Revised Project Description: Flood Retention Expandable Structure

Chehalis River Basin Flood Damage Reduction Project

Lewis County, Washington

April 25, 2024

Minor revisions to pages 26, 110-112,
and 127 made Nov. 27, 2025

Chehalis River Basin
Flood Control Zone District

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Contents

Tables

Figures

Appendices

Acronyms and Abbreviations

2D	two-dimensional
3D	three-dimensional
AEP	annual exceedance probability
ANSI	American National Standards Institute
AORC	Analysis of Record for Calibration AWS auxiliary water supply
BA	Biological Assessment
cfs	cubic feet per second
CHTR	collect, handle, transfer, and release
CSZ	Cascadia Subduction Zone
CVC	conventional vibrated concrete
District	Chehalis River Basin Flood Control Zone District
DSO	[WDOE] Dam Safety Office
DEIS	Draft Environmental Impact Statement
EM	Engineering Manual
ER	Engineering Regulation
FEMA	Federal Emergency Management Agency
FERC	Federal Energy Regulatory Commission
FFPF	Flood Fish Passage Facility
FRO	Flood Retention Only
FRE	Flood Retention Expandable
FRE-FC	Flood Retention Expandable-Future Construction
ft	foot/feet
ft-lbs/sec/ft^3	foot-pounds per second per cubic foot
ft/s	foot/feet per second
G	gravity
GARR	Gauge-Adjusted Radar Rainfall
GERCC	grout-enriched roller-compacted concrete
gpm	gallons per minute
H:V	Horizontal to Vertical
HEC-HMS	Hydrologic Engineering Center- Hydrologic Modeling System
HEC-RAS	Hydrologic Engineering Center-River Analysis System
HMR	Hydrometeorological Report
ICOLD	International Commission on Large Dams
km	kilometer(s)
LiDAR	light detection and ranging
LLO	low-level outlet
M	magnitude
MCE	Maximum Credible Earthquake
MRMS	Multi-Radar/Multi-Sensor System msl mean sea level
Mw	megawatt
NAVD88	North American Vertical Datum of 1988
NCEI	National Centers for Environmental Information
NEPA	National Environmental Policy Act
NLCD	National Land Cover Database

NMFS	National Marine Fisheries Service
NOAA	National Oceanic and Atmospheric Administration
NRCS	Natural Resources Conservation Service
NSSL	National Severe Storms Laboratory
NWS	National Weather Service
OSHA	Occupational Safety and Health Organization
PFM	potential failure mode
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
PRISM	Parameter-elevation Regressions on Independent Slopes Model
PSHA	probabilistic seismic hazard analysis
Qa	(modern) Quarternary alluvium
Qao	(older) Quarternary alluvium
QPE	quantitative precipitation estimate
RAWS	Remote Automated Weathers Stations
RCC	roller-compacted concrete
RCW	Revised Code of Washington
Reclamation	U.S. Bureau of Reclamation
RPD	Revised Project Description
RPDR	Revised Project Description Report
S&W	Shannon & Wilson
SCADA	Supervisory Control and Data Acquisition
SEPA	State Environmental Policy Act
Tcb	Crescent Formation pillow basalt
TCP	traditional cultural property
Tcs	Crescent Formation siltstone/claystone
TM	Technical Memorandum
USACE	U.S. Army Corps of Engineers
USDA	U.S. Department of Agriculture
USFWS	U.S. Fish and Wildlife Service
USGS	U.S. Geological Survey
WA	Washington
WAC	Washington Administrative Code
WDFW	Washington Department of Fish and Wildlife
WDNR	Washington Department of Natural Resources
WDOE	Washington Department of Ecology
WEST	WEST Consultants, Inc.
WRCC	Western Regional Climate Center
WSE	Watershed Science & Engineering
WSEL	water surface elevation

1 Introduction

1.1 Project Background

The purpose of this Revised Project Description Report (RPDR) is to replace the Combined Dam and Fish Passage Conceptual Design Report (HDR 2017a) and Combined Dam and Fish Passage Supplemental Design Report FRE Dam Alternative (HDR 2018) previously submitted by the Chehalis River Basin Flood Control Zone District (District) to the Washington State Department of Ecology (WDOE) and the U.S. Army Corps of Engineers (USACE) in support of their respective reviews of the Chehalis River Basin Flood Damage Reduction Project (Proposed Project) under the State Environmental Policy Act (SEPA) and National Environmental Policy Act (NEPA). The Proposed Project includes construction of a flow-through dam for flood control with volitional fish passage – the Flood Retention Expandable structure (FRE) – as well as flood-control operations, maintenance, and associated protection and mitigation activities including implementation of a proposed Mitigation Plan. While the overall flood damage reduction project also includes raising portions of the existing river levee adjacent to the Chehalis-Centralia Airport, no changes to that portion of the Proposed Project are under consideration. Therefore, this report includes no new information related to that portion of the overall Proposed Project.

WDOE and USACE released their respective Draft Environmental Impact Statements (DEISs) in 2020 on the basis of the aforementioned project description documents (HDR 2017a and 2018). The DEISs described potential impacts to environmental resources and resulted in identification of a traditional cultural property (TCP) by the Confederated Tribes of the Chehalis Reservation. To address these issues, the District has the revised FRE's alignment within the original Proposed Project footprint that meets the Proposed Project's purpose and need while minimizing effects to the identified TCP and addressing other environmental impacts identified in the DEISs, including providing for volitional fish passage during construction.

1.2 Purpose and Objectives

This RPDR is prepared for the District and replaces and supersedes HDR (2017a) and HDR (2018). Data and information that remain valid have been carried forward into this RPDR, which also describes components of the Proposed Project that have been revised to address the identified TCP and potential impacts identified in the federal and state DEISs. The primary objectives of this RPDR are to:

- Describe revisions to the proposed FRE alignment and associated fish passage configuration including construction phasing, river diversion and permanent and temporary access routes.
- Identify and describe potential opportunities for optimizing FRE operations to minimize the duration of flood management operations to minimize environmental impacts while still meeting flood damage reduction objectives.
- Describe changes and updates to material sourcing (quarries).
- Describe maintenance activities, electrical instrumentation, and controls.

Following delivery of this RPDR to WDOE and USACE, the design will continue to be developed (Preliminary Design) to support development of the Biological Assessment. The information provided within the RPDR will be the basis for continued development during Preliminary Design.

1.3 Previous Reports and Documents

As noted above, the RPDR references and replaces two previous reports:

- Combined Dam and Fish Passage Conceptual Design Report (HDR 2017a)
- Combined Dam and Fish Passage Supplemental Design Report FRE Dam Alternative (HDR 2018)

These previous reports provide references to multiple reports leading to the selection and design of the original Proposed Project.

Table 1-1 includes a chronological list (oldest to newest) of informational submittals directly associated and addressed within this RPDR. The District submitted these documents to WDOE, Washington Department of Natural Resources (WDNR), and USACE after issuance of the federal and state DEISs. The table identifies which documents remain valid or have been superseded and the corresponding RPDR section, if applicable. The table also indicates whether a portion of the document has been supplemented. If the document remains valid or has been supplemented, the document is appended to the RPDR in Appendix N.

A list or specific descriptions describing why each submittal supersedes, is valid or supplements, appends or is no longer applicable to the Proposed Project is provided following Table 1-1.

Table 1-2 includes a chronological list (oldest to newest) of information submitted by the District after issuance of the federal and state DEISs to WDOE, WDNR, and USACE that are not addressed within the RPDR but for which updated information will be provided to the agencies in a revised Vegetation Management Plan, revised Mitigation Plan, and revised Biological Assessment.

Table 1-1. Informational Submittals Associated with RPRD Submitted Post-Issuance of DEIS

Submittal Item	Agency Submittal Date	Document Date and Description	Superseded	Valid	Valid/ Supple-mented	Applicability - RPDR Section
1	4 Jun 2021	4 Jan 2021 – Ranking of Potential Quarry Sites for Proposed Flood Retention Structure on the Chehalis	X			N/A - 7,9
2	4 Jun 2021	Jan 2021 – Response to Office of the Chehalis Basin Board question regarding the extent to which the proposed FRE Project would be able to regulate the 100-year flood scenario under projected climate change conditions.		X		Appended
3	4 Jun 2021	1 Jun 2021 – Transfer of Use and Jurisdiction		X		Appended
4	4 Jun 2021	1 Mar 2021 – Existing All Species Fish Passage Facilities Research			X	Appended -14
5	3 Sep 2021	20 Aug 2021 – Description of Construction-Phase Fish Passage Facility (Updated 28 Jan 22)	X			N/A
6	3 Sep 2021	20 Aug 2021 – Quarry Operations (Draft) (Updated 17 Dec 2021)	X			N/A
7	3 Sep 2021	20 Aug 2021 – Large Woody Material Downstream Passage and Placement Clarification (Section 4, LWM for Downstream Habitat Enhancement Updated in the Mitigation Plan)			X	Appended
8	3 Sep 2021	24 Aug 2021 – Access Road Update and Best Management Practices (Superseded by 17 Dec 2021– Access Road Update and Best Management Practices)	X			N/A
9	3 Sep 2021	23 Aug 2021 – Temporary Construction Facilities (Superseded by 17 Dec 2021 – Temporary Construction Facilities)	X			N/A
10	3 Sep 2021	27 Apr 2021 – Slope Stabilization Mitigation	X			N/A - 7.1
11	3 Sep 2021	20 Aug 2021 – FRE Site Temporary and Permanent Power			X	Appended - 13

Submittal Item	Agency Submittal Date	Document Date and Description	Superseded	Valid	Valid/ Supple-mented	Applicability - RPDR Section
12	3 Sep 2021	20 Aug 2021 – Airport Levee Wetland Avoidance (Superseded by 22 Feb 2022 – Airport Levee Wetland Avoidance)	X			N/A
13	3 Sep 2021	24 Aug 2021 – FRE Site Selection			X	Appended -1
14	17 Dec 2021	17 Dec 2021 – FRE Facility – Conceptual Level Recreational Improvement Options		X		Appended
15	17 Dec 2021	17 Dec 2021 – Additional Information – Environmental Justice Benefits of the Proposed FRE Project		X		Appended
16	17 Dec 2021	17 Dec 2021 – Quarry Operations	X			N/A - 9,16
17	17 Dec 2021	17 Dec 2021 – Access Road Update and Best Management Practices (Draft) (Updated 29 June 2022)	X			11
18	17 Dec 2021	17 Dec 2021 – Temporary Construction Facilities	X			N/A - 16
19	17 Dec 2021	15 Nov 2021 – WDOE Information from FCZD Related to SEPA Final EIS, 5. Fish Passage Design – Response to Requested Information	X			N/A - 14.5
20	28 Jan 2022	28 Jan 2022 – Draft – Construction Phase Upstream Fish Passage Alternatives Selection (Updated 25 Feb 2022)	X			N/A
21	25 Feb 2022	25 Feb 2022 – Construction Phase Upstream Fish Passage Alternatives Selection and 10% Design	X			N/A
22	25 Feb 2022	22 Feb 2022 – Airport Levee Wetland Avoidance		X		N/A
23	25 Feb 2022	23 Feb 2022 – Dam Safety Standards and Seismic Fault Study Review	X			N/A - 8, Appendix L
24	30 Jun 2022	25 May 2022 – Construction Phase Upstream Fish Passage Alternatives Selection and 10% Design Addendum: Access Roads – Existing, Construction, and Operating Phases	X			N/A

Submittal Item	Agency Submittal Date	Document Date and Description	Superseded	Valid	Valid/ Supple-mented	Applicability - RPDR Section
25	30 Jun 2022	29 June 2022 – FRE Facility – Truck Trip Summary During and Post Construction	X			11
26	30 Jun 2022	29 June 2022 – Access Road Update and Best management Practices	X			11

Submittal Item and Description:

1. This submittal no longer applies to the Proposed Project as the proposed quarry locations and sizes have been revised. Revised quarry information is located in RPDR Section 7, Geotechnical Design, and Section 9, Aggregate Sourcing.
2. This submittal remains applicable, and a stand-alone document supporting the Proposed Project climate change operational capabilities and is appended to the RPDR.
3. This submittal remains applicable, and a stand-alone document supporting the Proposed Project’s Transfer of Use and Jurisdiction and is herein appended to the RPDR.
4. This submittal remains applicable and is supplemented in Section 14.5 - Reservoir Operational Sensitivity Analysis of the RPDR. Section 3 of the submittal discusses the proposed fish bypass configuration from the original FRE alignment while Section 15 of the RPDR discusses the proposed configuration of the temporary fish bypass facility at the revised FRE alignment.
5. This submittal is no longer applicable to the Proposed Project. The need for trap and haul activities during construction are no longer proposed.
6. This submittal was updated and superseded by Submittal Item 16.
7. This submittal remains applicable and will be supplemented in the Mitigation Plan. Section 2, LWM Passage and Section 3, Debris Removal, from the submittal remain applicable for the Proposed Project. Section 4, LWM for Downstream Habitat Enhancement will be updated and enhanced in the Mitigation Plan.
8. This submittal was updated and superseded by Submittal Item 26
9. This submittal was updated and superseded by Submittal Item 18.
10. This submittal is no longer applicable and is replaced by Section 7.1 - Landslide Evaluation of the RPDR.
11. This submittal is supplemented by RPDR Sections 13.1 - Electrical Service and 16.4 - Distribution Lines for Construction Power. Section 13.1 estimated permanent power demand supersedes Section 2.2 of the submittal. Figure 2 within the submittal is no longer valid. Sections 13.1 and 16.4 of the RPDR refer directly to applicable information within the submittal.
12. This submittal was updated and superseded by Submittal Item 22.
13. This submittal provides the history on why the proposed FRE was sited near river mile 108 of the upper mainstem of the Chehalis River. This RPDR describes the current revised alignment and configuration within the Proposed Project area.

14. This submittal remains applicable, and a stand-alone document supporting the Proposed Project's Conceptual Recreational Improvement Options and is herein appended to the RPDR.
15. This submittal remains applicable, and a stand-alone document supporting the Proposed Project's Environmental Justice Benefits and herein is appended to the RPDR.
16. This submittal is no longer applicable and has been revised by RPDR Section 9 - Aggregate Sourcing, and Section 16 - Construction Considerations.
17. This submittal is no longer applicable. It was updated and superseded by Submittal Item 26
18. This submittal is no longer applicable. Temporary Construction Facilities have been revised and are addressed in Section 16 - Construction Considerations of RPDR.
19. This submittal is no longer applicable. Fish passage information provided to WDOE in the submittal has been revised within Section 14.5 - Reservoir Operational Sensitivity Analysis of the RPDR.
20. This submittal is no longer applicable to the Proposed Project. The need for trap and haul activities during construction are no longer proposed.
21. This submittal is no longer applicable to the Proposed Project. The need for trap and haul activities during construction are no longer proposed.
22. This submittal remains applicable, and a stand-alone document supporting the Proposed Project's Airport Levee improvements is appended to the RPDR.
23. This submittal is no longer applicable and has been revised. Dam Safety information is now presented in Section 8, Structural Analysis and Design, of the RPDR. Seismic information is presented in Appendix L, Seismic Design Hazards TM, of the RPDR.
24. This submittal is no longer applicable to the Proposed Project. The need for trap and haul activities during construction are no longer proposed.
25. This submittal is no longer applicable to the Proposed Project. The Constructability TM attached to RPDR Section 16 - Construction Considerations, provides revised information regarding the estimated number of truck trips and water usage required for construction.
26. This submittal is no longer applicable to the Proposed Project. Section 11 - Civil Design and Earthwork, provides revised temporary and permanent access road information and associated BMPs.

Table 1-2. Updated Location of Other Information Provided After DEISs

Agency Submittal Date	Document Date and Description	Location of Updated Information
May 2020	18 May 2020 – SEPA DEIS Review: FRE Facility Temporary Reservoir Inundation and Vegetation Analysis Clarification	Vegetation Management Plan
Jul 2020	Jul 2020 – Draft Aquatic and Terrestrial Mitigation Opportunities Assessment	Mitigation Plan
Aug 2020	Aug 2020 – Wetland Mitigation Opportunities Assessment	Mitigation Plan
Nov 2020	Nov 2020 – Conceptual Vegetation Management Plan	Vegetation Management Plan
4 Jun 2021	26 Feb 2021 – Mitigation Capacity and Species Benefits	Mitigation Plan
4 Jun 2021	10 Mar 2021 – Avoidance and Minimizations of Rainbow Falls/Fisk Falls Lamprey Fishery Impacts and Related Cultural Effects	Mitigation Plan
4 Jun 2021	4 Jun 2021 – District's Committed AMM Measures Catalog (Microsoft Excel Database)	Biological Assessment/Vegetation Management Plan/Mitigation Plan
4 Jun 2021	18 Feb 2021 – Task 2: Short Term Aquatic Species Benefits	Biological Assessment
3 Sep 2021	26 Aug 2021 – Plant Replacement Plan	Vegetation Management Plan
3 Sep 2021	6 Aug 2021 – Commitment to No Net Loss of Aquatic Habitat	Mitigation Plan
3 Sep 2021	20 Aug 2021 – Large Woody Material Downstream Passage and Placement Clarification (Section 4, LWM for Downstream Habitat Enhancement)	Vegetation Management Plan
10 Sep 2021	Sep 2021 – Updated Biological Assessment	Biological Assessment
10 Sep 2021	Sep 2021 – Table of Substantive Changes from Draft BA	Biological Assessment
10 Sep 2021	Aug 2021 – Water Temperature Model Sensitivity Analysis	Mitigation Plan
17 Dec 2021	Dec 2021 – Vegetation Management Plan	Vegetation Management Plan
16 Jun 2022	June 2022 – Draft Flood Retention Expandable Facility Mitigation Plan: Aquatic Species and Habitat, Riparian and Stream Buffer, Large Woody Material, Surface Water Quality	Mitigation Plan
16 Jun 2022	June 2022 – Draft Wetland Mitigation Plan	Mitigation Plan

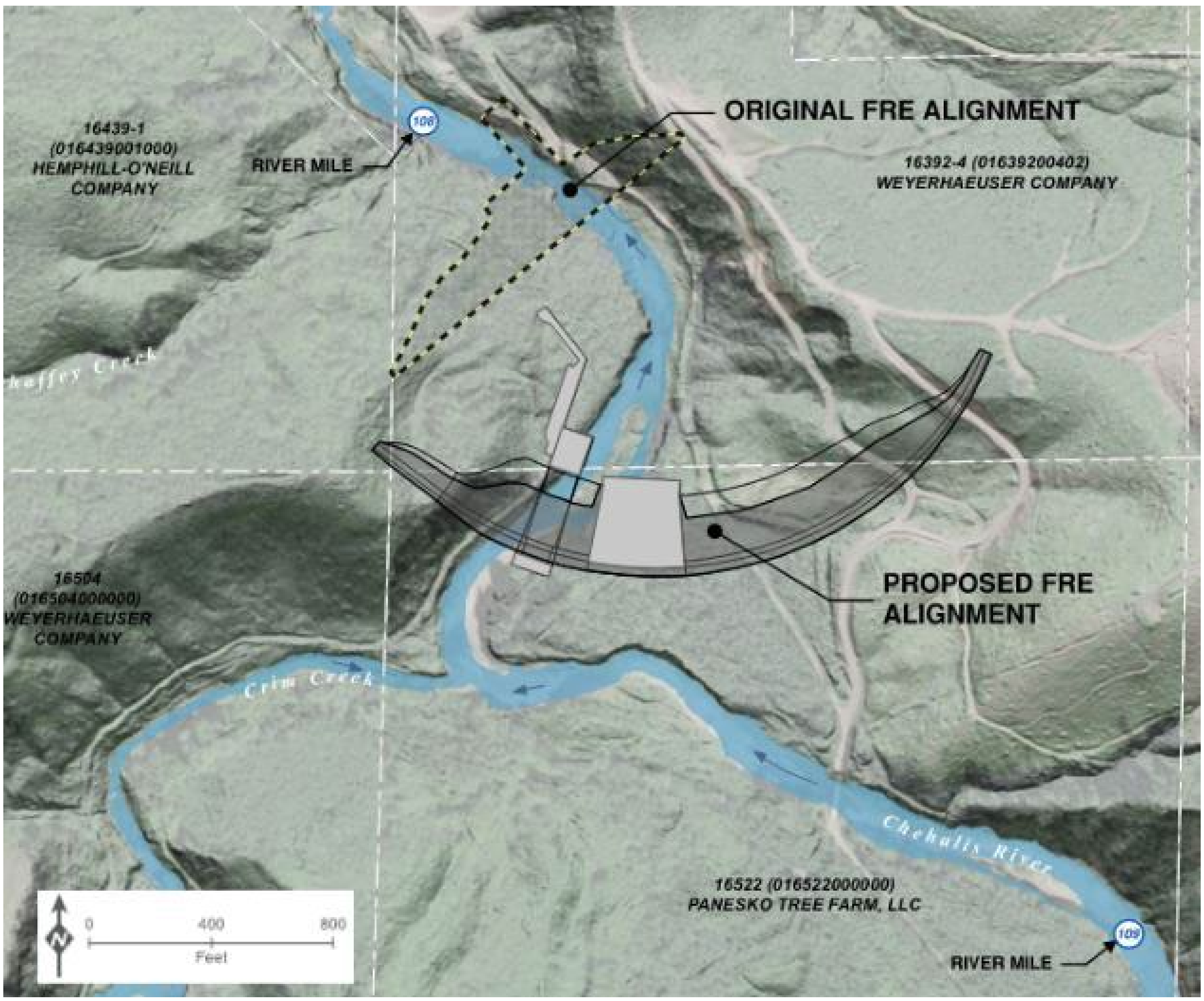
1.4 Proposed Project Purpose and Need

The Proposed Project's purpose and need have not changed from those previously established and documented within the WDOE and USACE DEISs.

1.5 Proposed Project Alignment, Size, and Feature Changes

To minimize impacts to an identified TCP, the FRE has been realigned from river mile 108.1 upstream to river mile 108.4 within the original footprint of the Proposed Project (Figure 1-1).

Figure 1-1. Original vs. Revised FRE Alignment



The proposed revised FRE alignment maintains the same spillway elevation as the original FRE alignment with no changes to the original proposed inundation area proposed except for removal of approximately 1,300 feet of inundation area length between river mile 108.1 and 108.4. Figure 1-2 illustrates a 32-acre reduction in the maximum temporary inundation area during flood operations (from 856 acres to 824 acres) as a result of the proposed revised FRE alignment.

Figure 1-2. Reduction in Proposed Project's Maximum Temporary Inundation Area

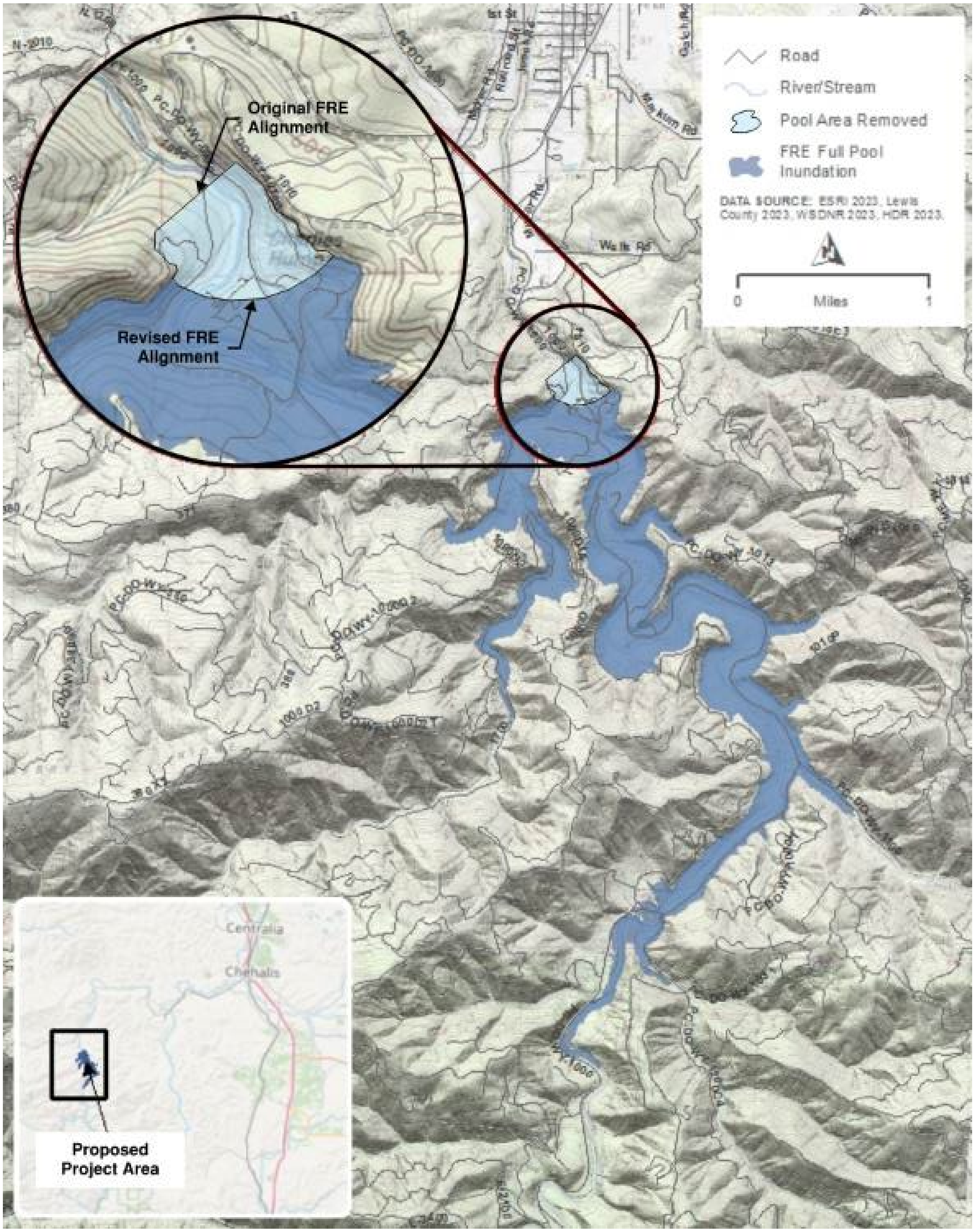


Table 1-3 provides a comparative summary of the physical changes between the original and revised FRE alignment.

Table 1-3. Proposed Project: Comparative Summary of Original and Revised Alignment

Element	Original	Revised
River Mile Location (approximate)	108.1	108.4
Maximum Temporary Flood Storage Volume (acre-feet)	65,000	62,000
Flood Storage Elevation (ft)	628	628
Length Abutment to Abutment (ft)	1,550	2,250
FRE Structure Height at Intake Channel (ft)	246	224
Fish Passage Conduit Structure Length (ft)	320	320
Spillway Width at Base of Dam (ft)	200	255

ft = foot/feet

Table 1-4 identifies the section of this document in which different features of the Proposed Project are described in detail, including any proposed revisions. Note that the RPDR section may refer to an appendix with additional information.

Table 1-4. Section of RDPR Addressing Proposed Project Feature Revisions

Proposed Project Feature	Description	RPDR Section
Operations	Expanded analysis and Proposed Project operations description.	4
Construction Phasing	Phased construction to accommodate volitional fish passage during construction.	6, 17
Temporary (Construction) and Permanent Impacts	Revised footprint, FRE realignment to minimize impacts to an existing TCP. Revised impacts to waters of the United States.	16, 18
FRE Configuration	Changed from straight axis gravity to curved axis to provide maximum distance between the FRE and important features associated with an existing TCP. Curved configuration provides potential for cross-section optimization during subsequent proposed FRE design phases.	10
Spillway and Hydraulic Energy Dissipation	Wider and stepped spillway with stilling basin for energy dissipation (formerly flip bucket).	6
Conduit and Fish Passage Structure	Addition of 9-foot-diameter evacuation conduit for operational flexibility to be used only during flood operations.	6, 8, 12, 14

Proposed Project Feature	Description	RPDR Section
Operations and Maintenance	Expanded definition of operation and maintenance requirements and plans. Description of operational capabilities.	14
Electrical and Controls	Additional design detail regarding power supply, emergency power, telecommunications, and automation.	13
Permanent Access Roads	Revised permanent access; additional permanent pool access analysis and data.	11
Quarries (material source volume requirements)	Revised quantities and new quarry sites and revised quarry boundaries.	7, 9

1.6 Proposed Project Description

The Proposed Project involves construction and operation of a flow-through dam for flood control, which is unlike a traditional detention dam. The Proposed Project's conduits and fish passage structure will be built at the same height as the existing riverbed. Under normal, non-flood operations, these conduits will remain open, allowing the river to flow freely through the structure and maintain its natural flow. Except during operations for infrequent major storm events, the mainstem of the Chehalis River will flow freely through the fish passage conduit structure system. Because flow-through dams minimally affect a river's natural flow under normal conditions, consequences such as blocking fish migration routes, sediment accumulation, restriction of water flow to downstream communities, and other negative cultural, environmental, and socioeconomic impacts are avoided or minimized.

The Proposed Project will not involve a permanent pool or reservoir. Rather, an area behind the dam will be inundated only temporarily when the structure is being operated for downstream flood reduction. When river stage and flow forecasts at the Grand Mound gage trigger operation, the conduit gates will slowly start to restrict the amount of water flowing through the conduits. This action will decrease peak flood flow out of the Upper Chehalis River and as described in the Proposed Project's purpose and need, will reduce flood flow and river stage elevations near Chehalis and Centralia to avoid catastrophic environmental and property damage. Following passage of the peak flood flow, the inundated area will be drained, and flow-through conditions re-established.

During temporary flood operations, upstream fish passage will be achieved using a collection and transport facility referred to herein as the Flood Fish Passage Facility (FFPF). Downstream fish passage will be temporarily delayed during flood operations.

As the needs of the Proposed Project differ from a conventional dam, the Proposed Project design team sought examples of a flow-through dam for flood control. Two structures were identified that offered design elements that are incorporated into the Proposed Project (Table 1-5).

Table 1-5. Examples of Flow-Through Dams for Flood Control

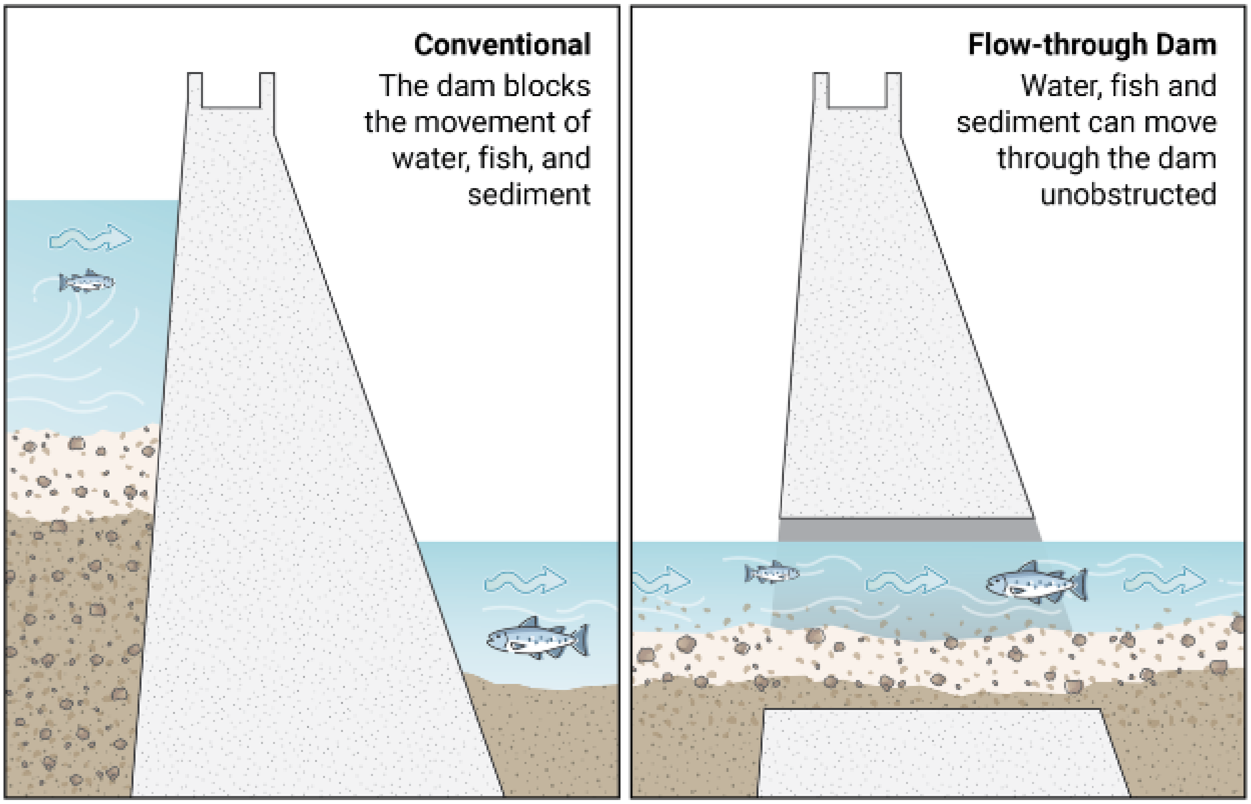
Name	Location	Design Similarities	Design Differences	Other Relevant Information
Masudagawa Dam	Shimane Prefecture, Japan	• Multiple shorter conduits at river elevation to mimic natural river conditions • Allows water and sediment to flow downstream • Climate change adaptation strategy	• The Proposed Project uses gates and valves to retain a pool during a flood event. Masudagawa simply backs up water when flows exceed the capacity of the conduits.	Used as a case study by the United Nations Climate Technology Centre and Network in their Guide to Adaptation Technologies for Increased Water Sector Resilience
Mud Mountain Dam	King County, Washington (WA)	• Primary conduit at river elevation • Secondary conduit to discharge and control flows higher in the temporary pool • Allows for short-term flood retention; water and sediment freely pass through during non-flood periods	• Mud Mountain has a single 9-ft conduit. The Proposed Project features multiple shorter and wider conduits specifically designed for volitional fish passage. • The FRE will be a concrete gravity structure made of roller-compacted concrete (RCC).	Rock and earth-filled dam, completed in 1948 by USACE

Mud Mountain Dam is similar to the Proposed Project in that it was constructed for short-term flood retention allowing water and sediments to freely pass through the structure during non-flood periods. The Proposed Project has been designed to improve certain operational elements over what is achieved by the Mud Mountain structure. For example, the Mud Mountain Dam's conduit located at river elevation is a 9-foot tunnel that is approximately 1,800 feet long and, at the time of construction in the 1940s, was not intended or designed for volitional fish passage. In contrast, the proposed FRE will have wider conduits and a shorter conduit length of approximately 320 feet. The wider and shorter conduits of the proposed FRE can more closely mimic the existing river channel flow conditions and roughness and will not impede fish migration.

Masudagawa Dam is a concrete gravity dam with a structure height of 48 meters (157.5 feet), approximately 67 feet shorter than the proposed FRE, and an approximate conduit length of 38 meters (124 feet). Similar to the proposed FRE, Masudagawa Dam incorporates two, 4.5-meter-wide (14.5-foot-wide), conduits located at river elevation that allow for free passage of normal river flows. However, Masudagawa Dam does not incorporate gates and valves and cannot regulate river flows during normal or flood operations. The proposed FRE, in contrast, will rely on gates to control the release of flow downstream during flood events which will allow for additional operational flexibility that can be used to avoid and minimize environmental impacts while still achieving downstream flood reduction objectives.

Figure 1-3 illustrates the major difference between a conventional dam and flow-through dam (and Proposed FRE).

Figure 1-3. Conventional Dam vs. Flow-through Dam



1.7 Proposed Project Operations

The *Chehalis Basin Strategy Operations Plan for Flood Retention Facilities* prepared by Anchor QEA, LLC (2017) and provided in Appendix M, formed the current basis for all operational analyses for the Proposed Project.

The Proposed Project team has assessed and identified several options for optimizing the Proposed Project's operations to reduce both the size and duration of temporary inundation during flood control events. This section summarizes those possibilities and is supported with additional details in Section 4 of the RPDR.

1.7.1 Operational Optimization

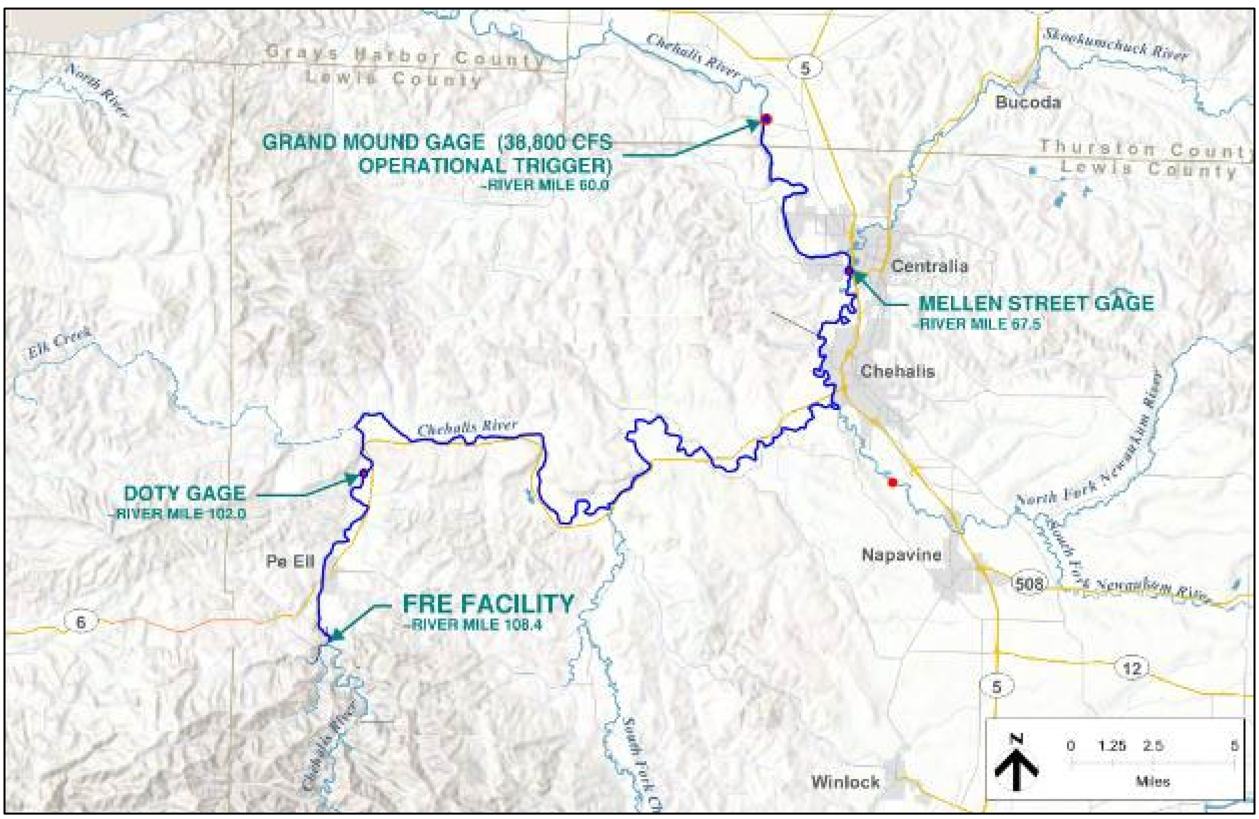
Rainfall across the Upper Chehalis Basin is not uniform, and no two storms are the same. For every storm event, precipitation will fall with varying intensities within the Upper Chehalis Basin and impact the tributary subbasins associated with Elk Creek, South Fork of the Chehalis, and Newaukum and Skookumchuck rivers differently. Recently recorded storm events that would have triggered operation of the Proposed Project suggest that reducing flow at the Proposed Project to 300 cfs is not always necessary when the primary precipitation contribution does not occur within the Willapa Hills above the Proposed Project.

The Proposed Project's operational trigger is a forecasted flow of 38,800 cubic feet per second (cfs) at the Ground Mound gage as described in Anchor QEA (2017). The trigger further anticipates initiating FRE gate operations (begin closing the gates at a rate of 200 cfs per hour) within 48 hours and maintaining those operations until the FRE flow out reaches a minimum flowrate of 300 cfs. For certain storm events, however, it may be possible to reduce the operational duration or allow for an increase in discharge flowrate beyond that described in Anchor QEA (2017) while continuing to achieve flood damage reduction objectives. Both optimization options would reduce flood storage and thus reduce the duration of operations resulting in minimizing associated environmental impacts.

HDR found that potential opportunities exist for better optimization of the reservoir operational rules in terms of maximum FRE releases, forecast triggers, and drawdown operations to achieve FRE performance objectives and water management goals given the sensitivity analysis results. Explicitly defining the goals, objectives, constraints, and performance metrics of system should be the next step in studying the flexibility and potential optimization of FRE operations.

Figure 1-4 shows the Doty, Mellen Street, and Grand Mound gage locations relative to the Proposed Project.

Figure 1-4. Key Gage and Map of Proposed Project Vicinity



2 Proposed Project Configuration

Primary components of the Proposed Project include the following:

- An RCC flow-through dam sized for 62,000 acre-feet of flood storage with estimated maximum dam structural height of 240 feet which could vary +/- 10 feet depending on final foundation excavation elevation.
- A crest length of approximately 2,250 feet.
- A foundation that will be approximately 290 feet wide at the widest point near the fish passage conduit and taper to as little as 35 feet wide at the abutments (edges). Exposed foundation post-construction will be backfilled to drain away from the FRE, topsoiled, and revegetated.
- A stepped overflow spillway, designed to pass flood flow up to and including the Probable Maximum Flood (PMF) without dam overtopping. The spillway includes an ogee crest overflow structure, stepped spillway chute, and stilling basin.
- Open channel diversion to manage flows during construction and allow volitional fish passage.
- Outlet works, including an evacuation conduit for flexible control of pool evacuation and low-level outlets for fish passage, flood regulation, and sediment transport.
- Fish passage facilities designed for volitional passage upstream and downstream prior to and after flood operations, and trap and haul fish passage during flood regulation periods only.

Conceptual design drawings of the Proposed Project are included in Appendix A.

The Proposed Project includes certain elements developed or designed to not preclude future expansion of the facility. Such design components include the foundation configuration, flow-through conduits and gates and water quality conduits. The parameters that were used to develop these designs are described in Appendix C. To be clear, the Proposed Project does not include future expansion and any such expansion would be subject to future federal and state approvals, tribal consultation, public review and environmental permitting; the Proposed Project is simply being designed to not preclude such expansion should it be proposed in the future.

The FRE would be constructed of RCC and considered a gravity dam structure (i.e., the dam cross-section has two-dimensional [2D] stability under the full range of seismic and hydraulic forces on the dam). Because of the revised FRE's enhanced curved shape and abutment conditions, it may be possible to decrease the cross-sectional area requirements as the design progresses due to arch-action typical of a curved concrete dam configuration. Therefore, the Proposed Project's footprint can be considered conservative in size.

The curved alignment of the revised FRE further distances the dam and related conduit structure from the TCP compared to a straight dam alignment. The curved configuration also improves the fish passage conduit, spillway, and stilling basin structure locations. Specifically, the curvature accommodates fish passage conduit alignment relative to the existing river while orienting the dam penetrations perpendicular to the dam's axis. A straight alignment would have resulted in increased reconstruction of the existing river alignment. The curved configuration also takes advantage of the favorable existing geological conditions and terrain at the left and right abutment locations.

The Proposed Project has been designed to meet downstream flood damage reduction objectives with the capability to reduce flood flows as described in the SEPA and NEPA DEISs. Because the Proposed Project will be constructed as an RCC gravity dam structure, a phased construction approach can be used and will accommodate an open-channel diversion to reroute the Upper Chehalis River while the primary fish passage and flood outlet structure is under construction. Once the primary outlet works structure is completed, the river will be rerouted into a realigned permanent channel and through the fish passage conduits while the remainder of the dam and spillway structures are completed.

The Proposed Project is designed to begin operations and temporarily store floodwater within 48 hours of a forecasted flow of 38,800 cfs at the downstream Grand Mound gauge. After flood regulation operations commence and the outlet works begin regulating outflows, fish passage through the outlet works will no longer be available. During these temporary storage events upstream fish passage is then provided via the Flood Fish Passage Facility (FFPF). At all other times, the Chehalis River will freely pass through the Proposed Project and allow fish to pass volitionally upstream and downstream.

3 Design Guidelines and Criteria

The following section summarizes the criteria used within the RPDR to design the proposed FRE dam and fish passage structure.

3.1 Survey Datum, Controls, and Topographic Information

The following datum was used for the revised FRE design:

State Plane Coordinate System, Zone:	Washington South
Horizontal Datum:	North American Datum of 1983 (HARN)
Vertical Datum:	North American Vertical Datum of 1988 (NAVD88; Geoid 03)

The existing ground surface is approximated using publicly available light detection and ranging (LiDAR) from the WDNR LiDAR Portal (Washington LiDAR Portal) Southwest WA Opsw 2019 data set. This data set is in North American Datum of 1983 (HARN) and NAVD88 (Geoid 03).

3.2 Hydrologic Criteria for Dam Design

3.2.1 Inflow Design Flood Requirement

All dams must include a spillway that allows safe routing of floods through the dam and reservoir without dam failure. Dam spillway requirements are dictated by two primary requirements and associated guidance documents: 1) the hazard classification of the dam (Federal Emergency Management Agency [FEMA] 2004a), and 2) incremental risks for loss of life or significant economic/environmental/lifeline impacts should the dam fail during a flood that is greater than the flood used for its spillway design (FEMA 2004b). Individual state dam safety offices as well as federal agencies with dam portfolios typically provide additional statutory requirements and guidelines specific to their portfolio characteristics. In all cases, that supplemental guidance must, at a minimum, comply with FEMA (2004a and 2004b) requirements.

A dam's hazard classification is first defined as outlined in FEMA (2004a). High hazard dams are those where failure would result in probable loss of life (one or more expected). Significant economic, environmental, and lifeline losses could also lead to a high hazard potential classification. Once a hazard classification is identified, the Inflow Design Flood (IDF) for a dam is defined by FEMA (2004b), as "the flood flow above which the incremental increase in downstream water surface elevation (WSEL) due to failure of a dam or other water impounding structure is no longer considered to present an unacceptable additional downstream threat. The IDF of a dam or other water impounding structures is the flood hydrograph used in the design or evaluation of a dam, its appurtenant works, particularly for sizing the spillway and outlet works, for determining maximum height of a dam, freeboard, and flood storage requirements. The upper limit of the IDF is the probable maximum flood."

Given the significant population and infrastructure at risk below the Proposed Project and based on HDR's experience performing many dam safety hazard classification studies, HDR has assumed the FRE structure would be classified as high hazard although a formal downstream hazard potential classification evaluation has not been completed. Because of the proximity of the population centers and significant infrastructure (including Interstate 5) downstream, HDR further assumed the RPD configuration of the FRE would require an IDF equivalent to the PMF. The design PMF has a maximum inflow of 72,215 cfs and an estimated total volume of 168,000 acre-feet as summarized further in Section 4.

3.2.2 Construction Flooding Hazards

A key design consideration for the FRE will be the systems required for management of normal and flood stream flows through the site during construction, which may require multiple stages of diversion and must support achieving both the design and environmental (fish passage) criteria and requirements established for construction of the dam and hydraulic structures.

The overall system for management of stream flows is a risk-informed decision. For projects involving large dams, it is common to provide temporary coffer dams, berms, and channels and structures capable of safely passing normal stream flows and floods with frequent return intervals as part of the project contract requirements. Risks associated with floods exceeding a specified recurrence interval are shared between the construction contractor and owner.

For the RPD, design of the staged stream flow and construction flood hazard management system includes all normal and flood flows up to and including the 25-year flood with 3 feet of freeboard.

Stream flow and flood hazard design criteria will continue to be evaluated through preliminary and final design and may be adjusted from the criteria adopted for the RPD.

3.3 Hydraulic Design Requirements

The hydraulic design requirements for the Proposed Project are a critical consideration for the proposed FRE and provide a primary basis for configuring the cross-sectional properties of the dam, fish passage/outlet works structure design, and spillway design. The hydraulic design criteria combined with the geology/geotechnical design of the excavation objective and the structural analysis of the dam provide the required layout information for the dam/hydraulic structure components (see Appendix C for additional information).

The hydraulic basis for the FRE design includes the following:

Table 3-1. Hydraulic Design Criteria

Parameter	Design Criterion	Comment/Reference
Flood Storage Volume	62,000 acre-feet	--
Minimum Flood Storage Reservoir Elevation	Natural Riverbed Elevation	--
Probable Maximum Flood (PMF)	69,800 cfs	2016 PMF
Spillway Crest Elevation	628.0 ft	--
PMF Peak Reservoir Elevation	643.3 ft	--
Dam Crest Elevation	651.0 ft	--
PMF Freeboard (Top of Parapet Wall)	654.5 ft	--
PMF Tailwater Elevation	473.25 ft	One-dimensional HEC-RAS model
Maximum Fish Passage Design Flow	2,200 cfs	5% exceedance flow; unrestricted fish passage for all flows up to 2,200 cfs
Climate Change, Maximum Fish Passage Design Flow	3,400 cfs	Climate Change Scalar of +55%
Minimum Fish Passage Design Flow	16 cfs	95% exceedance flow
Climate Change, Minimum Fish Passage Design Flow	14 cfs	Climate change Scalar of -14%
Primary Conduit Width and Height	12 ft 20 ft	One Primary or Workhorse Conduit, with Top-Seal Radial Gate
Primary Conduit Invert Elevation	427 ft	--
Secondary Conduit Width and Height	10 ft 16 ft	Four Secondary Conduits, with Bonneted Slide Gates
Secondary Conduit Invert Elevation	430 ft	--
Reservoir Evacuation Conduit Diameter	9 ft	For High Head Releases
Reservoir Evacuation Conduit Invert Elevation	432 ft	--

* See Appendix D for additional information.

The required flood storage volume is approximately equal to the flood volume of the 2007 flood of record. Refer to HDR (2017a) for additional details on the flood storage volume.

The hydrologic study performed by Watershed Science & Engineering (WSE 2016) and the hydrologic modeling of flood storage attenuation by Anchor QEA (2017) form the basis for hydraulic design of the FRE alternative.

Figure 3-1 shows the estimated reservoir stage versus area and storage curves for the proposed upstream FRE location. Figure 3-2 illustrates the hydraulic design criteria, elevations and storage volumes, on a typical section of the FRE.

Figure 3-1. Estimated Reservoir Stage vs. Storage and Area Curves

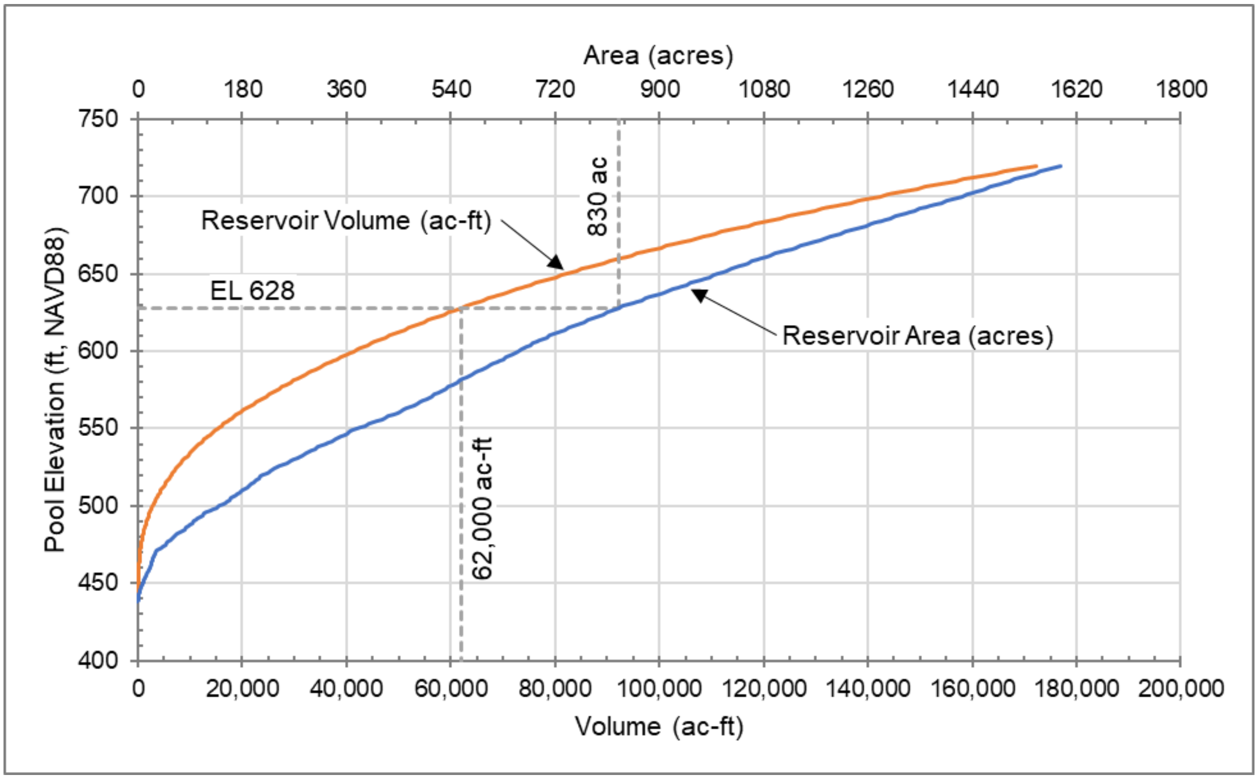
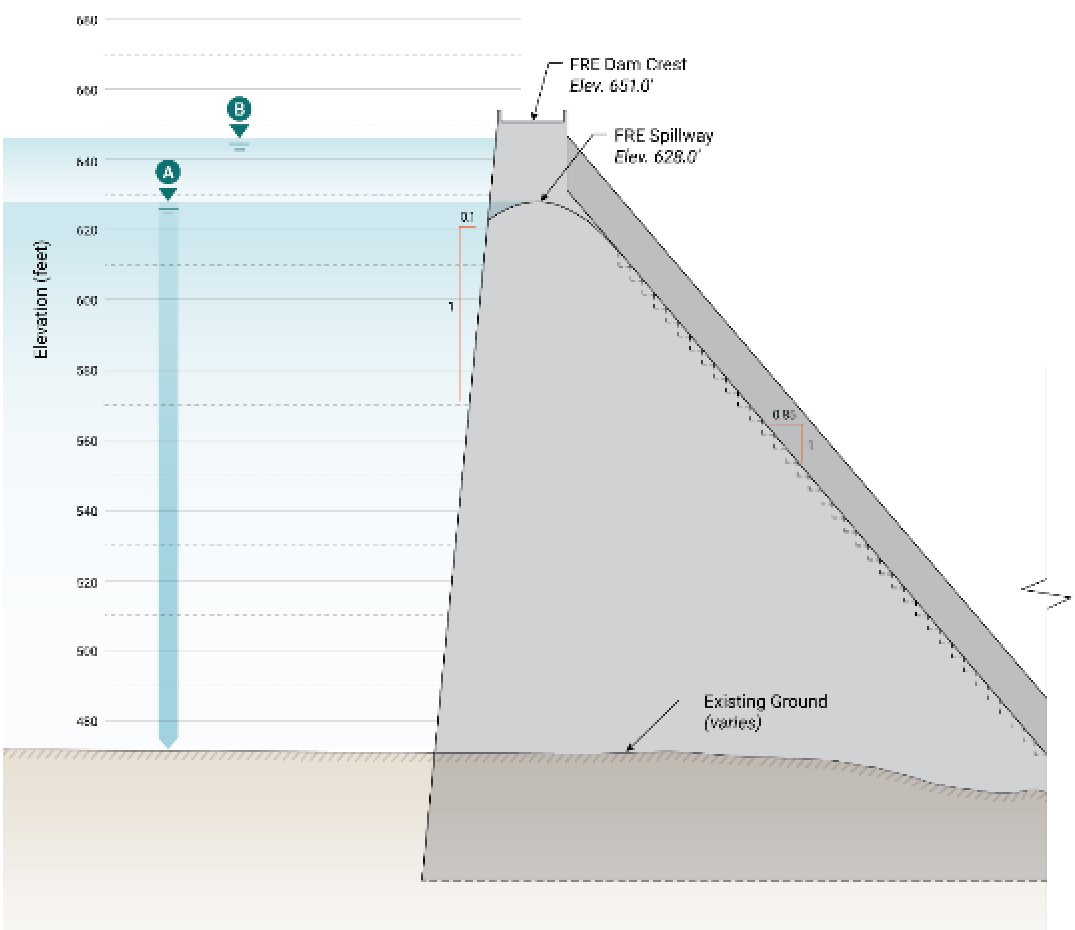


Figure 3-2. Hydraulic Design Criteria for FRE Typical Section

WATER LEVELS			
POINT	ELEVATION	STRUCTURE	SIGNIFICANCE
A	628.0'	FRE	Flood Storage: ~62,000 Acre-feet
B	643.3'	FRE	PMF Pool



3.4 Fish Passage Guidelines and Requirements

Design criteria and guidance for fish passage and protection at water retention facilities and channels has been developed by the Washington Department of Fish and Wildlife (WDFW), National Oceanic and Atmospheric Administration (NOAA) Fisheries, and U.S. Department of Fish and Wildlife (USFWS). These documents assist applicants in developing designs that promote safe and timely passage of aquatic species. The following documents provide design criteria and guidelines that have been used in the design described in this document. Additional documents used in development of the fish passage design are listed in Section 19 - References of this document.

- NOAA Fisheries. Guidelines for Salmonid Stream Crossings in WA, OR, and ID. (NOAA Fisheries 2022a).

- West Coast Region Anadromous Salmonid Design Manual (NOAA Fisheries 2022b).
- West Coast Region Guidance to Improve the Resilience of Fish Passage Facilities to Climate Change (NOAA Fisheries 2022c).
- Best Management Practices to Minimize Adverse Effects to Pacific Lamprey (USFWS 2010).
- Draft Fishway Guidelines for Washington State (WDFW 2000a).
- Draft Fish Protection Screen Guidelines for Washington State (WDFW 2000b).
- Water Crossing Design Guidelines (WDFW 2013).

3.5 FRE Structure Design Guidelines and Requirements

The current Proposed Project is being funded by the State of Washington. However, final design and construction may include a component of federal funding that would include federal review of the design. The design criteria outlined in this section is intended to satisfy both state and federal requirements.

The subsections below provide an overview of the FRE dam and hydraulic structures design guidelines and requirements that have been used to set the dam, fish passage structure, and spillway design configuration for the upstream project location. The specific design criteria for the dam are provided in Section 3.6.

3.5.1 Washington State Dam Safety Office Requirements

The WDOE Dam Safety Office (DSO) uses a risk-informed decision-making framework that incorporates consequence-dependent design levels such that increasingly stringent design criteria are applied as the potential for life loss or property damage increases. The procedure can also be tied to a downstream hazard classification (high, significant, or low) consistent with federal guidelines (FEMA 2004a).

Establishing the design/performance goal for the dam under the DSO guidelines is a stepped process. A numerical rating of the consequences of dam failure is estimated using guidance provided in Technical Note 2 of DSO’s dam safety guidelines (WDOE 1992). Multiple parameters are assessed under three broad consequence categories: 1) Capital Value of the Project, 2) Potential for Loss of Life, and 3) Potential for Property Damage. Numeric values are assigned to each parameter, and values are summed to estimate the consequence rating.

The FRE dam configuration presents unusual considerations in assigning an appropriate consequence rating because of the amount of the reservoir storage pool dedicated to flood storage only and the limited amount of time flood storage is used. The consequence rating for the FRE configuration would be low when the reservoir is not storing water, which would be most of the time. Similarly, because the FRE is being constructed so as not to preclude potential future expansion, HDR considered the consequence rating in that scenario, which would be lower when the water storage level is at normal maximum operational pool elevation, compared with when it is at flood pool capacity.

The consequence rating points is used to inform the hydrologic and seismic hazard design criteria, and the related geotechnical, structural, and hydraulic design/performance goals for the dams. The next step is to determine the loading annual exceedance probability (AEP) shown in the DSO guidelines. Based on current population at risk in the downstream corridor information, as well as experience with the federal hazard classification system for dams, the proposed FRE is classified as a high hazard potential structure.

A complete description of the consequences and loading criteria based on DSO guidelines for the structure is provided in Appendix F.

3.5.2 Federal Design Guidelines and Criteria Considerations

Federal agencies have well established guidelines for evaluating the safety of existing concrete dams such as the RCC configuration proposed for the Proposed Project. The federal agencies that have established design criteria and guidelines include the USACE, Reclamation, and the Federal Energy Regulatory Commission (FERC). Although there are some differences in the federal agencies' guideline details, the general approach is relatively consistent, and the agencies often refer to the guidelines developed by the other agencies. As previously noted, designs under the current scope of work are intended to satisfy design criteria of federal agencies where it is reasonable to do so; however, when in question, the USACE guidance will be given precedence.

3.5.2.1 USACE Engineering Manuals and Engineering Regulations

The USACE has comprehensive design guidance in the form of engineer manuals (EMs) and engineer regulations (ERs) that would be applicable. Of note are the following:

- EM 1110-2-2200, Gravity Dam Design (USACE 1995)
- EM 1110-2-2006, Roller-Compacted Concrete (USACE 2000)
- EM 1110-2-2100, Stability Analysis of Concrete Structures (USACE 2005)
- EM 1110-2-1603, Hydraulic Design of Spillways (USACE 1990)
- EM 1110-2-1602, Hydraulic Design of Reservoir Outlet Works (USACE 1980)
- EM 1110-2-1601, Hydraulic Design of Flood Control Channels (USACE 1991)
- ER 1110-2-1156, Safety of Dams Policy and Procedures (USACE 2014

With regard to ER 1110-2-1156, the USACE notes the following:

> *"Current USACE criteria must be used on all federally funded designs. When the design is being prepared for a sponsor on a cost reimbursable basis, the district DSO may consider use of state criteria. Deviations from USACE criteria require written concurrence from the USACE DSO."*

3.5.2.2 Reclamation Guidelines and Design Standards

In addition to publishing numerous dam design books and guidelines, Reclamation is a leader in development of concrete dam design methods and criteria, and in developing and incorporating risk-informed dam safety and design methods within its guidelines. Applicable Reclamation design guidance is as follows:

- Design of Small Dams (Reclamation 1987).

- Design of Gravity Dams (Reclamation 1976).
- Roller-Compacted Concrete, Design and Construction Considerations for Hydraulic Structures (Reclamation 2017).
- Public Protection Guidelines: a Risk Informed Framework to Support Dam Safety Decision-Making (Reclamation 2022a).
- Consequence Estimating Methodology, Interim, Guidelines for Estimating Life Loss for Dam Safety Risk Analysis (Reclamation 2014a).
- Dam Safety Risk Analysis Best Practices Training Manual (Reclamation and USACE 2019).
- Hydraulic Laboratory Report HL-2005-06, Research State-of-the-Art and Needs for Hydraulic Design of Stepped Spillways (Reclamation 2006).
- Engineering Monograph No. 25, Hydraulic Design of Stilling Basins and Energy Dissipators (Reclamation 1984).

3.5.2.3 FERC Guidelines

FERC regulates non-federal dams licensed for hydropower generation in the United States. A significant number of the dams under FERC's jurisdiction are large concrete gravity or arch dam structures, and FERC therefore has a well-established methodology and guidance on the design and safety evaluation of concrete structures:

- FERC Guidelines, Chapter 3, Gravity Dams (FERC 2016).

3.5.3 International Guidelines

There have been significant large concrete dam projects built around the world over the past 20 years including the largest RCC and concrete-faced rockfill dams in existence today. Significant advances in RCC technology have occurred, and the International Commission on Large Dams (ICOLD) Committee on Concrete Dams has captured the most current state of practice for RCC and other concrete dam design in several recent publications that will serve as important information for the Proposed Project dam alternatives.

- B145 Physical Properties of Hardened Concrete in Dams (ICOLD 2009).
- B165 Materials for Concrete Dams (ICOLD 2013).
- B177 Roller-Compacted Concrete Dams (ICOLD 2019).

3.6 Structural Design Guidelines and Requirements

A concept level risk-informed approach was established to evaluate and confirm the cross-section requirements of the RCC dam. The non-overflow and overflow (spillway) sections of the FRE represent conservative cross-sections and related structural height (see Appendix F), and the largest normal reservoir loading condition being considered for the site. A cross-section meeting the risk-informed design criteria for the maximum height under consideration will be capable of equal or better seismic performance for lower heights.

The risk-informed design criteria consider the structural response to various levels of seismic loadings. This approach is in accordance with the risk-informed criteria for large concrete dams under the federal guidelines for dam safety (Reclamation 2022a and USACE 2014). Further description of the risk-informed design criteria is provided in Appendix F.

The dam will be designed to meet required deterministic factors of safety for sliding, bearing, overturning, and flotation per the state and governing federal entity.

The concept-level design of the dam hydraulic structures and appurtenant structures was developed based on a deterministic design approach. The USACE design criteria was used in general unless noted otherwise. Hydraulic structures associated with potential failure modes (PFMs) will be designed using risk-informed design criteria during subsequent phases of design. Other codes used in design that USACE references include:

- American Institute of Steel Construction Specification for Design Fabrication and Erection of Structural Steel for Buildings
- American Welding Society Standard D.1.1, Structural Welding Code
- American Concrete Institute 318, Building Code Requirements for Reinforced Concrete
- American Institute of Steel Construction Allowable Stress Design 13th Edition
- Occupational Safety and Health Administration (OSHA)

3.7 Geotechnical Design Guidelines and Requirements

The geotechnical design for the revised FRE alignment has been based primarily on the extrapolation of geology and geotechnical information developed at the original FRE alignment along with limited site exploration and testing work at the revised alignment location. Additional site characterization work will be completed in 2024 at the revised alignment to confirm these extrapolations and address uncertainties, focusing in particular on the potential for siltstone and claystone materials in the vicinity of the right abutment to require specialized excavation and treatment.

Upon completion of additional site characterization, the following guidelines, manuals, and reference documents will provide the basis for the preliminary design of the RCC dam and associated fish passage/outlet works and spillway structures:

- Soil and Rock Slope Requirements
    - Design Standards No. 13, Chapter 4 Static Stability Analysis (Reclamation 2011).
    - Best Practices in Dam and Levee Safety Risk Analysis, Chapter D-7, Foundation Risks for Concrete Dams (Reclamation and USACE 2019).
    - Section C - Foundations of Reclamation's 1976 publication Design of Gravity Dams provides guidance on analysis of dam foundation stability (Reclamation 1976).
    - Bulletin B 88 – Rock Foundations for Dams (ICOLD 1993)
    - Bulletin B 129 – Dam Foundations (ICOLD 2005).
    - Methods of Geological Engineering in Discontinuous Rocks (Goodman 1976).

  - Analytical and Graphical Methods for Analysis of Slopes in Rock Masses (Hendron et al 1980).
  - Rock Slope Engineering (Hoek and Bray 1981; Wyllie and Mah 2004).
- Construction Dewatering
  - Design Standard No. 13, Chapter 21 Water Removal and Control: Dewatering and Unwatering Systems (Reclamation 2014b).
  - EM 1110-2-1914, Design, Construction, and Maintenance of Relief Wells (USACE 2022a).
  - Relief Wells for Dams and Levees Considering Landward Head, ASCE Journal of Geotechnical and Geoenvironmental Engineering, ISSN 1090-0241 (Keffer et al. 2023).
- Foundation Grouting
  - Design Standard No. 13, Chapter 15 Foundation Grouting, September (Reclamation 2014c).
  - EM 1110-2-3506 Engineering and Design Grouting Technology, (USACE 2017).
  - EM 1110-1-3500, Chemical Grouting, (USACE 1995).
- Foundation Treatment
  - Bulletin 129, Dam Foundations (ICOLD 2005).
  - Engineering and Geology Field Manual, Chapter 21, Foundation Preparation, Treatment, and Cleanup (Reclamation 2001).
  - Guidance for Surface Preparation of Dam Foundations (U.S. Society on Dams 2022).

3.8 Mechanical Design Guidelines and Requirements

The design of the mechanical components concentrated on general sizing of the trashracks, gates, and valves. Operating and miscellaneous metal components were not detailed at this phase of design. For further details on each component see Section 12 and Appendix H: Conceptual Chehalis Outlet Works Initial Analysis and Conceptual Gate Design Technical Memorandum (TM). The following additional design standards were followed for the outlet works components.

3.8.1 Trashracks

The trashracks were designed balancing the conflicting requirements of debris capture and fish passage. Optimization of the trashrack openings will be completed in subsequent design phases.

- Anadromous Salmonid Passage Design Manual. Fisheries West Coast Region. (NMFS 2022).
- Design Standards No. 6, Hydraulic and Mechanical Equipment, Chapter 12: Trashracks and Trashrack Cleaning Devices Phase 5 Final (Reclamation 2016) with some exceptions for fish requirements.

3.8.2 Bonneted Slide Gates and Tainter Valve

Design and sizing of both the bonneted slide gates and Tainter valves documented in this report followed EM 1110-2-2107 load combinations. Other design criteria documents include:

- American Society of Civil Engineers Paper No. 3000, Fixed Wheel Gates for Penstock Intakes (Skinner 1957).
- Design of Hydraulic Gates, 2nd Edition (Erbisti 2014).
- EM 1110-2-2107, Design of Hydraulic Steel Structures (USACE 2022b).
- EM 1110-2-6053, Earthquake Design and Evaluation of Concrete Hydraulic Structures (USACE 2007).
- EM 1110-2-2400, Structural Design and Evaluation of Outlet Works (USACE 2003).
- EM 1110-2-1602, Hydraulic Design of Reservoir Outlet Works (USACE 1980).
- ER 1110-2-1806, Earthquake Design and Evaluation for Civil Works Projects (USACE 2016).
- Guidelines for Evaluation of Water Control Gates (American Society of Civil Engineers 2017).
- Steel Construction Manual, AISC 360, 15th Edition (American Institute of Steel Construction 2017).

3.8.3 Gate Hoist

Mechanical design of the gate hoists and gate hydraulics was not completed as part of the design documented in this report. If a gantry type crane is used on the crest of the dam, the design shall consider referencing Construction Management Association of America Specification No. 70, Specifications for Top Running Bridge and Gantry Type Multiple Girder Electric Overhead Traveling Cranes.

3.9 Electrical Supply and Controls Design Guidelines and Requirements

The following codes and standards will be referenced for the electrical systems design as applicable:

- American National Standards Institute (ANSI)
- Applicable local codes and standards
- Institute of Electrical and Electronics Engineers
- National Electrical Code, ANSI/ National Fire Protection Association 70 latest edition
- National Electrical Manufacturer's Association, Power Switching Equipment, Publication SG-6
- National Electrical Safety Code, ANSI C2 latest edition
- National Fire Protection Association
- OSHA
- Underwriters Laboratories, Inc.

4 Hydrology

The understanding of the Chehalis Basin hydrology continues to be based on previous work completed. WEST Consultants, Inc. (WEST 2014) presented a thorough statistical analysis on gages in the basin. This work was confirmed and extended, but not changed, in WEST (2014), WSE (2017), and Anchor QEA (2017).

An understanding of the operational scheme for the Proposed Project was also presented in Anchor QEA (2017). A sensitivity analysis was performed on multiple parameters in the operations scheme to explore which changes may have potential to improve and minimize FRE operations. To support this work, HDR utilized the Hydrologic Engineering Center - Reservoir Simulation Software (HEC-ResSim) model developed by Anchor QEA (2016). This work supports an increased understanding of operational flexibility with a goal of qualifying minimization and avoidance operations that may decrease long-term impacts from the operations of the Proposed Project.

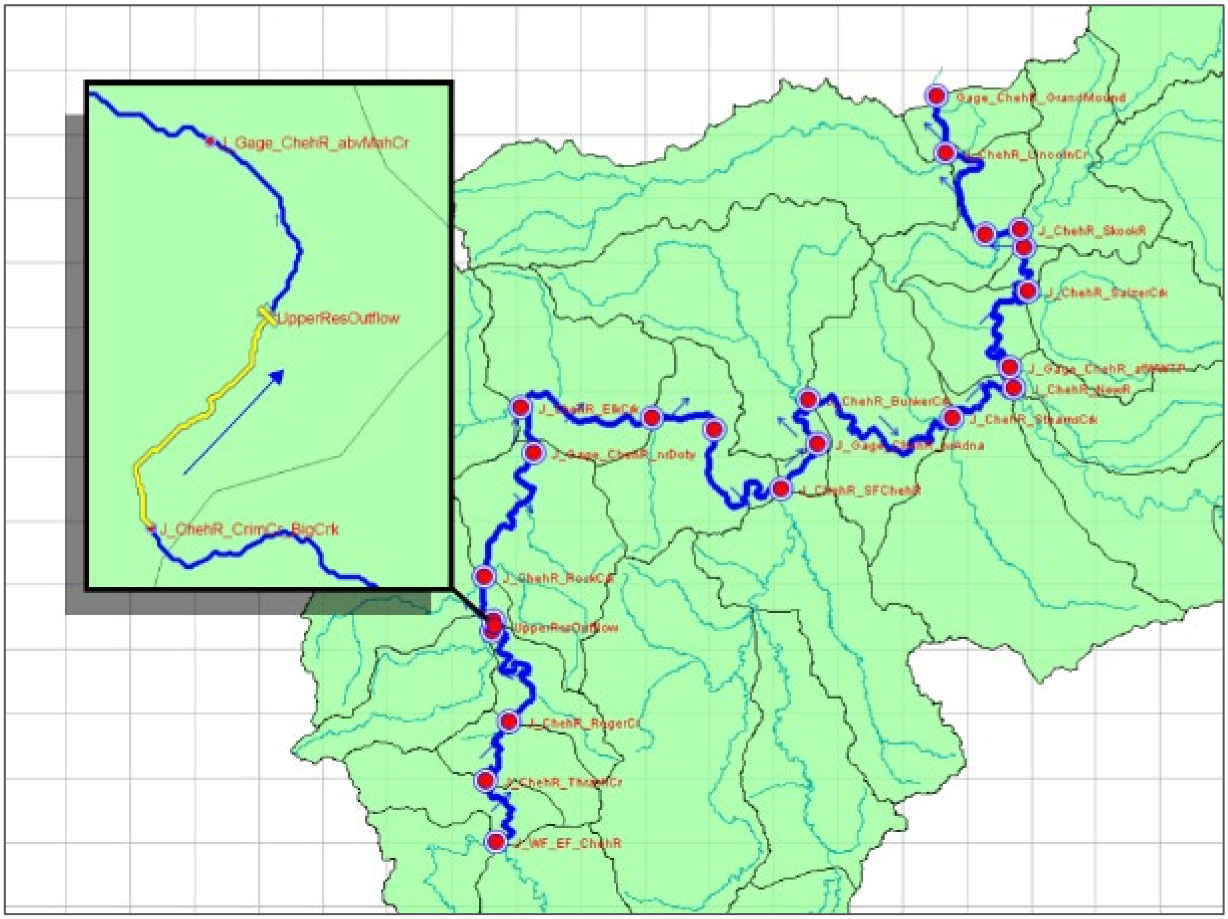
4.1 Operational Sensitivity Analysis Model

This section describes the model used for the operational sensitivity analysis for the Proposed Project. HDR utilized an existing HEC-ResSim operations model (Anchor QEA 2016). HDR enhanced this model by extending it downstream to quantify operational outcomes at points of interest through hydrologic routing.

Anchor QEA’s 2016 HEC-ResSim model was provided to HDR, along with its 2017 operations plan report (Anchor QEA 2017). HDR enhanced the model using HEC-ResSim software application version 3.3. Figure 4-1 depicts the HEC-ResSim model layout. The model includes the reservoir operation set developed and configured by Anchor QEA and an enhanced stream network extended from the FRE facility downstream to Grand Mound. The model reaches utilize both the Muskingum and Muskingum-Cunge routing methods. Dam reservoir operations were not modified for the analysis and remain as Anchor QEA wrote them in its ResSim model.

Figure 4-1. Snapshot of the Chehalis River Basin HEC-ResSim Model, Post-Enhancement

The enhancements including building out the model reaches and junctions downstream of the Proposed Project to analyze effects of operations.



5 Site Geologic and Geotechnical Characterization

5.1 Regional Geology

The Proposed Project's FRE dam site is located on the northern edge of the Willapa Hills, which represents a large upwarped anticline. The large gentle fold causes the rocks in this area to generally dip to the north at about 10 to 30 degrees. At the Willapa Hills core is the Crescent Formation, which consists of moderately to very strong intrusive and extrusive mafic igneous rocks that form steep slopes. During Crescent Formation deposition there were several episodes of subaqueous basalt flows with periods of quiescence in between flows where erosion of surrounding basalt occurred, and the eroded material was deposited in low lying areas as siltstone before being overlain by the next episode of basalt flows (Moothart 1992). Therefore, the Cresent Formation contains abundant siltstone lenses and volcanic breccia as well as basalt of varying quality, with baked flow contacts (Wells and Sawlan 2014; Moothart 1992). For more detail on the site geology and a geological map refer to the Geotechnical Data Report (Appendix E). The bedrock is overlain by Quaternary surficial deposits of primarily alluvial and colluvial origin and residual overburden soil resulting from the weathering of the bedrock that supports heavy vegetative cover.

5.2 Dam Site Geology

The most recent investigation near the revised alignment site, about 1,300 feet upstream of the original alignment, encountered similar conditions to those identified at the original alignment. This investigation included two borings and four geophysical survey lines and was intended primarily to confirm conditions along both alignments are similar; however, they did not significantly add to the dataset. Therefore, the dam site geology described below consists primarily of a summary of conditions at the original alignment that have been extrapolated to the revised alignment. For a more detailed discussion, refer to the Phase 1 Site Characterization TM (HDR and Shannon & Wilson [S&W] 2015).

5.2.1 Soil

Overburden soils consist of stream alluvium, colluvium, landslide deposits, and residual soil. Stream alluvium is typically silty fine sand, gravelly sand, and sandy gravel. Larger clasts range from pebbles to boulders, some as large as 3 to 4 feet. Modern Quaternary alluvium (Qa) is present in active stream channels and older Quaternary alluvium (Qao) is present in terraces more than 15 feet above the modern stream channel. Colluvium is typically sandy to gravelly clay or silt deposited on or at the base of hillslopes, primarily through gravity-driven transport. The colluvium often contains a high percentage of subangular boulders consisting of basalt and gabbro ranging widely in size and could be more than 2 feet in maximum dimension. Landslide deposits are made up of heterogeneous, mostly unsorted and unstratified debris. The soil in landslide deposits is highly variable, consisting of sandy silt (MH) to clayey or silty sand (SC/SM) to gravel with silt and sand (GP-GM). Clasts can range from gravel to boulders and be several feet in maximum dimension. Residual soil consists of lean clay (CL), elastic silt (MH), silt (ML), silt with sand (ML), sandy silt (ML), sandy silt with gravel (ML), silty sand (SM), silty sand with gravel (SM) and silty gravel (GM). The fines range from low to high plasticity. Residual soils frequently contain highly weathered clasts of bedrock that are angular to subangular.

5.2.2 Bedrock

The bedrock at the revised FRE alignment consists of both igneous volcanic rocks and Crescent Formation basalt and siltstone/claystone. Volcanic rocks (Tig) at the dam site have been identified as gabbro; which is high to very high strength, dark gray to black, occasionally white or black-speckled, aphanitic to medium grained rock. The Crescent Formation (pillow basalt [Tcb]) is characterized by massive basalt flows, pyroclastic flows, and tuffaceous sandstones. Crescent basalts are often in the form of pillow basalt flows but can also be locally intrusive. Several sequences of volcanism occurred during the deposition of Crescent Formation basalts resulting in interbeds of siltstone and claystone (Moothart 1992). Specifically, these materials were encountered as alternating sequences of pillow basalt (Tcb) deposition, and weathering and erosional events of the pillow basalt to silts and clays deposited within depressions and lithified to Crescent Formation siltstone/claystone (Tcs) and occasionally claystone breccias consisting of basalt clasts in a claystone matrix by subsequent events of pillow basalt flow deposition.

The Crescent Formation basalts (Tcb) range from weak to very strong, with strength increasing with depth. They are dark gray to gray-green fine to medium grained, with smooth to rough, closely to widely spaced, high to low angle joints with occasional mineral and rare clay infilling. The basalt was typically fresh to slightly weathered with occasional moderately to highly weathered zones. Iron oxide staining occurs locally with the basalt locally slightly vesicular. The Crescent Formation basalt makes up a large portion of the subsurface lithology at the proposed upstream FRE site. The Crescent Formation siltstone/claystone (Tcs) ranges from very weak to moderately strong, dark gray to black, very fine to fine grained, with smooth to rough, closely to moderately spaced, low to high angle joints, and with occasional clay infillings. Rock core samples were mostly fresh to slightly weathered with zones of moderate and high weathering. Low angle bedding planes were observed in the rock core.

5.2.3 Geologic Structure References

No evidence of active faults was found within the immediate vicinity of the proposed FRE site. Previous studies (S&W 2009) noted evidence of a possible fault zone in the downstream toe of the revised FRE alignment. However, field reconnaissance could not confirm these findings. There are three inactive faults mapped near the site (S&W 2009). The closest fault along which Quaternary movement/activity has been postulated to have occurred is the Doty fault, which is an east-west trending zone of fault segments 8 miles north of the FRE site.

6 Hydraulic Design

6.1 Introduction

This section summarizes the hydraulic design of the spillway and outlet structures to be included in the upstream FRE dam configuration. This section also summarizes the hydraulic design of the construction stream diversion system. To support the hydraulic design, reservoir stage vs. storage characteristics were updated and included in flood routing and outlet capacity analyses and evaluations. Flows used in the hydraulic analysis are based on WEST (2014), WSE (2017), and Anchor QEA (2017). Details related to the hydraulic design of the spillway and outlet works are provided in Appendix D, the Spillway Alternative Selection TM (HDR 2024a), the Hydraulic Design of Fish Passage and Evacuation Conduits TM (HDR 2024b), and the Construction Bypass Hydraulic Modeling TM (HDR 2024c).

In order to not preclude possible future expansion of the FRE (see Section 2), and due to the inability to easily modify existing embedded conduits and penetrations in the dam, the hydraulic analysis and design considered the maximum possible future storage conditions in sizing a number of the critical outlet works components such as the dam safety evacuation conduit.

6.2 Emergency (Dam Safety) Reservoir Evacuation Rate

A preliminary estimation of the maximum emergency reservoir evacuation rate required was evaluated using Reclamation (2022a). The estimated evacuation rate is used in sizing the evacuation (low-level) conduit size, the two fish passage conduits with the high head bonneted slide gates, and the fish passage conduits stilling basin. The estimated peak evacuation rate to meet dam safety requirements is 7,400 cfs. The computed emergency reservoir evacuation rate is conservative and will be further evaluated as the design is further developed.

6.3 Spillway and Spillway Chute

The FRE spillway will be an uncontrolled Ogee crest, discharging to a stepped chute with 4-foot-high steps converging with the radius of the dam. The steps will be a conventional reinforced concrete facing incorporated into the RCC dam section. The crest is set at elevation 628 feet with a hydraulic length of 300 feet, designed to pass 69,800 cfs. The spillway will include a bridge supported by four 4-foot bullnose (Type 2) piers, which increases the total spillway width to 316 feet. The piers are 20 feet long and linearly taper to 3 feet wide at the downstream end resulting in a clear span between piers of 60 feet. The steps begin at the downstream end of the piers as 1-foot steps and gradually increase in height to 4-foot steps.

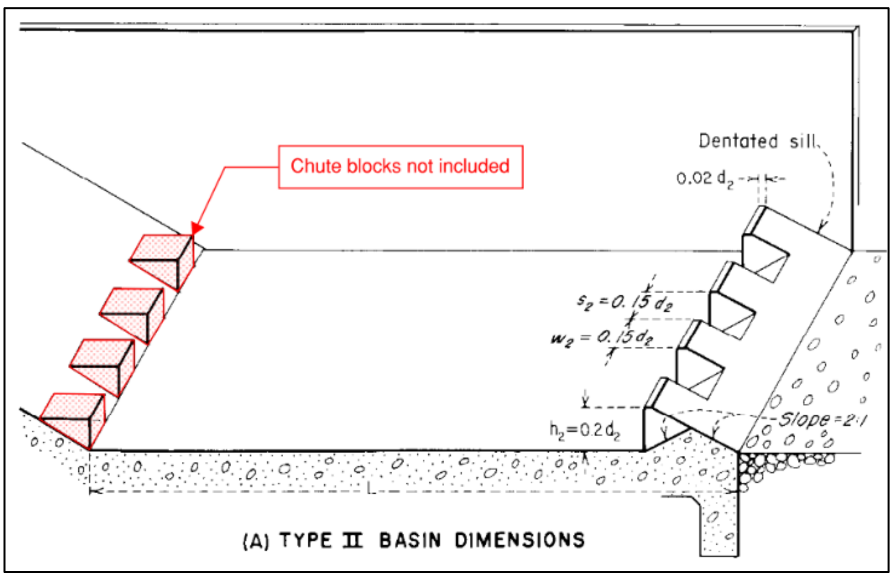
The spillway crest will have an elliptical crest shape that transitions seamlessly to the downstream spillway chute slope (0.85 Horizontal to 1 Vertical [H:V]) and is the same as the adjacent downstream slope of the dam. The crest is slightly overdesigned, with a design head to energy head ratio (H_e/H_d) of 0.90, to keep flow in contact with the spillway face.

The spillway chute converges, with the training walls parallel to the radial lines of the curved gravity dam configuration. The estimated maximum unit discharge over the spillway crest is 232 cfs/ft and increases to 260 cfs/ft due to chute convergence. The flow regime can be characterized as skimming flow at the design flow, where the flow skims in a reasonably coherent stream along a line connecting the tips of the steps. The steps dissipate energy through deflection of a portion of the nappe back into itself. Air entrainment is expected to occur approximately 116 feet from the spillway crest, beyond which the flow profile will be highly aerated. The spillway chute training wall heights are sized to contain the aerated flow with a factor of safety (1.5). The chute walls range from 15 feet high at the piers to 16 feet at the stilling basin (measured vertically from the tips of the steps).

6.3.1 Spillway Stilling Basin

The stilling basin will be 50 feet long, Type II basin (Reclamation 1987), which includes a 2.5-foot-high dentated endsill. A Reclamation Type II basin is shown on Figure 6-1. The proposed stilling basins are modified Reclamation Type II stilling basins because chute blocks are not included in the proposed stilling basins. Chute blocks are used to dissipate energy but because the spillway chute is a stepped spillway chute, enough energy will be dissipated on the spillway prior to flow entering the proposed stilling basin that chute blocks are not necessary. The stilling basin invert would be at an approximate elevation of 430 feet, which is deeper than hydraulically needed, but closer to the anticipated bedrock elevation. A top of wall elevation of 477 feet is required to adequately contain the aerated flow entering and the hydraulic jump that will develop in the stilling basin.

Figure 6-1. Type II Stilling Basin



Source: Reclamation (1987), edited

6.4 Fish Passage Conduits

The fish passage conduits are primarily intended to function with the FRE structure and were designed for fish passage flows scaled for climate change as described in the fish passage design flows (Section 15.3), where the gates are normally open for fish passage and only closed for flood retention. The conduits are designed to mimic the hydraulic characteristics of the normal river flows when compared to the existing rock channel downstream of the dam. When the fish passage conduit gates are closed, the evacuation conduit will be used for reservoir releases.

The conduit structure is 320 feet long and consists of one primary fish passage conduit and four secondary fish passage conduits. Figure 6-2 shows a schematic plan view of both the primary and secondary fish passage conduits. The primary fish passage conduit has a constant width of 12 feet from inlet to outlet. The secondary fish passage conduits have a constant width of 10 feet until the merging of two conduits into one as they approach the outlet. The convergence is a function of the radial orientation that aligns with the dam's curvature (the radius of the FRE is 1,200 feet) and maintains a minimum wall thickness between the conduits. The conduits are covered for their entire length (refer to Section 15.3.5 for discussion of artificial lighting within the fish passage conduits).

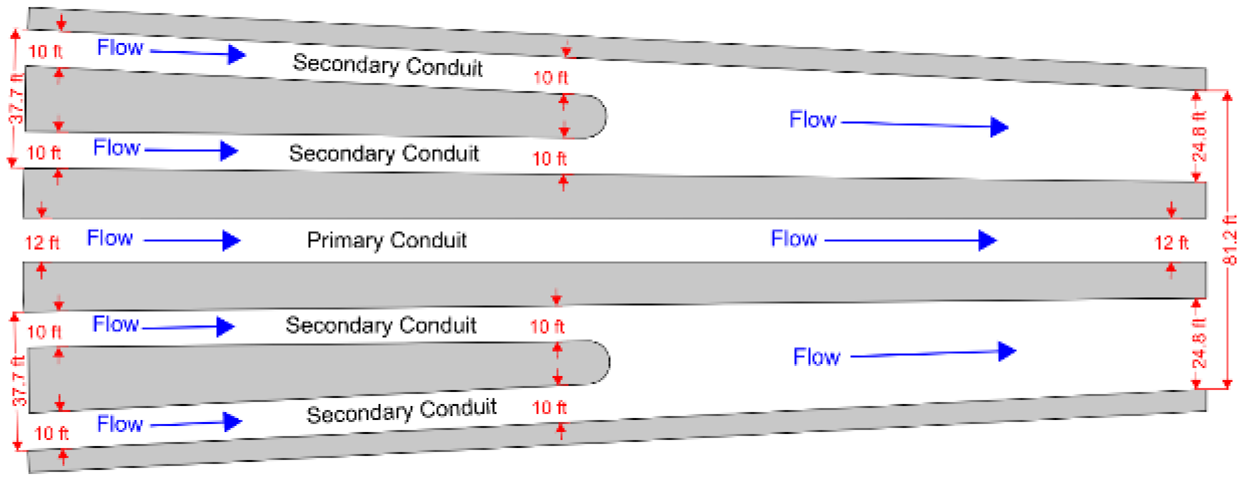
The fish passage conduits have been sized to pass the 95 and 5 percent exceedance probability river flows, which were scaled for climate change, as discussed in the Fish Passage Section 14.5. The conduits have elliptical entrance curves on the roof and sidewalls to improve flow convergence into the conduit, preventing flow separation at high flows. The invert profile incorporates a flat section between the inlet and the gate seal, followed by a half-percent slope, and a parabolic drop into the stilling basin.

The fish passage conduits will be full for flow rates greater than a 2-year return period. This is primarily due to the fish passage conduits tailwater and invert elevations.

A 2D hydraulic model was used to analyze the flow conditions through the fish passage conduits during the 95 and 5 percent exceedance flows, which were scaled for climate change. The model results were compared to the hydraulic characteristics of the same river flows through the existing rock channel downstream of the dam. Velocities in the proposed conduits do not exceed those in the naturally occurring section downstream in the Chehalis River. The flow depths in the proposed conduits are greater than those in the same reach downstream of the Proposed Project.

Chehalis River upstream of the conduits and downstream of the fish passage conduit stilling basin are proposed to be realigned to pass the river through the fish passage conduits and stilling basin. Crim Creek is proposed to be extended to its new confluence with the Chehalis River. The realigned Chehalis River channel upstream of the conduits is about 1,050 feet long and is referred to as the approach channel. The realigned Chehalis River channel downstream of the stilling basin is about 500 feet long and is referred to as the discharge channel. The lengthened portion of Crim Creek is about 600 feet long. A 2D hydraulic model was used to analyze the flow conditions in the proposed channels for the 95 and 5 percent exceedance flows. The model results were compared to the hydraulic characteristics of the reference reaches near the Proposed Project. Velocities and depths in the proposed channels roughly match those of the reference reaches.

Figure 6-2. Schematic Fish Passage Conduit Layout



Note: Overall length of the conduit structure is 320 feet. The conduits converge with the radius of the FRE which is 1,200 feet.

6.4.1 Stilling Basin

The stilling basin for the fish passage outlets was sized to function at the 100-year AEP flow event with all gates fully open, and during emergency (dam safety) reservoir drawdown with the two high-head bonneted slide gates operating at a combined flow of 7,400 cfs and a reservoir crest that exceeds the planned FRE crest by 63 feet (Appendix D).

6.4.2 Sediment Mobilization

The conduit structure stilling basin endsill elevation is set for fish passage and much larger than normally recommended and could cause rapid sedimentation of the stilling basin, meaning that the stilling basin will function as a sediment trap. However, as flowrates increase it is anticipated that some of the accumulated sediment would be flushed out. A preliminary analysis looked at the shear stress to determine what size of material can be mobilized at what flowrate. During a flood event, it is estimated that gravel sized material and smaller could be evacuated from the stilling basin. Additional information is available in the Fish Passage and Evacuation Conduits TM (Appendix D).

6.4.3 Climate Change

The climate change information has been incorporated using peak flow scalars that were derived from the 12 global climate models produced by WDOE's consultants (WSE 2023). The late-century ensemble average maximum scalar (+55 percent) has been applied to the historic high fish passage flow (5 percent exceedance). The mid-century average minimum scalar (-14 percent) has been applied to the historic low fish passage flow (95 percent exceedance). This is a reasonable approach for climate change conditions as the scalars are conservative but not overly conservative. Additional detail regarding the climate change flow can be found in Section 14.5.

6.5 Evacuation Conduit

After the fish passage gates are closed for flood retention, the low-level evacuation (dam safety) conduit will be used for reservoir releases.

A conceptual evaluation of the reservoir evacuation conduit was conducted. This evaluation included determining the flow capacity of the Howell Bunger valve (HBV) at different valve openings and reservoir elevations to determine when the valve could be safely operated. A conceptual baffle hood size also was assessed. This analysis is limited as it only considered one valve size, but the conceptual results can be used to inform reservoir operations and design development.

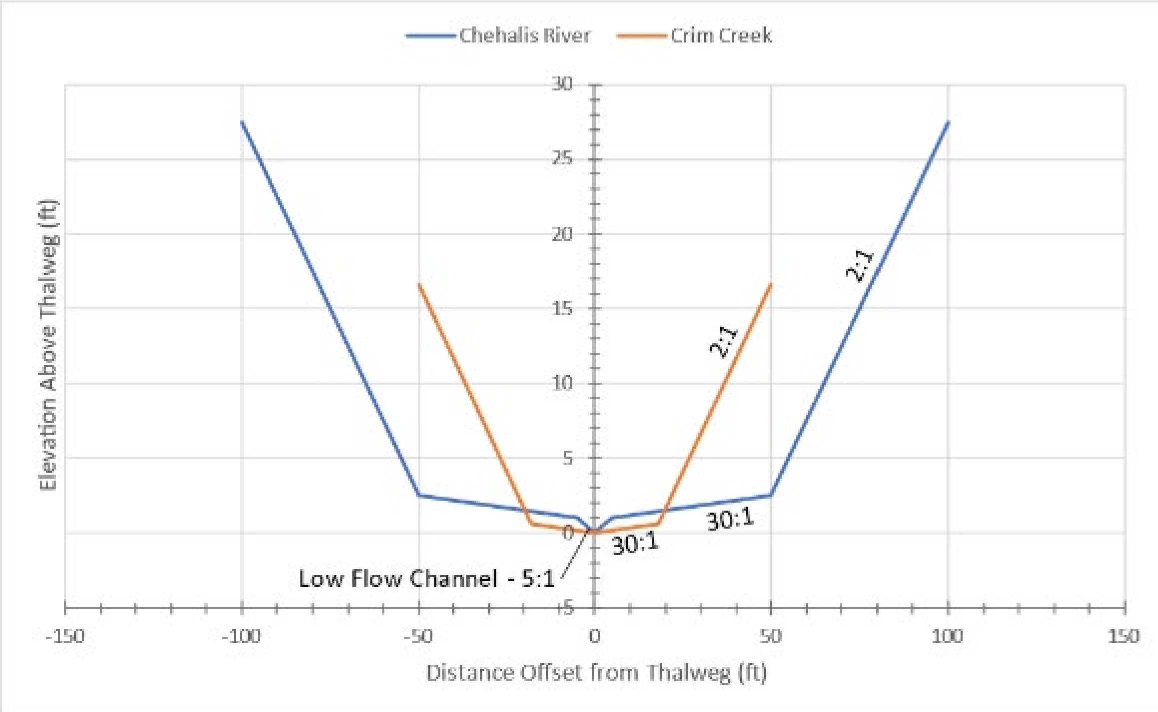
6.6 Construction Bypass Channel

Bypass channels would be used during the first phase of construction to temporarily divert Crim Creek and the Chehalis River. The proposed construction bypass channels would be sized to contain the 25-year AEP stream flow. The Chehalis River and Crim Creek would be diverted upstream of the dam to bypass the dam construction site. Figure 6-3 illustrates typical cross sections for the Chehalis River and Crim Creek bypass channels. These channel cross sections and slopes are conceptual based on the WDFW's stream simulation design approach to mimic observed geomorphology. The design flows used in the stream simulation design are provided in Section 15.3.1.3. Flow depths and velocities results at different flows are presented in HDR (2024c; Appendix D). The channel experiences high velocities under the peak flood events modeled. Some form of bank protection will be included to mitigate damage to the bypass channels if high-flow events occur while they are operational.

To analyze the range of hydraulic conditions within the proposed construction bypass channels under a range of flows, a 2D hydraulic model was employed. This informed, at a conceptual level, the minimum size of the channels and considerations for construction methods and quantities.

The preliminary design should further refine the channel grading to smooth the tie-in points at the upstream and downstream ends of the channels. The bypass channel needs to achieve a continuous low-flow channel for fish passage.

Figure 6-3. Typical Bypass Channel Cross Sections



7 Geotechnical Design

Limited geotechnical site characterization work has been completed at the revised FRE alignment location. Several phases of site characterization were previously completed approximately 1,300 feet downstream at the original alignment location. All of this site characterization information serves as the basis for extrapolating and developing anticipated geology and subsurface conditions for the Proposed Project and developing the geotechnical design for the RCC dam and spillway, fish passage, and outlet works structures components. This has resulted in a reasonable and conservative foundation design.

Geotechnical design is a critical element of the dam design and includes:

- Establishing excavation requirements (called excavation objectives) for the dam, spillway, and conduit structures
- Performing seepage and stability analyses of the dam foundation and abutments
- Developing foundation treatment requirements
- Addressing other geotechnical design considerations such as construction and permanent dewatering, temporary and permanent excavation slope stabilization, excess excavation material storage, and landslide stabilization

The initial investigation data of two borings (located about 250 to 350 feet downstream of the alignment due to lack of property owner permission) and four geophysical survey lines at the revised FRE alignment are not sufficient for accurate design, even at a conceptual level. Therefore, conservative estimates related to work quantities, construction schedule, estimated construction costs, and other construction risks are required at this phase. These uncertainties will be reduced as additional site characterization work is completed during the next phases of design. As additional data is gathered, the size of the footprint and cross section likely will be reduced because conservative estimates were required at this stage of the project. The following sections are summarized from the *Conceptual Geotechnical Design Report* (Appendix E) and reflect the extrapolated geotechnical considerations for the Proposed Project's geotechnical design.

7.1 Landslide Evaluation

Thirty-three landforms interpreted as landslides or possible landslides have been identified in the vicinity of the Proposed Project and temporary inundation area. HDR performed a preliminary evaluation of these landslides by reviewing previous reports and available LiDAR topographic information for the area. This evaluation focused on identifying landslides that may directly affect construction activities and/or long-term dam safety. This review formed the basis for developing recommendations for the planned 2024 field investigation program (Appendix E).

Figure 7-1 is a map of these landslides with preliminary recommendations for stabilization provided in Table 7-1. Table 7-1 further lists landslides identified as needing monitoring and further investigation. Stabilized landslides will also require long-term monitoring through the use of piezometers and inclinometers. Stabilization methods will include buttressing, installing drains, or a combination thereof.

Figure 7-1. Landslide Location Map

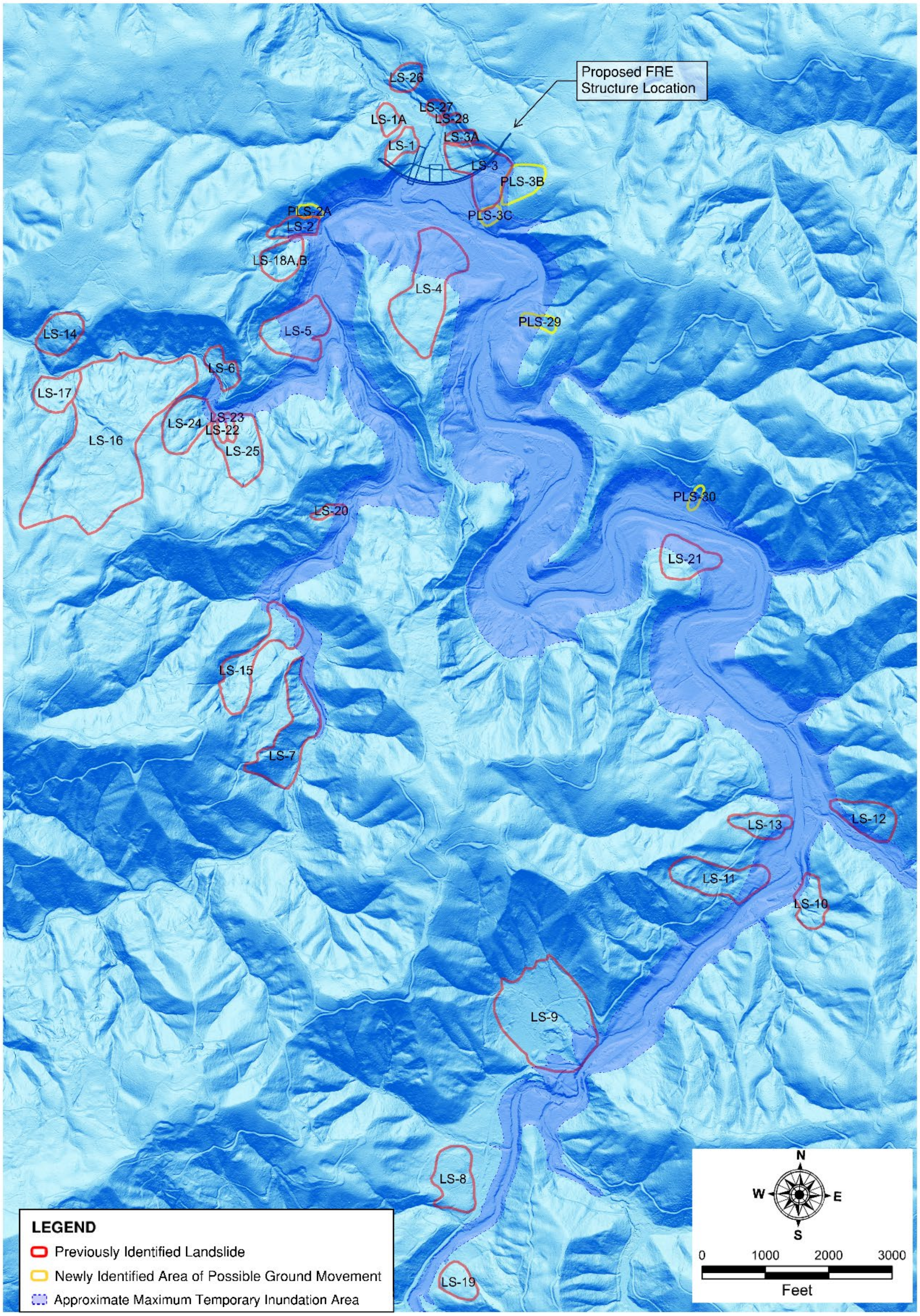


Table 7-1. Preliminary Landslide Recommendations

Landslide ID	Stabilize	Discussion
LS-1	**Yes**	Requires stabilization due to proximity to dam and appurtenant structures.
LS-1A	Potentially	Downstream of proposed dam alignment. Stabilize if area needed for site access.
LS-2	**Yes**	Likely unstable during drawdown and could become significant maintenance issue.
PLS-2A	Potentially	Include with LS-2 after additional field mapping and exploration.
LS-3, 3A	**Yes**	Requires stabilization due to proximity to dam and appurtenant structures.
PLS-3B, 3C	Potentially	Include with LS-3 after additional field mapping and exploration.
LS-4, 5	**Yes**	Likely unstable during drawdown and could become significant maintenance issue.
LS-6, 7, 8	No	Will not be regularly inundated and less proximal to proposed dam.
LS-9	No	Mostly landslide deposits and only slightly inundated by 100-year flood.
LS-10	No	Will not be regularly inundated and less proximal to proposed dam.
LS-11, 12, 13	No	Stability unknown. Less likely to present significant maintenance issue due to size and distance from dam site.
LS-14	No	Will not be regularly inundated and less proximal to proposed dam.
LS-15	No	Stability unknown. Less likely to present significant maintenance issue due to size and distance from dam site. Only small area inundated during 100-year flood.
LS-16, 17	No	Will not be regularly inundated and less proximal to proposed dam.
LS-18A, B	**Yes**	Likely unstable during drawdown and could become significant maintenance issue.
LS-19	No	Will not be regularly inundated and less proximal to proposed dam.
LS-20 to 25	No	Stability unknown. Small and distant from proposed dam site. Less likely to become a maintenance issue. Only small areas inundated during 100-year flood.
LS-26, 27, 28	Potentially	Downstream of proposed dam alignment. Stabilize if area needed for site access.
PLS-29	Potentially	Evaluate after additional field mapping and exploration.
PLS-30	No	Stability unknown. Small and distant from proposed dam site. Less likely to become a maintenance issue. Only small areas inundated during 100-year flood.

7.2 Excavation Objective

A large concrete dam must be constructed on a sound foundation. This requires establishing a foundation excavation objective that can be used to identify rock with the desired strength, deformability, and seepage characteristics. Ultimately, site characterization information will be used to define the excavation objective with a multi-attribute bedrock characterization model that considers rock type, weathering, rock strength designations, rock mass characterization systems (i.e., Rock Mass Rating), Rock Quality Designation, and results of surface and downhole geophysical testing (i.e., seismic refraction tomography used to identify compression and shear wave velocities within the rock). Between borings, the excavation objective is typically based on results of seismic refraction surveys such as compression wave velocity ≥ about 8,000 to 9,000 feet per second (ft/s), or shear wave velocity that defines the limit of ripability (> 5,000 ft/s).

An excavation objective has been developed for the Proposed Project based on expected loads and other requirements necessary as to not preclude future expansion as shown on Sheets 2EX-1 and 2EX-2 in Appendix A.

The Proposed Project's excavation objective was developed with limited site-specific data and relied heavily on site characterization information from the original FRE alignment location, which will be confirmed through additional site characterization work at the realignment location in 2024. When correlated to the two recent borings and compared to previous data, a compression wave velocity range of 4,000 to 5,000 ft/s generally corresponds to the top of weathered rock; a compression wave velocity range of 8,000 to 10,000 ft/s corresponds to competent rock that is suitable for the dam foundation. A conservative estimate for the depth of excavation was created along the dam axis profile. The excavation objective was then projected upstream and downstream to the limits of the dam/foundation contact reflecting changes in topography.

Conservatism was applied where the alignment passes through a large, mapped landslide in the right abutment. Allowance for construction equipment access as well as control channels for surface runoff are generally included in the excavation layout. Refinements will be made to the excavation objective during future design phases as additional subsurface data is gathered to address the uncertainties accounted for during conceptual design.

The overall excavation objective, although approximate, provides a reasonable, conservative depiction of required excavation, and a suitable basis for estimating the corresponding quantities of RCC and conventional vibrated concrete (CVC) materials including aggregate that will be required for dam construction. These estimates have been incorporated into the RPDR.

7.3 Foundation Treatment Systems

A system of foundation treatments will be necessary to provide a suitable bearing condition for the dam and full integration of seepage control provisions in the foundation and body of the dam. Foundation treatments include:

- Excavation surface preparation including cleaning, removal of unacceptable rock, shaping of the excavation with dental and shaping block concrete, and treatment of localized defects such as shears or abrupt changes in the rock surface.

- Consolidation grouting of the upper 10 to 20 feet of the rock below the dam to improve foundation modulus and seepage performance.
- Installation of a grout curtain cutoff to limit the amount of seepage under the dam.
- Installation of a drainage curtain downstream of the grout curtain to reduce uplift pressures acting on the base of the dam.
- Other systems such as temporary and permanent slope stabilization anchors.

HDR analyzed the available subsurface data from boreholes drilled during a 2023 site investigation and previous site investigations located within 600 feet of the revised FRE alignment to develop preliminary foundation seepage cutoff and foundation drainage recommendations aimed at reducing foundation seepage losses, minimizing adverse foundation seepage water pressures, and reducing the likelihood of seepage-related failure mode development. These recommendations are preliminary and may change considerably based on data gathered during future site investigations.

The existing data suggest that the primary foundation rock characteristics that will influence seepage are discontinuities such as fractures, unit contacts, and brecciated zones that may be related to either faulting or flow emplacement. The available geologic data suggest most features and areas requiring grout treatment can be adequately treated using cement-based high mobility suspension grouts; a standard suite of several high mobility suspension grout mixes with different viscosities, along with one mix that includes a sand component for treating larger voids is recommended.

The current design includes a two-row grout curtain that generally follows the dam alignment with the following design properties:

- Holes inclined at 75 degrees (15 degrees from vertical) with the azimuth of one row generally pointing toward the left abutment and the other row generally pointing toward the right abutment and varying across the curved alignment of the dam.
- Primary holes spaced 20 feet apart with secondary holes set on split spacing between each set of primary holes.
- Tertiary and quaternary holes added at selected locations based on observations made during drilling and grouting of the primary and secondary holes. The criteria for adding tertiary and quaternary holes have not yet been developed.
- Upstream and downstream rows spaced 10 feet apart (perpendicular to the axis of the grout curtain and dam alignment).
- Total grout curtain depth of up to 250 feet below the current existing top of rock. The depth of the grout curtain will decrease in the abutment areas based on the varying and reducing reservoir water levels along the dam/abutment contact.
- So as to not preclude future expansion, a lateral extent of up to 200 feet beyond each end of the FRE equates to a total grout curtain length requirement of 2,700 feet.

The sequence for implementation of the grouting program should generally follow these guidelines, with allowance for adjustments to accommodate the simultaneous occurrence of other construction processes, such as foundation excavation:

- Completion of primary holes prior to secondary holes.
- Completion of the downstream line of grout holes prior to completion of the upstream line.
- Addition of tertiary and quaternary holes based on observations of water pressure tests and grout takes during drilling and grouting of primary and secondary holes.
- Verification testing consisting primarily of water pressure tests conducted in holes drilled following completion of all holes in both the upstream and downstream rows.

The Proposed Project design will also include a foundation/abutment drainage curtain downstream of the grout curtain cutoff designed to relieve excessive hydrostatic pressure exerted on the dam when a reservoir is present. The drainage curtain will consist of a line of holes drilled downstream of and approximately parallel to the grout curtain. The drainage holes will extend upward through the dam and connect to a drainage gallery, from which water will be collected and discharged downstream into the river channel.

The system of drainage holes will be designed to target areas or features with sufficient hydraulic conductivity to allow water to flow into the drainage gallery. Based on the available geologic data, the areas that the drainage holes will target are generally located within the uppermost 100 feet of bedrock; a preliminary conceptual drainage curtain depth of 100 feet below the current top of rock is included in the conceptual design. The drainage curtain alignment will be parallel to the grout curtain alignment and located a minimum of 10 feet downstream of the downstream row of the grout curtain.

7.4 Aggregate Material Source (Aggregate Quarry Evaluation)

A summary of aggregate material sourcing is included in Section 9 Aggregate Sourcing with more detail provided in the *Conceptual Geotechnical Design Report* (Appendix E). Furthermore, construction materials and quarry operations are discussed in the Cost and Constructability Report (Appendix K).

A range of materials will be required for construction of the dam and related outlet and spillway structures. For the RCC and CVC, materials are primarily aggregate, cement, and supplemental cementitious material (such as fly ash or slag) for the dam and hydraulic structures. Other construction materials will include:

- Roadway fills, base coarse, and surface course (possibly including asphalt in some locations)
- Riprap and riprap bedding for channel lining
- Structural fills
- Some filter/drain materials

Based on limited previous investigations, it is anticipated that aggregate for RCC and CVC can be derived from nearby sources as described in Section 9. These sources should also be capable of providing roadway base and surface coarse aggregates as well as riprap and riprap bedding, structural fills and filter/drain sands, and gravels. Sources will be studied in greater detail during future design phases. Depending on contractor material processing operations and available space, strong consideration will be given to commercially supplying concrete sand and filter products in lieu of setting up site sand washing and potentially classifying operations. At this time, HDR assumes the sand materials will be produced onsite.

8 Structural Analysis and Design

8.1 General

This section provides a summary of the RCC dam structural analysis, and the hydraulic structures design conducted in conjunction with this report. Further details of the structural analysis and design are provided in Appendix F. RCC dam general design details and configuration are provided in Section 10.

The revised alignment is proposed to be curved to maximize the distance from the downstream cultural site and utilizes the best available/optimal geological conditions and terrain at the left and right abutments. For the revised FRE alignment, the cross-section will continue to be analyzed as a gravity concrete dam that is stable in 2D under all the anticipated loading conditions. Curving the dam will provide for arch action development and the ability to refine the dam cross-section. A 3D analysis is required for optimization of the dam's cross-section, but that analysis is deferred to preliminary design.

The following provides a summary of the design criteria for the Proposed FRE Project that were considered in performing the structural analysis of the dam as well as structural design of the fish passage/spillway structures:

- FRE – Design Criteria
  - Reservoir storage of up to elevation 628.0 feet (overflow spillway crest elevation).
  - Dam sized for flood storage with an estimated maximum dam structural height of approximately 240 feet, and a dam crest length of approximately 2,250 feet.
  - Fish passage conduits designed for free passage upstream and downstream prior to and after flood operations, and trap and haul during flood regulation periods
  - A low-level outlet through the dam for flood control operations
  - A central overflow spillway designed to safely pass the PMF without dam overtopping. The spillway would include an ungated Ogee overflow crest structure, stepped spillway chute, and Type II stilling basin.

A conceptual design of the hydraulic structures was completed. The hydraulic structures consist of the conduit structure trashrack and energy dissipation structure, spillway training walls, spillway bridge, and energy dissipation structure.

8.2 RCC Dam Design Load Cases

Design of concrete dams typically involves evaluation of a range of loading conditions including normal operations, flood loadings, and seismic loadings. Because of the primary flood control purpose of the dam, design and operation guidelines for typical flood control reservoirs under the jurisdiction of the USACE were considered for the dam design. Specifically, the load cases used are outlined in USACE EM 1110-2-2200 (1995).

8.3 RCC Dam Stability and Structural Analysis

The dam cross-sections selected for the revised dam alignment were based on the cross-sections developed for the original alignment and assumes an RCC compressive stress of 3,000 pounds per square inch (psi).

The HDR (2017a) conceptual design incorporated a risk-informed design approach as described in Section 3 to arrive at the cross-section of the dam. For the RPD, the same cross-section of the dam was used, but updated and the design criteria checked for the revised alignment. This process included development of a 2D finite element model (response spectrum analysis using SAP2000) to estimate maximum anticipated stress in the dam and at the dam/foundation contact. As the severity of the loading condition was increased (higher recurrence interval events) and the 2D model indicated potential for cracking and nonlinear response, an alternative model (time history analysis using EAGD-SLIDE) was used to estimate the potential for, and magnitude of, sliding along the base of the dam.

To determine the most conservative stress and stability criteria for the FRE structure, stress and stability analyses were evaluated for two representative sections of a potential future taller FRE-FC dam (maximum non-overflow section and maximum overflow section). Because the upstream face slope (0.1H:1V) and downstream face slope (0.85H:1V) would be the same, stresses would be higher and the resulting safety factors would be lower for a potential FRE-FC structure compared to those for the proposed FRE dam. The proposed FRE dam would meet and exceed the stress and stability criteria for a potential FRE-FC dam.

8.3.1 Rigid Body Stability Analysis

The stability of the maximum non-overflow and overflow were evaluated using the conventional gravity method of analysis to estimate stresses and stability factors in accordance with Reclamation guidelines (1976) and USACE EM 1110-2-2200 (1995).

A sliding plane was analyzed along the horizontal concrete/foundation interface at elevation 410 feet for the maximum non-overflow section and elevation 420 feet for the overflow section with a normal pool elevation of 628 feet and PMF pool elevation of 708.9 feet. Sliding stability along the sliding plane is based on the shear friction factor of safety. Reclamation (1976) defines the shear friction factor of safety as the ratio of resisting to driving forces. The sliding factor of safety was estimated for all load cases evaluated and compared to the minimum requirements for each load condition. The gravity analysis conclusion for the normal pool and PMF water elevations is the non-overflow and overflow sections of the dam would remain in compression along the base and the minimum sliding factors of safety be met.

A post-earthquake factor of safety was calculated for full uplift conditions and for friction angles that ranged from 35 to 55 degrees in 10-degree increments. The factor of safety was above 1.5 for all friction angles. The post-earthquake stability analysis performed in support of design development for the dam cross-sections verified that sufficient sliding resistance was available to meet the minimum required post-earthquake sliding stability factor of safety. This estimate was based on some strength degradation along the dam/foundation interface as well as conservative assumptions of uplift pressure along the base of the dam. As shown on the Proposed Project plans (Appendix A), the dam cross-section configurations for both the overflow (spillway) and non-overflow sections of the dam meet the established geotechnical/structural design criteria and would have a low probability of failure following a major earthquake event.

8.3.2 Response Spectrum Analyses

The Finite Element program SAP2000 was used to complete the response spectrum analysis of the dam non-overflow and spillway sections to assess their elastic response to site-specific seismic loading. The response spectrum analysis was completed for multiple recurrence intervals.

Using response spectrum analysis of the dam cross-section is a conservative approach for the verification of cross-sectional requirements as it only identifies the maximum compressive and tensile stresses that occur for the envelope of the seismic loadings. The number of times that the maximum stress condition occurs, and the process of crack propagation is not considered. More rigorous analysis with actual time histories were completed as described in the next section.

Overall, the analyses of the dam showed similar results to the HDR (2017a) analyses of the dam at the original site location.

8.3.3 Two-Dimensional Time History Analysis

EAGD-SLIDE was used to estimate sliding displacements of the non-overflow and spillway sections during seismic loading. The program provides time history response of a 2D model of a concrete gravity dam subjected to the given horizontal and vertical earthquake input motions. It allows sliding evaluation and accounts for foundation rock stiffness, foundation viscosity and radiation damping, effect of water compressibility, reservoir base wave absorption, dynamics of dam structure, and base sliding failure.

The results indicate that the sliding displacements for the range of seismic events evaluated meets project criteria for sliding.

8.4 Hydraulic Structures Design

Hydraulic structures conceptual designs consist of the conduit structure trashrack and energy dissipation structure, spillway training walls, spillway bridge, and stilling basin energy dissipation structure.

The analysis and design of the dam and hydraulic structures is summarized below with a further description of the analysis/design methodology, material properties, and loads and load cases, with the summary of results provided in Appendix D.

8.4.1 Conduit Structure Design

The conduit structure is referred to as the portion of the dam located left of the spillway through which the fish passage conduits, water quality conduits, and flood evacuation conduit pass. Drawings of the conduit structure are provided on sheets 3C-2 through 3C-8 and 3S-14 through 3S-39 in Appendix A. The number and size of conduits is based on design studies completed for the original alignment. A trashrack structure protects the entire upstream face of the dam to prevent large debris from damaging or blocking flow of water through the conduits yet allows fish passage through the dam. The fish passage conduits consist of a central 12-foot-wide by 20-foot-tall conduit and a pair of 10-foot-wide by 16-foot-tall conduits on each side of the central conduit. Water quality intakes and conduits included in the FRE design so as not to preclude potential future expansion are located above the fish passage conduits. Only the embedment requirements of the conduits in the FRE configuration are considered for the FRE conceptual design. A flood evacuation conduit has been included and is located in the adjacent monolith to the right of the fish passage conduits. The fish passage conduits extend through the dam and discharge immediately into an energy dissipation stilling basin.

8.4.1.1 Conduit Structure Analysis

A 3D analysis of the dam's conduit structures was completed to analyze the large and closely spaced openings for the fish passage conduits to ensure the proposed conduit spacing and size is acceptable. The analysis of the conduit structure was performed by developing a 3D linear-elastic finite element model in SAP2000 using a response spectrum analysis. The analysis at this design phase only considers the fish passage conduits with tensile stresses on the upstream face of the dam around the fish passage conduits being the main concern.

As part of the current design, larger gate opening sizes, and thus larger conduit sizes were evaluated. A sensitivity study was conducted by increasing the size of the openings to approximately one central 16 feet by 20 feet and four adjacent and outward 16 by 16 feet conduits. The larger openings resulted in higher tensile stresses ranging from an 18 percent increase for the 500-year seismic event to a 37 percent increase for the 10,000-year seismic event.

As expected, the results show that the tensile stress around the conduit openings both for the proposed conduit openings and larger conduit openings exceed the tensile capacity of the concrete for infrequent seismic events (greater than the 500-year event). Reinforcing steel will be needed around these openings to resist the tensile stresses and minimize crack propagation.

8.4.1.2 Trashrack Structure

A trashrack structure is needed to protect the entire upstream face entrance to the conduit structure from debris that could potentially damage the water quality intakes, evacuation conduit, and fish passage conduits' mechanical components while allowing for safe passage of fish. The trashrack minimum clearance spacing is 24 inches in both the vertical and horizontal direction. The current design includes steel trashracks supported by concrete columns that are laterally supported by concrete struts.

8.4.1.3 Stilling Basin

The fish passage conduits discharge into a 110-foot-long Reclamation Type III stilling basin with top of slab elevation 412.00 feet, training walls on each side to elevation 450.00 feet, and end sill elevation 436.00 feet. The width between the training walls matches the width of the downstream end of the conduit structure. The basin features a single transverse row of reinforced concrete baffle blocks, 6.75 feet tall by 5 feet wide by 8 feet long, and a sloped (2H:1V) end sill for energy dissipation.

8.4.2 Spillway Design

A spillway alternatives study was performed for the RPD, and a stepped spillway chute with a terminal stilling basin was identified as a preferred spillway configuration for the upstream dam location. A flip bucket design for the upstream dam location was concerning because of the potential for direct impacts to a downstream cultural site. The stepped spillway requires a widened spillway overflow section at the dam crest to decrease the unit discharges. However, a relatively small stilling basin at the downstream toe of the dam will be suitable to dissipate the energy compared to a smooth spillway for the anticipated range of flows that could occur.

The FRE spillway has been re-designed from HDR (2017a). The uncontrolled spillway Ogee crest is now 316 feet wide and will be designed to minimize negative pressures at the crest. The RCC will be capped with several feet of the conventional reinforced concrete to form the Ogee crest. The downstream chute will be converging due to the curvature of the dam and will be stepped with 4-foot step heights. The chute walls will be designed and constructed with conventional reinforced concrete.

Flow over the stepped spillway discharges into a Reclamation Type II stilling basin. The stilling basin top of slab is at elevation 430.00 feet, is approximately 266 feet wide (cross-channel) and has a minimum effective length in the upstream-downstream direction of 50 feet. The primary components of the stilling basin will be constructed of reinforced concrete and include an anchored structural slab, training walls, and a 2.5-foot high dentated endsill. Due to the stepped spillway hydraulics, the basin does not include chute blocks at the upstream end.

A spillway bridge is required over the spillway crest to provide access to both abutments of the dam. The current bridge design consists of the four piers (five spans) with prestressed concrete bridge girders.

9 Aggregate Sourcing

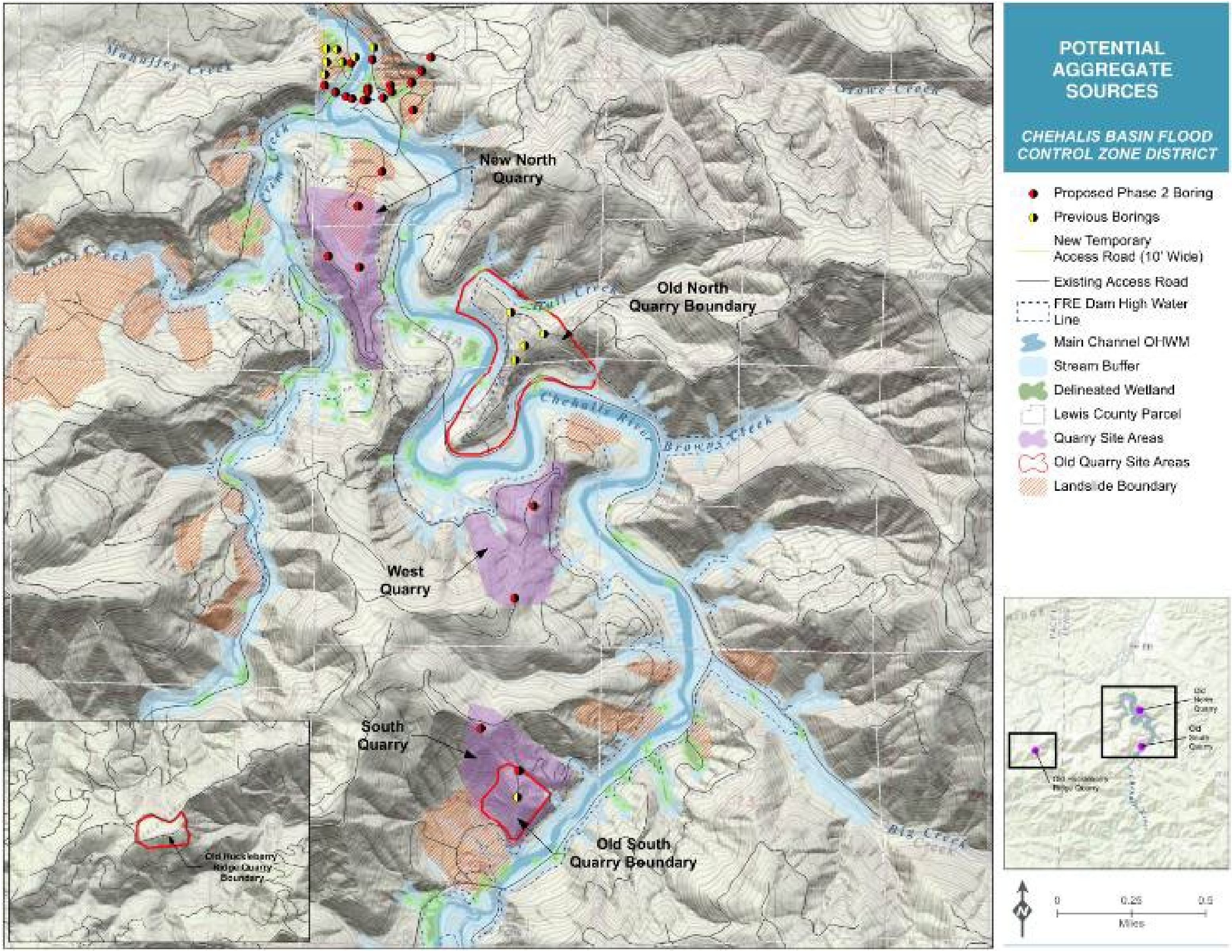
The District has stated that its goal is to develop and utilize a single quarry for all aggregate construction materials at the FRE site vicinity. However, to identify quarry locations with sufficient aggregate to support construction of the Proposed Project, subsurface investigations will be conducted in the future at each of three potential quarry sites with the goal of ultimately identifying one or, if necessary, two quarries for development as part of the Proposed Project.

The three quarry sites originally proposed and analyzed in the DEISs were the North Quarry, the South Quarry, and Huckleberry Ridge Quarry. Huckleberry Ridge was later determined to be less suitable based on haul distance, material quality, and material quantity, and thus dropped from further consideration by the District (HDR 2021b; Appendix E). Similarly, the North Quarry site has since been rejected because of uncertainty related to the quality and quantity of material, the amount of overburden removal that would be required, and the fact that another quarry has been identified closer to the proposed FRE providing a shorter trucking distance. Two new sites – the New North Quarry and the West Quarry – have been identified to replace the North Quarry and Huckleberry Ridge Quarry. Additionally, the South Quarry boundary has been enlarged to provide more area in which to conduct subsurface investigations in order to identify a quarry site within that larger boundary.

Details of how these three locations were identified are discussed in Section 7 of Appendix E.

Each of the three quarry sites have an approximate 65-acre boundary to allow subsurface investigations to be undertaken to identify the most ideal location for a 40-acre quarry within that larger boundary. The revised alignment location has resulted in a larger dam footprint, and therefore RCC and CVC quantity estimates for the Proposed Project have increased by approximately 70 percent. However, based on an assumption of an average 80-foot-thick rock source and 50 percent overburden (non-structural/usable materials), the total area of disturbance to produce required aggregate is expected to be 40 acres if all required materials originate from a single quarry or 80 acres if two quarries are required. The use of three quarry locations is not proposed.

Figure 9-1. Potential Aggregate Sources



Based on the results of future site investigations, laboratory testing, quarry development studies, and construction site layouts, there should be sufficient capacity at one or, if necessary, two of the three proposed quarry sites to construct the Proposed Project. The material sourcing approach supports identifying the minimum amount of quarry site area (a single quarry if possible) required for construction.

Proposed quarry operations are anticipated to cease with completion of the FRE construction and prior to first operation. The proposed quarries and access between the quarries and construction site will require several state, local, and regional permits as well as following Forest Practices rules and standards. Post quarry operations, the Surface Mine Reclamation permit issued by WDNR under the Surface Mine Reclamation Program (SMRP; Surface Mining and Reclamation | WDNR), will require the quarry site be restored to include addressing soil stability and proper water conditions and vegetation.

The *Conceptual Geotechnical Design Report* (Appendix E) contains a summary of the previous quarry evaluations.

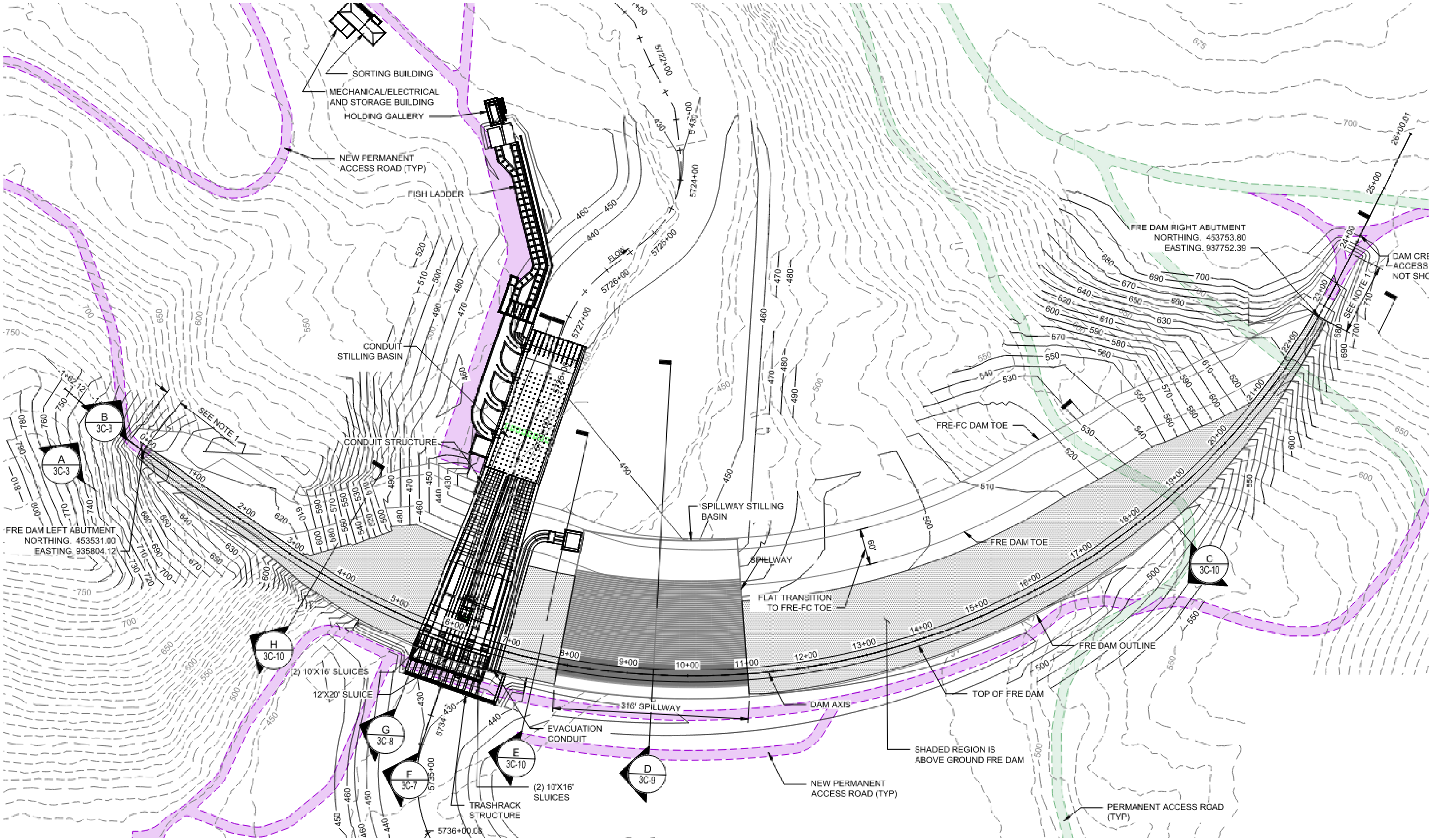
10 RCC Dam Design and Configuration

To meet the design requirements of the Proposed Project as described in Section 2, and based on the results of geotechnical, hydraulic, and structural analyses and evaluations, the FRE configuration was developed and is shown on Figure 10-1 with the complete set of drawings provided in Appendix A.

As can be seen on Figure 10-1, the fish passage/flood control outlet structure would be located at the base of the left side of the dam and approximately align with the existing stream channel. Flows through the structure for the FRE configuration would not require significant modification of the river alignment at either the intake or discharge ends of the structure. The spillway would be located to the right of the fish passage/flood control outlet structure (Figure 10-1). The alignment of the spillway structure provides for a strait channel downstream of the spillway stilling basin discharging to the natural stream alignment above the location of the TCP.

Additional details related to the dam are provided in the following subsections.

Figure 10-1. Plan View of the Configuration of the FRE Dam



Note: Reference sheet 3C-02 in Appendix A

10.1 RCC Dam Profile and Cross-section Properties

Design of the RCC dam requires careful consideration of each required element so that the constructability of the dam is maximized leading to minimized environmental disturbance, lower construction costs, and increased construction quality. A representative view of the dam along the dam axis profile (looking downstream) is shown on Figure 10-2. This detail was taken from sheet 3C-03 provided in Appendix A.

The profile view of the dam shows several important components of the dam's design including the location of the fish passage/flood control outlet structure, spillway crest, key elements of the overall seepage control strategy for the dam and foundation. The foundation and dam drainage systems are integrated in the dam gallery where all seepage is collected, measured, and discharged. Additional details on the elements of the seepage control strategy are shown on the dam cross-sections in Appendix A on sheet 3C-09 and 3C-10 and are described in the following subsections.

Elements of the dam design and construction work for the FRE were developed so as not to preclude future expansion as described in Section 2, including the extents of the foundation excavation objective, surface foundation treatments such as consolidation grouting and shaping blocks and dental concrete treatments, grouting curtain under the FRE outline, and block outs at each end of the drainage gallery (Appendix E).

10.1.1 Facing Systems

The full upstream face of the dam will be constructed with integrated CVC or grout-enriched roller-compacted concrete (GERCC) having an average thickness of 2.5 feet. The upstream facing is not stepped and will be formed with a smooth incline of 0.1H:1V and placed in conjunction with each RCC lift. The conventional concrete facing has increased freeze thaw durability and is commonly considered to have a lower excessive seepage risk compared to a GERCC facing.

The downstream face of the dam outside the spillway will use GERCC with an average thickness influence zone of about 2.5 feet while the spillway chute will use CVC and incorporate reinforcement as necessary to resist hydraulic loads and impacts on step surfaces. The downstream face will be stepped with each step being 4 feet tall with a 3.4-foot tread equating to a downstream face slope of 0.85H:1V. The facing step alignment will be the same within and outside of the spillway for the full length of the dam other than near the spillway crest where the step size decreases to form the uncontrolled Ogee crest configuration. The 4-foot step height was chosen to coordinate visually with the large dam, energy dissipation efficiency for the spillway, and constructability.

10.1.2 Crack Control Provisions

Cracking develops in a mass concrete dam because of thermal stresses associated with heat generated in the hydration of cement, and due to changes in the foundation profile or abrupt geometry changes in the dam that may be associated with hydraulic structures. Limiting cracking within the dam and controlling the location of the cracks is paramount to the successful seepage performance of a concrete dam. Important design provisions for controlling cracks are the spacing and detailing of the vertical control joints separating the dam "monoliths" and installation of other crack inducers located between the vertical control joints. Additional discussion of the crack control strategies incorporated into the new RCC dam are provided below.

10.1.2.1 Dam Monoliths and Control Joints

When the construction reaches a certain height that is above the initial stream diversion, the design anticipates RCC construction to progress across the full width of the dam (from abutment to abutment) and vertically through a continuous process of batching, mixing, placing and compacting the RCC and integrated CVC materials in multiple shifts, 5 to 7 days per week. Control joints will be installed at specified locations on a lift-by-lift basis as the work progresses. Each vertical control joint in the RCC dam as shown on sheet 3C-03 in Appendix A will result in the creation of a monolith. The design anticipates that the vertical control joints will be spaced from 50 to 70 feet on average and may be shorter or longer to coincide with outlet works structures, the spillway location and width, and changes in the foundation excavation profile. Dam control joint (monolith) spacing will be finalized during subsequent design phases and include consideration of results of 2D and 3D thermal analyses and a range of mix design requirements.

Vertical control joints will be formed by 12-inch-high, 16-gage galvanized plates inserted every-other RCC lift for the entire lift width and installed perpendicular and radial to the dam crest centerline. Each control joint will be accompanied by a vertical crack-inducing formed notch at the upstream face followed by a crack-inducing plate leading to a double 12-inch waterstop positioned across the formed joint.

While the curved gravity configuration of the dam limits potential downstream monolith displacement with the development of arch load transfer to the abutments, there is the possibility for upstream movement of cracked monoliths during a seismic event. To help further limit or eliminate the possibility for upstream movement of monoliths that may develop a crack through the section near the base of the chimney section, control joints may be configured to provide a number of 1-foot deep shear keys in the central portion of the dam cross-section. Such shear keys would be created by inserting the galvanized plates in a shear key pattern.

10.1.2.2 Crack Inducers

In addition to the dam control joints, intermediate upstream face crack inducers will be installed on approximately 10-foot spacing between the control joints along the CVC facing elements as a second line of defense related to cracking that may occur. Intermediate upstream CVC crack inducers are intended to control cracking alignment. Cracking associated with the inducers is only expected to propagate through the facing element and only a limited distance into the RCC mass.

10.1.3 Lift Joint Preparation

The near continuous RCC placement process allows several hours each day when fresh compacted RCC awaits being covered by new fresh RCC for compaction. Consequently, keeping completed lift surfaces clean, moist, and free of debris, delaying the initial set of the RCC, and maintaining a moist (saturated surface dry to slightly wetter) surface before subsequent RCC is spread and compacted is critical to achieving a good bond between lifts, creating cohesion, and allowing for tensile stress across joints.

Consequently, the RCC mix design will target maximizing the initial set time through high range and set retarding admixtures. Lift joint maturity and cleanliness will be monitored so that lift surfaces unable to bond (cold joints) between subsequent lift RCC placement will receive appropriate joint preparation that may involve removing debris, laitance, and uncompacted or damaged RCC, and spreading lift bedding or grout to provide a good bond between lifts. The RCC lift joint preparation is crucial to the structural integrity and stability of the dam.

10.1.4 Typical Non-overflow Cross-Section

Figure 10-3 shows the typical configuration of the RCC dam at the maximum cross-section location. This cross-section is a gravity configuration (meaning it is stable in 2 dimensions) that meets all requirements to perform safely under the anticipated range of normal, flood, and earthquake loading conditions at the Chehalis Dam site. As previously noted, the curved configuration of the dam will provide some opportunity to refine the cross-section properties during final design, taking advantage of the arch action that will develop during various loading conditions.: The current gravity configuration represents a maximum construction impact footprint. Optimization of the cross-section will be completed during later design phases.

Figure 10-2. Profile View along FRE Dam Axis

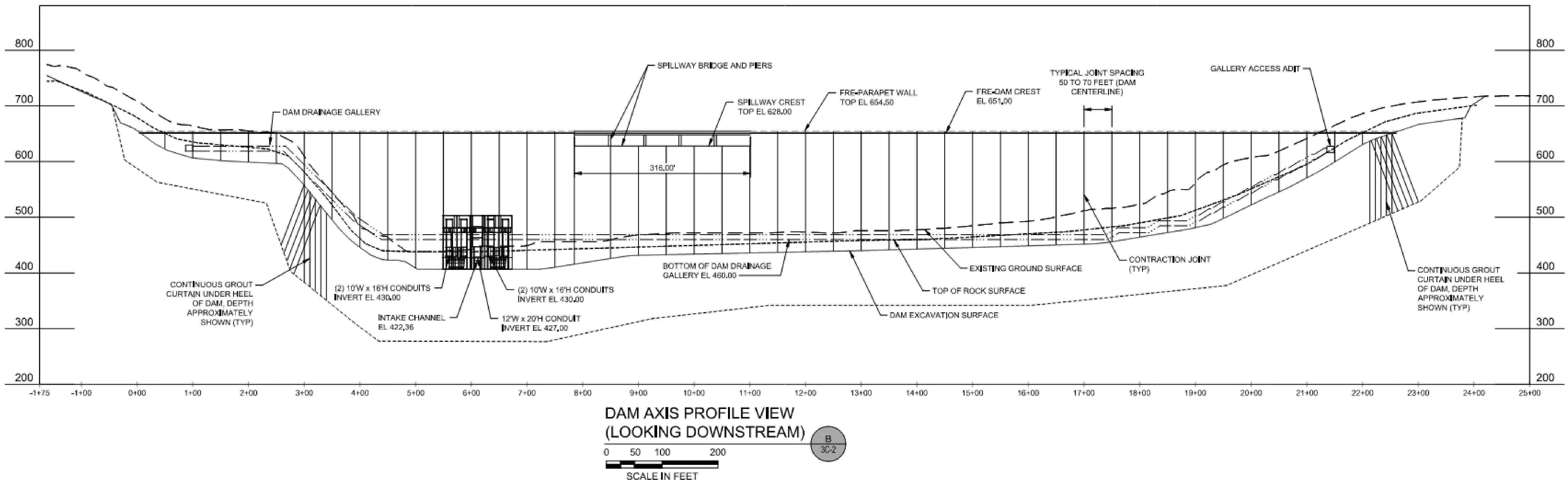
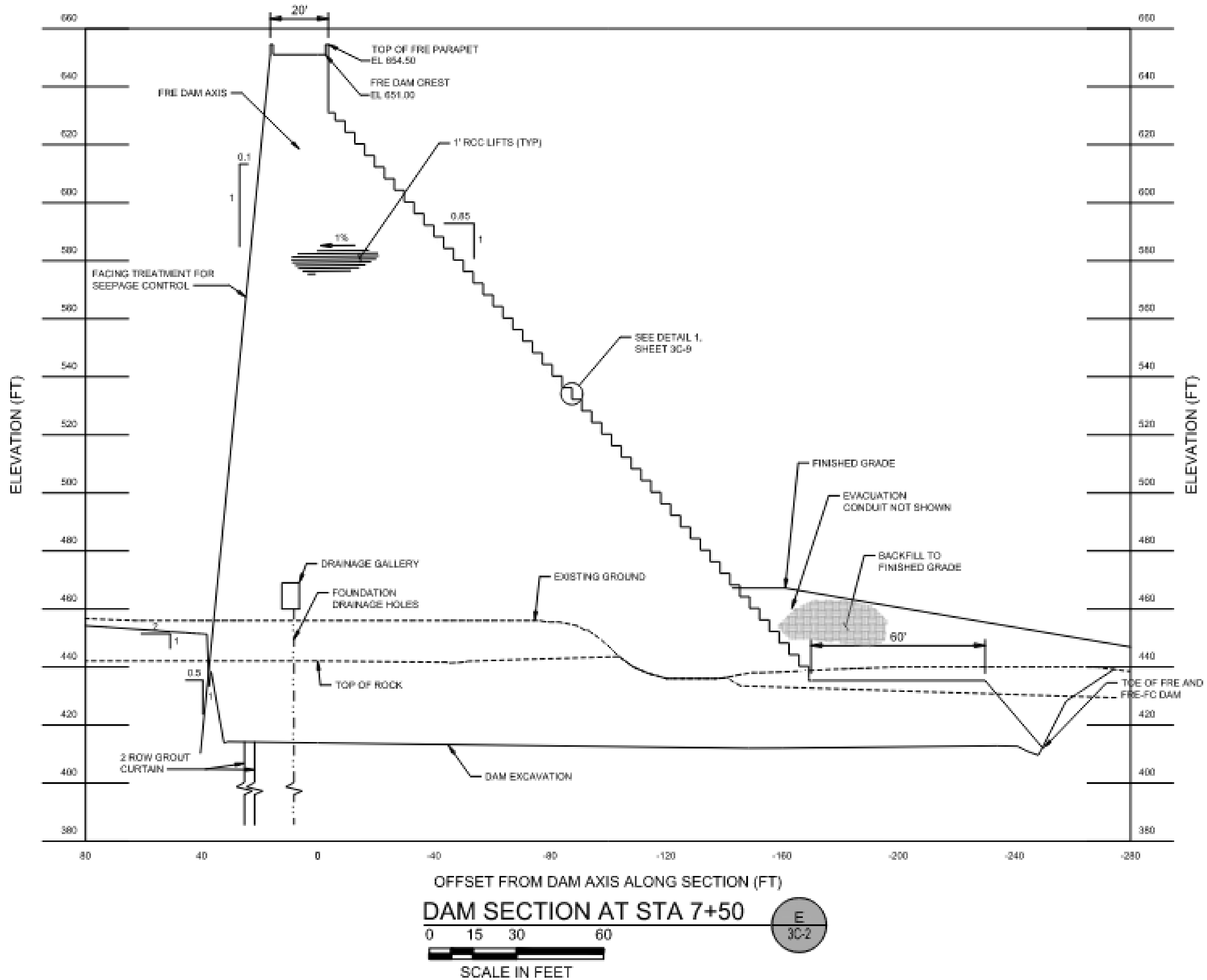


Figure 10-3. Maximum Non-Overflow Section of the FRE Dam

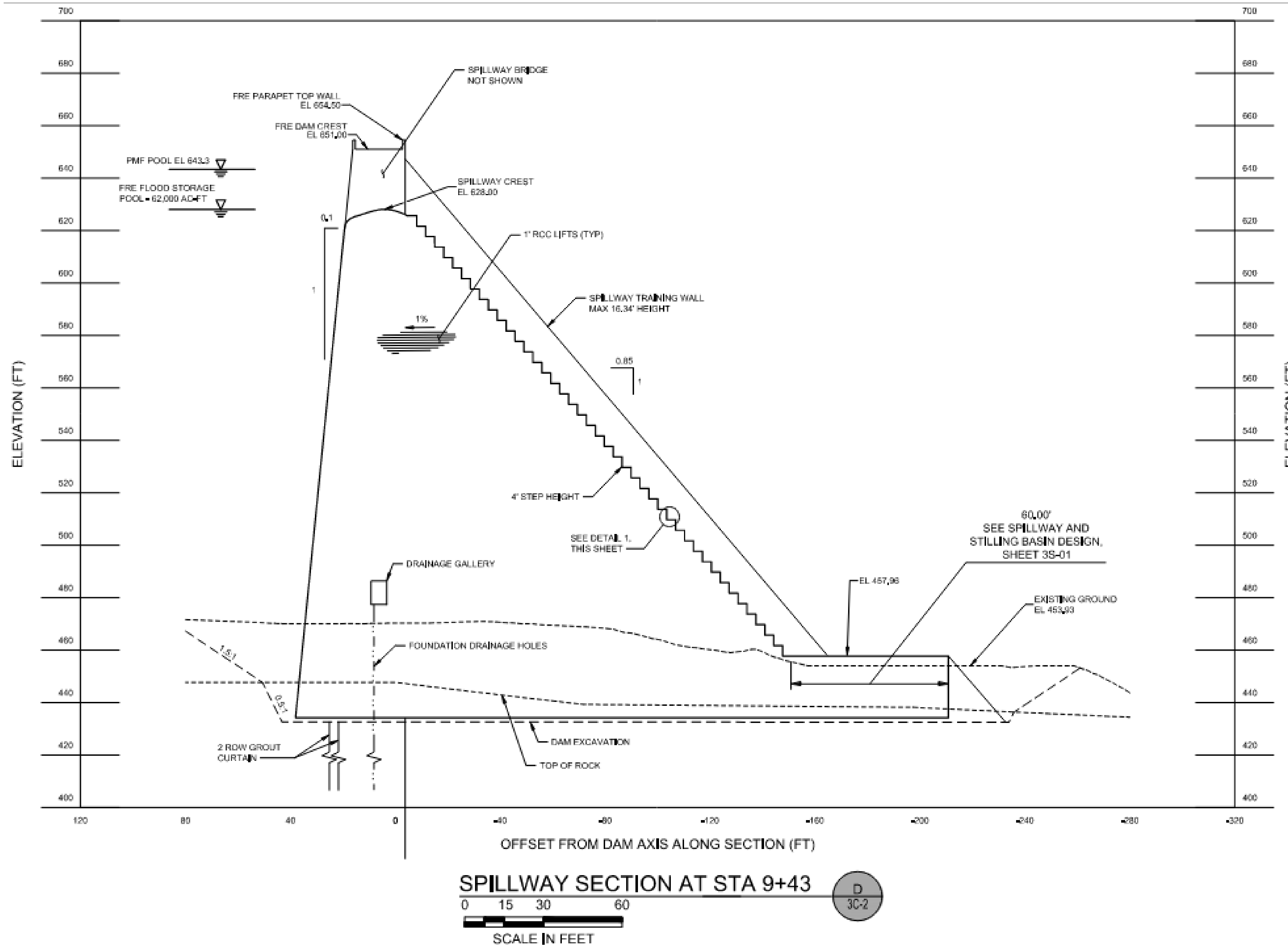


The cross-section will be constructed through the successive placement of 1-foot-thick layers or “lifts” of RCC from upstream to downstream across the section and from abutment to abutment. Upstream and downstream facing systems (described further below) will be constructed integrally with the RCC placement in the dam. The vertical control joints are formed in the monoliths to control cracking in the dam and are constructed integrally with the RCC placement operations. The hydraulic structures will be constructed either prior to, or integrally with the RCC placement in a manner that maximizes constructability and minimizes disruptions to the RCC placement and compaction sequence.

10.1.5 Typical Spillway Overflow Cross-Section

Figure 10-4 shows a typical cross-section of the RCC dam through the spillway. Similar to the non-overflow cross-section, the spillway cross-section has been designed to perform safely as a gravity section (2D) under the anticipated range of normal, flood, and earthquake loading conditions at the Chehalis Dam site. The cross-section along the spillway profile will be constructed through the successive placement of 1-foot-thick layers of RCC from upstream to downstream across the section and from abutment to abutment. The downstream face will be stepped with the steps constructed of reinforced CVC materials to resist hydraulic forces that will develop when the spillway operates. A step height of 4 feet and width of 3.4 feet is included in the current design configuration. Each RCC lift will have a slight slope to the upstream face to allow rainfall to drain off the surface of the RCC lift during construction.

Figure 10-4. Cross Section of the FRE Spillway



10.2 Seepage Control Provisions

The dam design includes a comprehensive and fully integrated strategy for seepage control through the dam and foundation. Regarding the cross-section of the dam, that system includes:

- The dam drainage gallery
- The foundation excavation and treatment system including consolidation grouting in the upper 10 to 20 feet of the foundation
- The foundation grout curtain
- The foundation drainage curtain (downstream of the grout curtain and discharging into the drainage gallery)
- The dams upstream facing system
- A curtain of drain holes in the dam to collect seepage through the dam and discharge that seepage to the drainage gallery
- Control joints and crack inducers installed in the dam and facing systems to control cracking at locations with water stops in the facing that prevent seepage into and through the dam at the crack locations

In addition to the design provisions listed above, the final element of the seepage control provisions in the dam design is the RCC mix design properties, and construction strategies related to bonding of lift surfaces in the dam described above. Well bonded lift surfaces combined with a uniform, densely compacted RCC material through the lift produces a very low permeable dam, eliminating horizontal seepage pathways.

10.2.1 Dam Drainage Gallery

The dam drainage gallery runs parallel to the dam's centerline axis from the left abutment to the right abutment and is 6 feet wide by 9 feet tall. Figure 10-2 shows the gallery along the dam axis profile. A 1-foot-thick concrete slab will be placed in the gallery to create a 1-foot-wide by approximately 1-foot-deep drain gutter along the upstream edge of the gallery. Drain holes extending from the galley and into the dam foundation downstream of the foundation grout curtain discharge to the drain gutter for observation, measurement, and safe discharge to a suitable downstream location.

Several adits will be installed to provide access from the downstream face of the dam to the gallery. These adits run perpendicular to the dam centerline axis. The adits are typically installed near the maximum section of the dam and also at the upper ends of the abutments providing multiple access and egress locations. Placing screened security doors over the adits at different elevations provides an added benefit of creating air circulation currents through the gallery system, minimizing or eliminating the need for supplemental air circulation systems in the dam. An access adit will also be located within the conduit structure. The access adits will have the same dimensions as the gallery.

The gallery will slope down the excavated rock surface in each abutment and into the gallery through the main body of the dam. The dimension of the access adits and gallery are suitable for foundation grouting operations that may be performed during construction or as part of supplemental seepage remediation activities.

The gallery follows the dam axis and intersects the conduit structure at approximate elevation 460 feet. The main dam gallery is elevated above the fish passage conduits. This elevated portion of the gallery provides access to the fish passage sluice gate actuator rooms.

The drain channel within the gallery will be sloped toward the adits to provide drainage to the downstream face of the dam. Seepage will be measured for monitoring at each adit discharge.

10.2.2 Foundation Drainage

Downstream of the foundation grout curtain and following the gallery alignment, a system of foundation drain holes will be installed (Figure 10-2). The combined grout curtain and drain hole system will provide a means for controlling reservoir seepage and limiting the uplift pressures on the base of the dam. The drain holes, drilled from the gallery level, will extend below the drainage trough and discharge foundation seepage that bypasses the grout curtain. These drain holes will be accessible for routine maintenance and monitoring to confirm safe performance of the structure over its design life. Sloping drain holes, similar to the orientation of the grouting holes, are planned to maximize the potential for intercepting foundation seepage and stress relief from the system of fractures in the foundation rock. While not anticipated, oversized drain holes that are properly developed with well screens and sand packs across intervals of highly fractured rock may be required in some locations to provide long-term stability and drain hole performance. When there is no water being stored, there would be little to no seepage anticipated into the gallery.

The foundation drains will be installed on 10- to 20-foot centers as needed based on foundation rock conditions. The drain holes will have an orientation determined by the orientation of fracture in the rock and other rock conditions and likely extend 10 feet below the grout curtain limits.

10.2.3 Dam Drainage

Dam drains will be installed on 10- to 20-foot centers, extended vertically through to the crest of the dam, and capped with removable inspection covers. Each dam drain penetration in the gallery ceiling will require installation of a drain receptor assembly inside the gallery to catch and route seepage to the trench trough in the gallery floor. Dam drains within the spillway limits will terminate at the top of the RCC and not be open to the surface. When there is no water being stored, there would be little to no seepage anticipated into the gallery.

10.3 Chimney Section, Crest Slab and Parapet Walls

The dam crest width will be 20 feet and include a 3.5-foot-tall by 1-foot-wide parapet wall on the upstream and downstream side. The parapet wall will provide additional freeboard to reduce the likelihood of overtopping during a flooding event, serve as a safety barrier for people, and keep vehicles from driving off the dam crest. Vehicle access is planned from both abutments. A spillway vehicle bridge is also included for the RPD. The crest slab and parapet walls will be reinforced and constructed with CVC mix. The crest slab will be placed directly over the final RCC lift, with no anchoring or rebar at the RCC to conventional concrete interface. Contraction joints will be included in both the crest slab and parapet wall. These joints will be aligned with both the dam control joints and upstream crack inducers.

10.4 Integration of Hydraulic Structures (Spillway, Outlets, Fish Passage)

Permanent hydraulic structures for the spillway, fish passage, water quality, and dam safety evacuation provide critical functions for dam safety, environmental and water supply operations and flood routing and flood management. These permanent hydraulic structures must protect the dam structure from damage or loss of integrity arising from uncontrolled or unmanaged water releases. The conduit structure designed for both flood regulation and fish passage through adjacent monoliths will be either constructed with or encased in conventional concrete and should be located and spaced to facilitate construction and make the adjacent RCC placement process as efficient as possible. Structure interference and transitions are important design considerations that should be evaluated on a case-by-case basis.

Effectively integrating permanent hydraulic structures into the structural design while considering constructability and construction sequencing as in this example reflect good RCC dam design practice.

10.5 FRE Structure Instrumentation and Monitoring

An instrumentation monitoring program (including the number, type, location, and expected performance of each instrument) will be developed during subsequent design phases (preliminary and then final design) and installed during construction to monitor key performance parameters for the critical PFMs.

10.5.1 General – Potential Failure Modes

Performance monitoring requirements for concrete dams are based on an evaluation of potential failure modes such as differential movements in the foundation, foundation rock block stability, sliding of the dam along weak or unbonded lift lines, the foundation contact, foundation erosion from spillway, or overtopping flows leading to undermining and instability of the dam. The instrumentation data will be designed to provide early indications should a PFM begin to develop. In addition to collecting data from installed instruments, the instrumentation will rely on visual inspections conducted on a routine basis, and inspections that are tailored to the relevant PFMs at the dam. If instrumentation data or visual inspection observations are outside of the expected behavior, the conditions will be evaluated in more detail. Some of the most common monitoring considered for RCC dams and implemented in the instrumentation design includes the following (Reclamation 1987):

- Uplift pressure monitoring in the drainage gallery (usually at five points in the upstream to downstream direction) in the foundation and at three or more lines based on the length of the dam.
- Drainage gallery flow monitoring with weirs in various locations to isolate flows in each abutment and internal drainage flows within the dam.

- Structural measurement points to monitor potential differential movements in the RCC dam or foundation and foundation rock instability including sliding.
- Internal movement monitoring to identify relative movement using plumblines, inclinometers, fixed survey monuments on crest of dam, single point and multipoint borehole extensometers, strain meters, joint meters or scribe marks across contraction joints in a foundation gallery, and collimation surveys.
- Temperature monitoring of the mass of the RCC dam during construction and generally continuing until the dam reaches a stable temperature.

10.5.2 Leakage and Uplift Pressures

It is important to understand how leakage through an RCC dam and foundation may change with time. If, over a period of time, the flow monitoring in the gallery indicates that flows are decreasing, it may indicate that the foundation drains are plugging and need to be cleaned. Drain plugging can lead to increased uplift pressures. If the drain flows increase, it may indicate a joint opening or cracks in the dam and foundation, possibly resulting in decreasing uplift pressures. The seepage inflows along with uplift pressure readings can be used to identify changes in foundation water pressures and help understand the cause of these changes. Increased uplift pressures can lead to movement of foundation blocks or sliding within the foundation, at the foundation contact or within the dam.

10.5.3 Structural Behavior Monitoring, Instrumentation, and Inspection

Direct evidence of concrete dam foundation instability may be the presence of control joint offsets or cracking that is not associated with temperature variations. Visual inspections, or data from joint meters or measurement points, could be used to detect evidence of movement. Increases or decreases in drain flows, changes in seepage flows, or changes in piezometer or observation well readings could also indicate that the dam foundation is becoming more susceptible to sliding failure. Piezometer data are sometimes needed to assess the stability of the structure if uplift pressures increase above that estimated during design. Collimation, extensometers, inclinometers, or plumbline instruments are sometimes used in large structures to detect structural movements.

A thorough visual inspection of the dam and appurtenant structures is normally required following an earthquake that produces strong shaking (ground acceleration estimated greater than 0.05g at the site). All applicable data, which could include uplift pressure readings, piezometers, observation well readings, drain flow measurements, extensometers, joint meters collimation, and foundation deformation meter readings, should be taken following an earthquake to identify changes.

Specific instrumentation locations and quantity of instruments to be installed will be determined during subsequent design phases when PFMs are further examined.

11 Civil Design and Earthwork

This section is a brief overview of existing and proposed forest roads for use as alternative access routes around the active construction area and quarry access and includes best management practices to reduce impacts of sedimentation from the use of existing roads during construction. Permanent access roads are intended to be used mostly for forest practices, recreation, and access to the facilities. Additional information can be found within the Access Roads & Best Management Practices TM provided in Appendix G.

11.1 General Earthwork

Excavation must be conducted in accordance with the strict requirements established by USACE, OSHA, and Washington OSHA. Shoring may be required at certain locations because of deep excavation and space constraints. Shoring design and trench safety would be the responsibility of the Contractor. Selection of a protective system (shoring) and/or angle of excavated slopes would be determined after considering applicable local, state, and federal safety standards and regulations and the geotechnical recommendation.

11.2 Erosion Control

Erosion control best management practices will be implemented for access road improvements for, during, and after construction activities. The District and Contractor(s) will comply with the National Pollutant Discharge Elimination System permit requirements, Washington Administrative Code (WAC) 173-201A: Water Quality Standards for Surface Waters of the State of Washington, and other federal, state, and local codes and regulations as incorporated into the permit issued for the Proposed Project. Temporary and Permanent erosion and sediment control measures will be implemented to complying with WDOW's Stormwater Management Manual for Western Washington, current Washington State Department of Transportation's Standard Specifications for Road, Bridge, and Municipal Construction and Standard Plans, and Lewis County Standards. Refer to the HDR (2023) for additional information.

11.3 Existing and Permanent Access Roads

Existing access roads were initially reviewed in two separate site visits during summer 2023. It was determined that all existing access roads to be used for construction and permanent access around the inundation pool are gravel surfaced, likely originating from onsite sources. The current road systems within the project area vary from well maintained, used for current logging operations, to needing moderate to significant improvements to be acceptable for use during and after construction activities. For this report, the existing access roads were assumed to be developed using typical State of Washington and Federal forestry road criteria and general requirements.

Permanent access roads, proposed for post-construction, have been developed using the assumed standards under which the existing road systems were developed (Table 11-1), being further developed as the Proposed Project design progresses. Aggregate for permanent road development and improvements to existing road systems is anticipated to come from on-site material sources developed during FRE construction. Permanent access roads will be used post-construction for maintenance around the inundation area including vegetation and debris management, access to logging operations outside of the proposed FRE pool, and recreational activities.

Temporary construction access roads have been developed for the various construction phases and then removed once construction has been completed (Table 11-1). See additional information pertaining to the construction access roads within Section 17 of this document.

Figures depicting the location of existing, temporary, and permanent access roads in the vicinity of the FRE structure and flood storage reservoir are located in Appendix G.

Table 11-1. Summary of Access Road Distances and Volumes

Type	Distance (miles)	Cut (cubic feet)	Fill (cubic feet)	Base Course (cubic feet)	Surface Course (cubic feet)
Permanent Inundated	15	515,000	400,000	105,000	60,000
Permanent NOT Inundated	4	140,000	110,000	30,000	16,000
New Temporary	2	70,000	55,000	14,000	8,000
Abandoned	1	NA	NA	NA	NA

11.4 Existing and Proposed Culverts

Existing roadway culverts within the proposed temporary pool were initially reviewed during two separate site visits in summer 2023 and preliminary conditional assessments of these culverts were performed. The existing culverts were found to be in average to good working condition and providing channeling water from tributaries to the Chehalis River. Additional individual assessments of each existing and proposed culvert location are necessary to verify capacity, fish passage considerations, and end treatment reinforcement options. The following State and Federal standards for design and modification of existing and proposed culverts will be employed:

- Water Crossing Design Guidelines (WDFW 2013).
- Washington State Forest Practice Rules (Title 222 WAC)
- Gravel Roads: Maintenance and Design Manual (USDOT FHWA).
- Forest Service Specifications for Construction of Roads and Bridges (U.S. Department of Agriculture [USDA] U.S. Forest Service 1996).
- Forest Road Practices Board manual - Section 3 Guidelines for Forest Roads (WDNR 2013).
- Forest Road Practices Board Manual (WDNR 2021a).

11.5 Temporary Site Stabilization Approach

The goal of temporary planting requirements is successful establishment of vegetation in newly graded, bare, disturbed or damaged areas for temporary and long-term stability of the soil. The Contractor will be required to implement topsoil reconditioning/import, and revegetation via hydroseeding or drill seeding, with seed mixtures developed and specified for use along embankment slopes and borrow sites away from the river. Separate seed mixes for temporary riparian areas near the proposed temporary river and stream bypasses will be developed along with seed mixes appropriate for application near permanent stream and river alterations. Success criteria will be based on plant density and defined in future project specifications. The Proposed Project vegetation management and mitigation plan discusses additional proposed vegetation management actions.

12 Mechanical Design

The Project's mechanical design includes the fish passage conduits' gate design and the associated upstream isolation gate along with the operating equipment, the water quality ports at key elevations, the evacuation conduit, and the evacuation conduit valve and guard gate.

The Proposed Project will include a Tainter valve (similar to a radial gate) for the 12-foot-wide by 20-foot-tall primary fish passage conduit, four bonneted slide gates as part of the 10-foot-wide by 16-foot-tall secondary fish passage conduits and a separate reservoir evacuation conduit with an energy dissipation valve on the downstream end discharging into the spillway stilling basin), and blind flanged water quality ports.

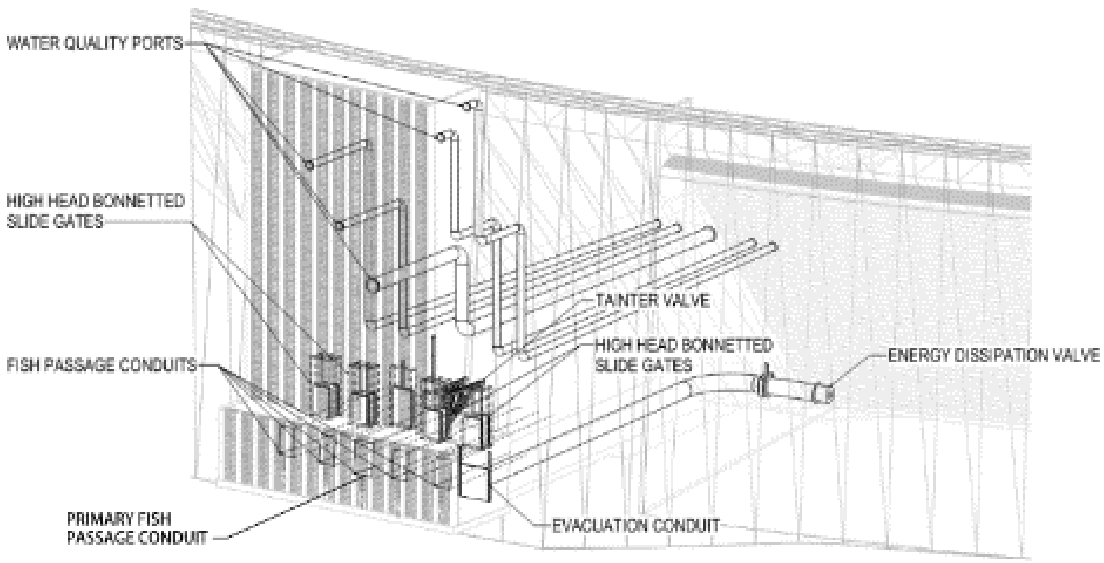
Table 12-1 describes each component, the evaluated size, and the primary purpose of each gate.

Figure 12-1 shows where the gates and conduits are located on an isometric figure of the FRE structure, along with showing the five water quality ports that are included so as to not preclude future expansion (FRE-FC). These ports will be covered and not operational for the proposed FRE.

Table 12-1. Designation of Low Level Outlet Structures for FRE

Gate Type (# of Gates)	Size	Range of Function	FRE (Purpose)
Tainter valve (1)	12'-0" Wide x 20'-0" High	• Max operating head: 100 ft (unseating) • Max static head: 300 ft • Ability to throttle from 10% open to full open within operating head only	Full operation and used for flow regulation below reservoir WSEL 510.
Bonneted Slide Gates (4)	10'-0" Wide x 16'-0" High	• Max operating head: 250 ft (seating condition only) • Max static head: 300 ft • Ability to throttle within operating head only	All four gates are in full operation, including sluicing. Also is used for emergency reservoir evacuation.
Water Quality Ports (5)	7'-0" Diameter (1) 1'-0" Diameter (4)	• Max operating head: varies at each location • Max static head: 300 ft • Seating condition only • Ability to throttle within operating head only	Not in use (blind-flanged).
Flood Evacuation Energy Dissipation Valve (1)	9'-0" Diameter	• Max operating head: 300 ft • Ability to throttle within operating head only	Full operation and used for flow regulation above reservoir WSEL 510.

Figure 12-1. Designation of Low Level Outlet Structures for FRE



12.1 Low-Level Outlet (LLO) Gate Type Recommendations

12.1.1 Gate Recommendation: Tainter Valve

A Tainter valve was selected for the conduit structure’s 12-foot by 20-foot primary fish passage conduit for the following purposes:

- Tainter valves would reduce friction on seating surfaces induced from the operating equipment. Vertical lift gates (roller gates, bulkheads) would require hoist machinery with a capacity to overcome not only the dead load of the gate but also the friction of the gate and the sealing surfaces when hoisted.
- Tainter valves would be advantageous regarding sediment in the water flow. It was stated in previous reports that sediment would be moving through this conduit with the water since it is a river basin. The use of a Tainter valve would allow for sluicing sediment at a river level and the natural curvature of the gate helps direct flow and thus sediment and debris past the gate. Utilizing a Tainter valve would also eliminate the need for gate slots. The gate slots would be problematic since sediment can accumulate within and cause issues with gate operation, as well as an ongoing maintenance item.
- Tainter valves would also allow for more reliable control to allow for different water flow regulations during flooding conditions. The geometry of the Tainter valve lip also benefits the hydrodynamic down pull when the gate is in operation. Most vertical gates must be specially designed at the bottom of the gate to reduce operational problems, but the Tainter valve has the geometry already built into its design.
- Implementation of a Tainter valve will result in leakage given the hydraulic head differentials at or exceeding 100 feet. Leakage is expected and allowable but limited to a maximum rate. The driving factor for limiting leakage is to prevent false attraction flow for fish into the fish conduits. The gates would only see the high hydraulic head differentials during flood

conditions. The anticipated leakage rates are within the allowable leakage rates and do not adversely affect the system. Prolonged periods of leakage and frequent occurrence can shorten the service life of the seal. Seal inspections should be performed after every flood event.

12.1.2 Gate Recommendation: Bonneted Slide Gates

A bonneted slide gate was selected for the four 10-foot by 16-foot additional fish passage conduits for the following reasons:

- A cast-in-place bonneted slide gate will allow for the vertical gate to have a sealing mechanism with a steel-on-steel connection, rather than a rubber gasket or other seal configuration. At the high-head applications required for the FRE and optional future FRE-FC conditions, traditional gate sealing for other types of vertical gates would be insufficient or ineffective.
- A bonneted slide gate would eliminate the need for gate slots within the concrete conduit structure, thereby prohibiting sedimentation buildup that would render gate-closing operations difficult. Mechanisms can be installed to introduce air to “blow out” trapped sediment in the bonnet before the gate is closed.
- A built-in piston, common to bonneted gate designs and used for gate operations, removes the need for full-height maintenance or operation shafts to be formed through the entire RCC dam geometry. A large vertical shaft through an RCC cross-section creates formwork, constructability, and seepage issues. The pistons are present and work within the bonnet, allowing for ease of installation in place. A dry chamber above the gate will still be needed for gate access and maintenance.
- Disadvantages associated with bonneted slide gates regard the maintenance that would have to be completed to provide sustained functionality. The bonneted slide gates may make inspection difficult. There would have to be an isolation bulkhead upstream, as well as a confined space entry or equivalent. There will be a dry chamber above the gates to ensure maintenance and access can be completed. All the throttling bonneted slide gates have an upstream isolation bonneted slide gate in place to provide personnel access if needed.

The secondary (10 feet by 16 feet) fish passage conduits with the high head bonneted slide gates are intended to be used for emergency releases at pools greater than elevation 530 feet. While high pool operation is possible, closing these gates at or below pool elevation 530 feet is recommended to protect the gates.

The evacuation conduit is a 9-foot diameter, concrete-encased, steel-lined pipe used to provide reservoir evacuation post-flood event or for emergency drawdown at pool elevation of 431 feet and above. The evacuation conduit is made up of a bell mouth intake, a trashrack, a bulkhead slot, a steel-lined and concrete-encased pipe, an isolation valve with a vent, a hooded energy dissipation valve and a valve house structure adjacent to the spillway training walls.

As part of the Tainter valve evaluation, several iterations of width were performed and became more viable with lower hydraulic operating head requirements post-workshop. The operating head assumptions as well as the width sensitivity and basis for current design are presented in Hydro

Mechanical TM located in Appendix H. The Tainter valve is designed to be operational only for the FRE condition but capable of withstanding hydraulic head (no-operation) up to the FRE-FC operational conditions. Results of the structural analysis showed that a 14- or 16-foot-wide gate would be a reasonable starting point for preliminary design. For larger gate widths, additional custom design fabrication uncommon in the United States and/or built-up steel member sections will be required unless the Tainter valve is to be decommissioned and bulkheaded off if an FRE-FC were constructed.

13 Electrical, Instrumentation, and Control Design

This section describes the general electrical, electrical distribution, communications, instrumentation and controls required for the proposed FRE.

13.1 Electrical Service

A new medium voltage electrical service is required for the Proposed Project to connect to the existing public electrical utility lines serving the Weyerhaeuser facility downstream of the proposed dam. A new electrical and telecommunications service duct bank will be buried along the service road and use the existing (or replacement) bridge to route the new services across the river to the project site. Coordination with the utility must confirm there is existing capacity at the Weyerhaeuser location.

A new 500kVA transformer will step down the medium voltage to 480V, three phase power for distribution around the site. The transformer will be next to the new FFPF mechanical/electrical building shown on drawing 4C-01 in Appendix A.

A 500kVA diesel generator will be installed for standby facility power. The generator will be exterior mounted in a weather enclosure, located adjacent to the mechanical/electrical building. Drawings show a portable generator, but actual determination of a portable or on-site generator will be determined during design development. The generator will require monthly inspection and operational testing. The generator will be connected to the distribution system via a manual transfer switch. Automatic transfer to the generator upon loss of utility power will be determined during future design development.

Refer to Section 16.4 - Distribution Lines for Construction Power for requirements for electrical power during construction and FRE Site Temporary and Permanent Power TM in Appendix N.

13.2 Power Distribution

The main switchboard will be in a new mechanical/electrical building and provide power to the FRE gates and valves, as well as a distribution panel to support the ancillary loads for gate and valve equipment, including general convenience power, lighting, and HVAC.

There will be a feed from the main switchboard to the distribution panel in the fish passage facility, designed to support the electrical loads required to operate the facility.

13.3 Telecommunications

Fiber will be routed to the site alongside the service power feed in the new utility duct bank. Fiber will terminate at the main control panel in the mechanical/electrical building and provide remote monitoring of the site. There is currently no plan to provide off-site remote facility operation or a separate hard line.

13.4 Instrumentation and Control

A Supervisory Control and Data Acquisition (SCADA) system will be installed on the site to control and/or monitor all aspects of the FRE, FFPF, intrusion detection, manual transfer switch, generator, public address, and gate monitoring. The SCADA system will be a programmable logic controller platform with a human machine interface in the mechanical/electrical building. All system statuses, historical data, and alarms will be available at the interface in addition to operator controls for site systems.

The SCADA system network will be a stand-alone system comprised of Ethernet switches, patch panels, and dedicated network ports. The Proposed Project will be remotely monitored. Provisions for remote monitoring are included in the design.

The FRE has nine gates and one cone valve controlled by the SCADA system that will operate automatically as previously described. Each gate has a local control panel allowing the operator to open, close, or stop the gate as desired. When the local control panel is placed in the REMOTE position, the SCADA system will have supervisory control. Signals received from the actuators are:

- Gate/Valve in Remote
- Open & Close Command
- Lockout Stop
- Control Power Available
- Actuator Thermal Overload
- Gate/Valve Fully Closed
- Gate/Valve Fully Open
- Gate/Valve Motion Delay (for audible alarm)
- Actuator Fault
- Actuator Overtorque
- Gate/Valve Position (Percent Open/Closed)

Other FRE instrumentation will include redundant reservoir level indicators, flow meters, and public address system calibrated to announce gate changes that may increase downstream flows.

Vandal resistant cameras will be placed at locations around the FRE to observe operations and the presence of personnel throughout the facility. Due to possible lighting restrictions, cameras will be equipped with night vision technology. Specialty cameras may be used for vehicle and license plate identification to log those that access the site. All video data will be stored on site with an uplink to a remote site.

The mechanical/electrical building contains distribution equipment for the site and will be monitored for temperatures, intrusion, and electrical characteristics to be defined in future design development. In addition, generator and manual transfer switch statuses will be monitored.

The flood fish passage facility will be controlled by the SCADA system. Motors, valves, pumps, conveyors, level indicators, flow meters, and ancillary equipment will be monitored and controlled either locally by operators at the equipment, or automatically by standard programmable logic controller algorithms programmed to coordinate the trap and haul operations.

14 Operations and Maintenance

This section summarizes operation and maintenance (O&M) considerations for the Proposed Project, which are documented in greater detail in Appendix J.

14.1 Facility Operational Strategy

Operation of the facility, including the FRE and FFPF, will occur in the following operating stages:

- **Facility startup** is defined as the transitional period prior to the start of the normal operating period where pre-season readiness and maintenance activities should be completed to prepare the facility.
- **Normal Operation** is defined as the main operational states expected during the annual normal operating period when floods historically occur. The two main operational states are:
    - Run-of-River Operation describes when the fish passage conduit gates are open, and the Chehalis River flows through the FRE unimpeded. This differs from the Non-Operational Period below in that the facility is actively monitoring forecasts and is fully prepared to initiate flood retention operations.
    - Flood Retention Operation describes when the fish passage conduit gates are closed to impound incoming floodwaters behind the FRE (refer to Section 14.2).
- **Facility Shut-Down** is defined as the transitional period following the end of the normal operating period where inspection, maintenance, storage, and documentation prepare the facility for inactivity.
- **Non-Operational Period** is defined as the period where the facility is inactive, on-site staff are reduced, forecasting is suspended, maintenance occurs, and the adaptive management process takes place. This period takes place annually during the dry season (a few months) where floods triggering operation historically do not occur. The Chehalis River flows through the FRE unimpeded during this period, however, certain fish passage conduits gates could be closed as flow varies to manage velocity and facilitate fish passage.

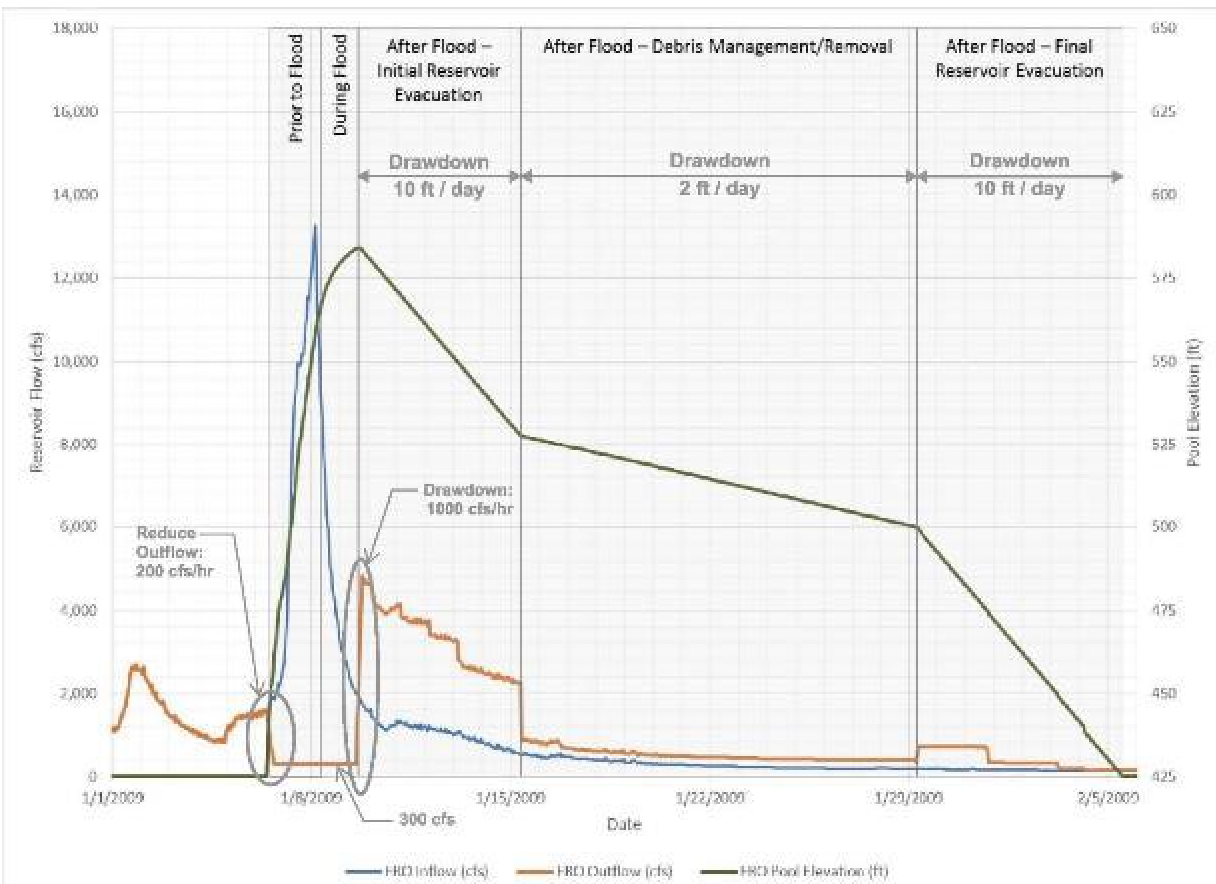
14.2 Flood Retention Operation

This section describes how the FRE will be operated for flood retention including the proposed gate closure and opening sequences.

14.2.1 Operating Rules

The operating rules for water retention and reservoir evacuation were developed by Anchor QEA to inform the Chehalis Basin Strategy Programmatic EIS and inform the flood retention structure alternative analysis (Anchor QEA 2017). These operating rules are incorporated into the design documented in this report and remain unchanged. As described in Anchor QEA (2017), impoundment events are triggered when the flow rate in the Chehalis River at Grand Mound, Washington gage 12027500, is forecasted to be 38,800 cfs or higher. Based on the hydrologic record from 1988 to 2016, the operational model indicates that these events are statistically equivalent to a 7-year recurrence interval (15 percent chance of occurrence in any year). Under future climate change conditions, it is estimated that these impoundment events would occur more frequently. An example of the discharge operating rules is provided in Figure 14-1. When flood retention operation is triggered, flow passing downstream through the FRE would be reduced per the gate operation procedure outlined in Section 14.2.2 below, causing the WSEL upstream of the FRE to rise. Upon reaching the maximum WSEL required for the retention event reservoir evacuation would begin and flow passing through the FRE would be increased per the gate operation procedure outlined in Section 14.2.2, causing the WSEL upstream of the FRE to fall. For a period during reservoir evacuation, the drawdown rate would be decreased to facilitate debris management as described in Section 14.3.3.

Figure 14-1. FRO Operation Modeling - January 2009 Flood



Source: Fig. 2.8 of Anchor QEA (2017) annotated

14.2.2 Gate Operation during Flood Retention and Evacuation

When flood retention operation is triggered as described in Section 14.2.1, flow through the FRE is controlled by the fish passage conduit gates and/or the evacuation conduit valve, depending on the WSEL upstream of the FRE. Flow control by these gates during retention and evacuation follows the ramping rates and flow rates defined in Section 14.2.1. Gate operation during flood retention and evacuation are summarized in the following steps:

Step 1: Flow projections at the Grand Mound gage trigger flood retention operations at the FRE structure. Begin simultaneously closing the secondary conduits and the primary fish passage conduit gate to a gate opening of approximately 50 percent.

Step 2: Close the secondary fish passage conduit gates.

Step 3: Begin closing the primary fish passage gate. Control river flow following operating rules until the reservoir reaches WSEL 510.

Step 4: At pool elevation 510 feet begin closing the primary fish passage conduit gate and begin opening the evacuation conduit valve while maintaining flow downstream in accordance with the operating rules.

Step 5: Flow control transition is complete. The primary fish passage conduit gate is fully closed. Flow releases downstream are controlled by the evacuation conduit valve and, where the FFPF auxiliary water system is supplied by gravity, by the FFPF auxiliary water facility, following the operating rules.

Step 6: Reservoir reaches the maximum WSEL required for the retention event. Flow releases downstream continue to be controlled by the evacuation conduit valve and, where the FFPF auxiliary water system is supplied by gravity, by the FFPF auxiliary water facility. Reservoir evacuation continues to follow the operating rules.

Step 7: Reservoir evacuation begins and follows the operating rules. At pool elevation 510 feet begin closing the evacuation conduit valve and begin opening the primary fish passage conduit gate while maintaining flow downstream in accordance with the operating rules. If in use, begin closing the FFPF auxiliary water supply as well.

Step 8: Flow control transition is complete. The evacuation conduit valve and FFPF auxiliary water supply are closed. Flow releases downstream are controlled by the primary fish passage conduit gate following the operating rules.

Step 9: Reservoir evacuation is complete. Fully open the primary fish passage conduit gate. Fully open the secondary fish passage conduit gates.

Step 10: Fish passage gates are fully open. Chehalis river returns to flow-through run-of-river through the fish passage conduits.

14.2.3 Potential for Operational Refinement

An operational sensitivity analysis was performed as described in Section 14.5 and Appendix B. This work demonstrated that there is potential to minimize environmental impact through operational refinements. Investigation into forecast information should consider, for example, contributions of local flows or stage reductions, before operation of the FRE may allow for improved performance and potential minimization of activation when flood reductions are not deemed critical. Operating rules for such events would incorporate changes to flow release rates and release durations and reductions in water storage volume and duration. These changes could result in shorter inundation periods, smaller reservoir footprint, less deposition of fine sediment over redds, shorter delays in downstream fish passage, shorter durations of non-volitional fish passage via the FFPF, and better growth rates and survivability for river-temperature reducing shade trees, minimizing environmental impacting. Potential refinement of flood retention operations will be studied in more detail in preliminary design.

14.3 FRE Facilities Operations and Maintenance

This section summarizes the operation and maintenance (O&M) considerations for the FRE and appurtenant facilities. These considerations are intended to provide the foundation for the O&M manual that will be developed in future design phases and will be advanced as the design progresses. For greater detail, refer to the Operations and Maintenance Considerations TM in Appendix J.

14.3.1 Authorization/Notification

Several personnel teams will be required for operation of the FRE for day-to-day operations, routine maintenance, record keeping, and flood retention operations. The number of O&M personnel may vary seasonally as there may be long periods (i.e., years) when the flood retention operation does not occur. However, many personnel would likely be required exclusively for flood retention operations and would be on call during the normal potential operating period (i.e., wet season) when flood retention could occur. As the design progresses, coordination with state and federal agencies will determine specific requirements for the number, qualifications/trainings, and degree of readiness of O&M personnel required for operation of the FRE.

As previously noted, forecasting at the USGS Grand Mound gage on the Chehalis River will trigger flood retention operations at the FRE. A project-specific forecasting and monitoring system should be developed to provide ample warning to regulators, personnel, and the public prior to initiating flood retention operations. A defined notification structure should be developed to communicate operational changes to downstream communities as they occur. Furthermore, a plan should be developed to address human safety within the reservoir footprint prior to operation.

14.3.2 Adaptive Management Strategy

The proposed facility is complex and unique and may require a steep learning curve as the various systems, maintenance requirements, and interaction with the natural operating environment become apparent. An adaptively managed operational strategy will be used to improve facility performance primarily over the first several years/instances of operation but will continue in some capacity as part of normal facility operations. Lessons learned from the prior operating season are valuable only if they are recorded, discussed, and incorporated into the following season’s operating strategy. The adaptive management strategy will be developed in cooperation with all parties involved. As impacts from climate change are realized, the adaptive management strategy is a critical tool to provide flexibility and resilience to the Proposed Project. Climate change will likely impact specific operating criteria for the Proposed Project, such as the normal operating period, frequency of operation, and statistical likelihood of different flow events.

The key phases of an adaptive management strategy include observing facility operations during the operating season, annually evaluating performance against predefined metrics, and developing and implementing refinements to operations prior to the next operating season.

14.3.3 Debris Management and Disposal

Debris management is critical to successful operation of the facility. During a flood, significant debris is expected to accumulate in the reservoir, specifically at the FRE trashrack. Debris will be removed prior to resuming run-of-river operations. Additionally, large woody debris smaller than 24 inches in diameter may be transported by natural processes past the trashrack and into the conduits and stilling basin during run-of-river operations. Debris removal will occur during reservoir drawdown, where the rate of drawdown will be slowed for up to 14 days to allow maintenance personnel to collect and remove debris from the trashrack and temporary reservoir. For further discussion see HDR (2021a).

14.3.4 Scheduled Maintenance Procedures

Regularly scheduled inspection and maintenance are integral to reliable facility performance and longevity. Inspection and maintenance intervals could range from daily to annually, depending on many factors such as likelihood of failure, severity of failure, ease of inspection, and manufacturer recommendations. Specific maintenance intervals will be developed for all major facility components and documented in the O&M Manual, with input from regulators, as the design progresses. Generally, additional inspection and maintenance should be performed following flood retention operations. It should also be noted that some regular maintenance activities such as maintenance/repair of the fish passage conduits, fish passage stilling basin, and constructed approach and discharge channels could require in-water work and would be scheduled during typical in-water work windows.

Inspection and preventative maintenance will be considered for the following facilities and may be expanded during preliminary and final design:

- Major concrete structures
- Access roads
- Electrical systems and mechanical equipment
- Stabilized landslides and landslides identified for monitoring within the basin

14.3.5 Emergency Response

A comprehensive emergency action plan (EAP) is required by WAC 173-175-520 and will be developed during preliminary and final design so that personnel are prepared to act in the event of incidents, failures, or damage/malfunctions that would endanger life or property. The EAP will address duties such as detecting and evaluating emergencies, establishing a chain of command, contacting appropriate first responders, taking preventative action, and otherwise managing the emergency in the quickest and safest means possible. Types of emergencies at the FRE could include the following:

- Loss of access
- Loss of communications
- Loss of power

- Extreme rain/flood event
- Major seismic event
- Fire in the watershed

14.3.6 Facility-Specific Health, Safety, and Environment

The FRE is a large water resources infrastructure facility, and safety considerations for personal protective equipment, confined spaces, fall protection, remote and automatic operations, and wet weather access to the site should be developed in accordance with industry standards and regulatory requirements.

Additionally, wildlife protection considerations will be investigated as the design progresses to minimize impacts to wildlife from operation and maintenance of the Proposed Project.

14.4 Flood Fish Passage Facility Operations Summary

As a component of the FRE facility, many operation and maintenance considerations for the FFPF are shared with the FRE. For example, development of an adaptive management strategy and scheduled maintenance of concrete structures and mechanical and electrical equipment will apply to the FFPF.

The FFPF will operate continuously, 24 hours per day, during FRE flood retention operations to maintain upstream fish passage. Operation will begin before flood retention operations are initiated and end after run-of-river conditions are resumed. It is expected that many of the personnel required during flood retention operations will be dedicated to the FFPF. Coordination with state and federal agencies during future phases of design will determine specific requirements for FFPF personnel, such as what qualifications/trainings are required and how often they must be renewed.

For greater detail, refer to the Operations and Maintenance Considerations TM in Appendix J.

14.5 Reservoir Operational Sensitivity Analysis

The work documented in this section describes an operational sensitivity analysis to test specific changes to the operating rules to examine the potential to minimize impact to the environment while still reducing flood damage risk. This sensitivity analysis is based on the hydrology presented in WEST (2014), WSE (2017), and the operating rules in Anchor QEA (2017). The operating rules for water retention and reservoir evacuation were developed to inform the Chehalis Basin Strategy Programmatic DEIS and inform the flood retention structure alternative analysis (Anchor QEA 2017). Understanding the relationship of operational triggers to potential reductions of downstream flows and WSELs is the first step in the analysis process for potential revisions to the operations plan that will better balance the competing needs of flood risk reduction and minimization of negative environmental impacts. Detailed analysis of, and revisions to, the FRE operations plan will be completed during the next project phase. A detailed technical memorandum for the current analysis is presented in Appendix B.

The operational trigger is defined as a flow forecast to exceed 38,800 cubic feet per second (cfs) at USGS Gage 12027500, Chehalis River Near Grand Mound, WA (Grand Mound Gage). Upon notification of this forecast flow, water retention would begin within 48-hours of the forecasted flood peak. The National Weather Service Northwest River Forecast Center (NWRFC) issues forecasts twice daily and would issue forecasts more frequently during these forecasted events.

The flow of 38,800 cfs at the Grand Mound Gage is defined as a major flood by the National Weather Service (NWS) and corresponds to documented flood impacts to property and infrastructure. NWS (2024) states that at this forecast stage:

> *"The Chehalis River in Thurston County will cause major flooding, inundating roads and farm lands in Independence Valley. Deep and swift flood waters will cover SR-12 and James, Independence and Moon Roads. Flooding will occur all along the river including headwaters, tributaries, and other streams within and near the Chehalis River Basin."*

The intent of the forecasted 38,800 cfs trigger at the Grand Mound Gage is to notify and initiate operational actions in sufficient time to capture the peak of a large flood event originating in the Willapa Hills (upstream and tributary to the Proposed Project) and reduce river stage and flows downstream during large, infrequent storm events to meet flood reduction goals as described in the purpose and need and flood reduction objectives.

However, the 38,800 cfs trigger was not intended to be taken in isolation. As a stated part of the operations described in Anchor QEA (2017), operational flexibility and adaptive management should be incorporated as part of the Proposed Project's Operation. An example of operational flexibility is a situation where a forecast that triggers operations 48 hours in advance does not develop into a flood of 38,800 cfs or greater at the Grand Mound Gage. Subsequent forecast updates at the Ground Mound Gage may indicate that FRE operations could be adjusted or cease. Existing hydrologic data from recent storm events suggest that for many storm events that meet the 38,800 cfs forecast threshold, it will be possible to reduce the FRE's operational duration or allow for an increase in the minimum discharge flowrate of 300 cfs described in the Anchor QEA (2017) report in Appendix M.

14.5.1 Operational Sensitivity Analysis Approach

Eight sensitivity scenarios were selected to consider a range of potential changes to the Anchor QEA (2017) operations that could lead to improved operational outcomes. The 300 cfs maximum outflow during operations was tested at higher values. Forecast triggers at the Grand Mound Gage and the Doty Gage were considered. Finally, the drawdown rates for emptying the pool were varied as well. Each of these scenarios was applied to three different storm/flood event patters using historic data in the basin. The February 1996, December 2007, and January 2022 events were examined using the Anchor QEA (2017) operational rules, and then tested with the eight sensitivity scenarios. These scenarios are summarized in Table 14-1. This work is further described in Appendix B of this report.

Table 14-1. Sensitivity Analysis Scenarios Tested for the February 1996, December 2007, and December 2022 Chehalis Basin flood events

Category	Scenario
Maximum Releases during Operations	Maximum FRE Release = 1,000 cfs Maximum FRE Release = 2,000 cfs Maximum FRE Release = 3,000 cfs
Modified Forecast Triggers	Grand Mound = 52,500 cfs Grand Mound = 52,500 cfs + Doty = 25,200 cfs Grand Mound = 38,800 cfs + Doty = 25,200 cfs
Modified Drawdown Rates	Drawdown Rate +20% Drawdown Rate - 20%

It is important to note that these sensitivity scenarios were chosen to test the effects of certain changes to the rules to see which might have the biggest impacts on the timing and volume of the temporary pool and downstream flood risk reduction effects. The modified parameters were not combined, and no attempt was made to improve any of them. Instead, this analysis was meant to demonstrate the technical feasibility of each scenario by testing a range of changes to key operational variables. These scenarios will be incorporated and further refined as the design of the Proposed Project continues with the specific intent of combining and optimizing these operational changes to minimize or avoid impacts due to the operation of the Project.

14.5.2 Modification of Minimum Outflow Release during Operations

HDR examined the differences in FRE performance while varying the minimum outflow release during operations. The proposed 2017 operations plan specifies that minimum outflow release through the FRE is limited to 300 cfs when the forecasted flow at Grand Mound, WA exceeds 38,800 cfs in the next 48-hours. HDR investigated three different maximum outflow peak release rates:

- Maximum FRE release = 1,000 cfs
- Maximum FRE release = 2,000 cfs
- Maximum FRE release = 3,000 cfs

HDR found that as the outflow release increases, the potential for reduction in downstream flows and WSEL decreases. However, the increase in downstream peak flows were less than 5 percent and increases in downstream maximum WSEL at Grand Mound, WA were less than 0.5 ft. Table 14-2 through Table 14-4 summarize these findings by event and corresponding results are shown in Figure 14-2 through Figure 14-4.

Table 14-2. Summary of Maximum Release Sensitivity Analysis Results: February 1996 Event

Scenario	Maximum by Locations						Difference from Unregulated		
	FRE		Doty, WA		Grand Mound, WA				
	Reservoir Elevation (ft)	Reservoir Release (cfs)	Flow (cfs)	WSE (ft)	Flow (cfs)	WSE (ft)	FRE Flow (%)	Doty WSE (ft)	Grand Mound WSE (ft)
Unregulated conditions	N/A	22,365	27,966	324.9	74,800	147.0	N/A	N/A	N/A
2017 operations plan = 300 cfs	610.3	7,394	10,808	316.9	71,035	146.8	-67	-8.0	-0.3
Maximum FRE release = 1,000 cfs	605.2	7,394	11,507	317.3	71,714	146.8	-67	-7.6	-0.2
Maximum FRE release = 2,000 cfs	597.4	7,125	12,507	317.9	72,725	146.8	-68	-7.0	-0.2
Maximum FRE release = 3,000 cfs	589.1	6,852	13,508	318.4	73,726	146.9	-69	-6.5	-0.1

Table 14-3. Summary of Maximum Release Sensitivity Analysis Results: December 2007 Event

Scenario	Maximum by Locations						Difference from Unregulated		
	FRE		Doty, WA		Grand Mound, WA				
	Reservoir Elevation (ft)	Reservoir Release (cfs)	Flow (cfs)	WSE (ft)	Flow (cfs)	WSE (ft)	FRE Flow (%)	Doty WSE (ft)	Grand Mound WSE (ft)
Unregulated conditions	N/A	50,482	64,882	336.8	79,100	147.2	N/A	N/A	N/A
2017 operations plan = 300 cfs	631.1	11,091	22,855	322.8	61,567	146.0	-78	-14.0	-1.2
Maximum FRE release = 1,000 cfs	630.7	10,092	23,553	323.1	62,269	146.1	-80	-13.7	-1.1
Maximum FRE release = 2,000 cfs	630.0	9,043	24,546	323.6	63,204	146.2	-82	-13.2	-1.0
Maximum FRE release = 3,000 cfs	629.3	9,103	25,544	324.0	64,340	146.2	-82	-12.8	-1.0

Table 14-4. Summary of Outflow Release Sensitivity Analysis Results: January 2022 Event

Scenario	Maximum by Locations						Difference from Unregulated		
	FRE		Doty, WA		Grand Mound, WA				
	Reservoir Elevation (ft)	Reservoir Release (cfs)	Flow (cfs)	WSE (ft)	Flow (cfs)	WSE (ft)	FRE Flow (%)	Doty WSE (ft)	Grand Mound WSE (ft)
Unregulated conditions	N/A	15,085	19,071	321.2	51,300	145.2	N/A	N/A	N/A
2017 operations plan release = 300 cfs	594.8	8,158	9,655	316.2	43,840	144.5	-46	-5.0	-0.7
Maximum FRE release = 1,000 cfs	587.8	7,878	9,378	316.0	44,535	144.6	-48	-5.2	-0.6
Maximum FRE release = 2,000 cfs	578.8	7,325	8,821	315.7	45,896	144.7	-51	-5.5	-0.5
Maximum FRE release = 3,000 cfs	569.8	7,325	8,823	315.7	46,832	144.8	-51	-5.5	-0.4

Figure 14-2. Maximum Release Sensitivity Analysis Results: February 1996 Event

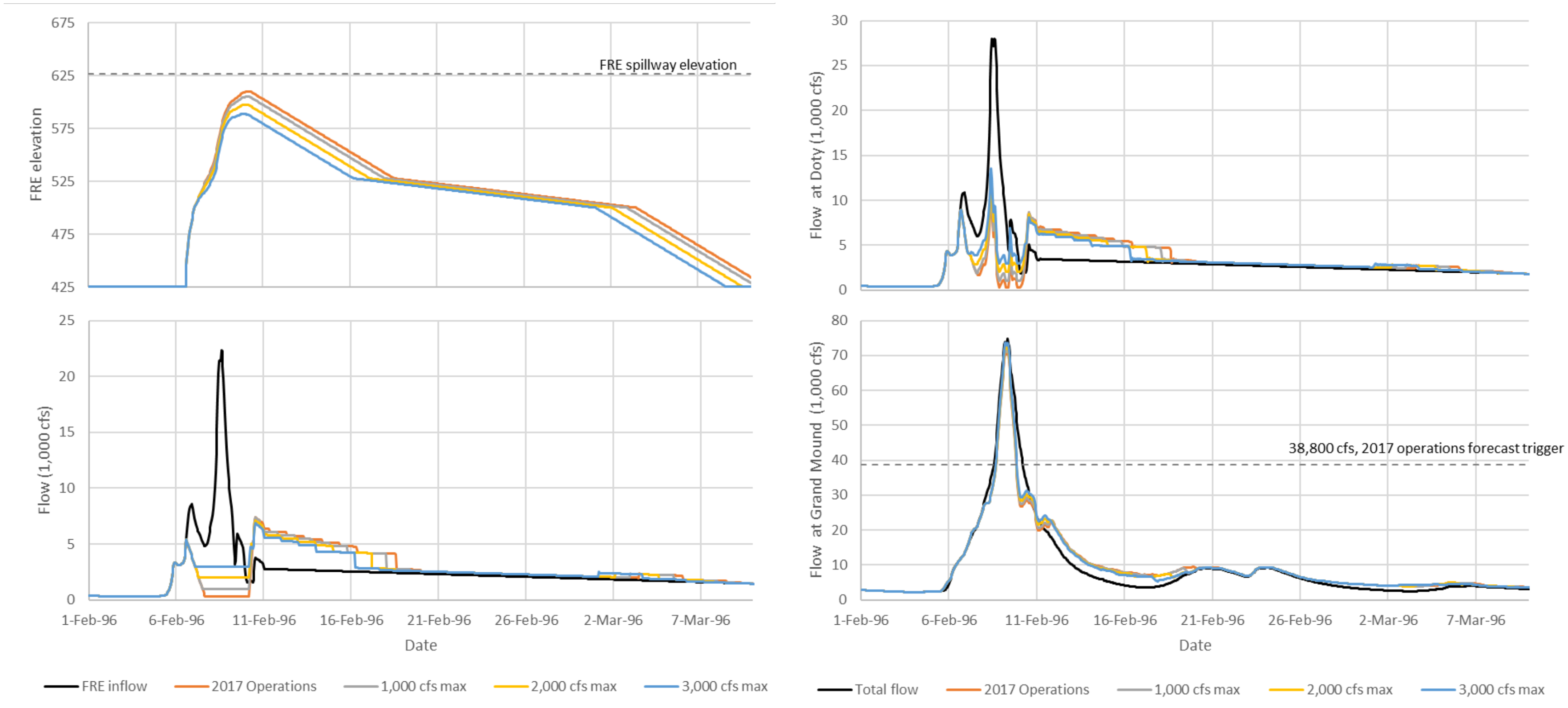


Figure 14-3. Maximum Release Sensitivity Analysis Results: December 2007 Event

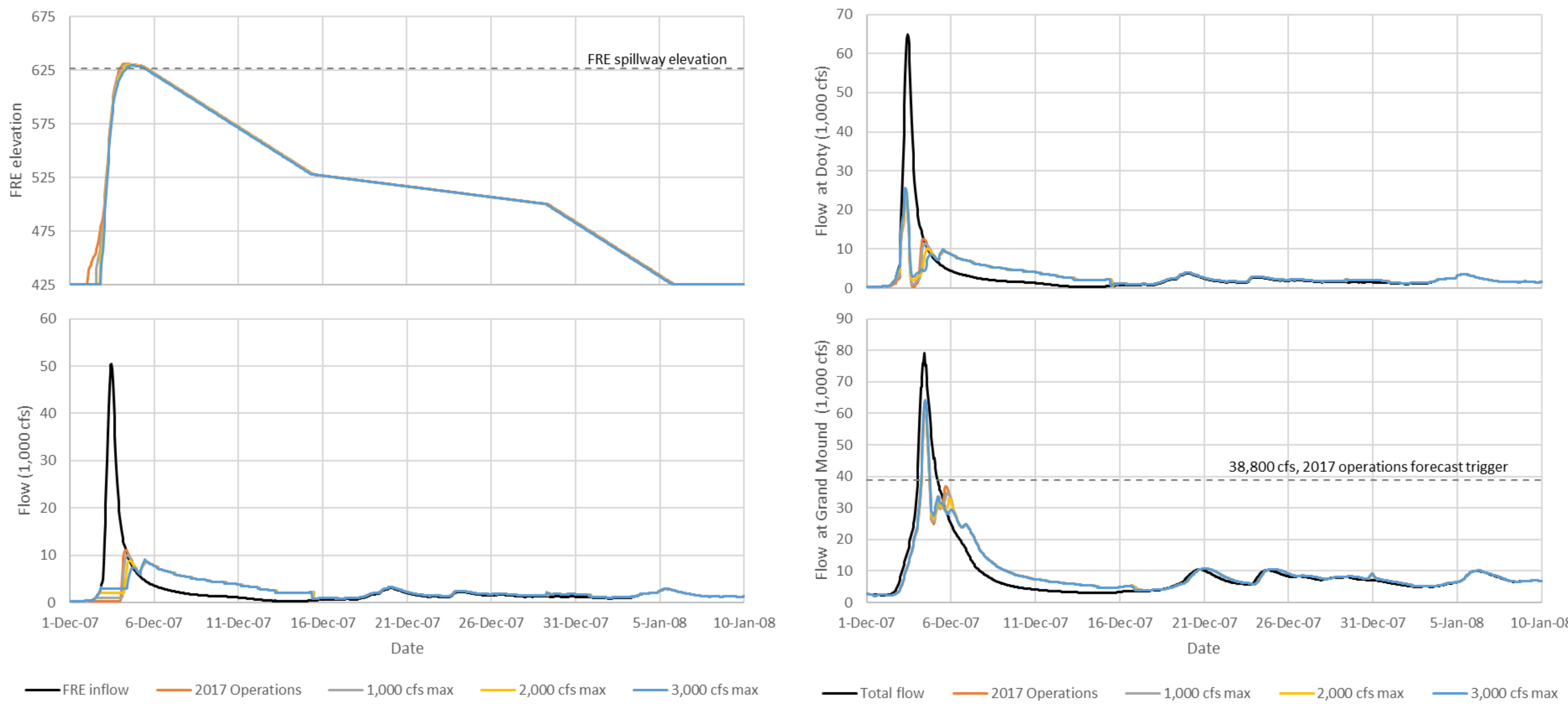
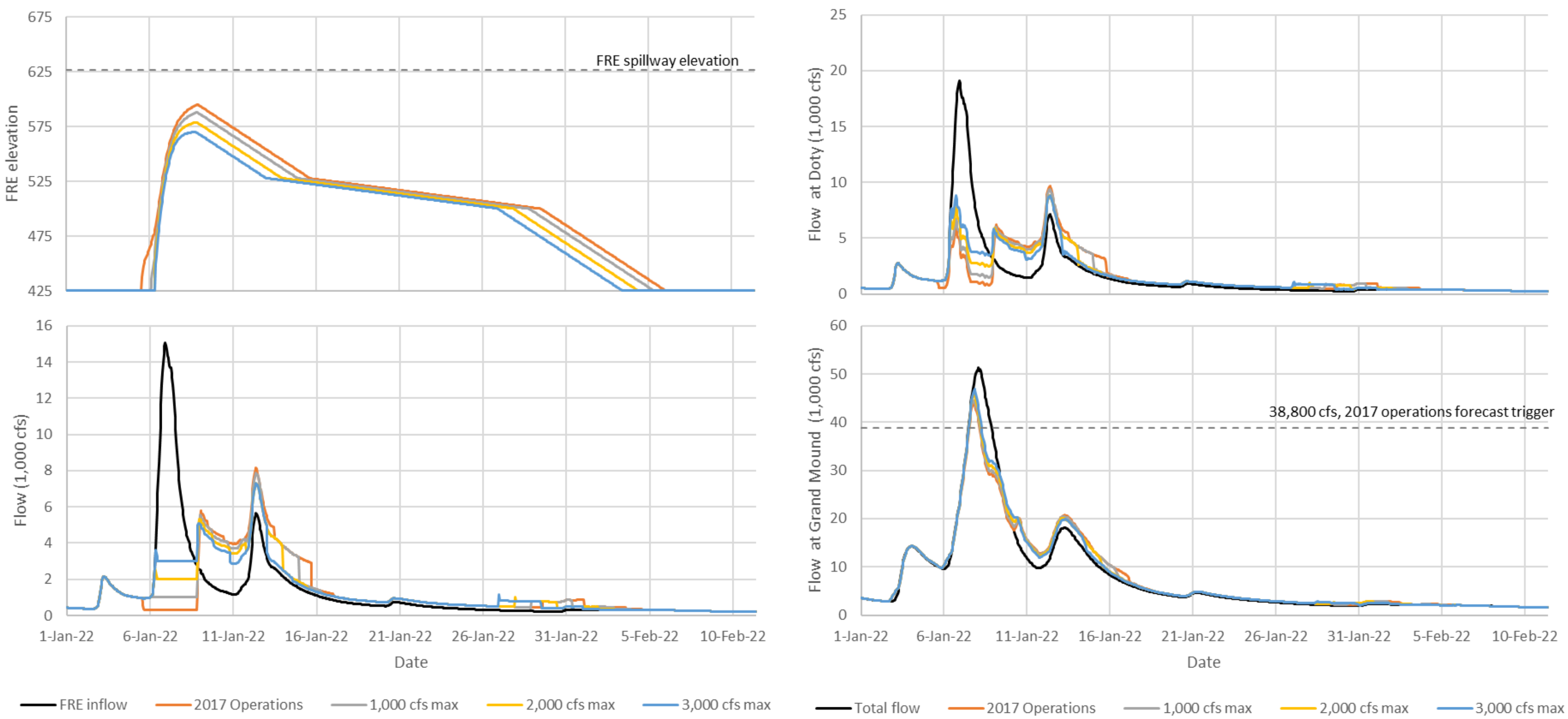


Figure 14-4. Maximum Release Sensitivity Analysis Results: January 2022 Event



14.5.3 Modified Forecast Triggers

The sensitivity of operations to different forecast triggers is described in this section. Storms can set up over or move across different parts of the Chehalis Basin. While the resulting flow at Grand Mound may be the same for different storms, the storm center locations can cause significantly different amounts of flows in the subbasins.

HDR examined the differences in FRE performance if different forecast triggers were used for FRE operation activation. Specifically, HDR investigated the potential impacts of a larger forecast trigger at Grand Mound, WA, the addition of a forecast trigger condition at Doty, WA, and the combination of the current Grand Mound trigger and additional trigger at Doty. In general, HDR found that larger or additional forecast triggers result in shorter durations of operation and reduced maximum FRE reservoir elevations. The WSEL is slightly increased (by as much as 0.7 ft) at Grand Mound, WA. Table 14-5 through Table 14-7 summarize these findings and corresponding results are shown in Figure 14-5 through Figure 14-7.

The addition of an operational threshold at Doty in combination with the Grand Mound trigger demonstrated that it will be possible to begin operations later and end operations earlier during a flood event, reducing both the duration and elevation of the temporary pool behind the FRE. Further operational optimization is almost certainly possible through exploring and defining additional trigger points and thresholds throughout the basin. These results suggest that it is possible that additional triggers within the basin will further minimize or even avoid any operations based on additional forecasts on tributaries.

Table 14-5. Summary of Forecast Trigger Sensitivity Analysis Results: February 1996 Event

Scenario	Maximum by locations						Difference from Unregulated		
	FRE		Doty, WA		Grand Mound, WA				
	Reservoir Elevation (ft)	Reservoir Release (cfs)	Flow (cfs)	WSE (ft)	Flow (cfs)	WSE (ft)	FRE Flow (%)	Doty WSE (ft)	Grand Mound WSE (ft)
Unregulated conditions	N/A	22,365	27,966	324.9	74,800	147.0	N/A	N/A	N/A
2017 operations plan	610.3	7,394	10,808	316.9	71,035	146.8	-67	-8.0	-0.3
52,500 cfs Grand Mound trigger	592.5	8,443	11,428	317.26	74,304	146.92	-62	-7.67	-0.04
38,800 cfs Grand Mound & 25,200 Doty trigger	589.6	11,856	12,249	317.72	71,035	146.71	-47	-7.21	-0.25
52,500 cfs Grand Mound & 25,200 Doty trigger	570.5	11,444	12,138	317.66	74,304	146.92	-49	-7.27	-0.04

Table 14-6. Summary of Forecast Trigger Sensitivity Analysis Results: December 2007 Event

Scenario	Maximum by Locations						Difference from Unregulated		
	FRE		Doty, WA		Grand Mound, WA				
	Reservoir Elevation (ft)	Reservoir Release (cfs)	Flow (cfs)	WSE (ft)	Flow (cfs)	WSE (ft)	FRE Flow (%)	Doty WSE (ft)	Grand Mound WSE (ft)
Unregulated conditions	N/A	50,482	64,882	336.8	79,100	147.2	N/A	N/A	N/A
2017 operations plan	631.1	11,091	22,855	322.8	61,567	146.0	-78	-14.0	-1.2
52,500 cfs Grand Mound trigger	631.1	11,084	22,855	322.8	61,567	146.0	-78	-14.0	-1.2
38,800 cfs Grand Mound & 25,200 Doty trigger	628.4	14,733	22,855	322.8	61,567	146.0	-71	-14.0	-1.2
52,500 cfs Grand Mound & 25,200 Doty trigger	628.40	14,726	22,855	322.8	61,567	146.0	-71	-14.0	-1.2

Table 14-7. Summary of Forecast Trigger Sensitivity Analysis Results: January 2022 Event

Scenario	Maximum by Locations						Difference from Unregulated		
	FRE		Doty, WA		Grand Mound, WA				
	Reservoir Elevation (ft)	Reservoir Release (cfs)	Flow (cfs)	WSE (ft)	Flow (cfs)	WSE (ft)	FRE Flow (%)	Doty WSE (ft)	Grand Mound WSE (ft)
Unregulated conditions	N/A	15,085	19,071	321.17	51,300	145.2	N/A	N/A	N/A
2017 operations plan	594.8	8,158	9,655	316.2	43,840	144.5	-46	-5.0	-0.7
52,500 cfs Grand Mound trigger	448.9	14,925	18,899	321.1	51,040	145.2	-1	-0.2	-0.0
38,800 cfs Grand Mound & 25,200 Doty trigger	448.9	14,925	18,899	321.1	51,040	145.2	-1	-0.2	-0.0
52,500 cfs Grand Mound & 25,200 Doty trigger	448.9	14,925	18,899	321.1	51,040	145.2	-1	-0.2	-0.0

Figure 14-5. Forecast Trigger Analysis Results: February 1996 Event

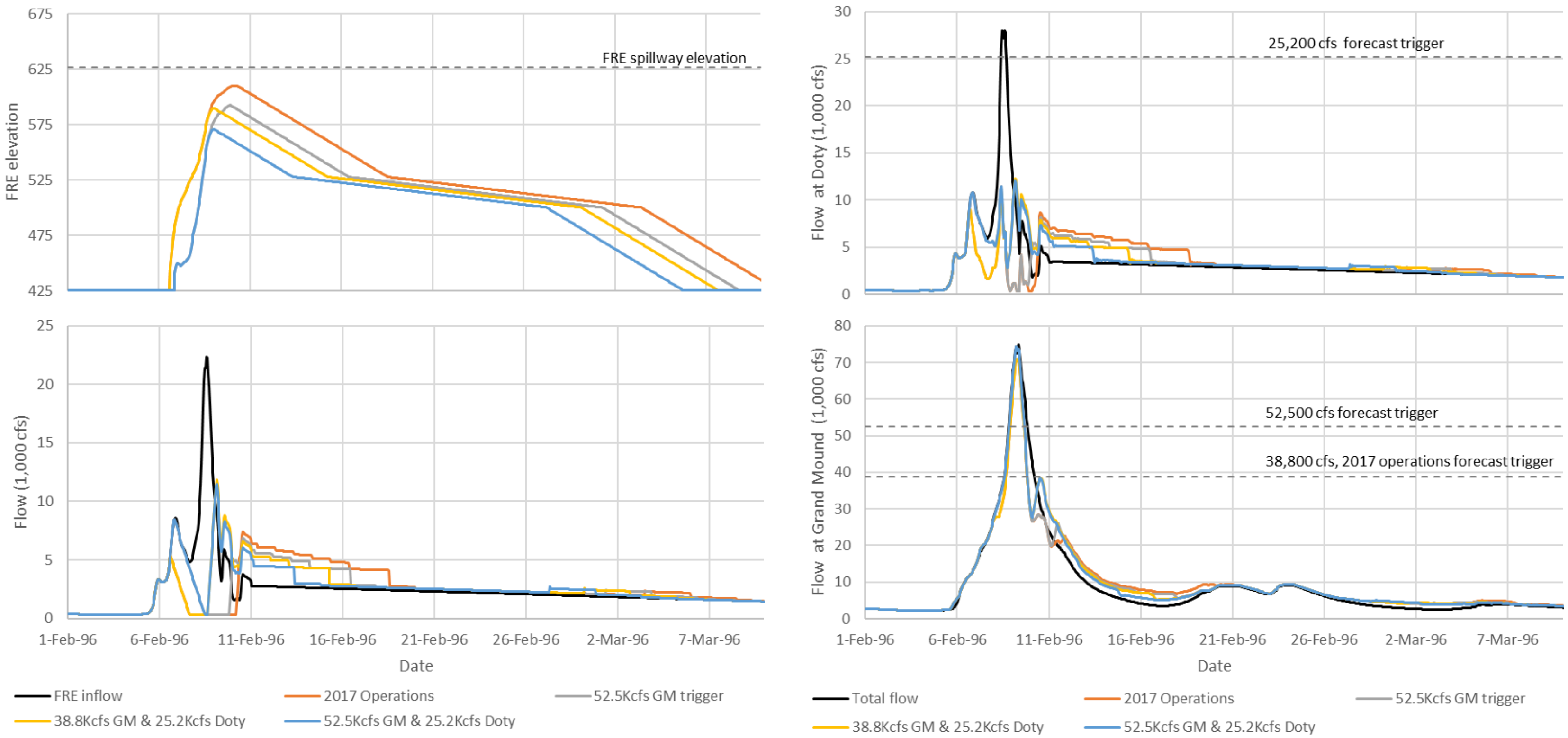


Figure 14-6. Forecast Trigger Sensitivity Analysis Results: December 2007 Event

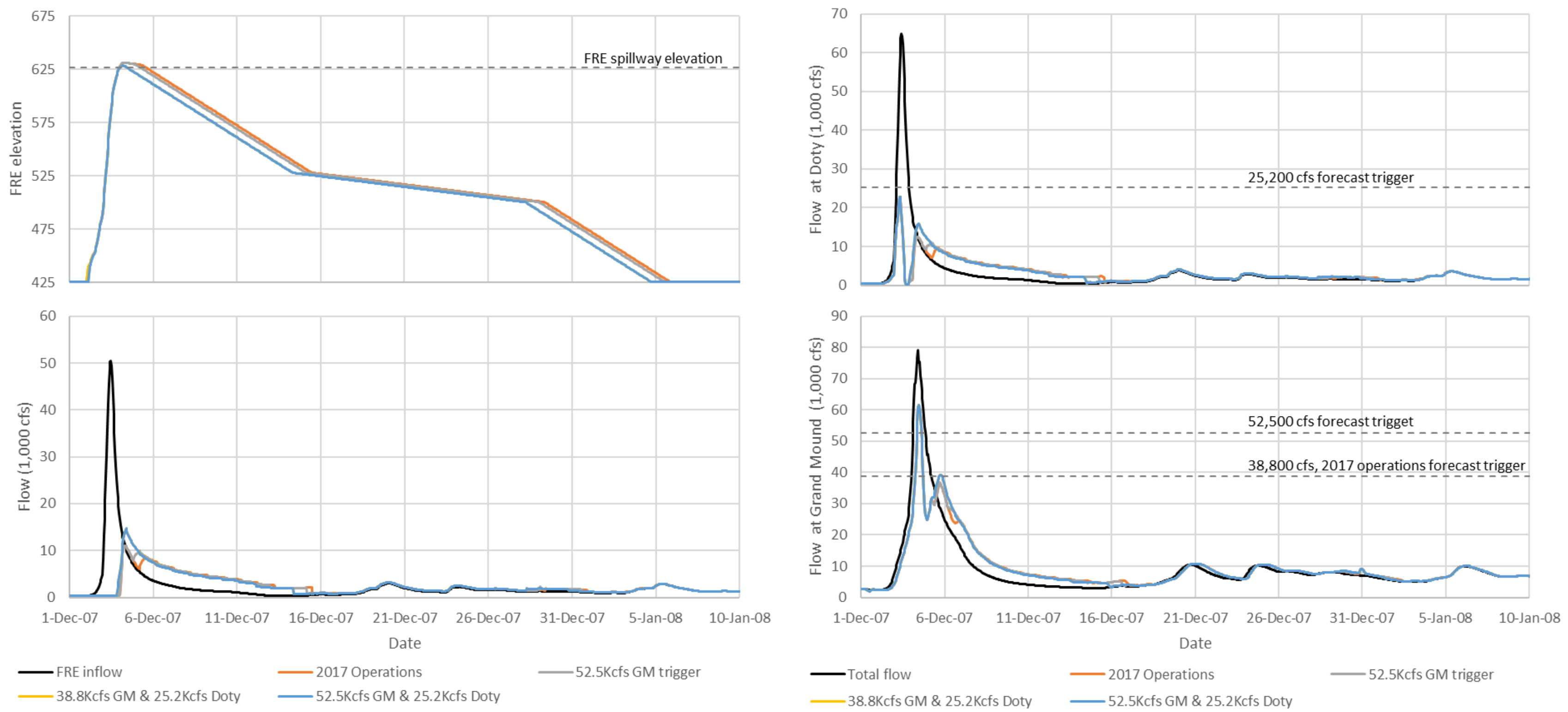
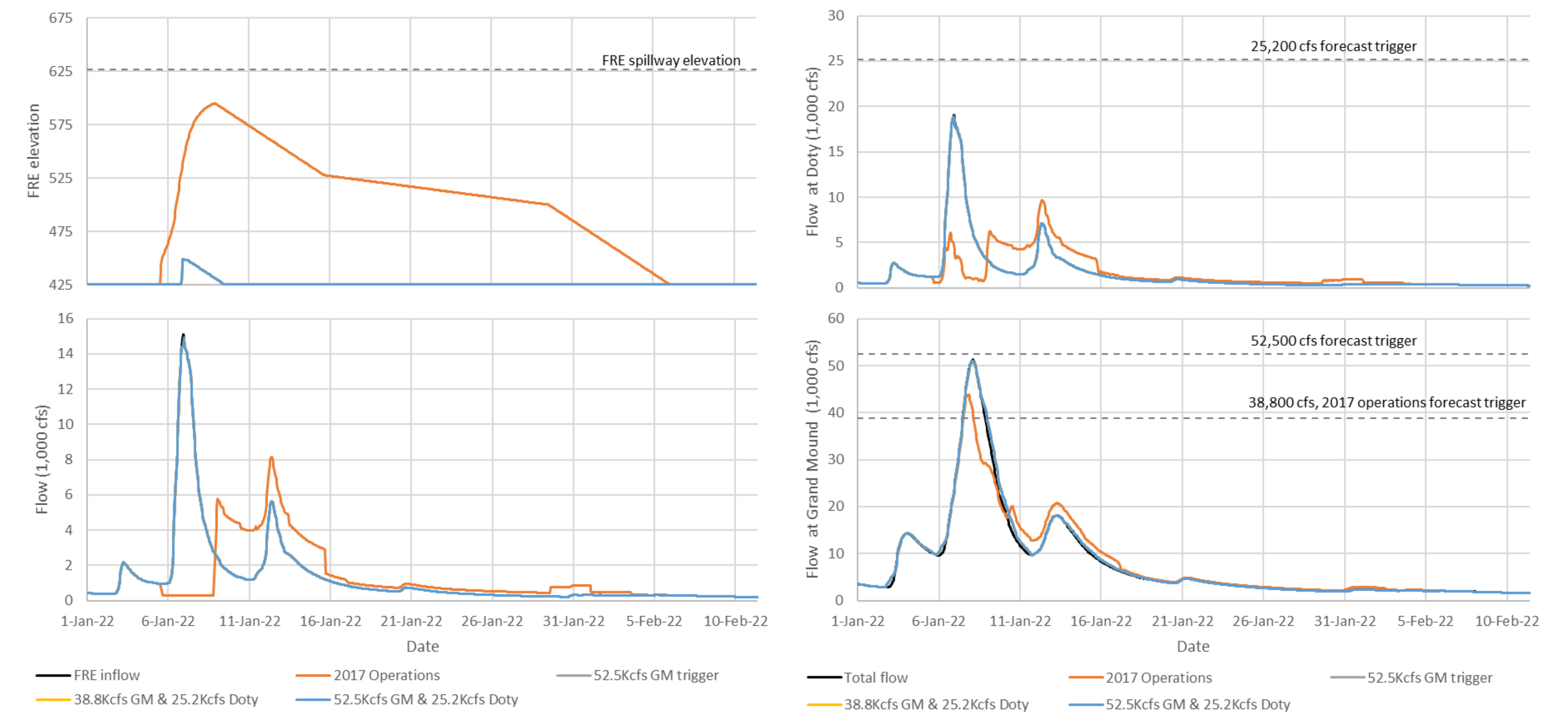


Figure 14-7. Forecast Trigger Sensitivity Analysis Results: January 2022 Event



14.5.4 Modified Drawdown Rates

The modification of the drawdown rate has a very direct impact on the detention time of the temporary pool stored behind the FRE. While varying the maximum release during flood operations had an impact on the ultimate volume and elevation of the pool, the drawdown rate determines how quickly the pool is released and operations return to run-of-river conditions. In this sensitivity analysis, the Anchor QEA (2017) drawdown plan was retained but varied plus and minus 20% to test the impact. In general, HDR found that increasing the drawdown rate by 20% resulted in the temporary pool draining approximately 5 days faster and decreasing the drawdown rate by 20% resulted in the pool draining approximately 5 days slower. Downstream peaks were unaffected by these modifications because the drawdown plan applies only to draining the temporary pool after a flood event has passed. The results are summarized in Table 14-8 through Table 14-10 and Figure 14-8 through Figure 14-10.

Table 14-8. Summary of Drawdown Rate Sensitivity Analysis Results: February 1996 Event

Scenario	Maximum by Locations						Difference from Unregulated		
	FRE		Doty, WA		Grand Mound, WA				
	Reservoir Elevation (ft)	Reservoir Release (cfs)	Flow (cfs)	WSE (ft)	Flow (cfs)	WSE (ft)	FRE Flow (%)	Doty WSE (ft)	Grand Mound WSE (ft)
Unregulated conditions	N/A	22,365	27,966	324.9	74,800	147.0	N/A	N/A	N/A
2017 operations plan	610.3	7,394	10,808	316.9	71,035	146.8	-67	-8.0	-0.3
Drawdown rate +20%	610.3	6,676	10,808	316.90	71,035	146.71	-70	-8.03	-0.25
Drawdown rate -20%	610.3	8,113	10,808	316.90	71,035	146.71	-64	-8.03	-0.25

Table 14-9. Summary of Drawdown Rate Sensitivity Analysis Results: December 2007 Event

Scenario	Maximum by Locations						Difference from Unregulated		
	FRE		Doty, WA		Grand Mound, WA				
	Reservoir Elevation (ft)	Reservoir Release (cfs)	Flow (cfs)	WSE (ft)	Flow (cfs)	WSE (ft)	FRE Flow (%)	Doty WSE (ft)	Grand Mound WSE (ft)
Unregulated conditions	N/A	50,482	64,882	336.8	79,100	147.2	N/A	N/A	N/A
2017 operations plan	631.1	11,091	22,855	322.8	61,567	146.0	-78	-14.0	-1.2
Drawdown rate +20%	631.11	11,091	22,855	322.8	61,567	146.0	-78	-14.0	-1.2
Drawdown rate -20%	631.11	11,091	22,855	322.8	61,567	146.0	-78	-14.0	-1.2

Table 14-10. Summary of Drawdown Rate Sensitivity Analysis Results: January 2022 Event

Scenario	Maximum by Locations						Difference from Unregulated		
	FRE		Doty, WA		Grand Mound, WA				
	Reservoir Elevation (ft)	Reservoir Release (cfs)	Flow (cfs)	WSE (ft)	Flow (cfs)	WSE (ft)	FRE Flow (%)	Doty WSE (ft)	Grand Mound WSE (ft)
Unregulated conditions	N/A	15,085	19,071	321.2	51,300	145.2	N/A	N/A	N/A
2017 operations plan	594.8	8,158	9,655	316.0	43,840	144.5	-46	-5.0	-0.7
Drawdown rate +20%	594.84	7,652	9,148	315.9	43,840	144.5	-49	-5.3	-0.7
Drawdown rate -20%	594.8	8,327	9,825	316.3	43,840	144.5	-45	-4.9	-0.7

Figure 14-8. Drawdown Rate Sensitivity Analysis Results: February 1996 Event

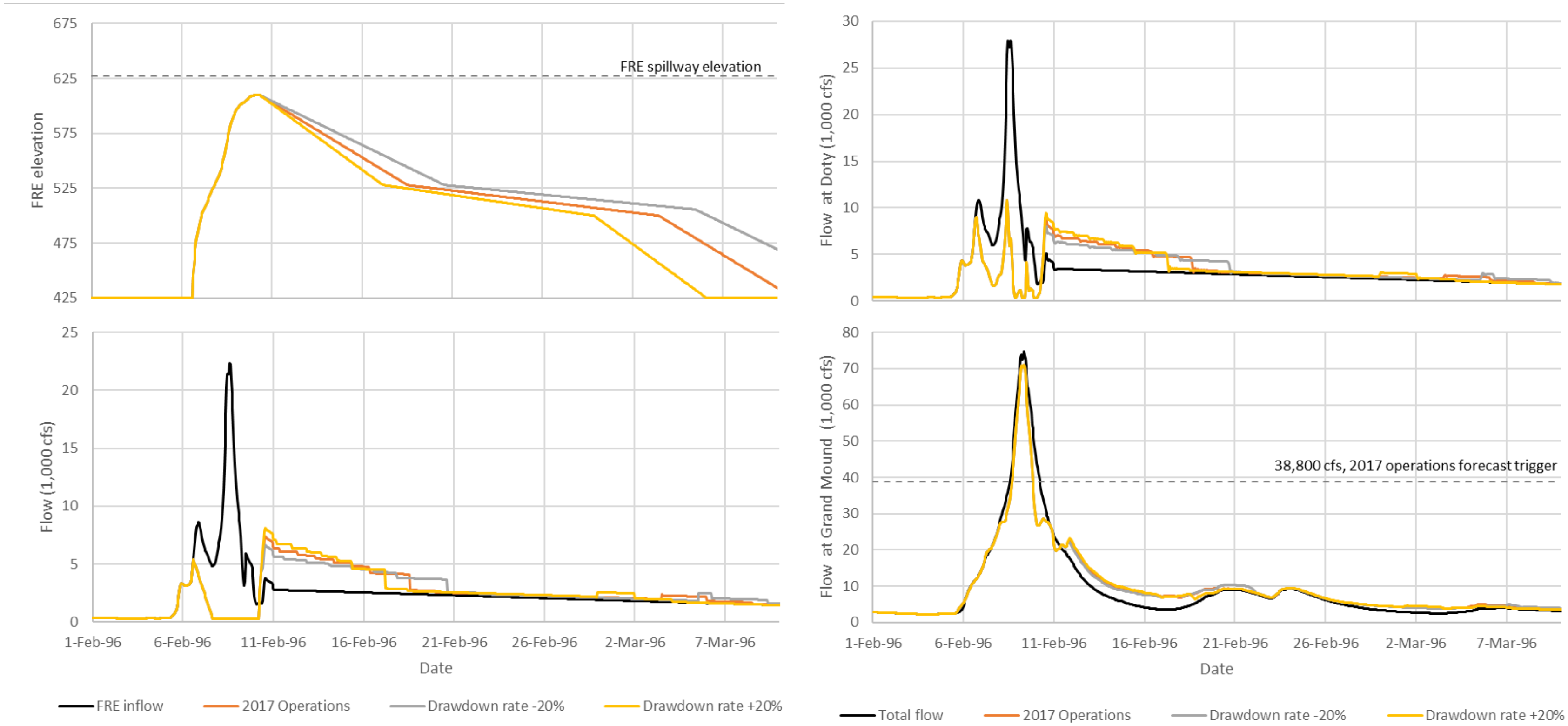


Figure 14-9. Drawdown Rate Sensitivity Analysis Results: December 2007 Event

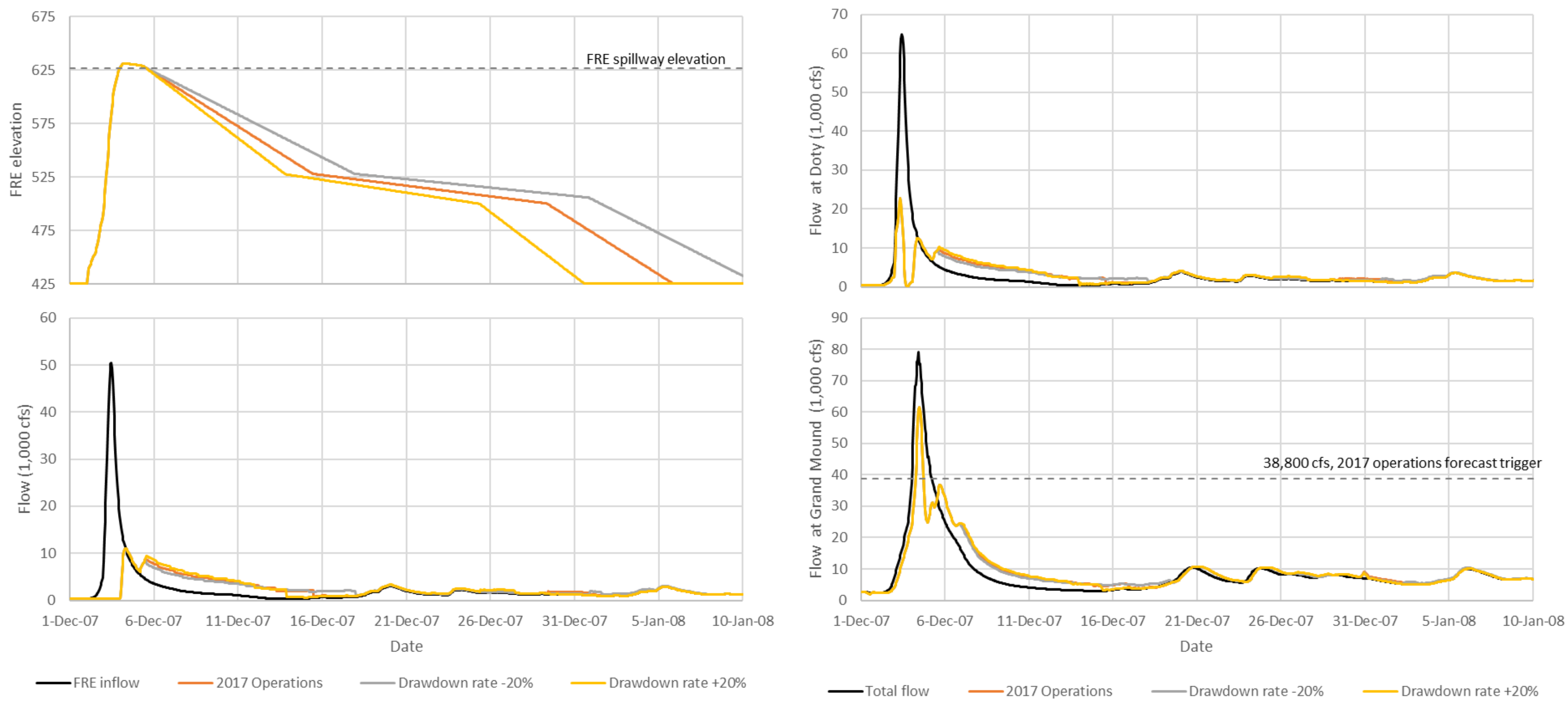
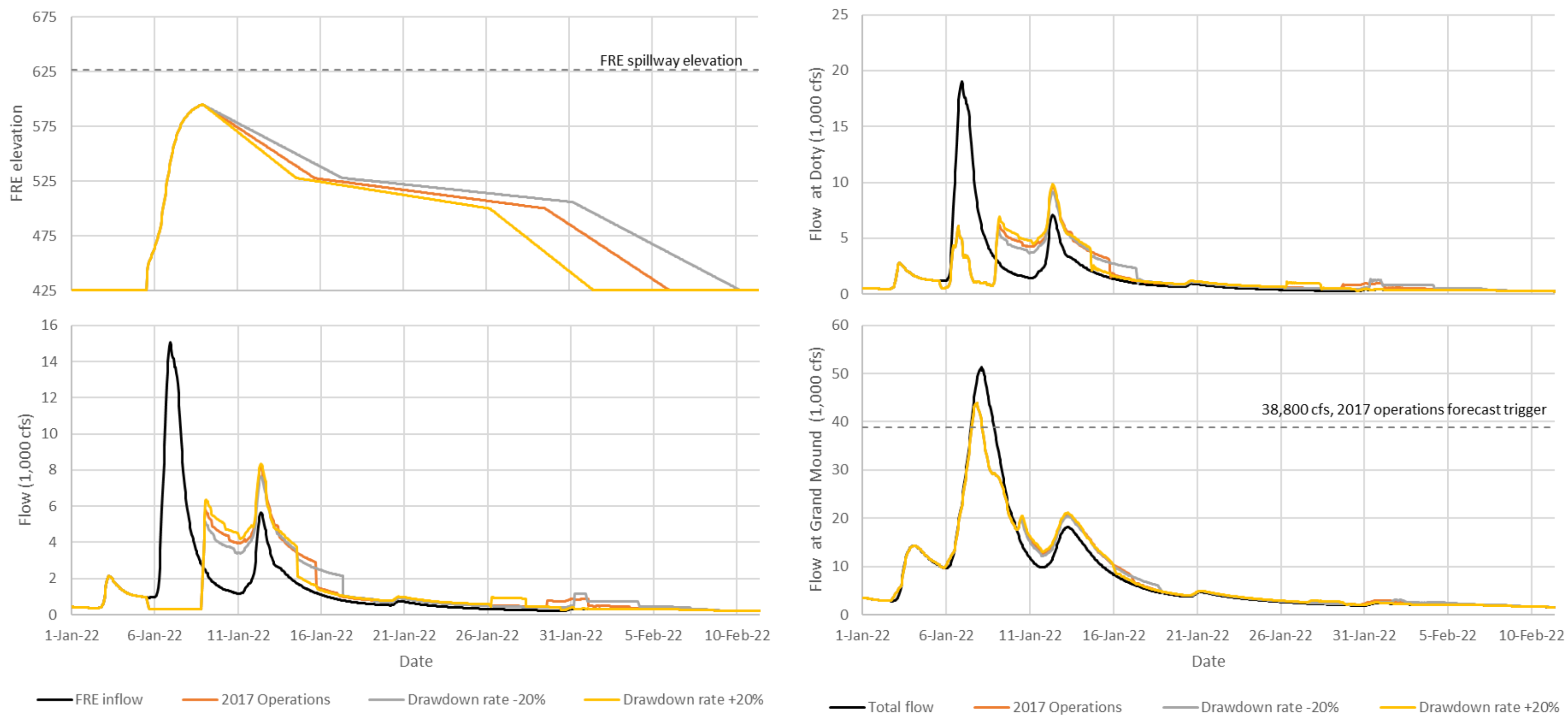


Figure 14-10. Drawdown Rate Sensitivity Analysis Results: January 2022 Event



14.5.5 Sensitivity Analysis Conclusions

This sensitivity analysis demonstrated it is technically feasible to enhance and improve operations of the FRE. Changing the maximum outflow from the FRE during a flood event appears to result in a reduction of both maximum pool depth and time of storage for the temporary pool. Additional minimization and avoidance of operations will likely be possible by adding and combining additional flow trigger locations and thresholds in the basin. The time of storage of the temporary pool can also be reduced by optimizing the drawdown plan to drain faster.

Through operational flexibility and optimization, the direct and indirect environmental impacts will be reduced for smaller, more frequent events, while still achieving flood damage reduction objectives. Furthermore, existing hydrological data indicates that there will be opportunities to improve operations to minimize impacts even during the large, less frequent, events. Further analysis will provide for additional clarity on the extent of operational flexibility for less frequent events.

15 Fish Passage

The FRE structure includes the following fish passage components, designed to provide passage for a range of species and life stages:

- Flood Fish Passage Facility (FFPF)
- Fish Passage Conduits
- Temporary channels
- Permanent channels

The fish passage design documented in this report includes updates of the design criteria to comply with current standards, re-examination and update of previous concept- level design development, performance assessment for a newly proposed bypass channel, and development of a plan to advance the fish passage design to a level necessary to inform the final Biological Assessment.

The fish passage design development and coordination efforts for the proposed project occurred over time with most of the work occurring between 2016-2024. First, design development occurred primarily in two stages. The conceptual design of the FFPF was completed for the original FRE alignment in 2017. This conceptual design and design criteria were based on current federal (NMFS) and state (WDFW) guidance documents at the time. The FFPF conceptual design effort was adapted to function similarly at the revised FRE alignment in 2023, but not significantly redesigned like the fish passage conduits and permanent channels were in 2023. Second, the overall fish passage conceptual design and design criteria were developed in coordination with the 2016-2017 Chehalis Basin Strategy Flood Damage Reduction Technical Committee Fish Passage Subcommittee (Subcommittee). This group was re-formed as the Fish Passage Technical Working Group (TWG) in 2023-2024. Design information presented in this section that was developed in collaboration with the Subcommittee and revisited by the TWG is noted as such. Otherwise design information presented in this section was developed in collaboration with the Subcommittee and has not yet been updated to reflect the current FRE alignment and current federal fish passage guidance nor reviewed by the TWG.

Additional detail can be found in Appendix I. History of fish passage design development for this Proposed Project, including detailed design information, can be found in the References section at the end of this report and the Fish Passage Design TM in Appendix I.

15.1 Purpose and Intent

The integration of fish passage systems is a central component of the flood damage reduction structure design. Fish passage facility design has occurred simultaneously with dam design efforts throughout the development of this RPDR. The purpose of this section is to summarize the results and conclusions of fish passage concept development performed in previous documents and in 2023 for this RPDR and identify a “roadmap” for fish passage design development supporting the final Biological Assessment. This information is intended to be used by the WDOE in development of the SEPA EIS, the USACE in development of the NEPA EIS, and by WDFW, NOAA Fisheries, and the Technical Working Group (TWG) to inform decisions regarding the integration and performance of potential fish passage technologies with the FRE structure being developed by the design team.

15.2 Design Criteria

This section summarizes the design criteria used for the design of fish passage facilities for the Proposed Project. Design criteria is categorized in this report as biological design criteria (15.2.1) or technical design criteria (15.2.2), and further subdivided into general criteria and criteria specific to the FFPF. General criteria apply to all fish passage components, unless noted.

Criteria for the Proposed Project were developed by referencing or engaging with the following sources or entities:

- Collaboration with the Fish Passage Technical Working Group comprised of members from WDFW, USFWS, USACE, NOAA Fisheries, WDOE, the Cowlitz Indian Tribe, and the District's Fish Passage Design Team
- Published design guidance and criteria developed by state and federal agencies
- Published research and historical data relevant to the Proposed Project
- Previous design development documents

For detailed discussion of specific criteria, refer to the Fish Passage Design TM included as Appendix I.

15.2.1 Biological Design Criteria

Biological design criteria for each component of the Proposed Project are summarized in the following subsections. General fish passage criteria apply to all project components where fish passage must be maintained (i.e., fish passage conduits, FFPF, permanent Chehalis River and Crim Creek channels, and Chehalis River and Crim Creek construction bypass), unless shown otherwise.

15.2.1.1 General Biological Design Criteria

General fish passage biological criteria which apply to all fish passage components of the Proposed Project are discussed in this section.

Target Species

Table 15-1 presents the target fish species and their respective life stages that were selected for the purposes of fish passage design development in this study. It should be noted that bull trout are believed to occur only downstream of the proposed dam location and were removed as a target species by the Subcommittee.

Table 15-1. Target Fish Species and Life Stages Selected for Design Development

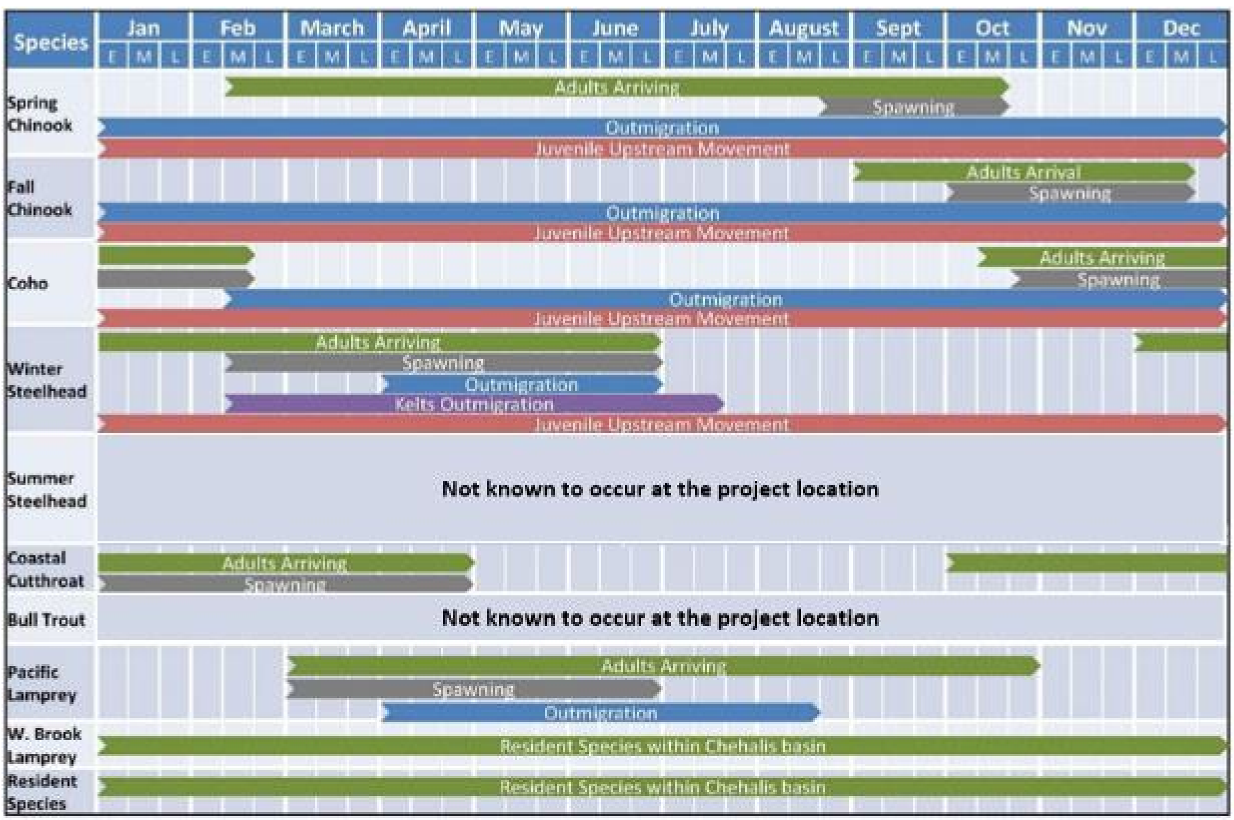
Species	Upstream	Downstream
Spring-run Chinook salmon	Adult, juvenile	Juvenile
Fall-run Chinook salmon	Adult, juvenile	Juvenile
Coho salmon	Adult, juvenile	Juvenile
Winter-run steelhead	Adult, juvenile	Adult, juvenile
Coastal cutthroat trout	Adult, juvenile	Adult, juvenile

Species	Upstream	Downstream
Pacific lamprey	Adult	Ammocoetes, Macropthalmia
Western brook lamprey	Adult	Ammocoetes, Macropthalmia
Resident fish, including: river lamprey, largescale sucker, Salish sucker, torrent sculpin, reticulate sculpin, riffle sculpin, prickly sculpin, speckled dace, longnose dace, peamouth, northern pikeminnow, redside shiner, rainbow trout, mountain whitefish	Adult	Adult

Migration Timing

Figure 15-1 presents the migration timing and duration for the different life stages of each selected species. Many of the target species are known to have unique migration behaviors and believed to pass upstream or downstream through the dam site at specific times of the year. The migration timing and duration for each selected fish species and life stage were discussed at Subcommittee meetings as new information was collected in the field and from literature sources.

Figure 15-1. Anticipated Migration Periods of the Targeted Species and Life Stages (Periodicity)



Species Abundance

Fish abundance was evaluated by WDFW and discussed during Subcommittee meetings. Abundance was described in terms of peak annual, peak daily, and peak hourly rates of migration.

The peak daily rate of migration for both upstream and downstream migrating fish influences the size of many fish passage component alternatives. A combined peak daily count of roughly 2,000 adult salmonids and a peak hourly count of 400 adult salmonids was used for design purposes for upstream migration.

Numbers for adult upstream migrating pacific lamprey, cutthroat trout, resident fish, and juvenile salmonids have not been estimated.

Abundance and daily peak numbers to be used for reference in designing downstream passage for juvenile salmon and steelhead are based on a design number of fish expected to migrate downstream to the location selected for the dam. A maximum daily abundance of 55,505 smolt was selected for design purposes for downstream migration.

Resident Fish Considerations

Finally, the Subcommittee, with support from the team's USFWS representative, assembled relevant biological data for the target resident species, lamprey, and salmonids. The Subcommittee was not able to find data on all target resident species. Through continued collaboration with the TWG, all fish passage is being designed to accommodate identified resident species to the extent possible, and without adversely affecting facility performance for listed priority species (salmonids and lamprey).

15.2.1.2 Flood Fish Passage Facility Biological Design Criteria

Biological criteria for the FFPF which differ from the general biological fish passage criteria discussed in Section 15.2.1.1 are identified in this section.

FFPF Target Species

Table 15-1 provides the fish species selected and life stages for design of the FFPF. For development of the FFPF, anadromous and resident species known to occur within the influence of the dam, in the inundation area of the associated reservoir, and upstream of the reservoir were selected for upstream passage only.

No downstream passage is provided by the FFPF. Downstream passage of juvenile salmon and steelhead is provided via the fish passage conduits when they are open. Downstream passage of outmigrating fish will be delayed during impoundment events coincident with flood retention activities. Because the primary flood control gates are almost closed and water is retained upstream of the dam, outmigrating fish entering the impoundment at this time would also be temporarily retained.

Upstream passage of juvenile salmon and steelhead is provided via the fish passage conduits when they are open. Upstream migration of juvenile species through trap and transport facilities has been documented and is expected to occur at some level during FFPF operations. Although the FFPF is not proposed to be specifically designed for upstream passage of juveniles, juveniles may pass through the facility and their collections is expected to occur to some degree. The same holding, sorting, and transport facilities for adults will also be used for juveniles. (HDR 2017a, Appendix G).

FFPF Migration Timing

Fish species migration timing and duration used for design of the FFPF follows the general biological fish passage criteria discussed in Section 15.2.1.1 and shown in Figure 15-1.

FFPF Species Abundance

Upstream migration rates used for FFPF design follow the general biological fish passage criteria discussed in Section 15.2.1.1.

FFPF Resident Fish Considerations

FFPF design criteria for resident fish do not differ from the general biological fish passage criteria discussed in Section 15.2.1. The FFPF is being designed to accommodate trap and transport of resident species to the extent possible, and without adversely affecting facility performance for priority species (salmonids, cutthroat trout, and lamprey). Trap and transport of resident species will be accommodated through incorporation of a separate low volume, low velocity entrance, fish ladder, hopper, and transport tank. Based on known swim speeds for resident species, the species will be able to enter the low volume, low velocity entrance and continue migrating upstream in the juvenile fish ladder via orifices.

FFPF Trapping and Holding

The design criteria for fish trapping and holding are based on established agency guidance and/or published research. Trapping and holding facilities include the holding gallery, hoppers, and transport tanks, and are designed to accommodate for the peak daily and hourly numbers of fish presented in Section 15.2.1.1.

Based on the peak number of fish expected, trapping and holding facilities are adequately sized and operated to not hold fish for greater than 72 hours. Fish holding during emergency situations where holding may be required for more than 72 hours will be addressed during the next phase of design development.

15.2.2 Technical Design Criteria

Technical design criteria for each component of the Proposed Project are summarized in the following subsections. General fish passage criteria apply to all project components where fish passage must be maintained (i.e., fish passage conduits, FFPF, permanent Chehalis River and Crim Creek channels, and Chehalis River and Crim Creek construction bypass), unless shown otherwise.

15.2.2.1 General Technical Design Criteria

Fish Passage Conduits

The fish passage conduits are intended to provide year-round, safe, volitional upstream and downstream passage for migrating adult salmon and steelhead, resident fish, and lamprey for the full range of fish passage flow conditions as required by NMFS criteria. The Subcommittee agreed in 2016 that the proposed flow velocity and depth through the conduits mimic the flow velocity and depth occurring naturally through the existing river reach at the dam. This approach was revisited and presented to WDFW, NOAA Fisheries, and the TWG in 2023, and no objections were voiced as of the publication of this document. This premise influenced the overall approach towards designing and evaluating performance of upstream and downstream passage through the conduits.

Lamprey Passage

As requested by participating resource agencies and Indian Tribes, incorporation of the best available science relating to the passage of lamprey was considered throughout the design. Best practices, lessons learned from experimental facilities on the Columbia River, and interviews with researchers who specialize in understanding lamprey behavior and navigational capabilities were used to inform lamprey passage facility requirements. A detailed list of resources used to form a basis of design for lamprey passage is included in Appendix I.

Trashracks

Trashracks are commonly used at fishway exits and entrances to exclude large debris from entering fish passage facilities. Trashracks are also used at the fish passage conduits. Design criteria for trashracks follow NMFS 2022 guidance. Refer to Appendix I for additional detail.

Constructed Channels

A reference reach design approach is utilized for the permanent Chehalis River approach and discharge channels and Crim Creek as well as for the construction phase Chehalis River and Crim Creek bypass channels. Refer to Appendix I for additional detail.

15.2.2.2 Flood Fish Passage Facility Technical Design Criteria

Technical design criteria for the FFPF that differ from the general technical fish passage criteria discussed in Section 15.2.2.1 are identified in the following subsections.

FFPF Fish Passage Conduits

Fish passage conduit design criteria is not applicable to the design of the FFPF facility as the fish passage conduits are not an available passage pathway when the FFPF is operating.

FFPF Fishways

Upstream fish passage designs at dams use widely recognized fishway design guidelines and references and are traditionally designed for the adult fish life stage. There are three major components to a fishway: fishway entrance, fish ladder, and fishway exit. Criteria for each of these components is based on established agency guidance and/or published research. Refer to Appendix I for additional detail.

The Subcommittee identified two types of fish ladders that were expected to provide the best performance for target and resident species: half-Ice Harbor fish ladder and vertical slot fish ladder. Hydraulic analysis of half-Ice Harbor- and vertical slot-type fish ladders indicate that the half-Ice Harbor-type ladder is believed to provide lower through-orifice velocities and therefore better passage performance for weaker-swimming fish species than the vertical slot-type fish ladder, it was selected as the preferred type of fish ladder.

FFPF Lamprey Passage

Lamprey passage design criteria used for design of the FFPF follow the general fish passage criteria discussed in Section15.2.2.1.

FFPF Trashracks

Trashrack design criteria used for FFPF design follow the general technical fish passage criteria discussed in Section 15.2.2.1.

FFPF Fish Screen and Bypass

A downstream passage system consists of five major components:

- Fish screens to protect juvenile fish from entrainment or impingement.
- A bypass channel. The bypass channel conveys the fish and is often located adjacent to the fish screens.
- A bypass entrance, located at the end of the fish screens.
- A bypass conduit, which conveys fish from the bypass entrance to a point of release downstream (bypass exit).
- A bypass exit, located at the end of the bypass conduit.

Design criteria for each of the above components are detailed in Appendix I.

The FFPF conceptual design includes the use of pumped flow from the dam stilling basin to supply flows to multiple FFPF components. The intake to the pump station will be screened to exclude fish according to NMFS (2022) guidelines.

FFPF Freeboard

The elevation of the finished ground at the sorting facility and the exterior walls of the fish ladder and the pump station will have a top elevation no less than 6 feet of freeboard above the 100-year flood elevation.

FFPF Operation

The FFPF is intended to collect migrating adult salmon and steelhead, juvenile salmon and steelhead, resident fish, and lamprey moving upstream during an impoundment event and safely transport them upstream of the FRE structure. Upstream fish passage via the FFPF will be designed to operate for 24 hours a day, 7 days a week for the full duration of each impoundment event.

Water will also be impounded in the reservoir when the natural flow of the river is greater than the capacity of the fish passage conduits, but not large enough to trigger an impoundment event at Grand Mound. In such situations are estimated to occur approximately once per year and last an average of 1 day. During these water retention events, the fish passage conduit gates would not be operated and remain fully open, and the FFPF would not operate.

Downstream passage of outmigrating fish will be delayed during impoundment events coincident with flood retention activities (i.e., fish passage conduits are closed). The passage of fish downstream would occur as the flood operations cease and the reservoir is drained, and the fish passage conduits are reopened.

Operation of the FFPF is described further in Section14 and Appendix J.

FFPF Auxiliary Water Supply

Fish ladder flow is supplemented with additional attraction flow by an AWS to meet the fish ladder entrance attraction guidelines provided by NMFS (2022).

15.3 Fish Passage Design

This section summarizes the fish passage facility design, including the fish passage conduits, FFPF, permanent river channels, and construction bypass channels.

It should be noted that the FFPF design has not substantially changed from the original conceptual design. Elements of the FFPF that are site-specific, including the fish ladder entrances, the FFPF water supply, and the physical location of the individual FFPF components (i.e., sorting building, fish ladder, etc.) were relocated to the current Proposed Project site. Other elements of the FFPF (i.e., the internal components of the FFPF) function the same as the original design, were not advanced during the course of this study, and remain valid.

15.3.1 Design Flows

Flows used for fish passage design at the project site are summarized in the following subsections.

15.3.1.1 Fish Passage Conduits and Permanent River Channels

The design flows used for the RPD for the fish passage conduits and permanent river channels were determined using NOAA Fisheries West Coast Region Guidance to Improve the Resilience of Fish Passage Facilities to Climate Change (NOAA 2022b).

Climate change information is incorporated into the fish passage design flows using peak flow scalars that were derived from the 12 global climate models produced by WDOE’s consultants for the SEPA EIS (WSE 2023). The high and low fish passage design flows used in the design of the fish passage conduits documented in this report are 3,400 cfs and 14 cfs, respectively. This approach to approximating fish passage design flows incorporating climate change conditions is conservative and consistent with a conceptual level of design development. These design flows will be further refined with additional climate change information in collaboration with NOAA Fisheries during preliminary and final design.

15.3.1.2 Flood Fish Passage Facility

Flows used in the design of the FFPF were documented in previously published documents and remain unchanged and are summarized in this section. These previously established design flows for the FFPF will be updated in future design development to be consistent with current NOAA Fisheries design guidance, including the incorporation of climate change (NOAA Fisheries 2022b).

Design Flows

NMFS (2011) required the high fish passage design flow to be the mean daily stream flow that is exceeded 5 percent of the time during periods when target fish species are migrating. Based on WDFW and NMFS guidance, a flow range between the 95 percent and 5 percent exceedance flows provides the widest range of flows for which facilities should be capable of passing fish, therefore, this flow range is set as the design criterion for the proposed facilities.

The 5 and 95 percent exceedance flows at the dam site were developed based on the mean daily flows for water years 1940 through 2012 from USGS gage 12020000 near Doty and then listed for each adult species using their respective upstream migration timing. The lowest 95 percent exceedance flow and the largest 5 percent exceedance determined the fish passage design flow range that both FRE upstream fish passage facilities will be designed for. The lowest 95 percent exceedance flow is 16 cfs, which occurs during the Fall Chinook migration period. The highest 5 percent exceedance flow is 2,197 cfs, which occurs during the Coho migration period. Therefore, fish passage facilities were designed to operate from a low fish passage flow of 16 cfs to 2,200 cfs.

Tailwater and Reservoir Fluctuation Ranges

Anticipated tailwater fluctuations for the FRE structure are significant factors in determining the type, size, and complexity of the FFPF. The fish ladder and fish ladder entrance of the FFPF must provide a continuous hydraulic connection throughout the anticipated range of tailwater elevations. In addition, the pump station supplying water for the FFPF that draws water from the tailwater pool must also accommodate the fluctuation in tailwater elevation without adversely affecting the water supply or endangering the facilities.

The design fish passage flows and select floods associated with their respective tailwater elevations in the stilling basins are provided in Table 15-2.

Table 15-2. Tailwater Elevations for Fish Passage Design Flows and Select Floods

Flow Event	Flow (cfs)	Tailwater Elevation (feet)
Low fish passage design flow	16	417.0
High fish passage design flow	2,200	419.3
2-year flood	7,300	427.4
10-year flood	10,300	430.1
25-year flood	12,200	431.7
100-year flood	15,000	433.9
PMF	69,800	444.0

The FRE reservoir will only hold a pool during impoundment events. The WSEL in the reservoir will vary corresponding to the dam operations plan (Anchor QEA 2016). Flow past the dam is controlled by a combination of the fish passage conduits, evacuation conduits, and AWS system for the FFPF during impoundment events, depending on WSEL, until water in the reservoir reaches the spillway crest elevation of 628.0. Water above the spillway crest elevation will pass uncontrolled over the spillway and downstream of the dam. More detailed information describing the potential flood storage and spill operations for the structural alternatives is presented in the dam operations plan (Anchor QEA 2016).

Water Supply Design

Multiple design elements of the FFPF fish passage facility require water to operate. The design flows for each element are provided in Table 15-3.

Table 15-3. Water Supply Flows for FFPF Elements

Design Element	Flow (cfs)
Adult AWS	200
Juvenile AWS	50
Adult fish ladder	25
Juvenile fish ladder	25
Lamprey ramp	4
Sorting facility	10
Intake backwash system	6

NOAA Fisheries (NMFS 2011 and NMFS 2022) states that attraction flows from the entrance of the fish ladder should be greater than 10 percent of the high fish passage design flow. The minimum attraction flow for the FFPF should then be at least 220 cfs. However, the Subcommittee decided in its March 22, 2017, meeting that, because the minimum outflow during the early portion of the impoundment period was 300 cfs, as defined in the operations plan (Anchor QEA 2016), the attraction water flow for the FFPF should be increased to 300 cfs. It was agreed that providing a single source of attraction water from the ladder entrances into the stilling basin will improve the fish passage performance of the facility given that it represents the only navigable pathway for fish to ascend upstream. This is commonly observed at other facilities in operation where attraction water from the ladder is the primary source of flow that fish experience as they navigate upstream.

Water is supplied to the FFPF via gravity throughout most of the FFPF operating period. When water levels in the reservoir are too low to supply water via gravity, water supply to the AWS is suspended and water supply to the adult fish ladder, juvenile fish ladder, lamprey ramp, and sorting facility is provided via pumping. The sorting facility consists of the sorting building, holding gallery, and surrounding area.

15.3.1.3 Construction Bypass

Since development of fish passage design flows following NOAA Fisheries guidance is not complete and updated hydrology, including revised exceedance and flood flows, were in development and not available at the time of hydraulic modeling of these channels the fish passage design flows used in HDR (2017a) have been adopted for use in the design documented in this report. The historic high fish passage design flow is 2,200 cfs, corresponding to 5 percent exceedance. The historic low fish passage flow is 16 cfs, corresponding to the 95 percent exceedance.

15.3.2 FFPF Upstream Release Sites

The locations of potential upstream fish release sites used as part of FFPF operation have not yet been identified. This is consistent with a conceptual level of design development. Potential specific locations will be developed in consultation with state and federal agencies based on existing redd data and review quality of each habitat and accessibility as part of WDFW Hydraulic Project Approval development. Additional factors that will influence fish release locations such as water quality, time of year, species, etc. are discussed in detail in Appendix I.

15.3.3 Fish Passage Hydraulic Modeling Results

Hydraulic model results for fish passage conduits and permanent and construction bypass channels demonstrate depths and velocities at the high and low fish passage design flows similar to their analogous and reference reaches. Model results are provided in Appendix D and were presented to the fish passage TWG on January 17, 2024 (Appendix I). The design of the conduits and channels was developed to a conceptual level of detail. This is reflecting in hydraulic modeling that utilizes uniform roughness for conduit and channel surfaces and does not incorporate large roughness elements. Nonetheless the velocity results indicate slower velocities along the margins of the channel, indicating that inclusion of roughness elements, velocity refugia, and variations in the channel cross section are likely to be successful in creating passage routes for weaker swimmers.

15.3.4 FFPF Fish Ladder Entrance & Stilling Basin Design

The entrances to the FFPF are located as far upstream in the river as possible (immediately downstream of the fish passage conduits) to improve the performance of the FFPF by minimizing the potential for false attraction. Multiple entrances are located within the conduit stilling basin to prevent fall back. Juvenile and resident fish are the weakest swimmers of the target species (e.g. – lower burst speeds, less energetic, etc.) therefore the juvenile/resident/lamprey entrance is located closest to the stilling basin endsill. All the water entering the river during portions of the FFPF operation comes out of the fish ladder entrances and passes over the stilling basin endsill. During the remaining periods of FFPF operation all the water entering the river downstream of the FRE structure comes from the fish ladder entrances and from the evacuation conduit. At all times during FFPF operation attraction water from the fish ladder entrances meets or exceeds NOAA Fisheries requirements (NOAA 2022a), reducing the potential for false attraction. When applicable during low outflow periods, all water will be released through the FFPF to provide a single source of attraction water and thus the only navigable pathway for fish to move upstream.

Uniform flow passes over the full width of the stilling basin endsill providing hydraulic conditions, such as lower velocities and less turbulence, which are favorable to fish passage. During FFPF operation the minimum depth over the endsill will be one foot. The channel downstream of the end sill is designed without a hydraulic drop, hydraulic jump, or excessive velocity that could create an impediment to fish access to the stilling basin and the fish ladder entrances. Detailed design of the end sill to accommodate the low fish passage design flow will occur in future phases of design development. At low flow the endsill must provide depths and velocities conducive to fish passage.

15.3.5 Lighting of Fish Passage Conduits

Lighting of the fish passage conduits was not examined as part of the design documented in this report. Concern regarding fish delay or holding due to the length of the fish passage conduits if they remain unlit was shared during the January 17, 2024 fish passage TWG meeting (Appendix I). It was noted in the meeting that the Lower Granite Dam on the Snake River has a fish passage tunnel under eight spill bays (approximately 200-300 ft total) that is artificially lit to encourage passage. TWG members shared that studies show no fish passage delay through the tunnel. At a minimum, artificially lighting the fish passage conduits will be included in the preliminary and final design. Other opportunities such as eliminating the ceiling of the fish passage conduits beyond (downstream) of the cross section of the FRE structure and studies of fish passage performance with such design features will be examined in future design development.

15.4 Fish Passage Performance

Fishways and other fish passage technologies are designed to provide continuous volitional fish passage at the location of an in-stream barrier. Performance at fish passage facilities is generally characterized by the proportion of fish that can locate and navigate a fish passage facility without being harmed or perishing. Research on fish passage performance is largely limited to facilities that consist of structures, such as fish ladders or floating surface collectors, or facilities composed of natural materials (e.g., rocks and boulders), such as nature-like fishways and roughened channels.

The construction bypass channels and permanent approach and discharge channels are fundamentally different from traditional fish passage facilities and more analogous to restoration and channel design projects. The design methodology for these channels is to mimic the physical characteristics (i.e., slope, cross section, bed material, complexity) and thus the hydraulic conditions (i.e., depth, velocity, flow paths) within the Chehalis River and Crim Creek in the vicinity of the Proposed Project. This methodology is derived from the WDFW's stream simulation design approach, which assumes that fish present in the natural channel are not expected to be challenged by the stream simulation channel that looks and performs similarly to adjacent natural channels (WDFW 2013). Additionally, these channels will convey 100 percent of the flow in system.

Two-dimensional hydraulic modeling of the construction bypass channels and the permanent river channels (Appendix D) confirm that at the fish passage design flows, flow depth and velocity within these channels are similar to, or more favorable than, the reference reaches used to design the channels. At the current level of design, there is no evidence to suggest that fish passage performance through the channels will be negatively impacted by the channels themselves, when compared to the existing river at the Proposed Project location. Therefore, fish passage performance and survival through the proposed channels is assumed to be 100 percent.

For anticipated fish passage performance through the fish passage conduits, see Table 4-2 in Appendix G of HDR (2017a).

15.5 Roadmap for Future Fish Passage Design

Future fish passage design efforts will complete the conceptual fish passage design and, prior to completion of the final Biological Assessment being prepared under the Endangered Species Act Section 10 consultation, will advance the fish passage design sufficiently to demonstrate the final design of the Proposed Project will meet current NOAA Fisheries and WDFW fish passage requirements.

The fish passage design will be fully integrated and compatible with the overall dam design. Future design phases will incorporate cross-discipline design development, design evaluations and analyses, coordination meetings, and configuration decisions to achieve a complete project.

15.5.1 Climate Change Incorporation

Fish passage design flows meeting the NOAA Fisheries guidance (2022b) will be established in collaboration with NOAA Fisheries representatives during preliminary design.

The quantity of auxiliary water flow will be revisited and updated during the preliminary design phase to meet the NOAA Fisheries attraction water flow requirement and the fish passage design flows incorporating climate change.

15.5.2 Flood Fish Passage Facility

The FFPF design, referred to in previous documents as the CHTR facility, has not been advanced since publication of the CHTR Preliminary Design Report (HDR 2018bc). The design will be advanced during the preliminary design phase to be consistent with the current FRE structure and location. Using revised fish passage design flows meeting NOAA Fisheries (2022b) and current WDFW and NOAA Fisheries fish passage design guidelines, the fish passage design will be updated during future phases of design development following preliminary design. Input from WDFW, NOAA Fisheries, and the TWG will be incorporated throughout future phases of design development, including preliminary design.

15.5.3 Fish Passage Conduits

The fish passage conduit design will be refined during preliminary design. Concepts identified at this time for refinement include, but are not limited to, staggered invert elevations, roughness elements, conduit size, length and spacing, and artificial lighting. Additional analyses include identifying low-velocity fish passage pathways, sediment transport analysis, and 2D hydraulic modeling. Further fish passage conduit design refinement will be required following preliminary design, including 3D hydraulic modeling, sediment transport modeling, additional roughness elements, artificial lighting, and staggered invert elevations. Input from WDFW, NOAA Fisheries, and the TWG will be incorporated throughout future phases of design development, including preliminary design.

15.5.4 Permanent and Construction Bypass Chehalis River & Crim Creek Channels

The permanent and bypass channel designs in both the mainstem Chehalis River and Crim Creek will be refined in preliminary design. Concepts identified at this time for refinement include, but are not limited to, channel roughness, slope, alignment, and velocity refugia. Additional analyses include identifying low-velocity fish passage pathways and 2D hydraulic modeling. Further refinement of the permanent and bypass channel design will be required following preliminary design, including additional hydraulic modeling, sediment transport modeling, additional roughness elements, artificial lighting, and staggered invert elevations. Three-dimensional hydraulic modeling of the permanent and bypass channels may also be required. Input from WDFW, NOAA Fisheries, and the TWG will be incorporated throughout future phases of design development, including preliminary design.

Future design development of the channels will also include design of the channel to resist erosion and to avoid subsurface flow, especially at low river flows, so that a minimum depth for fish passage is maintained in the channels. Stable elements such as large rock will be used to set a stable cross-section in the channels, including downstream of the fish passage conduit stilling basin endsill, to meet hydraulic and fish passage design requirements.

16 Construction Considerations

Construction and constructability considerations are developed in the Cost, Schedule, and Constructability Report provided in Appendix K. Key construction cost, schedule, and construction risk drivers include:

- Large project size and quantities of materials required
- Staged river diversion routing through the work area and construction flood risk
- Limited foundation characterization at the revised FRE alignment site
- Critical importance of aggregate supply partially due to the large project size
- Site development (access and staging) limited by topography, river hydrology, environmental sensitivities
- Uncertainty in cementitious material supply and industry trends
- An FRE design that incorporates certain design components to not preclude potential future expansion.

16.1 Construction Phase Flood Risks

Construction hydrology affects the flood risk during construction and can yield a significant cost and schedule impact. Construction hydrology has evaluated flows up to 25-year AEP routed in a diversion channel through the right side of the dam alignment as described in Chapter 6.6. A 5-year construction duration has been assumed for the RPD construction duration. Although very project specific, diversion capacities protecting work at a return period flow approximately 5 times the construction duration are commonly chosen. For example, design for a 25-year return period flow may be reasonable protection for a project with a 5-year construction exposure. Clearly the Proposed Project would bear risk beyond above that design threshold, and that risk should be addressed when considering cost and schedule, and also when developing specifications and contract requirements during final design. Appendix K discusses the new preliminary diversion planning which provides for 25-year recurrent flow protection until the bypass conduit is completed and practical diversion capacity reduces to approximately 10-year recurrent flow levels.

Flood event flow exceedance probabilities are dramatically higher annually than in the low-flow summer months. For example, 25-year recurrent flood flows between June and September are less than one tenth the annual 25-year AEP, and less than one fiftieth the annual 25-year AEP if only considering July and August. While not relevant to primary work which will last months across all seasons, the drier summer months are important to work that can be allowed in the river and for when critical diversion components need to be constructed, including when temporary flow diversions need to be made.

A Care and Diversion of Water during Construction specification will be developed during final design addressing construction contractor requirements during construction if water overtopped the cofferdam. The specification likely will include a requirement for the construction contractor to submit a Dewatering Plan, Diversion Plan, and a Fish Exclusion and Relocation Plan for how to clean and remove water and relocate fish behind a cofferdam following a potential overtopping event. The diversion plan will require flood monitoring, anticipated event preparation and evacuation, and the staging of equipment and materials including potentially hazardous materials outside of the 100-year

WSEL. The plan will require specific information such as duration of time following an event before all fish are relocated, how long fish can be held, where fish must be returned to, biodegradable oils for hydraulic equipment, specific cleaning equipment and materials, personnel training, and plan review times.

16.2 Construction Sequence

The sequencing of construction activities, including moving the river from its existing channel to the bypass channel and from the bypass channel to its permanent channel, impacts construction schedule and hence construction cost significantly. Investigating it and evaluating at least one feasible option for sequencing the construction clarifies risk factors and possible approaches. While more fully developed in Appendix K, the RPD assumes four phases of construction and two related relocations of the river channel as shown in Table 16-1.

Table 16-1. Risk Summary for Construction Sequence

Phase	Work	Duration (months)	Risk
0 - Preliminary	Preliminary work independent of the river	6-12	low
1 – River flow	Construct river bypass channel and right dam foundation. Chehalis River in existing channel.	10-12	moderate
2 – Channel flow	Construct outlet works including conduits, left dam foundation. Chehalis River in bypass channel.	20-24	high
3 – Bypass Conduit Flow	Remove bypass channel, construct dam foundation closure. Chehalis River in permanent channel.	10-12	high
4 – Bypass Conduit flow	Complete dam construction, and final outlet conduit configuration modifications. Chehalis River in permanent channel.	6-12	low

16.3 Construction Road Access

Updated temporary access roads are shown in Appendix G. Temporary Construction access routes will be removed or stabilized once construction has been completed. During this phase of the design process, most construction access roads developed will be permanent once construction is completed. However, construction roads not anticipated as permanent, will be developed using the same criteria as outlined in Section 11.3 in this document.

16.4 Distribution Lines for Construction Power

Similar to the original project description, the proposed FRE dam will require an electrical supply for construction and operations of the gates and other dam equipment. Construction power requirements may be provided either with on-site diesel-powered generators or through a distribution power line interconnection with the existing electrical grid, or a combination of both. Electrical power for operations will be provided by installation of a distribution power line to the electrical grid. The location of interconnection and route of the interconnecting distribution line will be determined by the local power supply utility. Overhead lines would be installed along existing roads within the first six months of year one of the construction schedule. See the FRE Site Temporary and Permant Power

TM (Appendix N) Sections 2.4 through 2.7 for four potential options for power distribution during construction. High power use construction functions include aggregate crushing operations, concrete and RCC plant operations, concrete and RCC mix temperature control energy, construction lighting during short daylight seasons and during shift work. Quarry proximity is not conducive to economical line power. Consequently, line power is likely to benefit other needs and to be fully useful, may need to be near the dam's upper right abutment and on the order of 3,000 to 5,000 kVA.

16.5 Temporary Construction Facilities and Trucking Information

Appendix K, Constructability TM, includes an appendix that provides discussion on the revised construction facilities and includes a conceptual layout of the construction facilities required to construct the proposed FRE. The Constructability TM further provides information regarding the estimated number of truck trips.

16.6 Water Usage during Construction

Construction water will be required for dust control, aggregate processing, concrete production, embankment fill, offices, warehouses, shops, tunneling operations, and various unlisted uses. Dam projects require a considerable amount of water with usage varying due to concrete specifications, aggregate in-situ properties, aggregate processing specifications, embankment compaction requirements, seasonal climate, number of on-site workers/staff, and various other project requirements. Based on other project experiences, water demand requirements are estimated to be 2,000,000 gallons per day (3 cfs) during construction activities. A water demand evaluation will be performed during final design to refine the estimate. The District is committed to avoid impacts to existing water supplies and water quality for local water withdrawals such as the City of Pe Ell while using water during construction.

The demand flow rate for construction water will vary throughout the course of construction as construction activities vary. Seasonal influences will also affect water demand. For example, construction water consumption for dust control will be much reduced during rainy months. Water storage tanks will likely be utilized by the construction contractor to help buffer some of the short term peak demands and facilitate continuous construction. Construction water will likely be obtained through surface water withdrawals from the Chehalis River. “Limited groundwater is present in (the vicinity of the project site) because the substrates are predominately bedrock with a thin layer of overlaid alluvial material” (Draft Biological Assessment and Essential Fish Habitat Assessment Chehalis River Basin Flood Damage Reduction Project; HDR 2021c). As such it is unlikely that groundwater would be employed for construction water. Fish screens meeting state and federal fish screening requirements would be employed for surface water withdrawals. The withdrawal location on the Chehalis River will likely be in the vicinity of the construction to minimize the temporary water supply infrastructure footprint. Temporary water supply infrastructure, including the withdrawal location, will be designed, installed, and operated in accordance with federal, state, and local laws and regulations. The proposed location of the construction water withdrawal will be identified as the design is further developed. Temporary water supply pipeline(s) will be installed to carry water to specific locations on the construction site, including water storage tanks and the concrete batch plant. All temporary water supply infrastructure such as water lines, pumps, and storage tanks will be removed upon completion of construction.

A feasibility study will be performed to identify water rights requirements for construction following WDOE guidelines. Water may be pulled directly from the Chehalis River, from a well drilled to obtain water or a combination of both sources. Public water supply lines within the area for project construction use are assumed to be unavailable.

17 Construction Schedule

A construction schedule has not yet been prepared but will be updated when project design advances. Appendix K provides a preliminary estimate of project duration in relationship to the phased construction, which totals 52 months, and is without consideration of in-water work constraints and impacts.

HDR anticipates the total time to construct the Proposed Project is 5 years within a probable range of between 4 and 6 years. Schedule contingency is warranted considering; a project schedule has not yet been developed, weather impacts, remaining foundation characterization, diversion exceedance potential, unexpected conditions, delays associated with equipment or material delivery, or other factors.

Key schedule and schedule risk drivers for the RPD include:

- Contract performance period
- Notice to Proceed date in comparison to in-water work constraints
- In-water work restrictions
- Climate and precipitation impacts on RCC and other weather sensitive work
- Access and staging development
- Construction phase sequencing
  - Phase 1 diversion channel construction
  - Phase 2 bypass outlet works conduit and necessary left-side work
  - Phase 3 foundation, dam, and stilling basin construction in the diversion channel closure section
  - Construction and completion of hydraulic structures following RCC
- Early quarry development and aggregate production outpacing demand
- Favorable river and flood flows during the work, avoiding exceedance events
- High-capacity RCC and concrete production and delivery systems
- A qualified and well-resourced contractor with strong project management capacity

17.1 Construction Sequence

Table 16-1 in Section 16.2 shows the estimated duration for the construction sequence by four diversion and construction phases; which generally include

- Site preparation, diversion channel construction, and right dam foundation construction
- Bypass outlet works conduit, fish passage, and left dam foundation construction
- Construction of the dam foundation closure beneath the diversion channel alignment
- Construct all remaining work

Work expected in Phases 1 through 3 would include earthworks, foundation grouting and preparation, and potentially RCC. RCC would be expected during Phases 2 and 3 and finishing during Phase 4. Concrete structures will be constructed beginning in Phase 2 and lasting through Phase 4.

17.2 Schedule and Construction Considerations

17.2.1 In-Water Work Window

Based on the project design it is anticipated that permitting variances will be required to extend normal in-water work windows. The WDFW approved in-water work window for the Chehalis Basin upstream of the South Fork is August 1 to August 31, and the USACE approved in-water work window for the same river reach is July 1 to August 31, preliminarily. To minimize impacts during construction by making use of the optimal hydrologic conditions as previously described, and to avoid impacts from continuous construction over a longer period of time, an extension of the in-water work window from July 1 to September 30 will be requested from WDFW and USACE. Following preliminary design when more refined construction schedule and cost and construction risk estimates are prepared, the in-water work window will be revisited in preparation for project permitting.

The Proposed Project is large and complex with large elements of construction risk. Consideration should still be given to:

- Early work packaging to allow access development, aspects of construction staging, limited quarry development, including test crushing or crushing of road base for roads and site use.
- Quarry test crush development (if not incorporated into early work packaging).
- Early contractor involvement.
- Value-based, best-value, contractor selection allowing contractor selection to consider cost and non-cost evaluation criteria.

18 Summary of Permanent and Temporary FRE Site Physical Impacts Including Waters of the United States

Construction and operation of the proposed FRE will cause temporary, permanent, and episodic impacts to waters of the United States (WOTUS). WOTUS within the project vicinity were delineated by Anchor QEA in 2018 and are summarized in *Wetland, Water, and Ordinary High Water Mark Delineation Report* (Anchor QEA 2018). WOTUS in the project vicinity include the Upper Chehalis River, associated tributaries such as Crim Creek, and associated wetlands. Appendix O summarizes permanent and temporary FRE site physical impacts.

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Appendix A. Maps and Drawings

April 25, 2024

Electronic version provided separately.

Appendix B. 2017 Reservoir Operations Plan Sensitivity Analysis TM

April 25, 2024

Electronic version provided separately.

Appendix C. FRE Hydraulic Design Assumptions – Additional Information TM

April 25, 2024

Electronic version provided separately.

Appendix D. Hydraulic Design

- Spillway Alternative Selection TM
- Fish Passage Conduits TM
- Construction Bypass TM

April 25, 2024

Electronic version provided separately.

Appendix E. Conceptual Geotechnical Design Report

April 25, 2024

Electronic version provided separately.

Appendix F. Structural Analysis and Design

April 25, 2024

Electronic version provided separately.

Appendix G. Civil Design - Access Roads & Best Management Practices TM

April 25, 2024

Electronic version provided separately.

Appendix H. Mechanical Design – Conceptual Lower Level Outlet Gate Design Report

April 25, 2024

Electronic version provided separately.

Appendix I. Fish Passage Design TM

April 25, 2024

Electronic version provided separately.

Appendix J. Operations and Maintenance TM

April 25, 2024

Electronic version provided separately.

Appendix K. Cost, Schedule, and Constructability

April 25, 2024

Electronic version provided separately.

Appendix L. Seismic Design Hazards TM

April 25, 2024

Electronic version provided separately.

Appendix M. 2017 Chehalis Basin Strategy Operations Plan for Flood Retention Facilities

Prepared by Anchor QEA, LLC (2017)

April 25, 2024

Electronic version provided separately.

Appendix N. Informational Documents Brought Forward

- Ranking of Potential Quarry Sites for Proposed Flood Retention Structure on the Chehalis River TM
- Response to Climate Change Flows
- Transfer of Use and Jurisdiction Letter
- Existing All Species Fish Passage Facilities Research TM
- FRE Site Temporary and Permanent Power TM
- FRE Site Selection TM
- FRE Facility – Conceptual Level Recreational Improvement Options TM
- Additional Information – Environmental Justice Benefits of the Proposed FRE Project TM
- Quarry Operations TM (Draft)
- Airport Levee Wetland Avoidance TM
- Dam Safety Standards and Seismic Fault Study Review TM

Electronic version provided separately.

Appendix O. Permanent and Temporary FRE Site Physical Impacts TM

April 25, 2024

Electronic version provided separately.