DESIGN DOCUMENTATION REPORT NO. 8

DETROIT DAM AND RESERVOIR
WILLAMETTE RIVER BASIN
NORTH SANTIAM RIVER, OREGON

US Army Corps
of Engineers ®
Portland District

Phase 1 Downstream Fish Passage – Selective Withdrawal Structure

Revised 60 Percent Design Documentation Report
ATR and BPA Review
December 2018

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EXECUTIVE SUMMARY

1. INTRODUCTION

The National Oceanic and Atmospheric Administration (NOAA) 2008 Willamette Biological Opinion (BiOp) identified the Reasonable and Prudent Alternative (RPA) to avoid jeopardy of Endangered Species Act (ESA)-listed fish in the Willamette Basin. RPA 5.2 requires investigation and implementation of improvements to downstream temperatures and Total Dissolved Gas (TDG) exceedances in the North Santiam River for ESA-listed fish species. Interim temperature control operations have been attempted annually since 2008 using existing project facilities and operating equipment. Operational temperature control only functions when the reservoir elevation is above the spillway crest, which limits its success. Furthermore, water passing over the spillway results in foregone power. These issues indicate the need for a structural solution to temperature control. Additionally, the NOAA 2008 Willamette BiOp RPA 4.12.3 requires investigation and implementation of a plan to safely pass juvenile fish downstream of Detroit Dam.

The process started in 2010 with the development of the Detroit and Big Cliff Long-Term Temperature Control and Downstream Fish Passage Engineering Documentation Report (EDR). The EDR identified an array of structural and operational alternatives to provide temperature control and downstream passage at Detroit Dam. The EDR was finalized in 2017 with a recommendation to move forward with a selective withdrawal structure (SWS) for temperature control, and two alternatives for juvenile fish collection: a weir box and a floating screen structure (FSS). In addition, this effort provided data to the Willamette Valley Project's Configuration and Operations Plan (COP) team for evaluation throughout the Willamette Basin.

Following completion of the EDR, three Design Documentation Reports (DDRs) were initiated for the purpose of developing design criteria and details for the SWS, weir box, and FSS. This DDR presents the required features of the proposed Detroit Dam and Reservoir (Detroit Dam, Project, or DET) SWS. The main feature of the SWS will be a concrete tower with two high intake weirs (HIWs) allowing for surface flow, and four low intake gates (LIGs) allowing for at-depth water to be taken from the reservoir and into a wet well. The water will then pass through the turbines when operating or through a penstock bifurcation and into the tailrace when the units are not running, thereby providing optimal water temperatures downstream at all times.

A second concurrent DDR is being prepared for the FSS. The FSS will screen all fish from the surface water before it enters the wet well of the SWS. The FSS screened water will then pass through the SWS as previously described. Once collected in the FSS, the juvenile fish will be transported downstream either by truck or bypass conduit. (Reference the Detroit Floating Screen Structure DDR for further information on the FSS criteria and design.) The FSS DDR assumes fish will be transported downstream via the trap-and-haul method. The High Head Bypass Product

Development Team (PDT) is currently investigating the feasibility of piped bypass for the downstream conveyance of juvenile fish at high head dams due to the potential biological and long-term operation benefits.

The third concurrent DDR was for a weir box. The weir box used flow into the SWS wet well to attract and trap fish, then entice them to exit the wet well near the surface and into the weir box. As the weir box design progressed to a 60% DDR level, the PDT found it difficult to achieve biologically effective hydraulic conditions; therefore, a decision was made to stop work on the weir box at that point. (Reference the Detroit Weir Box 60% DDR for further information.)

The recommended design and construction schedule for the SWS (Phase 1) and the FSS (Phase 2) of downstream fish passage is shown below. The FSS DDR in support of Phase 1 is being prepared concurrently with the SWS DDR to ensure that the SWS is configured correctly and can accommodate the future FSS. The SWS and FSS will be hydraulically connected and will work together as a system.

Design and Construction Schedule:

- Phase 1 of Downstream Fish Passage – SWS:
    - SWS DDR Draft Final: May 2019
    - FSS Draft Final DDR in support of Phase 1: July 2019
    - SWS Plans and Specifications: Feb 2019 – Jun 2020
    - SWS Construction: Nov 2020 – Apr 2024
- Phase 2 of Downstream Fish Passage – FSS:
    - FSS DDR in support of Phase 2: Jun 2021 – Dec 2022
    - FSS Plans and Specifications: Jan 2023 – Jun 2024
    - FSS Construction: Nov 2024 – Aug 2027

From the start of the DDR through the initial 60%, the SWS had been a freestanding tower. The tower was located approximately 140 ft upstream of Detroit Dam to avoid excavation directly adjacent to the dam and the potential need to seismically mitigate the dam monoliths. This location was preferred biologically because it placed the entrance of the FSS as close to the dam as possible, thus allowing the dam to act as a fish guidance structure to the FSS entrance, which optimized the biological performance of the FSS. However, the freestanding tower had extensive features, including an access bridge, two penstock conduits, and a regulating outlet (RO) bypass conduit that added cost and time to the construction schedule. Up through the 60% DDR development, it was assumed the freestanding tower, conduits, and access bridge would be constructed in the dry.

Upon completion of the SWS 60% DDR it became clear, through the National Environmental Policy Act (NEPA) Environmental Impact Statement (EIS) process, that a full drawdown of the Detroit Reservoir would have a considerable impact on the North Santiam downstream water supply, water quality, the local economy, recreation, and agriculture. In response to the constructability issue a team of engineering, construction, and project management personnel met to brainstorm alternative SWS construction methods and configurations that could be built in the wet. The meeting resulted in the following three potential build-in-the-wet alternatives:

1) Free Standing SWS Built in the Wet

2) SWS Attached to Dam Built in the Wet

3) SWS Attached to Dam Built with a Cofferdam

The PDT further developed the three alternatives and performed an alternatives analysis that resulted in a recommendation to switch to Option 2, SWS Attached to Dam Built in the Wet, to accomplish temperature control. The PDT's recommendation was briefed at the August 2018 NWP Fish Forum and received Corporate Board approval to switch from the freestanding SWS to an SWS attached to Detroit Dam. (See Appendix J, Detroit SWS Build in the Wet Alternatives Analysis for details of the alternatives analysis and the Fish Forum Detroit presentation and minutes.)

The Revised 60% DDR includes the design criteria, preliminary analysis, and layout of the SWS attached to Detroit Dam. Also included is the preliminary seismic evaluation of the existing dam with the added mass from the SWS.

2. PURPOSE

The purpose of this DDR is to provide a record of design criteria, assumptions, and methods related to the design, construction, and operation of the Detroit Dam SWS. The SWS is a multilevel intake structure intended to modify the outflow water temperature to more closely match the natural cycle of water temperatures in the North Santiam River. The natural cycle of water temperatures was altered when the Project began operation in 1953. The change from the natural cycle negatively impacted the lifecycles of the anadromous and native fish species downstream of the dam.

3. PROJECT DESCRIPTION AND LOCATION

Completed in 1953, Detroit Dam is one of 13 flood control dams located in Oregon's Willamette River Basin. The project was constructed primarily for flood control and hydroelectric power generation; however, other major benefits include recreation and conservation uses involving releases of stored water. There are small communities located downstream on the North Santiam River, with the largest being Stayton (population 7,644, approximately 44 miles). The city of Salem (population 167,419, approximately 50 miles) is along the Willamette River just beyond the point where it's joined by the North Santiam. Major features include a concrete dam with a spillway, regulating outlets, penstocks, and a detached powerhouse.

The North Santiam sub-basin drains about 760 square miles. Detroit and Big Cliff dams are two of Oregon's 13 multipurpose projects operated by the U.S. Army Corps of Engineers (USACE or Corps) in the Willamette Valley. Located in Marion County, in the rugged mountain forests below Mt. Jefferson, the two dams store the waters of the North Santiam River. Detroit Dam is located at river mile 60.9, approximately 50 miles southeast of Salem, Oregon. Big Cliff is a re-regulating dam located at river mile 58.1, about 3 three miles downstream of Detroit Dam. Big Cliff Lake is a small reservoir used to even out peak discharges of water used for power generation at Detroit Dam, thereby controlling downstream river level fluctuations.

Detroit Dam is a concrete gravity dam approximately 1,457 ft long with a maximum height of 450 ft above the lowest portion of its foundation. The spillway is a concrete ogee type with six Tainter gates located in the middle of the dam. There are four ROs located directly below the spillway; two at elevation 1,340 and two at elevation 1,265. A rarely used fifth RO, originally intended for hydraulic model testing, is located at the south end of the spillway at elevation 1,340. Two steel-pipe penstocks located on the north side of the spillway with entrances at elevation 1,403 go through the dam and exit on the downstream side to provide water to the two 50-MW Francis turbines in the powerhouse.

Detroit and Big Cliff dams were both constructed without adult fish ladders. The Minto Fish Collection Facility (Minto) was rebuilt in 2013 to provide trap-and-haul facilities to allow for the reintroduction of spring Chinook salmon and winter steelhead above Detroit Dam. The Minto Fish Collection Facility is located on the north bank of the North Santiam River at river mile 55, about 4 miles downstream of Big Cliff Dam and 7 miles downstream of Detroit Dam.

4. DESCRIPTION OF THE PROPOSED FACILITY

The SWS will include the following major features (Appendix A includes plates showing the SWS features and layout):

- A 370x40x108-ft concrete tower.
- Two HIWs and four LIGs to control flow.
- Penstock bifurcation to bypass the flow when turbines are not operating.
- Boat ramp for crew access to the FSS.
- Improvements to debris management.

The SWS is being designed to accommodate a future FSS for downstream fish passage. The excavation for the FSS will be performed during the construction of the SWS.

5. CONSTRUCTION ACCESS

Construction access is from Highway 22. Construction staging and access to the staging areas is discussed in detail in Sections 10 and 14 of this DDR.

6. CONSTRUCTION SCHEDULE

Construction will take place over 42 months. Notice-to-Proceed is anticipated in October 2020. Commissioning will occur immediately after construction is complete. Section 10 and Appendix I discuss the schedule in detail. See Appendix I, Cost Estimate and Schedule for more details.

7. OPERATIONS DURING CONSTRUCTION

This will be added to the 90% DDR.

8. COST

The estimated total project cost is $275M. See Appendix I, Cost Estimate and Schedule for more details.

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Pertinent Project Data for Detroit Dam and Reservoir

Detroit Dam is a 450-ft high, 1,457-ft long concrete gravity structure. The dam has a gated spillway structure that is 294.5 ft long and 28.0 ft high with six spill bays, each 42 ft wide. The spillway crest is at elevation 1,541.0 ft, full pool is elevation 1,569.0 ft, and minimum conservation pool is elevation 1,450.0 ft. Additionally, Detroit Dam has four regulating outlets (ROs), two with an invert elevation of 1,265.3 ft, two at elevation 1,340.0 ft, and two turbines with penstock intake elevation at 1,403 ft.

Pertinent Data - Detroit Dam and Reservoir

Year Completed	1953
Stream	North Santiam River
River Mile	60.9 (from Santiam mouth)
Drainage Area (square miles)	438
Dam Height (feet)	450
Dam Crest MSL	1,579.0
Maximum Pool	1,574.0 feet (472,600 acre-feet)
Full Pool	1,569.0 feet (455,100 acre-feet)
Maximum Conservation Pool	1,563.5 feet (436,000 acre-feet)
Spillway Crest	1,541.0 feet (363,200 acre-feet)
Minimum Conservation Pool	1,450.0 feet (154,400 acre-feet)
Minimum Power Pool	1,425.0 feet (115,000 acre-feet)
Turbines	Two 50-MW Francis turbines at penstock elevation 1,403.0 feet (4,300-5,300 cfs combined hydraulic capacity) Cavitation limit is between 1,100-1,000 cfs per unit, within normal pool operations range
Spillway Gates	Six radial Tainter gates (176,000 cfs combined hydraulic capacity)
Upper Regulating Outlets	Two at elevation 1,340 feet (13,050 cfs combined capacity)
Test Flume Conduit	One at elevation 1,340 feet (same dimensions as Upper Regulating Outlets, not currently used)
Lower Regulating Outlets	Two at elevation 1,265 feet that are not used

Pertinent Project Data for Big Cliff Dam

Big Cliff Dam is 280 ft long and 172 ft high. The spillway crest is at elevation 1,161.5, full pool is at elevation 1,206 and minimum pool is at elevation 1,182. The dam has three spill bays and one 18 MW capacity power generating unit (Table 2). Due to Big Cliff re-regulation operations, the lake level fluctuates as much as 22 feet daily.

Pertinent Data - Big Cliff Dam

Year Completed	1953
Stream	North Santiam River
River Mile	58.1 (from Santiam mouth)
Drainage Area (square miles)	452
Dam Height (feet)	172
Dam Crest (elevation feet MSL)	1,212.0
Maximum Pool	1,210.0 feet (5,300 acre-feet)
Full Pool	1,206.0 feet (4,700 acre-feet)
Minimum Power Pool	1,182.0 feet (2,300 acre-feet)
Turbines	One 18-MW Kaplan (2,800-3,200 cfs hydraulic capacity)*
Spillway Gates	Three radial Tainter gates (179,000 cfs combined hydraulic capacity)

Previous Memorandums

Project	Document No.	Description	Date
Detroit	1	DETROIT -BIG CLIFF PROJECT - Centralization of Control	
Detroit	*	DETROIT DAM	
Detroit		Minto Egg Collecting Station	
Detroit		Repair to Minto Dam Erosion & Alterations to Holding Ponds	
Detroit	*	DETROIT DAM	
Detroit		Steel Lining for Regulating Conduits	
Detroit		Design E Construction Evaluation	

Acronyms and Abbreviations

ATU	Accumulated Thermal Unit
BGC	Big Cliff Dam
BiOp	Biological Opinion
BOR	Bureau of Reclamation
BPA	Bonneville Power Administration
CE-QUAL-W2	Hydrodynamic and Water Quality Model in 2D (longitudinal-vertical)
CFD	Computational Fluid Dynamics
cfs	cubic feet per second
COP	Configuration and Operations Plan
CSRA	Cost and Schedule Risk Analysis
CWA	Clean Water Act
DDR	Design Documentation Report
DET	Detroit Dam
DSL	Department of State Lands
EC	Engineering Circular
EDM	Electronic Distance Measuring
EDR	Engineering Documentation Report
EIS	Environmental Impact Statement
EM	Engineering Manual
EPA	Environmental Protection Agency
ER	Engineering Regulation
ESA	Endangered Species Act
ESCP	Erosion and Sediment Control Plan
ETL	Engineering Technical Letter
°F	Degrees Fahrenheit
FBW	Fish Benefits Workbook
FCE	Fish Collection Efficiency
FCRPS	Federal Columbia River Power System
FDR	Flood Damage Reduction
FL	Fork Length
FS	Factor of Safety
FSC	Floating Surface Collector
FSS	Floating Screen Structure
ft	foot / feet
ft/s	feet per second
ft^3	cubic feet

HA	Hydro-acoustic
HDC	Hydroelectric Design Center
HEC ResSim	Hydrologic Engineering Center Reservoir System Simulation
HIW	High Intake Weir
HMT	Hatchery Management Team
HOR	Head of Reservoir
HSS	Hydraulic Steel Structures
IRRM	Interim Risk Reduction Measures
JSATS	Juvenile Salmon Acoustic Telemetry System
LIG	Low Intake Gate
m	meter
M&E	Monitoring and Evaluation
Minto	Minto Fish Collection Facility
MIS	Modular Inclined Screens
MSL	Mean Sea Level
MW	Megawatt
NEPA	National Environmental Policy Act
NMFS	National Marine Fisheries Service
NOAA	National Oceanic and Atmospheric Administration
NPDES	National Pollutant Discharge Elimination System
O&M	Operations and Maintenance
ODEQ	Oregon Department of Environment Quality
ODFW	Oregon Department of Fish and Wildlife
ODOT	Oregon Department of Transportation
OHA	Oregon Health Authority
P&S	Plans and Specifications
PDT	Product Development Team
PFMA	Potential Failure Mode Analysis
PIT	Passive Integrated Transponder
PLC	Programmable Logic Controller
POR	Period of Record
psf	pounds per square foot
RM&E	Research Monitoring and Evaluation
RMC	Risk Management Center
RO	Regulating Outlet
RPA	Reasonable and Prudent Action
SCADA	Supervisory Control and Data Acquisition
SIMPAS	Simulated Fish Passage Survival
SWS	Selective Withdrawal Structure
TBD	To Be Determined

TCD	Temperature Control Device
TCS	Temperature Control Structure
TDG	Total Dissolved Gas
TM	Technical Manual
TMDL	Total Maximum Daily Loads
TPC	Total Project Cost
USACE/Corps	United States Army Corps of Engineers
USFS	United States Forest Service
USFWS	United States Fish and Wildlife Service
USGS	United States Geological Survey
UWR	Upper Willamette River
WFFDWG	Willamette Fish Facility Design Work Group
WFOP	Willamette Fish Operations Plan
WFPOM	Willamette Fish Passage Operations & Maintenance
WQC	Water Quality Certification

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SECTION 1 - PURPOSE AND INTRODUCTION

1.1 SCOPE AND PURPOSE

The NOAA 2008 Willamette BiOp identified the RPA to avoid jeopardy of ESA-listed fish in the Willamette Basin. RPA 5.2 requires investigation and implementation of improvements to downstream temperatures and TDG exceedances in the North Santiam River for ESA-listed fish species. Interim temperature control operations have been attempted annually since 2008, using existing project facilities and operating equipment. Operational temperature control only functions when the reservoir elevation is above the spillway crest which limits its success. Furthermore, water passing over the spillway results in foregone power. These issues indicate the need for a structural solution. RPA 4.12.3 requires investigation and implementation of a plan to provide safe passage of juvenile fish downstream of Detroit Dam.

The DDR presents the required features of the proposed Detroit Dam SWS which is designed to provide control of the water temperature of the Project's outflows. This system will use a multilevel intake structure to modify the outflow water temperature to more closely match the natural cycle of water temperatures in the river. The natural cycle of water temperatures was altered when the Project began operation in 1953. The change from the natural cycle negatively impacted the lifecycles of the anadromous and native fish species downstream of the dam. The SWS will allow outflow water temperature to be modified to meet temperature targets throughout the year and will make the water available for the use of power generation.

The design of the SWS in this DDR will take into account the inclusion of a future FSS to provide downstream fish passage. A second DDR is being prepared concurrently for the design of the Detroit FSS.

1.2 REPORTS AND STUDIES USED IN THE DDR

Detroit and Big Cliff Long Term Temperature Control and Downstream Fish Passage Engineering Documentation Report (Detroit Temperature and Downstream Passage EDR), USACE, July 2017.

1.3 GENERAL DESCRIPTION

1.3.1 Location

The North Santiam sub-basin drains about 760 square miles. Detroit and Big Cliff dams are two of Oregon's 13 multi-purpose projects operated by USACE in the Willamette Valley. Located in Marion County, in the rugged mountain forests below Mt. Jefferson, the two dams store the waters of the North Santiam River. Detroit and Big Cliff dams were both constructed without adult fish ladders. The Minto Fish Collection Facility, located below Big Cliff Dam, was rebuilt in 2013 to provide trap-and-haul facilities to allow for the reintroduction of spring Chinook salmon and steelhead above Detroit Dam.

Detroit Dam is located at river mile 60.9 on the North Santiam River, approximately 50 miles southeast of Salem, Oregon. Big Cliff is a re-regulating dam located at river mile 58.1, about three miles downstream of Detroit Dam. Big Cliff Lake is a small reservoir utilized to even out peak discharges of water used for power generation at Detroit Dam, thereby controlling downstream river level fluctuations. The Minto Fish Collection Facility is located on the north bank of the North Santiam River at river mile 55, about four miles downstream of Big Cliff Dam and seven miles downstream of Detroit Dam.

Detroit Dam is a 450-ft high, 1,457-ft long concrete gravity structure. The dam has a gated spillway that is 294.5 ft long and 28.0 ft high with six spill bays, each 42 ft wide. The spillway crest is at elevation 1,541.0 ft, full pool is elevation 1,569.0 ft, and minimum conservation pool is elevation 1,450.0 ft. Detroit Dam also has four ROs, two with an invert elevation of 1,265.3 ft, two at elevation 1,340.0 ft, and two turbines with penstock intake elevations at 1,403 ft.

1.3.2 Project Authorization

Construction of the Detroit Dam, North Santiam River, Oregon, was authorized by the Flood Control Act of 1938, Pub. L. No. 75-761 (52 Stat. 1215). The law approved the "general comprehensive plan for flood control, navigation, and other purposes in the Willamette River Basin as set forth in House Document Numbered 544, Seventy-fifth Congress, third session". The Flood Control Act of 1948, Pub. L. No. 80-858 (62 Stat. 1175), modified the Flood Control Act of 1938 to provide for the installation of hydroelectric power-generating facilities at Detroit Dam, and included the construction of Big Cliff Dam as a part of the Detroit project in accordance with plans on file in the Headquarters Office, Chief of Engineers. These and subsequent laws have authorized the following project purposes at Detroit Dam: flood control, navigation, hydropower, water supply (irrigation, municipal, and industrial), water quality, fish and wildlife, and recreation.

The Corps is responsible for the construction and operation of the Project for its authorized purposes, and has exclusive control over all waters and Project lands adjacent to and beneath the water surfaces, to include withdrawn USFS lands, for carrying out these purposes. The use of withdrawn USFS lands for purposes extraneous to project operation remains under the jurisdiction of the USFS. To facilitate the management and control of project resources, and to eliminate the overlapping of administrative responsibilities, the operational areas of Detroit and Big Cliff Project lands that lie outside the USFS boundary will remain under the exclusive control of the Corps. The responsibility for administering all other Project lands within the boundary for recreation, fire protection, and land management is vested with the USFS, in accordance with a Memorandum of Understanding (MOU) between the Secretary of Agriculture and the Secretary of the Army, effective November 10, 1954.

1.4 PROPOSED SWS

A concrete SWS will be located on the upstream side the dam. See Figure 1-1 for weir and gate locations. Two HIWs will be used to withdraw warm water from near the surface at varying forebay elevations. Four LIGs will be used to withdraw cold water near the bottom of the reservoir. The warm and cold water will combine in the SWS wet well and be released to the Big Cliff pool to meet target outflow temperatures.

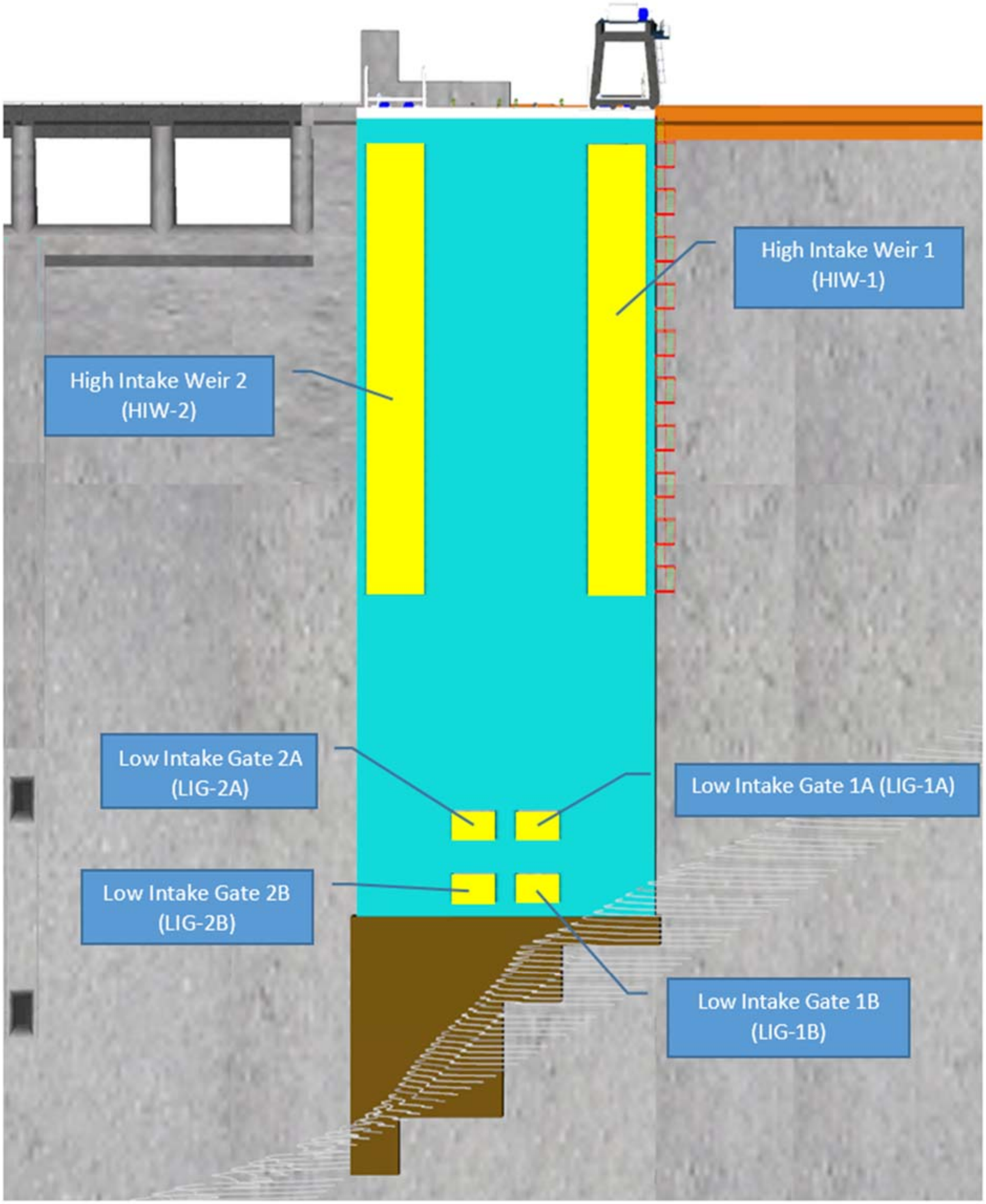

Figure 1-1. SWS Downstream Elevation

Components of the structure include (See Appendix A for Plates showing the SWS features and layout):

- A 370-ft tall x 40-ft x 108-ft concrete tower.
- Two HIWs and four LIGs to control flow.
- Penstock bifurcation to bypass the flow when turbines are not operating.
- A boat ramp for crew access to the FSS.
- Improvements to debris management.

The SWS is being designed to accommodate a future FSS for downstream fish passage. The excavation for the FSS will be performed during the construction of the SWS.

1.5 AGENCY COORDINATION

The design for this project will comply with NOAA's 2008 BiOp and is being coordinated through the regional Willamette Fish Facility Design Work Group (WFFDWG). Members include representatives from the Corps, Bonneville Power Administration (BPA), NOAA, Oregon Department of Fish and Wildlife (ODFW), and the United States Fish and Wildlife Service (USFWS).

Government-to-government coordination is also underway with the Confederated Tribes of the Grande Ronde, the Confederated Tribes of the Warm Springs Reservation of Oregon, and the Confederated Tribes of the Siletz Indians.

Coordination with other agencies when applicable will include the Oregon Department of Environmental Quality (ODEQ), State Historic and Preservation Office, Oregon Parks and Recreation Department, USFS, Oregon Water Resources Department, U.S. Bureau of Reclamation (BOR), Oregon Marine Board, Oregon Department of State Lands (DSL), Oregon Department of Transportation (ODOT), and Oregon Department of Forestry.

1.6 ENVIRONMENTAL COMPLIANCE

Compliance with various federal, state and local environmental regulations, such as the Clean Water Act (CWA) and ESA, are addressed in Section 9 of this DDR.

SECTION 2 - BIOLOGICAL DESIGN CONSIDERATIONS AND CRITERIA

2.1 GENERAL

The following section contains pertinent background and fish passage information that are being used during the design of the SWS, as well as the conveyance of fish collected in a fish collector being designed to be attached to the upstream face of the SWS. These fish will be transported by truck downstream to the Minto Fish Facility.

The NOAA 2008 Willamette BiOp identified the RPA to avoid jeopardy of ESA-listed fish in the Willamette Basin (NMFS 2008). Measure 5.2 in the RPA requires the Action Agencies to minimize water quality effects associated with operations of Detroit and Big Cliff dams by making structure modifications or major operational changes. While interim operational temperature control protocols have improved downstream water quality and temperatures, the current dam configuration does not provide the flexibility needed to meet downstream water temperature targets throughout the year.

The RPA contains categories of substantive measures for fish passage specific to Detroit Dam.

RPA 4.12.3 states: *"The Action Agencies will investigate the feasibility of improving downstream fish passage at Detroit Dam and if found feasible they will construct and operate downstream passage facilities. Temperature control will also be considered in designing the passage facility."*

The BiOp RPA also requires the collection and transport of fish from above Detroit to habitat downstream of Big Cliff and states: *"By March 2024, the Action Agencies will begin operating downstream fish passage facilities at Detroit that would enable collection and transport of fish from above Detroit to habitat downstream of Big Cliff Dam. Any necessary NEPA compliance required for implementation of proposed facilities will occur in conjunction with preparation of the Feasibility report."*

2.1.1 Criteria and Collaboration with Regional Fish Managers

Employed criteria for fish passage facilities, water flow, holding, and transportation can be found, in their entirety, in *Anadromous Salmonid Passage Facility Design* (NMFS 2011). Hydraulic criteria and considerations can be found in Section 4.3 of this DDR.

The Corps will coordinate the design of the SWS through the WFFDWG. WFFDWG members include the Corps, BPA and regional Federal, State and Tribal fish agencies and other partners.

The Willamette Fish Operations Plan (WFOP) is developed annually by the Corps in coordination with BPA and regional Federal, State and Tribal fish agencies and other partners through the Willamette Fish Passage Operations & Maintenance (WFPOM) coordination team. The WFOP is developed in accordance with the NOAA Willamette BiOp RPA Action 4.3 for the operation and maintenance of Willamette Valley dams and fish passage facilities to minimize impacts to fish. The WFOP is available online at the Willamette Fish Operations Plan Website:

http://pweb.crohms.org/tmt/documents/FPOM/2010/Willamette_Coordination/WFOP/

WFPOM coordination for the SWS construction will occur as needed taking into account the criteria in the WFOP. Minto adult trap operations and adult out-planting changes will be coordinated with the WFFDWG and WFPOM regional work groups prior to and during construction as well as through the NEPA process.

2.1.2 In-Water Work Window

The ODFW *Oregon Guidelines for Timing of In-Water Work to Protect Fish and Wildlife Resources* (June, 2008) recommends preferred in-water work windows for the protection of endangered species, sensitive species, and other game fish. Time periods were established to avoid the vulnerable life stages of these fish including migration, spawning and rearing. Every effort will be made within USACE authorities to minimize construction impacts to fish, wildlife, and habitat resources. While the construction site is located upstream of the dam, downstream impacts such as reduced river flows and water quality could be affected.

To minimize environmental impacts, the in-water work window is being coordinated through the NEPA process and will include an EIS for construction of this project and alternatives. The EIS evaluation of construction alternatives, associated construction schedule, and the Detroit Project operations for authorized purposes including fish and wildlife are the primary drivers for the development of the in-water work period for this project. Please see Section 9 for more information on environmental planning.

2.1.3 Species of Concern

The ESA-listed fish in the North Santiam River being addressed by this DDR include Upper Willamette River (UWR) spring Chinook salmon (*O. tshawytscha)* and ocean-maturing or winter-run steelhead (*O. mykiss*). Restoring the natural thermal regimes in the river reaches directly below Detroit and Big Cliff dams will provide a benefit to both of these ESA-listed species through the upstream run timing, adult collection at Minto, juvenile rearing, spawning, egg-incubation, and hatching.

Tables 2-1 and 2-2 are periodicity tables for spring Chinook and winter steelhead in the North Santiam River below Big Cliff taken from the North Santiam Subbasin WFOP.

Table 2-1. Periodicity Table for Spring Chinook in the N. Santiam River below Big Cliff Dam

Life Stage/Activity/Species	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Comments
Upstream Adult Migration													
Adult Spawning													
Adult Holding													
Egg Incubation through Fry Emergence													incubation & emergence accelerated 2-3 mo. because of warm water dam releases
													Emergence based on field observations and TU calculations
Juvenile Rearing													
All life stages													
Fry													peak period of rearing of fry based on trapping (1998) & field data (2011-2012);
Subyearling													subyearling primary rearing period (May-Aug) based on seining data
Fall migrant													subyearlings that do not migrate in first summer
Yearling													fish that remain through first summer & winter
Downstream Juvenile Migration													
Dec-Mar = fry													Fry movement based on field data (2011-2012)
April-mid July = subyearling													
Mar-May = yearling smolts;													
mid-Oct-mid Dec = fall migrants													
													Migration data based on PIT tag data, except fry movement

Represents periods of peak use based on professional opinion.
Represents lesser level of use based on professional opinion.
shaded cells represent information based on field data & direct knowledge
red cells represent critical periods when flow fluctuations should be avoided to prevent disruption of spawning,
to minimize disturbance of eggs during early incubation, and to minmize stranding or displacing newly emerged fry

Based on professional opinion, 90% of the life-stage activity occurs during the time frame shown as the peak use period.
Based on professional opinion, 10% of the life-stage activity occurs during the time frame shown as the lesser use period.

Table 2-2. Periodicity Table for Winter Steelhead in the N. Santiam River below Big Cliff Dam

Life Stage/Activity/Species	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Upstream Adult Migration												
Adult Holding												
Adult Spawning												
Egg Incubation through Fry Emergence												
Juvenile Rearing												
Downstream Juvenile Migration												

Represents periods of peak use based on professional opinion
Represents lesser level of use based on professional opinion
Represents information based on field data & direct knowledge
Represents critical periods when flow fluctuations should be avoided to prevent disruption of spawning,
to minimize disturbance of eggs during early incubation, and to minimize stranding or displacing newly emerged fry

Natural origin Spring Chinook salmon and winter steelhead are trapped at the Minto Fish Facility and released above the Minto barrier. Some hatchery origin spring Chinook are transported to designated release sites above the project dams.

The operation and maintenance of the Detroit and Big Cliff projects can impact downstream habitat conditions. The operations may alter flow conditions, both total flow and rate of change, and water quality, primarily temperature and TDG. The 2008 NOAA BiOp requires specific flow regimes below Big Cliff Dam. These operations include minimum and maximum flow targets, increasing and decreasing flow rate targets (ramp rates), and recommendations for operations during high flow periods. Flow rate and ramp rate requirements for Big Cliff Dam are detailed in the WFOP at the following link:

http://pweb.crohms.org/tmt/documents/FPOM/2010/Willamette_Coordination/WFOP/2018/final/2018%20WFOP%20Chapter%202%20North%20Santiam.pdf

Required minimum and maximum flows for Big Cliff Dam vary by time of year and are displayed in Table NS-2 of the WFOP. Minimum outflow from Big Cliff Dam is 1,000 cfs from July 16 to August 31. Spring spawning flows for winter steelhead are 1,500 cfs from March 16 to May 15, followed by incubation flows of 1,200 cfs lasting until July 15. Spring Chinook salmon spawning requires flows of 1,500 cfs from September 1 to October 15, followed by incubation flows generally through January 31. Maximum flows during spawning are 3,000 cfs, if possible.

Construction activities should minimize impacts to fish and habitat conditions upstream and downstream of the worksite to the greatest extent possible.

2.1.4 Other Species

Pacific lamprey *(Entosphenus tridentatus)* and ESA-listed bull trout (*Salvelinus confluentus*) are present in the Willamette Basin. There are a number of conservation efforts for lamprey and bull trout taking place to which the Action Agencies are signatories. Reintroduction projects for both species could expand in the future. Pacific lamprey and bull trout are not presently in Detroit Reservoir but have the potential to be present in the accessible range of salmonids which includes the lower North Santiam River.

Oregon chub, delisted in 2015, are present in the North Santiam River. Approximately 130,000 legal+ size and 200,000 fingerling size rainbow trout, as well as 25,000 kokanee, are stocked in the Detroit Reservoir on an annual basis to support sport fishery.

2.1.5 Water Quality

The NMFS 2008 BiOp considers elevated water temperatures caused by dam operations a primary limiting factor for the egg/emergence component of the UWR spring Chinook life stages in the North Santiam River due to premature hatching and emergence (NMFS 2008; ODFW and NMFS 2011). Interim temperature control operations consist of using the existing outlet configuration at the dam; spillway, powerhouse, and upper regulating outlets. The reservoir rule curve operations result in the forebay surface elevation dropping below the spillway during drawdown. As the pool continues to draft into autumn and early winter, the warm surface water approaches the penstock, resulting in releases of warmer than desired water in autumn/winter and until the lake has turned over (i.e., become isothermal).

Water temperature has been well-documented as a controlling factor in anadromous salmonid migrations (Major and Mighell 1967, Banks 1969, Quinn 1997, Dahl et al. 2004, Keefer et al. 2008). The interim operations fall short in the spring – warm water (from the upper part of the reservoir water column) is unavailable for release until the reservoir is above the spillway crest. This results in a release of cooler than desired water early in the year. Without water temperature control, the river reaches below Detroit and Big Cliff dams are much colder during the summer potentially causing migration delay to adult salmon.
A structural solution will be required to meet downstream temperature targets throughout the year and avoid jeopardy to ESA listed spring chinook and winter

steelhead. Interim temperature operations become even more limited during low water years where the pool elevation falls short of reaching the spillway crest or is at the spillway crest for shorter durations than the rule curve operation targets. Managing water temperatures in the river reaches below the dams with a structural solution that increases operational flexibility will better facilitate adult returns, spawning, and incubation closer to their historic timing. Structural improvements to temperature management will bolster the transport of adults throughout the run period to maintain genetic and life history diversity as well as improve habitat conditions downstream for rearing juvenile salmonids. Temperature management and fish passage designs identified in this report will strive for a no net impact to TDG. Water quality is discussed in detail in Section 5 of this report.

2.1.6 Downstream Fish Passage Considerations in SWS Design

The general goal for the SWS project is to increase the operational capability for downstream temperature management. A parallel DDR effort is underway to provide a high survival downstream fish passage alternative. Both projects share goals to not limit the ability to operate for flood risk management, power generation, and reservoir control.

Temperature control operations criteria and formal fish passage performance standards are currently being developed by the Corps in collaboration with NMFS and the regional fish managers. Detailed temperature control operations over the annual cycle continue to be evaluated to balance temperature control and fish passage once the SWS and FSS for downstream passage are constructed and ready for operation. More details on water quality can be found in Section 5.

The FSS PDT's working assumption of 95% fish collection efficiency (FCE) and 98% survival for fish transported below Big Cliff is based on the formal performance standards developed for Cougar downstream passage. FCE will be measured as the proportion of fish that are collected by the fish passage facility divided by the total number of fish in the "FCE measurement zone". The FCE measurement zone is an area upstream of the collector entrance that continues to be developed by the Action Agencies in collaboration with NOAA. This zone should be accurately defined, rooted in site specific ecology, and is necessary prior to FSS operation and development of post construction performance standard testing. These criteria and the project goals for the SWS and FSS PDTs have been a significant design driver in making these systems interconnected. Additionally, features such as guidance and/or exclusion nets and FSS pumped flow are being designed through the DET SWS and FSS DDRs respectively. Net design is directly linked to the FSS pumped outflow location and hydraulic performance which continues to be developed at this time. A formal document between the Corps and NMFS, similar to what has been agreed to for Cougar, that defines fish passage performance standards, adaptive management measures, and implementation criteria for these measures will be documented in the FSS DDR. .

The PDT is designing the SWS to be connected on the upstream face of the dam. The FSS is being designed by Architectural and Engineering (AE) contractors working closely with the SWS PDT. The AE is contracted to design the FSS to be compatible with the SWS connected to the dam with minimal disruption to the existing turbine operations. The design will be compatible with a lift system from the FSS to the deck of the dam for trap-and-haul of fish downstream to Minto Fish Facility and the provision to add 1000 cfs pumped flow in the future. The FSS will be connected to the two SWS HIWs and will move vertically with the annual forebay fluctuation cycle. The FSS will collect fish from the surface water flow that enters the SWS. The collected fish will be safely filtered from passage through the SWS and the two Francis turbine units. The turbines are the primary flow regulating mechanism for fish collection and temperature control.

The FSS intake weirs will be located just north of spill bay six facing south and perpendicular to the dam. The decision for a configuration that excludes the existing turbine intakes as a route of passage is supported by data on the poor passage conditions through the turbine route (Duncan and Carlson, 2011; Normandeau, 2011). In addition, study data shows significant numbers of juvenile-sized targets (Khan et al., 2012) and proportions of Juvenile Salmon Acoustic Telemetry System (JSATS)-tagged fish (Kock et al., 2015) entered this route under existing operations, and did so especially during the fall study period in 2013 (Beeman and Adams, 2015). Khan et al. (2012) reported approximately 86.5% of the fish passed through the turbines during the study period Feb. 2011 – Feb. 2012.

Once fish enter the FSS, they will be screened and diverted to a temporary holding facility until they are transported downstream by vehicle to the Minto Fish Facility or piped through the dam to a suitable location downstream of Big Cliff. (This is discussed further under “Downstream Fish Conveyance”, p. 2-13.) The primary drivers for SWS configuration at the dam (Fig. 4-4 and 4-5) include temperature control, compatibility with fish passage and power peaking operations, and constructability. The SWS design allows flexibility in operations to achieve temperature targets. Section 4 provides details on the SWS arrangement as well as hydraulic criteria and considerations. Section 5 provides the water quality and biological impacts discussion.

Distribution in the forebay

Movements of juvenile spring Chinook salmon (*O. tshawytscha*) and juvenile summer steelhead (*O. mykiss*) through Detroit Reservoir were studied over a two-year period from 2012 through 2014. The primary purpose of the study was to provide empirical data to inform decisions about future alternatives for improving downstream passage of salmonids at Detroit Dam. JSATS-tagged fish were released during spring and fall study periods to monitor fish migration. (Beeman and Adams, 2015.)

Groups of acoustic and passive integrated transponder-tagged (PIT-tagged) hatchery origin Chinook and summer steelhead, intended as surrogates for wild fish, were released to the two main tributaries several kilometers upstream of the reservoir (Table 2-3). Most inferences were based on an analysis period up to the 90th percentile of tag life. Transmitter life was about three months during year one. Year two fish had a shorter tag life, approximately 2.5 months. They were released over three-month periods in spring (March-May) and fall (Sept-Nov). Hydrophones were placed throughout the reservoir with two arrays near and at the dam. Array one was at the head of the reservoir and array six was near the log boom at the dam forebay. Detection probabilities were high for both species.

Table 2-3. Tributary Releases of Acoustic Tagged Fish

	2012		2013	
	Spring	Fall	Spring	Fall
Chinook (n)	468	514	394	606
Steelhead (n)	200	NA	229/*125	271

*125 released downstream Piety Island

Reservoir passage efficiency of Chinook and steelhead were estimated during both study years (Table 2-4). Tributary releases of Chinook had higher probabilities of reaching the reservoir than steelhead in the spring; however, once Chinook and steelhead were in the reservoir the probabilities of both species reaching the forebay of the dam were similar and fairly high. The movements of fall release groups of Chinook were similar to the spring. No steelhead were released to the tributaries during the fall of year one. Those that were released in year two had very low passage efficiency from release to the first array located at the head of the reservoir, as well as low passage efficiency through the reservoir to array six at the forebay to the dam.

Table 2-4. Tributary Releases – Reservoir Passage Efficiency (SE)

	2012		2013	
	Spring	Fall	Spring	Fall
Chinook	0.925 (0.013)	0.821 (0.018)	0.883 (0.018)	0.850 (0.015)
Steelhead	0.870 (0.030)	NA	0.855 (0.042)	0.286 (0.054)

The movement of both test species was directionally persistent (fish moving downstream tended to continue in that direction until reaching the dam) in the reservoir and fish accumulated in the forebay at the dam. Many fish made repeated trips from the head of the reservoir to the dam and back. This data helps support positioning the FSS that works in conjunction with the SWS and turbine operations close to the dam.

Qualitative examinations of tracks of fish within 105 meters of the dam indicate that the operating conditions affected fish paths near the dam. In general, fish exhibited milling behavior in the forebay near the dam where fish repeated travel along the face before returning upstream.

Position estimates of randomly selected Chinook and steelhead are provided in the report under various operating conditions which include: spill only, spillway +

powerhouse, powerhouse only, and regulating outlet + powerhouse operation. U.S. Geological Survey (USGS) provided randomly selected fish tracks and distribution of percent presence by operation. The data revealed fish densities and behavior within 105 meters of the dam. Fish passage rates were much greater during the spring and summer than in the fall and winter, and the difference was attributed to the availability and use of the spillway when operated during the spring and summer.

Fish densities within 25 meters of the dam were most concentrated near the dam during the spring when the spillway was operating, and were least concentrated near the dam in the fall when the spillway was not operating. The spring behavior is likely due to fish responding to surface flow when available. The mean depths of fish in the upper three reservoir elevation data bins were greater in the fall than the spring. Chinook behavior in the fall is influenced by thermal stratification of the reservoir as well as the lack of a surface route. However, data reported by Khan et al. (2012) during a “free flow” test conducted at the spillway during drawdown from September 23 through 27 showed a large increase in the total daily passage of juvenile sized fish.

During the USGS study in 2012, a two week weir spill operation was compared to the following two weeks of a normal spill condition. Weir spill is when water is passed freely over the spillway. Under normal spill operations, spill is controlled by the spillway tainter gates. With spill discharge controlled for, the passage rate during the weir condition was 3.1 times greater than normal spill. The success at the spillway in general as a passage route in the spring supports that properly designed surface collector could work at Detroit dam. The SWS and FSS will work together to pass surface flow and provide a free flow hydraulic drop with capture velocities of 8 fps at the FSS entrance. The entrance is being designed to maximize collection efficiency and minimize rejection of juvenile salmonids and steelhead kelt. Data from other locations such as the Foster weir, and mainstem Columbia surface routes at spillways and powerhouses suggests a hydraulic drop at the entrance may perform well.

The dam could be useful as a guidance structure to a surface route by positioning the FSS entrance close the dam. Maximizing surface flow at the entrance during the fall may influence behavior and allow opportunities for passage which might not otherwise be present when the reservoir surface dips below the spillway crest. One of the SWS design goals includes keeping the SWS profile as close to the dam as reasonably possible to maximize the potential for fish to discover the entrance to the FSS.

Dam passage occurred primarily during periods of elevated discharge and was most pronounced during the spring study period when spill occurred. Data and modeling support that the passage rate increased as spillway discharge increased. This suggests that maximizing surface flows through the FSS and into the SWS for temperature and turbine operations will maximize collection efficiency.

Horizontal distribution is significant for positioning the FSS as the FSS is being optimized for attraction to the entrance. The turbines will be the primary flow regulating mechanism for the FSS and SWS and large volumes of flow will maximize attraction.

The FSS, SWS, and turbine operations are designed to work with one another. More FSS operations details can be found in the FSS DDR.

Vertical distribution of salmonids in the area where the SWS is to be located is important for design and operation of the High Intake Weir (HIW) and Low Intake Gate (LIG) water intakes for temperature control and fish passage. - See Section 4 for a description of the HIW and LIG design and operations. The following is an excerpt of depth distribution results from Beeman and Adams 2015:

Depths of tagged fish within 25 m of the dam varied between species, reservoir elevation and diel period (fig. 1-28). When the reservoir elevation was less than 1,525 ft during the spring study period, which occurred as the reservoir was filling in March and April, Chinook salmon showed a large diel difference in hourly depths. Their individual mean hourly depths ranged from 1.3 to 107.0 ft, with mean values around 60 ft during the day and 27 ft during the night (table 1-4). When the reservoir elevation was greater than the spillway ogee of 1,541 ft during the spring study period (spill was present during much of this period), the mean of the median hourly depths of Chinook salmon ranged from 5.2 to 43.9 ft, were deeper during the day than during the night, and were highly variable (recall the fish depths were summarized as the mean among the median depths of each fish in each hour).

Depths of steelhead were shallower and less variable than those of Chinook salmon during the spring study period (fig. 1-28). Steelhead were only present within 25 m of the dam when the reservoir elevation was greater than 1,541 ft, except for one fish present when the reservoir elevation was between 1,450 and 1,500 ft. Their mean of the median hourly estimated depths ranged from 1.6 to 10.1 ft and were similar during the day and night during both elevation bins available.

Position estimates of Chinook salmon and steelhead were present over a wide range of reservoir elevations during the fall study period, but most fish were present when the reservoir elevation was less than 1,525 ft. Chinook salmon often were deeper during the day than at night, but their depths were highly variable (fig. 1-29, table 1-4). The mean of their median hourly depths ranged from 9.5 to 70.5 ft when the reservoir elevation was at least 1,450 ft, and from 15.4 to 50.9 ft when the elevation was less than 1,450 ft. Few steelhead were present in the reservoir during the fall study period, but the mean of their median hourly depths ranged from 7.1 to 68.2 ft when the reservoir elevation was between 1,450 and 1,500 ft.

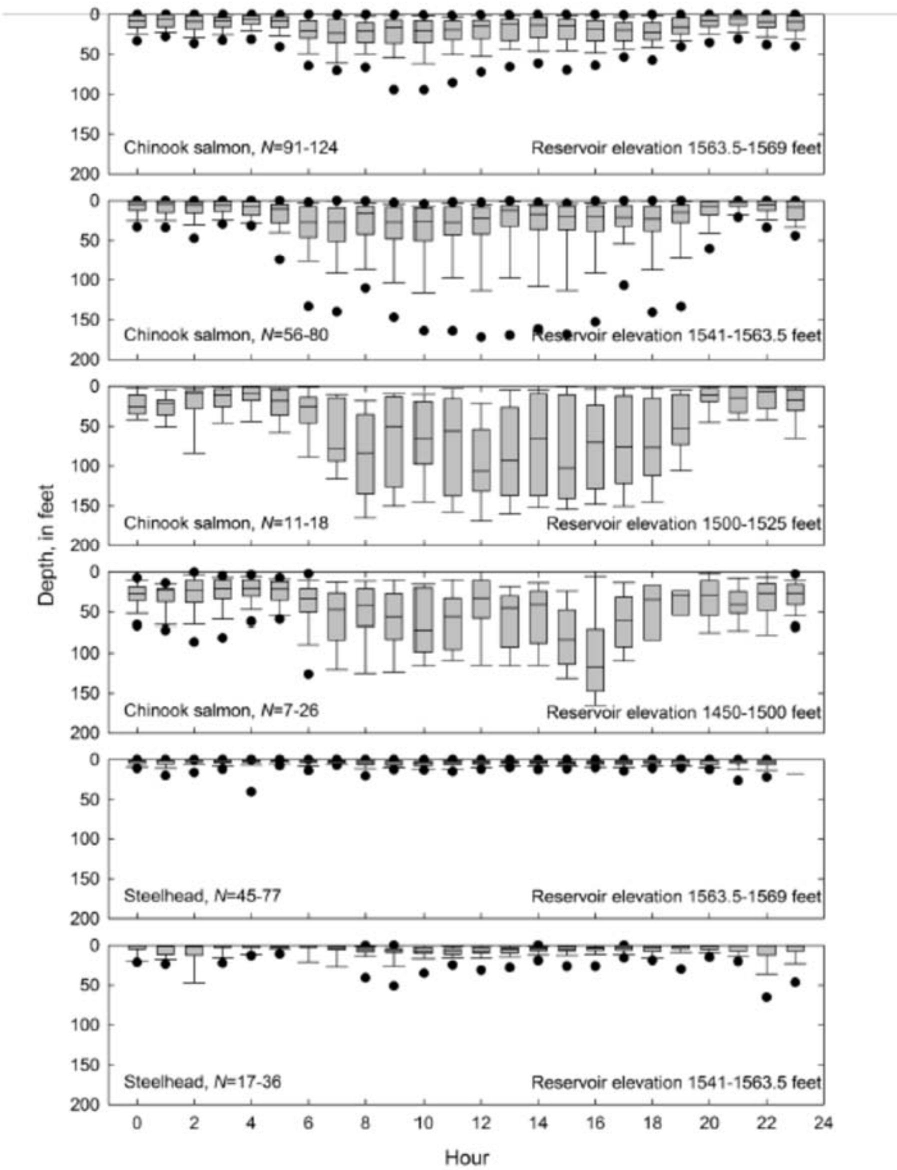

Figure 1-28. Boxplots of the hourly depths in feet of juvenile Chinook salmon and steelhead with position estimates within 25 meters of Detroit Dam, Oregon, during the 2013 spring study period. Data summarized are the median hourly depths of each fish present at the elevation ranges indicated. Boxes range from the 25th to the 75th percentiles with a line indicating the median, whiskers represent the 10th and 90th percentiles, and dots represent the 5th and 95th percentiles. Boxes without whiskers or dots contained insufficient data for them to be estimated. Sample sizes represent the number of fish (*N*) in the hourly boxes.

Figure 2-1. Depths of Juvenile Chinook Salmon and Steelhead With Position Estimates Within 25 Meters of Detroit Dam During 2013 Spring Study Period (Beeman and Adams, 2015)

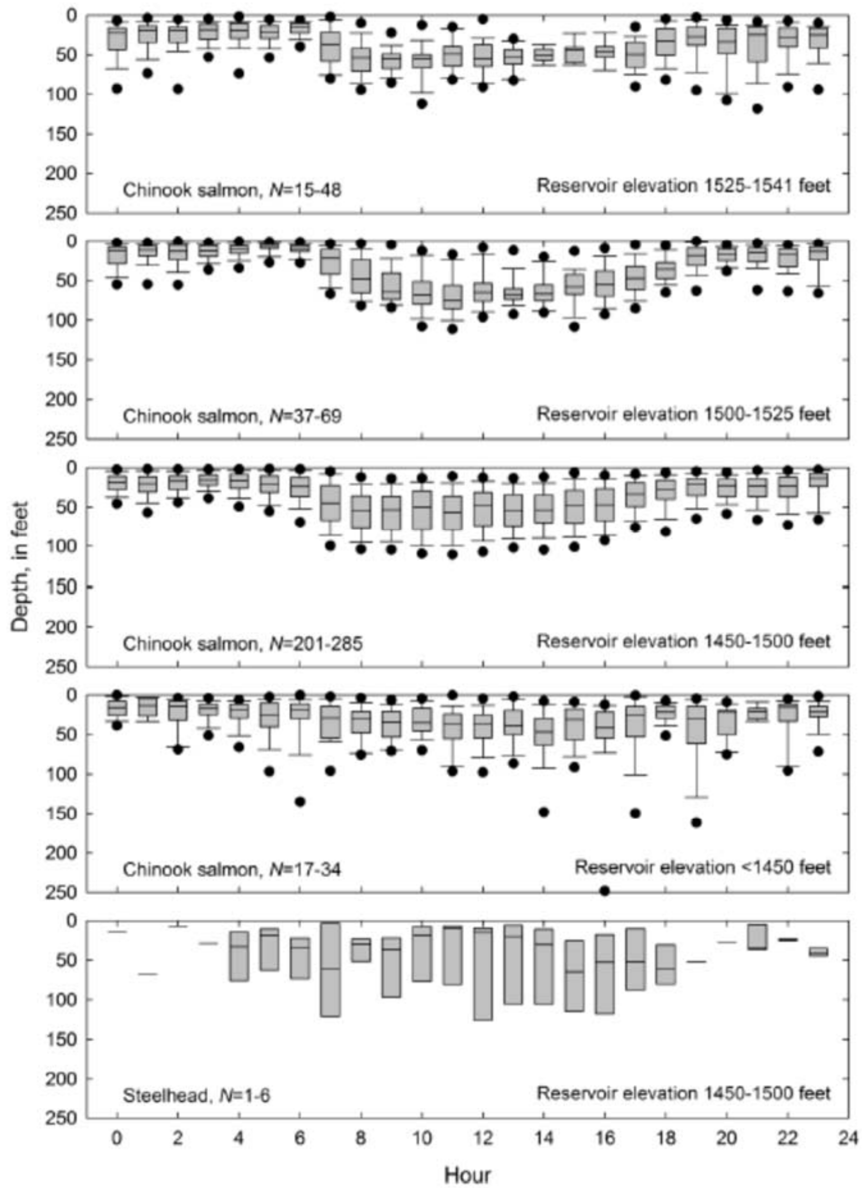

Figure 1-29. Boxplots of the hourly depths in feet of juvenile Chinook salmon and steelhead with position estimates within 25 meters of Detroit Dam, Oregon, during the 2013 fall study period. Data summarized are the median hourly depths of each fish present at the elevation ranges indicated. Boxes range from the 25th to the 75th percentiles with a line indicating the median, whiskers represent the 10th and 90th percentiles, and dots represent the 5th and 95th percentiles. Boxes without whiskers or dots contained insufficient data for them to be estimated. Sample sizes represent the number of fish (*N*) in the hourly boxes.

Figure 2-2. Hourly Depths of Juvenile Chinook Salmon and Steelhead During 2013 Fall Study Period (Beeman and Adams, 2015)

Table 1-4. Summary of the mean of the median hourly depths of each fish with position estimates within 25 meters of Detroit Dam, Oregon, during the 2013 spring and fall study periods.

[Reservoir elevations are expressed in feet. ≥, greater than or equal to; < less than; sample size, the number of fish from which the depths were estimated; SE, standard error; NA, not applicable. Elevation bins without data are not shown]

Season	Species	Reservoir elevation Bin	Diel period	Sample size	Mean (feet) Depth	Mean (feet) SE
Spring	Chinook salmon	≥1,563.5	Day	200	23.42	24.27
			Night	199	9.10	7.86
		1,541 to <1,563.5	Day	163	30.57	34.96
			Night	156	8.21	9.11
		1,525 to <1,541	Day	4	30.37	44.29
			Night	4	7.29	5.35
		1,500 to <1,525	Day	22	61.30	52.56
			Night	23	27.48	27.47
		1,450 to <1,500	Day	23	57.22	33.86
			Night	28	27.35	11.73
	Steelhead	≥1,563.5	Day	105	3.93	4.27
			Night	100	2.56	2.99
		1,541 to <1,563.5	Day	67	4.98	6.12
			Night	54	3.80	8.56
		1,450 to <1,500	Day	0	NA	NA
			Night	1	29.89	NA
Fall	Chinook salmon	1,541 to < 1,563.5	Day	1	35.50	NA
			Night	1	30.69	NA
		1,525 to <1,541	Day	53	54.57	15.01
			Night	56	24.09	16.93
		1,500 to <1,525	Day	84	59.59	20.05
			Night	98	15.75	12.43
		1,450 to <1,500	Day	344	54.27	24.45
			Night	395	22.24	13.48
		<1,450	Day	64	40.49	21.11
			Night	71	29.70	20.51
	Steelhead	1,525 to <1,541	Day	1	36.86	NA
			Night	0	NA	NA
		1,500 to <1,525	Day	4	14.65	9.57
			Night	2	15.58	7.93
		1,450 to <1,500	Day	7	39.09	42.85
			Night	10	29.02	27.47

Figure 2-3. Mean Median Hourly Depths of with Position Estimates Within 25 Meters of Detroit Dam During 2013 Spring and Fall Study Periods (Beeman and Adams)

The FSS utilizes fish behavior in design features. It will be connected to the SWS close to the dam and is based on the high probability of fish in the reservoir reaching the dam, the depths of fish, fish behavior near the dam, and maximizing the surface route opportunity for passage that works with SWS temperature operations. Fish depth varied by species, reservoir elevation, and diel period. Both species were at shallow depths throughout the study periods which suggest they would be available for passage if a surface route were available. The FSS entrances will be perpendicular to the dam utilizing the dam as a guidance feature. The design of the SWS for temperature control includes operational flexibility of the HIW to access surface water through the reservoir elevation range of 1,425 to 1,570 msl. The SWS and FSS will screen the surface water that enters the tower, meeting (the 90^{th} percentile of annual) power peaking flow rates. The FSS design flow range for meeting NMFS screen criteria is 1,000-4,500 cfs with the ability to operate up to 5,900 cfs. This design flow range will maximize the hydraulic signature in the forebay and minimize competing flow since the turbine route will be excluded and large volumes surface water will pass through the collector into the SWS, encouraging attraction and high fish collection efficiency.

Fish Interactions with Regulating Outlets and the SWS Lower Intake Gate

Khan et al., 2012 conducted a hydroacoustic evaluation of juvenile salmonid passage and distribution at Detroit Dam in 2011. A summary of results were reported in the *Detroit and Big Cliff Long-term Temperature Control and Downstream Fish Passage* Engineering Documentation Report, July 2017. Researchers using hydroacoustic data were able to discriminate between size classes of fish but not between salmonid species and non-salmonid species. For the purpose of analysis in the study, smolt-size fish were defined as 90mm < fork length < 300mm. The lengths are approximations based on acoustic target strength. The hydroacoustic data likely included juvenile Chinook salmon, as well as kokanee (land-locked sockeye salmon), since both were captured in the ODFW tailrace screw trap when it was operated between April and December 2011. Dam operations at Detroit during 2011 involved opening the RO in late summer and fall for downstream water temperature control, and during winter to discharge excess water beyond turbine discharge capacity.

Khan et al., 2012 estimated that 23,339 smolt-size fish (± 572 fish, 95% CI) passed via the RO when it was open from October 29 – November 12, 2011, January 2–6, 2012, and January 20 – February 3, 2012. During the October–November period, RO passage peaked at 1,086 fish on November 5, with a second peak on November 7 (1,075 fish). (The turbines were out of service from November 1–8 and all water passed the dam through the RO during this period.)

Diel distribution for RO passage was variable, indicating fish were passing the RO at all times of the day. For the two analysis periods for the RO, acoustic sizes corresponded to 105mm fish in November and ~60mm fish in January.

There is a potential risk of entrainment in the LIG for juvenile salmonids and kokanee that are annually stocked in Detroit reservoir for the sport fishery. The LIG have been configured to provide effective temperature operations and minimize velocities at the trashrack.

The LIG intake description, design information, and operations are provided in Section 4, Hydraulic Design. The four LIGs (Figure 4-4) on the SWS are sized based on assuring turbine capacity and potential cold water requirements to meet autumn target temperature. They are arranged in stacked pairs on the east face of the SWS at invert elevations 1,327 ft and 1,305 ft. The low level inlets will have operational slide gates with dimensions 10 ft tall by 15 ft wide and be able to collect 200-6200 cfs. The coarse trashracks are sized to reduce velocity and secured to a hood that will be positioned over a pair of LIG. Each trash rack hood will be 22 ft wide by 44 ft tall. Bubbler beams between the trashrack and the LIG will help evenly distribute velocity across the intake. The velocity at the trashracks of the LIG will be 4.3 ft/s at maximum turbine flow.

Detroit Dam has four ROs at slightly higher elevations. The operational gates are the two upper RO intakes at centerline elevation 1,340.0 ft. The RO gates are 5 ft 8 in wide and 10 ft high. (See Section 4 for more information on ROs). The hydraulic capacity of the upper RO tunnels is 13,050 cfs. The RO intake velocities at face of dam is 24 ft/sec at 5,000 cfs per RO and 15 ft/sec at 3,100 cfs per RO.

The RO velocities and flow volumes at maximum openings are considerably higher than the LIGs and the ROs would provide more attraction for fish. Other research suggests juvenile salmonids are generally reluctant to sound to pass dams, but when surface outlets are not available, they will sound through a relatively deep outlet. The vertical distance between maximum conservation pool (1,563.5 ft) and the upper LIG (1,327 ft) is 236 ft. The vertical distance at minimum power pool (1,425 ft) is 98 ft. Vertical distribution data at the dam showed variability over the study periods. Eliminating the turbine route, lower flows compared to the ROs at the LIG trashracks, and maximizing surface flow for fish attraction while balancing temperature operations will minimize risk of LIG fish entrainment.

Many fish smaller than the size in the behavior and passage studies will be out-migrating from the reservoir and it is uncertain they will behave the same way. Reconfiguration and operations of the water outlets for temperature control and fish passage may change vertical distribution near the dam to some extent over the annual cycle, benefiting collection. Post construction evaluations should include behavior and passage metrics from previous studies at Detroit to evaluate changes. The interconnected nature of the SWS, FSS, and turbine operations are necessary to optimize temperature control operations and fish collection efficiency through the reservoir elevation change with minimal disruption to the Detroit project authorized purposes.

2.1.7 Downstream Fish Conveyance

Bypassed fish released into the Detroit dam tailrace would be subjected to passage at Big Cliff Dam. A release location downstream of Detroit and upstream of Big Cliff would not meet survival performance criteria of 98% without significant modifications to Big Cliff operations and configuration. This passage alternative was deprioritized in the July 2017 EDR for this project.

The scope of the SWS design includes trap-and-haul to Minto as the primary location for releasing fish.

In general, NMFS requires that downstream migrating fish pass a water-withdrawal project under their own volition, meaning under their own swimming capabilities and of their own timing, and not in equipment that transports them to a downstream release location. This is due to the risks associated with the handling and transportation of juvenile migrants, as well as the potential interruption of maintenance and collection operations and transportation facilities given the long-term nature of these programs.

Vehicle transportation of downstream migrants to a safe location for release is expected to be a viable option for Detroit Dam. The trap-and-haul alternatives continue to be evaluated while working with the FSS AE contractor. The SWS/FSS PDT is investigating methods to move fish from the FSS to a vehicle. A dedicated non-personnel lift system for fish tanks, aka “transport pods”, from the FSS to the deck level of the dam is being designed.. More fish conveyance details from the FSS to transport vehicles and then to Minto Fish Facility will be provided in the 95% FSS DDR and Section 7, Mechanical Design. .

High head fish passage conveyance principally falls into two categories; trap-and-transport and volitional bypass. For the Phase 1 Detroit FSS DDR, the AE firm was tasked with developing a trap-and-transport method for downstream fish conveyance. In order to evaluate the feasibility and applicability of a volitional bypass the USACE Portland District formed the High Head Bypass PDT, which is currently developing a design parameters document. This document will guide the development of bypass alternatives by the PDT, which will be done in a separate EDR evaluation. The bypass feasibility and design will be evaluated by the High Head Bypass PDT in close collaboration with the Detroit downstream passage PDT.

2.1.8 Minto Fish Facility

Adult spring Chinook salmon and steelhead needed for ongoing fish management activities in the North Santiam subbasin are collected at the Minto Fish Facility located on the North Santiam River. The facility is owned by the Corps and operated by ODFW. The Minto Fish Facility consists of a fish ladder, presort pool and crowder, sorting flume, eight post-sort holding ponds fed by pumps, and many other features that accommodate both holding adult salmon and steelhead as well as acclimation of juveniles. The Minto Fish Facility is the primary location for a vehicle transport juvenile release site. Minto will operate to provide safe transfer of fish from vehicle trap and haul tanks to a designated raceway prior to release of fish downstream. Minor modifications may be needed to support the safe transfer of fish from the vehicle to the raceway.

Minto operations during construction may be impacted. Minimum flows for Minto operation have been identified at 700 cfs. Impacts to facility water intake at the intake screens may occur due to low flows and elevated debris. Increased attention during these periods, as well as debris removal with manual methods, may be necessary to minimize impacts. Impacts and alternatives are being evaluated for holding adult fish. Detailed plans will be included in the NEPA impacts analysis for construction alternatives.

2.1.9 Predation

SWS structures added to the forebay side of the dam should provide little additional habitat for predators.

2.1.10 Post Construction Evaluations

The goal of this project is to meet temperature control targets downstream of Big Cliff Dam while being compatible with downstream fish passage and minimizing impacts to the Detroit project authorized purposes. Monthly temperature targets for the North Santiam River below Big Cliff Dam are identified in the WFOP. See Table 5-1 (page 5-2) for temperature targets based on species and life stage. Design of the SWS will provide flexibility in operation to meet modified temperature targets if desired. Temperature targets may be refined through regional collaboration in the future as new information comes in to optimize adult migration conditions in the North Santiam River, juvenile rearing downstream of Big Cliff, FSS collection of fish in the forebay, downstream spawning, egg incubation and emergence timing.

Monitoring of downstream water temperature can be completed using USGS gages at Niagara or other temperature meters in the North Santiam River where appropriate. Temperature monitoring in the vicinity of the SWS and FSS intakes will be necessary for operations to achieve downstream temperature targets. Short and long-term field studies of juvenile and adult fish will be necessary to evaluate performance of the SWS and FSS. These may consist of active tag studies to evaluate route specific passage and behavior, passive studies with technology such as hydroacoustic and/or DIDSON cameras, and direct capture of juvenile and adult fish at locations to estimate species composition, run timing, and abundance. Post construction performance evaluations and monitoring will be developed through the regional Research, Monitoring, and Evaluation program as appropriate.

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SECTION 3 - GEOTECHNICAL DESIGN

3.1 GENERAL

This section summarizes the existing regional and site geologic conditions, the probable foundation conditions, and geotechnical design and construction considerations for the proposed SWS for downstream juvenile fish passage at Detroit Dam.

3.2 LIMITATIONS OF GEOTECHNICAL DATA

No new geotechnical explorations or testing have been performed for the design of the SWS or for the FSS excavation. The establishment of geological features and conditions, along with adoption of the geotechnical design values described here, are based upon review of extensive prior site explorations and testing, historic construction records, and other geotechnical efforts around the facility. These project-specific geologic references and records are listed in Section 3.3.2. Due to the nature of the site-specific geology, foundation conditions encountered during original dam construction are expected to be representative of areas located immediately upstream of the dam. For the SWS tower, approximately half the footprint of the tower foundation was previously excavated, prepared for concrete placement, and geologically mapped. The remaining half of the foundation is located within 20 ft of the previous preparation and mapping and is expected to be the same. The only new information that may be potentially beneficial to the design analysis is the measurement of shear wave velocity for seismic site classification. This work is underway through other studies and will be available by the end of FY 2019. The FSS excavation area is located roughly 200 ft from the dam and the previously recorded explorations. The information from the dam is sufficient to design the excavation. The purpose of the excavation is to provide space to permit the FSS to float at low pools. Finished rock surfaces need only be made to approximate grade to provide space for the floating structure. Slopes will be laid back as flat as practicable so as not to require any slope stabilization. There are no requirements for foundation preparation other than removal of loose rock to prevent sliding during seismic events. FSS explorations may be conducted during main tower construction for the purpose of permanent record documentation of subsurface conditions for dam safety more than relevance for excavation.

3.3 GEOTECHNICAL REFERENCES

3.3.1 Geotechnical Design Requirements

Geotechnical design will conform to the following Engineering Manuals (EMs) and Engineering Regulations (ERs).

U.S. Army Corps of Engineers (USACE). EM 1110-1-1807 Geotechnical Investigations, 20 February 1984.

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3.3.2 Project-Specific Geologic References

The following are used to characterize regional and site geology and the site foundation conditions.

Amec Foster Wheeler Environment & Infrastructure, prepared for U.S. Army Corps of Engineers Risk Management Center. Seismic Hazard Analysis for Six Dams in the Willamette Valley, Oregon. June 2017.

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U.S. Army Corps of Engineers (USACE), Portland District. Foundation Grouting Report, Detroit Dam, North Santiam River, Oregon. May 1953.

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U.S. Army Corps of Engineers (USACE), Portland District. Detroit Design Memorandum No. 4, Detroit-Big Cliff Earthquake and Fault Study, North Fork Santiam River, Oregon. September 1983.

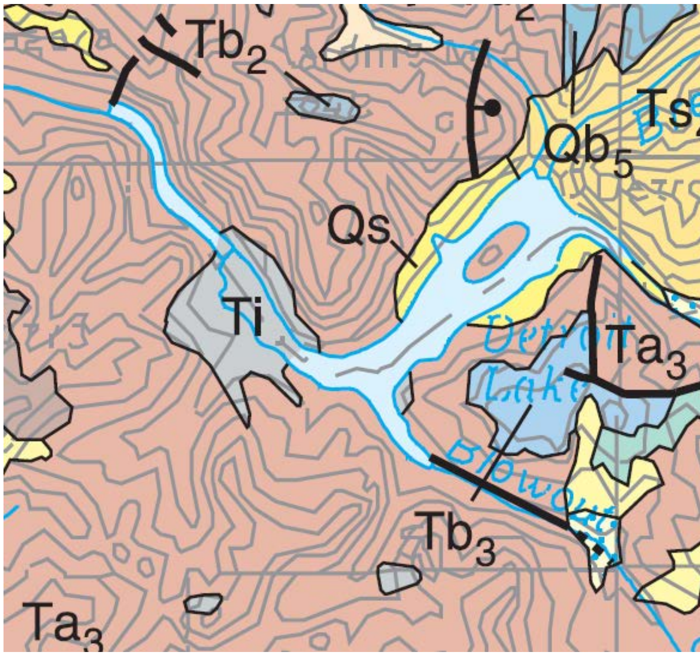
U.S. Army corps of Engineers (USACE), Portland District. Periodic Inspection No. 11 - Periodic Assessment No. 2, Detroit Dam (OR00004), North Santiam River, Oregon. December 2016.

U.S. Geological Survey (USGS). 2014 U.S. Seismic Design Maps https://earthquake.usgs.gov/hazards/designmaps/.

3.4 EXISTING GEOLOGIC CONDITIONS

3.4.1 Regional Geology

The dam site is located in the older Western Cascades geologic province, an Oligocene-Miocene age volcanic range composed of an 8,000-ft thick heterogeneous sequence of poorly stratified volcanic lava flows, volcaniclastic sandstone and mudstone, tuffaceous debris-flow deposits, and welded and non-welded tuffs of the Little Butte and Sardine formations. Non-conformities and unconformities are present. To the east is the High Cascade geologic province composed of the younger Pliocene to Holocene basaltic and andesite volcanic lava flows of the currently active volcanic range. Regional geologic mapping is shown in Figure 3-1. Geologic mapping in the Western Cascades has generally been at a reconnaissance level.



Geologic Units

Qs Quaternary sedimentary deposits
Qb_5 Quaternary basaltic and basaltic andesitic rocks
Tb_2 Miocene basaltic and basaltic andesitic rocks
Ta_3 Miocene andesitic rocks
Tb_3 Miocene basaltic and basaltic andesitic rocks
Ts_4 Oligocene sedimentary rocks

Ti Hall Diorite (Intrusive)

Figure 3-1. Regional Geologic Mapping In the Vicinity of Detroit Dam
Source: Sherrod and Smith, 2000

The older volcanic deposits were formed as a series of volcanic centers (mountains) composed of poorly stratified basaltic and andesitic lava flows, and fragmental breccias and tuffs. In the low lying areas between the volcanic centers, volcaniclastic sediments, tuffaceous debris flows and welded tuffs were deposited. Hypothetical geologic cross-section showing the distribution of typical rock types in the Cascades is shown in Figure 3-2. Geologically, Detroit Dam is located at shallow-depth magma chamber beneath one of the older intermediate strato-volcanoes. The dam is founded on the northwest margin and roof of an andesitic and micro-diorite stock.

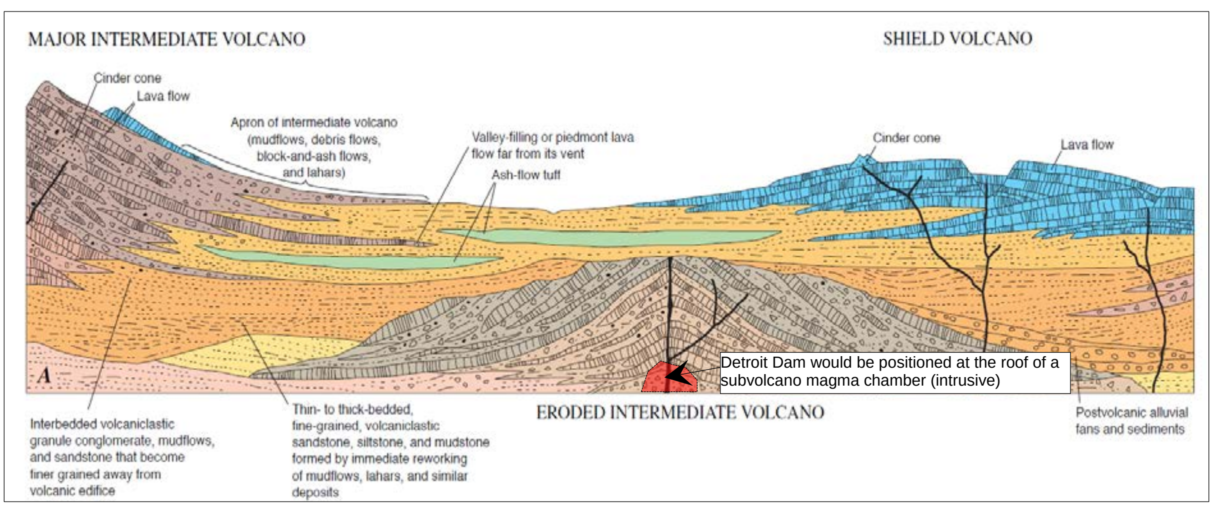


Figure 3-2. Hypothetical Geologic Cross-Section Through Western Cascade Volcanic Range - Source: Sherrod and Smith, 2000

These deposits were later faulted and intruded by basaltic-dacite dikes. They were later covered by younger middle and upper Miocene basaltic-andesitic lava flows with some pyroclastic interbeds.

Faulting and intrusives played an integral part in the structural phases of the rock with hydrothermal processes altering the rock along the major joints and faults. One of the larger shallow intrusions in the Oregon Cascades is the Hall Diorite located in the Detroit Reservoir area. This intrusion is the magma chamber below a major intermediate volcanic center that has been eroded away, exposing it. The damsite is along the northwestern margin of the pluton. Earlier design reports mapped and described the margin rock as a thick andesite and andesite breccia. Modern mapping and geologic interpretation is that this breccia represents a shallow brecciated intrusive phase of the intrusion (Pungrassami, 1969, Oregon State University MS Thesis). Brecciation of the intrusive may have occurred due to degassing of the magma as a result of a violent volcanic eruption of the overlying Miocene age volcano. The breccia fragments were re-welded together to form a massive rock that retains a visibly fragmented texture.

3.4.2 Site Geology

3.4.2.1 General

Detroit Dam is founded on Hall Diorite near the northern margin and roof of the pluton. The exposed portion of the pluton is about 2 to 3 square miles in size (Figure 3-3). A larger version of the map is included in the Geotechnical Appendix E. An abundance of dikes, sills, and smaller intrusions around the margin suggest that the magma chamber or pluton expands at depth. The Hall Diorite plutons intruded into the lower member of the Sardine volcanic country rock which is exposed above the pluton on both the right and left abutments. Total thickness of the pluton at Detroit is unknown but is likely greater than 1,000 ft. The lower member of the Sardine is composed of stratified tuffs, tuff breccias, andesite flows, and volcanic sedimentary rocks. Total thickness is about 3,000 ft.

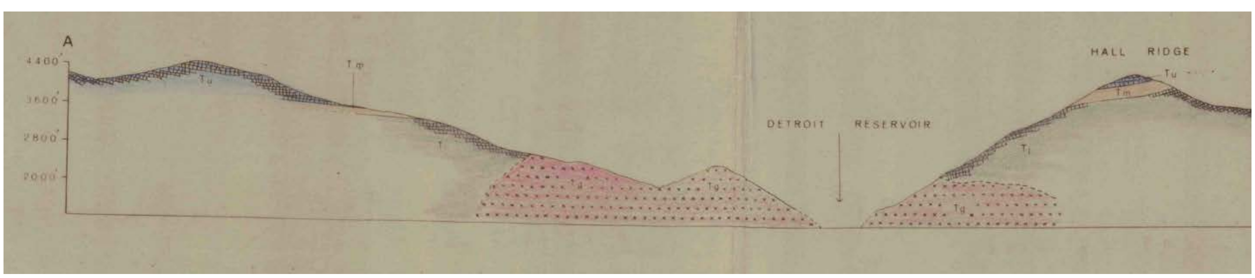


Figure 3-3. Geologic Section Through Detroit Reservoir
Source: Pungrassami, Thongchai, 1969

3.4.2.2 Explorations

The site was extensively investigated during the original design for the dam. Investigations included geologic mapping of the dam site and a large drilling program.

An aerial geology map of exposed bedrock at the dam site is provided in Appendix E, Page E-2. Orientation of about 150 joints shown on the geologic map were measured and tabulated. The tabulated list and field descriptions were developed during original geologic characterization and were published in the original design studies. New work will entail measuring joint orientations and conducting stereoplot analysis. Stereoplots for numerical number of joints and for length weighted (persistence) joints will be done. Purpose of the analyses is to ensure that, based on all known geologic information, the foundation is stable. The results will be provided in the 90% DDR.

Table 3-1 summarizes explorations conducted for the original design.

Table 3-1. Design Phase Explorations Conducted at Detroit Dam

Explorations	Amount
Geologic mapping of natural rock exposures and joints	
Diamond drill holes	
Vertical drill holes	62
Angle drill holes	12
Sizes smaller than 6-inch-diameter	
Number	71
Footage in overburden	2,391.6 ft
Footage in bedrock	4,661.2 ft
6-inch-diameter	
Number of holes	4
Footage in overburden	24.9 ft
Footage in bedrock	136.7 ft
36-inch-diameter calyx holes	
Number of holes	5
Footage	282.8 ft
Trenches	250 ft
Tunnels	460 ft

The foundation explorations conducted in the immediate vicinity of the proposed SWS and FSS is shown in Figure 3-4.

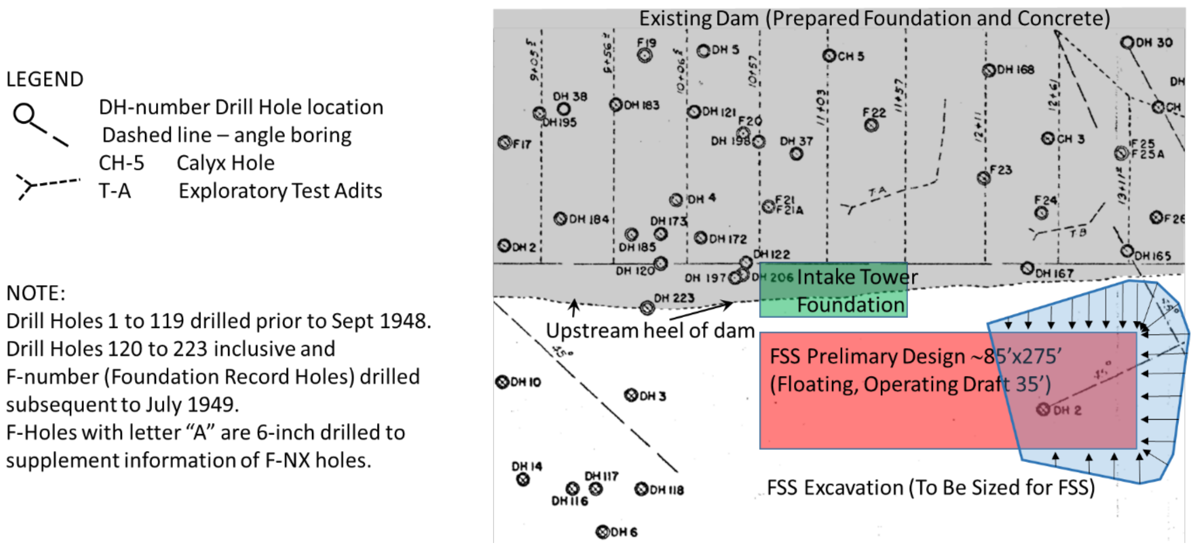


Figure 3-4. Existing Explorations Near SWS Tower and FSS (Conceptual Excavation)

3.4.2.3 Foundation Geologic Mapping

The foundation of Detroit Dam was geologically mapped before concrete was placed. Complied foundation map is shown in Figure 3-5 and in larger form in the Geotechnical Appendix E, Page E-21. The scanned foundation map is acknowledged to be poor quality and the geological hand written text (critical joint orientation) is not legible. The original has been located and a second scan attempted. While the new scan is at higher resolution and text is readable and useable, the image, unfortunately, was distorted. The map will either be rescanned or digitized for P&S.

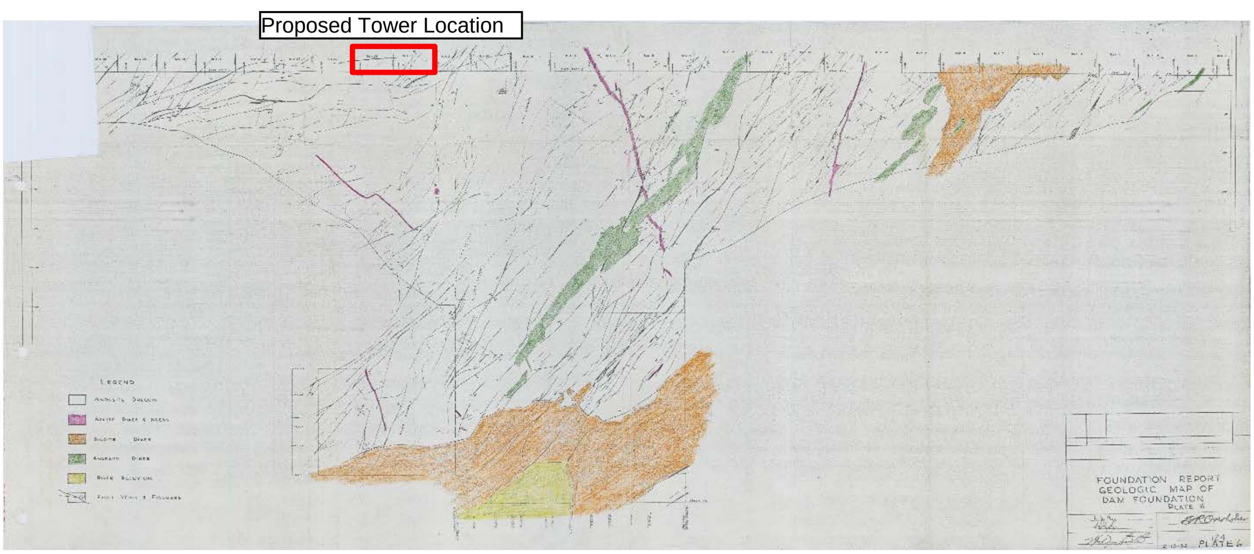


Figure 3-5. Foundation Geologic Map

The foundation of the dam was taken down to sound rock to support the massive loads of concrete gravity monoliths. Maximum height is 463 ft (load approximately 643 ksf). The proposed attached SWS tower will be on sound rock of the same geologic unit. Since the attached tower is hollow and much lighter in weight, the foundation is expected to be adequate. The existing detailed foundation mapping provides an insight into the foundation flaws and joint orientations. Enlarged detail of the original foundation map at the location of the intake tower is shown in Figure 3-6. About half of the foundation for the tower was prepared and mapped, and had concrete placed over it. No further geotechnical/foundation work is required. The remaining half, if not already excavated, will be excavated by controlled blasting in small lifts to the top of sound rock; it is expected to match the original final excavation. Blasting in small lifts is expected to cause less vibration than attempting to mechanically excavate by chiseling. Blasting provides greater control over creating the stair-stepped benches required to found the tower on. Geologic mapping indicates the structure will be founded on andesite and andesite breccia. Scattered drill holes located upstream and both left and right of the tower footprint indicate that this andesite continues upstream.

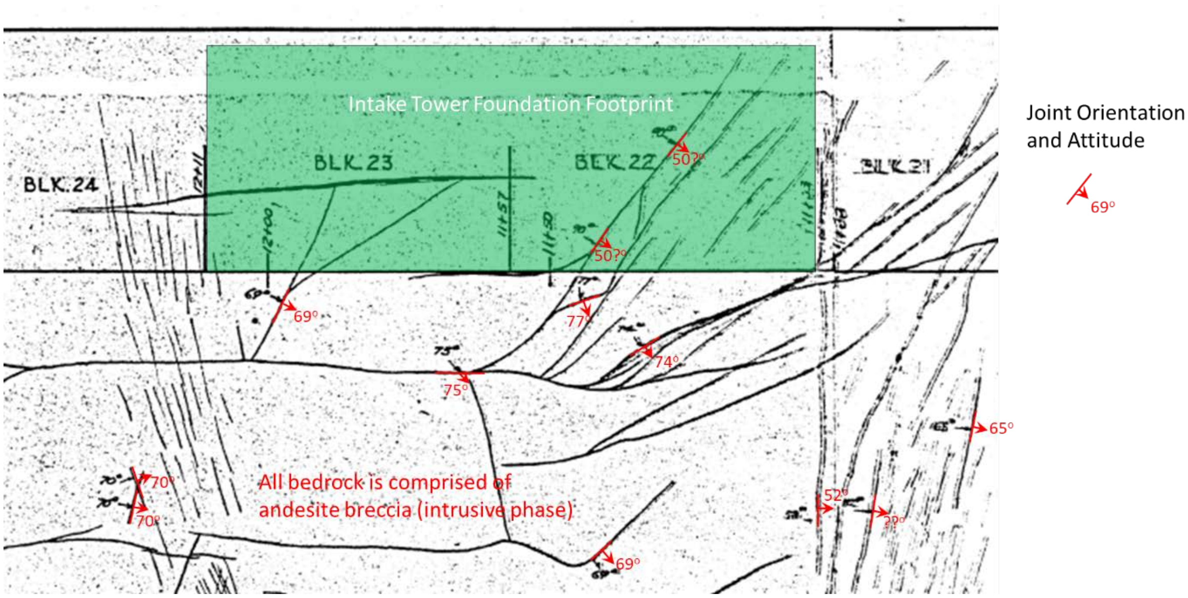
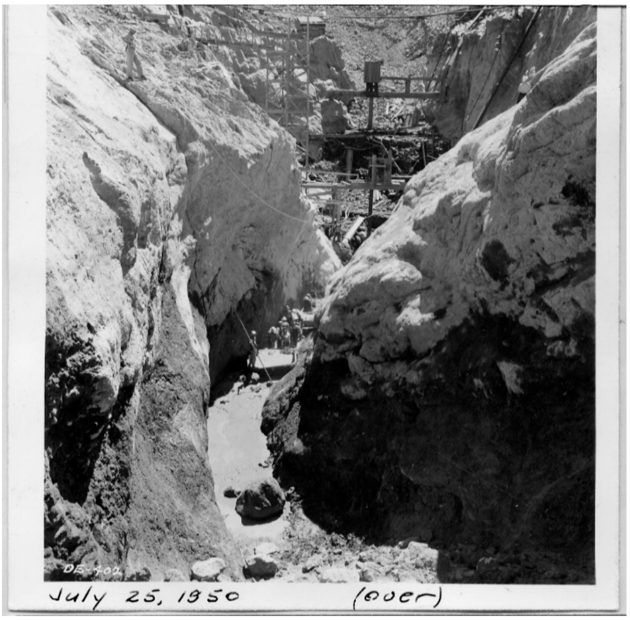


Figure 3-6. Detailed Foundation Geologic Map at the SWS Tower Location

3.4.2.4 Overburden

Depth of overburden at the site varied between 0 and 70 feet. Overburden consisted of weathered surface rock, talus, river alluvium, glacial debris, and remnants of higher, old cemented-terrace river gravels. All overburden and a few feet of weathered rock were excavated and removed from the foundation footprint of the dam. Two buried river channels were uncovered during foundation excavation. The main active river channel thalweg was a narrow channel incised nearly to a depth of 60 ft (Figure 3-7). This channel is located at blocks 18 and 19. Walls of the channel are near vertical to slightly overhanging. All gravels and boulders were removed from the channel within the footprint of the dam. However, the alluvial gravels and boulders are still present upstream of the dam outside of the footprint. The presence of this channel upstream of the dam at blocks 18 and 19 will not impact the foundation or constructability of the proposed SWS tower.



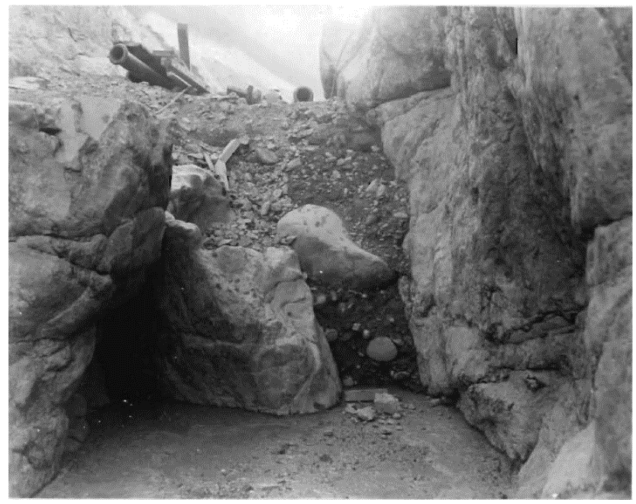


Figure 3-7. Incised river channel. Right upstream view monolith 19 (RO1). Left channel alluvial gravels and boulders backfill in the powerhouse area.

An older second channel was uncovered beneath blocks 28 and 29 around elevation 1,410 on the right abutment. The northern wall of the channel was vertical to overhanging. The channel was filled with a glacial deposit overridden by glacial ice. The deposit was compacted and impervious and required blasting to remove. The material filling this channel was removed and added to the upstream talus slope deposit. Implications of this higher channel suggest there may be a geologic weakness (closely spaced fissures) that may trend about 100 ft north of the FSS excavation. This possibility and potential impact to the FSS excavation will be covered in the 90% DDR. Currently, the plan is to lay the rock slope back 1V:1H and mitigate any adverse potential impacts.

The thickest remaining overburden is the weathered rock and dumped rock from highway construction on the right abutment upstream side (Figure 3-8) and on the left abutment (Figure 3-9). Thickness of the talus is not known, but protrusions of the underlying bedrock can be seen in the photos so it is not believed to be excessively thick.

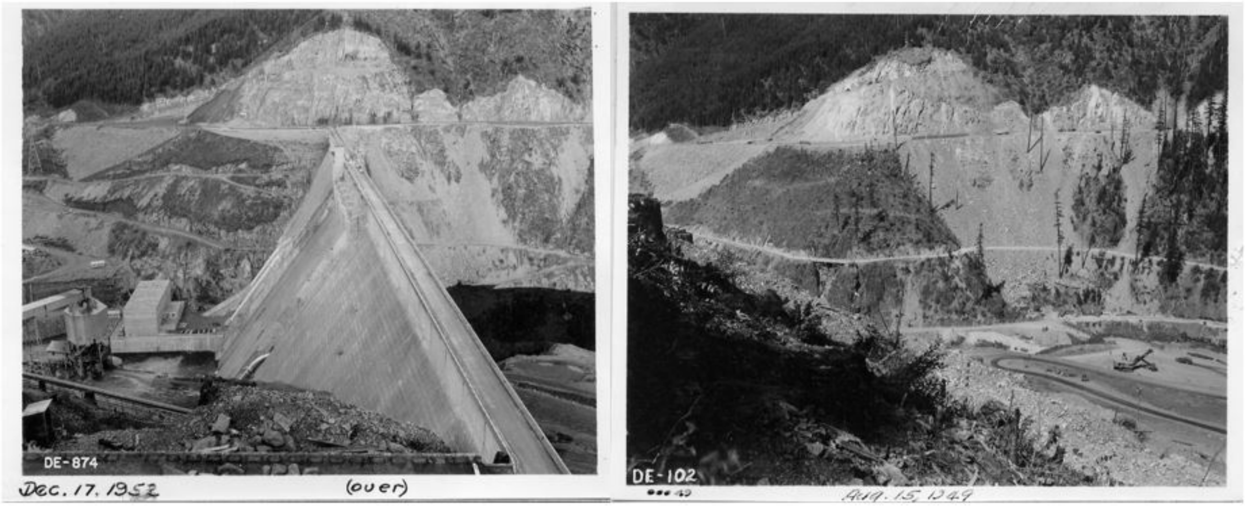


Figure 3-8. Right Abutment

Figure 3-9. View of Left Abutment Construction Fills

3.4.2.5 Bedrock

The bedrock at the dam consists of the following, in order of abundance: andesite breccia, diorite, aplite, andesite porphyry, hydrothermally altered phases of these rocks, and vein material composed of crushed vein matter, quartz and traces of hematite, lead, and zinc minerals. The andesite breccia occurs along the northwestern and western margin of the intrusion. It is unstratified and over 600 ft thick. The brecciated fragments are all reconsolidated in a solid rock mass. Bedrock is similar to jointed granite and has many of the properties of granite.

The rock is generally hard and brittle. The andesite breccia exposed in the foundation is variably jointed and faulted, and the rock mass has undergone intrusions and alteration from later phases of the intrusion process. Hydrothermal alteration due to migrating hot geothermal fluids occurred along major joints and fissures. This alteration has changed the minerals of the rock and formed clays which softened and weakened it. Weathering of the rock to the degree requiring excavation and removal occurred only near the surface and in a few isolated spots. Deepest weathering occurred along major fissure zones especially on the right abutment above elevation 1,400. Sound, unweathered, unaltered andesite was determined to be suitable for founding the 450-ft high Detroit Dam. Bearing loads of the existing dam are significantly higher than the maximum expected loads imposed by the proposed tower.

Extensive rock testing, both in a laboratory and in situ, has been conducted to characterize the foundation rock for the dam. This is documented in the revised Definite Project Report, Appendix B – Geology and Foundation Data.

3.4.2.6 Geologic Structures

Geologic structures at the site were mapped from the exposed foundation, in tunnels, drill holes and large diameter calyx holes. The major trend of the dikes and faults is N45ºW, nearly parallel to the river. Most joints, fissures, and faults dip steeply to the southwest. The larger northwest striking shears have been mineralized and presently consist of a few inches to nearly 5 ft of shattered rock in a hard matrix of quartz and epidote. Northeast striking faults and shears exposed in the foundation were generally tight and fresh. However, a more modern interpretation is that they might be related to the intrusive process. Subhorizontal shears, thought to be due to glacial unloading, were exposed in the left abutment.

Most important of these joints and fissures are the ones located higher up on the abutments. Typically, they dip steeply into the left abutment and downstream. On the right abutment above the dam, they were weathered to clay for a thickness of several inches and created stability problems that had to be corrected during construction (Figure 3-10). Three small-volume but significant rock slides occurred during construction. There was one fatality due to a slide at the rock quarry where a high angle joint daylight in the quarry.

Primary joint set: Strike N6E to N60W and dipping 74º to 78º westward.

All joints exhibit hydrothermal alteration which tends to form a 0.2 to 0.8-ft wide beached and softened zone. Adjacent to calyx hole CH-3 in the right abutment, geothermal fluids have bleached and softened a zone 26 ft wide. The clays in the zone tend to slake and crumble upon exposure to the atmosphere.

Secondary joint set: Strike N18W to N42W and dips 76º to 88º southwest

This set is commonly composed of many closely spaced fractures or zones of fragmented rock. The rock is only slightly altered.

Figure 3-10. Dominant Joint System Set – Right Abutment Highway

The foundation geologic map shows several steep joints or fissures beneath monolith 22-26. The original geologic map has been located, and review of the map shows that major joints dip steeply downstream and into the left abutment. Therefore, the minimal excavation for the attached SWS will not have any adverse impact on the stability of the existing dam. Once the tower foundation is in place, the open cut area between weathered rock and tower can be backfilled with uncompacted rock to mitigate any rock slope stability issues and prevent any unravelling of the rock slopes. However, the larger excavation for the FSS has a greater potential for adverse dipping joints to potentially form moveable rock wedges that could slide into the excavation. To reduce this risk, the permanent rock slope will be excavated to a flatter slope of 1V:1H. Once the final dimensions of the FSS are determined, the excavation will be sized and final slopes will be evaluated and designed. This work is expected to be accomplished at 90% design.

3.4.2.7 Geomechanical Properties

Foundation rock at Detroit has been extensively tested for the design of the original dam. Test results were included in the design project report and the foundation report. General geomechanical properties of the foundation rock are summarized in Table 3-2 below.

Table 3-2. Geomechanical Properties of Bedrock

<table>
<tr><th>Property</th><th>Number of Tests</th><th>Minimum</th><th>Maximum</th><th>Average</th></tr>
<tr><td>Specific Gravity</td><td>22</td><td>2.57</td><td>2.82</td><td>2.67</td></tr>
<tr><td>Absorption</td><td>22</td><td>0.05%</td><td>1.36%</td><td>0.59%</td></tr>
<tr><td>Compressive Strength</td><td>26</td><td>6,130 psi</td><td>48,200 psi</td><td>15,450 psi</td></tr>
<tr><td>Shear Strength</td><td></td><td></td><td></td><td></td></tr>
<tr><td>Shear Resistance (s)</td><td></td><td colspan="3">S = 1000 psi + 1.15 (Normal Load)</td></tr>
<tr><td>Angle of Friction (ϕ_U)</td><td></td><td colspan="3">(ϕ_U) = 49 degrees</td></tr>
<tr><td>Apparent Cohesion (C_A)</td><td></td><td colspan="3">C_A = 1,000 psi</td></tr>
<tr><td>Intact Rock (Unjointed)</td><td></td><td></td><td></td><td></td></tr>
<tr><td>Modulus of Elasticity</td><td>8</td><td></td><td></td><td>6.84×10^6 psi</td></tr>
<tr><td>Poisson's ratio</td><td>4</td><td></td><td></td><td>0.217</td></tr>
<tr><td>Rock Quality Designation (RQD)</td><td></td><td colspan="3">Range 75 to 100%</td></tr>
<tr><td colspan="5">Note:
1) No lab tests were run on the andesite porphyry because it was not recognized as a distinct rock type at the time. However, because it is less weathered, it is likely to have similar or higher strength parameters.
2) The angle of friction and apparent cohesion values are modern interpretations of the available data (i.e. ArcTan of 1.15 = 49 degrees)</td></tr>
<tr><td>Mass Rock (In tunnels) Results of Insitu Uniaxial Jacking Tests Performed in Tunnels</td><td>5</td><td colspan="3"></td></tr>
<tr><td colspan="2">Modulus of Elasticity (ETUNNEL)

Test conditions included considerable jointing on 3 planes, with spacing as little as 1-ft</td><td colspan="3">Range: 150,000 to 1,100,000 psi

(relatively low values due to movement on fracture planes as opposed to elastic deformation)</td></tr>
<tr><td colspan="2">Poisson's Ration (pp)</td><td colspan="3">Range 0.20 to 0.48</td></tr>
</table>

3.4.2.8 Shear Strength

Scale is important when considering the selection of design parameters. The shear strength parameters determined in the laboratory are a function of the relatively small size and surface area tested and the type of test conducted. Testing an intact specimen in triaxial cells provides an internal angle of friction that is relatively high due to the breaking of mineral bonds. In contrast, laboratory data from a direct shear test measures friction on natural fractured surfaces. This small fractured area captures 1st order asperities. However, in the case of a drilled shaft design that has a diameter similar to the average joint spacing planar friction, 1st order and 2nd order asperities may be required for design. An illustration from EM 1110-1-2908 is provided in Figure 3-11 below.

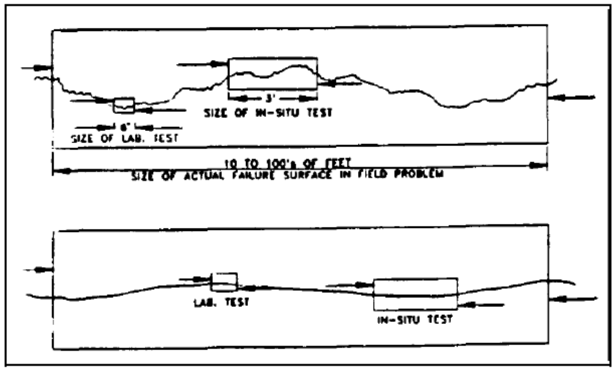


Figure 4-2. Effect of different size specimens selected along a rough and a smooth discontinuity surface (after Deere et al. 1967)

Figure 3-11. Excerpt from EM 1110-1-2908 regarding planar roughness

The planar friction and 1st order asperities were measured in the lab and it is known that the joints are at acute angles, so the 2nd order asperities can be ignored without significant error. Asperities generally increase the total angle of friction above that of a smaller surface as shown in the illustration from EM 1110-1-2908 below in Figure 3-12.

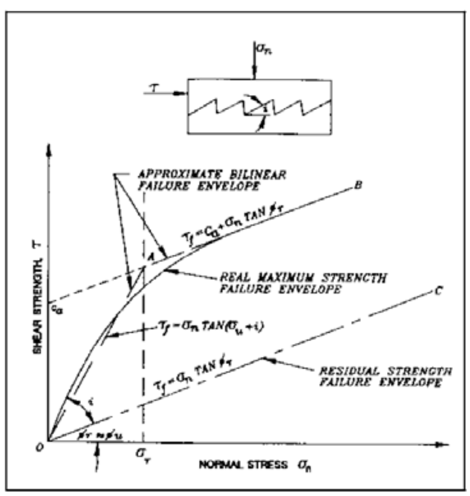


Figure 4-6. Typical approximate bilinear and real curvi-linear failure envelopes for modeled discontinuous rock

Figure 3-12. Excerpt from EM 1110-1-2908 regarding asperities

The shear strength parameters are defined below as;

$$\tau_f = \sigma_n \tan(\theta_u + i)$$

$$and$$

$$\tau_f = C_a + \sigma_n \tan \theta_r$$

where

$\tau_f = maximum\ (peak)\ shear\ strength\ at\ failure$

$\sigma_n = stress\ normal\ to\ shear\ plane\ (discontinuity)$

$\emptyset_u = basic\ friction\ angle\ on\ smooth\ planar\ sliding\ surface$

$i = angle\ of\ inclination\ of\ first\ order\ asperity$

$\emptyset_r = residual\ friction\ angle\ of\ material\ comprising\ the\ asperities$

$C_a = the\ apprent\ cohesion\ (shear\ \ intercept)\ derived\ from\ asperities$

For unweathered discontinuity surfaces, the basic friction angle and the residual friction angle are, for practical purposes, the same. Since we have slight weathering we can assume this principle holds true without significant error.

In order to use the correct failure envelope, we must make a choice about selecting a failure mode, i.e. intact rock or clean, free-draining discontinuities. In the case of clean discontinuities, design will be based on using the lower bound from the direct shear test, tabulated in Figure 3-13 below.

In the case of a bearing capacity failure, some combination of failure in the intact rock and discontinuity is likely; however, it is particularly difficult to analyze for two reasons: 1) the percentages of the failure path defined by discontinuities or intact rock are seldom known and 2) strains/displacements necessary to cause failure of intact rock are typically an order of magnitude (a factor of 10) smaller than those displacements associated with discontinuous rock. The latter is known as a strain incompatibility, meaning peak strengths of the intact rock proportion will already have been mobilized, and will likely be approaching residual strength before peak strengths along the discontinuities can be mobilized. For these reasons, selection of appropriate strengths must be based on sound engineering judgment and experience gained from similar projects constructed in similar geological conditions.

Due to the complexity of a combined mode of failure, it is prudent to treat the rock like a continuum and simply pick a single value. Given that intact rock likely has a high value of internal angle of friction, it is conservative to assume that the continuum for modeling a general shear failure has the same angle as that measured in the laboratory from direct shear tests.

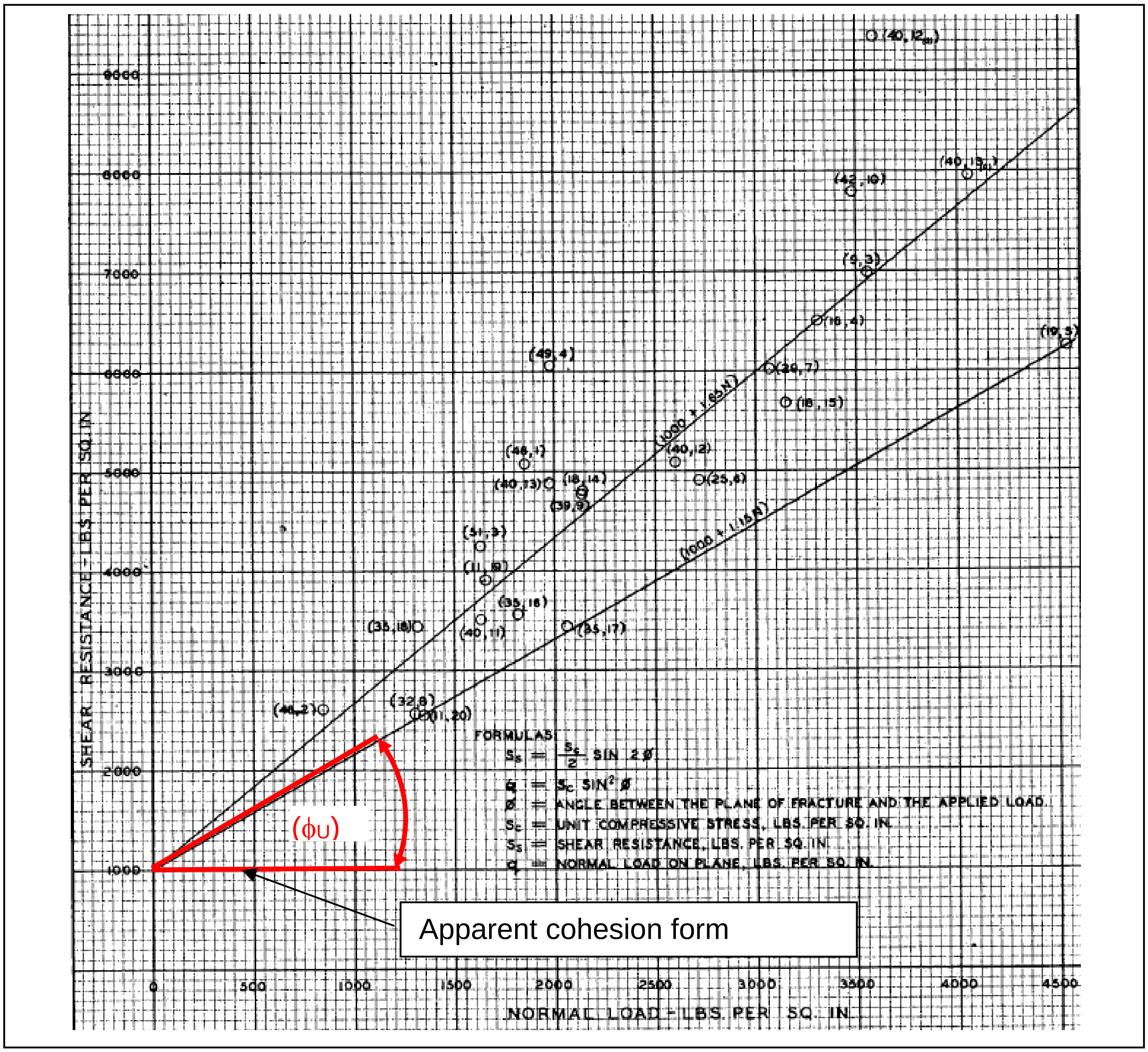


Figure 3-13. Shear Resistance of Foundation Rock. Excerpt from DE-20-37/12

Appendix B Chart 2 in the Definite Project Report

3.4.2.9 Groundwater

Seasonal groundwater may be an issue where excavations are dumped into rock/fills. Source of water is high pool and infiltration of precipitation. Water is not expected to be an excavation issue during or post construction; however, groundwater may be an issue in holes and drilled shafts where cement grout or tremie concrete is required to form permanent anchorages (rock bolts and anchors and drilled shafts). Water tightness and water in holes impacts inspections of holes, grout design mixes, and design strengths.

Previous packer tests will be reviewed and reevaluated using the current methodology. This issue will be addressed in the 90% report and plans and specifications (P&S).

3.4.3 Site Seismicity

3.4.3.1 Site Specific Seismic Studies

A site regional specific seismic study for the Willamette Valley was produced by Amec Geomatrix for USACE-HQ as a demonstration project in 2009. This report is included in Appendix E. In 2017, the report was further updated by Foster Wheeler for the Corps' Risk Management Center (RMC). It did not include Detroit but did include the nearby Green Peter Dam. The final version, provided to USACE in June of 2017, includes a detailed description of the tectonic setting (Amec, 2017). Review of the updated report shows that ground motions for the nearby Green Peter Dam have increased about 10% from the 2009 report. The three primary seismic sources for Detroit Dam are summarized as follows:

- <u>Cascadia Subduction Zone (CSZ) Interface</u>: Earthquakes that occur at the convergent boundary between the westward-moving North American and eastward-moving Juan de Fuca and Gorda Plates that run offshore from southern British Columbia to northern California. Earthquakes generated at this margin produce strong ground motions and long durations of shaking (upward of 5 minutes). A full rupture at the interface has the potential for generating earthquake magnitudes (M_w) in excess of 9.0 every 450 to 550 years, though partial rupture events in northern California and southern Oregon resulting in lower magnitudes may occur as frequently as every 200 years (Clague et al, 200). The most recent major CSZ earthquake occurred in 1700 (Goldfinger et al, 2012).

- <u>CSZ Intraplate</u>: Earthquakes that occur from deep within the subducting Juan de Fuca Plate having focal depths of 25 miles or more. The most recent recorded intraplate earthquake was the M_w 6.8 Nisqually earthquake that occurred northeast of Olympia, Washington in 2001.

- <u>Shallow Crustal</u>: Earthquakes originating from local crustal faulting. Several crustal faults have been identified within a 100 mile radius of Detroit Dam. The nearest known mapped fault is the Mount Angel fault which may have been the causative fault for the Scotts Mills 1993 earthquake. Many of these fault systems have no recorded recent seismic activity though slip rates and fault geometry suggest the potential for M_w on the order of 6.0 to 6.5.

3.4.3.2 Seismic Hazard Curve

Tabulation of estimates of ground motions at varying return periods using USGS 2014 National Seismic Hazard Mapping and the Site Specific seismic study (AMEC, 2009) are presented in Table 3-3 and plotted on Figure 3-14. Comparing the two curves, the USGS-2014 curve is significantly higher than the site specific study. Primary reasons are the USGS-2014 hazard mapping uses the latest Next Generation Attenuation relations. It is recommended that the newer USGS-2014 values be used.

Table 3-3. Seismic Hazard for Detroit Dam

USGS-2014		Site Specific AMEC-2009	
Return Period (years)	**PGA (g)**	**Return Period (years)**	**PGA (g)**
144	0.068	144	0.033
500	0.148	500	0.095
1,000	0.211	1,000	0.139
2,500	0.317	2,500	0.219
5,000	0.414	5,000	0.286
10,000	0.531	10,000	0.362
30,000	0.752	Not determined	Not determined
100,000	1.047	Not determined	Not determined

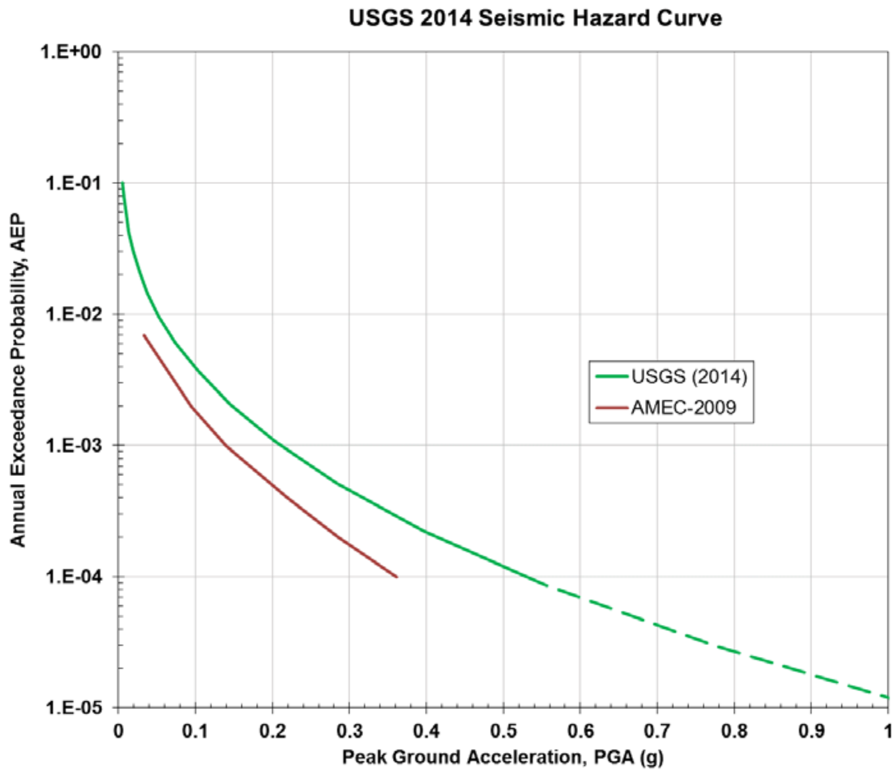


Figure 3-14. Seismic Hazard Curves

3.4.3.3 Response Spectra for Detroit Dam

Site-specific ground motion spectra for Detroit Dam, developed by AMEC (2009), is presented in Table 3-4. In addition, USACE RMC has initiated a contract that will update seismic ground motions at the dam. This work is expected to be completed in FY19. Until that work is completed, Site Class B is recommended as no other site specific data is available. However, this investigation may confirm that Seismic Site

Class A may be appropriate. Site Class A would reduce seismic ground motions and demand on the existing dam and newly attached tower. This is especially important in the seismic reanalysis of the two monoliths that will be modified. In addition, seismic stability of the loose talus slope/waste fill material placed in the reservoir will have to be considered. This material appears to have been dumped and is probably in a loose state at the natural angle of repose, meaning it likely has no seismic resistance. The concern is that a major seismic event may destabilize some of the material and cause it to slide downslope and accumulate in the area excavated for the FSS. While this would not cause an immediate concern, a later drawdown of the pool could result in the FSS grounding which could cause significant damage to the very expensive floating structure. Because of this, it may be more practicable to incorporate the removal of all loose fill material upslope of the FSS excavation during the SWS tower contract as a preventative measure.

Table 3-4. Ground Motion Spectra for Detroit Dam

Spectral Period (sec)	Pseudo-Spectral Acceleration (g), 5% Damping, for:							
	UHRS for Return Period of:						Deterministic Motions from Cascadia Interface M 8.7 to M 9.1	
	144 years	500 years	1,000 years	2,500 years	5,000 years	10,000 years	Median	84th Percentile
PGA	0.0331	0.0947	0.1392	0.2187	0.2856	0.3616	0.1191	0.2356
0.075	0.0522	0.1433	0.2247	0.3557	0.4779	0.6194	0.1732	0.3708
0.1	0.0595	0.1676	0.2602	0.4094	0.5540	0.7263	0.1974	0.4244
0.2	0.0732	0.2145	0.3318	0.5248	0.6934	0.8900	0.2680	0.5654
0.3	0.0655	0.2010	0.3157	0.5015	0.6561	0.8402	0.2638	0.5522
0.5	0.0458	0.1605	0.2708	0.4391	0.5787	0.7462	0.2355	0.4912
1.0	0.0223	0.1007	0.1759	0.3053	0.4184	0.5470	0.1551	0.3443
2.0	0.0083	0.0452	0.0952	0.1723	0.2490	0.3408	0.0794	0.1990
4.0	0.0023	0.0142	0.0315	0.0596	0.0966	0.1322	0.0225	0.0735

3.5 GEOTECHNICAL CONSIDERATIONS AND PARAMETERS

3.5.1 Dam Safety and Geotechnical Siting Consideration

The SWS tower being considered is attached to the upstream face of the dam. All work is restricted to the two penstock blocks; no other blocks will be modified. The tower footprint is about 40x108 feet or 4,030 square feet. About half of the tower foundation footprint has already been excavated, prepared, geologically mapped, and concreted. No more than about 2,000 square feet of foundation remains to be completed. This excavation will extend through weathered and fractured rock down to the top of sound rock. The quantity of fractured and weathered rock to be removed is small enough that it is expected it can be excavated by small controlled blasting techniques with less vibrational energy than mechanical chiseling. The depth of excavation will not go below the existing base of the dam or into the upstream sloping grout curtain and will not present a risk to the grout or drainage curtains. Once the foundation is dredged and free of loose materials, it will be concreted. Concrete will rise up higher than the elevation of the material removed. Consequently, this minor modification will not undercut the dam and will add mass to the upstream side of the dam which is not detrimental to dam safety. There is a minimal risk of undetected damage to the grout and drainage curtains. Monitoring drainage flows during construction and for 5 years after will confirm if and to what degree there was damage. Minor cracking is not a concern because the rock is non-erodible and non-soluble. Any leakage will be collected by the drainage curtain. It is only critical if uplift pressures increase above designed assumption and/or collected flow becomes excessive where it may impact sump pumping. Critical point is the first refill of the reservoir following construction. If increased flow is observed, then pool refill will be halted and a short segment of grout curtain repaired by drilling and grouting new grout holes from the grout gallery. Excavation for the FSS presents the greatest dam safety concerns. The excavation will require controlled blasting to minimize vibration and potential damage to the base of the dam and grout curtain. The depth of the excavation will extend a maximum of 50 ft below the base of the dam, and slope upstream and not extend any closer to the grout curtain.

No structural modifications will be made to any of the spillways or other blocks. These are the maximum height blocks that may have suspected undetermined dam safety seismic stability issues. These potential issues will be evaluated in the future by an Issues Evaluation Study (IES). An IES has not been scheduled.

To ensure dam safety of the existing structure, an exclusion zone has been established outside of where major construction will be located. This exclusion zone was established by extending a line at an angle of -45 degrees upstream from the heel of the dam. This line is believed to be conservative for the following reasons:

- Foundation rock is good.
- Shear strength of the rock is >49 degrees.
- Performance of existing near-vertical slopes along the road has been good except for deterioration along a prominent exposed fissure zone.
- Mapped fissure zone in the foundation dips steeply downstream and into the left abutment which is favorable for the slope immediately upstream of the dam. However, the joints do dip in the direction of the perpendicular slope. Evaluation will be made to determine if the apparent dip of these joints is steeper than the proposed rock slope or if any kinematically possible rock wedges could form.
- Design rock slope 1V:1H or less and it is not anticipated to require slope stabilization.

- Exclusion line is definable in design and construction.

Conceptual upstream excavation illustrating the geometry is shown in Figure 3-15. This excavation is for an older location of the FSS. Also, the rock excavation may be stair-stepped with 10 ft vertical lifts with 5 to 10 ft horizontal benches to facilitate actual construction. Although the excavation is not precise, it does show the relationship of the proposed upstream FSS excavations and the exclusion line to the dam, grout and drain curtains. This drawing will be revised after the size and location of the FSS excavation is determined. As can be seen, excavation is tight up against the dam and some exceptions will have to be allowed to permit localized excavations for penstocks or other tight spots. Each exception will be individually evaluated with the following consideration:

- Not more than one half of a monolith will be influenced by the exclusion line.

- Blasting within the zone will be stringent with only smaller blasting permitted.

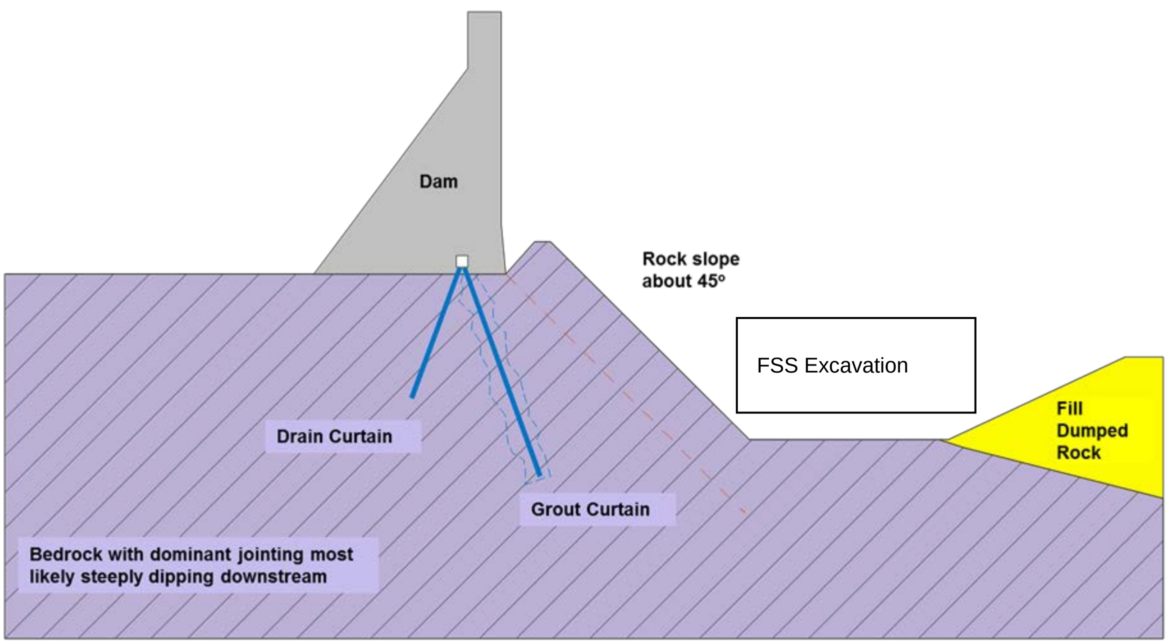


Figure 3-15. Upstream Excavation Exclusion Line

3.5.2 Design Parameters

No new subsurface explorations have been conducted for the Detroit SWS and Downstream Fish Passage project. Future subsurface explorations will be required to address several geotechnical foundation issues identified in this study. These are discussed in Section 3.6. The original foundation report did not include a final survey of the base of the concrete dam; however, there is an as-built foundation grade line shown. The as-built grade line shows that the finished foundation deviated up to 30 ft from construction drawings. In addition, benches at each monolith joint were deleted. This review found that the as-built foundation drawings are incorrect.

Original investigations have found the sound bedrock to be adequate for the 450-ft high concrete gravity dam. The most important factor for adequacy of foundation is the presence of highly jointed or fissured rock. About half of the footprint of the SWS has already been excavated, prepared, and mapped. The mapping shows a typical joint pattern that is characteristic at the site. Joints tend to cross the dam axis obliquely; jointing will be evaluated and considered in the design of the foundation.

USGS uniform hazard response spectra Site Class B will be used as no other site-specific data is available. However, on-site shear wave velocity measurements will be developed as part of updating the latest seismic loading. This work is currently under contract and is expected to be completed by end of FY 2019. If this investigation measures and confirms that shear wave velocity is sufficiently high to meet Site Class A, then a Site Class A may be used in the analysis. Site Class A would reduce seismic ground motions and demand on the structure.

Additional consideration is the seismic stability of upslope loose talus and waste fill. The sliding and accumulating of material on the upslope side of the solid concrete portion of the SWS tower may add a small incremental lateral load that will be considered in the design. However, the greatest concern is that material may slide down and accumulate in the FSS excavation which is addressed in the Section 3.5.3.

3.5.3 Overburden Excavation Upslope of the FSS Excavation

Overburden excavation is being considered to remove or lay back slopes in the talus deposit. Measurement of the existing talus slopes is about 49°. This angle represents the natural angle of repose of the talus/dumped rock under static conditions with a factor of safety (FS) of 1. Flattening the slope to 45° increases the FS to about 1.1. Since construction will be done in the wet, there will not be any personnel below the slope during construction or post construction. However, there remains an economic concern that a major seismic event greater than an Operating Basis Earthquake (OBE) may cause a large amount of this loose material to slide downslope and accumulate in the area of the FSS excavation. This is not a dam safety issue, however, if enough material accumulates the FSS could become grounded and possibly damaged when the pool is drawn down. It could occur during drawdowns below minimum flood pool; the operational consequence could mean it may not be possible to draw the pool down to minimum pool level until the material is dredged out. Dredging would require temporarily floating the FSS out of the way then dredging loose talus material from the excavation. Current design is considering removing, as much as is feasible, the loose talus materials upslope of the FSS excavation then providing extra space in the excavation to permit accumulation of anything that works its way free. The goal is to avoid any maintenance dredging as long as practicable – possibly for the design life of the project. The design will consider the tradeoff of the cost of additional excavation during construction with the cost of future mobilization and maintenance dredging.

3.5.4 Rock Excavation

Minimal rock excavation for the SWS tower is required. About half of the footprint for the tower has already been excavated, prepared, and concreted. The remaining half will require excavation of near-surface weathered rock to reach the top of sound rock on which the dam was founded. Foundation for the SWS will be benched to provide greater sliding resistance down slope (cross-valley direction).

Mass rock excavation will be accomplished by blasting for both the tower and FSS excavation. The bedrock was found to be susceptible to blast damage during the construction of the original dam. Consequently, stringent blasting criteria will be required. Special blasting techniques and smaller blast patterns removing smaller increments will be required within 5 ft of final foundation grade.

The excavation plan has not been fully developed as it will require further input and refinement from structural, geotechnical, dam safety, construction, and cost engineering. The latest concept being developed to meet structural requirements is to step the tower foundation illustrated in Figure 3-16.

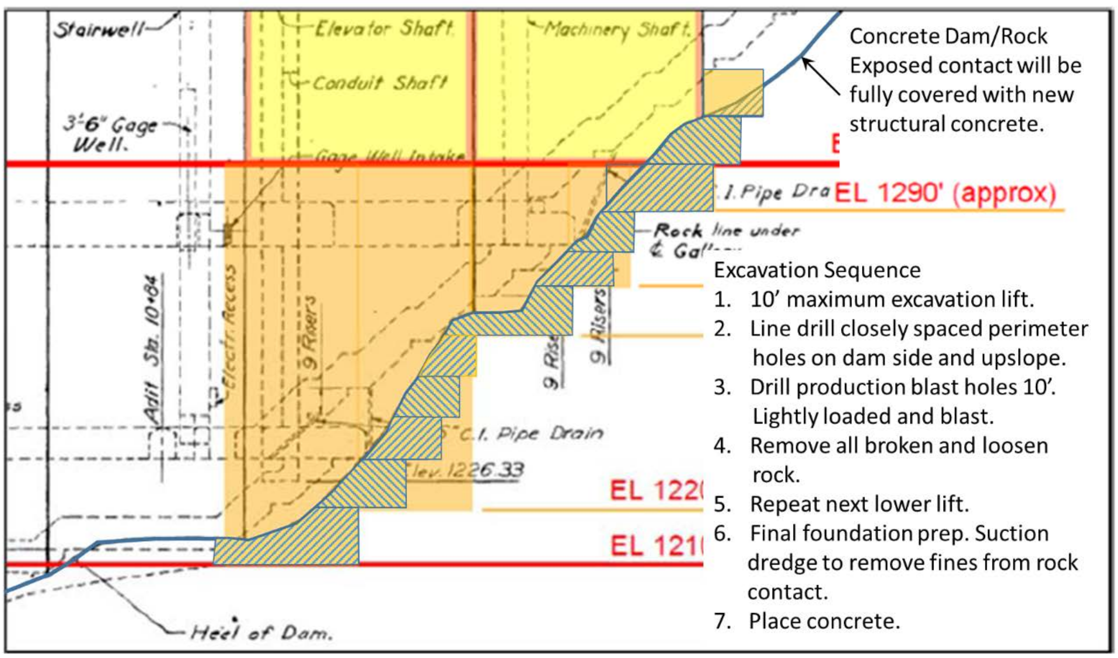


Figure 3-16. Preliminary Concept of Rock Excavation Sequencing

Conceptually, the sound rock will be excavated in small 10-ft lifts. The maximum quantity of rock to be removed at one time will be about 500 cy. Each lift will be line-drilled with closely spaced holes along the face of the dam and the upslope side to reduce energy and pressurized gases from extending toward or beneath the dam. Downslope and upstream pool will be daylighted out to the appropriate elevation to provide an area for blasted excavated rock to move toward.

This can be achieved by placing strict maximum vibration levels, setting peak particle velocities, requiring a specialized/experienced blaster to design and submit detailed blast plans, providing details on spacing and loading of individual holes, limiting sizes of charges, line drilling, sequencing individual charges to direct as much of the vibrational energy away from the dam as possible, and using bubble curtains. The key to completing the tower foundation is to focus on vibration control and foundation cleanup (removing all loose rock and possibly suction-dredging the exposed rock) to prepare a rough, but clean surface ready for concrete placement. Rock grade lines are a secondary concern where a stepped foundation is preferred to increase sliding resistance. Overbreak is unimportant so long as all of the loosened rock is removed. All overbreak areas will be backfilled with structural concrete for the base of the tower. The base contact on the existing concrete dam and rock is not expected to be damaged. However, if it is cracked, the exposed crack will be covered with 10 to 20 ft of structural concrete so that leakage along the crack is not considered a risk. To further meet dam safety concerns, the foundation will be assumed to be cracked with 0 cohesion for 10 ft beneath the dam for structural sliding and seismic analysis purposes. Any deficiency will be mitigated. All of this will ensure that the new construction will not compromise dam safety.

The FSS excavation only requires that vibration controls and minimum grade be maintained for the floating structure. Foundation preparation will not be required as no concrete will be placed on it. However, the base of the dam contact will not be covered by new concrete so there is greater concern for leakage. Cracking and leakage monitoring is covered in construction monitoring instrumentation.

Blasting is not expected to significantly damage the grout curtain. Grout curtains by nature are never 100% tight. There are always flaws and un-grouted joints and fractures. The high uplift pressure beneath monolith 15 is an example of a flaw in grouting. Because the high uplift occurs over a small portion of the base of one monolith, it has never been repaired. It is only monitored. Damage from this work would be detectable by increased uplift and seepage during construction. If it is significantly damaged to where uplift pressures are above design basis for the dam, then it will be repaired by drilling new grout holes from the grouting and drainage gallery.

Excavation away from the dam permits the slope to be flattened out to 1V:1H which would not require rock reinforcement. This would allow construction to proceed in-the-wet year round. The foundation will be designed to tolerate underwater construction, irregular, less precise grade lines and presence of loosened rock. To compensate for this, structural design will consider an initial reinforced slab anchored into the underlying bedrock then grouted between the slab and rock to ensure full contact support.

3.5.5 Rock Anchors and Rock Reinforcement

Rock anchors and reinforcements are not anticipated to be needed in the current 60% design. However, if they are used, they will be designed in accordance with EM 1110-1-2908 Rock Foundations, and EM 1110-1-2907 Rock Reinforcement.

3.5.5.1 Tower Rock Anchors

Seismic design of the tower requires a large concrete mass at its base that will be anchored to the underlying bedrock. Anchors would consist of large-diameter solid-steel bars totally encapsulated in concrete or cement grout. Anchors would be designed in accordance with the latest Post Tensioning Institute recommendations. Expected difficulties will be verifying water tightness and grouting vertical holes underwater that may or may not be dewatered. Grout would be hydraulic-cement based and pumped into the bottom of the hole through tubes to the surface to facilitate underwater construction. Grout volume and pressure will be closely monitored because high pressures will be required to overcome outside hydraulic pressure of the pool. Rock is nonthreatening to steel, so the encapsulated anchors would have a long but indeterminable service life.

3.5.5.2 Rock Anchors

It is currently proposed that the moorings for the FSS be restrained laterally by connection to rock anchors embedded in the east rock slope. Anchors will be designed to adequately support the tensile and compressive loads incurred by lateral loading to the moorings. Loading has not yet been established and the mooring's structural design is underway. The following properties will be considered for load-carrying rock anchors:

- Inclination
- Length
- Concrete-rock bond

The bond strength between rock and grout, in accordance with EM 1110-1-2908, will be governed by the compressive strength of the grout, given that 1/10 the average of the uniaxial compressive strength of the rock is 1,545 psi, well in excess of the maximum 600 psi specified in the EM.

3.5.5.3 Rock Reinforcement

Rock slope design is focused on avoiding long-term permanent rock bolts because they would be underwater, uninspectable, and unmaintainable. The intent is to design all slopes to be stable. Additional areas will be provided at the toe of the slopes to allow for the accumulation of dislodged rocks so that they will not interfere with the floating structure. Underwater slopes, wherever possible, will be flatter than 1V:1H. Locally, steeper slope will be considered in special situations where steepening the slope for one monolith is preferred over the alternative of excavating closer to the dam.

3.5.6 Mooring Foundations

The FSS will require mooring structures to limit horizontal movement of the floating structure. It is currently being proposed that the FSS be moored to the proposed SWS.

The design has not been finalized. All mooring foundation features are expected to remain on the dam or on shore close to the proposed intake. In some areas, bedrock is at shallow depth and the anchorage may bear on the bedrock. Foundations will likely consist of drilled rock sockets and shallow foundations.

3.5.6.1 Rock Sockets

Pilings socketed into rock are not described in the referenced EMs, nor are they considered in the current design configuration. However, there may be potential for deep rock anchors socketed into bedrock for future mooring structures. If used, rock sockets will be designed for base and shaft resistances as determined by the Hoek-Brown criteria (Hoek-Brown, 1974). Base resistance, q_{bl}, is given by:

$$q_{bl} = q_0 + q_u \left\{ \sqrt{m\frac{q_0}{q_u} + s} + \sqrt{m\frac{q_0 + q_u\sqrt{m\frac{q_0}{q_u} + s}}{q_u} + s} \right\}$$

Where q_0 is the surcharge load, and q_u is unconfined compressive strength. The unconfined compressive strength of the intake area dacite used in the design is 17,000 psi based on the available site geotechnical information. The Hoek-Brown parameters, m and s, are based on the rock mass rating (RMR) initially proposed by Bieniawski (1979). Based on the fracture patterns of the available rock cores in the area, an RMR of [70] will be used based on available information from historic boring logs.

The shaft resistance, q_{sl}, is given by:

$$q_{sl} = min(0.5c_w q_u.\ 0.05f_c')$$

Where c_w is the coefficient of weakness (Komarnitskii, 1968) and is related to the average number of fractures. f'_c is the specified nominal compressive strength of concrete or grout used in the socket.

3.5.6.2 Shallow Foundations

Shallow rock foundations will be designed in accordance with EM 1110-1-2908 Rock Foundations. The tower is assumed to be founded on sound rock. It is not anticipated that there will be a bearing capacity issue. Bearing load from the tower is less than the bearing load of the existing mass concrete dam. Bearing capacity analysis will be provided in the 90% design.

3.5.7 Existing Dam Safety Instrumentation

Dam safety instrumentation at Detroit includes uplift gages, foundation drains and weirs, tiltmeters, a survey system, and strong motion accelerometers. In general, instrumentation data results have followed consistent historic patterns and have not indicated any abnormal behavior. Although uplift pressures exceeding design assumptions are recorded in Monolith 15 during high pool in the summer, this condition was evaluated and determined to be localized and not a threat to the stability of the structure. Monolith 15 is located on the left non-overflow section and will not be impacted by any proposed modifications. Instrumentation plots and evaluation will be developed for the 90% design.

3.5.7.1 Uplift Gages

Uplift instrumentation is all located in the grouting and drainage gallery. Most instruments are read from the main gallery parallel to the dam axis and are usually spaced one per monolith; however, some monoliths contain 2 piezometers. Multiple uplift piezometers have been installed in transverse galleries in Blocks 15, 21, and 24 and are referred to as lines 1, 3, and 4 respectively. One row of uplift piezometers, referred to as line 2, was installed in Blocks 19 and 20, presumably between concrete lifts during construction, with pipes being routed within the monolith to locations in the main gallery.

Foundation uplift pressures measured by uplift gages and piezometers in the grouting and drainage gallery continue to follow historical patterns. The uplift gages are generally read monthly which is adequate; however, they were not read consistently on schedule from 2001-2010, so there are data gaps.

Other than Block 15, all the monoliths have uplift values below design uplift. Detroit Dam has had a long history of excessive uplift in Block 15. Gage 15A in the upstream grouting gallery, drilled just downstream of the grout curtain, has consistently shown uplift in excess of design limits since 1953. In August 1960, five additional drain holes were drilled in Block 15 to relieve the pressure, resulting in a drop of approximately 30 to 40 ft of head. In late 1963, plugged or partially plugged drains were reamed out but only minor increases in drain flows resulted. A temporary drop in pressure was noted in Gage 15A after reaming adjacent drain holes. A number of grout holes from Block 14 through 25 showed some leakage and were fitted with uplift gages in 1963. Several showed some uplift pressure, and five in Blocks 14 and 15 showed high uplift pressures.

An evaluation done in 1969 by Geology Section determined that the grout curtain conditions were not in compliance with contract requirements or current standards. The grout pipes of many blocks were found to be open, and many had flowing water. Also, it was determined that the drilling of additional drain holes parallel to the axis and inclined toward the right abutment appeared to offer the best possibility for reducing uplift pressures to acceptable levels. Between 1971 and 1973, all grout holes were filled, nine new uplift piezometers were drilled (one each in Blocks 9-16 and one in Block 23), and seven supplemental drains were drilled. Three of these newer drains are combination drains and piezometers that function as piezometers, but can be left open to serve as drains as necessary. During this work, the hole containing uplift gage 15A was deepened about 11 ft, and a combination uplift piezometer and drain 15aaa (later designated 15XXX) was drilled in block 15. Water pressure testing done in conjunction with these installations showed most of the holes and drains in Block 15 to be interconnected so that no dangerous uplift pressures could develop uniformly beneath the block. This means that the uplift pressure as measured by gage 15A does not represent a uniform uplift pressure, but represents very localized conditions that cannot exceed certain values before being relieved by the interconnected drainage system. Calculations for overturning of Block 15, submitted with the 1971 Periodic Inspection Report, indicate that even with high uplift readings of 15A (prior to its modification to a piezometer), Block 15 is stable. Historically, the piezometric elevation at 15A has been as high as 1,460 to 1,470 ft when the pool is at summer pool (1,563.5), compared to the design uplift value of 1,388 ft.

3.5.7.2 Foundation Drains

There are 149 foundation drains in Detroit Dam with a total depth of approximately 6,772 linear ft. The drains are 3-1/2-inch diameter cased in the concrete, and either 3-inch or 2-inch diameter holes in bedrock. Deepest recorded depths range between 10 ft and 136 ft.

In 1995, Portland District implemented a program to periodically inspect drains then recommend cleaning if warranted by blockages, or if it had been over 25 years since the previous cleaning. Detroit foundation drains had been previously cleaned under contract in 1983. The project had also periodically flushed the drains up until about 2004. Following the 2011 drain inspection, the Corps has been contracting with the BOR to clean the drains as funds become available. The BOR cleaned the drains with a high pressure water system that they own and use specifically for this purpose.

The Detroit Dam foundation drains were most recently inspected in May 2016. The inspection included depth soundings and flow readings. Out of 149 drains measured, 11 were plugged more than 50% of their length. These 11 blockages occurred in 10 different monoliths. Uplift pressures have not been increasing and most drains are still allowing water passage. The total flow measured at the 2016 inspection was 13 gpm. Flows vary from inspection to inspection and have historically ranged from 7 to 21 gpm. The details can be reviewed in the latest Periodic Assessment.

3.5.7.3 Weirs

Flows from the upper abutments and Block 15 are serviced by gravity drainage systems that discharge downstream from the dam. The five weirs used to monitor the flows from these drainage systems were installed in 1985. Weir flow has been consistent with seasonal fluctuations that correlate with pool elevation. Maximum flows are generally in the range of 8 to16 gpm. Plots can be viewed in the latest Periodic Assessment.

3.5.7.4 Tiltmeters

Three tiltmeters were installed in block 22 in 1979 to monitor upstream/downstream movement of the dam. These tiltmeters, installed in galleries near the top, middle, and bottom of the dam, provide a means to determine deflection or tilting in a single monolith. Readings are taken and averaged for an overall reading. In early 1988, an additional tiltmeter was installed in Block 15 at elevation 1,271 in the grouting and drainage gallery to monitor deflections related to the high uplift pressures recorded in that block.

The history of readings indicate that the dam responds to changes in pool level. The crest deflects downstream as pool is raised and upstream as pool is lowered.

3.5.7.5 Survey System

Electronic Distance Measuring (EDM) alignment and settlement survey monuments and targets were installed in 1981. Through 1990, EDM and settlement surveys were performed twice yearly, at high and low pools. Starting in 1992, survey readings were reduced to once every other year at high pool. Another schedule adjustment was made in 2009, and surveys are now taken at alternating 2-3 year frequencies (twice between 5-year periodic inspections).

Progressive downstream movement has not occurred. Normal maximum deflection range (upstream to downstream movement at any individual target) did not exceed about 0.55 inch through 2006. The subsequent two sets of readings taken during the full pool months of June showed downstream deflection exceeding this range with the 2008 reading approaching 0.9 inch. The 2010 and 2012 downstream deflection was less than 2008 and closer to the historical measurements. Elevation changes progressively decrease toward both abutments where height from foundation to dam crest is the least. The pattern of deflection and elevation changes corresponding to pool level changes suggests that only elastic movements are occurring and that settlement of the structure and thermal cooling of the concrete is complete.

3.5.7.6 Strong Motion Accelerometers

There are 3 seismic instruments installed at Detroit Dam. Two are SMA-1 model analog instruments, installed in 1973, and are located in the upper gallery and on the right abutment about 1,650 ft downstream of the dam. Detroit Dam has experienced shaking by several earthquakes over its 60 years of life. All five strong motion recorders were triggered at the dam by the 1993 Scotts Mills Earthquake (mag. 5.6 at 27 miles) (EERI, 1993). Peak ground motions recorded were 0.06g on bedrock at the downstream toe and 0.18g upper gallery near the crest.

3.5.8 Dam Safety Instrumentation and Monitoring During Construction

During construction, existing geotechnical and dam safety instrumentation will be frequently monitored. This will be augmented by new specific instrumentation during construction blasting, water up and first refill. Purpose of instrumentation is to ensure blasting vibrations do not damage the foundation, structure, grout and drainage curtains (monitored by observing foundation uplift pressures and foundation drainage flows), or cause damage to the dam. Construction monitoring instrumentation will consist of:

- Vibration monitoring during blasting

- Crack preconstruction inspection and during-construction monitoring in upstream grouting drainage gallery
- Piezometer/foundation uplift pressure during water up first refill
- Right abutment foundation drain flow during water up and first refill
- Crest survey during water up and first refill

General location of instruments are shown on Figure 3-16, Dam Safety/Geotechnical Construction Instrumentation Monitoring.

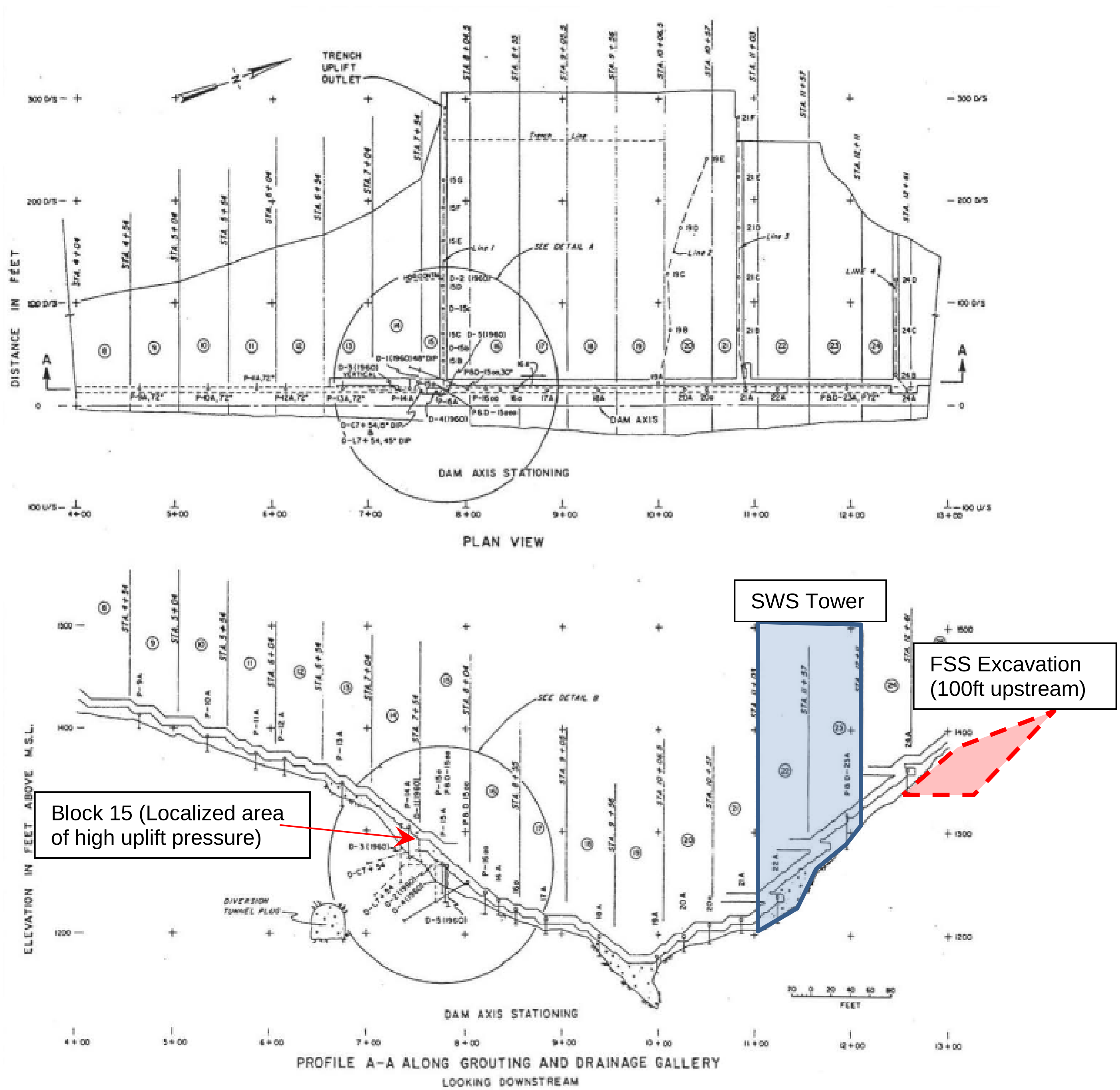


Figure 3-17. Permanent Dam Safety and Temporary Construction Instrumentation

3.5.8.1 Vibration Monitoring

Excavation will require blasting. Some rock overbreak is expected and it is possible to damage the concrete dam (i.e. concrete/rock contact). Vibration control and monitoring will be accomplished by industrial standards and equipment. Large massive concrete dams are not susceptible to vibration damage; however, the concern is focused on possible damage to the grout curtain and, more remotely, the drainage curtain further downstream. High vibrations and gas pressure from blasting can cause cracking in the grout curtain. Grout curtains typically are constructed from low-strength, high-water-content fluid cement that has a higher susceptibility to cracking. Vibration control can be achieved by placing strict maximum vibration levels, setting peak particle velocities, requiring a specialized/experienced blaster to design and submit detailed blast plans, limiting sizes of charges, line drilling, sequencing individual charges to direct as much of the vibrational energy away from the dam as possible, and using bubble curtains. Vibration monitoring will be accomplished by installing specialized vibration monitors in the grouting and drainage gallery as close as practicable to the areas that are being blasted. Additional requirements will be developed for the 90% design. At the completion of the blasting phase, blast vibration monitoring instruments will be removed.

3.5.8.2 Crack Inspection and Monitoring

Open galleries in and around areas to be excavated by blasting will be inspected before excavation begins. Significant cracks may be monitored by new crack displacement meters. Crack meters will be read after each blast. At the end of all blasting, a post construction gallery inspection will be conducted. Crack meters may be left in place for up to 5 years following water up/refill to verify that a fluctuating pool is not causing any displacements.

3.5.8.3 Piezometer/Foundation Uplift Pressure Monitoring

Existing piezometers and uplift pressure instruments will be read frequently during excavation and during water up and pool refill. Uplift pressures and changes in flows will be measured and plotted against pool levels. Analysis will focus on detecting changes (increases of pressure or flow) at a given pool level that may indicate damage to the grout curtain. If significant changes are detected, pool refill may be delayed until grout curtain can be repaired and/or new foundation drains installed to ensure uplift pressures are no higher than pre-construction. Piezometers and uplift pressure gages will be measured monthly for 5 years after the first complete refill.

3.5.8.4 Right Abutment Foundation Drains Monitoring

Existing foundation drains will be read frequently during excavation and during water up and pool refill. Changes in individual drain flows will be measured and plotted against pool levels. Measuring individual drains allows opportunity to narrow down potential problem areas. Analysis will focus on detecting changes (increases of pressure or flow) at a given pool level that may indicate damage to the grout curtain. If significant changes are detected, pool refill may be delayed until grout curtain can be repaired and/or new foundation drains installed to ensure uplift pressures are no higher than pre-construction levels. Foundation drains will be monitored individually for the first complete refill cycle and then aggregate flows on a monthly schedule for 5 years following the first complete refill.

3.5.8.5 Crest Survey

Existing crest survey monuments will be periodically read and compared to preconstruction location. Purpose is to ensure that construction does not cause any displacement of the dam. Crest survey will continue annually for 5 years after first complete refill.

3.6 GEOTECHNICAL INVESTIGATION

Currently, sufficient information exists on which to base the design, so limited additional geotechnical investigation is anticipated at this time. For the SWS tower, half of the foundation footprint for the heavily loaded SWS has already been prepared and covered with concrete. The remaining half is located within 20 ft of the existing foundation map, and foundation conditions are not expected to change significantly enough to warrant further investigation. Additional information required for the SWS is seismic site classification which is being developed by the RMC. The FSS excavation is less critical; its primary purpose is to provide room for the floating structure at low pool. Shape of the final excavated surface is not critical and does not have to be prepared or mapped. Slopes will be laid back on 1V:1H if possible to eliminate need for permanent rock stabilization. The only criterion is that loose rock be removed to prevent material from sliding down and accumulating beneath the floating structure during a seismic event. Seismic design of non-life safety slope is not critical. Consideration will be given to drilling and imaging two or three holes to document the rock conditions for dam safety purposes.

3.7 GEOTECHNICAL INFORMATIONAL DRAWINGS

The following plates are a compiled listing of relevant geologic and geotechnical drawings completed during original design and construction. The drawings are located in Appendix E. It should be noted that while the foundation excavation plans (Plates 15-18) are included in the set, they do not represent final as-excavated foundation. The foundation report discusses changes to the foundation excavation that includes deleting benches at each construction joint and the general raising of the final foundation grade, most notably in Blocks 24-28. Included drill logs also include the post final excavation foundation confirmation drilling.

Appendix E Page No.	Original Drawing No.	Title
App. E-1	N/A	Geologic Map and Section of the Western Detroit Reservoir Area, Quartzville and Detroit Quadrangles, Linn and Marion Counties, Oregon (most recent geologic mapping of the dam site and western portion of the reservoir)
App. E-2	DE-20-37/1	Foundation Exploration: Areal Geology
App. E-3	Fnd. Rpt. Plate 2	Foundation Report: Foundation Exploration
App. E-4		Foundation Exploration: Log Profiles
App. E-5	DE-40-21/1	Foundation Exploration: Rock Contours
App. E-6	DE-40-23/1	Foundation Exploration: Logs of Drill Holes (1)
App. E-7	DE-40-23/2	Foundation Exploration: Logs of Drill Holes (2)
App. E-8	DE-40-23/3	Foundation Exploration: Logs of Drill Holes (3)

App. E-9	DE-40-23/4	Foundation Exploration: Logs of Drill Holes and Trenches (4)
App. E-10	DE-40-23/5	Foundation Exploration: Calyx Holes – Tunnels (5)
App. E-11	DE-110-10	Foundation Exploration: Logs of Drill Holes
App. E-12	Fnd. Rpt. Plate 3	Preliminary and Contract Holes (sheet 1 of 3)
App. E-13	Fnd. Rpt. Plate 4	Preliminary and Contract Holes (sheet 2 of 3)
App. E-14	Fnd. Rpt. Plate 5	Preliminary and Contract Holes (sheet 3 of 3)
App. E-15	DE-121-1	Excavation Plan (Sta. 0+00 to 3+54) (Incorrectly Marked “As-Built”)
App. E-16	DE-121-2	Excavation Plan (Sta. 3+54 to 7+54) (Incorrectly Marked “As-Built”)
App. E-17	DE-121-3	Excavation Plan (Sta. 7+54 to 11+57) (Incorrectly Marked “As-Built”)
App. E-18	DE-121-4	Excavation Plan (Sta. 11+57 to 16+34.33) (Incorrectly Marked “As-Built”)
App. E-19	DE-121-5	Excavation Plan (Stilling Basin and Channel) (May be Correctly Marked “As-Built”
App. E-20	DE-121-6	Excavation Sections (Stilling Basin and Channel) (May be Correctly Marked “As-Built”
App. E-21	Fnd. Rpt. Plate 6	Geologic Map of Dam Foundation
App. E-22	Fnd. Rpt. Plate 7	Axis Profile Showing Various Grades (“As-Built”)
App. E-23	Fnd. Rpt. Plate 8	Geologic Plan of Diversion Tunnel
App. E-24	DE-20-51/1	Grout Hole Backfill: Left Abutment Section
App. E-25	DE-20-51/2	Grout Hole Backfill: Right Abutment Section
App. E-26	DE-20-51/3	Logs of Foundation Exploration: Piezometer Holes and Drains (1972)
App. E-27	DE-20-51/4	Uplift Measuring System and Supplemental Foundation Drains: Plan and Profile A-A
App. E-28	DE-20-51/5	Uplift Measuring System and Supplemental Foundation Drains: Profiles 1 Through 4 Data Tabulation

SECTION 4 - HYDRAULIC DESIGN

4.1 GENERAL

This section describes the hydraulic design of the SWS. The pertinent hydraulic or water conveying features include the HIWs, LIGs, new SWS tower, existing penstocks, and RO-penstock bifurcation conduit(s). Estimated flow capacities, loads, headlosses and preliminary operational considerations are provided for these features.

A future means of fish collection for targeted species that uses the surface water inflow is briefly summarized. A FSS is slated to be installed in the second phase, following construction of the SWS. A separate DDR is under development for the FSS.

Detroit Dam is 48.5 miles above the mouth of North Santiam River. The North Santiam River Basin tributary to Detroit Dam is a fan-shaped area of 438 square miles, located on the west slope of the Cascade Range about 60 miles southeast of Portland, Oregon. Principal tributaries to North Santiam River above Detroit Dam, in downstream order, are Marion, Pamelia, and Whitewater creeks; Breitenbush River; and Blowout and Khey creeks (Figure 4-1).

The basin terrain is mountainous and covered with a heavy stand of coniferous trees. Extremes in elevation within the basin are 1,200 ft at the dam to 10,495 ft on the summit of Mount Jefferson. The average elevation of that part of the basin tributary to the dam is 3,765 ft.

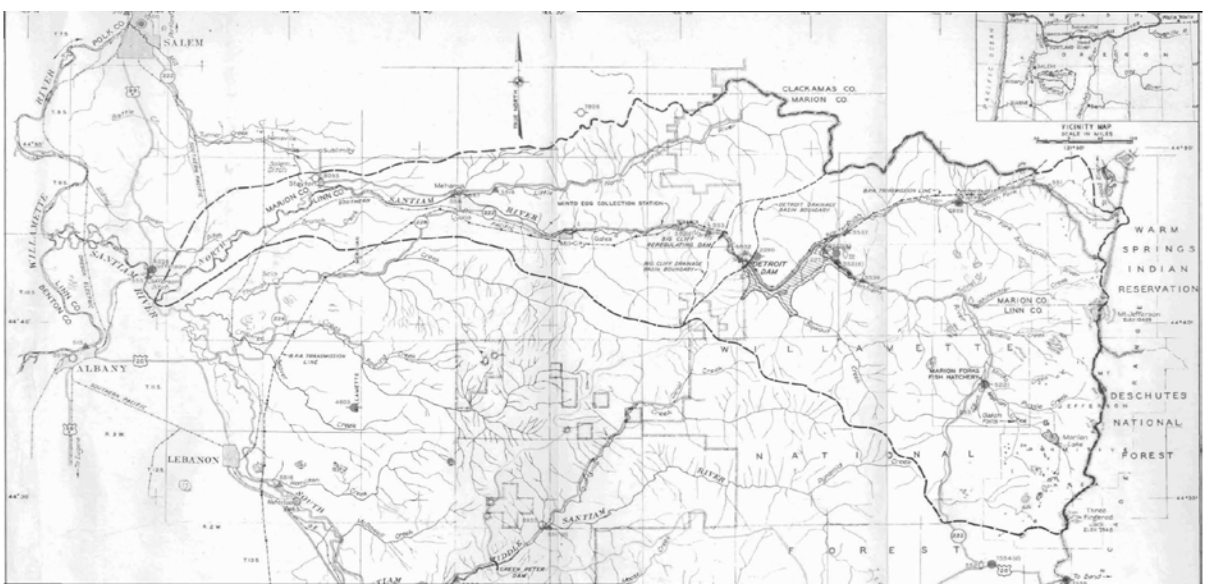


Figure 4-1. Detroit Reservoir Drainage Basin

4.1.1 Existing Project

Detroit Dam is a 450-ft high, 1,457-ft long concrete gravity structure. The dam has a gated spillway that is 294.5 ft long and 28.0 ft high with six spill bays, each 42 ft wide.

The spillway crest is at elevation 1,541.0 ft, maximum pool is elevation 1,574.0 ft, minimum conservation pool is elevation 1,450.0 ft, and minimum power pool is 1,425.0 ft.

Detroit Dam has four ROs, two with a centerline elevation of 1,265.3 ft, two at elevation 1,340.0 ft, and two turbines with penstock intake elevation at 1,403 ft. Facts and pertinent data tables for Detroit and Big Cliff dams are included in the Pertinent Data at the beginning of this report. An elevation view of Detroit Dam is shown in Figure 4-2. An interior view of Detroit Dam including water passage routes is shown in Figure 4-3.

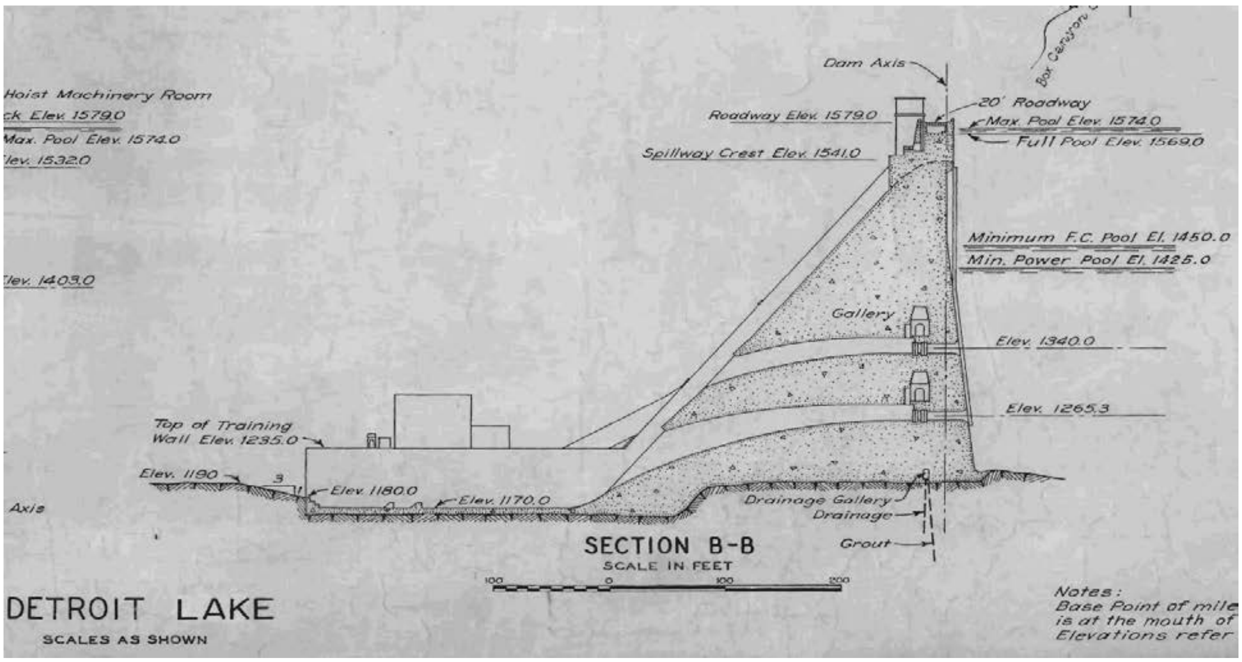


Figure 4-2. Elevation, Detroit Dam Structure

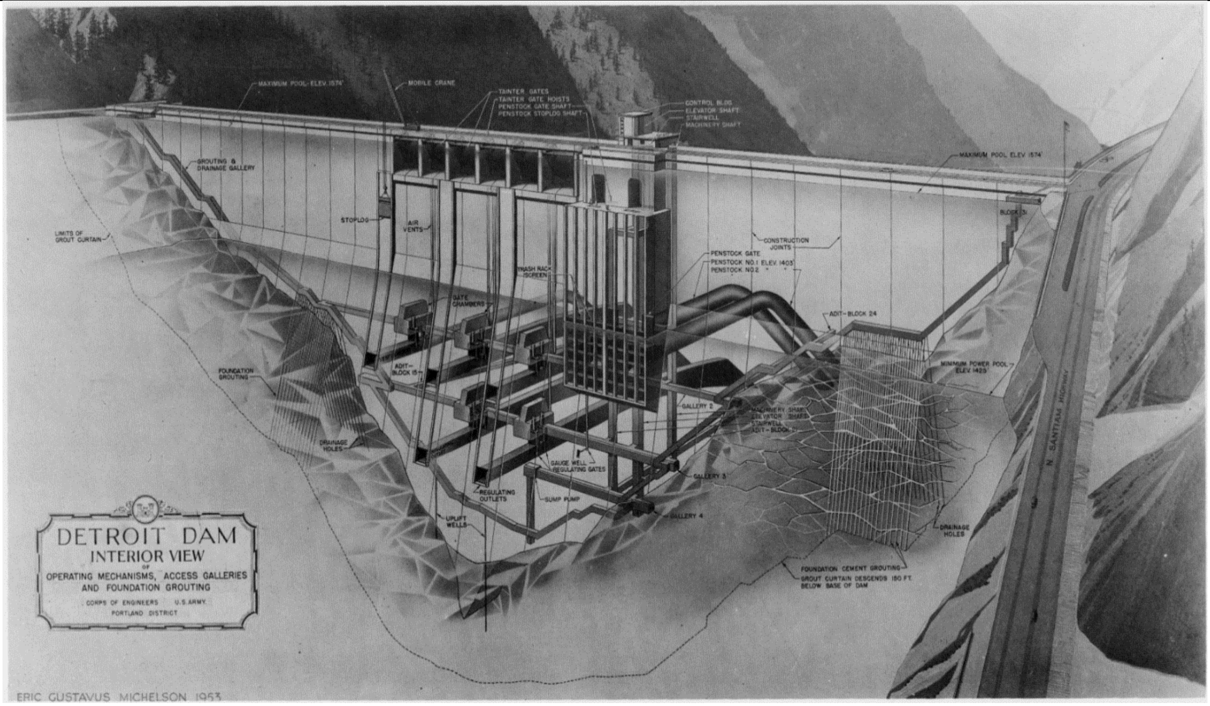


Figure 4-3. Interior View of Detroit Dam

4.1.1.1 Penstocks and Turbine Units

There are two penstocks that supply two 63.9 MW turbine units.

Both penstocks are 15 ft inside diameter with alignments over 400 ft long. The penstock drops 200 ft in elevation from a centerline intake of 1,403 ft down to 1,203 ft entering the powerhouse. Each penstock has a closure gate and an isolation bulkhead slot at the upstream end. Each penstock also has two air vents: a 24-inch vent just downstream of the closure gate and an 18-inch vent at the top of the sloped gradebreak.

The operational limit has been historically limited to 58 MW due to concerns about shaft integrity. The shaft concerns have been alleviated and the operational limit has reverted to the actual generator limit (63.9 MW). Hence, the maximum potential powerhouse discharge—that does not exceed cavitation limits--will increase from about 5600 cfs to 6200 cfs. This discharge would occur at transitional forebay elevations of about 1,480-1,490 ft.

4.1.1.2 Regulating Outlets

Detroit Dam has four ROs: two upper RO intakes at centerline elevation 1,340.0 ft and two lower RO intakes with a centerline elevation of 1,265.3 ft. All ROs are operated with a 200-ft head restriction above the respective centerline intake elevations. Hence, none of the upper ROs can be operated above the spillway crest elevation of 1541 ft.

Each RO tunnel is equipped with a service gate, emergency gate, and an isolation bulkhead slot. The RO gates are 5 ft 8 in wide and 10 ft high. The service gates are operated between 10-80% openings. Two bulkhead slots are shared by both upper and lower RO intakes located at equivalent dam stations. There is one bulkhead shared for all four intakes.

The upper RO and lower RO tunnels are approximately 190 ft and 260 ft long respectively. Downstream of the RO gates and air vent outlets, the 10 ft high by 5.67 ft wide tunnels are immediately expanded to 16 ft high x 7 ft wide. Beyond this point in the downstream direction, the invert is arched downward and the height of the tunnel is increased. The tunnels daylight at the downstream ogee slope for the spillway and ROs are protected from overhead spillway flow by means of flow deflectors located above the ROs.

Significant cavitation damage occurred in the concrete lining of the lower RO tunnels during high flows in 1953. In 1956, the first 65 ft of both lower RO tunnels (downstream of the gates) was steel lined, and operations were suspended in the lower regulating outlets until 2015, when pool levels fell below 1,450 ft. Relatively low flow rates (800 cfs) have been applied to augment temperature control during autumn months. Inspections have confirmed that as long as the head restriction (<1,465 foot pool) is maintained for lower RO operations, only modest and routine cavitation damage should occur on the steel plating immediately downstream of the service gates.

The hydraulic capacity of the upper RO tunnels is 13,050 cfs. Normal and maximum flood evacuation discharges are 10,000 and 17,000 cfs respectively.

4.1.1.3 Spillway and Stilling Basin

Detroit Dam has six spillway bays serviced by 42-ft wide Tainter gates. The overall spillway width is 294.5 ft wide and the crest invert elevation is 1,541 ft. When the Tainter gates are seated, the elevation of the top of the gates is 1,572 ft. The spillway design flood is 176,000 cfs.

Since the 2008 BiOp, the spillway has been operated when possible (typically late April to late September) to provide a higher, warmer water source and augment temperature control in the North Santiam River, downstream of the project. The future intent of the proposed SWS is to largely eliminate the need for this practice.

The stilling basin is 294.5 ft wide by 243.8 ft long. The invert is at elevation 1,170 ft, and there are two rows of baffle blocks located toward the downstream end that span the width of the basin. The stepped or sloped downstream endsill crests at an elevation of 1,190 ft, thus assuring a minimum depth of 20 ft within the stilling basin. (The Big Cliff pool varies between 1,180 and 1,210 ft, so the downstream pool can be at times below the water level in the Detroit stilling basin.)

4.1.1.4 Prototype Test Facilities at Detroit Dam: Test Spillway Chute

A test spillway chute capable of providing 24,000 cfs flow, consisting of a 42-ft, constant-width chute approximately 400 ft long, was provided by constructing an intermediate training wall to isolate spillway bay 6 (adjacent to the powerhouse) from the rest of the spillway. The training walls were outfitted with electrodes at 50-ft intervals for use in measuring velocity. Three observation windows were also installed on the outside wall of the chute for visual observation of the subsurface flow conditions. A movable footbridge, capable of moving up and down the test chute, spans the test chute and rides on rails at the top of the training walls.

The purpose of the high-velocity test chute was to measure the amount of bulking effect on the depth of flow that occurs in a full-scale, high-head, spillway chute due to entrainment of air and to attempt to relate the observed data to depth and velocity of flow. The results would then be compared to those obtained by scale models to determine if a correlation can be established for spillway design purposes. The effect of air entrainment on velocity of flow was also studied.

While portions of the test chute remain in place (observation windows and training walls), the facility has essentially been abandoned.

4.1.1.5 Prototype Test Facilities at Detroit Dam: Test Conduit

A test conduit capable of providing up to 4000 cfs and up to 318.5 ft of static head was provided by constructing an 8-foot-diameter conduit on the left side of the stilling basin. The intake is at elevation 1,340 ft (same intake elevation and shape as upper ROs) and the conduit terminates at elevation 1,245 ft along the left wall of the stilling basin. Emergency closure is provided by a single slide gate at elevation 1,340 ft (ROs have emergency gate and operating gates at elevation 1,340 ft). A regulating valve was provided along the top of the stilling basin wall (elevation 1,245 ft) to throttle flow through the test conduit.

When designed/implemented at Detroit, the prime advantage of the test conduit is the ability of the facility to provide high head for testing purposes, which would produce prototype cavitation pressures without the need for vacuum tanks. The original primary purpose of the test conduit was to test high-head control valves with particular emphasis on cavitation pressure, effects of venting, elimination of vibration, and determination of downpull forces on gate lips. Because of the head and discharge available at the test facility, the test conduit was also used to test the concrete composition on relative resistance to erosion, the length of steel liners requested downstream of regulating gates, and the design of transition from rectangular gates to a circular tunnel. A Feasibility Assessment was completed in May 2003 under a contract with the Office of Naval Research to determine whether a High Speed Drag Reduction Experiment could be located in the test conduit at Detroit Dam. The assessment revealed much of the test conduit would need to be refurbished and portions would need to be replaced to carry out the tests.

While portions of the test conduit remain in place, the facility has not operated in more than 30 years and has essentially been left abandoned. Stoplogs are in place on the upstream inlet (similar to RO bulkhead location), which may require diver assistance if they are to be removed.

Currently, a prime advantage of the test conduit is that the facility provides an existing route through the dam that could possibly be modified or adapted for a volitional fish passage bypass system. The test conduit is not used to meet Flood Damage Reduction (FDR) requirements; therefore, it provides an existing passageway through the dam that would only require modification, and not require drilling through the dam to provide a conduit for a volitional bypass system. Additional design work will be needed to determine if this test conduit could potentially be modified to allow for placement of a volitional bypass system through this portion of the dam.

Big Cliff Dam is 280 ft long and 172 ft high. The spillway crest is at elevation 1,161.5 ft, full pool is at elevation 1,206 ft, and minimum pool is at elevation 1,182 ft. The dam has three spill bays and one 18-MW capacity power generating unit (see Pertinent Data for Big Cliff). Due to Big Cliff re-regulation operations, the lake level fluctuates as much as 22 ft daily.

4.2 DESIGN REFERENCES

ENSR Corporation. Surface Bypass Program Comprehensive Review Report, prepared for USACE Portland District, December 31, 2007.

Miller, D.S. Internal Flow Systems, 3rd Edition. 2014

NMFS (National Marine Fisheries Service). Anadromous Salmonid Passage Facility Design. NMFS, Northwest Region, Portland, Oregon. 2011.

USACE. Detroit and Big Cliff Long Term Temperature Control and Downstream Fish Passage Engineering Documentation Report (Detroit Temperature and Downstream Passage EDR) March 2017.

USACE. Hydraulic Design Criteria 1987.

USACE, Portland District. Detroit Dam Water Temperature Control Structure Computational Fluid Dynamics Modeling, Draft Report. December 2013.

4.3 HYDRAULIC CRITERIA AND CONSIDERATIONS

4.3.1 General Criteria and Considerations

- Civil Works: The civil works of the passage facilities must be designed in a manner that prevents undesirable hydraulic effects (such as eddies and stagnant flow zones) that may delay or injure fish or provide predator habitat or predator access (NMFS 2011, Section 11.8.1.3). Also, hydraulic jumps and frequent decelerations should be avoided.

- The SWS is designed for a surface inflow up to 6200 cfs, per water temperature modeling results and required project operations.

- The high intake weirs will operate between pools 1425 – 1570 ft.

- The low level inlets are sized for up to 5920 cfs (with one gate redundancy) based on assuring turbine capacity and potential cold requirements to meet autumn target temperatures are met.

- Minimum North Santiam river flow from Big Cliff Reservoir is 1200 cfs. The Detroit Project does operate power peaking operations in which flow is shut off during low demand periods of the day. However, the Detroit project is normally operated to pass a sufficient daily quantity of flow to Big Cliff to assure minimum discharge criteria is met.

- Minimum gate opening criteria shall apply to low intake slide gates. A minimum 10 percent opening shall be applied.

4.3.2 Fish Passage Facility Flows and Head Differentials

The following fish passage facility sizing and flow criteria and assumptions were considered in the incorporation of the FSS into the design of the SWS:

- Except under bypass operations, all surface water shall be routed through the FSS before entry into the SWS.

- Surface water flow range for FSS is 1000-4500 cfs operated within NMFS criteria; the SWS wet well structure is designed to accommodate this range of flows.

- Maximum FSS flow capacity is 6200 cfs. FSS operations at 5600 cfs will exceed NMFS screen velocity criteria. Maximum capacity of the bypass operation is also 5600 cfs.

- The forebay to SWS wet well under FSS operations is expected to be between 3-5 ft.

- The normal head differential from the FSS conveyance channel through the SWS high intake weirs should not exceed 0.75 ft. The exception is when additional head differential between the forebay and SWS wet well is required to pull higher discharges through the LIGs.

- During interim or bypass operations (no FSS), the head differential between forebay and SWS will be limited to 0.4 ft to maintain a maximum trash rack velocity of 4 ft/second.

4.4 SWS

4.4.1 General

The SWS is designed to collect surface water within a range of flows from 1000 cfs to 6200 cfs. The low level inlets will also be able to collect 200-6200 cfs (with one gate redundancy) at 4.5 ft head differential. The total combined SWS discharge is 12,400 cfs.

This range is needed to accommodate a variety of possible fish collection options, provide adequate flows for water temperature control (see Section 5), and meet the operational requirements of the powerhouse and RO-Penstock Bifurcation conduit.

The proposed configuration of the At-Dam SWS is shown in Figure 4-4. The structure will use HIWs—using telescoping weir designs—to withdraw water from the surface of the reservoir over the range of forebay levels from 1,570 (full pool + 1 ft) to 1,425 (minimum power pool). In addition, there are four LIGs which will allow water to be withdrawn from the lower part of the reservoir. The LIGs will be located at two different levels to provide cold water for the temperature control system. The warm and cold water will join in the wet well to supply water to the two power penstocks at elevation 1,403 ft. A description of the design of the HIW and LIG are provided in sections 4.4.5 and 4.4.6 respectively.

Flows received by the HIW and LIG will be passed through the penstocks to the powerhouse and/or the RO-Penstock Bifurcation Conduits, described in section 4.4.4.

Design calculations can be found in Appendix C.

The FSS downstream fish passage feature will be included with the SWS in a second phase (Phase 2). This feature is further described in section 4.7.

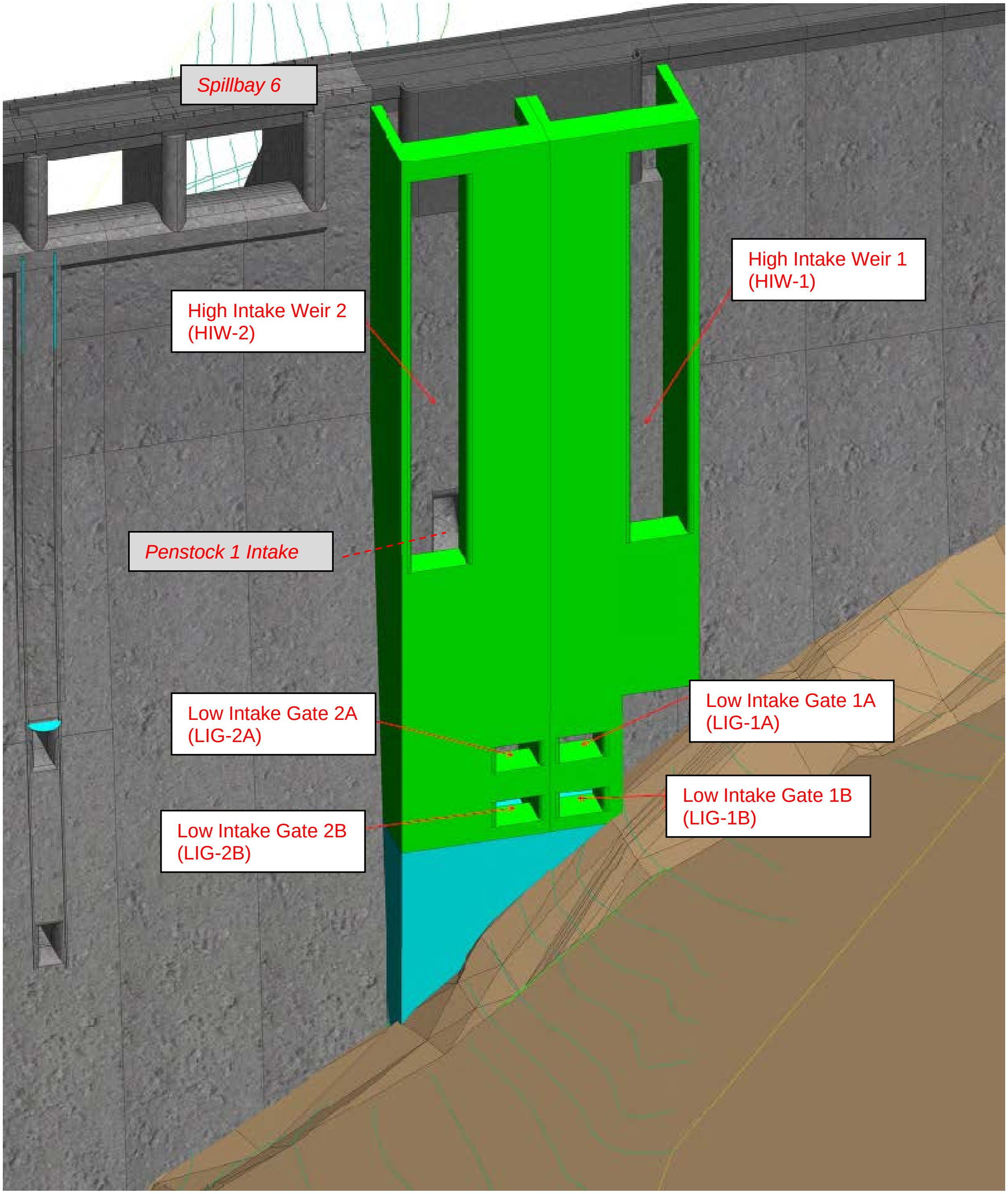


Figure 4-4. Isometric View of the Selective Withdrawal Structure (SWS)

4.4.2 SWS Location

The location of the SWS tower needs to be adaptable to the future FSS for downstream juvenile fish passage. The selected FSS alternative from the Detroit Temperature and Downstream Passage EDR has separate intake weirs positioned just north of spill bay 6.

The SWS location has moved multiple times through the design process as additional information has emerged.

The current location of the At-Dam SWS is on the upstream (east) face of the dam, and immediately adjacent to the existing penstock intakes. This way, flow moves directly from the SWS wet well into the penstocks, and the two penstock connection conduits developed in the previous design iterations are no longer needed.

A conceptual plan view of the SWS and FSS in context with the upstream face of the dam is shown in Figure 4-5. A brief description of the downstream passage features are described in Section 4-7.

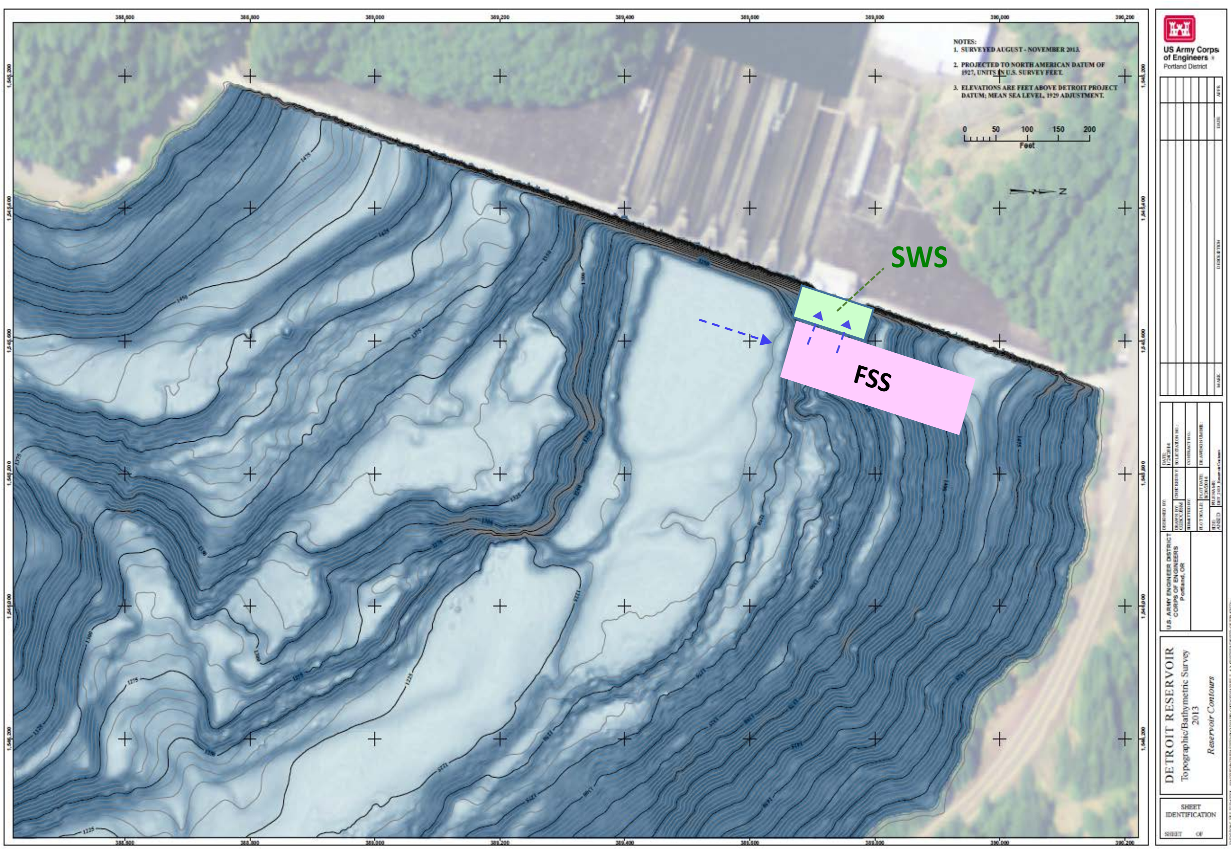


Figure 4-5. Conceptual Plan View of FSS with At-Dam SWS at Detroit Dam

4.4.3 Modifications to Existing Penstock Intake: Trash Rack Structure Removal

The revised 60% SWS design proposes to remove the 152 ft high x 108 ft wide concrete trash rack structure upstream (east) of the two penstock intakes (See Figure 4-6). This is required to remove flow constrictions and reduce internal headlosses for flow conveyance within the new tower. The saw cut lines for the trash rack structure are shown by the dashed redlines in Figure 4-6. This trash rack will be replaced by similar coarse trash racks on the new HIWs and in the two new trash rack hoods for the LIGs. The functionality of the existing bulkhead guides and the two closure gates will not be altered.

The existing penstock intakes have a coarse trash rack structure located 12.5 ft in front of the penstock intakes and 21.25 ft upstream of the dam axis. The trash rack structure consists of a grid of 2-ft wide concrete beams at approximately 12-ft centers. The concrete structure houses 112 steel trash rack panels. These panels are 10 ft square with ¼-inch wide x 6-inch bars at 6-inch centers.

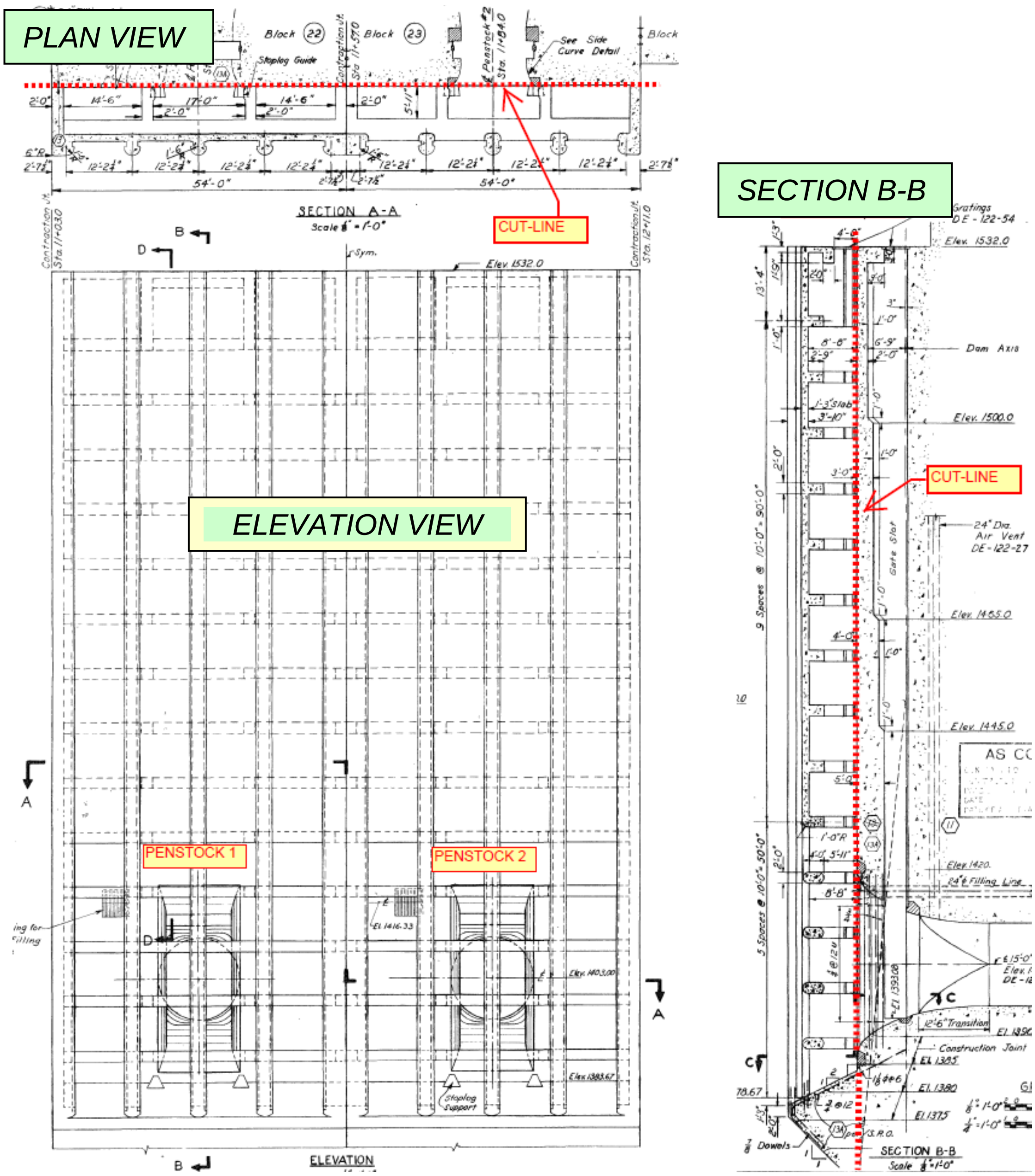


Figure 4-6. Schematic of Penstock Trash Rack Structure Removal

4.4.3.1 Internal Headloss Estimates within the SWS

The expected general flow patterns inside the SWS tower are shown in Figure 4-7. Not shown is the dividing wall between the two monoliths, which has openings at approximately 25% porosity allowing some cross-over flow between monoliths. This cross-over capability was neglected to attain more conservative internal headloss estimates.

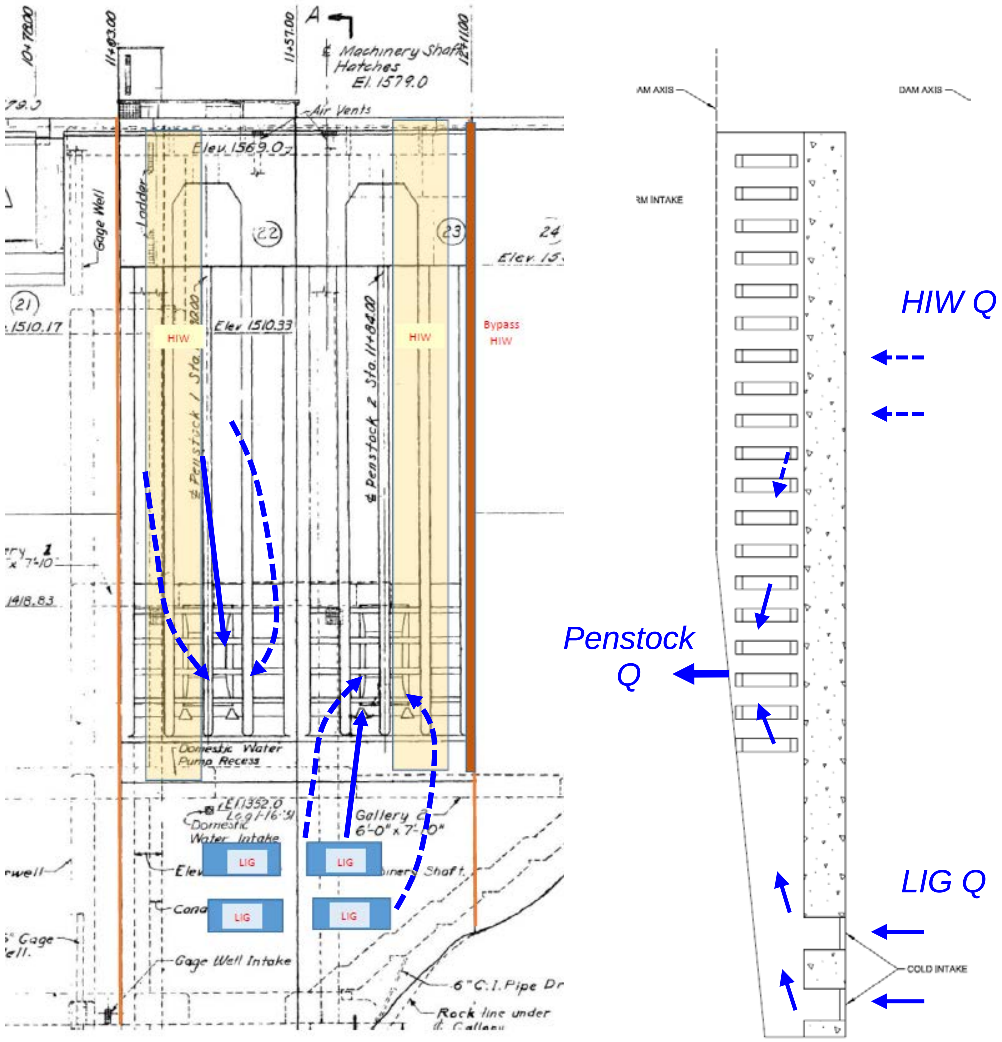


Figure 4-7. General Flow Patterns within SWS

For the estimation of the internal losses from the LIGs to the penstock intakes, an assumption of 2/3 of the total LIG inflow was applied for one side (or monolith) only. (As stated above, the cross-over flow benefit was neglected.) The inside tower depth is 18.25 ft at the penstock intake and 14.1 ft at the average LIG location (elevation 1,321 ft). To attain assumed cross-sectional flow areas, the average lateral width of the flow passing from LIG to penstock intake was assumed to be 30 ft (or 2 x LIG width). Upon entering the tower, the LIG flow will impinge on the existing dam face forcing the flow to spread laterally (north-south). Minor loss coefficients were assumed to be 1.2 for turning upwards at LIG location and 0.8 for turning back horizontally to the more rounded penstock intake. Values were approximated from tee coefficients (100% flow ratio from branch to main and vice-versa) in Miller (2011). The results are shown in Table 4-1.

Table 4-1. Estimated Internal SWS Headlosses from LIG to Penstock

Σ LIG Q (cfs)	2/3 Q (cfs)	Velocity near LIG (ft/s)	Velocity nr intake (ft/s)	VH1 (feet)	VH2 (feet)	Internal HL (feet)
1000	667	1.58	1.22	0.04	0.02	0.07
1500	1,000	2.36	1.83	0.09	0.05	0.15
2100	1,400	3.31	2.56	0.17	0.10	0.29
3000	2,000	4.73	3.65	0.35	0.21	0.59
3500	2,333	5.52	4.26	0.48	0.28	0.80
4000	2,667	6.30	4.87	0.62	0.37	1.04
4800	3,200	7.57	5.84	0.89	0.53	1.50
5600	3,733	8.83	6.82	1.22	0.73	2.04
6200	4,133	9.77	7.55	1.49	0.89	2.50

For the estimation of the internal losses from the HIWs to the penstock intakes, an inside tower depth of 18.25 ft at the penstock intake was applied. The average lateral width of the flow passing from a single HIW to penstock intake was assumed to be 35 ft (or 1.75 x HIW width). (Headlosses were computed per unit HIW to penstock intake. When total FSS flow is below 3100 cfs, then all flow was assumed to pass through one HIW instead of two.) Minor loss coefficients were assumed to be 1.2 for turning downwards at LIG location and 0.8 for turning back horizontally to the penstock intake. The results are shown in Table 4-2.

Table 4-2. Estimated Internal SWS Headlosses from HIW to Penstock

Σ HIW Q (cfs)	Unit HIW (cfs)	Velocity (ft/s)	VH (feet)	Internal HL (feet)
2000	2000	3.13	0.15	0.31
2500	2500	3.91	0.24	0.48
3100	1550	2.43	0.09	0.18
4000	2000	3.13	0.15	0.31
4500	2250	3.52	0.19	0.39
5000	2500	3.91	0.24	0.48
5600	2800	4.38	0.30	0.60
6200	3100	4.85	0.37	0.74

The estimated internal headlosses are notably higher when coming from the LIGs versus the HIWs. This is largely due to a presumed imbalance of flow from the LIGs and the reduced inside depth of the tower at the LIG location.

The above headloss estimations may be refined through subsequent CFD simulations in the P&S Phase.

4.4.4 Regulating Outlet (RO) - Penstock Bifurcation Conduit(s)

The proposed RO-penstock bifurcation conduits are intended to pass flow from the SWS to the tailrace when one or both units are not operating. The RO bifurcations will wye off the existing penstocks to be conveyed to the stilling basin when one or two units is not operating. The purpose of the RO bifurcations is to continue passing surface water through the FSS and SWS when one or more units is not operational. The RO-penstock bifurcation will pass flow up to 6200 cfs that normally would be discharged through either the existing ROs or the spillway. This way objectives of downstream juvenile fish passage and temperature control continue to be met during unit outages.

The number of bifurcations has not been decided. While the text presently assumes two bifurcations, the final number may be one.

In previous design iterations, three in-reservoir conduits were proposed to connect the SWS to the existing intakes for the penstocks and one upper RO intake. With the revised design, no in-reservoir connection conduits are required. The two penstock connection conduits have been eliminated with the At-Dam SWS location. The previous in-reservoir RO connection conduit has been replaced by the proposed RO – penstock bifurcation conduits.

The general location, plan and elevation schematics are provided in Figures 4-8 through 4-10. An isometric view is also shown in the Mechanical Design plates in appendix D.

4.4.4.1 Primary Features of RO-Penstock Bifurcation

Each RO-penstock bifurcation will have the following features:

- 13-ft diameter wye off existing 15-ft penstock
    - Assume a section 15-ft pipe removed
    - New flanges welded to 15-ft pipe
    - New fabricated wye section installed with dresser coupling
- Reducer to 11-ft diameter pipe
- 11-ft isolation gate valve
- 11-ft diameter cone or Howell-Bunger valve
- Means of isolating existing 15-ft diameter penstock(s) downstream of the new wye

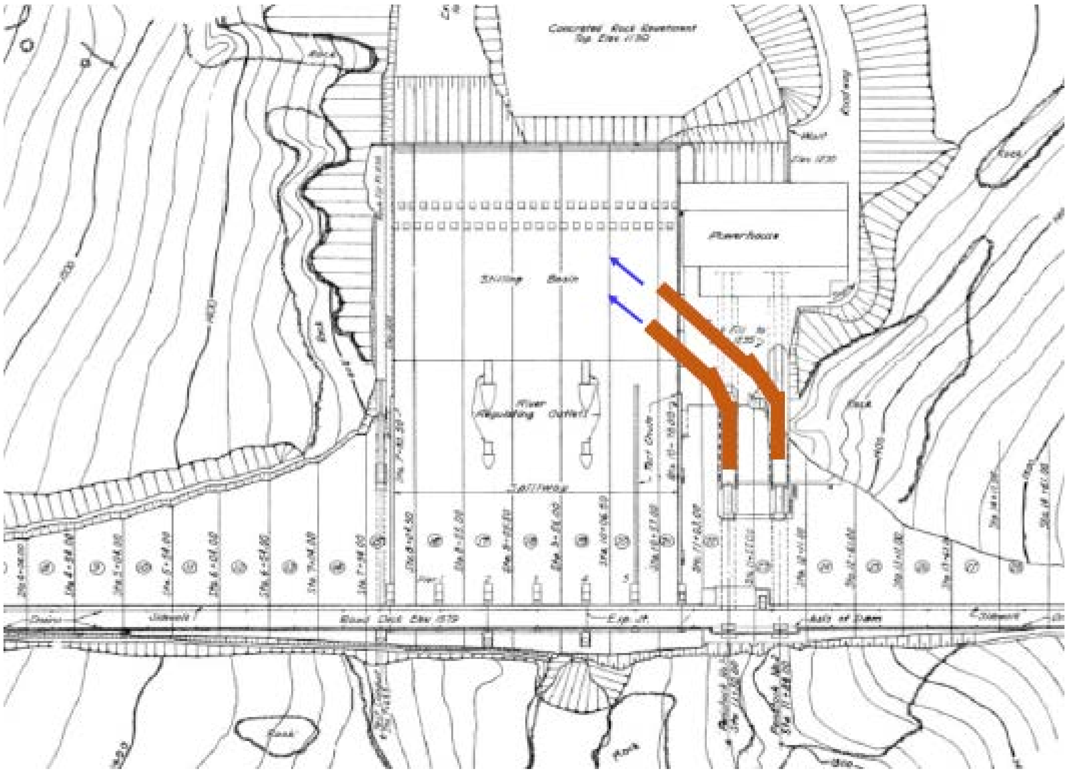


Figure 4-8. General Plan Schematic of RO – Penstock Bifurcation Conduit

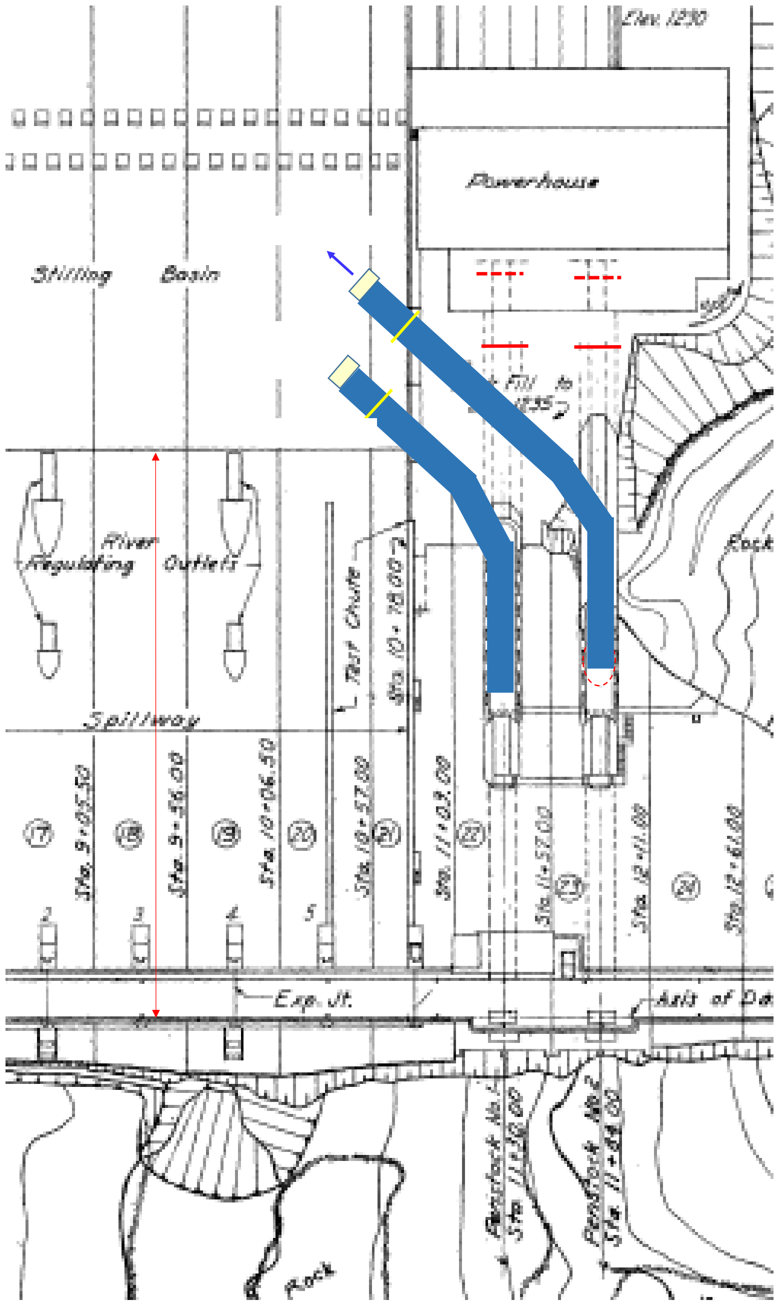


Figure 4-9. Schematic Plan Details of the RO – Penstock Bifurcation Conduit

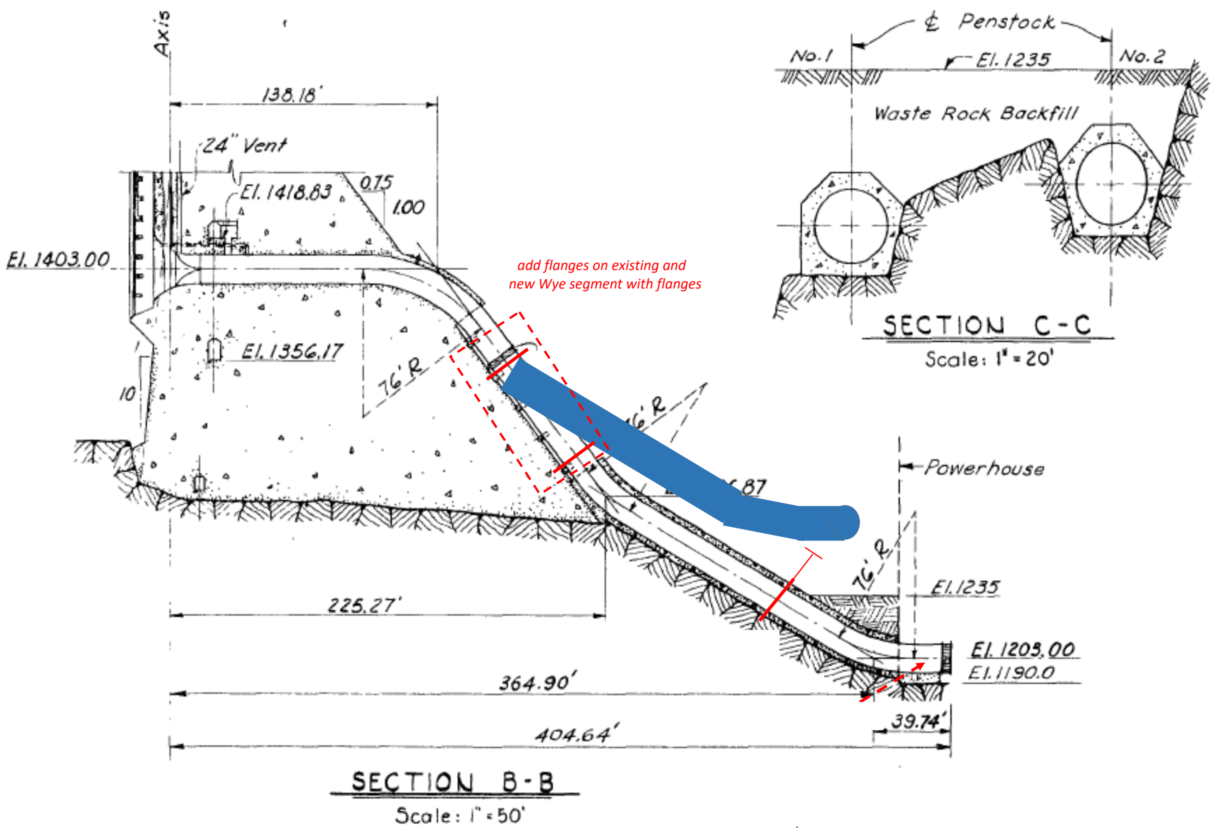


Figure 4-10. Schematic Elevation Details of the RO–Penstock Bifurcation Conduit

4.4.4.2 Turbine Penstock Headlosses: Existing versus Future

One pressing question is the difference in penstock system headlosses between the existing system and the future system with the FSS, SWS and RO-penstock bifurcation. All headlosses are estimated about 65% down the existing penstock to a match-line shown in informational drawing DE-122-21 (in latter section of Appendix A: Plates). This match-line is located 70 ft down the 53.1-degree slope. Both Penstocks 1 and 2 are the same upstream of this match-line and different downstream. The match-line represents the initial 253.6 ft of the 15 ft diameter conduit. The wye for RO-penstock bifurcation will be located about 30 ft upstream of the match-line.

Headlosses are estimated for the existing system down to the match-line. Table 4-3 shows the relatively modest estimated headlosses for routine dual turbine unit operations for the existing system. The form drag (i.e. minor) loss coefficients are largely obtained from Internal Flow Systems (Miller 2014).

Table 4-3. Headlosses in Existing Turbine Penstock to Match-line

Summary of Existing Penstock Conduit Headlosses to Match line for both Conduits (Assumming Dual Unit Operation)								
	Total Conduit Length =		253.6	feet				
	Σ Minor Loss Coefficients =		0.24					
Turbine	Max V	Vh	RE	f	fL/D	Headlosses (ft)		
Σ Q	Ave Vel.	Vel Head	Reynolds	friction	friction	Friction	Minor	Conduit
(cfs)	(ft/s)	(ft)	No.	factor	term	Vh x fL/D	ΣK x Vh	(ft)
4000	11.3	1.99	1.1E+07	0.0086	0.15	0.29	0.48	0.8
4500	12.7	2.52	1.3E+07	0.0086	0.14	0.36	0.61	1.0
5000	14.1	3.11	1.4E+07	0.0085	0.14	0.45	0.75	1.2
5600	15.8	3.90	1.6E+07	0.0084	0.14	0.55	0.95	1.5
6200	17.5	4.78	1.8E+07	0.0083	0.14	0.67	1.16	1.8

The combination of future changes (FSS, internal SWS tower and RO-penstock bifurcation) will bring about additional headlosses in the penstock and some loss to the power generating capacities. Table 4-4 shows the estimated headlosses for routine dual turbine unit operations under the proposed future system with the FSS, SWS and RO-bifurcation(s). The majority of increased headlosses will be incurred upstream of the unaltered penstock intake by the FSS and SWS. Losses caused by the angled bifurcation wye will be comparatively modest. In general, the overall differences are about 4 to 7 ft depending on flow rates.

Table 4-4. Headlosses in Future Turbine Penstock with FSS to Match-line

Summary of Penstock Conduit Headlosses under FSS Operations to Match line for both Conduits (Assumming Dual Unit Operation)										
	Total Conduit Length =		253.6	feet						
	Σ Minor Loss Coefficients =		0.29							
Turbine	Max V	Vh	RE	f	fL/D	Headlosses (ft)			FSS +	TOTAL
Σ Q	Ave Vel.	Vel Head	Reynolds	friction	friction	Friction	Minor	Conduit	SWS	System
(cfs)	(ft/s)	(ft)	No.	factor	term	Vh x fL/D	ΣK x Vh	(ft)	(ft)	(ft)
4000	11.3	1.99	1.1E+07	0.0086	0.15	0.29	0.58	0.9	3.7	4.6
4500	12.7	2.52	1.3E+07	0.0086	0.14	0.36	0.73	1.1	4.1	5.2
5000	14.1	3.11	1.4E+07	0.0085	0.14	0.45	0.90	1.3	4.5	5.9
5600	15.8	3.90	1.6E+07	0.0084	0.14	0.55	1.13	1.7	5.1	6.7
6200	17.5	4.78	1.8E+07	0.0083	0.14	0.67	1.39	2.1	6.0	8.1

Under normal unit operations, all flow entering the penstock intakes will be discharged through the units. Conversely, when the unit is down, all penstock flow will be diverted into the RO-bifurcation and discharged ultimately to the stilling basin. There will be some rare occasions where there is a simultaneous combined discharge through both the unit and RO-bifurcation attached to the same penstock. Historically, there is a relatively high number of occasions during late spring (April – June) in which flow was spilled (prior to the current temperature control operations) while simultaneously operating the turbine units. This situation occurs when the pool has neared or reached maximum conservation pool (1,563.5 ft). At this pool, the dual unit capacity was 4,000 cfs (increased to about 4600 cfs with the higher generation limit). When project inflow has exceeded 4000 cfs at the high pool, excess flow had to be sent over the spillway. Per the 2017 EDR (section 3.3), the measured survival rates over the spillway during 2009 tests varied to between 60% and 84% with about 50% injury. After being spilled, the remaining surviving fish are subjected to additional losses through the powerhouse at Big Cliff. For these reasons, incidental spill of moderate excess flow should be avoided up to 6200 cfs, or 3100 cfs per penstock, by running the extra flow through the FSS and RO-penstock bifurcation. In these cases, the headlosses could be as much as 7 ft higher than the current 4000 cfs dual unit operation. One way this might be avoided is by running excess flow through the RO-penstock bifurcation when the units are not operating.

Note: at minimum power pool (elevation 1,425 ft), the maximum dual turbine discharge is about 3800-4000 cfs before encountering the cavitation limits in the units. Given that the difference in existing versus future headloss is about 4 ft under 4000 cfs discharge, the minimum power pool may need to be raised 4-5 ft.

Preliminary Dimensions and Headlosses for the RO-Penstock Bifurcation Conduit(s)

The preliminary dimensions of the RO-penstock bifurcation conduits and headloss coefficients are shown in Table 4-5. The preliminary estimated conduit headlosses are shown as a function of an assumed-combined dual pipe discharge in Table 4-6. The headlosses include the existing 15-ft penstock up to the wye, the 15-ft x 13-ft wye, the 13-ft pipe section with bend, reducer to 11-ft, and 11-ft pipe section with isolation valve. The outlet control valve (either Howell-Bunger or cone valve) is not included as the discharge is a function of the approaching head and valve opening. The form drag (i.e. minor) loss coefficients are largely obtained from Miller (2014) and USACE (1987). The friction factor is based on an assumed pipe roughness (Ks) of 0.0001 ft for steel conduits from Hydraulic Design Criteria (USACE 1987). The total headlosses for the RO-penstock bifurcation conduit (including existing penstock conduit up to the wye) are listed in the yellow highlighted column. With the conservative 6-ft headloss estimated for the FSS and SWS, the additional system headlosses are totaled in the green highlighted column on Table 4-6.

Unlike the turbine penstock going to the power generating units, minimization of headloss is not a priority for the flow in the RO-penstock bifurcation. In fact, once the flow passes the wye into the RO-penstock bifurcation conduit, it is preferable to reduce the energy that will ultimately be discharged into the stilling basin. For this reason, a couple of in-line orifices may be considered in the 13-ft pipe section to help reduce the energy in the system before release to the tailrace.

Table 4-5. RO - Penstock Bifurcation: Dimensions and Headloss Coefficients

		Dimensions				Ks	Minor Loss Coeffcients				
#	Piece Type	Distance	Width	Height	Area	Roughness	Wye	Bend	Reducer	Reducer	Sum K
		(ft)	(ft)	(ft)	ft^2	(ft)	Kj	Kb*	Kc	Kg	Σ K
1	Wye	20.0	13	*round*	132.7	0.0001	0.81				0.81
2	Straight	80.0	13	*round*	132.7	0.0001					0.00
3	Bend	40.0	13	*round*	132.7	0.0001		0.11			0.11
4	Straight	20.0	13	*round*	132.7	0.0001					0.00
	Subtotal 13	160.0									0.92
5	Reducer	8.0	11	*round*	95.0	0.0001			0.1		0.10
6	Straight	30.0	11	*round*	95.0	0.0001					0.00
7	Iso. Gate	2.0	11	*round*	95.0	0.0001				0.05	0.05
8	Outlet	100.0	11	*round*	95.0	0.0001				**	
	Subtotal 15	140.0									0.15
	sum	300.0									1.07
*	Bend radius/Dia = 2.5, 60 degree angle assumed										
**	varies with valve control										

Table 4-6. Estimated RO - Penstock Bifurcation Conduit Headlosses up to Control Valve (with SWS + FSS)

Summary of RO- Penstock Birfurcation Conduit Headlosses under FSS Operations to Outlet Control Valve under Split Flows (Assumming Dual Pipe Operations)

		15-ft Penstock	13-ft Pipe	11-ft Pipe				Σ Minor Loss Coefficients:						
Conduit Inside Diameter (feet):		15	13	11				15-ft Pnstk	13-ft Pipe	11-ft Pipe				
Conduit Lengths (feet):		253.6	160	140				Σ K1	Σ K2	Σ K3				
Diameter/roughness (D/ks):		150,000	130,000	110,000				0.29	0.92	0.15				
Turbine	RO-BIF	V1	V2	V3	VH1	VH2	VH3	RE1	RE2	RE3	f1	f2	f3	
Σ Q	Σ Q	Penstk Vel.	13' Vel.	11' Vel.	Penstk VH	13' VH	11' VH	Pnstk RE	13' RE	11' RE	Pnstk fric.	13' fric.	11' fric.	
(cfs)	(cfs)	(ft/s)	(ft/s)	(ft/s)	(ft)	(ft)	(ft)	No.	No.	No.	factor	factor	factor	
2000	2000	11.3	7.53	10.52	1.99	0.88	1.72	1.1E+07	6.5E+06	7.7E+06	0.0086	0.0086	0.0091	
2000	3000	14.1	11.30	15.78	3.11	1.98	3.87	1.4E+07	9.8E+06	1.2E+07	0.0085	0.0085	0.0088	
2000	4200	17.5	15.82	22.10	4.78	3.89	7.58	1.8E+07	1.4E+07	1.6E+07	0.0083	0.0083	0.0086	
0	5600	15.8	21.10	29.46	3.90	6.91	13.48	1.6E+07	1.8E+07	2.2E+07	0.0084	0.0084	0.0085	
0	6200	17.5	23.36	32.62	4.78	8.47	16.52	1.8E+07	2.0E+07	2.4E+07	0.0083	0.0083	0.0084	

		Friction Terms (fL/D)			Σ Friction headlosses (feet)			Σ Minor Loss headlosses (feet)			Σ Head Losses (ft)		
Turbine	RO-BIF	$(fL/D)_1$	$(fL/D)_2$	$(fL/D)_3$	VH_i x $(fL/D)_i$			VH_i x ΣK_i			Σ RO-	FSS +	TOTAL
Σ Q	Q	Pnstk fric.	13' fric.	11' fric.	Penstk	13' fric.	11' fric.	Penstk	13' minor	11' min.	Penstock	SWS	System
(cfs)	(cfs)	term	term	term	losses	losses	losses	losses	losses	losses	Bifurcation	(ft)	(ft)
2000	2000	0.1461	0.1064	0.1162	0.29	0.09	0.20	0.58	0.81	0.26	2.2	3.4	5.6
2000	3000	0.1434	0.1044	0.1123	0.45	0.21	0.43	0.90	1.82	0.58	4.4	4.5	8.9
2000	4200	0.1411	0.1027	0.1097	0.67	0.40	0.83	1.39	3.58	1.14	8.0	6.0	14.0
0	5600	0.1422	0.1035	0.1079	0.55	0.72	1.45	1.13	6.36	2.02	12.2	5.1	17.3
0	6200	0.1411	0.1027	0.1074	0.67	0.87	1.77	1.39	7.79	2.48	15.0	6.0	21.0

4.4.4.3 Estimated Thrusts in the RO-Penstock Bifurcation Conduit(s)

The maximum normal thrusts on the RO-penstock bifurcation conduits were computed based on maximum design discharges and/or maximum pool head (1,574 ft) for the wye and the bend. Case 1 is hydrostatic under maximum head; Case 2 is maximum flow (3100 cfs per pipe) under maximum head minus head loss up to the feature. Case 2 (maximum flow at high pool) was the governing condition. The preliminary parameters for the wye and bends (both 13-ft diameter sections) are the following:

- Wye Elevation = 1,340 ft

- Wye Angle ϴ = 45º (may be closer to 30º)
- Bend Elevation = 1,246 ft
- Bend Angle ϴ = 75º (assumed, probably angle closer to 60º)

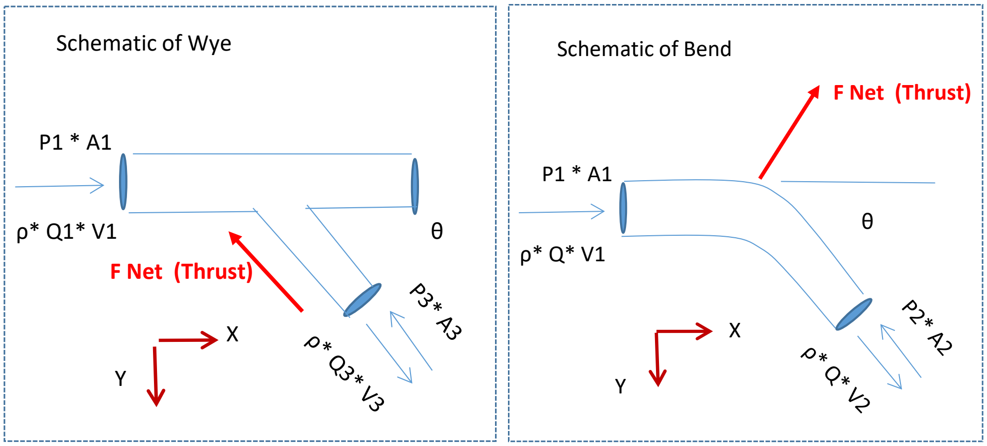


Figure 4-11. Schematic of Thrust Forces on Wye and Bend

Maximum Normal Thrust on RO - Penstock Bifurcation Conduits (Q = 3100 cfs per conduit)

F Net = sqrt(Fx²+Fy²)

bend	**Fx =**	**P1*A1 - P2*A2*cos {ϴ} - ρ*Q*(V2*cos {ϴ} - V1)**
bend	**Fy =**	**{P2*A2 + ρ*Q2*V2} *sin {ϴ}**
wye	**Fx =**	(P3*A3+ ρ*Q3*V3) * cos(θ)
wye	**Fy =**	(P3*A3+ ρ*Q3*V3) * sin(θ)

Wye:		
Fx =	1,422	kips
Fy =	1,422	kips
F Net =	2,011	kips

Bend:		
Fx =	2,070	kips
Fy =	2,697	kips
F Net =	3,400	kips

The thrusts resulting from the pending load rejection transient analyses will likely be larger than the above values. The load rejection will also cause subatmospheric pressures and an air vacuum valve or vent will be required for any high points or downward directed gradebreaks.

4.4.4.4 Jet Trajectories into Stilling Basin

Jet trajectories were estimated assuming hooded cone (or Howell-Bunger valves) into the 294.5-ft wide stilling basin. The amount of drop depends on the height of the 11-ft diameter outlet above the stilling basin sidewall. Some pertinent elevations:

- Maximum Forebay 1,574 ft
- Top of stilling basin side wall: 1,235 ft
- Stilling basin invert: 1,170 ft
- Endsill elevation: 1,180 ft
- Minimum Big Cliff Pool: 1,180 ft
- Assumed RO-Penstock Bif outlet CL: 1,245.5 ft (=1235'+5' + D/2)
- Outlet Slope = 10%
- Neglect upstream headloss (so outlet velocity = sqrt {2g * (max FB – min TW)}

With the above conservative assumptions, the maximum outlet velocity is about 143 ft/s. Tracking the top of the jet, the maximum fall distance is 71 ft. Assuming the outlet releases are perpendicular to the flow axis of the stilling basin, the jet will intersect minimum tailwater at about 105 ft from the north sidewall, or about 35% across the stilling basin.

Concerns that remain to be addressed include potential scour of the stilling basin floor, baffle block protection, and possible gasification of the flow.

4.4.4.5 Considerations between One or Two RO-Penstock Bifurcation Conduits

If one RO-penstock bifurcation is installed, there runs the possibility of being limited to 3100 cfs for surface water flow for temperature control. As the FSS is being designed for a maximum of 5600 cfs, the limit to fish collection flows would more likely fall to 2800 cfs since the screen hydraulics (i.e. sweeping velocities) are definitely superior at 2800 cfs in a single barrel when compared to 3100 split into two barrels.

The above scenario would occur during any outage of the unit or penstock in which the sole RO-penstock bifurcation was not installed. If the outage occurs during a period of high need for both surface water passage and fish collection (March – June) and dual units are operating, then there is a demand to push flow through the RO-penstock bifurcation as well as the turbine unit on the same penstock. This will detract from the head applied to the power generation.

The flow limitation described above would have much less impact during summer and late autumn months. During summer months, normal operations are single turbine unit due to limited reservoir inflow. During late autumn months, there is a high demand for cold water and the surface water collection will be reduced to 20 – 50%.

During winter months (except early December), Detroit Lake tends to be isothermal so the 3100 cfs (or 2800 cfs) limitation primarily pertains to fish collection.

Dual unit operations run between 3800 – 6200 cfs depending on pool elevation and cavitation limits.

Tables 4-7 and 4-8 show the total project and powerhouse flow rates that are exceeded at various percentages of time for each month based on hourly flow data recorded from 1985 – 2016. The left column for each table shows the percentile that is exceeded. These data pertain to only the times in which either the overall project or the powerhouse is passing flow. At the bottom of each table under the yellow highlighted heading, is the average percent of time in which the project or powerhouse has passed flow for each month.

Over the same data history, 3100 cfs has been exceeded 49% of the time during which flow has been discharged past the dam. During March through June, the discharge of 3100 cfs is exceeded 44%, 48%, 57% and 42% of the time respectively.

Table 4-7. Detroit Hourly Project Discharge (non-zero) Exceedance Data

Detroit Dam Hourly Project Discharge (non-zero) Exceedance Data (cfs)													
Period of Record: 1985 - 2016													
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Max	14,120	14,140	11,860	11,060	11,370	9,750	7,430	6,810	7,210	8,100	10,240	13,070	14,140
1%	13,690	13,369	9,240	9,350	9,080	5,980	5,525	4,820	4,960	5,970	9,400	11,180	9,970
5%	9,760	9,530	6,720	5,510	4,990	4,230	3,930	3,962	4,140	4,580	7,510	9,310	6,710
10%	7,747	6,912	4,690	4,360	4,440	3,920	3,900	2,720	4,100	4,510	6,300	7,760	5,290
25%	5,320	4,710	4,210	4,040	3,920	3,900	2,250	1,990	4,000	4,350	4,950	5,410	4,390
Average	4,728	3,705	3,231	3,159	3,288	2,768	2,199	1,993	2,511	3,307	4,391	4,773	3,425
75%	2,650	2,290	2,150	2,020	1,980	1,940	1,890	1,830	1,970	2,170	3,220	2,690	1,990
90%	2,270	1,120	1,700	1,970	1,950	1,490	1,090	970	1,320	2,010	2,330	2,450	1,810
95%	1,750	940	980	1,790	1,880	1,190	830	860	840	1,455	2,230	2,120	1,150
99%	1,080	800	720	950	1,300	630	450	620	508	720	1,650	1,530	650
Min	30	50	70	50	50	50	50	50	50	50	40	50	30
Total Percent of Time Project Discharged Flow during 1985 - 2016													
	67%	51%	50%	56%	71%	64%	49%	48%	64%	74%	75%	70%	62%

Table 4-8. Detroit Hourly Powerhouse Discharge (non-zero) Exceedance Data

Detroit Dam Hourly Powerhouse Discharge (non-zero) Exceedance Data (cfs) *Period of Record: 1985 - 2016*													
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Max	5,760	5,780	5,110	4,600	4,730	4,630	4,740	4,910	5,110	5,390	5,740	5,750	5,780
1%	5,500	5,400	4,780	4,580	4,460	4,400	4,613	4,770	4,210	4,630	5,480	5,520	5,410
5%	5,360	5,110	4,510	4,230	3,950	3,920	3,930	3,970	4,140	4,540	5,170	5,410	5,160
10%	5,290	4,890	4,320	4,090	3,930	3,910	3,910	2,360	4,100	4,490	5,010	5,350	4,910
25%	5,060	4,530	4,150	4,010	3,910	3,900	2,050	1,990	4,020	4,350	4,820	5,150	4,250
Average	3,868	3,254	3,041	2,892	2,929	2,653	2,360	2,059	2,598	3,394	3,790	4,055	3,155
75%	2,550	2,340	2,150	2,000	1,960	1,950	1,950	1,950	1,990	2,180	2,440	2,660	2,000
90%	2,250	2,190	2,080	1,970	1,950	1,940	1,890	1,350	1,770	2,090	2,280	2,420	1,950
95%	2,020	1,160	1,150	1,920	1,900	1,770	1,567	970	1,270	1,970	2,170	2,025	1,780
99%	1,310	1,110	970	1,160	1,410	1,212	591	580	970	1,190	1,800	1,460	970
Min	30	50	70	50	50	20	50	10	30	50	35	50	10
Total Percent of Time Units were Operated during 1985 - 2016													
	64%	47%	47%	53%	68%	55%	37%	40%	55%	65%	69%	66%	55%

4.4.5 HIWs

4.4.5.1 Description of Design

The telescoping HIWs are aligned across the width of the SWS with two weir opening slots:

- Two 20-ft wide weirs on the east face to collect screen flow from the FSS
- Flow bypassing the FSS (when ballasted) will use the same openings
- Weir crest invert range: 1,412.5-1,570 ft

Both weirs have three leaves with a height of 52.5 ft each to form the telescoping weirs. All inflow goes over the top leaf only (there is no separation of leaves below the crest). This arrangement allows for the gate system to track with the forebay to withdraw at the appropriate elevation for mixing. The system will pass water over a weir crest submerged by the forebay. The weirs will have rounded crests to reduce injury for entrained fish (pertinent during interim or ballasted FSS operations).

With previous SWS iterations, separate bypass weirs were designed for periods when the FSS is out of service. With the At-Dam SWS, there is no room or structural capacity to house the bypass weirs. Consequently, the coarse trash racks designed for the bypass or interim operations will need to be on the face of the dam over the HIW openings to accommodate the ballasted FSS operations.

Drawings showing the high intake weirs are shown in the structural plates.

4.4.5.2 HIW Operations

The operation of the telescoping weirs will allow for skimming the uppermost portions of the water column for temperature control. The telescoping weir leaves will stack and can be lowered or raised to adjust the weir crest elevation as needed to accommodate the demand for warm water and forebay elevation in the reservoir.

The flow over the weirs is a function of the weir length, upstream head on the weir (submergence) and head differential between forebay and wet well. However, there are additional factors with the presence of the FSS and coarse trash rack on the face of the weirs. There are three basic HIW operations that need to be addressed:

1. Interim operation prior the installation of the FSS,
2. Operations with FSS in place, and
3. Bypass operations with ballasted FSS.

4.4.5.3 Interim HIW Operations

With interim operations, two coarse trash racks will be on the face of the SWS tower just upstream of the HIW openings. This is to accommodate the future ballasted FSS operations. Since the area of the trash racks will effectively be no larger than the HIW openings, there will need to be a head restriction of approximately 0.5 ft between forebay and SWS. This is to assure a maximum average velocity of about 4 ft/s approaching the trash racks.

With the HIW head restriction, flow through the LIG will be limited to 2000 - 2800 cfs with 3 - 4 fully open LIG. During autumnal times of year—when cold water demand is high—the SWS will have to alternate operations between a combination of HIW and LIG, followed by strictly LIG operations. This is similar in respect to the current operations that alternate between spillway and turbine units to attain an average temperature target in the Big Cliff Reservoir.

A diagram illustrating the interim operation is shown in Figure 4-12. Referring to the figure: ‘Hw’ represents submergence of the weir below forebay and ‘DH’ represents the head differential from forebay to wet well. Position of HIW prior to the FSS installation will be determined based on flow requirements, temperature distribution in the reservoir, and forebay elevation.

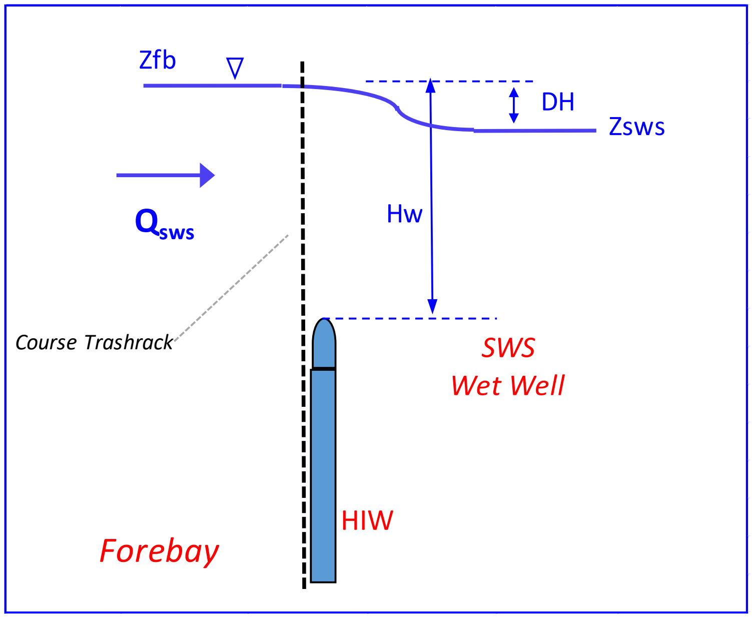


Figure 4-12. Submerged High Intake Weirs under Interim Operations

$$Q = Cw \cdot Cv \cdot B \cdot Hw^{1.5}$$

$$Cv = \left\{1 - \left(\frac{Hw - (DH - HLtr)}{Hw}\right)^{1.5}\right\}^{0.385}$$

$$HLtr = Ktr \cdot \left(\frac{Q}{B \cdot N_{hiw} \cdot Hw \cdot}\right)^2 \cdot \frac{1}{2g}$$

$$V = \frac{Q}{B \cdot \{Hw - DH/2\}}$$

Where:

Q = Discharge through SWS upper weir (cfs)
Cw = Weir coefficient = 3.33 for standard sharp crested weir
B = Weir opening width (feet)
Cv = Villemonte coefficient for weir submergence
Hw = Weir submergence or upstream head on weir (feet),
(= difference between forebay and weir crest)
DH = Head drop from forebay (or FSS) into SWS (feet)
V = Average velocity over weir (fps)
HLtr = Headloss through trash racks
Ktr = Trash rack loss coefficient = 0.5 (assuming 10% plugging)
N_{hiw} = Number of high intake weir open (1 or 2)

Table 4-9 shows various interim operations of the standard sharp crested 20-ft wide weirs. The standard sharped crest weir coefficient is 3.33. The table shows the required weir submerge for different flow rates while maintaining the head restrictions.

Table 4-9. Interim SWS High Intake Weir Operations

Q_{FSS} Total Flow over HIWs (cfs)	N_{HIW} No. of HIW	DH Head Diff. FB to SWS (ft)	H_w Weir Subm. (feet)	Flow Per Weir (cfs)	HIW Velocity (ft/s)	Average Depth below FB (feet)
6,200	2	0.5	38.3	3,100	4.0	19.2
5,600	2	0.5	34.9	2,800	4.0	17.5
5,000	2	0.5	31.5	2,500	4.0	15.7
4,500	2	0.5	28.6	2,250	3.9	14.3
4,000	2	0.5	25.7	2,000	3.9	12.8
3,500	2	0.5	22.7	1,750	3.9	11.4
3,000	2	0.5	19.7	1,500	3.8	9.9
2,750	2	0.5	18.2	1,375	3.8	9.1
2,500	2	0.5	16.7	1,250	3.7	8.4
2,500	1	0.5	31.5	2,500	4.0	15.7
2,000	2	0.5	13.6	1,000	3.7	6.8
2,000	1	0.5	25.7	2,000	3.9	12.8
1,500	1	0.5	19.7	1,500	3.8	9.9
1,000	1	0.5	13.6	1,000	3.7	6.8

4.4.5.4 HIW with Normal FSS Operations

With normal FSS operations, all surface flow entering the SWS will already be screened. Surface flow will pass through the FSS for fish collection and the screened excess flow will be sent to the SWS by means of a 35-ft deep plenum connecting FSS to SWS.

The FSS operations will require more head drop from the forebay than interim operations. The FSS requires more headloss with the intake weirs, screen losses and conveyance losses to the SWS. In most cases, the FSS headlosses will be sufficient to allow the LIG to operate without restriction when cold water is needed. The HIW of the SWS will normally be set to roughly match the plenum invert to reduce further headloss. In cases where cold water flow needs to be greater than roughly 3000 cfs, the high intake weirs will need to be raised above the plenum invert to assure enough head differential between forebay and SWS exists to draw the required LIG flow.

A diagram illustrating the normal HIW operation with FSS deployed is shown in Figure 4-13. Referring to this figure: Hw represents submergence of the weir below the water level in the FSS plenum. DH represents the head differential from forebay to wet well.

FSS Δ is the head difference between forebay and FSS plenum, and DHhwr is the head drop from the plenum to the SWS wet well. DH is the sum of FSS Δ and DHhwr.

Zfb = Forebay Elevation
Zsws = Water level in SWS
DH = Total head differential between FB and SWS = FSS Δ + DHwr
FSS Δ = Difference between Forebay and FSS plenum (interpolated from 60% FSS DDR)
FSS Δ = Incorporates FSS headloss through intake weirs, screens and plenum
DHlig = Estimated required head differentials between FB and SWS to meet flow targets through LIGs (computed separately in LIG computation sheet)
DHwr = Head drop over SWS HIW weir after FSS plenum
DHwr is assumed to be 0.5 ft
However IF: DHlig > FSS Δ+ 0.5', THEN: DHwr = DHlig - FFF Δ

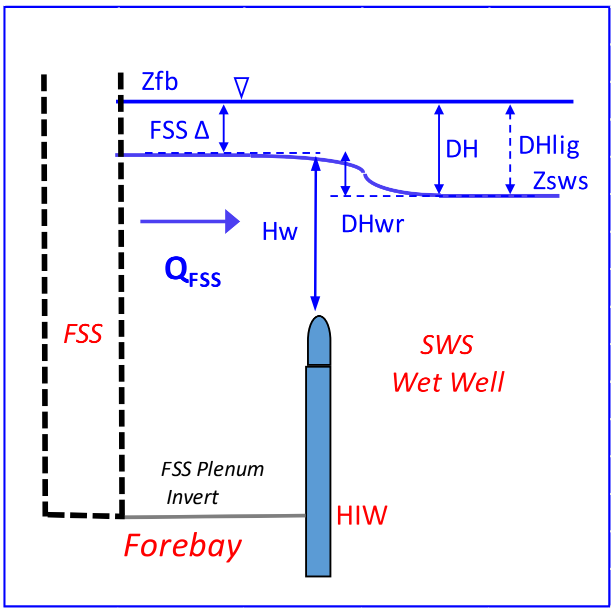


Figure 4-13. High Intake Weirs under Normal FSS Operations

Table 4-10 shows performance of the High Intake Weirs under normal FSS operations.

For a given unit FSS barrel flow, the FSS Δ was interpolated from the 60% FSS DDR headlosses estimated from forebay to plenum for three different flow conditions. Based on conversations with the FSS A/E, the FSS headloss values were raised by 20%.

The required head differentials for the LIGs is a summation of estimated flows through the trash rack hood, LIG and internal SWS losses. This is discussed further in the section concerning LIGs.

The same equations as used with the interim operations are applied with the following adjustments:

$$Cv = \left\{1 - \left(\frac{Hw - DH_{wr}}{Hw}\right)^{1.5}\right\}^{0.385}$$

$$\mathrm{DH} = \mathrm{Zfb} - \mathrm{Zsw} = \mathrm{FSS}\,\Delta + DHwr$$

$$DH \geq DHlig$$

$$DHwr = Zfb - Zsws - FSS\,\Delta$$

Table 4-10. High Intake Weirs under Normal FSS Operations

Q_{FSS} Total Flow over HIWs (cfs)	N_{FSS} No. of FSS Barrels	FSS Δ FB - FSS Plenum (feet)	DH_{LIG} Req. LIG DH from FB (ft)	DH_{wr} Head Diff. FSS Plenum to SWS (ft)	Maximum Allowable H_{wr} (feet)	H_{wr} Weir Subm. (feet)	Flow Per Weir (cfs)	HIW Velocity (ft/s)
6,200	2	3.2	0.0	0.5	31.8	34.6	3,100	4.5
5,600	2	3.0	0.0	0.5	32.0	31.6	2,800	4.4
5,000	2	2.7	0.2	0.5	32.3	28.6	2,500	4.4
4,500	2	2.4	0.8	0.5	32.6	26.0	2,250	4.3
4,000	2	2.1	1.4	0.5	32.9	23.4	2,000	4.3
3,500	2	1.9	2.1	0.5	33.1	20.8	1,750	4.2
3,000	2	1.8	2.5	0.7	33.2	15.9	1,500	4.7
2,750	2	1.7	2.6	0.9	33.3	13.7	1,375	5.0
2,500	2	1.6	2.6	1.0	33.4	12.3	1,250	5.1
2,500	1	2.7	2.6	0.5	32.3	15.4	2,500	8.1
2,000	2	1.6	3.6	2.0	33.4	7.9	1,000	6.3
2,000	1	2.1	3.6	1.5	32.9	8.8	2,000	11.4
1,500	1	1.8	4.5	2.7	33.2	5.7	1,500	13.2
1,000	1	1.5	5.4	3.9	33.5	3.8	1,000	13.0

4.4.5.5 HIW with Ballasted FSS Operations (i.e. Bypass Operations)

The FSS will be ballasted during routine maintenance periods, when the FSS requires immediate repairs, or when the forebay drops below 1,445 ft. When fish collection operations are suspended, the SWS temperature control operations will need to be maintained. As noted previously, there are no separate bypass high intake gates with the proposed At-Dam SWS, so the surface water must be drawn through the HIW beneath the ballasted FSS.

The current FSS design has a ballast depth of 8 ft (possibly subject to change). A coarse trash rack will track with the bottom of the ballasted FSS plenum to assure large debris is not pulled into the SWS. The trash racks will use the same guides on the face of the SWS provided for the interim operations.

The bypass operation will be similar to the interim operation, in which the head differential between forebay and SWS is restricted to about 0.75 ft. This is to assure a maximum average velocity of about 4 ft/s approaching the trash racks. Like the interim operations, the SWS during bypass mode will have to alternate operations between a combination of HIW and LIG, followed by strictly LIG operations during the autumn months. However, bypass operations during autumn months are not anticipated except during FSS emergencies.

A diagram illustrating the bypass HIW operation with a ballasted FSS is shown in Figure 4-14. Referring to the figure: 'Ywr' represents depth of the high intake weir below the ballasted FSS plenum and 'DH' represents the head differential from forebay to SWS wet well.

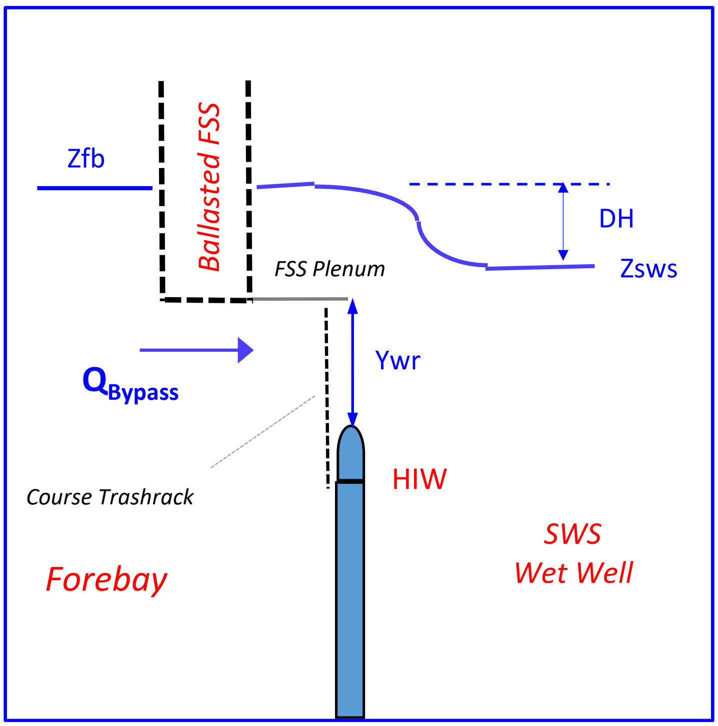


Figure 4-14. High Intake Weirs Bypass Operations under Ballasted FSS Operations

With the ballasted FSS plenum forming an upper ceiling to the approaching flow, the high intake weirs under bypass mode will be operating like inverted slide gates. Thus, the following orifice equations are applied:

$$Q = Cd \cdot B \cdot Ywr \cdot Nhiw \cdot \sqrt{2g \cdot (DH - HLfss)}$$

$$HLfss = (Ki + Ktr) \cdot \left(\frac{Q}{B \cdot Ywr \cdot Nhiw}\right)^2 \cdot \frac{1}{2g}$$

In which:

Q = Discharge through bypass HIW

DH = Head difference between Forebay and SWS

Cd = Discharge coefficient = 0.7

Ywr = Depth of HIW crest below plenum of ballasted FSS

HLfss = Headloss around edge of FSS and through trash rack

Nhiw = Number of open HIW

Ki = Loss coefficient for flow passing around FSS = 0.25

Ktr = Loss coefficient through trash rack = 0.5 (assume 10% plugged)

The normal 0.5 blunt edged intake loss coefficient is reduced to 0.25 for flow passing under the FSS barge since flow is not constricted from the bottom. The loss coefficient for the trash rack assumes a routine 10% plugging of the coarse trash rack bars.

Table 4-11 shows the bypass operations of the HIWs under a ballasted FSS.

Based on the results, the height of the trash racks will need to be at least 50 ft.

Table 4-11. High Intake Weir Bypass Operations under Ballasted FSS

Total Flow Over Weir(s) (cfs)	Nhiw Number of Weirs in Operation	Water Elev. Difference Forebay to Wet Well (ft)	Ywr Depth of Weir below Plenum *Cd = 0.7*	Flow Per Weir (cfs)	HIW Velocity (ft/s)	Average Depth below FB (feet)
6,200	2	0.75	40.1	3,100	3.9	28.0
5,600	2	0.75	36.2	2,800	3.9	26.1
5,420	2	0.75	35.0	2,710	3.9	25.5
5,000	2	0.75	32.3	2,500	3.9	24.2
4,500	2	0.75	29.1	2,250	3.9	22.5
4,000	2	0.75	25.8	2,000	3.9	20.9
3,500	2	0.75	22.6	1,750	3.9	19.3
3,000	2	0.75	19.4	1,500	3.9	17.7
2,500	2	0.75	16.2	1,250	4.0	16.1
2,000	1	0.75	25.8	2,000	3.9	20.9
2,000	2	0.75	12.9	1,000	4.0	14.5
1,500	1	0.75	19.4	1,500	3.9	17.7
1,500	2	0.75	9.7	750	4.0	12.8
1,000	1	0.75	12.9	1,000	4.0	14.5

4.4.5.6 Uplift Forces on Weir and Weir Leaves

With flow passing over the crest of the weirs, uplift will be the primary hydraulic force for design consideration. The combination of weir leaf height (52.5 ft) and potential head differential (DH) between forebay and SWS creates a significant potential for uplift. The upper most leaf will also be the outermost, or upstream, leaf. Hence the full forebay pressure head will act upon the bottom of the deeply submerged weir leaf. Large head differentials reduce the countering pressures on both the crest of the weir leaf and the pressure head acting on the top of the bottom support of the weir leaf.

The uplift loads on the HIW will be higher under normal FSS operations than under interim or bypass operations due to the higher head differential (DH) from forebay to SWS. Maximum normal cases consider head differential at 12 ft and extreme cases assume 19.2 ft—based on a possible rapid turbine unit start-up scenario (see section 4.5.5). A free body diagram of the pertinent uplift parameters is shown in Figure 4-15.

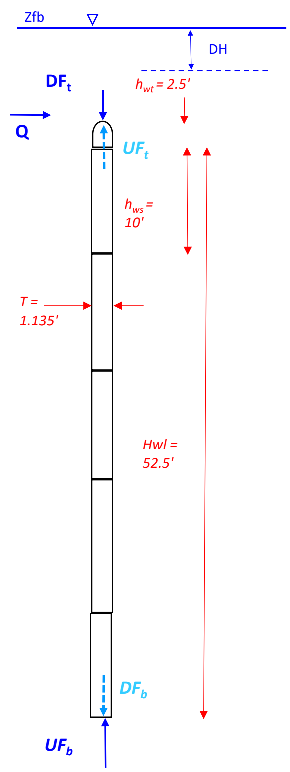


Figure 4-15. Uplift Free Body Diagram and Head Differentials

Table 4-12 lists the estimated uplift loads per weir for normal FSS operations for HIW inflows of 6200, 1000 and 0 cfs. It also includes maximum normal (DH = 12 ft, highlighted in blue) and extreme (DH = 19.2 ft, highlighted in yellow) for the same HIW inflows. The maximum uplift load was about 36 kips per HIW under the extreme condition with no HIW inflow. As noted, material weight and friction are not included in these loads.

Table 4-12. Estimated Uplift Loads per High Intake Weir

Q	Head Differential		Downward Heads		Uplift Heads			Total Forces		UP	
	FSS Δ	DH	DF_t	DF_b	ΣUF_s	UF_b	UF_t	Down-	Uplift	Net Uplift	
	Head Δ	Σ Head	downpull	downpull	Total	uplift	uplift	pull		force	
FSS	from	Δ from	on top	on top of	displaced	on leaf	under top	DFt + DFb	Ufb + Uft	per wier leaf	
Flow	FB to FSS	FB to	of gate	gate bot.	volume	bottom	of gate			(Excluding weight and	
Rate	plenum	SWS	Pr. head	Pr. head	supports	Pr. head	Pr. head	force	force	friction)	
(cfs)	feet	feet	feet	feet	ft^3	feet	feet	lbs	lbs	lbs	Kips
6200	4.6	5.4	31.4	81.7	57.0	91.7	29.3	170,441	190,181	19,740	19.7
6200	4.6	12.0	28.0	75.1	41.4	91.7	22.7	155,369	179,316	23,947	23.9
6200	4.6	19.2	24.4	67.9	57.0	91.7	15.5	139,050	169,584	30,534	30.5
4000	3.9	4.7	20.5	71.2	57.0	79.8	18.8	137,855	155,886	18,031	18.0
1000	1.5	3.5	6.7	57.7	57.0	62.8	5.3	96,210	109,317	13,107	13.1
1000	1.5	12.0	6.4	49.2	57.0	62.8	0.0	83,098	101,399	18,301	18.3
1000	1.5	19.2	6.4	42.0	57.0	62.8	0.0	72,390	101,399	29,009	29.0
0	0	3.0	0.0	49.4	57.0	52.5	0.0	73,539	85,388	11,849	11.8
0	0	12.0	0.0	40.4	57.0	52.5	0.0	60,154	81,634	21,480	21.5
0	0	19.2	0.0	33.2	57.0	52.5	0.0	49,446	85,388	35,942	35.9

4.4.5.7 Bypass and Interim High Intake Weir Trash Racks

Each Bypass HIW will have two 20-ft wide coarse trash racks located on the east face of the SWS and just upstream of each HIW. As indicated from the calculations for the bypass HIW operations, the bypass trash racks will need to be at least 50 ft tall.

The trash racks will have 0.5 inch-bars at 6-inch-centerline spacing. Two-inch wide crossbeams at 30-inch spacing are assumed based on the Bonneville powerhouse trash racks. (Bonneville PH trash racks see velocities over 5 ft/s.). The clean trash rack porosity is 86%.

The maximum design total bypass HIW flow is 6200 cfs with an approximate average velocity of 4.0 ft/s.

Headlosses (HL) across the BHIW trash racks are estimated at various levels of plugging using headloss coefficients for flat bars (Miller 1990). Kb is the estimated headloss coefficient for flat bars, the curve for Kb was extended through correlation with the curve for round bars (Kr), which is presented in a greater range of porosities (Kb ≈ 1.3 x Kr). The estimated headloss is Kb x velocity head (0.25 ft), listed in Table 4-13 for various levels of plugging.

Table 4-13. Estimated Head Differentials across Trash Racks for HIWs
<u>Head differences (HL in feet) across Bypass HIW Trash Rack bars:</u>

Bar Condition		Resultant	Bar Loss Coeff		HL
	% plugged	Porosity	Kr	Kb	(ft)
Clean	0%	86%	0.16	0.2	0.1
plugged	10%	77%	0.37	0.5	0.1
plugged	25%	64%	0.85	1.1	0.3
plugged	50%	43%	3.3	4.3	1.1
plugged	75%	21%	17.5	22.7	5.6

Like the bypass trash racks, the interim SWS HIW trash racks will cover the same spaces and use the same guides; however, the trash rack may or may not track with the forebay and HIW, it is preliminarily expected to be full height (Elevation 1400 to 1570).

4.4.6 LIGs for Cold Water

4.4.6.1 Description of Design

The four LIGs to the SWS are arranged in stacked pairs on the east face of the SWS at invert elevations 1,327 ft and 1,305 ft. The purpose of these inlets is to draw cold water from the lower part of the reservoir, which remains cold year round, as well as to augment when needed for normal operations. The inlets will consist of a horizontally aligned series of slide gates. There will be four gates with dimensions of 10 ft tall by 15 ft wide.

These gates will be split evenly between the two monoliths of the SWS and each pair will share a trash rack hood. Two trash rack hoods are separated to avoid crossing monolith boundaries. Each trash rack hood will be 22 ft wide by 44 ft tall (Area = 968 sq ft), sized to maintain a maximum average velocity of 4.4 ft/s under maximum normal case.

The criteria for the minimum opening of 10% for RO/slide gates will apply to the use of the LIGs. Some of the dimension and discharge capacities (assuming full gate openings) for the four gates is shown in Table 4-14 assuming a) 0.5-ft and b) 4-ft differentials between forebay and wet well. The LIG are designed as submerged orifices. The flows shown in Table 4-14 were computed using a preliminary discharge coefficient of 0.82 for a submerged rectangular orifice. The discharge coefficient is based on a standard beveled short tube provided in Brater and King (1976). This discharge coefficient will be refined, as the intent is to round the edges of the intake to assure a more hydraulically efficient opening. For partially opened gates, a discharge coefficient of 0.70 is assumed. Estimated trash rack headlosses are deducted from the head differential (ΔH) in the computation of LIG discharge. The equations for the LIGs are shown below:

$$Q = Cd \cdot B \cdot Go\sqrt{2g \cdot (\Delta H - HLtr)}$$

$$\Delta H = Forebay\ Elev\ - SWS\ elev$$

$$HLtr = Ktr \cdot \left\{\frac{\mathrm{Q_i + Q_{i+1}}}{\mathrm{Atr}}\right\}^2 \cdot \frac{1}{2g}$$

$$V = \frac{Q}{B \cdot Go}$$

Where:

Q = Low intake discharge (cfs)
Cd = Discharge coefficient
= 0.82 full open
= 0.70 throttled
B = Opening width (15 ft)
Go = Gate opening (feet)
Gmax = 100% gate opening or max Go = 10 ft
ΔH = Head differential across intake (feet)
V = Average velocity at gate opening
HLtr = Headloss through trash rack
Atr = area of trash rack = 968 ft^2
Ktr = trash rack loss coefficient = 1.47
assuming10% plugging and 66% porosity baffle
$Q_i + Q_{i+1}$ are individual LIG flows in shared trash rack hood

Table 4-14. Low intake Gate Dimensions and Discharge Capacities under Different Operations and Head Differentials

a. FSS Operations under 4-feet head from FB to SWS							
Lower Inlet ID Number	Gate Width (B) (ft)	Full Gate Height (Gmax) (ft)	Centerline Elev. @ 100% Open (ft)	FB - SWS (ΔH) (ft)	HLtr Trashrack headloss (ft)	Max Flow Per Inlet (Q) (cfs)	Gate Velocity (V) (fps)
1A	15.0	10.0	1,332.0	4.0	0.10	1,948	13.0
2A	15.0	10.0	1,332.0	4.0	0.10	1,948	13.0
1B	15.0	10.0	1,310.0	4.0	0.10	1,948	13.0
2B	15.0	10.0	1,310.0	4.0	0.10	1,948	13.0
Total	60.0	40.0	1,321.0	4.0		7,792	13.0

b. Bypass Operations under 1.5-feet head differential							
Lower Inlet ID Number	**Gate Width (B) (ft)**	**Full Gate Height (Gmax) (ft)**	**Centerline Elev. @ 100% Open (ft)**	**FB - SWS (ΔH) (ft)**	**HLtr Trashrack (ft)**	**Max Flow Per Inlet (Q) (cfs)**	**Gate Velocity (V) (fps)**
1A	15.0	10.0	1,332.0	1.5	0.04	1,193	8.0
2A	15.0	10.0	1,332.0	1.5	0.04	1,193	8.0
1B	15.0	10.0	1,310.0	1.5	0.04	1,193	8.0
2B	15.0	10.0	1,310.0	1.5	0.04	1,193	8.0
Total	60.0	40.0	1,321.0	1.5		4,772	8.0

c. Bypass Operations under minimum 0.5-feet head diff.							
Lower Inlet ID Number	**Gate Width (B) (ft)**	**Full Gate Height (Gmax) (ft)**	**Centerline Elev. @ 100% Open (ft)**	**FB - SWS (ΔH) (ft)**	**HLtr Trashrack (ft)**	**Max Flow Per Inlet (Q) (cfs)**	**Gate Velocity (V) (fps)**
#1	15.0	10.0	1,332.0	0.5	0.01	689	4.6
#2	15.0	10.0	1,332.0	0.5	0.01	689	4.6
#3	15.0	10.0	1,310.0	0.5	0.01	689	4.6
#4	15.0	10.0	1,310.0	0.5	0.01	689	4.6
Total	60.0	40.0	1,321.0	0.5		2,755	4.6

4.4.6.2 Downpull and Uplift Forces on Low intake Gates

Downpull force occurs when pressure acting on the top of the gate exceeds the pressure acting on the bottom of the gate. The pressures acting on the bottom of the gate are lowered as a function of velocity and flow separation passing under the gate. Downpull is usually significant at high head RO gates in which the upstream head may be hundreds of feet, whereas the pressure is slightly subatmospheric on the downstream side of the gate. The velocity, and hence the downpull, is significant under a typical RO gate. In the case of the LIG, the normal velocity is limited by head differentials between 2-6 ft from upstream to downstream. Downpull forces are moderate under normal operations, however become significantly large under maximum normal (12 ft) and extreme (19.2 ft) head differential cases. Uplift loads are also significant, albeit lower than downpull.

Table 4-15 shows the computed downpull and uplift for each size of LIG as a function of head differential. The buoyancy of the gates are accounted for in the calculations; however, the weight of gate and friction forces have not been included.

Table 4-15. Low intake Gate: Net Downpull and Uplift Forces

Max DOWNPULL		(Max Net hydraulic downpull forces on gates as function of head difference)									
		DH (ft) =	19.2	DH (ft) =	12	DH (ft) =	6	DH (ft) =	2	DH (ft) =	0
Gate Height (ft)	Wsl (ft)	Net DPf Max Net Downpull (feet)	Net DP Hydraulic Downpull Kips	Net DPf Max Net Downpull (feet)	Net DP Hydraulic Downpull Kips	Net DPf Max Net Downpull (feet)	Net DP Hydraulic Downpull Kips	Net DPf Max Net Downpull (feet)	Net DP Hydraulic Downpull Kips	Net DPf Max Net Downpull (feet)	Net DP Hydraulic Downpull Kips
10	16.67	24,608	24.6	14,605	14.6	6,268	6.3	711	0.7	-	0.0

MAX UPLIFT		(Max Net hydraulic uplift forces on gates as function of head difference)									
		DH (ft) =	19.2	DH (ft) =	12	DH (ft) =	6	DH (ft) =	2	DH (ft) =	0
DH (ft): Gate Height (ft)	Wsl (ft)	Net DPf Max Net Downpull (feet)	Net DP Hydraulic Downpull Kips	Net DPf Max Net Downpull (feet)	Net DP Hydraulic Downpull Kips	Net DPf Max Net Downpull (feet)	Net DP Hydraulic Downpull Kips	Net DPf Max Net Downpull (feet)	Net DP Hydraulic Downpull Kips	Net DPf Max Net Downpull (feet)	Net DP Hydraulic Downpull Kips
10	16.67	11,564	11.6	8,417	8.4	5,795	5.8	4,046	4.0	3,172	3.2

4.4.6.3 Vibration Analyses on Cables for LIGs

EM-1110-1602 states that the standard 45-degree sloping RO style gate bottom should be free of vibration induced by vortices shed from the gate lip. There remains the concern regarding vibration of the cables in these cable suspended gates. With the maximum uplift load estimated at 11.6 kips, USACE Structural Design engineers estimate that the weight of the gate needs to exceed up 26 kips to overcome uplift and friction loads. Assuming a 1.4 FS, each gate will need to weigh approximately 36 kips. For purposes of the vibration analyses, 11.6 kips is deducted from this weight.

From references Hydroelectric Design Center (HDC) and EM-1110-2-1602, the following equations were solved to estimate the forcing frequencies (Ff).
EM 1110-2-1602

Equation 4-3 (for flat bottom gates, conservative)

$$Ff(1) = \frac{V \cdot St}{Lp}$$

In which:
V = approach velocity = 27.6 ft/s,
St = Strouhal Number = 1/7
Lp = gate bottom width = 17.8 in

Equation 4-4

$$Ff(2) = \frac{\sqrt{2g \cdot Es}}{7 \cdot 2Y}$$

In which:
Es = Steel Modulus = 30 x 10^6 psi,
Y = max projection of gate into flow = 9.0 ft (10% open)
g = gravity = 32.2 ft/s^2

The larger of the above two values was applied as the forcing frequency (Ff = 2.65 Hz).

The natural frequency (Fn) of the cables are given in EM 1110-2-1602, Equation 4-6:

$$Fn = \frac{1}{2\pi}\sqrt{\frac{g \cdot Es}{12 \cdot Lc \cdot \sigma}}$$

In which:

Lc = Cable length = 265 ft,

σ = unit stress of cable

$$\sigma = \frac{W}{N \cdot Ac}$$

W = weight of LIG = 24 kips

N = number of cables per LIG = 8

Ac = cross-sectional area of cable

The ratio of natural frequency to forcing frequency was compared two ways to avoid resonance from occurring in cables:

Fn/Ff ≥ 5

TR < 1.1

TR = undamped magnification factor defined in equation below.

$$TR = \frac{1}{1-\left\{{}^{Ff}/_{Fn}\right\}^2}$$

The expected cable diameter is one inch, however the cable diameter was varied between 0.75 – 1.5 in. In all cases, the Fn/FS ratio was greater than 10^5 and TR was equal to 1.00 up to 7 digits. Therefore vibration of the gates will not be an issue. The calculations are shown in Appendix C.

4.4.6.4 Trash Racks for LIG with Trash Rack Hoods

Coarse trash racks will be provided for the LIG openings to protect the SWS and penstock from large debris. With head drops typically around 3 ft and potentially up to 19.2 ft through the LIG openings, trash racks cannot be feasibly placed in the gate openings, as the LIG velocities are too high and the trash racks would incur serious drag and vibration issues. As there are two LIGs on each monolith, there will need to be two equivalently sized LIG trash racks for each side.

The trash racks will have 0.5 inch-bars at 6-inch-centerline spacing. Two-inch wide crossbeams at 30-inch spacing are assumed based on the Bonneville powerhouse trash racks. (Bonneville PH trash racks see velocities over 5 ft/s). The clean trash rack porosity is 86%.

The maximum LIG inflow is assumed to be 6200 cfs under maximum normal operations. Under an extreme condition (rapid turbine startup), the LIG discharge may be as high as 7200 cfs.

Assuming a one-gate redundancy in a normal operation, three of the four LIGs are operating and one of the trash racks will be pulling 2/3 of the total flow or 4133 cfs per trash rack hood. In the extreme case, a redundant hydraulic power unit (HPU) is assumed to assure all four LIG's are operating so that the flow is balanced through all four LIGs and the total flow is 3600 cfs per trash rack hood. However, the trash rack will be designed to withstand a potential 4800 cfs (2/3 x 7200) in case one LIG is down for other reasons.

Both trash racks are sized to 44 ft tall x 22 ft wide (Area = 968 sq ft) so that the maximum average velocity is less than 4.4 ft/s. Each trash rack will have a conveyance hood that connects the LIGs to their respective trash rack. The rooves on both hoods will be sloped toward the east and away from the dam. The rock surface in this area has a dual slope, where it slopes both east away from the dam and to the south toward an underwater canyon.

The preliminary extension distance of the hoods from the SWS outer walls is 12 ft and will require baffles between the trash racks and LIG openings. The design will be refined through CFD modelling. Baffling is required on both trash rack hoods to assure more uniform velocities through the trash racks and to help reduce the extension of the hoods out from the SWS face. The baffling will consist of a series of horizontally aligned bars (like bubbler beams in a fish ladder diffuser) with an assumed 66% porosity. The centerline elevations of both trash rack hoods will be approximately 1,320 ft, which will improve the cold water injection and reduce the potential attraction of the target species into the LIGs.

Headlosses across the LIG trash racks are estimated at various levels of plugging using headloss coefficients for flat bars in Miller (1990). Kb is the estimated headloss coefficient for flat bars, the curve for Kb was extended through correlation with the curve for round bars (Kr), which is presented in a greater range of porosities (Kb ≈ 1.3 x Kr). The estimated headloss is Kb x velocity head (0.27 ft). The 66% baffle porosity is also included in the analyses. The maximum head differentials occur at 75% plugging, which yields approximately 7 ft under maximum normal condition and 9 ft under the extreme case. The trash rack and baffle coefficients and estimated headlosses for the maximum normal case are shown in Table 4-16.

Table 4-16. Estimated Trash Rack plus Baffle Headlosses (Maximum Normal Case)

Trash Rack Headlosses as Function of Plugging Maximum Normal Case: 4133 cfs per Trash Rack Hood					
Bars Loss Coeff.		Porosity	Kr	Kb	HL (ft)
Clean	0%	86%	0.16	0.2	0.1
plugged	10%	77%	0.37	0.5	0.1
plugged	25%	64%	0.85	1.1	0.3
plugged	50%	43%	3.3	4.3	1.2
plugged	75%	21%	17.5	22.7	6.4
Baffle Porosity =		66%	0.8	1.0	0.3

Total Max DH at 75% plugging=			6.7
For normal LIG OP, assume 10%:	Σ Kb =	1.5	0.4

4.5 OPERATIONS

4.5.1 Powerhouse

The SWS will, in general, operate for powerhouse flows. The water will mix in the wet well, from the upper (HIW) or lower (LIG) gates, or a combination of gates, depending on what is needed for temperature control, and will be released from the SWS into the powerhouse penstocks.

The existing closure bulkheads will be maintained at the dam for conduit inspection and maintenance purposes.

4.5.2 Regulating Outlet (RO) - Penstock Bifurcation Operation

When one or both units is not operating, surface flow can continue to be drawn through the FSS and SWS through the RO-penstock bifurcation.

4.5.3 Operation of SWS

An operations section for the SWS is being developed and will be included in the 90% DDR.

The operating head differentials (DH) between forebay and SWS wet well will vary with the type of operation:

- Normal conditions:
    - DH = 3-6 ft for normal FSS operations
    - DH = 0.5-0.75 ft under interim or future bypass operations, when operating the HIWs with LIG less than 2700 cfs
    - DH = 0 ft when there is no unit or RO bypass operations (during the periods of day when power demand is low)
- Abnormal conditions:
    - DH ≈ -0.5 ft during powerhouse load rejections (SWS higher than FB)
    - DH = 19.2 ft under a rapid unit startup

4.5.4 Speed of High Intake Weir Operations

Per BPA criteria, the turbine operations are ramped up over a 15-minute period. (The ramp down rate is also over a 15-minute span.) The maximum possible turbine flow is

6200 cfs at a pool of 1,480 ft. During interim or bypass operations, the high intake weirs may need to open to as much as 40 ft. Therefore, the HIW will need to be able to open at a rate of 2.67 ft per minute or 1 ft every 22.5 seconds.

4.5.5 Speed of Cold Water LIG Operations

The speed of the LIG will be set to 10 ft per minute. While the normal turbine operations are ramped up or down over a 15-minute span, there remains the potential for rapid unit startups. To avoid aversely drawing down the SWS tower wet well, the LIG speed needs to be 10 ft per minute. Because of the comparative ease in responsiveness, the LIGs are effectively the pressure regulators of the SWS.

Under rapid unit startups, the turbines can conceivably be ramped up in 9 seconds per unit (possible misoperation or intended). BPA may intend to maintain the ability to perform rapid turbine start-ups in the future. If this is to be done, HDC recommends that at least a 60-second delay be imposed (via interlock) to prevent immediate consecutive rapid unit start-up operations. The 60-second delay would separate the end of the first rapid unit start-up and the start of the 2nd rapid start-up.

Under the proposed modification to the rapid startup protocol—in which the 60-second delay is installed—the maximum total LIG discharge will be 7200 cfs with a maximum SWS head differential of 19.2 ft. Without the delay, the total LIG discharge would go up to 9000 cfs—which would be too much for the LIG trash racks and raise the maximum head differential to 36 ft.

The LIGs are intended to respond via programmable logic control (PLC) when the differential between forebay and SWS starts increasing above 5.5 ft. If the gates do not open sufficiently fast, the level in the wet well could be drawn down below the maximum head differential of 19.2 ft. A simplified transient analysis was conducted, assuming the maximum possible turbine flow at 6200 cfs (at pool of 1,480 ft). The simplification was that the transport time between turbine units and SWS was neglected and the effects of the transient pressure waves were not considered. The inside SWS dimensions were assumed to be 27 x 86 sq ft. The PLC will engage additional gates as the differential from the forebay and SWS increases from the normal 5.6 to 6 ft (pending information from the FSS design). The maximum design head differential was raised 30% from the actual simulated value of 16 ft to 19.2 ft to provide additional margin in the design parameters and further avoid potentially dire consequences to the tower's integrity if the maximum head differentials are exceeded.

USACE is investigating to determine if this potential rapid turbine startup operation can be prevented forever, possibly via control interlock. The resulting magnitude of 19.2 ft head differential could cause major design ramifications with the SWS tower integrity, gate hoists, and FSS screens.

4.5.6 Powerhouse Load Rejection

Powerhouse load rejections have occurred on several occasions in the past. Under a load rejection, the wicket gates for both units will close rapidly in 9 seconds. When the load rejection occurs, the tower will have some combination of high intake weirs and low level inlet gates open. The SWS will act largely as a moderating standpipe with pressure relieving outlets. However, a transient analyses is needed to determine the maximum and minimum head differentials that may occur between the SWS and forebay, as well as transient pressures in the new RO-penstock bifurcation conduit(s).

The transient analysis uses a water hammer and mass oscillation model to simulate a load rejection at the maximum possible combined turbine discharge (6200 cfs at pool 1,490 ft).

The existing 24-inch and 18-inch air vents do provide some minor surge benefits, and more importantly, prevent column separation in the existing penstock. The transient analyses will determine the maximum (positive and negative) pressure differentials between forebay and SWS, and whether additional measures are required. One possible measure may be the inclusion of pressure relief valves in the RO-penstock bifurcation conduits.

During load rejections, the head differential will at times be reversed and the level in the SWS will be higher than the forebay. Preliminary (simplified) estimates show that the maximum rise in the SWS above forebay should be around 0.5 ft.

The range of oscillating heads in the existing penstock and proposed RO-penstock bifurcation will be much higher than in the SWS.

4.6 HYDRAULIC MODELING

4.6.1 Computational Fluid Dynamics (CFD) for SWS

The CFD model can be used to define flow characteristics in three dimensions inside and outside the SWS under a range of flow conditions. The model can show velocity magnitudes and streamlines, areas of flow turbulence, and possible locations of pressure anomalies. The CFD model can also be used as a tool to help evaluate the proportions needed to attain the right mix of warm and cold water that will be discharged to Big Cliff Reservoir where the mixing is completed.

A CFD model was developed for the previous 2013 SWS configuration design (model report), which was similar to the proposed At-Dam SWS design. The model indicated that the SWS tower would be effective for water temperature mixing.

With the design modifications from the 2013 design, a new CFD model is under development. Significant differences include: Smaller internal SWS volume, no SWS to RO-conduit connection, different locations of the upper and lower intakes, LIG trash rack hoods, and the RO-penstock bifurcations. CFD, and possibly physical modelling of the RO-penstock bifurcation outlet discharges into the stilling basin, is intended to be performed during the plans and specs phase.

The CFD model of the SWS has been used to evaluate the weir box option, and the results were a factor in the discontinuation of the weir box design effort.

4.6.2 CFD General Forebay Model

A CFD model was developed for the forebay to assist in determining design loads for the potential FSS and anchorage to the previous 2013 SWS design.

The far field forebay CFD model will likely need to be revisited with the updated design to assist in the determination of performance standards for the future Detroit FSS.

4.7 DOWNSTREAM FISH PASSAGE FEATURES

The fish passage features include an FSS.

4.7.1 Weir Box (Discontinued Option)

A weir box was envisioned for the first phase, to be built in conjunction with the SWS. However the design of the weir box option has been discontinued due to concerns regarding biological and hydraulic viability of success, along with increasing incremental costs.

The weir box was intended to collect juvenile fish captured within the SWS by means of a continuous 100 cfs withdrawal from the tower. The factors for discontinuing the design effort on the weir box are detailed in the executive summary of the 60% Weir Box DDR.

4.7.2 FSS (Phase 2)

The FSS is being concurrently designed in a separate DDR to assure that it works well with the SWS (see Figure 4-5). The FSS will meet NMFS screen velocity criteria under a design range of inflows between 1000 and 4500 cfs. The system is designed for inflow up to 5600 cfs. The current design features of the FSS system include:

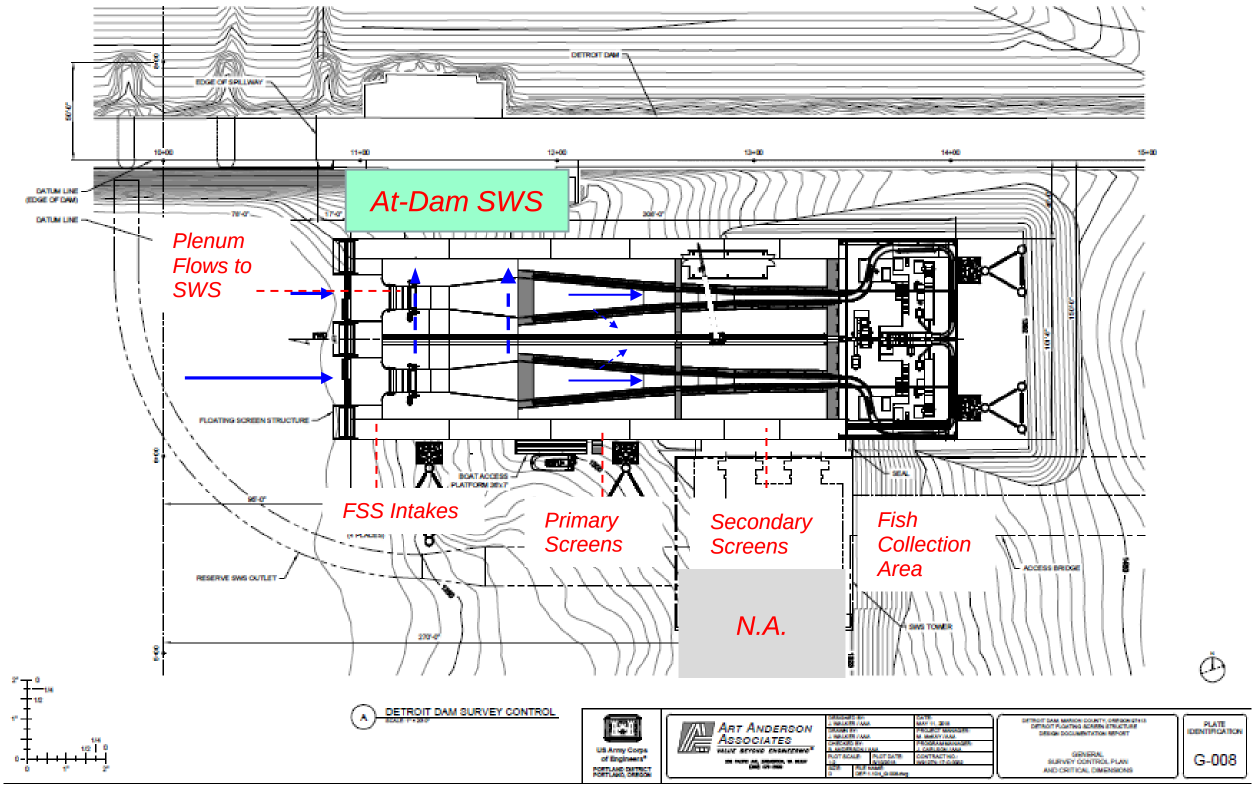


Figure 4-16. Plan Schematic of Floating Screen Structure

1. Coarse trash racks (8-inch spacing) will be placed 20 ft upstream of the FSS intake weirs. Each of the two FSS barrels will have a trash rack 24 ft wide and 35 ft deep. The maximum design velocity through the trash racks is 4 ft/s.

2. FSS intake weirs: Two 12-ft-wide elliptically shaped weirs extend 10-26 ft deep below the forebay. These will be located about 40 ft out from the face of the dam and adjacent to the SWS. The FSS weirs will draw flow from the south and along the face of the dam. There will be 1.5-2 ft of head drop across the FSS intake weirs to both attract fish and to assure capture into the facility.

   The FSS weirs will be adjustable to collect the proportion of flow required for surface water collection. If the surface water needs are less than half of the maximum FSS design discharge, then only one FSS unit will be operated, as the FSS system works better hydraulically when operated closer to maximum design flow. During important fish passage periods (such as spring), the priority in FSS weir operations will be fish passage. During these periods, if optimum temperature operations call for drawing from 10 ft deep, but the proportion of flow calls for 20 ft, then the deeper opening controls. The system is designed for a reasonable capacity for flexibility so that operations can be adjusted if actual conditions are notably different than anticipated at that time.

3. FSS screen system:

    - Upstream FSS section (primary screens): Two converging deep wall screens that narrow the channel.

    - Downstream FSS section (secondary screens): Two narrow wall screens on an adverse grade to dewater the flow leading to a fish collection area.

    - The velocity in the upstream and downstream FSS sections will vary throughout the channels and with various inflows. However, the velocity trend will be longitudinal acceleration in moving downstream in the screened section.

    - Baffled porosity plates will be permanently adjusted during commissioning to regulate the discharge through the screens at various locations along the screened channels. They will remain set and left thereafter.

    - All dewatered flow will be channeled to the SWS through a large plenum and will pass over the SWS HIWs.

    - The upper SWS weirs will receive flow from the FSS at a minimum loss of head and control the backwater from the FSS.

4. Total head differential from forebay to SWS wet well is anticipated to be 2 - 5 ft.

5. The upper design inflow limit is 5600 cfs[1], envisioned not to impact maximum peaking power operations. Screen velocities will exceed NMFS criteria for FSS intake flows above 4500 cfs.

6. Fish collection area: After the screen flow is dewatered to 20 cfs (per barrel), the flow will pass through debris separator bars, followed by adult separation and collection of the target juvenile fish species into pods for downstream transport. Pumps will be used to collect the tertiary dewatering needed for fish collection and discharge to the plenum.

As already noted, the FSS system is under development in a separate, concurrent DDR.

[1] If the turbine units require 6200 cfs, the remainder (600 cfs) will be taken through the low intake gates.

SECTION 5 - WATER QUALITY

5.1 GENERAL

The following section discusses water quality conditions and target temperatures in Detroit Lake and downstream of Detroit Dam; including minimum and maximum flow rates through structures, biological impacts, impacts to drinking water, and the predicted impact of the SWS and FSS at Detroit Dam.

At full pool elevation (1,569 ft), Detroit reservoir covers an area of 3,580 acres and is centered at the confluence of the North Santiam and Breitenbush Rivers. Detroit Lake is considered a warm monomictic lake that lacks ice cover in the winter, has an extensive single stratification period during the warmest parts of the year, and mixes to become nearly isothermal during the coldest part of the year. Dissolved oxygen (DO) gradients during the summer at Detroit Reservoir are essentially orthograde due to the reservoir's oligotrophic nature. The DO generally increases with depth, reaching its maximum concentration at around 30 meters, which continues to the bottom. In Big Cliff Reservoir, the re-regulating reservoir downstream of Detroit, DO concentrations are relatively high as well due to the well-oxygenated water it receives from Detroit Dam (2000).

The NMFS 2008 BiOp considers elevated water temperatures, caused by dam operations, a primary limiting factor for the egg/emergence component of the Upper Willamette River (UWR) spring Chinook life stages in the North Santiam River due to premature hatching and emergence (NMFS 2008; ODFW and NMFS 2011).

5.1.1 Clean Water Act Discussion

The ODEQ has been delegated the implementation of the CWA by the U.S. Environmental Protection Agency (EPA). The ODEQ develops water quality standards and pollution control plans termed Total Maximum Daily Loads (TMDLs). The ODEQ issued a TMDL for the Willamette Basin addressing temperature, mercury, and bacteria impairments (ODEQ 2006). The ODEQ has named the Corps as a nonpoint source Designated Management Agency (DMA) in that it has legal authority over a sector or source contributing to the pollutants. The temperature water quality standard and TMDL are relevant to Detroit temperature control. The Corps submitted a Water Quality Implementation Plan (WQP) and subsequent annual reports outlining actions that comply with the TMDL (USACE 2010).

The thermal pollution limit (termed a load allocation) for Detroit Dam is “no increase in natural thermal potential temperatures when biologically-based numeric criteria are exceeded”. The ODEQ determined that the biologically-based numeric criteria are exceeded in the North Santiam River downstream of Big Cliff and Detroit dams from April through November. The ‘core cold water’ criteria of a 7 day average of the daily maximum of 16.0 °C applies year round and the anadromous spawning criteria of 13.0 °C applies September 1 to June 15. The ODEQ developed preliminary monthly target temperatures for Detroit / Big Cliff reservoirs with the anticipation that they would be revised.

There is uncertainty regarding the temperature standard and implementation of the TMDL. The EPA disapproved the 'natural conditions criterion' provision of ODEQ's temperature standard because of a federal court order (ODEQ 2013). As of October 2013, there is pending litigation on ODEQ's existing temperature TMDLs based on the natural conditions criteria which includes the Willamette Basin TMDL. The ODEQ (2013) states "at present, nonpoint source temperature reduction targets from existing approved TMDLs continue to apply and should be implemented."

The ODEQ has determined that Detroit Reservoir is impaired for "aquatic weeds or algae" based on a harmful algae bloom advisory (ODEQ 2010). They also have determined that the North Santiam River (river mile 0 to 45.25: mouth to the near Big Cliff Dam) is impaired due to low DO levels during the salmonid spawning season, September 1 to June 15. Low DO levels tend to occur during and following peaks in algal populations in Detroit Lake. The ODEQ has not yet issued a TMDL addressing these parameters.

5.2 WATER TEMPERATURE TARGETS

Temperature control operations on the North Santiam River have been performed over recent years using water temperature targets based on those developed and implemented on the South Fork McKenzie River at Cougar Dam. These target temperatures were originally developed only for spring Chinook salmon since no winter steelhead are present in the McKenzie subbasin. A review of these targets, in comparison to literature-based thermal preferences for winter steelhead, indicate that these temperature targets are appropriate for the North Santiam River and meet the needs of both winter steelhead and spring Chinook in the North Santiam Basin (Table 5-1). Beginning in 2017, new temperature targets were developed and adopted by a multi-agency team including ODFW and NMFS. These newer targets are cooler than the older targets during June-September.

Table 5-1. Temperature targets for Detroit outlet temperatures based on species and lifestage

Focal Species & Lifestage			Literature Based Criteria	2007-2016 Targets		2017 Proposed Targets	
				Min	Max	Min	Max
January	Spr Chinook	Juvenile Outmigrant	<59.0	40.1	40.1	38	42
February	Wtr Steelhead	Adult Migration	<60.8	41.0	42.1	38	42
March	Wtr Steelhead	Adult Migration	<60.8	41.0	42.1	42	44
April	Wtr Steelhead	Spawning	51.8	43.2	45.1	42	46
May	Wtr Steelhead	Spawning/Incubation	51.8	46.0	49.1	46	50
	Spr Chinook	Adult Migration	<60.8				

June	Wtr Steelhead	Incubation	58.1	51.1	56.1	48	54
	Spr Chinook	Adult Migration	<60.8				
July	Spr Chinook	Adult Holding	<60.8	54.1	61.2	52	55
August	Spr Chinook	Adult Holding	<60.8	54.1	60.3	52	55
	Spr Chinook	Spawning	40-64				
September	Spr Chinook	Spawning/Incubation	40-57	52.3	56.1	48	54
October	Spr Chinook	Incubation	40-57	<50		46	52
November	Spr Chinook	Incubation	40-57	<50		42	46
December	Spr Chinook	Incubation	40-57	41.0	41.0	41	46

Rounds (2010) developed an empirical model to estimate the river temperature without the impact of Detroit and Big Cliff Dams. The model is based on a flow-weighted mean of upstream river temperature and an assumed warming rate. For the period of record, October 1998 to May 2013, the 30-day average of the minimum and maximum for each day of the year follows a similar annual pattern as the Chinook / steelhead targets (termed “2017 Target” and “2008-2016 Target”) (Figure 5-1). The ‘no dam’ target appears to satisfy the TMDL specifications and provide a smoother pattern while preserving the overall magnitude and generalized seasonal pattern of the Chinook / Steelhead target.

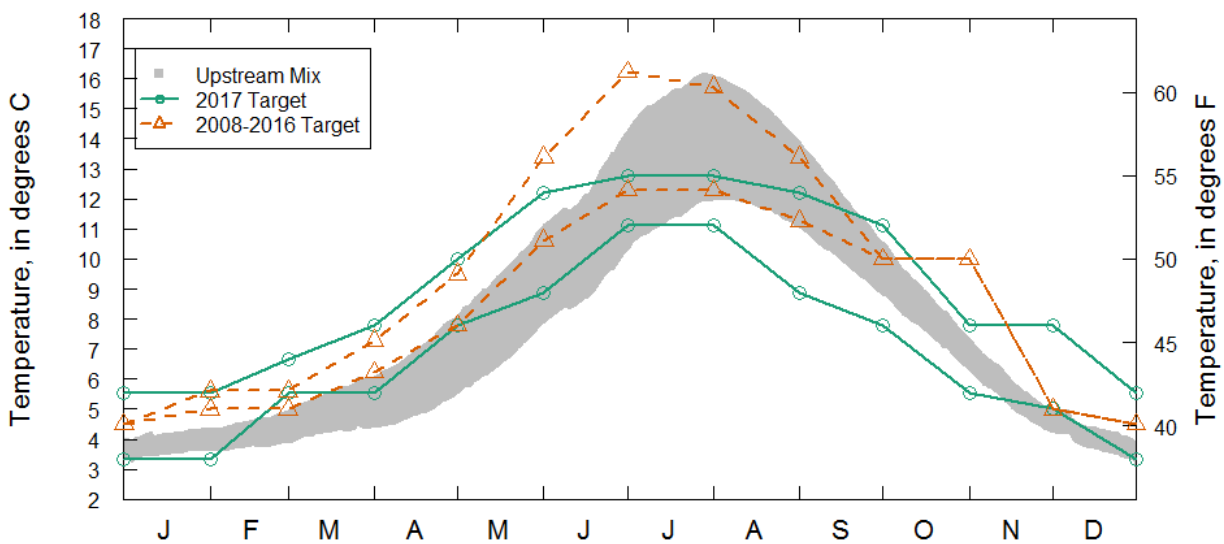


Figure 5-1. Comparison of the estimated no dam temperature range (30-day moving average of daily maximum and minimum temperature by day of the year) to the resource agency temperature targets for Detroit (instantaneous temperature)

Starting with a powerhouse fire in 2007, there has been a shift in the thermal pattern of releases from Detroit reservoir (Figure 5-2). Since 2008, the Corps has implemented special operations for temperature management. The goal is to decrease release temperature during Chinook spawning while enhancing downstream juvenile fish passage. Similar to the SWS, the special operation takes advantage of thermal stratification of the reservoir and different elevations of outlets. Beginning in June, about 40% of the flow is routed over the spillway (elevation 1,541 ft) and 60% through the powerhouse (elevation 1,395.5 ft). This pattern continues until the pool elevation falls below the spillway crest. Flow is then routed only through the powerhouse until about mid-October. From around October 22 until November 15, a blend of powerhouse and upper RO (elevation 1,340 ft) is discharged to meet downstream water temperature goals.

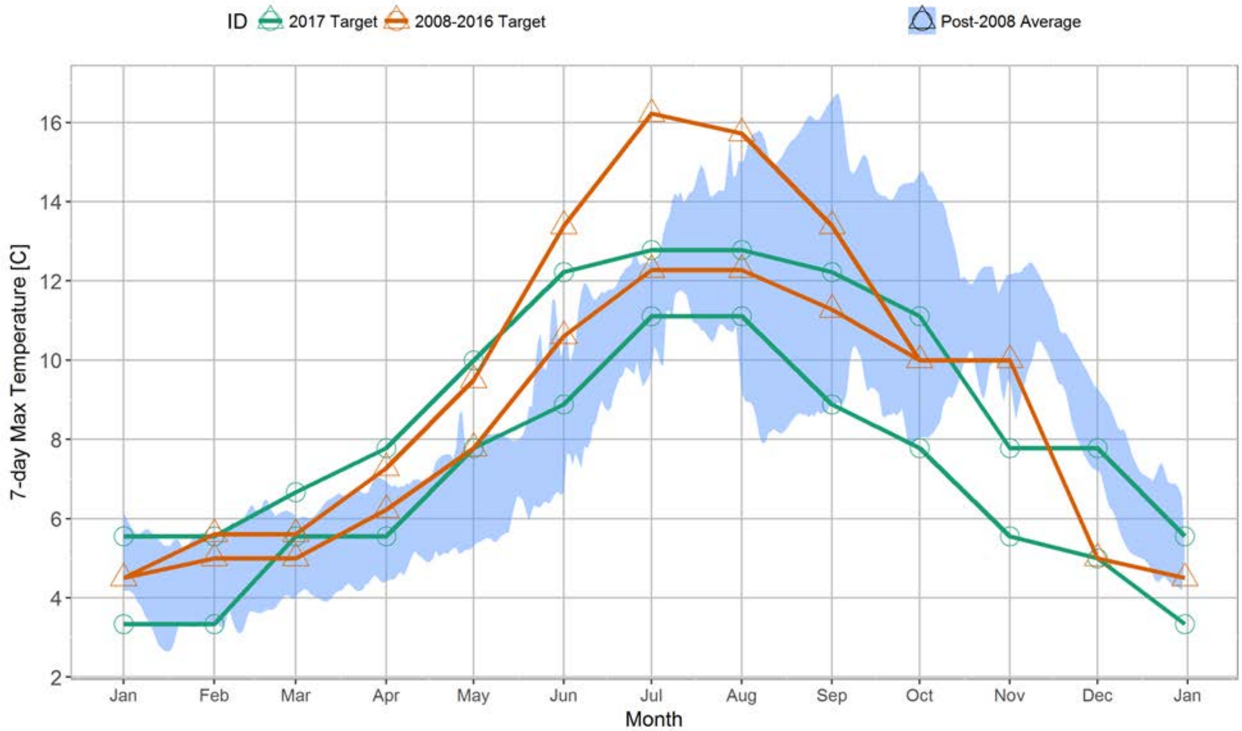


Figure 5-2. Historic temperatures downstream of Detroit Dam (USGS 14181500) (7-DADM = 7-day average of the daily maximum temperature). Orange and Green lines are temperature targets used during 2008-2016 and 2017, respectively

The design of the SWS does not limit operations to a specific temperature target and should be able to accommodate some flexibility. Regardless of the design, there are limits to the magnitude and pattern of release temperature because there is a limited supply of colder, deep water and the surface layer accumulates solar heat during the summer. Based on model sensitivity, lowering the LIGs below 1,340 ft. has little impact on release temperatures (see discussion below). Generally, the warmer the discharge temperature in July-August, the cooler the discharge temperature can be in October-November. Also, there is no temperature control possible when the reservoir is not stratified, approximately December through March. For the scenarios below, an operational temperature was developed using the 30-day average of the maximum daily average temperature predicted using the ‘no-dam’ calculation discussed above (upper limit of the grey shaded area in Figure 5-1). The operational target is used in the model to blend water from different outlets. The operational target flows the same annual pattern as the Chinook/steelhead target but does not have the rapid increases / decreases between months.

5.3 PREDICTED IMPACT OF SWS AND FSS

5.3.1 Fixed Outlet Depth Sensitivity

To provide information through the USACE EDR process and to assess downstream water temperature impacts from a temperature control structure (TCS) (referred to as a SWS in this report), USGS developed CE-QUAL-W2 models for application in Detroit Lake (Buccola, et. al., 2012; Buccola, et. al., 2015; Rounds and Buccola, 2015; Stonewall and Buccola, 2015). The CE-QUAL-W2 model is a two-dimensional, hydrodynamic model commonly used for stratified reservoirs. The USGS EDR model was used for sensitivity of Detroit discharge temperature to various elevations of the fixed, deeper outlet (termed ‘uro-float_400fmin’ in Buccola et al, 2012). The outlet temperature target was the daily variable maximum of the 7-day moving average of the daily average temperature of the predicted no-dam temperature. Without-dam temperature is estimated based on the flow weight average of the Breitenbush River, North Fork Santiam River and Blowout Creek from 1998 to 2013 plus a 0.11 °C / mile heating rate (9 miles estimated average distance from gage to Detroit Dam). The *hot-dry* and *cool-wet* design years were used.

The discharge temperature in October and November is sensitive to the elevation of the fixed, deep outlet. Outlet elevations of 1,400 and 1,371 ft have a more pronounced warming during the fall than lower elevation outlets (Figure 5-3). Outlet elevations deeper than 1,340 ft show minimal temperature improvement when compared to an outlet at 1,340 ft. The volume of the reservoir below an elevation of 1,340 ft is 6% of the maximum volume of reservoir. As such, the lower elevation outlets do not have access to a large volume of cold water.

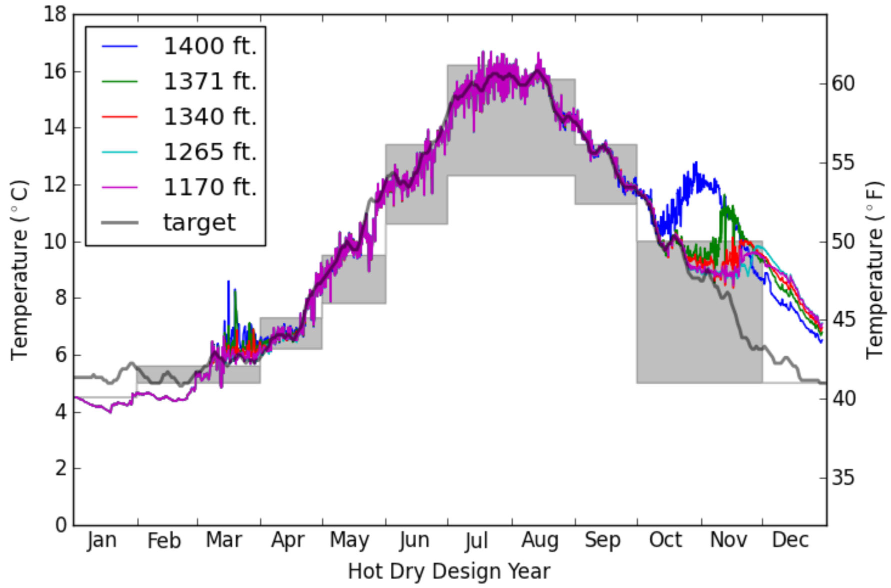


Figure 5-3. Predicted discharge temperature for different elevations of the fixed, deeper outlet. The grey boxes are the 2008-2016 target temperatures (range) presented in the EDR. The grey target line is a representation of no-dam temperature estimated which is described in the text

5.3.2 Minimum Flow Rates

The precision of temperature control can be impacted by minimum flow rates under certain conditions. While the minimum release rate from the lower ROs is 470 cfs due to vibration issues, simulations using a minimum flow rate of 100 cfs were tested in the temperature model to examine thermal impacts during low flow as temperature control is transitioning from surface-dominated release to a blended surface and deep release. Based on model results, at a total flow of 1100 cfs, the 100 cfs minimum could lead to a 1.0 °C of cooling. At a total flow of 2000 cfs, the 100 cfs minimum flow could lead to a 0.5 °C of cooling. The later flow rates are more representative of the anticipated summer peaking operation. This loss of precision of temperature control would likely be limited to short periods and have little impact on achieving temperature targets.

5.3.3 Maximum Flow Rates

Maximum flow rates for the weir gates and fixed gates are 4500 and 5600 cfs, respectively. The Corps anticipates there will be times of the year when the entire powerhouse flow will be directed either through the upper SWS gates (minimum 1000 cfs for fish passage in the FSS scenario) or through the lower SWS gates.

5.3.4 Temperature Model of Detroit Reservoir with a SWS and FSS

Based on the USGS water temperature models, a SWS was tested on two design years: *hot-dry* and *cool-wet*. Since the model is two-dimensional, the model must simplify the representation of outlet structures as shown in Table 5-2. Model line width determines the vertical impact of the outlet, with a longer line width having less impact vertically (i.e. the withdrawal is more vertically focused and has less of an impact to layers further away).

Table 5-2. Existing and proposed outlet properties with temperature model representation. SWS = Selective Withdrawal Structure, CL = centerline, FSS = Floating Screen Structure, HIW=High Invert Weirs, LIG = Low Intake Gates

Name	Reported Elevations	Temperature Model Representation	Flow Range	Notes
Spillway	1541 ft at crest	CL Elevation = 469.7 m (1541 ft) Line width = 25 m	0 – 191,640 cfs (0 – 5423 cms)	Used when water surface elevation is greater than crest and discharge exceeds powerhouse capacity.
Power-house	1395.5 ft (invert)	CL Elevation = 427.6 m (1403 ft) Line width = 6.8 m	1950 – 4960 cfs (55.2 – 140.4 cms)	Not represented in SWS temperature model (weir gates and lower intake gates feed powerhouse via a wet well which is not explicitly represented)
HIW SWS Gates	Min = 1410 ft Max = 1570 ft	***April-Sep:*** 6 floating outlets with CL from 0.19 to 4.27 m (0.63 to 14 ft) depth. ***Sep-April:*** 6 floating outlets with CL from 0.19 to 8.53 m (0.63 to 28 ft) depth. Line width = 6.8 m	0 – 5600 cfs (0 – 158.5 cms)	Scenario ***SWS***
FSS	Min = 1410 ft Max = 1570 ft	***April-Sep:*** 6 floating outlets with CL from 0.19 to 3.29 m (0.63 to 10.8 ft) depth. ***Sep-April:*** 6 floating outlets with CL from 0.65 to 6.74 m (2.1 to 22.1 ft) depth. Line width = 6.8 m	1000 – 5600 cfs (28.3 – 158.5 cms)	Scenario ***FSS_1000cfsMin*** specifies 1000 cfs minimum year-round; *Scenario* ***FSS_50prcMinSep-Dec*** specifies 1000 cfs minimum January-September and 50% minimum September-December
LIG Gates	1332 ft (centerline)	CL Elevation = 406 m (1332 ft) Line width = 6.8 m Assuming no maximum head restriction.	100 – 5600 cfs (2.8 – 158.5 cms)	Weir and fixed gates maximum combined flow of 5600 cfs

Upper Regulating Outlets (2)	1340 ft (invert)	CL Elevation = 410.0 m (1345 ft) Line width = 6.8 m Max head = 200 ft (61 m)	520 – 14,600 cfs (14.7 – 413.1 cms)	Used when water surface elevation is below spillway crest and discharge exceeds powerhouse capacity. This did not arise in the model scenarios; not explicitly represented in the model.
Test conduit	1340 ft (invert)			Not used in model
Lower regulating outlets (2)	1265 ft (invert)			Used only in deep drawdown scenario

For the design scenarios, the peaking operation of the powerhouse was simulated as 4900 cfs at all reservoir elevations. Based on model sensitivity runs, the greater the peaking flows the less temperature control is available because the greater flows draw water from a greater vertical zone in the reservoir. Therefore, the design scenarios use the maximum peaking flows as a conservative assumption. The daily duration of power generation was computed based on powerhouse capacity and daily outflow requirements. The timing of peaking was prioritized to the evening (18:00–21:00) and morning (06:00–09:00).

5.3.5 Thermal Structure of the Detroit Reservoir

The simulated thermal structure in Detroit Lake is presented in Figure 5-4 and Figure 5-5 for the two design years. The upper SWS gate is raised and lowered with the water surface elevation and is operated year round to provide at least a minimum flow for fish passage. The lower SWS gate was needed to meet the temperature target from July 15 to the end of the year. The spillway and upper ROs were only used for short periods during times of high flow. Although the USGS EDR model assumptions vary from those at the current 60% DDR design, temperature stratification within Detroit Lake is primarily a function of inflow, lake level, inflow temperature, and meteorology and would not likely change greatly due to the changes proposed in the 60% DDR. Therefore, Figures 5-3, 5-4, and 5-5 provide an informative context for outflow temperatures discussed later in this document.

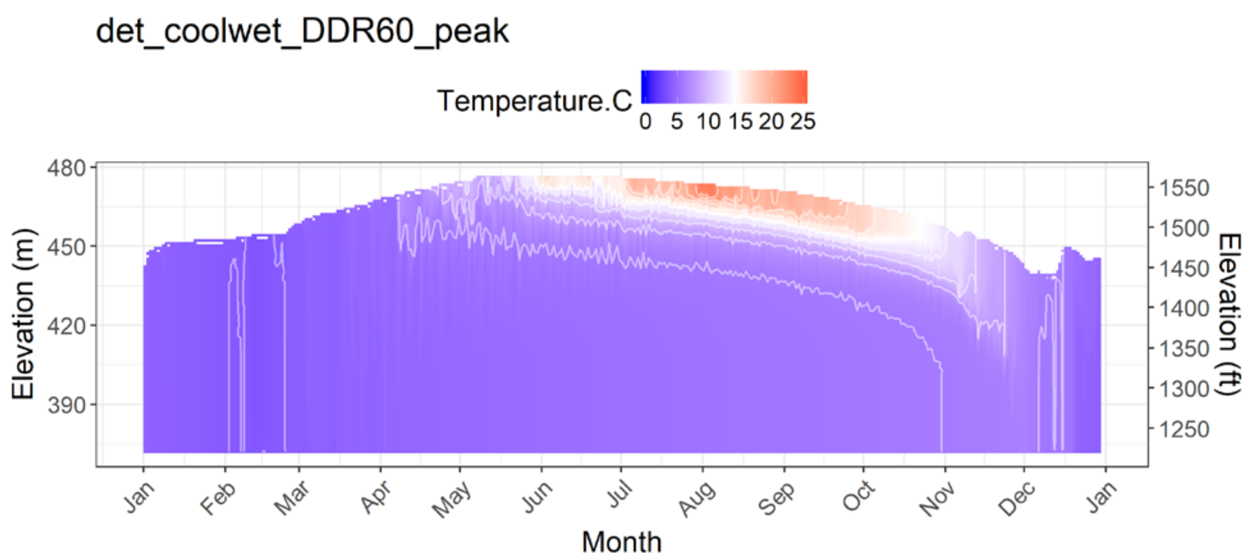


Figure 5-4. Annual temperature isopleths by elevation for Detroit Reservoir during the Cool-wet design year with outlet elevations as defined by the FSS-50prcMinSep-Dec scenario

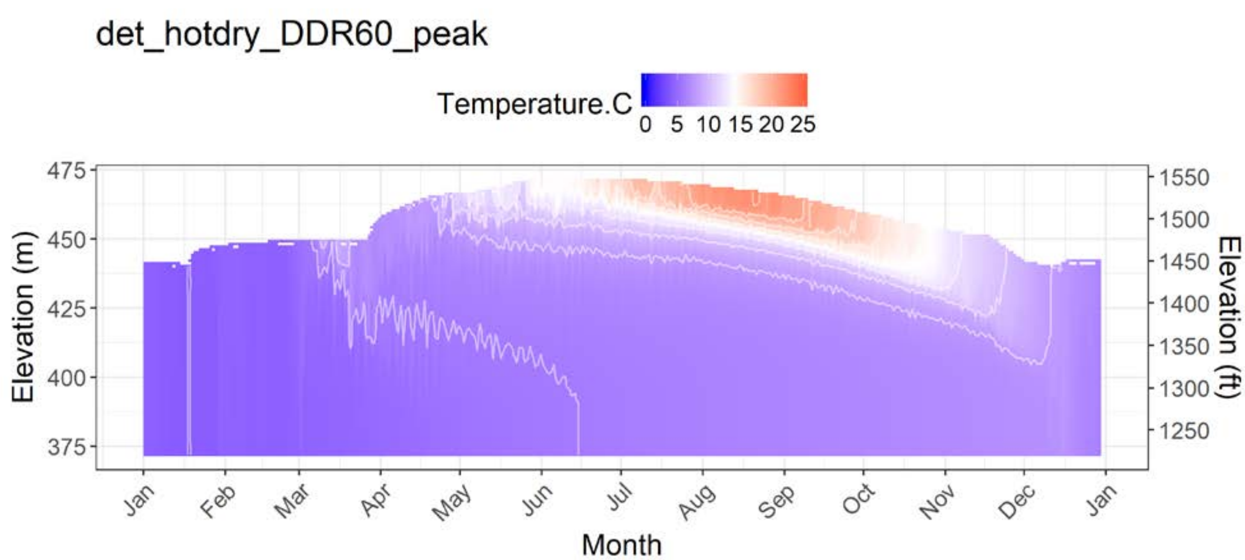


Figure 5-5. Annual temperature isopleths by elevation for Detroit Reservoir during the Hot-dry design year with outlet elevations as defined in the FSS-50prcMinSep-Dec scenario

5.3.6 60% DDR Detroit SWS Outlets and FSS Configurations

Using the USGS EDR models as a starting point, the 60% DDR SWS and FSS design scenarios were developed as follows:

- SWS
  - 6 outlets distributed over a 14-ft HIW depth April-Sep; spanning a deeper 28-ft depth Sep-April
  - No minimum flow through upper-weir outlet; blending with lower RO as-needed
  - Maximum flow through the upper weir outlets = 5600 cfs; flow over this amount routed to LIGs
- FSS
  - 6 outlets distributed over a 10.8-ft HIW depth April-Sep; spanning a deeper 22.1-ft depth Sep-April
  - Scenario *FSS_1000cfsMin* specifies 1000 cfs minimum for downstream fish passage year-round
  - Scenario *FSS_50prcMinSep-Dec* specifies 1000 cfs minimum January-September and 50% minimum of total outflow September-December
  - Maximum flow through the FSS and upper weirs = 5600 cfs; flow over this amount routed to LIGs
- Existing (identical to USGS EDR "Existing" scenarios (Buccola, et.al., 2015))
  - A maximum of 60% of total outflow to be routed to spillway (summer) or upper RO (fall) for temperature operations

Daily average outflows from the upper weir gates in the SWS and FSS scenarios are shown in Figures 5-6 and 5-7 respectively. Note that the daily average outflow and daily average peak outflow are shown in these figures to distinguish between peaking (hourly) values, which correspond to the 1000 cfs minimum hourly flowrate used as the design specification for the *FSS_1000cfsMin* scenario (Figure 5-7). For example, daily average outflow in the *FSS_1000cfsMin* scenario during late October and early November of the hot-dry year is about 200 cfs, while the daily average peak outflow is 1000 cfs during that time. The *FSS_50prcMinSep-Dec* scenario was developed to compare the temperature effects resulting from a minimization of competing flow between the LIG and HIW during the fall when downstream fish transport coincides with upstream adult migration. This increased flow through the HIW during the fall in scenario *FSS_50prcMinSep-Dec* compared to *FSS_1000cfsMin* is shown in Figures 5-7 and 5-8. No minimum flow to the HIW was assigned in the SWS scenarios, which resulted in zero flow from the warmer HIW and only cooler releases from the LIG in the fall (SWS in Figures 5-7, 5-8, 5-9). Daily average HIW outflow as a percentage of total outflow is shown in Figure 5-8.

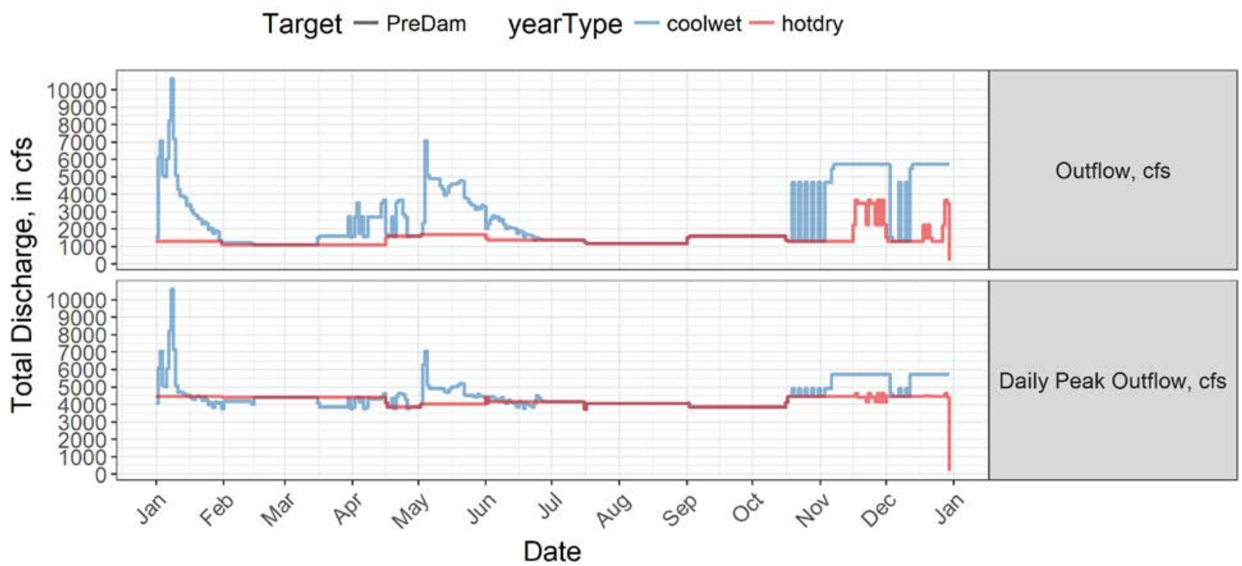


Figure 5-6. Simulated daily averages of total outflow (representing hourly peaking in top row graphs) and total non-zero outflow (bottom row graphs) in cool-wet and hot-dry design years for SWS and FSS scenarios at Detroit Dam

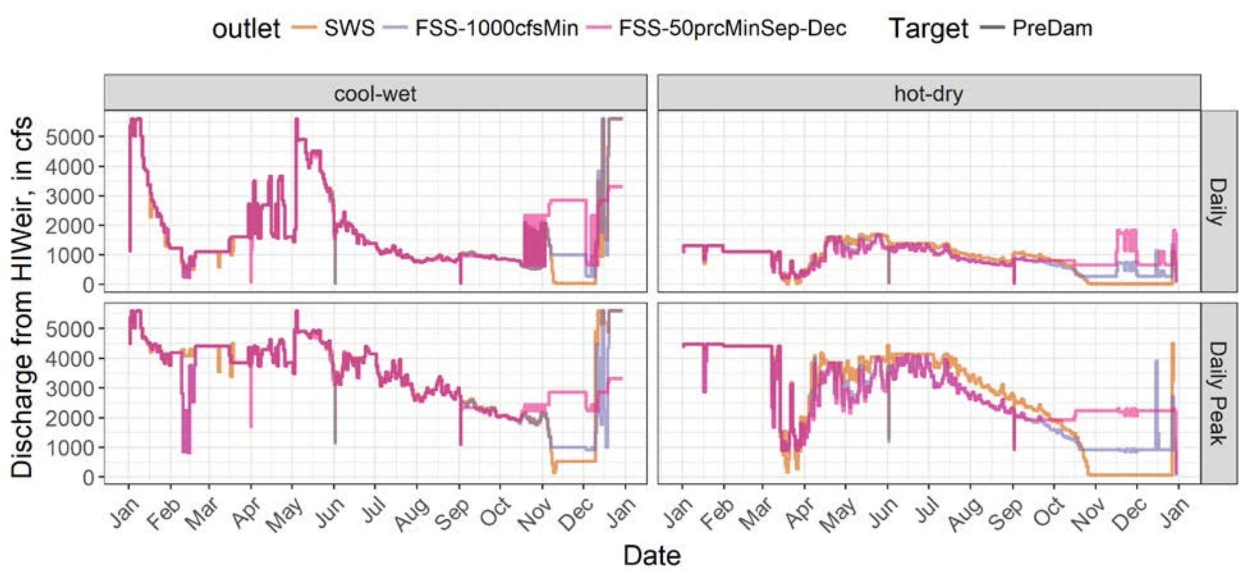


Figure 5-7. Simulated daily averages of non-zero upper weir flow (representing hourly peaking in top row graphs) and daily average upper weir (bottom row graphs) in cool-wet and hot-dry design years for SWS and FSS scenarios at Detroit Dam

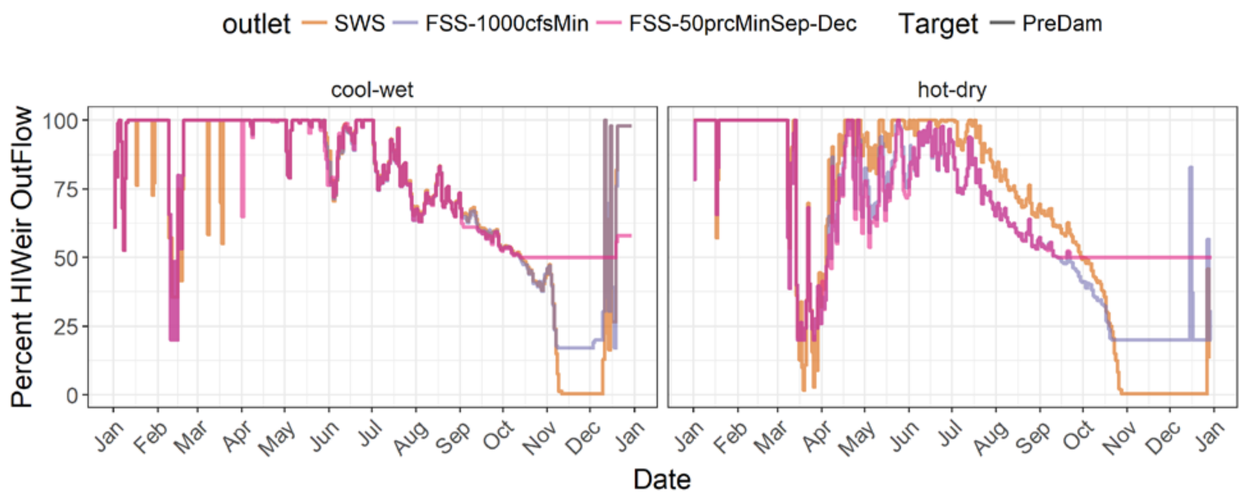


Figure 5-8. Simulated daily average upper weir outflow as a percentage of total outflow in cool-wet and hot-dry design years for SWS and FSS scenarios at Detroit Dam

To achieve the operational temperature, blending of water from the HIW and LIG is necessary from mid-July through October to meet the temperature target based on pre-dam conditions. Simulated temperatures from all scenarios (*Existing*, *FSS_1000cfsMin, FSS_50prcMinSep-Dec*, and *SWS*), and design years (*hot-dry* and *cool-wet*) are presented in Figures 5-9, 5-10, and 5-11. The temperature target used in all scenarios was based on the 7-day average of the long-term daily maximum pre-dam temperature estimates at the Detroit Dam location.

The model predicts that the SWS presented in the 60% DDR will generally be able to achieve the Chinook/steelhead targets in each of the design years up until about mid-October (*hot-dry*) to November (*cool-wet*). In the *hot-dry* scenario, by mid-October the LIG is generally accessing water warmer than operation target, which persists through December (Figures 5-9, 5-10, 5-11). This pattern is less notable in the *cool-wet* flow year. Greater flow from the HIW in *FSS_50prcMinSep-Dec* led to warmer releases compared with *FSS_1000cfsMin* and SWS scenarios from late September to mid-November in the *hot-dry* year, but minimal differences in the *cool-wet* year. By late November and December in the *hot-dry* year, the *FSS_50prcMinSep-Dec* was closer to the target temperature than other scenarios.

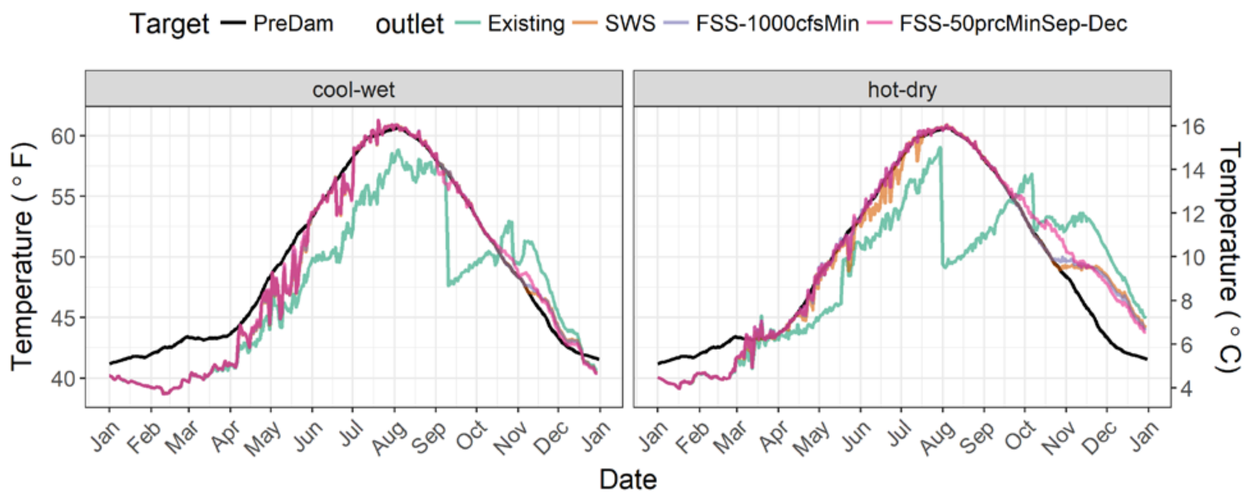


Figure 5-9. Simulated Detroit Dam release temperature in cool-wet and hot-day design years. The temperature target used for each scenario is the 30-day maximum of the long-term average without-dam temperatures at Detroit Dam ("PreDam")

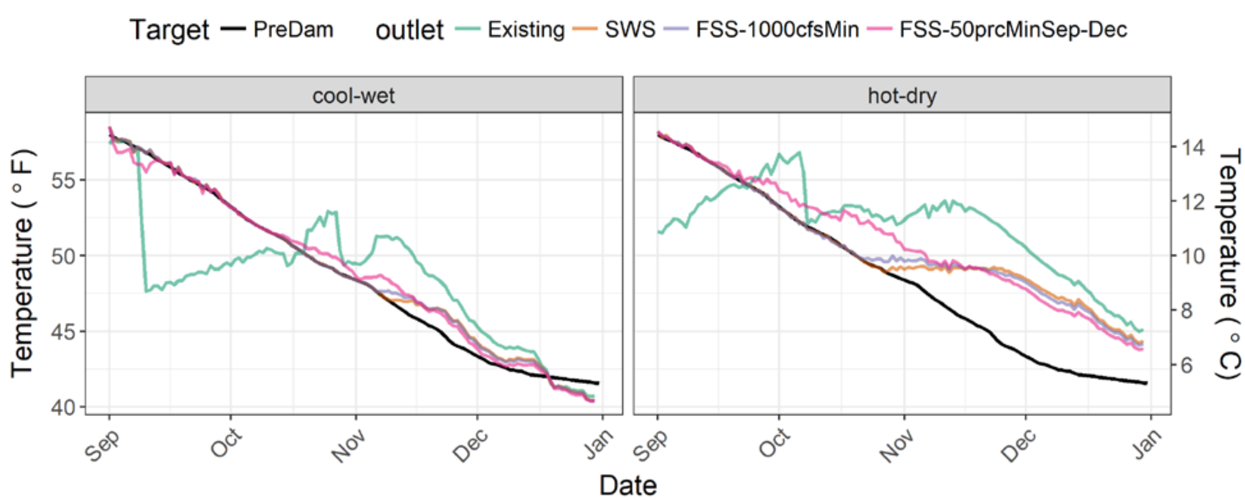


Figure 5-10. Simulated Detroit Dam release temperature for September-December in cool-wet and hot-dry design years. The temperature target used for each scenario is the 30-day maximum of the long-term average without-dam temperatures at Detroit Dam ("PreDam")

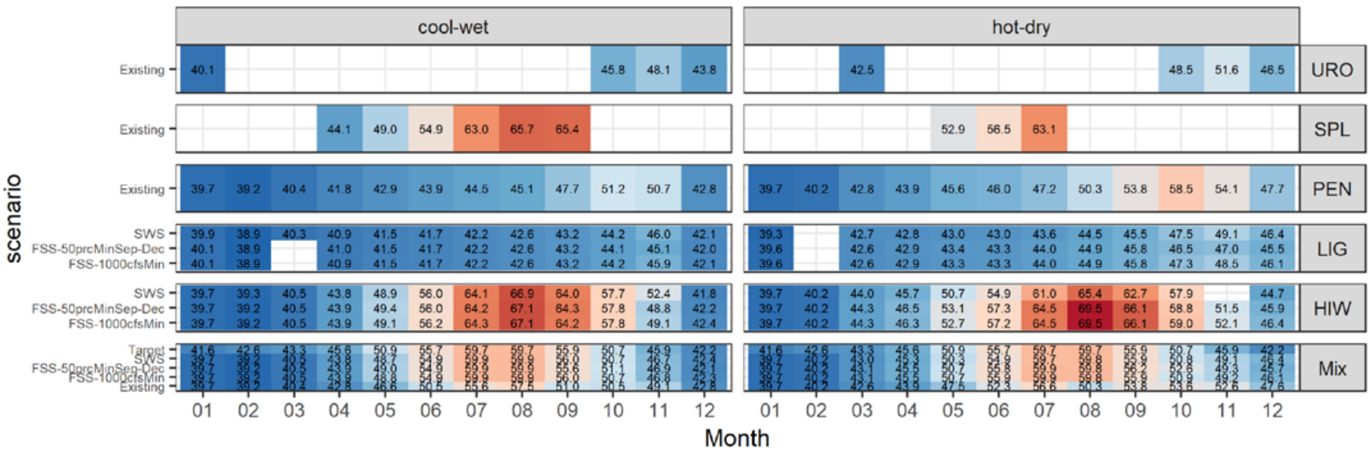


Figure 5-11. Simulated monthly average Detroit Dam release temperatures from each outlet in cool-wet and hot-dry design years. Explanation: URO: upper RO, SPL: Spillway, PEN: Penstocks, LIG: Low invert gates, HIW: High invert weirs, Mix: Mixed outflow temperature

5.3.7 Biological Impacts

Biological evaluation criteria for adult and juvenile Chinook salmon was borrowed from the Middle Fork Willamette 60% EDR (U.S. Army Corps of Engineers, 2015) for temperature control and fish passage alternatives. Each model scenario prediction is compared using evaluation criteria in Table 5-1 (page 5-2) and summarized below with the following temperature impacts for the proposed SWS and FSS compared to *Existing* conditions (Table 5-3 and Figure 5-12). Results from Table 5-3 and Figure 5-12 indicate the following conclusions related to SWS and FSS scenarios:

- Surface releases from SWS and FSS scenarios led to about half as much time below 52 °F during migration period (June) and greater than twice the amount of time in optimal conditions during the rearing period (May 1-Sep 15) compared with existing scenarios.

- In the *hot-dry* scenario year, SWS and FSS scenarios reduced the amount of time exceeding 50.2 °F during the incubation period by about 22-37% compared with the *existing* scenario.

- SWS and FSS scenarios had similar impacts (< 2% difference) in all life stages except for incubation, where *FSS_50prcMinSep-Dec* had 10% more time with temperatures above 50.8 °F compared to *SWS* and *FSS_1000cfsMin* scenarios.

Table 5-3. Summary table evaluating the temperature simulations at Detroit Dam for the percent of time in which each scenario met the life stage criteria and associated impact for chinook salmon in *Existing* and the proposed SWS and FSS configurations. Percentages are the mean of *hot-dry* and *cool-wet* scenarios

Life stage	Impact	Existing	SWS	FSS-1000cfsMin	FSS-50prcMinSep-Dec
Migration (<52.0; Jun-01 to Jun-30)	Delay	68	36	36	34
Holding (>60.8; Jun-01 to Sep-01)	Delay	0	2	2	2
Rearing (>57.2 & <60.8; May 01 to Sep 15)	Optimal	15	48	50	48
Spawning (>55.4; Sep-01 to Oct-31)	Sub-optimal	20	40	40	41
Incubation (>50.2; Sep-01 to Dec-31)	Sub-optimal	52	40	40	50

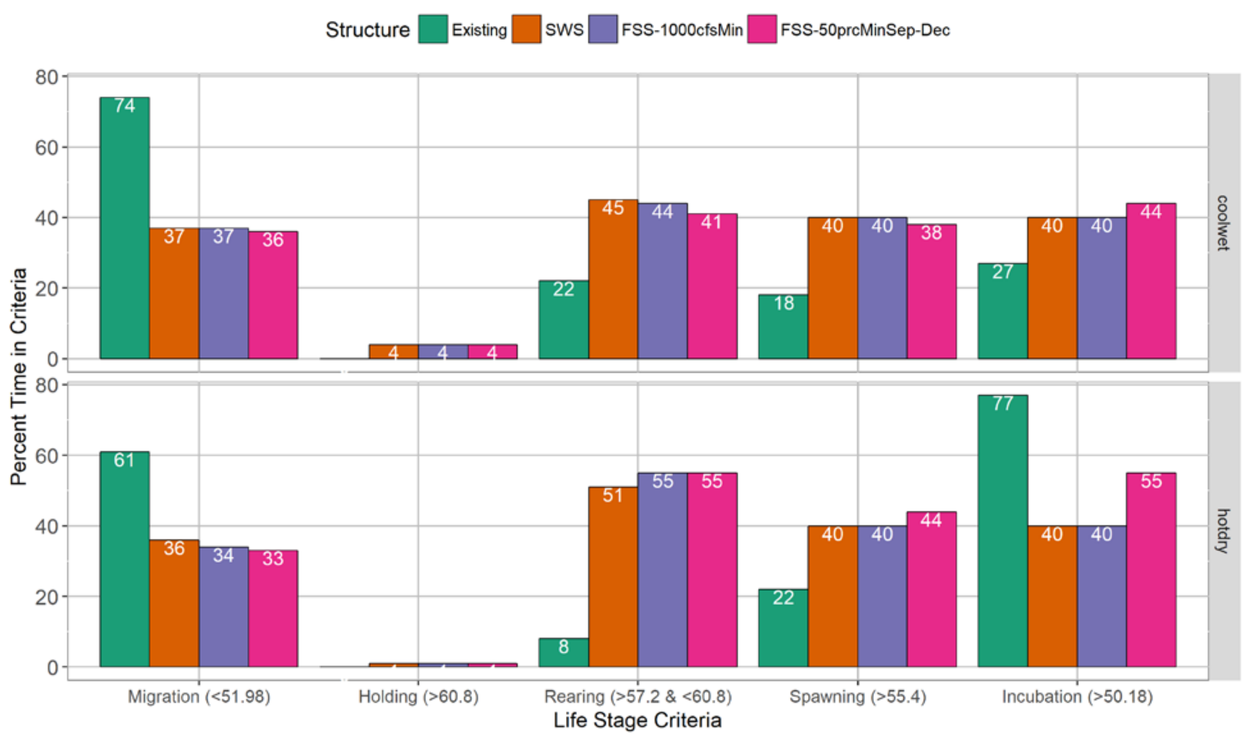


Figure 5-12. Summary table evaluating the temperature simulations at Detroit Dam for the percent of time in which each scenario met the life stage criteria and associated impact for chinook salmon in *Existing* and the proposed SWS and FSS configurations.

The timing of fall Chinook egg emergence is used to assess the relative impacts of temperature operations on early Chinook life stages. Early emergence could expose frye to additional predation or stress during the winter. Estimated emergence timing is calculated based on cumulative thermal units (degree-days) beginning on the presumed day when eggs are in the gravel and accumulated until the degree-day reaches 1750 °F-day accumulated thermal units.

The range of estimated egg emergence for each scenario mentioned above is compared with measured conditions downstream of Detroit and Big Cliff dams at Niagara (USGS 14189500; labeled "NiagaraPost08") and the estimated temperature without Detroit Dam in place (labeled "UpstreamMix") in Figure 5-13. Generally, spawning occurs from September to October, but emergence timing was calculated based (started) on three spawning dates: September 1 (early), September 20 (peak), and October 1 (late). The bar-ends in Figure 5-13 represent the emergence timing under the range of environmental forcings in *hot/dry* and *cold/wet* simulations, whereas the bar-ends for *NiagaraPost08* and *UpstreamMix* indicate the maximum and minimum of those data. Results indicate the following:

- No detectable difference between FSS and SWS scenarios, indicating that a HIW depth change from 7.5 to 12.5 ft makes little difference in emergence timing.

- Compared to "Existing" conditions scenarios, egg emergence from peak spawners (Sep 20) was delayed about 2 weeks under FSS and SWS scenarios (Figure 5-13).

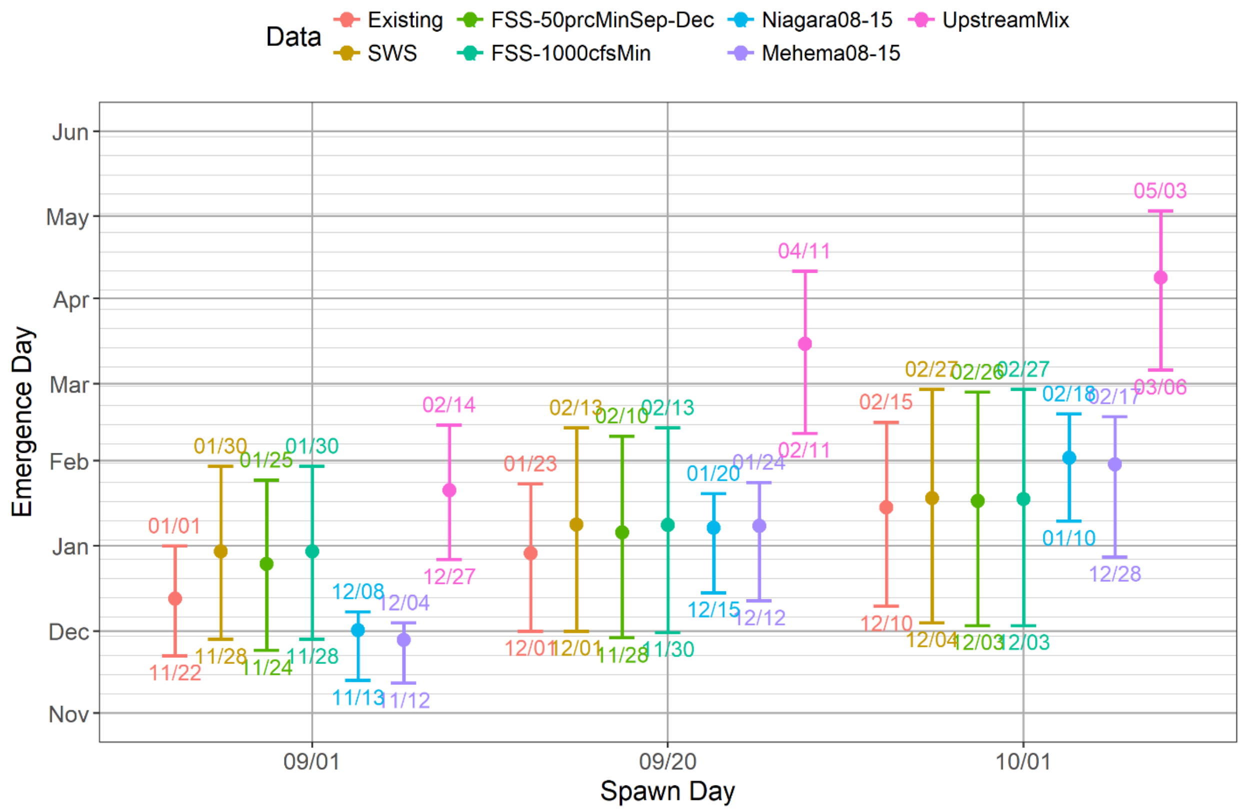


Figure 5-13. Estimated emergence timing date range (indicated by bar ends and text [MM/DD]) and mean (indicated by dots) immediately downstream of Detroit dam. Simulations are compared to measurements at USGS 14189500 ("NiagaraPost08") and without-dam estimates ("UpstreamMix")

5.3.8 Big Cliff Reservoir and North Santiam River

The USGS used CE-QUAL-W2 to represent Big Cliff Reservoir and the downstream North Santiam River by Buccola et al (2015). While the USGS study scenarios for the SWS and FSS are different from those in this 60% DDR, the results can help to inform potential impacts from this report. The USGS temperature scenario results from Detroit were used as input into the Big Cliff model. The residence time of Big Cliff reservoir can be as much as 2 days during the summer. The inflow and lake elevations of Big Cliff were simplified to a daily average because the rapid changes due to peaking operations at Detroit Dam lead to model instability. Beyond rapid fluctuations due to travel times, the temperature model predicts only minor temperature changes (approximately +/- 0.3 °C) between inlet and outlet temperatures at Big Cliff Reservoir for the Detroit SWS design scenarios. As part of the USGS study, a HEC-RAS temperature model was developed for the North Santiam River to estimate the impact of different flow and temperature releases from Big Cliff / Detroit Reservoirs (Stonewall and Buccola, 2015).

Estimated egg emergence timing is summarized from the USGS study in Table 5-4. While the structural scenario details have changed since the USGS study, the results for estimated egg emergence at Detroit Dam are closely fit to those calculated and shown for the 60% DDR results in Figure 5-13 (comparing Sep 20 *Existing* and *SWS* results in Figure 5-13 with *Existing* and *Sliding Weir* at Detroit Dam in Table 5-4). With this verification, Table 5-4 can be used to estimate the potential downstream impacts during the incubation period.

Table 5-4. Calculated average emergence day for the cool/wet, normal, and hot/dry environmental forcings at four locations from the Detroit Lake, Big Cliff Lake, and North Santiam River models (re-created from Buccola, et.al., 2015)

Scenario identifier	Scenario description	Scenario type	Average spring Chinook emergence day			
			Detroit Dam (RM 60.9)	Big Cliff Dam (RM 58)	Mehama (RM 38.7)	Greens Bridge (RM 14.6)
c1, n1, h1	*NoBlend*	operational	Dec. 2	Dec. 5	Dec. 7	Dec. 1
c2, n2, h2	*Existing*	operational	Jan. 6	Jan. 8	Jan. 3	Dec. 23
c3, n3, h3	*SlidingWeir*	structural	Jan. 13	Jan. 17	Jan. 9	Dec. 27
--	*WithoutDams*[1]	--	Mar. 30	Mar. 30	Feb. 18	Jan. 11

[1]Without-dams scenario estimated at Big Cliff Dam, then simulated in North Santiam River model.

5.3.9 Climate Change

A USGS study on climate change impacts to Detroit Lake temperatures was completed in 2016 (Buccola, et.al., 2016). Highlights from this study based on three global circulation models, rainfall-runoff models, and water temperature models include the following:

- Warmer atmospheric conditions (2 °C warming from base to future periods) and lower precipitation led to a 23% inflow reduction and 1 °C increase in mean annual inflowing stream temperatures. This led to an average annual warming of in-lake temperatures by 1.1 °C.

- A hypothetical floating surface withdrawal similar to that described in the 60% DDR improved temperature control in summer and autumn (0.6 °C warmer in summer, 0.6 °C cooler in autumn compared to existing structures) without altering release rates or lake level management rules (comparing scenarios “curmins” [*Existing* –like scenario] and “curmins_fl” [SWS-like scenario] in Figure 5-14).

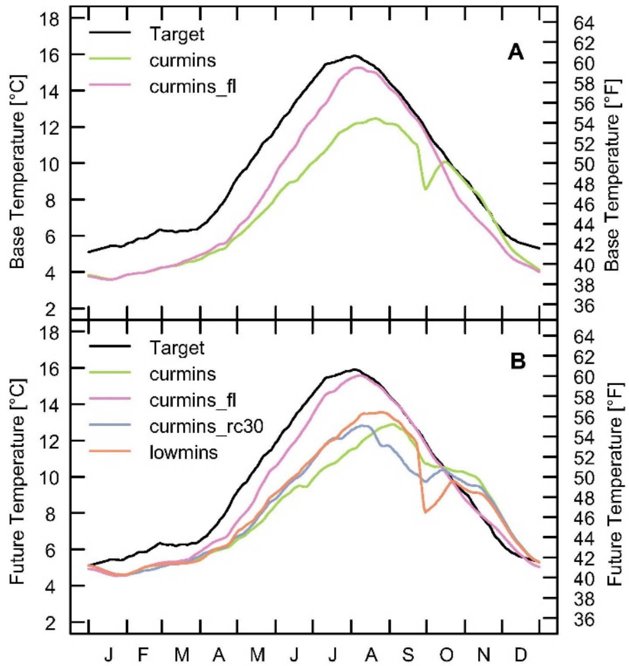


Figure 5-14. Simulated water temperature immediately downstream of Detroit Dam in base and future periods (recreated from Figure 12 in Buccola, et.al., 2016)

5.3.10 Water Quality Impacts During Construction

The water quality impacts during construction will be related to the potential draw down of Detroit Reservoir. There will likely be increased turbidity in the North Santiam River during the period the reservoir is drawn down. The level of turbidity will likely depend on the level of the lake, the duration of the draw down, and the flow rate during a draw down. Multiple alternatives, including various degrees of a drawdown, are in the process of being developed and analyzed for Detroit Lake. One initial scenario of a deep drawdown to 1,312 ft has been simulated using the CE-QUAL-W2 temperature model. Under this scenario, dam outflow was set equal to the inflow for the entire year and the lower RO was the primary outlet with an invert elevation of 1,265.33 ft [385.67 m]. Figure 5-15 shows the temperatures at 1,312 ft elevation throughout a *cool/wet* and *hot/dry* year scenario. The smooth black line is the temperature target based on without-dam temperatures; the step-leveled black line is the 2017 operational temperature target.

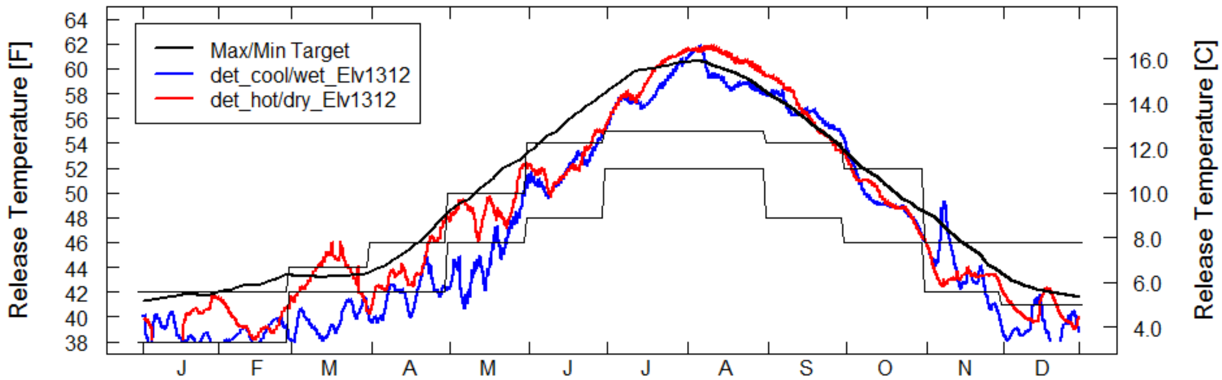


Figure 5-15. Water temperature immediately downstream of Detroit Dam under a deep drawdown scenario (water surface elevation of 1312 ft)

The estimated egg emergence timing of a 1,312-ft elevation in *cool-wet* and *hot-dry* year scenarios is compared with measurements downstream of Big Cliff Dam prior to and after temperature operations at Detroit Dam (*NiagaraPre08*, *NiagaraPost08*), and the estimated temperatures without the effects of dams (*UpstreamMix*) in Figure 5-16. These initial results indicate that egg emergence downstream of Detroit during an extreme drawdown would be within the lower range of the estimated without-dams temperatures (*UpstreamMix*).

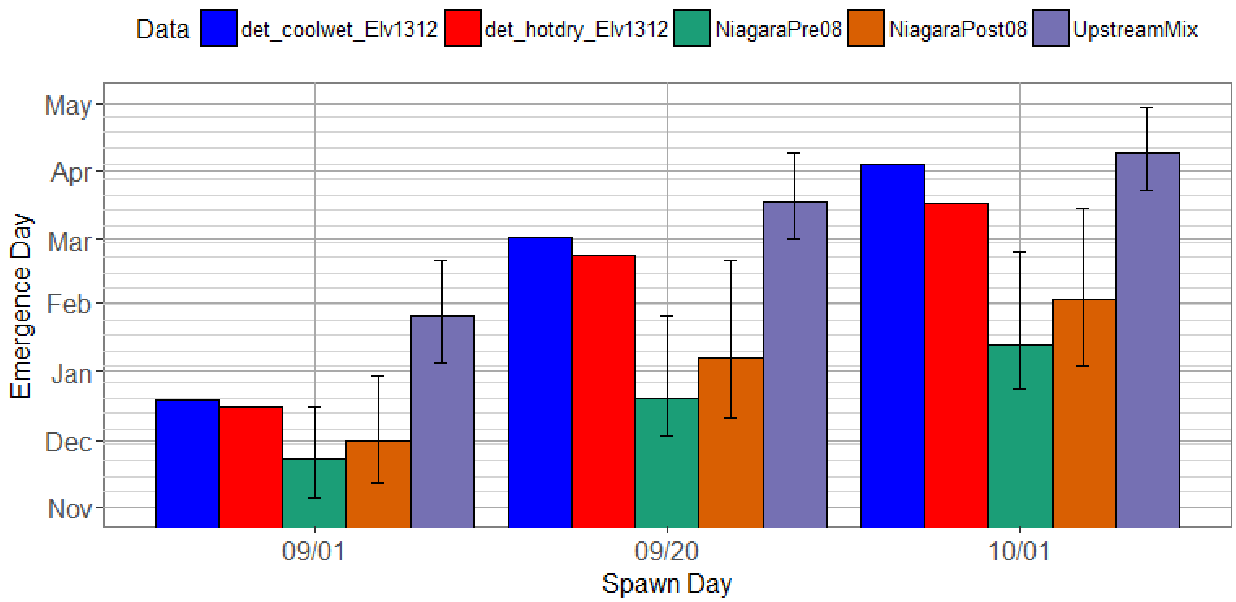


Figure 5-16. Water temperature immediately downstream of Detroit Dam under a deep drawdown scenario (water surface elevation of 1312-ft)

Since downstream flow targets would not necessarily be met under this scenario, some quicker heating of the river downstream of Big Cliff Dam could be realized. Additional water quality analysis will occur in the environmental assessment when more details about construction have been determined.

5.4 OTHER WATER QUALITY PARAMETERS

5.4.1 Total Dissolved Gas (TDG)

State of Oregon water quality standards for TDG are:

- 105% of saturation (relative to atmospheric pressure at the point of sample) in hatchery intake waters and other waters of less than 2 ft in depth.

- 110% of saturation in other waters.

TDG data has not been evaluated by ODEQ to determine CWA status of the North Santiam River (ODEQ 2010, most recent evaluation at this time).

USACE (2009) documented the impact of Detroit and Big Cliff operations on TDG concentrations downstream and concluded:

- Combined releases from Detroit spillway and ROs produced TDG concentrations that exceed 110% directly downstream of Big Cliff Dam.

- Big Cliff spillway operations result in TDG concentrations that exceed 110% directly downstream of Big Cliff Dam.

- Supersaturated TDG was quickly degassed in the river reach between Big Cliff Dam and Minto (4.0 miles downstream of Big Cliff Dam). Minto TDG concentrations were approximately 105% regardless of concentration directly downstream of Big Cliff Dam.

- Flow through the powerhouses at Detroit and Big Cliff Dams results in TDG concentrations < 105% directly downstream of Big Cliff Dam.

Continuous TDG data collected by USGS on the North Santiam at Niagara (0.8 miles downstream of Big Cliff Dam) since September 2011 shows a similar pattern. TDG concentrations tend to exceed 110% when there is flow over the spillway at Big Cliff Dam (Figure 5-17).

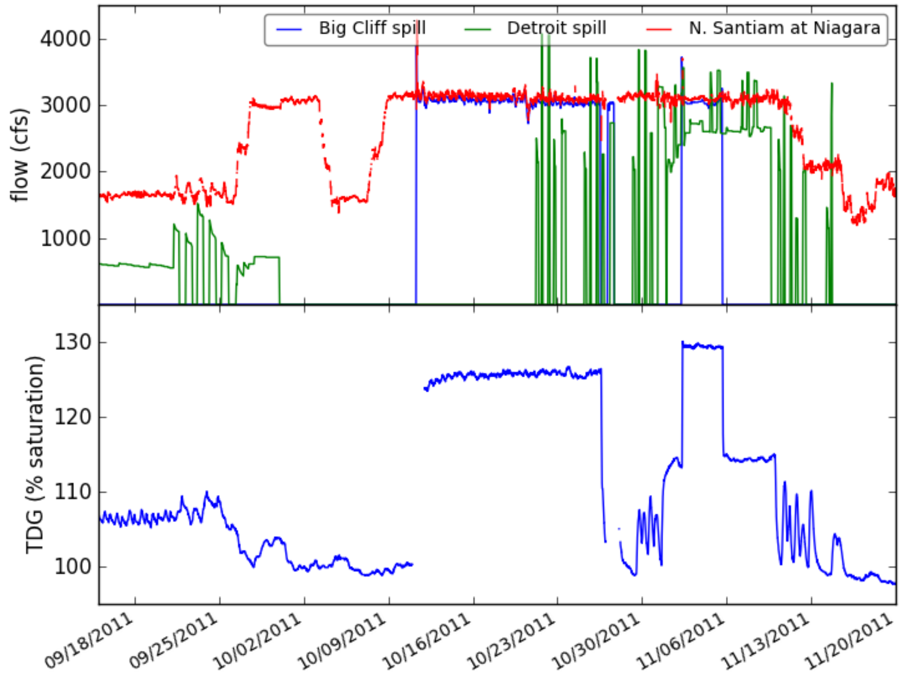


Figure 5-17. Comparison of flow and TDG concentrations at Niagara. 'Spill' indicates flow not through the powerhouses

Current temperature operations at Detroit use the spillway and upper RO which can lead to elevated TDG concentrations. The construction of the SWS will alleviate the need to use the spillway and upper RO for temperature operations. Flow over the spillway and through the ROs at Detroit and Big Cliff when the powerhouses are not available, or when flow exceeds powerhouse capacity, will likely continue to result in elevated TDG concentrations. However, these elevated concentrations should dissipate downstream before the river reaches Minto (4.0 miles downstream of Big Cliff).

5.4.2 Reservoir Eutrophication

Natural and anthropogenic nutrients entering lakes can cause algae blooms which can lead to low DO concentrations, high pH, algal toxins and aesthetic impairment. High nutrient influx can lead to low DO concentrations during algal respiration or decomposition, typically resulting in super-saturated oxygen concentrations during photosynthesis, and possible diurnal fluctuation. Larson (2000) summarized the limnological and water-quality studies in Detroit and Big Cliff reservoirs. There have been reports of intense algae blooms dominated by blue-green, nitrogen-fixing bacteria such as *Anabaena flos-aquae* (*Cylindrospermopsin*) during the spring and early summer of 1990 (Larson 2000) and May 30 to June 13, 2007. The latter resulted in a harmful algae bloom advisory by the Oregon Health Authority (OHA). In 2018, EPA and OHA guidelines for cyanotoxin-based water advisories became more stringent. This coincided with a harmful algae bloom and drinking water advisories for Stayton and Salem water supplies downstream of Detroit during June of 2018. The Corps is currently working with USGS to calibrate the existing CE-QUAL-W2 model of Detroit Lake for the predominant species, *Anabaena flos-aquae* (*Cylindrospermopsin*). This will help to assess the future impacts of year-round surface water releases at Detroit on this dominant (toxin-producing) species. This predominant algal species are blue-green algae, which affix nitrogen from the atmosphere to gain a biological advantage over other algal species. This is an indication that the limiting nutrient in Detroit Lake is nitrogen.

Monthly sampling May through September 2013, from various depths near the dam, resulted in an average total phosphorus concentration of 0.011 mg/L and total nitrogen concentration of 0.10 mg/L. Measurements indicate a ratio of nitrogen to phosphorus of about 10 but can be elevated at the surface during bloom events. During a nitrogen-fixing blue-green algae bloom in 2013, increased nitrogen and reduced phosphorus concentrations were observed near the surface of the lake (as compared to deeper measurements). Monthly depth profiles between June and September of DO ranged from 7.6 to 11.5 mg/L with a mean of 9.6 mg/L. Depth profile pH ranged from 6.3 to 8.8 with a mean of 7.3. The pH values greater than 8.5 occurred on June 26, 2013, between depths of 0 to 25 ft. Similar concentrations and pH levels were measured by USGS in 2003. USGS also measured Secchi disc transparencies between 3.3 and 7.8 m with a mean of 5.7 m and chlorophyll a concentrations between 1.0 and 5.0 with a mean of 3.3 µg/L.

Although the lake levels will remain similar to historic conditions after the construction of the SWS, the thermal profile of the reservoir and the residence time in the photic zone could change. Before interim temperature operations began in 2007, most flow in the summer was routed through the powerhouse intake (elevation 1,396 ft., 168 ft below the summer pool elevation target). After construction of the SWS, most of the flow until the July 15th flow will likely be routed through the weir gates which will be continuously adjusted to 15 ft below the water surface. Between July 15 and October 15, flow will be mixed from the weir gates and the LIGs (elevation 1,340 ft). After October 15, most of the flow will be routed through the LIGs. This operation could delay thermal stratification and the warming of the surface layer. It is not known how changes to the thermal structure and outlet elevation will impact algae concentrations. During interim temperature operations, there has not been a reported frequency of intense algae blooms; however, there has not been consistent monitoring.

5.4.3 North Santiam River Eutrophication

The USGS has been collecting continuous dissolved oxygen, pH, and chlorophyll a measurements on the North Santiam River (Figure 5-18 shows locations). PH is a good indicator of algal activity and is not as dependent on seasonal temperatures as DO concentration. Typically, pH downstream of Big Cliff is between 7.0 to 7.6. Summer pH tends to increase in the downstream direction and can exceed the pH 8.5 criteria (Figure 5-19). The impact of the reservoir on the increased pH downstream is not known. There does not appear to be a major shift in seasonal pattern or magnitude of pH concentrations at Geren Island, near Stayton, due to interim temperature operations which began in 2007 (Figure 5-20). The interim temperature operations resulted in warmer water released from Detroit Dam in July and August. Interestingly, pH concentrations in 2001 are similar to other years despite having the warmest September temperature for the period of record. This qualitative analysis of the pH data indicates that the change in thermal pattern from the Detroit Reservoir will not impact pH in the North Santiam River. Furthermore, if pH is a good indicator of algae productivity, then likely algal productivity will not increase, either. However, a quantitative analysis, like a water quality model, would reduce the uncertainty of the prediction.

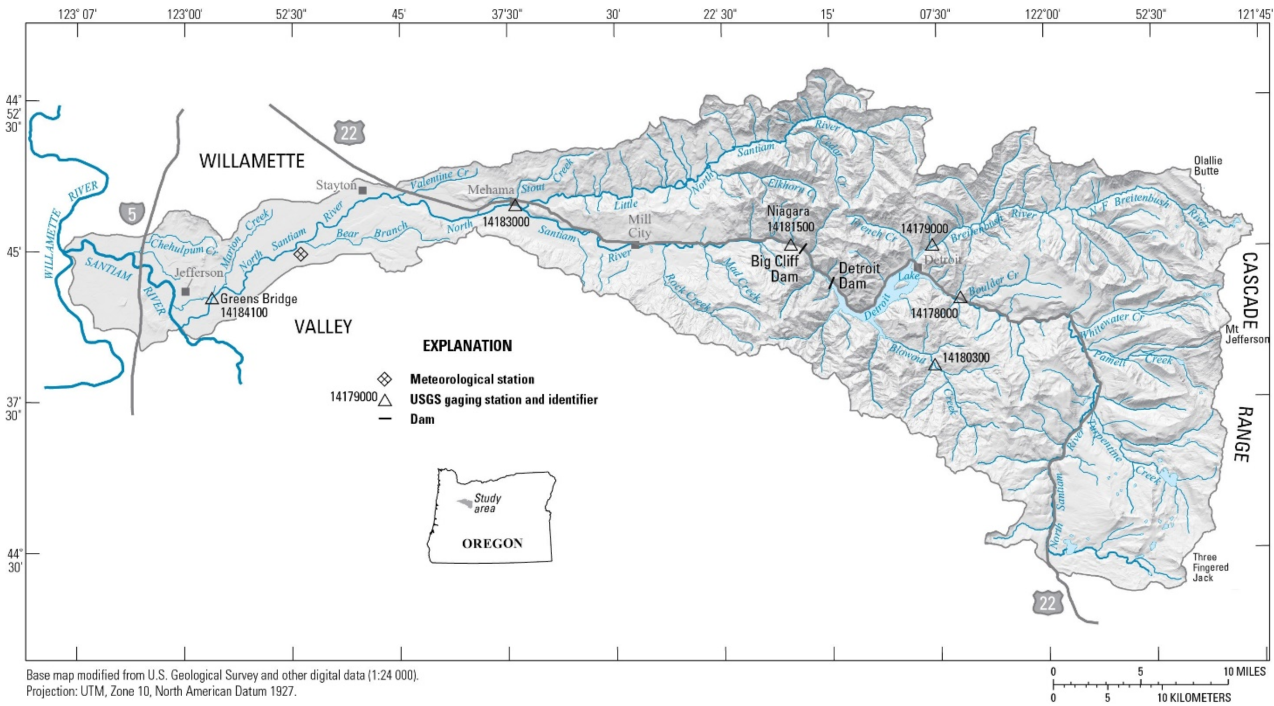


Figure 5-18. Map showing North Santiam and Santiam Rivers and the North Santiam River Basin, northwestern Oregon. (from Sullivan and Rounds, 2004)

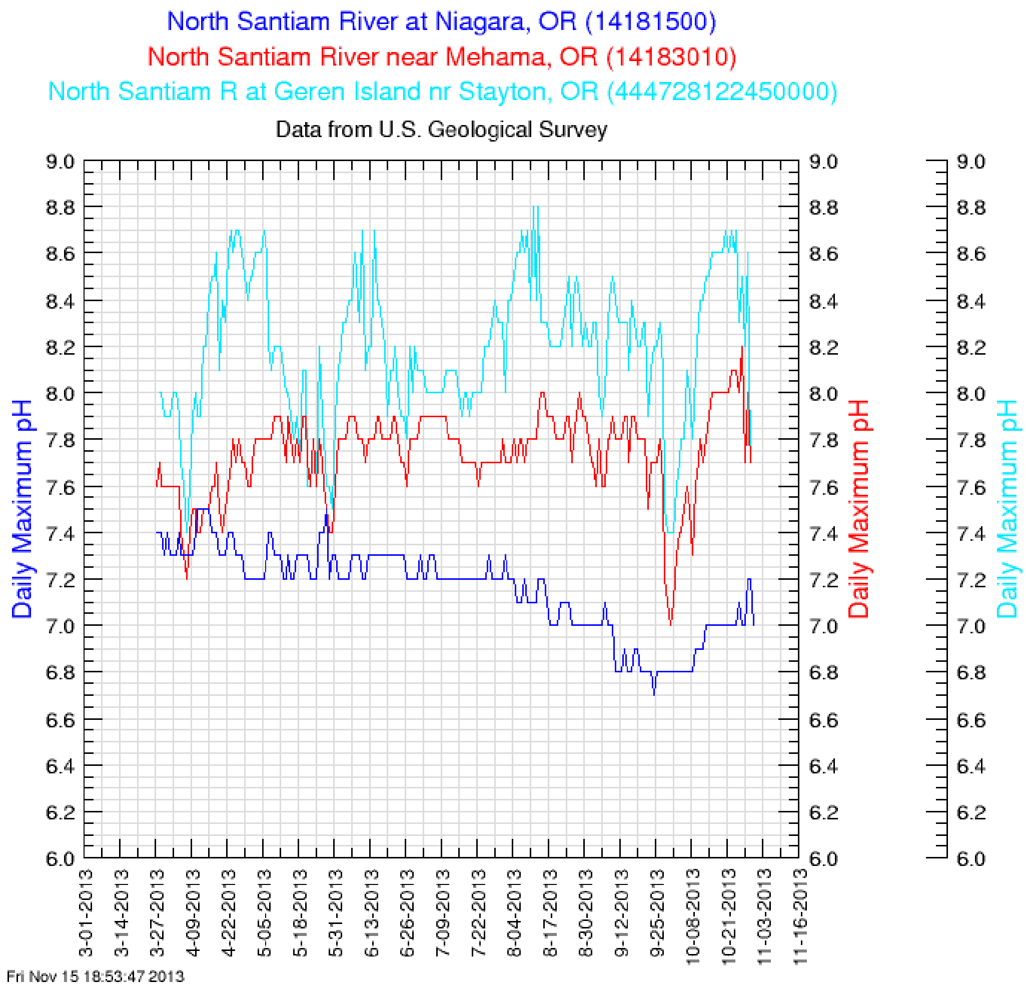


Figure 5-19. Daily maximum pH from USGS (2013). Data after July 2, 2013 are provisional

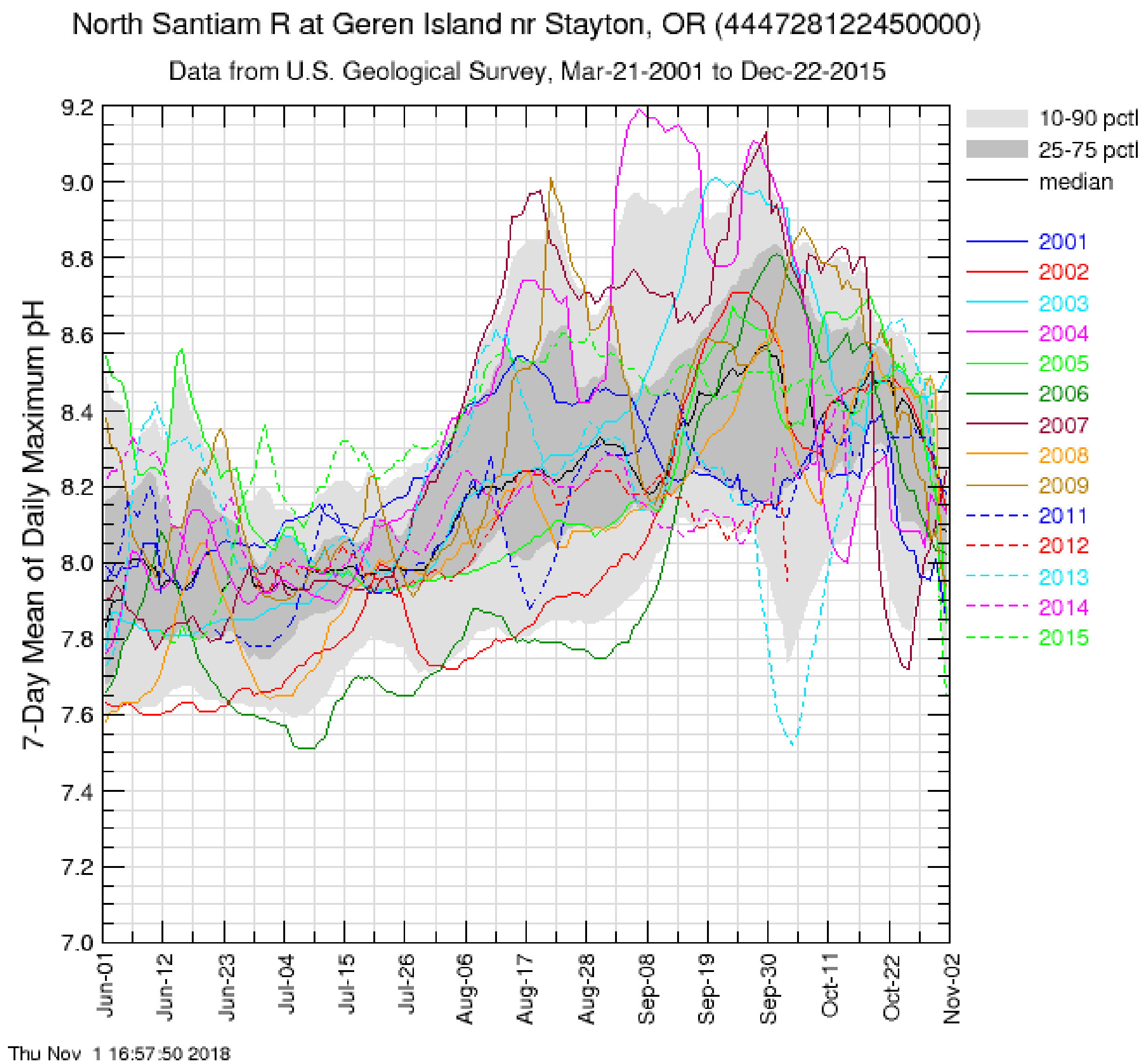


Figure 5-20. 7-day mean of daily maximum pH at Geren Island on North Santiam River, (USGS , 2018)

5.4.4 Drinking Water Impacts

The City of Salem and other communities withdraw raw drinking water from the North Santiam River. The City of Salem intake is near Stayton, approximately 27 miles downstream of Big Cliff Dam. High turbidity in the North Santiam River during the February 1996 flood caused the City of Salem to shut down its water-filtration system and use emergency water rations (Larson 2000). Increased turbidity is likely if Detroit Reservoir is drawn down during construction, but it is not known what turbidity concentrations would occur. It is possible that increased Total Suspended Solids from the reservoirs during a construction period drawdown could cover epiphyton, which could reduce plant productivity or result in DO levels in the North Santiam River downstream of the reservoirs. If an extended drawdown of the pool during construction results in less storage during the summer, the downstream flow targets may not be achieved. Lower flow in the river could lead to warmer temperatures and increased algal activity.

Risks after completion of the SWS include:

- Increased summer, peak temperatures which could lead to increased algal activity in the river and treatment ponds

- Although not expected, if there is an increase in the frequency and magnitude of blue-green algal blooms in the reservoir for any reason, it could lead to the production of toxins

5.5 REFERENCES

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SECTION 6 - STRUCTURAL DESIGN

6.1 GENERAL

This section describes design criteria, constraints, assumptions and analysis procedures that will be used in the structural design of the new SWS and its appurtenant features. This section also describes the evaluation of the existing dam monoliths affected by the new SWS. The SWS will ultimately operate integrally with a new FSS, which is a floating fish collection plant that provides juvenile downstream passage and will be moored adjacent to the SWS. The FSS is being designed concurrently with the SWS.

The SWS is a hollow, rectangular concrete structure that will be attached to the upstream face of the dam within the extents of the penstock monoliths, blocks 22 and 23. The total overall dimensions of the SWS are 108 ft wide, 40 ft deep, and approximately 369 ft tall. The SWS is divided into two mirrored halves at the expansion joint between blocks 22 and 23; each half is 54 ft wide, which is equal to the width of one block. The SWS is anchored to the upstream face of the dam for its entire height.

The internal layout of the tower is a generally open wet well for mixing water for temperature control downstream of the dam. The SWS will incorporate two different types of intakes to allow for temperature control: two (2) high intake weirs (HIW) and four (4) low intake gates (LIG). See Appendix A, Plate 4 for HIW and LIG locations.

The pair of HIWs are located on the east wall of the SWS with an opening of 20 ft wide and 157.5 ft tall (from elevations 1,412.5 to 1,570). The four LIGs have openings that are 15 ft wide and 10 ft tall. The bottom invert of the upper two LIGs are at elevation 1,327, and the bottom invert of the lower two LIGs are at elevation 1,305.

The flow out of the tower will be routed directly into the existing penstocks. The existing penstocks are to be bifurcated to maintain the ability to pass flows out of the tower while the units are not running. The new pipes are supported on the downstream face of the dam.

Trash racks for debris management will be provided in front of the HIWs, and the LIGs. Once the FSS is in place and operating, it will provide the debris management for flows into the HIWs, and the HIW trash racks will be removed. However, a trash rack structure will be required during times the FSS is in maintenance mode.

The top deck of the SWS will be accessible by the dam deck at elevation 1579. A fixed stair tower off the north face of the SWS will provide personnel access to and from the FSS. Boat access will also be provided from a location to be determined. The bridge is accessed from the dam’s south parking lot, and attaches to the top deck of the SWS. The bridge is a single-span with steel girders.

6.2 DESIGN STANDARDS AND REFERENCES

The structural design will conform to the following Engineering Circulars (ECs), EMs, ERs, Engineering Technical Letters (ETLs), Technical Manuals (TMs), and Industry Codes:

Aluminum Association (AA). 2010. Aluminum Design Manual.

American Concrete Institute (ACI). ACI 318-18, Building Code Requirements for Structural Concrete.

ACI. ACI 350-06, Code Requirements for Environmental Engineering Concrete Structures.

American Institute of Steel Construction (AISC). Steel Construction Manual (LRFD and ASD), 15th Edition.

American Society of Mechanical Engineers (ASME) ASME BTH-1-2014.

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USACE, EM 1110-2-2400, Structural Design and Evaluation of Outlet Works.

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USACE, EM 1110-2-6051, *Engineering and Design – Time-History Dynamic Analysis of Concrete Hydraulic Structures.*

USACE, EM 1110-2-6053, Earthquake Design and Evaluation of Concrete Hydraulic Structures.

USACE, EM 1110-2-6054, Inspection, Evaluation and Repair of Hydraulic Steel Structures.

USACE, ER 1110-2-1806, Earthquake Design and Evaluation for Civil Works Projects.

USACE, ER 1110-2-8157, Responsibility for Hydraulic Steel Structures.

U.S. Geological Survey (USGS). 2008. National Seismic Hazard Maps.

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Machinery’s Handbook, 28th Edition.

2009 AMEC Site Specific Seismic Study.

6.3 FOUNDATION DATA

Data for foundation rock and soil properties is detailed in Section 3, Geotechnical Design, of this DDR.

6.4 STRUCTURE CLASSIFICATION

6.4.1 Critical Structures

A critical structure is defined by ER 1110-2-1806 as, "structures, natural site conditions, or operating equipment and utilities at high hazard potential projects whose failure during or immediately following an earthquake could result in loss of life". The project hazard potential is classified according to Table B-1 from the aforementioned ER.

<u>Direct Loss of Life.</u> Direct loss of life is likely limited to personnel on the structure at the time of the earthquake and would most likely result from the collapse of the structure, or from falling overhead equipment. The performance criteria of all structures will be for damage control and collapse prevention. Overhead and large equipment must be seismically anchored to the structure. Therefore, there is no expected direct loss of life.

<u>Lifeline Losses.</u> The primary purpose of the SWS is downstream fish passage and temperature control of outflows downstream of the dam. The loss of these services would not result in loss of lifeline/essential services or indirect loss of life. The SWS is the main intake of the penstocks. The loss of these new intakes would result in inoperability of the turbine units, but this is considered an economic impact. With respect to flood control, all ROs and the gated spillway can be used to pass flows in the event that the SWS and/or the units are inoperable. Therefore, there is no threat of lifeline losses or indirect loss of life.

<u>Property Losses.</u> The property losses are considered to produce the most significant consequences in the event of a failure of the SWS or its appurtenant features. The ability to pass flow through the turbine units could be lost, requiring extensive repairs. Any repairs would likely require a partial or full drawdown of Detroit Lake, which would result in indirect economic impacts to the surrounding community. As such, there may be major or extensive consequences to public and private facilities.

<u>Environmental Losses.</u> Environmental losses are categorized by the impact downstream that may be caused by an incremental flood wave produced by a project failure beyond which would normally be expected. Flood control would be maintained by Detroit Dam and no incremental flood wave would be produced downstream of the dam. Therefore, this factor is categorized as minimal incremental damage.

Based on the evaluation of the expected losses, the hazard potential classification for this project is *Significant* (See Figure 6-1). This is largely influenced by the possibility of economic and property losses in the event of project failure.

The SWS structure is not classified as a critical structure since it is not a high hazard project. Detroit Dam and its existing features do have a high hazard potential and are classified as critical structures. The SWS will be designed as a non-critical structure. The gravity dam, monolith 22 and 23, combined with the attached SWS will be evaluated as a critical structure.

APPENDIX B

Table B-1
HAZARD POTENTIAL CLASSIFICATION
FOR CIVIL WORKS PROJECTS

Hazard Potential Classification	Category[1] Direct Loss of Life[2]	Lifeline Losses[3]	Property Losses[4]	Environmental Losses[5]
Low	None Expected	No disruption of services – repairs are cosmetic or rapidly repairable damage	Private agricultural lands, equipment, and isolated buildings	Minimal incremental damage
Significant	None Expected	Disruption of essential facilities and access	Major or extensive public and private facilities	Major or extensive mitigation required or impossible to mitigate
High	Probable (one or more)	Disruption of critical facilities and access	Extensive public and private facilities	Extensive mitigation cost or impossible to mitigate

[1] Categories are based upon project performance and are not applicable to individual structures within a project.

[2] Loss of life potential based upon inundation mapping of area downstream of the project. Analyses of loss of life potential should take into account the population at risk, time of flood wave travel, and warning time.

[3] Indirect threats to life caused by the interruption of lifeline services due to project failure or operation (*i.e.*, direct loss of (or access to) critical medical facilities).

[4] Direct economic impact of property damages to project facilities and downstream property and indirect economic impact due to loss of project services (*i.e.*, impact on navigation industry of the loss of a dam and navigation pool or impact upon a community of the loss of water or power supply).

[5] Environmental impact downstream caused by the incremental flood wave produced by the project failure beyond which would normally be expected for the magnitude flood event under which the failure occurred.

B-1

Figure 6-1. Hazard Potential Classification

6.4.2 Hydraulic Structures

The SWS is classified as a Reinforced Concrete Hydraulic Structure (RCHS).

Reinforced Concrete Hydraulic Structures are directly subjected to submergence, wave action, spray, icing or other severe climatic conditions.

The following structures are classified as Hydraulic Steel Structures (HSS):

- HIWs
- LIGs

The new penstock bifurcation is not considered an HSS. The design of these features is according to the American Society of Mechanical Engineers (ASME) Boiler and Pressure Vessel Code (BPVC).

6.5 SERVICE LIFE

6.5.1 SWS

The service life for this structure will be 100 years for major infrastructure projects in accordance with ER 1110-2-1806 Earthquake Design and Evaluation for Civil Works Projects.

6.5.2 HSS

The service life for all structures classified as HSS will be 100 years as required by ETL 1110-2-584, Design of Hydraulic Steel Structures.

6.5.3 Other Structures

The service life will be 50 years for all other miscellaneous structures not captured above.

6.6 SEISMIC DESIGN

6.6.1 Design Earthquakes

Design earthquakes will be considered as per ER 1110-2-1806 and EM 1110-2- 6053. Earthquake ground motions for the design and evaluation of USACE concrete hydraulic structures (CHSs) are OBE and Maximum Design Earthquake (MDE). Seismic forces associated with the OBE are considered unusual loads. Those associated with the MDE are considered extreme loads. Earthquake loads are to be combined with other loads that are expected to be present during routine operations.

The OBE is a level of ground motion that is reasonably expected to occur within the service life of the structure. This ground motion has a 50% probability of exceedance within its service life. An event with a return period of 144 years will be used for the OBE design earthquake.

The MDE is the maximum level of ground motion for which a structure is designed or evaluated. As a minimum, for other than critical structures, the MDE ground motion has a 10% chance of being exceeded in a 100-year period, (or a 1000-year return period). For critical structures, the MDE ground motion is the same as the maximum credible earthquake (MCE) ground motion. The SWS is not a critical structure, and therefore an event with a 1,000 year return period will be used for the MDE. The seismic evaluation of the existing dam, and the connections to the dam, are considered critical and will be designed for the MCE.

Seismic ground motion information available for design includes USGS data (2008 and 2014) and the results of a regional site specific study (2009). In addition, a new site specific study has been initiated by the USACE RMC. This new study will update and expand the 2009 ground site specific study by adding vertical ground motions and new time histories.

The regional seismic site specific study was completed in 2009 by AMEC-Quest. That study presents a regional seismic analysis that identifies and quantifies seismic hazards for 13 USACE dams in Oregon’s Willamette Valley, including Detroit Dam. The study used state-of-the-art methodologies and procedures for developing and selecting engineering design ground motion and response spectra. Seismic ground motions to be used for the seismic design at Detroit Dam are provided below (Tables 6-1, 6-2, and 6-3).

Table 6-1. Ground Motions Spectra Data

Spectral Period (sec)	Pseudo-Spectral Acceleration (g), 5% Damping, for:							
	UHRS for Return Period of:						Deterministic Motions from Cascadia Interface M 8.7 to M 9.1	
	144 years	500 years	1,000 years	2,500 years	5,000 years	10,000 years	Median	84th Percentile
PGA	0.0331	0.0947	0.1392	0.2187	0.2856	0.3616	0.1191	0.2356
0.075	0.0522	0.1433	0.2247	0.3557	0.4779	0.6194	0.1732	0.3708
0.1	0.0595	0.1676	0.2602	0.4094	0.5540	0.7263	0.1974	0.4244
0.2	0.0732	0.2145	0.3318	0.5248	0.6934	0.8900	0.2680	0.5654
0.3	0.0655	0.2010	0.3157	0.5015	0.6561	0.8402	0.2638	0.5522
0.5	0.0458	0.1605	0.2708	0.4391	0.5787	0.7462	0.2355	0.4912
1.0	0.0223	0.1007	0.1759	0.3053	0.4184	0.5470	0.1551	0.3443
2.0	0.0083	0.0452	0.0952	0.1723	0.2490	0.3408	0.0794	0.1990
4.0	0.0023	0.0142	0.0315	0.0596	0.0966	0.1322	0.0225	0.0735

Table 6-2. OBE and MCE Spectra Data (2009 Regional Study)

Spectral Period (sec)	Pseudo-Spectral Acceleration (g), 5% Damping, for: OBE Horizontal	OBE Vertical	Cascadia MCE Horizontal	Cascadia MCE Vertical
PGA	0.0331	0.0202	0.2356	0.1437
0.075	0.0522	0.0353	0.3708	0.2855
0.1	0.0595	0.0381	0.4244	0.3030
0.2	0.0732	0.0404	0.5654	0.3274
0.3	0.0655	0.0328	0.5522	0.2761
0.5	0.0458	0.0229	0.4912	0.2456
1.0	0.0223	0.0118	0.3443	0.1917
2.0	0.0083	0.0050	0.1990	0.1221
4.0	0.0023	0.0015	0.0735	0.0492

Table 6-3. Summary of Site Specific Design Parameters

	OBE	MDE	MCE
Encounter Probability	50% in 100 years	10% in 100 years	NA
Return Period	144 years	949 years	NA
PGA	0.0331g	0.139g	0.2356

6.6.2 Seismic Performance Criteria

Various performance levels are considered when evaluating the response of CHSs to earthquake ground motions. The performance levels commonly used are serviceability performance, damage control performance, and collapse prevention performance.

6.6.2.1 Serviceability Performance

Serviceability performance requires that the condition of all structures be serviceable and operable immediately following earthquakes producing ground motions up to the OBE level. The project should function without interruption and with little-to-no structural damage.

6.6.2.2 Damage Control Performance

Certain elements of the structure can deform beyond their elastic limits (non-linear behavior) if non-linear displacement demands are low and load resistance is not diminished when the structure is subjected to extreme earthquake events. Damage may be significant, but it is generally concentrated in discrete locations where yielding and/or cracking occurs. The designer should identify all potential damage regions and be confident that the structure is capable of resisting static loads and, if necessary, can be repaired to stop further damage by non-earthquake loads. Except for unlikely MCE events, it is desirable to prevent damage from occurring in substructure elements, such as piling and drilled piers, and other inaccessible structural elements.

All project features are expected to meet damage control performance objectives when subjected to an MDE event. Damage should be controlled to occur at or above the minimum flood pool (elevation 1450 ft).

6.6.2.3 Collapse Prevention Performance

Collapse prevention performance requires that the structure does not collapse regardless of the level of damage. Damage may be unrepairable. Ductility demands can be greater than those associated with the damage control performance. If the structure does not collapse when subjected to extreme earthquake events resistance can be expected to decrease with increasing displacements. Collapse prevention performance should only be permitted for extreme MCE events.

Critical project features, in addition to the serviceability and damage control performance objectives cited above, are expected to provide collapse prevention performance when subjected to an MCE event. Collapse prevention performance requires that critical project features do not collapse regardless of the level of damage.

6.6.3 Analysis Procedures

The seismic coefficient method is generally considered a preliminary analysis method and is typically used as a screening process for preliminary results. More accurate results are obtained by a Response Spectrum Analysis (RSA) which will be used in subsequent analyses as the design progresses. The structural response of the intake tower subject to earthquake loading has been preliminarily investigated using the response spectrum method.

6.6.4 Seismic Design Criteria

The seismic design of the SWS shall be in compliance with EM 1110-2-2104, ACI 318-14, EM 1110-6053 and other applicable criteria. The new SWS tower structure will not be considered a critical structure per ER 1110-2-1806. If the tower were to fail there are still lower RO and spillway gates that can regulate the forebay elevation and would not result in loss of life. The intake structure size and configuration is based on the inlets and collection wells proposed geometry, and the space and clearance requirements for the mechanical and electrical equipment. The objectives of effective structural planning is to maintain symmetry, minimize torsional effects, provide direct vertical paths for lateral forces, and provide a proper foundation.

Earthquake loadings generally govern the design of intake towers. Performance is considered acceptable if all brittle modes of failure are suppressed, and demand–to-capacity ratios are less than the allowable values.

The current design will be conducted as per EM 1110-2-2104 and ACI 318-14. Alternatively, cast-in-place CHSs may be designed using EM 1110-2-6053 with D/C ratios as listed in Table 6-1 of EM 1110-2-6053. The Demand Capacity Ratio (DCR) requirements for flexure limit the ductility demand to levels acceptable for lightly reinforced structures (Table 6-4). The DCR requirements for brittle modes of failure will suppress shear and sliding shear failures. Shear capacity for computation of DCR should be selected consistent with the level of displacement ductility demand associated with the peak flexural response.

Table 6-4. DCR Allowable Values for Reinforced Hydraulic Structures

Force	MDE
Flexure	2
Shear	1
Sliding Shear	1

Tower design requirements:

- Design as per EM 1110-2-2104 for the MDE since this is not a critical structure. Use 10% exceedance in 100 years which is a 949-year return event.

- Design as per EM 1110-2-2104 for the OBE.

Potential tower failure modes with regard to free standing intake towers are outlined in Figure 6-2.

The current SWS tower configuration is an attached configuration.

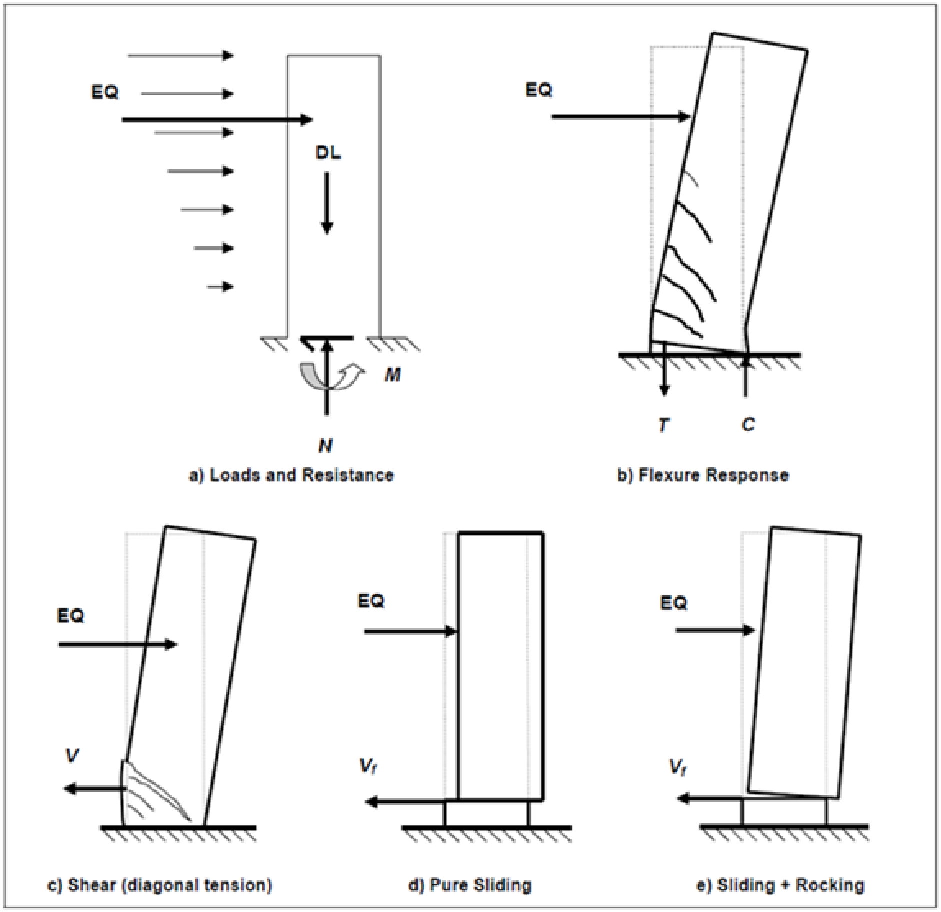

Figure 6-2. Potential Tower Failure Modes

6.6.4.1 Intake Tower Analysis

Reinforced concrete structures, such as intake towers, are commonly evaluated for earthquake ground motion effects using a linear-elastic response-spectrum analysis. Depending on the complexity of its geometry, an intake tower may be evaluated using a FEM model with frame or solid elements. The results for FEM models with frame elements are output as forces (moments, shears, and axial load) rather than stresses. However, the Detroit Dam tower is not a standard intake tower and the proposed construction method does not allow simple frame members to be used.

STAAD.pro was used to model the geometry of the proposed Detroit SWS. Solid elements were used to determine if the precast blocks will separate from each other during operation.

The results for FEM models using solid elements are output as element stresses, which must be converted into forces and moments at critical sections then compared with section capacities.

6.6.4.2 Acceptance Criteria

The earthquake load effects combined with the effects of dead and live loads are used to calculate total demands on the structure. SWS design using the 144-year and 1000-year ground motions should meet applicable criteria.

6.6.4.3 DCR <= Allowable Value

A demand to capacity comparison, utilizing a DCR as a performance indicator, establishes the basis for the design of reinforced concrete structures. For reinforced concrete structures, DCR is defined as the ratio of force-or-moment–demand to force-or-moment-capacity.

The ultimate strength U, or capacity, will be determined using the principles and procedures described in EM 1110-2-2104. Capacities are based on ultimate strength, or the nominal strength multiplied by a capacity reduction factor. The capacity reduction factor, as per ACI 318, is generally 0.9 for bending and 0.75 for shear.

The SWS will be designed such that DCR <= 1 as per EM 1110-2-2104 and ACI 318-14.

The DCR values for CHSs designed or evaluated using performance based design as outlined by EM 1110-2-6053 are provided in Table 6-4 (refer to page 6-10).

6.6.4.4 Brittle Modes of Failure Evaluation

To meet performance requirements, all brittle modes of failure should be suppressed. Brittle failure mechanisms include shear failure, reinforcing steel anchorage failure, and reinforcing steel splice failure, for which the structure should respond elastically. Flexural failures are generally considered to be ductile failures. Performance is considered acceptable provided all brittle modes of failure are suppressed and demand-to-capacity ratios are less than the allowable values per code. Brittle modes of failure are considered to be force-controlled actions (FEMA 356, 2000). For force-controlled actions, the capacity (nominal or ultimate strength) of the member at the deformation level associated with maximum flexural ductility, the demand must be greater than the force demands caused by earthquake, dead or live loads. The DCR should be equal to or less than one.

The following brittle modes of failure should be subject to investigation.

- Shear
- Sliding shear failure

- Reinforcing splice failure
- Reinforcing anchorage failure
- Compressive spalling failure
- Fracture of tensile reinforcement

6.6.4.5 Shear Capacity

The shear capacity of the SWS will be determined in accordance with the following equation:

$$\phi V_n = \phi V_c + \phi V_s$$

The shear strength Vn provided by concrete (Vc) and reinforcement (Vs) shall be computed in accordance with ACI 318.

6.6.4.6 Shear Friction

Shear friction may be checked with the following equation:

$$V_{SF} = \mu_{SF} (P + 0.25 A_s f_y)$$

Where:

V_{sf} = Sliding shear capacity or shear friction capacity, lb
μ_{SF} = Shear friction coefficient, as per ACI 318 = 1.0
P = Axial load on section, lb
As = Area of the longitudinal reinforcing across the potential failure plane, in^2.
fy = Yield strength of reinforcing steel, psi

6.6.4.7 Reinforcing Splice Failure

Reinforcing splice lengths will meet the requirements of ACI 318-14.

6.6.4.8 Reinforcing Anchorage Failure

Reinforcing anchorage will meet the requirements of ACI 318-14. The minimum anchorage length should not be less than 30 bar diameters for straight anchorages or less than 15 bar diameters for hooked anchorages.

6.6.4.9 Compressive Spalling Failure

With the use of prestressing strands there is a high likelihood of compression spalling around the anchor head; industry standard detailing of rebar around the anchor heads, anchor spacing and edge distance will be followed to ensure this does not occur.

6.6.4.10 Fracture of tensile reinforcement

Fracture of reinforcing steel can be prevented if sufficient flexural reinforcing steel is provided to produce a nominal-moment strength equal to, or greater than, 1.2 times the cracking-moment capacity of the section.

Sufficient reinforcing steel should be provided to assure that the nominal-moment capacity equals or exceeds 120 percent of the cracking-moment (EM 1110-2-6053).

With regard to the seismic design, sufficient reinforcing should be provided to assure that the nominal-moment capacity equals or exceeds 120 percent of the cracking-moment.

$Mn < 1.2\ Mcr$

6.6.4.11 Flexural Capacity

The construction method of precast blocks will not allow the tower to act as a traditional cast-in-place concrete tower. The flexural capacity will depend on the pre-stressing strands' ability to keep all the blocks compressed while under a hydrodynamic earthquake load or a differential hydrostatic loading condition.

6.6.4.12 Three Dimensional Solid Element Model

The 3-D analysis model was created in STAAD.pro and is comprised of solid elements. A 3D solid element model was developed as shown in the following Figures 6-3 and 6-4. Boundary conditions consist of compression springs and tension-only anchors.

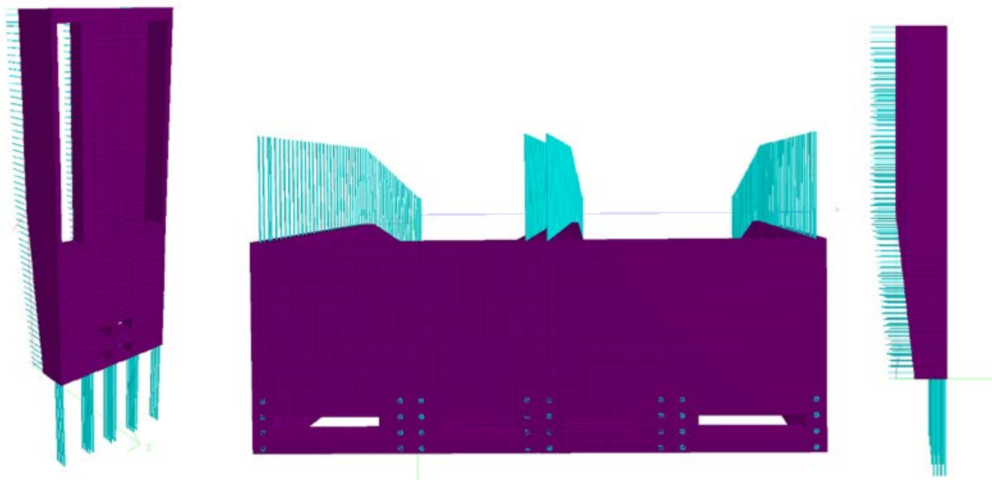

Figure 6-3. Conceptual 3D Solid Model

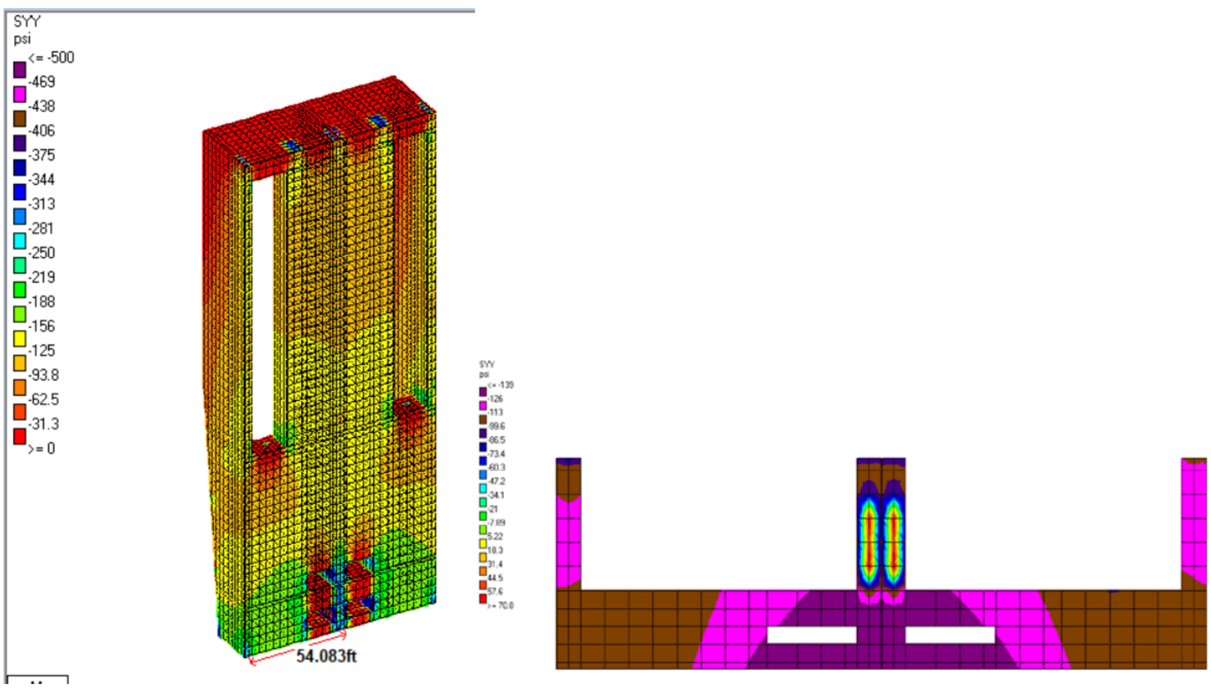

Figure 6-4. Typical Vertical Stress Plot (psi)

Model Assumptions:

Height = 279 ft (above base)
Wet well wall thickness = 4’ ft (N/S)
Wet well wall thickness = 13’-0” feet (E/W)
Top deck thickness = 5.0 feet
Mass concrete base = compression only supports
Tower above base = 108’x40’

The SWS should be additionally evaluated with regard to local effects due to usual, unusual, and extreme loading. Earthquake ground motion in both the upstream/downstream direction and cross canyon direction will be considered.

The following structural issues should be considered with respect to the 3D model and the structural design:

- Long vertical span lengths
- Long horizontal projection from the face of the dam
- Significant hydrodynamic added-mass requirements
- Significant vertical openings required for warm water intake gates
- Vertical wall segments between gate openings may require structural support
- Openings near the base of the tower

6.6.4.13 Serviceability Load Combinations

For the OBE

U = I.5 (D + L + Hs) +I.5E

where:

U = value of thrusts, shears, or moments due to the effects of dead load, live load, and earthquake

D = internal forces from self-weight

L = internal forces from live loads

Hs = internal forces from hydrostatic loading

E = internal forces from the OBE

6.6.4.14 Strength Load Combinations

For the MDE (1000 year)

where:

U = value of thrusts, shears, or moments due to the effects of dead load, live load, and earthquake

D = internal forces from self-weight

L = internal forces from live loads

Hs = internal forces from hydrostatic loading

E = internal forces from the MDE

U = D + L + Hs + E

6.7 LOADING SEISMIC EVALUATION OF EXISTING DAM

6.7.1 General

Attaching the SWS to the face of the dam adds seismic mass to monoliths 22 and 23. The added mass consists of the SWS structure and ancillary components, enclosed water and the tremie concrete foundation block. In a seismic event, the added mass could increase seismic inertial forces on the dam and create a potential dam safety concern. The increase in seismic mass is partially offset by the removal of the existing concrete trash rack structure at monoliths 22 and 23.

A seismic stability analysis was performed to evaluate the effects of the added seismic load on the sliding stability, overturning stability, bearing pressure, and stress in intake monoliths 22 and 23. The monoliths must meet pre-established stability criteria laid out in USACE EMs and ERs. In the event these criteria are not satisfied, measures must be taken to improve the stability and strength of the dam.

6.7.2 Input Parameters and Assumptions

Foundation, concrete, and dimensional parameters were based on as-built drawings and material records. Since the accuracy of these sources cannot be guaranteed, conservative assumptions were made where possible.

6.7.2.1 Load Cases

Monolith stability was evaluated with added SWS mass (concrete plus enclosed water) for the seismic load cases listed below. The majority of the SWS's vertical gravity load is transferred into the foundation through the tremie concrete block and does not significantly affect static stability. The lateral seismic load of the added mass is a contributor to stability and stress.

EM 1110-2-2100, 4-7a requires that the seismic loads be combined with the 50% coincident pool (elevation 1,531.4 ft.), defined as the elevation that the water is expected to be at or below for half of the time during each year. However, to be conservative for this analysis the seismic loads were combined with the 90% pool (elevation 1,563.8 ft.).

The following load cases were considered:

LC1 (Unusual) - 90% Pool, OBE

- Dead load of structure
- Reservoir at elevation 1563.8
- Water surface in SWS at normal head difference
- Full Uplift
- OBE Design Earthquake

LC2 (Extreme I) – 90% Pool, MCE

- Dead load of structure
- Reservoir at elevation 1563.8
- Water surface in SWS at normal head difference
- Full Uplift
- MCE Design Earthquake

LC3 (Extreme II) – 90% Pool, 10,000 yr. Earthquake

- Dead load of structure
- Reservoir at elevation 1563.8
- Water surface in SWS at normal head difference
- Full Uplift
- 10,000-year Design Earthquake

6.7.2.2 Seismic Parameters

Since the dam is a critical structure, it must meet the stability requirements defined in Section 6.10.2 for the OBE and MCE (EM 1110-2-2100, 4-7a). The seismic event with a 10,000-year return period is evaluated for potential future risk assessment purposes.

Each design earthquake was applied in the upstream and downstream directions. Stability for a cross-canyon earthquake was not evaluated. The added mass will have negligible effects on cross-canyon stability since adjacent monoliths act as a system, limiting cross-canyon movement. Vertical acceleration was considered, but its effects were found to be negligible.

It is assumed that seismic load does not transfer across monolith joints due to the lack of reinforcement across the joints. In reality, adjacent monoliths would act as a system, providing additional overturning and sliding resistance through shear friction at the joints.

6.7.3 Evaluation Methodology

Seismic stability and stress distribution is evaluated by a stepped approach in accordance with EM 1110-2-2100, Section 7-2. Evaluation of an existing dam should begin with a preliminary two-dimensional analysis. The initial phase should use parametric studies to assess the impact of each input parameter on factors of safety. If the structure meets USACE safety and performance objectives, then no further analysis is required. If these objectives are not met, more comprehensive analyses should be performed to include more accurate investigation of input parameters, as well as more complex analytical methods.

For this analysis three methods will be used, listed in order of increasing complexity and accuracy:

- 2D seismic coefficient method (pseudo-static/gravity method)
- 2D pseudo-dynamic simplified response spectrum method
- 3D RSA using a finite element model (FEM)

6.7.3.1 Seismic Coefficient Analysis

The seismic coefficient method has traditionally been used to evaluate the seismic stability of structures. This method may be used in the preliminary design and stability analyses. The seismic coefficient method is a cursory seismic analysis. Earthquake loading is treated as an inertial force applied statically to the structure through the center of gravity. Two types of loads are applied to the dam: inertia force due to the horizontal acceleration of the dam and hydrodynamic forces resulting from the reaction of the reservoir water against the dam. The magnitude of the inertia forces is computed by the product of mass and the seismic coefficient. The magnitude of the seismic coefficient is taken as a fraction of the peak ground acceleration expressed as a decimal fraction of the acceleration of gravity. The Hydrodynamic forces are computed using Westergaard's formula (EM 1110-2-2100, Eqn. 4-2). The monolith is assumed to be a 2D rigid body with 5% damping, neglecting dynamic amplification from monolith flexibility.

Sliding and overturning stability were evaluated in accordance with EM 1110-2-2100, Section 3-7 and 3-8 respectively, using two-thirds of the effective peak ground acceleration (2/3 EPGA), per Section 4-7b. The stress in the monolith was calculated at each lift line using linear-elastic beam theory. The peak ground acceleration (PGA) was used for stress determination.

The seismic coefficient method was used to evaluate global stability (ref. EM 1110-2-2100, 3-2 h.1.b) and to provide a preliminary estimate of stress to determine if post-tensioned (PT) anchors or other mitigation measures would be needed. Since the seismic coefficient method is not as reliable for calculating stress, more advanced methods should be used.

A full description of the seismic coefficient analysis and results can be found in Appendix B.

6.7.3.2 Pseudo-Dynamic Analysis

The pseudo-dynamic analysis is based on the simplified response spectra method as described by Chopra (1988). The pseudo-dynamic method is a RSA procedure, which estimates the peak response directly from the earthquake design spectrum. This analysis procedure includes the effects of dam-water-foundation interaction, known to be important in the earthquake response of dams. A pseudo-dynamic analysis is conceptually similar to a seismic coefficient analysis except that it recognizes the dynamic amplification of the inertia forces along the height of the dam. The oscillatory nature of the amplified inertia forces is not considered. The stress and stability analyses are performed with the inertia forces continuously applied in the same direction. The fundamental period of vibration and total damping are computed based on geometric and material properties, as well as consideration of the reservoir-dam-foundation interaction. The spectral acceleration is determined based on the fundamental period of vibration and total system damping.

A computer program, CADAM (Computer Analysis of Dams), was used for the pseudo-dynamic analysis. This program was developed by the Ecole Polytechnique de Montreal, Canada to perform 2D static, pseudo-static and pseudo-dynamic analyses of concrete gravity dams. Figure 6-5 shows an example of the user interface with the Detroit Dam model. The program models the monolith, concrete properties, reservoir and tailwater elevations, silt elevation, lift lines, post-tensioning, drainage galleries, added mass, applied forces, and foundation properties. CADAM reports the sliding and overturning factors of safety as well as lift line stress. Total response moments, forces and stresses are computed by adding static response, equivalent static response for fundamental vibration mode and equivalent static response for higher vibration modes. Rigid body linear-elastic beam theory is used to calculate forces, moments and stresses.

The CADAM program performs the pseudo-dynamic seismic evaluation in two phases:

(a) Stress analysis using peak spectral acceleration values

(b) Stability analysis using sustained spectral acceleration values

Since the pseudo-dynamic method does not recognize the oscillatory nature of seismic loads, it is generally accepted to perform the stability calculation using sustained acceleration values taken as 0.67 to 0.5 of the peak acceleration values. The sliding safety factors are computed considering crack lengths determined from the stress analysis.

A full description of the pseudo-dynamic analysis and results can be found in Appendix B.

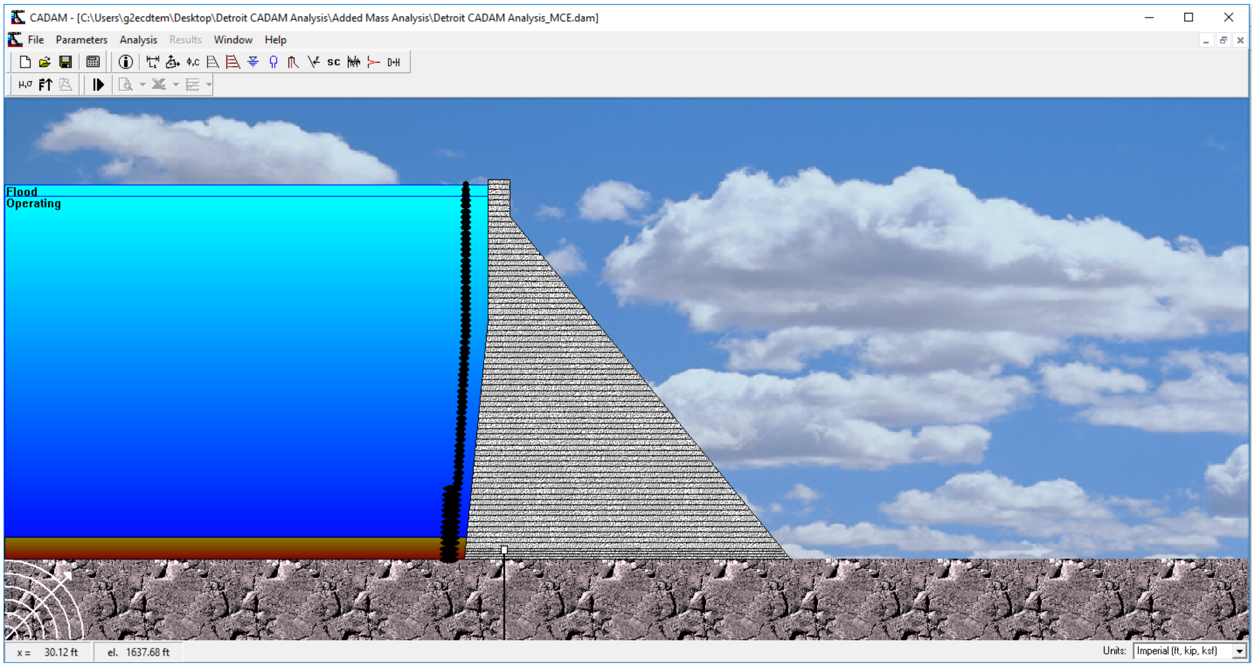

Figure 6-5. CADAM user interface showing pseudo-dynamic analysis of the dam

The CADAM pseudo-dynamic analysis provides an accurate computation of lift line stress at the upstream and downstream faces of the dam in areas where the monolith is uniform. For sections of dam with openings, galleries, and discontinuous geometry, the pseudo-dynamic method is less reliable because CADAM cannot model openings and galleries. For this reason, a 3D finite element RSA should be performed to accurately determine stress in these areas.

6.7.3.3 3D FEM RSA

Where applicable, linear elastic RSA will be used to conduct the evaluation. The linear elastic response spectrum method uses modal superposition dynamic analysis to determine the structural response. A 3D RSA using FEM software can provide a more accurate evaluation of monolith stress than the beam theory model used in CADAM. This is particularly useful at galleries, openings and in the chimney section. The RSA procedure estimates the spectral response accelerations from the earthquake design spectrum based on the significant modes of vibration. The number of modes required varies for each analysis. All modes with significant contribution to the total response of the structure will be included. Usually the number of modes is considered adequate if the total mass participation of the modes used in the analysis is at least within 90% of the total mass of the structure. The analysis model will consider a portion of the foundation as well as hydrodynamic loading using the hydrodynamic added-mass method. ANSYS, a finite element software program, will be used to perform this analysis for the 90% DDR review. The results will be used to determine final seismic mitigation measures.

6.7.3.4 Linear Elastic Time History Analysis

Linear elastic time history analysis may be required based on the findings of the more simplified analyses discussed above. If so, this analysis will be performed for the 90% DDR review.

6.7.4 Evaluation Results

6.7.4.1 Seismic Coefficient Results

a) OBE – Unusual Load Case

For the OBE, the dam is globally stable for sliding and overturning. All lift lines are individually stable for sliding and the stresses do not exceed the allowable. The seismic coefficient analysis indicates that no seismic mitigation measures are required for the OBE.

b) MCE – Extreme Load Case

For the MCE, the dam is globally stable for sliding and overturning. All lift lines are individually stable for sliding and the stresses do not exceed the allowable. The seismic coefficient analysis indicates that no seismic mitigation measures are required for the MCE.

c) 10,000-year Seismic Event – Extreme Load Case

For the 10,000-year event, the dam is globally stable for sliding and overturning. For ground motion in the upstream direction, the monolith is stable at lift lines and the stress is less than allowable. For downstream ground motion, the monolith is unstable for overturning and sliding at lift lines between elevations 1,524 ft and 1,529 ft, but stable at all other lifts. The initial seismic coefficient analysis indicates that PT anchors are required for the 10,000-year event.

d) Summary

The following tables (Table 6-5 and Table 6-6) summarize the findings of the seismic coefficient analysis. Detailed analysis and results can be found in Appendix B.

Table 6-5. Summary of seismic coefficient analysis global stability

		SLIDING			OVERTURNING			BEARING PRESSURE	
Seismic Event	Ground Motion Direction	Stable?	Factor of Safety	Allowable FS	Stable?	% Base in Compression	Required Base in Compression	Maximum Bearing Pressure, ksf	Allowable Bearing Pressure, ksf
OBE	U/S	YES	3.54	1.3	YES	100%	75%	35.6	389
	D/S	YES	4.00	1.3	YES	100%	75%	13.68	389
MCE	U/S	YES	2.03	1.1	YES	YES - 67%	Resultant w/in base	55.7	389
	D/S	YES	7.30	1.1	YES	YES - 100%	Resultant w/in base	27.0	389
10,000	U/S	YES	1.37	1.1	YES	YES - 40%	Resultant w/in base	91.0	389
	D/S	YES	15.5	1.1	YES	YES - 100%	Resultant w/in base	36.17	389

Table 6-6. Summary of seismic coefficient lift line stress and stability

		SLIDING				STRESS			
Seismic Event	Ground Motion Direction	Stable?	Min. Factor of Safety	Allowable FS	Unstable Lift Location	Max Compressive Stress, PSI	Allowable Comp. Stress, PSI	Max Tensile Stress, PSI	Allowable Tensile Stress, PSI
OBE	U/S	YES	3.13	1.3	n/a	164	1500	< 0	125
	D/S	YES	3.07	1.3	n/a	106	1500	< 0	125
MCE	U/S	YES	1.91	1.1	n/a	255	2700	96.7	312
	D/S	YES	6.19	1.1	n/a	274	2700	65.5	312
10,000	U/S	YES	1.20	1.1	n/a	528	2700	234	312
	D/S	NO	< 1.1	1.1	EL 1524' - EL 1529'	355	2700	115	312

6.7.4.2 Pseudo-Dynamic Results

a) OBE – Unusual Load Case

For the OBE, the dam is stable in sliding and the stress at lift lines is below allowable for both upstream and downstream ground motion. The pseudo-dynamic analysis indicates that no seismic mitigation measures are required for the OBE.

b) MCE – Extreme Load Case

For the MCE, the dam is stable in sliding and the stress at lift lines is below allowable for both upstream and downstream ground motion. The pseudo-dynamic analysis indicates that no seismic mitigation measures are required for the MCE.

c) 10,000-year Seismic Event – Extreme Load Case

For the 10,000-year event, the dam is unstable for sliding at lift lines between elevations 1,530 ft and 1,275 ft with upstream ground motion. The dam is stable for sliding with downstream ground motions. For ground motion in the upstream direction, tensile stress in the upstream face exceeds allowable at lift lines between elevations 1,235 ft and 1,545 ft. For ground motion in the downstream direction, tensile stress in the downstream face exceeds allowable at lift lines between elevations 1,420 ft and 1,545 ft. The initial pseudo-dynamic analysis indicates that PT anchors are required to mitigate lift line stress for the 10,000-year event. This conclusion will be further evaluated during the 90% DDR phase.

d) Summary

Table 6-7 summarizes the findings of the pseudo-dynamic analysis. Detailed analysis and results can be found in Appendix B.

Table 6-7. Summary of pseudo-dynamic lift line stress and stability

		SLIDING				STRESS			
Seismic Event	Ground Motion Direction	Stable?	Min. Factor of Safety	Allowable FS	Unstable Lift Location	Max Compressive Stress, PSI	Allowable Comp. Stress, PSI	Max Tensile Stress, PSI	Allowable Tensile Stress, PSI
OBE	U/S	YES	2.11	1.3	n/a	265	1500	37.9	125
	D/S	YES	2.54	1.3	n/a	210	1500	12.4	125
MCE	U/S	YES	1.27	1.1	n/a	451	2700	267	312
	D/S	YES	2.34	1.1	n/a	308	2700	250	312
10,000	U/S	NO	0.91	1.1	EL 1530' - EL 1275'	574	2700	431	312
	D/S	YES	1.34	1.1	n/a	471	2700	409	312

6.7.4.3 3D FEM Response Spectrum Results

A preliminary dynamic analysis of a typical monolith was performed using the linear elastic response spectrum method. The analysis model included a simplified non-overflow monolith approximately representing monolith 22, the attached SWS structure and the tremie concrete foundation.

The following element types were used to perform the analysis.

- Solid187 – Higher order 3D element
- Mass21 - Point mass element

Boundary conditions initially assumed a fixed base at elevation 1230.0 ft. The 3D analysis of a typical monolith will be extended to include the foundation. Foundation dimensions will be set as approximately 1.5 times the monolith height deep and 3 times the monolith height long.

Hydrodynamic effects were incorporated into the FEM by applying added-mass to nodes along the upstream face of the SWS. Additionally, hydrodynamic effects due to enclosed fluid (within the SWS) will be considered.

Additional analysis of detailed structural models will be performed for the 90% DDR review. The following load cases will be considered:

a) OBE – Unusual Load Case

b) MCE – Extreme Load Case

c) 10,000-year Seismic Event – Extreme Load Case

The FEM was used with respect to the seismic evaluation. Version 17.1 of the ANSYS program was used to conduct this analysis. ANSYS software provides a comprehensive solution for structural linear, nonlinear and dynamic analysis.

6.7.5 Seismic Retrofit Design

Based on preliminary analysis, seismic retrofits are not required to stabilize the dam with the attached SWS for the OBE and MCE events.

If the dam is to be stabilized for the 10,000-year event, PT anchors are recommended to be drilled and grouted vertically from the intake gate machinery room at elevation 1,569 ft and diagonally from the upstream face of the dam, as shown in Figure 6-6 below. The type, layout and embedment of PT anchors is preliminary and non-optimized, but intended to provide a concept for reducing tensile stress and improving sliding stability at lift lines. Preliminary anchor design calculations can be found in Appendix B.

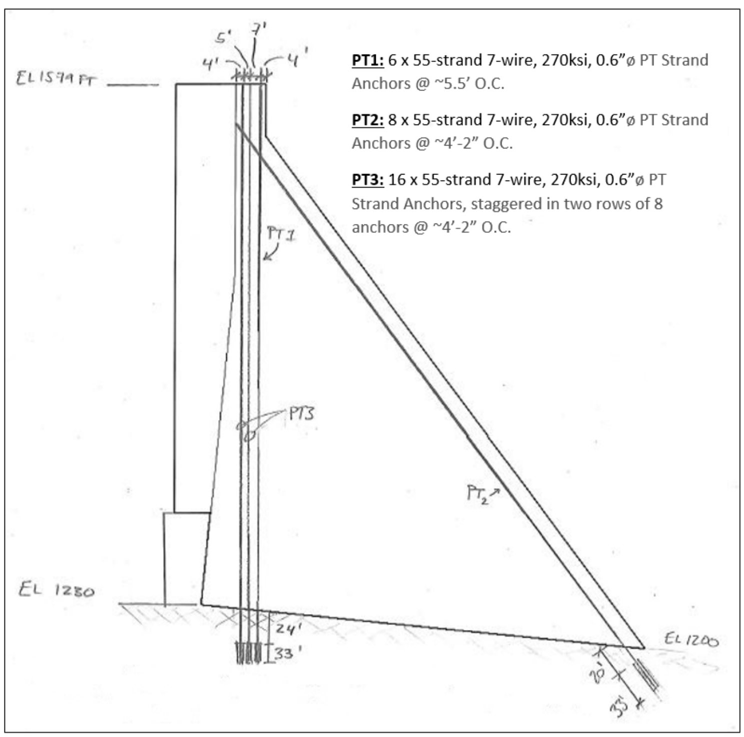

Figure 6-6. Typical section through intake monolith showing preliminary PT anchor layout to stabilize the dam for the 10,000-yr earthquake

It is anticipated that PT anchors would be challenging from both the design and construction perspectives. The anchors would need to be positioned to avoid galleries and the penstock conduit and be drilled and embedded to depths in excess of 400 ft as shown below in Figure 6-7. Drilling equipment would be used in tight areas in the machinery room and would be supported over the forebay to drill diagonal anchors from the upstream face.

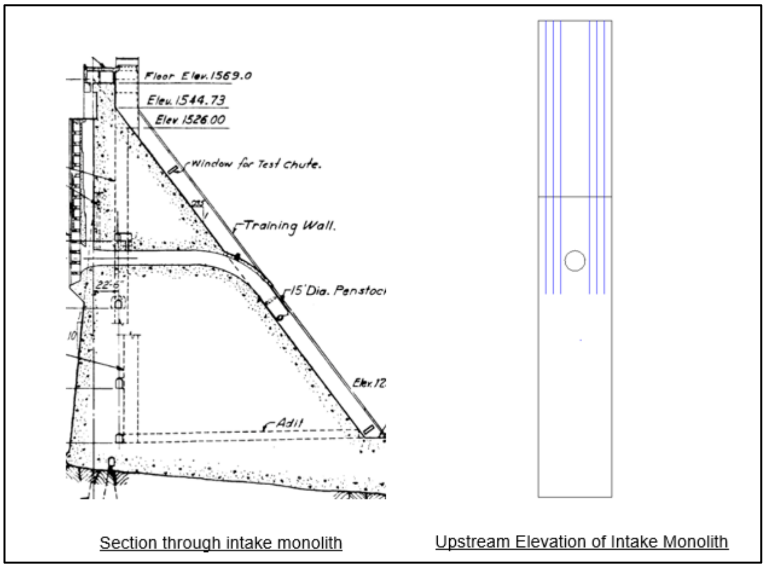

Figure 6-7. Section and elevation demonstrating need for PT anchors to avoid penstock conduit and galleries

6.7.6 Conclusions

Seismic coefficient analysis and pseudo-dynamic analysis indicate that intake monoliths 22 and 23 are stable for the MCE and the OBE. Peak seismic stresses are also below allowable stress limits for the OBE and the MCE.

Seismic coefficient analysis indicates the potential for sliding instability in the chimney section of the dam for the 10,000-year seismic event. Pseudo-dynamic analysis indicates sliding instability at lift lines and peak seismic tensile stress in excess of the allowable limits for the 10,000-year seismic event. It is expected that anchors and other seismic mitigation measures will be needed if the 10,000-year event is used for final design.

6.7.7 Recommended Further Analysis

The following recommendations will be addressed in the 90% DDR review:

a) Perform 3D RSA using FEM software.

b) Use FEM software to evaluate stress concentrations around galleries, chimney, discontinuous geometry and machine room.

c) Include added mass from machinery and machinery building in future analysis.

d) Investigate the seismically related stress at multiple sections along the length of monoliths 22 and 23 since cross section varies along the length.

e) Consult a contractor or designer with experience in PT anchor design to determine feasibility of anchoring the dam and to optimize design.

6.8 DESIGN LOADS

Loads will be categorized into three load categories:

- Usual: The Usual loading category represents daily or frequent operational conditions that require highly reliable performance. The design criteria for the usual loading category apply to load cases with the predominant load (or joint loads) having a mean return period (Tr) between 1 and 10 years.

- Unusual: The Unusual loading category represents infrequent operational conditions that require a defined level of performance, and that can be reasonably expected to occur within the service life of the project. The design criteria for the unusual loading category apply to load cases with the predominant (or joint loads) having a mean return period (Tr) between 10 and 300 years.

- Extreme: The Extreme loading category represents possible conditions that are not likely to occur within the service life of the project. The design criteria for extreme load cases are applicable if the predominant load (or joint loads) has a mean return period (Tr) greater than 300 years.

6.8.1 Dead Loads

Dead loads consist of the weight of concrete, metal, and fixed equipment. Concrete unit weight is assumed to be 150 lb/ft^3. Steel unit weight of 0.283 pounds per cubic inch is based upon AISC values for structural plates and shapes.

6.8.2 Live Loads

Live loads for any maintenance platforms, catwalks, sidewalks, etc., will be in accordance with ASCE 7 and relevant EMs, but not less than 100 psf as shown below in Table 6-8. Live loads will not be reduced.

Table 6-8. Live Loads

Type	Uniform Load (psf)	Concentrated Load (lbs)
Intake Structure Deck	500	HL-93 / Mobile Crane
Access Bridge	500	HL-93 / Mobile Crane
Walkways/elevated platforms	100	300

Stairways	100	300

6.8.3 Handrail and Guardrail Loads

All handrails and guardrails must be designed to resist a single 200-lb concentrated load applied in any direction or a 50-lb per linear foot distributed load along the top rail in any direction. Guardrails that act as a vehicle barrier shall be designed to resist a single 6,000-lb load applied horizontally in any direction to the barrier system at a height between 1 ft 6 in and 2 ft 3 in, whichever produces the maximum load effects.

6.8.4 Vehicle Loads

The vehicle loads considered in the design of the access bridge and the intake structure deck will be the HL-93, as defined by AASHTO, and a mobile crane used for operations and maintenance activities.

- HL-93. The HL-93 is the combination of the AASHTO-defined design truck or design tandem, whichever governs, and a 640 plf lane load.

- Mobile Crane. The Willamette Valley Project owns and operates an 80-ton Liebherr Mobile Crane (Model LTM 1070-4.2, Figure 6-8). This crane will be used as the mobile crane load. The crane's capacity exceeds what is required to perform all anticipated construction and maintenance operations. It would be overly conservative and uneconomical to design for the maximum capacity; only loads from the anticipated operations will be used during the design.

Figure 6-8. Project Crane

6.8.5 Hydrostatic Loads

Where applicable, the hydrostatic loads on the structure include internal and external pressures for all design load conditions. The unit weight of water is assumed to be 62.4 lbs/ft^3. The head drop between the pool and wet well is 3 to 6 ft for normal operations and a normal design head difference of 12 ft and an extreme design head of 19.2 ft. This design head is from Section 4 of this DDR.

6.8.6 Uplift

Uplift at the base of the hydraulic structure is assumed to be 100 percent of the adjacent river pressure over 100 percent of the base area. At internal planes, uplift is assumed to vary linearly from hydrostatic head at the external surface of a hydraulic structure to the hydrostatic head at any internal surface. Uplift pressures are assumed to remain unchanged during an earthquake.

6.8.7 Wind Loads

The wind loads are developed using ASCE 7 as modified by the IBC/OSSC. Wind loads for the Main Wind Force Resisting System (MWFRS), and other structures and appurtenances, will be developed using the Directional Procedure. The wind load is developed using the parameters listed in Table 6-9.

Table 6-9. Wind Load Parameters

Parameter	Value	Notes
Risk Category	I	Table 1604.5, OSSC
Basic Wind Speed, V	115 mph	Figure 1609C, OSSC
Wind Directionality Factor, K_d Main Wind Force Resisting System Other Structures and Appurtenances	 0.85 0.95	
Exposure Category	D	
Topographic Factor, K_{zt}	1.0	
Enclosure Classification	Varies	
Velocity Pressure Coefficient, K_z	Varies	Table 27.3-1, ASCE 7
Velocity Pressure, q_z Main Wind Force Resisting System Other Structures and Appurtenances	 31.1 psf 34.7 psf	Equation 27.3-1, ASCE 7
Combined Net Pressure Coefficient, GC_{pn}	Varies	Per ASCE 7

6.8.8 Snow Loads

Snow loads are developed using ASCE 7 as modified by the IBC and/or OSSC, with reference to the publication Snow Load Analysis for Oregon (SEAO, January 2011). According to the updated 2011 Snow Load Tables, the 50-year snow load at Detroit Dam is 28 psf. This is set as the site-specific ground snow load, p_g. Since the ground snow load is greater than 20 psf, the 5 psf rain-on-snow surcharge load is not applied. The pertinent snow load parameters are summarized in Table 6-10 below. Roof snow loads shall not be less than the ground snow load.

Table 6-10. Snow Load Parameters

Parameter	Value	Notes
Ground Snow Load, p_g	28 psf	
Exposure Factor, C_e	1.0	Terrain Category D, assume sheltered exposure
Thermal Factor, C_t	1.2	Unheated / Open Air Structure
Importance Factor, I_s	0.8	Risk Category I
Sloped Roof Factor, C_s	1.0	

6.8.9 Operation, Maintenance, Construction, and Temporary Loads

Cranes, trucks, boats, barges, and other maintenance and construction equipment loads will be considered. All decks, rails, and other structural components affected by these loads will be designed in accordance with the proper codes to meet design loads.

6.8.10 Debris, Silt, and Trash Loads

Debris, silt and trash loading will be a factor in some loading conditions and will need to be further defined. Trash racks will be designed to keep all larger debris from entering the SWS through the weir gate openings. Floating trash racks are being investigated as a possible solution to allow travel in unison with the top weir gates. Maintenance and cleaning without the need for an automated raking system are a design necessity. The LIGs will have a portable trash rack system that is removable when cleaning or when maintenance becomes necessary.

Historically, woody debris has been collected inside the restrictive boom area in front of the penstocks at monoliths 22 and 23. This is according to the Detroit Lake Periodic Inspection No. 10 report, created by CENWP-ENC-HC. A recent bathymetry survey of the area directly upstream of the dam shows build-up of what is most likely a combination of woody debris and silt. The actual volume of debris at the location of new construction is unknown and will need to be investigated further either during P&S or as a requirement for the Contractor. Conservative lateral pressures have been assumed at the base of the structure based on the existing height of the debris.

The debris and trash loading is assumed to be 1.0 kips per foot, which is applicable for all Willamette Valley Projects. This load has been used at various projects, including Big Cliff Dam Tainter Gate Repair, and is based on a log connecting with a structure at a certain current velocity. This load is expected to act at the top of the surface water elevation as the majority of the debris and trash will be floating at the surface.

6.8.11 Wave Loads

The wave load is a 3-ft additional surcharge. The calculations were performed as part of the conceptual and feasibility studies. This was calculated with a nominal 3 second wind gust speed at 33 ft above the water surface. A survival wind speed of 95 mph was specified in the Marion County area and was used in conjunction with the fetch length that was found at 2.3 miles. The calculations are included in the Structural Design Appendix B. The wave force is applied as an additional load to the hydrostatic load.

6.8.12 Ice Loads

According to historical data and USACE Detroit Dam periodic inspection reports, ice load is not applicable for this structure.

6.8.13 Machinery Loads

Machinery loads from Mechanical Design are for gate hoisting equipment and other maintenance hoists. Machinery loads will be updated as the design progresses.

6.8.14 Seismic Loads

Seismic loads include structure inertial and hydrodynamic forces. These loads were developed as indicated in ER 1110-2-1806 Earthquake Design and Evaluation for Civil Works Projects, EM 1110-2-6053, Earthquake Design and Evaluation of Concrete Hydraulic Structures and other related USACE EMs. For the preliminary phase of analysis the Seismic Coefficient Method was used. For future analyses, a response spectra analysis will be performed using the site-specific response spectra curves for the OBE, MDE, and MCE as appropriate.

6.8.15 Hydrodynamic, Hd

The hydrodynamic forces in the seismic design will be calculated using Westergaard's method per ETL 1110-2-584 and EM 1110-2-2100:

- The direction of the ground motions is assumed to be in both horizontal directions but the vertical direction is presumed to be negligible.

- Hydrodynamic forces will be estimated by the use of Westergaard's equation, which is defined as:

$$P_E = (7/12)k_h\gamma_w h^2$$

Where:

P_E = Hydrodynamic resultant force per unit length
k_h = Horizontal; seismic coefficient
γ_w = Unit weight of water
h = Water depth

The resultant, $P_{E,}$ is applied at 0.4h above the top of ground in the opposing direction of the earthquake as shown below in Figure 6-9.

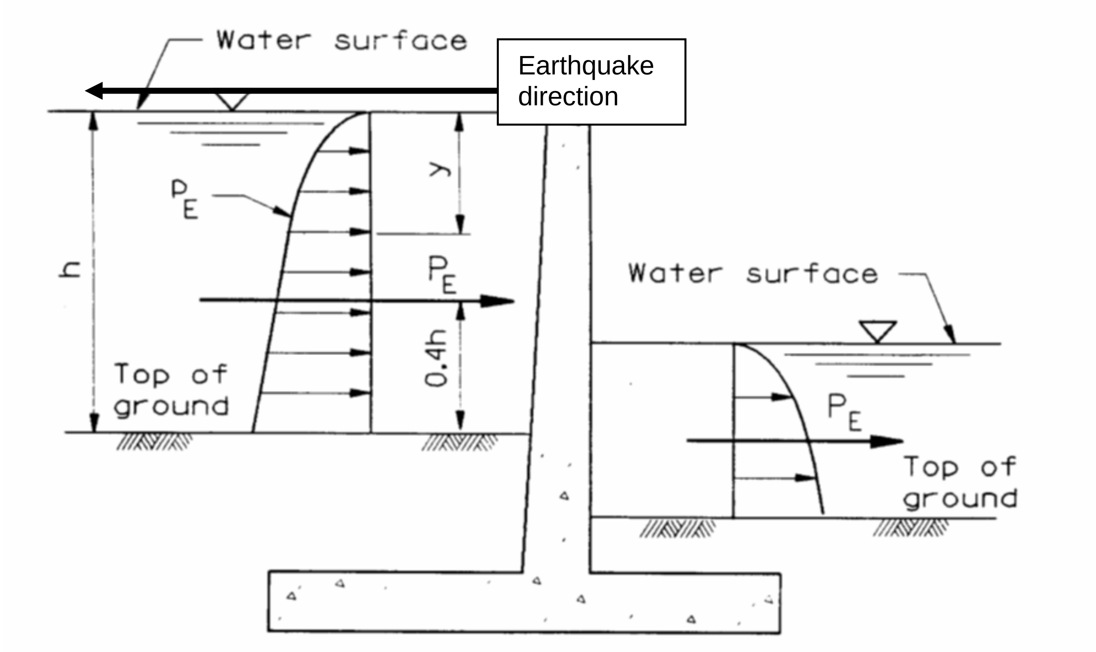

Figure 6-9. Hydrodynamic Forces for Freestanding Water

The resultant should not be used in a strength evaluation so it is necessary to find the actual pressure distribution. The lateral pressure at a depth y may be found by use of the Westergaard equation as represented in ETL 1110-2-584, paragraph 3.2.3.6, equation 3-2:

$$p = (7/8) * \gamma_w * a_c * \sqrt{(Hy)}$$

Where:
- p = Lateral pressure at a distance y below the pool surface
- γ_w = Unit weight of water
- a_c = Max. acceleration (a fraction of gravitational acceleration g)
- H = Pool depth to dam foundation
- y = Distance below the pool surface

While a rigid structure will be subjected to a maximum acceleration equal to the peak ground acceleration (PGA) during earthquake ground shaking, a flexible structure will experience an average acceleration that depends on the vibration period of the structure and on characteristics of the earthquake ground motion. For structures with periods between 0.02 seconds and 1 second (the typical range for most CHSs) the average structure acceleration will be greater than the PGA, with Dynamic Amplification Factors (DAF) as high as two to three[2]. The selected DAF will be estimated based on the fundamental period of vibration with respect to the structure of interest.

[2] EM 1110-2-6053, Section 7-2.

Hydrodynamic effects (water-structure interaction) may be accounted for as a hydrodynamic added-mass, where appropriate, using the following equation.

mai = 7/8 ρw Ai (H(H-zi))))^1/2

Where:
Ai = area tributary to a single node (at point i)
H = depth of water
zi = height above the base of the dam
ρw = mass density

Enclosed fluid effects

Hydrodynamic effects due to enclosed fluid (within the SWS) will be considered. This effect will be based upon a published discussion of the original Westergaard paper[3]. The hydrodynamic effect will consist of pressure increase or decrease acting on parallel confining walls. The following force will be uniformly distributed and computed as follows:

$F = ½\ \gamma_w\ V\ a_c$

Where:
γ_w = Unit weight of water
V = Volume of enclosed fluid/water
a_c = Max. acceleration (a fraction of gravitational acceleration g)

6.8.16 Side Seal Friction, Fs

This frictional force is a result of the side seal being in contact with the steel seal plates embedded in the SWS slots. It is a function of the coefficient of friction, which is assumed to be μ = 0.5, the amount of hydrostatic force on the seal, and the amount of preset deflection that is set in the seal. The equation below, from ETL 1110-2-584, will be used to estimate side seal friction from cantilevered bulb seals. The frictional force from rubber seals in bearing is determined by the product of the normal force and the coefficient of friction.

$$F_s = \mu_s S\, l + \mu_s \gamma_w \frac{d}{2}\left(l_1 \frac{h}{2} + h\, l_2 \right)$$

6.8.17 FSS Loads

An AE Firm is preparing a DDR for the design of the FSS, and to determine design loads, which also includes loads for hydraulic connections between the FSS and the

[3] Trans. ASCE Vol. 98, General Westergaard Theory

SWS. Based on preliminary research of comparable-sized portable floating fish collectors, an estimate of 100 kips per connection is assumed with two total connections to the SWS. Each connection is assumed to act at the water surface elevation in a horizontal direction, with the direction dependent on the loading condition.

6.9 STRUCTURAL DESIGN

6.9.1 Engineering Properties of Construction Materials

The engineering properties of construction materials are shown in Table 6-11.

Table 6-11. Engineering Properties of Construction Materials

Concrete: All Cast-in-Place (CIP) Structures	
New concrete in contact with or containing water Existing Concrete Modulus of elasticity (E) Poisson's ratio	f'c=5,000 psi f'c=3,000 psi 4,030,000 psi 0.2
Steel Reinforcement: All Structures	
New : ASTM A615 Grade 60	fy=60,000 psi
Structural Carbon Steel and Structural Stainless Steel: Areas of use shown on drawings	
ASTM A36 (carbon steel)	fy=36,000 psi
ASTM A 992 (carbon steel)	fy=50,000 psi
ASTM A709 (carbon steel)	Fy = 50,000 psi
ASTM A 240 (stainless steel)	fy=30,000 psi
ASTM A 276 (stainless steel)	fy=30,000 psi to 45,000 psi depending on Type selected
Structural Aluminum: Areas of use shown on drawings	
Type 6061-T6	fy=40,000 psi
Type 5052-H32	fy=28,000 psi

ASTM = American Society for Testing Materials
f'c = Specified compressive strength of concrete
fy = Specified yield strength (steel or aluminum)

6.9.2 Stability Analysis

Stability analysis of the SWS is performed in accordance with ER 1110-2-1806 Earthquake Design and Evaluation for Civil Works Projects, EM 1110-2-2100 Stability Analysis of Concrete Structures, and EM 1110-2-2200 Gravity Dam Design. The objective of the stability analysis is to maintain horizontal, vertical, and rotational equilibrium of the structure. Stability is ensured by providing an adequate FS against sliding at all possible failure planes; providing specific limitations on the magnitude of the foundation bearing pressure; providing constraints on the permissible location of the resultant force on any plane; and, providing an adequate FS against flotation of the structure. The site information is categorized as "Ordinary site information."

6.9.2.1 Load Cases for Stability

<u>LC1-Summer Pool, Normal Head Difference</u>

- Dead load of structure
- Reservoir at elevation 1569
- Water surface in SWS at normal head difference
- Earth load
- Full Uplift
- Wave loads

<u>LC2-Winter Pool, Normal Head Difference</u>

- Dead load of structure
- Reservoir at elevation 1450
- Water surface in SWS at normal head difference
- Earth Load
- Full Uplift
- Wind & Wave loads

<u>LC3-Summer Pool, Maximum Head Difference</u>

- Dead load of structure
- Reservoir at elevation 1569
- Water surface in SWS at max head difference
- Earth load
- Full Uplift
- Wave loads

<u>LC4-Winter Pool, Maximum Head Difference</u>

- Dead load of structure
- Reservoir at elevation 1450
- Water surface in SWS at max head difference
- Earth load
- Full Uplift
- Wind & Wave loads

<u>LC5-Maximum Pool, Normal Head Difference</u>

- Dead load of structure
- Reservoir at elevation 1574
- Water surface in SWS at normal head difference
- Earth load
- Full Uplift

- Wave loads

LC6-Minimum Pool, Normal Head Difference

- Dead load of structure
- Reservoir at elevation 1425
- Water surface in SWS at normal head difference
- Earth load
- Full Uplift
- Wind & Wave Loads

LC7-Construction – Construction methods not yet known to fully develop this load case

- Dead load of structure (partially or fully completed)
- Reservoir at elevation 1300
- Earth load
- Heavy construction equipment required on or near the structure during construction
- Wind load in the direction that would produce the most severe foundation pressures

LC8-Summer Pool, OBE

- Dead load of structure
- Reservoir at elevation 1569
- Water surface in SWS at normal head difference
- Earth load
- Full Uplift
- Wave Loads
- OBE Design Earthquake

LC9-Winter Pool, OBE

- Dead load of structure
- Reservoir at elevation 1450
- Water surface in SWS at normal head difference
- Earth load
- Full Uplift
- Wave Loads
- OBE Design Earthquake

LC10-Summer Pool, MCE

- Dead load of structure
- Reservoir at elevation 1569

- Water surface in SWS at normal head difference
- Earth load
- Full Uplift
- Wave Loads
- MCE Design Earthquake

LC11-Winter Pool, MCE

- Dead load of structure
- Reservoir at elevation 1450
- Water surface in SWS at normal head difference
- Earth load
- Full Uplift
- Wave Loads
- MCE Design Earthquake

The stability analysis loading and their loading classification are summarized below in Table 6-12.

Table 6-12. Stability Analysis Load Case Summary

Load Case Number	Loading Description	Loading Condition Classification
LC1	Summer Pool, Normal Head Difference	Usual
LC2	Winter Pool, Normal Head Difference	Usual
LC3	Summer Pool, Maximum Head Difference	Unusual
LC4	Winter Pool, Maximum Head Difference	Unusual
LC5	Maximum Pool, Normal Head Difference	Unusual
LC6	Minimum Pool, Normal Head Difference	Unusual
LC7	Construction	Unusual
LC8	Summer Pool, OBE Earthquake	Unusual
LC9	Winter Pool, OBE Earthquake	Unusual
LC10	Summer Pool, MCE Earthquake	Extreme
LC11	Winter Pool, MCE Earthquake	Extreme

6.9.2.2 Sliding

The seismic sliding stability is estimated using the seismic coefficient method. The seismic coefficient is equal to two-thirds (2/3) of the effective peak ground acceleration (EPGA) and is shown below in Table 6-13. The minimum required FS for seismic sliding stability is 1.3 and 1.1 for the OBE and MCE, respectively. For all other non-seismic load cases, the sliding stability FS shall not be less than 1.5, 1.3, and 1.1 for usual, unusual, and extreme load cases, respectively.

EPGA may be determined as per ERDC/CHL CHETN-VI-41 or EM 1110-2-6053.

EPGA = Ss/2.5

Table 6-13. Sliding Stability EPGA

EQ	2/3 EPGA	Source
144 yr	0.0195	Site Specific 2009
1000 yr	0.0885	Site Specific 2009
MCE	0.1573	Site Specific 2009

6.9.2.3 Flotation

The required factors of safety for flotation are 1.3, 1.2, and 1.1 for usual, unusual, and extreme load cases, respectively.

6.9.2.4 Bearing Capacity

The computed bearing pressures on the foundation material must not exceed the allowable bearing pressures. Allowable bearing pressures are computed in conjunction with the Geotechnical Design. The allowable bearing pressures for usual load cases include a FS of X. Increases of 15 percent and 50 percent to the allowable bearing pressures are permitted for unusual and extreme load cases, respectively. A summary of the allowable bearing pressures will be included in the 90% DDR in Table 6-14.

Table 6-14. Allowable Bearing Pressures

Load Condition	Allowable Bearing Pressure (psf)
Usual	XX,000 psf
Unusual	XX,000 psf
Extreme	XX,000 psf

6.9.2.5 Overturning

Overturning stability is evaluated by rotational modes of failure and the location of the resultant force along the structure's base. When evaluating rotational modes of failure, both static and dynamic modes –such as rocking– are considered. The criteria for acceptable overturning stability is determined by the amount of the base still in compression; a summary of the required criteria for each loading condition is in Table 6-15.

For MDE loading conditions, the resultant location must be within the base. If the resultant location is outside the base, a dynamic rotational stability analysis should be performed.

Table 6-15. Requirements for Overturning Stability

Load Condition	Requirements for Stability
Usual	100% of Base in Compression
Unusual	75% of Base in Compression
Extreme	Resultant Within Base

6.9.3 Foundation

The foundation of the SWS will consist of tremie concrete. The tremie concrete placement method is a procedure through which concrete is placed by gravity feed below the water level. A mass-concrete block will be cast against stepped rock at the reservoir bottom. The design of this foundation will be performed through hand calculations and the application of standard engineering principles. The final design will be verified with the use of a STAAD model that integrates the applied loads from the SWS.

The length of the foundation parallel to the dam face shall be 112'-0", extending 2'-0" beyond each side of the SWS. This additional length will provide space for construction elements such as formwork and guidance structures for the pre-cast blocks of the SWS.

The width of the foundation extending perpendicular to the dam face shall extend 12'-0" beyond the face of the SWS. This extension will provide the additional mass required to prevent the overturning of the foundation without the need for anchorage into the face of the dam. Furthermore, the extension will provide a bearing surface for the cold water intake trash rack structure. Due to the sloping face of the dam, the total width of the foundation will vary from approximately 37'-0" at the top (Elevation 1300') to 28'-0" at the bottom (Elevation 1210').

The rock at the bottom of the reservoir will need to be benched for the casting of the foundation. Additionally, the foundation will be keyed in to the rock to a depth of approximately 5'-0". The foundation height will be stepped following the benched rock and will vary from approximately 90'-0" to 10'-0". Refer to Figure 6-10 for approximate bench elevations.

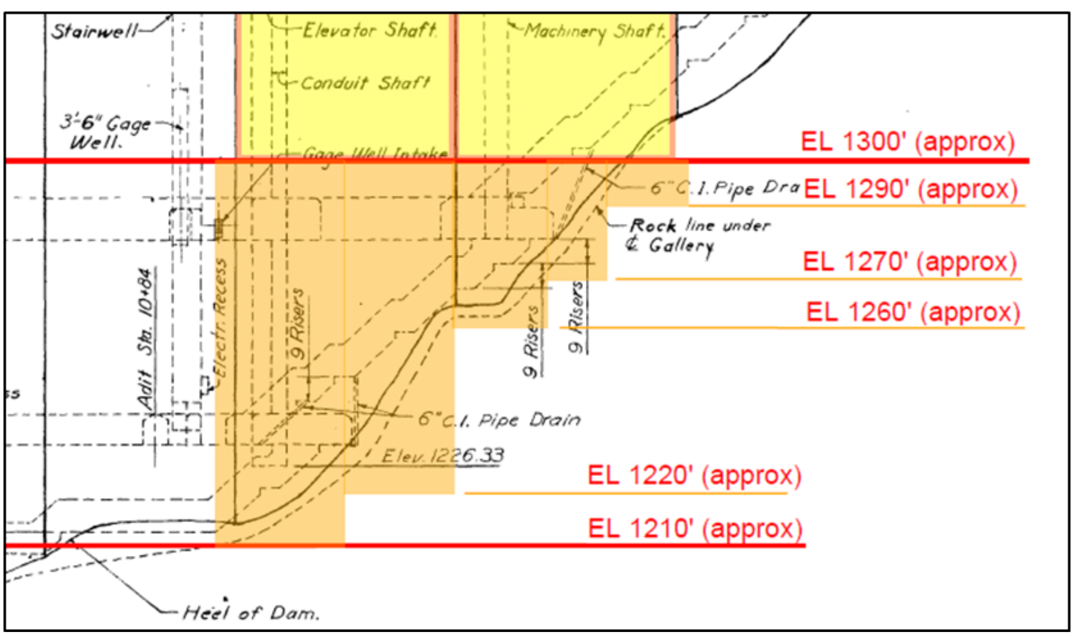

Figure 6-10. Stepped SWS Foundation

The primary overturning forces to be considered are seismic activation of the self-weight of the foundation and hydrodynamic forces acting directly on the foundation. The SWS will impart predominantly only vertical forces onto the foundation as the majority of the lateral loads will be resolved in the SWS anchorage to the dam face, as discussed below. Consequently, the governing load case will occur with a seismic event at maximum pool. By inspection, the following load cases specified in Section 6.10.2.1, will govern:

Load Case 8 – Summer Pool, OBE Earthquake

Load Case 10 – Summer Pool, MCE Earthquake

Design criteria for sliding, bearing, and overturning shall be designed as specified in the above sections. Additionally, the base of the foundation being keyed into the rock, as described above, will provide lateral restraint against sliding in a seismic event, and the thrust caused by the self-weight of the foundation bearing on the sloped surface of the dam.

6.9.4 Anchorage

The SWS foundation and anchorage will be designed using a STAAD model with hydrostatic loading, and hydrodynamic loading as the main overturning forces.

The STAAD model isometric model used for the anchorage design was shown previously in Figure 6-3. This model was built using solid elements that connect to each other on each face using 4 nodes. Hydrodynamic and hydrostatic loads were applied to the outside faces of the tower.

The maximum hydrostatic head difference was provided by Hydraulic Design and was taken as 12 ft. With regard to the STAAD model, the hydrostatic head difference is applied around the entire tower in each direction at each face.

The external hydrodynamic force on the SWS was determined using the Westergaard equation with a base acceleration of 0.139 and 0.236 to respectively check the MDE and the MCE load cases. Both external and internal hydrodynamic loading will be considered. The internal hydrodynamic force due to enclosed fluid will use an appropriate method.

STAAD reports the reaction forces at the base and the anchor locations. Using these reactions the anchorage options will be evaluated. The base of the tower will be wider than the tower to reduce the reactions at the anchorage.

The anchorages to the existing dam will be designed in accordance with EM 1110-2-2104 and ACI 318-14 chapter 17 *Anchoring to Concrete*. Design forces will be from the STAAD analysis. However, these forces were not available for the 60% DDR, so a static analysis was performed to estimate the reinforcement required.

The SWS was designed to be a self-supporting structure on the tremie foundation, so the anchors are passive only. Therefore, only seismic loads are applied to the anchors.

From the analysis, the size of anchors required was equal to the area of a #18 bar spaced at 1-ft on center from elevation 1,300 ft to 1,520 ft. After elevation 1,520 ft the spacing incrementally increases to 5 ft on center. The embedment into the concrete was calculated based on the length required to develop the reinforcing, which was 16.1 ft. If bundled bars are used, the development length will likely increase. Concrete breakout was not checked as part of this analysis, because it is anticipated that the design forces will decrease when results from the STAAD model are available.

To install the anchors, holes will be drilled into the dam at the required spacing. The anchors will be set into the hole, then filled with either grout or epoxy. It is anticipated that the reinforcement will only extend out of the dam far enough for a mechanical connector to be installed. The mechanical connectors will allow for the SWS reinforcement to be installed without interference from the embed anchors. After the SWS reinforcement is placed, the remainder of the anchor will be installed and tied into the SWS reinforcement.

6.9.5 HIWs

The HIWs are horizontally framed telescoping weirs and are welded steel assemblies. The HIW openings are 20 ft wide. Each HIW consists of three identical leafs with a nominal height of 52.5 ft per leaf, for a total maximum weir height of 157.5 ft to cover the full height of the opening. A single leaf will be made up of 6 segments that can be disassembled from the top deck of the SWS. Each segment will be sized to fit on a standard truck for transportability.

Each leaf is individually operated with wire rope hoists. The hoists are located within a guide slot to place the ropes outside of the flow path. The weirs will include a series of guide-rollers to maintain alignment within the guide slots and minimize the friction and risk of the gate racking and jamming. A seal is required along the perimeter of the HIWs so that colder water from deeper in the water column does not leak into the wet well. The top invert of each HIW leaf will be rounded for hydraulic performance.

6.9.5.1 HIW Design Criteria

The HIW are designed using a combination of hand calculations and finite element analyses using STAAD.Pro. The gates are analyzed for all limit states including strength (LRFD), serviceability, fatigue, fracture, and survivability under extreme events in accordance with ETL 1110-2-584.

<u>Cyclic Loading.</u> The primary stresses in the HIW are driven by the differential head from the reservoir to the water elevation within the SWS. The water elevation within the SWS will fluctuate with changes in the flow out of the SWS, such as startup and shut down of the turbine units or flow through the bifurcation. These changes are limited to 6 to 8 feet for normal operations, but could see several cycles a day. Considering an

average rate of only one cycle per day, the structure would experience 36,500 cycles throughout its 100-year service life. It is reasonable to assume that the gates see more than one cycle per day, therefore fatigue is an applicable limit state.

Materials. The structural members of the gate will be fabricated out of ASTM A709 carbon steel. Grade 50T and 50F steel will be specified for non-fractural members and fracture critical members, respectively. Under no circumstance will weathering steel (grade 50W) be permitted. Non-structural members, such as seal clamp bars, do not need to be ASTM A709 and A36 is acceptable.

HSS Performance Factors. The HIW are normally submerged and are not easily accessible for inspection and maintenance. Therefore, the HSS performance factor, α, will be 0.85.

Load Combinations for HIW.

Load combinations are in accordance with Section E.5, ETL 1110-2-584. Additional load cases have been added to account for case-specific loading conditions. Similarly, load cases that do not apply have been indicated as such and were not included in the design.

Case 1: Gate Closed, Wave (Strength Limit State)

$$1.2D + 1.6G + 1.4Hs2 + 1.6Wa$$

Case 2: Gate Opening, Normal Operations (Strength Limit State)

$$1.2D + 1.6G + 1.4Hs2 + 1.6Wa + 1.6\,Fr + 1.2\,Hd$$

Case 3: Gate Closed, OBE Earthquake (Strength Limit State)

$$1.2D + 1.2G + 1.2Hs1 + 1.0EQ_{MDE}$$

Case 4: Gate Jammed (Strength Limit State)

$$1.2D + 1.2G + 1.0Hs1 + 1.2Q3$$

Case 4a: Fatigue I, Infinite Life, Normal Hydrostatic and Wave (thermal effects not applicable)

$$1.0Hs1 + 1.0Wa$$

Case 4b: Fatigue II, Finite Life, Unusual Hydrostatic and Wave (thermal effects not applicable)

$$1.0Hs2 + 1.0Wa$$

Case 5: Gate Closed, MDE Earthquake

$$1.2D + 1.2G + 1.2Hs1 + 1.0EQ_{MDE}$$

6.9.5.2 HIW Analysis Results and Design Summary

See Structural Appendix B for analysis results.

6.9.6 LIG

There are four LIGs with openings that are 15 ft wide by 10 ft tall each. Two of the LIGs have a bottom invert elevation of 1,327, and the other two have a bottom invert elevation of 1,305 (see PLATE 4). The LIGs are framed similar to the HIWs, horizontally framed, welded steel assemblies.

The LIGs are subject to a uniform hydrostatic pressure resulting from the head difference between the reservoir and water elevation in the SWS. Although the normal head difference is anticipated to be relatively small, the bottom of the gate will be shaped to reduce flow induced vibrations. Significant uplift and downpull forces are still expected and results will be provided by the hydraulic engineer.

Each LIG is individually operated using wire ropes, which will be located within the guide slots out of the flow path. Similar to the HIWs, the LIGs will include a series of guide-rollers and seals. A custom lifting beam will be built to retrieve the LIGs to the top deck for inspection and maintenance. Lifting lugs will be provided on top of the LIGs to receive the latching mechanism for the lifting beam.

6.9.6.1 LIG Design Criteria

The LIG are designed similarly to the HIW using a combination of hand calculations and finite element analyses using STAAD.Pro. The gates are analyzed for all limit states including strength (LRFD), serviceability, fatigue, fracture, and survivability under extreme events in accordance with ETL 1110-2-584.

<u>Cyclic Loading.</u> Like the HIW, the primary stresses in the LIG are driven by the differential head from the reservoir to the water elevation within the SWS. The number of cycles is expected to be similar to that of the HIW and therefore fatigue is also considered for the LIG.

<u>Materials.</u> The structural members of the gate will be fabricated out of ASTM A709 carbon steel. Grade 50T and 50F steel will be specified for non-fractural members and fracture critical members, respectively. Under no circumstance will weathering steel (grade 50W) be permitted. Non-structural members, such as seal clamp bars, do not need to be ASTM A709 and A36 is acceptable.

HSS Performance Factors. The LIGs will be constantly submerged under normal operations and will not be easily accessible for inspection and maintenance. Therefore, the HSS performance factor, α, will be 0.85 in accordance with Section 3.1.1 of ETL 1110-2-584.

Minimum Weight. Since the LIGs are operated with wire ropes, they will need to be heavy enough to close with only the force of their own weight. The total force resisting closure will be the sum of hydraulic uplift, buoyancy, seal friction and roller friction. An FS of 1.4 will be applied to the force resisting closure to determine a minimum total weight of the gate.

Load Combinations for Low Intake Gates.
Similar to the warm water intake gates, the load combinations for the cold water intake gates are in accordance with Section E.5, ETL 1110-2-584. Load cases have been added or removed as applicable to these gates.

Case 1: Strength Limit State Ia – Gate Closed, Wave

$$1.2D + 1.6G + 1.4Hs2 + 1.6Wa$$

Case 2: Strength Limit State Ib – Gate Closed, Thermal

Not Applicable

Case 3: Strength Limit State II – Gate Open, Wind

Not Applicable

Case 4: Extreme – Gate Open, Gate Jammed

$$1.2D + 1.2G + 1.0Hs1 + 1.2Q3$$

Case 5: Extreme – Gate Closed, Earthquake (MDE)

$$1.2D + 1.2G + 1.2Hs1 + 1.0EQ_{MDE}$$

Case 6: Strength III – Gate Closed, Earthquake (OBE)

$$1.2D + 1.2G + 1.2Hs1 + 1.0EQ_{OBE}$$

Case 7: Strength Limit State IV – Maintenance, Laying Gate Flat / Lifting Gate from Flat

$$1.2D + 1.6G + 1.2Q$$

Case 4a: Fatigue I, Infinite Life, Normal Hydrostatic and Wave (thermal effects not applicable)

1.0Hs1 + 1.0Hd

Case 4b: Fatigue II, Finite Life, Unusual Hydrostatic and Wave (thermal effects not applicable)

1.0Hs2 + 1.0Hd

6.9.6.2 LIG Analysis Results and Design Summary

See the Structural Design Appendix B for analysis results.

6.9.7 HIW Trash Rack

The trash racks for the HIW's are attached to the upstream face of the SWS. There will be two trash racks, each composed of eight 20-ft sections, which will be fabricated as 10-ft sections for transportation, and combined in the field prior to being lowered into place. The trash racks will be 20 ft wide and a total of 160 ft tall.

The HIW trash rack is necessary as an intermediate function until the FSS is installed; completion is expected in 2027. However, once the FSS is installed, there will be a continued need for 40 ft of trash rack hanging below the flume and tracking with the FSS. Therefore, the trash rack has been designed to make minor modifications to two of the 20-ft sections, allowing them to be available for continued use once the FSS is complete.

<u>Materials.</u> ASTM A992, Grade 50 Steel will be used for rolled shapes, and ASTM A572, Grade 50 Steel will be used for Plates.

<u>Loads.</u> The trash racks were designed to keep out debris, silt, and trash loads from objects floating downstream; should debris build up to a point that it blocks flow by 75% prior to being manually cleaned, the trash rack could experience an 8-ft head differential. This head differential between the forebay and the SWS causes a differential pressure of 500 psf across the trash rack. There is also a potential for snow loads in the winter, as the forebay will be at a lower pool, and earthquake loading due to seismic event.

6.9.8 LIG Trash Rack

The trash racks for the LIG's are attached to the face of a concrete pipe 10 ft from the upstream face of the SWS. The concrete pipe will be constructed at the same time as the SWS, in (5)-ft lifts. There will be two trash racks, each composed of 11 4-ft sections, which will be combined in the field and attached to the pre-constructed pipe.

<u>Materials.</u> ASTM A992, Grade 50 Steel will be used for rolled shapes, and ASTM A572, Grade 50 Steel will be used for Plates.

Loads. The trash racks were designed to keep out debris, silt, and trash loads from objects floating downstream; should debris build up to a point that it blocks flow by 75% prior to being manually cleaned, the trash rack could experience a 10-ft head differential. This head differential between the forebay and the SWS causes a differential pressure of 624 psf across the trash rack. The last anticipated load is an earthquake load due to a 975-year seismic event.

6.9.9 Personnel Access

The FSS will require a secondary means of access. It was determined that a steel framed staircase on the north side of the tower would best serve this purpose. The staircase begins at the top of the tower and ends at the bottom invert of the HIWs. The staircase runs at a 41 degree angle; each landing is 9‘2” below the next, and the rise-to-run is 9-in to 11-in.

Because the staircase will be continuously submerged under the varying forebay elevation, the treads and landings will be made of fiberglass reinforced plastic with a non-slip surface to make it safe for personnel access. The FSS will have gangway-type surface that tracks with the staircase, such that is can be reached at any elevation of the forebay.

Loads. The stairs were designed to handle a 1,000-lb load on the center step between landings, and their own self weight. The handrails are designed for a 200-lb load in any direction.

6.9.10 Fish Off-loading

Fish offloading strategies are still being developed. Options being considered are: a vertical hoist off the face of the dam; amphibious vehicles; and a “trolley car” system on grade at the north bank. The DDR will be updated once a decision has been made.

6.9.11 Downstream Penstock Bifurcation

To pass the required flows for water temperature control while the units are not operating, a bifurcation has been proposed off the existing penstocks. The bifurcation conduits wye off the existing penstocks and convey the temperature regulated water to the spillway as shown in Plate 2. For further information regarding the penstock bifurcation conduits, see Section 4.4.4.

The wye off the existing penstocks and the bend of the pipes into the stilling basin result in large thrust forces as described in Section 4.4.4(d). The thrust forces are to be resolved by incorporating large concrete thrust blocks that will encompass the bifurcation pipes. The required mass of concrete for the bend thrust block is approximately 80’ x 30’ x 10’ (W x L x H) which is shown in Plate 2. The required thrust block for the wye pipe has not been determined and will be updated in the 90% submittal.

6.10 SWS CONSTRUCTABILITY

6.10.1 Demolition Requirements

Demolition is limited in scope to the removal of the existing steel and concrete trash rack structures that are located at the penstocks. The pieces can be saw cut and removed in a manner that maintains structural integrity of remaining elements. Pieces can then be lifted to the top of the dam using a mobile lift crane. Lifts will need to be limited in size so they can be lifted by a crane that is capable of lifting within the limits of the dam roadway deck.

6.10.2 Work Platforms

Access to the work area can be reached from the top of the deck; however, space is limited to the 20'-0" roadway width. Precasting of SWS tower sections can be done from barges located on water adjacent to the upstream side of the dam. Concrete will be pumped from trucks located at the top of the dam to casting barges located on the water, adjacent to the tower footprint. Precast concrete sections will need be match-cast prior to placement on the tower, and the barges must be designed to support the weight of two blocks of tower height. Cast-in-place concrete can be tremie poured from the dam deck into place at the tower, and rebar in these sections can be lifted into place from the deck. Divers will be required to place the rebar, place formwork for cast-in-place concrete, drill passive anchors into the dam, guide precast units into place, and place hoisting frame tower elements underwater.

6.10.3 Hoisting Frames

To place the large precast sections, a multi-bent hoist frame will need to be designed and built to support a gantry crane capable of lifting the precast pieces from the barge into place at the tower location. The crane will need to be able to move the precast units in two directions in plan, so that pieces can be lifted off the casting barge onto the tower, as well as maneuvered between the adjacent tower sections. The hoist frame will consist of space truss towers anchored to bedrock. As such, some towers will be greater than 390 ft tall from bedrock to top of dam. No other viable alternatives exist to lift the large precast sections, which weigh approximately 275 tons. Any attempt to cantilever a casting yard and/or crane off the top of the dam yields tensile forces at the top of the dam in excess of 1 million pounds of force. It may be possible, pending input from a contractor, that a crane of sufficient size can be located on barges adjacent to the casting barges, to lift the precast into place. At the crane sizes and pick radii required, no stock crane/barge system has been found that is capable of making the pick; however, design of such a system may be possible during the construction phase and should be considered at that time.

6.10.4 Tremie Slab

The tremie slab will be poured from the dam deck. Formwork will be constructed underwater, and will consist of a soldier beam system consisting of steel H-piles drilled into bedrock, and using steel I-beam lagging between the posts. The slab pours will be limited to 10ft vertical lifts, letting the concrete harden between lifts. This will limit the lateral pressure exerted on the soldier beams, and will allow this system to achieve the 100-ft vertical height needed for the tremie slab.

SECTION 7 - MECHANICAL DESIGN

7.1 GENERAL

This section discusses the mechanical equipment primary features and functions on the SWS. This section also includes the design criteria and assumptions used in development and design of this equipment. The major mechanical components consist of HIW hoists, LIG hoists, trash rack hoists, an overhead maintenance crane, a bifurcation valve for penstock water passage, and an FSS hopper crane.

7.2 REFERENCES

40 CFR 112 – Oil Pollution Prevention, July 12, 2013

CMAA 70, Specifications for Top Running Bridge and Gantry Type Multiple Girder Electric Overhead Traveling Cranes, 2010

EM 1110-2-1424, Lubricant and Hydraulic Fluid, 29 January 2016

EM 1110-2-2610, Mechanical and Electrical Design for Lock and Dam Operating Equipment, 30 June 2013

EM 1110-2-2704, Engineering and Design - Cathodic Protection Systems for Civil Works Structures, 12 July 2004

EM 1110-2-3200, Engineering and Design – Wire Rope for Civil Work Structures, 30 Nov 2016

ER 1110-2-8159, Life Cycle Design and Performance

ETL 1110-2-584, Engineering and Design – Design of Hydraulic Steel Structures, 30 June 2014

NMFS Anadromous Salmonid Passage Facility Design, 2011

RR-W-410F, Federal Specification, Wire Rope and Strand, 6 Dec 2007

7.3 MECHANICAL PLATES

Table 7-1. Mechanical Plates at 60% DDR

No.	Description
SKM001	Index
SKM100	Detroit SWS General View
SKM101	Detroit SWS Plan View
SKM102	Detroit SWS HIW Assembly View
SKM103	Detroit SWS LIG Assembly View
SKM104	Detroit SWS Maintenance Crane

SKM105	Detroit SWS Bifurcation Valve

See Appendix A for mechanical plates.

7.4 SCOPE OF WORK

The SWS will operate alongside the FSS to control water temperatures downstream of Detroit Dam. The FSS will provide the surface inflow to the SWS and the SWS mechanical components will regulate the quantity and temperature of flow. The SWS will include the following mechanical features:

- Hoists for two HIWs, each consisting of three telescoping leafs. Each leaf will be operated by an independent hoist, and each hoist will allow the leaf to travel from the bottom of the ogee to the SWS deck (approximately 180 ft).

- Hoists for four LIGs. Each LIG will be driven by an independent Hydraulic Power Unit (HPU). The HPU will operate two independent piston cylinders connected to two 1” wire ropes (four wire ropes per gate). A lifting beam will be deployed by the maintenance crane to bring the LIGs up to the SWS deck.

- An overhead crane. The overhead crane will be rated for 70 kips and used for the following purposes:

  - To transfer HIW segments from the HIW gate slot to a flatbed truck on the roadway

  - To raise the LIGs to deck level for inspection/maintenance

  - To lift the access hatch from the roof of the maintenance room

- Two hoists for the trash racks (one per HIW). Salient features of the trash rack hoists shall be provided at the 90% DDR.

- A bifurcation valve to reroute penstock flow to the stilling basin.

- The FSS hopper crane. Salient features of the FSS hopper crane shall be provided at 90% DDR.

  Salient features of the described mechanical components can be found in this chapter. Mechanical Plates can be found in Appendix A. Refer to Figures 7-1 and 7-2 for gate arrangement and mechanical components atop the SWS.

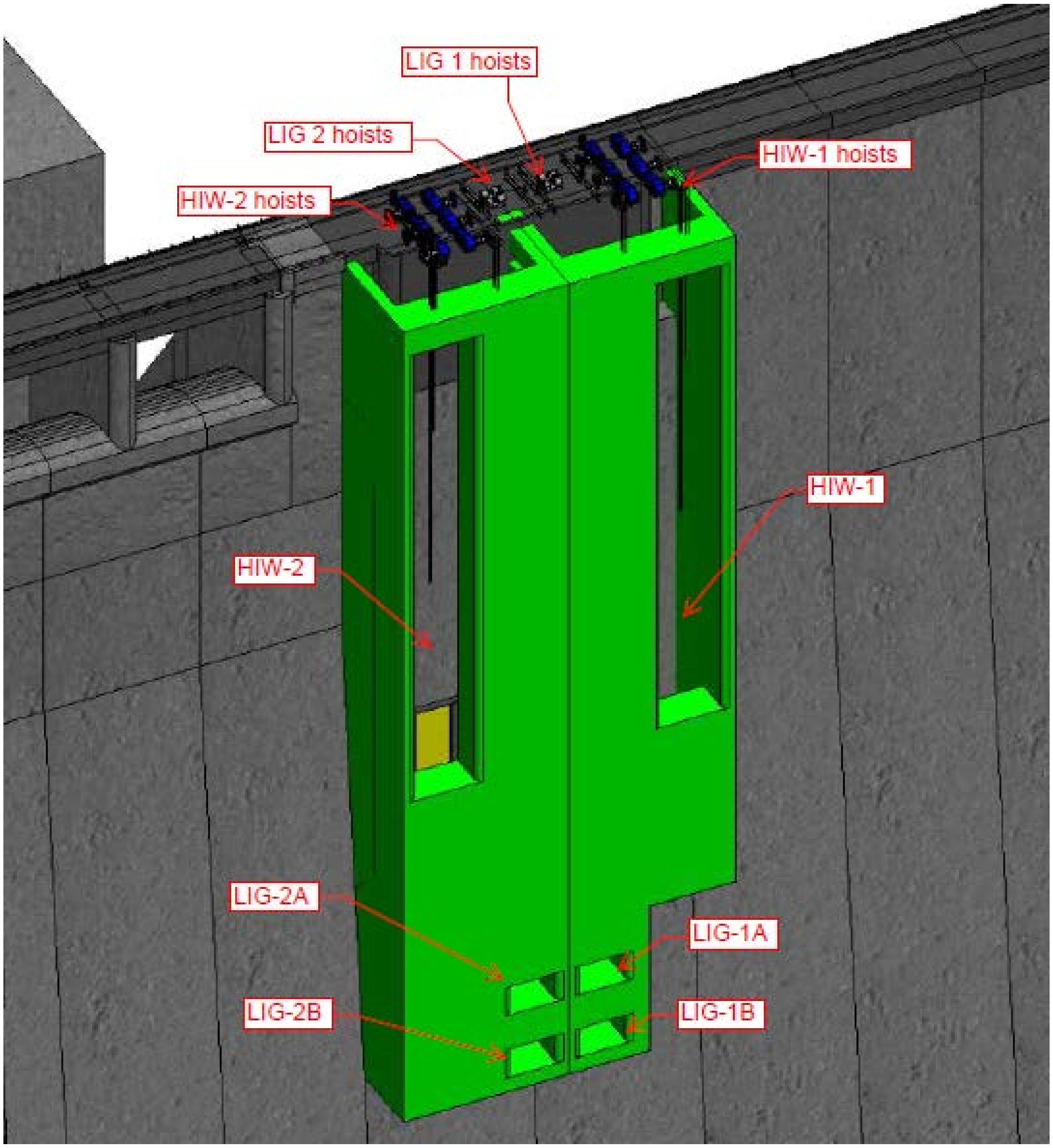

Figure 7-1. SWS Deck Isometric View

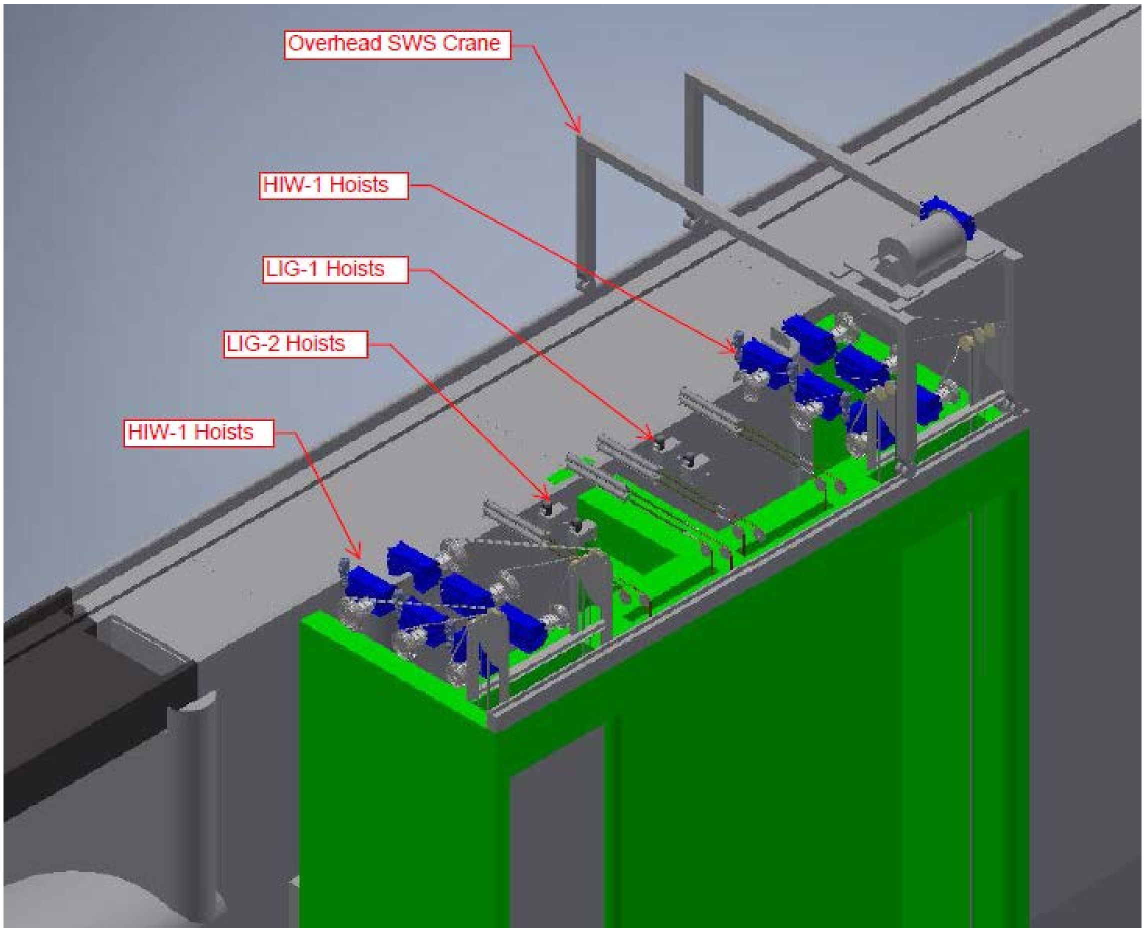

Figure 7-2. SWS Deck Isometric View

7.5 CONSTRAINTS

One of the driving limitations of the mechanical equipment is the spatial and seismic constraints atop the SWS. To meet seismic constraints as outlined in Chapter 6, Structural Design, the SWS could not exceed a 40'x108' footprint. All the mechanical components (excluding the overhead crane and the FSS fish hopper), must fit inside a mechanical room that is less than 40'x108' while meeting spatial requirements as outlined in EM-385-1. The gate machinery must be placed as low to the deck as possible to minimize seismic risk, and the overhead crane must not exceed 30 ft in height. Therefore, when the HIWs must be removed, each leaf will have to be broken up into 10-ft segments and transported segment by segment to a flatbed on the road deck. Refer to Figure 7-2 for the gate equipment (sheave packs not shown, nor trash rack hoists), that must fit inside the mechanical room. Refer to Figures 7-3 and 7-4 for a plan and elevation view of the mechanical equipment atop the SWS.

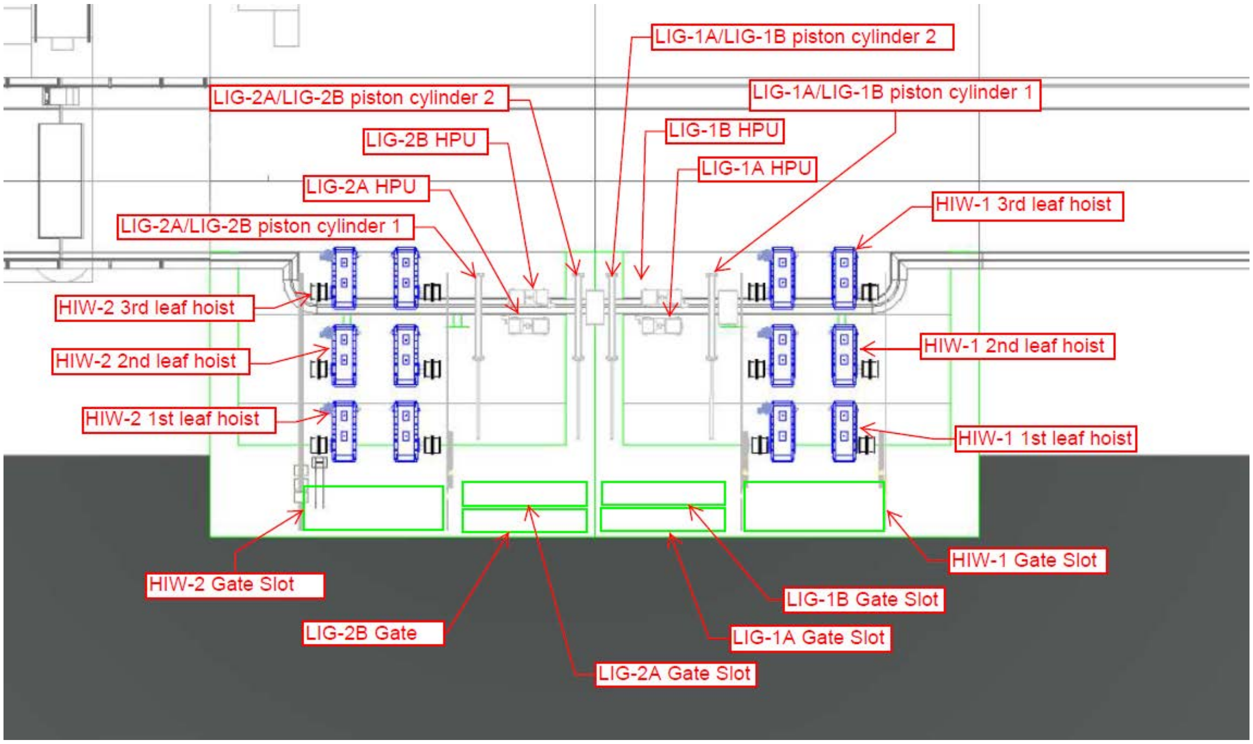

Figure 7-3. Hoist Equipment Plan View

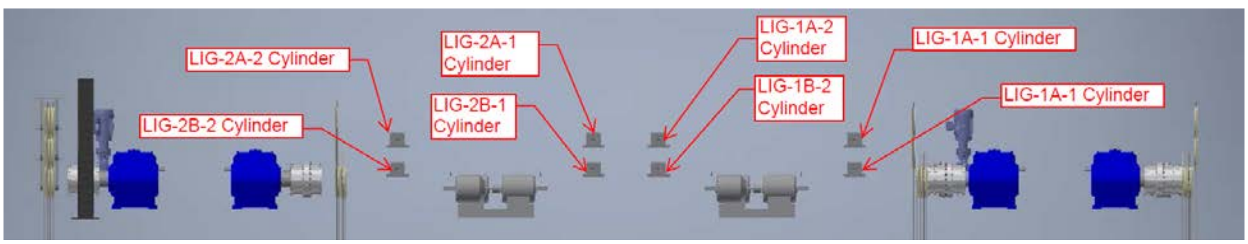

Figure 7-4. Hoist Equipment Elevation View

Table 7-2. Elevations and Heights of Weir Gates and Associated Equipment

Max crane lift – bottom of hook elevation	1635 feet
Overhead Hoist platform elevation	1615 feet
Nominal top of gate (HIW)– maximum weir invert elevation	1570 feet
Nominal top of gate (HIW)– minimum weir invert elevation	1412.5 feet
Height of individual weir gates	53.25 feet
Nominal height of individual lower gates (LIG)	11.25 feet
Nominal width of individual lower gates (LIG)	16 feet

For the HIWs and LIGs, head differential will range 3 ft to 12 ft for normal operating conditions. For extreme operating conditions, head differential on the gates can peak to 19.2 ft. The abnormal condition would result from rapid unit start up, where the turbine could start up or shut off in 9 seconds. To counteract this, the HIWs would need to raise at 2.7 ft/min and the LIGs would need to raise at 10 ft/min. Although the rapid unit startup may not happen in the gate's lifetime, it should still be considered a design criteria. The hoists must be robust and capable of withstanding the gate speeds and friction resulting from the extreme head differential. Additionally, because the HIWs and the LIGs are being classified as HSS, the hoist machinery must be able to raise the gates to the SWS deck for HSS Inspections, in accordance with ETL 1110-2-584.

Another constraint that must be considered is the trash rack, and how to deploy the trash rack without interfering with the FSS seal. Between the FSS and the SWS a continuous flow open channel occurs, a sealing device will be used to prevent water flow that has not passed through the FSS. The FSS to SWS seal will act as an effective trash rack for the HIW while the FSS is in operation. However, when the FSS is in maintenance mode, a secondary trash rack will need to deploy. The secondary trash rack will be 50 ft in length and will only need to track the first telescoping HIW. For the 90% DDR, the SWS PDT will need to work with the AE contractor to develop an effective solution for interfacing the trash rack with the FSS to SWS seal. In terms of deployment, the PDT is still determining whether the trash rack should be lowered by a lifting beam, wire rope hoist, or other means. Salient features of the trash rack, trash rack hoists, and location will be provided at the 90% DDR. Refer to Figure 7-5 for the FSS to SWS seal mechanism concept. Due to the depth of the LIGs, the trash rack will not include a raking device.

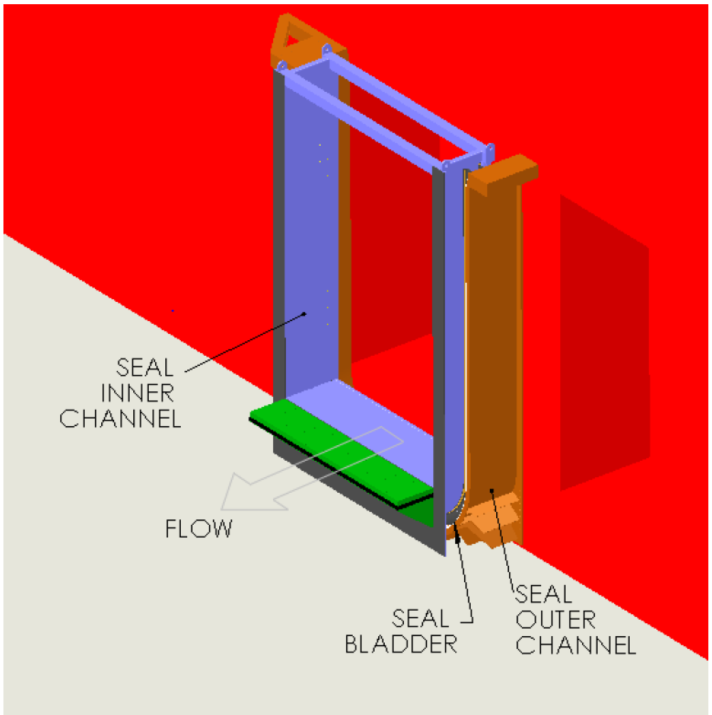

Figure 7-5. FSS to SWS Seal Concept

The final constraint to be considered is environmental. The mechanical equipment will design oil containers to be below 55 gallons so it is not accounted for on the SPCC plan on site. To ensure that contamination is minimal, a means to capture oil will be considered along with equipment to detect and trap any spills such as drip trays and float sensors. The mechanical PDT will consider alternatives (like open gearing, custom gearboxes, etc.) if the quantity of oil exceeds the allowable oil atop the SWS. Additionally, self-lubricated composites shall be preferred over greased bushing to mitigate any contamination into the river. More information regarding environmental regulation will be provided at 90% DDR.

7.6 OPERATIONS AND MAINTENANCE ASSUMPTIONS

Operational requirements of the gates will change seasonally and will mainly involve minor operations due to gradually changing temperature requirements, turbine operations, and pool elevation requirements. It is expected that most operations will be small, incremental changes. The PDT has determined there will be no bulkheads for the HIWs. If a gate is pulled from its slot for maintenance or inspection, the slots will be open and outflow temperature may be skewed during that time. Each piece of equipment will be guarded against hydrocarbon contamination and acceptable maintenance clearances will be maintained.

The design life for all mechanical equipment will be 50 years.

7.7 HOIST SYSTEM LOAD CASES

Engineering Manual 1110-2-2610, Chapter 7-4 allows the general design criteria for Tainter gates to be applied to vertical lift gates. Loads to account for in the hoist design include gate dead weight, hydrodynamic loads, ice, silt, sliding or rolling friction load, and side seal friction load.

Per EM 1110-2-2610, section 9-2.m.(5), two load cases, Load Case A (LCA) and Load Case B (LCB) will be considered in the hoist design. LCA is the normal operating condition where the normal working load is distributed evenly over the hoist and LCB is the maximum overload condition.

7.7.1 Load Case A – Normal Operations

LCA is the normal operating load case; it is calculated based on the maximum design differential head of 12 ft, and presumes the worst case silt load. It assumes regular maintenance will be performed on the mechanical systems over the 50-year design life. The normal operation loads on the hoist are a function of the external loads applied to the gate (hydrostatic forces, gravitational forces, friction forces, etc.). To calculate the load required to lift the vertical gate (tension in hoist ropes) a free body diagram is created (Figures 7-6 and 7-7). Operating loads are applied to the free body diagram and a summation of forces is performed to solve for the tension in the wire ropes. The anticipated loads will be used to size the rope and hoist machinery.

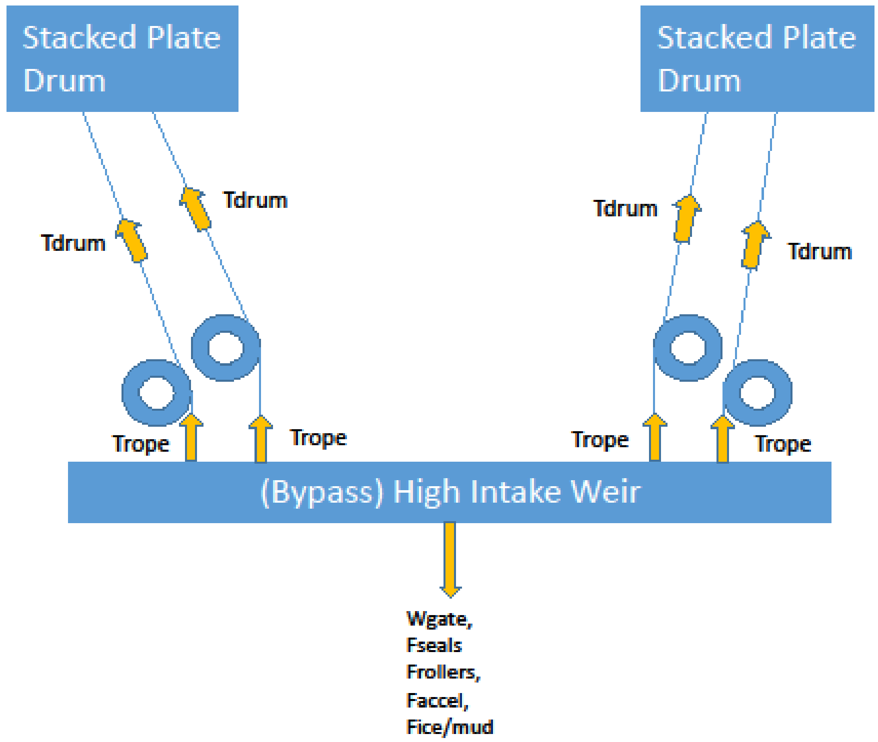

Figure 7-6. HIWs Hoist Free Body Diagram (FBD)

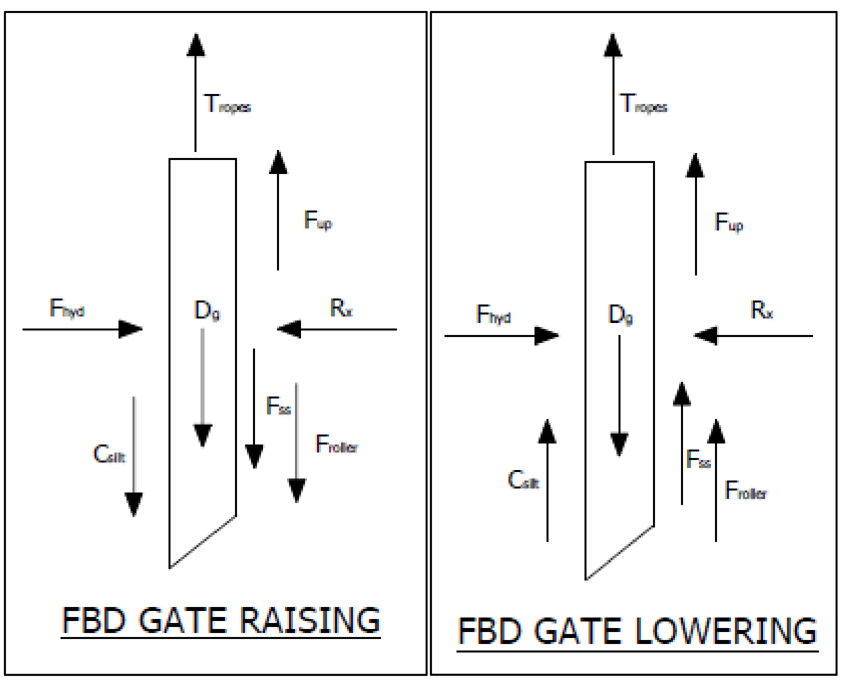

Figure 7-7. LIG Hoist Free Body Diagram (FBD)

7.7.2 Load Case A – Design Criteria

Per the requirements of EM 1110-2-2610, custom designed components of the hoist system must be designed for a factor safety of 5 based on the ultimate tensile strength.

Pre-engineered (off-the-shelf) components should be selected for this load case based on the Original Equipment Manufacturing (OEM) engineered ratings. Pre-engineered components must have an adequate FS and robustness for the application.

7.7.3 Load Case A – Duration

The HIWs will generally track the reservoir elevation. This means that during a normal year, where the reservoir follows the rule curve (see Chapter 15, Hydrologic Design, for detail), the HIWs will complete one full cycle from fully extended to fully retracted and back to fully extended. However, since the reservoir level is prone to fluctuation, and the outflow from the dam is not constant, the HIWs may be raised or lowered multiple times per day to maintain the desired flow rate into the tower; it is assumed these minor adjustments will add up to one full gate cycle per year. If the gates are on a 25-year inspection cycle, they will undergo at least one full cycle for inspection during the design life of the hoist. It is assumed the HIWs will be cycled five times during startup and commissioning. In total, the hoist must be designed to complete at least 175 full gate cycles (extend, retract, extend).

The LIGs are anticipated to be raised from the closed to open position once a week, for an approximate total of 2600 cycles in the design life.

7.7.4 Load Case B –Maximum Motor Torque

LCB will be defined as the maximum torque of the motor applied to each hoist.

For hoists with a single drum configuration, or for a hydraulically operated gate, no load sharing is applicable and 100% of the motor load will be applied. For hoists with the configuration of a single hoist motor and two wire rope drums (one on either side of the gate), the load will be split 70/30 between the drums as described in EM 1110-2-2610 part 9-2 m.5(C).

This load case is not directly covered in ETL 1110-2-584 for the structural design of gates; it is, however, most comparable to Section E.5, ETL 1110-2-584, Case 4, which assumes the gate is jammed and prevented from moving. The max load from the hoist is taken as the force applied from the locked-rotor torque of the motor.

7.7.5 Load Case B –Design Criteria

Per the requirements of EM 1110-2-2610 custom designed components for the hoist system under LCB must be designed to not exceed 75 percent of the yield points, with the exception of wire rope, which must not exceed 70% of the breaking strength. Pre-engineered components will use manufacturer ratings which take into account appropriate safety factors.

7.7.6 Load Case B –Duration

The hoists will be designed to experience LCB as the static torque five times in the life of the system. Since the HIWs will have a load pin, LCB should not be experienced longer than 30 seconds per instance.

7.8 HIW

The HIWs will be designed to operate with a single hoist for each of the three leaves. The gates will use an operating range of 157.5 ft, which will employ three telescoping gates approximately 53.25 ft long, with a width of 21 ft, not including the rollers. Calculated leaf weights are estimated at 70,000-lb each.

Each leaf will be lifted and lowered at a velocity of 2.67 ft/min by a hoist consisting of two electric motors, brakes (need for redundant braking will be determined at the 90% milestone), two stacked plate wire rope drums (one on each end), two parallel reducers and two right angle reducers. If possible, open gearing will be avoided since open gearing increases risk of contaminants. The hoists will be reeved such that the wire ropes are terminated on each end of the drums. As the rope leaves the drum, it will wrap over a sheave that is attached above the gate slot (one sheave per gate side), then the wire rope will connect to a hoist connection on the side of the leaf (Figures 7-8 & 7-9).

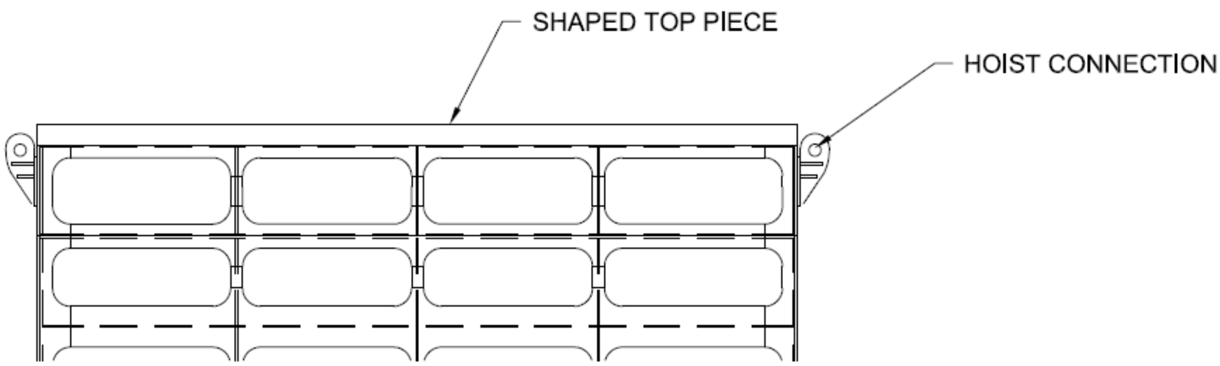

Figure 7-8. Downstream Elevation View of HIWs

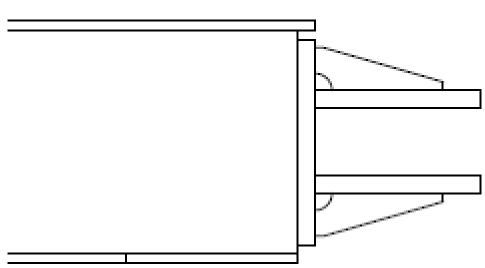

Figure 7-9. HIW Lifting Lug

Each HIW leaf is composed of five 10-ft segments bolted together. Each gate slot will have dogs, enabling each weir gate to be dogged off at each gate segment. For disassembly the leaf will be raised to the hoist machinery deck, and removed, segment by segment.

The HIWs will have stainless steel rollers on the ends to assist in raising and lowering the gates (rollers were selected over guides due to less friction). The roller assembly will consist of stainless steel rollers with an estimated diameter of 15 in, separated by a self-lubricated composite bushing with an estimated diameter of 2 inches. Rollers will be fastened on the sides of the leaves parallel and perpendicular with the flow path. Quantity of rollers per leaf will be determined at 90% DDR.

7.8.1 HIW Hoist Motors and Brakes

- Hoist motors will be sized to not exceed 75% of their rated full load under LCA loading. This requirement is based mainly on lessons learned from projects where cold weather conditions increased the amperage draw of the hoist motor beyond full load values when starting cold. Electric brake motors will be rated class A continuous duty.

- Hoist motors will be selected to C-face mount to the primary reducers.

- Motors will be selected to have integral brakes with manual release devices installed inside the brake enclosure. Integral brakes help minimize the mounting and assembly requirements. Manual releases allow manual override of the brake in the event a failure of the release mechanism occurs. A mount drum brake would also be acceptable in the hoist design as it would reduce transducer extraneous leap issues.

- Motor brakes will be sized for 150% of motor rated torque per requirements of EM 1110-2-2610.

7.8.2 HIW Enclosed Gear Reducers

- A system of enclosed gear reducers will be used to obtain the required gear ratio. The system will be sized for continuous operation under LCA and sized not to exceed 75 percent of the yield strength under LCB.

- Enclosed gear reducers will be required to meet the American Gear Manufacturers Association (AGMA) requirements applicable to the type of gearing used in the reducer.

  - Reducers should be helical, herringbone, cycloidal, spiral-bevel, or a combination.

- Reducers will be selected based on nominal output torque, and checked for adequate mechanical and thermal power rating. A service factor of 1.0 for uniform loading will be applied.
- Gears will be AGMA quality 11 or higher.
- Units will be sealed from the environment and, where feasible, provided with hygroscopic desiccant breathers.
- A thermostatically controlled unitary of block heaters should be installed to maintain operational viscosity of oil, and reduce moisture within the enclosure. Maximum Watt density is 10 W/sq. in.
- Reducers will have drains/oil sampling ports with a valve, plug, and oil level dipstick or sight glass with graduation marks. The smallest graduation typically available in sight glasses for this application is 1/4".

7.8.3 HIW Coupling

- Couplings used in the drive train will be limited to gear or grid couplings which are considered to be reliable, heavy duty styles of couplings with high torque capacities.
- Couplings will be selected based on the load and speed ratings without additional factors of safety applied.
- Couplings for position indication devices and limit switch devices will be zero backlash motion control style. Coupling-to-shaft connections will have primary and backup features for transferring torque to the shafts to which they are mounted (typically compression fits plus a keyway for motion control couplings).

7.8.4 HIW Shafts

- Shafts will be sized to meet the general load case design criteria with stress concentrations included for keyways and other stress areas.
- The distortion energy theory will be used to determine the total state of stress at points of interest for components under combined loading.
- Under the LCA torque, shafts will be sized to meet the torsional deflection limit of 0.08 deg/ft and the bending deflection limit of 0.01 deg/ft required in EM 1110-2-2610.
- Shafts will be checked for thermal expansion effects to allow appropriate selection of coupling and bearings to accommodate thermal movement.

- Shaft guards, as required per 29 CFR 1910.219 Machinery and Machinery Guarding, will be provided in areas with personnel access to the line shaft, which is a high speed shaft that presents a significant personnel safety risk.

- Keyways and keys for the shaft will be designed per ASME B17.1.

7.8.5 HIW Bearings

- Anti-friction bearings will be selected to have a minimum L-10 bearing life of 75,000 hours under LCA. LCB will be checked against the bearing max load rating.

- Bearings will be sized for an axial load equal to a minimum of 15% radial load, which has proven successful in previous Tainter gate projects. It is conservative, but will account for unexpected misalignment or inappropriate installation allowances for shaft thermal expansions. The use of flat wheels with self-lubricating, self-aligning, spherical bushings has been successful at compensating for gate deflection at the ends. These are available in many bearing and lubricant combinations to suit a variety of applications. Self-lubricating, self-aligning, spherical bushings have been used successfully in offshore, industrial, and dam applications.

7.8.6 HIW Hoist Drums

- Custom designed components will be designed for LCA and LCB design criteria. Refer to Section 7.5.2 for LCA and LCB Criteria.

- The diameter will be sized to meet the D/d (diameter of the drum/diameter of the wire rope) bend ratio, as outlined in EM 1110-2-3200.

7.8.7 HIW Wire Rope

- One-inch diameter wire ropes will be selected to meet the LCA design criteria and the EM 1110-2-3200 max load criteria for LCB.

- The wire ropes will be stainless steel to provide resistance to weather and the submerged environment (302/304 SS).

- The wire rope construction will be selected based on Figure 2-6 (X-chart, found in EM 1110-2-3200) and what constructions are readily available. For most submersible gates, this results in the selection of 6 x 19 class, 6 x 25 IWRC, regular lay stainless steel rope.

- Wire rope end connections will be performed by poured spelter sockets (zinc or resin/epoxy), as they can accommodate the full breaking strength of the rope and custom rope sockets. Per EM 1110-2-3200 resin speltering is safer and requires less personal protective equipment and is more easily performed on site when compared to zinc speltering. A swage socket would also be acceptable for design. Each wire rope will travel over a sheave and down the gate slot. At each rope end connection will be a U-bolt. The U-bolts will be used as a means of tensioning the wire rope, during and after installation. Refer to Figure 7-10 for the U-Bolt type gate connection used from a previous projects.

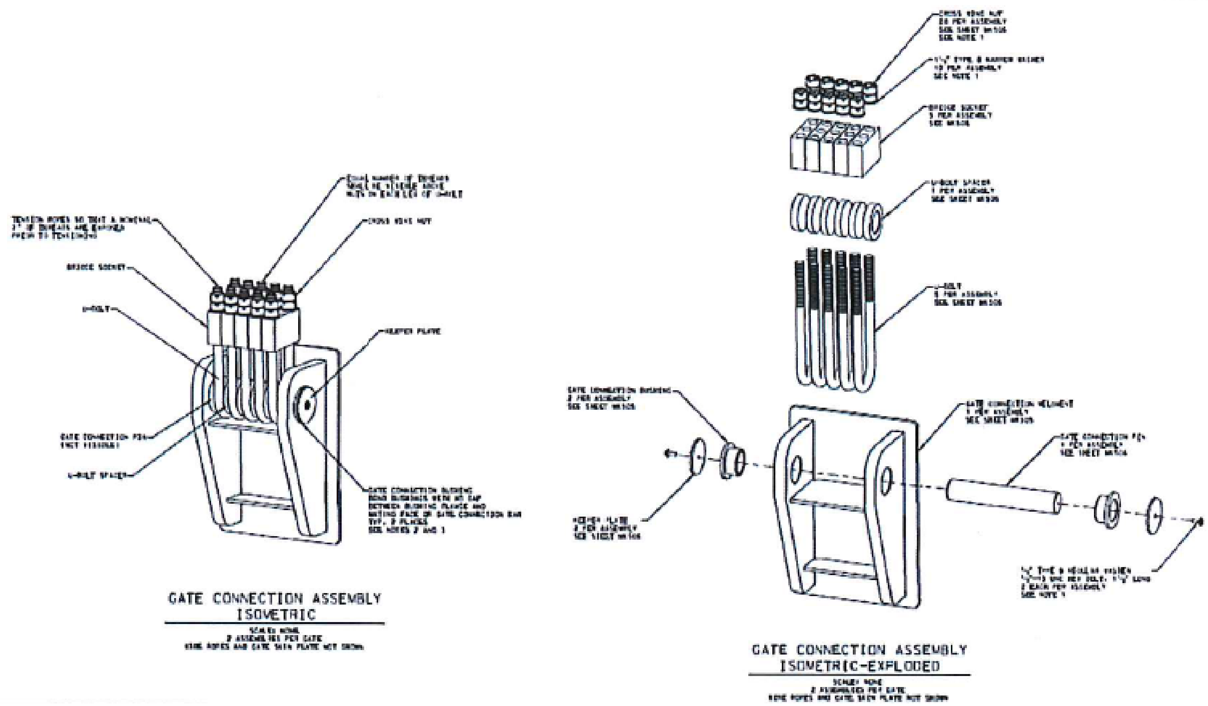

Figure 7-10. U-Bolt Rope Tensioning System

7.9 LIG

All four LIGs will be operated by a HPU. Each gate will be lifted and lowered by two horizontally mounted piston cylinders (one at each end). Each gate will have one stationary HPU placed in the center of each piston cylinder. Attached at the end of each hydraulic cylinder piston rod will be two stainless steel wire ropes. The wire ropes will travel over a sheave and down the gate slot. The HPUs will raise at a speed of 10 ft/min. Between the sheave and the end of the rod will be a turnbuckle. The turnbuckle will be used as a means of tensioning the wire rope, during and after installation. (Refer to the Figure 7-11 for LIG machinery isometric view.)

The piston cylinder will have a stroke length of 11 ft, meaning that to raise the LIGs to the machinery platform the gates would have to be fully lowered and then the wire ropes removed from the piston rod before a lifting beam can be lowered into the gate slot to pick up the gate. The wire rope attaches to the end of the piston rod, via a turnbuckle. See Mechanical Plate SKM103 in Appendix A for LIG attachment details.

All LIGs will have stainless steel rollers on the ends to assist in raising and lowering the gates (rollers have been selected over guides due to less friction). The roller assembly will consist of a stainless steel roller separated by a self-lubricated composite bushing. Quantity and sizing of the rollers per gate will be determined at the 90% DDR.

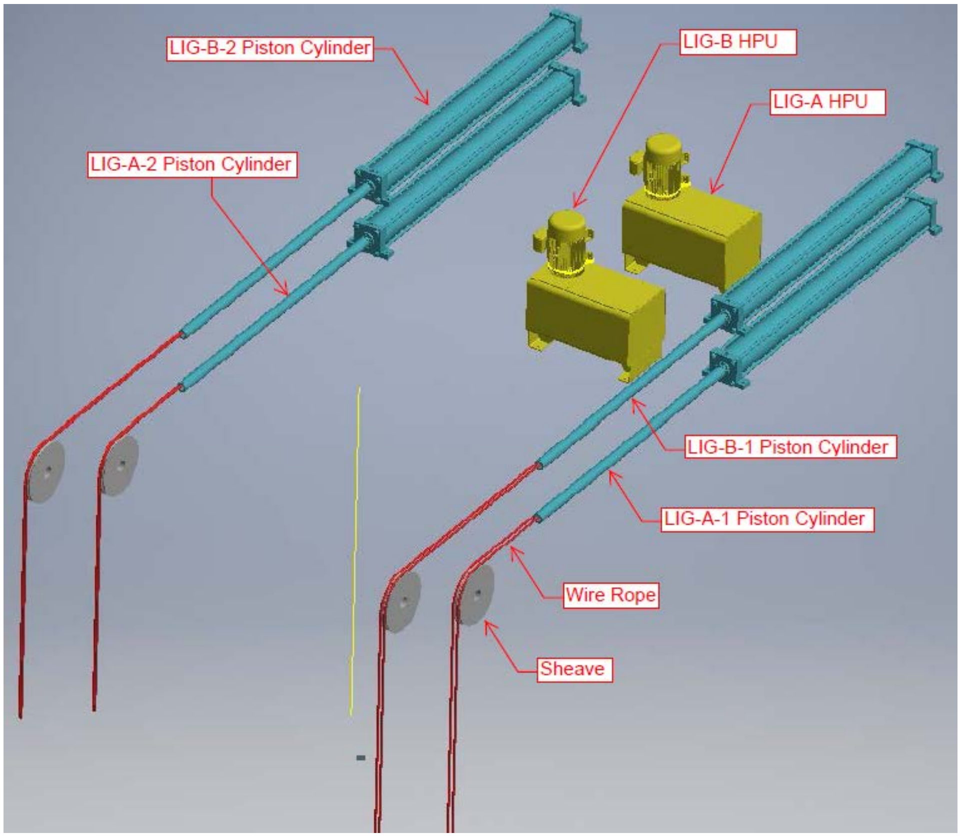

Figure 7-11. LIG Machinery Isometric View

7.9.1 LIG Hydraulic Piston Cylinder

- Eight (8) hydraulic cylinders will be furnished and installed to raise/lower the four (4) LIGs. The hydraulic cylinders shall be high pressure, non-cushion type, suitable for max shock operating pressure of 3000 psi.

- All cylinders will be double-acting, meaning hydraulic pressure is applied in both directions. The body of the hydraulic cylinder shall be manufactured by a suitable material that meets LCA and LCB pressure criteria.

- Piston rods will be stainless steel with an end attachment for wire rope.

7.9.2 LIG Hydraulic Pump

- A hydraulic pump will be used to provide flow-rate and pressure to the system.

- One pump will have the means of providing enough pressure for two cylinders. A second pump shall be provided as a back-up in the HPU.

- Pumps will be rated 150% of the highest working pressure.

7.9.3 LIG Motor

- LIG motors will be sized to not exceed 75% of their rated full load under LCA loading. This requirement is based mainly upon lessons learned from projects where cold weather conditions increased the amperage draw of the hoist motor beyond full load values when starting cold. Motors will be rated class A continuous duty.
- Motor brakes will be sized for 150% of motor rated torque per requirements of EM 1110-2-2610.

7.9.4 LIG Valves

The following valves will be used for the LIG HPUs.

- Directional control valve: to move and stop flow in any desired direction.
- Flow control valve: to regulate flow quantity traveling to hydraulic cylinder.
- Pressure control valve: to regulate pressure and prevent over-pressure to cylinder.
- Shut-off valve: to stop flow to cylinder. Used solely for emergencies or maintenance.

7.9.5 LIG Hydraulic Fluid Oil

The type of hydraulic fluid oil will be determined at 90% DDR and in accordance with EM 1110-2-1424, Lubricants and Hydraulic Fluids.

7.9.6 LIG Reservoir

Each HPU will be provided with its own reservoir. The reservoir will be sized for 3x the discharge rate of the acting cylinders. More salient features of the reservoir will be provided at 90% DDR.

7.9.7 LIG Piping

- A factor safety of 8 will be provided for all piping, based off the operating pressure.
- Hydraulic fluid flow-rate in the pipes will not exceed 10 cfs.
- Piping will be stainless steel.

7.9.8 Position Indication

Information regarding the position indication will be provided at 90% DDR.

7.10 OVERHEAD SWS CRANE

An overhead crane is required atop the SWS. The overhead crane will have a span of 66' and a wheel base of 35'. The main hoist will have 35 ton capacity with a 250' lift and a speed range variable between 10 ft/min and 0.1 ft/min. The main hoist will be mounted on a single trolley and the maximum allowable trolley speed will be 20 ft/min. The overhead crane will serve the following purposes:

- To transfer HIW segments from the HIW gate slot to a flatbed truck on the roadway

- To raise the LIGs to deck level for inspection/maintenance

- To lift the access hatch from the roof of the maintenance room

The overhead crane will likely be a performance spec and subcontracted out for design during EDC.

7.11 TRASH RACKS

NO TRASH CLEANING SYSTEM WILL BE PROVIDED FOR DETROIT SWS.

The LIG systems are assumed to be self-cleaning as debris will accumulate in the areas below the racks during times of zero flow. If a differential is seen at the LIG system, a shutdown may be required to allow debris to sink to the bottom. Since existing debris will be removed as part of the construction effort it is estimated there will be at least another 50 years of debris capacity at the base of the SWS.

For the HIWs, the FSS to SWS seal will serve as a debris deterrent into the SWS (See Figure 7-5 (page 7-6) for details). Only when the FSS is out of service for maintenance will a separate trash rack be required. Details of the secondary trash rack will be provided at 90% DDR.

7.12 PENSTOCK BIFURCATION

A custom pipe wye will be installed to allow the rerouting of penstock flow to the stilling basin. This feature essentially acts like a RO when flows through the powerhouse are not desired. Each penstock will separate and reduce to an 11-ft diameter pipe or two 8-ft diameter pipes. One or both of the penstocks will be modified. (See Section 4, Hydraulic Design for modification details and sketches.)

Each penstock modification will include a custom fabricated wye to minimize separation angle, to allow a long sweeping curve, and to provide room for three valves. The valves will include one penstock isolation valve or blank flange provided at the powerhouse, one knife gate just upstream from the discharge end of the pipe, and one energy dissipating valve placed at the end of the pipe. Discharge will be directed into the stilling basin. If each penstock is separated into two 8-ft diameter pipes then quantities will double, however, lead time and ease of fabrication may improve. Greater detail will be available in the 90% review.

7.13 MAINTENANCE ROOM

A maintenance room will be built on the deck of the SWS. This building will protect the HIW and LIG machinery (hoists and sheaves) from rain, debris, humidity and extreme temperature changes. The maintenance room will not cover the overhead crane, and will not extend to the road way (therefore, it should not impede logging trucks). Dimensions of the maintenance room and additional salient features will be provided at the 90% DDR.

For gate inspection, the maintenance room ceiling will have a hatch that can be removed by the crane. When removed, the hatch will provide enough clearance to allow each gate segment to pass through and be lowered onto a flatbed truck on the roadway. There will be one hatch per gate (four hatches total).

The proposed design arrangement will be selected and partially detailed for the 90% review. Details of the maintenance room are not provided in the 60% DDR Mechanical Plates.

SECTION 8 - ELECTRICAL DESIGN

8.1 GENERAL

This section presents the basic electrical components of the SWS. The primary electrical features are electrical service to the structure, electrical distribution for equipment at the structure, and control & indication of the mechanical equipment (HIW, LIG, and penstock bifurcation equipment).

8.2 REFERENCES

The electrical design will follow EMs, ERs, ETLs, TMs, and Industry Codes listed below where applicable.

EM 1110-2-2610, Mechanical and Electrical Design for Lock and Dam Operating Equipment, 30 June 2013.

National Fire Protection Association NFPA 70, National Electrical Code, 2017.

National Fire Protection Association NFPA 70E, Standard for Electrical Safety in the Workplace, 2018.

National Fire Protection Association NFPA 780, Standard for the Installation of Lightning Protection Systems, 2017.

The IESNA Lighting Handbook – 10th Edition.

UFC 3-310-04, Seismic Design for Buildings, 2013.

UFC 3-520-01, Interior Electrical Systems, 2015

UFC 3-550-01, Exterior Electrical Power Distribution, 2016

8.3 SEISMIC CONSIDERATIONS FOR ELECTRICAL EQUIPMENT

Typical seismic restraints for floor-mounted equipment will be required. Distribution transformers will be seismically tested, seismically qualified, and meet or exceed requirements of the Uniform Building Code (UBC) and International Building Code (IBC).

8.4 ELECTRICAL SERVICE

8.4.1 Description of Service Alternatives

Three alternatives were identified at the start of this project for providing power to the new SWS, which also accounts for the future connection of the FSS after the SWS is complete. The three alternatives include:

1. Add redundant 13.8kV feeds from new breakers at the existing SJ station service switchgear (refer to Plates E-2 and E-3 in Appendix H) in the powerhouse to new 15kV switchgear in the dam control house. The existing SJ station service switchgear is rated for 1200A at a 13.8kV operating voltage.

2. Add redundant 480V feeds from the existing SQ station service switchgear in the powerhouse to new 480V switchgear in the dam control house. The existing SQ station service switchgear is rated for 1200A at a 480V operating voltage.

3. Add redundant 480V feeds from the existing dam motor control center DQ1 to new 480V switchgear in the dam control house. The existing DQ dam motor control center is rated for 400A at a 480V operating voltage.

The total amount of power needed for the SWS and FSS combined is estimated to be approximately 3MW. The FSS will account for 2.5MW of the total load with the remaining 500kW needed for the SWS. Based on the amount of power needed, alternative 1 is the only viable option for providing service to the SWS and FSS due to the lack of available capacity in the existing equipment to be used by alternatives 2 and 3.

8.4.2 Proposed Service Alternative

The first alternative proposes to supply redundant feeders to the SWS by connecting to the 13.8kV system at the existing SJ switchgear located within the powerhouse (elevation 1213). The existing SJ switchgear will have to be reconfigured or replaced to incorporate two new XJ feeder breakers which would provide the redundant feeds to the SWS. The SJ switchgear modifications will be completed by the HDC. The redundant feeds will supply a new double-ended 15kV switchgear lineup ‘DSJ’ located in the existing control house at the top of the dam (elevation 1579.75). The feeders will be routed from the powerhouse to the control house using the existing vertical cable chase next to the elevator shaft, which goes from the bottom of the dam to the top. Space in the cable chase will be made available for the new 15kV feeders by removing the existing 480V DQ1 feeders from the powerhouse. The new 15kV switchgear will provide redundant feeds to the SWS substation ‘DSQ’ and redundant feeds for the FSS.

The SWS substation ‘DSQ’ will be configured as a double-ended switchgear lineup with two 13.8kV/480V, 3-phase, step-down transformers, feeding into the main-tie-tie-main configured 480V switchgear, located in the existing machine room at the dam (elevation 1569.0). The 480V switchgear will supply motor control centers and panelboards which will provide power, protection, and controls to all motor loads used at the SWS. DSQ will also provide power to the existing dam switchboard DQ1 which is currently fed from the station service 480V switchgear in the powerhouse.

8.4.3 Requirements

The following load list provides the basis of the estimate for the SWS. These loads have not been finalized, and are expected to change as the design progresses:

- HIW hoists: six hoists, each driven by a ##hp, 480V, 3-phase motor for a total of six (6) ##hp motors.
- LIG Hydraulic Power Units (HPU): four HPUs, each driven by a ##hp, 480V, 3-phase motor for a total of four (4) ##hp motors.
- Motor Heaters: one heater for each motor, brake, gearbox, and limit switch rated ###W at 120V for a total of ## heaters.
- SWS Crane: TBD.
- Fish Transport System: TBD.
- Penstock Bifurcation: TBD.
- Welding Outlets: outlets rated 60A, 480V, 3-phase, total quantity to be determined by final layout.
- Convenience Outlets: general purpose receptacles rated at 120V, total quantity to be determined by final layout.
- General Lighting: LED flood/site lights for exterior locations, and linear LED general purpose lights for interior locations.
- FSS: 2.5MW (determined by 90% FSS DDR).

8.4.4 Assumptions

- Equipment ratings, equipment layout, and available spare circuit breakers will need to be verified to support any design.
- Gate operation will be automatic based on water elevation, temperature and required flow. All ten motors could potentially operate simultaneously if an abnormal operation of the two powerhouse generating units initiates a rapid start-up, where the motors would operate to prevent the head differential between the forebay and SWS from exceeding 12 ft.
- Lighting loads will be considered continuous loads as defined by the NEC for the purpose of this study.
- The FSS will be provided under a separate contract after the SWS is built. Spare capacity for the future FSS is included in the SWS design.

8.5 ELECTRICAL DISTRIBUTION AND EQUIPMENT

Distribution: Power for equipment at the SWS will be distributed in a simple radial configuration at 480V to various loads through a motor control center, or panelboard, to supply combination starters, a lighting transformer, and 480V welding receptacles. A

120/240V panelboard will be provided for lighting, receptacles, and other small branch circuits.

Penstock Bifurcation: TBD.

Standby Generator: The SWS will be the source of water for the penstocks and the associated penstock bifurcation. Continuous operation of the SWS during loss of station service power is desired for passing water and also maintaining fish survival once the FSS is complete. It is assumed that the existing powerhouse and dam generators do not have capacity to serve the SWS and FSS standby loads during loss of station service power, therefore a new generator will be required to service the SWS and FSS. The PDT will need to determine an appropriate location for the generator that considers environmental and space restrictions.

Automatic Transfer Switch: An automatic transfer switch (ATS) will be required to sense the loss of normal station service power and automatically start the engine generator. When the generator reaches an acceptable voltage and frequency, the ATS will switch to the generator source in an open transition. The load will be served by the generator until the ATS detects the return of station service power to acceptable values.

Power Meter: A digital power meter at the 15kV service will be furnished to enable remote monitoring of power status and possibly load shedding to ensure standby generator capacity is not exceeded.

Grounding and Bonding: The electrical system will be grounded through a high resistance grounding system at each DSQ transformer secondary. The installation will comply with article 250 of the NEC, UFC 3-520-01, and UFC 3-550-01.

Raceways: Rigid galvanized steel conduit (RGS) will be required for all exposed interior and exterior work. Schedule 40 PVC rigid conduit will be specified for direct burial and for concrete encased applications.

8.6 ELECTRICAL FEATURES

Control System: A PLC system will provide automatic and/or remote control of the SWS. The PLC will also provide status/indication and alarming to the powerhouse control room. Touch screens located at the SWS and powerhouse control room will display local alarms and system status. An industrial field bus will be used to connect all sensors, switches, remote I/O, power meters, and other devices supporting the communication protocol. Process devices such as level sensors will require a 4-20mA transmitter, as field bus units are not commonly available. The processor power supply shall be backed up by a small uninterruptable power supply (UPS) so PLC operation will not experience disruptions during generator testing or short power failures. Local, manual push-button controls will be provided for each respective hoist on the SWS to facilitate maintenance and serve as a means for back-up controls to the PLC system.

Instrumentation: HIW positions will be determined using encoders or string potentiometers (exact type TBD) with rotary cam limit switches used for overtravel limits. LIG positions will be determined using string potentiometers connected to the hydraulic cylinders at the top of the SWS; roller lever limit switches attached to the cylinder will be used for overtravel. Level transmitters will also be required at the forebay and in the SWS wet well for measuring head differential.

<u>Supervisory Control and Data Acquisition (SCADA)</u>: The system will be incorporated into the existing SCADA system at Detroit Dam for remote control and/or monitoring of SWS system parameters and alarms. This will require coordination with HDC's Automated Control/Cyber Security (ACCS) branch. The Generic Data Acquisition and Control Systems (GDACS) Technical Advisory Board will be consulted for integration requirements.

<u>Cybersecurity</u>: The District's cybersecurity staff will be consulted throughout the design process to ensure compliance with the USACE civil works control systems' cyber security requirements.

<u>Communications</u>: Telephone service will be furnished for communications to the control room at Detroit Dam.

<u>Lighting</u>: General lighting will be provided using LED fixtures for the interior electrical/mechanical equipment rooms and the exterior spaces. Based on the IES Handbook recommendations, an average of 20 footcandles will be provided for the interior equipment rooms and an average of 5 footcandles will be provided for the exterior spaces.

<u>Security</u>: Door position switches will be provided for any exterior doors leading to interior spaces. Closed circuit television (CCTV) cameras will be provided for remote monitoring of the structure and surrounding area. The door position switches and cameras will be integrated with the existing dam security system for local and remote monitoring/alarming.

<u>Lightning Protection System</u>: A lightning protection system will not be provided, based on the Lightning Risk Assessment from NFPA 780, Annex L. The risk assessment will be re-evaluated at each design milestone based on changes to the structure.

SECTION 9 - ENVIRONMENTAL AND CULTURAL RESOURCES

9.1 GENERAL

This section addresses environmental and cultural resources and permitting requirements as they apply to the Detroit Dam SWS. This system will use a multilevel intake structure to modify the outflow water temperature to more closely match the natural cycle of water temperatures in the Santiam river. The natural cycle of water temperatures was altered when the Detroit Dam Project began operation in 1953. The change from the natural cycle disturbed the life cycles of the anadromous and native fish species downstream of the dam on the North Santiam River near Detroit, Oregon.

9.2 REFERENCES

DEQ (Oregon Department of Environmental Quality). 2000. NPDES permit. Application No. 977457. WQ File No. 64495. Salem, Oregon.

DEQ (Oregon Department of Environmental Quality). 2005. Erosion and Sediment Control Manual. GeoSyntec Consultants Project Number SW0106-01. April 2005. http://www.deq.state.or.us/wq/stormwater/escmanual.htm

DEQ (Oregon Department of Environmental Quality). 2008. Stormwater Management Plan Submission Guidelines for Removal/Fill Permit Applications Which Involve Impervious Surfaces. DEQ Northwest Region, Portland, Oregon. http://www.deq.state.or.us/wq/sec401cert/docs/stormwaterGuidlines.pdf

NMFS (National Marine Fisheries Service). 2000. Guidelines for electrofishing waters containing salmonids listed under the Endangered Species Act. NMFS, Portland, Oregon.

NMFS (National Marine Fisheries Service). 2008a. Endangered Species Act Section 7(a)(2) Consultation Biological Opinion & Magnuson-Stevens Fishery Conservation & Management Act Essential Fish Habitat Consultation on the "Willamette River Basin Flood Control Project". NMFS, Northwest Region, Portland, Oregon.

NMFS (National Marine Fisheries Service). 2011. Anadromous Salmonid Passage Facility Design. NMFS, Northwest Region, Portland, Oregon.

ODFW (Oregon Department of Fish and Wildlife). 2008. Oregon Guidelines for Timing of In-Water Work to Protect Fish and Wildlife Resources. ODFW, Northwest Region North Coast Watershed District

USACE (U.S. Army Corps of Engineers). 2002. Excerpted from the Civil Works Environmental Desk Reference. http://www.usace.army.mil/CECW/Documents/cecwp/envdref/2002ProfilesofLaws.pdf

USFWS (U.S. Fish and Wildlife Service). 2008. Final Biological Opinion on the Willamette River Basin Flood Control Project Endangered Species Act Section 7

Consultation on the Continued Operation and Maintenance of the Willamette River Basin Project and Effects to Oregon Chub, Bull Trout, and Bull Trout Critical Habitat Designated Under the Endangered Species Act. USFWS, Portland, Oregon.

9.3 ENVIRONMENTAL PLANNING

9.3.1 National Environmental Policy Act (NEPA)

All actions that are federally funded, permitted, or constructed must satisfy the requirements of the National Environmental Policy Act of 1969, as amended (42 U.S.C. 4321 et seq.). The project team should seek to avoid and minimize environmental impacts in the design and construction of the Detroit Downstream Fish Passage Project. To comply with NEPA, a draft EIS will be distributed for a 45-day public review and comment period. The draft EIS will address the alternatives analysis as well as the temporary and permanent environmental impacts associated with project elements. Major project elements are describe in Section 1.4. After the public notice period has closed, any comments will be addressed in the final EIS, and a Record of Decision (ROD) will be completed based on the assessment. No decision on a proposed action will be made until 60 days after notice of the final EIS availability has been published in the Federal Register by the EPA.

9.3.2 Endangered Species Act (ESA)

In accordance with Section 7(a)(2) of the ESA of 1973, as amended, federally funded, constructed, permitted, or licensed projects must take into consideration impacts to federally listed or proposed species. Listed species under the jurisdiction of the USFWS which may occur in Linn and Marion Counties include[4] (Threatened (T), Endangered (E), Proposed (P), or Candidate (C)):

- North American wolverine (P),
- Water howellia (T),
- Streaked Horned lark (T),
- Bradshaw's desert-parsley (E),
- Yellow-billed Cuckoo (T),
- Marbled murrelet (T),
- Nelson's checker-mallow (T),

[4] Source – Center for Biological Diversity - http://www.biologicaldiversity.org/programs/population_and_sustainability/T_and_E_map/

- golden paintbrush (T),
- Willamette daisy (E),
- Kincaid's Lupine (T),
- Northern spotted owl (T),
- Whitebark pine (C),
- bull trout (T), and
- Fender's blue butterfly (E).

Listed species under the jurisdiction of NMFS include:

- Upper Willamette River Chinook salmon (*O. tshawytscha*), and
- Upper Willamette River steelhead (*O. mykiss*).

The Detroit Dam SWS is incorporated in the July 11, 2008, NMFS and USFWS ESA Section 7(a)(2) Consultation BiOps on the "Willamette River Basin Flood Control Project". The Detroit Dam SWS designs should adhere to the NMFS 2011 Anadromous Salmonid Passage Facility Design Standards. Additionally, a summary identifying the potential amount and extent of take (defined as “to harass, harm, pursue, hunt, shoot, wound, kill, trap, capture, or collect or attempt to engage in any such conduct”) associated with construction and operation of the Detroit Dam SWS will be submitted to NMFS and USFWS. Even if the net effect of the Project is beneficial, the consultation pathway will depend on whether any of the effects qualify as "take" under the ESA. Based on conversations with NMFS General Counsel, even if the effects rise to the level of "take," NMFS currently believes take coverage can be provided through the existing BiOp rather than an individual consultation.

9.3.3 Magnuson-Stevens Fishery Conservation and Management Act (MSA)

In compliance with the Magnuson-Stevens Fishery Conservation and Management Act, an Essential Fish Habitat assessment will be prepared and included as part of the summary described under 9.2.b and sent to and reviewed by NMFS. Formal consultation was completed and incorporated in the above referenced 2008 NMFS BiOp.

9.3.4 Fish and Wildlife Conservation Act (FWCA)

To maintain compliance with the Fish and Wildlife Conservation Act (FWCA), input from the USFWS and state fish and wildlife agencies concerning this proposal is being provided through the WFFDWG and their review will be requested during the public notice comment period for the draft EIS. Further, the Detroit Dam SWS is being developed in close collaboration with NMFS and USFWS, and their staff has had, and will continue to have, input throughout the design of the facility. All elements of the project design should pass review by the resource agencies. Additionally, some requirements of this Act have been simultaneously addressed in conjunction with the ESA consultations referenced above. The project team did informally coordinate with the USFWS and NMFS on applicability of FWCA and they concurred with the Corps' determination.

9.3.5 Coastal Zone Management Act (CZMA)

This Act is not applicable to the Detroit Dam SWS due to its location being outside the geographic boundaries of the Act.

9.3.6 Marine Protection, Research, and Sanctuaries Act Title I (MPRSA) (Section 103)

This project will not involve ocean dumping or any other action impacting the marine environment. Therefore, coordination under this Act is not required for this proposed action.

9.3.7 CWA (Sections 401, 402, 404r, 404b (1))

A 404(b) analysis will be completed for this project. To comply with Section 404 of the CWA, dredge and fill activities proposed at the Detroit Dam TCS will require an individual State 401 WQC from the ODEQ for temporary and permanent impacts to wetlands and waters of the State. This requires submission of fees and a Joint Permit Application (JPA) for Removal and Fill, which is accepted by both ODEQ and the DSL. Because impervious surfaces are involved, the ODEQ 401 program also requires submission of a post-construction Stormwater Management Plan (SWMP) for permanent treatment of nonpoint discharge from the facility. The ODEQ has accepted specific design criteria from five manuals; these approved design manuals and the checklist of information that will be required in the SWMP are referenced in the ODEQ Stormwater Management Plan Submission Guidelines.

Temporary impacts to water quality should be avoided and/or minimized, to the greatest possible extent, during the Project's construction and staging. An Erosion and Sediment Control Plan must be developed and implemented in compliance with the Corps' existing general National Pollutant Discharge Elimination System (NPDES) 1200-CA permit issued by ODEQ for during-construction stormwater management. A guide for proper installation and maintenance of appropriate Best Management Practices (BMPs) for both uplands and in-water work can be found in the DEQ Erosion and Sediment Control Manual. Low Impact Development techniques that include infiltration and protection of existing soils and vegetation should be implemented wherever appropriate. As much as possible, site grubbing and clearing should be kept to the minimum required for the permanent project footprint.

Additionally, all in-water work will require a work isolation plan for control of turbidity and plans for fish salvage and exclusion. The plans will be submitted with the JPA and reviewed during ODEQ's WQC evaluation. The ODEQ often defers to the ODFW and

NMFS regarding appropriateness of proposed fish salvage and exclusion measures, and may require just a document of acceptability issued to the agencies from those organizations. Turbidity monitoring reports will be required during all in-water work.

The project will result in permanent impacts to wetlands and waters. These include: permanent fill and removal of in-water materials essential to constructing the SWS foundation. Changes to channel dynamics are expected to remain localized and should avoid inducing significant up or downstream channel or bank instability. An ODFW blasting permit will be required; blasting should be scheduled to occur during the in-water work window. As appropriate, additional BMPs should be applied to minimize impacts to listed species. This plan must address all contributing impervious areas and provide treatment designed per a DEQ-accepted manual or its equivalent. Impervious surfaces contribute to water quality degradation because they act as deposition and conveyance surfaces for accumulated air and traffic pollutants. Water quality treatment to avoid these impacts should be described in the SWMP.

Point source discharges for the facility operation will need to be covered under an NPDES permit issued by the DEQ.

Restoration of water quality function will be required to address the impacts to waters of the State. Restoration of riparian vegetation and stream banks must be reflected in a site restoration and enhancement plan to be included with the JPA. Although none are expected, any additional wetland impacts will also require mitigation. Any mitigation will be reviewed by DSL and DEQ when considering replacement of water quality function. The 2008 BiOp also describes water quality and habitat restoration measures that should be considered in the mitigation and restoration plan development. Opportunities to meet these obligations likely exist on site.

9.3.8 Clean Air Act (CAA)

Section 118 (42 U.S.C. 7418) of the Clean Air Act (CAA) specifies that each department, agency, and instrumentality of the executive, legislative, and judicial branches of the Federal Government (1) having jurisdiction over any property or facility or (2) engaged in any activity resulting, or which may result, in the discharge of air pollutants, shall be subject to, and comply with, all Federal, State, interstate, and local requirements respecting the control and abatement of air pollution in the same manner, and to the same extent as any non-governmental entity. Corps activities resulting in the discharge of air pollutants must conform to National Ambient Air Quality Standards (NAAQS) and State Implementation Plans (SIP), unless the activity is explicitly exempted by EPA regulations. Construction of the Detroit Dam SWS is anticipated to remain in compliance with the CAA and the SIP. This is not a transportation project, it will not qualify as a major stationary source of emissions of criteria pollutants, and the project does not appear to be located in a non-attainment area for limited air quality. Any emissions that do occur during and after construction from motor vehicles or facility functions are expected to be de minimis and from activities of a similar scope and operation to those of the original facility.

9.3.9 Applicable Local and State Statutes

Under the Clean Water Act, the Corps is required to comply with state and/or local requirements, including obtaining permits and paying reasonable service charges and respecting the control and abatement of water pollution. This will include obtaining a Section 401 WQC from the DEQ. The WQC will likely require that in-water work occur within the ODFW preferred time window, which for the North Santiam River above Detroit Dam is June 1 - August 31. Under State law, DEQ requires that the activity is compatible with local land use plans. This can be achieved if Marion and Linn Counties sign the City/County Planning Department Land Use Affidavit section of the JPA for the WQC. Under federal law, the Corps is required to comply only with the local requirements governing control and abatement of water pollution, and is not obligated to comply with local land use laws. Therefore, any requirements by the County must be based solely on water quality-related matters. The Corps may need to obtain a permit from the DSL for the discharge of fill material into waters of the United States. Any mitigation should be based on direct habitat losses, with the use of adaptive management (including monitoring) to ensure mitigation for wetlands incurs no net loss. The Corps should attempt to align any DSL requirements consistent with its own CWA Section 404(b)(1) evaluation of the impacts.

9.3.10 National Historic Preservation Act (NHPA)

Section 106 of the NHPA requires that federally assisted, or federally permitted undertakings, account for the potential effects on sites, districts, buildings, structures, or objects that are included in, or are eligible for inclusion in, the National Register of Historic Places. Detroit Dam was built in 1953 and is recommended eligible to the National Register of Historic Places. It will be necessary to ensure that project construction is consistent with “in-kind” maintenance of the structure and will not impact eligibility. Any proposed drawdown to an elevation below the minimum conservation pool of 1,450 ft has the potential to expose documented archeological sites and to reveal new sites. Exposed areas will need to be inventoried prior to construction and known archeological sites will need to be monitored to update site conditions to current State Historic Preservation Office standards. During a drawdown, law enforcement, or rangers, will need to increase patrols along the shoreline to guard against potential looting as sites are exposed. Consultation on the Area of Potential Effect, which is assumed to include the dam, staging areas, and areas exposed by the deep drawdown, will take place with the State Historic Preservation Office and the tribes.

SECTION 10 - CONSTRUCTION

10.1 GENERAL

This section presents the basic construction considerations, restrictions, and coordination of the major features for the Detroit Dam SWS.

10.2 SCHEDULE

10.2.1 General Information

Construction is scheduled to begin in fiscal year 2021. The current schedule allows for 3 years of construction. The construction sequence will be determined by the Contractor. The assumption is that the 3 years will roughly be divided into one year for preparation and excavation, one year to build the concrete structure, and one year to install the mechanical and electrical features. A Gantt chart schedule is located in the Cost Appendix.

10.2.2 In-Water Work

As outlined in the Oregon Department of Fish and Wildlife Guidelines for Timing of In-Water Work to Protect Fish and Wildlife Resources, the in-water work period above Detroit Dam is June 1 to August 31. All of the work for temperature control will be in or above the reservoir. This project will not be able to comply with the in-water work period.

10.3 RESERVOIR ELEVATION

To mitigate impacts to downstream water users the following reservoir operations are currently being considered:

- <u>Normal Operations</u>: Maintain dam operations as usual – In this scenario, the contractor will be required to construct the entire structure in such a way as to meet contractual obligations regardless of varying pool elevations.

- <u>One Year Drawdown to Elevation 1,400 ft</u>: After Labor Day the pool would be lowered to below the typical low pool at elevation 1,450 to Elevation 1,400 for approximately one year. The drawdown would be timed for the second year of construction, when the concrete structure is built.

The majority of construction will have to be completed in the wet with both options. The drawdown would reduce the depth of diving by as much as 250 ft, and could allow for some construction to be completed in the dry.

10.4 CONSTRUCTION METHODOLOGY

The Early Contractor Involvement acquisition strategy is being pursued for this project; this would bring the construction contractor onto the design team early in the P&S phase to advise the team on constructability.

Excavation will be similar to clamshell dredging but in deeper water, about 200 to 350 ft. Blasting will be required for rock removal. Underwater grade control will be developed for the 90% design.

Foundation construction could be completed by installing formwork like super sacks, concrete blocks, or steel, and then tremie placing concrete.

The concrete wet well could be constructed by precasting blocks, stacking them in position, and attaching the structure to the dam. The blocks could be precast on top of the dam and lifted into position by a crane staged on top of the dam. They could also be cast on a barge in the forebay.

Mechanical features will require special attention to ensure that the gates operate correctly over the full range of motion. To meet the tight tolerances, the gate guides for the Cougar tower were successfully installed as a secondary pour instead of as an imbed in the initial concrete pour.

The electrical features will be located above water on top the dam along with the mechanical equipment. Electrical installation will not be impacted much by in-the-wet construction.

10.5 DIVING

In-the-wet construction of the tower will require diving support for the majority of the construction period. Due to water depths, the saturation diving method will be required for the majority of these dives. A drawdown to Elevation 1,400 could allow for a several month period where the more cost effective mixed-gas diving method could be used.

10.6 CONTRACTOR OPERATIONS

Contractor Work, Office, Staging, and Parking Areas: The topography of the area both upstream and downstream of Detroit Dam is a steep canyon, making it difficult to find staging areas. The only work area directly adjacent to the proposed structure is the roadway on top of the dam and the parking lot to the north. Those two areas will have to be closed to public access during construction. Other possible staging and construction yard areas include the Detroit Dam operations yard, the Mongold boat ramp, and the Minto North area.

Detroit Dam Road and Parking Lot: The roadway on top of Detroit Dam is the most useful staging area to support construction. The area would likely be used to position cranes, concrete pump trucks, forklifts, and other material handling equipment. Trucks could then deliver material that is ready to be installed. The Corps has an agreement with logging companies to use this road to provide loggers access to the forest south of the Detroit Reservoir. Maintaining access for loggers would significantly reduce the benefit of using this staging area.

Detroit Dam Operations Yard and Road: The yard is located below the dam next to the powerhouse. Access to the area is acceptable for semi-load deliveries. The one-way transit time to the top of the dam is approximately 15 minutes for a loaded truck. The yard is about one acre and there are two wide spots on the access road that could add approximately two more acres.

Mongold Boat Ramp: The boat ramp is located 2.5 miles east of the dam; it is four miles from the dam on Highway 22 or by boat. This boat ramp is the most logical choice for deploying marine equipment to support construction. The boat ramp has parking for more than a hundred trucks with boat trailers. This area could be used to stage material and construction trailers. Access to this area will need to be coordinated with the USFS.

In addition to the parking area, at low water there is significant space available with a gentle slope toward the water. There is also a grassy area upstream of the Mongold parking lot that is potentially available. Using this area for construction and/or staging of materials would likely require reducing or closing the swimming area at Mongold.

Oregon Parks and Recreation estimates that approximately 110,000 people visit the Mongold boat ramp each year.

Minto North: The five-acre property due north of the Minto Fish Collection Facility was purchased by the Corps in 2010 to provide staging, earth disposal, and a septic location for the fish facility. There is one acre of usable space for staging material. The property is adjacent to Highway 22, 6.5 miles west of Detroit Dam.

Marine Equipment: It is anticipated that marine equipment will be required to support construction. Barges or modular flexi-floats could be used to provide staging areas adjacent to the dam and materials staging closest to the jobsite; however, unless just-in-time material deliveries are closely coordinated, this would still require an intermediate staging area. To reduce the impact to the Mongold boat ramp a new ramp could be constructed just south of the dam.

Environmental Controls: All Federal, State, and local laws and regulations will be complied with concerning this work. All runoff from construction site activities will be controlled with BMPs provided by the contractor and approved by the Government along with controls implemented under the NPDES permit and the Erosion and Sediment Control Plan (ESCP). Capture of job site runoff in retention ponds will allow settlement of sediments and removal of contaminants.

SECTION 11 - OPERATIONS AND MAINTENANCE

11.1 GENERAL

The Detroit SWS will be operated and maintained by the USACE as described in this section of the DDR.

11.2 FEATURES

This subsection describes the operations and maintenance (O&M) of the facility features. Reliability and maintainability of the SWS is being considered during design. Where possible, components/materials that reduce maintenance requirements and improve reliability are being selected. Components that require inspection, adjustment, or periodic replacement will be safely and easily accessible to maintenance personnel.

11.2.1 SWS Concrete Structure

This section will be completed for the 90% DDR.

11.2.2 HIWs

This section will be completed for the 90% DDR.

11.2.3 LIGs

This section will be completed for the 90% DDR.

11.2.4 Penstock Bifurcation

This section will be completed for the 90% DDR.

11.2.5 Mechanical Equipment

This section will be completed for the 90% DDR.

11.2.6 Electrical Equipment

This section will be completed for the 90% DDR.

11.2.7 Boat Ramp

This section will be completed for the 90% DDR.

11.3 FISH TRANSPORT

The FSS is currently being re-configured to hydraulically connect to an SWS that is attached to the dam. The fish transport section will be completed in the 90% DDR.

11.4 PERSONNEL ACCESS

The FSS is currently being re-configured to hydraulically connect to an SWS that is attached to the dam. The personnel access section will be completed in the 90% DDR.

11.5 DEBRIS MANAGEMENT

The debris management plan will be included in the 90% DDR.

11.6 OPERATIONAL COSTS

This section will be completed for the 90% DDR.

11.7 SAFETY

Components that will be used as isolation points for hazardous energy control will have provisions for installation of clearance locks/tags. New equipment will be placed in locations that do not restrict personnel access or require use of personnel fall protection equipment for normal operation or maintenance of equipment. Electrical and control systems will be checked and inspected every 5 years in accordance with manufacturer's specifications.

11.8 ENVIRONMENTAL

Environmental protection will be considered during design of the SWS. Risk of an oil/grease spill into the river will be mitigated, where feasible. When practicable, Environmentally Acceptable Lubricants (EALs) will be employed. Floating oil containment booms will be located in close proximity to the SWS to minimize reservoir contamination.

11.9 DOCUMENTATION

A System Operations and Maintenance Manual (SOMM) will be produced for the facility during the EDC phase. The SOMM will include as-built drawings of the SWS.

11.10 TRAINING

Prior to commissioning of the fish facility, project personnel will receive training on all aspects of O&M for the facility. Preliminary manuals and drawings will be on hand during the training.

11.11 COMMISSIONING

Commissioning tests will be performed following construction of the facility to verify all functional aspects are operational prior to placing in service.

SECTION 12 - COST ESTIMATES

12.1 GENERAL

This section presents the cost estimate for the Detroit Dam SWS, as presented in this report. The Total Project Cost (TPC, design and construction) estimated at the 60% DDR phase is $275 million. The construction contract, including escalation to the midpoint of construction and a 30 percent contingency, is estimated to cost $220 million.

12.2 CRITERIA

ER 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 Corps. 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.

12.3 BASIS OF THE COST ESTIMATE

The cost estimate is based on discrete costs for equipment, manpower, and materials where quantities and/or costs for such items can be assumed with reasonable confidence at this design level, and parametric unit costs where such assumptions cannot reasonably be made.

A formal Cost and Schedule Risk Analysis (CSRA) will be performed while this 60% DDR document is under review. Results and conclusions of the CSRA will be included in the 90% DDR report and the 30% contingency will be replaced with a risk based contingency calculated by the CSRA.

12.4 COST ITEMS

The major cost items are concrete placement to build the structure of the tower, the mechanical gate system, and the marine equipment required to construct the tower in the reservoir. Site conditions and in-the-wet construction is a significant cost driver and source of cost risk.

12.5 CONSTRUCTION SCHEDULE

It is anticipated that the total construction schedule will be approximately 3 years in duration. Additional information about the construction schedule can be found in Section 10.

12.6 ACQUISITION STRATEGY

The cost estimate assumes competitive pricing will be obtained by an unrestricted request for proposals with a best value trade off source selection.

12.7 SUBCONTRACTING PLAN

The cost estimate is based on the work being accomplished by a general construction contractor with marine construction experience as the prime contractor. It is expected that the contractor will self-perform clearing of the area for the foundation, construction of the foundation, and concrete placement. It is anticipated that the general contractor will subcontract design of contractor-designed features, metal fabrication, mechanical systems fabrication and installation, electrical systems fabrication and installation, coring or other borings into the dam, and diving.

12.8 PROJECT CONSTRUCTION

Access to the project is from Highway 22. This allows road access directly to the top of the dam for trucks, equipment, and personnel. Marine access to the project is from the Mongold boat ramp four miles past Detroit Dam on highway 22. From the Mongold boat ramp marine vessels will travel four miles in the reservoir to the upstream face of the dam.

Steel, concrete, aggregate, and other materials required for the project are readily available from commercial sources. The nearest established suppliers are in the Salem area, approximately 45 miles from the site. The cost estimate assumes concrete will be trucked to the site from concrete plants in Salem. Material is also available from Eugene (93 miles), Albany (55 miles), and Portland (90 miles). Specialty items such as actuators and electrical equipment will likely be shipped in from other states. The estimate assumes inclusion of the Buy American Act.

12.9 COST AND SCHEDULE RISK ANALYSIS

A formal cost and schedule risk analysis will be completed between the 60% and 90% DDRs.

12.10 FUNCTIONAL COSTS

<u>Planning Engineering and Design (30 Account)</u>: 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 (P&S), District review, contract advertisement, award activities, and engineering during construction. This effort is estimated to cost $26 million.

<u>Construction Management (31 Account)</u>: 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 $25 million.

Annual Operations and Maintenance: Annual operations and maintenance are estimated to be $25,000 and assumed to be similar to the Cougar temperature control tower O&M costs.

SECTION 13 - REAL ESTATE

13.1 GENERAL

This section identifies the minimum real estate requirements to complete the selected plan for the Detroit Dam SWS. The planning report documenting the minimum real estate requirements for the construction of the Detroit Dam and Reservoir Project is identified in the *Detroit Lake Real Estate Planning Report* dated August 30, 1946.

The Project is located in a transition area that includes residential, industrial, and rural areas. The Project contains 7,595.74 acres of fee, easement, withdrawn, permit, and license lands. The Corps' operations buildings are located below the dam structure. The total acreage of the Project includes the lands on which the Marion Forks Fish Hatchery and the Minto Egg Collection Station, outgranted to ODFW, and Packsaddle Park, outgranted to Marion County, are located. The major land use in the area is timber production.

13.2 PROJECT AUTHORIZATION EXISTING LANDS, EASEMENTS, RIGHTS-OF-WAY, RELOCATIONS AND DISPOSALS

As authorized by law, the Corps is responsible for the construction and operation of the Detroit Dam and Reservoir Project for the primary purposes of flood control, navigation, consumptive water use, and power production. In carrying out the authorized project functions, the Corps has jurisdiction over all Project areas, including National Forest Lands withdrawn from the public domain. The Department of Agriculture, USFS has jurisdiction over the use of the Corps' withdrawn lands for purposes extraneous to construction and operation of the Project.

To eliminate the overlapping of administrative jurisdictional responsibilities of the two agencies, the Corps and the USFS entered into an MOU on November 10, 1954, that remains in full force and effect today.

The MOU provides for the following administrative jurisdictional responsibilities:

- The Corps retains exclusive control of all waters for operation of the reservoir and all Project lands adjacent to and beneath the water surfaces to the extent required for execution of the functions related to the operation of the Project, including but not limited to flood control, navigation, irrigation and power production.

- The USFS may authorize occupancy and use of Project lands and waters, or the administration of their resources, for recreation or other purposes, provided such occupancy and use do not interfere with the Corps' authorized project purposes. The USFS has historically administered all recreational opportunities within the Project area.

- The MOU specifically withheld certain portions of the Project area for exclusive jurisdiction of the Corps. As identified in the MOU, "…downstream from the south line of the north half of the south half of Section 7, Township 10 South, Range 5 East, Willamette Meridian and bounded on the east by the west boundary of the North Santiam Highway (Highway 22)" are for the exclusive use of the Corps and the USFS may not authorize any use of these lands for recreation or other purposes.

The land area under exclusive jurisdiction of the Corps is outlined in red on Figure 13-1. The boundary for the Detroit Reservoir Project is shown in the insert map outlined in black.

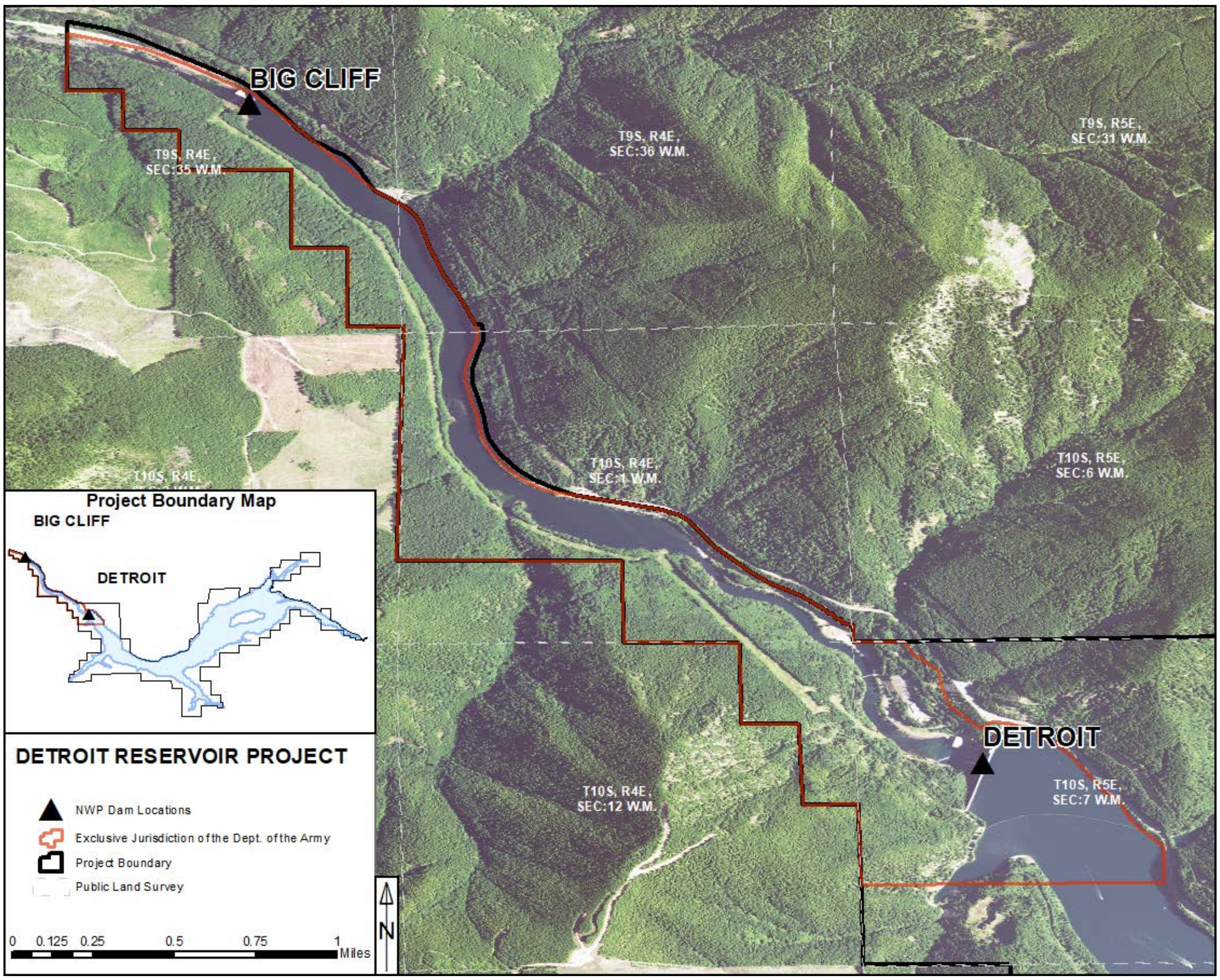

Figure 13-1. Detroit Reservoir Project Boundary Map

13.3 CONSTRUCTION FEATURES AND RIGHT-OF-WAY REQUIREMENTS

Prior to issuance of the solicitation for a construction contract, the District Chief of Real Estate is required to certify, in writing, to the district element responsible for the solicitation, that sufficient real property interests are available to support construction pursuant to the contract.

SWS:

For the construction of the SWS attached to the dam, right-of-way (ROW) requirements are held in fee by the United States of America and under the exclusive administrative jurisdiction of the Corps.

Temporary Staging Areas for Construction:

The PDT identified the Northwest Visitor Parking Lot for the construction staging area. ROW requirements for the Northwest Visitor Parking Lot are held in fee by the United States of America and under the exclusive administrative jurisdiction of the Corps.

The Real Estate Division contacted ODOT on May 31, 2018, to inquire about an ownership or easement interest to the Northwest Visitor Parking Lot. The ODOT stated that they have historically maintained the parking lot, to include plowing, sweeping and lighting. However, ODOT was not able to produce an ownership or easement interest from the Corps or the USFS to validate an interest in the Northwest Visitor Parking Lot. To maintain a good working relationship with ODOT, it is recommended the Real Estate Division continue communication and to advise them of the closing of the Northwest Visitor Parking Lot for the construction area staging needs.

The PDT identified the Dam Operations Yard as a second construction staging area. ROW requirements for the Dam Operations Yard are held in fee by the United States of America and under the exclusive administrative jurisdiction of the Corps.

Potential Boat Ramp Locations:

North side – The PDT identified a potential boat ramp location on the north side of the reservoir. ROW requirements for the potential boat ramp location on the north side of the reservoir are held in fee by the United States of America and under the exclusive administrative jurisdiction of the Corps.

Optional south side – The PDT identified a potential boat ramp location on the south side of the reservoir. The Corps retains exclusive control of all waters for operation of the reservoir and all Project lands adjacent to and beneath the water surfaces to the extent required for execution of the functions related to the operation of the Project; including, but not limited to, flood control, navigation, irrigation and power production.

The Real Estate Division will coordinate the exact location of the optional boat ramps with the USFS to minimize the potential for conflict with public use of this area.

Spoil Disposal Areas:

The PDT plans to use spoil and excavated rock to construct the necessary boat ramps required for construction. If additional spoil areas are needed, Government-controlled or owned lands at nearby Willamette Valley Project sites will be used.

General Access:

The Project is accessible by State Highway 22 which extends through the area. Construction activities will require temporary closure of the road across the dam for an extended period of time. There are many different types of users who cross the dam access road on a regular basis (for uses of the USFS, fire suppression, logging, and recreational) to access Kinney Creek Road (NF-2212). The Corps granted an indefinite permit to the USFS for use of the roadway across the dam, and the Corps will need to coordinate the temporary closure with the USFS.

The impacts of temporarily closing the road across the dam require an additional route for users to cross the river and reservoir. The PDT identified an alternate route south of the dam, known as the old southern access road, to accommodate the dam access road closure. Details of the road are located in Section 14.4. ROW requirements to rehabilitate the route and bridge crossing are held in fee by the United States of America.

SECTION 14 - CIVIL DESIGN

14.1 GENERAL

This section describes the criteria and design considerations for the civil work. The civil project features are shown on C-001 in Appendix A. These features include construction staging areas, a rehabilitated access road, a proposed boat ramp, and debris booms. The scope for civil design for this project includes delineation of site access, haul routes and construction staging areas, temporary environmental controls, and improvements to the site needed for construction, as well as daily operations and maintenance of the facilities.

14.2 REFERENCES

The civil design conforms to the following federal and state reports, regulations, and standards.

USACE, 2004. Recreation Facility and Customer Service Standards, EM 1110-1-400. November 1, 2004.

USACE, 2008. Safety and Health Requirements Manual, EM 385-1-1. September 15, 2008.

USACE, 2009. Policies for Referencing Project Elevation Grades to Nationwide Vertical Datums, Engineering Regulation, ER 110-2-9160. March.

U.S. Department of Transportation, Federal Highway Administration, 2009. Manual on Uniform Traffic Control Devices for Streets and Highways.

State of Oregon Department of Environmental Quality (ODEQ), 2013a. Construction Stormwater Best Management Practices Manual, 1200-C NPDES General Permit. March.

ODEQ, 2013b. Construction Stormwater Erosion and Sediment Control Manual, 1200-C NPDES General Permit. January.

U.S. EPA, 2011. Environmentally Acceptable Lubricants, EPA 800-R-11-002. November 2011.

14.3 SURVEYS AND COORDINATION SYSTEM

The topographic and hydrographic data used in the design of the SWS were derived from various upland topographic surveys and reservoir hydrosurveys conducted between 2009 and 2017. A list of the surveys with technical details are provided in Appendix A, Sheet G-003 (Survey Plan) and Sheet G-004 (Survey Narrative). Control points and details are also identified on the Survey Narrative.

The vertical datum for the project is referred to as the Detroit Dam Project Datum and is 4.23 ft below the North American Vertical Datum of 1988 (NAVD 88). The elevation conversion from NAVD 88 to the Project Datum, in feet, is as follows:

Elevation in NAVD 88 – 4.23 = Elevation in Project Datum

The horizontal coordinate system for the design is based on the State Plane Oregon North FIPS 3601 with units of U.S. survey feet. The original survey and as-built drawings were based on the North American Datum of 1927 (NAD 27). The more recent topographic, hydro-, and LiDAR surveys were based on the North American Datum of 1983 (NAD 83).

14.4 HAULING AND SITE ACCESS

The Detroit Dam Powerhouse and the top of dam are both accessed directly from North Santiam Highway No. 162, Oregon Route 22 (OR-22). OR-22 is a primary route between Interstate 5 (I-5) in Salem, OR and Central Oregon. For the first 13 miles east of I-5, from Salem to Stayton, North Santiam Highway is a four-lane divided freeway. For the remaining 30 miles, from Stayton to the Detroit Dam, OR-22 is classified as a Rural Principal Arterial highway by the Federal Highway Administration (FHWA) and ODOT. Additionally, OR-22 is designated as a freight route by ODOT, signifying that the annual tonnage hauled by truck on this route is moderate to high and the highway provides connectivity to freight generating areas. It is therefore expected that large trucks hauling materials and equipment to the project site will encounter similar vehicles along the route.

OR-22 is classified as a Group 1 highway by ODOT. The largest trucks that can transit along this highway without a permit are 60-ft long truck-tractors with semi-trailers, 65-ft long truck-tractors with stinger-steered pole trailers, or 75-ft long truck-tractors with multiple semi-trailers. The maximum height and width of non-permit vehicles is 14 ft and 8.5 ft, respectively, while the maximum permissible weight is 80,000 pounds, with no more than 20,000 pounds on any one axle or 34,000 on any tandem axle. Exceptions can typically be made with an oversize vehicle permit on a case-by-case basis. A review of weight restricted bridges has not revealed any impediments for hauling to the project site. The only restricted bridge along the haul route from I-5 to the Detroit Dam project site is over the Santiam River in Mill City; however, this bridge is located off of the OR-22 mainline and does not restrict standard or permitted vehicles on the highway.

14.5 CONSTRUCTION TRAFFIC

Construction traffic and haul roads around the Detroit Dam will be in compliance with the safety and health requirements specified in EM 385-1-1 (USACE, 2008). This manual specifies use of the Manual of Uniform Traffic Control Devices (U.S. Department of Transportation, 2009) for highway construction signage. The construction contractor will be required to prepare and implement a traffic control and safety plan that will address access into the project site and staging areas, as well as construction traffic entry and exit points onto OR-22 and other public roads. Measures will be taken to minimize impact on the local traffic and recreational uses of the area.

14.6 ROADWAY IMPROVEMENTS

14.6.1 Old Southern Access Road

The roadway on top of Detroit Dam is used to gain access to Kinney Creek Road (NF-2212) and the forest south of Detroit Lake. USACE has an agreement with the USFS that covers the Forest Service's right to repair, maintain, use, control, and improve Kinney Creek Road. The construction of the SWS at the Detroit Dam will result in the temporary closure of access to NF-2212 across the top of the dam. As regular access to the area south of Detroit Lake needs to be maintained for forest service personnel, commercial logging vehicles, and public recreation, an alternate route onto NF-2212 south of the dam must be made accessible during construction. The only viable option in the vicinity of Detroit Dam is the rehabilitation of an old construction road located near the southern abutment on the downstream side of the dam, shown on Appendix A C-001 (Site Plan). This old southern access road can be reached via the Detroit Downstream Access Bridge over the North Santiam River located to the west of the Detroit Dam powerhouse yard. According to the Draft Design and Load Rating of the Detroit Downstream Access Bridge report, this bridge "does meet criteria for AASHTO and Oregon State legal loads therefore no load posting is required." The bridge structure is safe for infrequent use by non-permit USFS vehicles and commercial logging trucks, but may require further investigation if it is to be used for the mobilization of logging equipment, large firefighting equipment, or daily construction traffic. Site work may be required due to the limited space and steep vertical curves of the two bridge approaches.

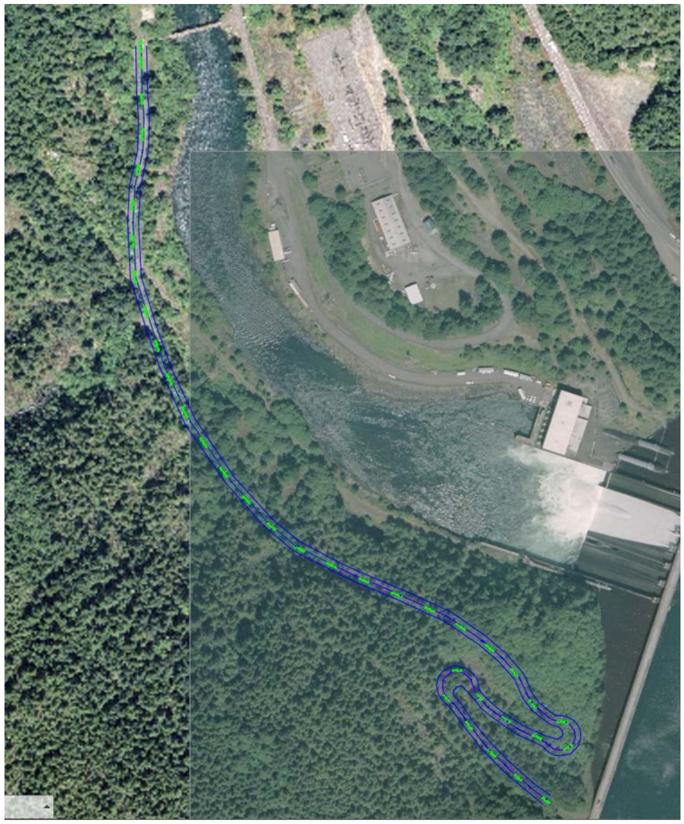

Figure 14-1. Old Southern Access Road

A table-top evaluation of the current condition of the old southern access road and the feasibility of rehabilitation was conducted in August 2018. A report documenting this evaluation is presented in Appendix F, Civil Calculations and a detailed plan view can be found in Appendix A CS-101 (Southern Access Road Rehabilitation). The report concluded that the old southern access road can be realigned and rehabilitated to provide alternative access to NF-2212 for logging trucks equivalent in size to interstate semi-trailers (WB-62). The rehabilitated road would drop approximately 353 ft in elevation over 3,450 linear feet, for an average grade of 10.2%. However, the grade varies from as little as 3% to as high as 13% over the length of the roadway, depending upon existing topography, so as to minimize earthwork volumes. It is estimated that the rehabilitation of the old southern access road will require 3,450 cubic yards (CY) of cut and 10,700 of fill, for a net total of 7,250 CY of fill. The majority of the fill is needed to construct two switchbacks and to repair an eroded slope near the midpoint of the roadway.

14.6.2 Proposed Boat Ramp

As the nearest boat ramp is over 4 lake-miles away from the project location and is heavily used by the public, a new, project-dedicated boat ramp may be constructed in the vicinity of the dam. This boat ramp will be used for site access during construction as well as maintenance and crew access once the FSS is operational. Additionally, the boat ramp will be critical infrastructure if an amphibian vehicle is used to transport fish around the dam. Four locations and alignments were considered for boat ramp construction assuming the same general design parameters: 15-ft width, maximum grade of 15%, preferred side slopes of 1.5H:1V with maximum side slopes of 1H:1V for fill, and widened turnouts every 200 linear feet for safety purposes and to aid in debris removal. All boat ramp options started at the relative elevation of the roadway and ended at an elevation of 1400 ft for continued access at the low power pool of 1425 ft. If net fill is required, construction of the boat ramp can serve as the disposal area for an estimated 22,500 CY of overburden and rock that will be excavated to create the foundation for the SWS and FSS docking area. The four boat ramp options are described in detail below and presented in Appendix A C-001 (Site Plan) and CS-102 (Boat Ramp Alternatives). A more detailed analysis of the boat ramp options is presented in Appendix F, Civil Calculations.

14.6.2.1 Option 1: South Boat Ramp near Cumley Creek

Boat Ramp Option 1 lies partially within a ravine on the southern shore of Detroit Lake. Currently, Cumley Creek enters the reservoir through a culvert approximately 35 ft beneath Forest Service road NF-2212. Because of the need to allow flow from Cumley Creek to enter the lake during low pool, it is not feasible to construct the boat ramp over the entire length of the ravine to the limits of the lake. Instead, the proposed boat ramp follows the trajectory of the Cumley Creek ravine then angles to the west and intersects NF-2212 approximately 500 ft north of the existing culvert. The proposed boat ramp starts along NF-2212, approximately 1/2 mile by road from the Northwest Visitors Parking Lot at an elevation of roughly 1,600 ft. The ramp proceeds at a constant 14.2% slope for 1,400 linear feet, where it terminates approximately 375 ft from the upstream side of the Detroit Dam, on the south side of the spillway. It is estimated that a net fill of 155,000 CY of material is needed to construct Boat Ramp Option 1. The primary advantages of this option include the simple alignment, beneficial disposal of all excavated rock, and the minor amount of excavation required. The disadvantages are the large additional volume of fill required after disposal of excavated rock, and that all boat traffic would have to traverse in front of the spillway to access the project area.

14.6.2.2 Option 2: South Boat Ramp on Old Construction Road

The second boat ramp option follows the alignment of an old dam construction access road on the south side of Detroit Lake. This proposed boat ramp begins along USFS road NF-2212, approximately one mile by road from the Northwest Visitors Parking Lot. Based on available topographic maps, it appears that the old construction access road intersects NF-2212 approximately 400 ft to the southeast from the edge of the reservoir where the elevation is approximately 1,675 ft. This would require the rehabilitation of 400 ft of the construction road with a grade of approximately 15% to get from NF-2212 to the edge of the reservoir. Once in the reservoir, the proposed boat ramp runs to the north-northwest for 500 ft, then turns west for the next 500 ft, where it then turns north again. The alignment of the final 700 ft of Boat Ramp Option 2 is identical to that for Option 1. The average grade for the total length of the boat ramp is 10.9%. A large diameter culvert approximately 100 ft long will be installed around station 9+50 of the boat ramp to allow for flow from Cumley Creek to enter the reservoir during low pool. Like Option 1, the boat ramp terminates approximately 375 ft from the upstream side of the dam, on the south side of the spillway. It is estimated that a net fill of 41,000 CY of fill material is required to construct Boat Ramp Option 2. The advantages of option 2 are that it will reuse the old construction access road, provide disposal of all excavated rock, and require only minimal excavation. Disadvantages include steep side slopes along the first 500 linear feet, the additional road rehabilitation needed to intersect with NF-2212, and that boat traffic must cross in front of the spillway to reach the project area.

14.6.2.3 Option 3: North Boat Ramp with Straight Alignment

The third option is to construct the boat ramp in a straight line along the north shore of Detroit Lake. Boat Ramp Option 3 starts at an existing vehicle pullout along OR-22 North Santiam Highway, roughly 1,500 ft to the southeast of the Northwest Visitors Parking Lot, at an elevation of approximately 1,590 ft. The ramp proceeds at a steady 14.6% downgrade over 1,300 linear feet to the northwest until it achieves its final elevation. Although relatively straight, the centerline of the ramp follows the existing topography in an attempt to eliminate large cuts or fills, which results in several minor turns along the length of the boat ramp. The upslope (north) side of the ramp is entirely in rock cut while the downslope (south) side of the ramp is entirely in fill. Steep side-slopes are necessary on the downslope side of the ramp due to the existing grades in the area. The boat ramp terminates approximately 375 ft from the upstream side of the Detroit Dam, on the north side of the spillway. It is estimated that approximately 1,700 CY of rock will be excavated and 8,600 CY of fill will be required to construct the boat ramp, for a net fill of approximately 6,900 CY. Advantages for Boat Ramp Option 3 include the small volume of earthwork required compared to the southern options and boat traffic will not need to traverse in front of the spillway to access the project area. Disadvantages include steep side slopes along the entire length of the ramp, the need for additional disposal of excavated rock, and that construction would impact traffic along highway OR-22. Additionally, the ramp alignment passes below the location of at least one known slide, which could both impact ramp construction and affect the stability of the highway above.

14.6.2.4 Option 4: North Boat Ramp with Switchbacks

Boat Ramp Option 4 is located on the north shore of Detroit Lake, immediately to the north of the SWS. The ramp starts at the existing Northwest Visitor Parking Lot, at an elevation of approximately 1,580 ft., and heads directly down the north slope of the lake using switchbacks to stay within the vicinity of the project area. The ramp proceeds downward at a constant grade of 14.4% for approximately 1,250 linear feet, until it reaches the final elevation. Two switchbacks are located from stations 4+00 to 5+00 and 8+50 to 9+50, so that the ramp alignment never ventures further than 450 ft to the east from the starting location and does not proceed beneath the known slide that is approximately 725 ft to the east of the upstream dam face. Both switchbacks are constructed by turning the road slightly into the existing slope so that grade can be maintained while reducing the amount of fill necessary. All turns were constructed with a centerline turning radius of 30 ft. Outside of the switchbacks, the ramp centerline alignment roughly follows the existing topography so as to minimize large cuts and fill.

The upslope (north) side of the boat ramp is entirely in rock cut and the downslope (south) side of the ramp is entirely in fill. A steep side-slope is necessary on the downslope side of the ramp due to the existing grades and the need to construct a boat ramp further down the slope below the fill. The boat ramp terminates approximately 475 ft from the upstream side of the Detroit Dam, on the north side of the spillway. In total, it is estimated that approximately 4,400 CY of rock will be excavated and 4,800 CY of fill will be needed to construct this boat ramp, which results in a net fill of 400 CY. The major advantages for Boat Ramp Option 4 include a balanced cut-fill approach and boat traffic will not need to cross in front of the spillway to access the project area. Disadvantages include steep side slopes, potential impacts to highway OR-22, and the loss of a disposable area for excavated rock.

14.7 POSSIBLE CONSTRUCTION STAGING AND CONCRETE BATCH PLANT AREAS

It is assumed that primary construction staging will occur in the roadway atop the Detroit Dam, as this will be closed to the public and is immediately adjacent to the SWS footprint. Several other possible construction staging and concrete batch plant areas were identified early on in the design process. The more viable areas are described below. Additional in-depth evaluations of these areas will need to be conducted as the design progresses to select the site that most closely meets the Project objectives and has the least impacts to the environment, cultural resources, the local communities, and the recreational uses of the reservoir. The site evaluations will include, but are not limited to, the following factors:

- Ability to obtain access agreements or easements for properties not owned by USACE
- Environmental impacts and mitigation requirements
- Cultural resource impacts and mitigation requirements
- Available size
- Site improvement requirements and costs
- Distance to/from the job site
- Impacts to recreational uses of the reservoir and surrounding lands, especially for sites that may need to be closed or have limited access to the public
- Impacts associated with increased construction traffic and road closures
- Impacts to the local communities and their economy, if public sites are closed or limited during the entire construction duration

14.7.1 Northwest Visitor Parking Lot

The Detroit Dam visitor parking lot immediately adjacent to OR-22 is a potential staging area. It is located approximately 200 ft to the north of the SWS footprint and directly connected to the road atop the Detroit Dam. There are approximately 0.55 acres available for contractor offices and parking, and equipment storing and staging. The parking lot is paved so there would be minimal site improvements needed to ready the site for construction staging use. However, the parking lot may be widened and extended on the eastern end to provide a platform for fish transport equipment or electrical equipment storage.

Figure 14-2. Northwest Visitor Parking Lot

14.7.2 Original Precast Yard

The precast yard used during the original dam construction is also a potential site for a construction staging area. This area is located southeast of the dam, approximately 1,500 to 2,000 ft south of the SWS construction area. As the reservoir bottom in this area varies from 1,575 ft to 1,475 ft, a large portion of this area is typically dry during the fall/winter pool levels. There may be more than 6.5 acres available for staging in this area, though the actual space available is dependent upon water levels. Moreover, the area has steep slopes which would also limit the available area for construction staging to roughly 3 acres or less without a major re-grading effort. Some road improvements and site clearing and grading will be needed to prepare the site for staging. In addition, a temporary cofferdam would likely be needed to isolate the staging area from the reservoir and to prevent flooding. Use of original precast yard for construction staging could be done in conjunction with construction of either of the two south boat ramp options to reduce overall earthwork costs.

Figure 14-3. Precast Yard – Dry at Fall/Winter Pool Levels

14.7.3 Detroit Dam Operations Yard

This area is downstream of the dam and located outside of the powerhouse security fence. Approximately two acres would be available for staging. Some road improvements and site clearing and grading may be needed to prepare the site for staging. The yard is located approximately 2,000 ft west of the SWS construction area, but the driving distance from this potential staging area to the job site is over 2 miles as the access road from this staging area to OR-22 is approximately 0.8 miles, and the dam crest is another 1.25 miles from the junction.

Figure 14-4. Detroit Dam Operations Yard

14.8 STAGING AREA REQUIREMENTS

Vehicle staging, cleaning, maintenance, refueling, and fuel storage will take place in a vehicle staging area located as far as possible from the reservoir, river, and any wetlands. BMPs and other spill prevention measures will be implemented to prevent and minimize any releases to the environment. The staging locations will avoid all designated historic properties and archaeological sites. All disturbed areas will be returned to their pre-construction condition at the completion of construction.

During construction, the contractor will provide temporary security fencing or other security measures around the established staging and construction areas to prevent public access and theft. The length of the fencing required will depend on final staging area configuration. Signage and mobile lighting around the work area will be the responsibility of the contractor. Security measures might also be required around the construction area to keep the public away.

14.9 DEBRIS BOOM

Debris management on Detroit Lake currently consists of a single floating boom to block surface debris from reaching the dam. This boom is in poor condition and in need of replacement. The existing debris boom does not extend below the water surface, so debris can move under the boom and float downstream until it reaches the upstream dam face. The primary method of removing debris from Detroit Lake is to open the spillways during high pool and allow the debris to pass through the dam.

Further analysis is being done to determine if another design of debris boom could be more effective at stopping floating and submerged debris from reaching the dam face and the proposed SWS and FSS, where debris may be more harmful to operations. Additionally, dependent upon the preferred boat ramp option, the debris boom may need to be relocated or divided into multiple booms to allow for boat access to the FSS. Appendix A, Sheet C-001, shows the potential anchor locations for the primary and secondary debris booms if either of the south boat ramp options is selected. To provide added debris removal, the inclusion of an auxiliary debris boom further upstream near the FSS is also being investigated. The final debris boom design and anchor locations will be determined as part of the overall debris management plan for the project.

14.10 STORMWATER POLLUTION PREVENTION AND EROSION CONTROL

USACE's NPDES Construction General Permit (CGP) 1200-CA from the ODEQ addresses stormwater discharges from construction sites of one acre or more. The permit requires that an ESCP be prepared before construction begins. The ESCP will be prepared in accordance with ODEQ guidance (ODEQ, 2013a and 2013b). The ESCP is generally completed by the construction contractor as a pre-construction submittal and describes the measures, including BMPs, to be implemented during construction to control erosion, prevent sediment discharges in stormwater, and minimize the potential for hydrocarbon or chemical contamination of site soils and water bodies. The ESCP will address all areas of disturbance from the construction activities, including equipment staging, material stockpiling, and the concrete batch plant. The contractor must comply with all conditions of the CGP and implement the ESCP. The ESCP will be kept on the site and be updated by the contractor as needed.

Erosion and sediment control BMPs will be implemented to stabilize exposed areas and contain runoff, such as the installation of silt fencing to ensure that sediment from construction activities is prevented from entering wetlands or the surrounding water bodies. Stormwater will be collected and sediment removed before being released to the reservoir or river. Disturbed work areas will be mulched, and inactive material stockpiles will be covered during rains that produce runoff. If any disturbed ground and stockpiles are held over the winter, they will be protected with fiber-bonded mulch or similar methods to prevent erosion. These sediment and erosion control measures will be maintained and replaced as necessary until construction is completed and permanent vegetation and storm runoff control measures are established and effective.

Other BMPs that will likely be implemented include containment of equipment fueling areas and locating these areas as far from wetlands or waters as possible to prevent discharges in the event of a spill. Daily inspections of the fueling area and construction equipment will occur to ensure there are no leaks. Oil absorbing pads, drip pans, or similar devices will be placed beneath the equipment when working in waters or staged overnight to catch any leakage. Fuel spill control devices, such as a Wiggins Fast Fuel system, or equivalent, will be used. Equipment hydraulic fluids will be substituted with biodegradable fluids as appropriate.

Special construction measures will be required when working above/near water to prevent pollutant discharges, such as the use of EALs on construction equipment and machinery (EPA, 2011). These requirements will be developed during the P&S stage of design.

Section 9, Environmental and Cultural Resources, of this DDR addresses requirements for in-water work to control turbidity and protect fish.

14.11 SITE RESTORATION

Areas that are disturbed during construction will be restored to existing conditions upon the completion of work unless stated otherwise in the drawings and specifications.

SECTION 15 - HYDROLOGIC DESIGN

15.1 GENERAL

This section describes the historical hydrology at Detroit Dam including the spatial and temporal variability of regional precipitation, variability in annual inflow to Detroit Reservoir, and presents impacts to reservoir operations including releases and elevations.

The North Santiam River Basin, tributary to Detroit Dam, is a fan-shaped area of 438 square miles, located on the west slope of the Cascade Range approximately 60 miles southeast of Portland, Oregon. The basin terrain is mountainous and covered with heavy stands of coniferous trees. Extremes in elevation within the basin are 1,200 ft at the dam to 10,495 ft on the summit of Mount Jefferson. The average elevation of the section that is tributary to the dam is 3,765 ft. In general, this area is underlain with basalt and many outcrops penetrate the thin organic soil cover. Principal tributaries to the North Santiam River above Detroit Dam, in downstream order are: Marion, Pamelia, and Whitewater Creeks; Breitenbush River; and Blowout and Khey Creeks.

15.2 REFERENCES

Hydrological Analysis was based on the following ECs, EMs, EERs, ETLs, and Engineer Pamphlets (EPs):

EM 1110-2-1415, Hydrologic Frequency Analysis, 5 March 1993.
EM 1110-2-1619, Risk-Based Analysis for Flood Damage Reduction Studies, 1 August 1996.
EP 1110-2-7, Hydrologic Risks, 1 May 1988.
EP 1110-2-8, Explaining Flood Risks, 30 April 1992.
ER 1105-2-101, Planning - Risk Analysis for Flood Damage Reduction Studies, 3 January 1996.
ER 1110-2-1450, Hydrologic Frequency Estimates, 31 August 1994.

Hydrology Report, Willamette FIS Update (Phase One), US Army Corps of Engineers, Portland District, 06 May 3013.

Reservoir Regulation Manual, Detroit and Big Cliff Reservoirs, North Santiam River, 1953.

Reservoir System Simulation (HEC-Res Sim), Version 3.1 RC3, Hydrologic Engineering Center, September 2010.

Statistical Software Package (HEC-SSP), Version 2.0, Hydrologic Engineering Center, October 2010.

United States Corps of Engineers, Northwestern Division, Dataquery:

Detroit Dam, Outflow Discharge, Manual Collection (QRDRXZZAZD).

Detroit Dam, Forebay Elevation, Manual Collection (HFDRXZZAZD).

United States Geological Survey, National Water Information System: Web Interface:

Breitenbush River Above French Creek Near Detroit, 14179000.
North Santiam River Below Boulder Creek, Near Detroit, 14178000.
Blowout Creek Near Detroit, 14180300.

15.3 HYDROLOGICAL SUMMARY

15.3.1 Precipitation Records

In the immediate area of the North Santiam River Basin above Detroit Dam there are six climatological stations all with automatic precipitation recorders. Those stations are located at Santiam Junction, Santiam Pass, Marion Forks, Breitenbush, Detroit, and Detroit Dam. Normal annual precipitation over the portion of the North Santiam River Basin above Detroit Dam is 82 in. Within that part of the basin, the normal annual precipitation ranges from 65 in in the vicinity of Detroit to about 100 in at the higher elevations on the west slope of Mount Jefferson.

Climatological records for the station at Detroit show that about 60% of the average annual precipitation occurs during the winter months, November through February. The average November precipitation slightly exceeds that of each of the following three months. During the summer months, June through September, only 10% of the average annual precipitation occurs. The maximum monthly precipitation recorded at Detroit was 27.76 in in November 1942. Complete absence of precipitation has occasionally been recorded during the months of July, August, and September. The highest 24-hour precipitation of record, 4.81 in, occurred in November 1909. Cumulative annual and monthly precipitation data for Detroit and Detroit Dam from the Reservoir Regulation Manual are summarized in Tables 15-1 and 15-2.

Table 15-1. Detroit Dam Cumulative Precipitation Data (inches)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1949	---	---	---	---	---	---	0.36	.015	2.81	6.42	12.11	12.15	---
1950	---	13.05	14.64	7.84	2.96	6.31	0.70	1.26	2.60	20.99	17.68	13.59	---
1951	21.14	11.83	12.09	2.52	4.71	0.35	0.07	0.63	1.96	16.13	13.28	18.27	102.98
1952	12.01	8.68	10.31	2.94	4.67	6.08	0.00	0.25	0.82	0.51	2.71	15.40	64.38
1953	28.32	13.07	9.90	5.33	7.39	3.51	0.18	3.25	0.97	---	---	---	---

Station: Detroit Dam; Marion County; Oregon

Latitude: 44º 44’ Longitude: 122º 14’ Elevation 1585 ft

Table 15-2. Detroit Cumulative Precipitation Data (inches)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1894	---	---	---	---	2.73	5.41	0.39	0.00	5.77	11.34	6.11	8.12	---
1895	14.10	3.25	6.47	5.49	8.59	0.63	1.00	0.20	3.46	0.10	8.01	18.67	69.97
1896	14.86	9.52	8.92	7.85	10.48	1.18	0.00	1.80	1.08	4.97	22.95	13.35	96.96
1902	6.97	13.41	8.71	6.82	6.61	1.05	3.87	0.47	2.25	3.00	11.91	15.85	80.92
1903	---	---	---	---	---	---	0.13	0.09	2.20	2.33	14.67	3.32	---
1904	---	16.83	16.10	4.40	1.95	1.31	1.70	0.52	---	---	---	---	---
1909	---	---	---	---	---	---	---	---	---	---	23.23	9.70	---
1910	9.89	15.69	5.10	3.24	3.58	1.47	0.00	0.24	1.13	6.02	18.85	9.28	74.49
1911	12.18	4.96	2.91	4.47	6.43	1.16	0.04	0.03	6.05	2.41	12.71	9.38	62.73
1912	18.79	11.99	5.68	5.43	5.60	4.08	0.65	5.33	2.60	7.90	12.29	12.61	92.95
1913	14.08	3.16	11.91	4.66	3.99	4.96	1.58	0.93	4.15	7.79	10.84	4.74	72.79
1914	16.65	7.67	6.95	6.15	2.50	3.37	0.67	0.10	7.74	8.10	7.09	3.14	70.13
1915	9.57	6.87	4.50	3.65	7.91	1.41	1.09	T	1.90	5.10	19.47	14.23	75.70
1916	10.99	9.72	13.64	5.76	6.18	4.04	2.73	0.98	2.68	2.59	12.64	9.35	81.30
1917	5.61	7.23	8.14	8.05	3.76	2.88	0.04	T	2.08	0.65	9.45	25.84	73.73
1918	15.18	7.17	2.76	1.40	1.41	0.07	0.77	0.45	0.00	4.28	5.84	4.22	43.55
1919	14.23	10.61	6.88	3.31	2.88	1.17	0.01	0.01	3.43	6.88	14.51	9.70	73.62
1920	7.21	0.26	8.79	7.96	0.56	1.16	0.84	1.27	5.97	3.33	6.20	14.23	57.78
1921	9.75	7.83	8.79	5.95	3.52	1.80	T	0.35	4.80	5.85	18.82	5.60	73.06
1922	8.87	6.11	9.74	4.73	1.11	0.90	0.00	0.88	1.99	3.85	3.74	16.58	58.50
1923	21.47	3.89	4.70	3.66	3.95	2.58	3.02	1.50	1.22	4.06	7.55	13.11	70.71
1924	7.85	7.87	4.53	1.96	0.95	2.85	T	0.67	4.32	9.68	17.14	10.00	67.81
1925	16.69	11.53	4.42	5.42	4.70	1.68	0.04	0.53	1.20	0.83	8.20	5.63	60.92

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
1926	6.20	12.79	1.09	3.37	4.31	0.45	0.00	3.41	3.42	6.58	17.58	7.41	66.61
1927	17.59	15.18	6.45	3.74	3.86	3.25	0.08	0.86	8.49	6.00	18.11	7.40	91.01
1928	9.99	2.74	9.84	5.62	1.65	0.96	0.02	0.00	1.50	4.33	8.69	10.30	55.64
1929	8.75	3.07	6.14	7.70	2.24	3.88	0.00	0.00	1.04	1.70	0.56	20.11	55.19
1930	5.70	11.30	4.25	4.03	4.83	1.21	0.00	0.00	2.08	4.96	7.23	4.42	50.01
1931	6.73	5.06	13.55	4.71	2.13	3.67	0.00	0.01	2.00	7.26	11.43	9.50	66.05
1932	10.95	4.62	9.18	5.79	2.80	0.39	0.69	0.87	0.20	6.96	11.94	6.13	60.52
1933	9.32	5.81	7.92	2.62	5.68	4.36	T	0.47	6.07	4.98	3.04	24.70	74.97
1934	13.15	2.57	9.31	4.23	2.75	1.07	0.33	0.20	1.60	8.77	15.21	11.69	70.88
1936	---	---	---	---	---	3.45	0.62	0.00	2.20	0.38	0.42	13.17	---
1937	7.66	11.91	4.39	12.28	3.13	6.52	0.99	1.38	2.02	7.17	15.94	15.52	88.91
1938	10.64	8.83	13.42	5.20	1.88	1.29	0.26	0.37	2.00	4.00	10.81	10.68	69.38
1939	8.62	12.60	7.88	0.98	1.62	2.37	0.57	0.90	1.34	5.68	1.50	13.74	57.80
1940	5.50	16.26	8.24	4.27	2.22	1.04	1.30	0.06	5.13	6.28	11.65	6.58	68.53
1941	8.59	2.88	2.60	3.76	7.99	2.64	1.03	2.15	5.60	4.32	9.77	15.13	66.46
1942	6.20	8.18	4.17	4.22	5.72	3.63	0.94	0.21	0.04	3.25	27.76	19.76	84.04
1943	12.40	9.10	9.79	8.35	2.95	4.69	0.53	3.02	0.15	12.40	5.76	3.50	72.64
1944	6.78	6.81	5.50	5.85	4.14	1.90	0.06	0.55	4.56	1.68	8.70	4.18	50.71
1945	10.09	13.85	11.91	9.24	8.08	0.76	0.22	1.20	4.06	2.83	21.49	15.26	98.99
1946	13.74	9.75	9.87	3.77	1.77	3.40	0.41	0.49	2.18	11.50	15.72	13.79	86.39
1947	10.46	4.70	8.51	5.39	0.92	8.13	2.08	0.96	1.72	16.00	11.83	7.99	78.69
1948	14.42	14.20	7.41	6.87	6.59	1.95	1.19	0.92	4.36	4.65	14.06	18.29	94.91
1949	3.19	19.60	5.49	2.63	6.51	1.53	0.25	0.14	2.82	6.89	11.16	11.93	72.14
1950	21.75	11.03	13.17	4.70	2.14	3.46	0.64	0.98	2.25	19.80	15.87	13.34	109.13
1951	18.55	10.70	8.78	2.17	2.81	0.06	0.14	0.78	1.84	14.32	12.18	15.30	87.63
1952	9.65	8.49	8.01	---	---	4.16	0.00	0.37	1.26	0.41	2.26	13.31	---
1953	27.09	12.64	8.60	4.34	5.87	2.55	0.07	3.16	0.70	---	---	---	---
Means	11.53	8.92	7.72	5.03	4.00	2.46	0.63	0.76	2.89	5.83	11.73	11.45	72.95

Station: Detroit; Marion County; Oregon
Latitude: 44° 42’ Longitude: 122° 04’ Elevation 1475 ft

Winter precipitation over that portion of the basin above Detroit Dam frequently occurs in the form of snow, which accumulates to great depths at higher elevations. About 10% of the mean annual precipitation is estimated to occur as snow at 2,000 ft elevation, 50% at 5,000 ft elevation, and 75% at 7,000 ft elevation. The snow at lower elevations usually melts off several times each winter, whereas at the higher elevations it accumulates continuously throughout the winter. On rare occasions, as in the winter of 1948-49, snow accumulates to great depths at the lower elevations. At Detroit, the average observed annual snowfall has been 56 in. Extremes in recorded snowfall at Detroit are 195 in during the winter of 1915-16, and 29 in in a 24-hour period in January 1895. (Reservoir Regulation Manual, Detroit and Big Cliff Reservoirs, North Santiam River, 1953.)

15.3.2 Streamflow Records

USGS currently operates three gages upstream of Detroit Reservoir. Information on these gages are shown below in Table 15-3.

Table 15-3. USGS Upstream Gages

Gage Name	Station Number	Data Begin Date	Data End Date	Drainage Area (mi^2)
Breitenbush River Above French Creek Near Detroit	14179000	01 Jun 1932	Current	108
North Santiam River Below Boulder Creek, Near Detroit	14178000	01 Jan 1907	Current	216
Blowout Creek Near Detroit	14180300	01 Oct 1998	Current	26.0

USACE maintains data of the total inflow and outflow of Detroit Reservoir. The daily mean inflow data QIDRXZZAZD from August 01, 1960 to June 30, 2013, and daily mean outflow data QRDRXZZAZD from August 01, 1960 to December 01, 2010, was used to determine the statistics below.

15.3.2.1 Inflows

The ten largest mean daily inflows for each month over the 53 years of record (1960-2013) are shown in Table 15-4. The largest mean daily inflow of record, 55,893 ft^3/s, occurred in December 1964. The largest mean daily flow outside of the four winter months (November - February) was 17,954 ft^3/s in April 2002. The second largest was 16,357 ft^3/s recorded in March 1972. As shown in Table 15-4, high inflows drop precipitously in magnitude in the months of July through September.

For each of the 53 years of record, the maximum mean daily discharge was selected from each month (of which the 10 largest are shown in Table 15-4). Statistics of these data are shown on Table 15-5. The average maximum mean daily inflow is largest in the month of December (11,220 ft^3/s) and decreases steadily through June. In July through September, the average maximum mean daily inflow drops abruptly before quickly rising again in October and November. The smallest maximum mean daily inflow on record is below 1,800 ft^3/s in all months of the year.

For each of the 53 years of record, the minimum mean daily discharge was selected from each month. Statistics of these data are shown on Table 15-6. As shown by the table, inflows can be low in any month of the year.

Table 15-4. Ten Largest Inflows on Record for Each Month (Mean Daily Values)

October High Inflows (cfs)	Year	November High Inflows (cfs)	Year	December High Inflows (cfs)	Year	January High Inflows (cfs)	Year
10,009	1997	26,790	1960	55,893	1964	27,479	1972
8,697	1967	24,963	1999	26,363	1977	26,794	2011
7,470	1994	23,562	1996	22,757	1980	26,295	1965
6,405	1982	21,443	1977	21,923	2005	22,558	1997
6,079	1973	19,258	1995	20,597	1996	21,449	2006
5,179	2007	18,894	1962	20,046	2006	20,532	1995
4,900	1968	16,684	2006	18,863	2011	20,045	1971
4,839	1996	16,486	1970	17,055	1995	18,683	2009
4,818	2012	13,580	1984	17,026	1998	18,187	1970
4,810	1985	13,296	1963	16,455	2007	17,787	1980
February High Inflows (cfs)	**Year**	**March High Inflows (cfs)**	**Year**	**April High Inflows (cfs)**	**Year**	**May High Inflows (cfs)**	**Year**
48,288	1996	16,357	1972	17,954	2002	11,260	2008
29,393	1961	15,846	1993	14,386	1996	10,110	2009
26,717	1986	15,017	2012	11,517	1990	6,859	1971
23,933	1995	12,667	2003	10,247	1997	6,799	1963
20,243	1982	11,328	2005	9,599	1962	6,739	1999
17,895	1963	10,430	1997	8,768	2013	6,700	1969
17,195	1968	10,003	1983	8,459	1993	6,209	1996
16,227	1972	9,669	2010	8,009	2012	6,188	1976
15,595	1997	8,998	1966	7,009	2011	6,020	1972
13,120	1981	8,920	1979	7,000	1965	5,599	1964
June High Inflows (cfs)	**Year**	**July High Inflows (cfs)**	**Year**	**August High Inflows (cfs)**	**Year**	**September High Inflows (cfs)**	**Year**
9,129	2010	3,150	2008	2,300	1968	3,029	1971
8,398	1974	2,700	1974	1,890	2004	2,370	1973
8,214	1981	2,520	1999	1,390	2008	2,090	1997
5,829	2008	2,329	1983	1,360	1999	1,958	1986
5,705	1985	2,160	1971	1,231	1971	1,930	2004
5,493	1984	2,128	1975	1,206	1975	1,700	1961
5,000	1964	2,090	1976	1,200	1974	1,662	1978
4,930	1999	1,920	1969	1,123	1983	1,400	1996
4,809	2000	1,899	1961	1,100	1964	1,360	1972
4,540	2011	1,818	1984	1,080	1972	1,335	1977

Table 15-5. Statistics of the Maximum (Mean Daily Inflows)

Month	Largest Occurrence (cfs)	Average Occurrence (cfs)	Minimum Occurrence (cfs)
October	10,009	2,630	524
November	26,790	8,752	982
December	55,893	11,220	1,257
January	27,479	11,086	893
February	48,288	8,810	1,430
March	16,357	6,247	1,730
April	17,954	5,463	1,700
May	11,260	4,482	1,780
June	9,129	3,139	805
July	3,150	1,392	654
August	2,300	898	481
September	3,029	1,071	530

Table 15-6. Statistics of the Minimum (Mean Daily Inflows)

Month	Largest Occurrence (cfs)	Average Occurrence (cfs)	Minimum Occurrence (cfs)
October	865	564	300
November	1,683	877	410
December	2,473	1,348	523
January	2,731	1,472	593
February	3,101	1,507	593
March	2,500	1,578	820
April	2,980	1,881	812
May	3,190	1,777	643
June	3,030	1,249	569
July	1,230	732	300
August	870	585	365
September	760	540	380

Presented on Table 15-7 are statistics of the mean monthly inflows. A mean monthly inflow larger than 4300 ft^3/s has occurred in 1 or more years for each month from November through June (see column labeled "Largest Occurrence"). The average of the mean monthly inflows is considerably lower from July through October compared to the remainder of the year.

Plotted on Figure 15-1 is the maximum, average, and minimum mean daily unregulated inflow for each day of the year. The largest "maximum" value plotted for each month in

Figure 15-1 shows minimum, maximum, and average daily inflows to Detroit Dam throughout the year. Also plotted on Figure 15-1 is the Detroit rule curve, which provides the target pool elevation at Detroit Reservoir (shown on right y-axis). As illustrated on Figure 15-1, maximum inflows occur when the target pool level (rule curve) is at minimum flood control pool (elevation 1,450 NVGD).

Shown on Figure 15-2 are the 5 through 95 percent chance exceedance values of the 5-day running average of the unregulated inflows, again plotted by day of the year. In general, the inflow, averaged over 5 days, exceeds 1,400 to 3,100 ft^3/s about 50% of the time during the period from November through the middle of May. From late October through mid-June, the 5-day average inflow is around 600 to 2,200 ft^3/s or greater about 25% of the time. Only occasionally (about 5% of the time for several days in November, December, January, and February), does the 5-day average inflow exceed 10,000 ft^3/s.

Table 15-7. Statistics of the Mean Monthly Inflows

Month	Largest Occurrence (cfs)	Average Occurrence (cfs)	Minimum Occurrence (cfs)
October	2,183	961	462
November	5,050	2,459	563
December	9,706	3,508	695
January	6,951	3,511	727
February	8,453	3,118	744
March	7,131	2,838	1,268
April	4,743	2,901	1,463
May	5,680	2,737	1,133
June	4,314	1,906	666
July	1,695	976	531
August	1,045	691	431
September	1,102	688	455

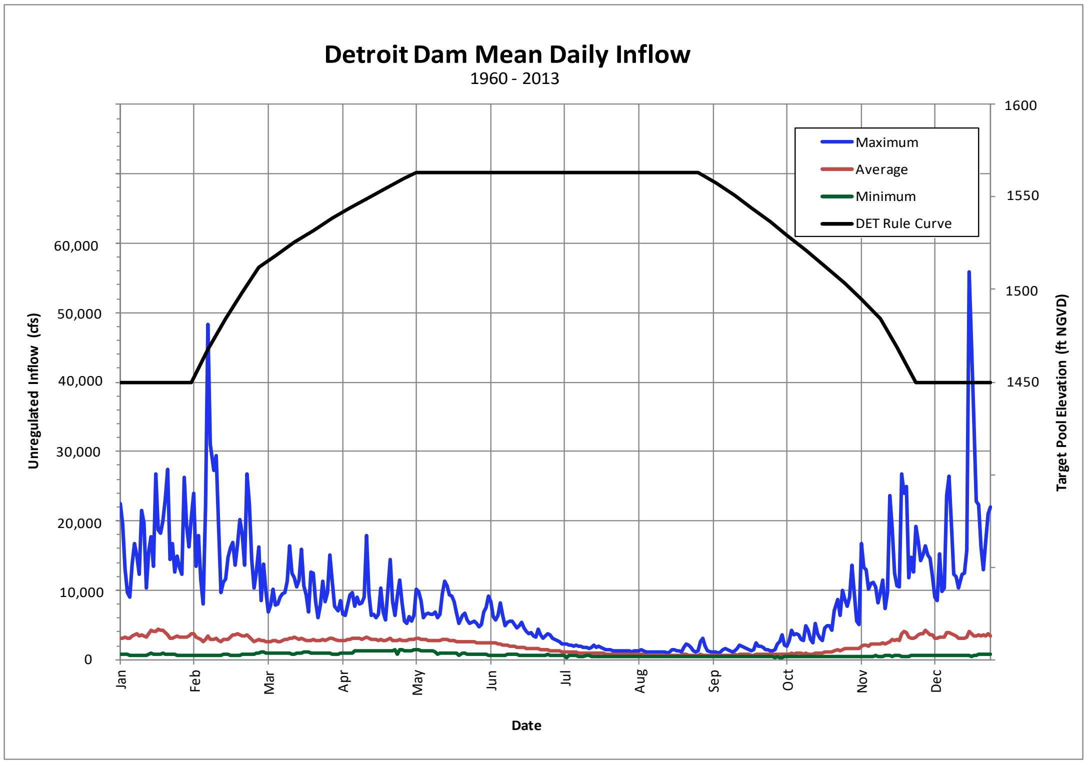

Figure 15-1. Detroit Reservoir Mean Daily Inflow

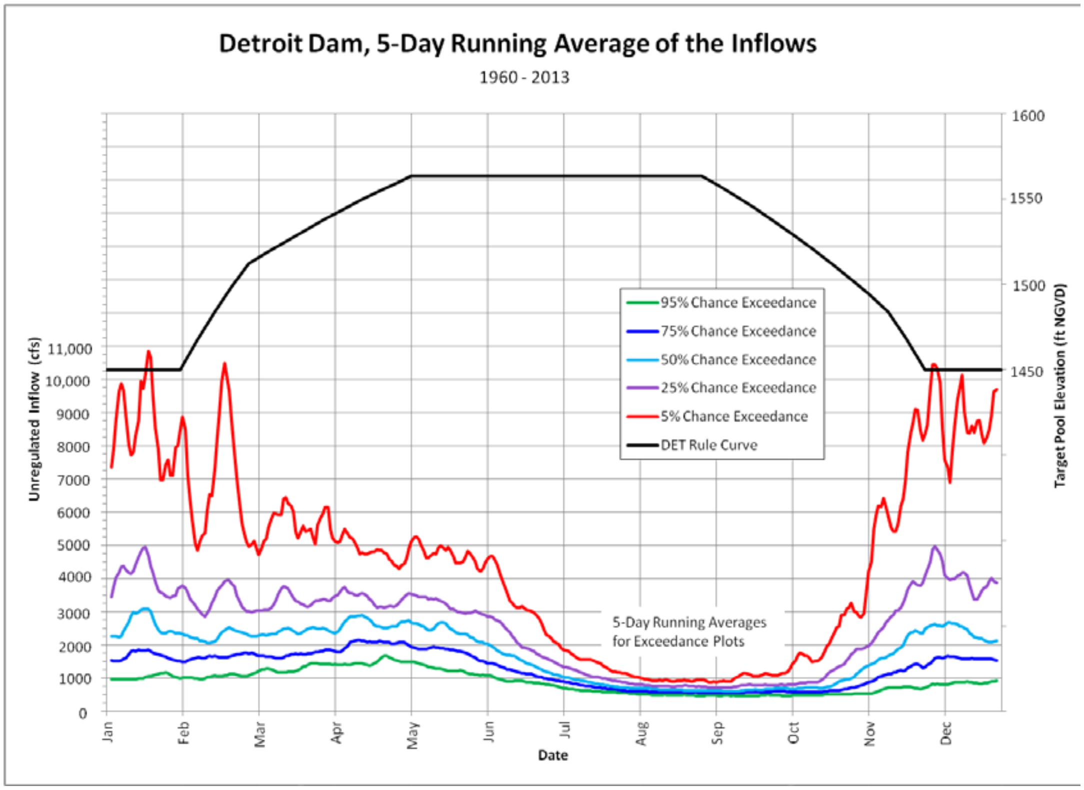

Figure 15-2. Detroit Reservoir Inflow Exceedance

15.3.2.2 Releases

Plotted on Figure 15-3 is the maximum, average, and minimum mean daily regulated discharge from Detroit Reservoir for each day of the year with the Detroit rule curve. As expected, the largest outflows occur during the same months as the largest inflows.

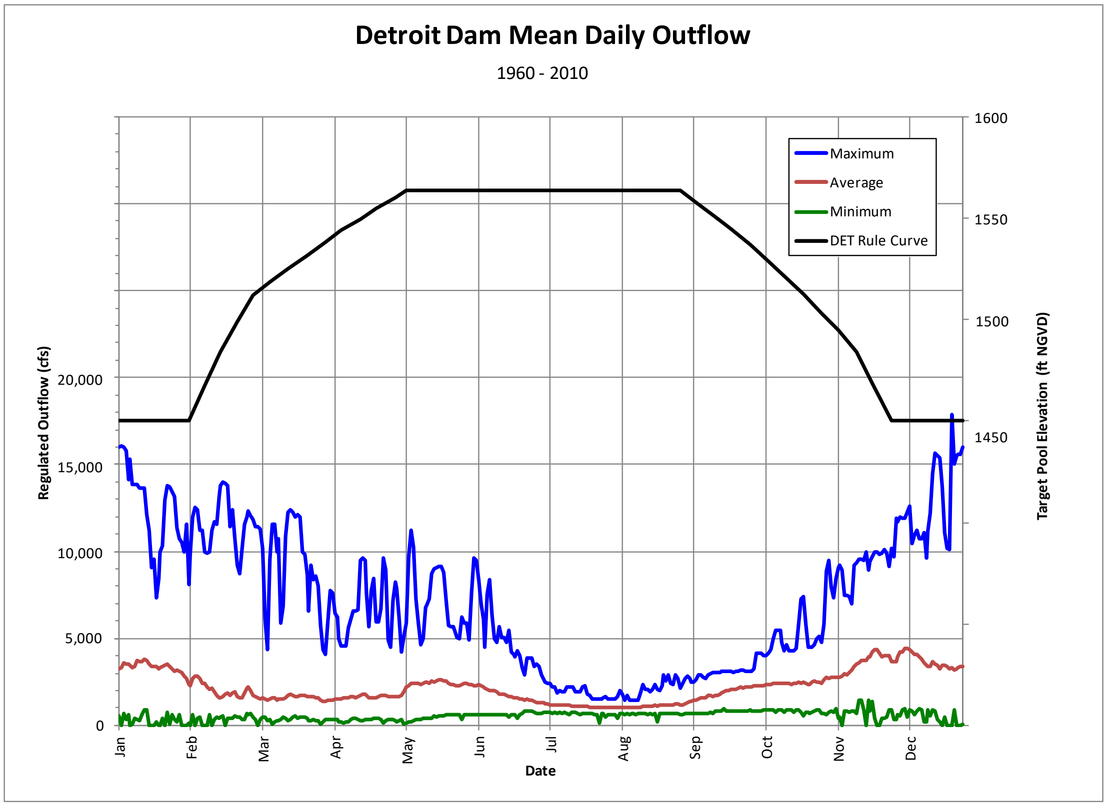

Figure 15-3. Detroit Reservoir Mean Daily Outflow

Shown on Figure 15-4 are the 5 through 95 percent chance exceedance values of the daily outflows, again plotted by day of the year. In general, the outflow is between 2,000 to 4,000 ft^3/s about 50% of the time during the period from November to February. From late October through early February, the outflow is around 3,000 to 5,000 ft^3/s or greater about 25% of the time. The outflow exceeds 10,000 ft^3/s less than 5% of the time in November through February.

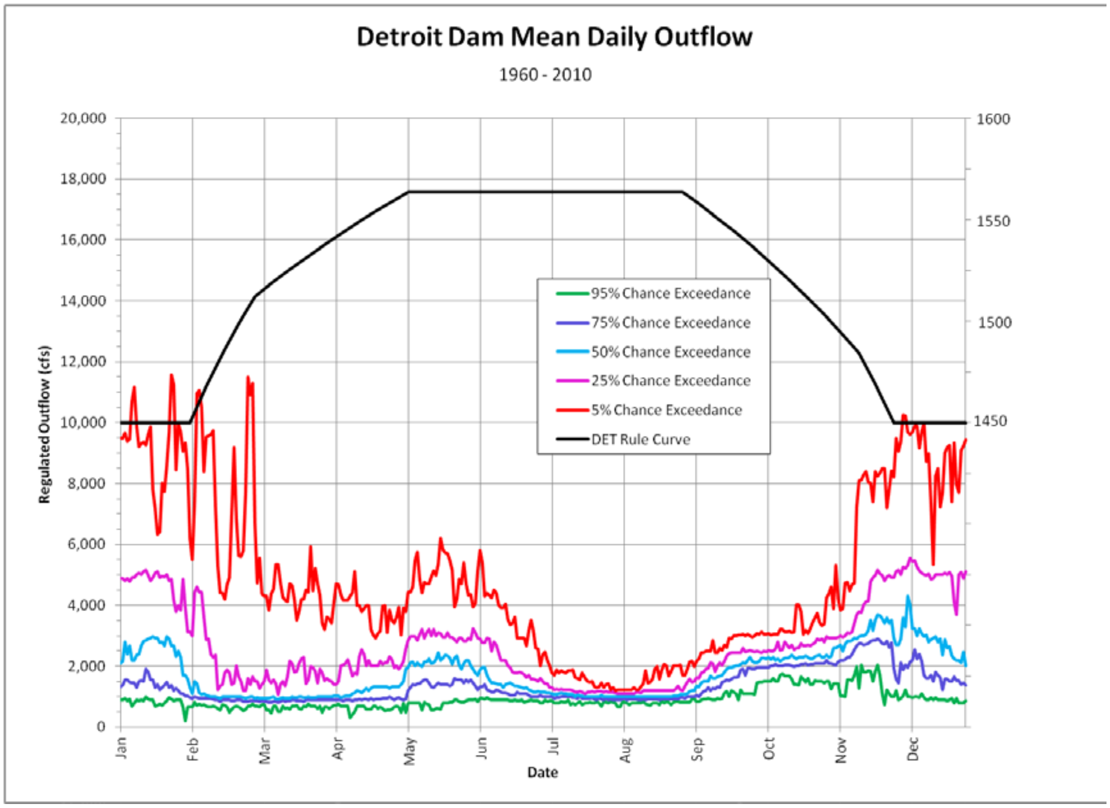

Figure 15-4. Detroit Reservoir Outflow Exceedance

15.3.3 Elevations

15.3.3.1 Pool Elevations

Plotted on Figure 15-5 are the maximum, average, and minimum mean daily pool elevations from observed data at Detroit Reservoir for each day of the year with the Detroit rule curve. All elevations are in NGVD29.

Shown on Figure 15-6 are the 5 through 95 percentile non-exceedance values, and the minimum and maximum of the project elevations from modeled data, using the derived Period of Record historical flows from 1935-2008 (US Army Corps of Engineers, 2013). (Note that over the history of the project, the standard operations have changed over time. For example, the minimum project outflows were increased according to the 2008 BiOp from the minimum outflow specified in the water control manual. The modeled results assume current standard operations for all years of historical inflows, even for years prior to the project construction.) Figure 15-6 shows the variability in reservoir elevations throughout the years.

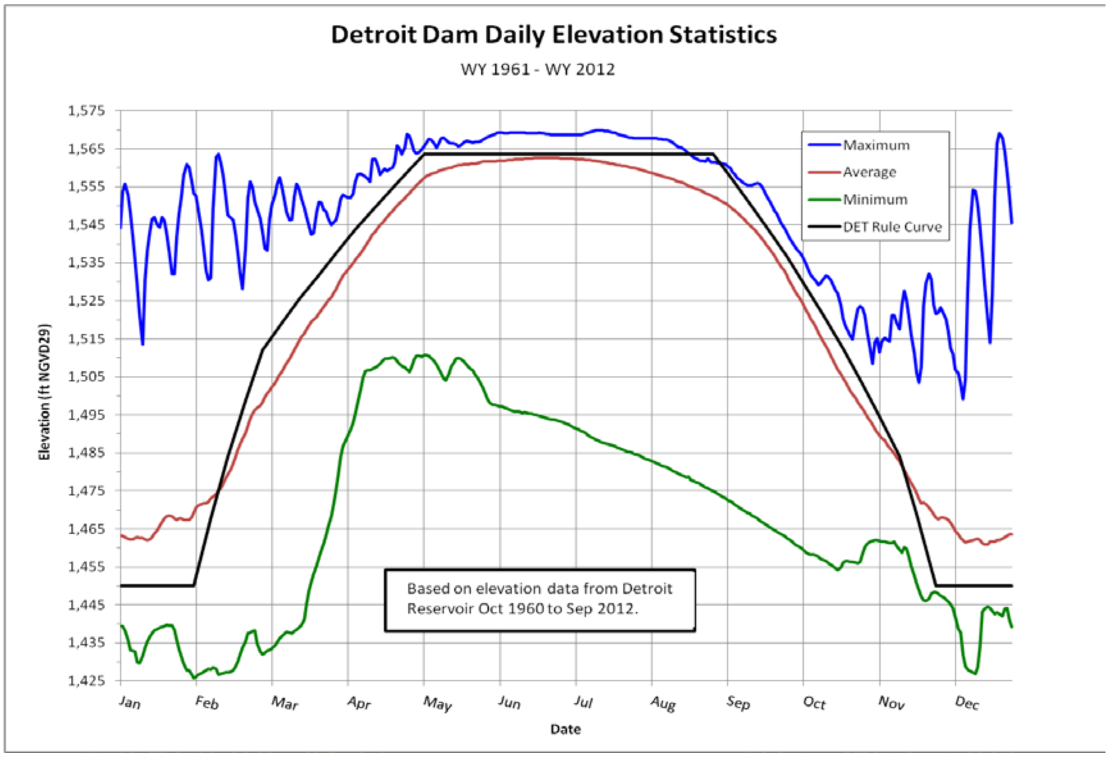

Figure 15-5. Detroit Reservoir Mean Daily Pool Elevations from Observed Data, 1961-2012

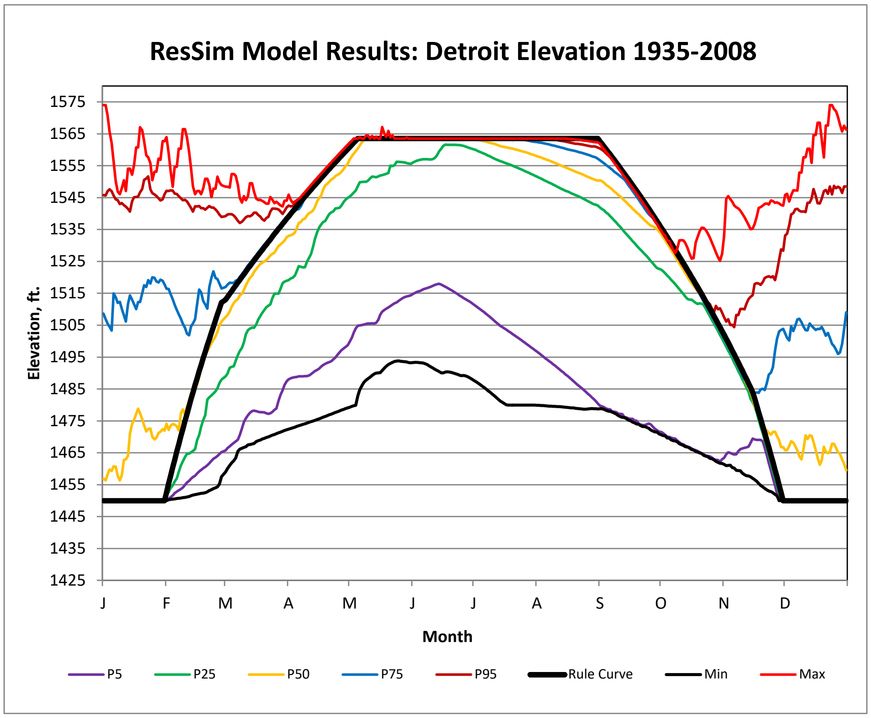

Figure 15-6. Detroit Reservoir Pool Elevation Non-Exceedance Values based on Modeled Reservoir Operations Using Historical Inflows for 1935-2008

15.3.3.2 Change in Elevation

Figure 15-7 shows the probability of a pool rise exceeding 8 ft in any 1-week period from the observed reservoir elevation data. Note that the pool can rise significantly in late January and early February.

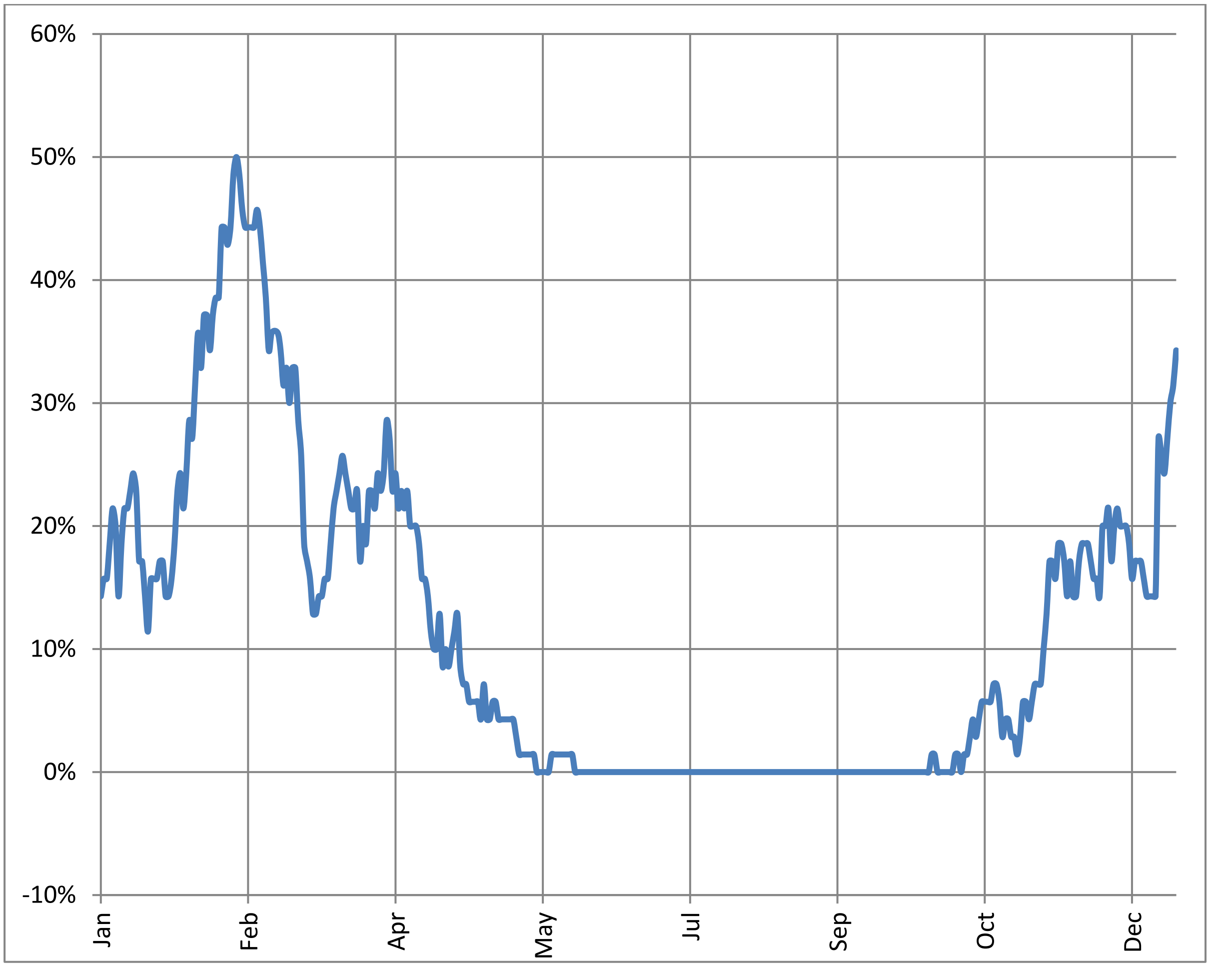

Figure 15-7. Probability of Pool Rise Greater Than 8 feet in a 1-Week Period