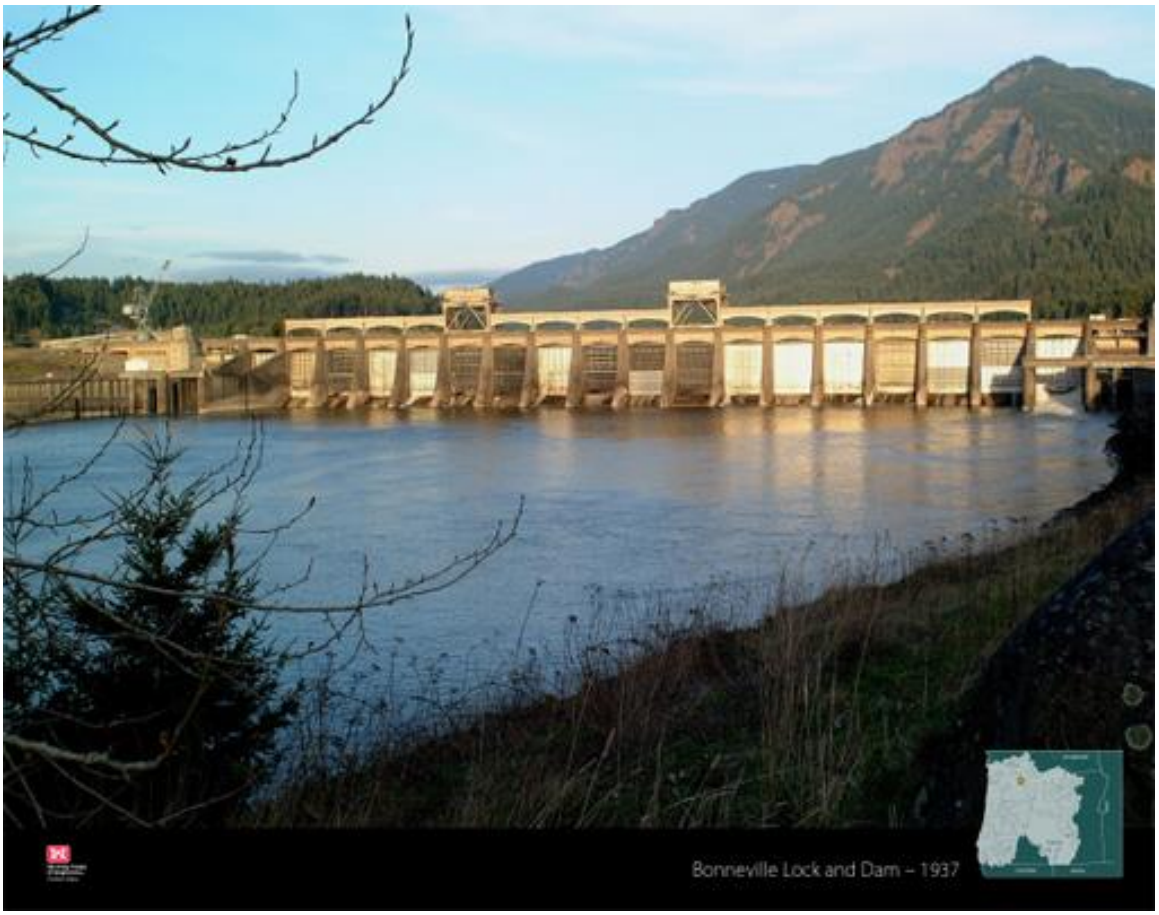
BONNEVILLE DAM
COLUMBIA RIVER BASIN
CASCADE LOCKS, OREGON

US Army Corps of Engineers®

Bonneville POWER ADMINISTRATION

BON Spillway Rock Mitigation

Phase 1A Report – Final



Prepared By:
US Army Corps of Engineers
Portland District

January 2022

THIS PAGE INTENTIONALLY LEFT BLANK

EXECUTIVE SUMMARY

In the last decade, hydrosurveys conducted in the Bonneville Dam spillway have found rocks migrating upstream towards and into the stilling basin. Model studies have concluded the movement of the rocks is due to the hydraulic patterns within the spillway. The fast-moving layer of water near the surface caused by the flow deflectors creates a hydraulic pattern akin to a vertical eddy and water velocities near the bedrock of the spillway are large enough in magnitude to move material in the upstream direction. Loose rocks in this current can migrate to the apron, up the apron, and into the stilling basin. Once rocks have entered the stilling basin, they cause damage to the concrete from ball milling effects. Rocks in the stilling basin must be removed mechanically by cranes and divers as part of non-routine maintenance that is required to extend the life of the spillway and stilling basin.

This project aims to design and construct a solution to the rock migration problem that will prevent all rocks from entering and causing damage to the stilling basin. In addition, this project also aims to replace the non-routine maintenance contract framework for rock removal with a long-term, recurring maintenance contract framework for rock removal.

Numerical modeling was utilized to assist in evaluating the three most feasible alternatives: barriers on the apron, a continuous baffle block extension, and raising of apron low spots. The raising of apron low spots caused excessive shoreline velocities along Bradford Island. The barriers and baffle block extension were physically modeled. The modeling effort concluded that barriers are the only hydraulically viable alternative to exclude rocks from entering the stilling basin. The structural design of the barrier underwent an iterative design process. The revised barrier design consists of a rectangular cross-section and no anchors. Construction is anticipated to be modular, with precast concrete cells to be submerged and filled with concrete. Each barrier is 13 feet wide, 17 feet tall, and 80 feet long. The southern barrier will be placed on the apron, in-line with the spillway pier between bays 11 and 12, at approximately -35 feet Mean Sea Level. The northern barrier would also be placed on the apron, but its precise alignment is yet to be determined.

The total project cost (design and construction) estimated at the 90% Phase 1A milestone is $6.65 million including a 41.9% contingency. With a 41.9% contingency and 5.5% escalation, the construction contract is estimated at $5.12 million. The construction contract in-water work period is expected to last approximately one month.

CONTENTS

TABLES

FIGURES

APPENDICES

PERTINENT DATA

BONNEVILLE PROJECT

Note: Datum for vertical elevations is NGVD29. At Bonneville Project, to convert to NAVD88, add 3.34 feet to NGVD29.

1. Project Description

Stream: Columbia River (river mile 145.6)
Location: Cascade Locks, Oregon
County: Multnomah County in Oregon/Skamania County in Washington
Owner: United States Government
Project Authorization:
National Industrial Recovery Act of 1933 .... Initial appropriation
Rivers and Harbors Act of 1935.................................... Formal authorization for construction
Bonneville Project Act of 1937…... Authorized Second Powerhouse
Supplemental Appropriations Act, August 1985 .... Authorized Navigation Lock 2
Primary Authorized Purposes ....Power, Navigation
Operated for other purposes .... Fish & Wildlife Conservation, Water Quality, Recreation

2. Drainage Area .... 240,000 square miles

3. Reservoir (Datum is NGVD 29 (feet))

Length of Reservoir .... 46.9 miles
Full pool elevation ....77.0 feet
Normal operating pool elevation range .... 71.5 to 76.5 feet
Minimum pool elevation ....70.0 feet
Pool at spillway design flood (1,600,000 cfs) .... 82 feet
(assuming flood fighting, and no overtopping and 2005 spillway rating curve)
Normal high tailwater elevation ....35.0 feet
Normal low tailwater elevation ....7.0 feet

4. First Powerhouse

Length ....1,027 feet
Width...190 feet
Height (roof to bedrock) ....190 feet
Number and type of main turbines units.... 10 Main, 1 Station Service, Minimum Gap Runner
Total unit capacity limit at rated power factor .... 599.4 MW

5. Second Powerhouse

Length (including erection bay & service bay) .... 985.5 feet
Width (U/S face of intake to D/S face of draft tube) .... 221.25 feet
Height (roof to bedrock) .... 210 feet
Number and type of units .... 8 Main Units, 2 Fish Units, Kaplan
Total unit capacity limit at rated power factor .... 558.2 MW

6. Spillway

Type .... Gate controlled, ogee, concrete gravity

Length (overall) .... 1,450 feet
Gate type .... Two leaf, vertical sliding
Number and size of gates .... 18 gates, 50 ft-wide x 60 ft-high
Crest elevation ....24 feet
Deck elevation ....97 feet
Spillway Design Flood[1] .... 1,600,000 cfs

7. Navigation Lock 1 (Not in Service)

Type .... Single lift
Length ....500 feet
Width..76 feet
Vertical lift .... 30 to 70 feet

8. Navigation Lock 2

Type .... Single lift
Length ....675 feet
Inside width ....86 feet
Vertical Lift .... 30 to 70 feet

9. Real Estate (2018)

Easement .... 9,383 acres
Acquired (Fee) .... 1,574 acres
Public Domain Withdrawal (Fee) .... 723 acres

[1] The spillway was designed to pass the Spillway Design Flood; however due to the condition of low points at the Project, level of flood fight actions, the ability to open the gates to a free flow condition, the spillway discharge capability would vary.

ACRONYMS AND ABBREVIATIONS

Acronym	Description
ALT	Alternative
ASTM	American Society for Testing and Materials
BiOp	Biological Opinion
BONSRM	Bonneville Spillway Rock Mitigation
CFD	Computational Fluid Dynamics
cfs	Cubic Feet per Second
CHL	Coastal and Hydraulics Laboratory
COVID-19	Coronavirus Disease 2019
EM	Engineering Manual
ERDC	Engineering Research and Development Center
ft^3	Cubic Feet
HTRW	Hazardous, Toxic, and Radiological Waste
IWWW	In-Water Work Window
kcfs	Thousand Cubic Feet per Second
KSF	Kips per Square Foot
lbs	Pounds
MCACES	Micro Computer Cost Estimating System
MSL	Mean Sea Level
NMFS	National Marine Fisheries Services
NTP	Notice to Proceed
NWP	Corps, Portland District
O&M	Operation and Maintenance
OBE	Operational Basis Earthquake
PDT	Product Development Team
PGA	Peak Ground Acceleration
PMF	Probable Maximum Flood
PNNL	Pacific Northwest National Laboratory
psi	Pounds per Square Inch
ROV	Remotely Operated Vehicle
STP	Special Technical Publication
TDG	Total Dissolved Gas
USACE	United States Army Corps of Engineers

Action Agencies	U.S. Army Corps of Engineers, Bonneville Power Administration, U.S. Bureau of Reclamation
AFF	Adult Fish Facilities
AWS	Auxiliary Water Supply
B2CC	Bonneville Second Powerhouse Corner Collector
BPA	Bonneville Power Administration
CBT	Columbia Basin Telecommunications
CENWD	Corps, Northwestern Division
CENWD-PDW	Corps, Northwestern Division, Columbia Basin Water Management Division
CENWD-PDW-HP	Corps, Northwestern Division, Columbia Basin Water Management Division, Hydrologic Engineering and Power Branch
CENWD-PDW-R	Corps, Northwestern Division, Columbia Basin Water Management Division, Reservoir Control Center
CENWP	Corps, Portland District
CENWP-HDC	Corps, Portland District, Hydroelectric Design Center
CENWP-ENC-H	Corps, Portland District, Engineering and Construction Division, Hydraulics & Hydrology Branch
CENWP-ENC-HC	Corps, Portland District, Engineering and Construction Division, Hydraulics & Hydrology Branch, Dam & Levee Safety Section
CENWP-ENC-HD	Corps, Portland District, Engineering and Construction Division, Hydraulic Design Section
CENWP-ENC-HR	Corps, Portland District, Engineering and Construction Division, Hydraulics & Hydrology Branch, Reservoir Regulation & Water Quality Section
CENWP-ENC-HY	Corps, Portland District, Engineering and Construction Division, Hydraulics & Hydrology Branch, River & Hydrologic Engineering Section
CENWP-ODB	Corps, Portland District, Operations Division, Bonneville Project
CENWP-ODN-W	Corps, Portland District, Waterways Maintenance Section
CENWP-SO	Corps, Portland District, Safety & Occupational Health Office
CENWW	Corps, Walla Walla District
cfs	cubic feet per second
CHPS	Community Hydrologic Prediction System

COOP	Continuity of Operations Plans
CRITFC	Columbia River Inter-Tribal Fish Commission
CROHMS	Columbia River Operational Hydromet Management System
CRS[1]	Columbia River System
CWA	Clean Water Act
CWMS	Corps Water Management System
EAP	Emergency Action Plan
EIS	Environmental Impact Statement
EPA	Environmental Protection Agency
ESA	Endangered Species Act
ESP	Extended Streamflow Prediction
FCOP	Flood Control Operating Plan (Columbia River Treaty)
FCRPS[1]	Federal Columbia River Power System
FMS	Fixed Monitoring Station
FPOM	Fish Passage Operation and Maintenance coordination team
FOP	Fish Operations Plan
FPP	Fish Passage Plan
GIS	Geographic Information System
GDACS	Generic Data Acquisition and Control System
GOES	Geostationary Operational Environmental Satellite
IRRM	Interim Risk Reduction Measures
JBS	Juvenile Bypass Facility
LIDAR	Light Detection and Ranging
LFS	Lamprey Flume System
LPS	Lamprey Passage System
MGR	Minimum Gap Runner
msl	mean sea level
MW	Megawatt

NEPA	National Environmental Policy Act
NGVD29	National Geodetic Vertical Datum 1929
NMFS	National Marine Fisheries Service, also known as NOAA Fisheries
NWDP	Northwestern Division, Portland (WCDS terminology)
NOAA	National Oceanic and Atmospheric Association
NRCS	Natural Resources Conservation Service
NWRFC	Northwest River Forecast Center
NWS	National Weather Service
ODEQ	Oregon Department of Environmental Quality
ODFW	Oregon Department of Fish and Wildlife
OWRD	Oregon Water Resources Department
OWSC	Oregon Water Science Center
PA	Periodic Assessment
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
PNCA	Pacific Northwest Coordination Agreement
Pub. L.	Public Law
RCC	Reservoir Control Center
RIOG	Regional Implementation Oversight Group
RMJOC	River Management Joint Operating Committee
Reclamation	U.S. Bureau of Reclamation
ROCASOD	Record of Consultation and Statement of Decision
ROD	Record of Decision
RPA	Reasonable and Prudent Alternative
RWCDS	Regional Water Control Data System
SCADA	Supervisory Control and Data Acquisition
SDF	Spillway Design Flood
Services	NOAA Fisheries and USFWS
SMF	Smolt Monitoring Facility
SNOTEL	Snowpack Telemetry
SPF	Standard Project Flood
STS	Submersible Traveling Screens
SWE	Snow Water Equivalent

SYSTDG	System Total Dissolved Gas (model)
TDG	Total Dissolved Gas
TMDL	Total Maximum Daily Load
TMT	Technical Management Team
UMT	Upstream Migrant Transportation
USACE	United States Army Corps of Engineers
USFWS	United States Fish and Wildlife Service
USGS	United States Geological Survey
VHF	Very High Frequency (radio)
WCDS	Water Control Data Systems
WCM	Water Control Manual (Bonneville)
WDOE	Washington State Department of Ecology
WMP	Water Management Plan

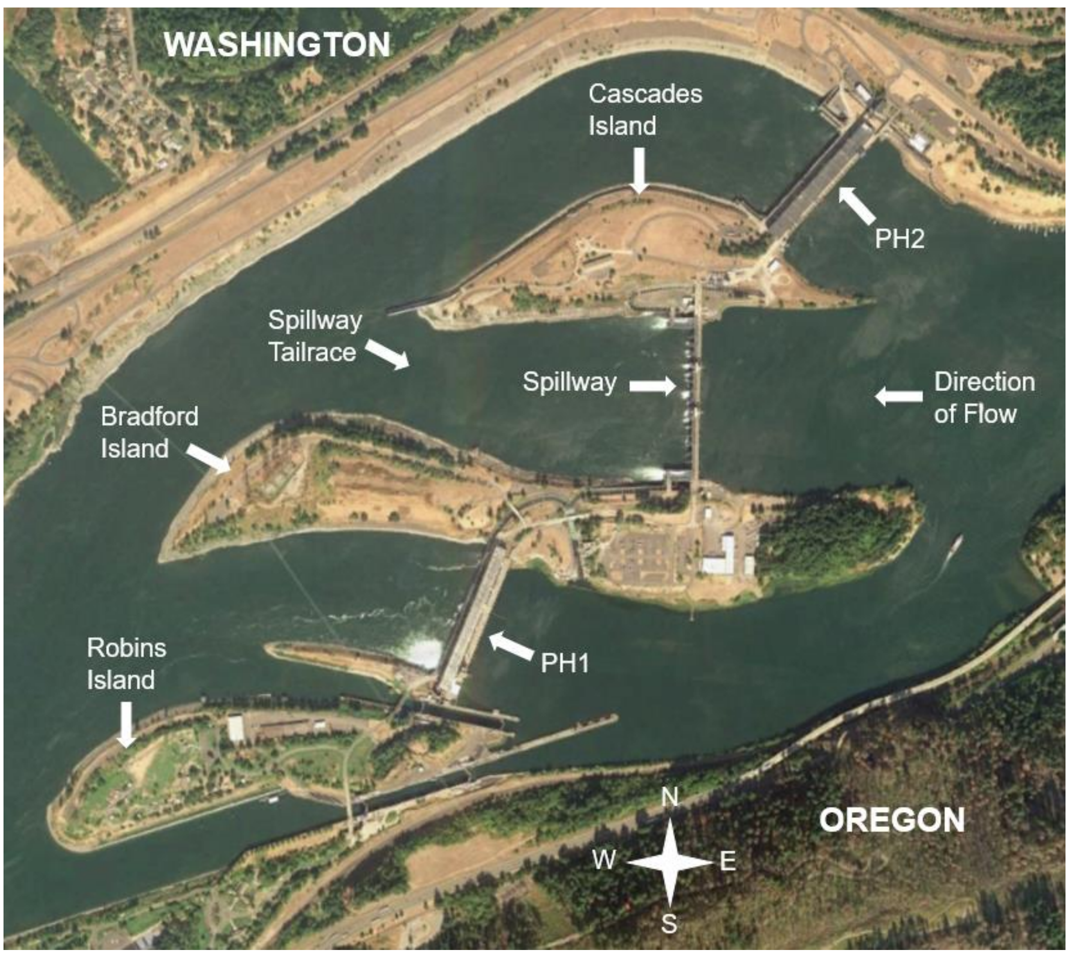
SECTION 1 - PROBLEM STATEMENT

1.1 BACKGROUND INFORMATION

Bonneville Dam (Figure 1-1) straddles the Columbia River between Oregon and Washington approximately 40 miles east of Portland, Oregon at River Mile 146.1. Bonneville Dam, also referred to as "Bonneville" in this document, is owned and operated by the United States Army Corps of Engineers (USACE). Bonneville, its ancillary components, and its operation staff constitute the Bonneville Project, also referred to as "the project" in this document.

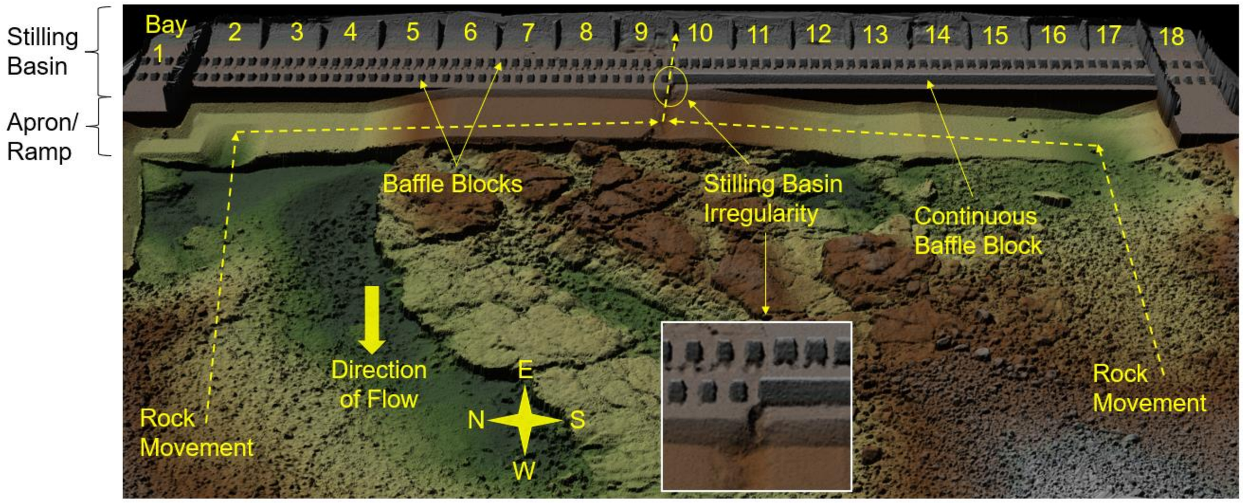
Bonneville spans the width of the Columbia River, connecting Robins, Bradford, and Cascades Islands. Bonneville is a run-of-the-river dam, meaning it has little to no storage in its forebay; it passes the water that it receives.

Figure 1-1. Bonneville Project.



The Bonneville Spillway (Figure 1-2) spans the section of river between Bradford Island and Cascades Island. The spillway consists of 18 bays, a concrete stilling basin, a concrete apron (also referred to as a ramp), and bedrock. Each bay is 50 feet wide with 10-foot-wide piers and has an ogee crest elevation of 24 feet mean sea level (MSL). All elevations in this document are measured from MSL. Each bay has an independently controlled vertical lift gate. Spanning the width of the stilling basin (237.5 feet) are two rows of baffle blocks for energy dissipation.

Figure 1-2. Bonneville Spillway.



The apron was constructed to provide bedrock scour protection downstream of the stilling basin and serves as a place to position a cellular cofferdam to permit dewatering of the stilling basin. The elevation of the apron is variable and follows the top of excavated bedrock. The southern half of the apron has been used twice for placement of cofferdams for repairs and modifications to the baffle blocks (1937 and 1954). The apron has notable features that pertain to this project. First, the north and south ends have low spots. These two low spots correspond to where unfavorable foundation conditions of weaker rock and more soil-like foundation materials were over excavated and removed. Second, the highest point on the apron coincides with the most erosion resistant foundation bedrock. The apron's high point also coincides with the lowest point in the stilling basin edge; this location has the smallest differential elevation between the apron and the stilling basin.

The Bonneville spillway channel is marked by three zones downstream of the apron, moving north to south across the channel. The middle section is more scour/erosion resistant, volcanically derived outcroppings of sandstones and conglomerates. This is evident from bathymetric relief imagery (Figure 1-2) where fault and shear zones are scoured out to form 10- to 20-foot-deep linear trenches. This exposed bedrock is one of the sources of rocks for bedload. Over the last few years, the hydrosurveys have shown a general movement of approximately 3000 cubic yards of rock on each side of the outcropping towards the apron.

As a whole, the Bonneville spillway has numerous problems and is considered to be in poor condition. Damage to the Bonneville stilling basin has occurred since the construction of the dam. The cause of the damage has changed over the years as changes have been made to the project and spillway operations. Dam Safety studies have concluded that the risk of loss of life is low; however, economic and environmental consequences are very high. Since the Bonneville spillway is currently functional and with no immediate threat of a catastrophic failure, it is not a national priority to be rehabilitated. This study is focused on enhancing operation and maintenance (O&M) measures to extend the life of the spillway until higher level decisions can be made to move forward on the spillway's larger and more expensive issues.

1.2 SPILLWAY MODIFICATIONS

Major repairs were made to the southern part of the basin during the early 1940s and mid-1950s. In 1954, the south half of the spillway was repaired due to damage caused by the 1948 flood. The repair was executed in the dry, with a cofferdam built around the south half of the spillway with large coffer cells placed on the apron. During the repair, one of the rows of baffle blocks on the south half of the spillway was modified to a continuous baffle block. In the process of removing the cofferdam, an end sill irregularity was created between bays 9 and 10 (Figure 1-2, inset). The cause of the irregularity formation is not known.

Continuously generated power was not needed so spill during nonpeak times was required since Bonneville is a run of the river project with limited upstream facilities. Spill flow was concentrated in a few bays due to ease of operations. During the mid- to late-1950's, uniform spill patterns were recommended because of the erosion experience in the stilling basin.

During the 1970s, modifications started to be implemented that improved water quality and overall fish passage. The spillway is a major fish passage route for out-migrating juveniles. To assist with fish passage and survival through the spillway, flow deflectors were installed. Spillway flow deflectors are designed to minimize the saturation of total dissolved gasses (TDG) caused by spillway releases. A properly designed flow deflector forces the spill flow to skim the surface of the receiving tailwater. The purpose is to prevent the highly aerated water from being exposed to the hydrostatic pressures within the lower depths of the stilling basin. The Biological Opinion (BiOp) developed by the National Marine Fisheries Services (NMFS) currently mandates that spillway releases be maximized and within TDG criteria, during the juvenile fish out-migration season, which typically extends from April 10 to August 31.

The Bonneville Laboratory investigations resulted in construction of spillway flow deflectors on thirteen of Bonneville's 18 spillway bays. All thirteen deflectors were constructed at elevation 14 feet on bays 4-15 and bay 18; they are 12 feet long, with a 6-foot radius transition from the spillway slope to the horizontal surface of the deflector. From the 1970s to the 1990s, spill patterns were developed to aid in fish passage and survival needs.

Because of spill requirements for juvenile fish passage and the related TDG issues, a re-investigation of spillway flow deflectors was conducted in 1999/2000. The purpose of these investigations was to evaluate the existing deflector design under current operating conditions and to design a deflector for the remaining five non-deflected spillway bays. The goal was to optimize the deflector design to perform well for both voluntary (or juvenile fish spills) and involuntary spills. The model investigations resulted in a 12.5-foot deflector at elevation 7 feet MSL. The lower deflector elevation was necessary to prevent plunging flow from occurring under low tailwater conditions.

In 2002, five new flow deflectors were installed at elevation 7 feet on bays 1-3, 16, and 17 and the flow deflector in bay 18 was modified from elevation 14 feet to elevation 7 feet. Currently, all bays have flow deflectors. Bays 1-3 and 16-18 have flow deflectors at elevation 7 feet and bays 4-15 have flow deflectors at elevation 14 feet.

1.3 FLOW DEFLECTOR COMPLICATIONS

The effect of deflected spillway flows on ogee and stilling basin erosion was not thoroughly documented during either of the deflector model investigations. However, some general observations were made during the original physical model effort by seeding the model with sand and gravel. In general, there are two distinct flow patterns generated by the deflected spillway release that would appear to cause erosion at the toe of the ogee adjacent to the spillway piers, and across the pier nose. Erosion at these locations is clear in stilling basin surveys. The first flow condition occurs when the tailwater elevation is relatively high. The deflected spillway jet generates a large circulation cell, which has the potential to carry bedload material (large rocks and cobbles) from the downstream river channel upstream along the floor of the basin to the ogee toe. The heavier material is deposited in the corner pockets formed by the ogee toe and the adjacent pier walls. As the tailwater elevation lowers during the late summer periods, the spillway flow begins to plunge from the deflectors and the circulation cell beneath the jet becomes much more intense and focused at the ogee toe and corner pockets. The very intense circulation of flow, combined with any bed load material that may have been deposited under the higher tailwater conditions, generates a ball milling action with an extreme potential for erosion.

The newer deflectors at elevation 7 feet will be less likely to cause plunging flow and will reduce the potential for ball milling of deposited material. However, the initial thirteen deflectors, which were designed for higher involuntary spill flows and higher tailwater elevations will now be subjected to operations throughout the fish passage season that results in skimming and then plunging flow conditions.

Although the hydraulic characteristics of deflected spillway flow have been thoroughly investigated and well defined, the potential for erosion has not. Uncertainties include the sources and availability of bed load material for transportation into the stilling basin and the carrying capacity of the vertical circulation cells generated by the deflected flow. The lower deflectors may reduce the potential for erosion caused by ball-milling of deposited material but may increase the potential to transport material from the downstream channel up to the spillway.

1.4 ROCK MOVEMENT

The condition of the Bonneville spillway has been monitored on a yearly basis since 2006. Hydrosurveys are conducted annually if spill hits 150 kcfs or higher. The 2011 spill season was the first time in recent times where the rock movement resulted in material in the stilling basin. After the 2011 spill season, 1150 cubic yards of material were found deposited in the stilling basin. Dive surveys showed that the material was well rounded rock, ranging in size from gravel up to 4 feet in diameter. The source of the rocks is believed to be cobbles and boulders being derived from erosion/scour of the underlying bedrock. Earlier surveys have shown movement of material in the spillway downstream of the apron, but this was the first time that material was found between the flow deflectors and baffle blocks.

The 1:55 scale Bonneville spillway physical model was used to investigate rock movement after the 2011 spill season. Spill volumes of 125, 150, 175, 200, and 300 kcfs were evaluated. No rock movement was noted at 125 kcfs but at 150 kcfs, rock movement was initiated. Observations from the physical model showed that rocks generally start on the bedrock downstream of bays 16 and 17, move on to the apron, move up the apron towards the center of the spillway, and maneuver into the stilling basin. Near the middle of the spillway, the apron and stilling basin invert are nearly level, making it possible for the rocks to move into the stilling basin. Most rocks entered at bay 9, just north of the continuous baffle block, and moved laterally towards bays 2 and 17.

The movement of the rocks is due to the hydraulic patterns within the spillway. The fast-moving layer of water near the surface caused by the flow deflectors creates a hydraulic pattern akin to a vertical eddy. Physical model studies have estimated water velocities near the bedrock of the spillway to be on the order of 60 feet per second in the upstream direction. Loose rocks in this current can migrate to the apron, up the apron, and into the stilling basin. Once in the stilling basin, rocks cause damage to the concrete from ball milling effects. Because the spillway is used for five months of the year, the cumulative ball milling damage is potentially serious. Rocks in the stilling basin cannot be removed via a flushing spill operation; they must be removed mechanically by cranes and divers as part of non-routine maintenance that is required to extend the life of the spillway and stilling basin.

If spill exceeds the 150 kcfs ‘threshold,’ rocks tend to move upstream and make their way into the stilling basin. Since Bonneville typically spills more than 150 kcfs every year, rocks are found within the stilling basin most years. Table 1 summarizes the hourly spill levels since 2011. The current contract framework behind the removal of the rocks is via a non-routine maintenance contract. Each year, a contract is drawn up, issued, and executed to remove the rocks that have migrated into the stilling basin since the last rock removal contract was executed. The most recent contract was awarded in January 2021 for $411,736.

Table 1-1. Annual Spill Levels at Bonneville.

	Hours of Spill					
	20 – 50 kcfs	**50 – 75 kcfs**	**75 – 100 kcfs**	**100 – 125 kcfs**	**125 – 150 kcfs**	**150 + kcfs**
2011	125	201	1333	177	499	1484
2012	32	337	1026	773	787	821
2013	235	339	2148	853	121	26
2014	334	120	1680	1401	215	204
2015	73	180	2631	783	43	0
2016	45	163	2141	939	115	93
2017	104	264	1386	295	266	2104
2018	132	100	1553	896	118	837
2019	1	151	1707	1012	571	18
2020	83	351	1246	743	868	191

1.5 JUSTIFICATION

The Bonneville Spillway Rock Mitigation (BONSRM) project aims to design and construct a solution to the rock migration problem that will prevent all rocks from entering and causing damage to the stilling basin. If necessary, rocks may still need to be mechanically removed from the system, but on a less frequent basis.

In addition to a solution that will prevent rocks from entering the stilling basin, this project also aims to replace the existing non-routine maintenance contract framework for rock removal with a long-term, recurring maintenance contract framework for rock removal. The new contract framework will include provisions for the government to determine how frequently removal of rocks will occur.

1.6 CRITERIA AND CONSTRAINTS

A virtual charrette was held June 30, 2020. The project was discussed at length with the stakeholders and product development team (PDT) members in attendance. The criteria and constraints for this project were established during and shortly after the charrette by PDT members and project stakeholders.

1.6.1 Criteria

Criteria are guidelines for the project and design that define its success. The project and design must adhere to these items to be considered successful. The criteria for this project are listed below.

1.6.1.1 *Exclude Rocks*

The project’s end goal is to design and construct a solution to the rock migration problem that will prevent rocks from entering the stilling basin (upstream of the baffle blocks).

1.6.1.2 *Spillway Capacity*

The capacity of the Bonneville spillway cannot be diminished from its existing condition.

1.6.1.3 *In-Water Work Window*

The in-water work window (IWWW) for Bonneville Dam must be adhered to.

1.6.1.4 *Movement of Adult Fish*

The recommended alternative cannot negatively impact the movement of adult fish.

1.6.1.5 *Movement of Juvenile Fish*

The recommended alternative cannot negatively impact the movement of juvenile fish. The Bonneville spillway is a major passage route for downstream-migrating juvenile fish. The recommended alternative must either be deep enough in the water column such that juvenile fish do not encounter it, or the recommended alternative must be fish-friendly if juvenile fish do encounter it.

1.6.1.6 *Additional O&M*

The recommended alternative cannot create cumbersome O&M requirements upon the project. This implies the recommended alternative be of the 'set it and forget it' variety.

1.6.1.7 *Recurring Maintenance Contract*

The current administrative process by which rocks are removed from the Bonneville spillway is by executing a non-routine maintenance contract. Since rocks are typically found in the spillway every year, a non-routine maintenance contract needs to be drawn up, awarded, and executed each year. A long-term, recurring maintenance contract framework would be better suited for this type of work. As such, in conjunction with the recommended alternative, a recurring contract for rock removal in the Bonneville spillway is to be developed and implemented in place of the existing non-routine maintenance contract framework. The recurring maintenance contract framework will include provisions for the government to determine how frequently removal of rocks will occur.

1.6.1.8 *Design Flood*

The recommended alternative must be functionally capable of withstanding the 100-year (or 500 kcfs) flood event with little or repairable damage.

1.6.1.9 *Design Seismic Event*

The recommended alternative must be functional following a 144-year operational basis earthquake (OBE) seismic event with no or minimal repair. The recommended alternative must consider that repair may take place several years following an earthquake.

1.6.1.10 *Dam Safety*

The recommended alternative must meet a minimum level of dam safety requirements.

- Do no harm. The recommended alternative must not damage or harm the existing structure (Bonneville dam) or its functionality.
- The recommended alternative should be reversible if it is found to be causing unanticipated problem(s).
- The recommended alternative should not impair future use of the apron for placement of a cofferdam to conduct stilling basin repair work.

1.6.2 Constraints

Constraints are the real-world limitations that are imposed on the project and design. Constraints must be kept in mind when designing, evaluating, and constructing the recommended alternative. The constraints for this project are listed below.

1.6.2.1 *Existing Condition – Future Without Project*

Bonneville Project has fully operated for over 80 years without the proposed improvement. There is no incremental loss of life risk attributable to the accumulation of rocks in the stilling basin. However, long-duration spills result in ball milling and scouring of the stilling basin by the accumulated rocks. This scouring is causing accelerated deterioration and premature need for expensive rehabilitation of the spillway. Currently, the operational solution is for periodic over-water rock removal. This is a significant maintenance project that requires one or two months to accomplish in one IWWW. The most recent contract was awarded for $411,736. This is the approximate annual baseline cost of the current condition without project alterative (see section 4.4 for further cost details).

The project is considered enhanced non-routine maintenance. The purpose of this project is to slow and/or halt degradation of the stilling basin to extend its useful life of the spillway while still meeting court order environmental fish spills while operational and meeting dam safety considerations. Therefore, any alternative must meet:

- No measurable risk to human life during construction, operation, surveillance, or maintenance above existing baseline condition.
- No increase in environmental risk (fish mortality) above existing baseline condition.

- No impact or risk of the damage to the apron which is likely to be required for dewatering for future rehabilitation or repairs of the spillway above existing baseline condition.

Project alternatives will be evaluated predominantly on economic, operational, environmental considerations (simplest, most reliable, lowest construction cost, and lowest maintenance cost) for the next 50 to 100 years while other studies on how to rehab the spillway are completed.

1.6.2.2 *Fish Passage Spill Volumes*

The Bonneville spillway is a major passage route for downstream-migrating juvenile fish. Bonneville maximizes spill during the juvenile migration season to encourage juvenile fish passage through the spillway. Fish passage spill volumes are on the edge of spill levels that are known to move rocks (150 kcfs). The project is going to continue to spill for fish, implying rocks are likely to move towards, if not into, the stilling basin every year.

1.6.2.3 *Run-of-the-River Spill Volumes*

Bonneville is a run of the river project. As flow increase additional flow is ran through the powerhouse. If there is a lack of power demand or the capacity of the powerhouses is exceeded the spillway is open to pass inflow. This creates moments of high spill volumes during the freshet. These volumes regularly exceed volumes that have been known to cause rock movement (150 kcfs), implying rocks are likely to move towards, if not into, the stilling basin every year.

1.6.2.4 *Under-Keel Clearance*

The recommended alternative design must maintain the minimum under-keel clearance of vessels that need access to the spillway area, even during periods of low tailwater.

1.6.2.5 *Wet Construction*

The construction of the recommended alternative must occur during the IWWW (1-Dec through 28-Feb). This is the period of highest daily precipitation, winds, and surface waves. Construction is expected to experience some percentage of adverse weather days. All construction will occur in open water below the spillway without a cofferdam in place. There will be no wave dampening devices available during the IWWW to protect or reduce potential adverse impact of waves on overwater crane or diver activities, which are expected to occur at depths of approximately 75 feet below the water surface.

1.6.2.6 *Flushing Spill*

The recommended alternative cannot inhibit the project’s ability to conduct flushing spill. Flushing spill allows the project to clear rocks off the apron, but rocks already in the stilling basin cannot be removed via flushing spill.

1.6.2.7 *Vertical Clearance*

Vertical clearance over the spillway tailrace is inhibited due to low-hanging high-power lines. Movement of tall equipment or materials into the spillway from downstream must be aware of the limited vertical clearance.

1.6.2.8 *Shoreline Velocities*

The recommended alternative cannot increase shoreline velocities along Bradford and Cascades Islands. The recommended alternative cannot exacerbate damage to either river-bank riprap or fish ladders along Bradford and Cascades Islands. Left bank riprap has been repaired at least three times since constructed, suggesting that the riprap may be undersized and shoreline velocities cannot be increased any further.

SECTION 2 - ALTERNATIVES

A virtual charrette was held June 30, 2020 and the project was discussed at length with the stakeholders and PDT members in attendance. Several alternatives were suggested and discussed. Many were screened out due to various reasons and three were deemed the most feasible.

2.1 CONSIDERED ALTERNATIVES

Listed below are the possible alternatives that were proposed.

2.1.1 ALT 1: Do Nothing

This alternative consists of keeping the status quo. It would allow rocks to freely move, given sufficient flows, onto the ramp and into the stilling basin. The non-routine maintenance contract framework for rock removal would be retained.

2.1.2 ALT 2: Barrier

This alternative consists of a structural barrier placed on the north and south portions of the ramp to prevent rocks from moving up and ramp and into the stilling basin. The barrier would have to be designed to be structurally robust to withstand spillway flows.

2.1.3 ALT 3: Extended Continuous Baffle Block

This alternative consists of an extension to the continuous baffle block that currently sits downstream of bays 10 through 17. The continuous baffle block would be extended to the north by the length of half a bay (approximately 25 feet) to prevent rocks from maneuvering from the apron to the stilling basin. The continuous baffle block extension would have to be structurally robust to withstand spillway flows.

2.1.4 ALT 4: Raised Apron

This alternative consists of raising the low spots on the north and south ends of the apron sufficiently high so that rocks are unable to neither move onto nor make their way up the apron.

2.1.5 ALT 5: Removal of All Rocks

This alternative consists of the methodical collection and disposal of all loose rocks downstream of the spillway that could eventually migrate into the stilling basin.

2.1.6 ALT 6: Angled Barriers

This alternative takes the idea of the barrier (ALT 2) but sets them at an obtuse angle to the direction of rock movement. This would encourage the rocks to slide along the barrier and off the apron.

2.1.7 ALT 7: Pit for Rocks

This alternative consists of excavating a large sink or pit for rocks to fall into as they migrate towards the apron. Since rocks are on both the north and south sides of the spillway, two pits would likely have to be excavated. If the pit were significantly large enough, the rocks it collects would, in theory, be stuck there indefinitely.

2.1.8 ALT 8: Series of Barriers

This alternative consists of constructing a series of barriers either on the apron or leading up to the apron. These would prevent rocks from migrating to and up the apron.

2.1.9 ALT 9: Spillway Jetty

This alternative consists of constructing a jetty within the spillway tailrace that disrupts the hydraulics that cause the rocks to move in the first place.

2.2 ALTERNATIVES SCREENED OUT

Listed below are the alternatives that were screened out and the associated rationale.

2.2.1 ALT 5: Removal of All Rocks

This alternative was screened out due to cost. The time and effort it would take to locate and extract every loose rock downstream of the spillway was deemed prohibitively expensive. In addition, without the armoring rock protection, the bottom of the river would likely to scour even deeper further reducing tailwater in the stilling basin exasperating the hydraulic problem with and exposed more rocks from the underlying bedrock.

2.2.2 ALT 6: Angled Barriers

This alternative was screened out due to structural concerns. Angled structures in the spillway would have to cope with prolonged asymmetrical hydraulic loading. The structural integrity and anchoring of the barriers might be subject to failure with any orientation other than parallel to flow.

2.2.3 ALT 7: Pit for Rocks

This alternative was screened out due to cost. Excavating two large pits through the bedrock deep enough to collect all loose rocks was deemed prohibitively expensive. In addition, the pits would have to be constructed immediately adjacent to the apron and could pose a potential undermining issue for the apron. This could potentially damage and render the apron unable to provide its intended purpose. Undermining critical dam features is not acceptable from a dam safety perspective.

2.2.4 ALT 8: Series of Barriers

This alternative was screened out due to redundancy. Stopping the rocks with one barrier at a bottleneck location (i.e., on the apron) would work just as effectively as a series of barrier on or leading up to the apron.

2.2.5 ALT 9: Spillway Jetty

This alternative was screened out due to scope. Construction of a jetty is a major undertaking and requires months, if not years, of intensive research and testing spanning many engineering disciplines and external agencies. The scope of this project does not extend to such an alternative.

2.3 MOST FEASIBLE ALTERNATIVES

With several alternatives screened out, the most feasible alternatives are listed below.

2.3.1 ALT 1: Do Nothing

This alternative consists of keeping the status quo. It would allow rocks to freely move, given sufficient flows, onto the ramp and into the stilling basin. The existing non-routine maintenance contract framework for rock removal would be retained. This alternative is viable. Rocks identified within the stilling basin are removed every year, albeit retroactively. This alternative requires coordination and awareness to keep on top of the rock migration issue and ensure minimal damage to the stilling basin.

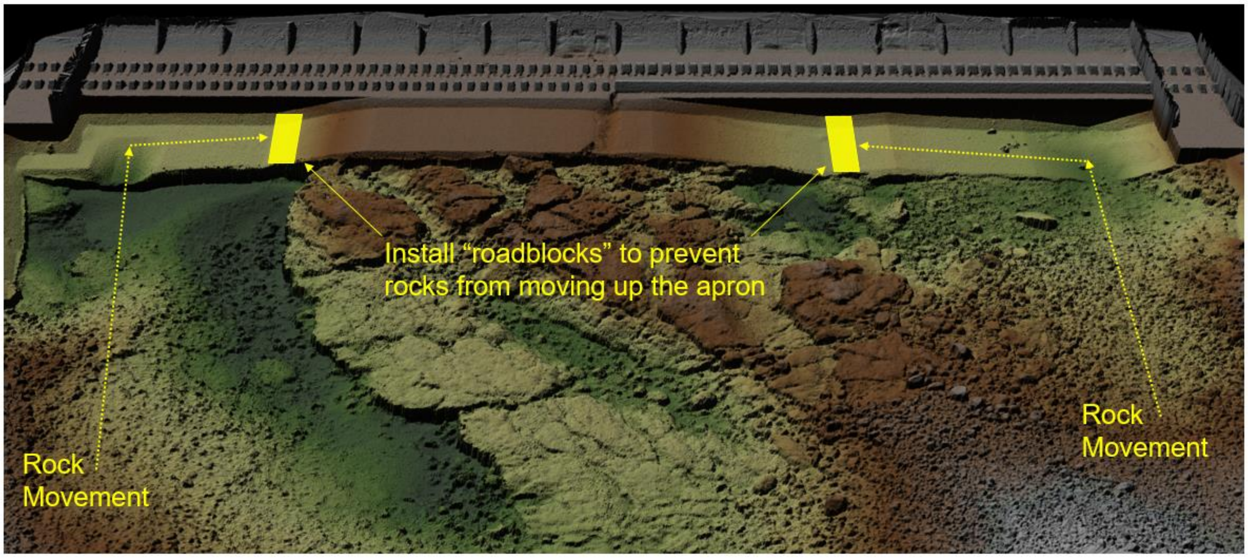
2.3.2 ALT 2: Barrier

This alternative consists of a structural barrier placed on the north and south portions of the apron to prevent rocks from moving up and into the stilling basin (Figure 2-1). The barrier would not be anchored to the underlying apron slab or bedrock. It would be designed to be sufficiently heavy and structurally robust to withstand turbulent spillway flows. Small movements of a few feet are not expected to impair its functionality. Any gaps between the barrier and the apron or end sill wall will be sealed. The tops of the barriers must not exceed the sill height of the stilling basin. The barrier must also be parallel to flow to minimize the hydraulic load acting upon them. Spillway flows up to 100-year would be designed for and 144-year seismic would be designed for. Minor repairable damage would be acceptable. Two will be installed; one on the north and one on the south portions of the ramp. If stopped by the barrier, rocks would likely accumulate on the down-ramp side of the barrier.

Attached to this alternative is the replacement of the existing non-routine maintenance contract framework for rock removal with a long-term, recurring maintenance contract framework for rock removal. The new contract framework will include provisions for the government to determine how frequently removal of rocks will occur. Annual hydrosurveys will be conducted to assess the quantity of rock blocked by the barriers.

The hydrosurveys will also assist in determining if execution of a rock removal contract is warranted to prevent rocks from overtopping the barriers.

Figure 2-1. Conceptual "Barrier" Alternative.

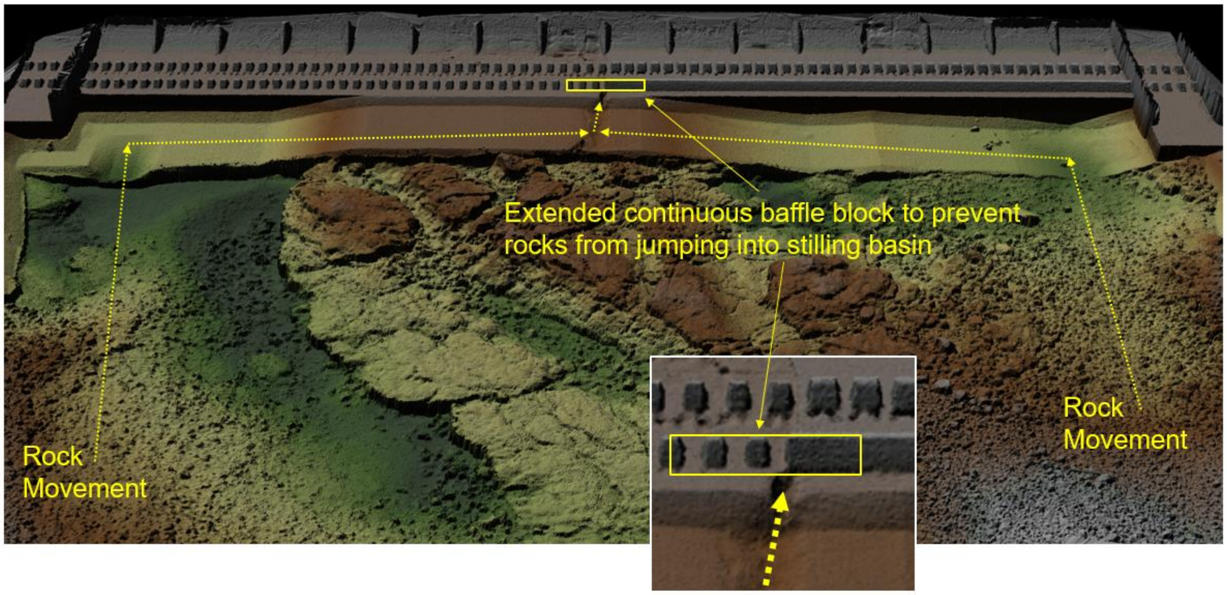


2.3.3 ALT 3: Extended Continuous Baffle Block

This alternative (also referred to as "Baffle") consists of an extension to the continuous baffle block that currently sits downstream of bays 10 through 17. The continuous baffle block would be extended to the north by the length of half a bay (approximately 25 feet) to prevent rocks from entering the stilling basin. Work would be complex and must be conducted in water depths of 30 to 40 feet. The continuous baffle block extension (Figure 2-2) would have to be structurally robust to withstand spillway flows. If stopped by the baffle, rocks would likely accumulate at the top of the apron, near the stilling basin irregularity.

Attached to this alternative is the replacement of the existing non-routine maintenance contract framework for rock removal with a long-term, recurring maintenance contract framework for rock removal. The new contract framework will include provisions for the government to determine how frequently removal of rocks will occur. Annual hydrosurveys will be conducted to assess the quantity of rock blocked by the extended baffle block. The hydrosurveys will also assist in determining if execution of a rock removal contract is warranted to prevent rocks from overtopping the extended baffle block.

Figure 2-2. Conceptual "Baffle" Alternative.

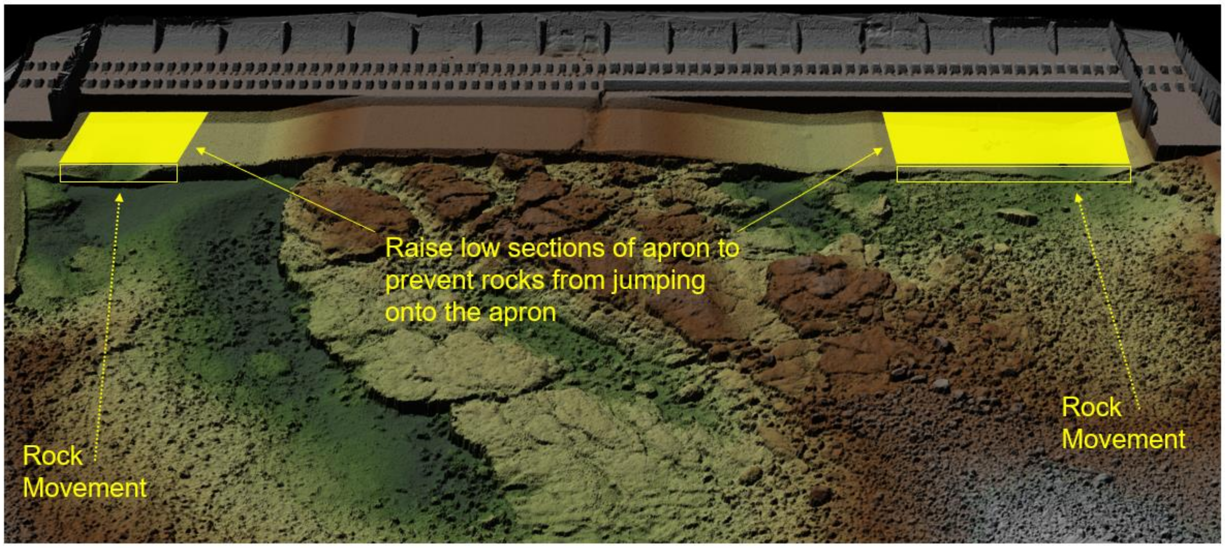


2.3.4 ALT 4: Raise Apron

This alternative (also referred to as "Raise") consists of raising the low spots on the north and south ends of the apron sufficiently high so that rocks are unable to neither move onto nor make their way up the apron (Figure 2-3). This alternative would require a large quantity of concrete to be placed underwater. Total quantity of concrete is expected to greater than the barrier alternative. Underwater form work will be required in water depths up to 60 to 70 feet. Divers may be required to anchor the forms to the concrete apron. The south side of the apron has a wider low spot which will require more concrete to raise than the north side. How high to raise the apron and hence the number of lifts, will have to be determined. It should be raised enough that if rocks accumulate downstream of the raised spots, they are not able to pile up and maneuver onto the apron.

Attached to this alternative is the replacement of the existing non-routine maintenance contract framework for rock removal with a long-term, recurring maintenance contract framework for rock removal. The new contract framework will include provisions for the government to determine how frequently removal of rocks will occur. Annual hydrosurveys will be conducted to assess the quantity of rock blocked by the raised apron locations. The hydrosurveys will also assist in determining if execution of a rock removal contract is warranted to prevent rocks from overtopping the raised apron locations.

Figure 2-3. Conceptual "Raise" Alternative.



2.4 PROS & CONS

The pros and cons of each alternative are listed below. It is estimated that the baffle alternative will be the cheapest, the raised apron alternative will be the most expensive, and the barrier alternative will be in between.

2.4.1 Barrier

Pros:

- Relatively easy to construct
  - No underwater form work
  - Mass concrete inside of a prefabricated concrete block could be tremied in the wet
  - Can be completed in one IWWW
- Relatively easy to inspect (remotely operated vehicle (ROV) / diver / hydrosurvey)
- The shape of the barrier is flexible
- Located deep enough in water column to be fish-friendly
- Relatively long life expectancy since barrier does not see extreme turbulence

Cons:

- Sealing gaps around or underneath might be challenging
- May require divers to do final assembly underwater
- Might be difficult to repair or move should it get damaged or dislodged
- Rocks collect at two different locations (North and South)

- Anchoring has been abandoned due to geotechnical concerns and higher expected cost.

2.4.2 Extended Continuous Baffle Block

Pros:

- Relatively easy to inspect (ROV/diver/hydrosurvey)
- Construction can be completed in one IWWW
- Rocks collect at one location, rather than two
- Fish friendly since continuous baffle block already in place

Cons:

- Shorter life expectancy than barrier due to seeing extreme turbulence like other baffle blocks
- Somewhat complex construction
    - Concrete to be poured in the wet
    - Intricate concrete forms require divers to position and anchor
    - Conduct work in water 30 – 40 feet in depth

2.4.3 Raise Apron

Pros:

- Simple design
- Fish-friendly since deep enough in water column
- Relatively easy to inspect (ROV/diver/hydrosurvey)
- Long life expectancy since deep in the water column

Cons:

- Expensive due to large quantity of concrete to pour
- Require form work in water 60 – 70 feet in depth
- Forms will have to be anchored to the existing concrete apron underwater by divers
- Multiple concrete lifts may be required to obtain required height
- May take multiple IWWWs to complete construction
- Rocks accumulate at two different locations

SECTION 3 - EVALUATED ALTERNATIVES

The approach that was undertaken for alternative evaluation was one that prioritized hydraulic functionality. Since hydraulics are the root cause of the rock movement, hydraulic modeling was utilized as the first evaluation tool to establish which alternatives are feasible and which are not.

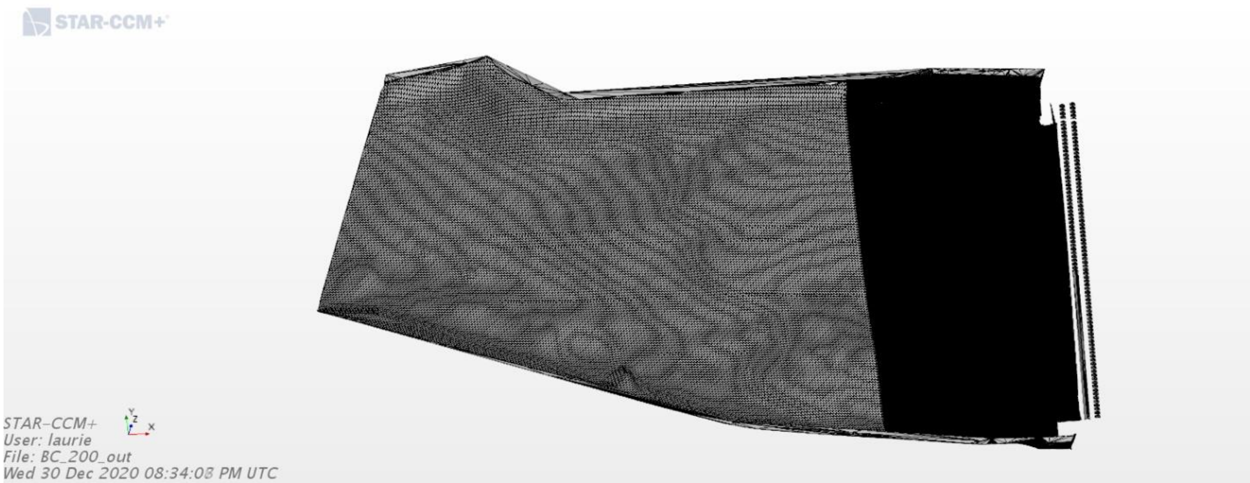
All evaluated alternatives focus on the south side of the spillway. The south side of the spillway lends the least resistance to rock movement and as such, a larger quantity of rocks move onto the apron and up into the stilling basin via the southern route. This is due to the shape and elevation of the apron's southern low spot. The southern low spot is flatter and wider than the apron's northern low spot, thereby making it more accessible for loose material to move onto the apron and travel to the stilling basin.

Note that though the modeling focused only on the southern half of the spillway, the preferred alternative will account for the entirety of the spillway, as some rocks do travel up the northern half of the apron.

3.1 NUMERICAL MODELING

The 3D Computational Fluid Dynamics (CFD) model originally developed by Pacific Northwest National Laboratory (PNNL-20056, Rakowski et. al. 2010) was modified to evaluate the flow conditions in around the Bonneville spillway. In particular, the spillway channel was extracted from the larger tailrace model, see Figure 3-1. See Appendix A for the full Modeling Report.

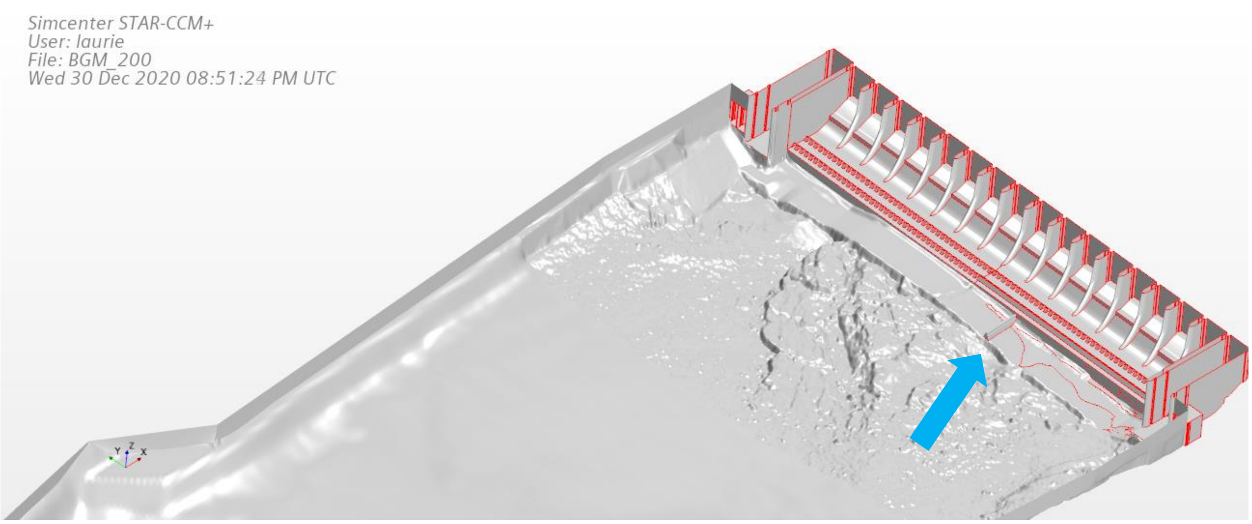
Figure 3-1. Truncated Spillway Channel Model.



3.1.1 Set-Up

To develop CFD metrics to evaluate the alternatives, one first need to understand the geometry of the spillway and the previous physical model results of rock movement. The physical model showed that rocks moved on to the ramp at bays 16/17 and moved up the ramp to bays 9/10 where they then maneuvered into the stilling basin. The ramp slopes up from bays 16/17 to bays 9/10. At bays 9/10, the ramp is just a little bit lower than the stilling basin invert as opposed to at bay 16/17, where there is a significant difference between the ramp and the stilling basin inverts. The other difference is the flow deflector elevations. Bays 4 through 15 contain flow deflectors set at 14 feet MSL and bays 1 through 3 and 16 through 18 contain flow deflectors set at 7 feet MSL (see Figure 3-2). Figure 3-2 shows the barrier alternative on the south side of the apron/ramp.

Figure 3-2. Spillway Model including Barrier Alternative.



The CFD model was previously validated by PNNL. Model runs were made with the truncated model to verify that flow conditions in the stilling basin were the same. All alternatives (Figures 3-2 through 3-4) were incorporated into the truncated model with no structural alternatives in place, referred to in this document as “clean.”

Figure 3-3. Continuous Baffle Block Alternative.

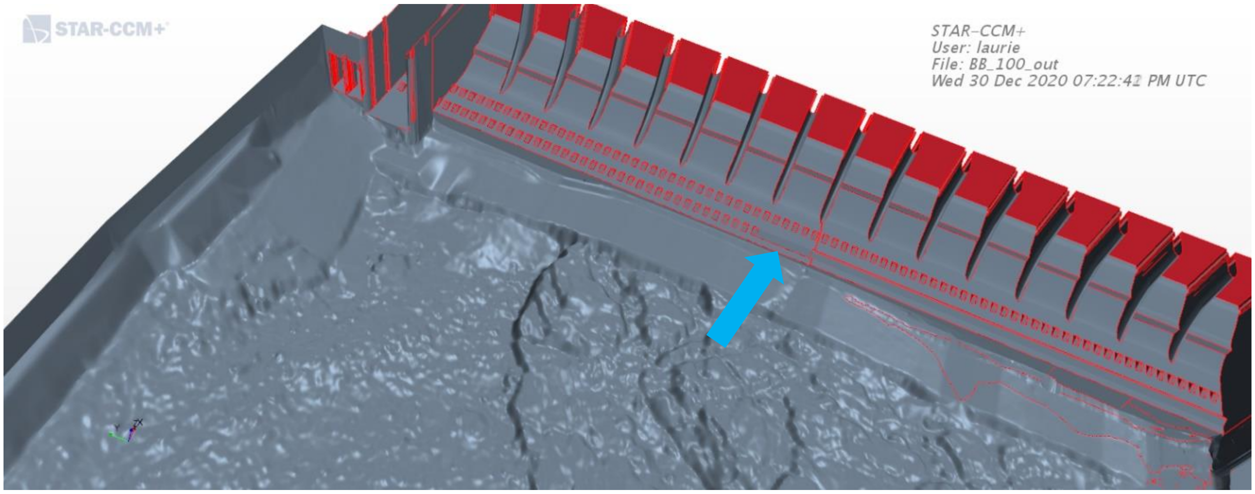
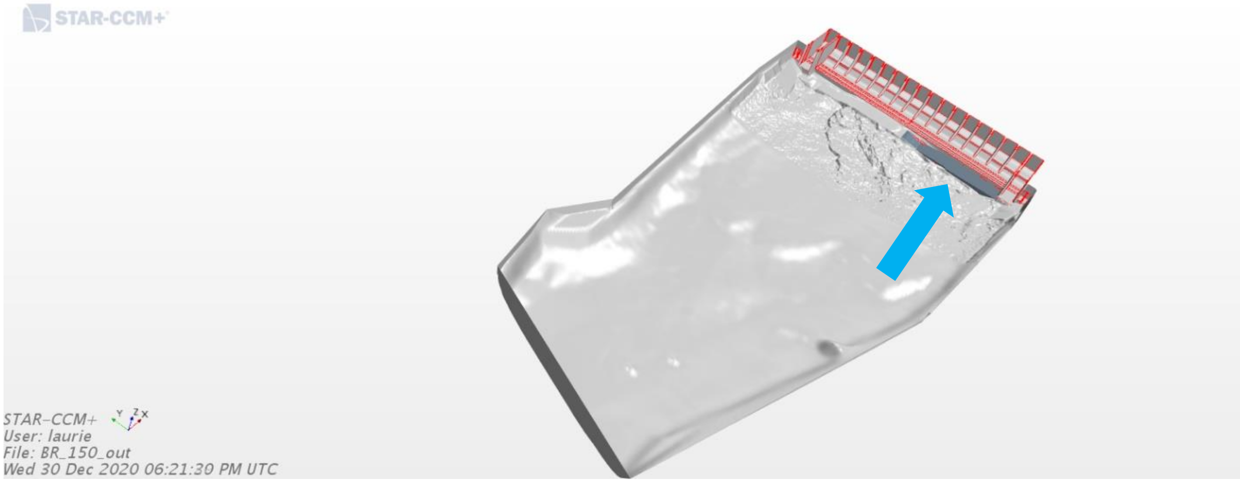


Figure 3-4. Raised Apron Alternative.



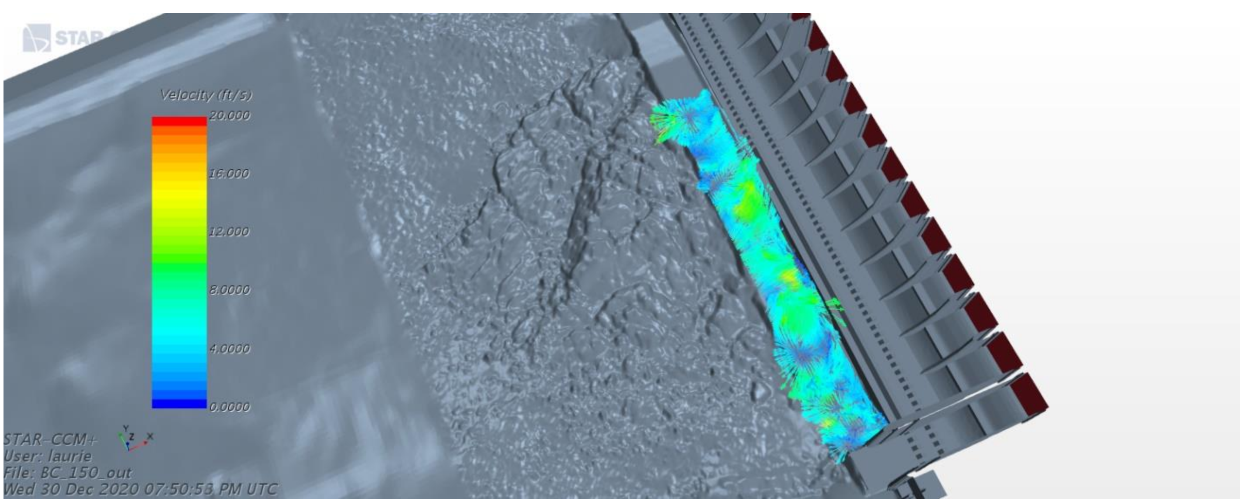
3.1.2 Runs

Model runs were made for the Clean, Barrier, Continuous Baffle Block, and Raised Apron at spillway flows of 200, 150, 125, and 100 kcfs.

3.1.3 Results

The CFD modeling shows that if the rocks can move up as far as the bay 9/10 discontinuity on the ramp, sufficient energy exist to allow rocks to maneuver into the stilling basin at spill volumes of 125 kcfs and higher. See Appendix A for the full Modeling Report. For the clean model runs and the extended baffle block runs, it is uncertain if sufficient energy exists at 125 kcfs to move rocks all the way up to the bay 9/10 discontinuity. However, there is sufficient energy at 150 kcfs and 200 kcfs (see Figure 3-5).

Figure 3-5. Velocity on Apron/Ramp, 200 kcfs.



The CFD modeling shows that preventing rocks from moving up the ramp to the bay 9/10 discontinuity is the most effective way to prevent rocks from entering the stilling basin. The barrier alternative meets this requirement as does the raised ramp.

The raised ramp alternative was found to increase shoreline velocities along Bradford Island and thereby could increase risk of additional erosion at Bradford Island Fish Ladder. Due to the Shoreline Velocities constraint (section 1.6.2.8), the raised ramp alternative was eliminated from evaluation.

The barrier alternative is the only hydraulically viable alternative tested in the numerical modeling effort.

3.2 PHYSICAL MODELING

The U.S. Army Engineer Research and Development Center (ERDC), Coastal and Hydraulics Laboratory (CHL) engaged with the USACE Portland District (NWP) to evaluate the performance of the barrier and extended baffle block alternatives on the existing 1:55 Bonneville Spillway Tailrace Physical Model (Figure 3-6), located at ERDC in Vicksburg, Mississippi.

Due to the COVID-19 pandemic, NWP personnel were unable to travel to ERDC and witness the physical modeling in-person. ERDC technicians installed livestream capabilities so that NWP personnel could virtually witness each test in real time. In addition to the livestream, videos and photos were taken above and under water during each test. These data were used to assess the movement and final position of two piles of green- and red-colored rocks.

Scaled versions of both structural alternatives were manufactured and tested against an array of flow conditions. A benchmark alternative was also evaluated by applying the same flow conditions with the physical model as is, allowing the comparison of results and performance evaluation of each structural alternative. Physical model testing occurred November 6-17, 2020. See Appendix A for the full Modeling Report.

Figure 3-6. Existing 1:55 Scale Bonneville Spillway Physical Model.

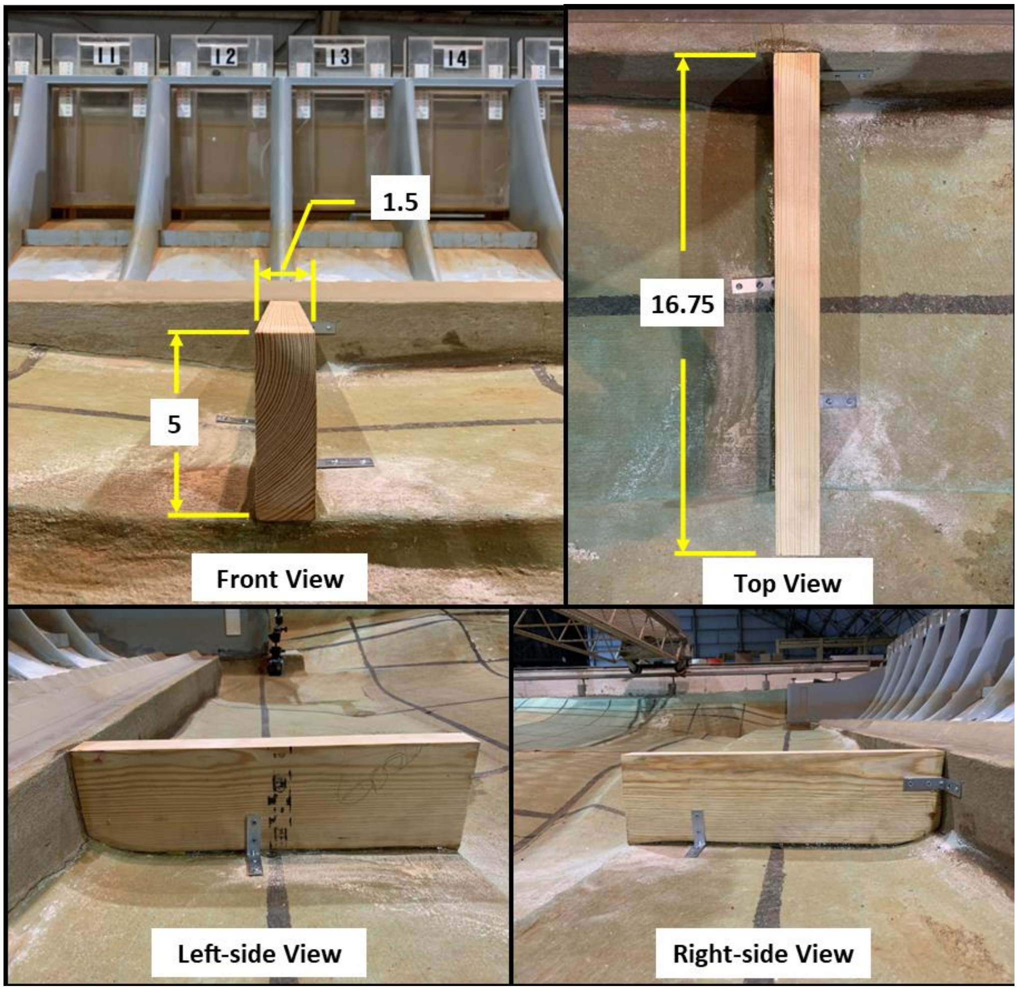
3.2.1 Set-Up

After cleanup of the Bonneville Spillway model, multiple cracks were identified across the spillway. To get the model back to specifications, the existing filler was removed and the area thoroughly cleaned before being repaired by applying a resin compound. A heavy-duty clear silicone sealant was used to seal other cracks in the structure.

Three physical model alternatives were tested. They consisted of two structural alternatives (barrier and extended baffle block) and a clean/baseline alternative with no structural modifications in place.

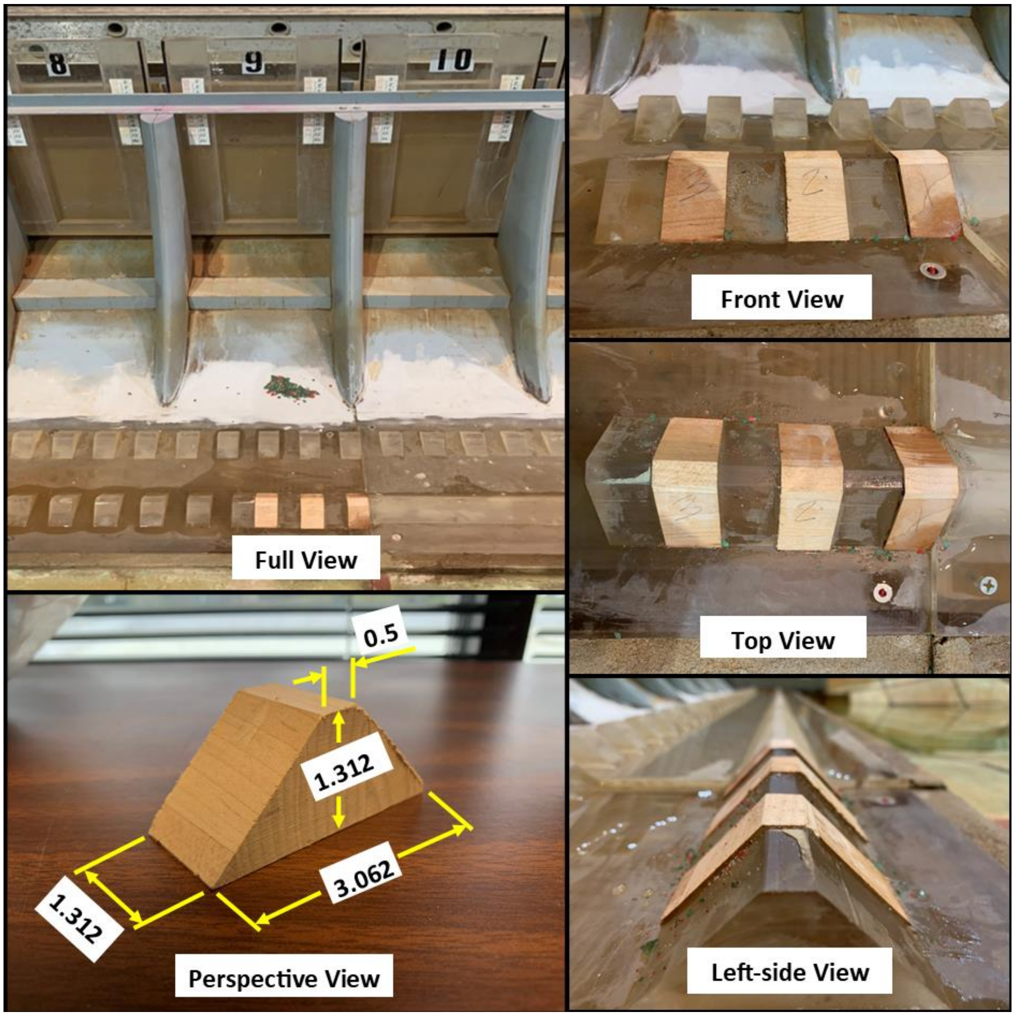
The barrier alternative was modeled with a rectangular piece of wood measuring 1.5 inches wide x 16.75 inches long x 5 inches high. This barrier model (Figure 3-7) was modified so that the base of the barrier matched the contour of the apron. The barrier was limited in height to the same elevation of the stilling basin's floor. Three L-shaped steel braces held the barrier model in place. A layer of clear heavy duty and waterproof sealant was applied to the bottom and along the side of the model to prevent the movement of water and debris underneath it. The barrier was placed in line with pier 12, between bays 12 and 13.

Figure 3-7. Barrier Model (dimensions in inches).



Modeling the extended baffle block alternative consisted of filling the area between baffle blocks near the center of the stilling basin (in front of bay 9). This created an extension of the existing continuous baffle block (bays 10 to 17) northward to encase baffles downstream of bay 9. This modification was intended to prevent rocks that did get onto the apron from maneuvering into the stilling basin. In the model, three wooden pieces represented this alternative, which are of same size as the existing baffles. As illustrated in Figure 3-8, they have a cross-sectional shape of an isosceles trapezoid with top and bottom widths of 0.5 and 3.062 inches, respectively. The height and length of the models each measured 1.312 inches. Nothing was used to hold the blocks in place since they fit tightly between the existing baffle blocks.

Figure 3-8. Extended Baffle Model (dimensions in inches).



NWP determined all camera positions (Figure 3-9 and Figure 3-10) to capture views where greater rock transport was expected, as has been observed in previous model visits. These locations were not changed during the study.

Figure 3-9. Overhead view of physical model and camera setup.

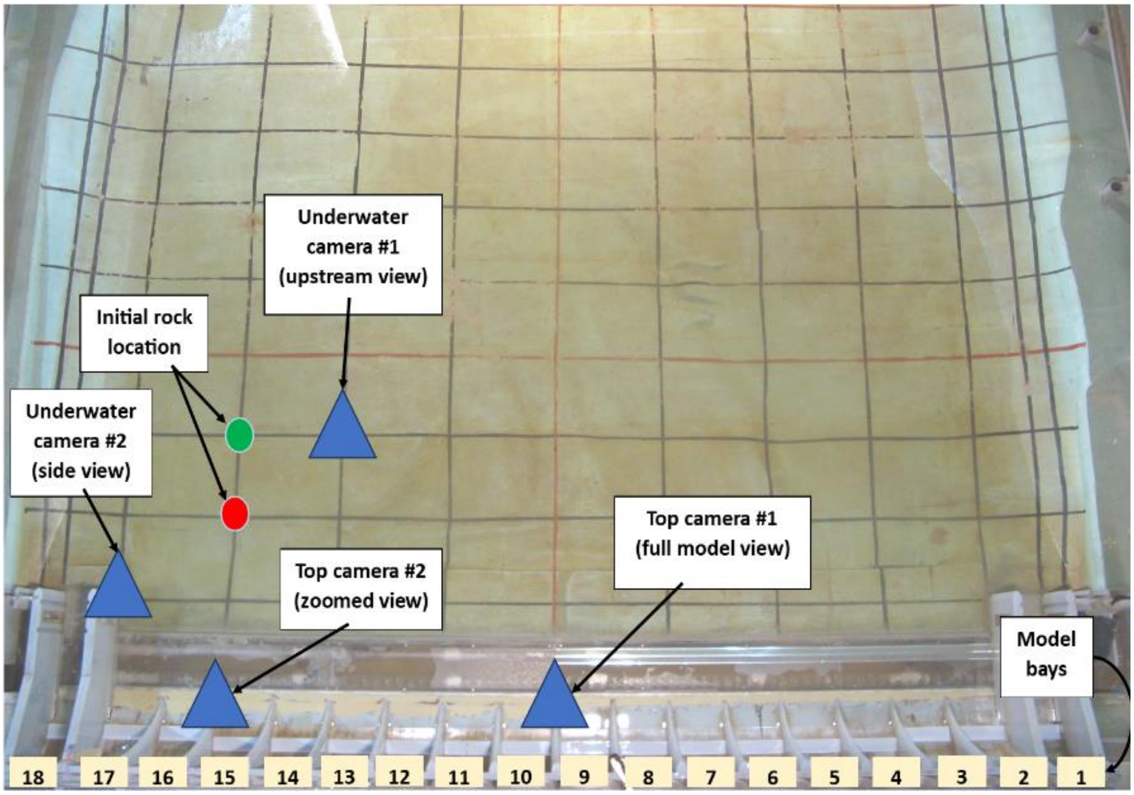
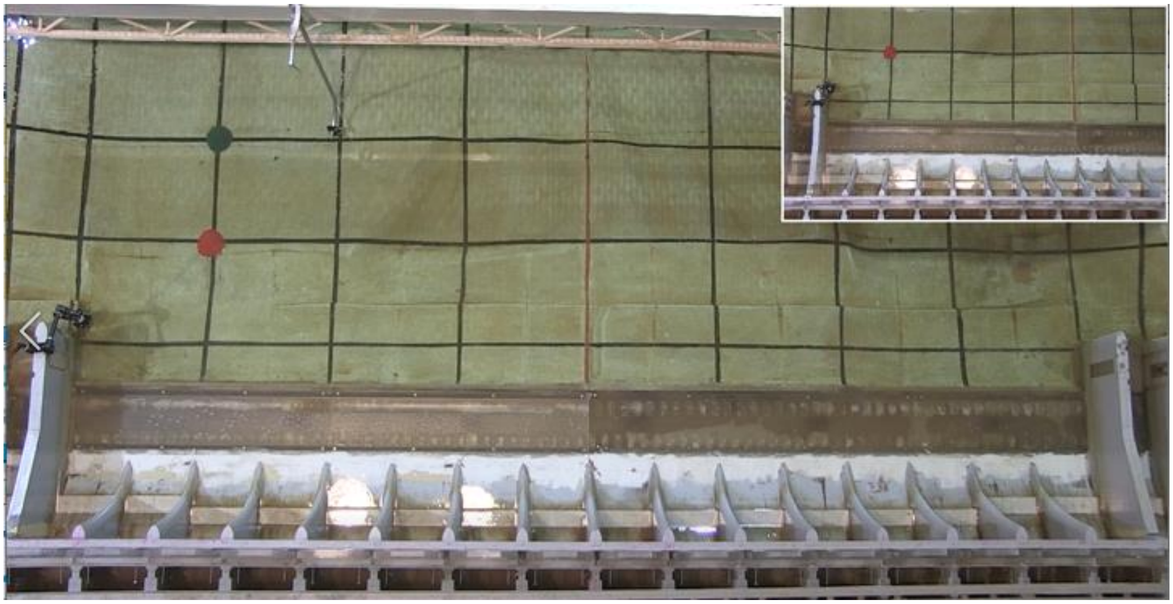


Figure 3-10. Overhead view of physical model from cameras.



Note: Larger image shows view from top camera #1; inset shows zoomed view from top camera #2. Initial rock piles are shows for illustrative purposes.

3.2.2 Runs

The physical modeling testing consisted of thirteen tests, each with a specific flow pattern for the three model alternatives (Table 3-1). A forebay elevation of 74 feet was kept constant for all tests. The tailwater elevation and model discharge were set before every test. A vertical gate controlled the tailwater elevation and model discharge was controlled with a gate valve. Model stability took 15 to 20 minutes for each test.

150 kcfs was tested because it is deemed the "threshold" for rock movement. 200 kcfs was tested to verify that rocks still moved at higher spills and because Bonneville often spills at or above that flow per season. 500 kcfs was tested because it is the design flood stipulated. Note a 500 kcfs test was attempted but was abandoned after 30 minutes due to model instability. It was decided that all further 500 kcfs tests would not be attempted.

Table 3-1. Schedule and test conditions for the physical modeling study.

Test No.	Condition	Flow pattern (kcfs)	Tailwater elevation (feet MSL)	Date	Duration (hours)	Remarks
0	Wet run	-	-	6-Nov	8	
1	Clean	200	25	9-Nov	3	
2	Clean	200	20	9-Nov	3	
3	Clean	200	30	10-Nov	3	
4	Clean	150	23	10-Nov	3	
5	Baffle	200	25	12-Nov	3	
6	Baffle	150	23	12-Nov	2	
7	Baffle	500	38	13-Nov	0.5	1
8	Clean	200	25	13-Nov	0.5	2
9	Barrier	200	25	16-Nov	3	
10	Barrier	150	23	17-Nov	3	
11	Barrier + Flushing	25	12	17-Nov	1.5	3
12	Clean	500	38	-	-	4
13	Barrier	500	38	-	-	4

Remarks:
1: Cancelled due to model instability.
2: Demonstration for NWP and partner agencies.
3: Consisted of five, 10-min runs. Used gates 4 and 5 to stabilize the model. Used gates 13 - 17 to perform flushing spill operation.
4: Not performed due to model instability.

Each test of this study consisted of twelve general steps presented in Table 3-2. The initial placement of the rocks downstream of bays 15/16 correspond to the approximate location of loose material within the Bonneville spillway identified by hydrosurveys. Each test was run for approximately three hours or until all rocks moved completely to the apron. The underwater cameras were used for monitoring the rock movement and final position, while the overhead cameras recorded the entirety of the test. A test concluded with the closure of the water intake and documentation of the final position of rocks. The model was then prepared for the next test.

Table 3-2. Physical modeling procedure per test.

Step	Description
1	Clean model and basin of debris and rocks
2	Spot check deflector elevations
3	Set spillway gate openings
4	Set structural alternative
5	Take picture from overhead camera
6	Establish flow conditions – forebay and tailwater stabilized
7	Take picture from overhead camera
8	Put 2 cups of red and green rocks in model
9	Start 3-hour run (22.25-hour prototype) and video recording. Monitor rock movement with underwater camera during first and last hour.
10	Shut flow down.
11	Take picture from overhead camera.
12	Document and take pictures of rocks new location.

3.2.3 Results

Under identical flow conditions (200 kcfs spillway flow & 25 feet tailwater elevation), the barrier alternative outperformed the extended baffle block alternative and the baseline alternative.

The baseline alternative, with no structural modifications in place, allowed a significant quantity of rock to enter the stilling basin and accumulate largely in bay 9. At the conclusion of test 1, the rocks had organized into three groups with locations shown in Figure 3-11. A portion of the green rocks moved to the left of their original position (inset A), mainly due to an eddy with counterclockwise circulation centered in that same area. The second group (inset B) is a mixed-colored rock pile siting in a contoured low spot downstream of the apron in line with bays 12/13. The low spot was named the “pit” and will be referred to as such for the continuation of this report. While few rocks were scattered on the apron (inset B and C), a third group of red and green rocks was found in front of bay 9 (inset D). In addition, traces of both rocks were found at the spillway in front of bays 3, 8, 16 and 17.

By visual inspection, the extended baffle block alternative allowed the same quantity of rock, if not more, into the stilling basin, again accumulating largely in bay 9 (Figure 3-12). Rocks of both colors were also located in front of bays 8, 10, 11, 14 and 16. More rocks moved onto the apron in front of bays 10 to 15. A few red rocks also ended behind the wooden baffles blocks in the stilling basin. Not many rocks were found in the pit. The route the rocks took to bypass the extended baffle block was not identified.

Figure 3-11. Final rock positions at the conclusion of test 1.

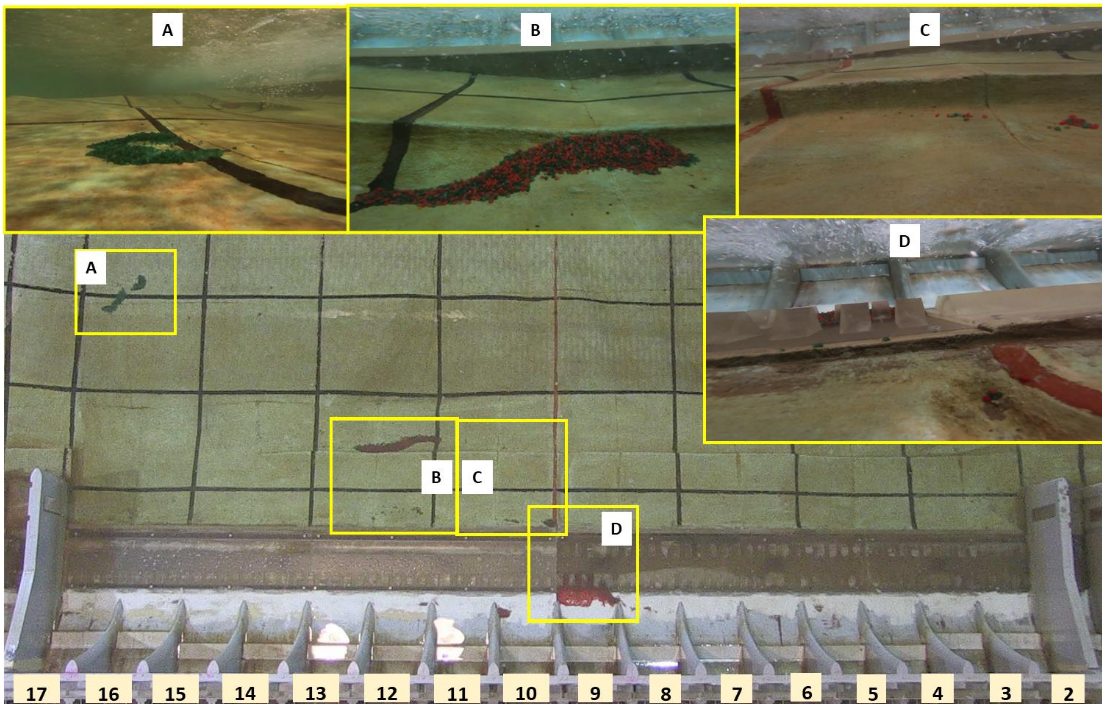
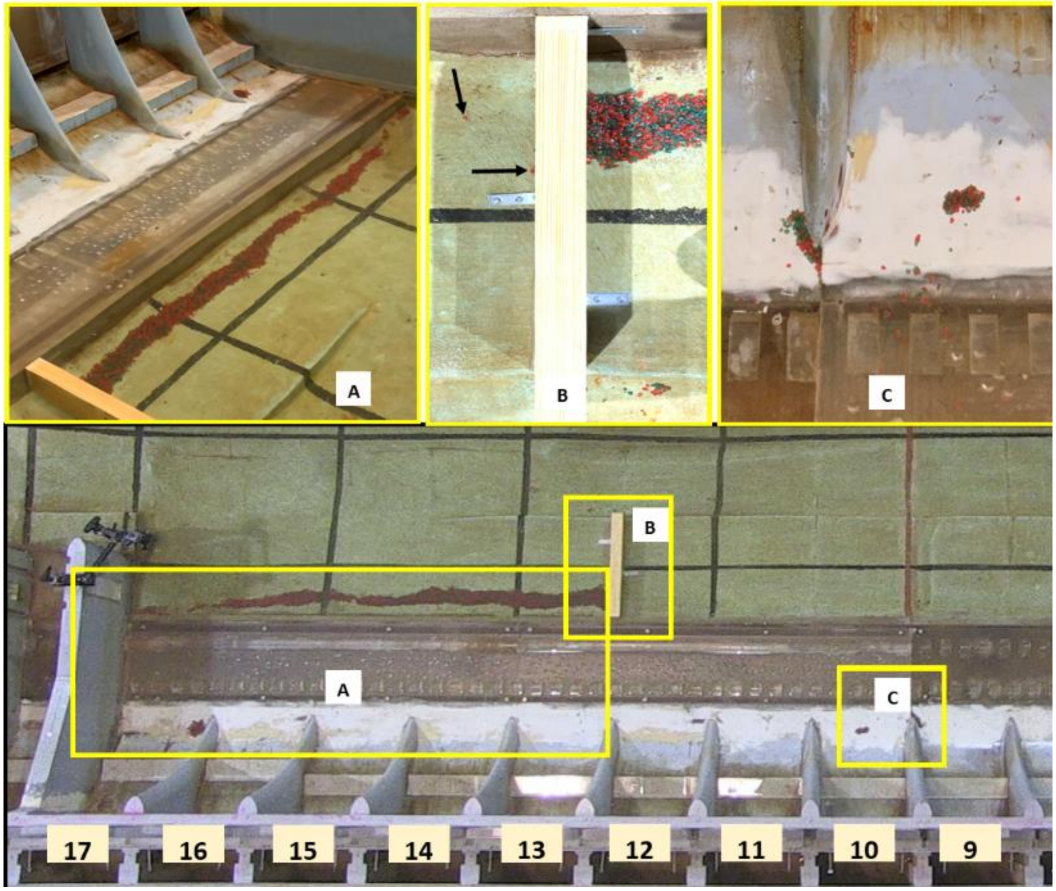


Figure 3-12. Final rock positions at the conclusion of test 5.

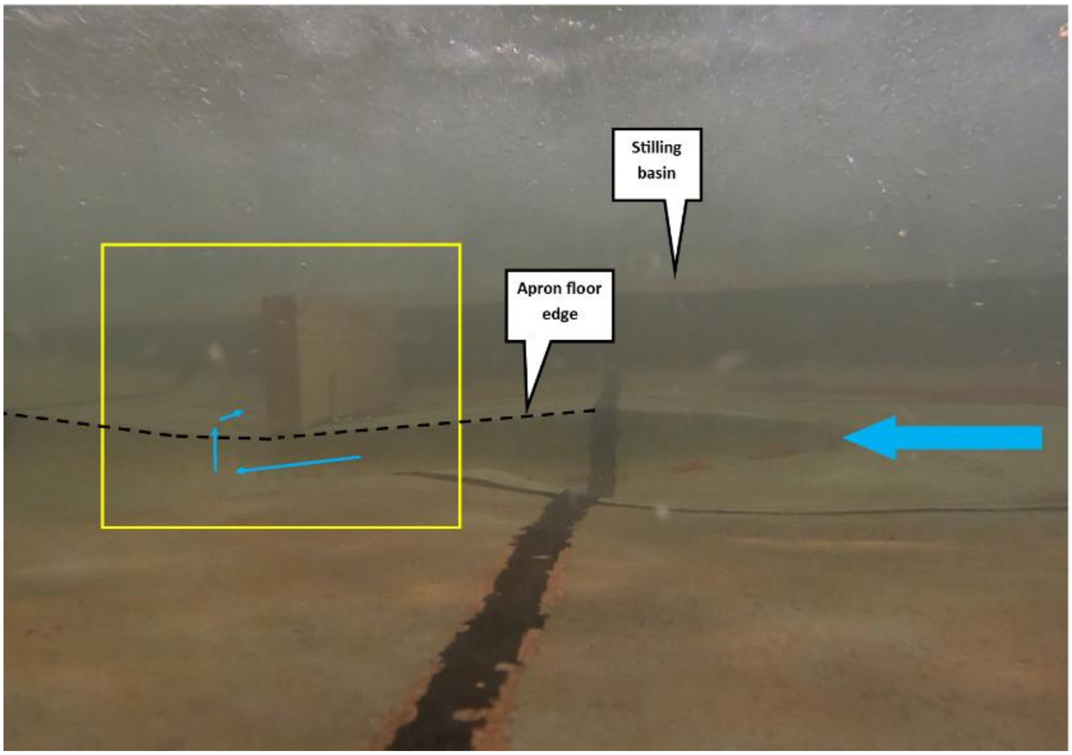
By visual inspection, the barrier alternative allowed far fewer rocks to migrate into the stilling basin (Figure 3-13). Most of the rocks – of both colors – were spread on the apron from bays 13 to 17 (inset A). A few rocks of both colors entered the stilling basin and were found in bays 9 and 10 (inset C), and bay 17. Only two rocks were found on the up-ramp side of the barrier (inset B).

Figure 3-13. Final rock position at the conclusion of test 9.



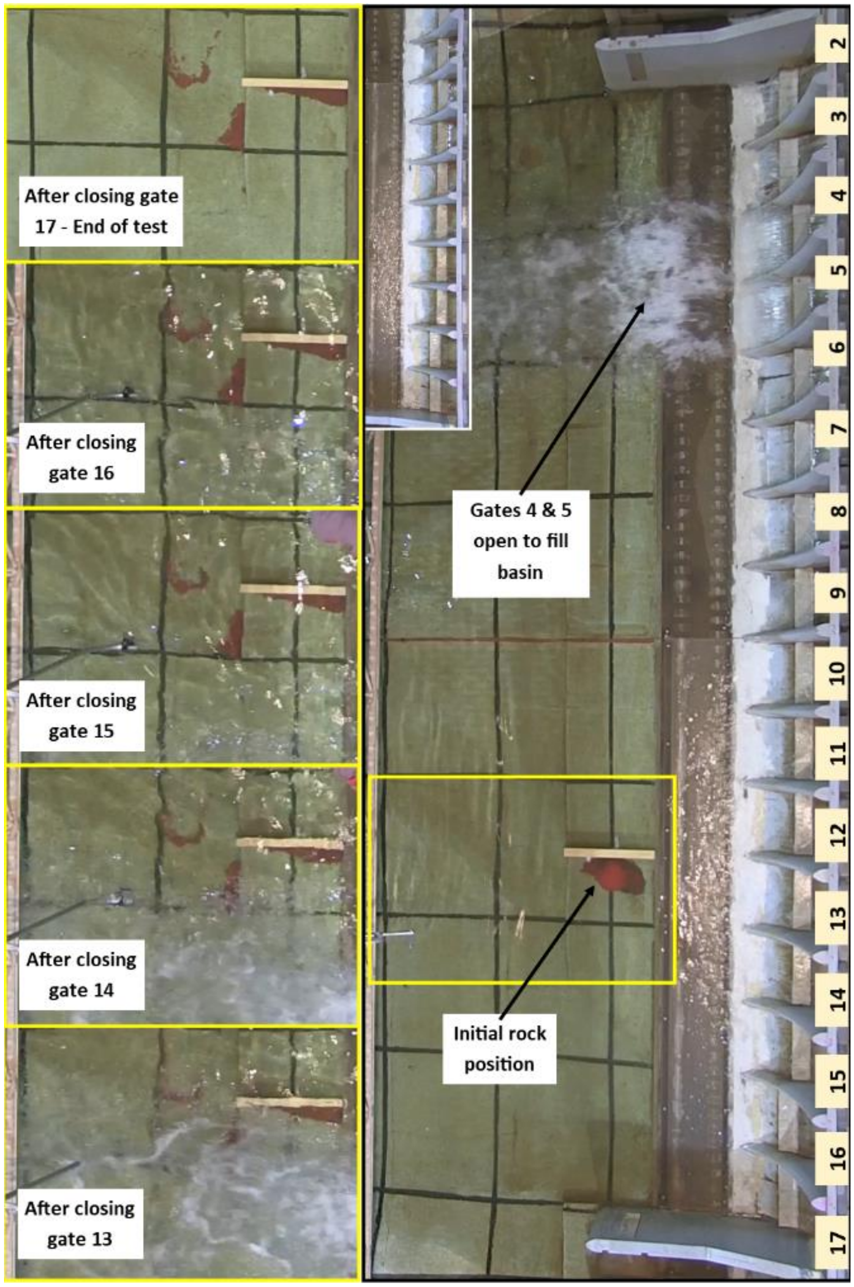
The results of test 9 were perplexing as there seemed to be no method by which the rocks could bypass the barrier. Upon closer inspection of the underwater video taken during the test, the bypass route was identified. Rocks that found their way to the pit were able to maneuver onto the ramp on the opposite side of the barrier and continue to migrate into the stilling basin. The bypass route taken by the rocks is shown in Figure 3-14 with blue arrows. This bypass route is unable to occur in the prototype and will be discussed in section 3.3.

Figure 3-14. Barrier bypass route.



A flushing spill test was conducted with the barrier alternative in place (Figure 3-15) and was successfully able to flush most of the rocks placed next to the barrier off the apron. A portion of the rocks remained next to the barrier and against the stilling basin end sill.

Figure 3-15. Flushing spill, test 11.



3.3 BARRIER BYPASS ROUTE

During physical modeling, the barrier was tested at a location on the apron in line with pier 12, between bays 12 and 13 (Figure 3-7). At the conclusion of test 9, it was determined that most of the rock movement was stopped by the barrier. Underwater video showed that a few rocks that found their way to the pit were able to move onto the apron on the opposite side of the barrier (the bay 12 side) and continue to migrate into the stilling basin.

Note the bypass route identified pertains to rocks that moved from their initial locations (on the bedrock in front of bays 15/16) directly to the pit, without moving onto the apron at all. Underwater video showed that rocks that moved onto the apron and were stopped by the barrier were not able to move off the apron, enter the pit, and then move back onto the apron on the other side of the barrier. The “bedrock to pit to apron” bypass route was shown to be present; the “apron to pit to apron” bypass route was not shown to be present.

The question remains: is the behavior seen in the physical model viable for the prototype? Comparing prototype bathymetry (Figures 1-2 and 3-2) against physical model bathymetry (Figure 3-6), it is obvious the physical model bathymetry is much smoother relative to the prototype. This is understandable since the physical model can’t accurately capture every rock outcropping and undulation of the prototype. It is believed that the bypass route was possible for the rocks to take in the physical model due to the smoothness of the bathymetry. Likewise, it is believed rocks that attempt this route in the prototype would get caught up in the roughness of the bathymetry and not be able to move to the pit and thus, onto the apron and into the stilling basin.

Plate 1 shows three latitudinal cross sections across the prototype bathymetry: at the downstream edge of the apron, 7 feet downstream from the edge of the apron, and 14 feet downstream from the edge of the apron. Plate 1 shows that at pier 12, the apron and bedrock bathymetry immediately downstream are close in elevation. However, at pier 11 (between bays 11 and 12) there is a distinct drop immediately downstream of the apron.

Plate 2 shows longitudinal cross sections through the centerlines of piers 8 through 12. Like Plate 1, Plate 2 also shows a relatively close elevation difference between apron and downstream bedrock bathymetry at pier 12 but a sizable elevation difference at pier 11.

To ensure that rocks will not move onto the apron even if they are to make their way to the pit, the barrier is recommended to be located where there is a sizable elevation difference between the apron and bedrock inverts. Thus, the barrier shall be located on the apron, in line with pier 11.

3.4 MODEL VALIDATION

Validation of the tools used in this effort is a combination of prototype results, physical model results and CFD results. The prototype has shown a tendency for the spillway hydraulics to move rocks onto the ramp at 125 kcfs and higher and a definite tendency of moving rocks into the stilling basin when spill is at 150 kcfs for any length of time (counted in hours). The CFD modeling done by PNNL confirmed how the rocks move.

This has been documented in previous modeling reports. The physical modeling confirmed specifically how the rocks move into the stilling basin.

For the modeling effort specific to this project, the strength of each tool was used. The CFD model was used to provide velocity information in the flow field. The CFD model showed how the velocity magnitude changes on the ramp between 100 kcfs and 150 kcfs spill. The CFD model also highlighted the velocity changes on the spillway shorelines for the raised ramp alternative, which could have negatively impacted the stability of the shorelines and led to the elimination of that alternative. The physical model allowed the PDT to place rocks in the spillway, watch their movement, and document their final locations at various flow rates and with various structural alternatives in place.

3.5 MODELING CONCLUSIONS

The numerical modeling concluded that preventing rocks from moving up the ramp to the bay 9/10 discontinuity is the most effective way to prevent rocks from entering the stilling basin. The barrier alternative meets this requirement as does the raised ramp; the baffle alternative does not. The raised ramp alternative was found to increase shoreline velocities along Bradford Island and thereby could increase risk of additional erosion at Bradford Island Fish Ladder. Due to the Shoreline Velocities constraint, the raised ramp alternative was eliminated from evaluation. The barrier alternative is the only hydraulically viable alternative modeled in CFD.

The physical modeling concluded that none of the alternatives fully prevented rock transport into the stilling basin. However, the barrier alternative outperformed the baffle alternative in terms of excluding rocks from entering the stilling basin.

Underwater video suggested the rocks that did bypass the barrier did so due to the smoothness of the physical model bathymetry and the small elevation difference between the bedrock and the apron at pier 12. After bathymetric analysis of the prototype, the PDT is confident the bypass route is extremely unlikely to occur in the prototype due to the roughness of the prototype bathymetry. Regardless, the barrier is recommended to be shifted north and be placed in line with pier 11 to maximize the elevation difference between the bedrock and the apron to prevent rocks from maneuvering onto the apron in the unlikely event that rocks migrate to the pit.

Numerical and physical hydraulic modeling indicated the barrier alternative (ALT 2) was the only hydraulically viable alternative and is thereby considered to be the preferred alternative.

SECTION 4 - PREFERRED ALTERNATIVE

The preferred alternative arrived to by the modeling efforts is ALT 2: Barrier. This section outlines the design and development of the preferred alternative.

4.1 BASIC MODEL OR DRAWINGS

See section 4.2.2 for preliminary structural drawings.

4.2 DISCIPLINE SPECIFIC CONSIDERATIONS

4.2.1 Hydraulic & Coastal Design

Discipline-specific considerations for Hydraulic & Coastal Design have been captured in previous sections.

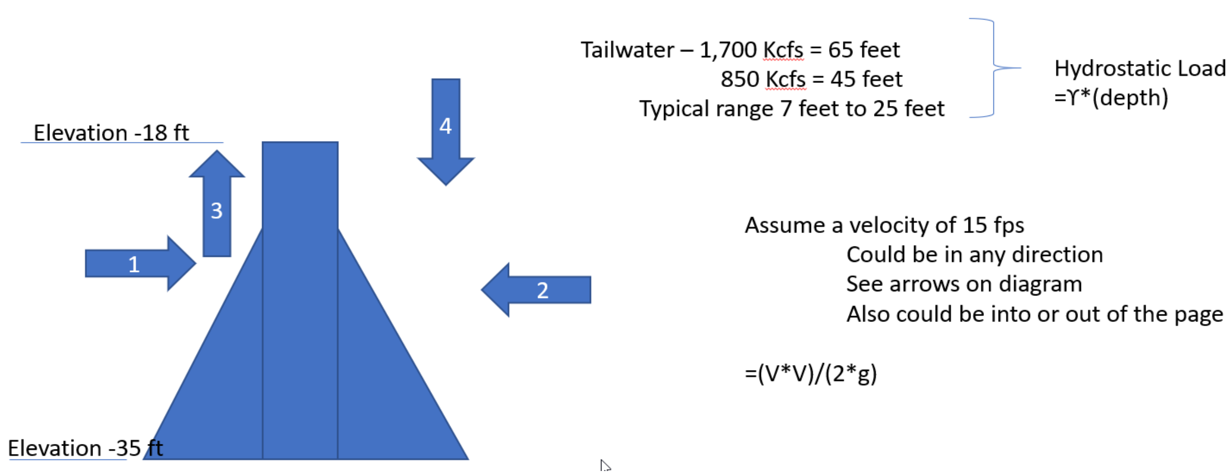
4.2.2 Structural Design

The structural design of the barrier underwent an iterative design process. The initial barrier design had floatation and seismic stability issues, which were addressed in the alternate barrier design.

4.2.2.1 *Initial Barrier Design*

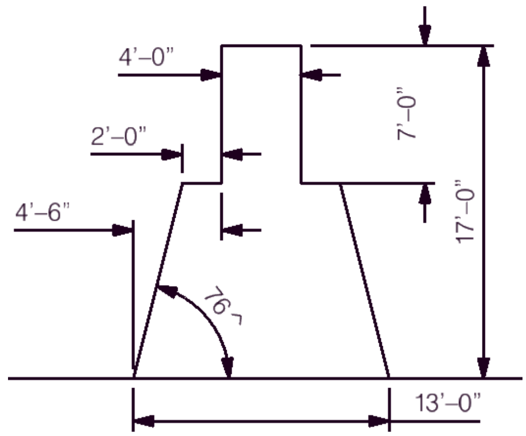
Numerical and physical modeling was utilized to evaluate the design of a reinforced concrete barrier which would be built on the apron of the Bonneville spillway. The barrier would be located in-line with the spillway pier between bays 11 and 12. The purpose of the barrier would be to prevent rocks from migrating into the stilling basin. The evaluated barrier concept and associated cross section appears in Figure 4-1.

Figure 4-1. Hydraulic modeling information provided.



The model dimensions were replicated in MicroStation to inform the structural evaluation of the barrier.

Figure 4-2. Dimensioned structural cross-section for evaluation.



The hydraulic modeling found that there was little to no structural loading from flowing water through all spill ranges. This means that the driving loads for evaluation are stability related gravity and seismic loads.

A stability analysis of the shape shown in Figure 4-2 identified that the structure contained insufficient mass to meet stability factors of safety for floatation. At normal maximum tailwater elevation 40 feet, the factor of safety for floatation was found to be 1.1 compared to a usual factor of safety of 1.3 as required in Engineering Manual (EM) 1110-2-2100. The factor of safety at flood loading (tailwater elevation 65 feet) was found to be 1.08 compared to an extreme factor of safety of 1.1 required in EM 1110-2-2100. Based on this analysis, it was determined that anchors would be required to meet floatation factors of safety.

Anchors were selected and designed to meet normal, unusual, and extreme loading conditions including seismic loading on the structure. The PDT met with Dam Safety personnel and determined that the 950-year seismic event would be the appropriate level of design. This results in a peak ground acceleration (PGA) of 0.27g. The average tailwater elevation of elevation 20 feet was considered the concurrent tailwater elevation for this level of seismic analysis.

The analysis showed that the structure as depicted in Figure 4-2 would require four (4) 3-inch diameter anchors spaced every 5 feet to provide sufficient resistive force for uplift and seismic loading. An evaluation of overlapping conical failure (pull out failure) of the rock mass foundation required the anchors to be drilled more than 100 feet into the foundation. Consultation with the geotechnical engineer identified that the rock beneath the spillway structure is more appropriately considered a heavily consolidated clay/weak rock material. Section 4.2.3 describes the foundation concerns. The concern with installation of anchors in this rock type is the high potential for consolidation of the foundation material and a loss of stress in the anchors. Additionally, these anchors will be subjected to cyclic loading with fluctuating tailwater and spill levels. This will subject the anchors to fatigue cycles that are likely to result in early failure of the anchors. For these reasons, the PDT has elected to not design anchors to stabilize the barrier.

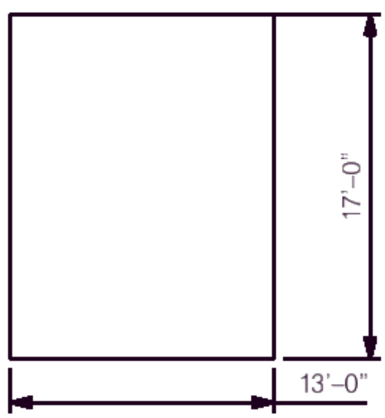
4.2.2.2 *Revised Barrier Design*

The initial evaluation of the barrier showed two key issues:

1. Floatation Stability factors of safety could not be met.
2. Seismic Stability factors of safety for overturning and sliding could not be met without anchors.

To increase the floatation stability of the structure, additional mass is required. A rectangular structure, with similar overall dimensions was evaluated. The rectangular cross-section shown in Figure 4-3 increases the cross-sectional area of the barrier from 133 square feet to 221 square feet. This results in an increase of 66% in the mass of the structure resisting uplift loads.

Figure 4-3. Rectangular cross-section.

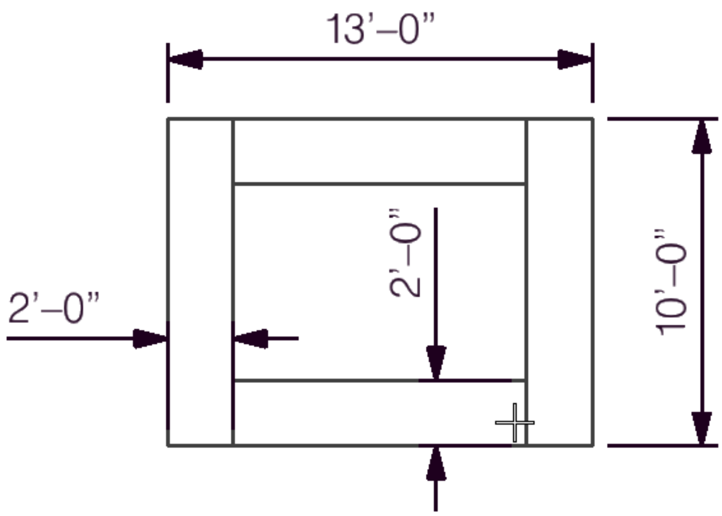


An evaluation of flotational stability for the structure pictured in Figure 4-3 shows that the factor of safety for normal loads (tailwater elevation 40 feet) is 1.32 with a required factor of safety of 1.3 for normal loading per EM 1110-2-2100. The factor of safety for extreme flood loading (tailwater elevation 75 feet) is a factor of safety of 1.22 with a required factor of safety of 1.1 for extreme loading per EM 1110-2-2100.

The rectangular structure shown in Figure 4-3 was evaluated for the 975-year seismic loading event with a PGA of 0.27g. The PDT goal was to prevent building a structure that would require repairs after a large earthquake event. The overturning factor of safety was found to be 0.996. The sliding factor of safety was found to be 0.05 with a required factor of safety for extreme loadings of 1.1 per EM 1110-2-2100. The barrier dimensions (width to height) as depicted in Figure 4-3 do not resist seismic sliding forces due to a lack of sufficient base slip resistance. Concrete keyways and rock anchors were evaluated as a method of preventing the structure from sliding in a seismic event. As previously addressed, the use of rock anchors is not considered acceptable by the PDT. The use of a rock keyway into the foundation would require the existing approximately 5-foot-thick apron to be cut out. The sliding force on the remaining slab would likely fail the slab resulting in additional repairs required after the earthquake event. Based on the potential for damage to the apron slab and a desire to avoid the use of anchors, the PDT has elected to allow the wall to move in a seismic event. The amount of wall movement that will occur has not been determined at this phase of the project. An OBE event earthquake would not result in wall sliding or movement. Wall repairs are anticipated after a larger seismic event.

The resulting barrier has a cross section of 221 square feet and a wall length of 80 feet. This results in a total volume of concrete required of approximately 655 cubic yards of concrete. The wall construction would likely consist of 24-inch-thick precast concrete cells. Assuming a 10-foot-long concrete box cells with 24-inch-thick walls and 17 feet tall results in a precast concrete cell that weighs 97 tons. Figure 4-4 shows the precast concrete cell dimensions. The cell would be constructed of precast panels anchored together to form the cell which would then be placed using a barge crane. The precast concrete cell would utilize 383 yards of 5000 pounds per square inch (psi) precast concrete. This results in a required 655-383 = 272 yards of tremie concrete.

Figure 4-4. Plan view of precast concrete cell for barrier construction.



The precast concrete cells would be set at the required location on the apron. The wall segments will be keyed together to ensure proper wall alignment. The bottom section of the wall is typically sealed off with sandbags on the outside of the wall to prevent tremie concrete from leaking out of the form. This is a diver operation for both bag placement and leak monitoring during tremie placement. The interior of this cell would then be filled with tremie concrete to the required elevation.

The top of the barriers will be set at approximately elevation -18 feet. The minimum historic tailwater (between 1974 – 1999) was 7 feet, yielding 25 feet of water between the top of the barriers and the water's surface. This does not restrict access by vessels to the area due to shallowness because the stilling basin floor is also at -18 feet and the baffle blocks (continuous and individual) rise to an even higher elevation. Even with the barriers in place, the limiting structure for under-keel clearance in the spillway is the baffle blocks.

4.2.3 Geotechnical Design

No new geotechnical investigations were conducted for this project. There is already a wealth of geological/geotechnical information collected and published from original design and construction of the spillway and even more modern information collected for the design of the second powerhouse which is a large structure founded on the same geologic unit. Total past investigations spanned years of explorations consisting of 500 to 1000 borings, extensive laboratory testing, construction observations, and post-construction performance observations. Only the summaries of this information could be reviewed for this project. The most important geologic/geotechnical aspects that directly impacts the decision-making process are summarized here.

4.2.3.1 *Foundation Rock Description.*

The spillway and second powerhouse are founded on sedimentary rock. Materials comprising the bedrock were derived from local volcanic sources.

Approximately 2/3 of the clastic materials which make up this formation were originally deposited as volcanic glass. The remaining 1/3 was predominantly lithic fragments. Materials were transported by water and/or slurries where they were deposited probably at the distal edge of an ancient volcano(es) on slopes or lakes. Stratification is described as extremely varied and discontinuous. Bedding thicknesses range from less than 0.1 inch to greater than 20 feet but most of the unit is composed of beds less than 3 feet thick. Bedding contacts are variable ranging from sharp distinct to gradual and indistinct. Many of the beds were reported to show scour and fill. From the second powerhouse, only major conglomerate beds can be traced across the site.

Volcanic glass is unstable and it has completely devitrified to form new, replacement minerals predominantly clays – montmorillonite (>85%), kaolinite and halloysite (<5%), zeolites (10%), and minor amount of other minerals (5%). These clays were formed from insitu chemical processes and have engineering properties significantly different than normal clays which were deposited in lakes. They are best classified as weak-rock.

Sedimentary beds do not form large continuous sheets but form multitudes of thinner discontinuous beds. Unlike most marine sedimentary rock, this material was subjected to hot fluid circulation from the nearby volcanoes when they were being buried and consolidated. This combination of hot fluids, unstable volcanic glass and other unstable mineralogy led to the volcanic glass completely devitrifying and other minerals being altered. This clay alteration has significantly weakened the rock. The foundation rock now has geotechnical engineering properties between that of a true clay deposit and unaltered sedimentary rock. These foundation rocks should be considered weak-rock.

Primary rock type descriptions for the Second Powerhouse Foundation Report are as follows. These descriptions have been shortened.

Mudstones (bentonitic or claystone is also commonly used in earlier site studies) are described as massive, unstratified rock which are very fine-grained aphanitic with little or no texture, fabric, or other relict features. The overall strength of the mudstones is controlled by the aphanitic matrix materials which are predominantly autogenetic clay and zeolite minerals derived from the alteration of the original volcanic glass constituents. Field testing of cores show crater quality; this equates to an approximate intact unconfined compressive strength of 1,000 to 3,000 psi. See Williamson, et.al., ASTM STP 984.

Mudstones can be scratched with a fingernail. Knife cuts usually create smooth, shiny to semi-shiny surfaces indicative of the high clay content. Sand and gravel sized clasts are locally present but only as minor constituents. The overall strength of the mudstones is controlled by the aphanitic matrix materials which are predominantly autogenetic clay and zeolite minerals derived from the alteration of the original volcanic glass constituents. Mudstones form massive beds between 3 and 20 feet in thickness. The fracture intensity of mudstones ranges from slightly fractured to intensely fractured. Slickensided joints are common in mudstones. The mudstones are likely to have originated as volcanic mudflows which originally consisted of dominantly volcanic glass shards.

A minor variation but with significant engineering importance is a green clay-like material which forms a concordant bed. This material is known to be weak and is often the root cause of geotechnical problems (slides, over-excavations, foundation issues).

The sandstone/siltstone unit consists of stratified rocks which range from aphanitic to medium-grained. They are composed primarily (> 50%) of sand and/or silt sized particles. They are generally gray to grayish green in color and [field testing of cores] crater to dent quality (this equates to an approximate intact unconfined compressive strength of 1,000 to 8,000 psi). About half of this group can be scratched with a fingernail, the other half cannot. The framework of these rocks contains lithic and crystal particles rather than vitric, and consequently have not been altered. The sandstones/siltstones range from matrix supported to framework supported (which probably accounts for the range in hardness). Matrix materials generally consist of the same altered materials which make up the mudstones. Sandstones and siltstones which are matrix supported have a high susceptibly to slaking. Some of the sandstones contain no matrix.

Conglomerates (agglomerates in older reports) consist of rounded, gravel sized or larger clasts within a sandstone or claystone matrix. The conglomerates are generally grayish in color. The clasts consist of hard unaltered lithics. Both framework supported (ortho-) and matrix supported (para-) conglomerates (or agglomerates as used by Holdredge, 1937) are present but not differentiated.

Proportion of rock types were made during the second powerhouse design and construction. Tabulated values for the major rock types found in the spillway may be found in Figure 4-5.

Figure 4-5. Major rock types found in the Bonneville spillway.

PERCENTAGE OF ROCK TYPES

Rock Type:	Mudstone	Sandstone/ Siltstone	Conglomerate
AREA			
SPILLWAY	50%	20%	30%
2PH			
design	20%	65%	5%
actual	50%	40%	10%

All strata have been deformed and now dip about 10 to 15 degrees upstream and into the left abutment. Foundation rock has also been faulted and sheared which has further weakened the rock. Faults and shears are shown on spillway foundation map. In addition, it is suspected that bedrock weakened by faulting and shearing has been eroded out forming linear trenches shown in the downstream bathymetry.

4.2.3.2 *Geoengineering Material Properties.*

Geoengineering material properties were extensively studied for the second powerhouse. Significant parameters are:

- Bearing Capacity for the Second Powerhouse.

"The overall adopted bearing capacity for the foundation rock is 33 KSF. This value is well within the range of structural loads exerted on the same foundation rock unit for the spillway dam foundation. Loading by the spillway dam ranges from 22 to 60 KSF. The 40-year performance record of this structure with no adverse cracking or excessive settlement is testimony to the validity of the adopted value."

- Modulus of Deformation for the Second Powerhouse

"During design, modulus of deformation was estimated to be in the range of 300,000 to 450,000 psi." "During the construction phase of the work, ongoing foundation explorations, i.e., Menard pressure meter testing, Borehole Extensometer and underground benchmark data all indicated an overall modulus of deformation value for the foundation rock closer to 200,000 psi." See Figure 4-6 for adopted geotechnical values for the Bonneville second powerhouse.

Figure 4-6. Adopted Geotechnical Values for the Bonneville second powerhouse.

PROPERTY	ADOPTED VALUE
Field Unit Weight	130 - 140 pcf
Dry Density	112.5 pcf
Percent Clay Sizes	20 - 40 %
Clay Mineral	Montmorillonite
Significant Exchange Ion	Ca^{++} and Mg^{++}
Percent Swell	2%
Swell Pressure	18 - 0.13 TSF
Rate of Slaking	very fast
Natural Water Content	20%
Liquid Limit	45 - 65
Plasticity Index	5 - 25
USCS classification	CL
Modulus of Elasticity	3,000 - 4,500 psi
Unconfined Compressive Strength	1,000 - 1,500 psi
Direct Shear Strength	
Residual	phi = 20°, c = 0.5 TSF
Discontinuity	phi = 32°, c = 5.0 TSF
Triaxial Strength	phi = 35°, c = 15.0 TSF
Seismic Velocity (intact cores)	8,000 ft/sec
Permeability	2x10 ft / min
Rock / Grout Bond Strength	Tb = 117 psi, Tf = 87 psi
Concrete / Rock Shear Strength (at 0.25 TSF normal load)	1.26 TSF

Construction and post-construction observations have shown there is a higher risk of foundation problems. What is documented:

- A soft, green, bentonitic clay-rich strata was encountered at the north end of the spillway. Foundation grade lines were adjusted to remove most but not all of it. This area where normal flows have formed a scour hole has lowered the riverbed about 30 feet.
- Similar soft, green, clay-rich strata were encountered during construction of the second powerhouse. Several slope failures occurred in steep construction slopes.
- The north spillway gate repair pit center bridge pier piles are believed to be founded on this same unit. The bridge pier began settling almost immediately

following construction. Settlement continued steady for about 70 years until earth load was partially removed. Pattern is consistent with incipient foundation failure (i.e., bearing capacity failure or plastic flow of the foundation).

- Geologic cross-sections from original construction indicates this soft stratum probably projects to below the proposed location of the north rock barrier. The presence of this stratum within potential anchorage depth is likely and should be considered.

4.2.3.3 *Geoengineering Design Considerations.*

Geotechnically, anchoring the barrier to the apron slab may be problematic. There is little or no zone engineering guidance on how this clay-rich rock will perform under thousands of hours of repetitive load cycling due to hydraulic loading under turbulent spill condition. Concern is that whether it is possible that repetitive loading could induce fatigue on the anchor grout/rock interface or induce creep in the heavily stressed rock column surrounding an anchor which could lead to gradual loss of anchor tension.

- Foundation anchorages is considered difficult and expensive to do. It is uncertain whether an anchor can be tensioned underwater by divers.

- Anchor heads would be continuous underwater with no way to inspect, conduct non-destructive testing, or conduct lift off tests to determine if there has been loss of tension of anchorage.

Avoiding barrier anchorage (accepting the risk and consequences of sliding) removes uncertainties and risks associated with designing and counting on anchors. The apron is a greater critical feature from an operational and dam safety point of view than the rock barrier which is more of enhanced maintenance. Therefore, avoiding anchoring with all the associated uncertainties and costs is preferred over avoiding or reducing operation and maintenance costs of rock removal.

Placing the barrier on the apron avoids the complication of excavating below the foundation grade. Excavating below the foundation grade could potentially lead to undercutting of the apron. In addition, placing the barrier on the apron is geotechnically beneficial as the heavy bearing load is distributed out by the 5-foot-thick reinforced concrete apron slab, thereby reducing bearing load at the actual concrete/rock interface. The reinforced slab also keeps the foundation rock tightly confined. The foundation was inspected, treated, and approved in the dry during original construction.

4.2.4 Dam & Levee Safety

The preferred barrier alternative is classified as a non-critical structure. The barrier causes no measurable risk to human life during construction, operation, surveillance, or maintenance above existing baseline condition. The barrier causes no increase in environmental risk (fish mortality) above existing baseline condition. The placement of the barrier onto the apron, rather than anchoring, risks damage to the apron, but the risk is low and the potential damage to the apron would be minor. Avoiding damage to the apron reduces the chance for repair operations, which would likely require an extensive dewatering procedure.

4.2.5 Reservoir Regulation & Water Quality

No discipline-specific considerations were needed at this point in project development.

4.2.6 Fish Passage

The preferred barrier alternative will be installed deep enough in the water column such that fish are not expected to encounter it. Therefore, the barrier is not an impediment to either upstream or downstream fish passage.

4.3 PERFORMANCE CALCULATIONS

The stability analysis is located in the structural design appendix (Appendix B) for all evaluated alternatives.

4.4 FEASIBILITY LEVEL COSTS

4.4.1 General

This section presents the cost estimate for the Bonneville Spillway Rock Mitigation as presented in this Phase 1A. The total project cost (design and construction) estimated at the 90% Phase 1A is $6.65 million, including a 41.9% contingency. With a 41.9% contingency and 5.5% escalation, the construction contract is estimated at $5.12 million. The construction contract is expected to take about a month. The risk analysis and total project cost summary sheet can be found in Appendix C.

4.4.2 Criteria

Engineer Regulation 1110-2-1302, Engineering and Design Civil Works Cost Engineering, provides policy, guidance, and procedures for cost engineering for all Civil Works projects in the USACE. For a project at this phase, the cost estimates are to include construction features, lands and damages, relocations, environmental compliance, mitigation, engineering and design, construction management, and contingencies. The cost estimating methods used are to establish reasonable costs to support a planning evaluation process. The design is at a preliminary level and the cost estimate is at a similar level.

4.4.3 Basis of the Cost Estimate

The cost estimate is based on engineering calculations from the design team and data presented in the Phase 1A. The estimate is calculated with the Micro Computer Cost Estimating System (MCACES) MII, using historical data, labor and equipment crews, quantities, production rates, and material prices. Prices are updated for July 2021 in MII and escalated to the midpoint of construction on the total project cost summary sheet.

4.4.4 Cost Items

The cost estimate includes costs for engineering for plans and specifications, construction costs, engineering during construction, construction management for supervision and administration, escalation costs, and contingency to account for unforeseen details at this level. Other possible costs are not shown separately, such as lands and damages, relocations, cultural resources, environmental mitigation, environmental compliance, and hazardous, toxic, and radiological waste (HTRW) costs. These costs are either not applicable or integrally part of the construction costs and are included in the construction features. Escalation costs to account for inflation are applied according to EM 1110-2-1304, Civil Work Construction Cost Index system.

4.4.5 Cost and Schedule Risk

An abbreviated cost and schedule risk analysis has been completed to determine a risk-based contingency to add to the cost estimate. The analysis identified the following project risks that need to be addressed during the design phase.

- Design Development: The design is still developing and may have adjustments or additional scope to complete the mission.
- Concrete Pumping Location: Weight limits on and around the spillway may present risks to where a concrete pump can be placed to perform the work.

4.4.6 Acquisition Strategy and Subcontracting Plan

The cost estimate assumes that competitive pricing will be obtained from the small business community. The cost estimate is based on the work being accomplished by a marine contractor being the prime contractor. The estimate assumes that the prime will subcontract out diving and concrete work.

4.4.7 Functional Costs

4.4.7.1 *Planning Engineering and Design (30 Account)*

Engineering and design costs are determined from the budgets for the expected design and engineering effort. These costs include engineering costs for design and development of a contract package (plans and specifications), NWP review, contract advertisement, award activities, and engineering during construction. This effort is estimated to cost $810,000 for the plans and specifications phase, including 41.9% contingency.

4.4.7.2 *Construction Management (31 Account)*

Construction management costs are determined from the budget of the expected effort for supervision, administration, and quality assurance for the construction contract. This effort is estimated to cost $717,000, including 41.9% contingency.

4.4.8 Schedule

Construction is expected to take approximately 35 days for all aspects onsite. A product development and construction schedule is located in Appendix C. The notice to proceed (NTP) is expected to be issued between June and August to allow the contractor to design and fabricate the precast panels for the concrete forms prior to the IWWW beginning December 1.

4.4.9 Historical Dredging Costs

Three dredging contracts to remove rock from the spillway surface were completed since FY18, with an average escalated bid price of $613,357, as show in Figure 4-7.

Figure 4-7. Historic Rock Dredging Prices

Contract	Contractor	Average Bid	Total Award	Mob/Demob	Rock Removal	Escalated Total Award	Escalated Ave. Bid	Escalation To FY22Q1
18C0014	HME	$ 629,940	$ 371,450	$ 122,756	$ 248,694	**$ 416,290**	**$ 705,984**	89%
19C0009	J.E. McAmis	$ 653,000	$ 514,000	$ 302,000	$ 212,000	**$ 548,836**	**$ 697,256**	94%
21C0010	J.E. McAmis	$ 427,700	$ 411,736	$ 253,000	$ 158,736	**$ 420,525**	**$ 436,830**	98%
Average		$ 570,213	$ 432,395	$ 225,919	$ 206,477	**$ 461,884**	**$ 613,357**	

Figure 4-8 shows the expected future contract costs of continued rock dredging, using the past dredging costs, escalated for each fiscal year, excluding functional costs.

Figure 4-8. Expected Future Dredging Costs

Adjusted Future From Historical (CWCCIS 10/31/2021)				Running	
Year	From Average Award	Total From Award	From Average Bid	Total From Bid	Escalation
FY23	$ 474,011	$ 474,011	$ 585,186	**$ 585,186**	103%
FY24	$ 485,861	$ 959,873	$ 599,815	**$ 1,185,000**	105%
FY25	$ 498,004	$ 1,457,876	$ 614,805	**$ 1,799,805**	108%
FY26	$ 510,454	$ 1,968,330	$ 630,175	**$ 2,429,980**	111%
FY27	$ 523,216	$ 2,491,546	$ 645,931	**$ 3,075,911**	113%
FY28	$ 536,295	$ 3,027,841	$ 662,078	**$ 3,737,989**	116%
FY29	$ 550,034	$ 3,577,875	$ 679,038	**$ 4,417,027**	119%
FY30	$ 564,334	$ 4,142,209	$ 696,692	**$ 5,113,719**	122%
FY31	$ 579,009	$ 4,721,218	$ 714,809	**$ 5,828,529**	125%
FY32	$ 594,417	$ 5,315,634	$ 733,830	**$ 6,562,359**	129%
FY33	$ 610,463	$ 5,926,098	$ 753,641	**$ 7,316,000**	132%
FY34	$ 626,945	$ 6,553,042	$ 773,988	**$ 8,089,988**	136%
FY35	$ 643,875	$ 7,196,917	$ 794,889	**$ 8,884,876**	139%
FY36	$ 661,259	$ 7,858,176	$ 816,350	**$ 9,701,226**	143%
FY37	$ 679,517	$ 8,537,692	$ 838,890	**$ 10,540,116**	147%
FY38	$ 698,545	$ 9,236,238	$ 862,382	**$ 11,402,498**	151%
FY39	$ 718,101	$ 9,954,339	$ 886,524	**$ 12,289,022**	155%
FY40	$ 738,209	$ 10,692,547	$ 911,347	**$ 13,200,369**	160%

SECTION 5 - CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

After screening of considered alternatives was complete, the most feasible alternatives were subjected to numerical and physical hydraulic modeling. The hydraulic modeling indicated that the barrier alternative (ALT 2) was the only hydraulically viable alternative and was thereby considered to be the preferred alternative.

The current state of the barrier design consists of two rectangular, concrete prisms 13 feet wide, 17 feet tall, and 80 feet long. The barriers are anticipated to be modular, consisting of precast-concrete cells which will be placed underwater then filled with concrete. The cells will be keyed together to ensure proper alignment. One barrier each will be placed on the north and south side of the apron. The location of the south barrier will be in line with pier 11. The precise location of the north barrier is yet to be determined. Due to geotechnical concerns regarding the quality of the substrate beneath the apron and the criticality of the apron itself, it was decided that avoiding anchoring with all the associated uncertainties and costs is preferred over avoiding or reducing operation and maintenance costs of rock removal.

The barrier is classified as a non-critical structure. It is expected to cause no measurable risk to human life during construction, operation, surveillance, or maintenance above existing baseline condition. The barrier will be installed deep enough in the water column such that fish are not expected to encounter it, hence the barrier is not expected to impede to either upstream or downstream fish passage.

The total project cost (design and construction) is estimated to be $6.65 million, including a 41.9% contingency. With a 41.9% contingency and 5.5% escalation, the construction contract is estimated at $5.12 million. The construction contract in-water work period is expected to last approximately one month. Rock removal prices are likely to increase over time due to escalation.

5.2 RECOMMENDATIONS

It is recommended that a barrier design be implemented in the Bonneville spillway to prevent rocks from entering and causing damage to the stilling basin. It is also recommended that Phase 1 further investigate aspects of the barrier design that could not be investigated in Phase 1A. Such aspects include but are not limited to: investigation of alternative barrier anchoring methods, a comprehensive seismic stability analysis of the barrier, further hydraulic/structural/geotechnical modeling of the barrier final shape as necessary, determination of the location of the north barrier, development of contingency plans in the event the barrier slides or fails, and a break-even analysis between the status quo non-routine maintenance rock removals and the lifetime costs of the barrier. In addition, constructability issues such as optimization of barrier module size and concrete pumping logistics should be addressed to aid in the development of a more refined cost estimate.

It is recommended that a long-term, recurring maintenance contract framework for rock removal be developed to replace the existing non-routine maintenance contract framework for rock removal. The recurring maintenance contract framework would streamline the rock removal contract process and allow the frequency of rock removal events to be determined at the discretion of the government.

SECTION 6 - REFERENCES

National Oceanic and Atmospheric Administration, National Marine Fisheries Service. 2020. *Endangered Species Act Section 7(a)(2) Biological Opinion and Magnuson-Stevens Fishery Conservation and Management Act Essential Fish Habitat Response for the Continued Operation and Maintenance of the Columbia River System*.

Pacific Northwest National Laboratory. Rakowski, Serkowski, Richmond, Perkins. 2010. *Computational Fluid Dynamics Modeling of the Bonneville Project: Tailrace Spill Patterns for Low Flows and Corner Collector Smolt Egress*.

Pacific Northwest National Laboratory. Rakowski, Serkowski, Richmond, Romero-Gomez. 2012. *Bonneville Project: CFD of the Spillway Tailrace*.

U.S Army Corps of Engineers Northwestern Division. 2021. *2021 Fish Passage Plan: Lower Columbia & Lower Snake River Hydropower Projects*.

U.S. Army Corps of Engineers. 2001. Design Document Report No. 46. *Bonneville Spillway Flow Deflectors and Gate Hoists*.

U.S. Army Corps of Engineers. 2005. EM 1110-2-2100. *Stability Analysis of Concrete Structures.*

U.S. Army Corps of Engineers. 2014. *Bonneville Dam Water Control Manual*.

U.S. Army Corps of Engineers. 2016. ER 1110-2-1302*. Civil Works Cost Engineering*.

U.S. Army Corps of Engineers. 2021. EM 1110-2-1304*. Civil Works Construction Cost Index System (CWCCIS)*.

Williamson, et.al. 1988. "The Unified Rock Classification System." In Rock Engineering Systems for Engineering Purposes. *ASTM STP-984*. L. Kirkaldie (ed.). American Society for Testing and Materials: 7-16.