velocity	Ub	Ub2	Dn50	D50 (min)	D50 (max)	D50 (ave)	D30 (min)	D30 (max)
1.486	0.891	0.795	0.18	0.19	0.21	0.20	0.14	0.15
2.346	1.408	1.981	0.44	0.48	0.52	0.50	0.34	0.37
3.058	1.835	3.367	0.75	0.82	0.89	0.85	0.57	0.62
3.686	2.211	4.890	1.08	1.19	1.29	1.24	0.83	0.90
4.255	2.553	6.518	1.45	1.59	1.72	1.65	1.11	1.20
4.781	2.869	8.230	1.83	2.01	2.17	2.09	1.40	1.52
5.273	3.164	10.009	2.22	2.44	2.64	2.54	1.71	1.85
5.736	3.442	11.845	2.63	2.89	3.13	3.01	2.02	2.19
6.175	3.705	13.728	3.04	3.35	3.62	3.49	2.34	2.54
6.594	3.956	15.652	3.47	3.81	4.13	3.97	2.67	2.89

Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 999 Summary Sheet.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

Operational Scenario

Number of Operational Gates	11.00		
Flow	214.39	m^3s^{-1}	**7,570cfs**
Gage Level	0.70	m	**2.3 ft**
River Level	26.58	mAD	**87.2 ft**
Gate Opening	1.52	m	**60 inches/5 ft**
Stilling Basin	As existing		

Downstream Rating Curve

Depth of Flow	Water Level	Discharge	Average Velocity	Froude
D	Zwl	Q	v	Fr
m	mAD	m^3s^{-1}	ms-1	-
0.00	24.40	0.00	0.00	0.00
0.30	24.70	30.02	1.49	0.87
0.60	25.00	95.23	2.35	0.97
0.90	25.30	187.03	3.06	1.03
1.20	25.60	301.88	3.69	1.07
1.50	25.90	437.57	4.26	1.11
1.80	26.20	592.59	4.78	1.14
2.10	26.50	765.77	5.27	1.16
2.40	26.80	956.17	5.74	1.18
2.70	27.10	1163.06	6.18	1.20
3.00	27.40	1385.80	6.59	1.22

Radial Gate with Raised Cill

Max Unit Discharge	qmax	4.16	m^2s^{-1}
Design Unit Discharge	q	4.11	m^2s^{-1}
Flow	Qmax	223.20	m^3s^{-1}

Stilling Basin Level Design

Condition 1 - 100% Design Flow with 100% Design Flow Downstream

Condition 2 - 10% Design Flow with 0% Design Flow Downstream

Condition 3 - 30% Design Flow with 30% Design Flow Downstream

Summary of Critical Basin

Basin Level	23.40	mAD
Critical Condition	Condition 1	
Froude	10.30	
Critical Condition	Condition 2	

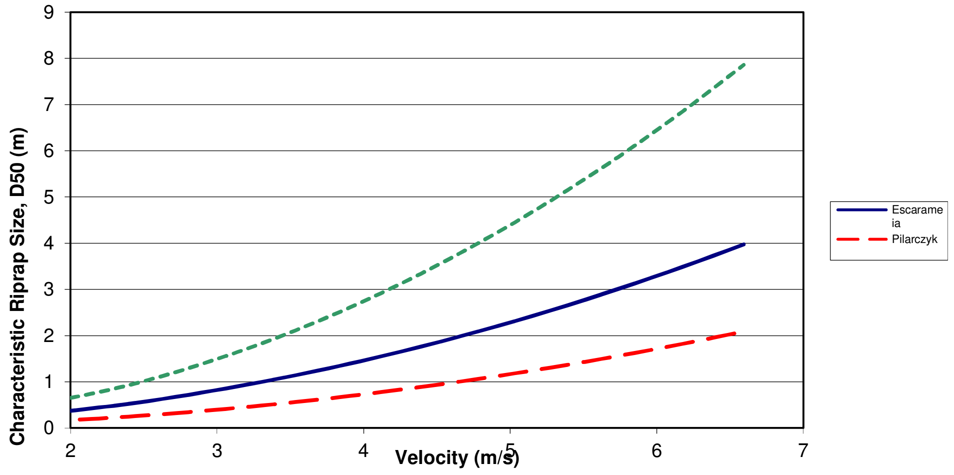
Project **Priest Lake Dam**	MOTT MACDONALD				
Calculations for 999 Summary Sheet.	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
	Checked by	PJH	Date	31/07/2017	of

Conjugate Depth	2.11	mAD
Critical Condition	Condition 1	
Max Basin Length	6.72	mAD
Critical Condition	Condition 1	

USBR Stilling Basins

Minimum Basin Length	6.60	m
Minimum End Sill Height	1.00	m
Minimum End Sill Thickness	0.00	m

Erosion Stone Sizing


Characteristic Riprap Size versus Velocity
Section 4

Project

Priest Lake Dam

Calculations for	Division	WCD	File No.		
	Calc by	JM	Date	31/07/2017	Number of sheets
007 Stream Power.	Checked by	PJH	Date	31/07/2017	of

Stream Power

C742, Annandale (2006)

1. INPUTS

Specific gravity of Water	ϒ	1	-	
Flow rate per unit width (original)	q0	1.764	m^2s^{-1}	
Flow rate per unit width (new)	q1	1.871	m^2s^{-1}	
Flow rate per unit width (design)	q2	6.223	m^2s^{-1}	
Flow rate per unit width (winter)	q3	4.161	m^2s^{-1}	
Upstream Water Level (original)	WL_{US0}	26.792	mAoD	
Upstream Water Level (new)	WL_{US1}	26.944	mAoD	
Upstream Water Level (design)	WL_{US2}	27.752	mAoD	
Upstream Water Level (winter)	WL_{US3}	26.579	mAoD	
Downstream Water Level (original)	WL_{DS0}	24.470	mAoD	
Downstream Water Level (new)	WL_{DS1}	24.473	mAoD	
Downstream Water Level (design)	WL_{DS2}	25.670	mAoD	
Downstream Water Level (winter)	WL_{DS3}	25.394	mAoD	
Jump length (original)	L_0	7.911	m	
Jump length (new)	L_1	8.191	m	
Jump length (design)	L_2	11.954	m	
Jump length (winter)	L_3	6.603	m	
Mass Strength Number	M_s	0.020	-	**Using SPT results**
Block Size Number	K_b	28.317	-	**Assuming non-cohesive granular soiland d50=1ft**
Inter-particule/Inter-block shear strength number	K_d	0.268	-	**Assuming phi=15 degrees**
Orientation and shape number	J_s	1	-	**Assuming diameter of root bulb bounded by fine fibrous roots = 0.1m**

2. OUTPUTS

Erodability index	K	0.152		$M_sK_bK_dJ_s$
Critical stream power	P_{sc}	0.243	W/m^2	If K>0.1, K^0.75 else 0.48*K^0.44
Energy Slope (original)	S_{e0}	0.293	m	$(WL_{US0}-WL_{DS0})/L_0$
Energy Slope (new)	S_{e1}	0.302	m	$(WL_{US1}-WL_{DS1})/L_1$
Energy slope (design)	S_{e2}	0.174	m	$(WL_{US2}-WL_{DS2})/L_2$
Energy slope (winter)	S_{e3}	0.179	m	$(WL_{US3}-WL_{DS3})/L_3$
Stream Power (original)	P_{s0}	0.5177	W/m^2	Yq_0S_{e0}
Stream Power (new)	P_{s1}	0.5646	W/m^2	Yq_1S_{e1}
Stream Power (design)	P_{s2}	1.0839	W/m^2	Yq_2S_{e2}
Stream Power (Winter)	P_{s3}	0.7466	W/m^2	Yq_3S_{e3}

mottmac.com

C.2 Outlet Dam Structure Stability Analysis

MOTT MACDONALD

Memorandum

Project: Priest Lake Water Management Study

Our reference: 376997 **Your reference:**

Prepared by: C Brodbaek **Date:** 02/23 2018

Approved by: S Philips **Checked by:** S Philips

Subject: Priest Lake Outlet Dam - Preliminary Stability Analysis

1 Introduction

As a part of concept development and preliminary studies for the Priest Lake Water Management study in Bonner County, Idaho, a preliminary geotechnical stability assessment of the existing outlet dam located on Priest River in the southern part of Priest Lake.

The preliminary analyses included in this Memorandum are performed to verify stability of the existing dam and at a feasibility level confirm stability of proposed dam modifications due to a potential 6-inch pool raise from EL +3.0 ft to +3.5 ft.

2 Basis of Design

2.1 Existing Outlet Dam Design Drawings

Existing design drawings of the outlet dam and the radial gates are included in Appendix A to this Memorandum. The gates are T-1 Overflow Type from Waterman Industries.

2.2 Geotechnical Criteria

The stability against sliding and foundation bearing pressures shall satisfy the safety levels indicated in Table 1, per USACE EM 1110-2-2200 *Gravity Dam Design.*

Table 1. Dam stability criteria per EM 1110-2-2200 Gravity Dam Design

Load Condition	Resultant location at base	Minimum Sliding FS	Foundation Bearing Pressure
Usual	Middle 1/3	2.0	≤ allowable
Unusual	Middle 1/2	1.7	≤ allowable
Extreme	Within base	1.3	≤ 1.33 x allowable

2.3 Geotechnical conditions

Information on geotechnical conditions at the outlet dam is included in the 1978 Priest Lake Outlet Structure replacement project drawings. The locations of geotechnical drill hole #1 and #2 are shown in Figure 1.

Geotechnical drill logs for drill holes #1 and #2 are shown in Figure 2. A third drill hole log, DH #4, is also shown on the drill logs. The location of DH #4 is, however, unknown.

Ground conditions: The drill logs indicate that the soils at the site consists of layers of soft to medium stiff silts and clays and loose sands with blow counts in the range 3 to 7 for 6-inch penetration in the upper 30 ft of soil. This is an indication of normal consolidated to lightly over-consolidated alluvial sediments. Based on review of available soils information the sands at the site may be susceptible to liquefaction during a seismic event.

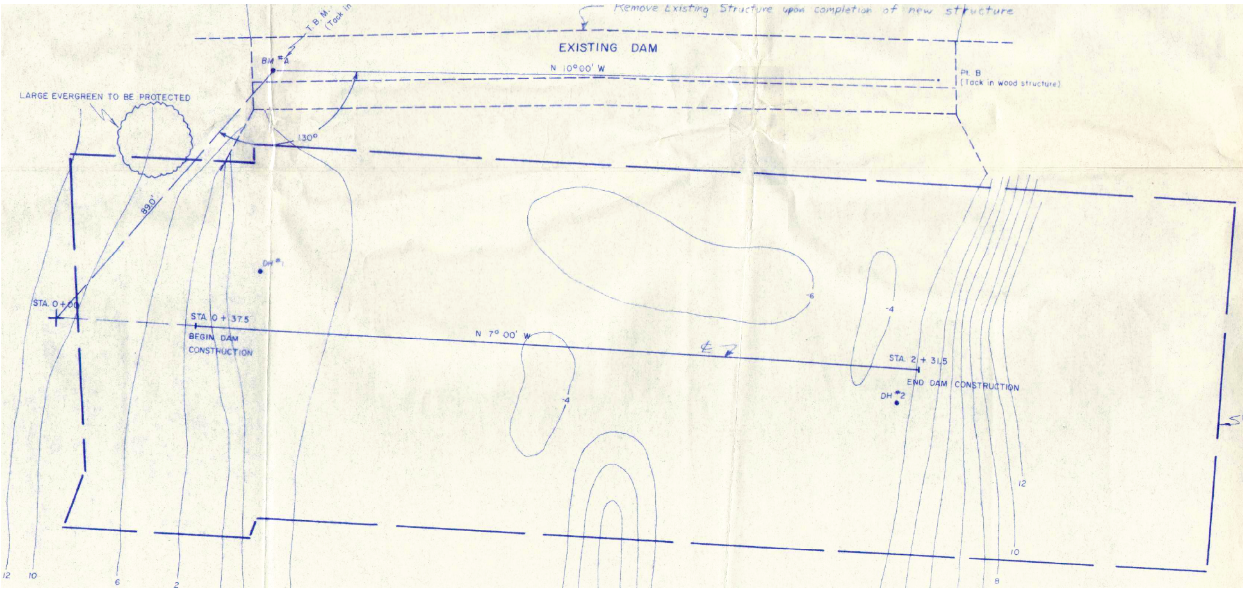


Figure 1: Location of drill holes DH #1 and #2.

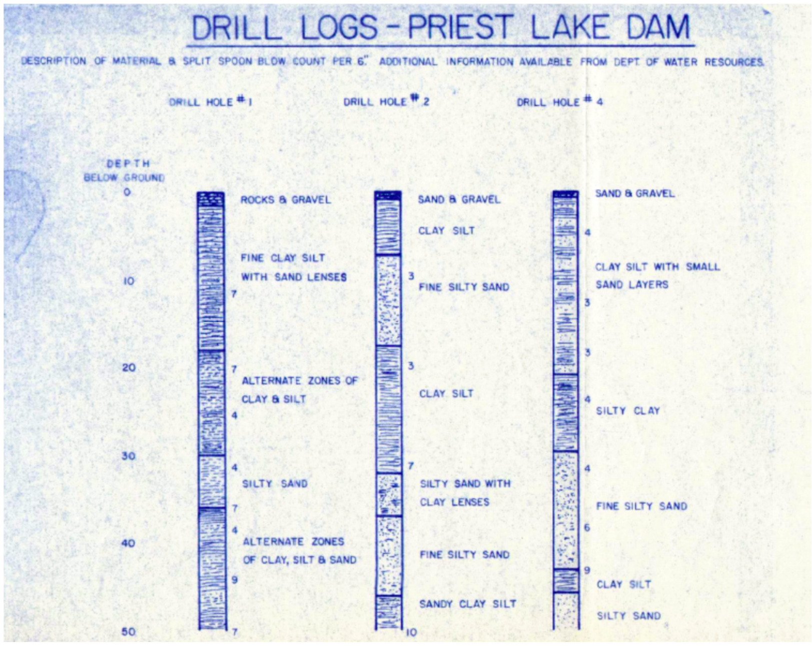


Figure 2: Drill hole logs for at outlet dam.

2.4 Hydraulic Criteria

Hydraulic criteria for dam stability assessment include the following Summer Recreation lake levels:

Present pool lake level:	3.00 ft
Proposed future pool lake level:	3.50 ft
Tailwater level during low flow periods:	-4.35 ft

Design tailwater level is based on observations at a site visit during week of September 18th, 2017. The condition is shown in Figure 3 with one gate partially open. It is estimated the flow during this condition is approximately 100 cfs.

Figure 3: Tailwater during low flow periods. 100 cfs estimated on site.

2.5 Seismic Criteria

The main focus of the project is to evaluate potential pool raises between zero and six inches. Seismic performance requirements would need to be established with IDWR in subsequent phases of the design.

For the present feasibility assessment seismic analysis is performed to verify seismic stability of the improved outlet dam with a 6-inch pool raise, assuming the foundation soils do not liquefy during a seismic event or the impact from liquefaction on stability of the dam is acceptable.

Seismic analysis is performed utilizing the Maximum Credible Earthquake (MCE) from USGS website ASCE 7-10:

MCE (2,475 year return period): PGA = 0.30 g for a site class E (soft and loose soils)

A horizontal pseudo-static seismic coefficient, k_h, is applied for dam stability analysis, corresponding to an *Extreme* load condition in Table 1:

k_h = 0.5 x PGA = 0.15 g

Detailed assessment of liquefaction during a major seismic event should be performed once additional geotechnical explorations is performed during subsequent phases of the project. Depending on severity of liquefaction impact on the dam structure during a seismic event – ground improvements to mitigate liquefaction may be required.

2.6 Unit Weights and Materials

The following unit weights are applied for the stability analysis:

Reinforced concrete: 150 pcf

Natural saturated soils: 110 pcf

Fill - sand, gravel and rip rap: 120 pcf

River freshwater: 62.4 pcf

3 Dam Stability Analysis

3.1 Existing Dam

Stability analysis of the existing dam is performed based on the original construction drawings of the outlet dam replacement project in 1978. Additional scour protection was placed downstream of the dam extending 18 feet from the edge of the existing sill after the outlet dam was constructed due to excessive scour following commissioning of the dam.

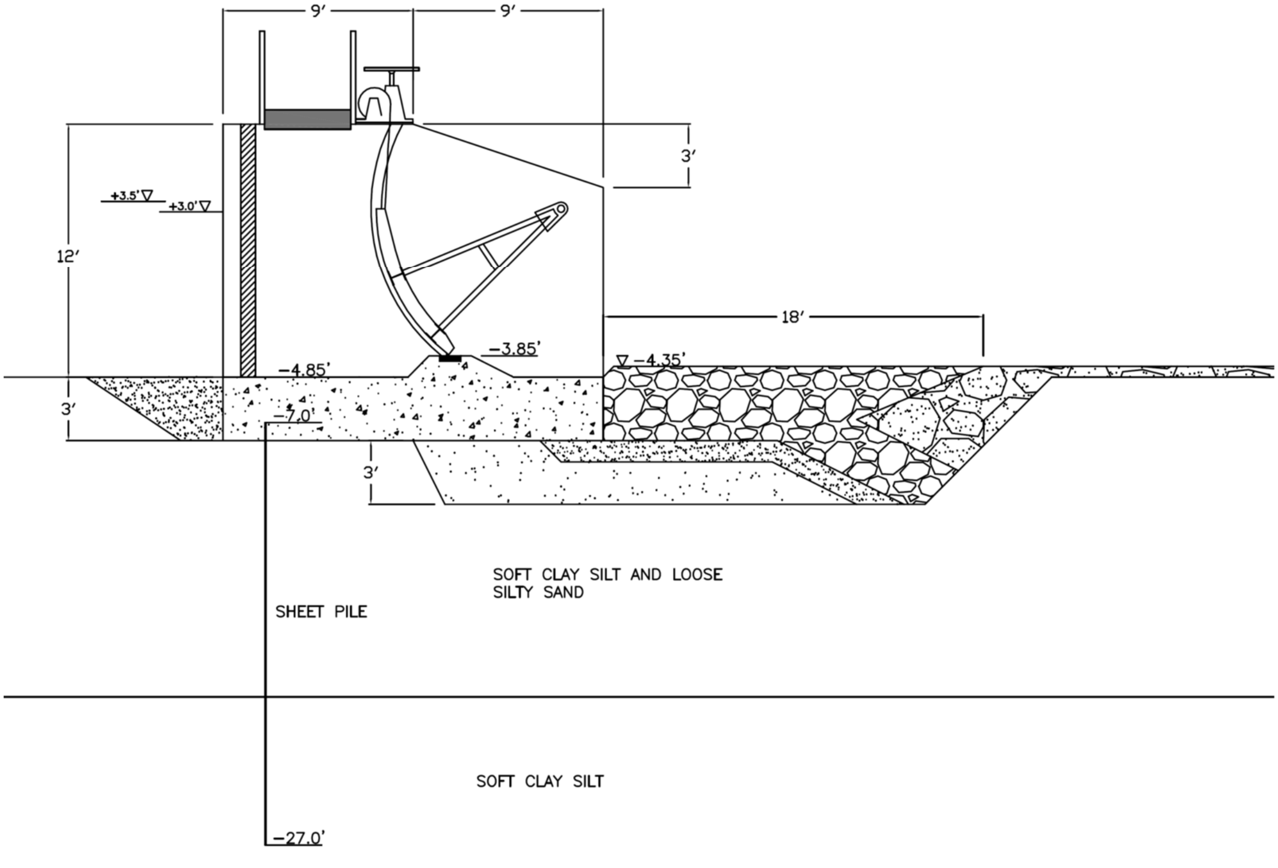


Figure 4: Existing outlet dam.

The following basic assumptions are made:

1. Effect from sheet pile is neglected due to uncertainty of penetration, sheet pile interlocking, water tightness, connection to sill is not detailed and is likely not a positive structural connection.
2. The three-foot thick filter sand and gravel below sill would in principle provide pressure relief. However, siltation since the dam was constructed may have caused the pressure relief to be

ineffective. Two scenarios are run accordingly: a) Filter sand fully effective for pressure relief and b) Filter sand ineffective for pressure relief.

Summary of forces applied in stability analysis is shown in Figure 5 and Figure 6 with definitions and values listed in Table 2, Table 3 and Table 4.

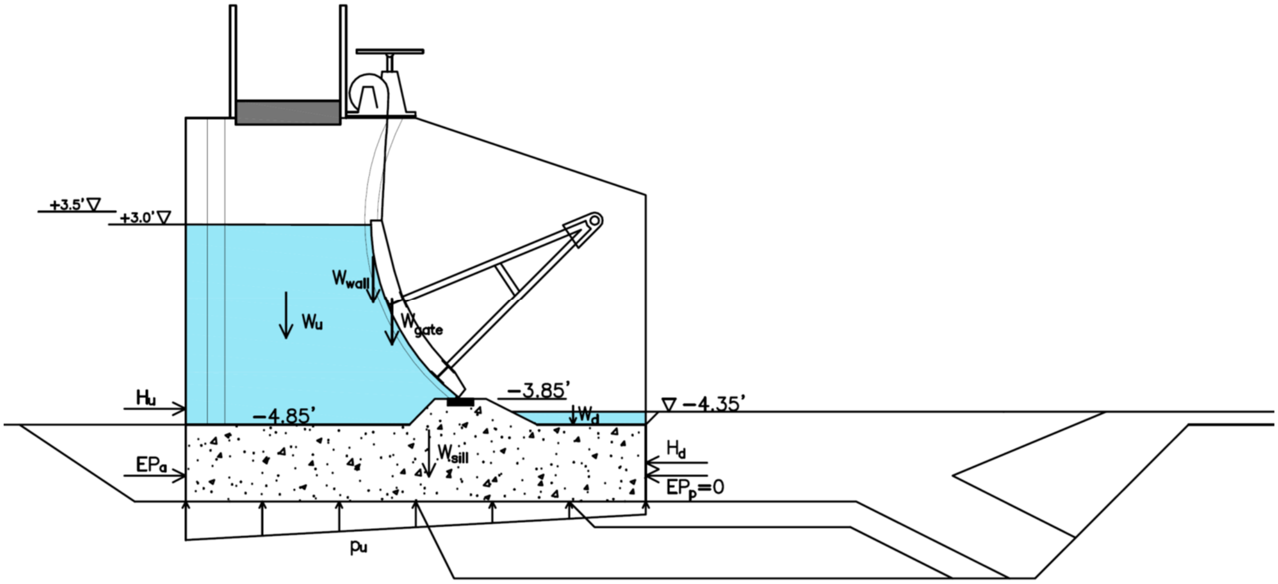


Figure 5: Stability analysis force definition - Non-effective relief drains.

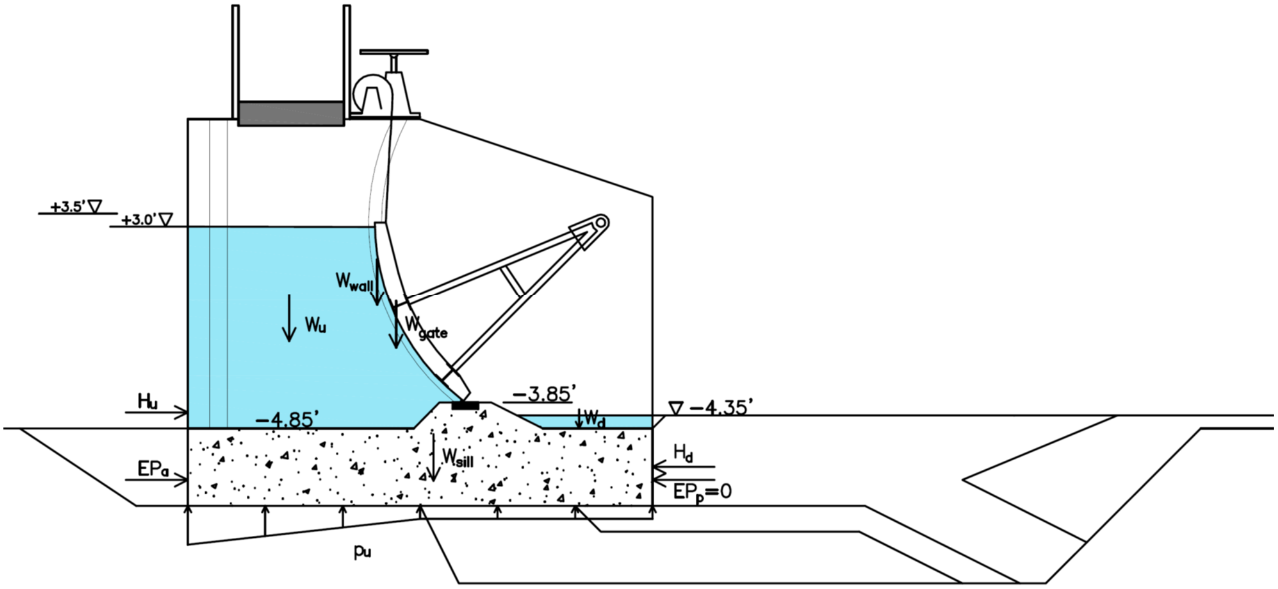


Figure 6: Stability analysis force definition - Effective relief drains.

Table 2: Self-weights - Outlet dam stability analysis.

Item	ID	Water EL (ft)	Unit weight	Weight (kip)	CG (ft) from toe
Concrete sill	W_{sill}		150 pcf	151	8.9
Side walls	W_{wall}		150 pcf	45	9.4
Ponding water - Upstream	W_u	3.00	62.4 pcf	66	13.7
Ponding water - Upstream	W_u	3.50	62.4 pcf	70	13.8
Ponding water - Downstream	W_d	-4.35	62.4 pcf	2.4	2.3
Gate face plate	W_{gate}		490 pcf	1.3	9.0
Horizontal ribs			31 lb/ft	1.0	9.0
Misc. steel				0.5	9.0

Table 3: Earth and hydrostatic pressures.

Item	ID	Water EL (ft)	Unit weight (pcf)	Total Horizontal Force (kip)
Upstream water pressure	H_u	3.0	62.4	64.3
Upstream water pressure	H_u	3.5	62.4	70.3
Downstream water pressure	H_d	-4.35	62.4	-6.7
Upstream earth pressure	EP_a	N/A	57.6	1.5
Downstream earth pressure	EP_p	N/A	57.6	0.0

Table 4: Uplift sill pressures.

Item	ID	Water EL (ft)	Unit weight (pcf)	Uplift pressure (psf)
Uplift bottom of sill	p_u	3.0 3.5 -4.35	62.4	677 708 218

The stability analysis results for the existing dam at pool EL +3 feet and +3.5 feet are shown in Table 5. It is seen that the existing dam is stable under current conditions, albeit it does not meet the minimum requirements to safety for sliding. For a maximum pool EL at +3.0 ft the factor of safety against sliding varies between 1.2 and 1.6. For raised maximum pool EL at +3.5 ft the factor of safety against sliding varies between 1.1 and 1.4.

Table 5: Dam stability results for existing dam.

	Existing Dam without effective foundation drain		**Existing Dam with effective foundation drain**		
	Pool at EL 3.0 ft	**Pool at EL 3.5 ft**	**Pool at EL 3.0 ft**	**Pool at EL 3.5 ft**	**Criteria**
Factor of Safety - Sliding	1.2	1.1	1.6	1.4	Minimum FS = 2.0
Factor of Safety - Overturning	1.6	1.5	1.9	1.9	None
Uniform Bearing Pressure	0.46 ksf	0.47 ksf	0.57 ksf	0.59 ksf	Results indicate low bearing pressure
Resultant location at base, eccentricity from center of sill	1.2 ft	1.4 ft	0.9 ft	1.1 ft	Eccentricity within middle 1/3 of base width, i.e. less than B/6 = 3 ft

3.2 Existing Dam with New Concrete Scour Apron

To improve the scour protection downstream of the dam a 3.5 ft thick and 30 ft long concrete scour apron is proposed. The scour apron will be in direct contact with the existing sill and will improve the sliding resistance of the existing dam accordingly. The new concrete scour apron would be placed on a gravel bed and the downstream end of the apron would be protected with rip rap as shown in Figure 7.

A filter fabric below the gravel bed may be required to prevent migration of foundation soils. A minimum fabric to ground friction coefficient of 0.35 is assumed.

The total effective weight of 3.5 ft of concrete and 1 ft of gravel is 364 psf. The total sliding resistance provided by the apron is F_{apron} = 0.35 x 17.5' x 30' x 364 psf = 67 kips and the sliding resistance provided by the existing dam is 72 kips. Hence the total sliding resistance of the existing dam and new concrete scour apron is 139 kips.

The total lateral thrust for an increased pool at EL 3.5 ft is H_{tot} = 65 kips. Hence the factor of safety against sliding including the new concrete scour apron is FS = 139 kips / 65 kips = 2.1 which is greater than minimum factor of safety against sliding of 2.0.

Additional pseudo static lateral thrust during a seismic event has been calculated to be of the order 36 kips. Hence the total lateral design thrust during an MCE seismic event is 101 kips corresponding to factor of safety of FS = 139 kips / 101 kips = 1.37, which is greater than 1.3 required for an extreme event.

A factor of safety of 1.37 requires that the foundation soils do not liquefy during a seismic event. Additional geotechnical explorations should be performed in subsequent phases of the project, including explorations to identify whether the soils at the site are liquefiable as noted in Section 2.5 of this memo.

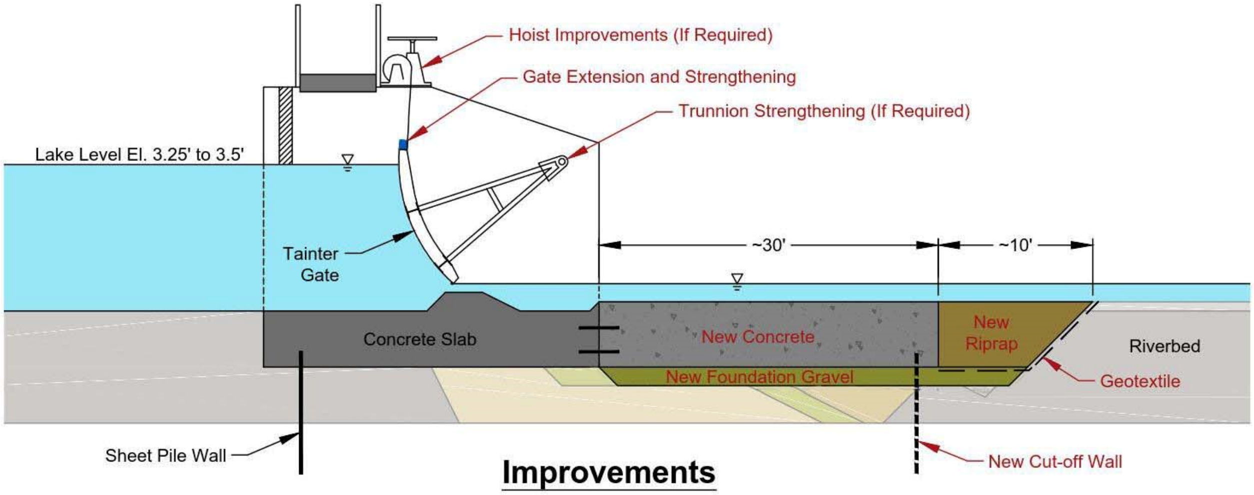


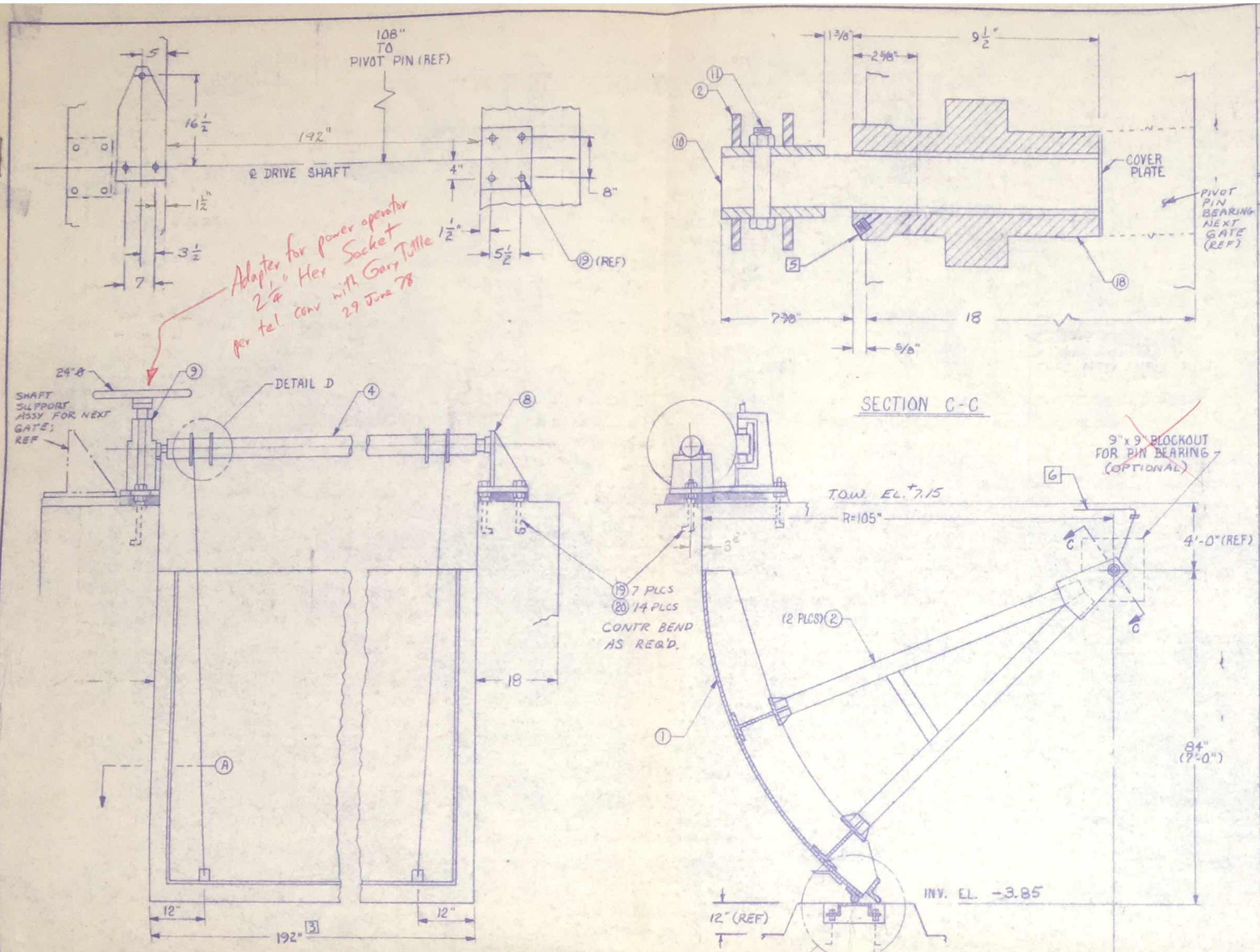
Figure 7: Existing dam with new scour concrete scour apron.

References

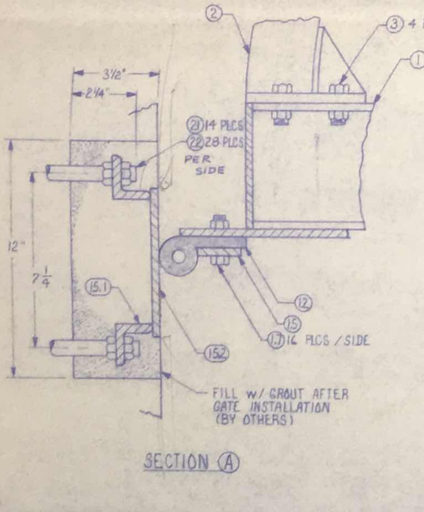
U.S. Army Corps of Engineers (USACE). Gravity Dam Design; EM 1110-2-2200; 30 June 1995.

USGS U.S. Seismic Design Maps available at https://earthquake.usgs.gov/designmaps/us/application.php.

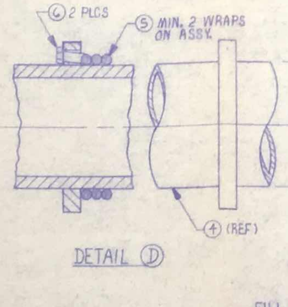
Appendix A



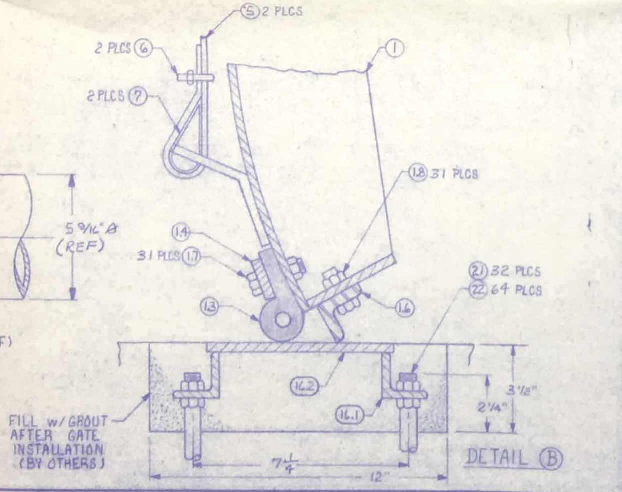
Adapter for power operator
2¼" Hex Socket
per tel conv with Gary Tuttle
29 June 78



SECTION Ⓐ



DETAIL Ⓓ



DETAIL Ⓑ

MATERIAL

1. CAST IRON-ASTM A-126 CL. B
6. STRUCTURAL STEEL-ASTM A-36
8. STEEL HARDWARE ASTM A-307
9A. STN. STL. ASTM A-276 TYPE 304
10. STEEL-AISI 1020
11. STEEL PIPE ASTM A-120
12. COMMERCIAL BRASS
13. RUBBER ASTM-D-2000
14. SEAMLESS STEEL TUBE

5. GALVAN
6. PAINT:
ANDREW BROWN SYNFLEX ENAMEL; COLOR - RED

ITEM	DESCRIPTION	MAT'L	FIN	PART NO. OR SIZE	QTY/ GATE
1	FACE PLATE ASSY.	-	-		1
1.1	FACE PLATE WELDMENT	6	4	PER DETAIL PAGE 3	1
1.2	SIDE SEAL (J- BULB	13	2	4" J-BULB - REF 100168	2
1.3	BOTTOM SEAL (J-BULB)	13	2	4" J-BULB REF 100168 x 192" LG	1
1.4	BOTTOM SEAL RETAINER	6	4	1/4" x 2" BAR x 185 ½" LG.	1
1.5	SIDE SEAL RETAINER	6	4	1/4" x 2" BAR x 97" LG	2
1.6	STOP BAR	6	4	3" x 2" x ½" ANGLE	1
1.7	SEAL ATTACHING HDW.	8	5	3/8" NC x 1½" LG. H.H. BOLT & NUT	63 EA.
1.8	STOP BAR ATTACHING HARDWARE	8	5	½" NC x 2" LG. H.H. BOLT & NUT	31 EA.
2	RADIAL ARM WELDMENT	6	4	DETAIL PAGE 3	2
3	ATTACHING HARDWARE	8	5	3/4" NC x 2" LG. H.H. BOLT & NUT	16 EA.
4	DRIVE SHAFT ASSY	—	4	DETAIL PAGE 2	1
4.1	SHAFT	6	—	2"Ø BAR	2
4.2	PIPE	11	—	5"Ø SCH. 40 PIPE x 193¼ LG.	1
4.3	SHAFT SUPPORT	6	—	3/8" ℞ - 2 7/16" I.D. x 5" O.D.	4
4.4	CABLE STOPS	6	—	3/8" ℞ - 5 5/8" I.D. x 7" O.D.	4
5	CABLE	6	4	3/8"Ø 7 x 19 FLEX. CABLE x 16 FT. LG.	2
6	CABLE CLIP	6	4	FOR 3/8" Ø CABLE	4
7	THIMBLE	6	4	FOR 3/8" Ø CABLE	2
8	SHAFT SUPPORT ASSY.	-	-	DETAIL PAGE 2	1
8.1	BRACKET	6	4	½" ℞	1
8.2	BEARING	-	2	BROWNING FB-250 (2"Ø BORE)	1
8.3	ATTACHING HARDWARE	8	5	5/8" NC x 2" LG. H.H. BOLT & NUT	4 EA.
9	LIFT	1	6	TYPE L-1 w/ 24 Ø HANDWHEEL	1
10	PIVOT PIN	9A	2,3	2½ Ø x 16.75 LG	2
11	ATTACHING HARDWARE	8	5	3/4" NC x 5" LG. H.H. BOLT & NUT	4 EA
12	KEY	6	2	3/8" x 3/8" x 2" LG.	2
13					
14					
15	SIDE SILL ASSY (L & R)	6	4	DETAIL PAGE 2	1 EA
15.1	ANGLE	-	-	2" x 2" x 1/4" ANGLE	14
15.2	PLATE	-	-	1/4" x 6" BAR	1
16	BOTTOM SILL ASSY.	6	4	DETAIL PAGE 2	1
16.1	ANGLE	-	-	2" x 2" x 1/4" ANGLE	32
16.2	PLATE	-	-	1/4" x 6" BAR	1
17					
18	PIVOT PIN BEARING (BNZ BUSHED)	1,12	1,3	W-1348	2
19	ANCHOR BOLT	10	5	3/4 NC x 12 LG	7
20	HEX NUT	10	5	3/4" NC	14
21	ANCHOR BOLT	10	5	5/8" NC x 10" LG.	60
22	HEX NUT	8	5	5/8" NC	120

NOTES:

1. DESIGN HD.: 8 FT (INVERT TO TOP OF WATER)
2. HOIST CAPACITY @ 25 LB. PULL = 2765 LBS (7,685 IN-LBS)
3. FOR FAB. & INSPECTION ONLY; OVERALL DIM. TO OUTSIDE OF SEALS - 192¼ ± 1/8"
4. APPROX. GATE WEIGHT: 3530 LBS
5. TAP 1/8-NPT THREADS FOR LUBE LINE
6. CONTR. PROVIDE LUBE LINE, FITTINGS, & BRACKETS AS REQ'D.
7.

Approved by W.
29 June 1978

REF.:
PRIEST LAKE OUTLET
BONNER COUNTY, IDAHO
IDAHO DEPT. OF WATER RESOURCES,

(11) GATES REQ'D @ PRIEST LAKE OUTLET PLAN PG 97-2020

REV. NO.	DATE	BY

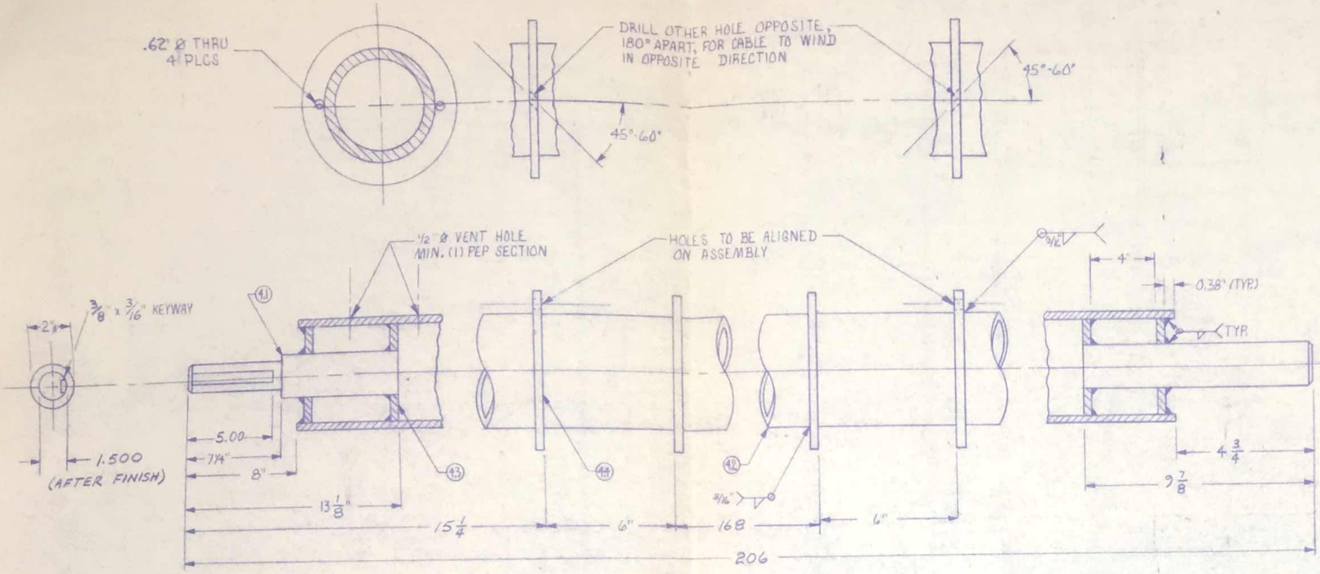
192" x 84" (16 FT. x 7 FT)
WATERMAN MODEL
T-1 OVERFLOW TYPE
RADIAL GATE

FOR:
ROBERT B. GOEBEL CONST.
TERMINAL BOX 3086
SPOKANE, WASH. 99220
P.O. #2606

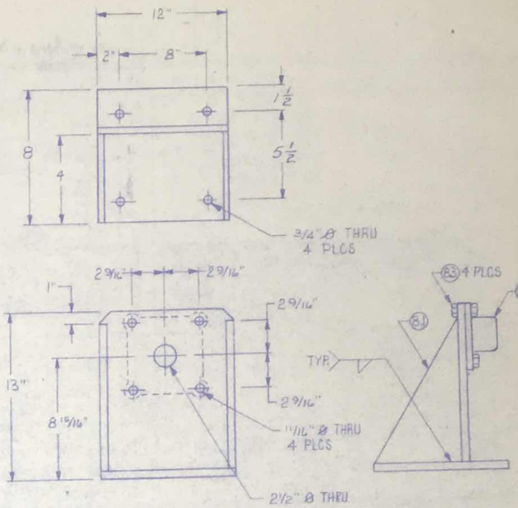
JOB NO R-7211
QUOTE NO NQ78-D-024
DRAWN BY G.T.
CHECKED BY —
SCALE NONE
DATE 6-21-78
DR. NO. RC78128
PAGE 1 OF 3

RED TOP

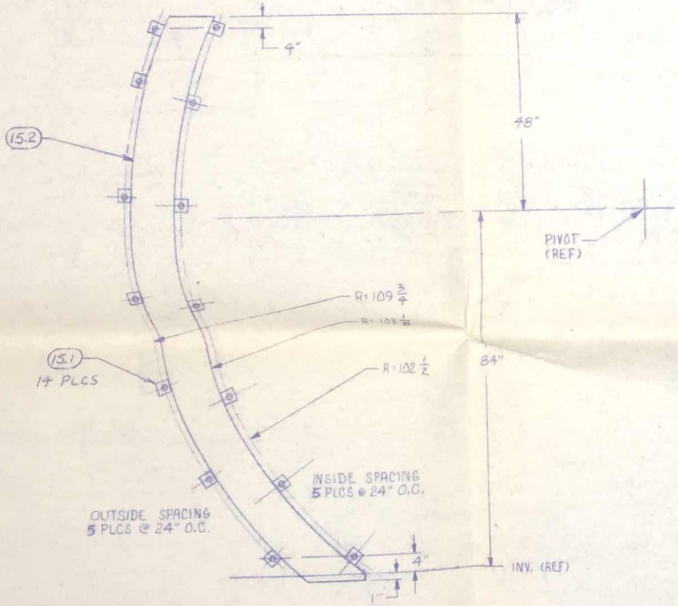
WATERMAN INDUSTRIES, INC.
EXETER, CALIFORNIA 93221



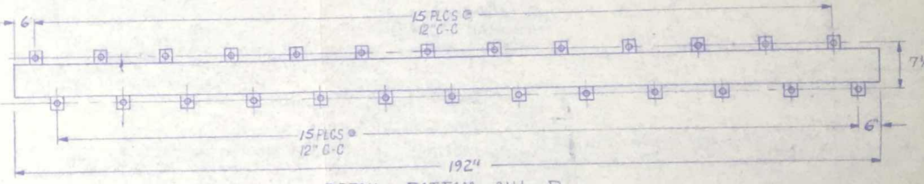
DETAIL DRIVE SHAFT ASSY.



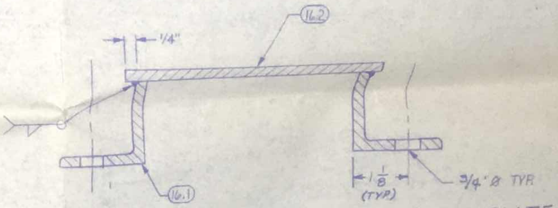
DETAIL SHAFT SUPPORT ASSY.



DETAIL SIDE SILL PLATE



DETAIL BOTTOM SILL ℄



TYPICAL SECTION BOTTOM SILL & SIDE PLATE

REF.:
PRIEST LAKE OUTLET
BONNER COUNTY, IDAHO
IDAHO DEPT. OF WATER RESOURCES

REV. NO.	DATE	BY

(11) GATES REQ'D

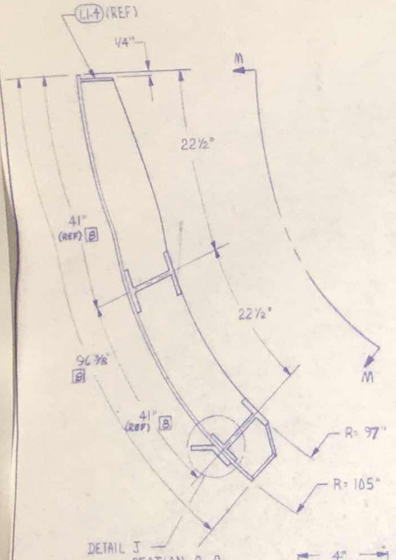
192" x 84" (16 FT x 7 FT)
WATERMAN MODEL
T-1 OVERFLOW TYPE
RADIAL GATE

FOR:
ROBERT B. GOEBEL CONST.
TERMINAL BOX 3086
SPOKANE, WASH. 99220
P.O. #2605

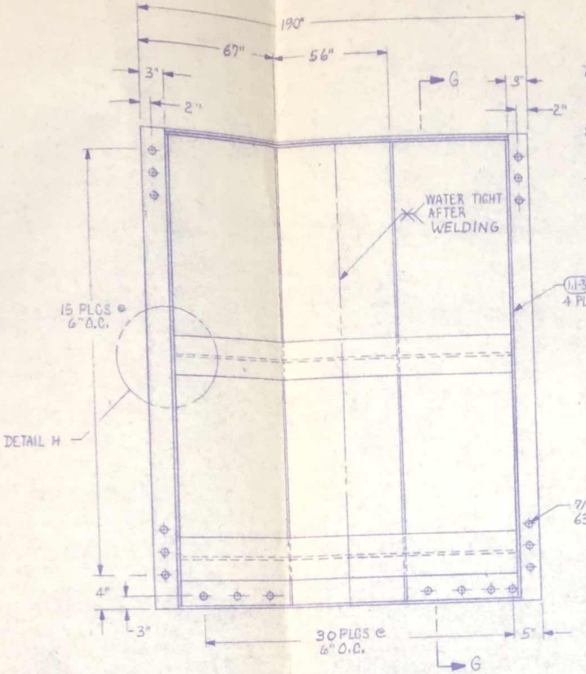
JOB NO. R-7211
QUOTE NO. NQ78-D-024
DRAWN BY G.T.
CHECKED BY —
SCALE NONE
DATE 6-27-78
DR NO RC78128
PAGE 2 OF 3

RED TOP WATERMAN

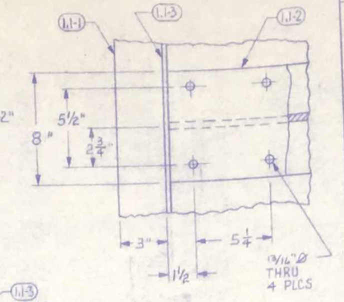
WATERMAN INDUSTRIES, INC.
EXETER, CALIFORNIA 93221



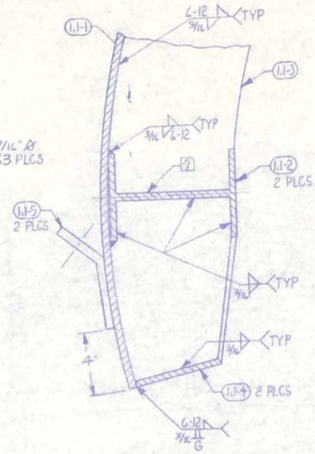
SECTION G-G



VIEW M-M



DETAIL H



DETAIL J

MATERIAL KEY

6. STRUCTURAL STEEL- ASTM A-36
14. SEAMLESS STEEL TUBE

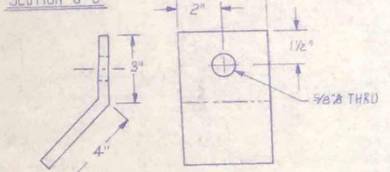
FINISH KEY

4. GALVANIZE PER ASTM A 123

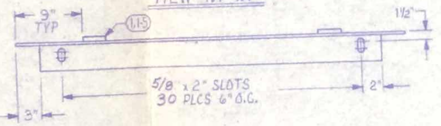
ITEM	DESCRIPTION	MATL	FIN	PART # OR SIZE	QTY
1.1	FACE PLATE WELDMENT	–	4	-	1
-1	FACE PLATE	6	-	1/4" R : 95" x 96 3/8" $\boxed{8}$	2
-2	HORIZONTAL RIB	6	-	WF 8x31.0 LB/FT WF BEAM x 183 1/4" LG.	2
-3	VERTICAL RIB	6	-	3/8" R	4
-4	TOP & BOTTOM BAR	6	-	1/2" x 6" BAR x 184" LG.	2
-5	LIFTING LUG	6	–	1/2" x 4" BAR x 7" LG.	2
2	RADIAL ARM WELDMENT	-	-		2
2.1	ARM	6	4	3 OD" x 3 OD" x 1/4" WALL SQ TUBE x 92 1/2" LG.	4
2.2	ARM BRACE	6	4	3 OD" x 3 OD" x 1/4" WALL SQ TUBE	2
2.3	END PLATE	6	4	1/2" R 8"x8"	4
2.4	GUSSET	6	4	3/8" R	A/R
2.5	PIVOT PLATE	6	4	1/2" R	4
2.6	HOUSING	14	4	4" O.D. x 2 1/2 I.D. x 6" LG	2

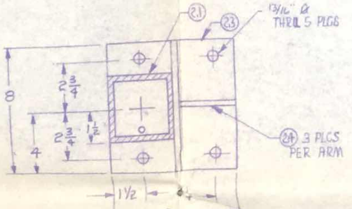
NOTES:

$\boxed{8}$ LENGTH (ALONG ARC) AFTER ROLLING MEASURED @ R= 105"
$\boxed{2}$ DRILL (3) 1/2" Ø DRAIN HOLES IN EACH HORIZONTAL RIB

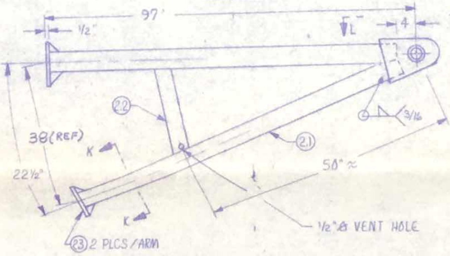


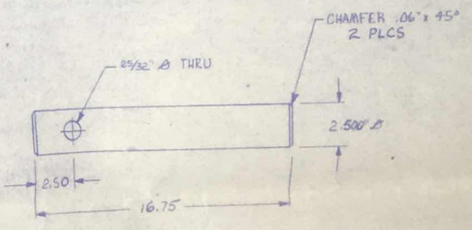
LIFTING LUG DETAIL



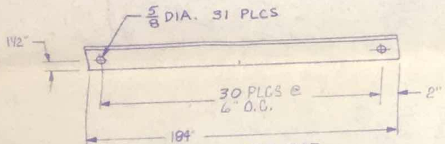


SECTION K-K

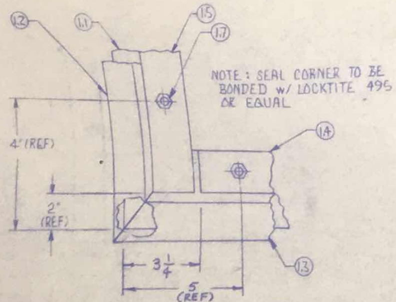




DETAIL PIVOT PIN

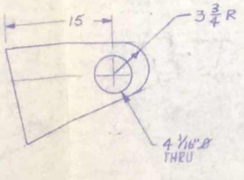


DETAIL STOP BAR

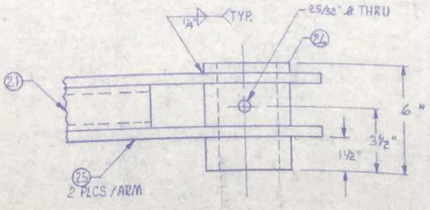


NOTE: SEAL CORNER TO BE BONDED w/ LOCKTITE 495 OR EQUAL

ASSY. DETAIL - CORNER SEAL



DETAIL PIVOT PLATE



VIEW L-L

LEFT ARM SHOWN: HOUSING & END PLATE OPPOSITE ON RIGHT ARM

REF:
PRIEST LAKE OUTLET
BONNER COUNTY, IDAHO
IDAHO DEPT. OF WATER RESOURCES

(11) GATES REQ'D

192" x 84" (16 FT x 7 FT)
WATERMAN MODEL
T-1 OVERFLOW TYPE
RADIAL GATE

FOR:
ROBERT B. GOEBEL CONST.
TERMINAL BOX 3086
SPOKANE, WASH. 99220
P.O. # 2605

REV. NO.	DATE	BY

JOB NO. R-7211
QUOTE NO NQ78-D-024
DRAWN BY G.T.
CHECKED BY —
SCALE NONE
DATE 6-21-78
DR NO. RC 78128
PAGE 3 OF 3

RED TOP WATERMAN

WATERMAN INDUSTRIES, INC.
EXETER, CALIFORNIA 93221

IDAHO DEPARTMENT OF WATER RESOURCES

BOISE, IDAHO

1978

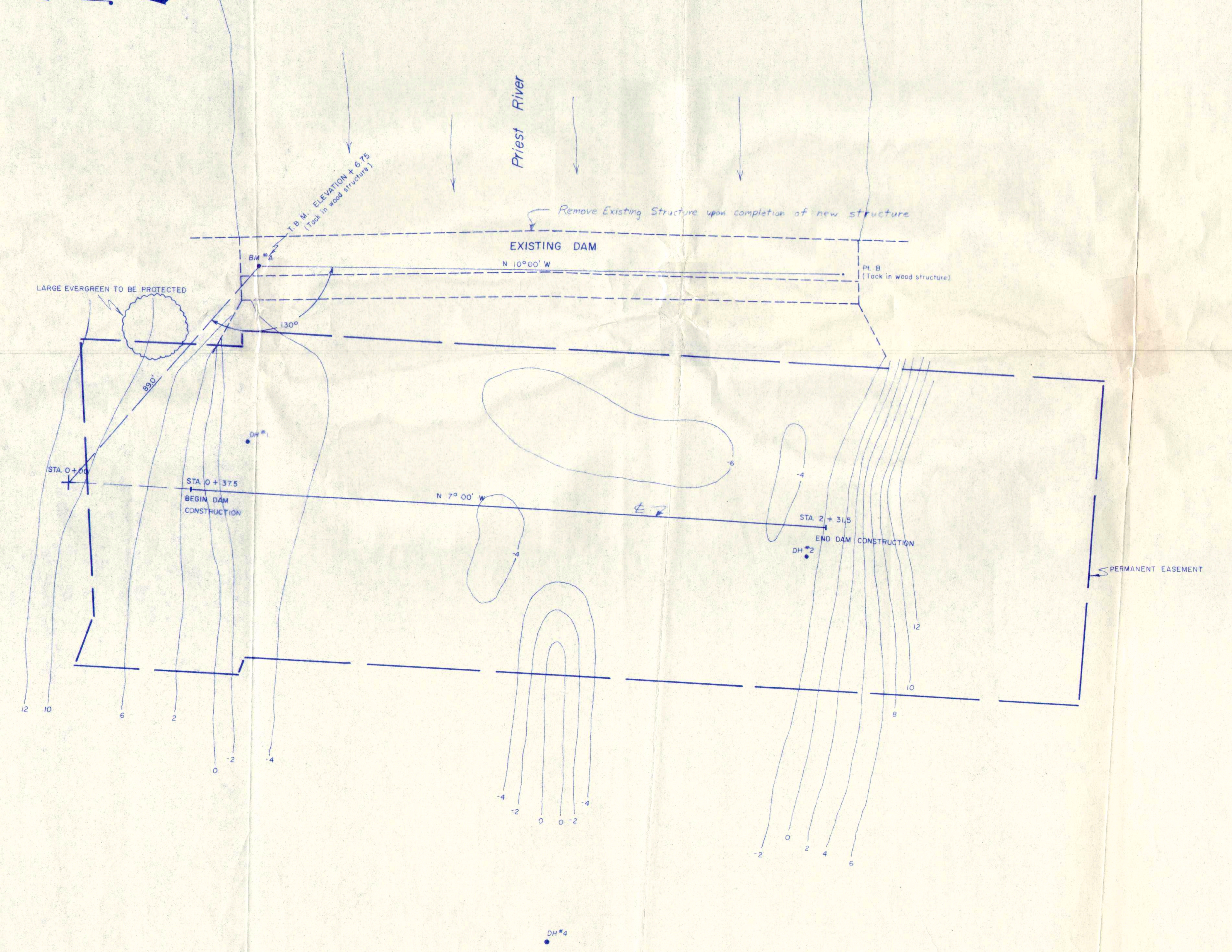
PLANS FOR REPLACEMENT OF CONTROL STRUCTURE

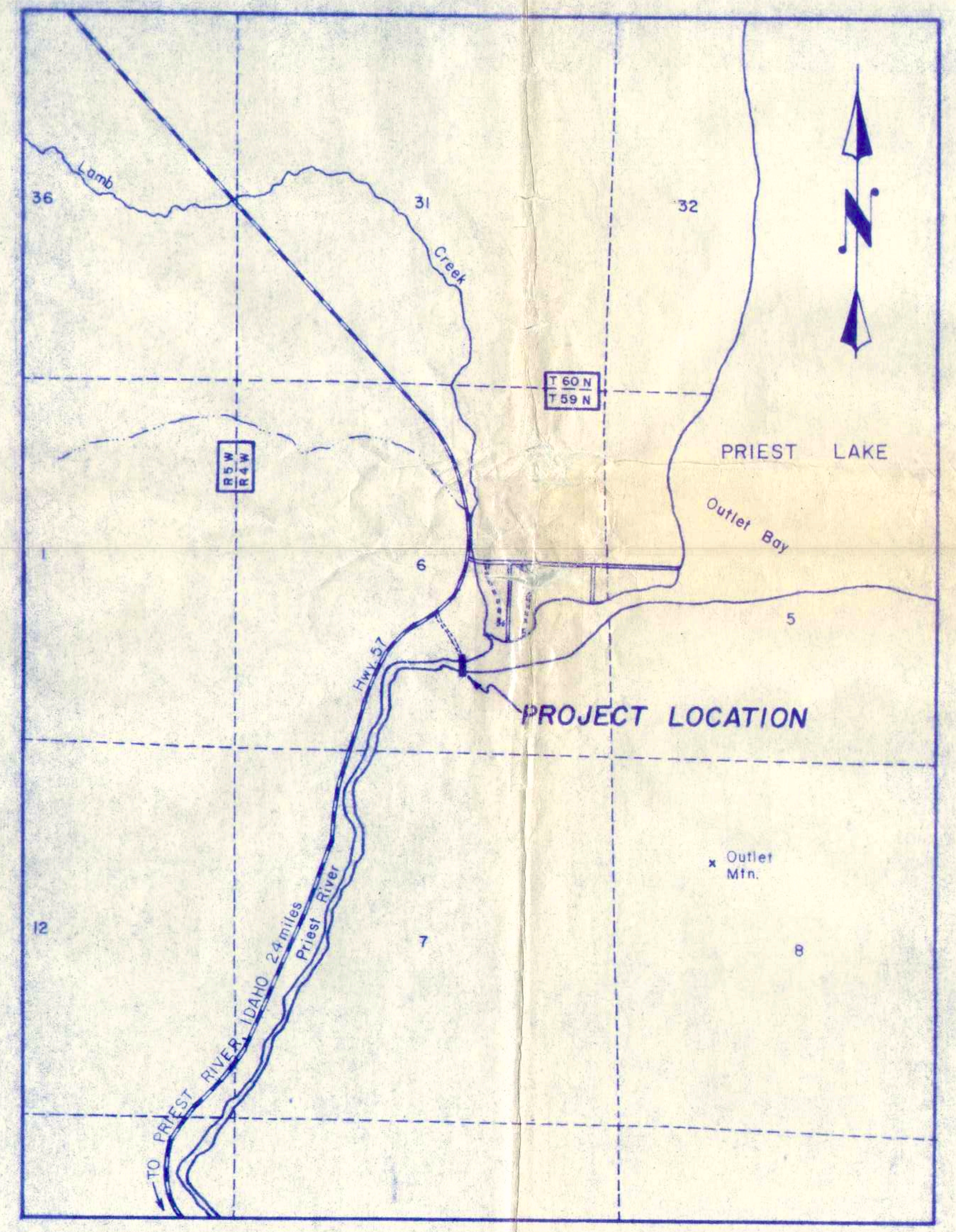
PRIEST LAKE OUTLET

BONNER COUNTY, IDAHO

PLAN VIEW

SCALE 1" = 20'





PROJECT LOCATION MAP

SCALE 1" = 2000'

3/20/78 Norman C Young

MICROFILMED
JUL 23 1991

Not filmed in place
Drawing too large
to be filmed.

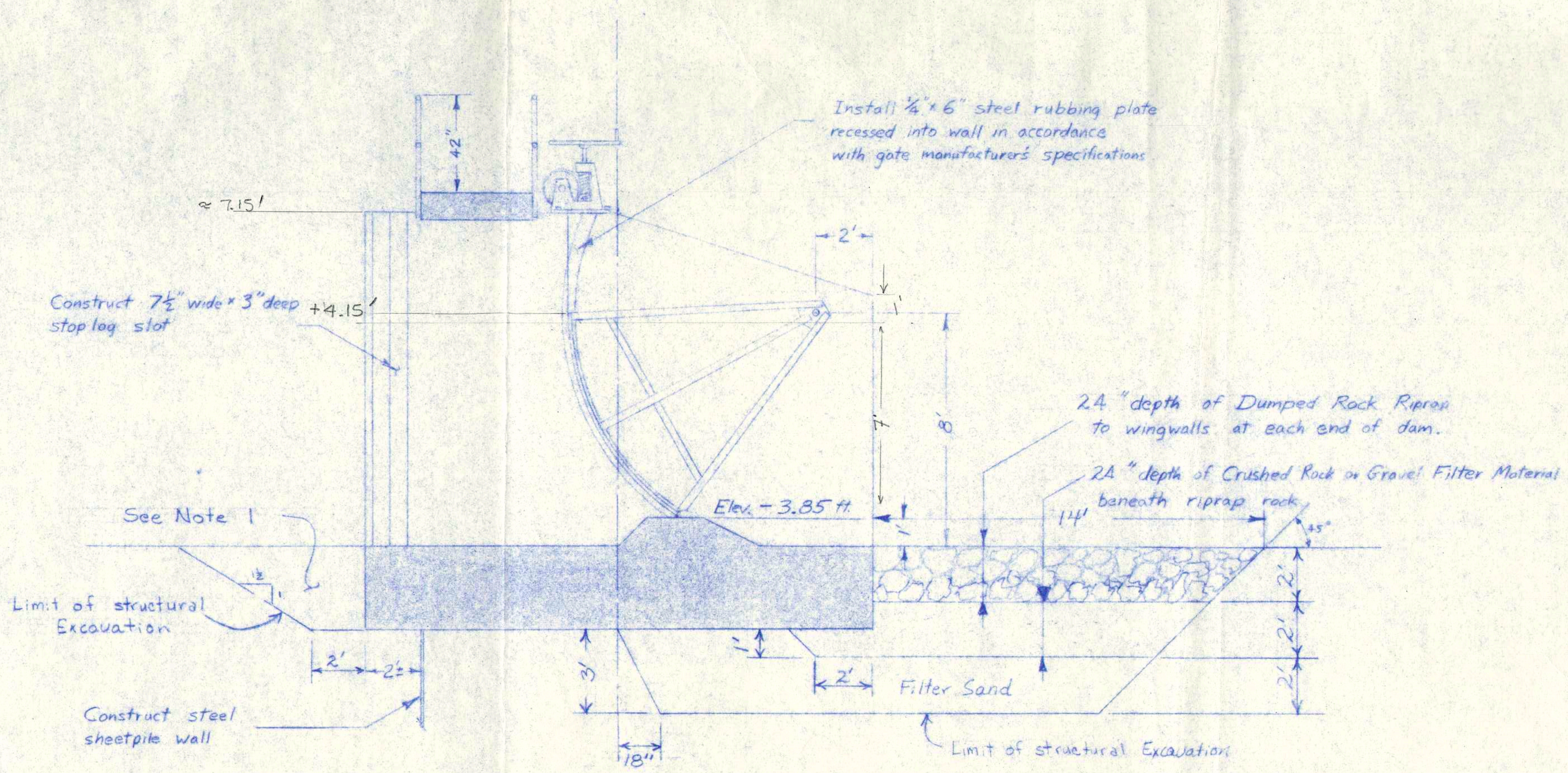
17-3-78

IDAHO DEPARTMENT OF WATER RESOURCES		
SCALE: as shown	APPROVED BY:	DRAWN BY RC PB
DATE: Mar 1978		REVISED
PRIEST LAKE OUTLET — TITLE PAGE		
97-2020		DRAWING NUMBER 97-2020-1

24X36 PRINTED ON NO. 1000H CLEARPRINT

DOWNSTREAM PROFILE
Scale: 1" = 8'-0"

PLAN VIEW
Scale: 1" = 8'-0"



TYPICAL SECTION
Scale: 1" = 4'-0"

NOTES

1. Place compacted backfill, Class II compaction.
2. Place compacted backfill with no more than 5% passing a No. 200 sieve above level of crushed rock or gravel to top of wingwalls. Class I compaction. Limits of fill extend vertically from crushed rock or gravel as shown
3. Place Class I compacted impervious backfill behind walls near abutment and along water side of upstream walls.

APPROVED
IDAHO DEPARTMENT OF WATER RESOURCES
3/20/78 Norman C. Young
DATE ADMINISTRATOR, RESOURCES ADMIN. DIVISION

17-3-78
William R. Gossett

IDAHO DEPARTMENT OF WATER RESOURCES		
SCALE: as shown	APPROVED BY: W. Gossett	DRAWN BY WG JB
DATE: 17 Mar. 1978		REVISED
PRIEST LAKE OUTLET —— PLAN & PROFILE		
		DRAWING NUMBER 97-2020-2

TYPICAL INTERIOR WALL DETAIL

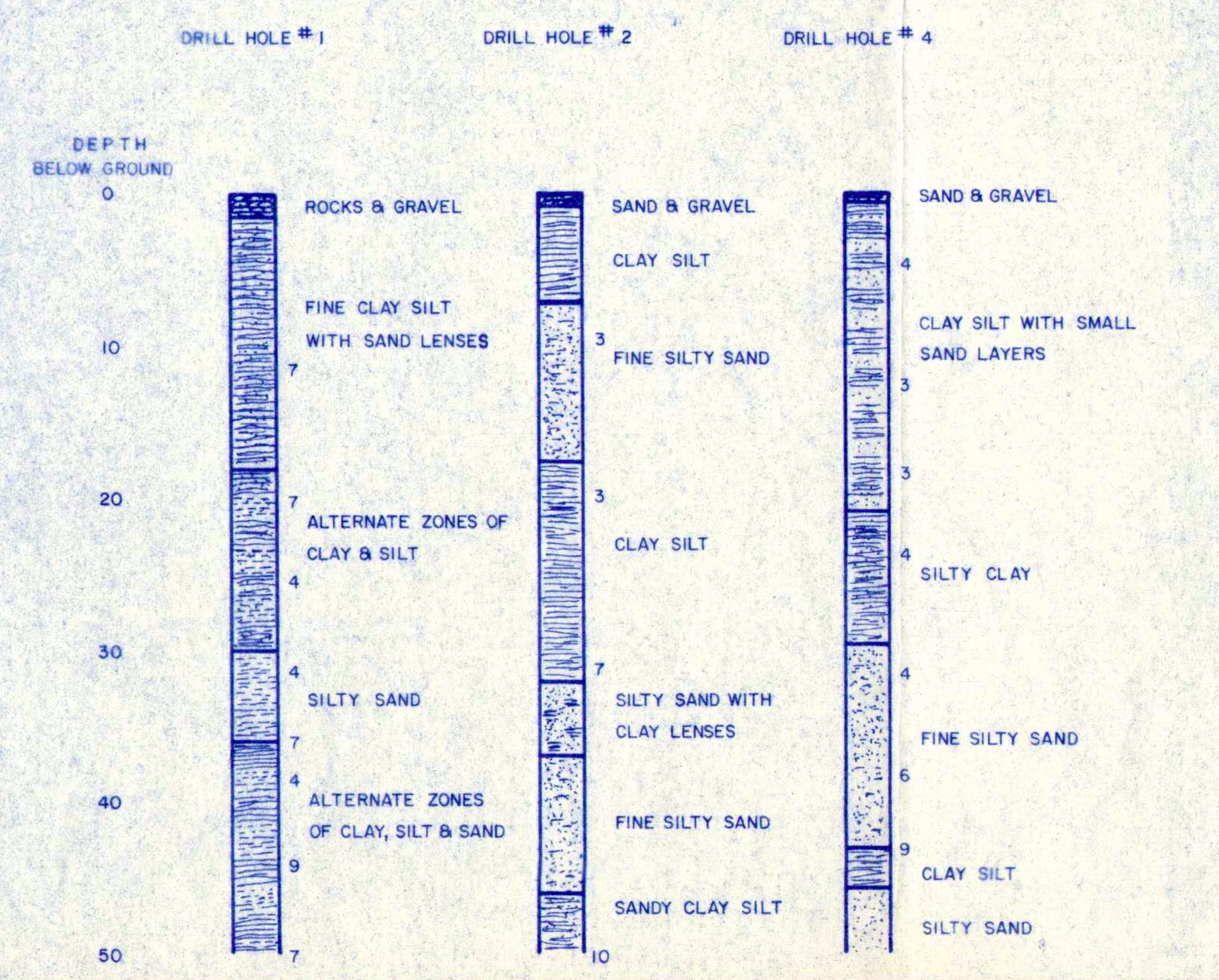
SCALE: 1"=2'

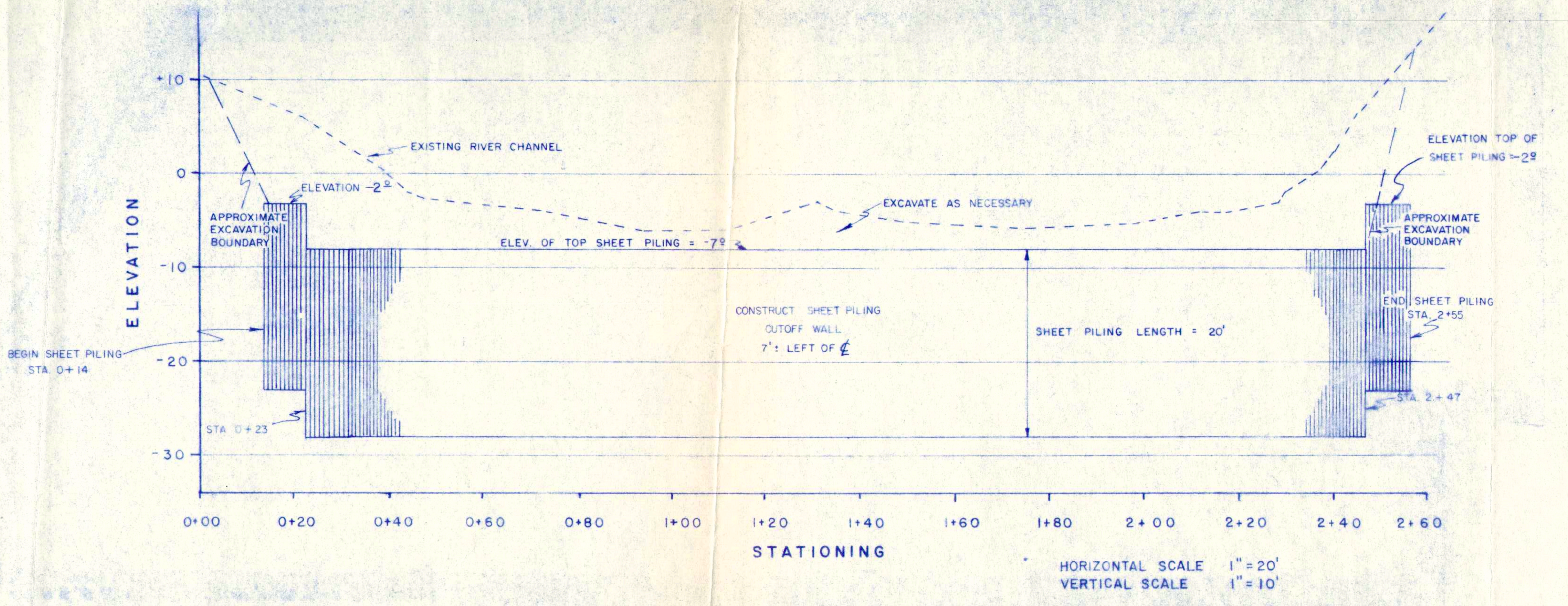
TYPICAL END WALL DETAIL

SCALE: 1"=2'

DRILL LOGS - PRIEST LAKE DAM

DESCRIPTION OF MATERIAL & SPLIT SPOON BLOW COUNT PER 6". ADDITIONAL INFORMATION AVAILABLE FROM DEPT OF WATER RESOURCES.





HORIZONTAL SCALE 1" = 20'
VERTICAL SCALE 1" = 10'

SHEET PILE WALL PROFILE

17-3-78
William R. Yoatt

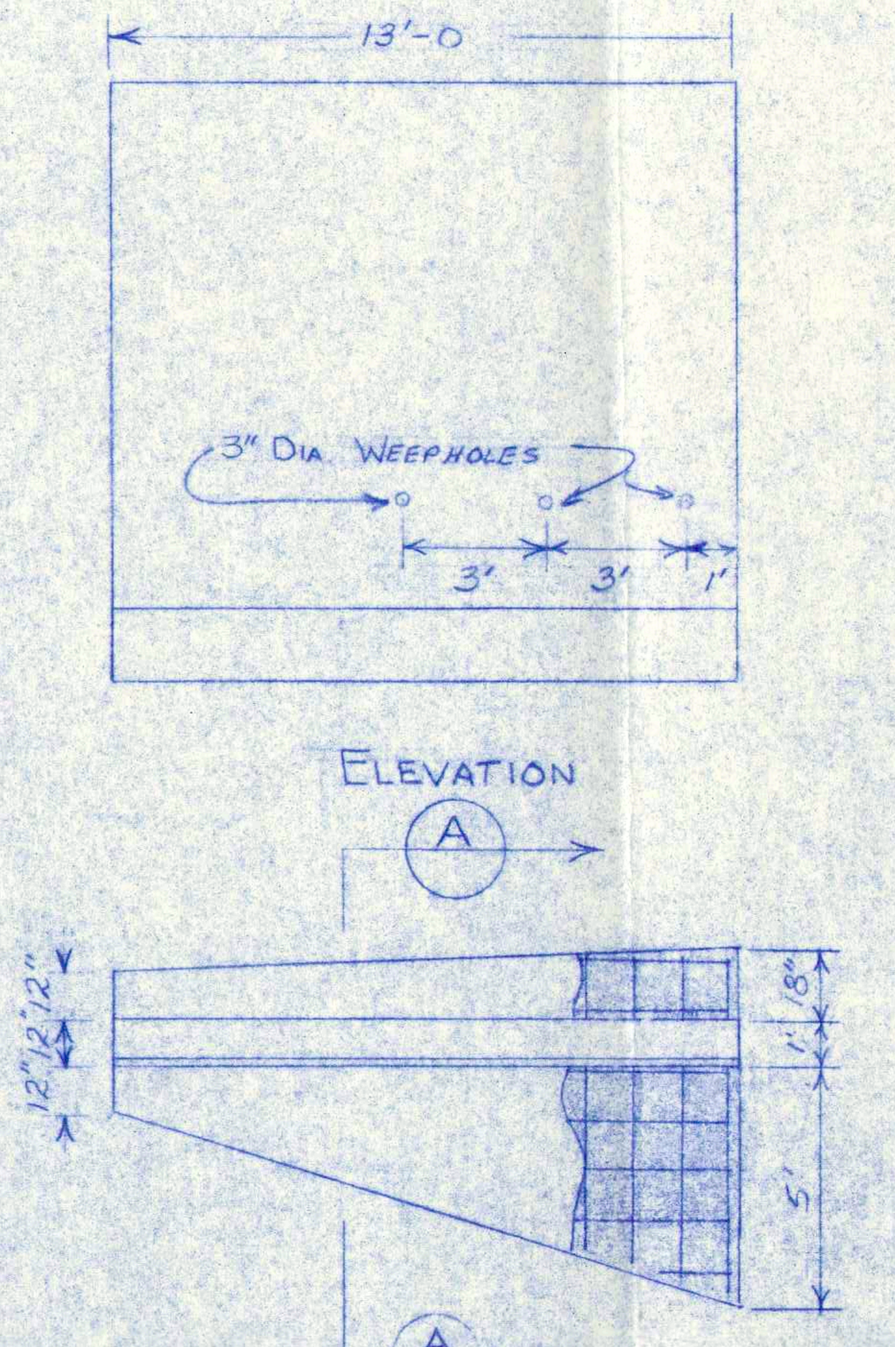
APPROVED
IDAHO DEPARTMENT OF WATER RESOURCES
3/20/78
DATE ADMINISTRATOR, RESOURCES ADMINISTRATION

IDAHO DEPARTMENT OF WATER RESOURCES

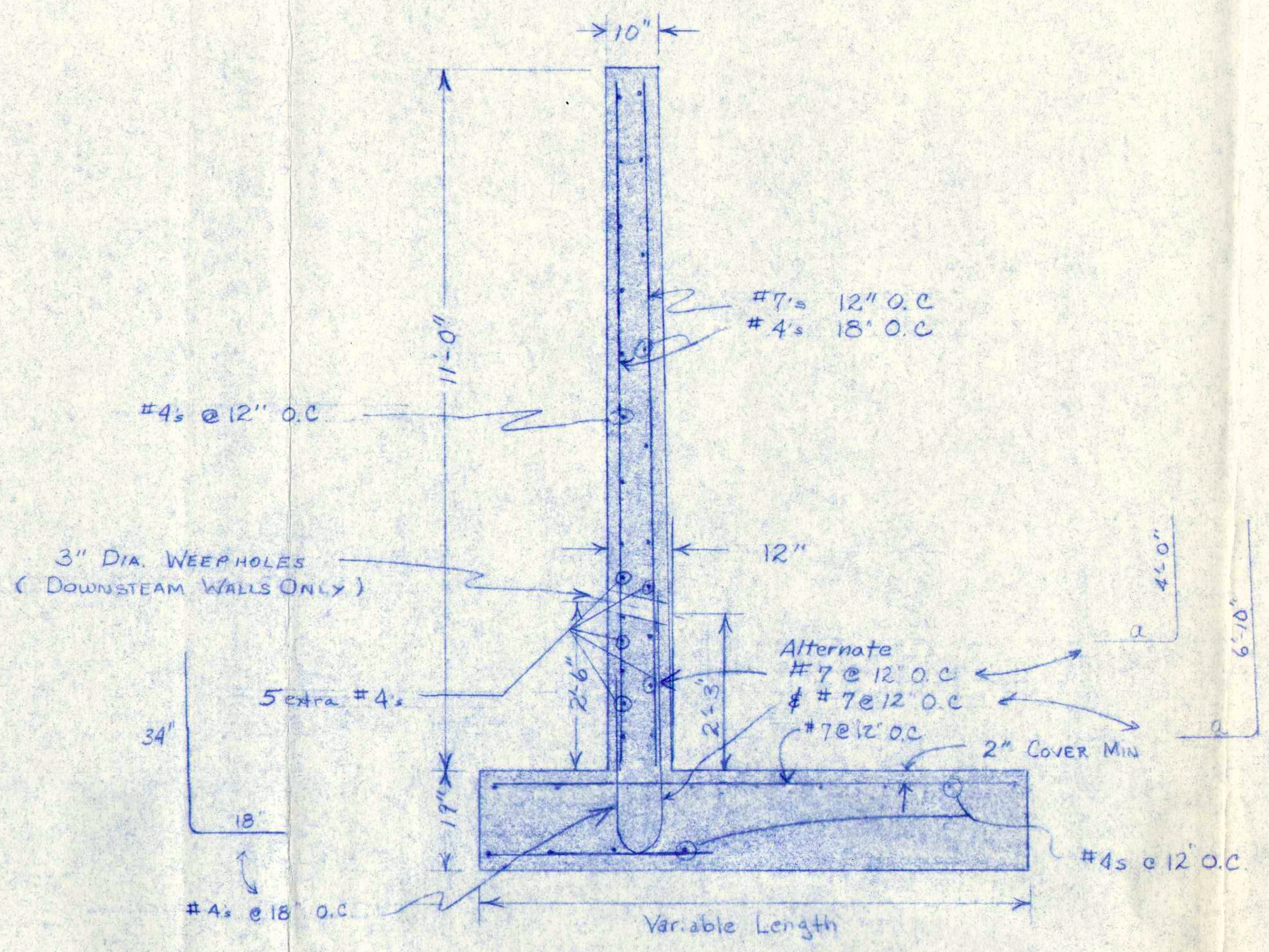
SCALE: as shown	APPROVED BY:	DRAWN BY PE, KA
DATE: Mar. 1978		REVISED

PRIEST LAKE OUTLET – STRUCTURAL DETAIL

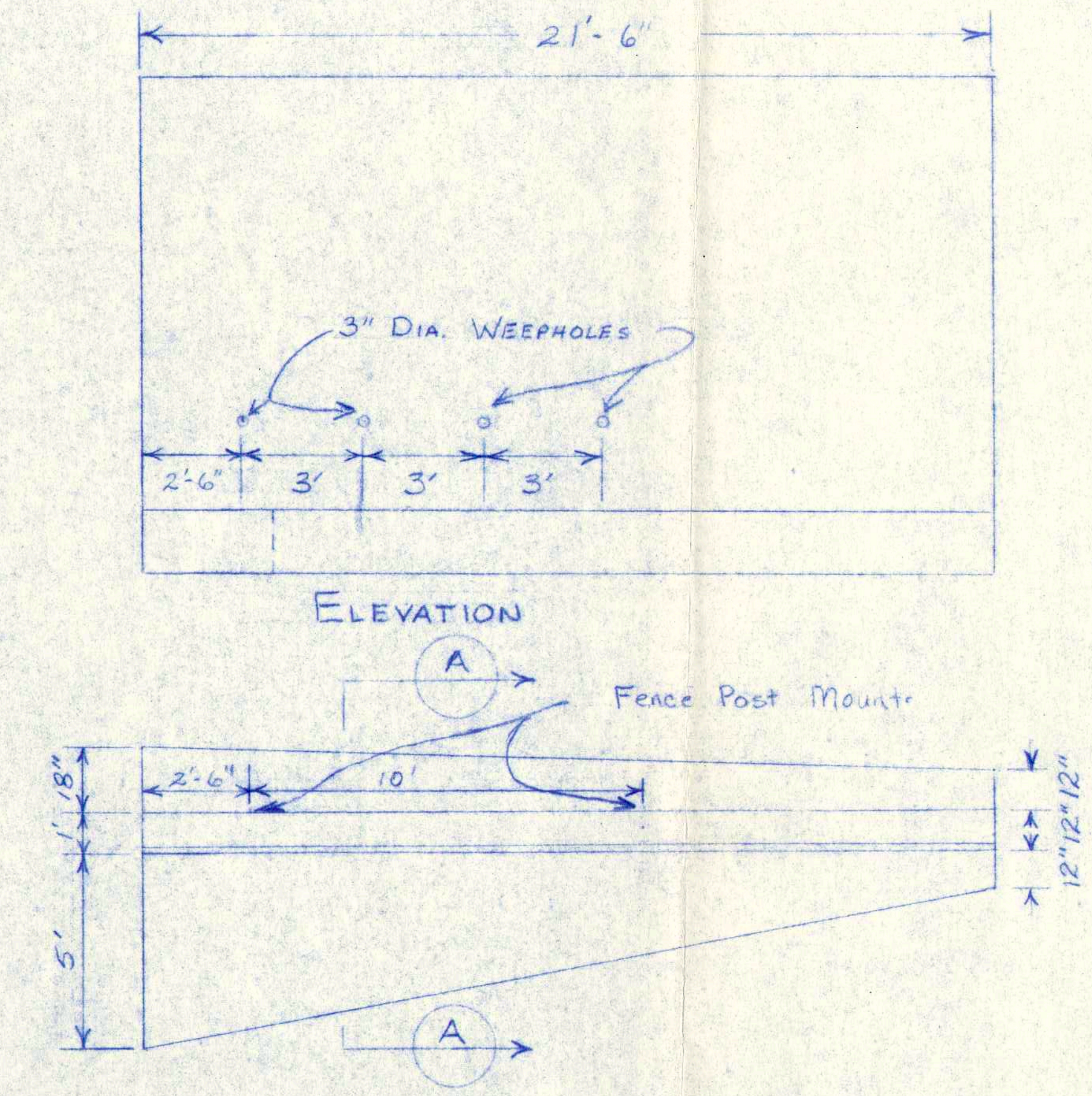
DRAWING NUMBER
97-2020-3



Left Wing Wall Downstream
Scale 1" = 4'-0"

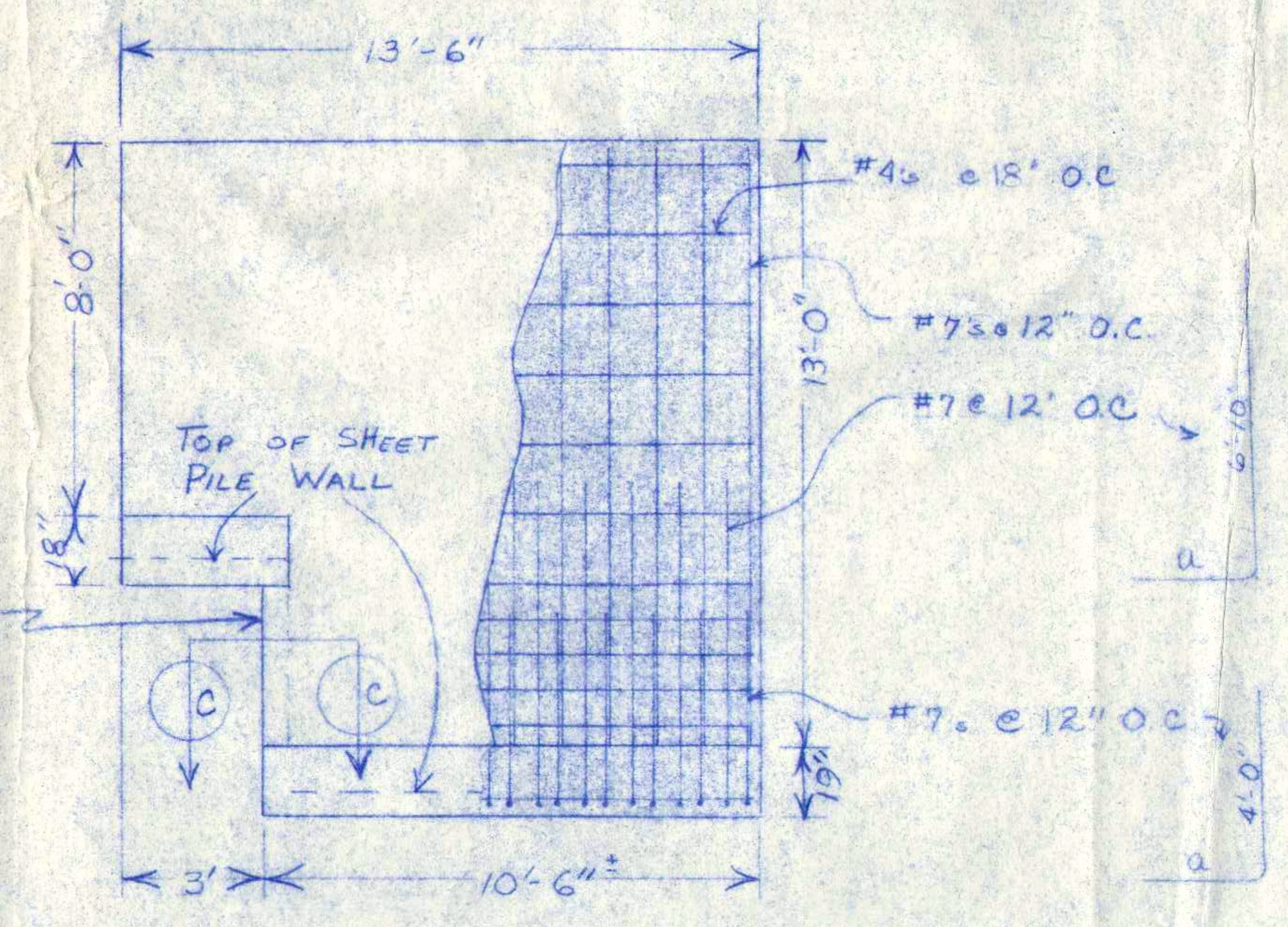


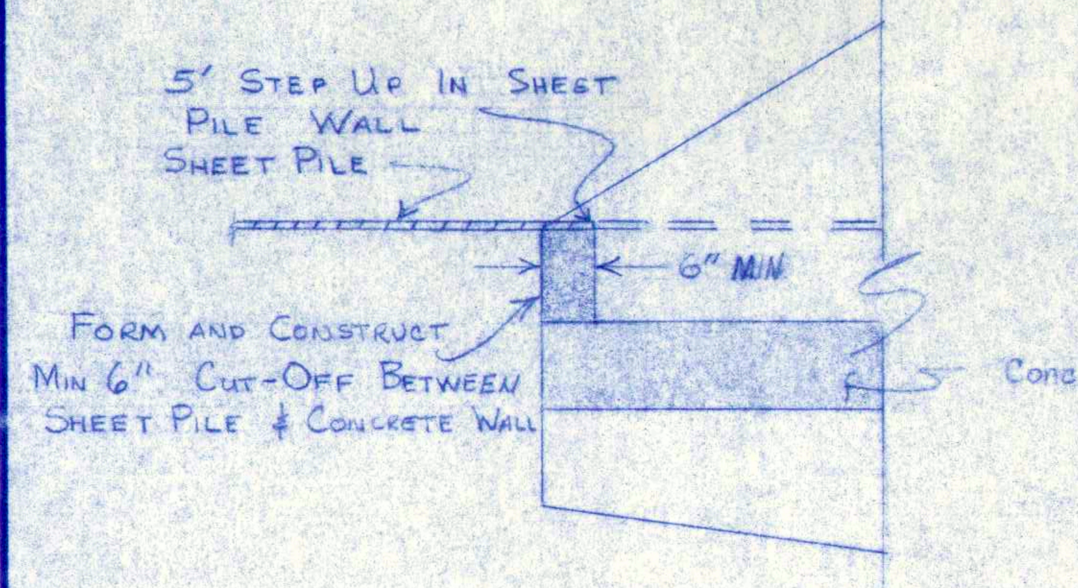
Section A-A
Typ. Downstream Wing Walls
Scale 1" = 2'-0"



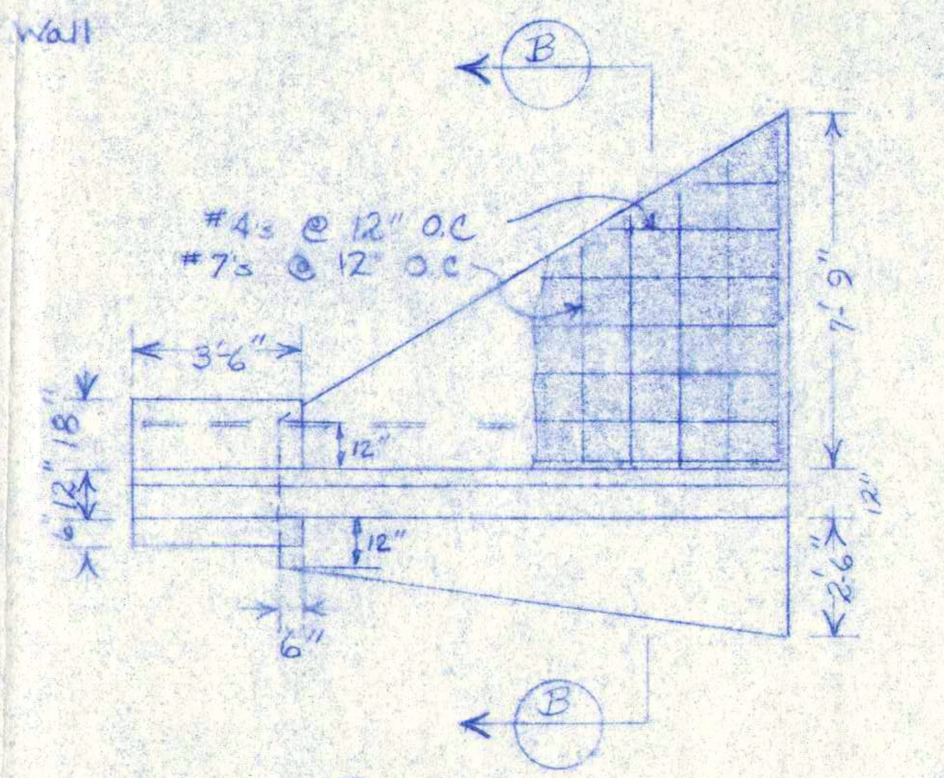
Right Wing Wall - Downstream
Scale 1" = 4'-0"

NOTE:
Dimension "a" varies with length of wing wall base

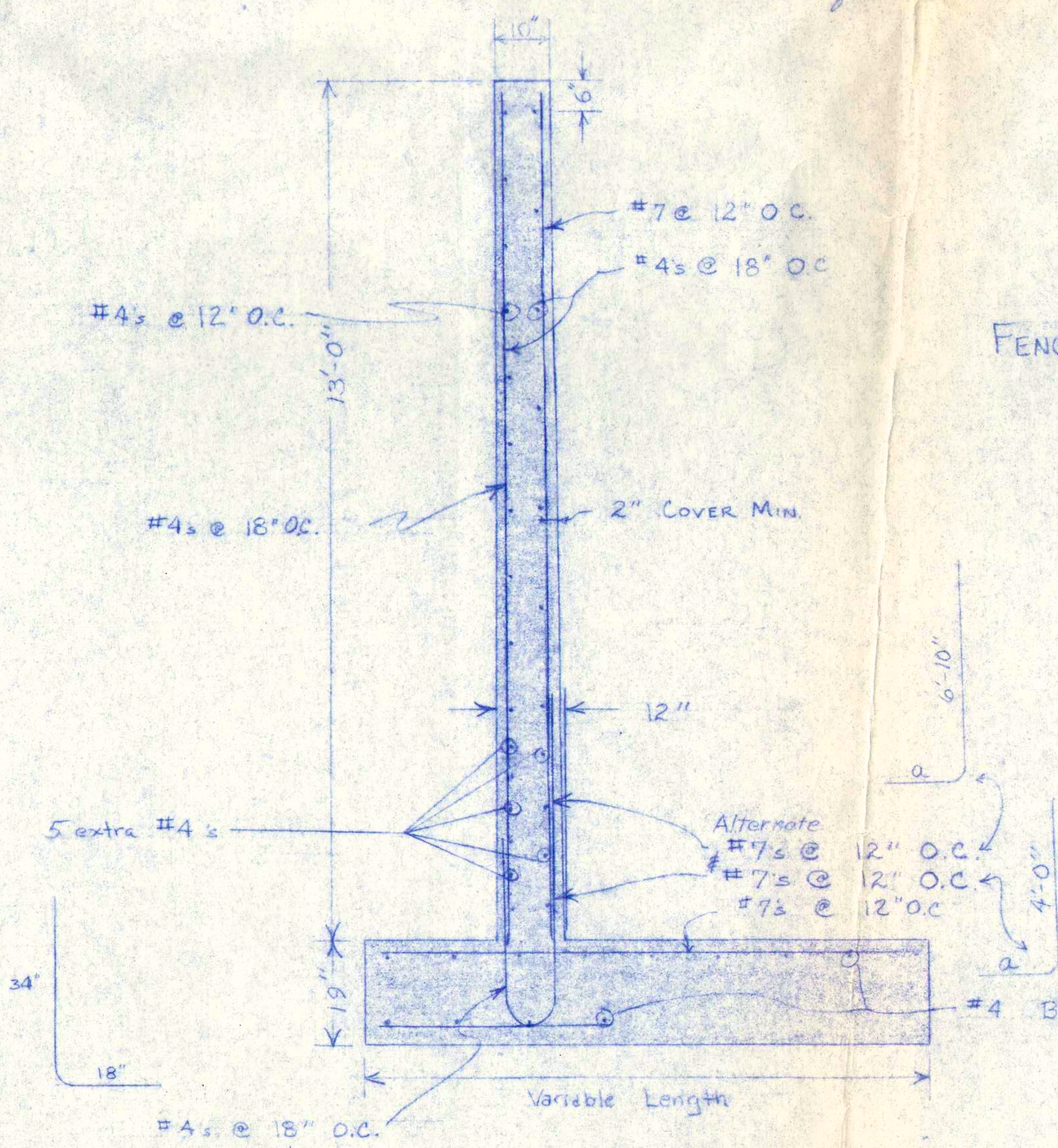




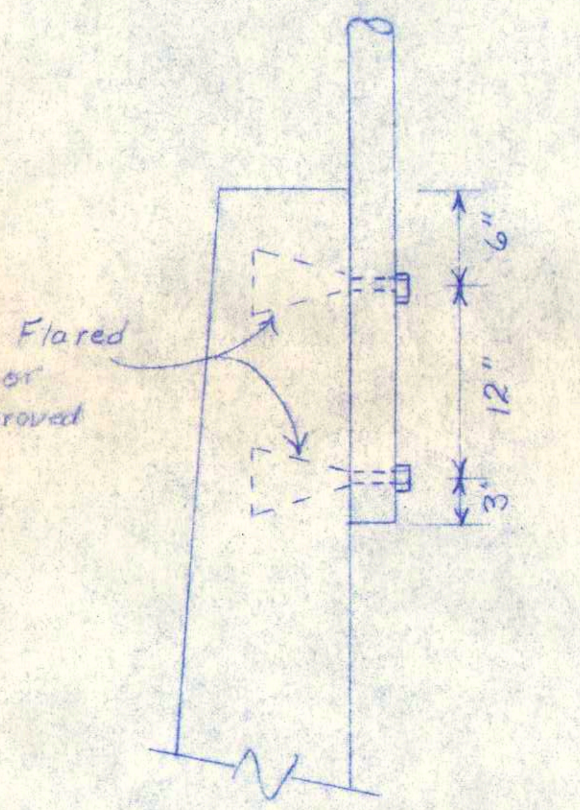
Section C-C
Scale 1" = 2'-0"



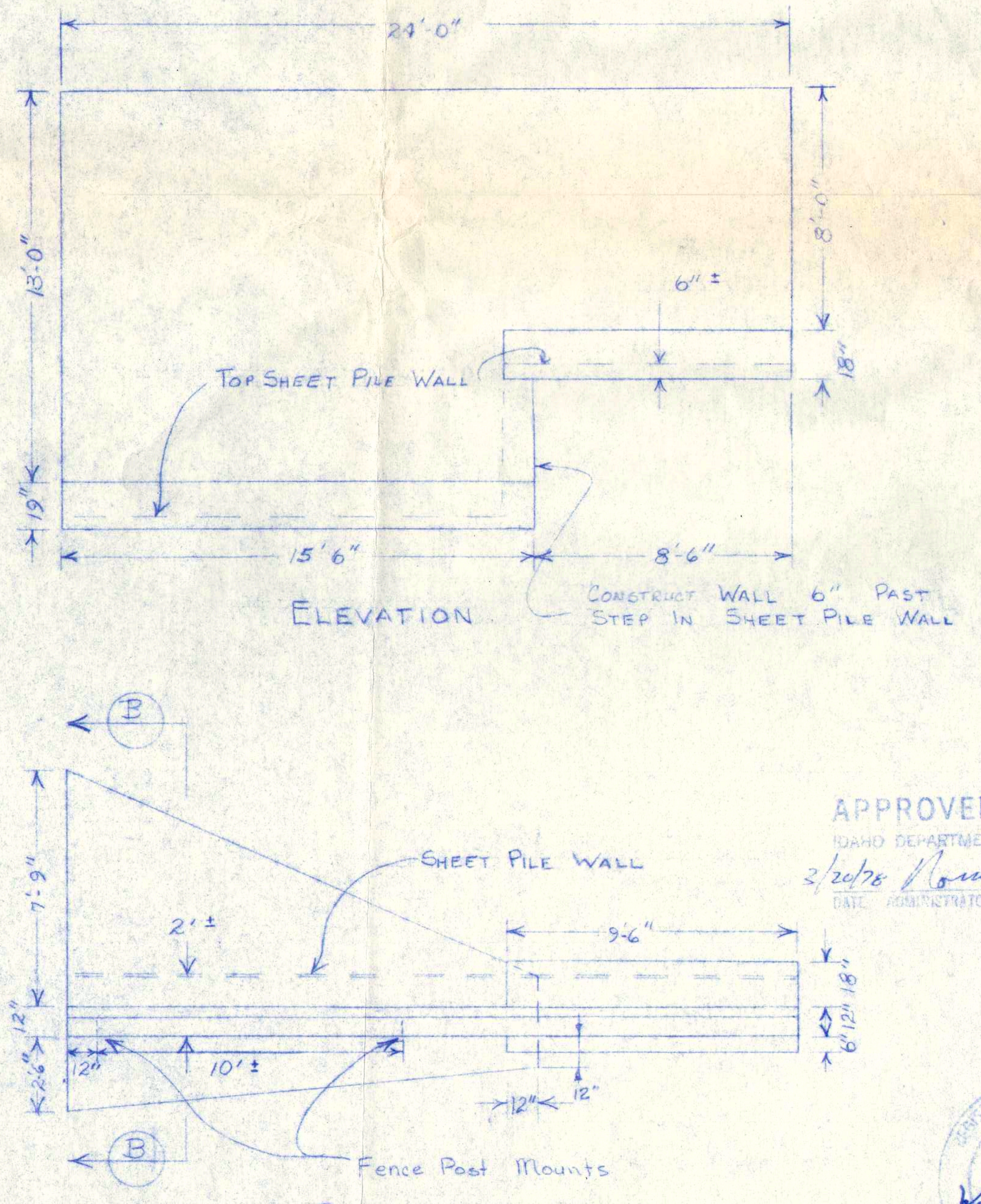
Left Wing Wall - Upstream
Scale 1' = 4'-0"



Section B-B
Scale 1" = 2'-0"



Fence Mount Detail
Scale 1" = 1'



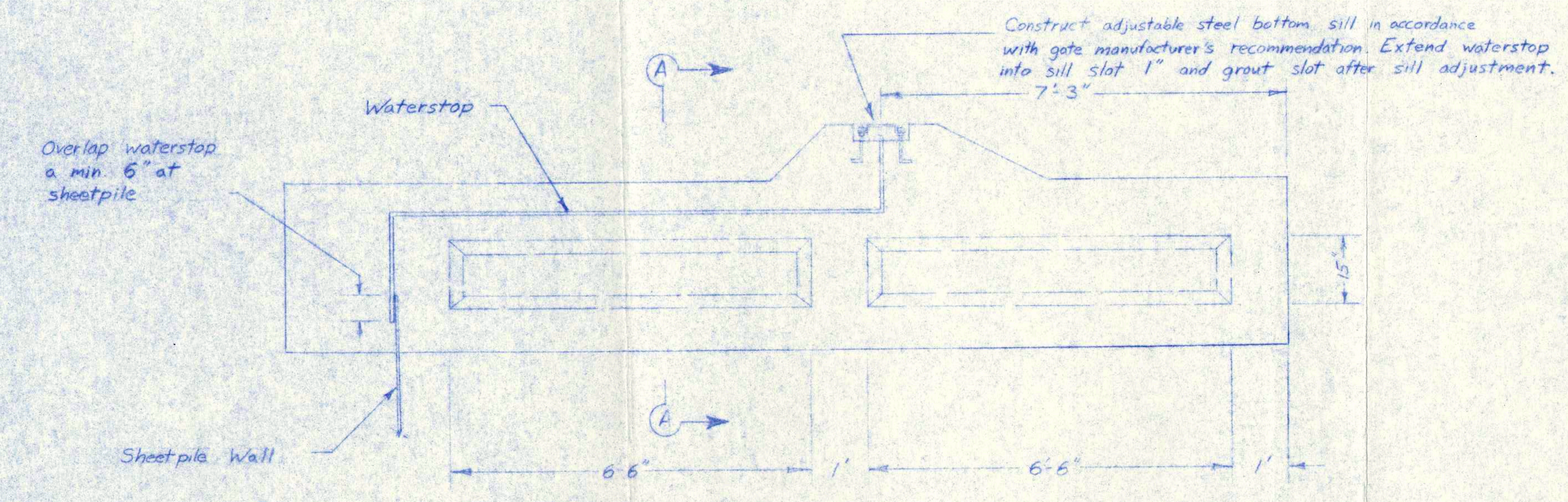
Right Wing Wall - Upstream
Scale 1" = 4'-0"

APPROVED
IDAHO DEPARTMENT OF WATER RESOURCES
3/20/78 Norman C Young
DATE ADMINISTRATOR, RESOURCES ADMIN. DIVISION

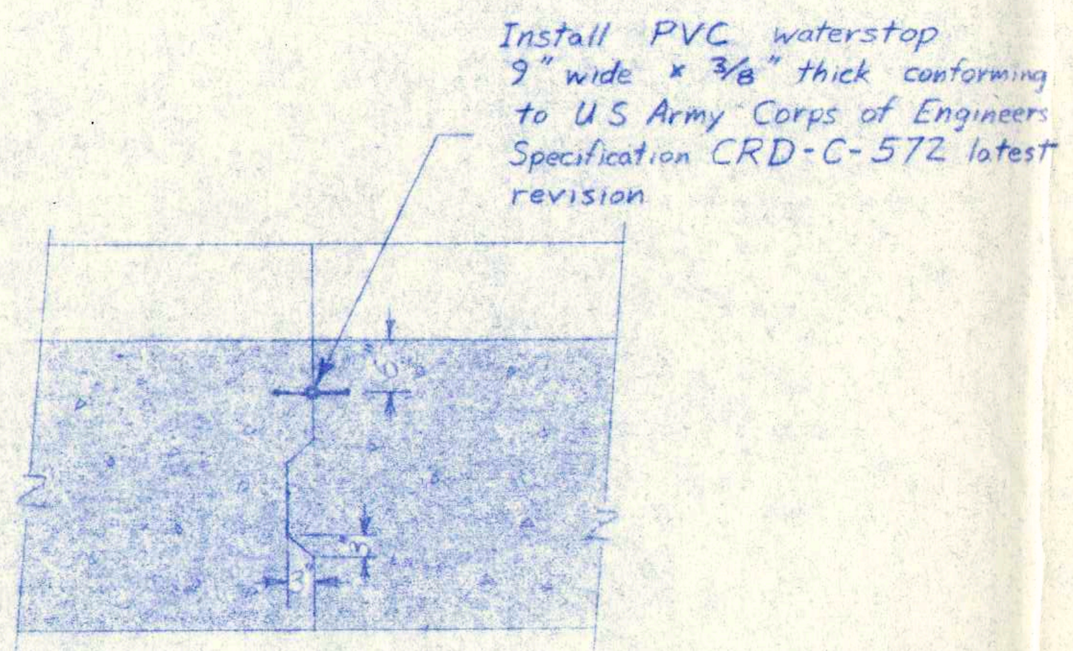
REGISTERED PROFESSIONAL ENGINEER
17-3-78
2811
William R Gossett
STATE OF IDAHO
WILLIAM R. GOSSETT

NOTE:
Construct Expansion Joint at each contact between wingwalls and dam end wall.
See Typical Detail on Sheet 5.

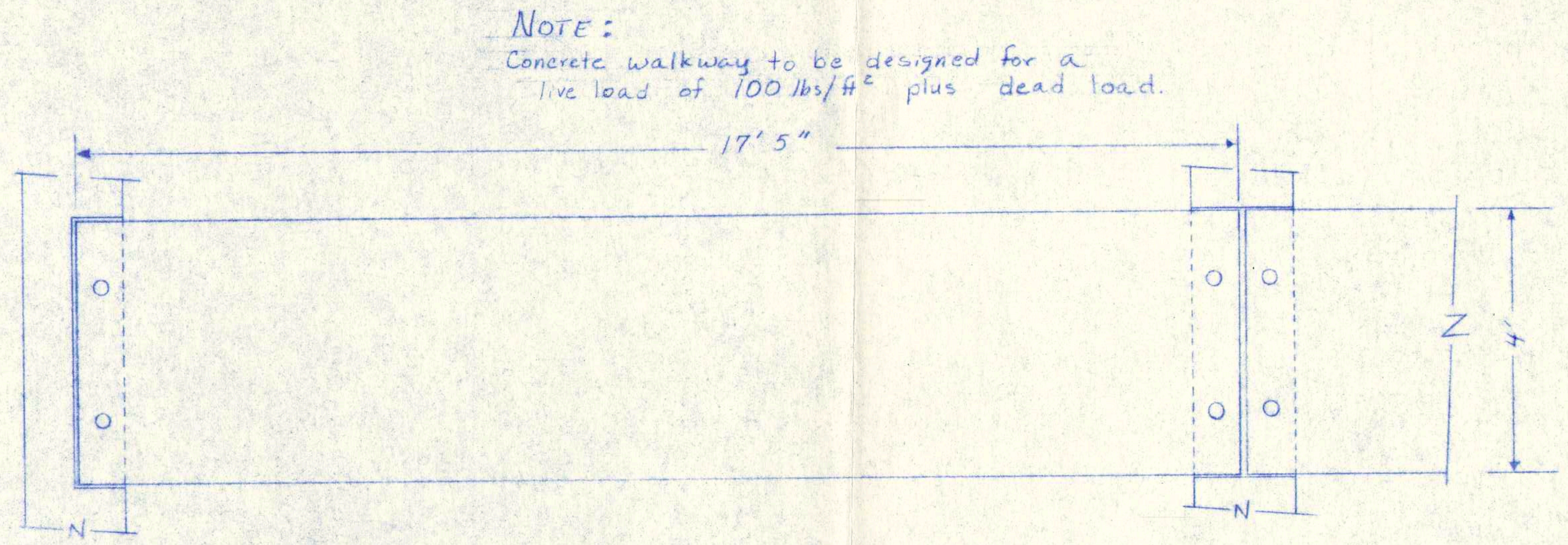
IDAHO DEPARTMENT OF WATER RESOURCES		
PRIEST LAKE OUTLET — WINGWALLS		
Drawn by: J.B. Approved: W. Gossett		
SCALE as shown		SHEET No.
DATE 17 Mar 78		97-2020 4



CONTRACTION JOINT DETAIL
Scale: 1" = 2'-0"

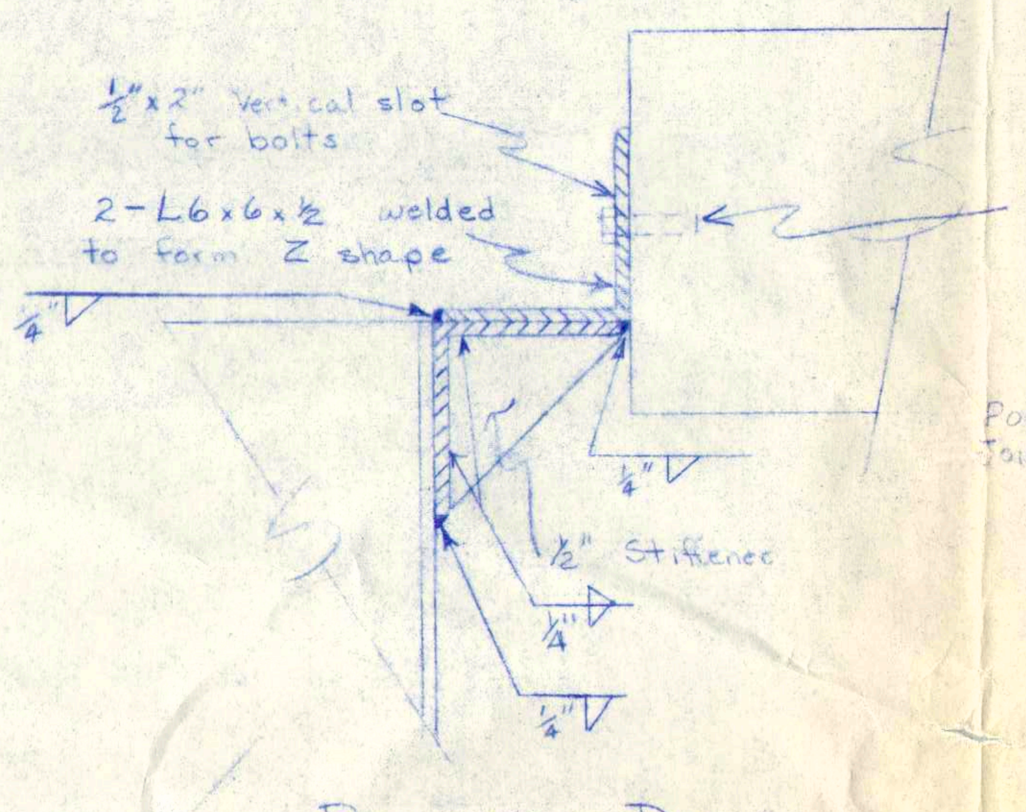


SECTION A-A

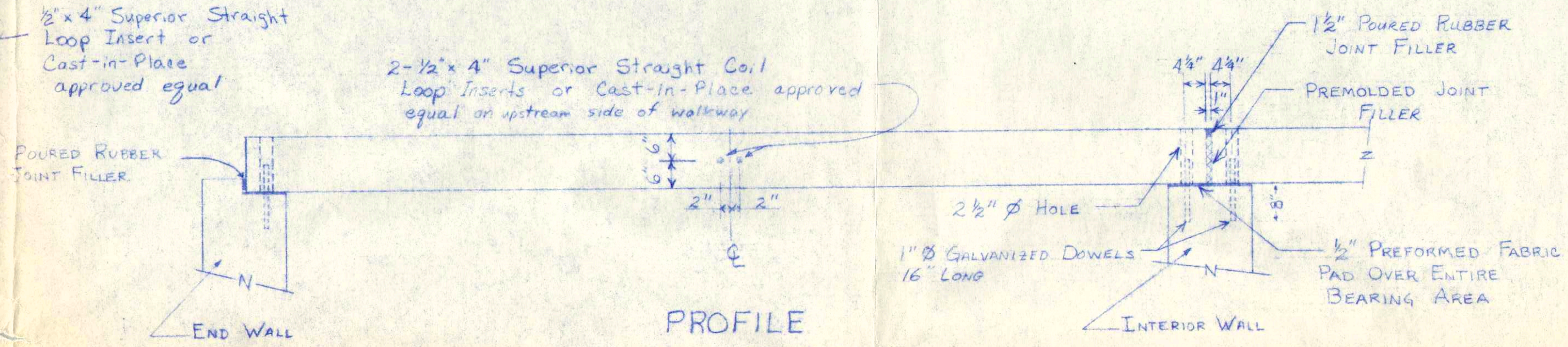


PLAN
SCALE 1" = 2'-0"

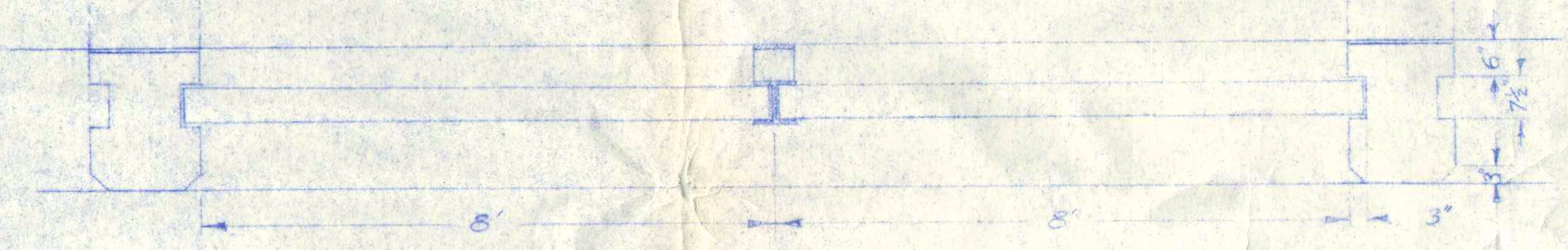
BOLT INSERTS OR WELD PLATES SHALL BE INSTALLED ALONG BOTH SIDES TO ACCOMODATE HANDRAILING. HANDRAIL DESIGN SHALL BE APPROVED BY OWNER PRIOR TO CONSTRUCTION.



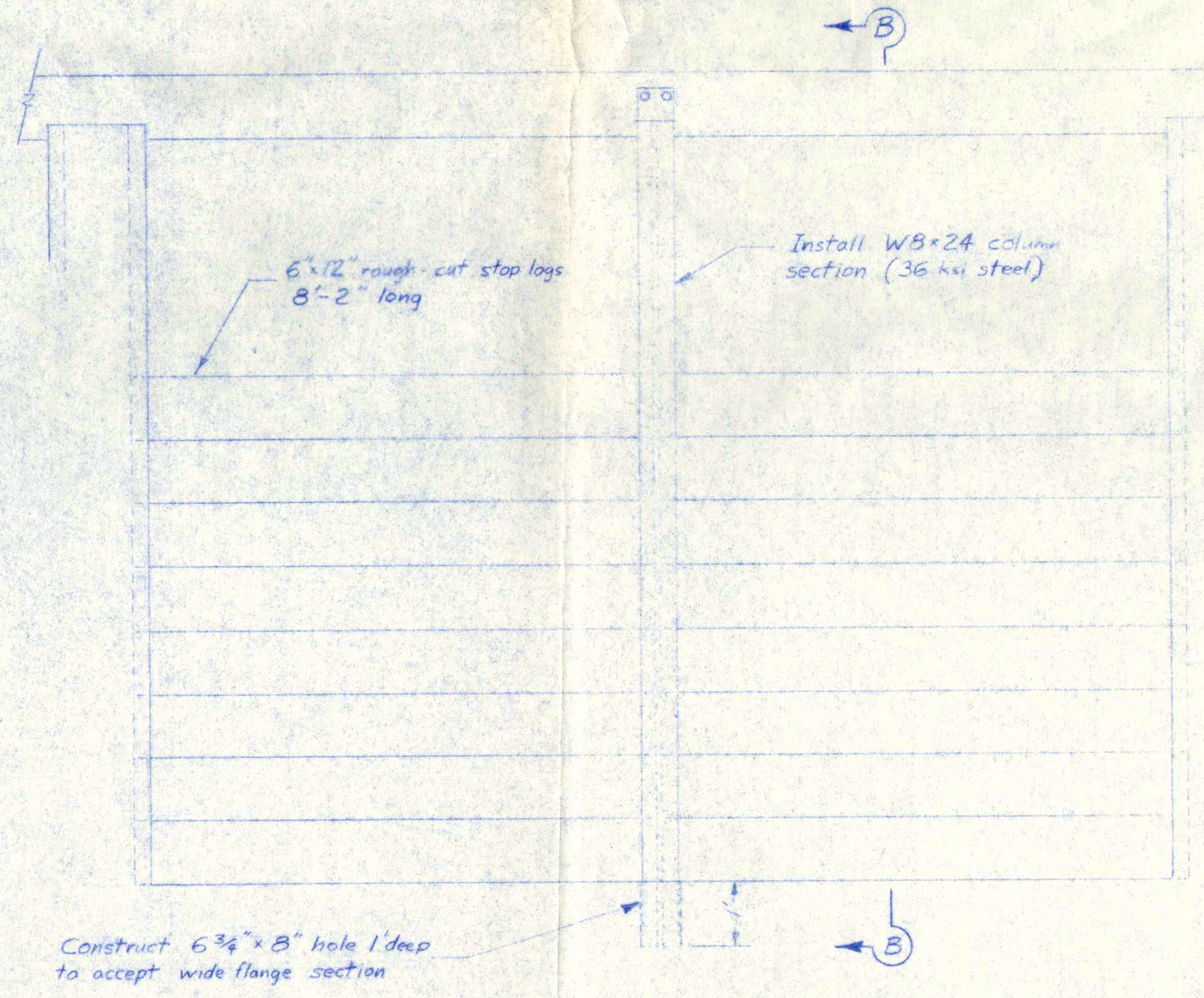
BRACKET DETAIL
SCALE 1" = 1'



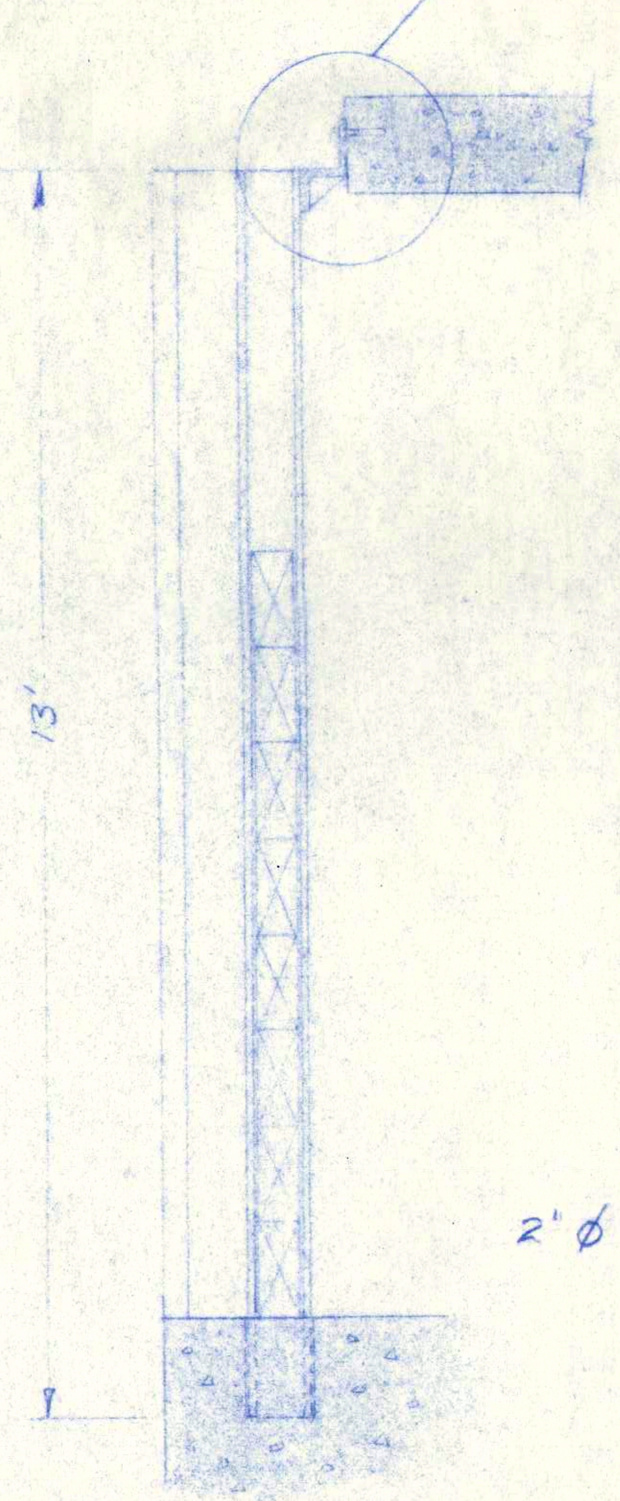
PROFILE
SCALE 1" = 2'-0"



PLAN

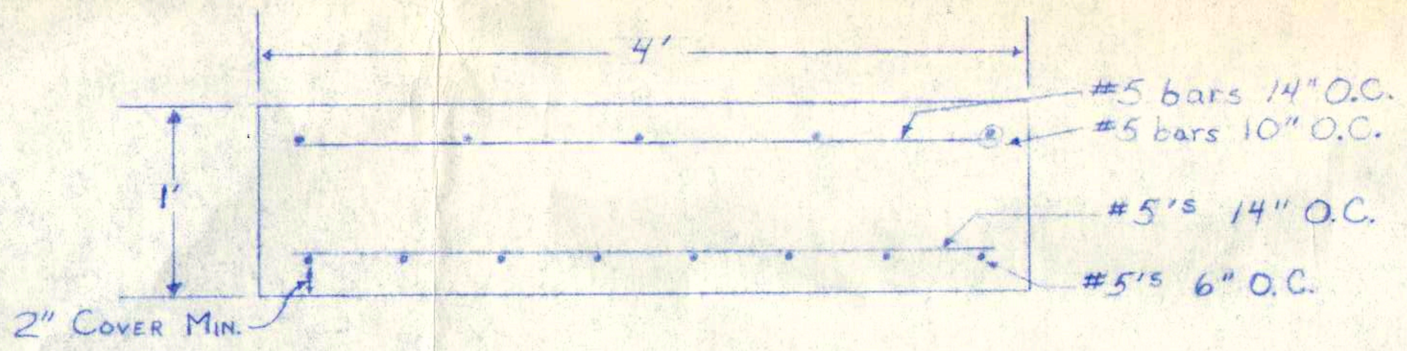


PROFILE



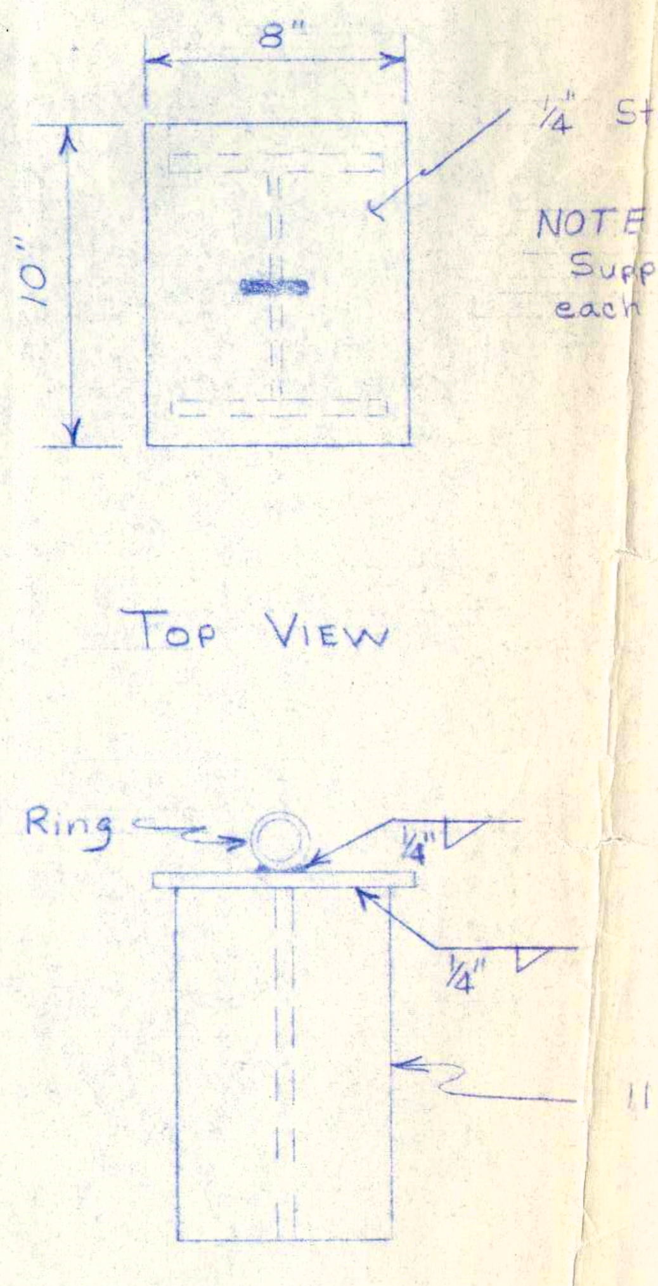
SECTION B-B

STOP LOG SECTION DETAIL
Scale: 1" = 2'-0"



SECTION C-C
SCALE 1" = 1'-0"

PRECAST CONCRETE WALKWAY

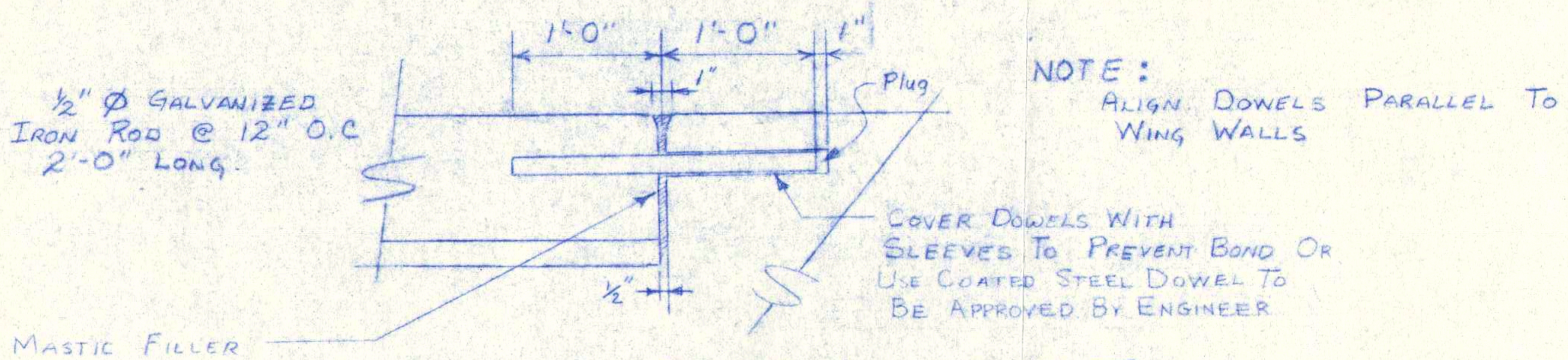


¼" Steel plate

NOTE:
Supply one plug for each gate bay.

11" Section W 8x24

PLUG DETAIL
SCALE 1" = 1'



NOTE:
Align Dowels Parallel To Wing Walls

TYP. EXPANSION JOINT
NO SCALE

APPROVED
IDAHO DEPARTMENT OF WATER RESOURCES
3/20/78 Norman C. Young
DATE ADMINISTRATOR, RESOURCES ADMIN. DIVISION

17-3-78
William R

IDAHO DEPARTMENT OF WATER RESOURCES
PRIEST LAKE OUTLET — MISC. DETAILS
DRAWN BY: W.G., K.H. APPROVED: W.
SCALE as shown
DATE 17 Mar 1978
SHEET NO. 97-2020 5

IDAHO BLUEPRINT & SUPPLY CO., INC., BOISE, IDAHO

C.3 Outlet Dam Structure Assessment Summary

MOTT
MACDONALD

Priest Lake Water Management Study

Outlet Structure Assessment – 9/25/17

IDAHO WATER RESOURCE BOARD

Final

Assessment Purpose & Criteria

1 Evaluate Pool Raise effect on Structure

2 Determine repairs or upgrades needed to accommodate a pool raise

1 IDWR & USBR Standards

2 Basis of Analysis Memorandum dated June 16, 2017 (Section 19.Outlet Dam)

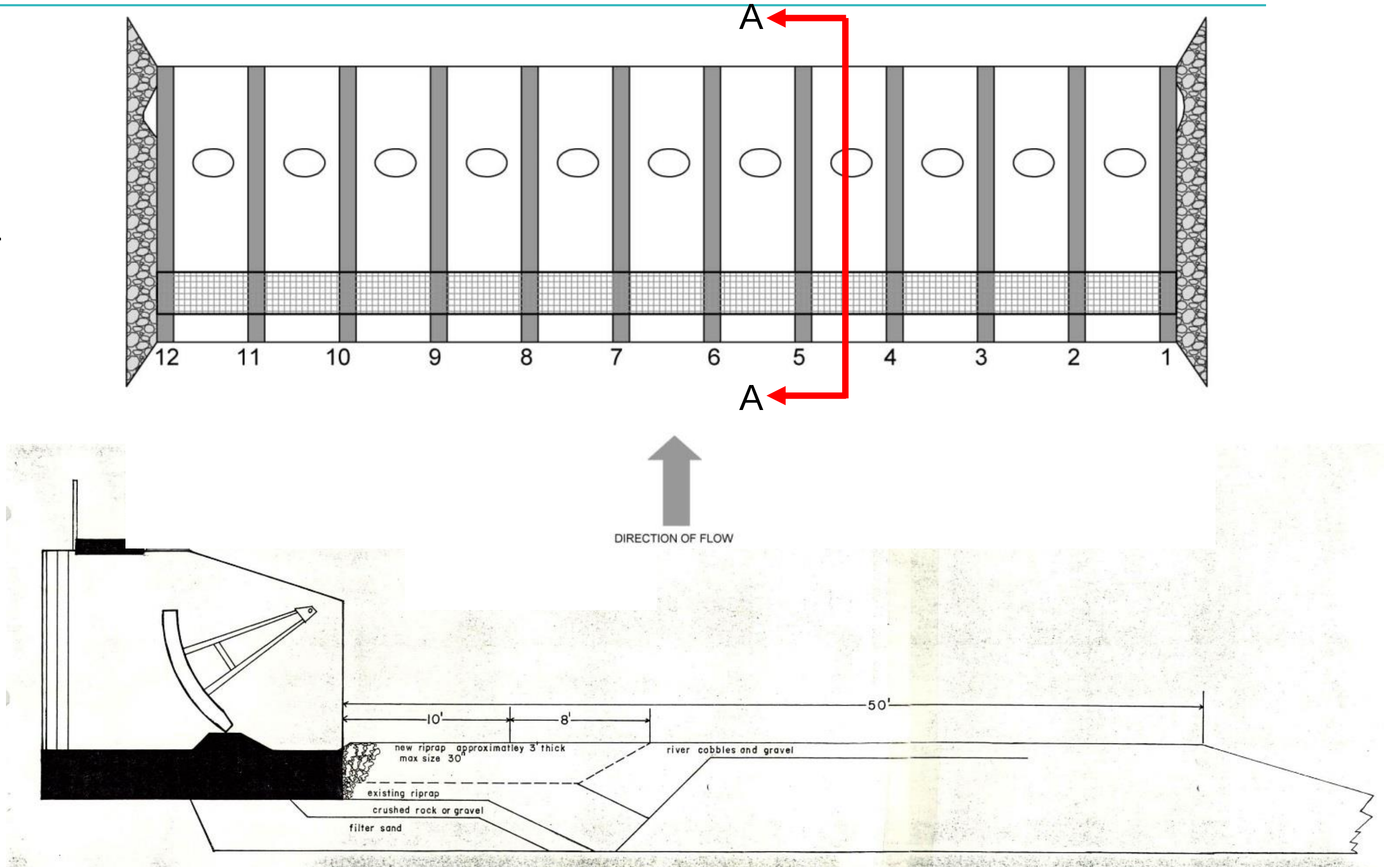
OUTLET STRUCTURE EXISTING (May 2017 High Flow)

OUTLET STRUCTURE EXISTING (Sept 2017 Low Flow)

Outlet Structure – Background

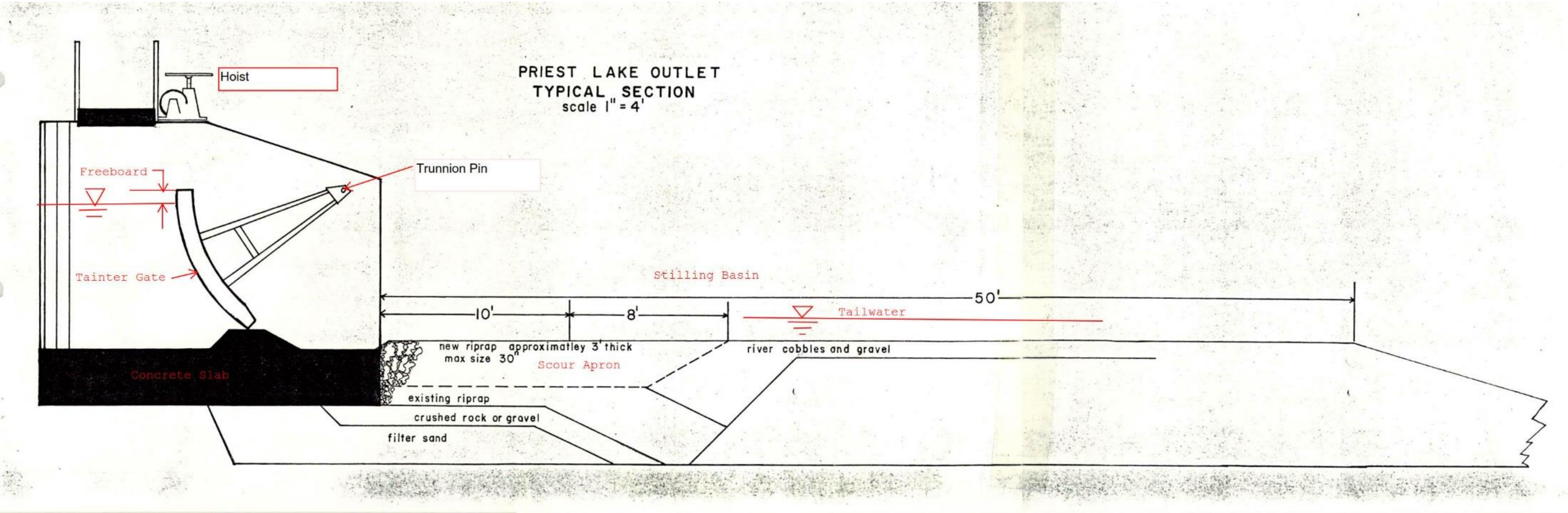
- Constructed in 1978
- Radial gates manually operated on 11 equally sized spillway bays
- Repairs to downstream scour protection in 1979 ~larger stone installed to increase scour protection
- Gates are 7' tall with 0.15' freeboard

- Dam size classification per IDAPA 37.03.06: Large
- Dam hazard category per IDAPA 37.03-06: Significant



SECTION A-A

OUTLET STRUCTURE EXISTING

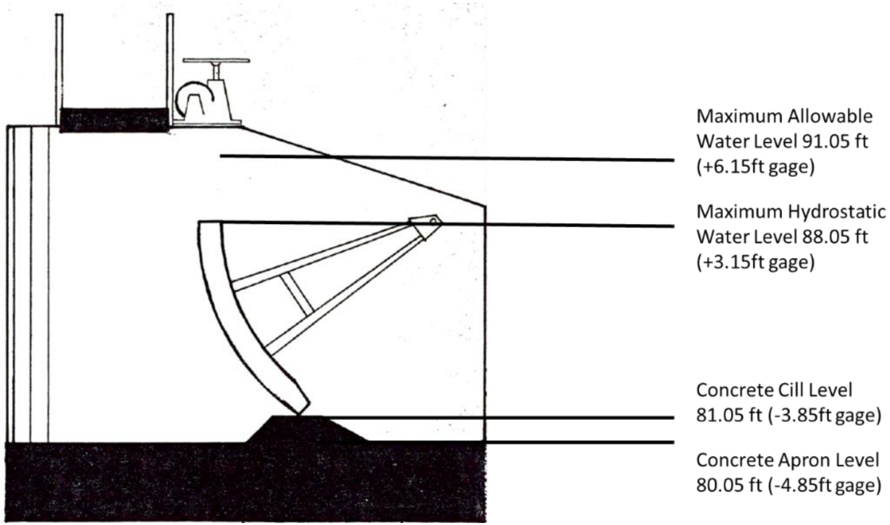


Outlet Structure Assessment

<u>Goals/Criteria</u>

- Raise Pool to
  - 3.25 ft gage
  - 3.50 ft gage
- Provide larger tolerance on vertical operating range; ~0.15' in lieu of 0.05'
- Freeboard = min 3"
- Reduce risk for operator error
- Improve gate operations for more effective water management
- Reduce risk of erosion of d/s scour protection

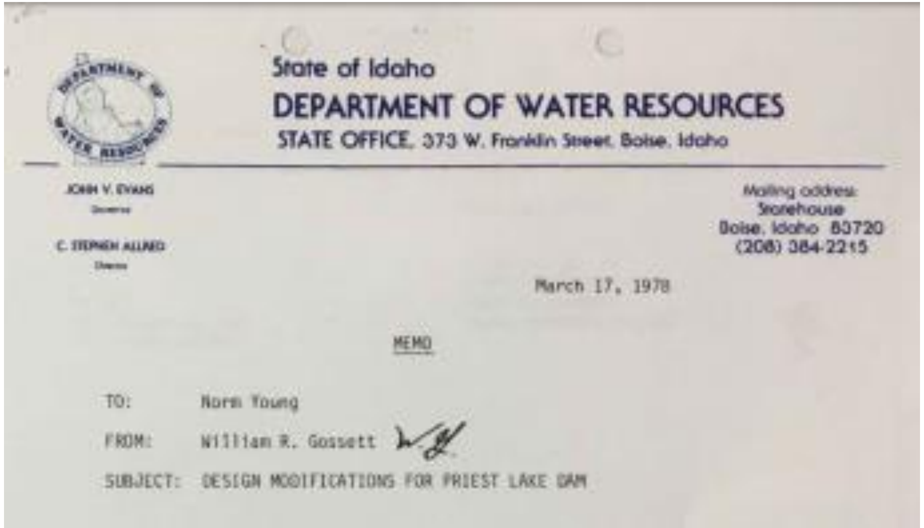
<u>Existing Conditions</u>



OUTLET STRUCTURE DESIGN BACKGROUND - 1978

1978 Design Reviews (U.S. Army Corps of Engineers & Ch2MHill)

- Concerns raised during the review of the 1978 design included the following:
  - Reliance on sheet pile cutoff wall to resist sliding
  - Concerns on stability safety factors in particular sliding
  - Lack of a stilling basin and adequacy of scour protection
  - Recommendation for further investigation of scour potential under a range of operating conditions



State of Idaho
DEPARTMENT OF WATER RESOURCES
STATE OFFICE. 373 W. Franklin Street, Boise, Idaho

JOHN V. EVANS

C. STEPHEN ALLRED

Mailing address:
Statehouse
Boise, Idaho 83720
(208) 384-2215

March 17, 1978

MEMO

TO: Norm Young

FROM: William R. Gossett

SUBJECT: DESIGN MODIFICATIONS FOR PRIEST LAKE DAM

OUTLET STRUCTURE – HYDRAULIC ASSESSMENT

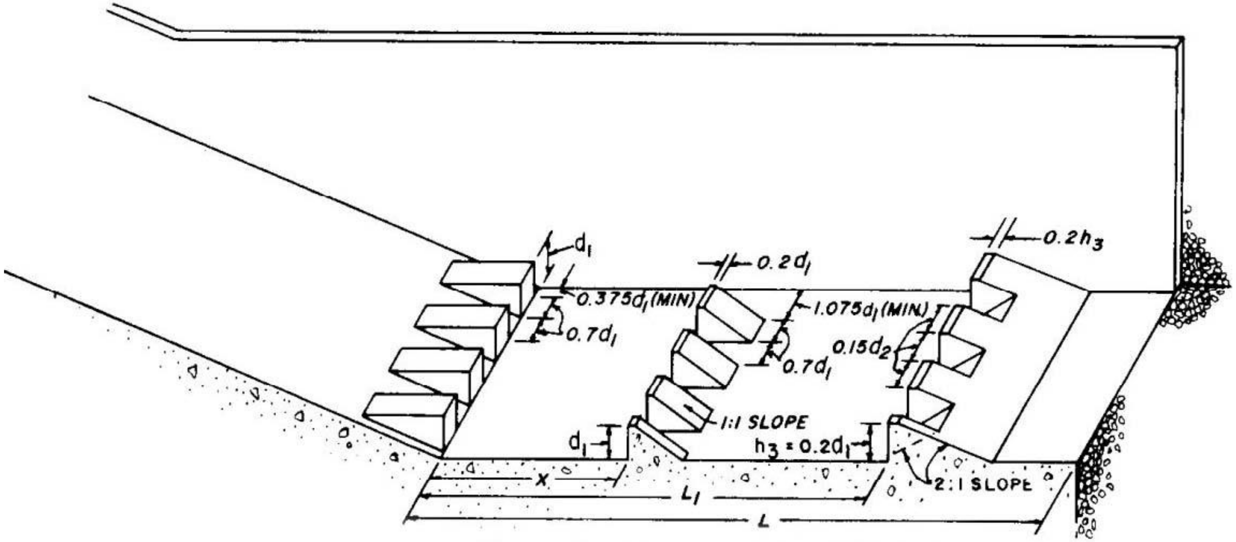
- Mott MacDonald conducted an assessment of hydraulic conditions.
- HYDRAULICS
  - Evaluated Spring, Summer and Fall Flow Conditions
  - Hydraulic jump forms beyond concrete slab and some instances beyond riprap scour protection
  - High velocities a consideration for all flow conditions; in particular is concern for high pool, low flow condition
  - Stream power (erosion potential) is increased by 9% for 6” pool raise discharge condition
  - High pool, low flow gate operation within center spillway bays better than at outer channel margins
- SCOUR ASSESSMENT
  - Stone is undersized for certain discharge flow and gate operation conditions
    - Analysis indicates D50 of 1’ to 2.5’; current D50 estimated to be 1’.
  - Larger D50 and layer thickness for riprap scour protection is needed
    - Larger stone would reduce risk of scour during future operations for spring or summer conditions
  - Length of scour apron is shorter than standards indicate
  - Concrete stilling basin is a more ideal system to mitigate scour hazard and hydraulic jump; especially in light of dependence of current system on human operations.
- Summary
  - Sensitivity to specific hydraulic conditions and gate operations
    - High flow: Variable location for hydraulic jump formation depending on gate operations and discharge.
    - High Pool, high flow, few gates open = High velocities & scour potential.
  - Stone is undersized and susceptible to scour during gate operations
  - Improved scour apron and more formalized concrete stilling basin should be considered

OUTLET STRUCTURE – Stilling Basin Improvement Discussion

- Concrete Apron
  - Assist to reduce scour potential in stilling basin and reduce reliance on operator to minimize risk. Could also benefit stability of structure (sliding resistance) if designed as such.
- Baffle Block
  - Consideration to keep hydraulic jump closer to the structure. Could reduce length of stilling basin.
- End Sill
  - Reduce risk of scour at the end of the concrete apron; deflects currents up to reduce erosive circulation currents
- Stone Apron
  - Extended beyond concrete apron as needed to reduce risk; could be used to reduce length of concrete apron.
- Cutoff Wall (Sheet pile)
  - Recommended to ensure edge of concrete apron is protected from scour; current structure does not have a cutoff wall on the downstream edge; only on upstream edge.

Scour Apron Mitigation Concepts

ALT #1 – USBR TYPE IVA STILLING BASIN



Pros:

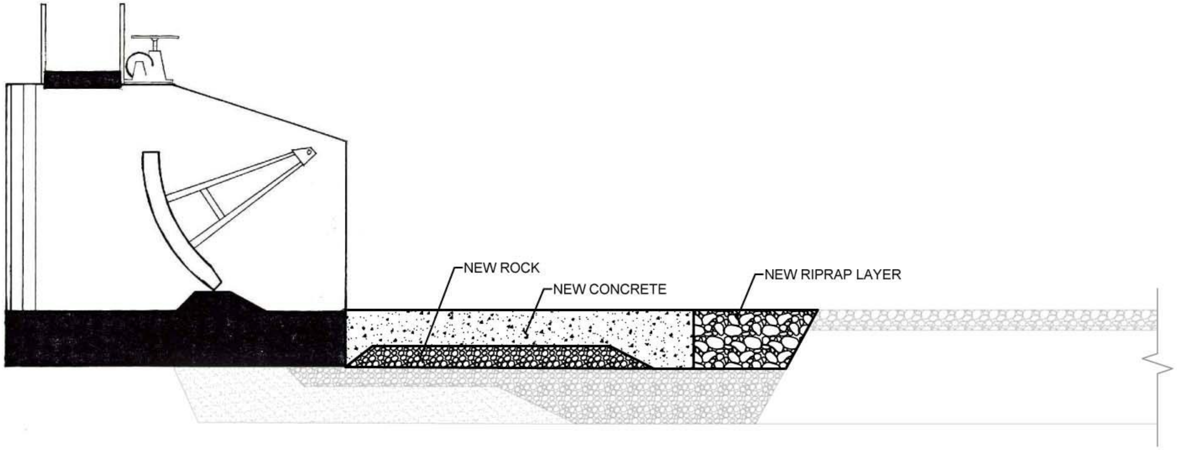
- High Performance
- Lowest Risk
- Most Robust Alternative
- Improved Dam Stability

Cons:

- High Expense
- Expansion of scour apron length

Scour Apron Mitigation Concepts

ALT #2 – EXTEND CONCRETE APRON



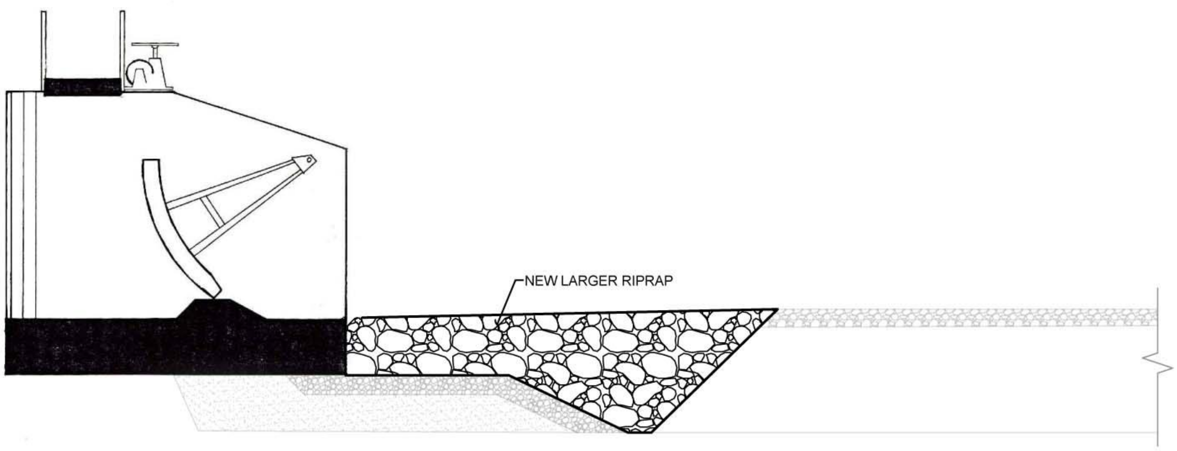
Note: Baffles, End Sill and Sheet Pile cutoff wall not shown.

Pros:
- Improved Stilling Basin Operations
- Lower Risk
- Potential for improvement to dam stability

Cons:
- Expense
- Less effective than Alt 1

Scour Apron Mitigation Concepts

ALT #3 – NEW LARGER RIPRAP



Pros:

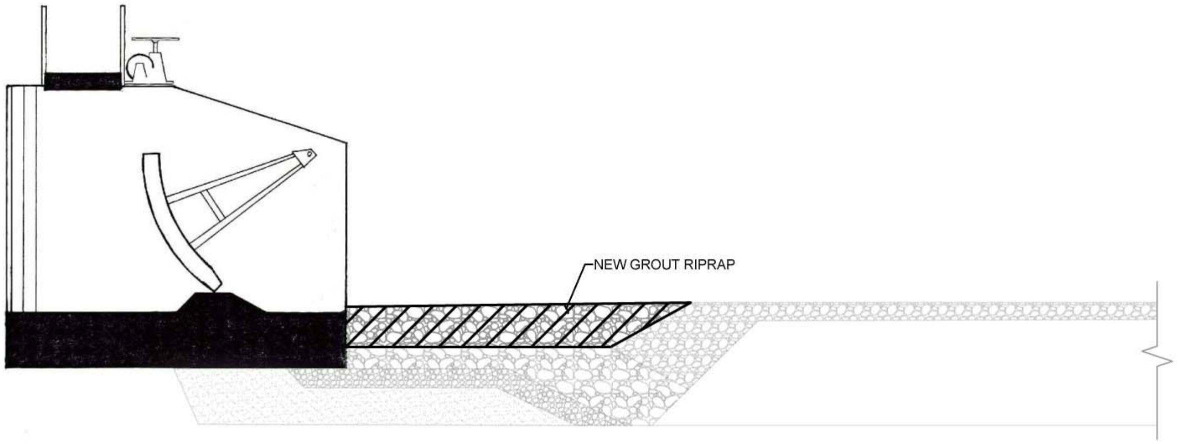
- Improved Stilling Basin Operations
- Less time for construction

Cons:

- Deeper excavation required
- Smaller improvement to dam stability

Scour Apron Mitigation Concepts

ALT #4 – GROUT EXISTING RIPRAP

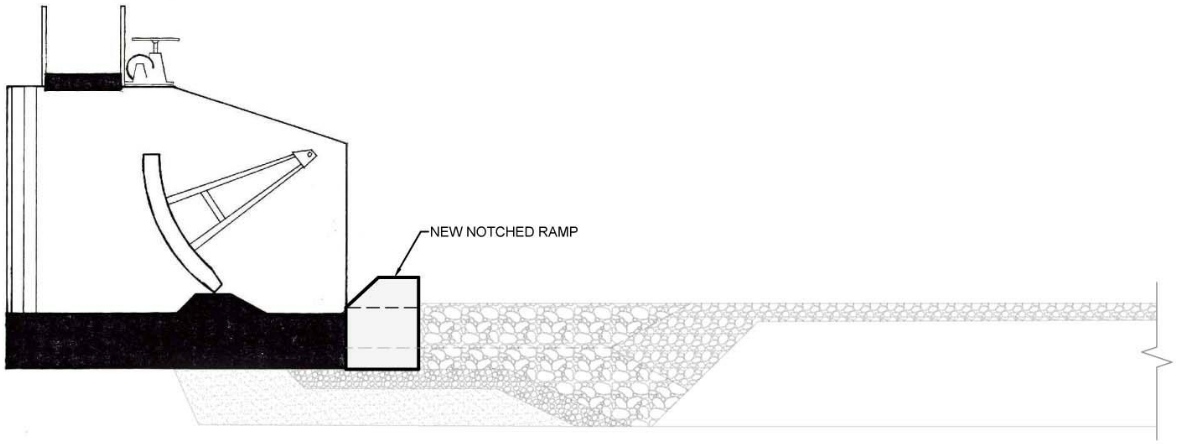


Pros:
- Lower Cost

Cons:
- Less effective
- Higher risk
- Constructability
- No improvement to dam stability

Scour Apron Mitigation Concepts

ALT #5 – NOTCHED RAMP

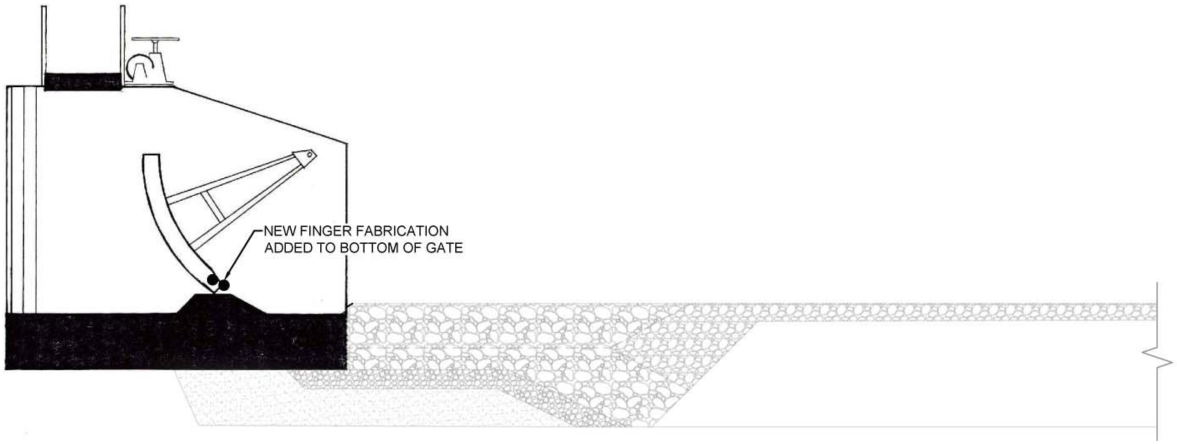


Pros:
- Lower cost
- Less impact on stream

Cons:
- CFD and/or physical modeling required to analyze effectiveness
- Less effective
- Higher risk (still reliant on existing stone) or combined with new stone
- Minimal improvement to dam stability

Scour Apron Mitigation Concepts

ALT #6 – GATE RETROFIT

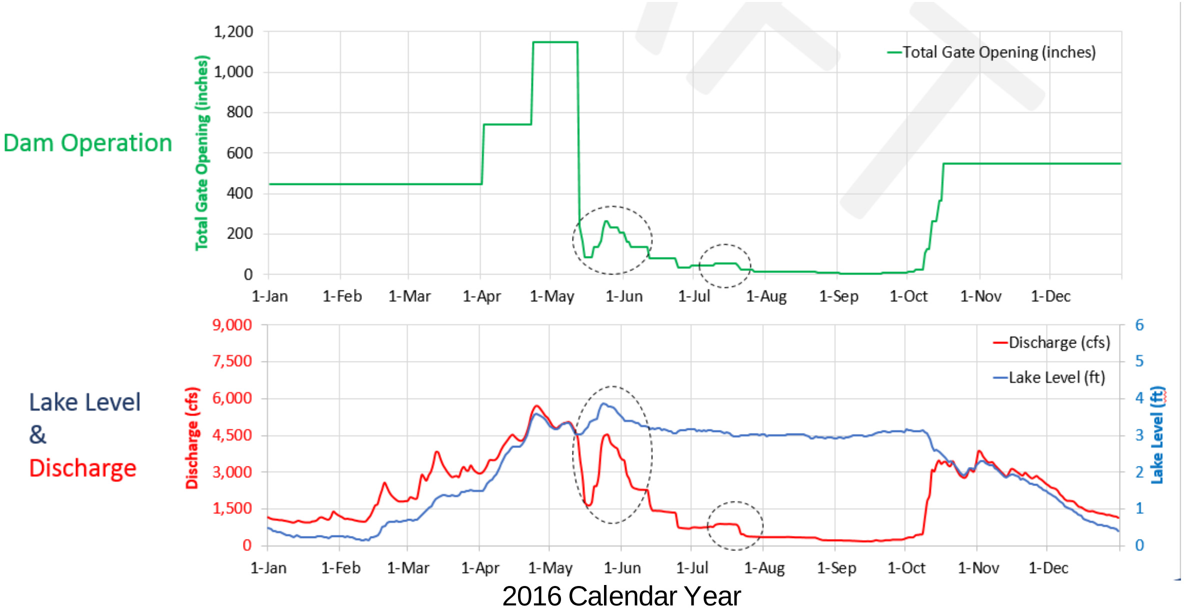


Pros:
- Lower Cost
- No stream modifications

Cons:
- CFD modelling will be required to analyze effectiveness
- Higher risk than other alternatives
- No improvement to dam stability
- Prior experience and performance not known

OUTLET STRUCTURE – GATE OPERATIONS

- Winter. Gates fully open
- Spring. Gate opened and managed to achieve 3.0’ gage by July 1.
- Summer. Small number of gates used to manage pool and discharge.
- Fall. Opened in Oct to release storage between early Oct and Nov 1.



Operational Modifications Assessment

Operational Strategy

- Summer high pool with larger flows. Operate larger number of gates to reduce erosion risk potential (high velocities).
- When smaller number of gates are required to be operated, utilize gates near center of structure.

Powered Operation & Automation Considerations

- Concepts
  - Retrofit with motor and gearbox to existing or a modified drive with starter panel
  - Valve Actuator – self contained unit; remote operation
- Assessment
  - Requires a housing structure, communication, power, etc…
  - Large investment required for fully automated.
  - Risks of operations still requiring onsite attention
- Alternative Concept
  - Provide power operation with remote monitoring but not remote operation could be an alternative.
  - Focus improvements on more refined onsite operation to improve real time operations of the dam
  - Improve discharge and pool measurement and monitoring system for gate operations and to improve rating curve
- Summary
  - Power operation could increase operational flexibility but likely not a requirement for pool raise; could be a good improvement to supplement other improvements
  - Automation is not a requirement for the pool raise project; could be expensive to implement as well.

Gate Modifications Assessment

Radial Gate & Trunnion

- Slight alter in angle of resultant load and increase in load magnitude
- Slight reduction in factor of safety
- Additional structural analysis in next phase to quantify reduction
- Likely doesn’t require substantial modification for smaller increase in pool

Gate Extension

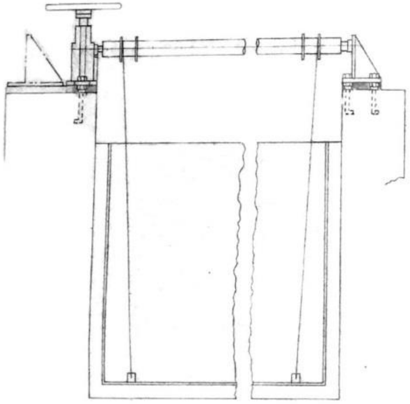
- Required for pool raise as current freeboard is only 2”.
- Extension likely to be stiffened steel plate with isolation of new steel and exist iron gate
- Freeboard. Freeboard for new pool level of at least 3” to 4” should be considered.
- Increase in gate height likely 5” to 8” including increase in freeboard.

Hoist

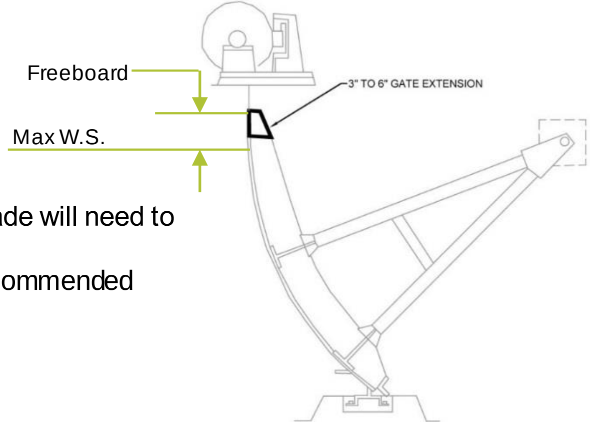
- Added weight of gate extension (<1,000 lbs)
- Frictional force increase from added weight
- Hoist lift load increase ~ 12%
- Slight reduction in factor of safety; more detailed analysis in next phase and review of capacity of system.
- Not significant increase in load and within safety factor.

Summary

- Modification of gates is feasible to accommodate a 3” to 6” pool raise; exact extent of upgrade will need to refined in the next phase of project.
- Allowance for gate strengthening with gate extension (stiffener plates) in cost estimates recommended
- Gate modification should be conducted to accommodate a 6” higher pool level.



Gate Elevation

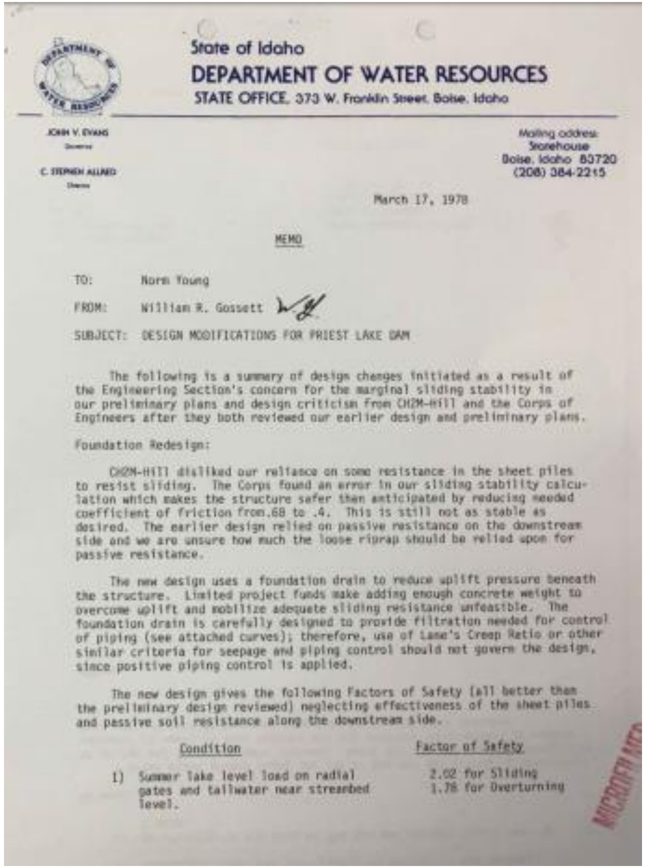


Gate Section

Dam Stability Analysis – 1978 Design/Assessment

Ch2MHill & U.S. Army Corps of Engineers Review

- STABILITY
  - Overturning
  - Sliding Resistance: Dependent upon the sheet pile wall and downstream scour apron for providing lateral resistance to achieve the required Factor of Safety.
  - Sliding & Piping – dependent on filter layer and the downstream riprap scour apron remaining in place
  - Improvements: Downstream concrete key recommended (not implemented).
  - Assessment: Recommend a key or weight of structure used to resist sliding and not sheet pile wall.
- STILLING BASIN
  - No end sill or concrete apron to control location and formation of hydraulic jump
  - Riprap may be undersized and susceptible to erosion and therefore destabilization of the dam



State of Idaho
DEPARTMENT OF WATER RESOURCES
STATE OFFICE, 373 W. Franklin Street, Boise, Idaho

JOHN V. EVANS

C. STEPHEN ALLRED

Mailing address:
Statehouse
Boise, Idaho 83720
(208) 384-2215

March 17, 1978

MEMO

TO: Norm Young

FROM: William R. Gossett

SUBJECT: DESIGN MODIFICATIONS FOR PRIEST LAKE DAM

The following is a summary of design changes initiated as a result of the Engineering Section's concern for the marginal sliding stability in our preliminary plans and design criticism from CH2M-Hill and the Corps of Engineers after they both reviewed our earlier design and preliminary plans.

Foundation Redesign:

CH2M-Hill disliked our reliance on some resistance in the sheet piles to resist sliding. The Corps found an error in our sliding stability calculation which makes the structure safer than anticipated by reducing needed coefficient of friction from.68 to .4. This is still not as stable as desired. The earlier design relied on passive resistance on the downstream side and we are unsure how much the loose riprap should be relied upon for passive resistance.

The new design uses a foundation drain to reduce uplift pressure beneath the structure. Limited project funds make adding enough concrete weight to overcome uplift and mobilize adequate sliding resistance unfeasible. The foundation drain is carefully designed to provide filtration needed for control of piping (see attached curves); therefore, use of Lane's Creep Ratio or other similar criteria for seepage and piping control should not govern the design, since positive piping control is applied.

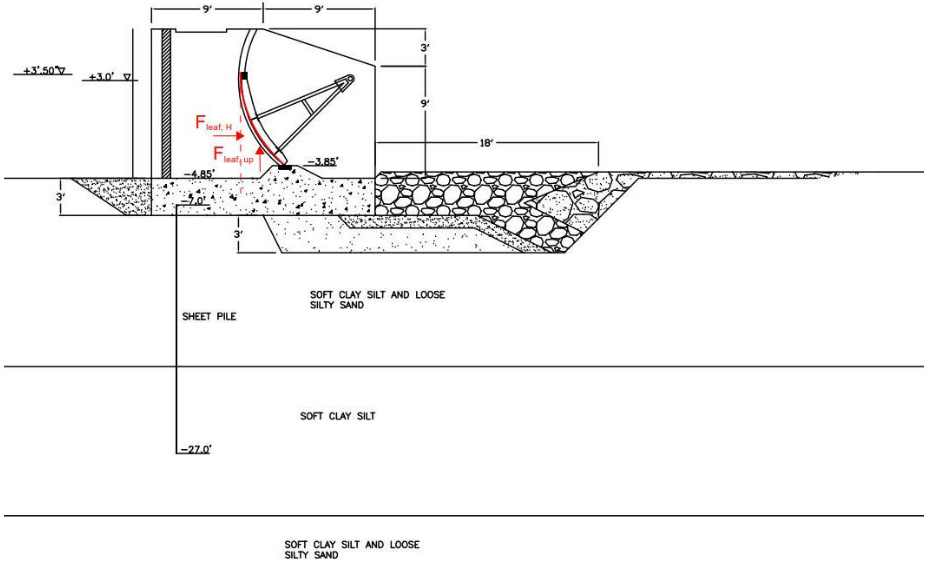
The new design gives the following Factors of Safety (all better than the preliminary design reviewed) neglecting effectiveness of the sheet piles and passive soil resistance along the downstream side.

	Condition	Factor of Safety
1)	Summer lake level load on radial gates and tailwater near streambed level.	2.02 for Sliding 1.76 for Overturning

Dam Stability Analysis - Background

Dam Stability Analysis

- Criteria: As outlined in the Priest Lake Basis of Analysis.
- Standards: IDWR and USBR.
- Available Data:
  - Construction Plans
  - Borings
  - Inspection Reports
  - Site Visual Assessment
- Review of Historical Data
- Global Stability Analysis
  - Sliding
  - Overturning
  - Bearing Pressure
  - Resultant Location at base (eccentricity “e”)



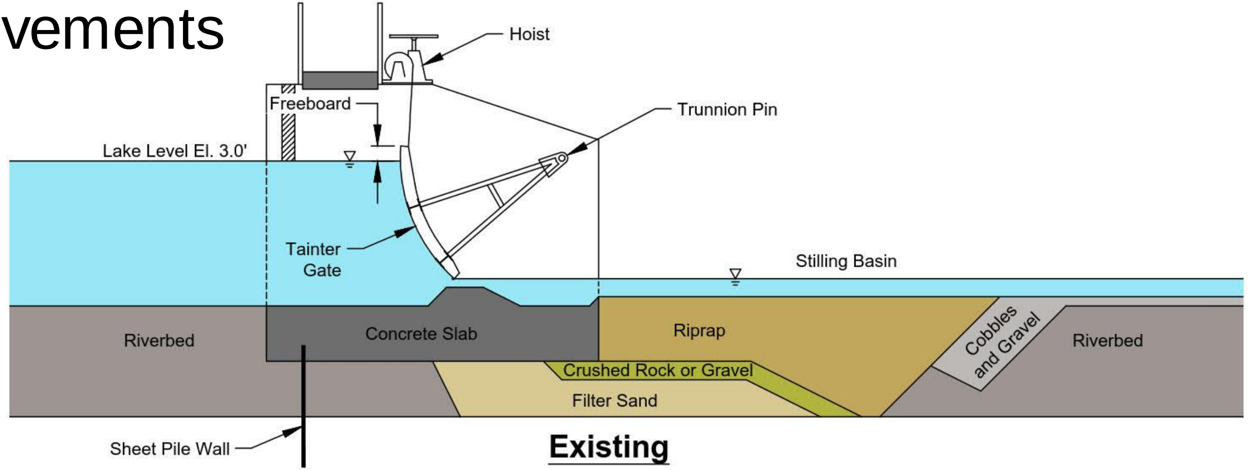
Dam Stability Analysis - Results

- Sliding Stability
  - Sliding stability is of concern and relies upon sheet pile wall that has a marginal structural connection to outlet structure. Increased pool will increase sliding load and thereby further reduce Factor of Safety. New improvements to stilling basin and scour protection likely needed which could improve sliding force resistance.
- Seepage
  - Sheet pile wall provides hydraulic cutoff within center of structure; review at river bank ends of structure.
- Overturning
  - Ok.
- Bearing Pressure
  - Ok, within allowable.
- Resultant location on base
  - Ok, eccentricity within middle 1/3 of base
- Seismic
  - No upgrades but Improvements for stilling basin to not decrease seismic stability
  - Does pool raise improvements constitute a need to address seismic stability for current code requirements? Or will improving seismic stability relative to original conditions suffice for pool raise.

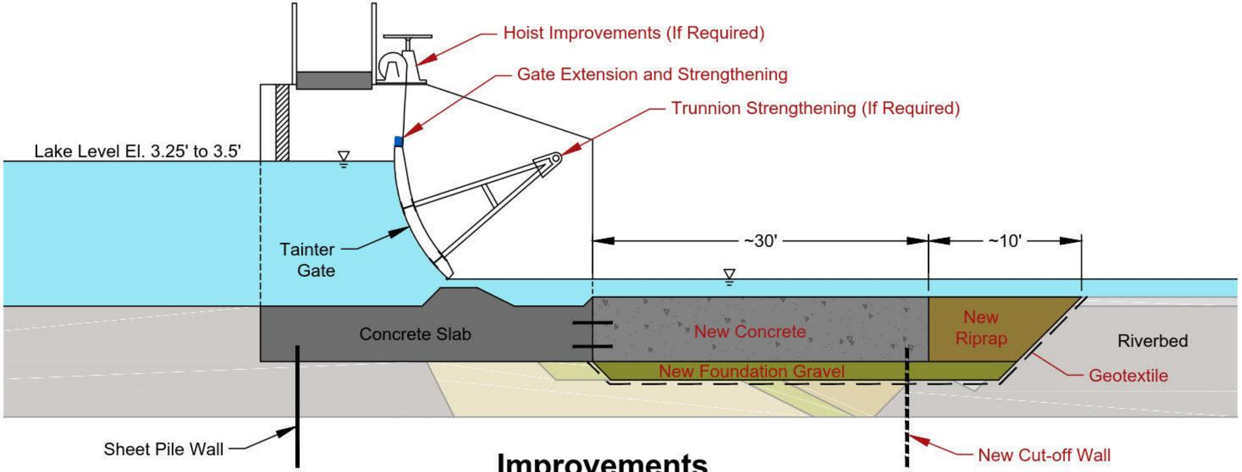
Dam Assessment – Conclusions

- Structure Stability
  - Improvement for stability likely needed to meet current standards to not rely upon sheet pile wall for sliding resistance.
  - Sheet pile wall does provide reduction in seepage.
- Stilling Basin
  - Improvements needed to mitigate risk of scour and corresponding impact on dam stability.
  - Alternatives. Scour apron with concrete slab is recommended; see graphic on subsequent slide.
- Gates
  - Increase height of gate for 3” to 6” pool raise looks feasible.
  - Modification to gate will be needed at top of existing gate (plate extension)
  - Gate Structure and Trunnion: Likely OK, additional analysis in next phase needed to finalize assessment and determination any retrofit needs.
  - Budget line item as contingency will be developed.
  - Gate modifications should be conducted relative to a 6” pool raise.
- Gate Operations
  - Power operation should be considered; likely not required for pool raise. If power is added, include on all gates. Provide measure for backup power during power outages.
  - No automation (or telemetry for remote operation) needed for pool raise.

Outlet Dam Improvements Summary



Existing



Improvements

Next Phase

Additional Data Needs

- Hydrographic Survey of the channel within the scour apron and basin downstream of outlet structure to identify locations of riprap erosion
- Surveying of the top of gate, sill level and gage on spillway sidewall.
- Stage discharge monitoring at outlet structure and pool. Calibration of data.

Additional Analysis

- Developed refined and updated design criteria for improvements.
- Gate operator assessment; replacement vs. retrofit. Motor size type and size determination.
- Detailed structural assessment of the gates.
- Finalize stability and seepage analysis.
- Evaluate and refine the size, length and thickness of the scour apron. Refine design of sheet pile cutoff wall vs. thickened slab. Evaluate need for end sill.

MOTT
MACDONALD

Priest Lake Water Management Study

Outlet Structure Assessment – 9/25/17

IDAHO WATER RESOURCE BOARD

Final

D. Thorofare Assessment

D.1 Thorofare Geomorphic Assessment

D.2 Thorofare Hydraulic and Sediment Transport Analysis

D.3 Thorofare Improvement Alternatives Screening Summary

D.1 Thorofare Geomorphic Assessment

MOTT MACDONALD

Priest Lake Water Management Study

Thorofare Geomorphic Assessment – 8/8/2017

Idaho Water Resource Board

THOROFARE

ISSUES

- Boat access to Thorofare & navigation is challenging at the Thorofare mouth
- Deteriorated breakwater structure

STUDY GOALS

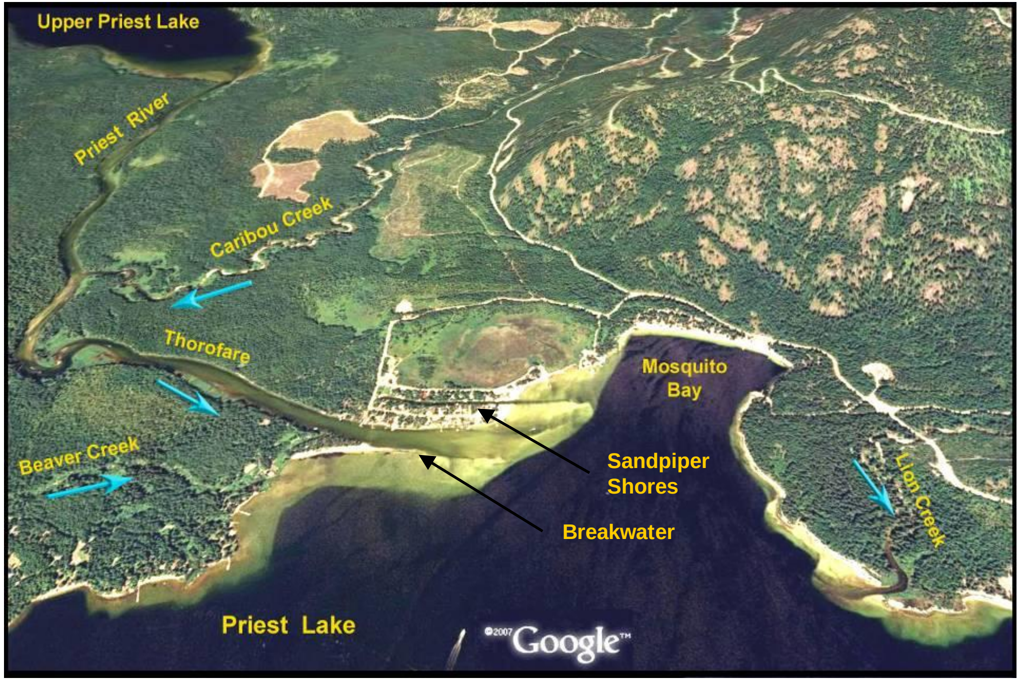
- Providing sustainable modifications to improve Thorofare access, navigability, & water quality (minimize maintenance dredging needs)

GEOMORPHIC ASSESSMENT GOALS

- Evaluate flow & sedimentation processes at Thorofare mouth to aid in evaluation of Thorofare improvement alternatives

THOROFARE BACKGROUND

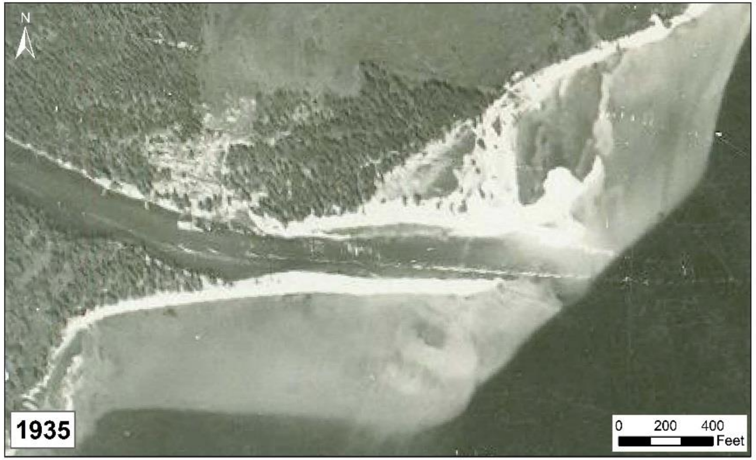
- Thorofare is a 2.7 mile long channel connecting Upper & (Lower) Priest Lake
- Thorofare is by far the single highest flow volume tributary to the (Lower) Priest Lake, approximately 40% of total (IWRB 1995)
- Sediments deposited at Thorofare mouth have created shallow depths & made navigation challenging. There is no recent records of maintenance dredging at the Thorofare mouth
- Concerns about navigation access & need for dredging Thorofare mouth have been voiced dating back to 1994 public meeting (IWRB 1995)
- Thorofare is designated as a State Natural River & dredging above the mouth is prohibited (IWRB 1995)
- 2009 Study Conclusion: replacement of breakwater with solid structure needed to maintain access to Thorofare (CHE 2009)



THOROFARE HISTORY

- Historical aerial imagery suggests that alignment of Breakwater was different in 1935 and Thorofare mouth was narrower

- Widening the Thorofare mouth is likely to have reduced the flow velocities and subsequently sediment transport capacity of Thorofare



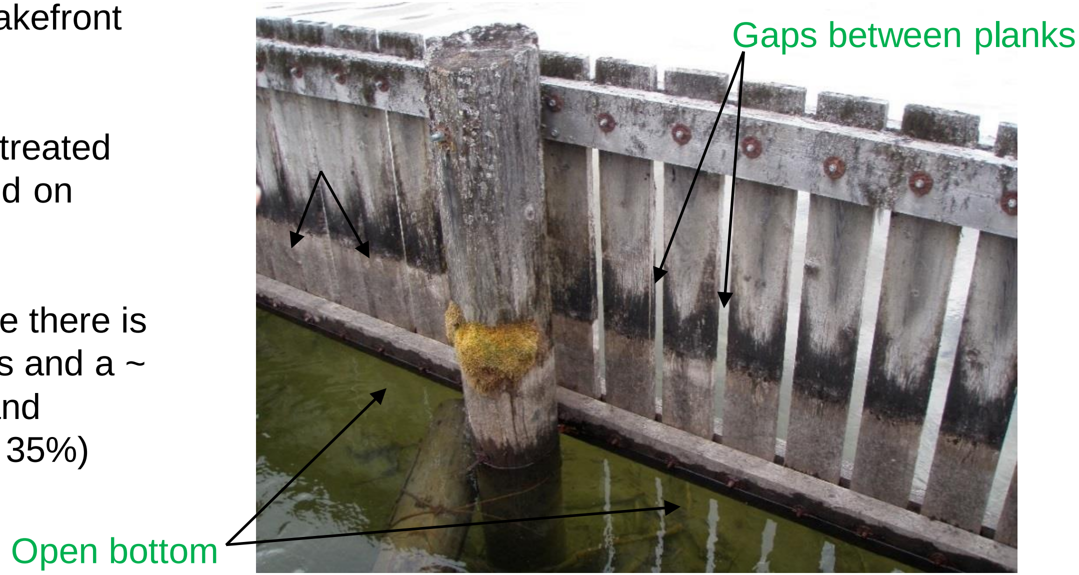
THOROFARE HISTORY

- Historical aerial imagery from 1935 shows Thorofare delta before construction of the breakwater

- It appears that the configuration of the lacustrine delta has changed very little from 1935 until present, despite the human modifications (including the breakwater) in the area (Geo 2009)

THOROFARE BREAKWATER

- Original timber pile breakwater (BW) was constructed by USFS to facilitate access to Thorofare in 1933 (IMR 1989)
- Currently, Breakwater serves an additional function of providing wave shelter to lakefront properties in Sandpiper's Shore
- Breakwater structure composed of untreated timber piling and plank boards installed on cross-breams
- Breakwater is considered porous since there is a ~ 1-in gap between the plank boards and a ~ 10-in gap between bottom of planks and Thorofare bed (BW porosity ~ 20% to 35%)



BREAKWATER HISTORY

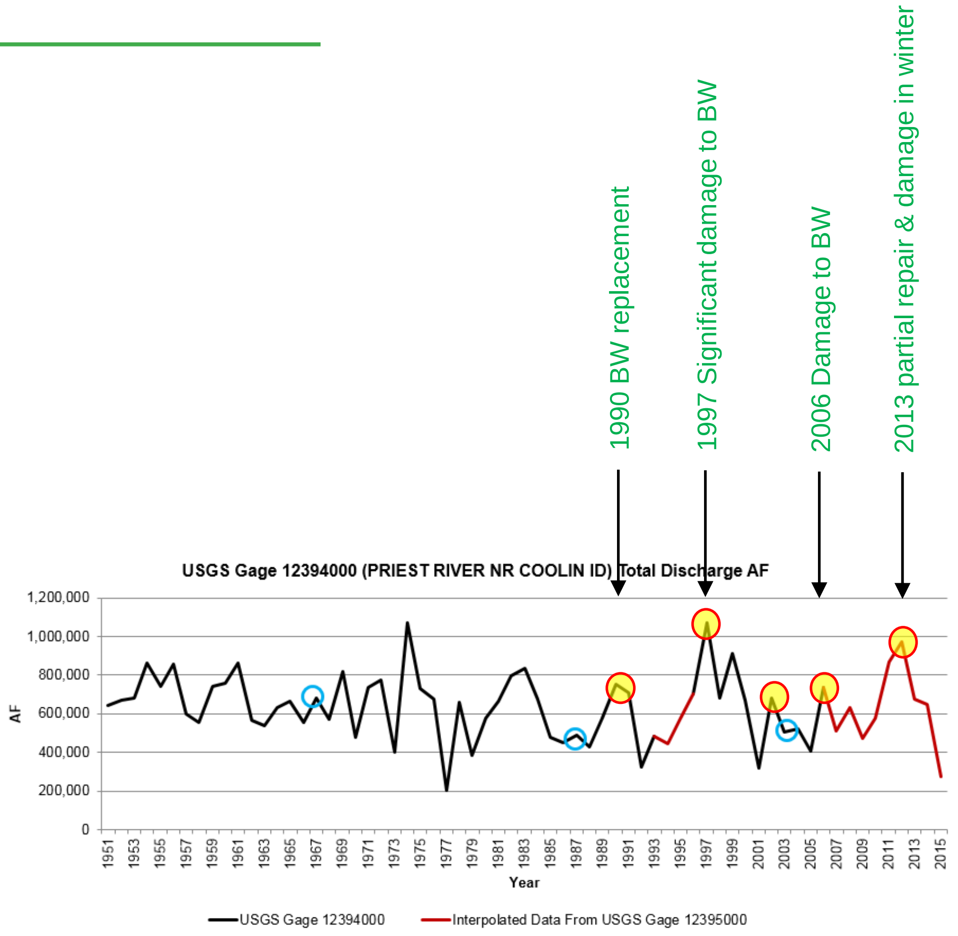
- 1920’s: Original timber pile breakwater (BW) constructed
- 1980: Easternmost 200 ft of BW was replaced
- 1990: BW replacement by InterMountain Resources
- 1997: Partial repair after damage due to spring runoff
- 2006: Partial repair after damage due to spring runoff
- 2013: Longer plank boards (14’ vs. old 8’ boards) were installed in solid ground. However, the flow in the subsequent winter scoured the bed underneath the boards. (Source: Copper Bay Construction Co.)

Observations:

- Damages to breakwater have occurred approximately every 7 to 10 years, resulting in need for (partial) repair
- Non-engineered repairs of breakwater have not withstood strong spring run-offs

Summary:

- Service life of breakwater ~ 30 to 40 years with periodic repairs
- Portions of Breakwater currently nearing end of service life



1990 BW replacement

1997 Significant damage to BW

2006 Damage to BW

2013 partial repair & damage in winter

THOROFARE NAVIGATION

- Bonner County marks the navigable part of channel annually with buoys before start of the recreational season

- Observations indicated that deeper part of channel changes its course within the Thorofare mouth annually & sometimes seasonally

- Boaters have identified most challenging area for navigation is the Thorofare mouth and passing over the sand bar

- Concerns about navigation access & need for dredging Thorofare mouth date back to 1994 public meeting (IWRB 1995)

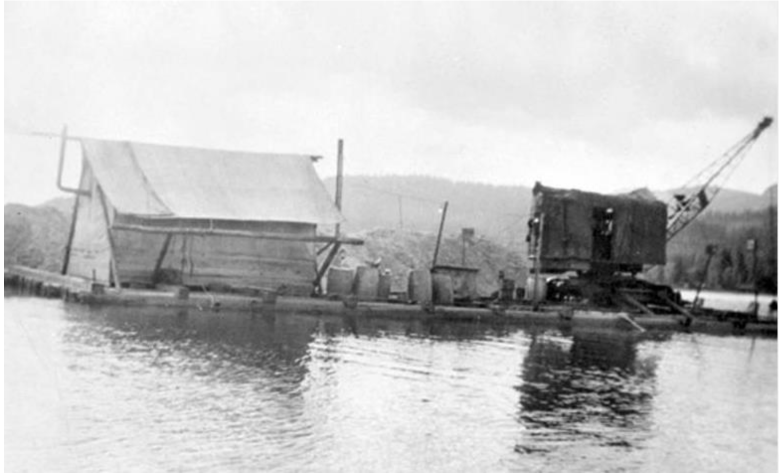
- Minimum desired water depth for recreational navigation is 4 ft (per Basis of Analysis criteria).

THOROFARE DREDGE HISTORY

- 1930’s: historical photo showing mechanical dredging of Thorofare using a barge

- 1940 – 1990’s: anecdotal accounts suggest episodic mechanical dredging

- 1990s – present: no official records of dredging but a few permit exist.



Summary:

- Regular maintenance dredging program has not been in place since 1990s; this has placed greater focus on effectiveness/performance of breakwater in directing the flow & avoiding flow spreading

THOROFARE FLOW BACKGROUND

- Thorofare flow is not measured on a continuous basis. Only available measurements of Thorofare flow date back to 1994-1995 (DEQ 1997).
- This chart shows Thorofare hydrograph for 1994-1995 using gaged data for Upper Priest River & modeled flows for tributary creeks plus numerous discrete flow measurements on the Thorofare.
- Discrete flow measurement of 2755 cfs on 5/11/2017 conducted as part of Mott MacDonald study.
- This table provides summary for Thorofare flow analysis (DEQ 1997)

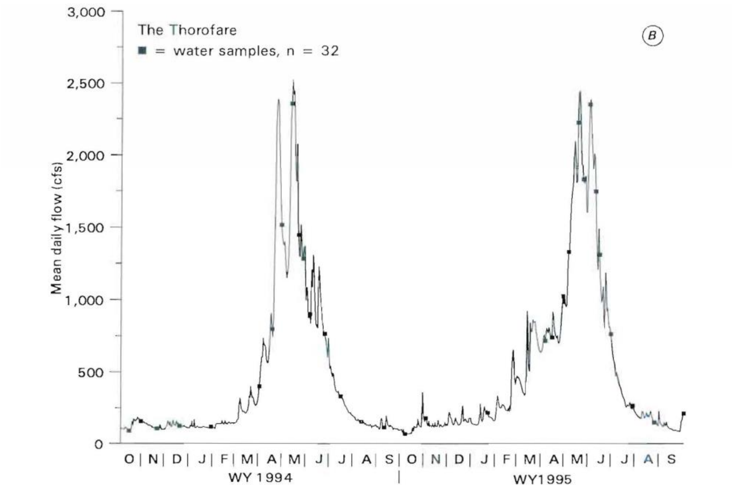


Figure 4-4. *A*, mean daily cfs hydrograph for Two Mouth Creek; and *B*, The Thorofare. Data for water years 1994 and 1995.

	Annual mean daily flow (cfs)	Spring mean daily/maximum (cfs)	Annual Volume (ac-ft)	% of total inflow to Priest Lake
1994	428	1216/2522	309,650	42.2
1995	510	1201/2443	369,550	38.3

THOROFARE FLOW

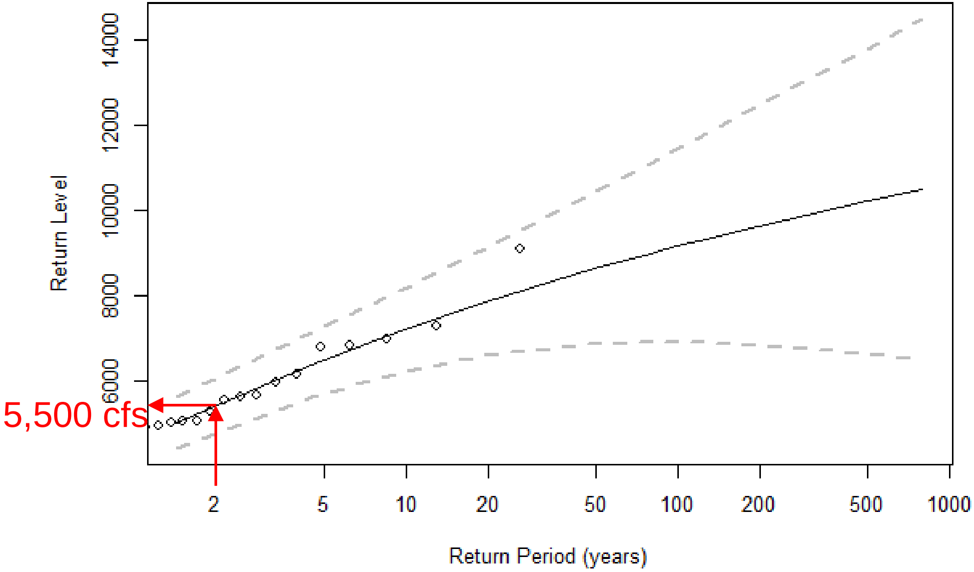
- In lieu of adequate flow measurements at Thorofare, we will rely on discharge measurements downstream of the outlet dam at the Priest River Near Coolin gage (USGS Station#12394000) and correlation between the two

- Thorofare is assumed to account for 40% of discharge measured at the Dickensheet gage (DEQ 1997)

- Extreme Value Analysis shown for Dickensheet gage (for records of 1980-2006) suggests that the 2-yr discharge is 5,500 cfs

- Channel-forming flow (assumed to be equal to 2-yr flow) for Thorofare can be calculated as
  Qcf = 0.4 x 5,500 cfs = 2,200 cfs

Extreme flows for Discharge Priest River near Coolin



5,500 cfs

Note:

- Need confirmation from Matt Anders on assumptions

THOROFARE FLOW SPREADING

- Shallow sand bar at the mouth on 3/15/2005, WL = 0.33’ USGS
- Significant flow spreading & flow cutting into the sand bar and underneath the breakwater
- Aerial photo suggests ~40% of flow is going underneath the Breakwater, % to be verified with numerical modeling

Photos courtesy of Tom Weitz

THOROFARE FLOW SPREADING

- Episodic flow spreading & flow cutting underneath the breakwater can be observed in aerial images

- This channel migration (cutting through & underneath the Breakwater) and breakwater structure damage has occurred over past 40 years impacting navigability

THOROFARE FLOW SPREADING

- Thorofare flow forms channels underneath the breakwater.

- The channel locations & width vary with time

THOROFARE BANKS

- Erosion along the bank (likely due to high flow events and vessel wakes) was observed in limited stretches along Thorofare

- Most of the Thorofare bank is highly vegetated and to some degree stabilized

Summary:

- Bank erosion contributes to sedimentation at Thorofare mouth but it is not a significant source of sediment present at entrance to Thorofare

THOROFARE HYDROGRAPHY

- Only existing hydrographic survey of Priest Lake dates back to 1995 (DEQ 1997). Unfortunately, that survey did not cover the Thorofare (to the best of our knowledge)

- MM completed a hydrographic survey of Thorofare in May 2017

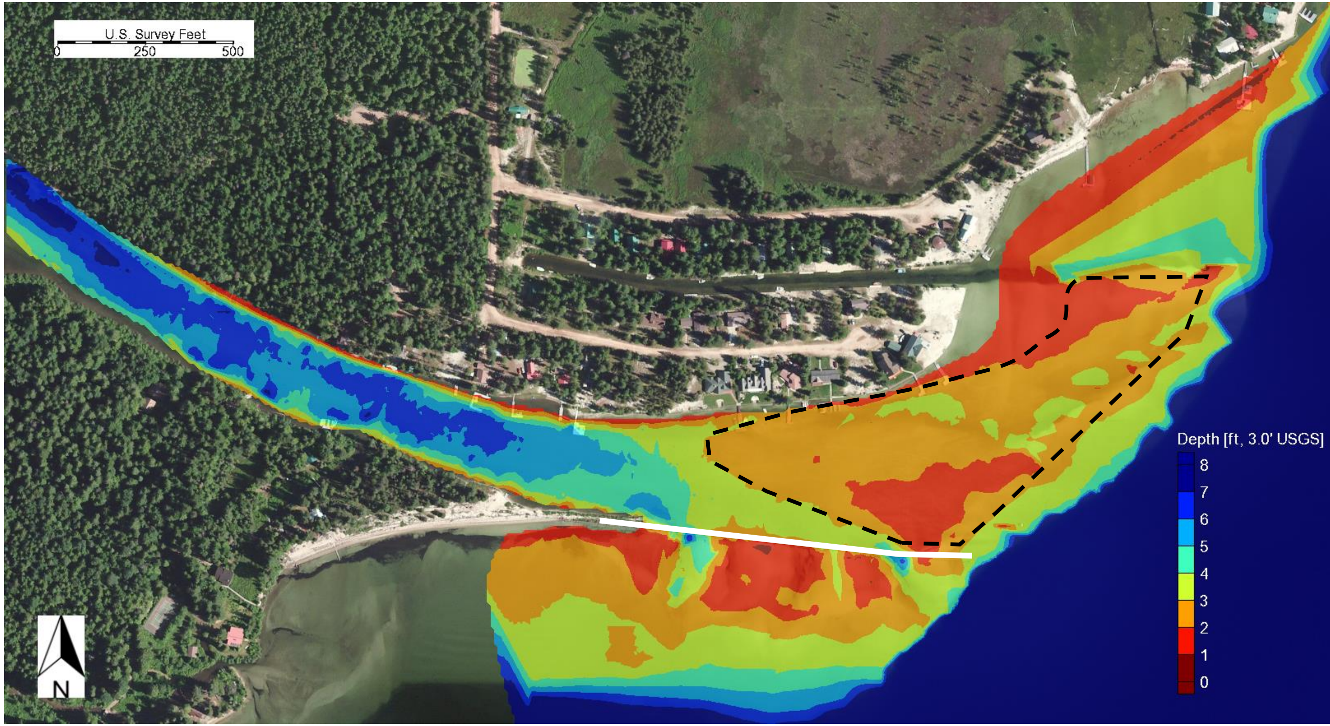
- Color contours here represent available water depth during summer w.r.t. Lake Level at 3.0' USGS gage

- Water depths at the Mouth outlined by black dashed line is mostly shallower than 3 ft, with some areas shallower than 2 ft

Conclusion:

- Dredging & improvements to better confine Thorofare flow likely needed to maintain navigable access

- Accurate marking of Thorofare mouth by bouys would be important to help boaters access Thorofare



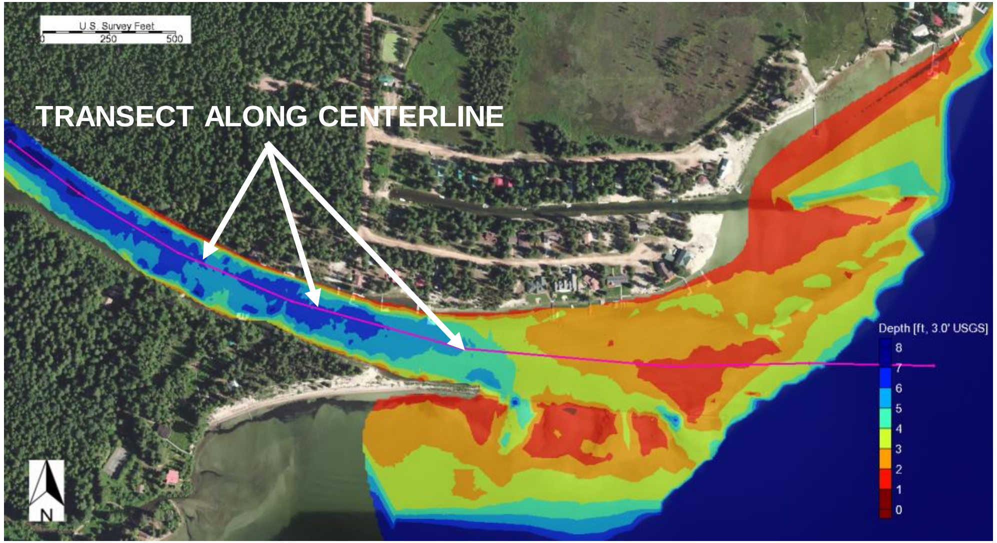
THOROFARE HYDROGRAPHY

- Water depth during summer along the Thorofare centerline is shown on profile view
- Upstream of Thorofare mouth (& breakwater), water depths larger than 5 ft start to decrease gradually closer to the breakwater
- Shallow depth at the mouth due to sediment deposition over approximately 300 ft along the centerline
- Moving further east past the mouth, there is a sudden drop to depths of 35 ft and deeper

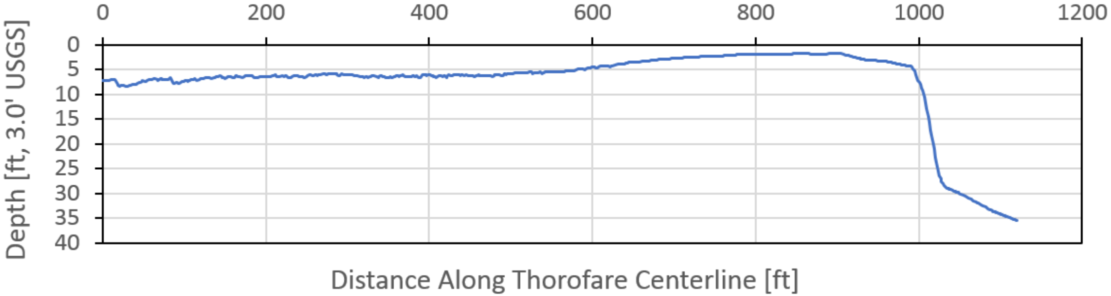
Summary:

- Thorofare has a low gradient upstream of the breakwater
- Available depths at entrance are on average 2' shallower than desired for navigation

Site Plan View



Profile View



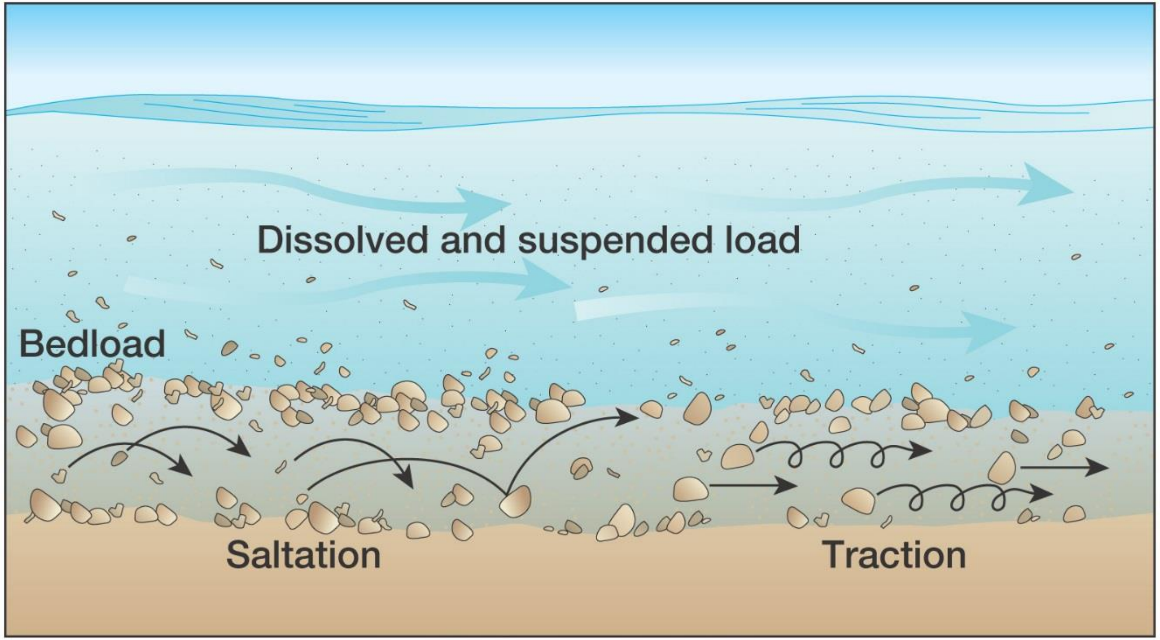
RIVER PROCESSES – SEDIMENT TRANSPORT

Transport of sediments in rivers occurs in two modes:

- Suspended Load: small particles (sand & clay-sized) are carried within the water by turbulent flow. Some minerals will be dissolved and will be carried in solution.
- Bed Load: sand-sized and heavier material may be bounced along the river bed in a "leap-frog" motion (saltation) or rolled along the bed (traction).

Summary:

- Majority of sediment transported in Thorofare (upstream of Breakwater) are bed load sediments and not suspended sediments

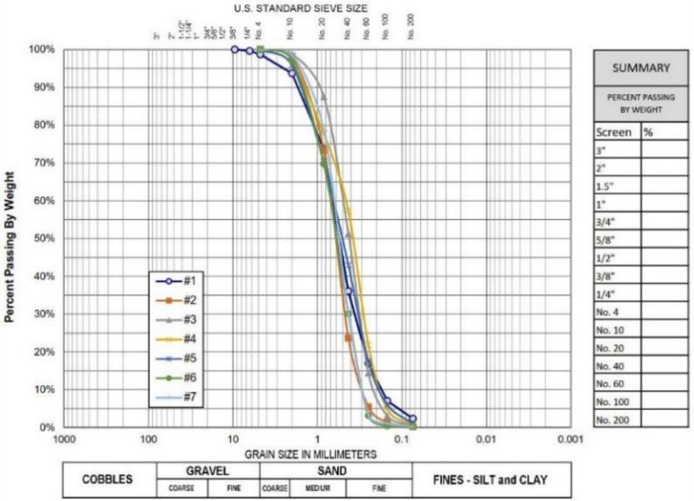


THOROFARE SEDIMENTS

- 7 grab samples were collected & analyzed in at Thorofare mouth. Analysis showed well-graded fine to coarse sand
- d_{50} = 0.4 to 0.6 mm & d_{84} = 0.8 to 1.2 mm
- Minimal variation relative to location was observed (MM 2017)
- Bed grain size from fine gravel to coarse sand throughout the lower ½ of Thorofare to the breakwater was observed (Geo 2009)

Summary:

- Sediment is very uniform fine to coarse sand with little variability between Thorofare & lake shore

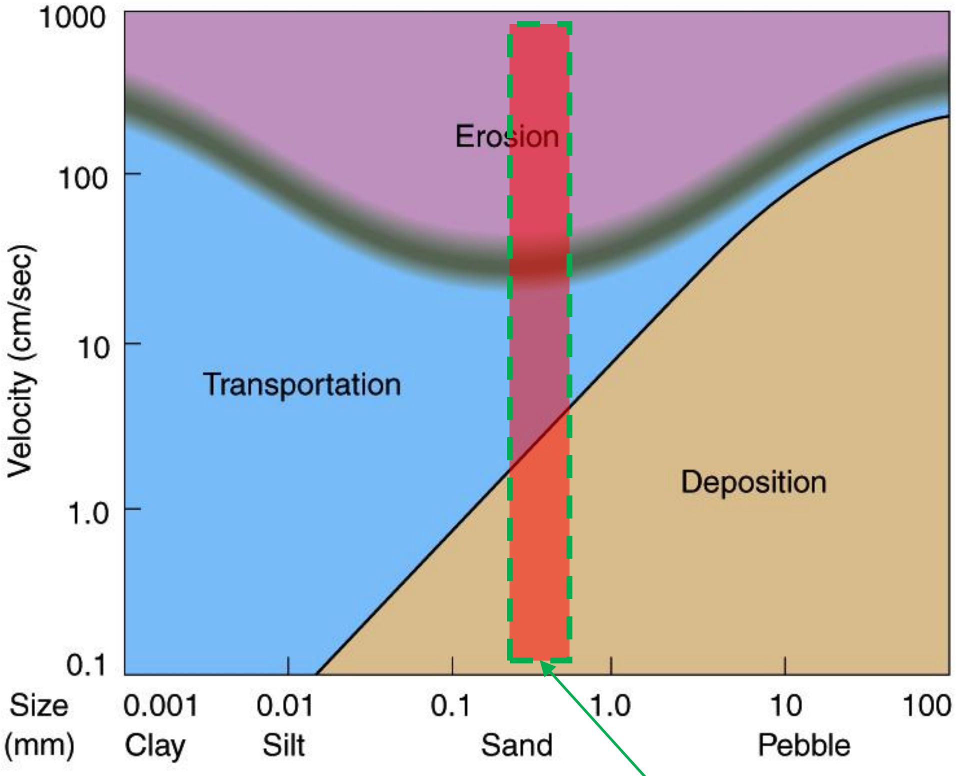


Location of grab samples

THOROFARE SEDIMENT TRANSPORT CAPACITY

- Priest Lake Sediment samples: Well-graded fine to coarse sand, 0.4 mm < d_{50} < 0.6 mm

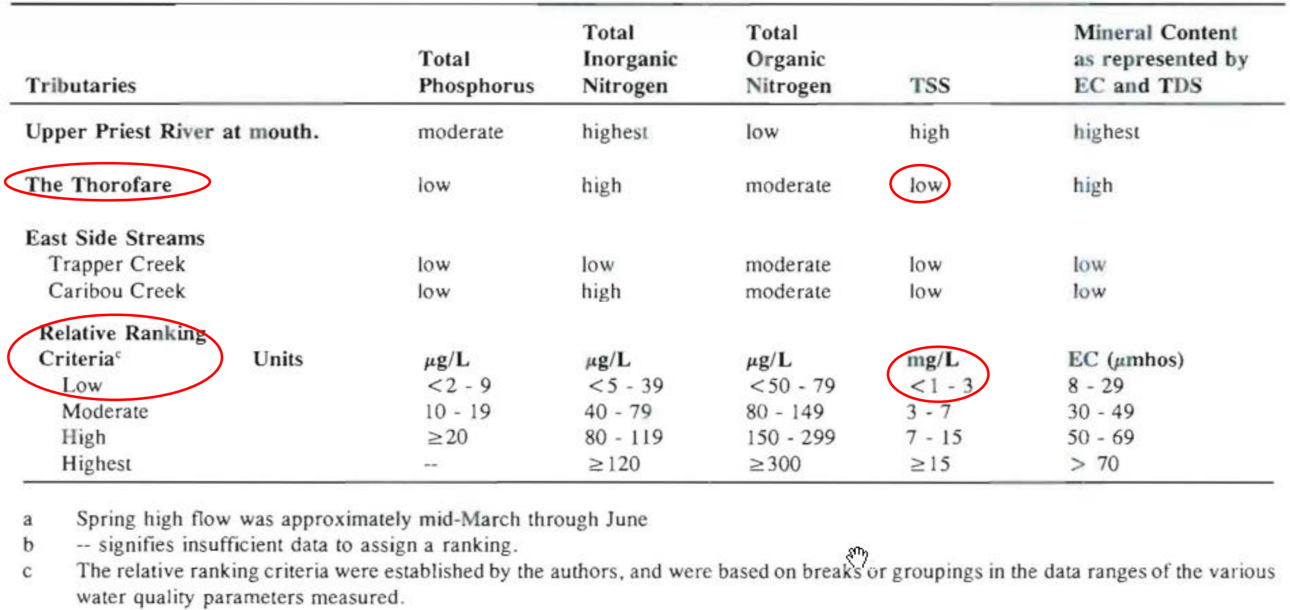
- For a constant sediment size, reduction in flow velocity leads to reduction in sediment transport/erosion capacity or lead to deposition



Range of median grain size for Thorofare mouth

THOROFARE GEOMORPHOLOGY

Summary:
Thorofare & Caribou Creek both rank low (< 3 mg/L) for total suspended sediment (TSS) transport. Most of the sediment must be transported as bed load.



Tributaries		Total Phosphorus	Total Inorganic Nitrogen	Total Organic Nitrogen	TSS	Mineral Content as represented by EC and TDS
Upper Priest River at mouth.		moderate	highest	low	high	highest
The Thorofare		low	high	moderate	low	high
East Side Streams						
Trapper Creek		low	low	moderate	low	low
Caribou Creek		low	high	moderate	low	low
Relative Ranking Criteria[c]	**Units**	**µg/L**	**µg/L**	**µg/L**	**mg/L**	**EC (µmhos)**
Low		<2 - 9	<5 - 39	<50 - 79	<1 - 3	8 - 29
Moderate		10 - 19	40 - 79	80 - 149	3 - 7	30 - 49
High		≥20	80 - 119	150 - 299	7 - 15	50 - 69
Highest		--	≥120	≥300	≥15	> 70

[a] Spring high flow was approximately mid-March through June

[b] -- signifies insufficient data to assign a ranking.

[c] The relative ranking criteria were established by the authors, and were based on breaks or groupings in the data ranges of the various water quality parameters measured.

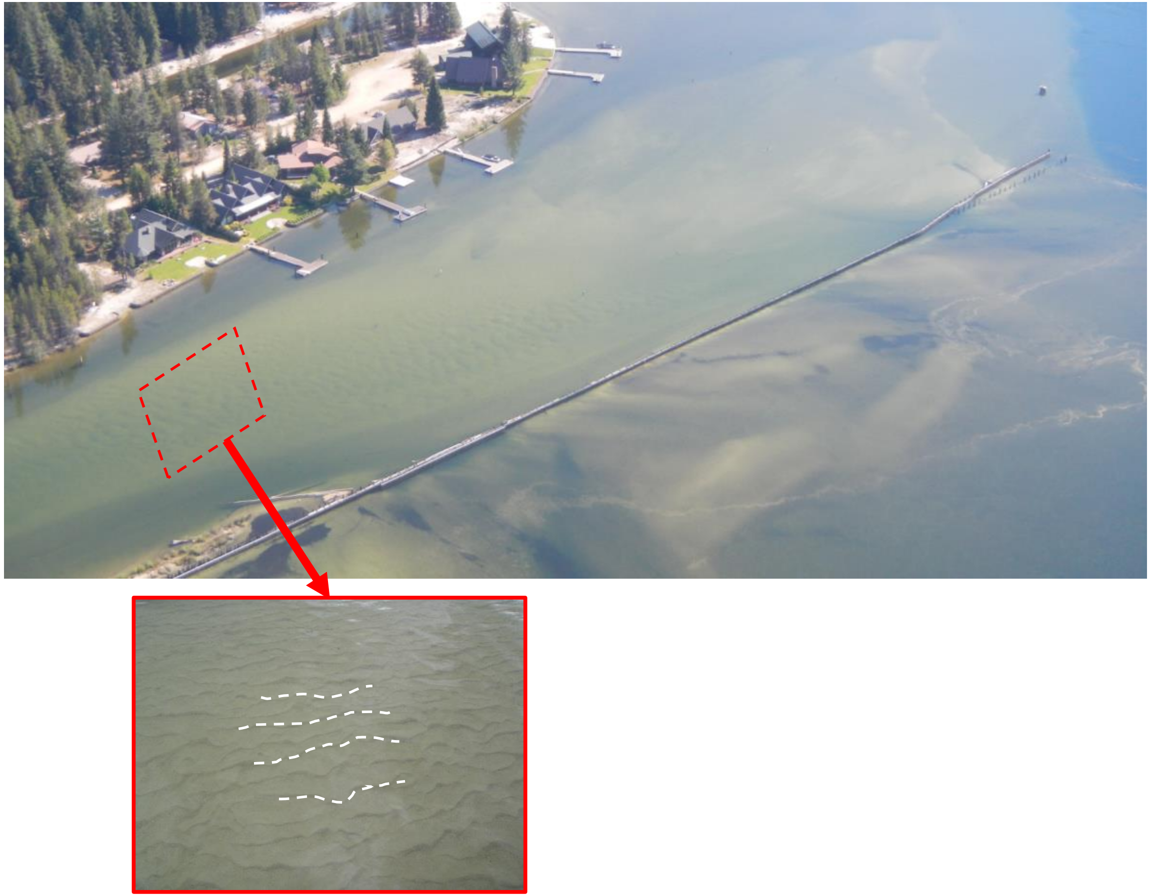
DEQ (1997)

THOROFARE GEOMORPHOLOGY

- Upper Priest Lake is a settling basin for incoming suspended sediment, and Thorofare ranks low for in total suspended sediment (TSS). Also, Caribou Creek ranks low in TSS (BC 2003)

- Thorofare has little to no gradient (IWRB 1995)

- Ripples, characteristics of low-gradient, sand-bed rivers

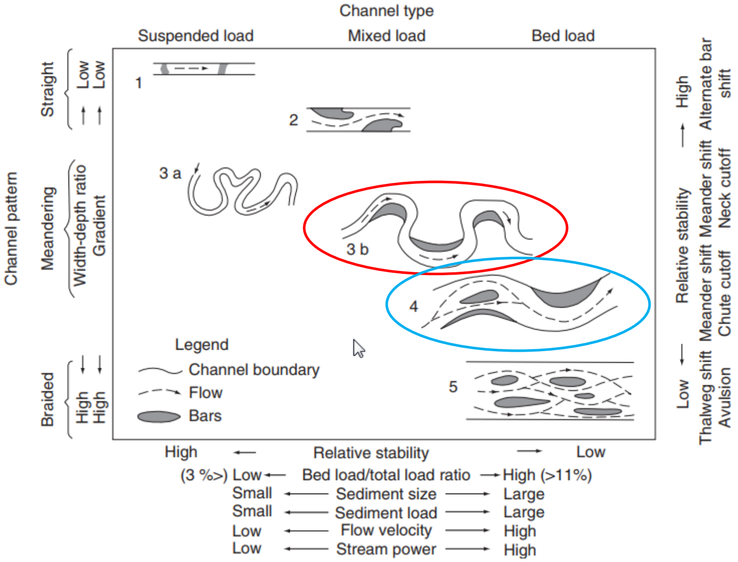


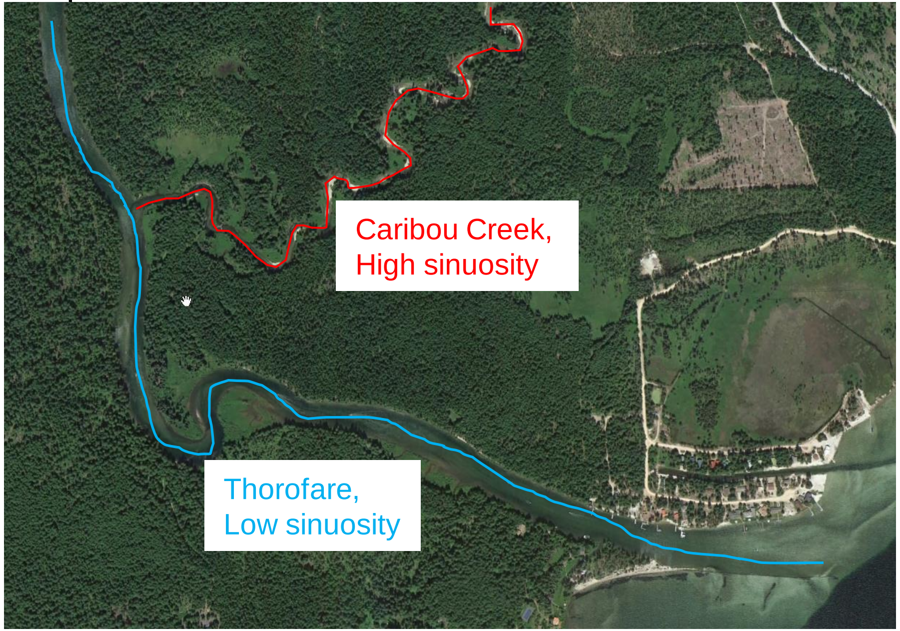
THOROFARE GEOMORPHOLOGY

Thorofare is an alluvial channel with sediment transported with channel forming flows

Summary:

Low sinuosity of Thorofare indicates stronger bed load compared to suspended load.





Caribou Creek, High sinuosity

Thorofare, Low sinuosity

CARIBOU CREEK GEOMORPHOLOGY

Caribou creek is highly sinuous yet relatively stable stream channel near its confluence with Thorofare

Caribou bed is composed primarily of loose sand and gravel. Caribou banks are composed of cohesive silt and clay with vegetation providing further bank stabilization

Lower 5.5 miles of Caribou Creek have low to moderate gradient (less than 1.7%)

Estimated mean daily spring high flow: 400-600 cfs, summer base flow: 20 – 50 cfs (for WY 95 based on Lion Creek)

Logging practices, dirt roads, & steep mountains produce a constant supply of sediment (loose sand and gravel bed) to Caribou creek

Lack of sediment bars and loose bed material indicates readily transport of material downstream

Grain sizes decrease from coarse gravel to coarse sand in lower ¼ mile of Caribou Creek

Sources: Geo (2009) & IDEQ (2001)

Caribou Upstream Grain Sizes

Caribou Downstream Grain Sizes

Caribou Creek

BED LOAD SEDIMENT TRANSPORT

- Bed load sediment transport is dominant sediment source in the Thorofare.

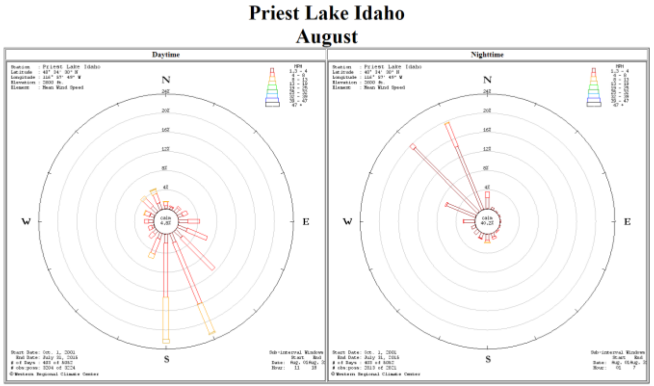
- Sources come from tributaries and Thorofare bank adjustment.

- Majority of bedload is likely transported during spring runoff high flow and certain winter high flow conditions.

- Flow diversion at the breakwater reduces sediment transport capacity of Thorofare resulting in sediment deposition in the outlet area near the breakwater.

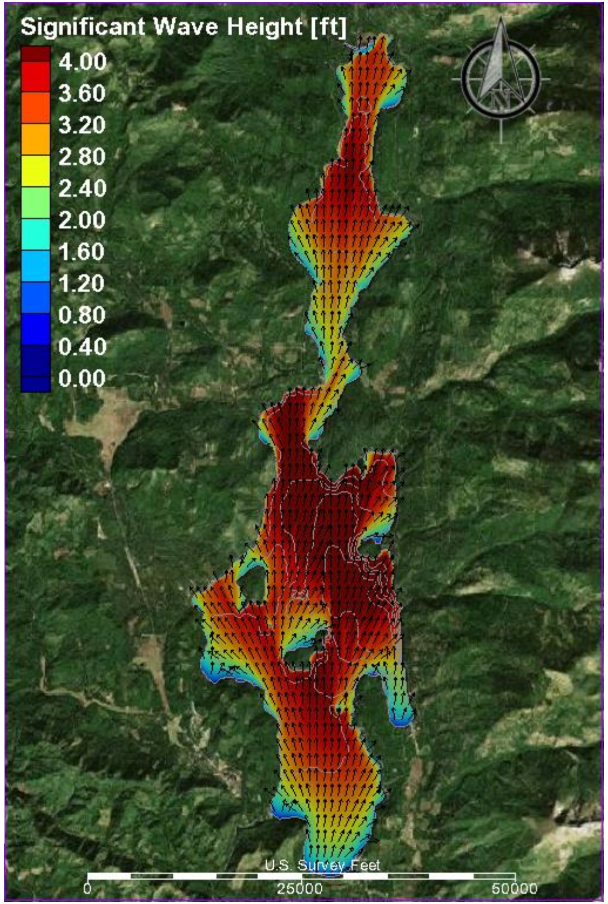
LAKE SHORELINE - WIND WAVES

- Dominant winds blowing from south sustained during daytime (Mar to Nov) can generate waves of up to 4 ft at the Breakwater
- Breakwater provides protection to Sandpiper Shores lakefront residences from southerly wind waves
- The direction of wind reverses at night time and dominant winds blow from North which will not create waves at the Breakwater



Daytime Wind Rose

Nighttime Wind Rose

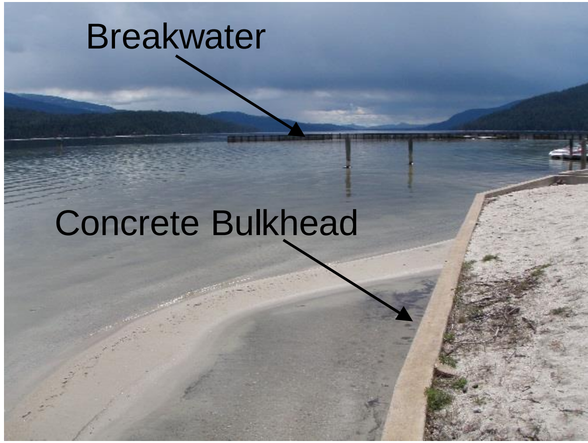


LAKE SHORELINE - SANDPIPER SHORES

- Sandpiper Shores property shorelines (at the Thorofare mouth across from the Breakwater) are currently protected with hard structures (vertical bulkhead, gabion baskets, rock walls)
- It is unclear if the existing structures were originally constructed or were installed after a change in Thorofare dynamics
- The existing breakwater provides wave sheltering to Sandpiper Shores

Note:

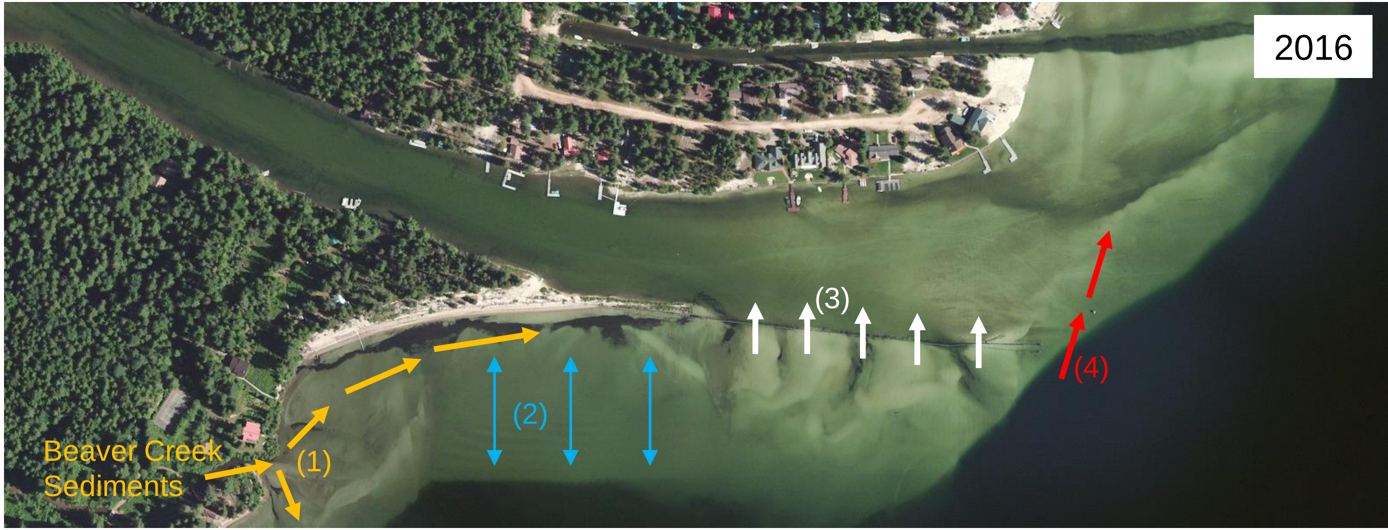
- Need input from Steering Committee & residents on history & timeline



LAKE SHORELINE - WAVE-DRIVEN SEDIMENT TRANSPORT

Wave-driven sediment sources & transport directions include:

1) Beaver Creek brings sediments to its mouth, some of it moves eastward parallel to breakwater due to waves & currents
2) Waves move sediments perpendicular to shoreline
3) Waves may push some sediments through & underneath the breakwater
4) Portion of sediments will move past the breakwater end



Without monitoring sediment input from Beaver Creek & having consecutive surveys of the shoreline, quantification of sedimentation transport in this area is difficult.

LAKE SHORELINE - WAVE-DRIVEN SEDIMENT TRANSPORT

- Sand waves south of Breakwater indicate direction of dominant wind-waves

- Change in alignment of sand bar at the eastern end of Breakwater suggests wind-waves likely to transport sediments through and past the eastern end of Breakwater

THOROFARE GEOMORPHOLOGY SUMMARY

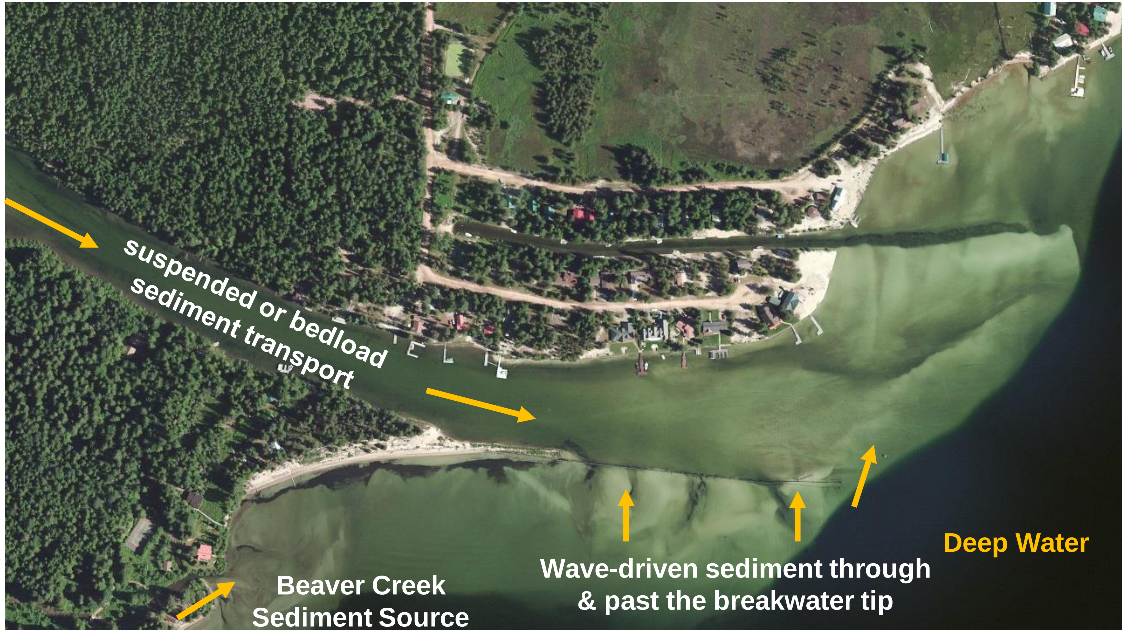
- Thorofare mouth has formed a lacustrine delta into Priest Lake.

- Sediment deposition is result of decreased transport capacity as low-gradient Thorofare meets zero-gradient Lake and flow spreading.

- Deposition has been accentuated by three factors at the mouth:

1) Widening of Thorofare channel along Breakwater
2) Reduced Thorofare discharge & velocity as water passes through or under existing timber breakwater
3) Wind-driven sediments get pushed through and around the breakwater eastern end

- Thorofare flow passes through the Breakwater and has episodically scoured the bed underneath the breakwater leading to significant spreading of the flow and reduced sediment transport capacity at the Mouth



CONCLUSIONS

Flow confinement in the Thorofare is important for improvement of navigation sustainability.
Solid Breakwater is preferred vs. a Porous Breakwater.
Blocking sediment movement from South into the Thorofare mouth will eliminate that sediment source and will reduce maintenance dredging needs.

Thorofare ranks low in suspended sediment transport; majority of sediment is bedload.

Based on results of geomorphologic evaluation, Thorofare improvement concepts to be considered may include:

- (1) No Action (maintain existing breakwater);
- (2) Replace porous breakwater with a solid breakwater at current location;
- (3) Replace breakwater & change location;
- (4) Replace breakwater and extend eastward

Scoping of alternatives to be finalized after completion of hydraulic analysis through a screening processes.

MOTT MACDONALD

Priest Lake Water Management Study

Thorofare Geomorphic Assessment – 8/8/2017

Idaho Water Resource Board

Working Document – v0

D.2 Thorofare Hydraulic and Sediment Transport Analysis

MOTT MACDONALD

Priest Lake Water Management Study

Hydraulic & Sediment Transport Analysis – 8/17/2017

Idaho Water Resource Board

Hydraulic Analysis & Sediment Transport Analysis – Introduction

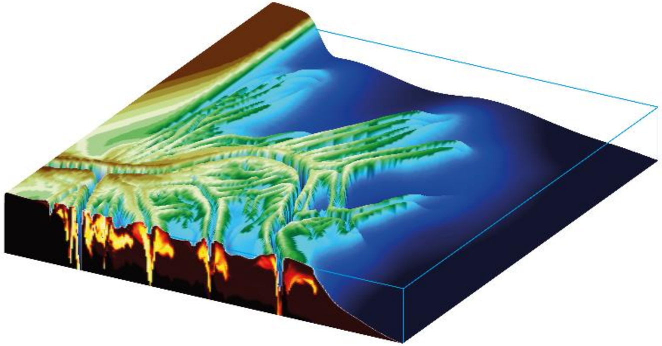
- Purpose – Simulate existing hydraulic conditions to establish baseline for evaluation of alternatives for improving Thorofare for navigation and sustainability.

- Method – Utilize available discharge, bathymetric survey, water levels and sediment size to conduct a conceptual level hydraulic numerical analysis.

NUMERICAL MODELING SYSTEM (DELFT 3D)

- Delft3D-FLOW is a three-dimensional (3-D) hydrodynamic and transport simulation program which calculates non-steady flow and transport phenomena that result from tidal, meteorological, and river forcing on a curvilinear, boundary fitted grid.

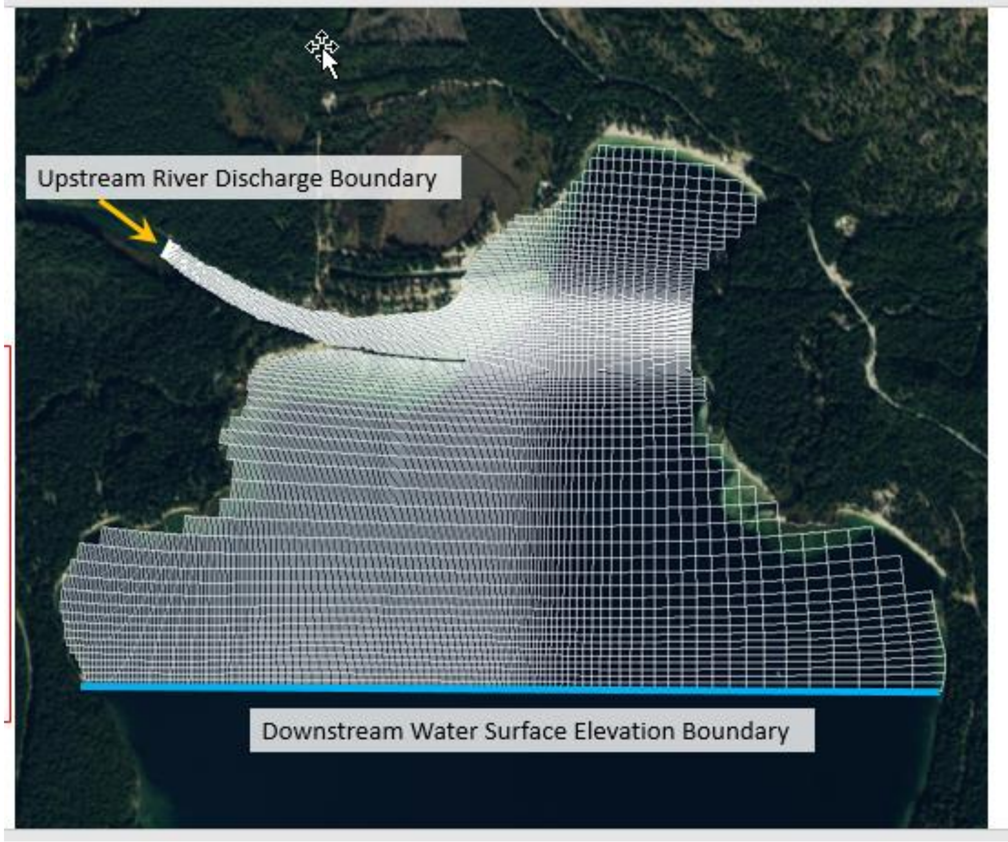
- Delft3D is widely accepted and used in industry and academia



Source: https://www.deltares.nl

MODELING DOMAIN

- Curvilinear Grid shown
- 211 x 111 Grid Points
  Approximately 11120 land/null cells
- Spatial resolution varies from approximately 16 ft (5 m) at Thorofare to 262 ft (80 m) near the Water Surface Boundary
- Using three-dimensional (3-D) mode and using six vertical layers

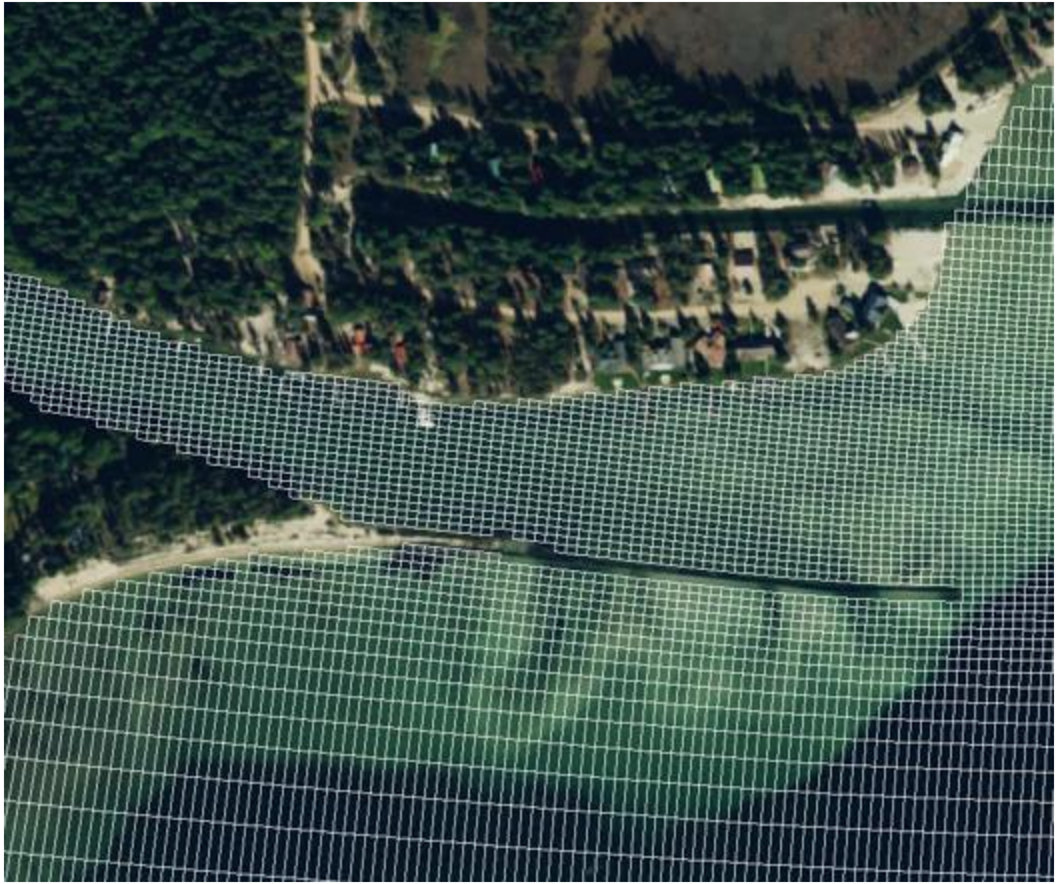


MODEL RESOLUTION AT THE THOROFARE

- Computational element size approximately 16 ft (5 m) in breakwater/Thorofare mouth vicinity

- An average of 20 computational elements across Thorofare Mouth

- Breakwater is modelled as a porous plate with a friction coefficient of 0.274

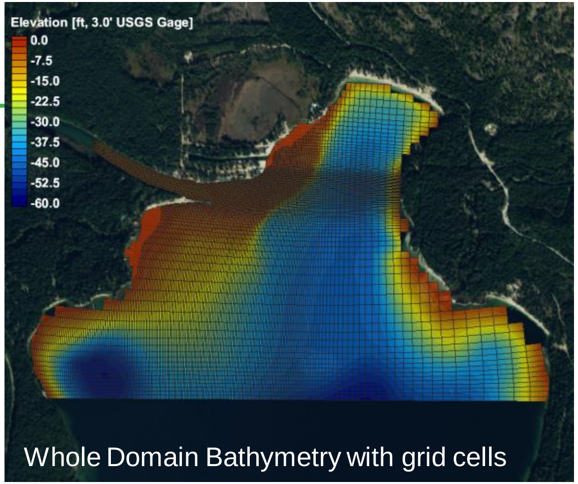


MODELING BATHYMETRY

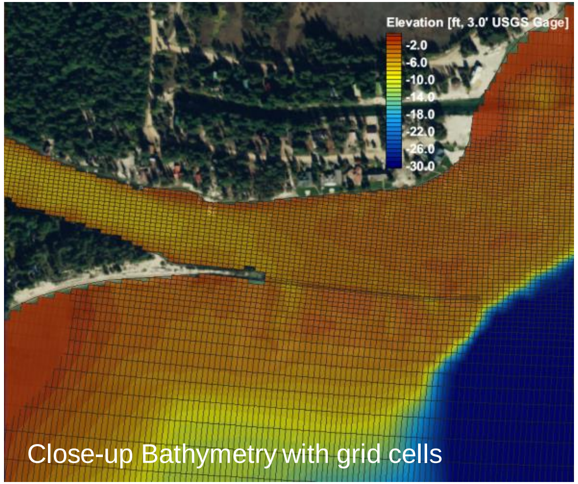
Two Bathymetry Sources:

1) Recent and local trackline Survey on May 10, 2017. Approximately 231,783 soundings, by Mott MacDonald Team

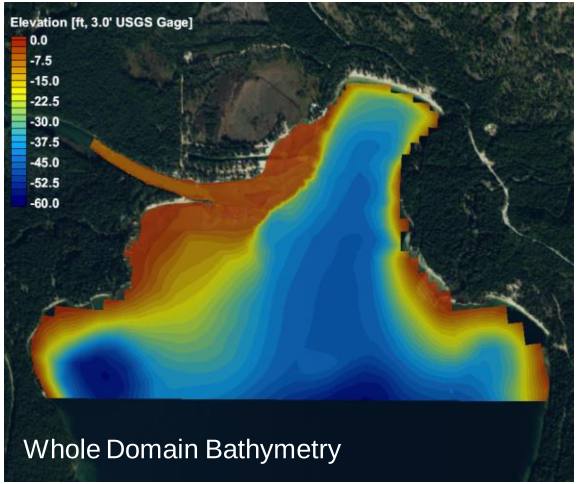
2) 1997 DEQ survey covering the entire Priest Lake



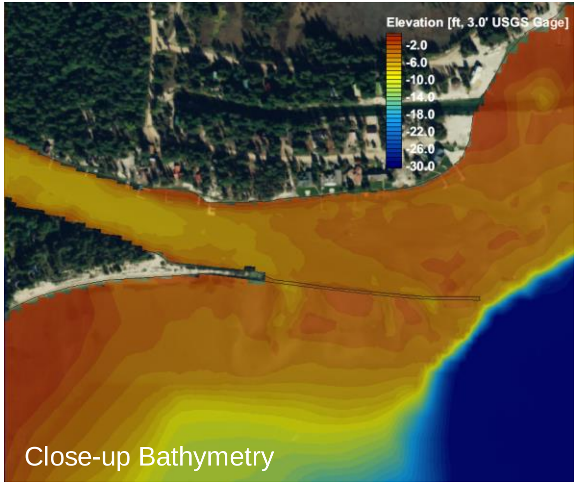
Whole Domain Bathymetry with grid cells



Close-up Bathymetry with grid cells

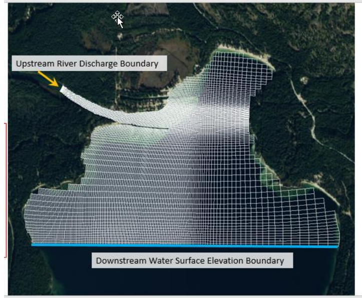


Whole Domain Bathymetry

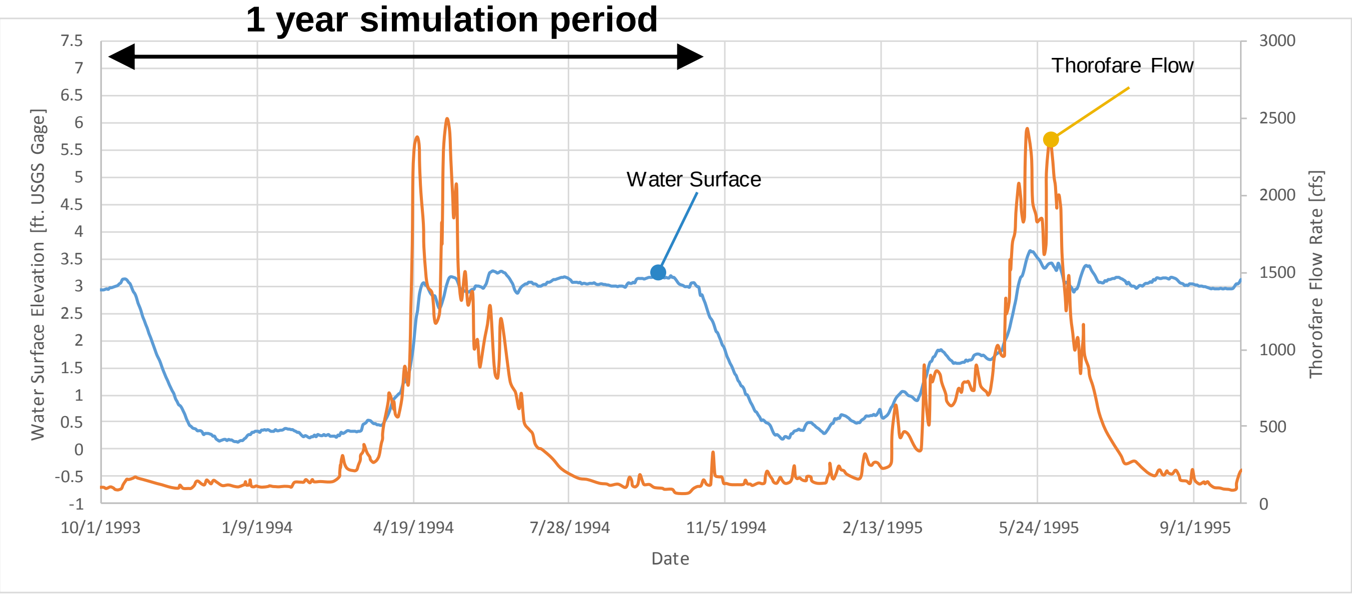


Close-up Bathymetry

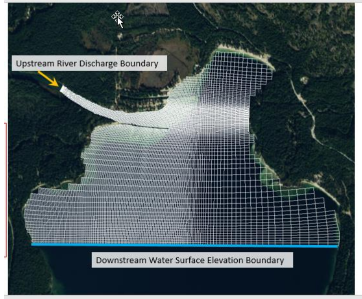
MODEL INPUT - BOUNDARY CONDITIONS



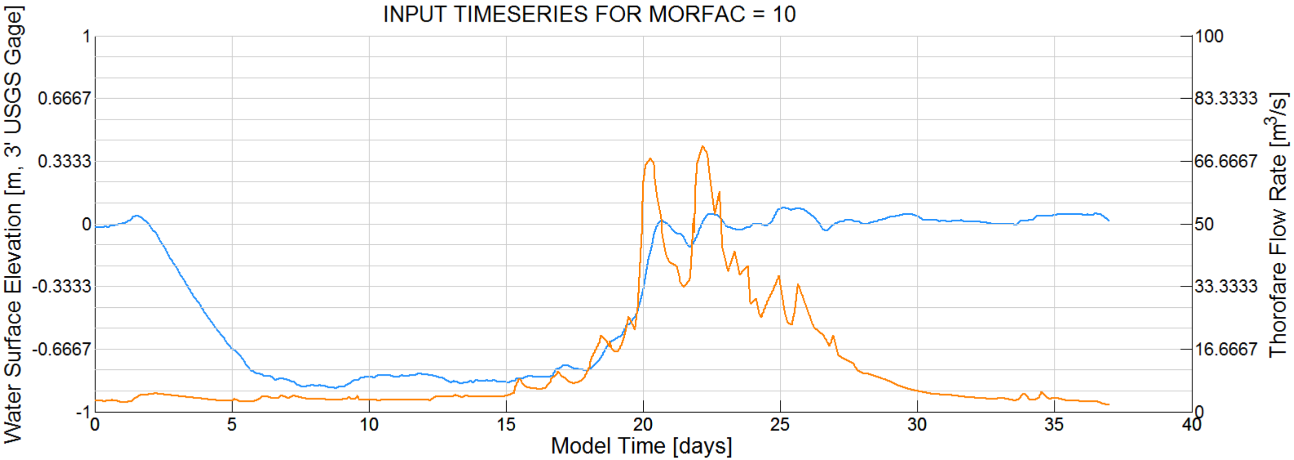
- Run a 1-year simulation in order to simulate a wide range of hydrodynamic conditions as well as see any sufficient mid to long term sedimentation.
- 1994 water year simulation



MODEL INPUT - BOUNDARY CONDITIONS



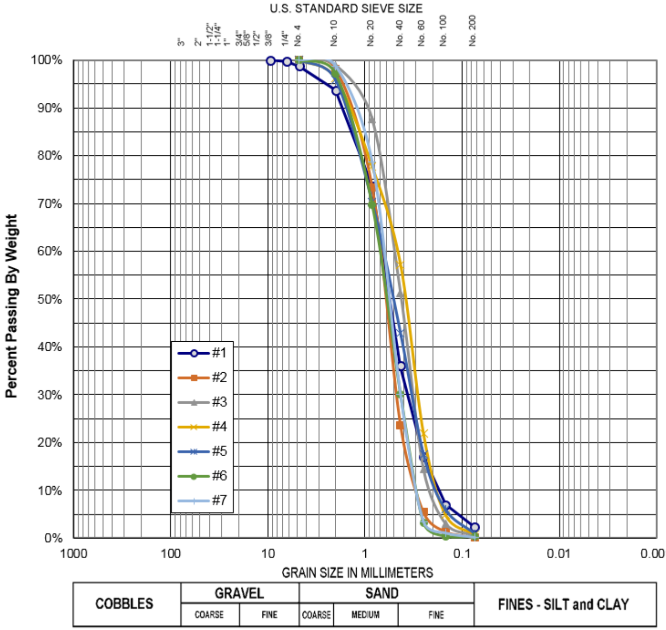
- Delft3D offers Morphological Acceleration Factor (MORFAC)
- Scales morphological results based on MORFAC
- If MORFAC = 10, morphological results would be equivalent to 10x the simulation period.
- With MORFAC = 10, 37 simulated days is enough to simulate ~1 year (370 days).
- Compress the 370 day boundary conditions into 37 days



Simulation Period is now 37 days

MODEL INPUT – SEDIMENT SIZE

- Five (5) Grab Samples taken on May 11, 2017. All samples are fairly uniformly graded sand, but consistent amongst each location
- To model the minor variations in grain size, 3 sediment classes were modeled in Delft3D
- 3 Sediment Classes modelled as sand:
  - 25% of mass assign to d = 0.800 mm
  - 50% of mass assign to d = 0.500 mm
  - 25% of mass assign to d = 0.350 mm

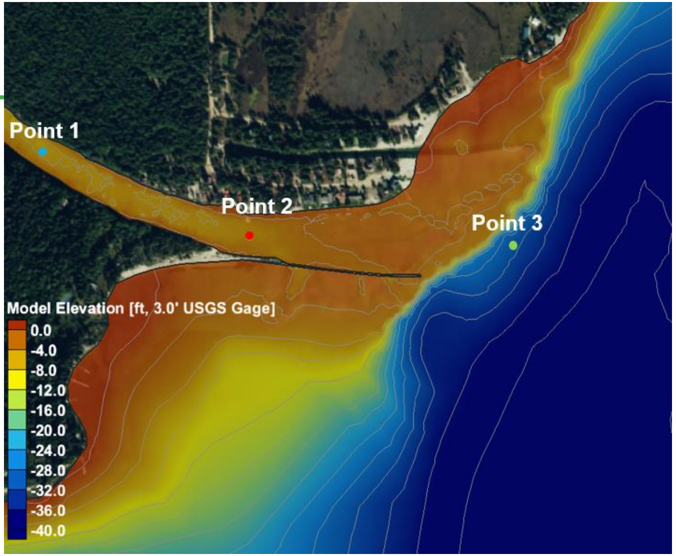


SUMMARY

PERCENT PASSING BY WEIGHT

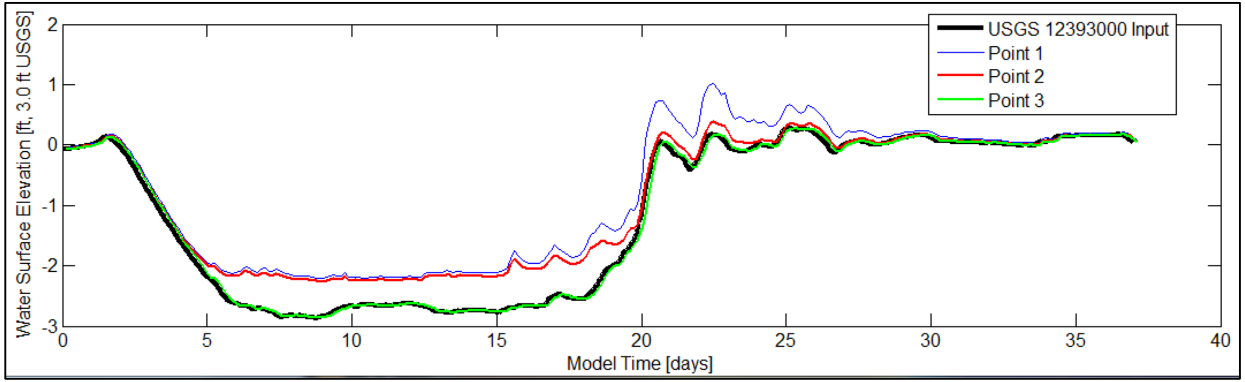
Screen	%
3"	
2"	
1.5"	
1"	
3/4"	
5/8"	
1/2"	
3/8"	
1/4"	
No. 4	
No. 10	
No. 20	
No. 40	
No. 60	
No. 100	
No. 200	

MODELING RESULTS – EXISTING CONDITIONS

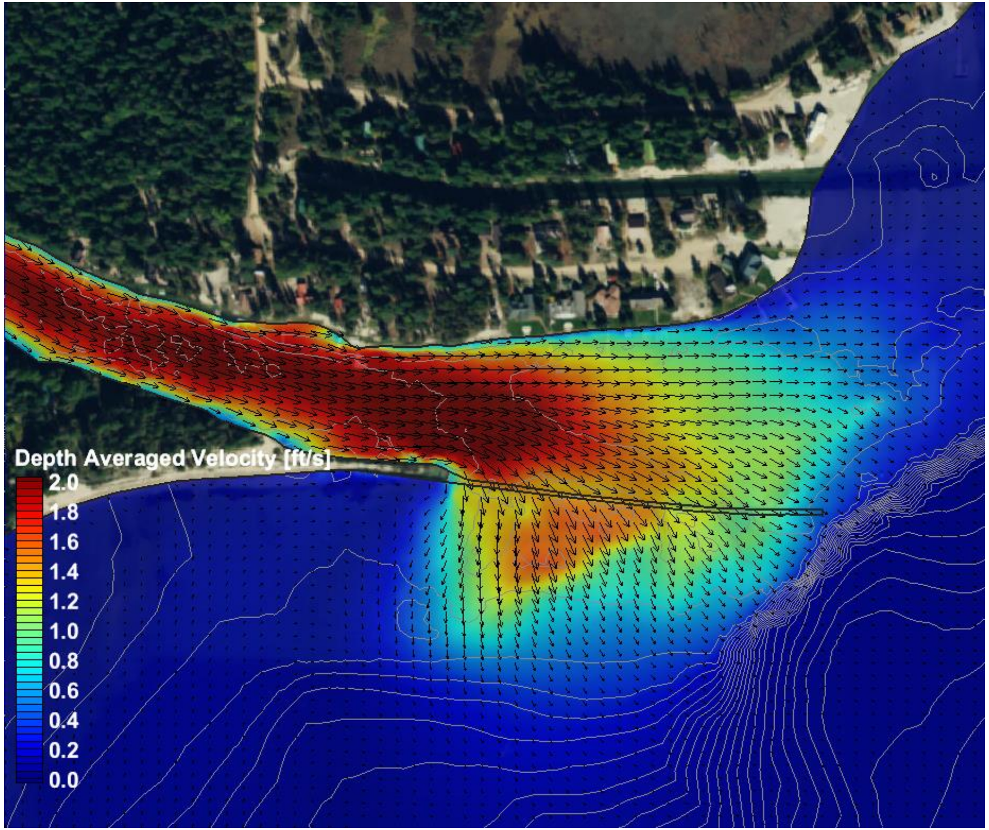


Water Surface Elevation

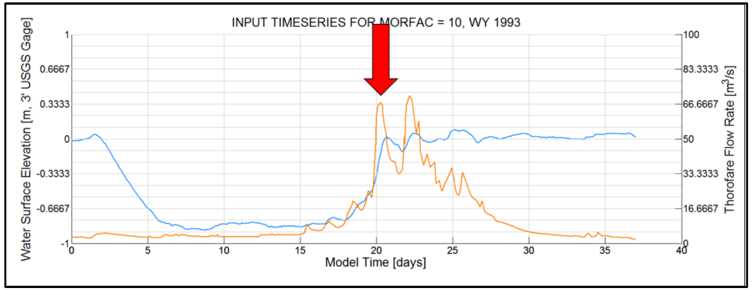
- Water Surface Elevations extracted along the river to check if water surface elevations matched the model inputs. Results show that there is one (1’) foot difference between the of water surface elevation at beginning of the breakwater to the deep end of the mouth



MODELING RESULTS – EXISTING CONDITIONS (Velocity Snapshot #3)

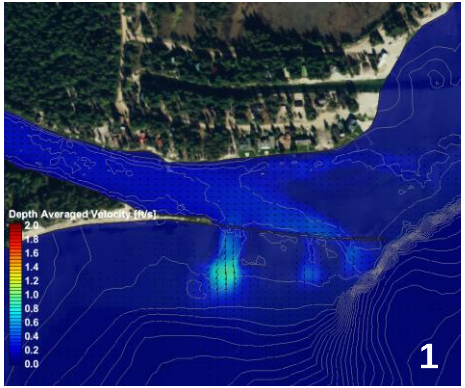


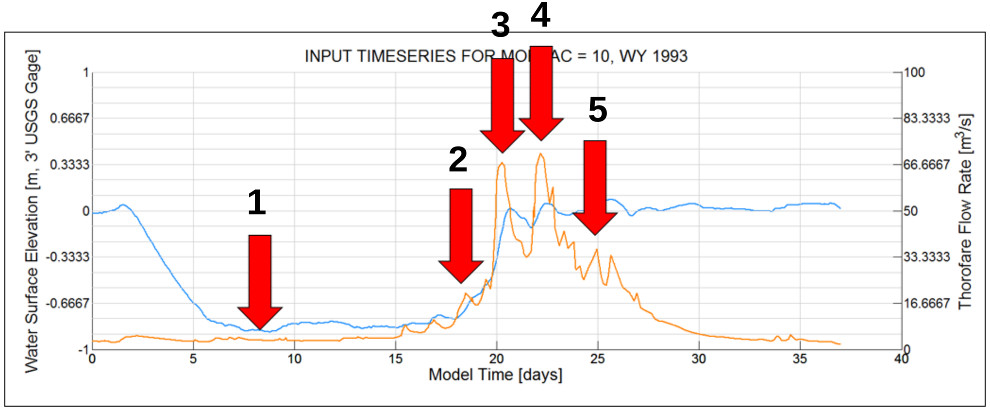
3

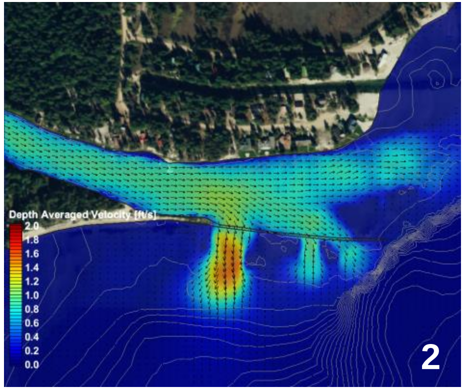


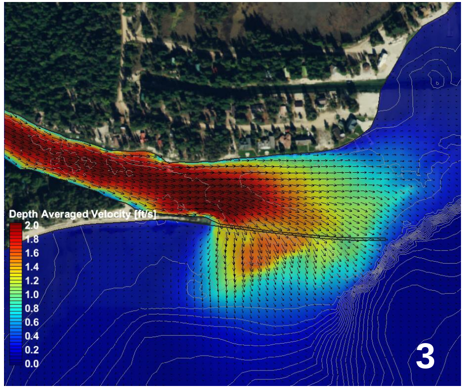
t = 483 hours (20.2 days)
Peak Velocity @ Thorofare Mouth = 2.2 ft/s

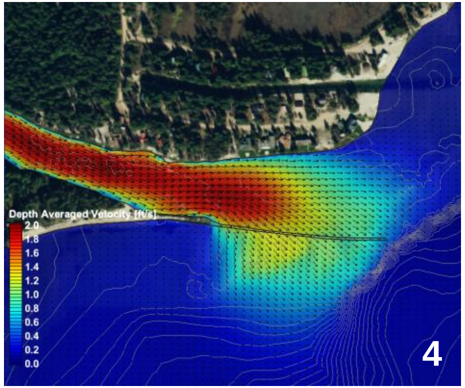
MODELING RESULTS – EXISTING CONDITIONS SUMMARY (Velocity)

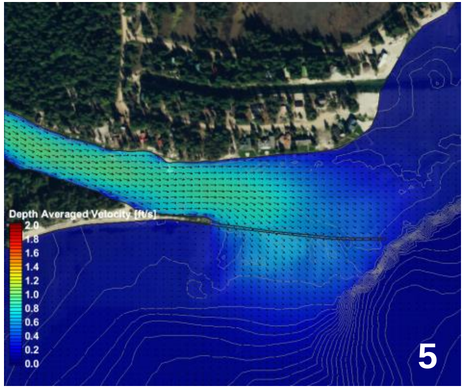






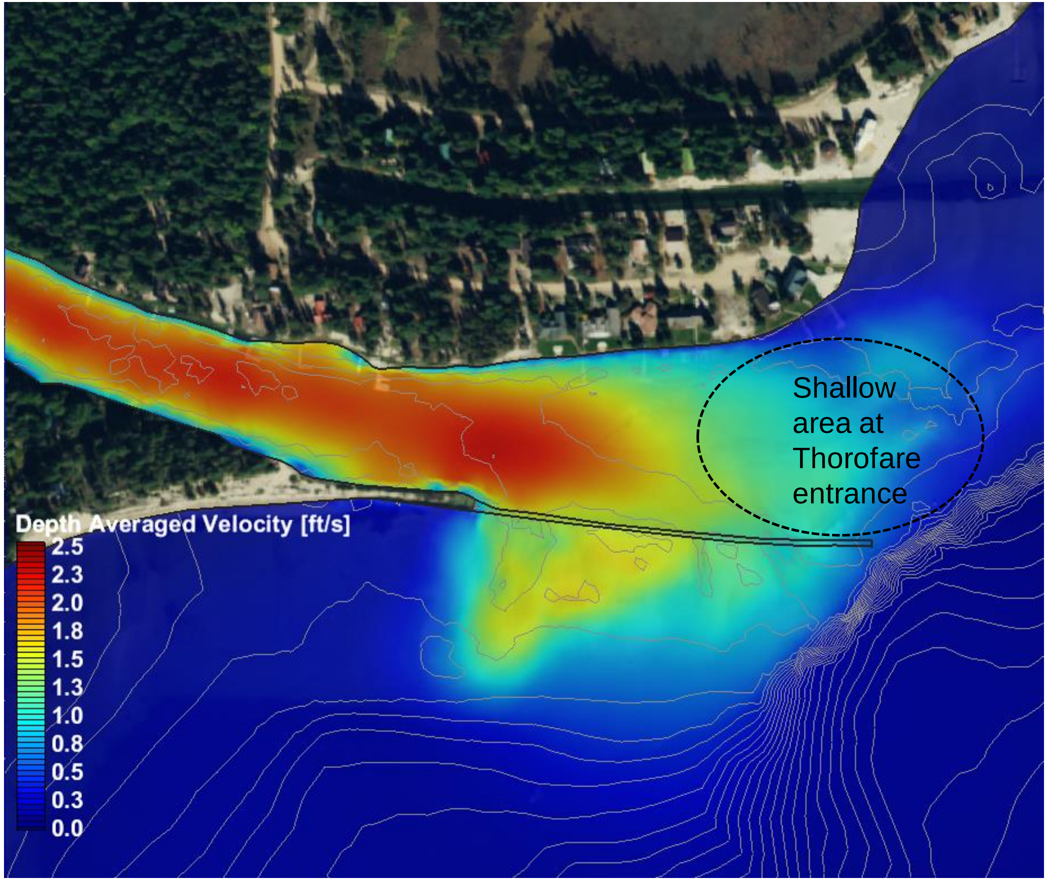




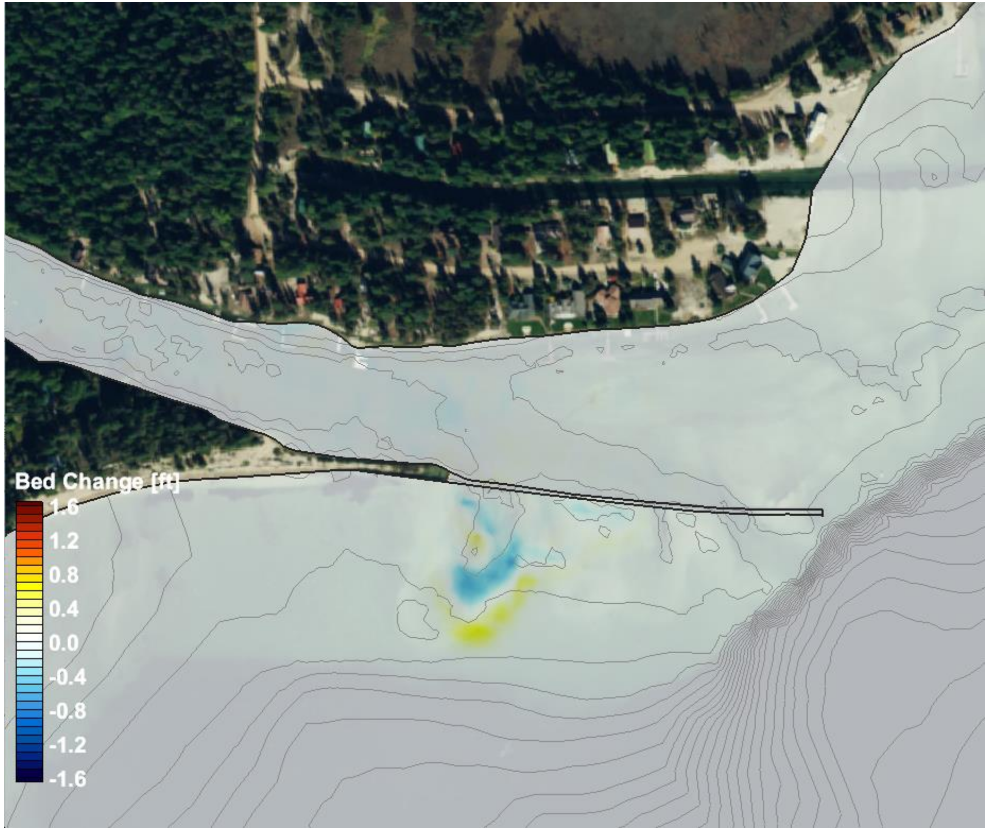


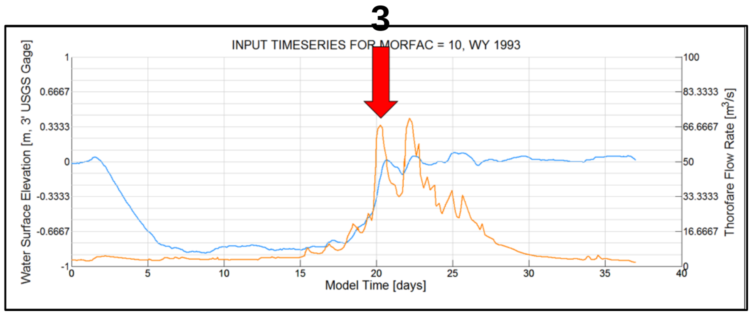
MODELING RESULTS – EXISTING CONDITIONS RESULTS (Velocity)

- Maximum Velocity: 2.2 ft/s
- Flow spreading along breakwater is represented in model results
- Significant drop in velocity at approximately ½ the length of the breakwater
- Velocity reduction zone corresponds with area of reduced depth (shoal) at entrance to Thorofare



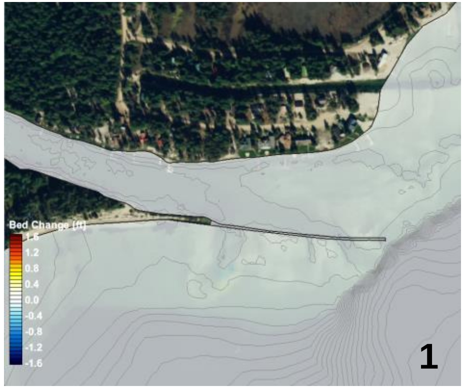
MODELING RESULTS – EXISTING CONDITIONS (bed change snapshot #3)



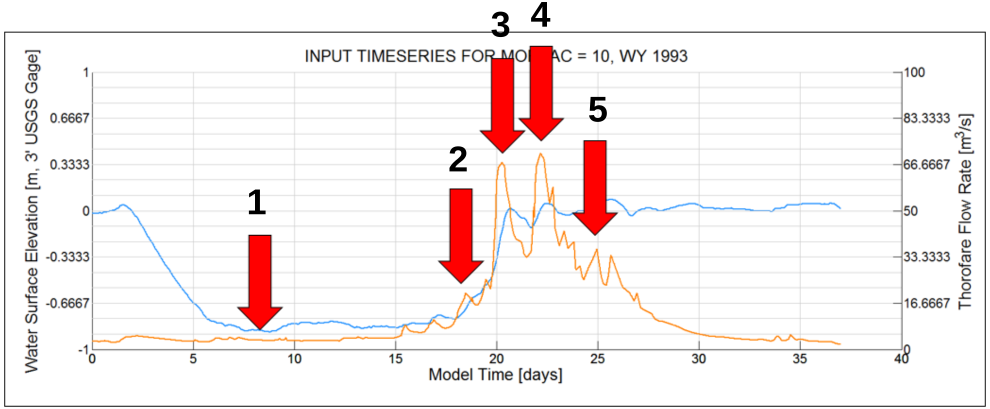


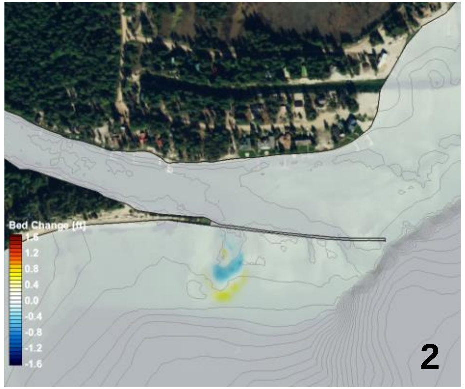
t = 483 hours (20.2 days)

MODELING RESULTS – EXISTING CONDITIONS (sediment transport summary)



1





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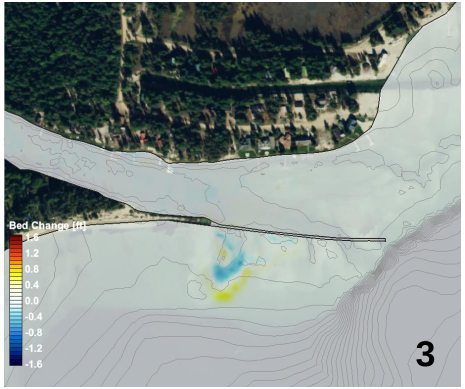

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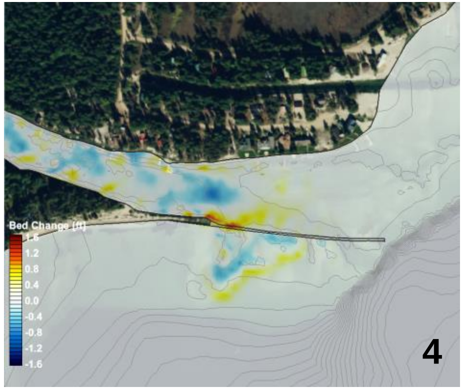
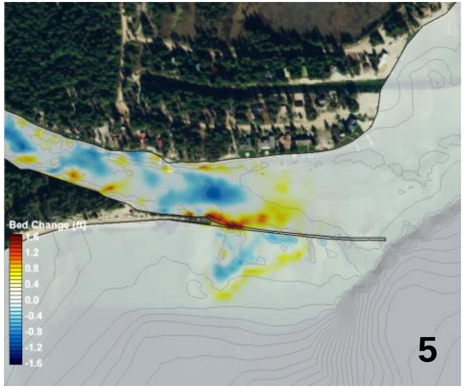
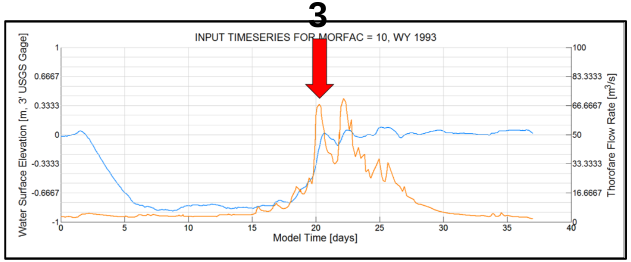


4

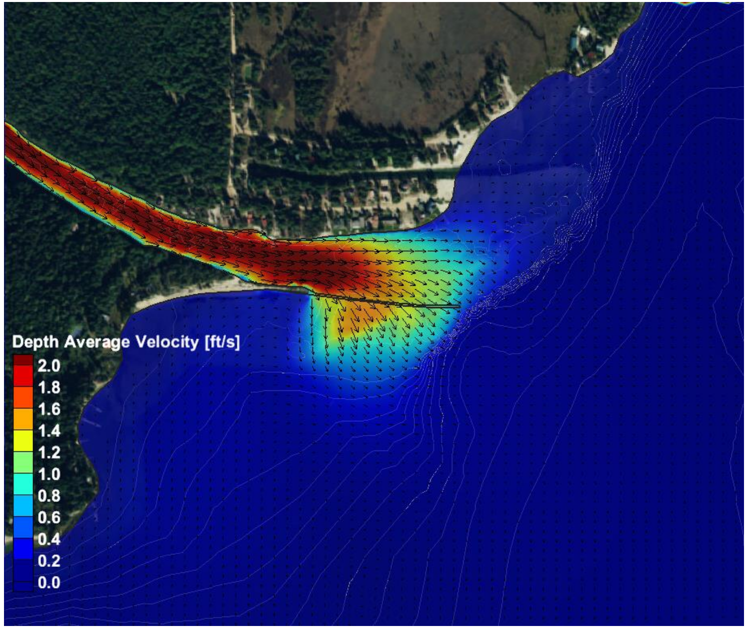


5

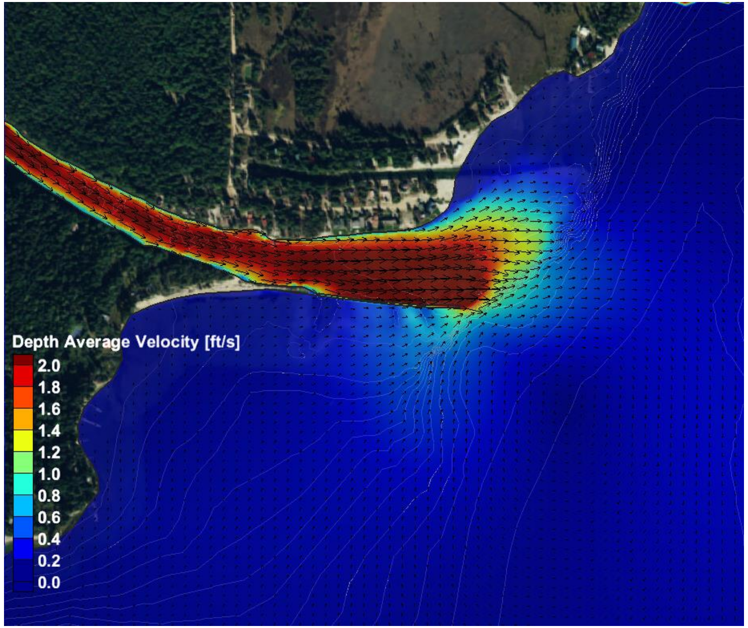
MODELING RESULTS – SENSITIVITY TESTING



Snapshot of depth-averaged velocity for Porous vs. Solid Breakwater



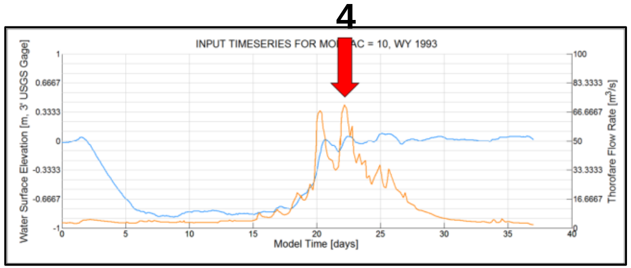
Porous Breakwater

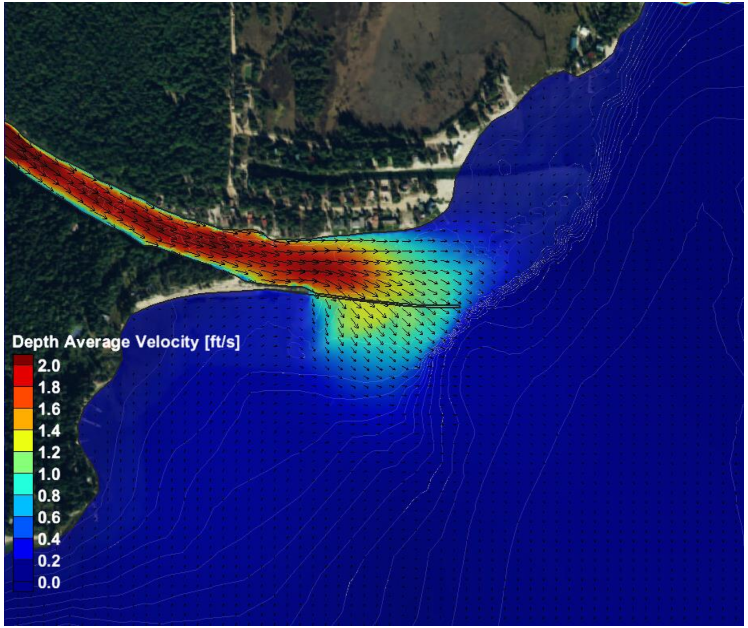


Solid Breakwater

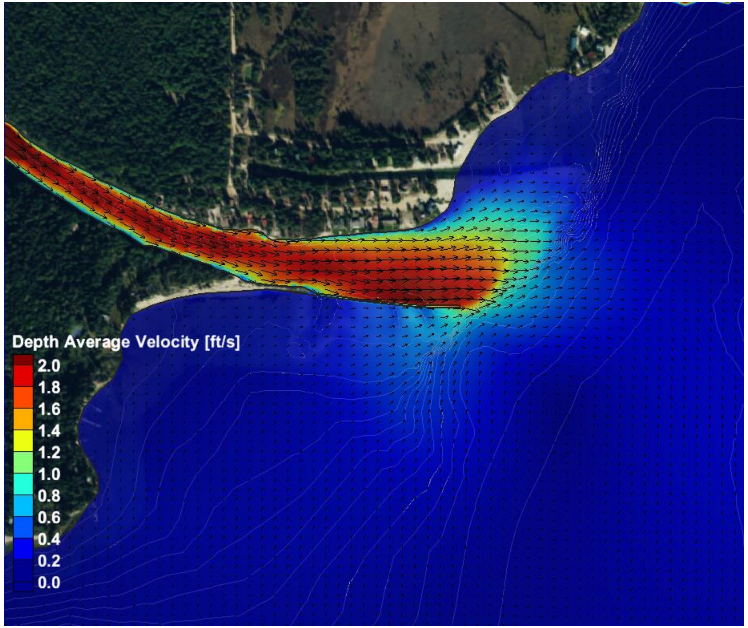
MODELING RESULTS – SENSITIVITY TESTING

Snapshot of depth-averaged velocity for Porous vs. Solid Breakwater



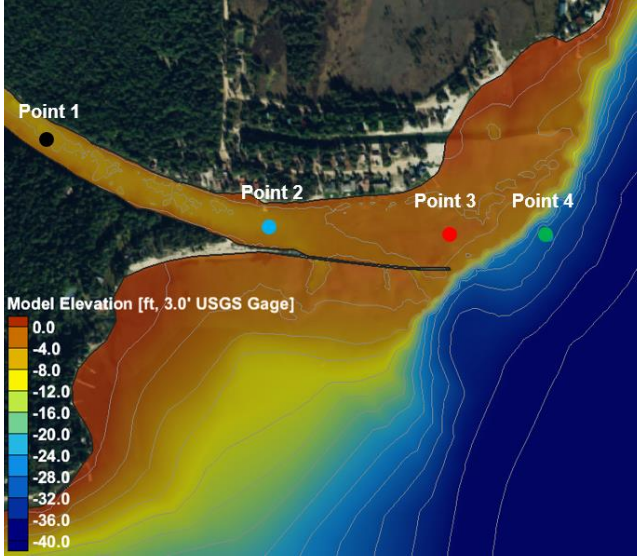


Porous Breakwater

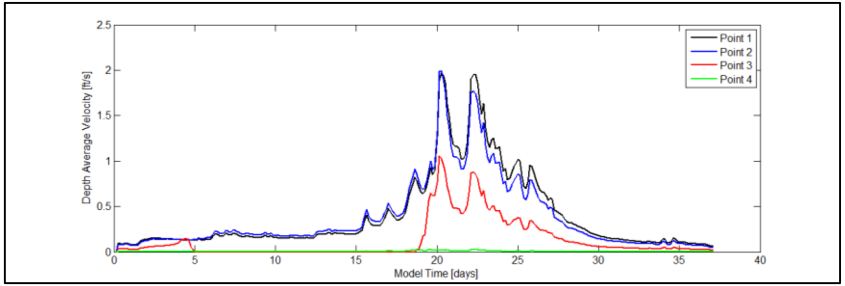


Solid Breakwater

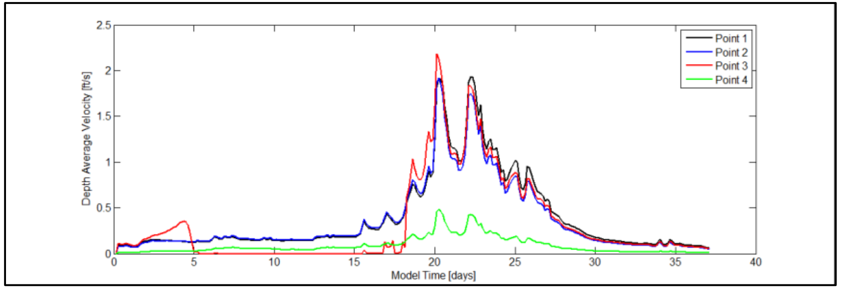
MODELING RESULTS – SENSITIVITY TESTING



- Time histories of depth-averaged velocity in Thorofare extracted at Points 1 to 4 for Porous vs. Solid Breakwater
- Solid Breakwater better confines the flow to Thorofare compared to Porous Breakwater and as a result, higher velocities are observed at Points 3 and 4



Porous Breakwater

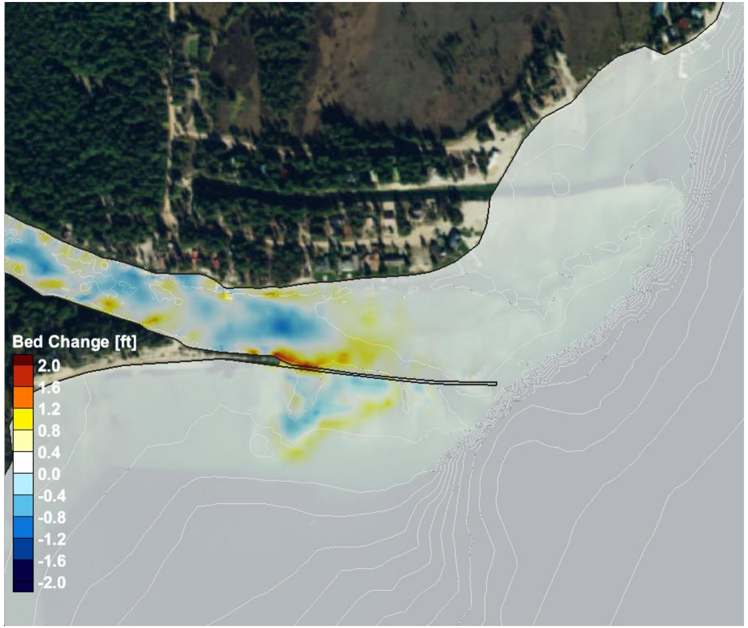


Solid Breakwater

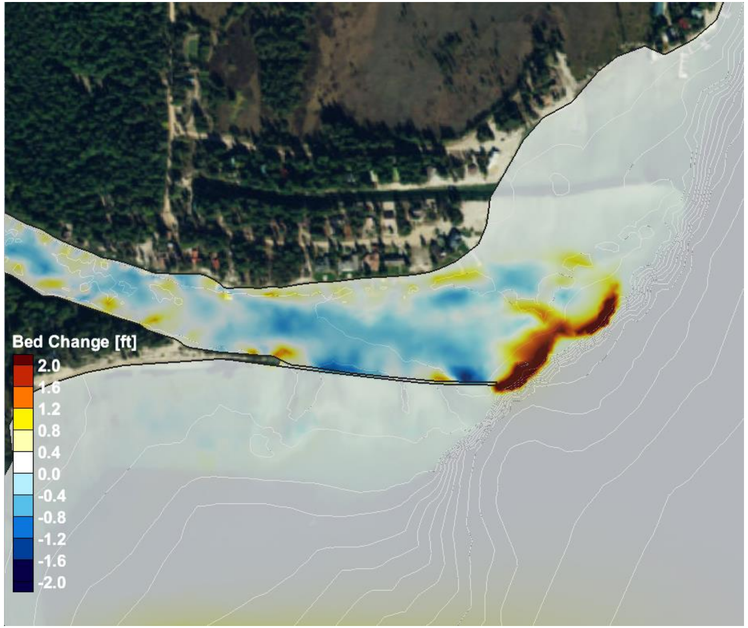
MODELING RESULTS – SENSITIVITY TESTING

Maximum Bed Change (sedimentation/erosion) during the simulation period

Solid Breakwater transports material and deposits them in deeper water past the Breakwater end.



Porous Breakwater



Solid Breakwater

CONCLUSIONS – HYDRAULIC ANALYSIS

- Numerical modeling simulation can reasonably represent hydraulic & sediment transport processes for existing conditions
- Numerical modeling simulation can be used to evaluate alternatives on a qualitative comparative analysis basis
- Maximum velocity in Thorofare coincides with maximum spring runoff
- Sensitivity testing showed that Solid Breakwater compared to Porous Breakwater better confines the flow to Thorofare and as a result, higher velocities will occur in the Thorofare mouth
- Sensitivity testing showed that Solid Breakwater compared to Porous Breakwater transports more sediments along Thorofare and deposits them in deeper water
- Flow diversion in Thorofare doesn’t occur with non-porous feature.
- Improvement alternatives should consider a solid breakwater and either extended length and/or a change in orientation (alignment angle relative to the existing).

MOTT MACDONALD

Priest Lake Water Management Study

Hydraulic & Sediment Transport Analysis – 8/17/2017

Idaho Water Resource Board

Working Document – v0

D.3 Thorofare Improvement Alternatives Screening Summary

MOTT MACDONALD

Priest Lake Water Management Study

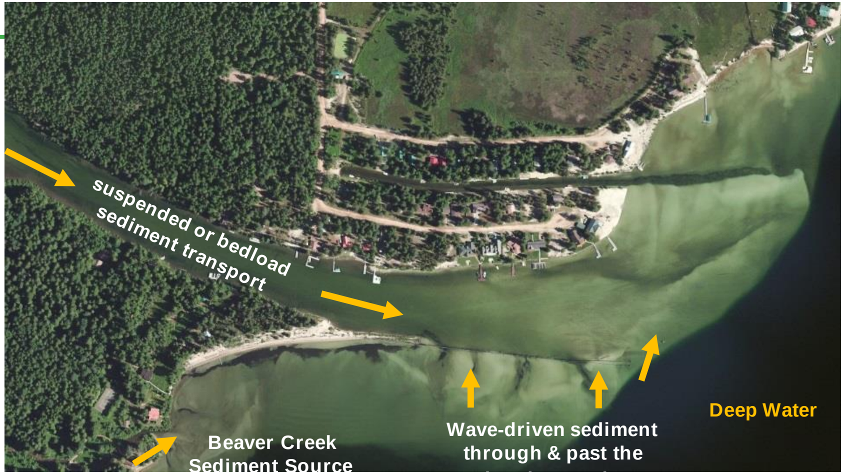
Thorofare Improvements Alternatives Screening Summary – 9/13/2017

IDAHO WATER RESOURCE BOARD

THOROFARE – POTENTIAL IMPROVEMENT ALTERNATIVES

- Conduct evaluation of alternatives shortlisted through screening process
- Incorporate input provided through stakeholder and public outreach process
- Conduct additional numerical modeling to evaluate alternatives to aid in the selection of a recommended improvement.

THOROFARE NAVIGATION IMPROVEMENTS – SCREENING ANALYSIS BACKGROUND



ISSUES

- Motorized boat access (recreational & emergency services) and navigation is challenging at the Thorofare mouth; need to increase depths
- Deteriorated breakwater structure with periodic maintenance and repair history

CONSIDERATIONS

- Thorofare flow splitting ~ less effective sediment transport
- Sediment migration from lake shoreline into Thorofare
- Repeated failures of breakwater increasing extent of Thorofare flow splitting

IMPROVEMENTS

- Geomorphic & Hydraulic analysis indicates the following:
  - Reduce sediment migration from lake shoreline into Thorofare (tighten up area from spit to lake deep water)
  - Utilize an impermeable sediment retention structure or breakwater

THOROFARE – POTENTIAL IMPROVEMENT ALTERNATIVES

Improvement Alternatives:

1) No Action (maintain existing)
   - No repairs, improvements or dredging

2) Removal of Breakwater
   - Complete Removal with dredging

3) Rehabilitate Existing Porous Breakwater
   - Rehab existing damaged areas, continue repairs in future, conduct dredging to restore navigation

4) Replace Existing Porous Breakwater with Sediment Retention Feature
   - Construction new feature to replace breakwater and conduct dredging to restore navigation; see next slide for details

5) In channel flow diversion
   - Construction new feature to supplement breakwater and conduct dredging to restore navigation; see next slide for details

▨ = Focus Area for Thorofare Improvements

ALTERNATIVE NO. 1 – NO ACTION

- Continued shoaling at entrance
- Continued flow diversion at breakwater
- Leakage of sediment from lake shoreline into Thorofare
- Formation of channels through breakwater
- Difficult to maintain navigation channel

ALTERNATIVE NO. 2 – REMOVE BREAKWATER

- **Continued shoaling at entrance**
- **Increased flow diversion at breakwater**
- **Greater leakage of sediment from lake shoreline into Thorofare**
- **Increased formation of distributary channels through breakwater**
- **Maintaining navigation channel extremely difficult**

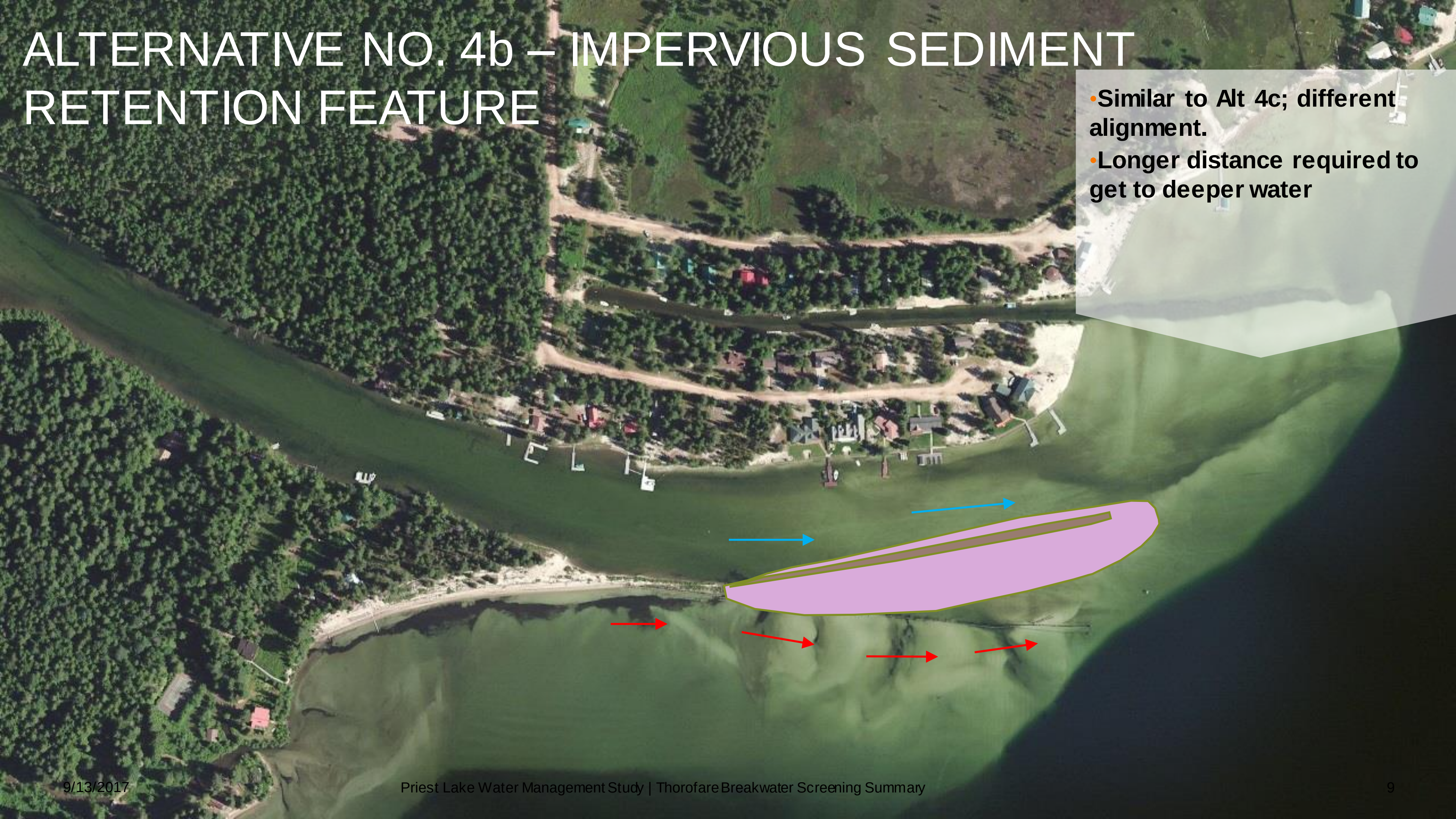
ALTERNATIVE NO. 3 – REPAIR POUROUS BREAKWATER

- **Similar to historical conditions (past 10 years)**
- **Continued maintenance**
- **Continued lack of depth at entrance channel**
- **Frequent dredging**

ALTERNATIVE NO. 4a – IMPERVIOUS SEDIMENT RETENTION FEATURE

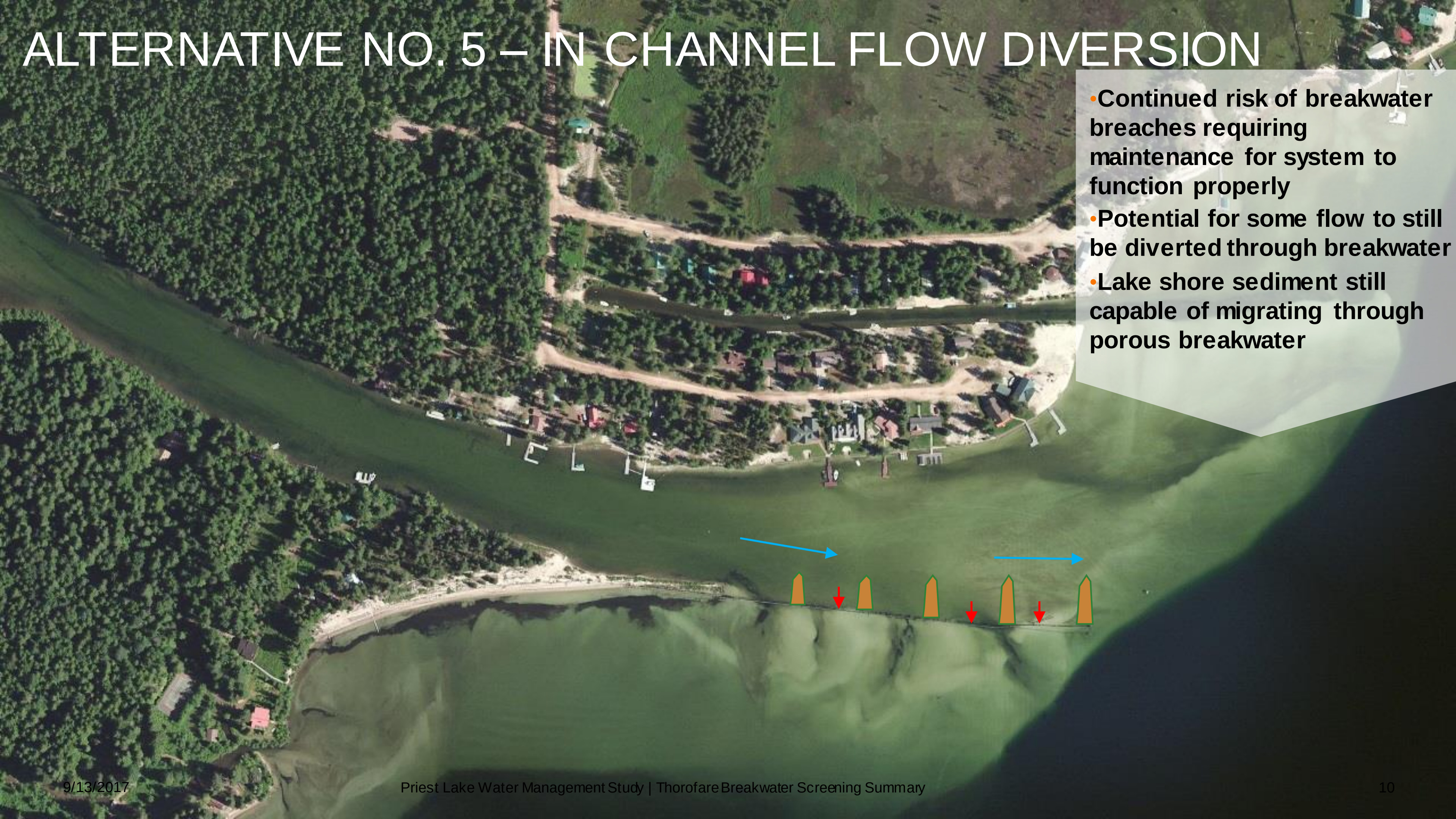
- Reduced lake shore sediment transport into Thorofare
- Increased flow velocity in Thorofare to transport sediment into Lake

ALTERNATIVE NO. 4b – IMPERVIOUS SEDIMENT RETENTION FEATURE



- Similar to Alt 4c; different alignment.
- Longer distance required to get to deeper water

ALTERNATIVE NO. 5 – IN CHANNEL FLOW DIVERSION



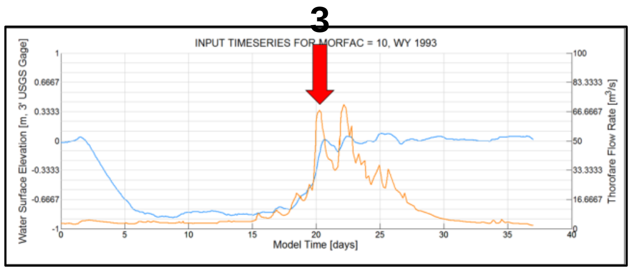
- **Continued risk of breakwater breaches requiring maintenance for system to function properly**
- **Potential for some flow to still be diverted through breakwater**
- **Lake shore sediment still capable of migrating through porous breakwater**

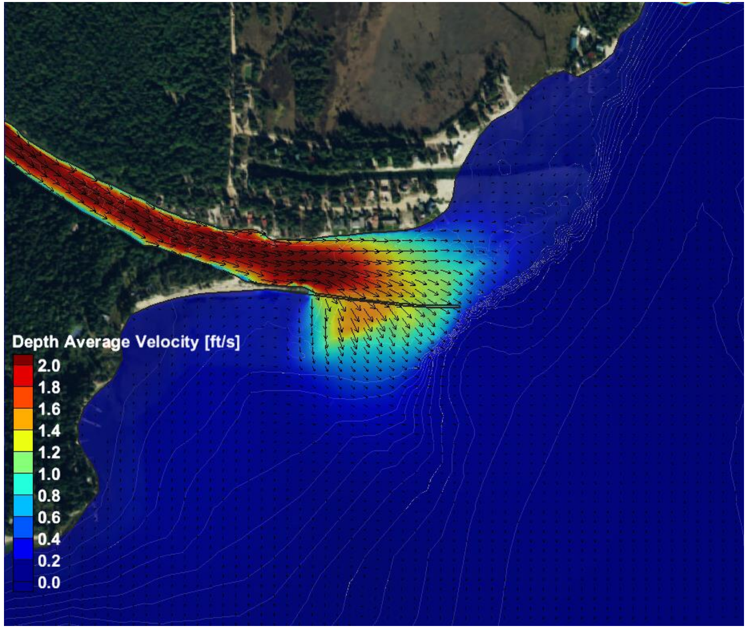
ALTERNATIVE 6 – Partial In Channel Flow Deflection

- Continued shoaling at entrance
- Increased flow diversion at end of breakwater
- Increased formation of distributary channels through end of breakwater
- Maintaining navigation channel difficult

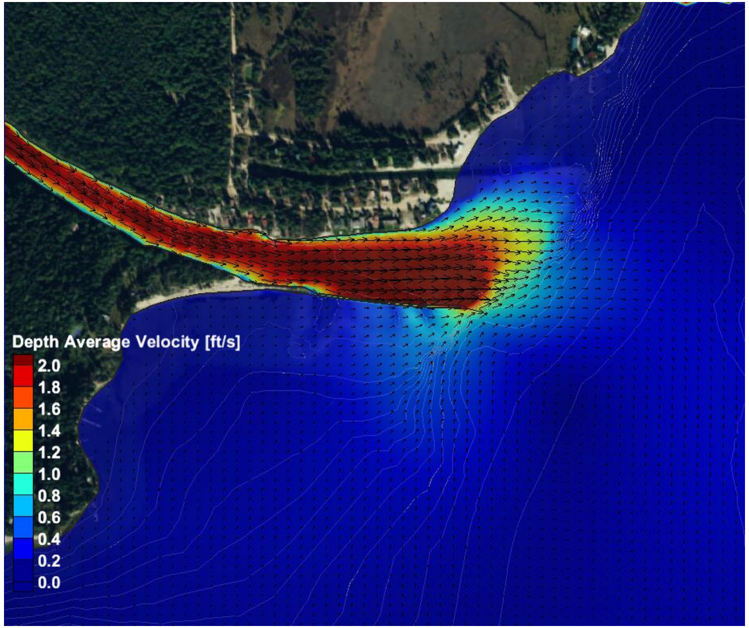
MODELING RESULTS – SENSITIVITY TESTING

Snapshot of depth-averaged velocity for Porous Breakwater vs. Solid Feature





Porous Breakwater

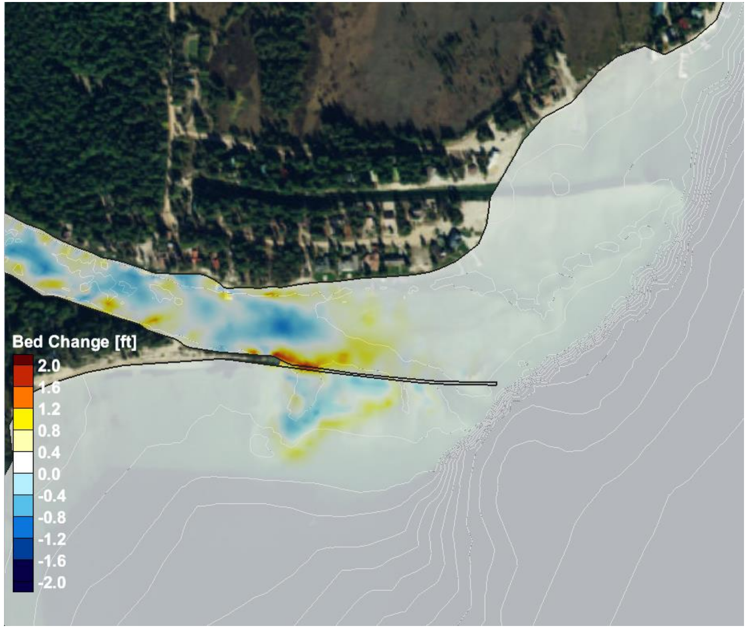


Solid Feature

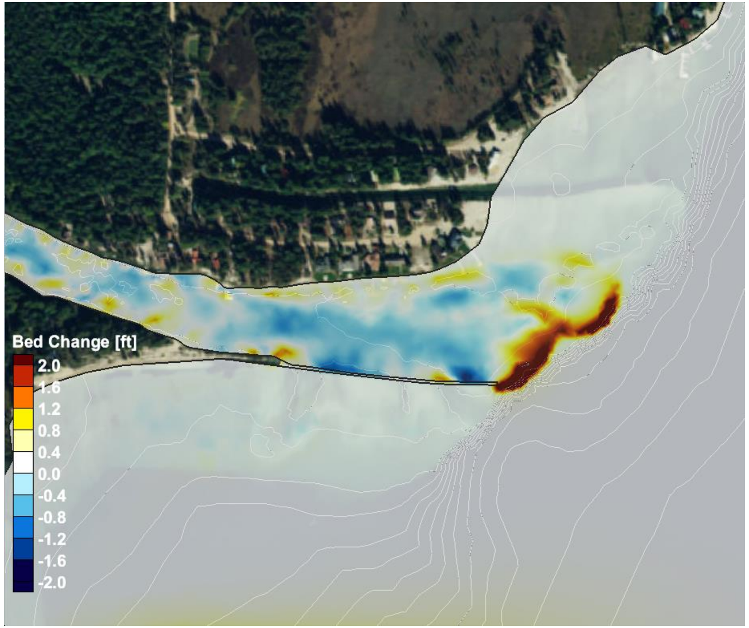
MODELING RESULTS – SENSITIVITY TESTING

Maximum Bed Change (sedimentation/erosion) during the simulation period

Solid Feature: transports material and deposits them in deeper water past the feature end.



Porous Breakwater



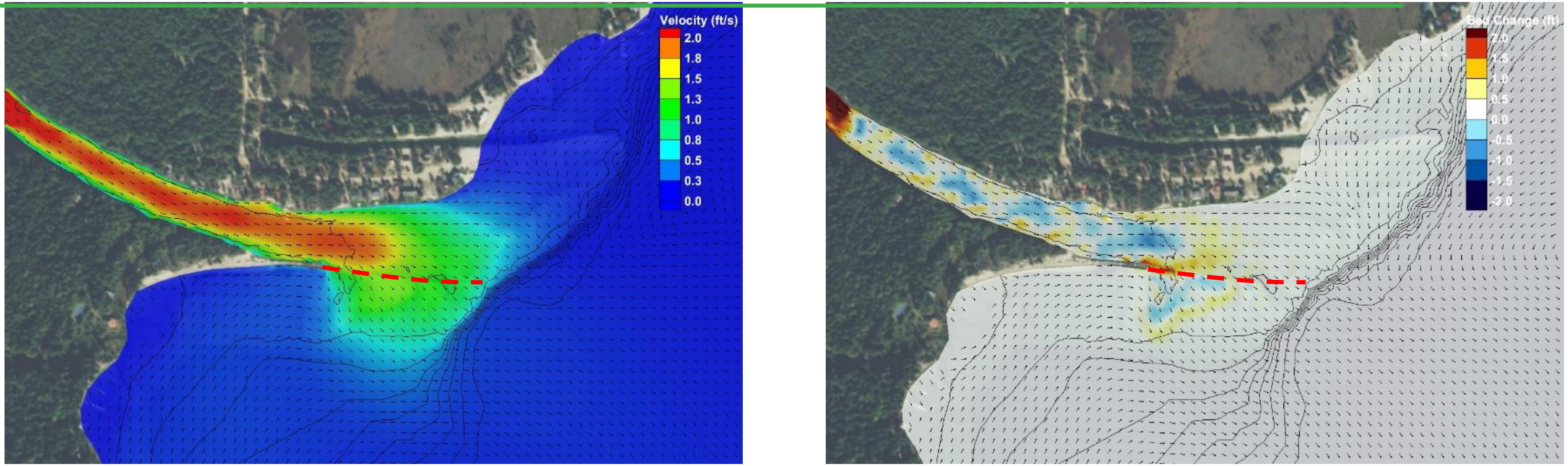
Solid Feature

THOROFARE –ALTERNATIVES EVALUATION

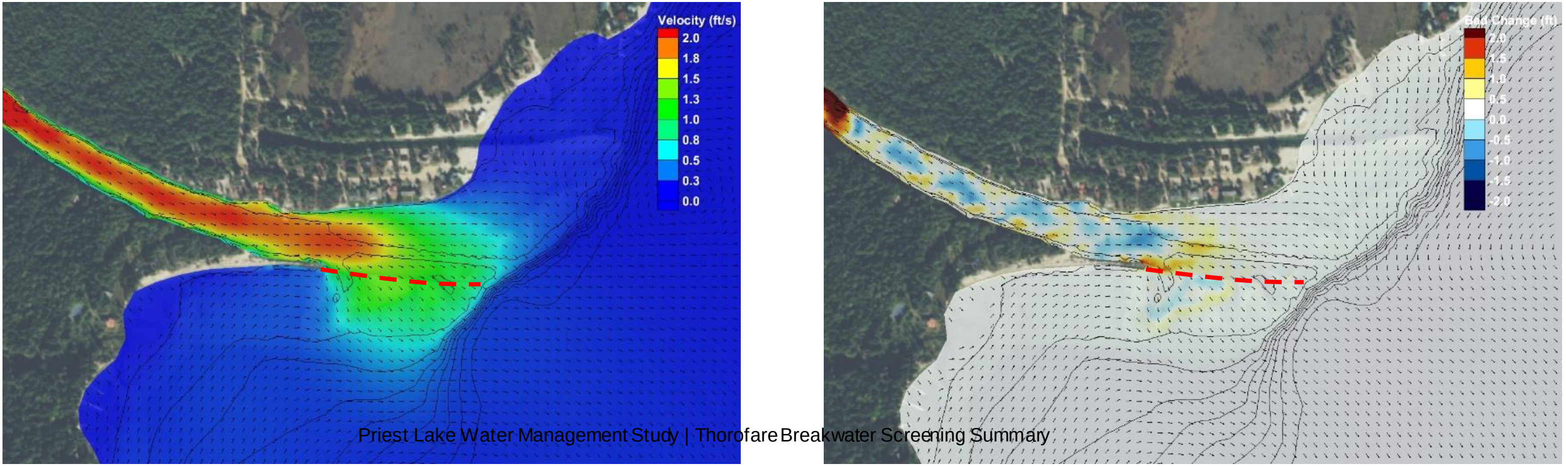
- Prescreening Assessment
  - Alternative 2 not feasible as it doesn't meet any of the stated study goals. Therefore not evaluated in subsequent more detailed analysis.
  - Alternatives 1 and 3 are similar from hydrodynamic perspective.
- Evaluate alternatives utilizing numerical modeling to analyze Thorofare discharge and sediment transport
- Utilize hydraulic modeling results from hydraulic assessment to further evaluate alternatives.
  - See Hydraulic & Sediment Transport Analysis summary appendix.
- More detailed analysis conducted on solid vs. porous breakwater to evaluate performance of new alternative relative to existing conditions
- Evaluate alternatives with and without an assumed navigation channel dredging.
  - For navigation channel dredging size assumptions, see Thorofare Dredging Constructability Assessment

MODELING RESULTS – Existing Pervious Breakwater (Alts 1 & 3)

Pervious BW

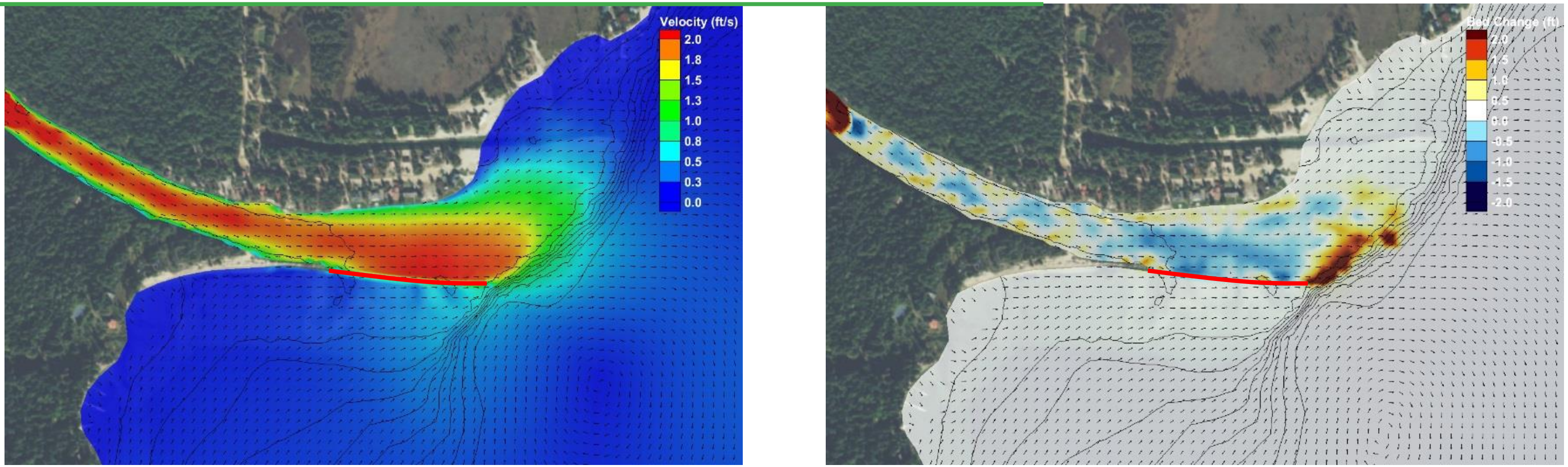


Pervious BW **with Dredge**

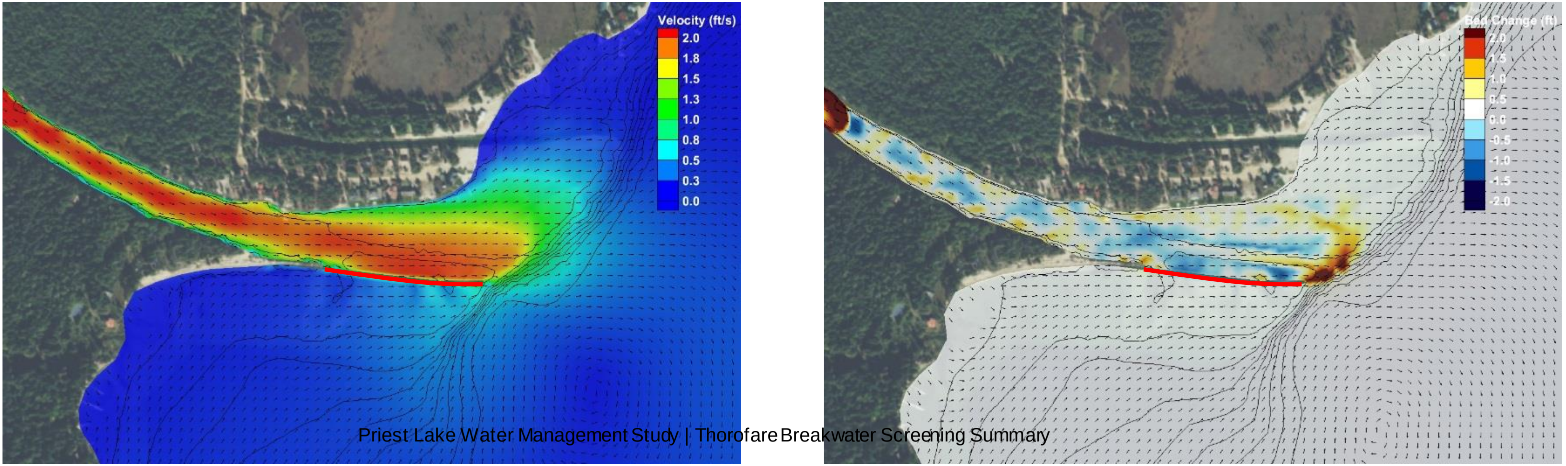


MODELING RESULTS – New Solid Breakwater (Alt 4)

Solid Breakwater

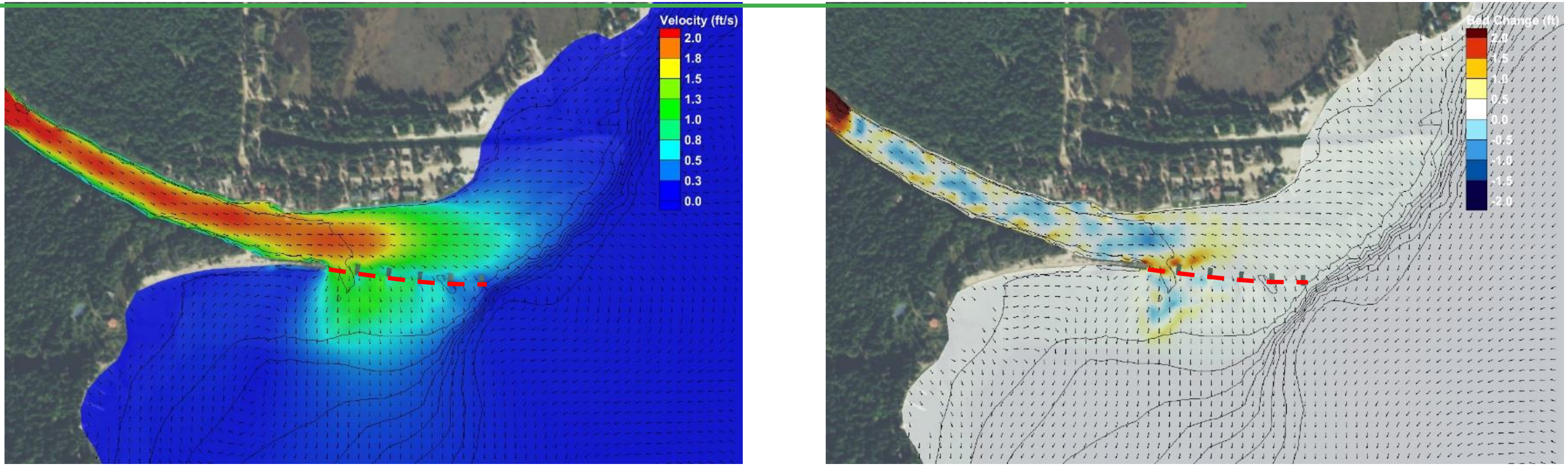


Solid Breakwater **with Dredge**

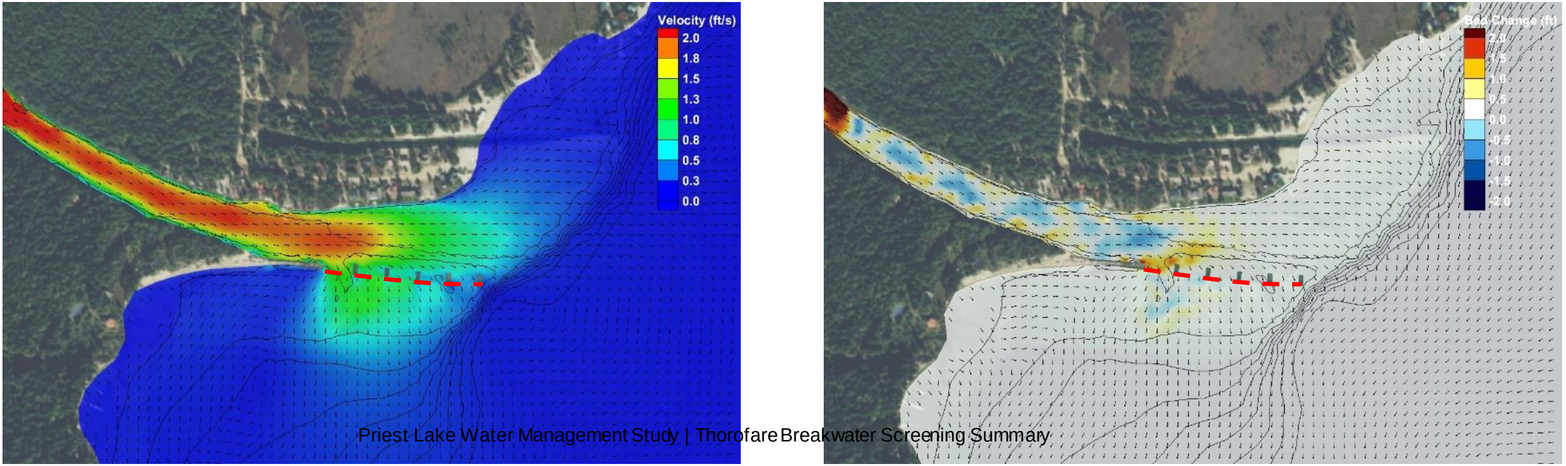


MODELING RESULTS – In Channel Flow Diversion (Alt 5)

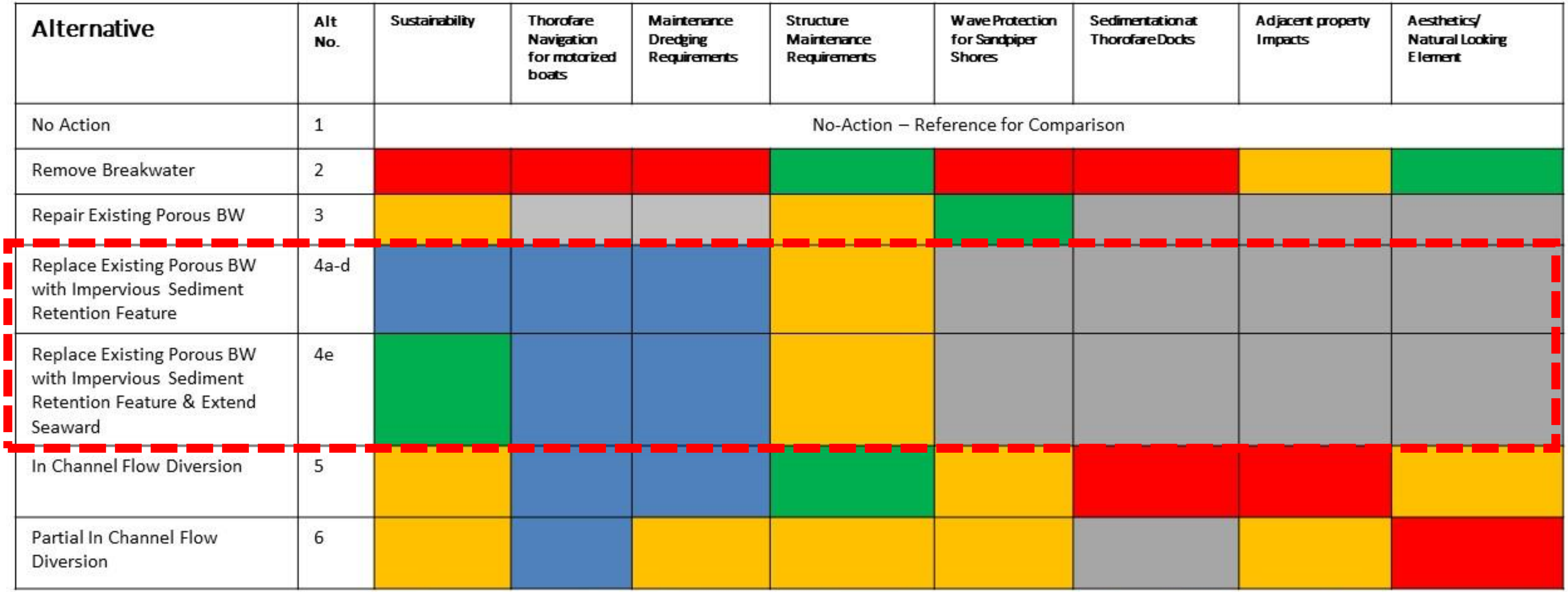
Pervious BW with Barbs



Pervious BW with Barbs **and Dredge**



THOROFARE/BREAKWATER IMPROVEMENT – ALTERNATIVE SCREENING



Alternative	Alt No.	Sustainability	Thorofare Navigation for motorized boats	Maintenance Dredging Requirements	Structure Maintenance Requirements	Wave Protection for Sandpiper Shores	Sedimentation at Thorofare Docks	Adjacent property Impacts	Aesthetics/ Natural Looking Element
No Action	1	No-Action – Reference for Comparison							
Remove Breakwater	2	🟥	🟥	🟥	🟩	🟥	🟥	🟨	🟩
Repair Existing Porous BW	3	🟨	⬜	⬜	🟨	🟩	⬜	⬜	⬜
Replace Existing Porous BW with Impervious Sediment Retention Feature	4a-d	🟦	🟦	🟦	🟨	⬜	⬜	⬜	⬜
Replace Existing Porous BW with Impervious Sediment Retention Feature & Extend Seaward	4e	🟩	🟦	🟦	🟨	⬜	⬜	⬜	⬜
In Channel Flow Diversion	5	🟨	🟦	🟦	🟩	🟨	🟥	🟥	🟨
Partial In Channel Flow Diversion	6	🟨	🟦	🟨	🟨	🟨	⬜	🟨	🟥

Change with respect to Status Quo (No Action):

🟩 Significantly Positive

🟦 Moderately Positive

⬜ No/Negligible Change

🟨 Moderately Negative

🟥 Significantly Negative

THOROFARE –ALTERNATIVES EVALUATION

- Recommended Alternative Concept to be considered for additional analysis
    - Alternative 4 – Replace existing porous structure with impervious sediment retention feature (structure)
    - Conduct additional evaluation to determine alignment, extension and material type (pile, rubble mound, cobble/gravel, etc..)
    - See Thorofare Breakwater Evaluation Summary for more details

MOTT MACDONALD

Priest Lake Water Management Study

Thorofare Improvements Alternatives Screening Summary – 9/13/2017

IDAHO WATER RESOURCE BOARD

Working Document

E. Implementation

E.1 Thorofare Improvements – Dredging Assessment and Constructability

E.2 Thorofare Improvements – Breakwater Alternatives Evaluation

E.3 Conceptual Engineering Plans

E.4 Construction Cost Estimates

E.5 Environmental Permitting Requirements

E.6 IWRB October 24 Meeting Presentation

E.1 Thorofare Improvements – Dredging Assessment and Constructability

MOTT MACDONALD

Priest Lake Water Management Study

Thorofare Improvements – Dredging Assessment & Constructability – 9/26/17

IDAHO WATER RESOURCE BOARD

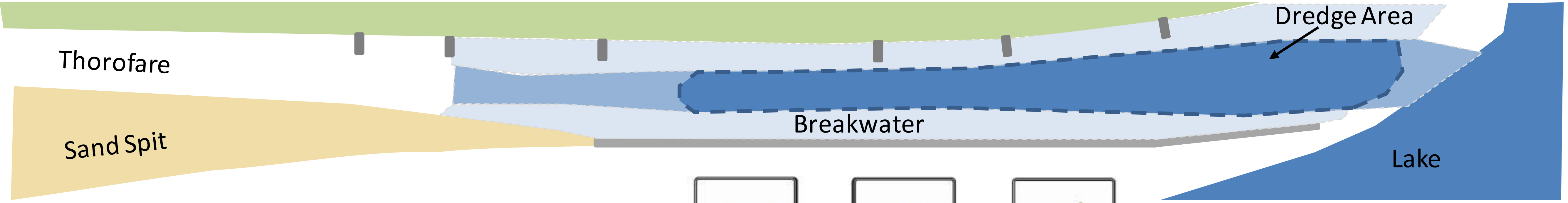
Dredging Assessment

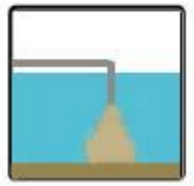
- Purpose. Review dredge location, volumes, dredging equipment and disposal/beneficial reuse options and concepts to aid in the development of a recommended Thorofare Navigation Improvement concept and corresponding construction costs.

Priest Lake Water Management Project

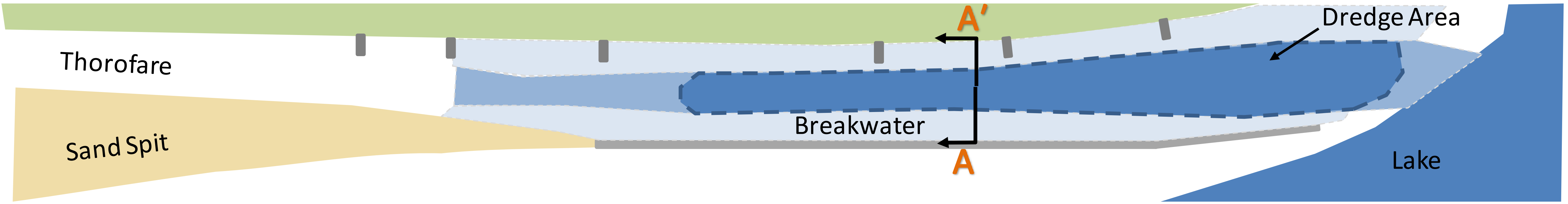
Dredging and Disposal Concept Screening

08/14/2017



			
Open Water Placement 	✓	✓	✗
Nearshore Beach Nourishment 	✓	✓	✗
Barge to Upland Reuse 	✗	✓	✓
Shallow Water Habitat 	✓	✓	✓
Offsite Upland 	✗	✓	✓

Dredging Extents and Depths

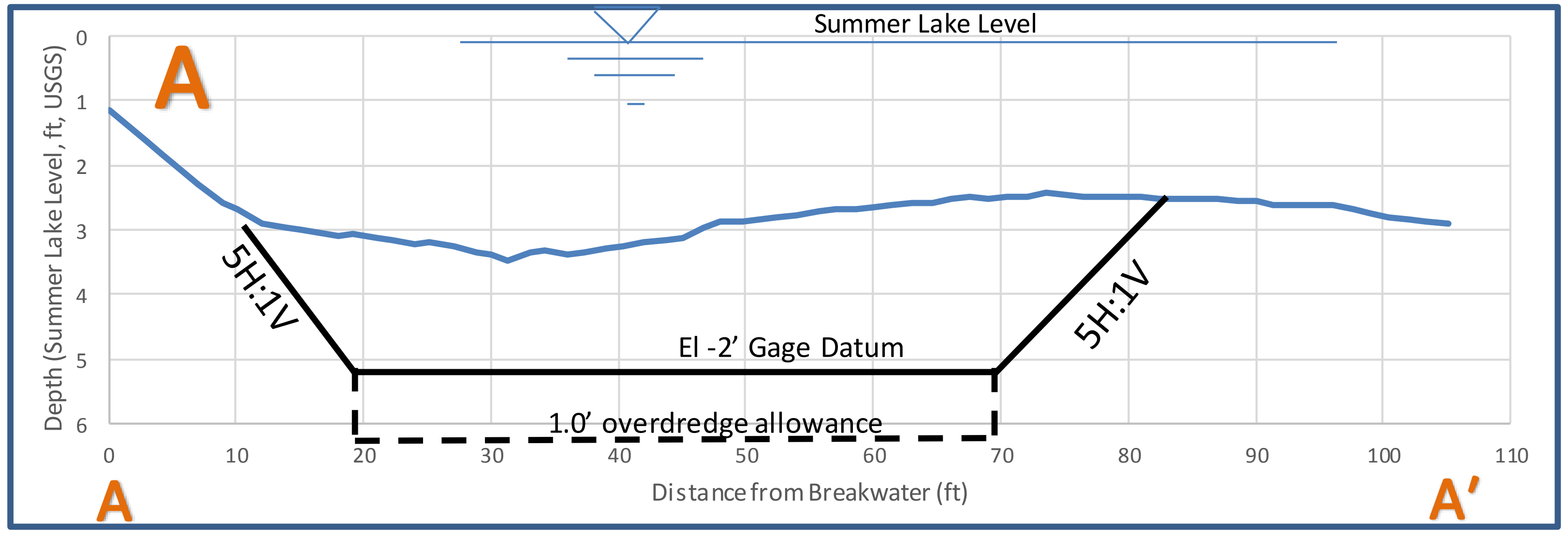


Dredging Width

- 50'

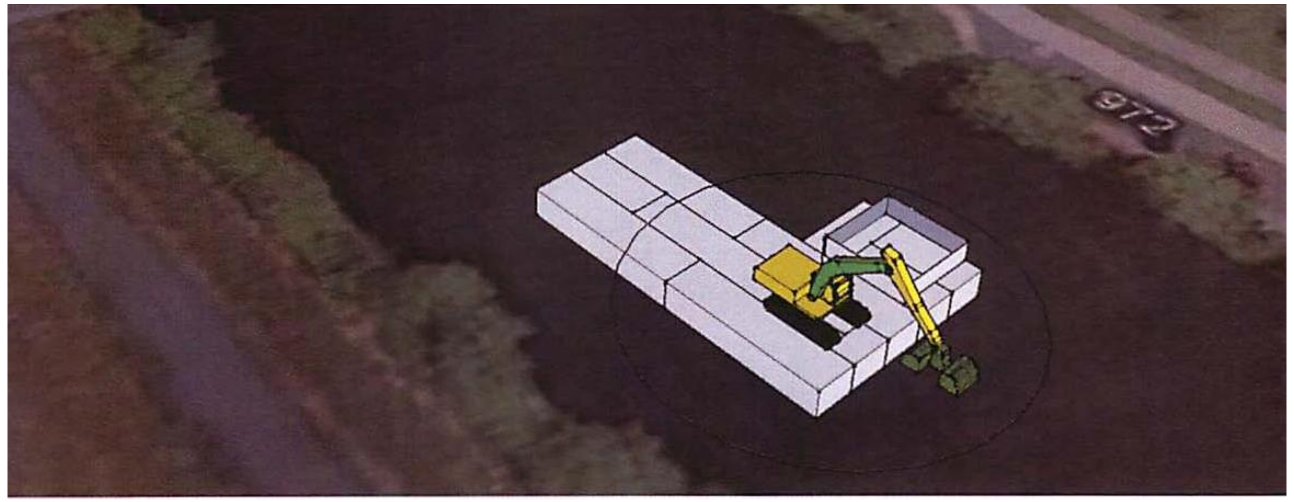
Estimated Dredging Volumes

- 4.0' Depth (El -1' Gage Datum)
  - 5,000 to 7,500 cy
- 5.0' Depth (El -2' Gage Datum)
  - 9,000 to 11,000 cy

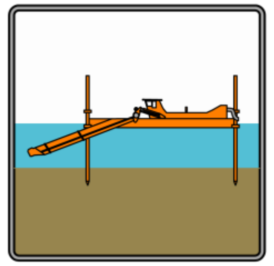


Note: Dredging width to be finalized; 50' is estimate for assessment phase only. Likely range is 40 to 75' width.

Floating Clamshell Dredging



- Requires 4’ to 5’ water depth for loaded barge
- Limited to summer season construction due to depth requirements
- Ease of access from nearby boat launch
- Potential lower production rate dredging and disposal operation depending on disposal site locaiton
- Required offload & Re-handling facility nearby for upland disposal.
- Aquatic placement directly offloaded.



Hydraulic Cutter Head Dredging

Mobilization – Portable Hydraulic Dredge

- Minimal site impacts; HDPE pipeline along lake shoreline and floating to upland nearshore or open water placement.
- Feasible for fall construction season
- Limited by identification of nearby placement site
- Ease of mobilization to the site from nearby boat launch for placement with crane

Operations – Portable Hydraulic Dredge

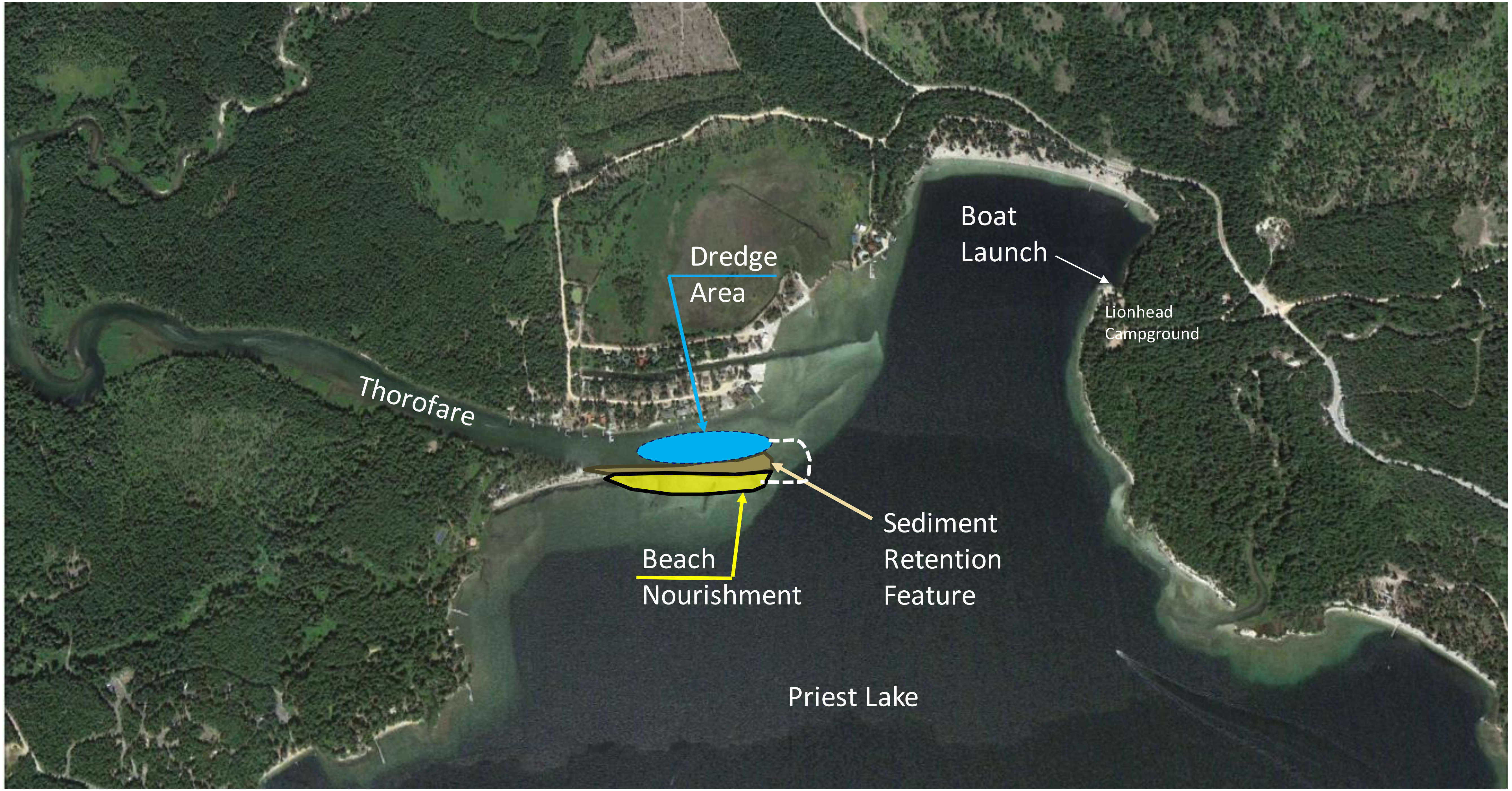
Temporary Berm Dredging

- Feasible for low water period (late fall/winter season) if direct upland access were available.
- Temporary berm required; can be used for breakwater repair/replacement (dual purpose)
- Requires land side access; likely not available

Nearshore Placement

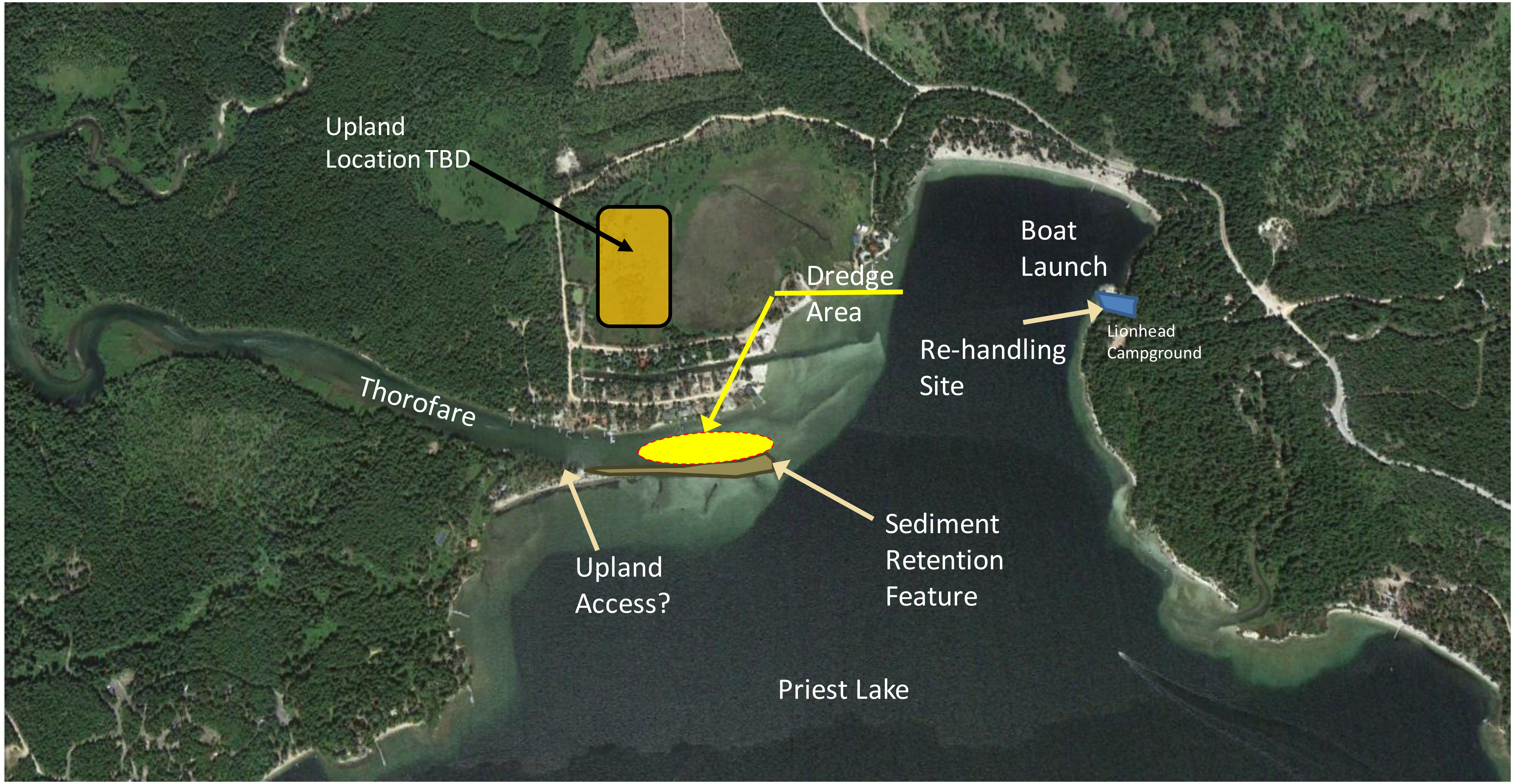
-Either Hydraulic or barge based. Hydraulic more economical
-Determine capacity of available and feasible sites
-Shallow water placement for nearshore beneficial reuse

Beach Nourishment



-Either Hydraulic or barge based. Hydraulic more economical
-Determine capacity of available and feasible sites
-Shallow water placement for nearshore beneficial reuse

Upland Disposal



-Either Hydraulic or barge based. Very limited sites for hydraulic (nearby Lake only). Barge more feasible but requires re-handling area such as boat ramp. Material rehandled into dump trucks for disposal offsite. A lot more cost. Berm dredging could be feasible if upland access was made available.

Construction Access

- Access to the site will be critical for ease of construction and to reduce construction costs.
- Boat Ramp such as Lionhead campground for launching of marine equipment and staging will be needed.
- Upland access from adjacent private parcel could aid in reducing construction costs for both dredging and the sediment retention feature construction.


Boat Ramp – Lionhead Campground


Adjacent Upland Parcel

Regulatory Considerations

- Water Quality (401 Certification). Both dredging and aquatic placement areas will need to meet turbidity and water quality protection requirements. BMP's to be developed to ensure compliance with WQ standards.
- Sediment Evaluation Framework (SEF). Sampling & testing of proposed dredged sediments to be conducted during regulatory process. Ensure material meets requirements for in-water placement and protection of water quality. U.S. Army Corps of Engineers is the lead regulatory agency on this process.
- Aquatic Placement. Preliminary consultation with regulatory agencies indicates nearshore nourishment may be acceptable but deep water disposal likely not.
- Upland Placement. Find location of nearby, upland land for receipt of material. No wetlands.

Dredging Summary

- Disposal/Placement Site. Identification of potential sites is critical to cost evaluation and determination of preferred method.
- Regulatory.
  - Placement of dredged material below ordinary high water for beach fill but not disposal in deep water.
  - Sediment Evaluation Framework – sampling & testing of materials required; site is located in low risk area
- Desired Depth. Greater depths will increase volume and may result in diminishing returns for performance.
- Advanced Maintenance Dredging. Additional dredging depth to extend time period between dredging actions; limit to depth for added value.
- Dredging Methods. Dependent on cost, timing of water levels and disposal concepts. Berm method appears to not be feasible on its own due to access limitations from the uplands.

Recommendations

- Pursue multiple dredging methods with both upland and aquatic placement to provide maximum flexibility to minimize construction.
- Investigate upland access at the Thorofare and use of nearby Fish & Game boat launch.
- Identify aquatic and nearby upland disposal sites to reduce construction costs.
- Combine dredging concept with sediment retention feature (Breakwater replacement) to ensure long term sustainability of the navigation through the Thorofare.

MOTT MACDONALD

Priest Lake Water Management Study

Thorofare Improvements – Dredging Assessment & Constructability – 9/26/17

IDAHO WATER RESOURCE BOARD

DRAFT_v0

E.2 Thorofare Improvements – Breakwater Alternatives Evaluation

MOTT MACDONALD

Priest Lake Water Management Study

Thorofare Improvements (Breakwater Alternatives Evaluation) – 9/28/2017

Idaho Water Resource Board

Thorofare Breakwater Replacement Assessment

- Purpose of Assessment
  - Review breakwater replacement structure/feature types to aid in the development of a recommended Thorofare Navigation Improvement concept and corresponding construction costs.
- Background: Refer to the following documents
  - Geomorphologic Assessment
  - Hydraulic & Sediment Transport
  - Thorofare Improvement Screening Evaluation (9/13/17)
- Alternative Type Focus
  - Screening analysis indicated a impervious structure with either an extension and/or a rotation is preferred. Structure material type to be evaluated.

THOROFARE NAVIGATION IMPROVEMENTS – SCREENING ANALYSIS BACKGROUND

ISSUES

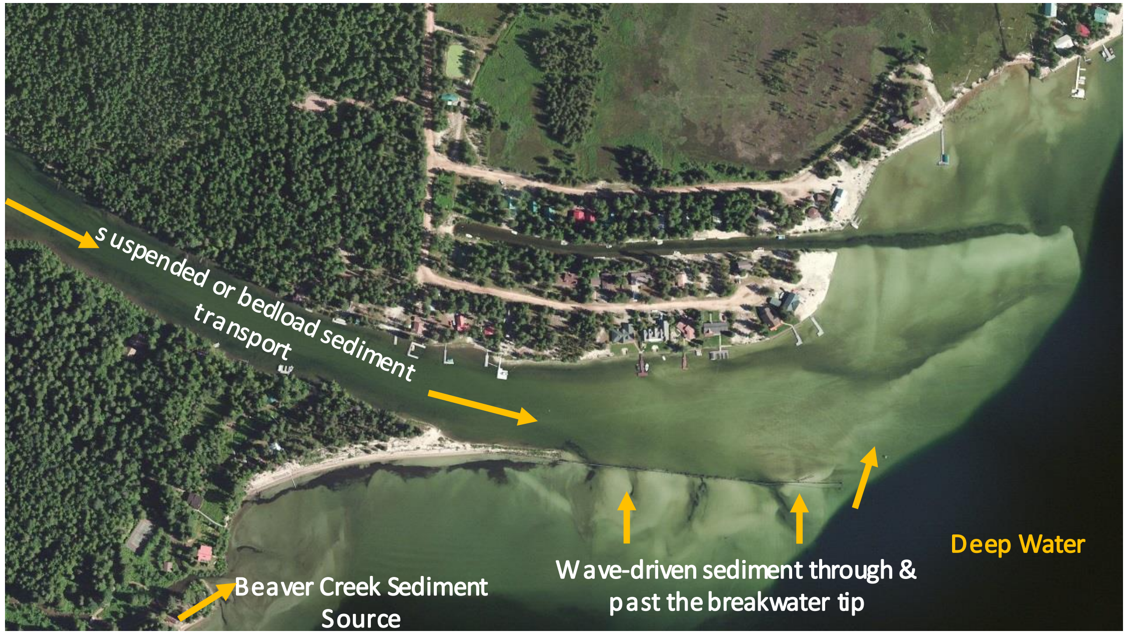
- Motorized boat access (recreational & emergency services) and navigation is challenging at the Thorofare mouth; need to increase depths
- Deteriorated breakwater structure with periodic maintenance and repair history

CONSIDERATIONS

- Thorofare flow splitting ~ less effective sediment transport
- Sediment migration from lake shoreline into Thorofare
- Repeated failures of breakwater increasing extent of Thorofare flow splitting

IMPROVEMENTS

- Geomorphic & Hydraulic analysis indicates the following:
    - Reduce sediment migration from lake shoreline into Thorofare (tighten up area from spit to lake deep water)
    - Utilize an impermeable sediment retention structure or breakwater
    - Evaluate an extension or rotation of the new structure to improve effectiveness to reduce sedimentation of Thorofare



THOROFARE – POTENTIAL IMPROVEMENT ALTERNATIVES (additional details)

- Impervious Sediment Retention Feature
  - Purpose?
    - Eliminate diversion flows from Thorofare at entrance to lake
    - Eliminate lake shore sediment from migrating into Thorofare during low Thorofare flow conditions
    - Potential for non-structural feature (in lieu of breakwater); soft stabilization
  - Material Type?
    - Option 1: Core rock, Large Wood Debris (root wads), cobble/gravel, dredged material on lake shore side (beneficial reuse of dredged material). Incorporate plantings?
    - Option 2: Rubble mound (stone with sand-tight core)
    - Option 3: Pile structure (sheetpile or other)
    - Option 4: Geotextile Tube (covered with cobble/gravel and dredged materials)
  - Profile (height). TBD but likely above high lake level.
- Flow Diversion
  - Purpose?
    - River Training structure. Divert flow, reduce cross sectional flow area to increase flow velocity. Reduce diversion flows through breakwater. Increase sediment transport capability of Thorofare
  - Material Type?
    - Large Wood Debris, Stone, Piles,
  - Profile (height). TBD, likely medium. Potentially lower than current breakwater.

THOROFARE/BREAKWATER IMPROVEMENT – ALTERNATIVE SCREENING

Alternative	Alt No.	Sustainability	Thorofare Navigation for motorized boats	Maintenance Dredging Requirements	Structure Maintenance Requirements	Wave Protection for Sandpiper Shores	Sedimentation at Thorofare Docks	Adjacent property Impacts	Aesthetics/ Natural Looking Element
No Action	1	No-Action – Reference for Comparison							
Remove Breakwater	2	🟥	🟥	🟥	🟩	🟥	🟥	🟨	🟩
Repair Existing Porous BW	3	🟨	⬜	⬜	🟨	🟩	⬜	⬜	⬜
Replace Existing Porous BW with Impervious Sediment Retention Feature	4a-d	🟦	🟦	🟦	🟨	⬜	⬜	⬜	⬜
Replace Existing Porous BW with Impervious Sediment Retention Feature & Extend Seaward	4e	🟩	🟦	🟦	🟨	⬜	⬜	⬜	⬜
In Channel Flow Diversion	5	🟨	🟦	🟦	🟩	🟨	🟥	🟥	🟨
Partial In Channel Flow Diversion	6	🟨	🟦	🟨	🟨	🟨	⬜	🟨	🟥

Change with respect to Status Quo (No Action):

- 🟩 Significantly Positive
- 🟦 Moderately Positive
- ⬜ No/Negligible Change
- 🟨 Moderately Negative
- 🟥 Significantly Negative

THOROFARE/BREAKWATER IMPROVEMENT – ALTERNATIVE SCREENING

Alternative	Alt No.	Sustainability	Thorofare Navigation for motorized boats	Maintenance Dredging Requirements	Structure Maintenance Requirements	Wave Protection for Sandpiper Shores	Sedimentation at Thorofare Docks	Adjacent property Impacts	Aesthetics/ Natural Looking Element
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Remove Breakwater	2								
Repair Existing Porous BW	3								
Replace Existing Porous BW with Impervious Sediment Retention Feature	4a-d								
Replace Existing Porous BW with Impervious Sediment Retention Feature & Extend Seaward	4e								
In Channel Flow Diversion	5								
Partial In Channel Flow Diversion	6								

🟧 = Alternatives considered for additional refinement

ALTERNATIVE NO. 4a – IMPERVIOUS SEDIMENT RETENTION FEATURE

- Reduced lake shore sediment transport into Thorofare
- Increased flow velocity in Thorofare to transport sediment into Lake

ALTERNATIVE NO. 4b – IMPERVIOUS SEDIMENT RETENTION FEATURE

- Similar to Alt 4a; different alignment.
- Alternative alignment could improve sustainability of Thorofare dredging and maintenance of the required navigable depth

ALTERNATIVE NO. 4c – IMPERVIOUS SEDIMENT RETENTION FEATURE

- Similar to Alt 4b; different alignment.
- Longer distance required to get to deeper water

ALTERNATIVE NO. 4d – IMPERVIOUS SEDIMENT RETENTION FEATURE

- Similar to Alt 4a (no rotation); different type of construction materials. Variation in aesthetics, capital cost, maintenance, etc…

ALTERNATIVE NO. 4e - Small Rotation w/ Extension

- Small rotation (15 deg) with extension may improve effectiveness of structure to retain sediment.
- Placement of dredged material on lake side of structure could be feasible with extension

ALTERNATIVE NO. 4e - Large Rotation w/ Extension

- Larger rotation (30 deg) may require removal of existing structure
- Placement of dredged material on lake side of structure could be feasible with extension

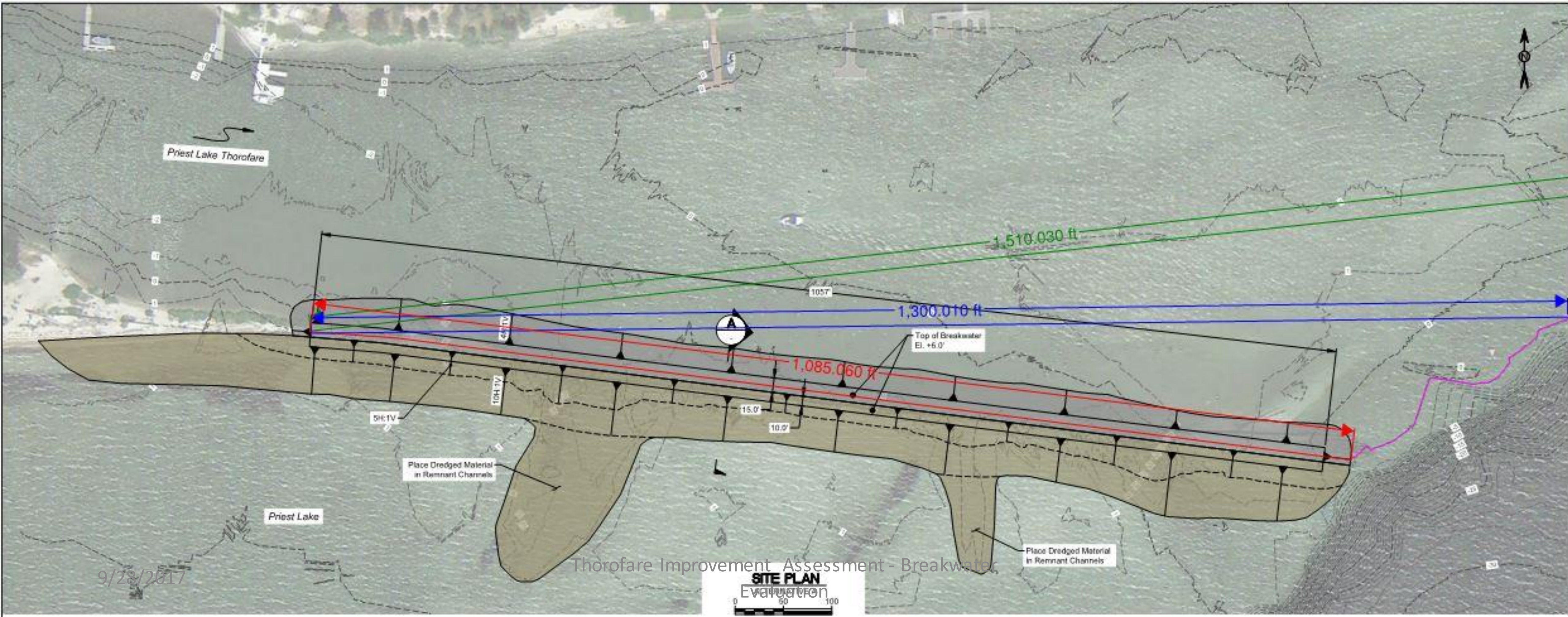
Alignment

- Evaluation of alignment to assess cost/benefit for increasing effectiveness of navigation

🟩 = 30 deg rotation

🟦 = 15 deg rotation

🟥 = 0 deg rotation



Alignment Modification Considerations

- Cost Implications
    - 30 degree rotation = 40 to 65% more expensive than 0 degree rotation
    - 15 degree rotation = 25 to 40 % more expensive than 0 degree rotation
- Performance
    - Modeling of alternatives – 15 degree determined to be the most effective relative to cost. 30 deg option results in substantial increase in cost.
    - Increased velocity in Thorofare is anticipated with the 15 and 30 degree rotations
    - Increased velocity will require additional scour protection along the north side of the sediment retention berm

Extension

Distance beyond existing breakwater = 100 to 150 ft

Extension Considerations

- Cost Implications for Extension
  - Estimated at $1,800 to $2,000 per foot
  - Total Length ~ 100 to 150 ft for non rotated option; substantially longer for rotated options
  - Total Additional Cost = $180,000 to $300,000 additional cost

- Extension Feasibility relative to Alignment Alternatives Considerations
  - Alt C: Easier to install and improved performance
  - Alt B: Slightly more difficult to build in deeper water but feasible without much impact on performance
  - Alt A: More challenging to build in deeper water and less effective. Likely would require a modification to Alt B at the end to transition.

- Overall Assessment
  - Extension should be considered in final alternative development as a means to aid in diverting long shore lake shore sediment transport toward deeper water away from the Thorofare and to provide an opportunity to place dredged material on the lake side of the breakwater.

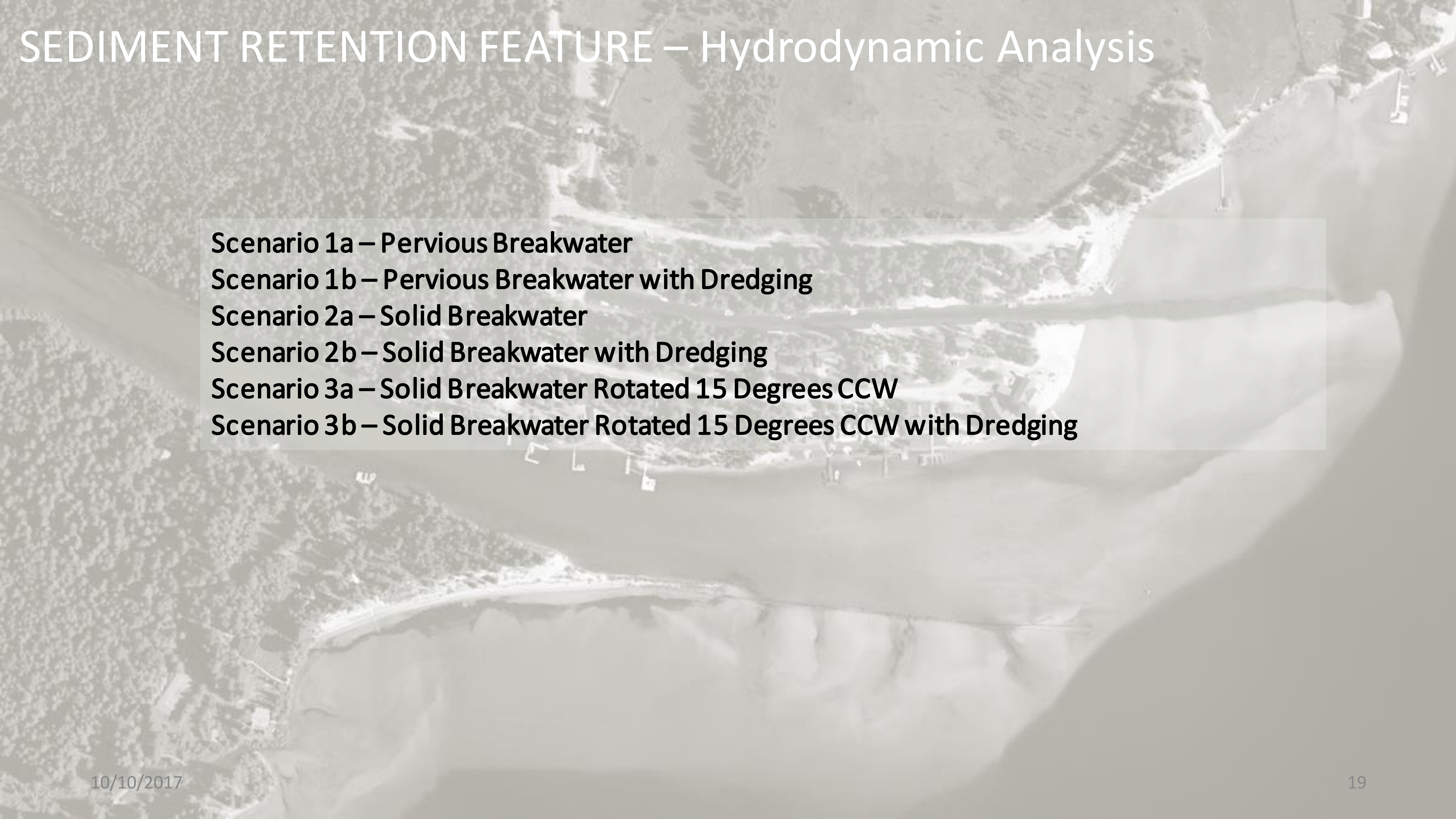
Dredged Material Placement Considerations

- Smaller rotation with extension may increase volume for placement of dredged material and eliminate need to demolish existing breakwater
- Could improve habitat in the nearshore
- Reduce maintenance and reduce costs for new breakwater

Dredged Material Placement Considerations

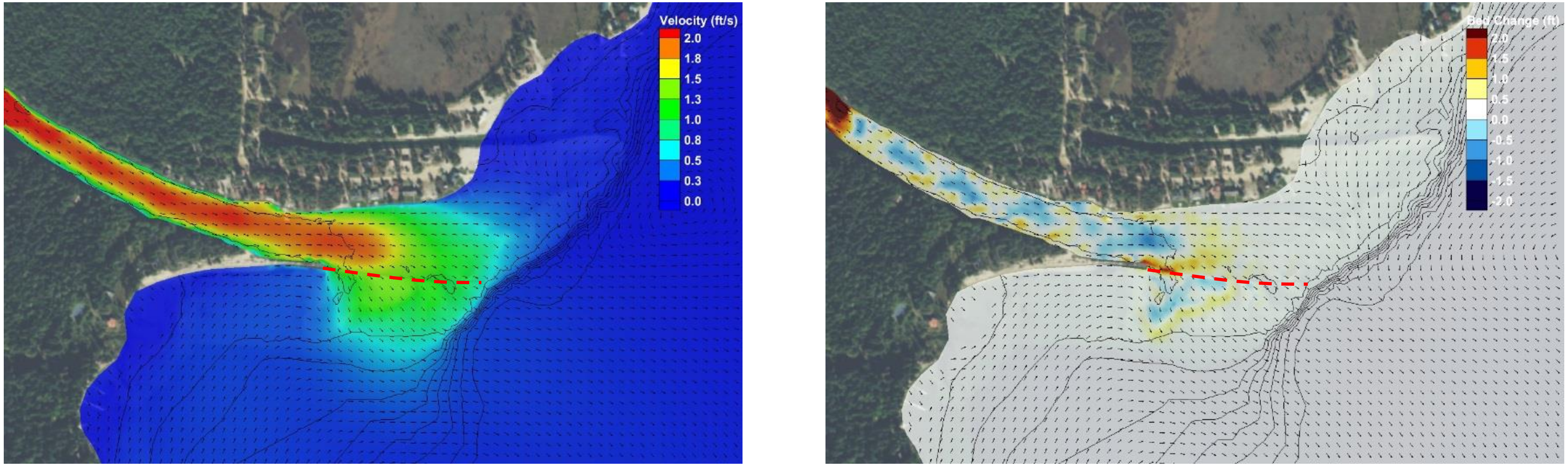
- Rotated has some benefit by increasing capacity for placement of dredged materials
- Larger Rotation may require removal of existing structure

SEDIMENT RETENTION FEATURE – Hydrodynamic Analysis

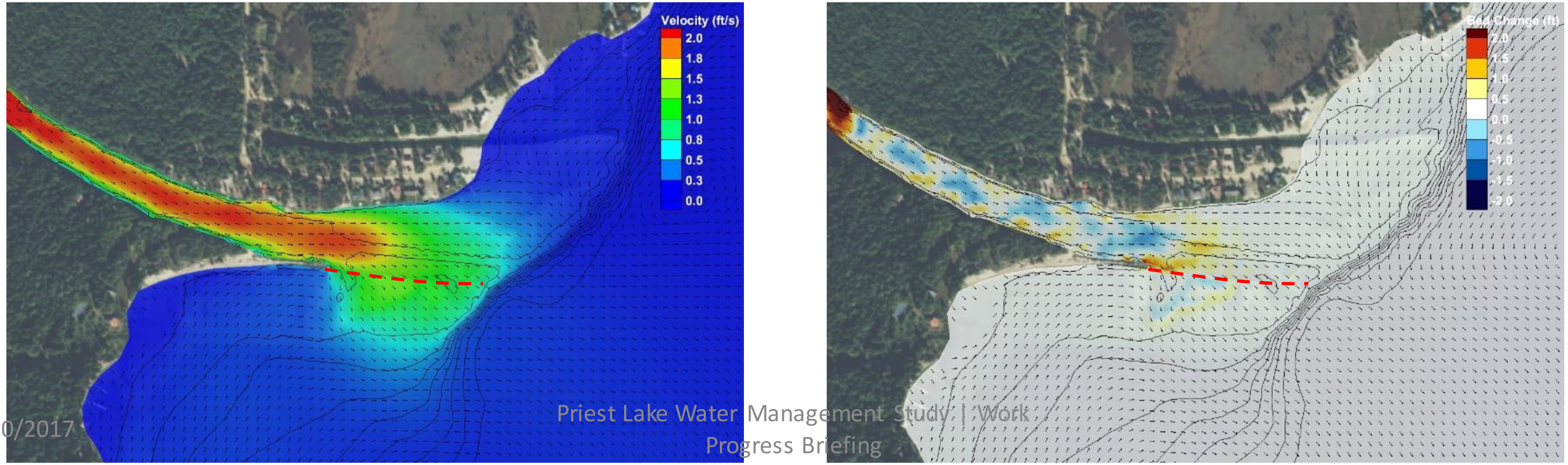


Scenario 1a – Pervious Breakwater
Scenario 1b – Pervious Breakwater with Dredging
Scenario 2a – Solid Breakwater
Scenario 2b – Solid Breakwater with Dredging
Scenario 3a – Solid Breakwater Rotated 15 Degrees CCW
Scenario 3b – Solid Breakwater Rotated 15 Degrees CCW with Dredging

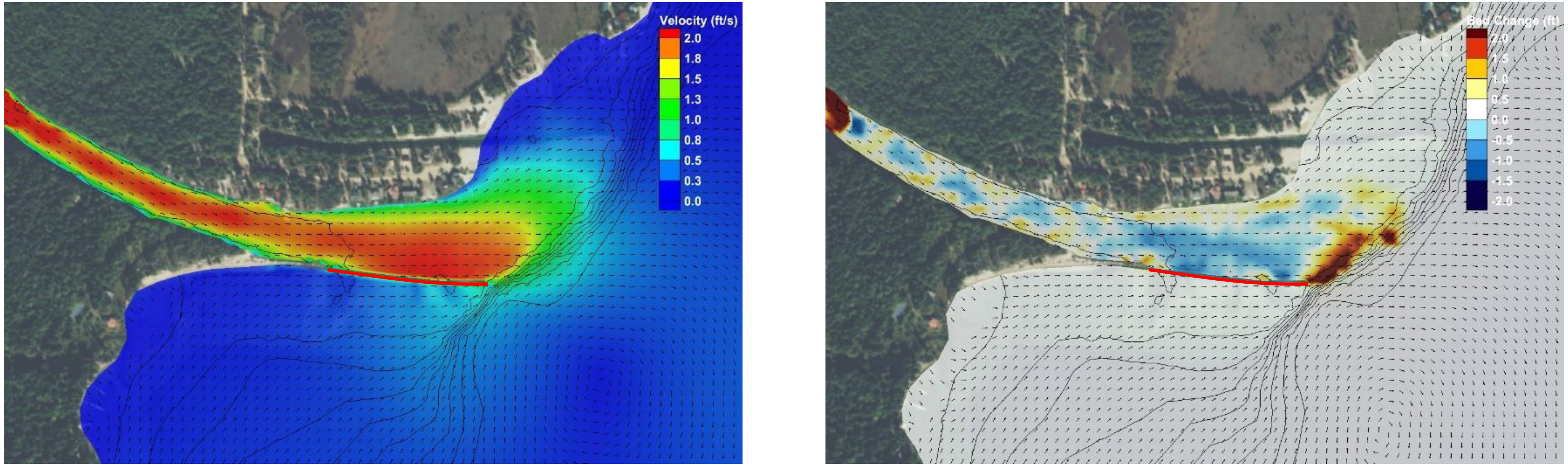
Pervious BW



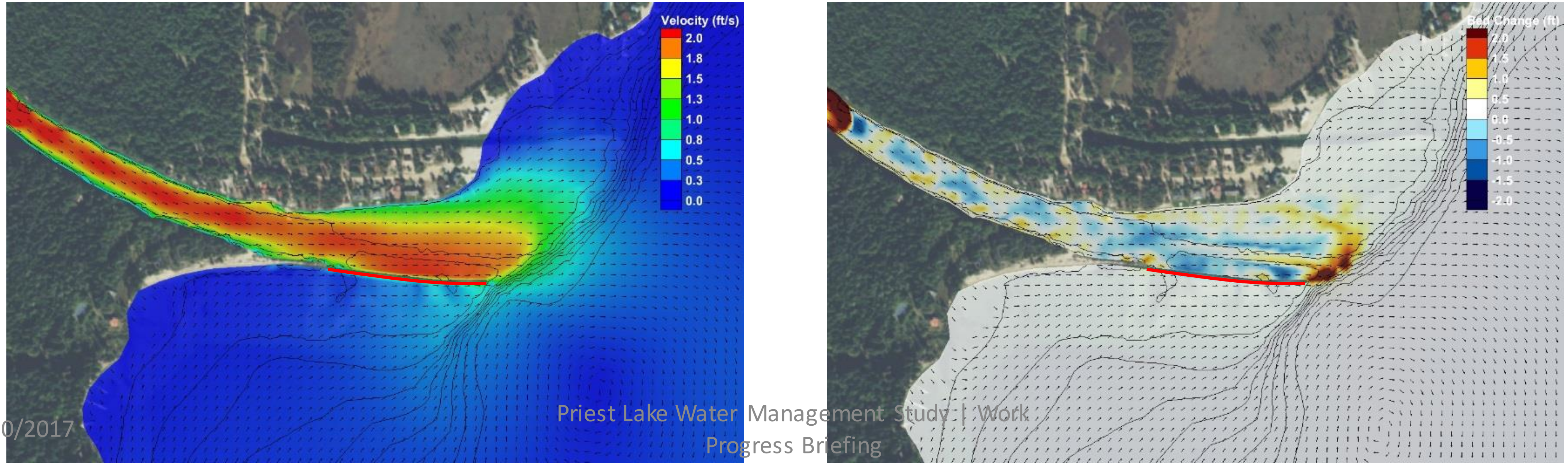
Pervious BW **with Dredge**



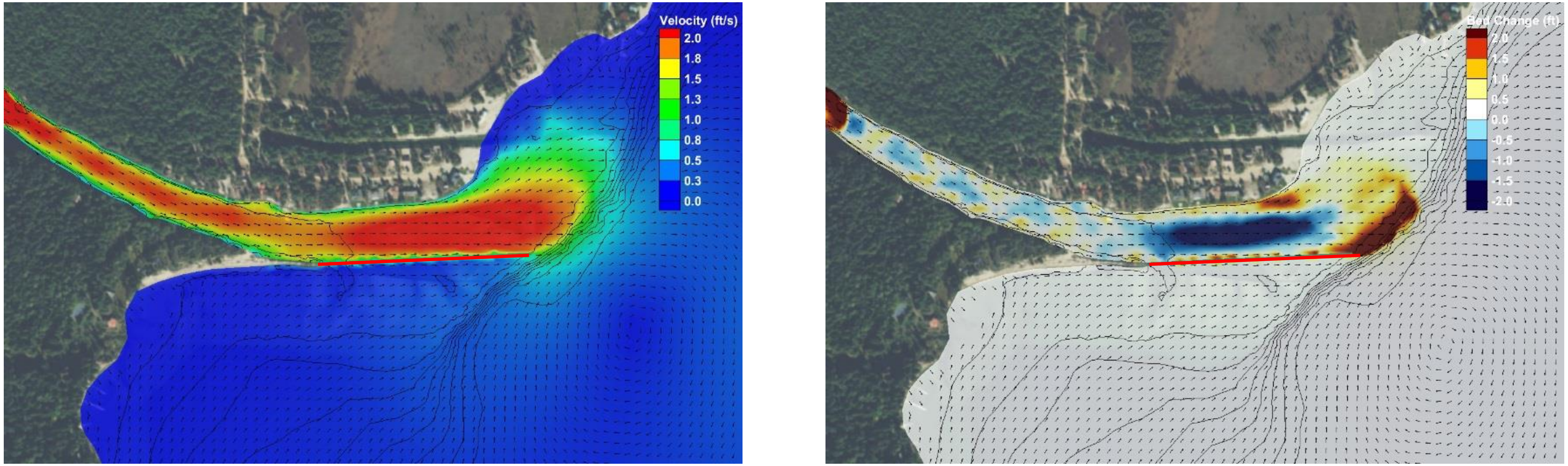
Solid Breakwater



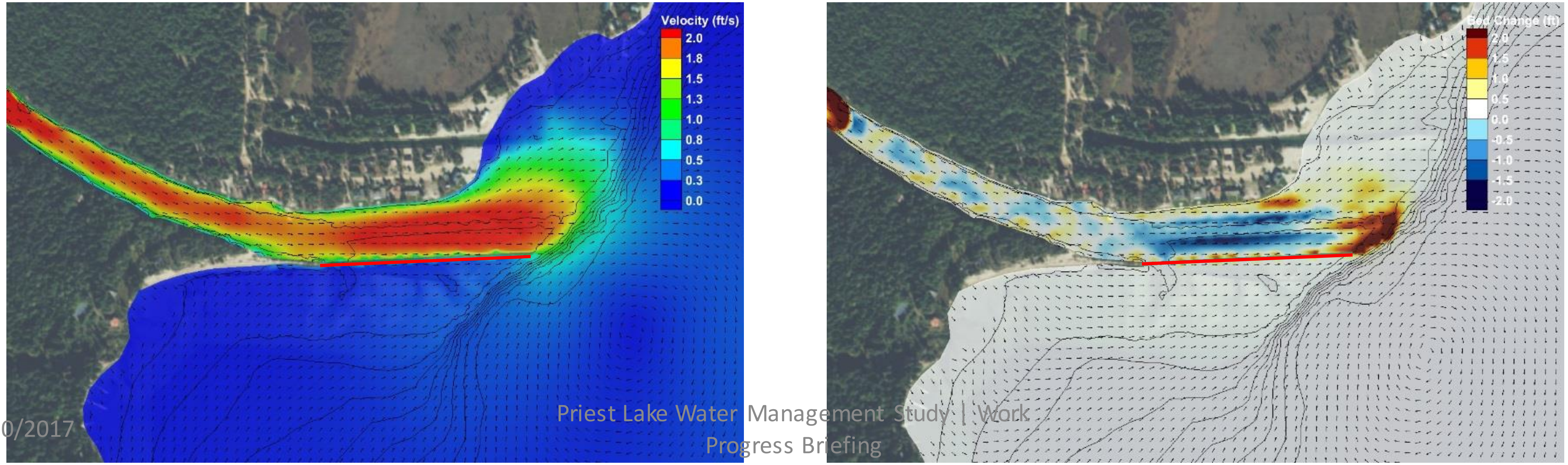
Solid Breakwater **with Dredge**



Solid Breakwater Rotated 15 Degrees CCW



Solid Breakwater Rotated 15 Degrees CCW **with Dredge**



Thorofare Improvement Discussion

- Breakwater/Sediment Retention Feature Engineering Recommendations
    - Slightly Rotated (<15 degrees) is best performing and likely best benefit/cost
    - Extension to help increase effectiveness and containment of dredged material placement should be considered
    - Preferred Concept? Likely Alt A or B.
    - Alternative selection in next phase; all have similar magnitude of cost. Selection after conducting additional data collection and more detailed regulatory agency consultation.

- Regulatory Considerations & Takeaways from Govt Agency Coordination
    - Bio-engineered Techniques
    - Purpose & Need for Breakwater and dredged material placement
    - Outlet Structure Discharge criteria justification
    - Wetland Impact Assessment
    - Aquatic Habitat Enhancement

DESIGN CONSIDERATIONS – ALTERNATIVES EVALUATION

- Currents forces focused at structure
- Toe Scour

- Wave Impacts
- Beach Adjustment
- Sediment Transport

Sediment Retention Feature - Criteria

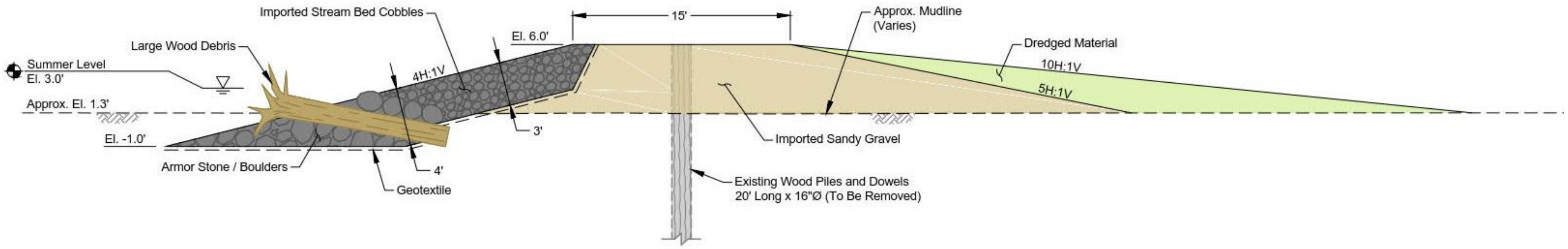
- Impermeable to retain sediment
- Durable to resist hydrodynamic forces of flood flows from Thorofare and storm waves from the south
- Provide option to place dredged material along the lake side of structure
- Long service life with reduced level of maintenance and repair relative to current breakwater structure

Sediment Retention Feature – Bio-Engineered

Alternative A

Sediment Retention Feature – Bio-Engineered

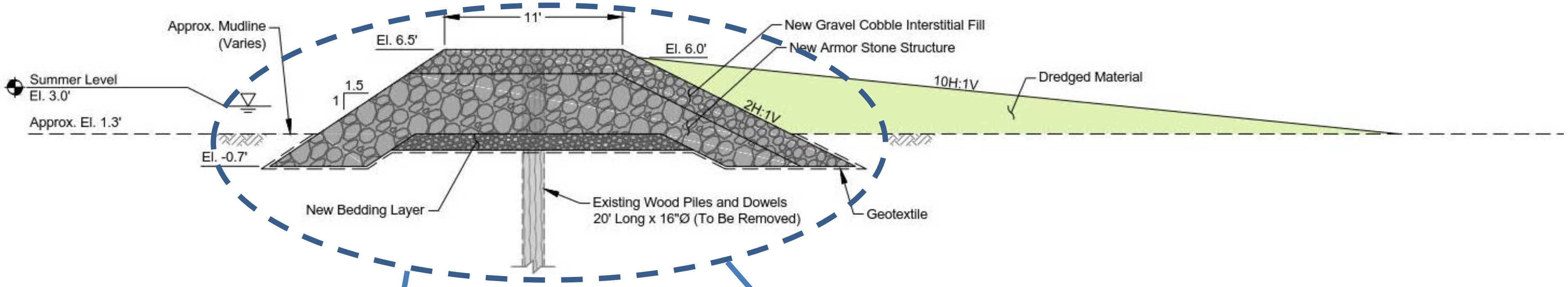
Alternative A



- Advantages
  - Durability of construction material; long lasting if high quality stone source used
  - Some possibility to plant the upper later with shrubs if backfilled with sand/gravel and beach fill installed on Lake shore side
- Limitations
  - Higher potential for maintenance after large or extreme wind storm/flood events
  - Larger footprint

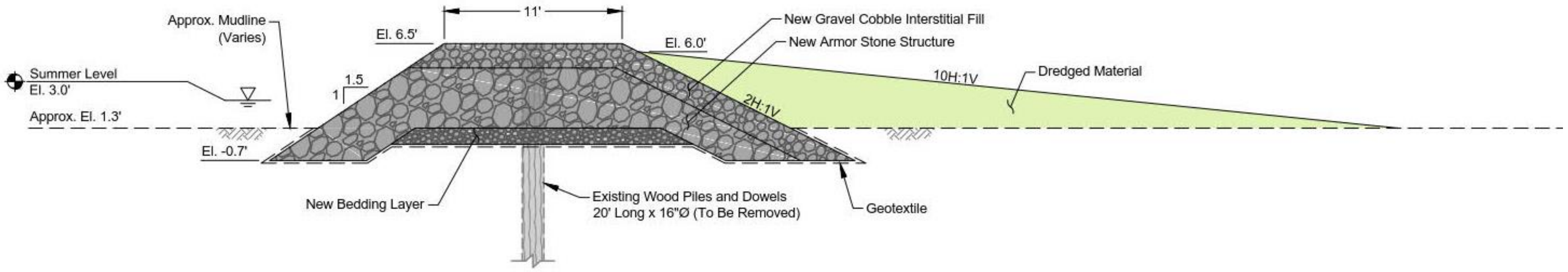
Sediment Retention Feature – Armor Stone

Alternative B



Sediment Retention Feature – Armor Stone

Alternative B

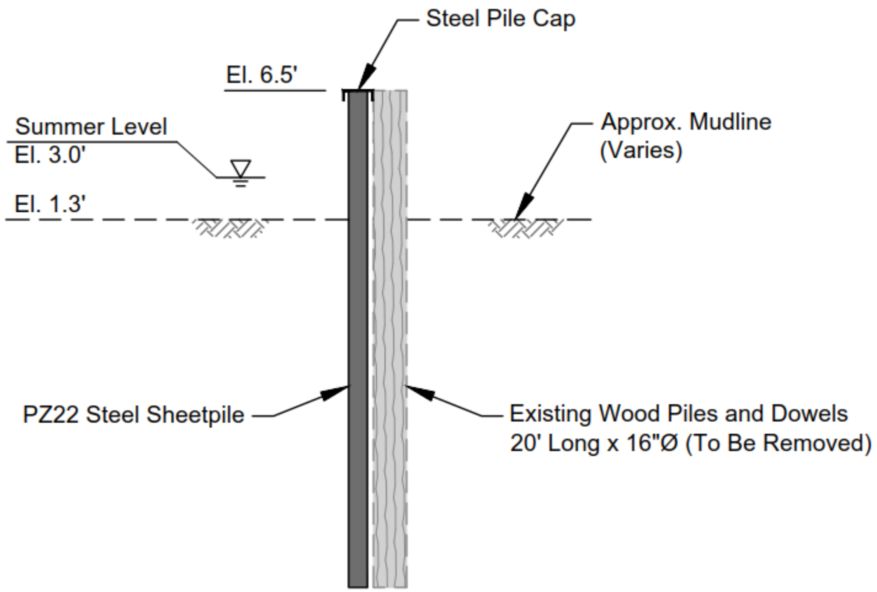


- Advantages
  - Durability of construction material; long lasting if high quality stone source used
  - Smaller footprint than Alt A.
  - Some possibility to plant the upper layer with shrubs if backfilled with sand/gravel and beach fill installed on Lake shore side

- Limitations
  - Partially Reflective
  - Embedment of rock for scour protection; underwater excavation and stone installation

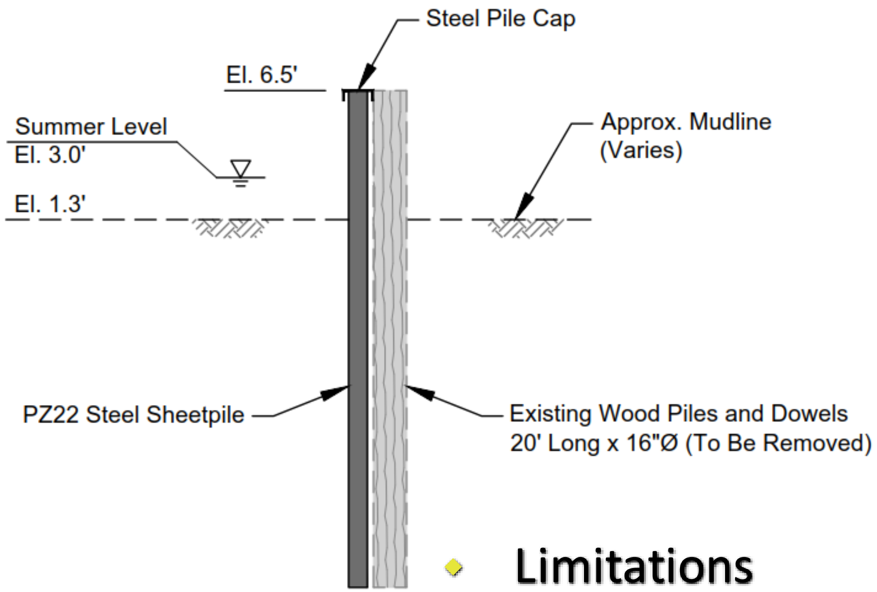
Sediment Retention Feature – Sheet Pile

Alternative C



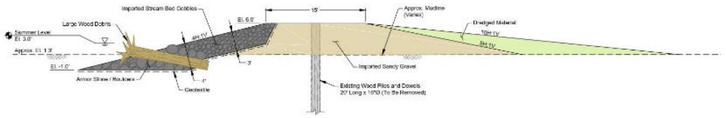
Sediment Retention Feature – Sheet Pile

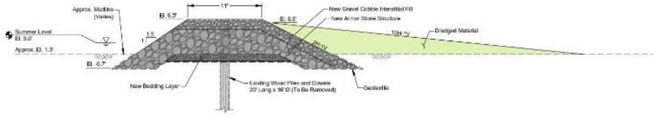
Alternative C

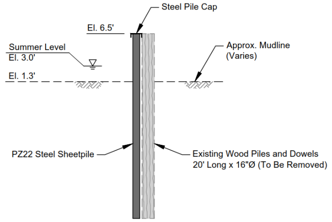


- **Advantages**
  - Ease of construction; less time
  - Smallest footprint
  - Easiest alternative to extend length out into deeper water
- **Limitations**
  - Reflective structure; may be difficult to maintain a beach on lake side.
  - Requires a crane for installation, other alternatives are smaller equipment (excavators)

Construction Material Type Comparison







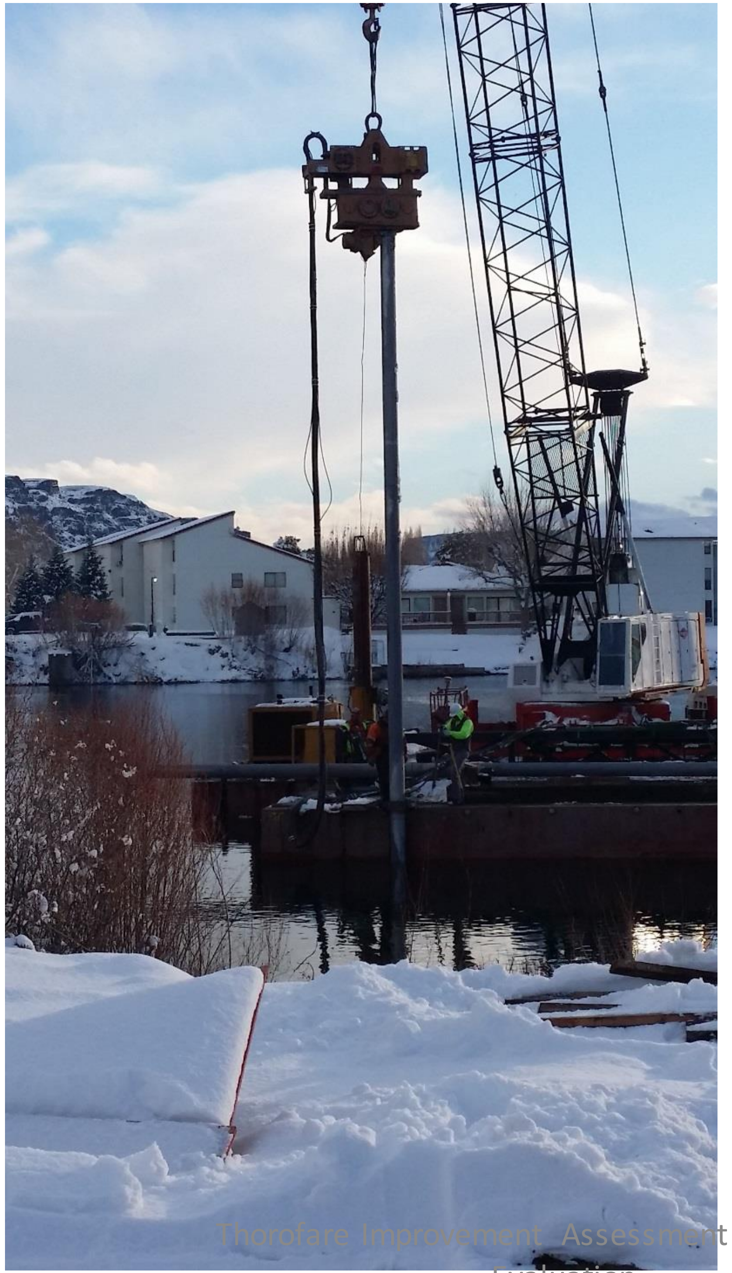
Alt Type	Constructability	Compatibility w/ Site Conditions	Cost/FT	Maintenance
A Bio-Engineered	Poor to Good (near entrance) Good elsewhere	High	$1,000 to $1,300	Medium/Low
B Stone	Good to Excellent	High-Medium	$900 to $1,200	Low/Med
C Sheet Pile	Good to Excellent	Medium	$1,100 to $1,400	Low

Breakwater Evaluation Summary

- Dredging Considerations. Combine dredging concept with sediment retention feature (Breakwater replacement) to ensure long term sustainability of the navigation through the Thorofare.
- Breakwater Orientation. Extension and rotation likely beneficial and recommended for further review and consideration.
- Type. Impermeable to keep hydrodynamics separated between lake and Thorofare and to prevent sediment from passing through structure.
- Cost. Cost of the alternatives are similar order of magnitude. Project funding can be developed based on preferred alignment concept. Develop final material type during next phase based on more detailed discussion with regulatory agencies.

Breakwater Constructability Review

- Similar to dredging assessment, equipment access will depend upon landside access is availability directly to work area.
- If no landside access, then flexifloat access from nearby boat ramp will be required. Flexifloat would be used for both equipment and materials transfer to the Thorofare work area



Regulatory Considerations

- Construction Methods. Pile driving measures for protection of bull trout.
- Environmental Enhancement. Are enhancement measures needed to offset the need for mitigation?
- Disturbance of lakebed sediments
- Best Management Practices and water quality protection during construction.
- See GeoEngineers Technical Memorandum for additional details.

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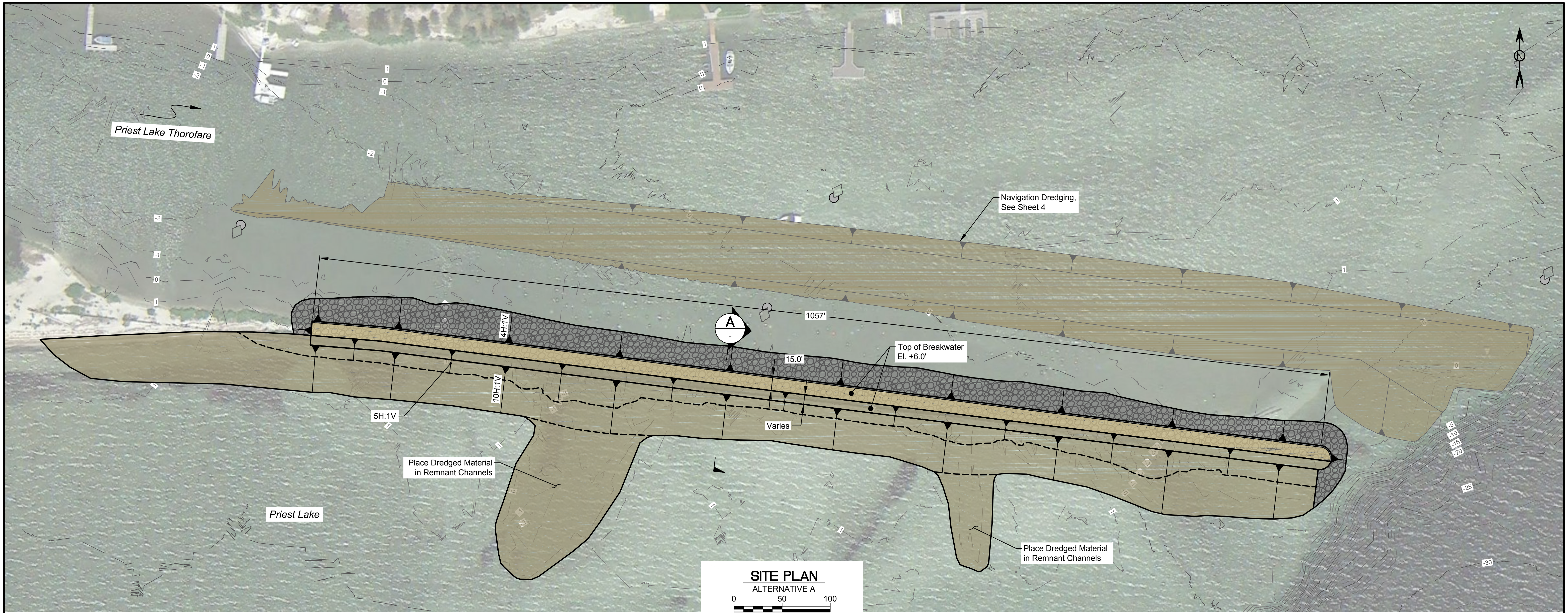
Priest Lake Water Management Study

Thorofare Improvements (Breakwater Alternatives Evaluation) – 9/28/2017

Idaho Water Resource Board

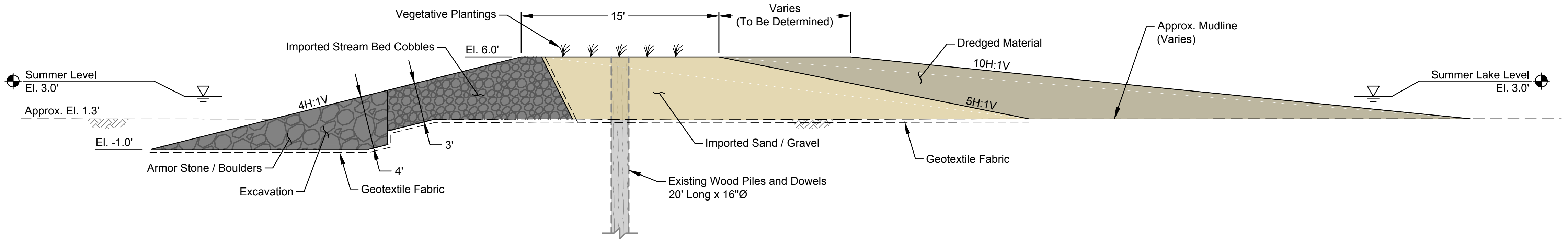
Working Document – v0

E.3 Conceptual Engineering Plans



SITE PLAN

ALTERNATIVE A



A - TYPICAL SECTION

NOTES

1. Bathymetry Source: 2017 Mott MacDonald Survey
2. Horizontal Datum: NAD83, West, Feet
3. Unit and Vertical Datum: Feet Referenced to USGS Gage 12393000 Datum. Datum of Gage 2,434.64 Feet above NGVD29
4. Aerial Source: Google Earth
5. 15° rotated alignment now shown.

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IDWR

Rev	Date	Drawn	Description	Ch'k'd	App'd
	12/20/17	CT	DRAFT		

Project Number 376997
B/O 1
Total 5

Designed	J. LaFave	Eng check	S. Phillips
Drawn	C. Taylor	Coordination	
Dwg check	Y. Nouri	Approved	

Scale at ANSI D: As Noted
Status
Rev
Security
Drawing Number

Title
Priest Lake
Water Management Study
Thorofare Improvements

Breakwater Alternative A
Sand and Gravel Berm

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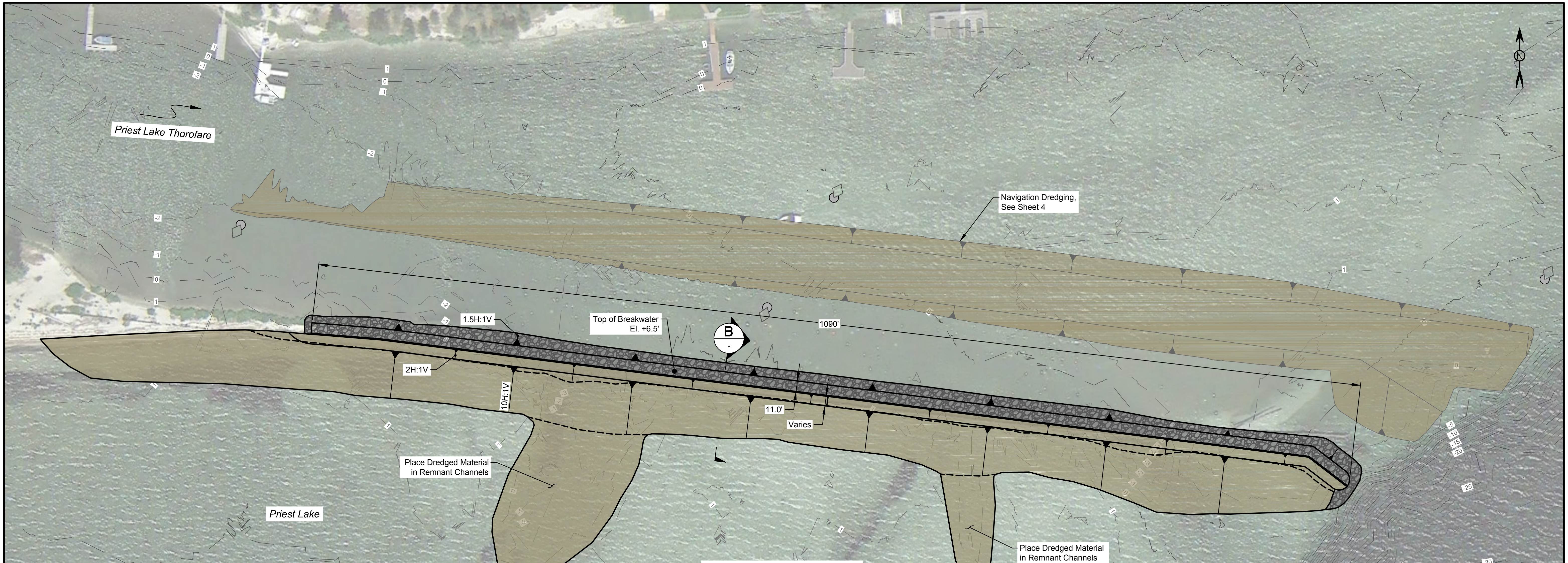
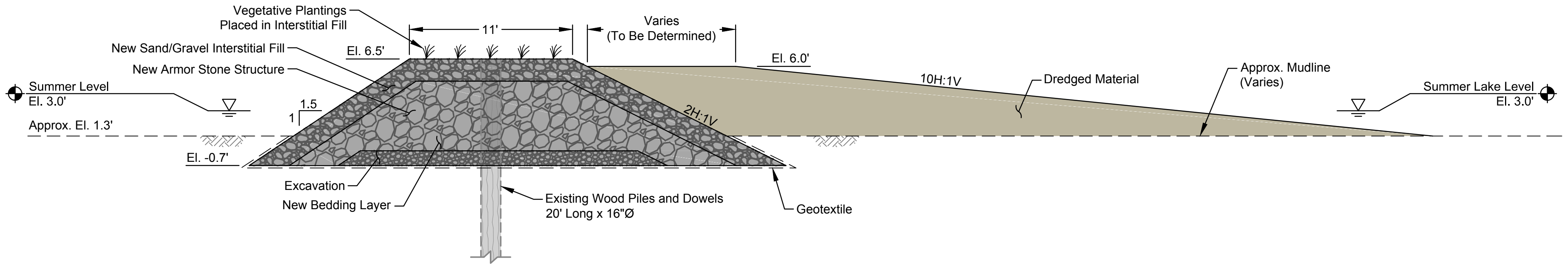


SITE PLAN
ALTERNATIVE B
0 50 100
SCALE IN FEET



B TYPICAL SECTION
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SCALE IN FEET

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4. Aerial Source: Google Earth
5. 15° rotated alignment now shown.

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Dwg check	Y. Nouri		Approved		

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As Noted			

Project Number	B/O	Total
376997	2	5

Drawing Number

Title
Priest Lake
Water Management Study
Thorofare Improvements

Breakwater Alternative B
Rubblemound Breakwater

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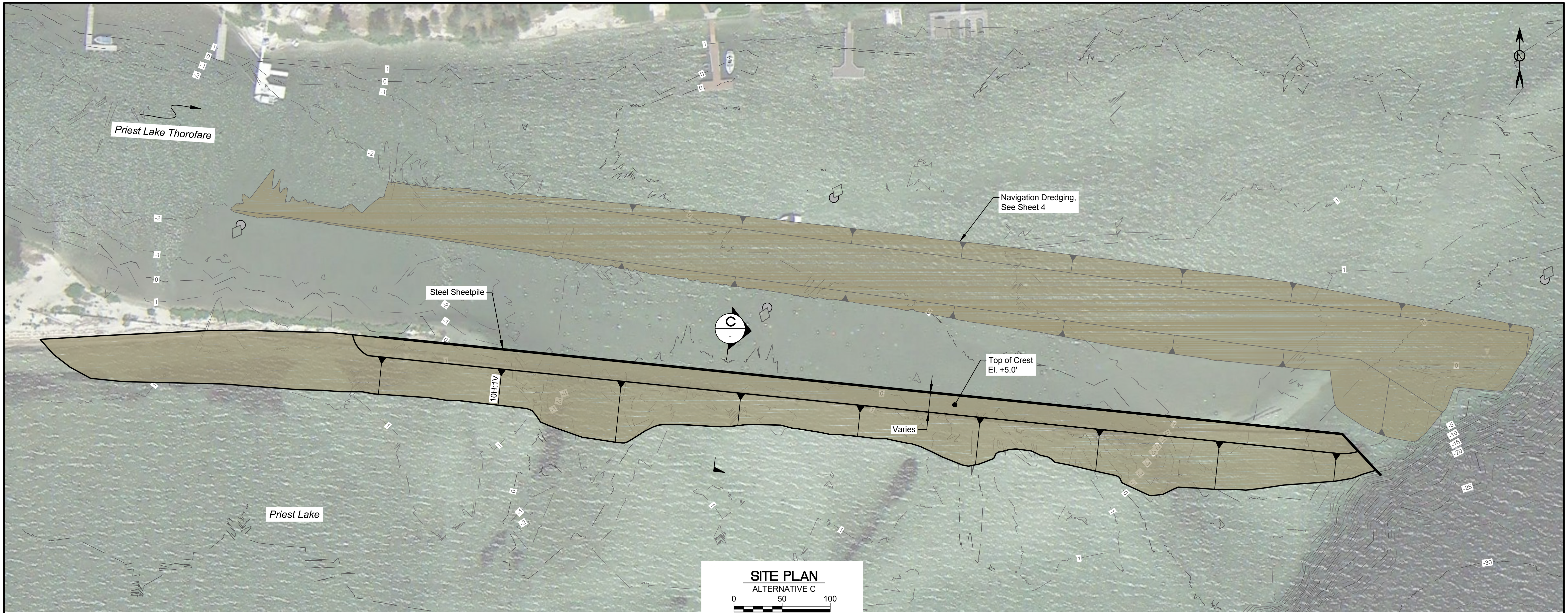
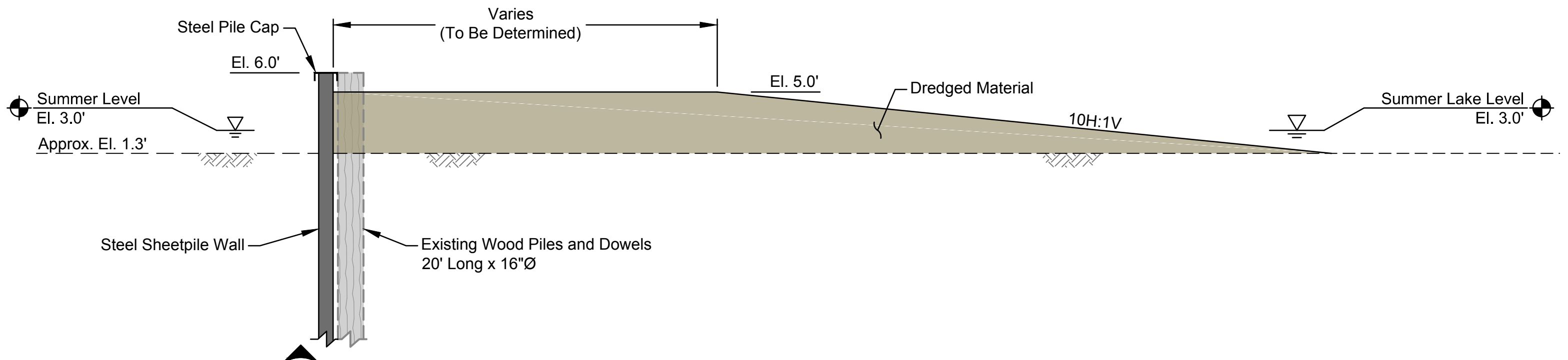


SITE PLAN

ALTERNATIVE C

0 50 100

SCALE IN FEET



C - TYPICAL SECTION

0 5 10

SCALE IN FEET

NOTES

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3. Unit and Vertical Datum: Feet Referenced to USGS Gage 12393000 Datum. Datum of Gage 2,434.64 Feet above NGVD29
4. Aerial Source: Google Earth
5. 15° rotated alignment now shown.

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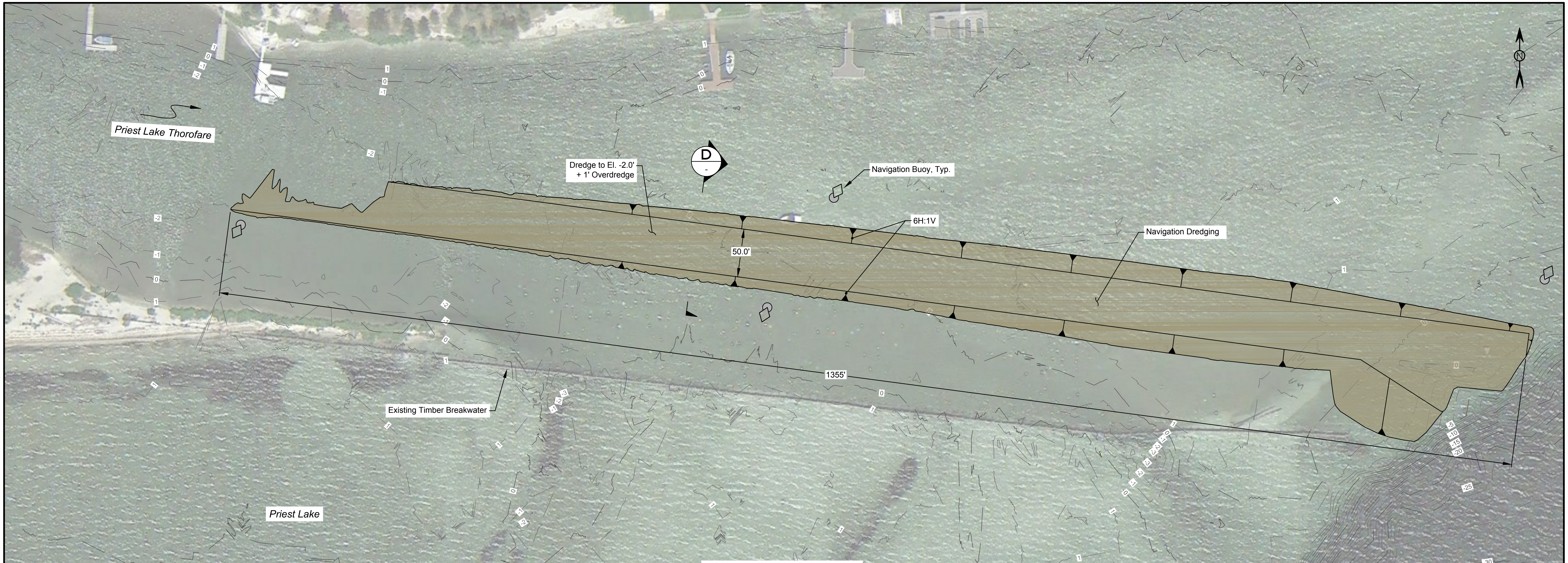
Drawn	C. Taylor	Coordination	
Dwg check	Y. Nouri	Approved	
Scale at ANSI D As Noted	Status	Rev	Security

Project Number	B/O	Total	Drawing Number
376997	3	5	

Title

Priest Lake
Water Management Study
Thorofare Improvements

Breakwater Alternative C
Sheetpile Wall



SITE PLAN

NAVIGATION DREDGING

0 50 100

SCALE IN FEET

NOTES

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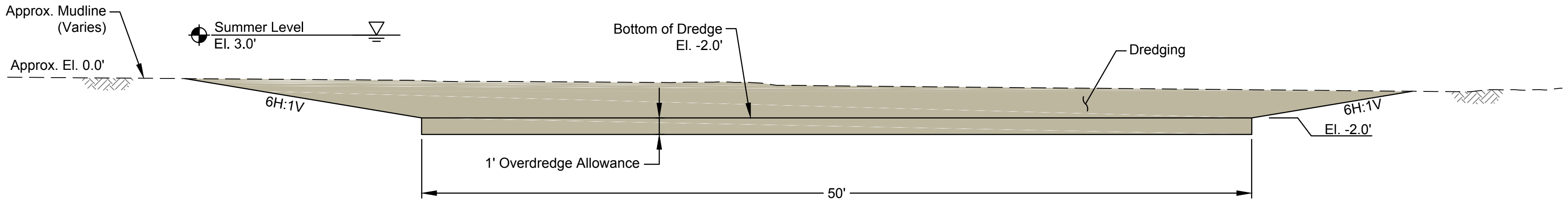
2. Horizontal Datum: NAD83, West, Feet
3. Unit and Vertical Datum: Feet Referenced to USGS Gage 12393000 Datum. Datum of Gage 2,434.64 Feet above NGVD29
4. Aerial Source: Google Earth
5. Slopes, width, and location of dredging to be optimized during next phases of engineering.



D - TYPICAL SECTION

NAVIGATION DREDGING

0 5 10

SCALE IN FEET

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Rev	Date	Drawn	Description	Ch'k'd	App'd
	12/20/17	CT	DRAFT		

Designed	J. LaFave		Eng check	S. Phillips	
Drawn	C. Taylor		Coordination		
Dwg check	Y. Nouri		Approved		

Scale at ANSI D	Status	Rev	Security
As Noted			

Project Number	B/O	Total	Drawing Number
376997	4	5	

Title

Priest Lake
Water Management Study
Thorofare Improvements

Navigation Dredging

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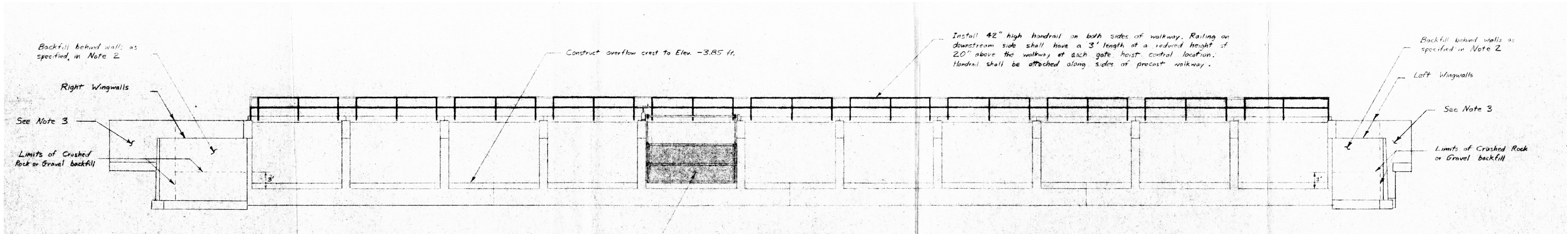
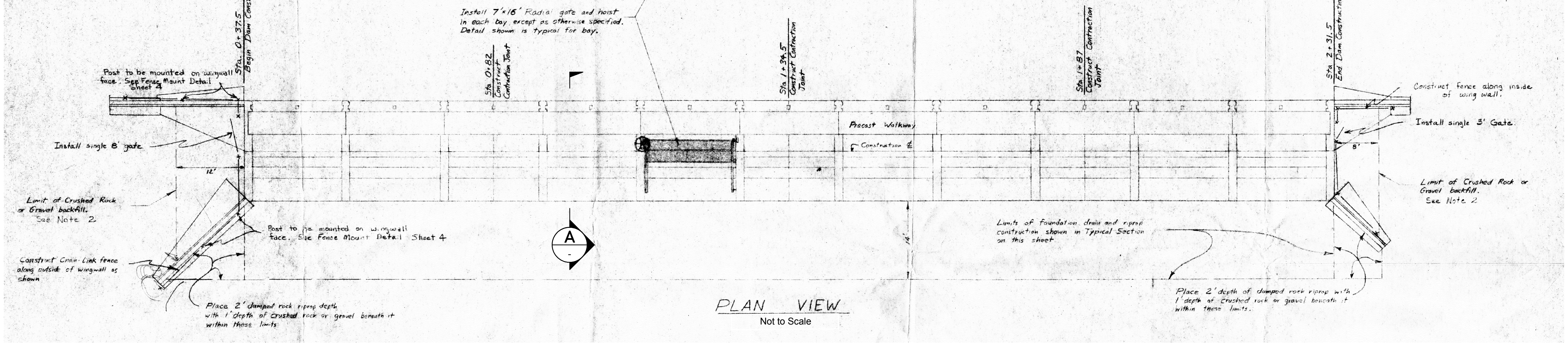
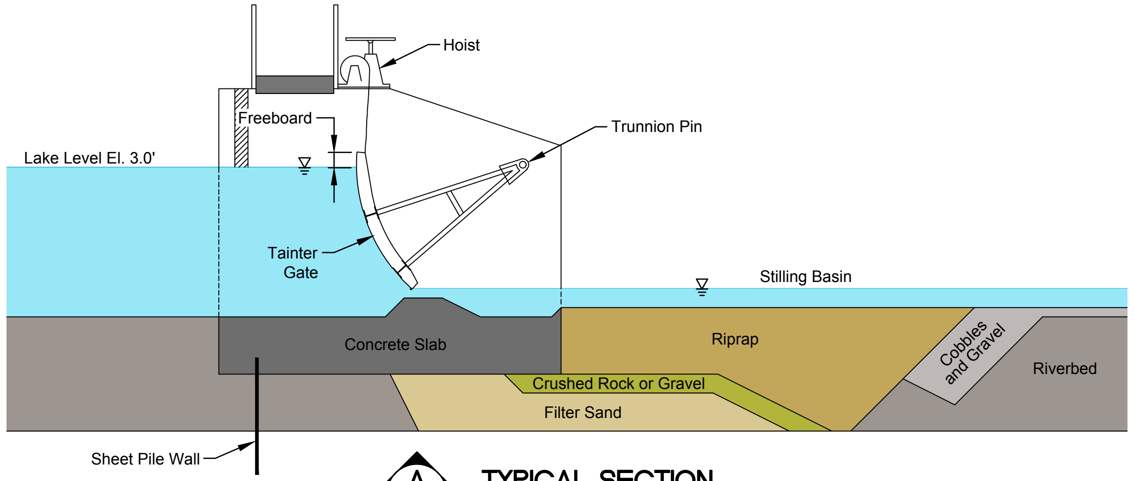


DOWNSTREAM PROFILE
Not to Scale



PLAN VIEW
Not to Scale



A
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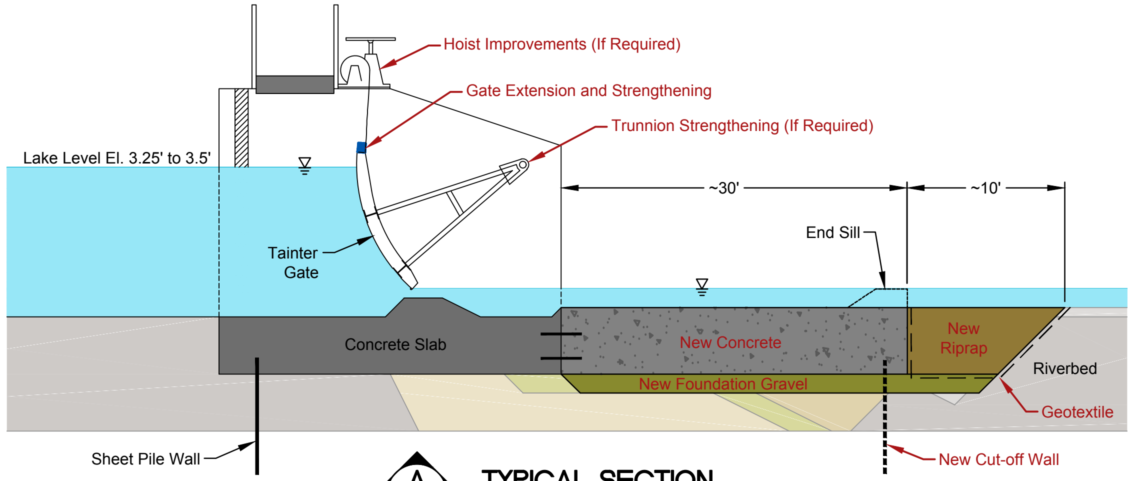
TYPICAL SECTION
EXISTING
Not to Scale



A
-
TYPICAL SECTION
IMPROVEMENTS
Not to Scale

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	12/15/17	CT	DRAFT		

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Drawn	C. Taylor	Coordination	
Dwg check	Y. Nouri	Approved	

Scale at ANSI D: As Noted
Status
Rev
Security

Project Number: 376997
B/O: 5
Total: 5
Drawing Number

Title
Priest Lake
Water Management Study

Outlet Structure Improvements

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E.4 Construction Cost Estimates

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Priest Lake Water Management Study
Preliminary Cost Estimate Worksheet
SUMMARY

Prepared By: **JL/SP**
Date: **01/17/18**

Schedule 1 - Outlet Structure

Item	Description	Quantity/Units	Unit	Unit Cost	Total
1	Outlet Structure	1	LS	$2,635,000.00	$2,710,000.00

Schedule 2 - Thorofare Alternatives

Item	Description	Quantity/Units	Unit	Unit Cost	Total
1	Alternative A - Sand/Gravel Berm	1	LS	$2,504,000.00	$2,504,000.00
2	Alternative B - Rubblemound Breakwater	1	LS	$2,295,000.00	$2,295,000.00
3	Alternative C - Sheetpile Wall	1	LS	$2,443,000.00	$2,443,000.00

TOTAL COSTS

Item	Description	Quantity/Units	Unit	Unit Cost	Total
1	Alternative A - Sand/Gravel Berm	1	LS	$5,139,000.00	$5,139,000.00
2	Alternative B - Rubblemound Breakwater	1	LS	$4,930,000.00	$4,930,000.00
3	Alternative C - Sheetpile Wall	1	LS	$5,078,000.00	$5,078,000.00

Thorofare Alternative Refinements (7.5 ° Rotation and 84' Extension)

Item	Description	Quantity/Units	Unit	Unit Cost	Total
4	Alternative A - Sand/Gravel Berm - EXTENSION	1169	LF	$2,302.82	$2,692,000.00
5	Alternative B - Rubblemound Breakwater - EXTENSION	1169	LF	$2,111.21	$2,468,000.00
6	Alternative C - Sheetpile Wall - EXTENSION	1169	LF	$2,247.22	$2,627,000.00

TOTAL COSTS (w/ Rotation and Extension)

Item	Description	Quantity/Units	Unit	Unit Cost	Total
1	Alternative A - Sand/Gravel Berm	1	LS	$5,327,000.00	$5,327,000.00
2	Alternative B - Rubblemound Breakwater	1	LS	$5,103,000.00	$5,103,000.00
3	Alternative C - Sheetpile Wall	1	LS	$5,262,000.00	$5,262,000.00

Notes:

1) All Costs include an estimated amount for Engineering, Permitting, Data Collection, and Contingency.
2) Mitigation costs not included (if required)

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Priest Lake Water Management Study
Preliminary Cost Estimate Worksheet
Alternative A - Sand/Gravel Berm

Prepared By: **JL/SP**
Date: **01/17/18**

Schedule 1 - Outlet Structure

Item	Description	Quantity/Units	Unit	Unit Cost	Total
1	Mobilization/Demobilization (8%)	1	LS	$143,840.00	$143,840.00
2	Environmental Protection	1	LS	$25,000.00	$25,000.00
3	Stream Diversion & Dewatering	1	LS	$100,000.00	$100,000.00
4	Excavation	700	CY	$40.00	$28,000.00
5	Concrete Apron	650	CY	$1,000.00	$650,000.00
6	Concrete Anchors	76	EA	$250.00	$19,000.00
7	Riprap	250	CY	$100.00	$25,000.00
8	Foundation Gravel	300	CY	$55.00	$16,500.00
9	Geotextile Fabric	300	SY	$8.00	$2,400.00
10	Sheetpile Wall	5680	SF	$25.00	$142,000.00
11	Gate Modification	11	EA	$10,500.00	$115,500.00
12	Gate Electrical (11 Gates)	11	EA	$15,000.00	$165,000.00
13	Gate & Trunnion Strengthening	1	LS	$110,000.00	$110,000.00
14	Backup Generator & Security Fencing	1	LS	$20,000.00	$20,000.00
15	Surveying	1	LS	$15,000.00	$15,000.00
16	Site Restoration	1	LS	$5,000.00	$5,000.00
			Schedule 1 Subtotal		$1,582,240.00

Schedule 2 - Thorofare Alternative A - Sand/Gravel Berm

Item	Description	Quantity/Units	Unit	Unit Cost	Total
1	Mobilization/Demobilization (10%)	1	LS	$131,970.00	$131,970.00
2	Environmental Protection	1	LS	$35,000.00	$35,000.00
3	Dredging & Disposal	10000	CY	$25.00	$250,000.00
4	Sediment Retention Feature				
4.1	Existing Breakwater Demolition	1	LS	$170,000.00	$170,000.00
4.2	Excavation	1400	CY	$50.00	$70,000.00
4.3	Sand/Gravel	5500	CY	$75.00	$412,500.00
4.4	Streambed Cobble	1510	CY	$75.00	$113,250.00
4.5	Armor Stone	1410	CY	$95.00	$133,950.00
4.6	Bedding Stone	0	CY	$75.00	$0.00
4.7	Geotextile Fabric	4000	SY	$8.00	$32,000.00
4.8	Vegetative Plantings	1	LS	$35,000.00	$35,000.00
4.9	Large Woody Debris	12	EA	$1,500.00	$18,000.00
4.10	Sheetpile Wall	0	LF	$0.00	$0.00
4.11	Steel Pile Cap	0	LF	$0.00	$0.00
5	Surveying	1	LS	$35,000.00	$35,000.00
6	Site Restoration	1	LS	$15,000.00	$15,000.00
			Schedule 2 Subtotal		$1,451,670.00

Outlet Structure Subtotal	$1,582,240.00
Thorofare Subtotal	$1,451,670.00
Combined Subtotal	$3,033,910.00
Data Collection	$100,000.00
Engineering & Permitting	$607,000.00
Construction Administration	$304,000.00
Contingency (30%)	$911,000.00
Sales Tax (Bonner County, 6%)	$183,000.00
GRAND TOTAL:	**$5,139,000.00**

Notes:
1) Breakwater Length = 1056 FT. Total length is 1330 FT - Uplands with be strengthened with additional beachfill.
2) Width of outlet structure is 194 FT
3) Outlet Structure Sheetpile wall is 224 FT
4) Dredged material placed adjacent to the breakwater replacement

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Priest Lake Water Management Study
Preliminary Cost Estimate Worksheet
Alternative B - Rubblemound Breakwater

Prepared By: **JL/SP**
Date: **01/17/18**

Schedule 1 - Outlet Structure

Item	Description	Quantity/Units	Unit	Unit Cost	Total
1	Mobilization/Demobilization (8%)	1	LS	$143,840.00	$143,840.00
2	Environmental Protection	1	LS	$25,000.00	$25,000.00
3	Stream Diversion & Dewatering	1	LS	$100,000.00	$100,000.00
4	Excavation	700	CY	$40.00	$28,000.00
5	Concrete Apron	650	CY	$1,000.00	$650,000.00
6	Concrete Anchors	76	EA	$250.00	$19,000.00
7	Riprap	250	CY	$100.00	$25,000.00
8	Foundation Gravel	300	CY	$55.00	$16,500.00
9	Geotextile Fabric	300	SY	$8.00	$2,400.00
10	Sheetpile Wall	5680	SF	$25.00	$142,000.00
11	Gate Modification	11	EA	$10,500.00	$115,500.00
12	Gate Electrical (11 Gates)	11	EA	$15,000.00	$165,000.00
13	Gate & Trunnion Strengthening	1	LS	$110,000.00	$110,000.00
14	Backup Generator & Security Fencing	1	LS	$20,000.00	$20,000.00
15	Surveying	1	LS	$15,000.00	$15,000.00
16	Site Restoration	1	LS	$5,000.00	$5,000.00
				Schedule 1 Subtotal	$1,582,240.00

Schedule 2 - Thorofare Alternative B - Rubbemound Breakwater

Item	Description	Quantity/Units	Unit	Unit Cost	Total
1	Mobilization/Demobilization (10%)	1	LS	$120,586.00	$120,586.00
2	Environmental Protection	1	LS	$35,000.00	$35,000.00
3	Dredging & Disposal	10000	CY	$25.00	$250,000.00
4	Sediment Retention Feature				
4.1	Existing Breakwater Demolition	1	LS	$170,000.00	$170,000.00
4.2	Excavation	1410	CY	$50.00	$70,500.00
4.3	Sand/Gravel	0	CY	$75.00	$0.00
4.4	Streambed Cobble	1410	CY	$75.00	$105,750.00
4.5	Armor Stone	4110	CY	$100.00	$411,000.00
4.6	Bedding Stone	670	CY	$75.00	$50,250.00
4.7	Geotextile Fabric	3170	SY	$8.00	$25,360.00
4.8	Vegetative Plantings	1	LS	$20,000.00	$20,000.00
4.9	Large Woody Debris	12	EA	$1,500.00	$18,000.00
4.10	Sheetpile Wall	0	LF	$0.00	$0.00
4.11	Steel Pile Cap	0	LF	$0.00	$0.00
5	Surveying	1	LS	$35,000.00	$35,000.00
6	Site Restoration	1	LS	$15,000.00	$15,000.00
				Schedule 2 Subtotal	$1,326,446.00
				Outlet Structure Subtotal	$1,582,240.00
				Thorofare Subtotal	$1,326,446.00
				Combined Subtotal	$2,908,686.00
				Data Collection	$100,000.00
				Engineering & Permitting	$582,000.00
				Construction Administration	$291,000.00
				Contingency (30%)	$873,000.00
				Sales Tax (Bonner County, 6%)	$175,000.00
			GRAND TOTAL:		**$4,930,000.00**

Notes:

1) Breakwater Length = 1056 FT. Total length is 1330 FT - Uplands with be strengthened with additional beachfill.
2) Width of outlet structure is 194 FT
3) Outlet Structure Sheetpile wall is 224 FT
4) Dredged material placed adjacent to the breakwater replacement

MOTT MACDONALD

Priest Lake Water Management Study
Preliminary Cost Estimate Worksheet
Alternative C - Sheetpile Wall

Prepared By: JL/SP
Date: 01/17/18

Schedule 1 - Outlet Structure

Item	Description	Quantity/Units	Unit	Unit Cost	Total
1	Mobilization/Demobilization (8%)	1	LS	$143,840.00	$143,840.00
2	Environmental Protection	1	LS	$25,000.00	$25,000.00
3	Stream Diversion & Dewatering	1	LS	$100,000.00	$100,000.00
4	Excavation	700	CY	$40.00	$28,000.00
5	Concrete Apron	650	CY	$1,000.00	$650,000.00
6	Concrete Anchors	76	EA	$250.00	$19,000.00
7	Riprap	250	CY	$100.00	$25,000.00
8	Foundation Gravel	300	CY	$55.00	$16,500.00
9	Geotextile Fabric	300	SY	$8.00	$2,400.00
10	Sheetpile Wall	5680	SF	$25.00	$142,000.00
11	Gate Modification	11	EA	$10,500.00	$115,500.00
12	Gate Electrical (11 Gates)	11	EA	$15,000.00	$165,000.00
13	Gate & Trunnion Strengthening	1	LS	$110,000.00	$110,000.00
14	Backup Generator & Security Fencing	1	LS	$20,000.00	$20,000.00
15	Surveying	1	LS	$15,000.00	$15,000.00
16	Site Restoration	1	LS	$5,000.00	$5,000.00
			Schedule 1 Subtotal		$1,582,240.00

Schedule 2 - Thorofare Alternative C - Sheetpile Wall

Item	Description	Quantity/Units	Unit	Unit Cost	Total
1	Mobilization/Demobilization (10%)	1	LS	$128,700.00	$128,700.00
2	Environmental Protection	1	LS	$35,000.00	$35,000.00
3	Dredging & Disposal	10000	CY	$25.00	$250,000.00
4	Sediment Retention Feature				
4.1	Existing Breakwater Demolition	1	LS	$170,000.00	$170,000.00
4.2	Excavation	0	CY	$50.00	$0.00
4.3	Sand/Gravel	0	CY	$75.00	$0.00
4.4	Streambed Cobble	0	CY	$75.00	$0.00
4.5	Armor Stone	0	CY	$100.00	$0.00
4.6	Bedding Stone	0	CY	$75.00	$0.00
4.7	Geotextile Fabric	0	SY	$8.00	$0.00
4.8	Vegetative Plantings	0	LS	$0.00	$0.00
4.9	Large Woody Debris	0	EA	$1,500.00	$0.00
4.10	Sheetpile Wall	1056	LF	$550.00	$580,800.00
4.11	Steel Pile Cap	1056	LF	$200.00	$211,200.00
5	Surveying	1	LS	$25,000.00	$25,000.00
6	Site Restoration	1	LS	$15,000.00	$15,000.00
			Schedule 2 Subtotal		$1,415,700.00
			Outlet Structure Subtotal		$1,582,240.00
			Thorofare Subtotal		$1,415,700.00
			Combined Subtotal		$2,997,940.00
			Data Collection		$100,000.00
			Engineering & Permitting		$600,000.00
			Construction Administration		$300,000.00
			Contingency (30%)		$900,000.00
			Sales Tax (Bonner County, 6%)		$180,000.00
		GRAND TOTAL:			**$5,078,000.00**

Notes:

1) Breakwater Length = 1056 FT. Total length is 1330 FT - Uplands with be strengthened with additional beachfill.
2) Width of outlet structure is 194 FT
3) Outlet Structure Sheetpile wall is 224 FT
4) Dredged material placed adjacent to the breakwater replacement

E.5 Environmental Permitting Requirements

GeoEngineers

523 East Second Avenue, Spokane, Washington 99202, Telephone: 509.363.3125
www.geoengineers.com

Memorandum

To: Shane Phillips, PE and Younes Nouri, PE; Mott MacDonald

From: Jason Scott and Tim Hanrahan; GeoEngineers, Inc.

Date: December 12, 2017

File: 22593-001-00

Subject: Priest Lake Study Potential Environmental Permitting Requirements

INTRODUCTION

Modifications of the Priest Lake Thorofare Breakwater structure and potential dam modifications will require an extensive and thorough permit review process. Regulatory authorization will be required from Bonner County, the State of Idaho and the Federal Government. Despite the fact that written authorization is necessary from several levels of government and many agencies, permitting should be a coordinated effort between the design team and collective regulatory body to maintain project contiguity. While each regulatory authorization process has specific requirements that are unique, many requirements overlap and some are dependent on others. Therefore, for a project of this scope to achieve the appropriate authorizations, it is critical that:

1. A technical team, comprised of representatives from each appropriate regulatory agency and members of the design team, be assembled and maintained throughout the project
2. Meetings and updates occur regularly so individuals don't lose track of progress and decisions that have been made
3. Design parameters remain flexible throughout the permitting process
4. Documents and communication records are clear, up to date, and diligently maintained in a chronological sequence

Specific regulatory authorizations we expect include Bonner County, state of Idaho and Federal agencies.

BONNER COUNTY

Bonner County is the local government jurisdiction. We expect the county to require a conditional use permit to be issued for the potential thorofare and/or dam improvements. Conditional use permits are considered for projects with unique characteristics and are considered individually. To apply for the Conditional Use Permit we expect project-specific plans and details will need to be submitted to Bonner County as specified in Title 12, Subchapter 2.2 of the Bonner County Code.

STATE OF IDAHO

Idaho Department of Environmental Quality (IDEQ) is responsible for implementing Section 401 (water quality certification) of the Clean Water Act on behalf of the Environmental Protection Agency (EPA). Water quality certification will be required before construction of the thorofare improvements can proceed. This process to be closely coordinated with the Section 404 process (see below) because Section 401 conditions are often used, in part, as conditions for Section 404 authorization. IDEQ has up to one year to provide a Section 401 certification decision.

Idaho Department of Lands (IDL) is responsible for issuing a Lake Encroachment Permit under the Idaho Lake Protection Act. The intent of the permit is to weigh the benefit of thorofare improvement structure(s) within the context of private property, navigation, fish and wildlife habitat, aquatic life, recreation, water quality, and aesthetic beauty. Because Priest Lake is a navigable waterway, IDL will need to review the project details and issue a permit before the project can proceed. We expect this process will be iterative with substantial input from multiple agencies and the public at large and will be associated with the Section 404 permit process.

Idaho Department of Water Resources (IDWR) is responsible for issuing a stream channel alteration permit under the Idaho Stream Channel Protection Act. It is somewhat unusual that a Stream Channel Alteration permit and Lake Encroachment Permit are necessary for the same project but because the Thoroughfare has perennial flow, defined bed and banks, and is a documented migration corridor for bull trout, the Stream Channel Alteration permit applies. The intent of the permit is to protect surface water resources, biological communities, and public safety. Like the Lake Encroachment Permit, we expect this process to be iterative with substantial input from multiple agencies and the public at large and will be associated with the Section 404 permit process.

IDWR is also responsible for dam safety in the State of Idaho. Accordingly, any potential modifications to the outlet dam will need authorization through IDWR. Dam modifications must be designed by a Professional Engineer licensed in the State of Idaho and must meet or exceed the prescriptive design and construction requirements described in IDAPA 37.03.06.

Idaho Department of Fish and Game (IDFG) is not responsible for issuing a specific permit but, as a co-manager of fish and wildlife resources, consultation with them will be mandatory throughout the duration of the project. Some of the permit conditions (local, State, and Federal) are likely to be recommendations made by IDFG.

FEDERAL GOVERNMENT

We expect the *U.S. Army Corps of Engineers (USACOE)* to be the lead Federal agency for the project because Priest Lake is considered “waters of the United States.” Since potential dredging and filling will occur below the ordinary high-water mark, a Section 404 Clean Water Act permit will be required. The USACOE could also require a Section 10 of the Rivers and Harbors Act permit because Priest Lake is designated navigable by the State of Idaho. It is possible the Section 10 permit won’t be required because there is question regarding Priest Lake’s federal navigable designation but the USACOE will make the decision on applicability.

Because the USACOE will likely have jurisdiction, the Federal National Environmental Policy Act (NEPA) process will be engaged. The NEPA process will be guided by the USACOE and is intended to give consideration to the environment before moving a project forward. The process many paths and associated timelines ranging from issuance of a Categorical Exclusion (CE) to preparation of an Environmental Impact Statement (EIS). In this case we expect the NEPA could involve the preparation of an Environmental Assessment (EA) and ultimately issuance of a Finding of No Significant Impact (FONSI). However, it must be noted that the process and findings are a Federal responsibility.

Through the NEPA process, the USACOE will be required to coordinate with other regulatory agencies to fulfill their due diligence. As an example, presence of bull trout (*Salvelinus confluentus*) is documented in the project area and the fish are listed as Threatened, under the Endangered Species Act (ESA). We expect consultation with the US Fish and Wildlife Service (USFWS) will be necessary to comply with Section 7 and Section 10 of the ESA. Further, given the project area's location within the aboriginal territory of several Indian tribes, it is probable the area was inhabited, at least seasonally, and it is possible that cultural resources exist within the project boundaries. Coordination with the tribes and other interested parties will be initiated to comply with Section 106 of the National Historic Preservation Act (NHPA) so the project is developed in a manner that avoids or minimizes impacts to cultural resources.

Regulatory processes are controlled by each respective agency based on the unique circumstances of the project and their regulatory authority. There is some predictability in the permitting processes; however, it is difficult to predict specific permit conditions, public involvement, plan change requests, additional research needs, and participation by requested entities. For those reasons it is difficult to determine timelines for completion and associated budgets.

E.6 IWRB October 24 Meeting Presentation

MOTT MACDONALD

Priest Lake Water Management Study

IWRB Board Briefing Meeting– 10/24/2017

IDAHO WATER RESOURCE BOARD

PRESENTATION OUTLINE

- Criteria, Scope & Schedule
- Thorofare Geomorphology & Hydraulics
- Thorofare Improvements Alternatives Evaluation
- Navigation Dredging
- Thorofare Improvement Summary
- Pool Raise Assessment
- Estimated Costs & Recommendations

STUDY CRITERIA, SCOPE, & SCHEDULE

Study Criteria

1

Lake Level Management:
Maintain Lake Level at 3.0' during Recreation Season in Dry Years, improve habitat & minimize shoreline impacts

2

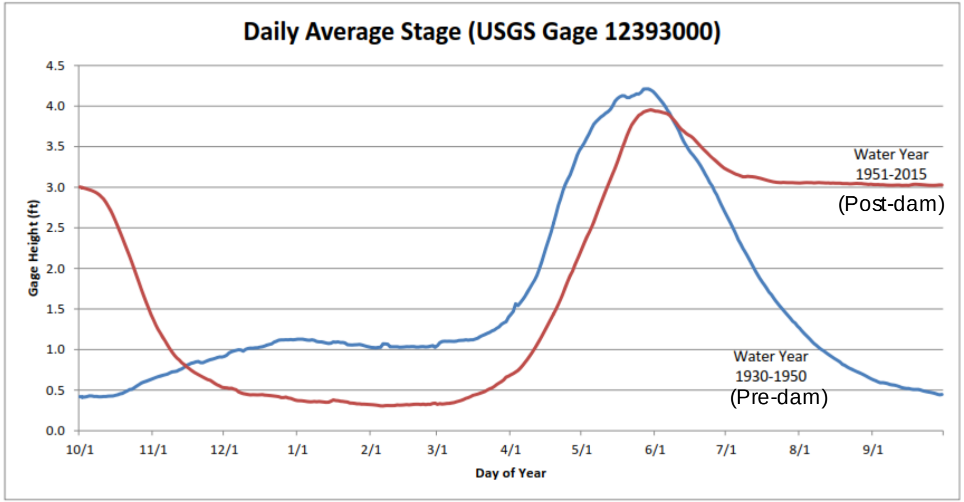
Minimum Outlet Structure Flows:
Maintain current minimum discharge flow requirements downstream of the dam

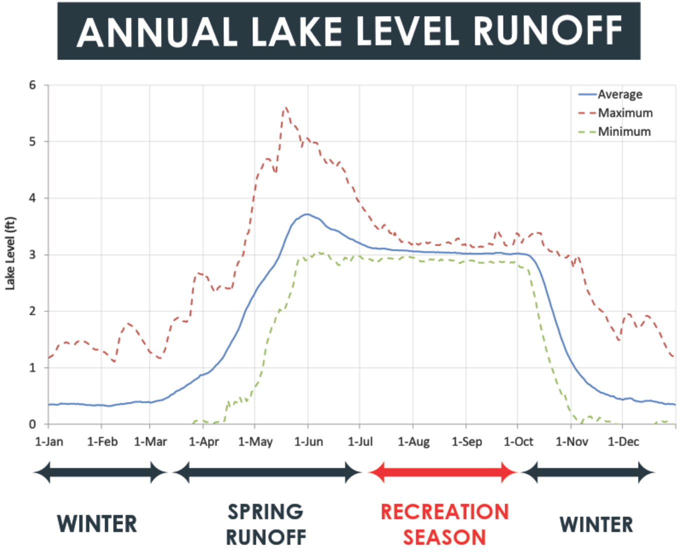
3

Thorofare Sustainability:
Promote self-sustaining improvements to Thorofare access, navigability and water quality

HISTORICAL DATA COLLECTION & ASSESSMENT

- Lake level Evaluation
- Historical Information provided by IDWR
- 1951 = Outlet Structure Construction





NEW DATA COLLECTION - BATHYMETRY

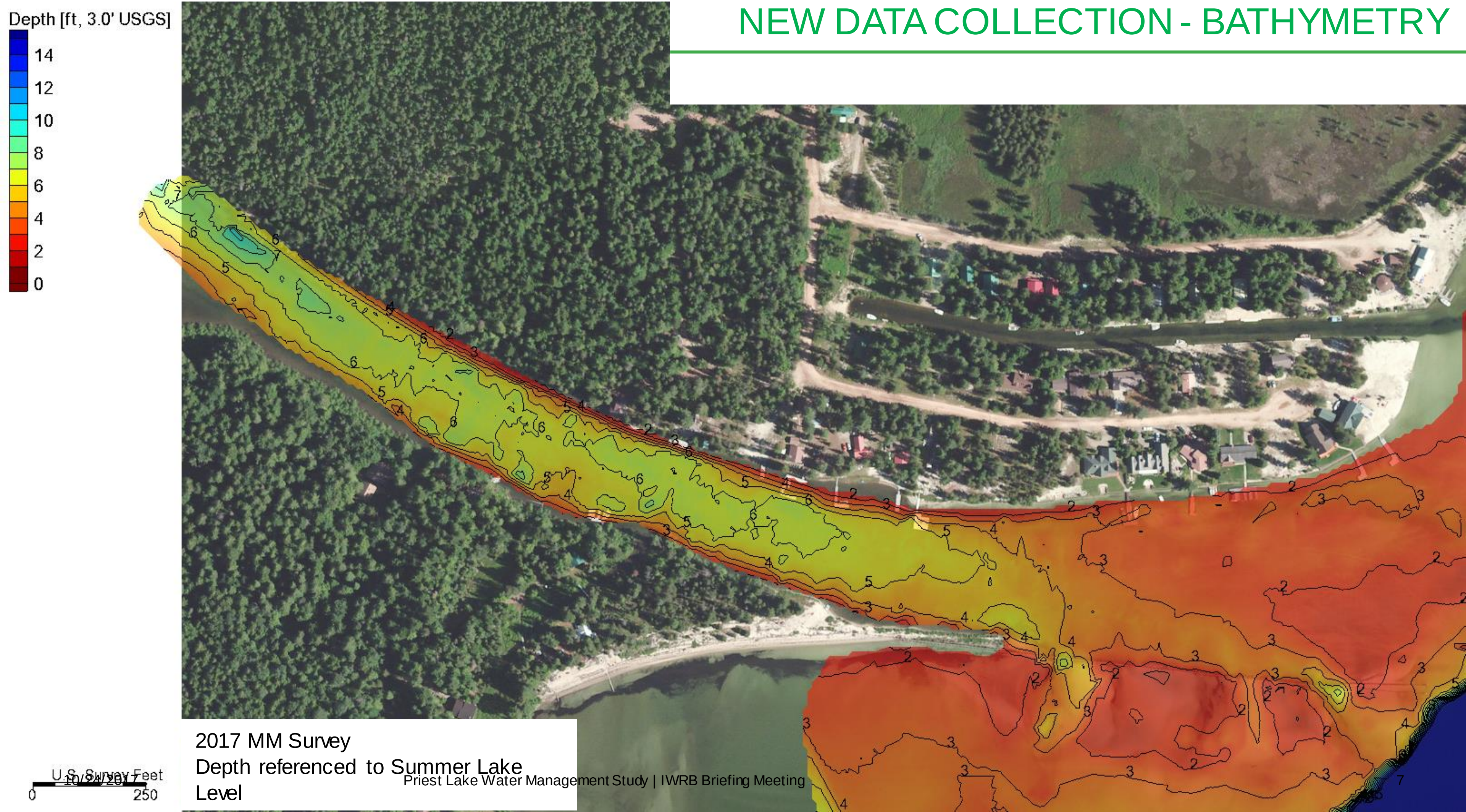
- Hydrographic Survey
  - Data Processing Complete
  - Final Plotting & Deliverable in process

- ADCP
  - Data Processing

Figure 2 Survey Boat (ADCP visible on portside stern – transit configuration).

Figure 3 RTK Base set over Benchmark

NEW DATA COLLECTION - BATHYMETRY

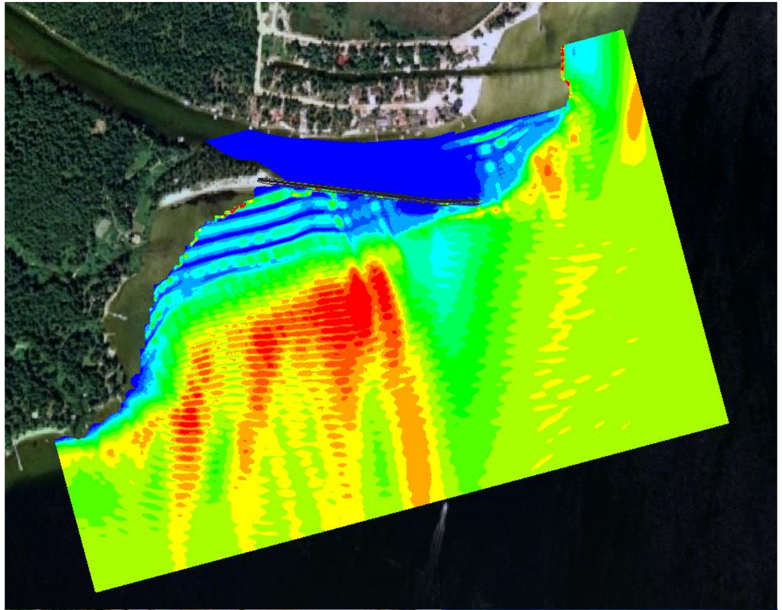


2017 MM Survey
Depth referenced to Summer Lake Level

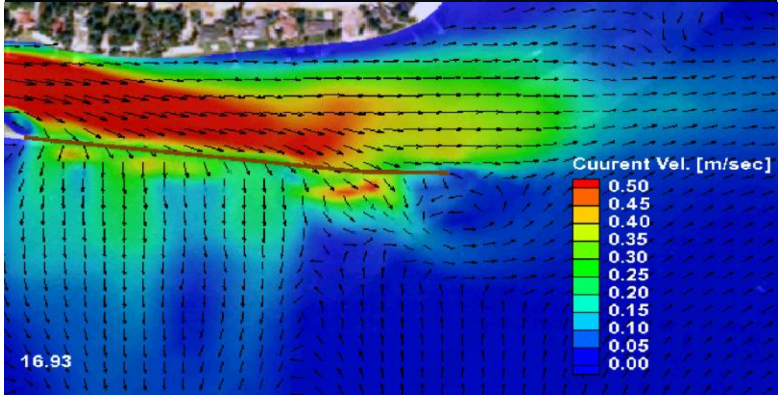
HYDRODYNAMIC ANALYSIS

- Both Thorofare & Lake
- Basis for evaluation of improvements

Wave Analysis



Currents



BASIS OF ANALYSIS/CRITERIA

Developed in coordination with IDWR/Bonner County

Considerations

- Standards
- Water Levels
- Historical Operations
- Future Operations
- Dry Year vs. Every Year Water Management
- Outlet Dam Discharge & Timing thereof
- Recreational Period
- Navigation, Vessel Size & Thorofare Use
- Species/Habitat Considerations
- Climate Change Considerations
- Outlet Dam Operation Criteria
- Property Ownership
- Dam Safety

PUBLIC & STAKEHOLDER OUTREACH

- Stakeholder Group Outreach
  - Steering Committee
  - Local Stakeholders
  - Govt Agency Outreach
- Public Open House
  - Mid July
  - Late September
- Community Postings
- Websites
- Social Media

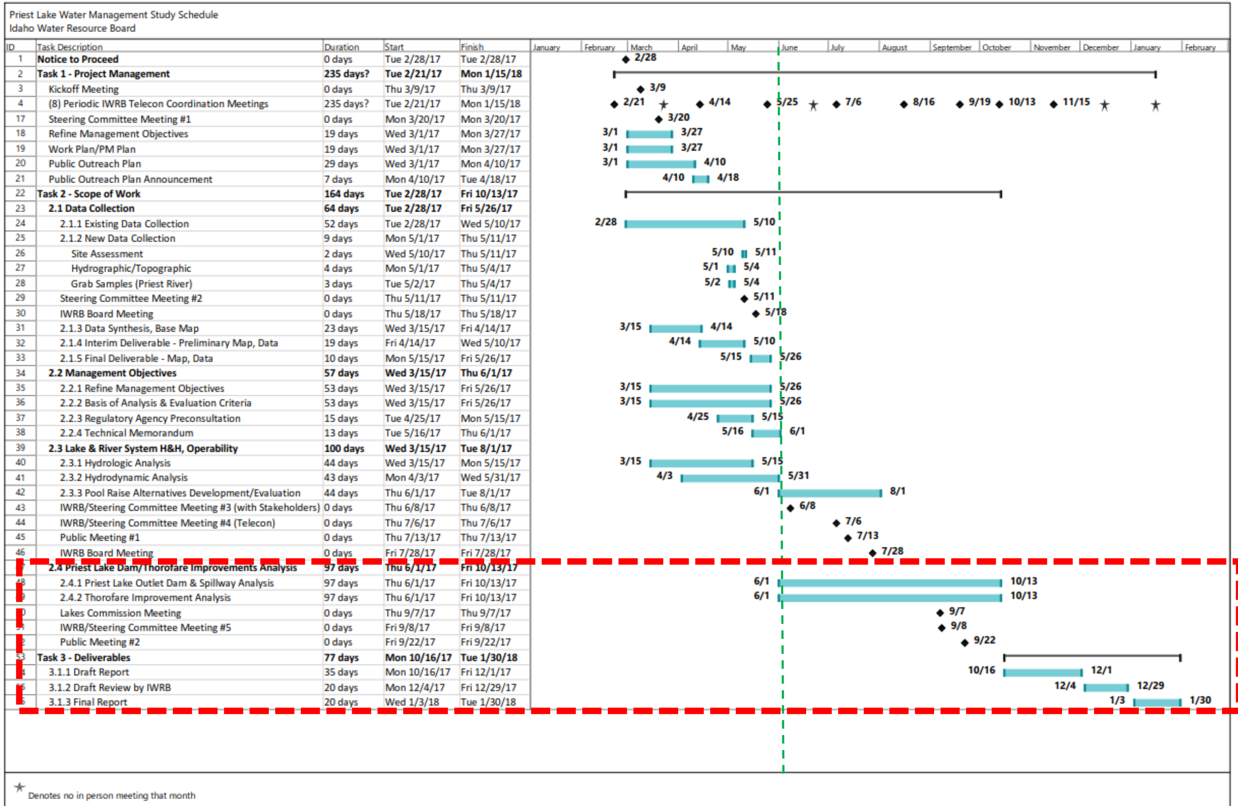
SCHEDULE

Task 1- Project Management

- Public Outreach Plan
  Complete
- Project Management Plan
  Complete – Send final to IDWR

Task 2 – Scope of Work

- 2,1 Data Collection
  100% completed;
- 2.2 Basis of Analysis
  100% Completed
- 2.3 Lake & River System H&H
  90% Completed
- 2.4 Thorofare/Dam Improvements
  75% Completed
- Meetings
  Steering Committee Meetings
  Stakeholder meetings – Periodic
  IWRB Meeting – Oct 24th
  Reporting – Nov (Draft Report), Final in December



Priest Lake Water Management Study Schedule
Idaho Water Resource Board

ID	Task Description	Duration	Start	Finish
1	**Notice to Proceed**	0 days	Tue 2/28/17	Tue 2/28/17
2	**Task 1 - Project Management**	**235 days?**	**Tue 2/21/17**	**Mon 1/15/18**
3	Kickoff Meeting	0 days	Thu 3/9/17	Thu 3/9/17
4	(8) Periodic IWRB Telecon Coordination Meetings	235 days?	Tue 2/21/17	Mon 1/15/18
17	Steering Committee Meeting #1	0 days	Mon 3/20/17	Mon 3/20/17
18	Refine Management Objectives	19 days	Wed 3/1/17	Mon 3/27/17
19	Work Plan/PM Plan	19 days	Wed 3/1/17	Mon 3/27/17
20	Public Outreach Plan	29 days	Wed 3/1/17	Mon 4/10/17
21	Public Outreach Plan Announcement	7 days	Mon 4/10/17	Tue 4/18/17
22	**Task 2 - Scope of Work**	**164 days**	**Tue 2/28/17**	**Fri 10/13/17**
23	**2.1 Data Collection**	**64 days**	**Tue 2/28/17**	**Fri 5/26/17**
24	2.1.1 Existing Data Collection	52 days	Tue 2/28/17	Wed 5/10/17
25	2.1.2 New Data Collection	9 days	Mon 5/1/17	Thu 5/11/17
26	Site Assessment	2 days	Wed 5/10/17	Thu 5/11/17
27	Hydrographic/Topographic	4 days	Mon 5/1/17	Thu 5/4/17
28	Grab Samples (Priest River)	3 days	Tue 5/2/17	Thu 5/4/17
29	Steering Committee Meeting #2	0 days	Thu 5/11/17	Thu 5/11/17
30	IWRB Board Meeting	0 days	Thu 5/18/17	Thu 5/18/17
31	2.1.3 Data Synthesis, Base Map	23 days	Wed 3/15/17	Fri 4/14/17
32	2.1.4 Interim Deliverable - Preliminary Map, Data	19 days	Fri 4/14/17	Wed 5/10/17
33	2.1.5 Final Deliverable - Map, Data	10 days	Mon 5/15/17	Fri 5/26/17
34	**2.2 Management Objectives**	**57 days**	**Wed 3/15/17**	**Thu 6/1/17**
35	2.2.1 Refine Management Objectives	53 days	Wed 3/15/17	Fri 5/26/17
36	2.2.2 Basis of Analysis & Evaluation Criteria	53 days	Wed 3/15/17	Fri 5/26/17
37	2.2.3 Regulatory Agency Preconsultation	15 days	Tue 4/25/17	Mon 5/15/17
38	2.2.4 Technical Memorandum	13 days	Tue 5/16/17	Thu 6/1/17
39	**2.3 Lake & River System H&H, Operability**	**100 days**	**Wed 3/15/17**	**Tue 8/1/17**
40	2.3.1 Hydrologic Analysis	44 days	Wed 3/15/17	Mon 5/15/17
41	2.3.2 Hydrodynamic Analysis	43 days	Mon 4/3/17	Wed 5/31/17
42	2.3.3 Pool Raise Alternatives Development/Evaluation	44 days	Thu 6/1/17	Tue 8/1/17
43	IWRB/Steering Committee Meeting #3 (with Stakeholders)	0 days	Thu 6/8/17	Thu 6/8/17
44	IWRB/Steering Committee Meeting #4 (Telecon)	0 days	Thu 7/6/17	Thu 7/6/17
45	Public Meeting #1	0 days	Thu 7/13/17	Thu 7/13/17
46	IWRB Board Meeting	0 days	Fri 7/28/17	Fri 7/28/17
7	**2.4 Priest Lake Dam/Thorofare Improvements Analysis**	**97 days**	**Thu 6/1/17**	**Fri 10/13/17**
48	2.4.1 Priest Lake Outlet Dam & Spillway Analysis	97 days	Thu 6/1/17	Fri 10/13/17
9	2.4.2 Thorofare Improvement Analysis	97 days	Thu 6/1/17	Fri 10/13/17
0	Lakes Commission Meeting	0 days	Thu 9/7/17	Thu 9/7/17
1	IWRB/Steering Committee Meeting #5	0 days	Fri 9/8/17	Fri 9/8/17
2	Public Meeting #2	0 days	Fri 9/22/17	Fri 9/22/17
53	**Task 3 - Deliverables**	**77 days**	**Mon 10/16/17**	**Tue 1/30/18**
4	3.1.1 Draft Report	35 days	Mon 10/16/17	Fri 12/1/17
5	3.1.2 Draft Review by IWRB	20 days	Mon 12/4/17	Fri 12/29/17
6	3.1.3 Final Report	20 days	Wed 1/3/18	Tue 1/30/18

★ Denotes no in person meeting that month

THOROFARE – GEOMORPHOLOGY & HYDRAULICS

THOROFARE

ISSUES

- Boat access to Thorofare & navigation is challenging at the Thorofare mouth
- Deteriorated breakwater structure

STUDY GOALS

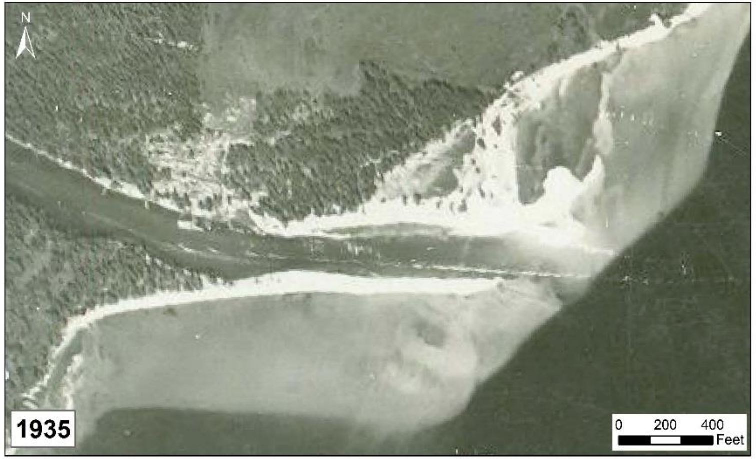
- Providing sustainable modifications to improve Thorofare access, navigability, & water quality (minimize maintenance dredging needs)

GEOMORPHIC ASSESSMENT GOALS

- Evaluate flow & sedimentation processes at Thorofare mouth to aid in evaluation of Thorofare improvement alternatives

THOROFARE HISTORY

- Historical aerial imagery suggests that alignment of Breakwater was different in 1935 and Thorofare mouth was narrower
- Widening the Thorofare mouth is likely to have reduced the flow velocities and subsequently sediment transport capacity of Thorofare



THOROFARE BREAKWATER

- Original timber pile breakwater (BW) was constructed by USFS to facilitate access to Thorofare in 1933 (IMR 1989)
- Breakwater serves an additional function of providing wave shelter to lakefront properties in Sandpiper's Shore
- Breakwater structure composed of untreated timber
- Breakwater is considered porous since there is a ~ 1-in gap between the plank boards and a ~ 10-in gap between bottom of planks and Thorofare bed (BW porosity ~ 20% to 35%)

BREAKWATER HISTORY

Background:

- Timeframe: 1920’s to 2013
- Multiple Replacements & Major Repairs

Observations:

- Damages to breakwater have occurred approximately every 7 to 10 years, resulting in need for (partial) repair
- Non-engineered repairs of breakwater have not withstood strong spring run-offs

Summary:

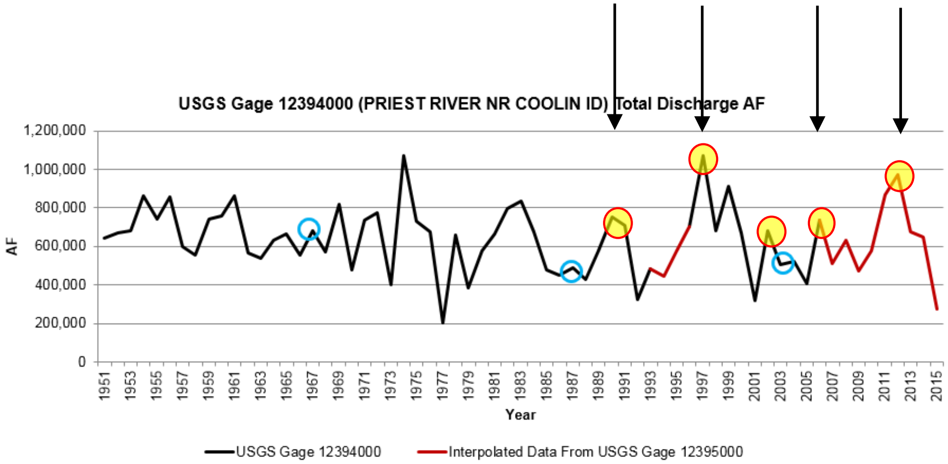
- Service life of breakwater ~ 30 to 40 years with periodic repairs
- Portions of Breakwater currently nearing end of service life

1990 BW replacement

1997 Significant damage to BW

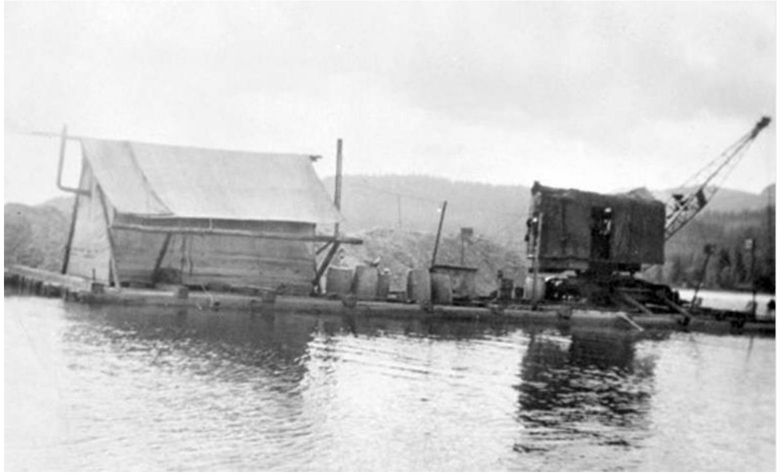
2006 Damage to BW

2013 partial repair & damage in winter



THOROFARE DREDGE HISTORY

- 1930's: historical photo showing mechanical dredging of Thorofare using a barge
- 1940 – 1990's: anecdotal accounts suggest episodic mechanical dredging
- 1990s – present: no official records of dredging but a few permit exist.



Summary:

- Regular maintenance dredging program has not been in place since 1990s; this has placed greater focus on effectiveness/performance of breakwater in directing the flow & avoiding flow spreading

THOROFARE FLOW SPREADING

- Shallow sand bar at the mouth on 3/15/2005, WL = 0.33' USGS
- Significant flow spreading & flow cutting into the sand bar and underneath the breakwater
- Aerial photo suggests ~40% of flow is going underneath the Breakwater, % to be verified with numerical modeling


3/15/2005


3/15/2005

Data Collection -AERIAL PHOTO

MAY 29, 2017

THOROFARE FLOW SPREADING

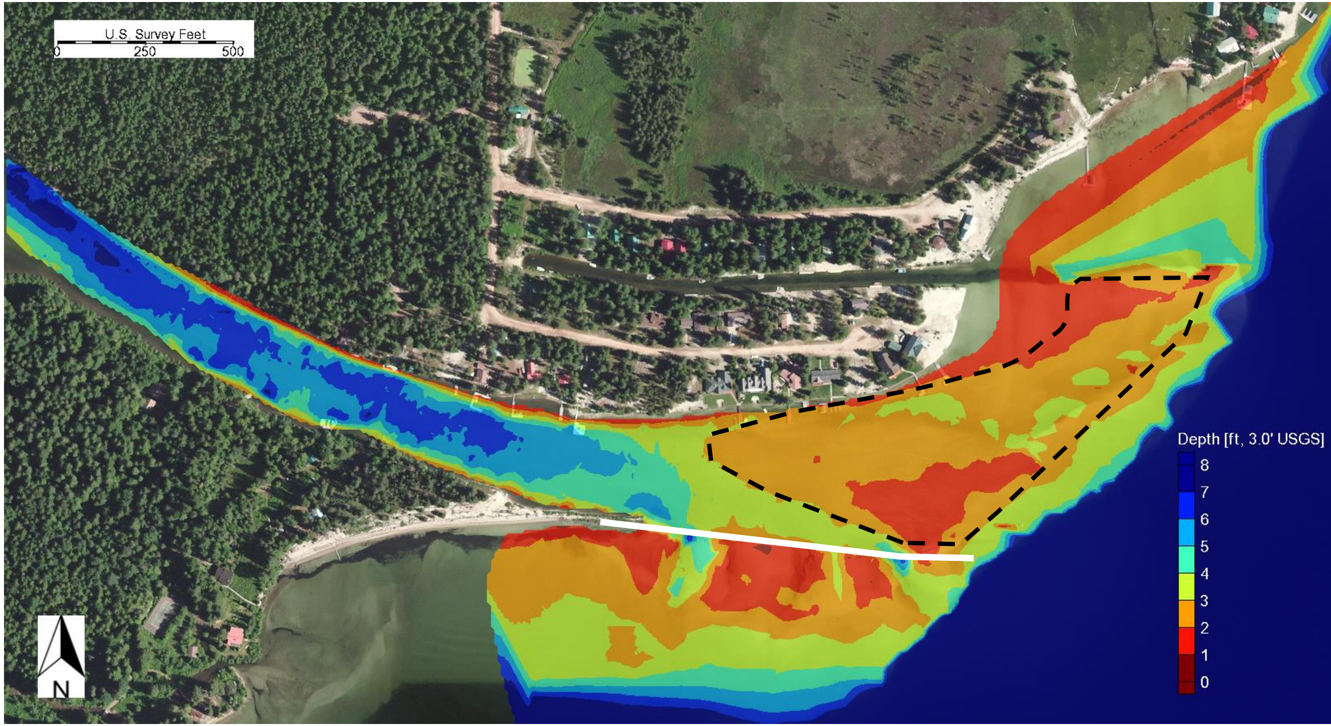
- Thorofare flow forms channels underneath the breakwater.
- The channel locations & width vary with time

THOROFARE HYDROGRAPHY

- Water depths at the Mouth outlined by black dashed line is mostly shallower than 3 ft, with some areas shallower than 2 ft

Conclusion:

- Dredging & improvements to better confine Thorofare flow likely needed to maintain navigable access
- Accurate marking of Thorofare mouth by buoys would be important to help boaters access Thorofare



BED LOAD SEDIMENT TRANSPORT

- Bed load sediment transport is dominant sediment source in the Thorofare.
- Sources come from tributaries and Thorofare bank adjustment.
- Majority of bedload is likely transported during spring runoff high flow and certain winter high flow conditions.
- Flow diversion at the breakwater reduces sediment transport capacity of Thorofare resulting in sediment deposition in the outlet area near the breakwater.

LAKE SHORELINE - WAVE-DRIVEN SEDIMENT TRANSPORT

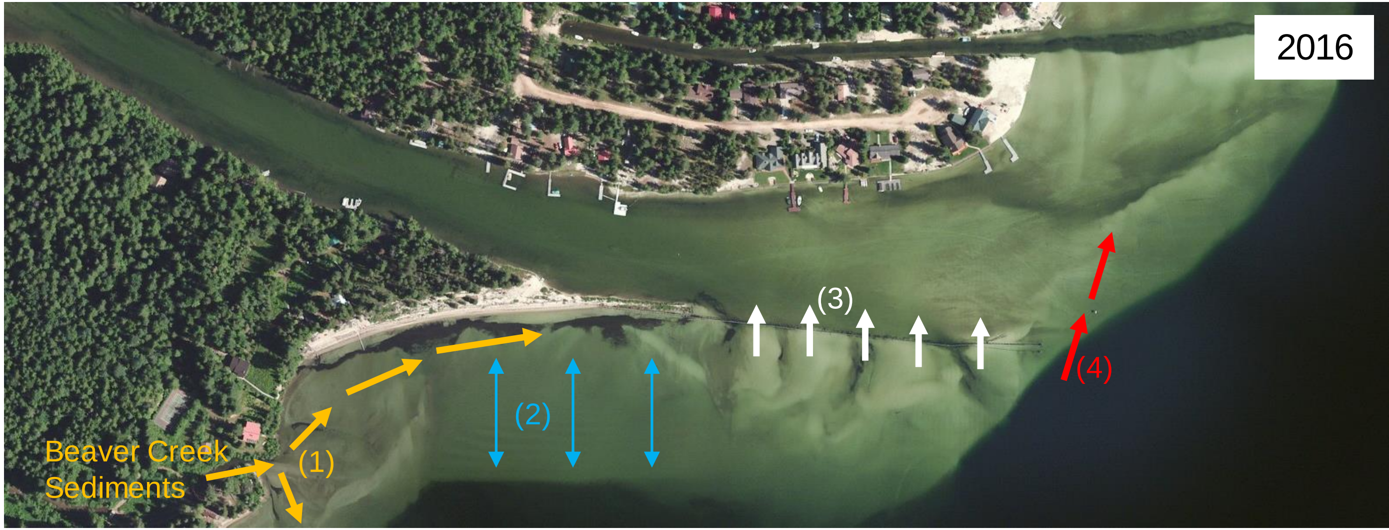
Wave-driven sediment sources & transport directions include:

1) Beaver Creek

2) Waves move sediments perpendicular to shoreline

3) Waves may push some sediments through & underneath the breakwater

4) Portion of sediments will move past the breakwater end

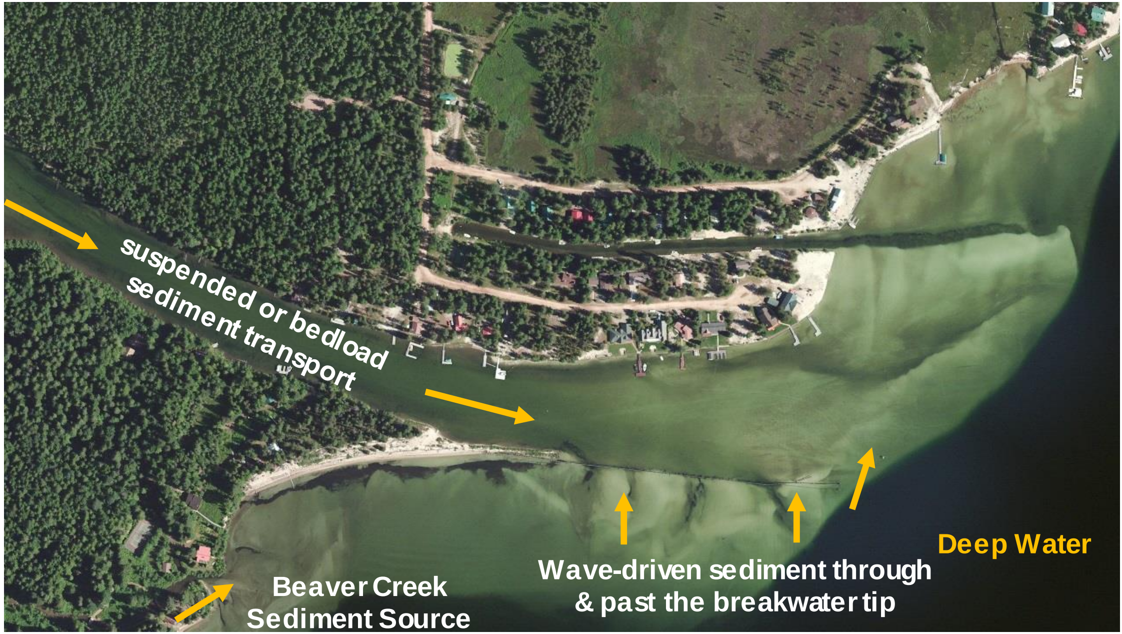


THOROFARE GEOMORPHOLOGY SUMMARY

- Sediment deposition is result of decreased transport capacity as low-gradient Thorofare meets zero-gradient Lake and flow spreading.

- Deposition has been accentuated by three factors at the mouth:

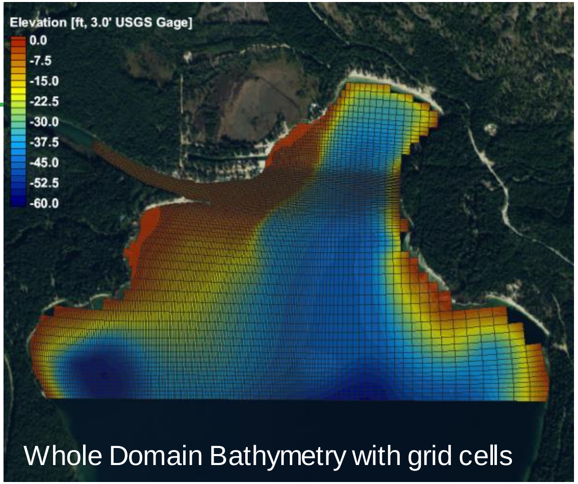
1) Widening of Thorofare channel along Breakwater
2) Reduced Thorofare discharge & velocity as water passes through or under existing timber breakwater
3) Wind-driven sediments get pushed through and around the breakwater eastern end



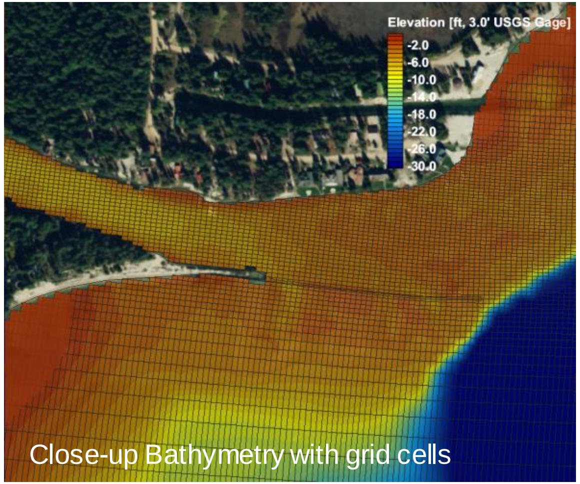
HYDRAULIC MODELING

- Delft3D-FLOW is a three-dimensional (3-D) hydrodynamic and transport simulation program which calculates non-steady flow and transport phenomena that result from river forcing on a curvilinear, boundary fitted grid.

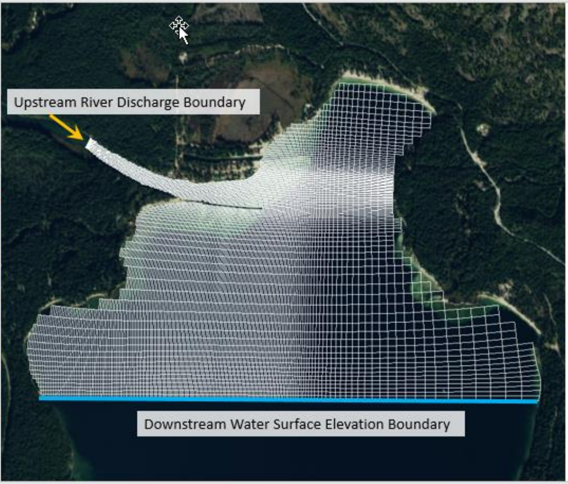
- Delft3D is widely accepted and used in industry and academia



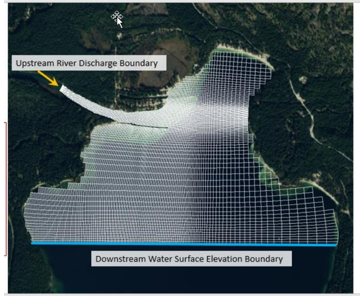
Whole Domain Bathymetry with grid cells



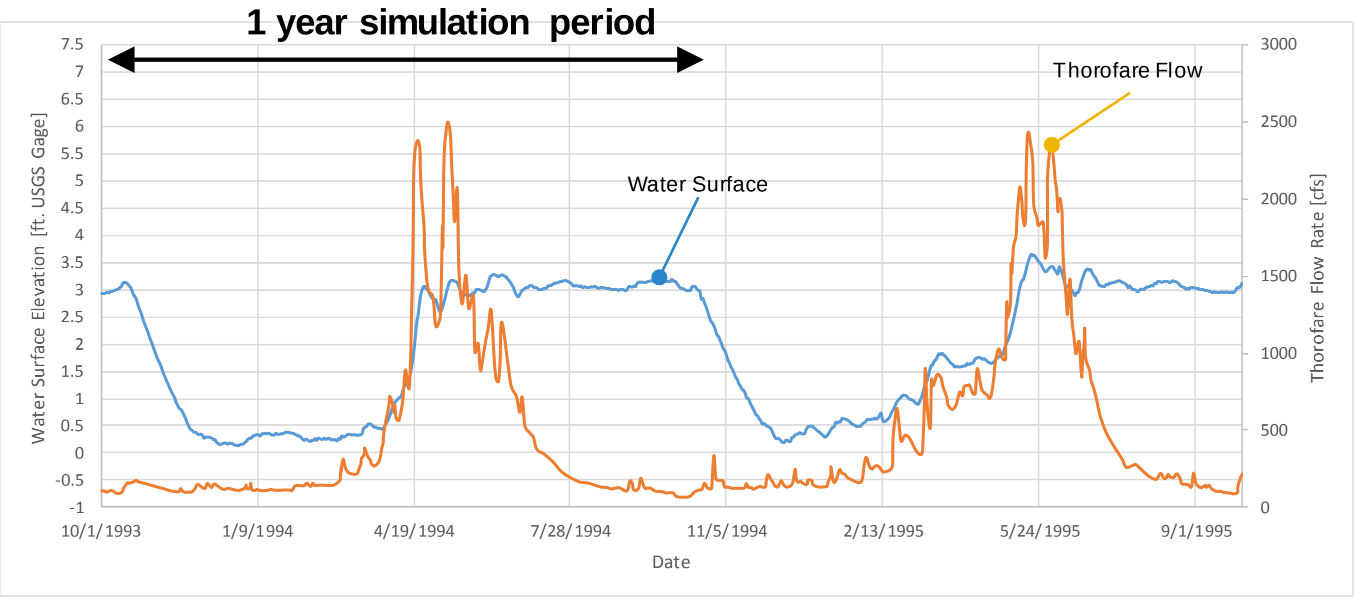
Close-up Bathymetry with grid cells



MODEL INPUT - BOUNDARY CONDITIONS

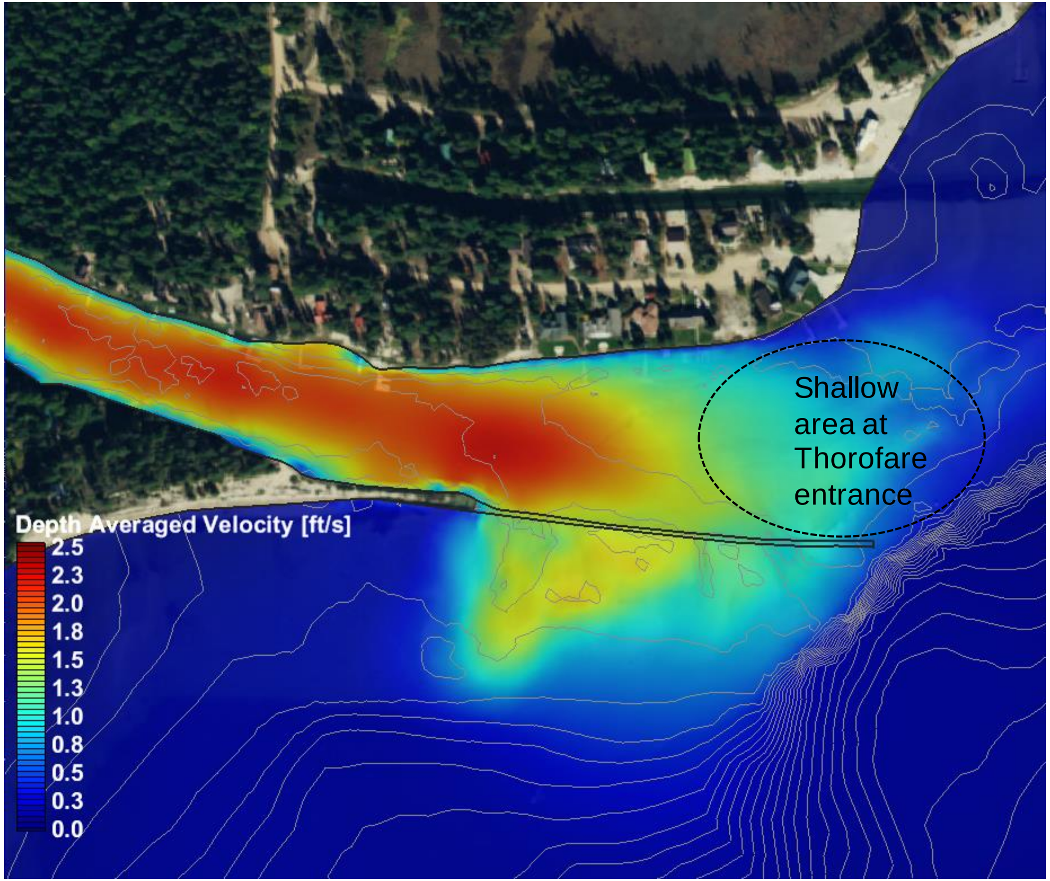


- Run a 1-year simulation in order to simulate a wide range of hydrodynamic conditions as well as see any sufficient mid to long term sedimentation.
- 1994 water year simulation

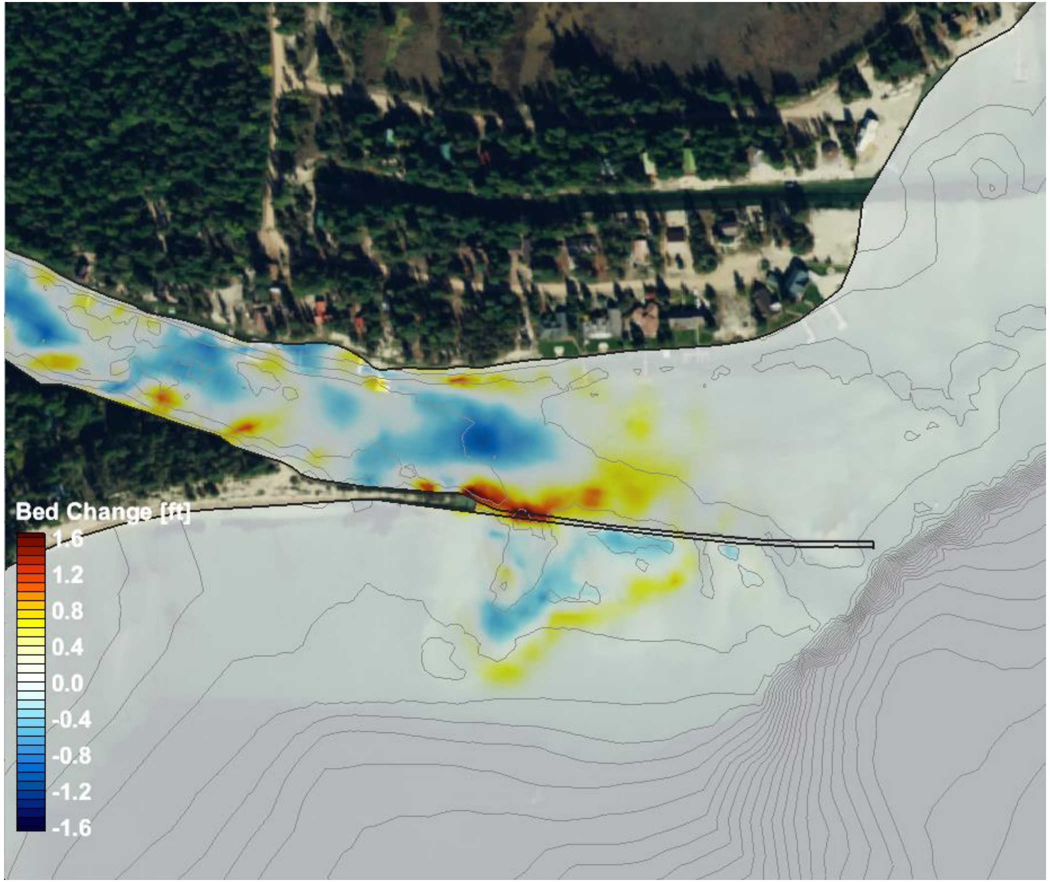


MODELING RESULTS – EXISTING CONDITIONS RESULTS (Velocity)

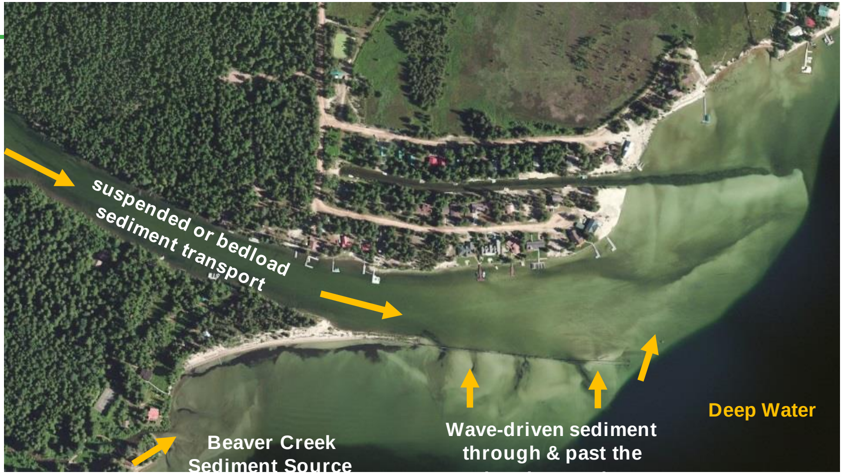
- Maximum Velocity: 2.2 ft/s
- Flow spreading along breakwater is represented in model results
- Significant drop in velocity at approximately ½ the length of the breakwater
- Velocity reduction zone corresponds with area of reduced depth (shoal) at entrance to Thorofare



MODELING RESULTS – EXISTING CONDITIONS (sediment transport summary)



THOROFARE NAVIGATION IMPROVEMENTS – SCREENING ANALYSIS BACKGROUND



ISSUES

- Motorized boat access (recreational & emergency services) and navigation is challenging at the Thorofare mouth; need to increase depths
- Deteriorated breakwater structure with periodic maintenance and repair history

CONSIDERATIONS

- Thorofare flow splitting ~ less effective sediment transport
- Sediment migration from lake shoreline into Thorofare
- Repeated failures of breakwater increasing extent of Thorofare flow splitting

IMPROVEMENTS

- Geomorphic & Hydraulic analysis indicates the following:
  - Reduce sediment migration from lake shoreline into Thorofare (tighten up area from spit to lake deep water)
  - Utilize an impermeable sediment retention structure or breakwater

THOROFARE – POTENTIAL IMPROVEMENT ALTERNATIVES

Improvement Alternatives Considered:

1) No Action (maintain existing)
   - No repairs, improvements or dredging

2) Removal of Breakwater
   - Complete Removal with dredging

3) Rehabilitate Existing Porous Breakwater
   - Rehab existing damaged areas, continue repairs in future, conduct dredging to restore navigation

4) Replace Existing Porous Breakwater with Rotated Sediment Retention Feature
   - Construction new feature to replace breakwater and conduct dredging to restore navigation; see next slide for details

5) In channel flow diversion
   - Construction new feature to supplement breakwater and conduct dredging to restore navigation; see next slide for details

▨ = Focus Area for Thorofare Improvements

ALTERNATIVE NO. 1 – NO ACTION

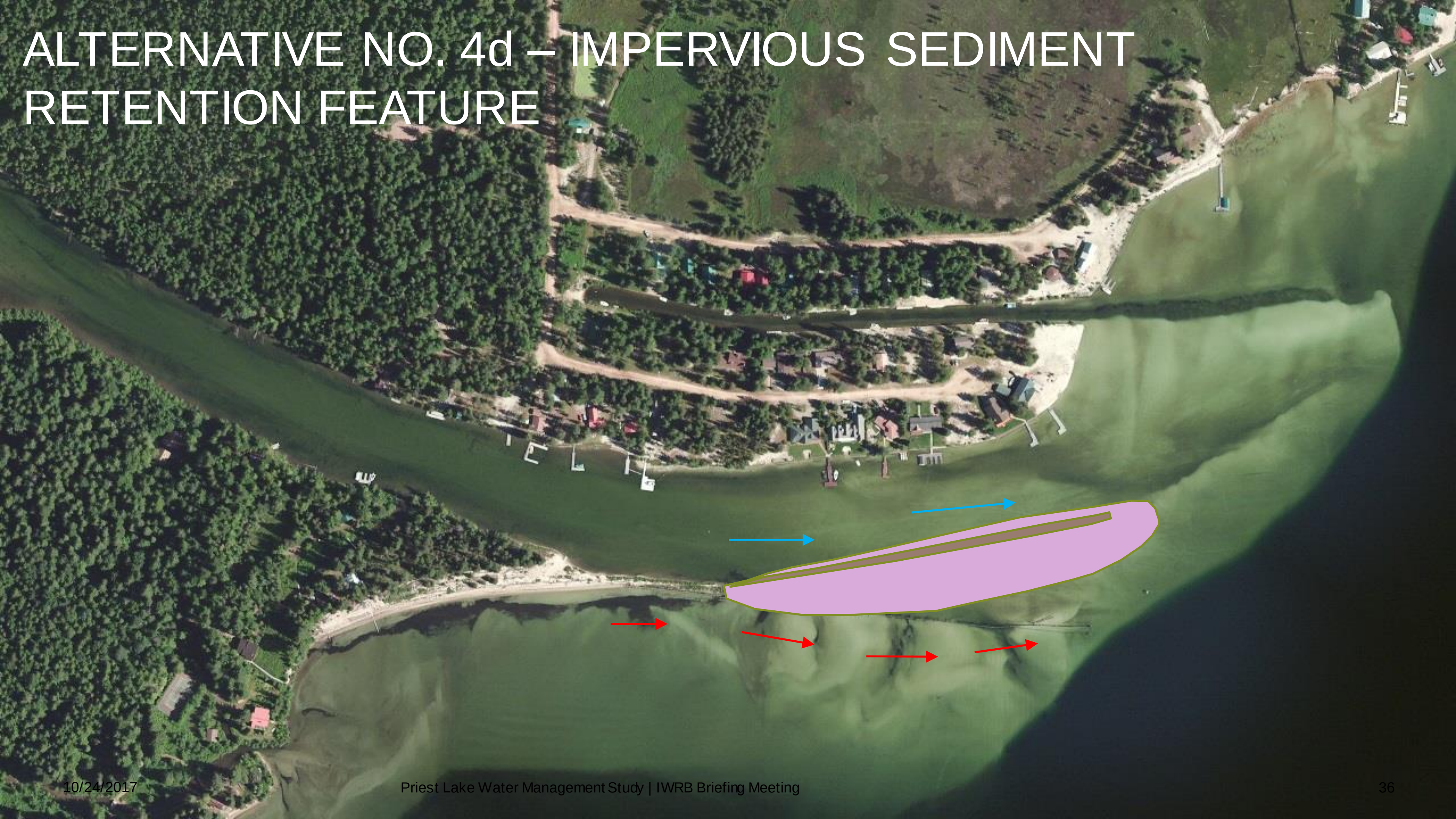
Continued Shoaling; Summer Depths < 2 ft

ALTERNATIVE NO. 2 – REMOVE BREAKWATER

ALTERNATIVE NO. 3 – REPAIR POUROUS BREAKWATER

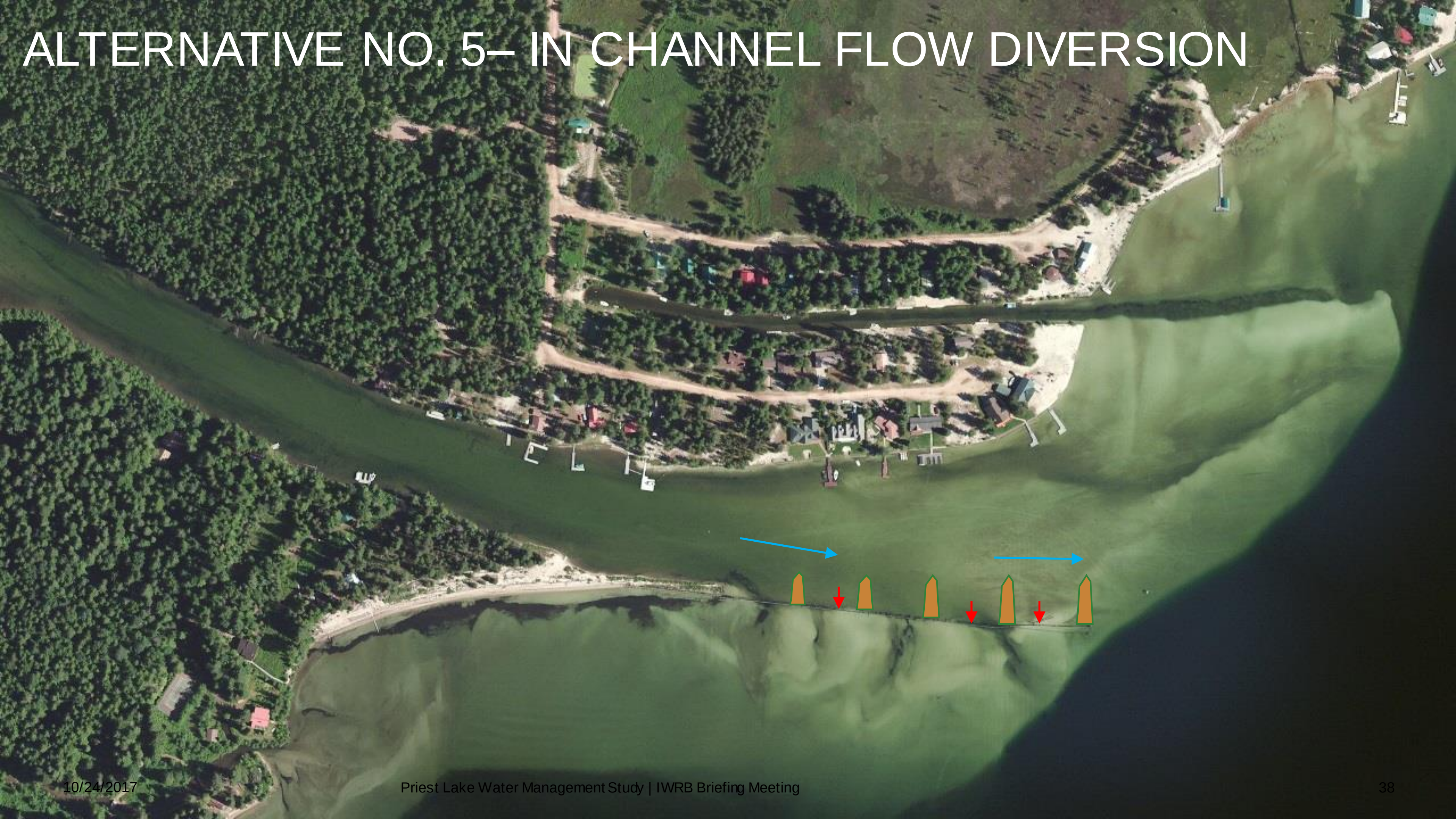
ALTERNATIVE NO. 4a – IMPERVIOUS SEDIMENT RETENTION FEATURE

ALTERNATIVE NO. 4d – IMPERVIOUS SEDIMENT RETENTION FEATURE



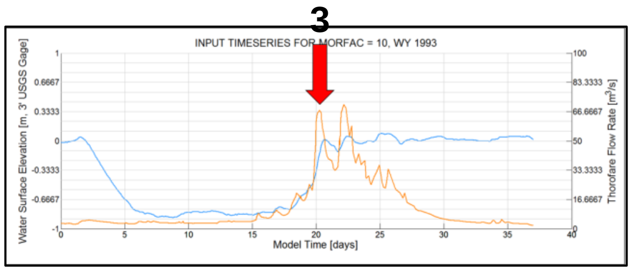
ALTERNATIVE NO. 4b – IMPERVIOUS SEDIMENT RETENTION FEATURE

ALTERNATIVE NO. 5– IN CHANNEL FLOW DIVERSION

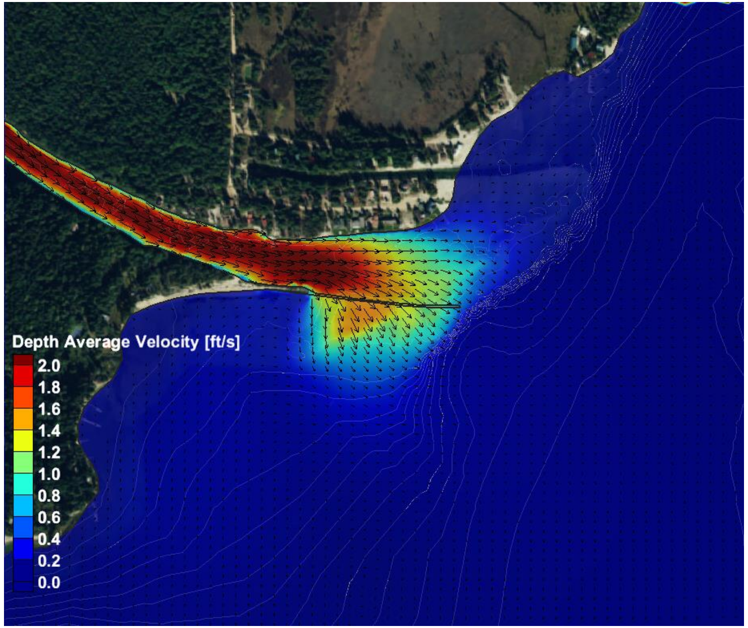


ALTERNATIVE NO. 6 – Partial In Channel Flow Deflection

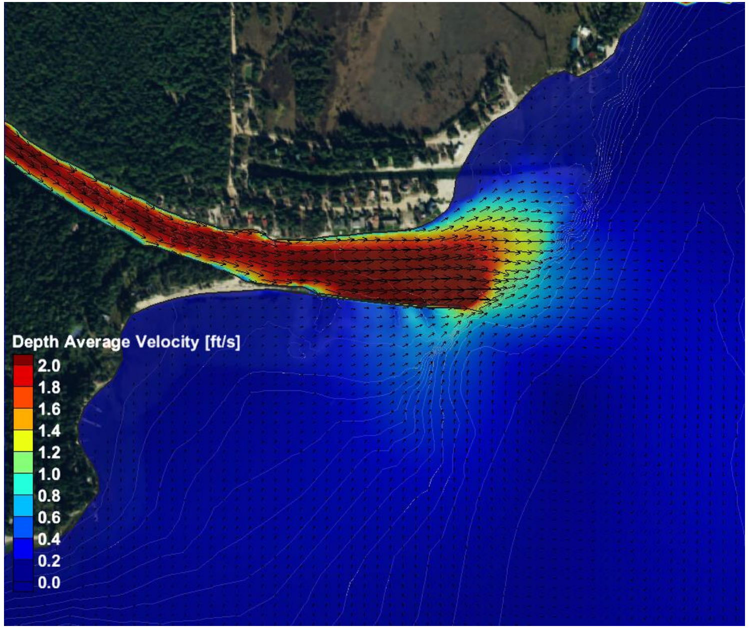
MODELING RESULTS – SENSITIVITY TESTING



Snapshot of depth-averaged velocity for Porous Breakwater vs. Solid Feature



Porous Breakwater

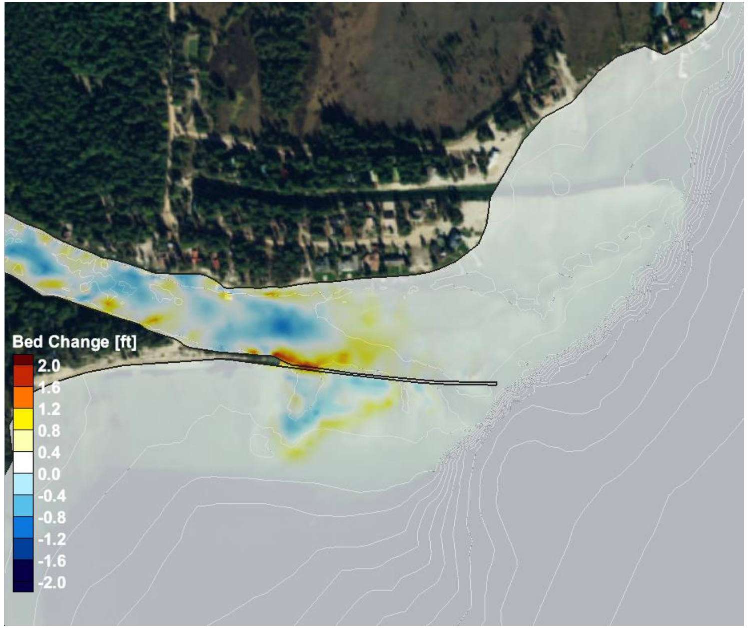


Solid Feature

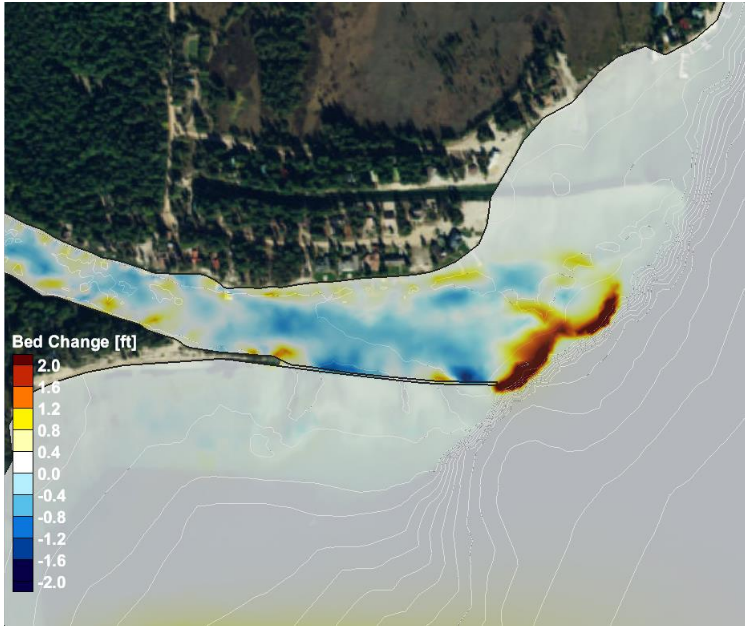
MODELING RESULTS – SENSITIVITY TESTING

Maximum Bed Change (sedimentation/erosion) during the simulation period

Solid Feature: transports material and deposits them in deeper water past the feature end.



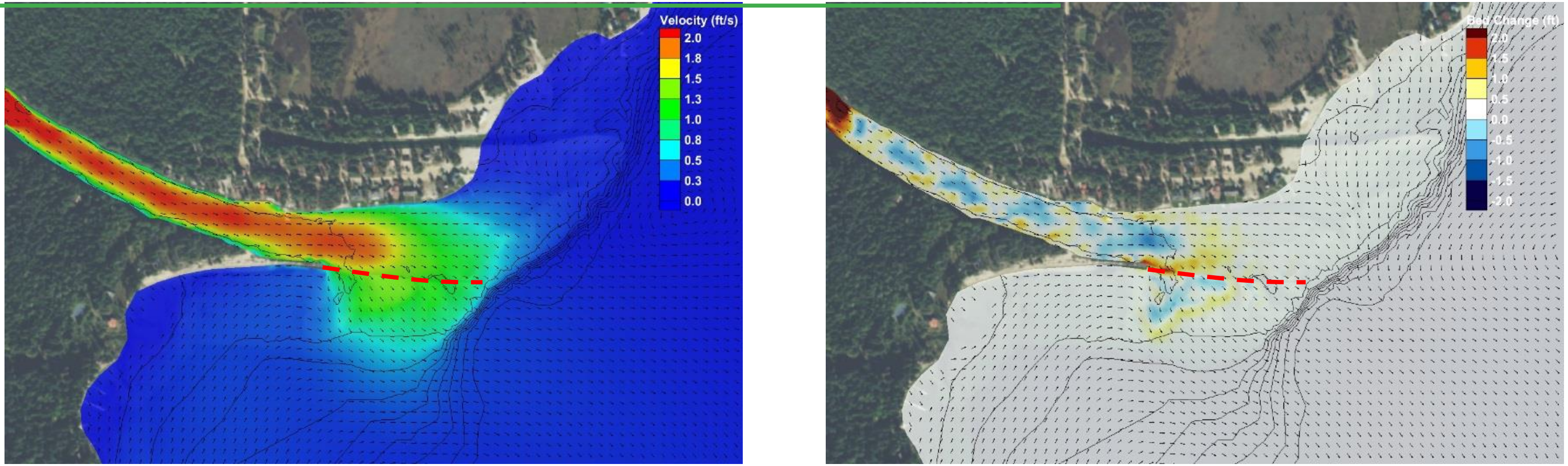
Porous Breakwater



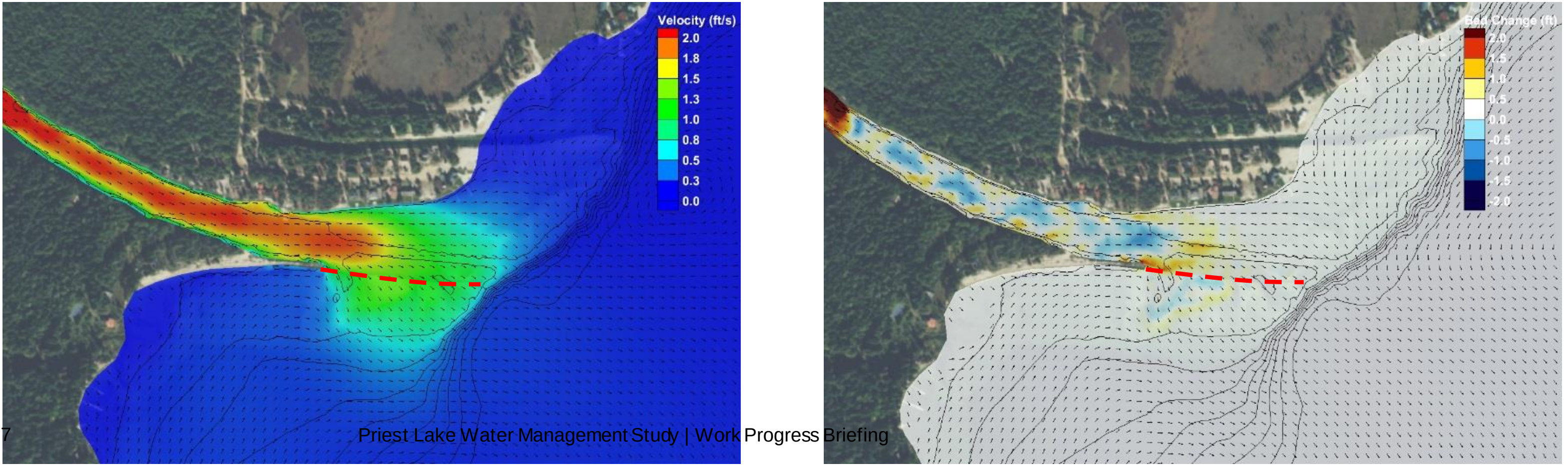
Solid Feature

MODELING RESULTS – Existing Pervious Breakwater

Pervious BW

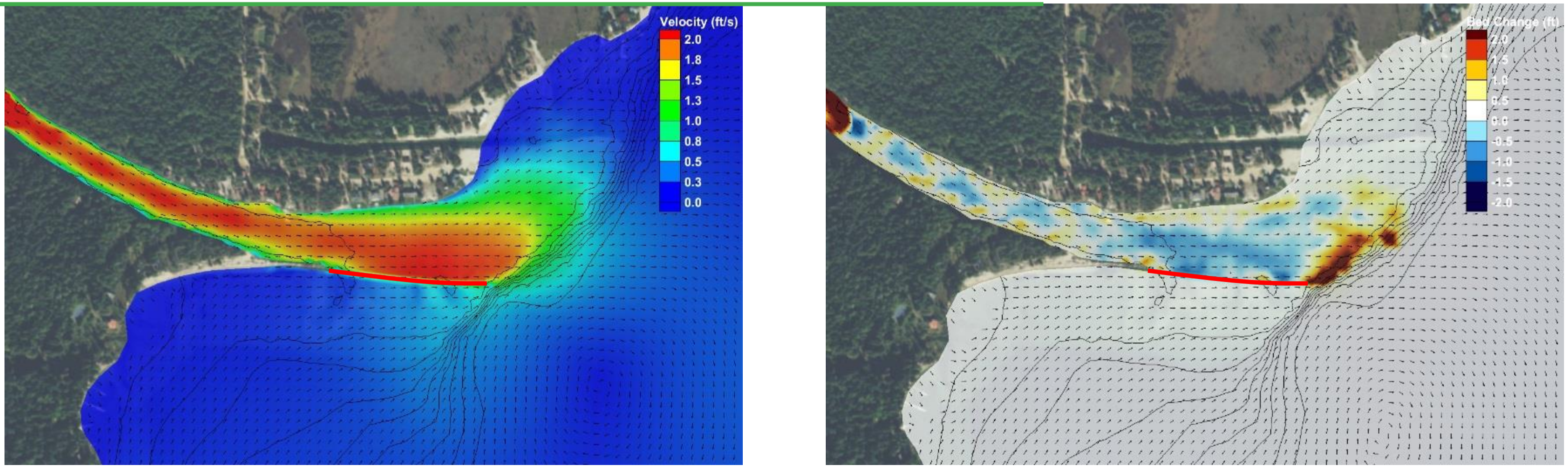


Pervious BW **with Dredge**

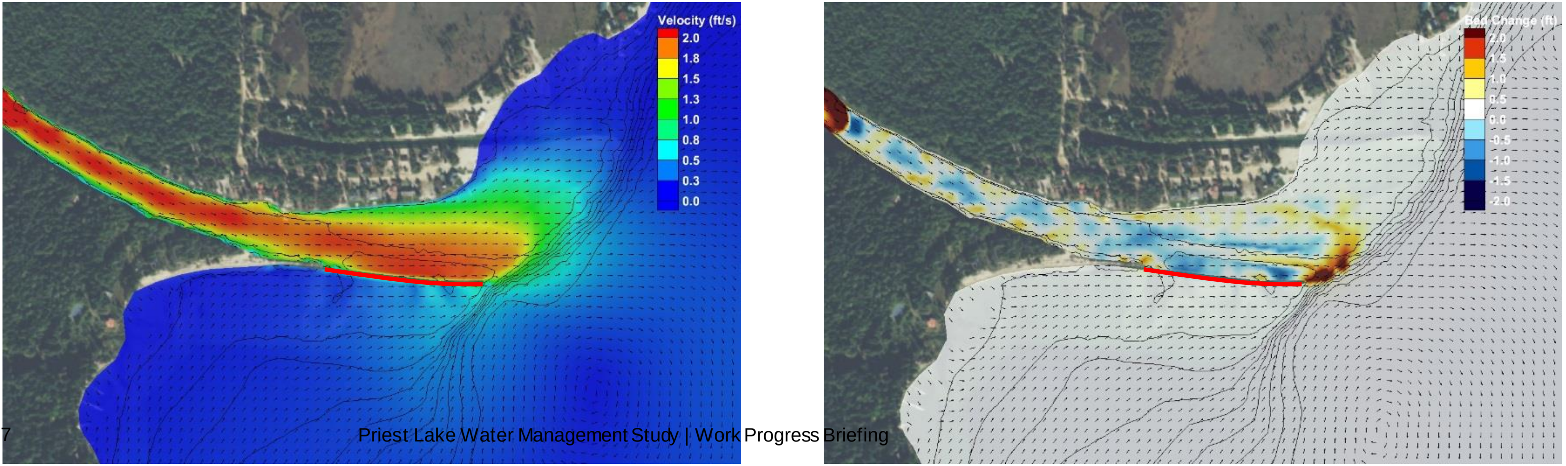


MODELING RESULTS – New Solid Breakwater

Solid Breakwater

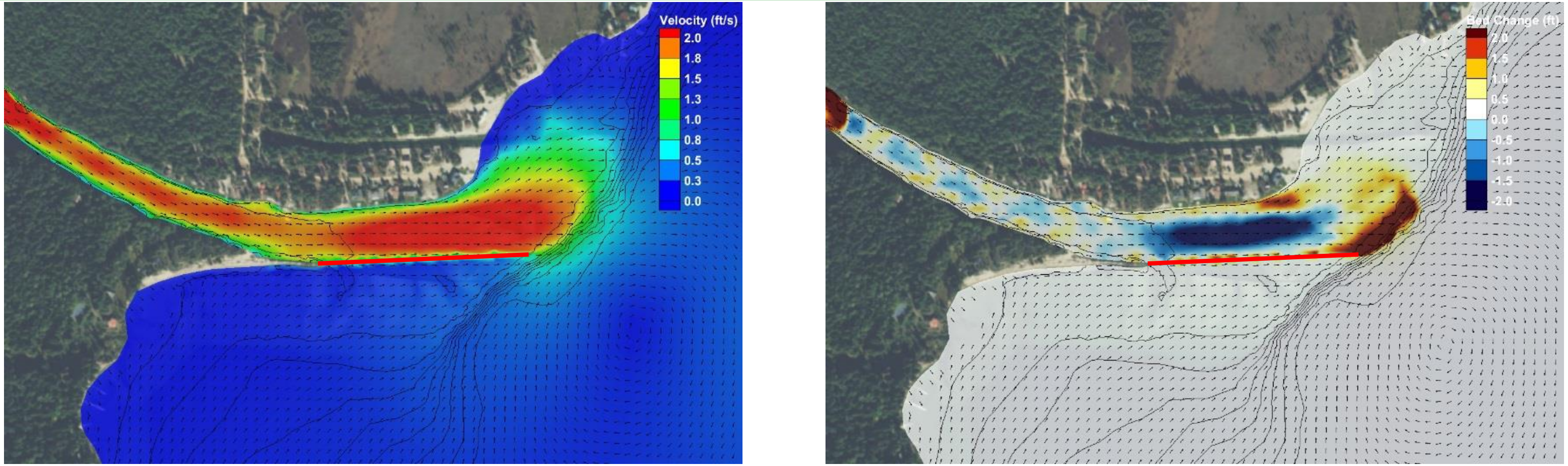


Solid Breakwater **with Dredge**

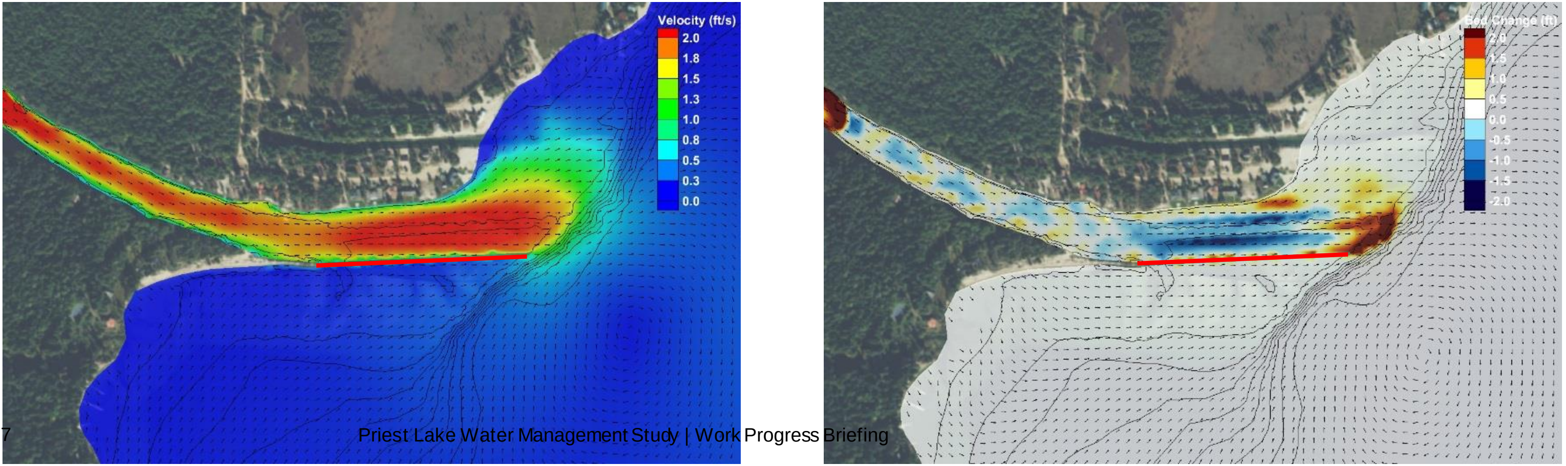


MODELING RESULTS – Solid Breakwater 15 deg rotation

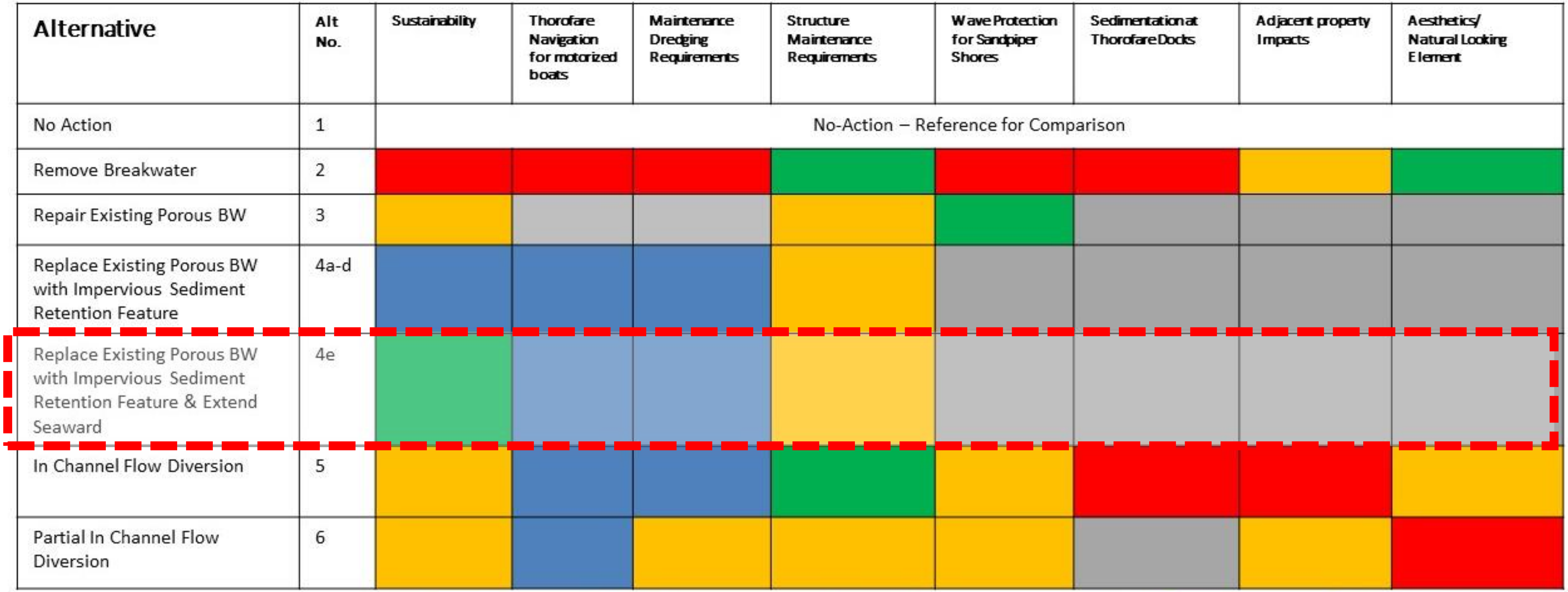
Solid Breakwater Rotated 15 Degrees CCW



Solid Breakwater Rotated 15 Degrees CCW **with Dredge**



THOROFARE/BREAKWATER IMPROVEMENT – ALTERNATIVE SCREENING



Alternative	Alt No.	Sustainability	Thorofare Navigation for motorized boats	Maintenance Dredging Requirements	Structure Maintenance Requirements	Wave Protection for Sandpiper Shores	Sedimentation at Thorofare Docks	Adjacent property Impacts	Aesthetics/ Natural Looking Element
No Action	1	No-Action – Reference for Comparison							
Remove Breakwater	2	🟥	🟥	🟥	🟩	🟥	🟥	🟨	🟩
Repair Existing Porous BW	3	🟨	⬜	⬜	🟨	🟩	⬜	⬜	⬜
Replace Existing Porous BW with Impervious Sediment Retention Feature	4a-d	🟦	🟦	🟦	🟨	⬜	⬜	⬜	⬜
Replace Existing Porous BW with Impervious Sediment Retention Feature & Extend Seaward	4e	🟩	🟦	🟦	🟨	⬜	⬜	⬜	⬜
In Channel Flow Diversion	5	🟨	🟦	🟦	🟩	🟨	🟥	🟥	🟨
Partial In Channel Flow Diversion	6	🟨	🟦	🟨	🟨	🟨	⬜	🟨	🟥

Change with respect to Status Quo (No Action):

🟩 Significantly Positive

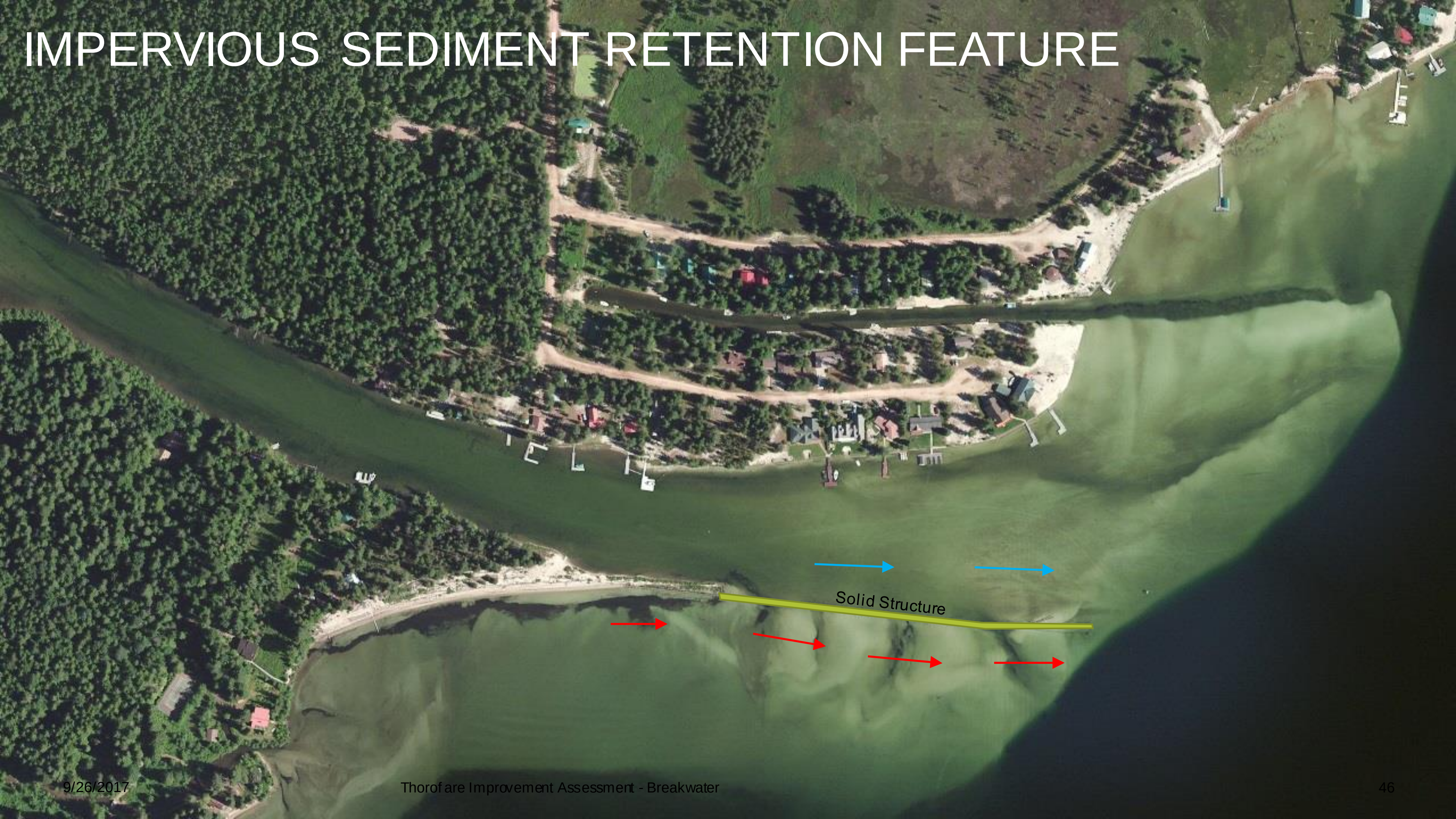
🟦 Moderately Positive

⬜ No/Negligible Change

🟨 Moderately Negative

🟥 Significantly Negative

IMPERVIOUS SEDIMENT RETENTION FEATURE



Extension

Distance beyond existing breakwater = 100 to 150 ft

Rotation

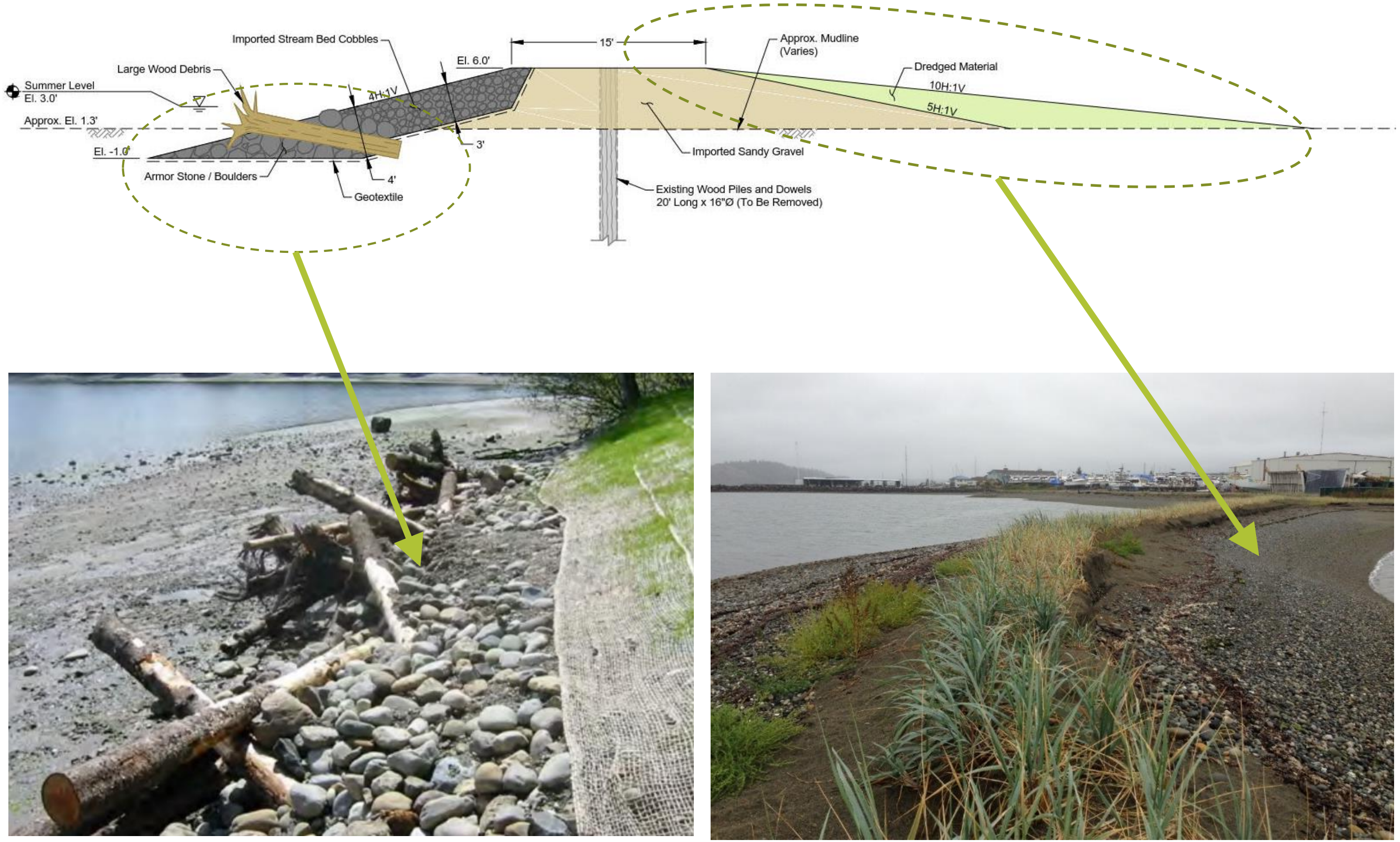
Existing, 15 degree, 30 degree

Preferred Concept

w/ dredged material

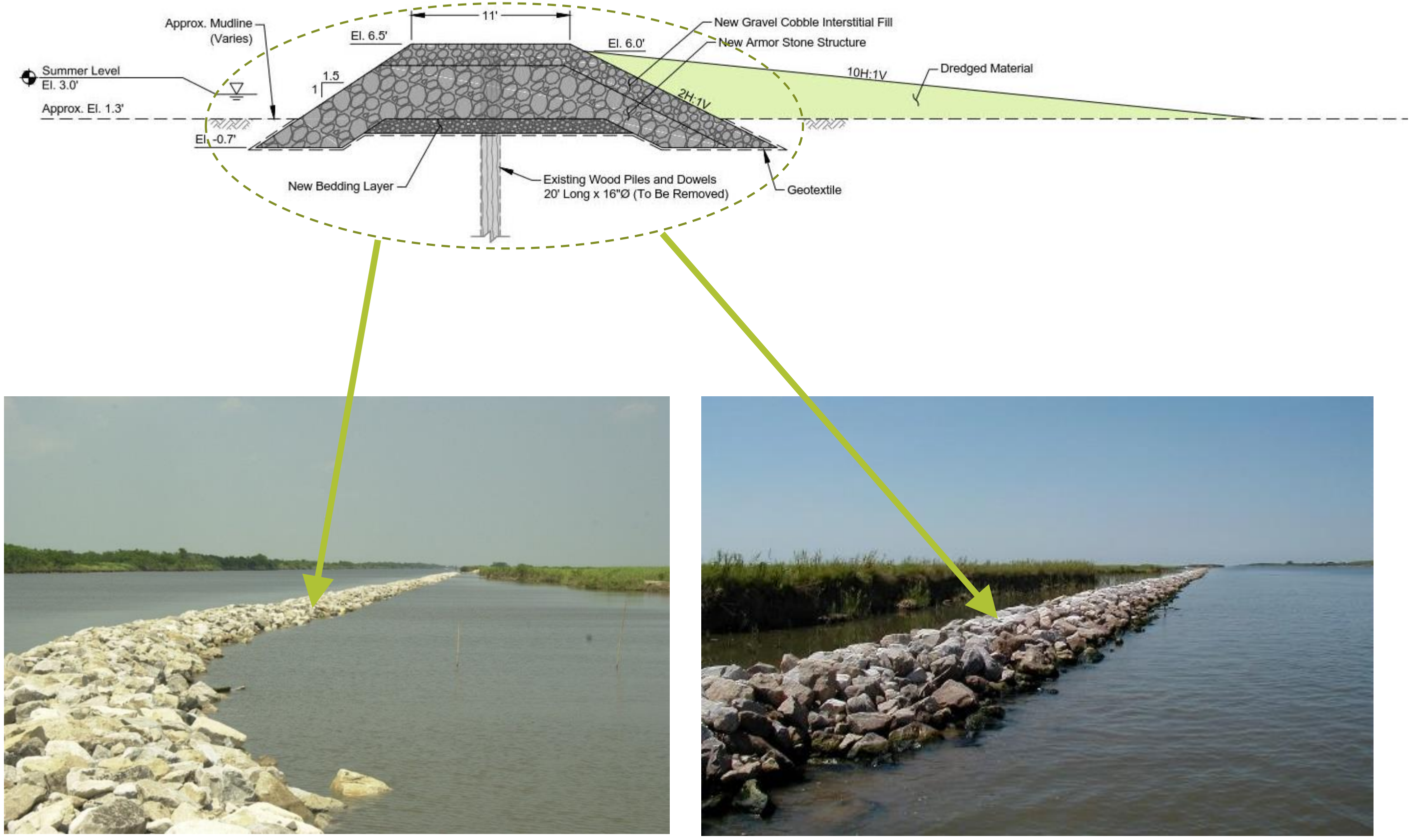
Sediment Retention Feature – Bio-Engineered

Alternative A



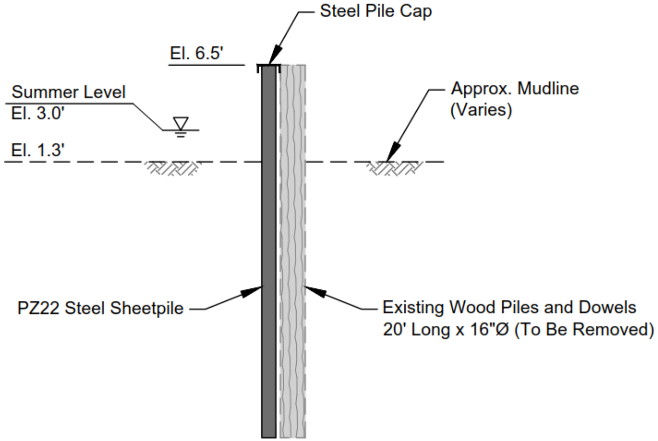
Sediment Retention Feature – Armor Stone

Alternative B

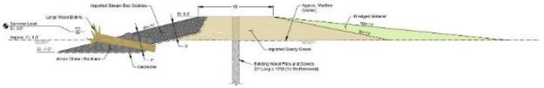


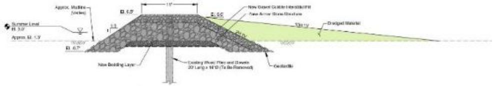
Sediment Retention Feature – Sheet Pile

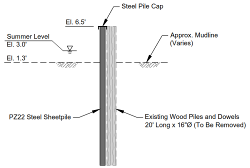
Alternative C



Construction Material Type Comparison







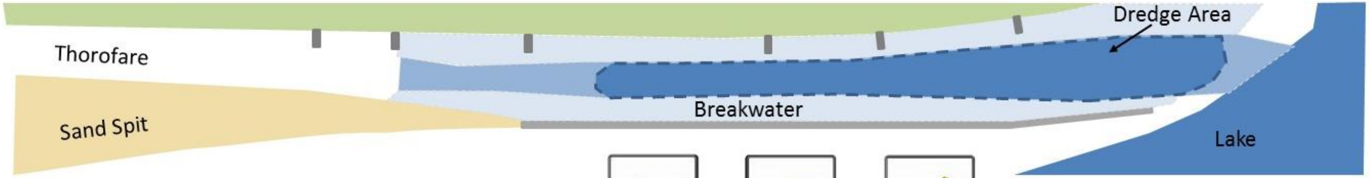
Alt Type	Constructability	Compatibility w/ Site Conditions	Cost/FT	Maintenance
A Bio-Engineered	Poor to Good (near entrance) Good elsewhere	High	$1,300 to $1,600	Medium/Low
B Stone	Good to Excellent	High-Medium	$1,100 to $1,400	Low
C Sheet Pile	Good to Excellent	Medium	$1,200 to $1,500	Low

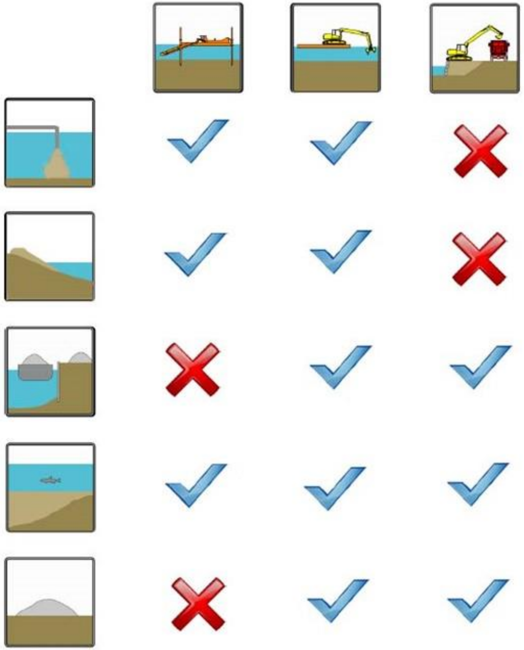
Constructability

- Equipment access will depend upon landside access is availability directly to work area.
- If no landside access, then flexifloat access from nearby boat ramp will be required. Flexifloat would be used for both equipment and materials transfer to the Thorofare work area

NAVIGATION DREDGING

Dredging & Disposal Concept Screening



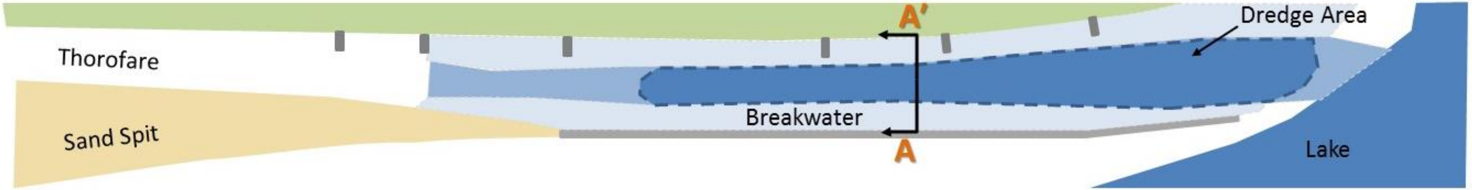


DREDGING

PLACEMENT/
DISPOSAL

Open Water Placement	✓	✓	✗
Nearshore Beach Nourishment	✓	✓	✗
Barge to Upland Reuse	✗	✓	✓
Shallow Water Habitat	✓	✓	✓
Offsite Upland	✗	✓	✓

Dredging Extents and Depths

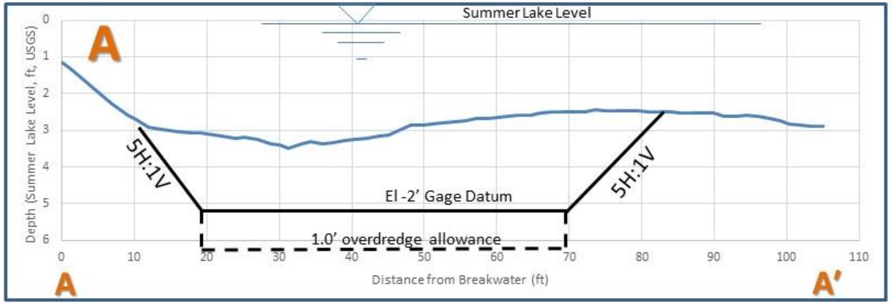


Dredging Width

- 50’

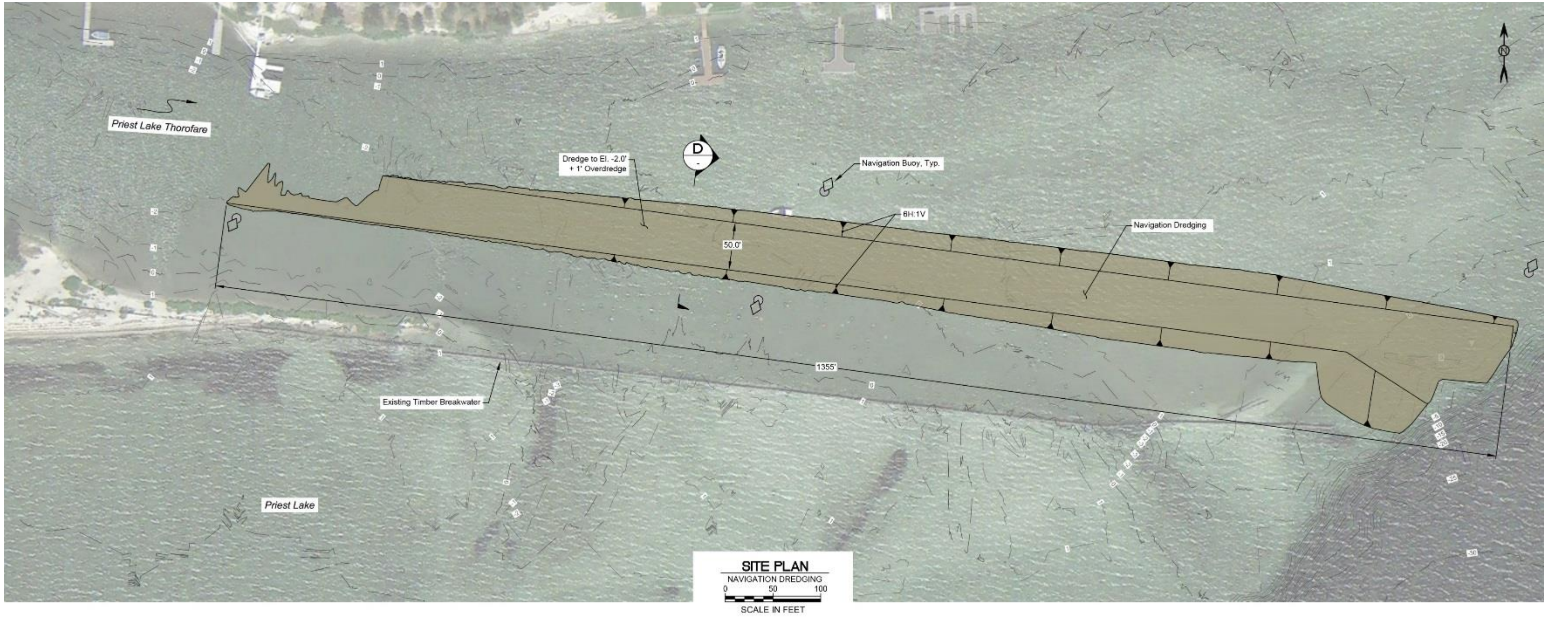
Estimated Dredging Volumes

- 4.0’ Depth (El -1’ Gage Datum)
  - 5,000 to 7,500 cy
- 5.0’ Depth (El -2’ Gage Datum)
  - 9,000 to 11,000 cy



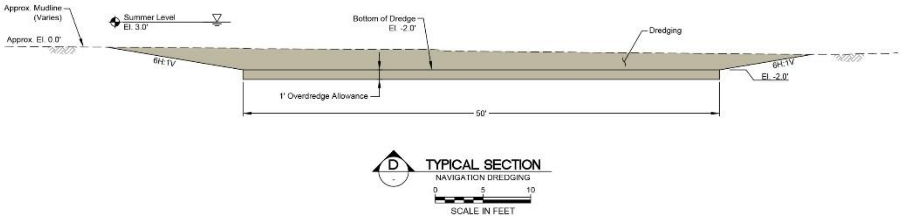
Note: Dredging width to be finalized; 50’ is estimate for assessment phase only. Likely range is 40 to 75’ width.

NAVIGATION DREDGING



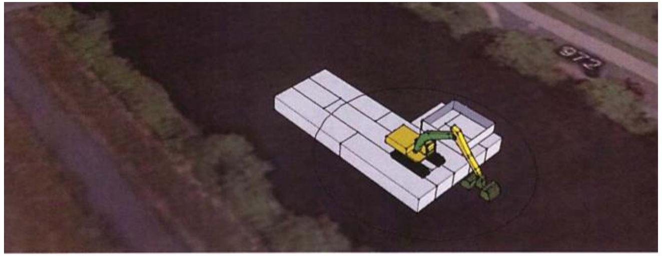
NOTES

1. Bathymetry Source: 2017 Mott MacDonald Survey
2. Horizontal Datum: NAD83, West, Feet
3. Unit and Vertical Datum: Feet Referenced to USGS Gage 12393000 Datum. Datum of Gage 2,434.64 Feet above NGVD29
4. Aerial Source: Google Earth
5. Slopes, width, and location of dredging to be optimized during next phases of engineering.



CONCEPTUAL

Floating Clamshell Dredging



- Requires 4’ to 5’ water depth for loaded barge.
- Limited to summer season construction due to depth requirements.
- Ease of access from nearby boat launch.
- Potential lower production rate dredging and disposal operation depending on disposal site location.
- Required offload & Re-handling facility nearby for upland disposal.
- Aquatic placement directly offloaded.

Hydraulic Cutter Head Dredging

Mobilization – Portable Hydraulic Dredge

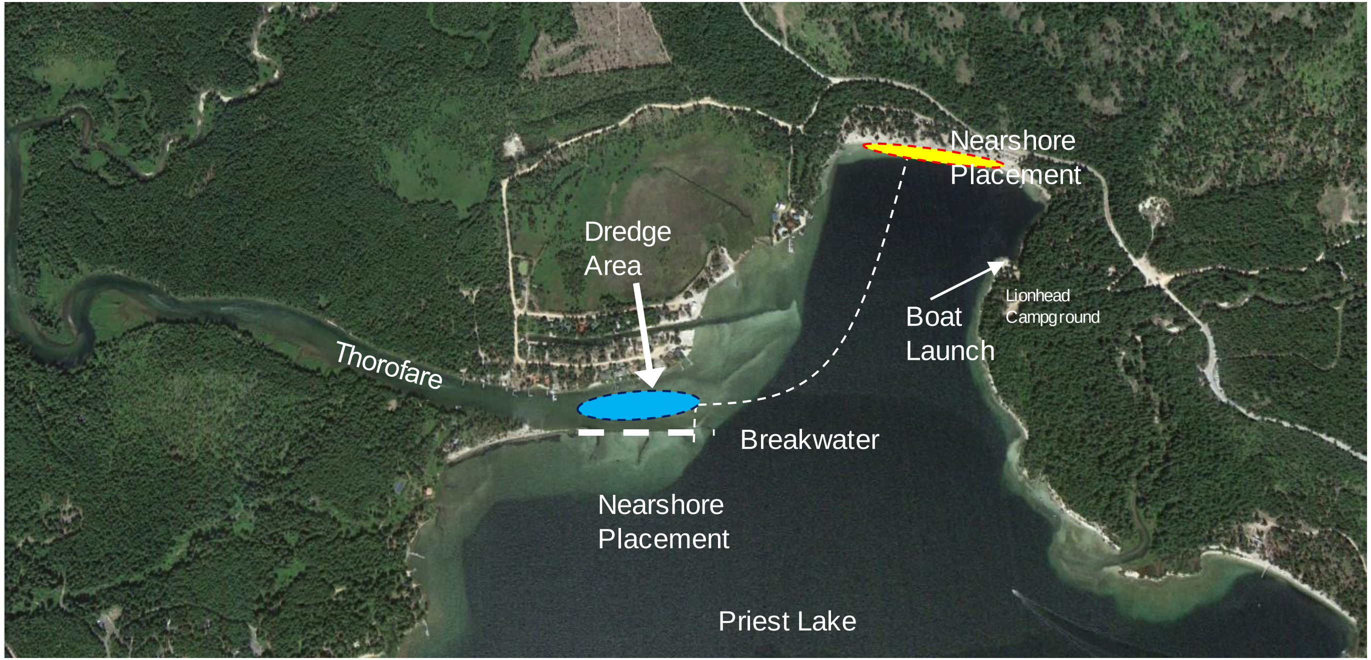
- Minimal site impacts; HDPE pipeline along lake shoreline and floating to upland nearshore or open water placement.
- Feasible for fall construction season.
- Limited by identification of nearby placement site.
- Ease of mobilization to the site from nearby boat launch for placement with crane.

Operations – Portable Hydraulic Dredge

Temporary Berm Dredging

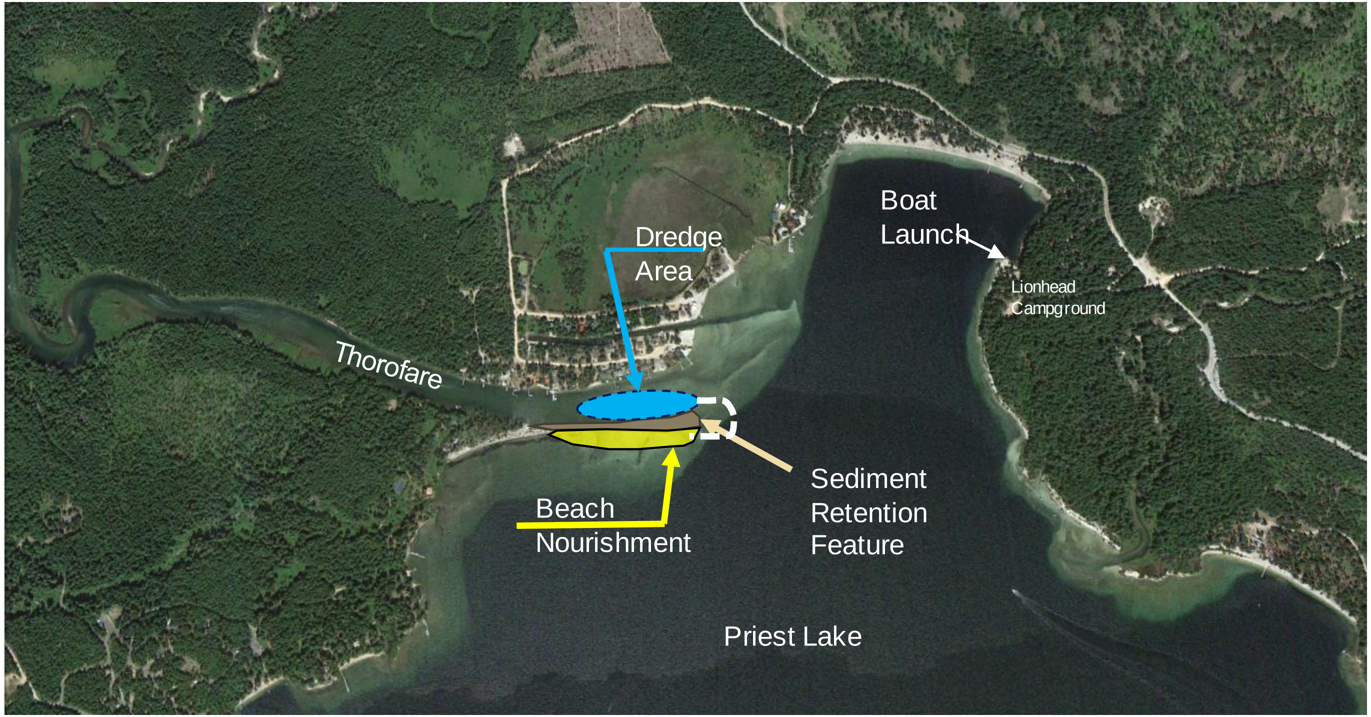
- Feasible for low water period (late fall/winter season) if direct upland access were available.
- Temporary berm required; can be used for breakwater repair/replacement (dual purpose).
- Requires land side access; likely not available.

Nearshore Placement



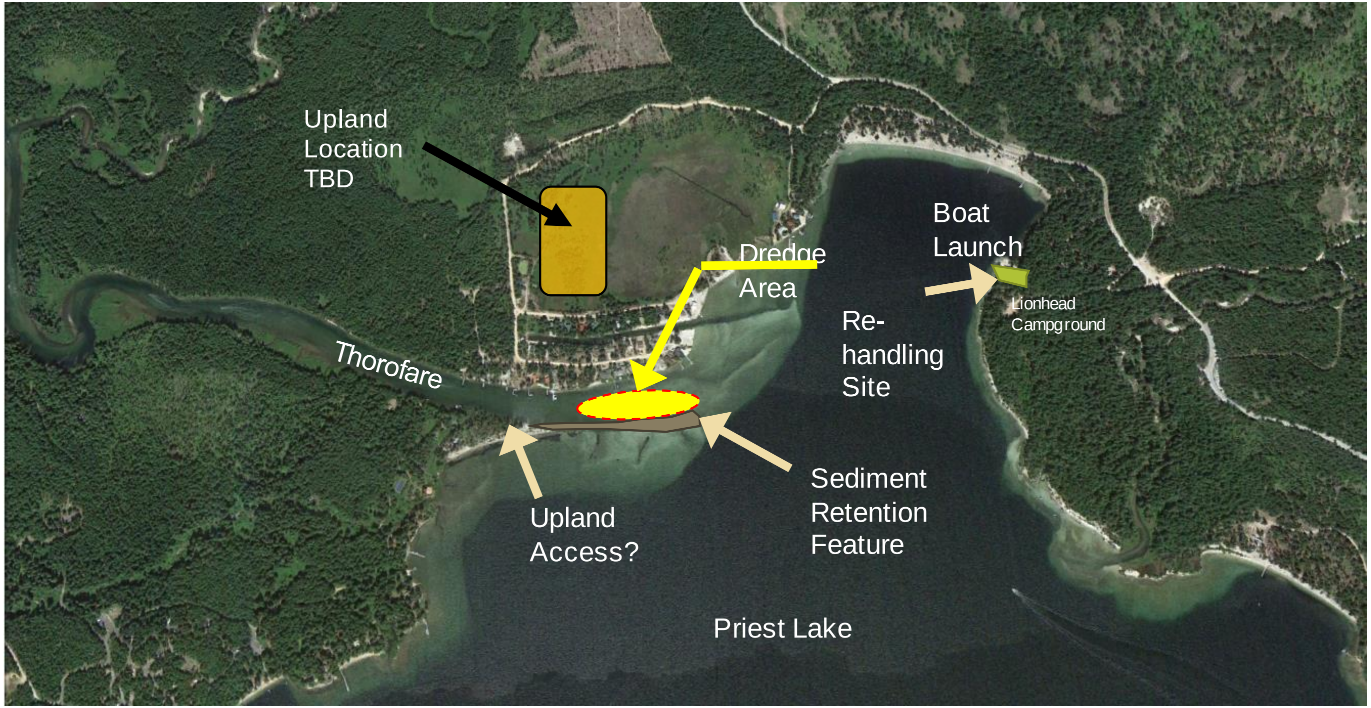
-Either Hydraulic or barge based. Hydraulic more economical
-Determine capacity of available and feasible sites
-Shallow water placement for nearshore beneficial reuse

Beach Nourishment



-Either Hydraulic or barge based. Hydraulic more economical
-Determine capacity of available and feasible sites
-Shallow water placement for nearshore beneficial reuse

Upland Disposal



-Either Hydraulic or barge based. Very limited sites for hydraulic (nearby Lake only). Barge more feasible but requires re-handling area such as boat ramp. Material re-handled into dump trucks for disposal offsite.

Construction Access

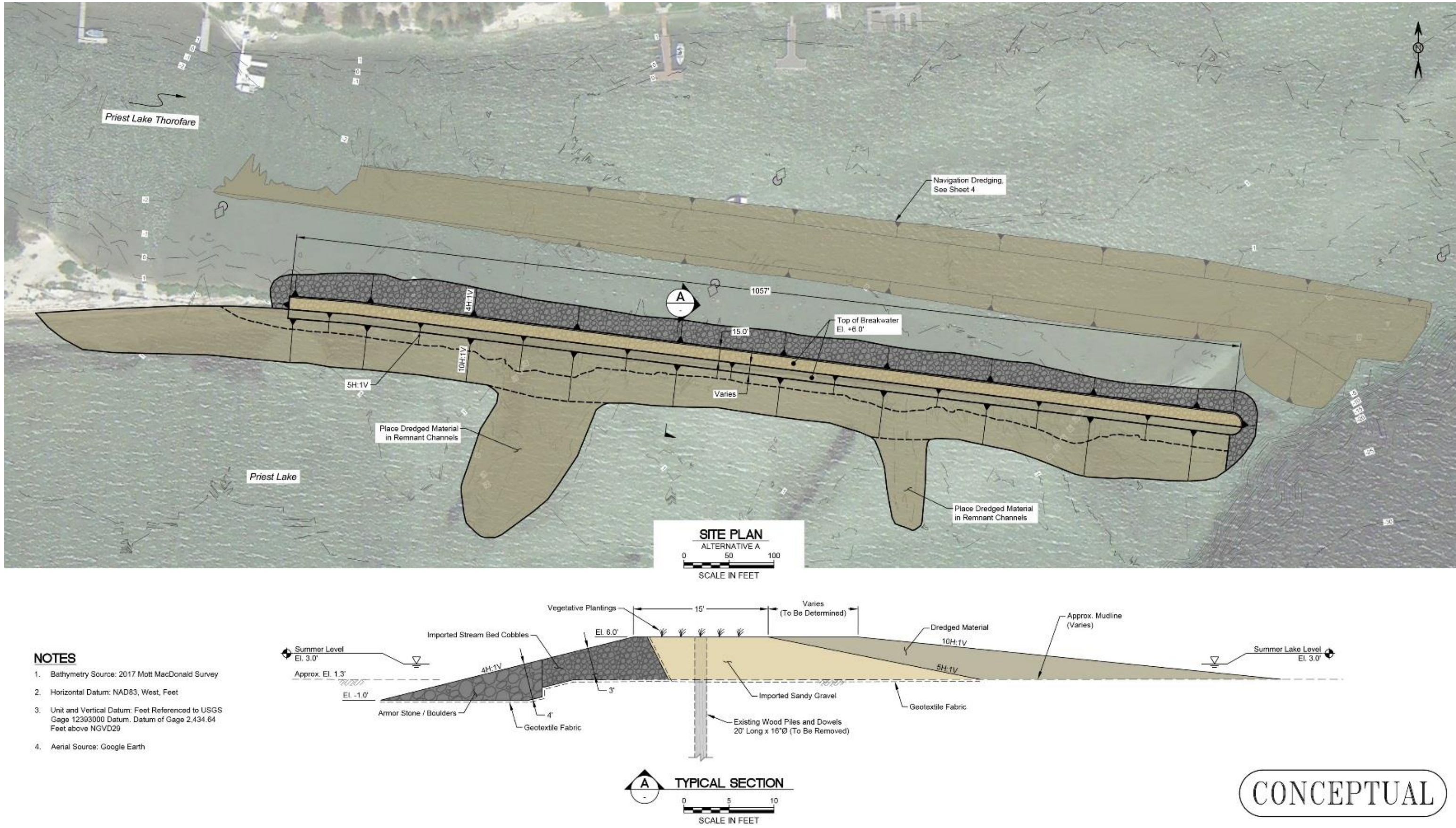
- Access to the site will be critical for ease of construction and to reduce construction costs.
- Boat Ramp such as Lionhead campground for launching of marine equipment and staging will be needed.
- Upland access from adjacent private parcel could aid in reducing construction costs for both dredging and the sediment retention feature construction.

Boat Ramp – Lionhead Campground

Adjacent Upland Parcel

THOROFARE IMPROVEMENTS SUMMARY

Alternative A – Gravel/Cobble Berm



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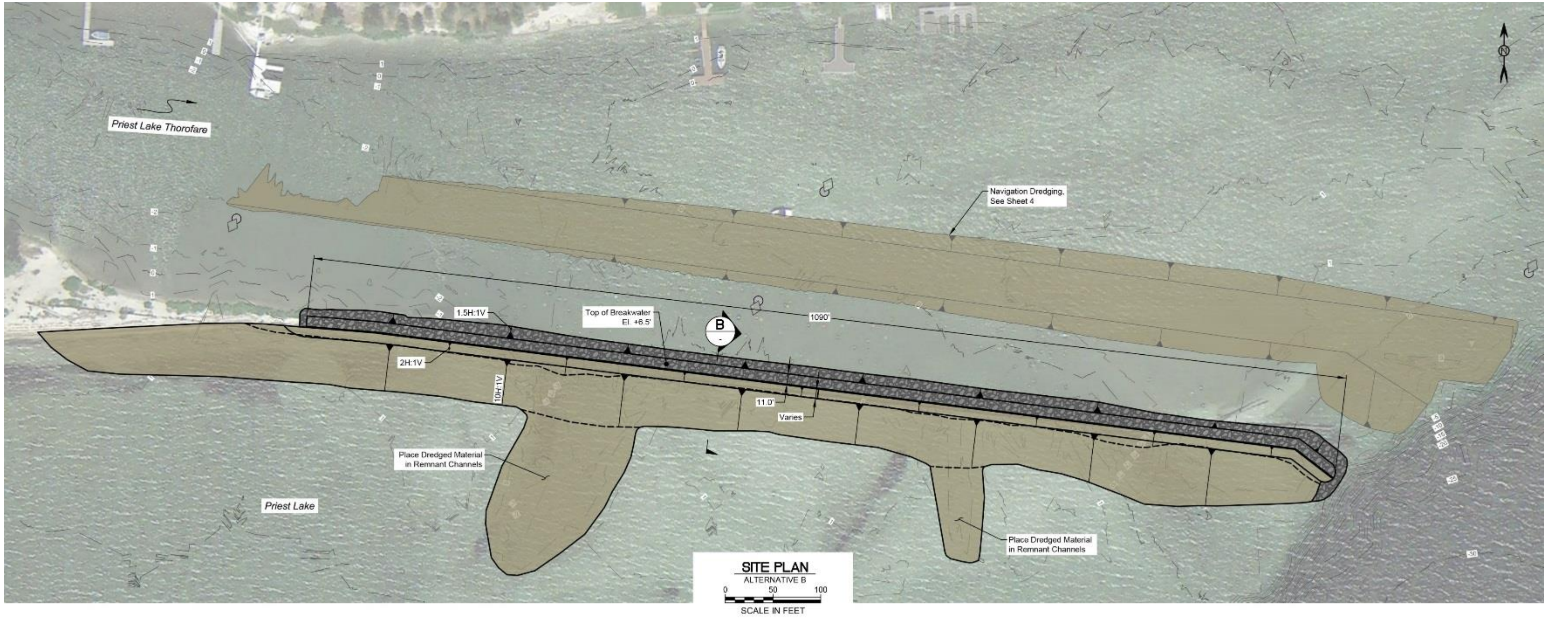
4. Aerial Source: Google Earth

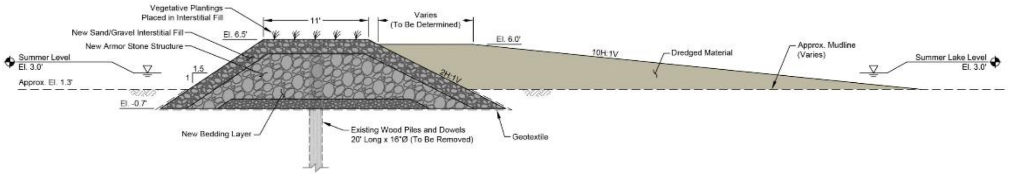
Alternative B - Rubblemound



SITE PLAN
ALTERNATIVE B

NOTES

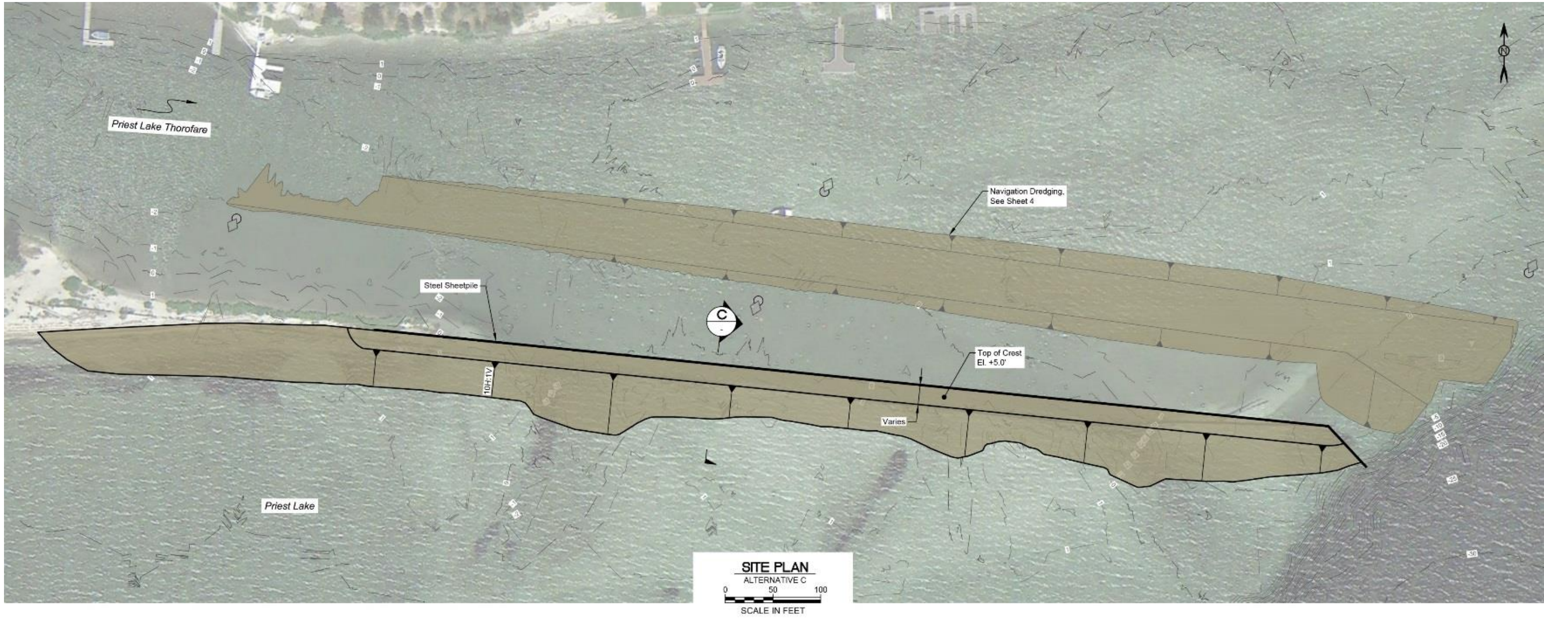
1. Bathymetry Source: 2017 Mott MacDonald Survey
2. Horizontal Datum: NAD83, West, Feet
3. Unit and Vertical Datum: Feet Referenced to USGS Gage 12393000 Datum. Datum of Gage 2,434.64 Feet above NGVD29
4. Aerial Source: Google Earth



B TYPICAL SECTION

CONCEPTUAL

Alternative C – Sheet Pile



SITE PLAN
ALTERNATIVE C

NOTES

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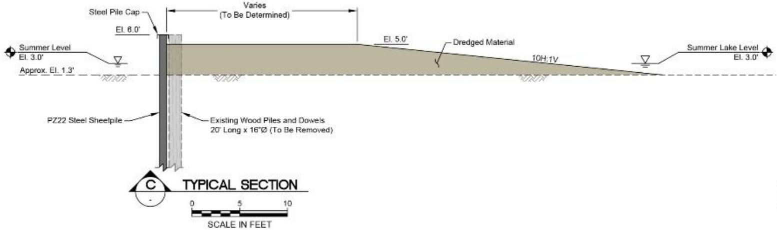


TYPICAL SECTION

CONCEPTUAL

Regulatory Considerations

- Water Quality (401 Certification). Both dredging and aquatic placement areas will need to meet turbidity and water quality protection requirements. BMP's to be developed to ensure compliance with WQ standards.
- Sediment Evaluation Framework (SEF). Sampling & testing of proposed dredged sediments to be conducted during regulatory process. Ensure material meets requirements for in-water placement and protection of water quality. U.S. Army Corps of Engineers is the lead regulatory agency on this process.
- Aquatic Placement. Preliminary consultation with regulatory agencies indicates nearshore nourishment may be acceptable but deep water disposal likely not. Needs to relate to purpose and need for in-water placement.
- Upland Placement. Find location of nearby, upland land for receipt of material. No wetlands.

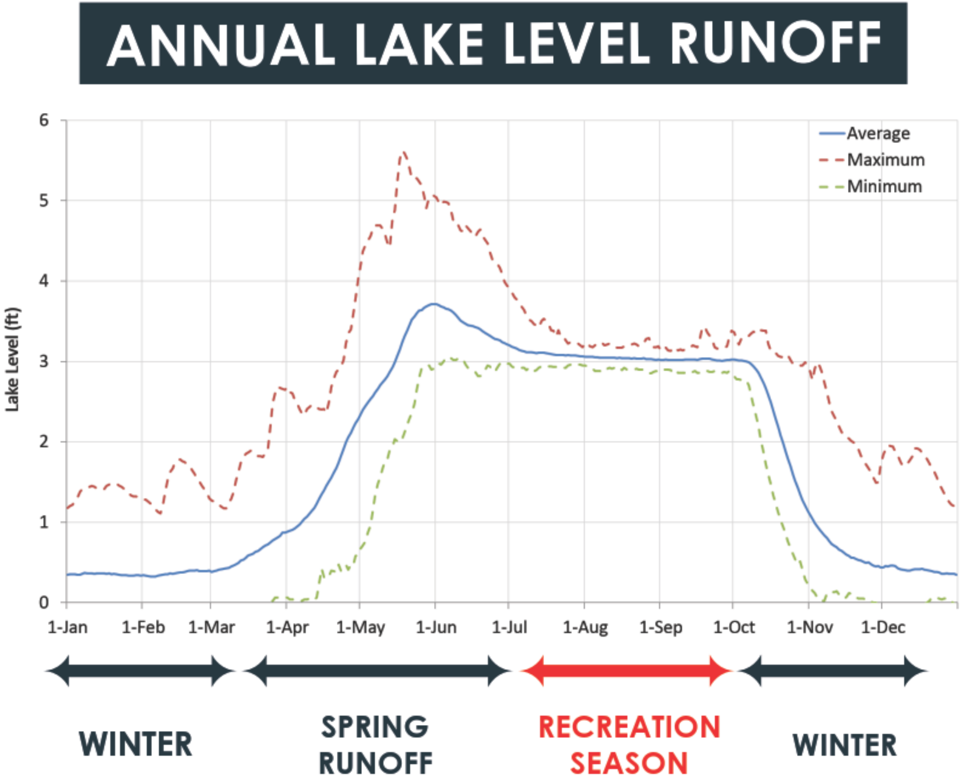
POOL RAISE ASSESSMENT

Water Level Management – Background & Purpose

- In 2015, The discharge from the dam was reduced below the current policy of minimum 60 cfs to maintain the lake level and meet statutory requirements.
- In 2016, which had seemed to be a typical year, the same minimum discharge concerns occurred.

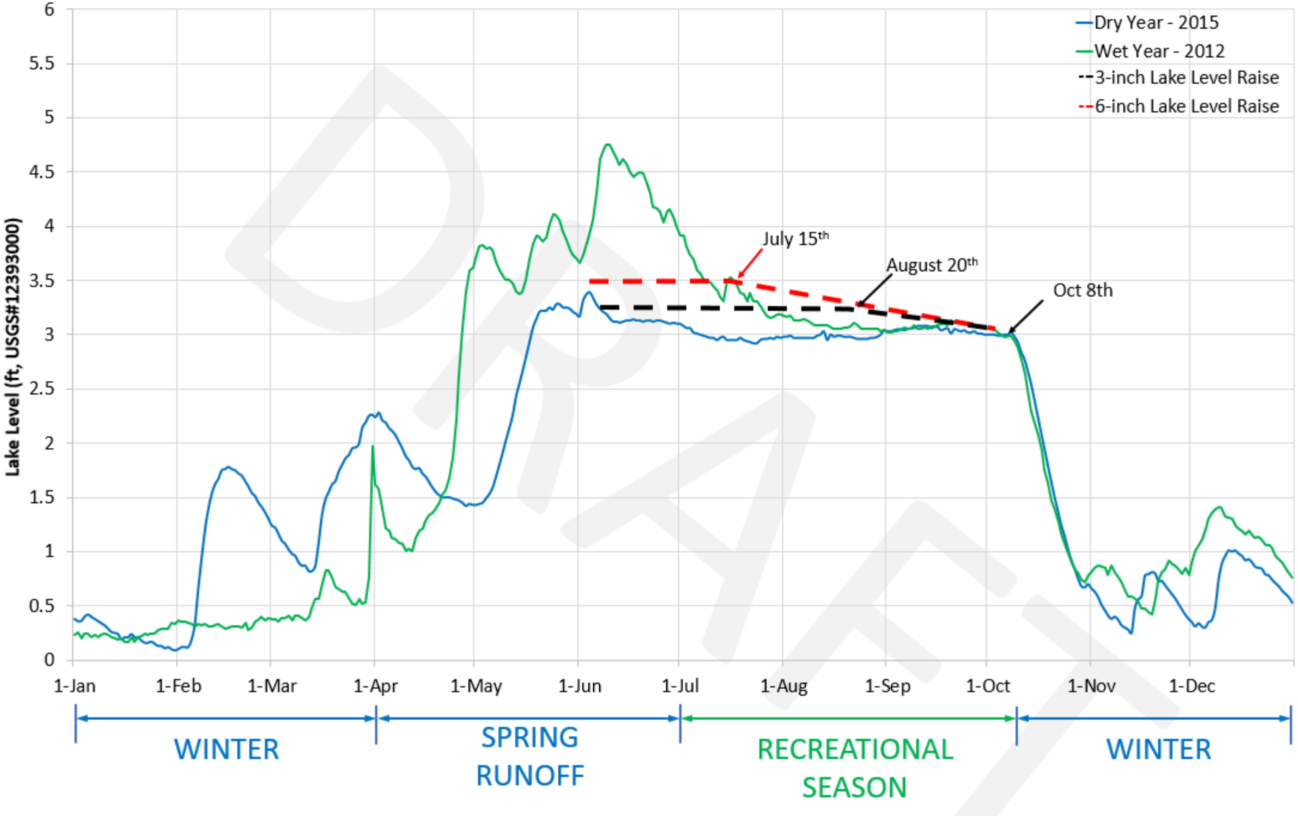
Study Purpose:

- Evaluate possible changes for a dry year water management scheme consisting of either a 3-inch or 6-inch higher lake level during part of the summer recreational season



Priest Lake Water Level - Background

- >3.0’ during the recreational season; not uncommon.
- Summer 2012, lake level was 6-in higher than 3.0’ required level till July 15th and 3” higher until July 20th.
- Temporary pool raise during dry years can be thought of as managing lake level similar to natural lake level in wet years with a slight increase during month of August



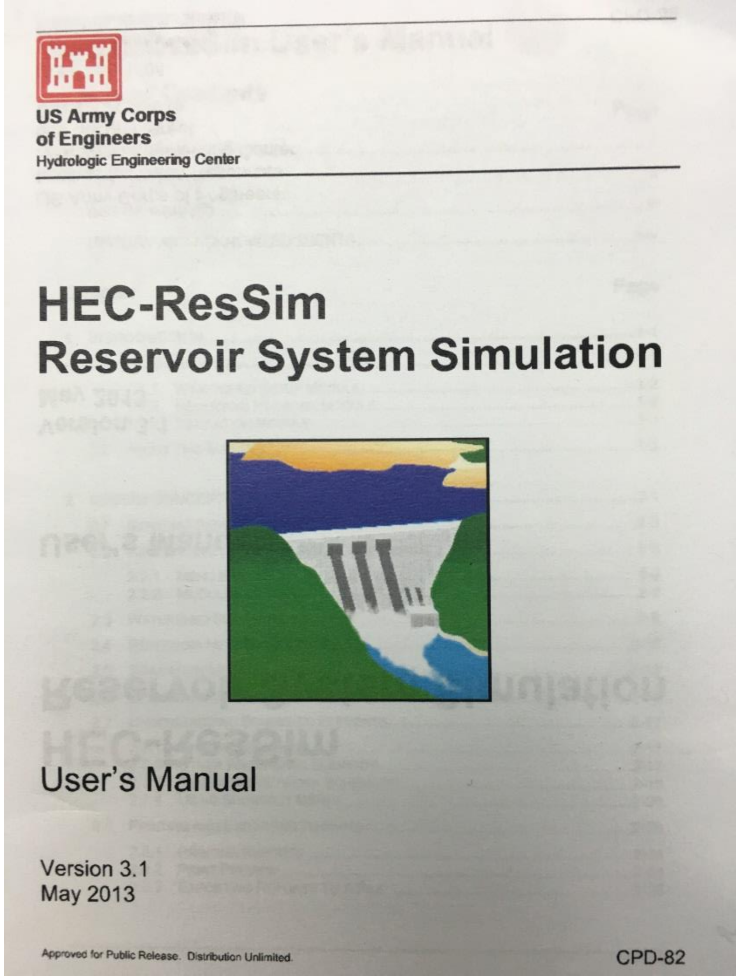
LAKE MANAGEMENT OPTIONS

- Water Management Analysis
- Evaluation of Outlet Dam operations
- Input Data
- Simulations
- Preliminary Conclusions

- 3” Pool Raise likely will work relative to historical dry years to meet the defined criteria.

- Integration of real time streamflow data into dam operations

- Provide larger tolerance in operations to allow more flexibility (currently operated to maintain as close to 3.0’ as possible). Allow variation of 3 to 4”.



US Army Corps
of Engineers
Hydrologic Engineering Center

HEC-ResSim
Reservoir System Simulation

User’s Manual

Version 3.1
May 2013

Approved for Public Release. Distribution Unlimited.

CPD-82

Dry Year Pool Raise Assessment

Purpose: Evaluate changes from existing conditions & potential for impacts on the following elements due to pool raise (3” or 6”).

SHORELINE FEATURES

Recreational Beaches

- Pier Access
- Benches
- Beaches
- Stairs
- Fire Pit

Boat Covers

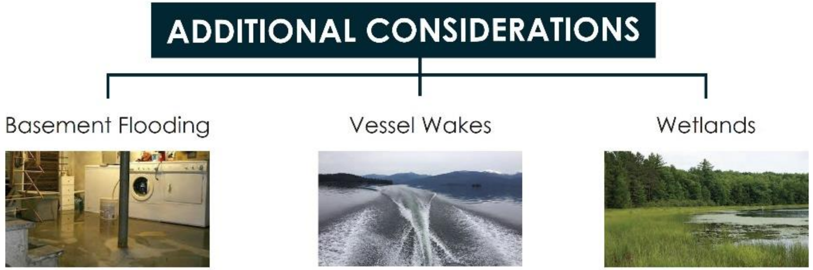
- Fixed Height Structures
- Height of Vessel Adjustments

Natural Shoreline

- Natural Shoreline Features
- Rocks, Vegetation
- Large % of Overall Lake Shoreline

Public/Private Boarding Access

- Beach
- Boat Ramps



ADDITIONAL CONSIDERATIONS

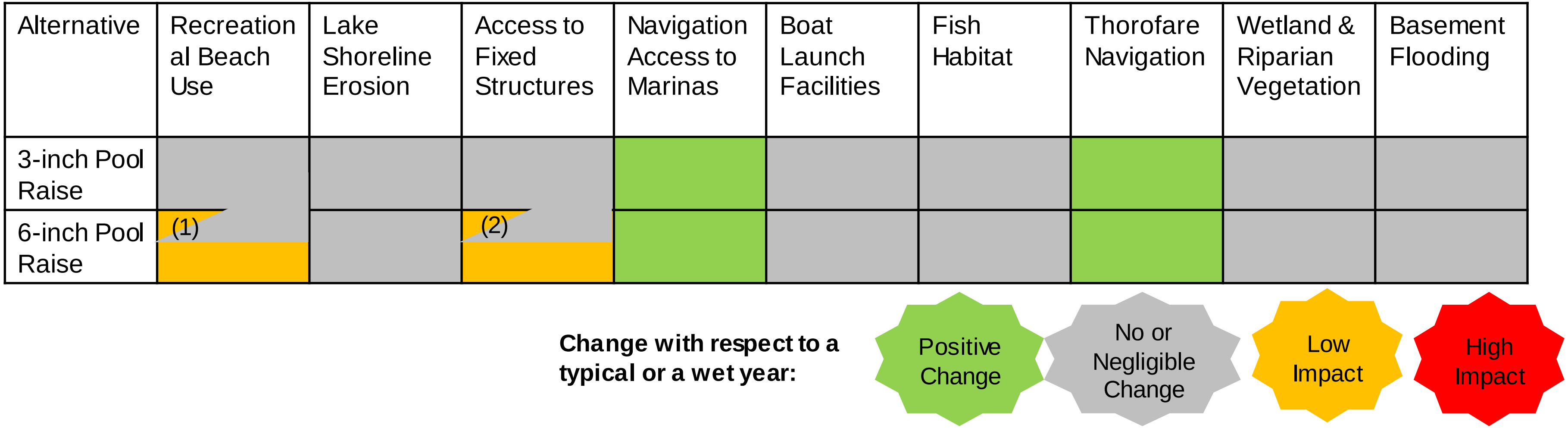
Basement Flooding

Vessel Wakes

Wetlands

Dry Year Pool Raise Assessment - Summary

Temporary pool raise is being considered as an improvement measure **only** for dry and marginally dry years. Therefore, any possible impact will be limited to these years.



Alternative	Recreational Beach Use	Lake Shoreline Erosion	Access to Fixed Structures	Navigation Access to Marinas	Boat Launch Facilities	Fish Habitat	Thorofare Navigation	Wetland & Riparian Vegetation	Basement Flooding
3-inch Pool Raise	⬜	⬜	⬜	🟩	⬜	⬜	🟩	⬜	⬜
6-inch Pool Raise	🟧 ⬜ (1)	⬜	🟧 ⬜ (2)	🟩	⬜	⬜	🟩	⬜	⬜

Change with respect to a typical or a wet year:

- 🟩 Positive Change
- ⬜ No or Negligible Change
- 🟧 Low Impact
- 🟥 High Impact

Footnotes:
(1): There will be no impact on majority of the beaches. Localized areas will see loss of usable dry beach.
(2): There will be no impact on majority of fixed structures. A low percentage of structures will see low impacts.

Assessment Purpose & Criteria

1 Evaluate Pool Raise effect on Structure

2 Determine repairs or upgrades needed to accommodate a pool raise

1 IDWR & USBR Standards

2 Basis of Analysis Memorandum dated June 16, 2017 (Section 19.Outlet Dam)

OUTLET STRUCTURE EXISTING (May 2017 High Flow)

OUTLET STRUCTURE EXISTING (Sept 2017 Low Flow)

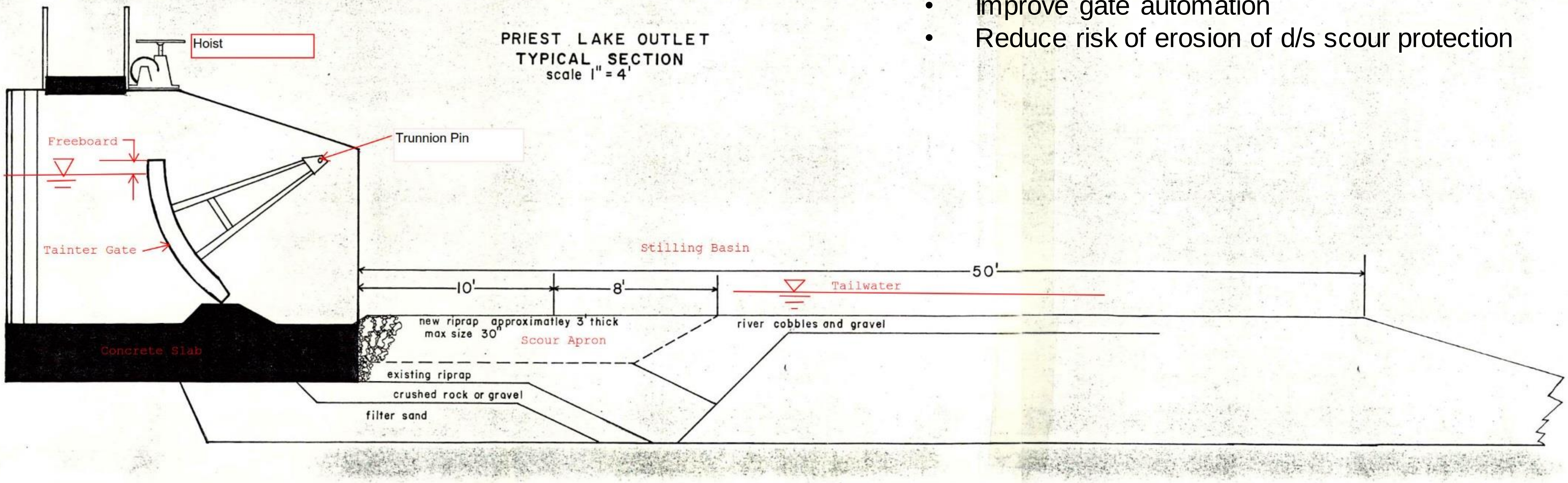
OUTLET STRUCTURE ASSESSMENT

Background

- Constructed in 1978
- Radial gates manually operated on 11 equally sized spillway bays
- Repairs to downstream scour protection in 1979 ~larger stone installed to increase scour protection
- Gates are 7’ tall with 0.15’ freeboard

Goals/Criteria

- Evaluate alternatives to Raise Pool to:
  - 3.25 ft gage
  - 3.50 ft gage
- Provide larger tolerance on vertical operating range; ~0.15’ in lieu of 0.05’
- Freeboard = min 3” is recommended
- Reduce risk for operator error
- Improve gate automation
- Reduce risk of erosion of d/s scour protection

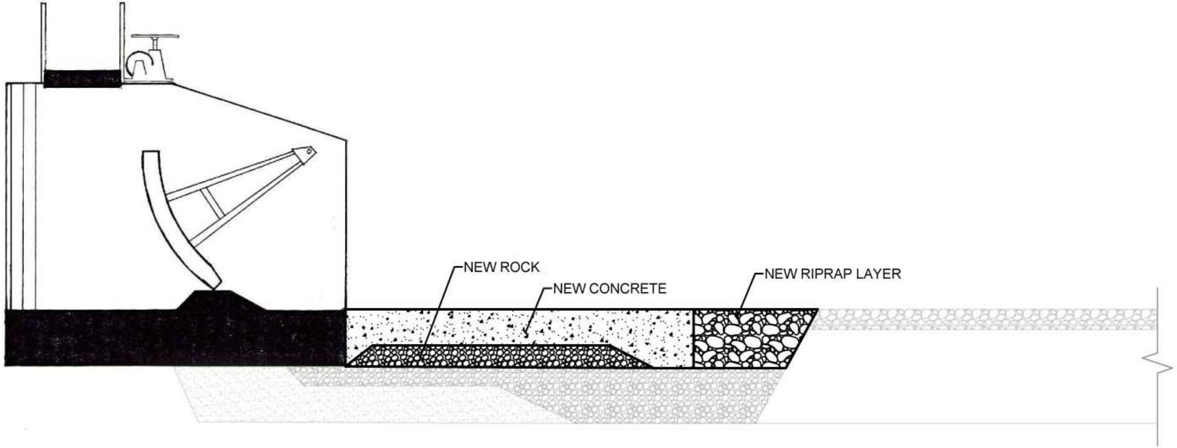


OUTLET STRUCTURE – HYDRAULIC ASSESSMENT

- Mott MacDonald conducted an assessment of hydraulic conditions.
- HYDRAULICS
  - Evaluated Spring, Summer and Fall Flow Conditions
  - Hydraulic jump forms beyond concrete slab and some instances beyond riprap scour protection
  - High velocities a consideration for all flow conditions; in particular is concern for high pool, low flow condition
  - Stream power (erosion potential) is increased by 9% for 6” pool raise discharge condition
  - High pool, low flow gate operation within center spillway bays better than at outer channel margins
- SCOUR ASSESSMENT
  - Stone is undersized for certain discharge flow and gate operation conditions
    - Analysis indicates D50 of 1’ to 2.5’; current D50 estimated to be 1’.
  - Larger D50 and layer thickness for riprap scour protection is needed
    - Larger stone would reduce risk of scour during future operations for spring or summer conditions
  - Length of scour apron is shorter than standards indicate
  - Concrete stilling basin is a more ideal system to mitigate scour hazard and hydraulic jump; especially in light of dependence of current system on human operations.
- Summary
  - Sensitivity to specific hydraulic conditions and gate operations
    - High flow: Variable location for hydraulic jump formation depending on gate operations and discharge.
    - High Pool, high flow, few gates open = High velocities & scour potential.
  - Stone is undersized and susceptible to scour during gate operations
  - Improved scour apron and more formalized concrete stilling basin should be considered

Scour Apron Mitigation Concepts

ALT #2 – EXTEND CONCRETE APRON



Note: Baffles, End Sill and Sheet Pile cutoff wall not shown.

Pros:
- Improved Stilling Basin Operations
- Lower Risk
- Potential for improvement to dam stability

Cons:
- Expense
- Less effective than Alt 1

Operational Modifications Assessment

Operational Strategy

- Summer high pool with larger flows. Operate larger number of gates to reduce erosion risk potential (high velocities).
- When smaller number of gates are required to be operated, utilize gates near center of structure.

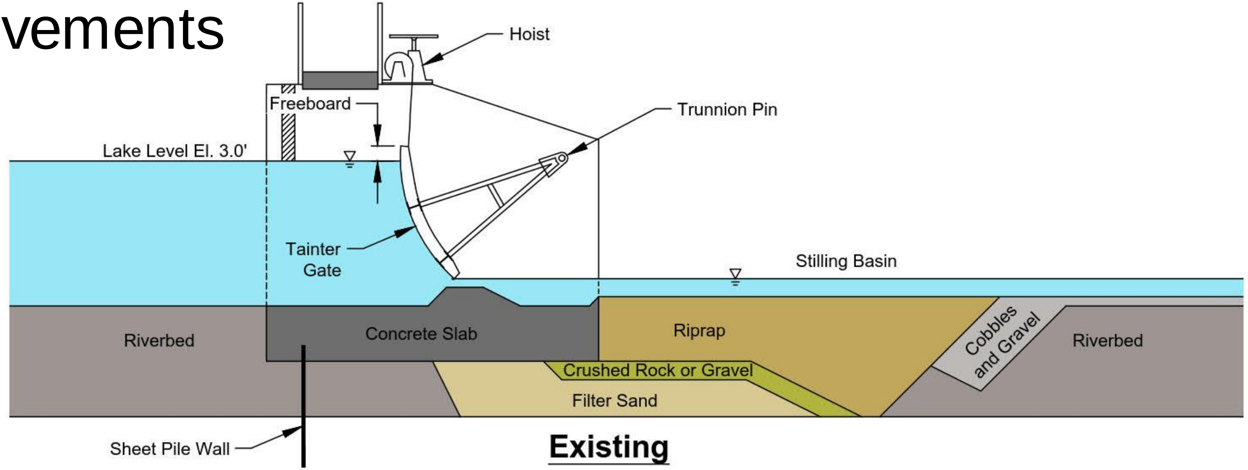
Powered Operation & Automation Considerations

- Concepts
  - Retrofit with motor and gearbox to existing or a modified drive with starter panel
  - Valve Actuator – self contained unit; remote operation
- Assessment
  - Requires a housing structure, communication, power, etc…
  - Large investment required for fully automated.
  - Risks of operations still requiring onsite attention
- Alternative Concept
  - Provide power operation with remote monitoring but not remote operation could be an alternative.
  - Focus improvements on more refined onsite operation to improve real time operations of the dam
  - Improve discharge and pool measurement and monitoring system for gate operations and to improve rating curve
- Summary
  - Power operation could increase operational flexibility but likely not a requirement for pool raise; could be a good improvement to supplement other improvements
  - Automation is not a requirement for the pool raise project; could be expensive to implement as well.

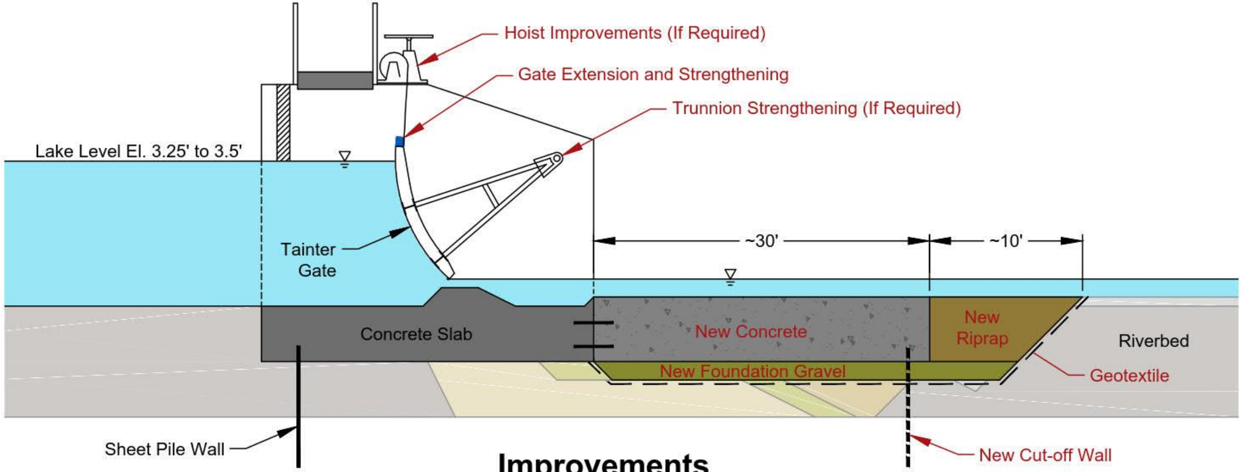
Dam Assessment – Conclusions

- Structure Stability
  - Improvement for stability likely needed to meet current standards to not rely upon sheet pile wall for sliding resistance.
  - Sheet pile wall does provide reduction in seepage.
- Stilling Basin
  - Improvements needed to mitigate risk of scour and corresponding impact on dam stability.
  - Alternatives. Scour apron with concrete slab is recommended; see graphic on subsequent slide.
- Gates
  - Increase height of gate for 3” to 6” pool raise looks feasible.
  - Modification to gate will be needed at top of existing gate (plate extension)
  - Gate Structure and Trunnion: Likely OK, additional analysis in next phase needed to finalize assessment and determination any retrofit needs.
  - Budget line item as contingency will be developed.
- Gate Operations
  - Power operation should be considered; likely not required for pool raise.
  - No automation needed for pool raise.
  - Have greater flexibility in pool level tolerance for gate operations to “capture” summer runoff events and store water while assessment of water needs is made.

Outlet Dam Improvements Summary



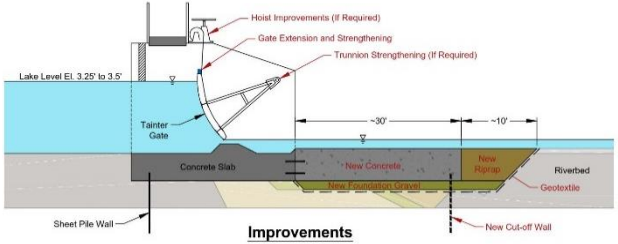
Existing

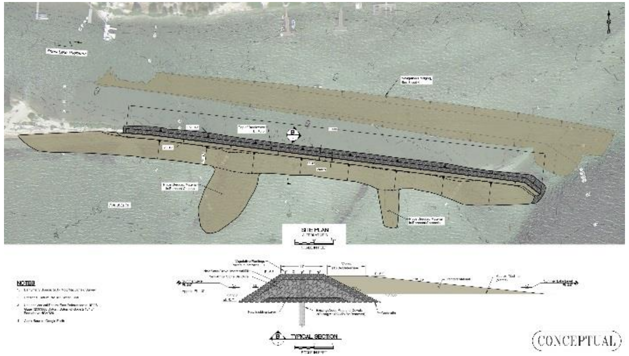


Improvements

Estimated Costs

- Outlet Structure Improvements
  - $2.4 million
- Thorofare Improvements
  - Alt A – $2.6 million
  - Alt B – $2.4 million
  - Alt C – $2.5 million
- Total Cost
  - ~ $5 million
- Notes
  - 2018 dollars





Recommendations

- Outlet Structure Improvements
    - Scour apron improvements
    - Gate Modification
    - Strengthening where needed

- Thorofare Improvements
    - Sediment Retention Feature w/ Dredging
    - Final Alternative Type selected upon collection of additional data, refined analysis, and additional consultation with regulatory agencies.
    - Rubblemound (Alt B) is the longest lasting w/ least maintenance alternative

MOTT MACDONALD

Priest Lake Water Management Study

IWRB Board Briefing Meeting– 10/24/2017

IDAHO WATER RESOURCE BOARD

Working Document

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